House of Sweden

Structural Study of Alternative Floor Systems

2900 K St. NW
Washington, DC 20007

The Pennsylvania State University
Department of Architectural Engineering
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EXECUTIVE SUMMARY

Technical Report 2 is the Structural Study of Alternative Floor Systems Report. This report was generated to investigate alternative floor systems for the House of Sweden. Four alternative systems were considered and preliminary designs were conducted and compared to the original post-tensioned system. The north-east corner of the north building was taken as a representative area for the preliminary designs. The four alternative systems are:

- Composite Steel Deck with Non-Composite Beams
- Composite Steel Deck with Composite Beams
- Pre-Cast Hollow Core Slab on Pre-Cast Beams
- Two-Way Reinforced Concrete Slab

When the systems were compared, none of the alternatives systems were immediately recognized as a viable alternative to the existing system. This is due to the 22' cantilever that exists on the north side of the building. This cantilever presented a design challenge that was met by devising a steel tube hanger system for the composite steel systems and a non-prismatic beam with hollow core slabs for the concrete systems.

Overall, the composite steel deck with non-composite beams was not a viable system; however, this system was only analyzed as a baseline for the composite steel beam system. The two-way reinforced concrete slab might be a possible system, but is hard to construct and has the very deep non-prismatic beam. The hollow core slab is viable due to the ease of construction, but steel might want to be investigated to reduce the depth of the system. The most viable alternative is the composite steel deck with composite beams because of the weight is very small, the construction is fairly easy, and the erection time is short. It was noted that the existing system, is still the best option in overall depth, construction time, budget, and cantilever solution.

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INTRODUCTION

This Structural Study of Alternative Floor Systems contains a description of the slab conditions currently existing in the House of Sweden, including gravity loading and deflection criteria. It provides a synopsis of the structural components including gravity and lateral load systems. Through analysis of the serviceability and strength of alternative floor systems, this report discusses the feasibility of implementing these systems in a later re-design that might become part of the overall proposal.

BACKGROUND

House of Sweden (Cover Figure) is located in Georgetown, Washington D.C. at the intersection of Rock Creek and the Potomac River. This development is built on a single mat foundation with a parking garage level and then two separate towers rise out of the site. The south building consists of 5 stories and a mechanical penthouse; the north building is 6 stories and a mechanical penthouse. Construction of the two buildings began on August 4, 2004 and finished on May 12, 2006. It was delivered in a design-bid-build method where the design of the south building was commissioned as a competition in Sweden.

Wingardh Arkitektkontor AB completed the winning design for the south building and houses the Swedish Embassy along with an exhibit hall, convention center, rooftop terrace, and apartments. They designed this building to be “a shimmering jewel in the surrounding parkland.” To accomplish this goal, the base of the building is clad in light stone, while the upper floors are clad in glass laminated with a traditional Nordic blond wood pattern. This glass façade is backlit at night to create the illusion of the structure floating above the river.

Housed in the north building are offices and apartments, which incorporate expansive balconies and long stretches of ribbon windows to maximize exterior views. The façade employs the same type of light stone on the podium, but the upper floors are clad in metal panels. This lets the north building relate to the south building, yet keep its own identity.

Both building envelopes are steel stud construction with faced blanket insulation and gypsum wallboard attached. A standoff system is used on the north building to attach light stone panels to the podium of the building and metal paneling to the upper floors. This same standoff system is used on the south building to attach light stone paneling on the lower level. The upper levels employ a different standoff system of laminated glass panels as cladding. None of these cladding systems are used as a barrier system, which is why the insulation is faced to prevent moisture penetration.

DOCUMENT AND CODE REVIEW

The following documents were either furnished for review or otherwise considered for this report:

- ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* published in 2006 by the American Society of Civil Engineers
- IBC 2006 *International Building Code* published in January 2006 by the International Code Council, Inc.
- ACI 318-08 *Building Code Requirements for Structural Concrete* published in January 2008 by the American Concrete Institute
- AISC 13th Edition *Steel Construction Manual* published in December 2005 by the American Institute of Steel Construction, Inc.
- PCI 6th Edition *Design Handbook* published in 2004 by the Precast/Prestressed Concrete Institute
- *Post-tensioned Concrete Floors* authored by Sami Khan and Martin Williams published in 1995 by Butterworth-Heinemann Ltd
- *Notes on ACI 318-08 Building Code Requirements for Structural Concrete* published in 2005 by the Portland Cement Association
- *Two-Way Post-Tensioned Design Example* published by the Portland Cement Association
- Construction Documents originally dated October 28, 2003 by VOA and TCE

STRUCTURAL SYSTEM DISCUSSION

Foundation

Cast-in-place piles support a mat foundation. These piles are 16" in diameter with a concrete compressive strength of f'_c = 6,000 psi and exist under the north perimeter of the parking garage. The mat foundation exists over the entire parking garage. It is a minimum of 38" thick, and 42" at the columns with a concrete compressive strength of f'_c = 4,000 psi and rests on a 2" thick mud slab. It is reinforced with rebar varying from #18 bars to #6 bars and at a variety of spacings. This foundation is either set on the piles at the north perimeter, or held with tie-downs. Columns from both the north and south buildings will be supported on the mat foundation.

Framing System

House of Sweden is located in Georgetown, Washington, DC; therefore, the use of a post-tensioned concrete structural system was an obvious choice to help minimize the slab thickness and maximize the number of floors. Most of the floors above grade are two-way post-tensioned concrete flat slabs.

The north building has 6 levels above grade. The first floor slab is a 9"-10.5" thick reinforced with #4 and #5 bars and the drop panels are 5", 8", or 10" thick and reinforced with #7 and #8 bars. The second through sixth floors are 7"-8" thick with drop panels reinforced with #5 and #6 bars. Typical concrete strength on these floors is 6 ksi or 8 ksi. Concrete strength and slab thickness vary on each floor, which means that the slabs were not placed as single, monolithic pours and they had to be completed in sections. Because of the irregular building shape, there is no typical bay spacing, although many bays were kept at 30' x 30', possibly accounting for the change in slab strength and thickness.

The south building has 5 levels above grade. The first floor slab is a 9"-12" thick reinforced with #4-#6 bars and the drop panels are 8", 10", or 12" thick and reinforced with #6- #9 bars. The second through fifth floors are 10"-12" thick with drop panels reinforced with #5 and #6 bars. Typical concrete strength is 6 ksi or 8 ksi. Concrete strength and slab thickness vary on each floor, which means that the slabs were not placed as single, monolithic pours and they had to be completed in sections. Because of the irregular building shape, there is no typical bay spacing, although many bays were kept at 32' x 22', possibly accounting for the change in slab strength and thickness.

The penthouse roof of the north building is similar to the floor slabs. It is a two-way, post-tensioned slab, 7" thick with a concrete strength of 6 ksi. It has drop panels reinforced with #4 and #5 bars. This roof was designed to hold a 30 psf snow load, plus snow drift load around the mechanical equipment.

The main roof of the south building is similar to the floor slabs. It is a two-way, post-tensioned slab, 10" or 12" thick with a concrete strength varying from 6 ksi to 8 ksi. The drop panels are reinforced with #5 and #6 bars. This roof was designed to hold a 30 psf snow load plus snow drift load around the mechanical equipment and the penthouse to the north. Since the south half of the roof has a convention space, it was designed to hold a 100 psf terrace load plus a 25 psf paver load.

For ease of calculation, the north building was used as a representative building for the alternative slab designs. Calculations were completed using regular bay spacings of 30'x30' with the 22' cantilever and 24"x24" square concrete columns in the Northeast corner. Refer to Figure 1. for the specific location.

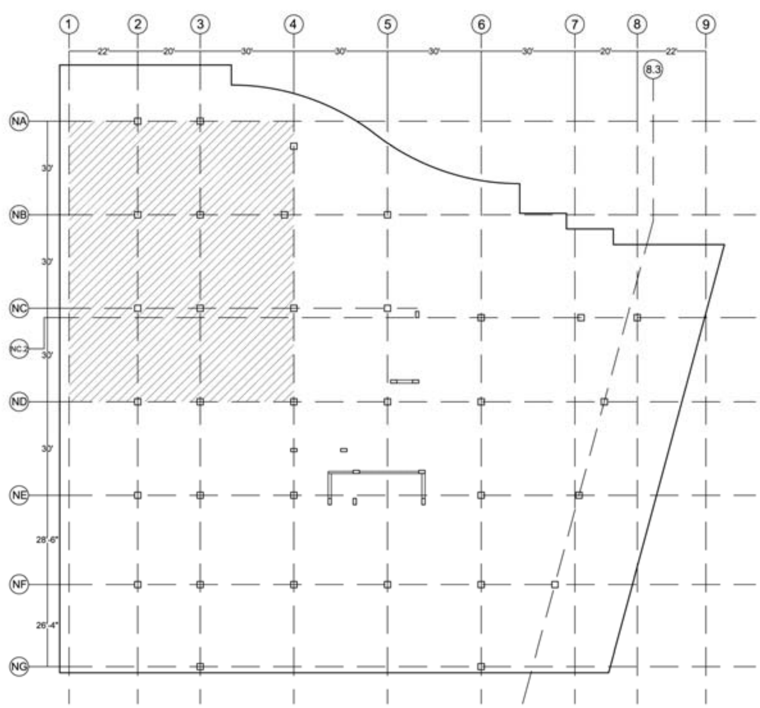


Figure 1. North Building Alternative Slab Area of Design

Lateral System

Shear walls make up the lateral system of the north building from the garage to the fourth floor (Figure 2.). These walls vary in width and are 8 ” or 12” thick with concrete strength of 6 ksi reinforced with #4 bars at 12” spacing in two curtains. These shear walls stop below the fifth floor where the structure becomes a concrete moment frame. This system resists lateral loads in the north-south and east-west direction depending upon the orientation of the wall.

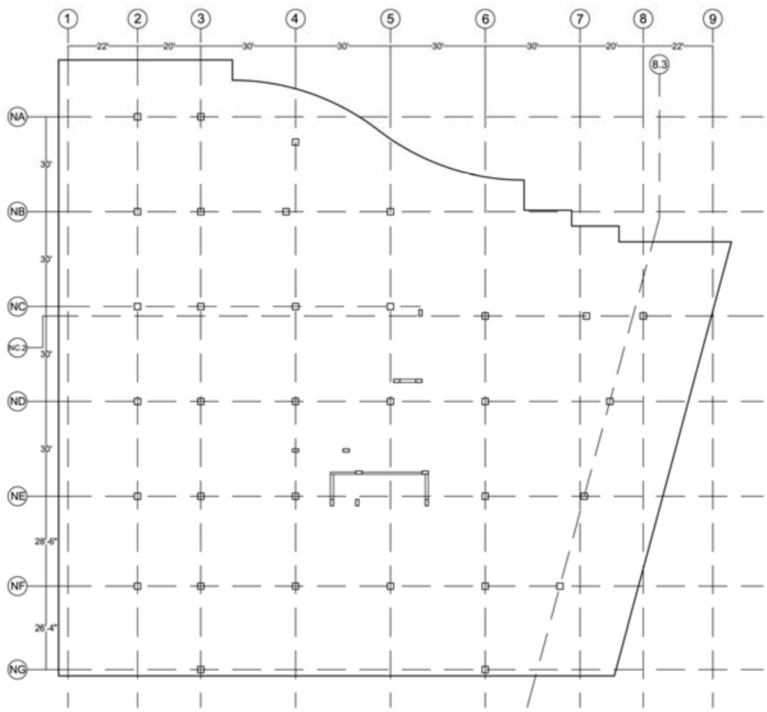


Figure 2. North Building Column and Shear Wall Layout

Shear walls exist in the garage under the south building and are 12” thick with a concrete strength of 6 ksi reinforced with #4 bars at 12” spacing in two curtains. However, these walls do not extend past the garage level, and the building lateral system becomes a concrete slab-frame moment system to resist lateral loads in both the north-south and east-west directions Refer to Figure 3. for a typical floor plan.

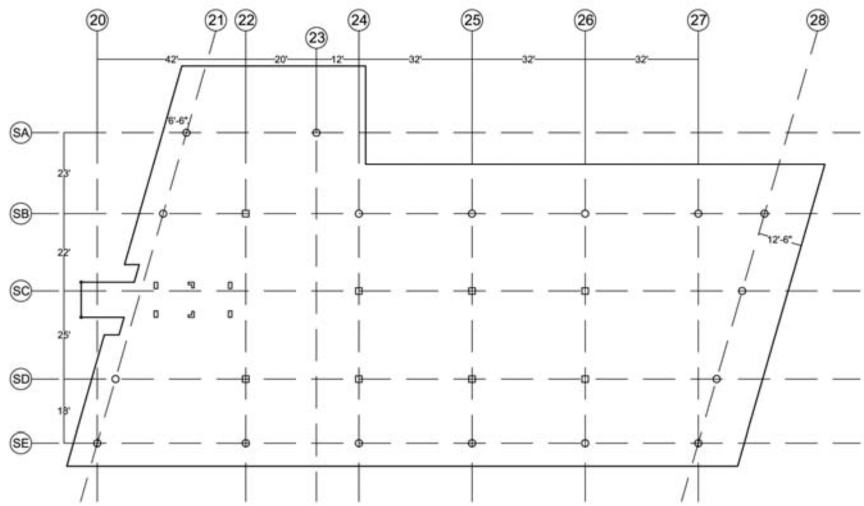


Figure 3. South Building Column Layout

GRAVITY LOAD DISCUSSION

To analyze the gravity system of the House of Sweden, the static and dynamic loading on the structure had to be determined. The following is a summary of the approximate design gravity loads and criteria used to spot check the House of Sweden's gravity system. Load references are listed in the tables.

Deflection Criteria

Floor Deflection – IBC 2006 Table 1604.3

Typical Live Load Deflection for Floor Members	L/360
Typical Total Deflection for Floor Members	L/240

Floor Dead Loads		
Occupancy	Design Load	Reference
Normal Weight Concrete	150 pcf	ACI 318-08
Roof Pavers	25 psf	Structural Drawings
Ballast, Insulation, and waterproofing	8 psf	AISC 13th Edition
Glass Curtain Wall	6.4 psf	Glass Association of North America
Studs and Batt Insulation	4 psf	AISC 13th Edition

Roof Live Loads		
Occupancy	Design Load	ASCE7-05 Load
Public Terrace	100 psf	100 psf
Snow Load**	30 psf*	20 psf*
Rain Load**	---	41.6 psf

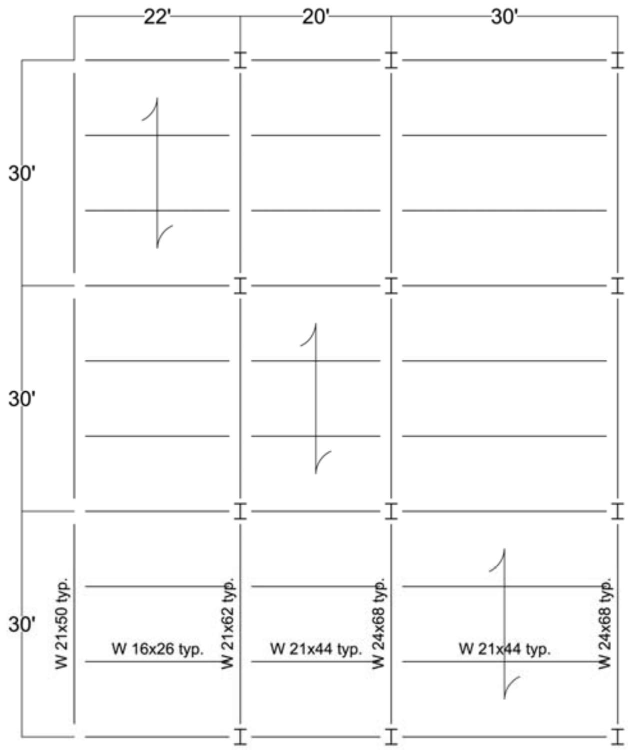
**Snow drift will accumulate around the penthouse and on the lower roof of the north building. This load was calculated and can be found in the Appendix B along with the flat roof snow load and rain load calculations.

Floor Live Loads		
Occupancy	Design Load	ASCE7-05 Load
Penthouse Machine Room	150 psf*	Not listed specifically, but light storage warehouses - 125 psf*
Residential	40 psf + 20 psf for partitions*	40 psf*
Stairways	100 psf	100 psf
Corridors	100 psf	100 psf
Commercial and Plaza Area	100 psf*	Offices - 50 psf, Corridors above 1st floor - 80 psf, Lobby - 100 psf*
Elevator Machine Room	300 lbs of concrete load on 4 square inches	300 lbs of concrete load on 4 square inches
Loading Dock	400 psf	Not listed specifically
Parking Garage	50 psf and 2000 lbs of concrete load on 20 square inches*	40 psf and 3000 lbs of concrete load on 20 square inches*

*For load discrepancies, worst case scenario loading was used.

ALTERNATIVE FRAMING DISCUSSION

System 1: Composite Steel Deck with Non-Composite Beams



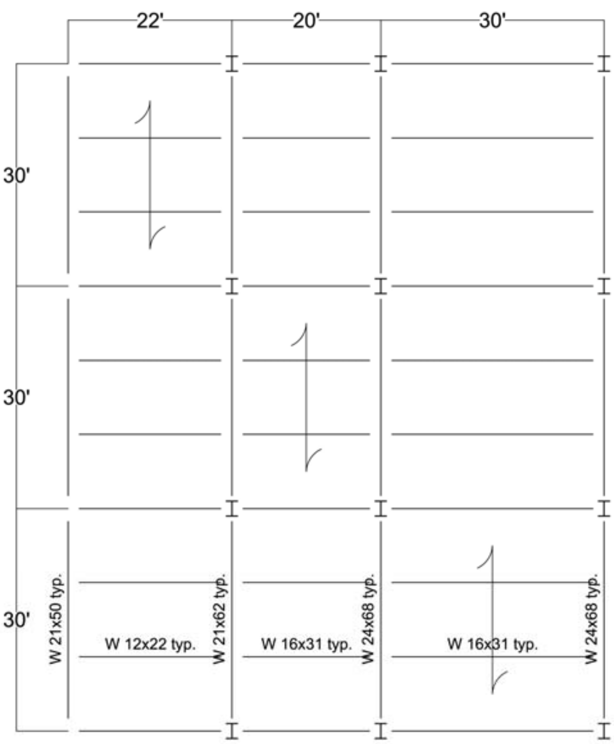
This system was analyzed as a reference point for System 2: Composite Steel Deck with Composite Beams. The design is 3000 psi concrete reinforced with welded wire fabric on top of a 2" metal 18 gauge Volcraft deck. The beams are 10' on center.

To address the large cantilever, a hanger system was devised out of Round HSS steel. The tube is anchored at the top of the exterior column and connects to the 6th floor at a 46.1° angle. The 7th floor is only an 11' cantilever so it is able to carry its own weight to the column.

The overall depth of the system is 24½". The necessary 2-hour fire rating for the deck is met by the 5¼" concrete slab thickness; however, the steel members will need spray fire-proofing to meet code. The positives of this system are the ease of erection of steel and the elimination of formwork due to the metal deck. Due to the non-Composite action of the beams, they are on the heavier side and therefore, this system is not viable due to the excessive depth and weight.

Refer to the chart in the System Comparison Discussion section for a comparison between all the systems and a look at the viability of this system.

System 2: Composite Steel Deck with Composite Beams



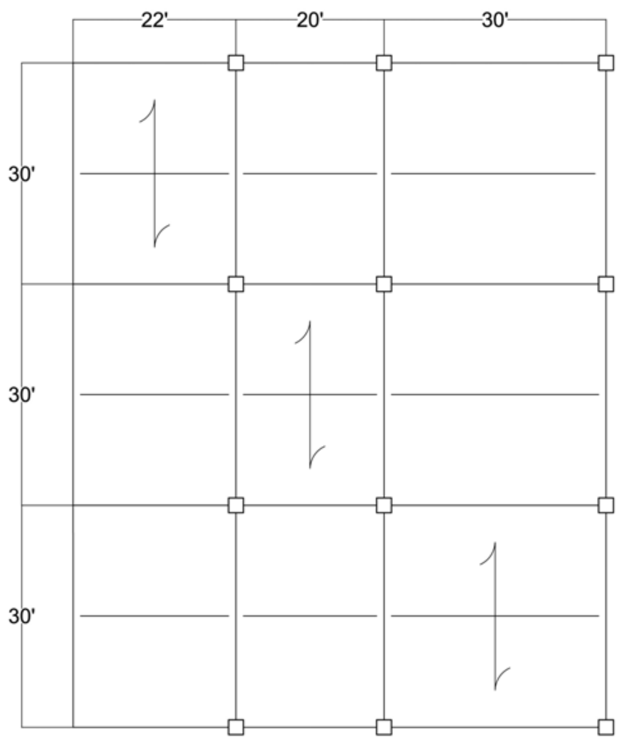
This design is 3000 psi concrete reinforced with welded wire fabric on top of a 2” metal 18 gauge Volcraft deck. It produced beams at 10’ on center that are thinner in depth and less weight than the non-composite system as was anticipated. The girders are the same in both the composite and non-composite systems because deflection controlled the design.

To address the large cantilever, a hanger system was devised out of Round HSS steel. The tube is anchored at the top of the exterior column and connects to the 6^{th} floor at a 46.1° angle. The 7^{th} floor is only an 11’ cantilever so it is able to carry its own weight to the column.

The overall depth of the system is 24½”. The necessary 2-hour fire rating for the deck is met by the 5¼” concrete slab thickness; however, the steel members will need spray fire-proofing to meet code. The positives of this system are the ease of erection of steel and the elimination of formwork due to the metal deck. Due to the fact that deflections controlled the design, the beams and girders are on the heavier side but this system may warrant further investigation due to the handling of the cantilever and the overall weight of the system.

Refer to the chart in the System Comparison Discussion section for a comparison between all the systems and a look at the viability of this system.

System 3: Pre-Cast Hollow Core Slab on Pre-Cast Beams



This design is pre-cast hollow core slabs, 6" deep with a 2" normal weight concrete topping. The slabs are 4' wide and pre-stressed with a strand designation of 66-S. The beams are also pre-cast and spaced at 15' on center. The exterior beams are 20" deep and 26" wide; the interior beams are 44" deep and 28" wide. The columns are kept as the original 24x24 design.

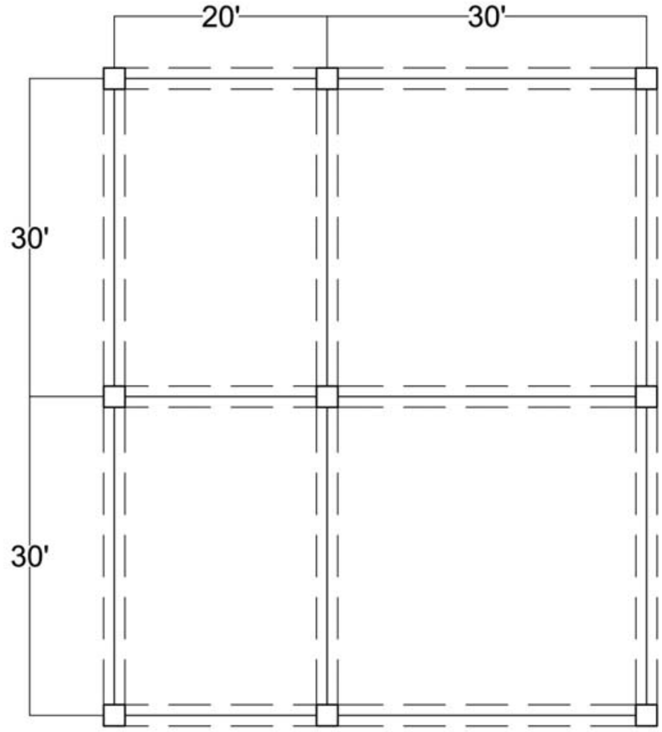
To address the large cantilever, a non-prismatic beam was designed to carry the pre-cast hollow core slabs. The total depth at the column is 44" and extends to the edge of the cantilever, where the beam tapers to 8". The pre-cast hollow core slabs will be supported on this beam and all the weight will be transferred to the exterior columns. Shear reinforcing is necessary throughout the entire beam and will be provided by #3 stirrups. Flexural reinforcing is fairly standard, 12 #9 bars, but they should be located at the top of the beam to counteract the moment of the cantilever.

The overall depth of the system is 44". The necessary 2-hour fire rating for the deck is met by the 8" concrete slab thickness. No steel is used in this design, so no extra fireproofing is necessary for any members. The positives of this system are the ease of erection, the elimination of formwork due to the pre-cast components, and the short lead time. Due to the excessive depth of this system, this may not be a viable solution; however, using steel instead if the pre-cast beams, or possibly using a smaller beam spacing could make this system a more feasible solution.

Refer to the chart in the System Comparison Discussion section for a comparison between all the systems and a look at the viability of this system.

System 4: Two-Way Reinforced Concrete Slab

This design is 6000 psi concrete with beams spanning between all the columns. The columns were kept as the original 24x24 design. A slab thickness of 10½" was determined with 24x20 beams. Due to the large bay spacing, the moments induced in the slab are quite high so much reinforcing is necessary. #8 bars were used in the column strips with up to 25 bars necessary to support the exterior span negative interior moment in the column strip. The exterior panels needed less reinforcing with only up to 9 bars necessary to counteract the exterior span negative interior moments. Deflection was not calculated due to the use of the deflection table in ACI to find the slab thickness.



To address the large cantilever, a non-prismatic beam was designed to carry the pre-cast hollow core slabs. The total depth at the column is 44" and extends to the edge of the cantilever, where the beam tapers to 8". The pre-cast hollow core slabs will be supported on this beam and all the weight will be transferred to the exterior columns. Shear reinforcing is necessary throughout the entire beam and will be provided by #3 stirrups. Flexural reinforcing is fairly standard, 12 #9 bars, but they should be located at the top of the beam to counteract the moment of the cantilever.

The overall depth of the system is 20" for the main building and 44" for the cantilever beams. The necessary 2-hour fire rating is met through the use of concrete and the clear cover for the reinforcing steel. The positives of this system are the small depth of the floor system in the main building and the elimination of fireproofing in the design versus steel design. More study should be conducted to see if this system is viable due to the elimination of the post-tensioning while keeping the design similar to the existing.

Refer to the chart in the System Comparison Discussion section for a comparison between all the systems and a look at the viability of this system.

SYSTEM COMPARISION DISCUSSION

System	System 1 Two-Way Post-Tensioned Concrete Slab (Existing)	System 2 Composite Steel Deck with Non-Composite Beams	System 3 Composite Steel Deck with Composite Beams	System 4 Pre-Cast Hollow Core Slab on Pre-Cast Beams	System 5 Two-Way Reinforced Concrete Slab
Cost per ft^2	\$21.55	\$36.99	\$24.70	\$25.85	\$25.05
Slab Depth	8"	5¼"	5¼"	8"	10½"
Structural Depth	22"	24½"	24½"	44"	44"
Structural Weight	100 psf	48.4 psf	47.1 psf	120 psf	158 psf
Cantilever Solution	Post-Tensioning	Steel Tube Hangers	Steel Tube Hangers	Non-Prismatic Beam	Non-Prismatic Beam
Fireproofing	No Additional Fireproofing Required	Fireproofing Necessary on Beams	Fireproofing Necessary on Beams	No Additional Fireproofing Required	No Additional Fireproofing Required
Lead Time	Short	Long	Long	Long	Short
Construction Difficulty	Hard	Easy	Medium	Easy	Hard
Formwork	Necessary	None Necessary	None Necessary	Necessary for Cantilever	Necessary
Additional Study	--	No	Yes	Yes	Yes
Overall Feasibility	Existing System	No	Most Possible	Possible	Least Possible

CONCLUSION

This report analyzed four alternative slab systems and compared them to the existing system for feasibility. The four alternative systems are composite steel deck with non-composite beams, composite steel deck with composite beams, pre-cast hollow core slab on pre-cast beams, and two-way reinforced concrete slab. The existing system is two-way post-tensioned concrete.

The composite steel deck with non-composite beams is not a viable system, but it was analyzed as a reference for the composite steel deck with composite beams.

The two-way reinforced concrete slab may be a viable system if more research is conducted. This system eliminates the need for post-tensioning while keeping the slab at approximately the same overall depth. The lead time, budget, and construction difficultly are fairly close to the existing. The structural weight of the system is fairly high and reduces the possibility of use as an alternative system.

Pre-cast hollow core slab on pre-cast beams are a possible alternative. The main issue with this system is the depth of the pre-cast beams. As further study, looking at the possibility of using steel beams instead of the pre-cast beams may decrease the overall depth and weight of this system and create a higher likelihood of use as an alternative system.

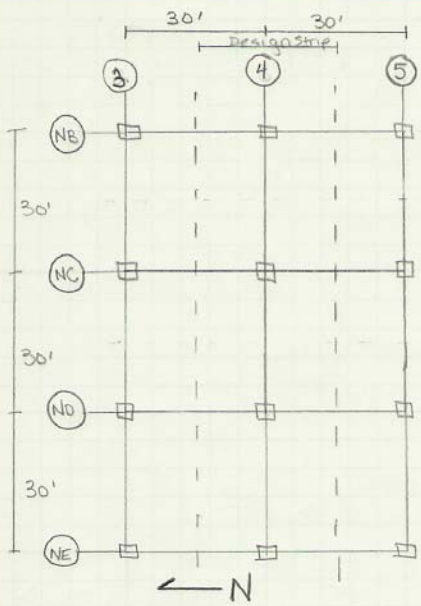
Composite steel deck with composite beams is the most viable alternative. The budget and structural depth are comparable to the existing system. The need for formwork is eliminated, but the need for fireproofing is created. The lead time is longer than that of the post-tensioned slab, but the erection time is shortened and the schedule will not change much.

Overall, the composite steel deck with composite beam system is the most viable alternative and warrants further study. A look at the comparison chart shows that no specific system is a better alternative to the existing system. The existing system is also able to address the cantilever with an overall structural depth of 22" and no hanger system to hinder the exterior cladding of the building. Further study may be conducted into one of the alternatives, but the existing post-tensioned system appears to be the most economical and effective system for the House of Sweden.

APPENDIX A – Two-Way Post-Tensioned Concrete Slab (Existing)

Two-Way Post-Tensioned Concrete Slab (Existing)

4th Floor Slab | Spot Check | North Building



Columns : 24" x 24"
Story height : 10'-10"

f'_c = 6 ksi (slab), 6 ksi (columns)
* slab strength varies on plans but for this report, just the 6 ksi will be used
f_y = 60 ksi
f_{pu} = 270 ksi
f'_{ci} = 4500 psi
w = 150 pcf (N-W concrete)
LL = 100 psf (commercial occupancy)

PT: Unbonded tendons
1/2" ϕ, 7 wire strands, A = 0.153 in^2
estimated prestress losses = 15 ksi
$f_{se} = 0.7(270) - 15 = 174$ ksi
$P_{eff} = Af_{se} = 0.153(174) = 26.6$ k/tendon

14" deep drop panels

Preliminary Slab Thickness

$$\frac{\text{span}}{\text{depth}} = \frac{L}{45} = \frac{360}{45} = 8''$$

Use Equivalent Frame Method

Shear Strength of Slab

d = 6.94" (3/4" clear cover and #5 bars)

Factored Load

LL reduction : Interior Bay
$A_T = 30(30) = 900$ ft^2
$K_{LL} = 1$
$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 0.75(100) = 75 \text{ psf}$$

$$w_u = 1.2(8/12)(150) + 1.6(75) = 240 \text{ psf}$$

consider a 12" wide strip taken at distance d from face of support

$$V_u = 0.240(12.4) = 2.98 \text{ K}$$
$$\phi V_c = \phi 2\sqrt{f'_c}\,bwd = 0.75(2)\sqrt{6000}(12)(20.9) = 29.1 \text{ K} > V_u \checkmark$$

Two-Way Post-Tensioned Concrete Slab (Existing)

4th Floor Slab | Spot Check | North Building | 2

Shear strength at distance $d/2$

$$V_u = 0.240[(30)(30) - 1.29^2] = 216\,K$$

$$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d = 0.75(4)\sqrt{6000}(4 \cdot 15.47)(20.9)/1000 = 301\,K > V_u \checkmark$$

Frame members of Equivalent Force

- Flexural Stiffness of slab-beams at both ends, K_{sb}

$$\frac{C_{N1}}{\ell_1} = \frac{24}{30(12)} = 0.07 \qquad \frac{C_{N2}}{\ell_2} = \frac{24}{30(12)} = 0.07$$

by interpolation from Table A6
with drop panels thickness = 1.75h

C_1/ℓ_1	C_2/ℓ_2	K_{NF}	C_{NF}	m_{NF}	K_{FN}	C_{FN}	m_{FN}
0.07	0.07	5.99	0.51	0.1093	4.65	0.63	0.0732

$$E_{cs} = 57000\sqrt{f'_c} = 57000\sqrt{6000} = 4.42 \cdot 10^6$$

$$I_s = \frac{\ell_2 h^3}{12} = \frac{30(12)(8)^3}{12} = 15360\ in^4$$

$$K_{sb} = K_{NF}\frac{E_{cs} I_s}{\ell_1} = \frac{5.99(4.42 \cdot 10^6 \cdot 15360)}{30(12)} = 1130 \cdot 10^6\ in\text{-}lb$$

- Flexural Stiffness of Column Members at both ends K_c

Table A7 $\quad t_a = 18'' \quad t_b = 18'' \quad t_a/t_b = 1.0$
$H = 10'\text{-}10'' = 130'' \quad H_c = 112'' \quad H/H_c = 1.16$

K_{AB} 5.84
C_{AB} 0.604

$$I_c = \frac{c^4}{12} = \frac{24^4}{12} = 27648\ in^4$$

$$E_{cs} = 57000\sqrt{6000} = 4.42 \cdot 10^6$$

$$K_c = 5.84\, E_{cc} I_c/\ell_c = \frac{5.84(4.42 \cdot 10^6)(27648)}{130} = 5490 \cdot 10^6\ in\text{-}lb$$

Two-Way Post-Tensioned Concrete Slab (Existing)

4th Floor Slab | Spot Check | North Building | 3

• Torsional Stiffness of Torsional Members, K_t

$$K_t = \frac{9 E_{cs} C}{[\ell_2 (1 - c_2/\ell_2)^3]} = \frac{9(4.42 \cdot 10^6)(4224.7)}{360(1 - 24/360)^3} = 574 \text{ in-lb}$$

so high due to drop panels

• Equivalent Column Stiffness, K_{ec}

$$K_{ec} = \frac{\Sigma K_c \cdot \Sigma K_t}{\Sigma K_c + \Sigma K_t} = \frac{2(5490 \cdot 10^6)(2)(574 \cdot 10^6)}{(2)(5490 \cdot 10^6) + 2(574 \cdot 10^6)} = 1039 \cdot 10^6 \text{ in-lb}$$

• Slab beam joint Distribution Factors

at exterior joint

$$DF = \frac{1130}{1130 + 1039} = 0.521$$

at interior joint

$$DF = \frac{1130}{1130 + 1130 + 1039} = 0.343$$

$COF = 0.509$

Partial frame analysis of equivalent frame

$w_D = 100$ psf $w_L = 75$ psf $w_{bal} = 75$ psf

• FEM for slab beams

Dead load

$$m_{NF} w_u \ell_2 \ell_1^2 = 0.1093(100)(30)(30)^2 = 295 \text{ ft-K}$$

Live Load

$$m_{NF} = 0.1093(75)(30)(30)^2 = 221 \text{ ft-K}$$

Balanced Load

$$m_{NF} = 0.1093(75)(30)(30)^2 = 221 \text{ ft-K}$$

Refer to Spreadsheet for Moment Distribution

Two-Way Post-Tensioned Concrete Slab (Existing)

Dead Load Moments						
Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.521	0.343	0.343	0.343	0.343	0.521
COF	0.509	0.509	0.509	0.509	0.509	0.509
FEM	295	-295	295	-295	295	-295
DIST	-153.7					153.7
CO		-78.23			78.23	
DIST		128.02			-	
CO			65.16	-65.16		
DIST			-	123.54		
CO			62.88	-62.88		
DIST			-32.01	32.01		
CO		-16.29			16.29	
DIST		5.59			-5.59	
CO	2.84					-2.84
DIST	-1.48					1.48
CO		-0.75			0.75	
DIST		0.26			-0.26	
CO			0.13	-0.13		
DIST			-0.05	0.05		
CO		-0.02			0.02	
DIST		0.01			-0.01	
CO	0.00					0.00
DIST	0.00					0.00
Neg. M	142.7	-256.4	267.6	-267.6	256.4	-142.7
M @ Midspan	137.95		69.91		137.95	

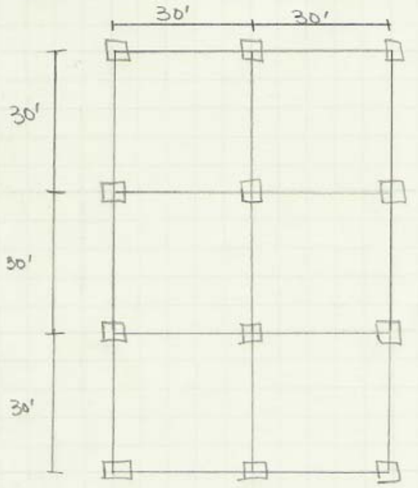
Live Load Moments						
Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.521	0.343	0.343	0.343	0.343	0.521
COF	0.509	0.509	0.509	0.509	0.509	0.509
FEM	221	-221	221	-221	221	-221
DIST	-115.1					115.1
CO		-58.61			58.61	
DIST		95.91			-	
CO			48.82	-48.82		
DIST			-	92.55		
CO			47.11	-47.11		
DIST			-	23.98		
CO		-12.20			12.20	
DIST		4.19			-4.19	
CO	2.13					-2.13
DIST	-1.11					1.11
CO		-0.57			0.57	
DIST		0.19			-0.19	
CO			0.10	-0.10		
DIST			-0.03	0.03		
CO		-0.02			0.02	
DIST		0.01			-0.01	
CO	0.00					0.00
DIST	0.00					0.00
Neg. M	106.9	-192.1	200.5	-200.5	192.1	-106.9
M @ Midspan	103.63		52.66		103.63	

Balanced Load Moments						
Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.521	0.343	0.343	0.343	0.343	0.521
COF	0.509	0.509	0.509	0.509	0.509	0.509
FEM	221	-221	221	-221	221	-221
DIST	-115.1					115.1
CO		-58.61			58.61	
DIST		95.91			-95.91	
CO			48.82	-48.82		
DIST			-92.55	92.55		
CO			47.11	-47.11		
DIST			-23.98	23.98		
CO		-12.20			12.20	
DIST		4.19			-4.19	
CO	2.13					-2.13
DIST	-1.11					1.11
CO		-0.57			0.57	
DIST		0.19			-0.19	
CO			0.10	-0.10		
DIST			-0.03	0.03		
CO		-0.02			0.02	
DIST		0.01			-0.01	
CO	0.00					0.00
DIST	0.00					0.00
Neg. M	106.9	-192.1	200.5	-200.5	192.1	-106.9
M @ Midspan	103.63		52.66		103.63	

Two-Way Post-Tensioned Concrete Slab (Existing)

Slab | Spot Check | North Building | 5

Look at the Slab between column lines 3-5 and NB-ND
on the 4th Floor
- Commercial Occupancy -



Loads

Framing Dead Load = Selfweight
Live Load = 100 psf Commercial

Materials

Concrete: NW 150 pcf
$f'_c = 6000$ psi
$f'_{ci} = 4500$ psi

Rebar: $F_y = 60{,}000$ psi

PT: Unbonded tendons.
$\frac{1}{2}"\phi$, 7wire strands, $A = 0.153 in^2$
$f_{pu} = 270{,}000$ psi
Estimated prestress losses = 15 ksi
$f_{se} = 0.7(270) - 15 = 174$ ksi
$P_{eff} = Af_{se} = 0.153(174) = 27$ k/tendon

Determine Preliminary Slab Thickness

8" (Determined Previously)

Loading
DL = 100 psf
L_o = 100 psf

LL reduction: Interior Bay
$A_T = 30(30) = 900 ft^2$
$K_{LL} = 1$
$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 0.75 L_o = 75$ psf

Design of E-W Interior Frame

- Total Bay width between centerlines = 30'
- No pattern loading

Calculate Section Properties

$A = bh = 360(8) = 2880 in^2$
$S = bh^2/6 = 360(8)^2/6 = 3840 in^3$

Two-Way Post-Tensioned Concrete Slab (Existing)

Slab | Spot Check | North Building | 6

Set Design Parameters

At time of Jacking
$f'_{ci} = 4500$ psi
Compression $= 0.60 f'_{ci} = 2700$ psi
Tension $= 3\sqrt{f'_{ci}} = 201$ psi

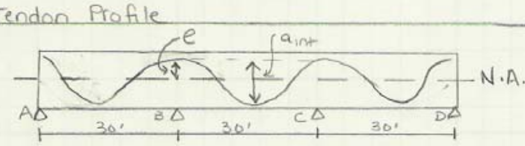
At service loads
$f'_c = 6000$ psi
compression $= 0.45 f'_c = 2700$ psi
Tension $= 6\sqrt{f'_c} = 465$ psi

Average Precompression Limits
$P/A = 125$ psi min
$= 300$ psi max

Target Load Balances
60%-80% of DL for slabs
For this example $0.75\,w_{DL} = 75$ psf

Cover Requirements
3/4" clear cover top and bottom

Tendon Profile



Tendon ordinate	Tendon (CG) location
Interior support - top	7.0"
Interior span - bottom	1.0"

$a_{int} = 7.0" - 1.0" = 6.0"$
Eccentricity, e, is the distance from the center of the tendon to the N.A. It varies along the span.

Prestress Force Required to Balance - 75% of the DL

$w_b = 0.80 w_{DL} = 0.75(100)(30) = 2250$ plf

Force needed in tendons to counteract load

$P = w_b L^2/8a = 2250(30)^2/8(\frac{3.75}{12}) = 810$ K

Two-Way Post-Tensioned Concrete Slab (Existing)

Slab | Spot Check | North Building | 7

Check Precompression Allowance

Determine number of tendons needed

tendons = 810 / 27 = 30

Use 30 tendons

Actual Force for banded tendons

$P_{actual} = 30(27) = 810\ k$

Adjust balanced load

$w_b = (810/810)(2250) = 2.25\ k/ft$

Determine actual precompression stress

$P_{actual}/A = 810(1000)/2880 = 281\ psi$ > 125 psi min ✓, < 300 psi max ✓

$\boxed{P_{eff} = 810\,K}$

Check Slab Stresses

- Stage 1: Stresses Immediately after Jacking (DL + PT)

Midspan Stresses

$f_{top} = (-M_{DL} + M_{bal})/S - P/A$

$f_{bottom} = (+M_{DL} - M_{bal})/S - P/A$

Interior Span

$f_{top} = (-69.91 + 52.66)(12)(1000)/3840 - 281$

$= -335$ psi (compression < $0.6f'_{ci} = 2700$ psi ✓

$f_{bottom} = (69.91 - 52.66)(12)(1000)/3840 - 281$

$= -227$ psi (compression) < $0.6f'_{ci} = 2700$ psi ✓

Exterior Span

$f_{top} = (-137.95 + 103.93)(12)(1000)/3840 - 281$

$= -387$ psi (compression) < $0.6f'_{ci} = 2700$ psi ✓

$f_{bottom} = (137.95 - 103.93)(12)(1000)/3840 - 281$

$= -270$ psi (compression) < $0.6f'_{ci} = 2700$ psi ✓

Two-Way Post-Tensioned Concrete Slab (Existing)

4th Floor Slab | Spot Check | North Building | 8

Support Stresses

$f_{top} = (+M_{DL} - M_{bal})/S - P/A$

$f_{bottom} = (-M_{DL} + M_{bal})/S - P/A$

* used difference of Moment Distribution Moments

$f_{top} = (524.02 - 392.56)(12)(1000)/3840 - 281$

$= 130 \text{ psi (tension)} < 3\sqrt{f'_{ci}} = 201 \text{ psi}$ ✓

$f_{bottom} = (-524.02 + 392.56)(12)(1000)/3840 - 281$

$= -692 \text{ psi (compression)} < 0.6 f'_{ci} = 2700 \text{ psi}$

• Stage 2: Stresses at Service Load (DL + LL + PT)

Midspan Stresses

$f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{bottom} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

Interior Span

$f_{top} = (-69.91 - 52.66 + 52.66)(12)(1000)/3840 - 281$

$= -499 \text{ psi (compression)} < 0.45 f'_c = 2700 \text{ psi}$ ✓

$f_{bottom} = (69.91 + 52.66 - 52.66)(12)(1000)/3840 - 281$

$= -62.5 \text{ psi (compression)} < 0.45 f'_c = 2700 \text{ psi}$ ✓

Exterior Span

$f_{top} = (-137.95 - 103.63 + 103.63)(12)(1000)/3840 - 281$

$= -712 \text{ psi (compression)} < 0.45 f'_c = 2700 \text{ psi}$ ✓

$f_{bottom} = (137.95 + 103.63 - 103.63)(12)(1000)/3840 - 281$

$= 150 \text{ psi (tension)} < 6\sqrt{f'_c} = 465 \text{ psi}$ ✓

Support Stresses

$f_{top} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

$f_{bottom} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{top} = (524.02 + 392.56 - 392.56)(12)(1000)/3840 - 281$

$= 1360 \text{ psi (tension)} > 6\sqrt{f'_c} = 465 \text{ psi} \rightarrow$

$f_{bottom} = (-524.02 - 392.56 + 392.56)(12)(1000)/3840 - 281$

$= -1920 \text{ psi (compression)} < 0.45 f'_c = 2700 \text{ psi}$ ✓

Two-Way Post-Tensioned Concrete Slab (Existing)

4th Floor Slab | Spot Check | North Building | 9

Ultimate Strength

Primary post-tensioning moments, M_1

$M_1 = P * e$
$e = 0$ in. at the exterior support
$e = 3.0$ in at the interior support (N.A. to center of tendon)

$M_1 = (810)(3)/12 = 203$ ft-K

Secondary post-tensioning moments, M_{sec}

$M_{sec} = M_{bal} - M_1$
$= 392.56 - 203 = 190$ ft-K at interior supports
$=$ at midspan

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

At Midspan: $M_u = 1.2(137.95) + 1.6(103.63) + 1.0(95) = 426$ ft-K
At Support: $M_u = 1.2(-524.02) + 1.6(-392.56) + 1.0(190) = -1070$ ft-K

Determine minimum bonded reinforcement

- Positive Moment Region
  Interior Span: $f_t = 15\text{psi} < 2\sqrt{f'_c} = 2\sqrt{6000} = 155\text{psi}$
  no positive reinforcement required

  Exterior Span: $f_t = 180\text{psi} > 155\text{psi}$

Minimum positive moment reinforcement required

$y = f_t/(f_t + f_c)h$
$= [180/(180 + 622)]8 = 1.80$ in

$N_c = M_{DL+LL}/S + 0.5\, y\, l_2$
$= (137.95 + 103.63)(12)/3840 + 0.5(1.80)(30)(12))$
$= 245$ K

$A_{s,min} = N_c/0.5 f_y$
$= 245/0.5(60)$
$= 8.17 \text{ in}^2$

$A_{s,min} = 8.17/30 = 0.272 \text{ in}^2/\text{ft}$

Use #5 @ 12" oc Bottom $= 0.31 \text{ in}^2/\text{ft}$

Two-Way Post-Tensioned Concrete Slab (Existing)

4th Floor Slab | Spot Check | North Building | 10

• Negative Moment Region

$A_{cf} = \max \; 8(30+30)/2\,(12) = 2880$
$\qquad\quad 8(30)/12 = 2880$

Interior Supports

$A_{s\,min} = 0.00075\,A_{cf}$
$= 0.00075(2880)$
$= 2.16\text{ in}^2$

Try 7 #5 top $(A = 2.17\text{ in}^2)$

Exterior Supports

$A_{cf} = \max \; 8(30/2)(12) = 1440$
$\qquad\quad 8(30)(12) = 2880$

Try 7 #5 top $(A = 2.17\text{ in}^2)$

$A_{s\,min} = 0.00075\,A_{cf}$
$= 0.00075(2880)$
$= 2.16\text{ in}^2$

maximum bar spacing

$= 1.5h = 1.5(8) = 12''$

Check that minimum reinforcement is sufficient

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

d = effective depth
$A_{ps} = 0.153(30) = 4.59\text{ in}^2$
$f_{ps} = f_{se} + 10{,}000 + (f'_c bd)/300 A_{ps}$
$a = (A_s f_y + A_{ps} f_{ps})/0.85 f'_c b$

At Supports

$d = 18'' - 3/4'' - 1/4'' = 17''$
$f_{ps} = 174000 + 10000 + (6000(30)(12)(17))/300(4.59)$
$= 210{,}667\text{ psi}$
$a = (2.17(60) + 4.59(211))/0.85(6)(30)(12)$
$= 0.59$

$\phi M_n = 0.9(2.17(60) + 4.59(211))(17 - 0.59/2)$
$= 1376\text{ ft-k} > 1070\text{ ft-k}$ Minimum reinforcement ok

7 #5 @ 12" oc Top at supports

Two-Way Post-Tensioned Concrete Slab (Existing)

At Midspan (end span)

$d = 8 - 1.5 - .25 = 6.25"$
$f_{ps} = 184000 + (6000(30)(12)(6.25))/300(4.59)$
$= 193{,}804$ psi
$a = [9.3(60) + 4.59(195)]/0.85(6)(30)(12) = 0.79$

$\phi M_n = 0.9[9.3(60) + 4.59(195)][6.25 - 0.79/2]/12$
$= 638$ ft-K > 426 ft-K Minimum reinforcement ok

#5 @ 12" oc Bottom at end spans

In Conclusion

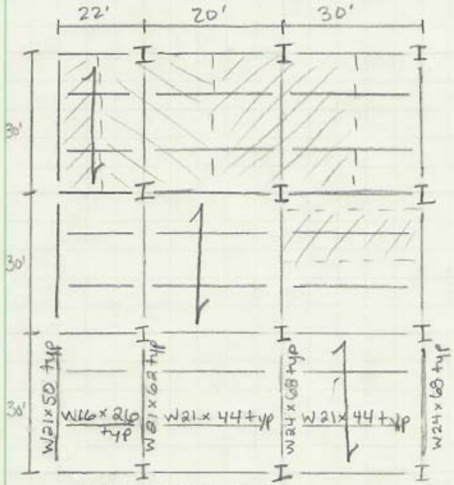
Design: 8" post-tensioned slab with drop panels in the E-W direction, 30 tendons are bundled to give 810 K. In the N-W direction, 30 tendons are uniformly distributed to total 810 K.

Use #5 @ 12" oc Bottom reinforcing at end spans
Use 7#5 @ 12" oc Top reinforcing at supports

APPENDIX B – Composite Steel Deck with Non-Composite Beams

Composite Steel Deck with Non-Composite Beams

Alternative Slab 1 | Composite Steel Deck | Non-Composite Beams



10' beam spacing
Assume fully laterally braced

- Volcraft Steel Roof + Floor Deck Catalog

Loading: 80 psf + 20 psf partitions
Dead load of structure
$w_u = 1.6(100) = 160$ psf

Materials: $f'_c = 3000$ psi
$F_y = 50$ ksi (beams + girders)

2" metal deck Volcraft
18 gauge

5¼" concrete (3¼" above rib)
LWC - dead weight of deck + concrete = 42 psf

recommended welded wire fabric
6×6 - W1.4×W1.4

Superimposed live load
@ 10' clear span = 176 psf > 160 psf ✓

Design the Beams

- Interior beam

Tributary width: worst case 10'
$L_r = 100(0.25 + 15/\sqrt{2(30\times10)}) = 86.2$ psf
$w_u = 1.6(86.2) + 1.2(42) = 188$ psf

$w = 188(10') = 1.88$ K/ft

$M_n = \frac{wl^2}{8} = \frac{1.88(30)^2}{8} = 212$ ft-K $V_n = \frac{wl}{2} = \frac{1.88(30)}{2} = 28.2$ K

Steel Construction Manual - Table 3-2
W18×35 $\phi M_n = 249$ ft-K $> M_n = 212$ ft-K ✓ $\phi V_n = 159$ K $> V_n = 28.2$ K ✓

- Exterior "Cantilevered" Beam

Tributary width: 10'
$w = 1.6(100) + 1.2(42) = 210$ psf
$w = 2.10$ K/ft

$M_n = \frac{wl^2}{8} = \frac{2.10(22)^2}{8} = 127$ ft-K $V_n = \frac{wl}{2} = \frac{2.10(22)}{2} = 23.1$ K

Steel Construction Manual - Table 3-2
W12×26 $\phi M_n = 140$ ft-K $> M_n = 127$ ft-K ✓ $\phi V_n = 84.3$ K $> V_n = 23.1$ K ✓

Composite Steel Deck with Non-Composite Beams

Alternate Slab 1 | Composite Steel Deck | Non-Composite Beams

Check Deflections in Beams

• Interior Beam (worst case span)

$\Delta_{max} = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{5 w l^4}{384 EI} = \frac{5(0.1 \cdot 10)(30')^4 (12)^3}{384(29000)(510)} = 1.23'' > 1''$ max NO!

Try W21×44

$\Delta_{LL} = \frac{5(0.1 \cdot 10)(30')^4 (12)^3}{384(29000)(843)} = 0.75'' < 1''$ ✓

$\Delta_{DL} = \frac{5(0.143 \cdot 10)(30)^4 (12)^3}{384(29000)(843)} = 1.06'' < 1.5''$ ✓

$\Delta_{max} = l/240 = 30(12)/240 = 1.5''$

• Exterior Beam

$\Delta_{max} = l/360 = 22(12)/360 = 0.73''$

$\Delta_{LL} = \frac{5 w l^4}{384 EI} = \frac{5(0.1)(10)(22)^4 (12)^3}{384(29000)(204)} = 0.89'' > 0.73''$ max NO!

Try W16×26

$\Delta_{LL} = \frac{5(0.1 \cdot 10)(22)^4 (12)^3}{384(29000)(301)} = 0.60'' < 0.73''$ ✓

$\Delta_{max} = l/240 = 22(12)/240 = 1.1''$

$\Delta_{DL} = \frac{5 w l^4}{384 EI} = \frac{5(0.142 \cdot 10)(22)^4 (12)^3}{384(29000)(301)} = 0.86'' < 1.1''$ ✓

Composite Steel Deck with Non-Composite Beams

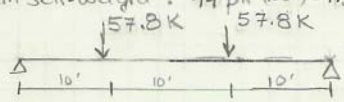
Alternative Slab 1 | Composite Steel Deck | Non-Composite Beams

Design the Girders

- Interior Girder

Tributary width: worst case - 30'

beam self-weight: 44 plf (30') = 1.32 K   1.2(1.32) = 1.44 + 2(28.2) = 57.8 K



$V_n = 57.8$ K

$M_n = Pa = 57.8\ (10) = 578$ ft-K

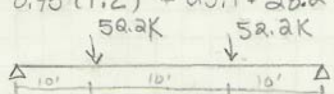
Steel Construction Manual - Table 3-2
W21×68 $\phi M_n = 600$ ft-K > $M_n = 578$ ft-K ✓ $\phi V_n = 273$K > $V_n = 57.8$ K ✓

- Exterior Girder

Tributary width: 11' + 10'

beam self-weight: 44 plf (10) + 26 plf (11) = 0.73 K

0.73 (1.2) + 23.1 + 28.2 = 52.2 K



$V_n = 52.2$K

$M_n = Pa = 52.2\ (10) = 522$ ft-K

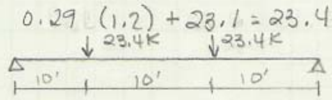
Steel Construction Manual - Table 3-2
W21×62 $\phi M_n = 540$ ft-K > $M_n = 522$ ft-K ✓ $\phi V_n = 252$K > $V_n = 52.2$K ✓

- "Cantilevered" Girder

Tributary width: 11'

beam self-weight: 26 plf (11) = 0.29 K

0.29 (1.2) + 23.1 = 23.4 K



$V_n = 23.4$K
$M_n = Pa = 23.4\ (10) = 234$ ft-K

Steel Construction Manual - Table 3-2
W 18×35 $\phi M_n = 249$ ft-K > $M_n = 234$ ft-K ✓ $\phi V_n = 159$ K > $V_n = 23.4$K ✓

Composite Steel Deck with Non-Composite Beams

Alternative Slab 1 | Composite Steel Deck | Non-Composite Beams

Check Deflections in Girders

• Interior Girder (worst case 30' width)

$\Delta max = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{30(30)^3(12)^3}{28(29000)(1480)} = 1.16'' > 1''$ No!

Try a W24 × 68

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{30(30)^3(12)^3}{28(29000)(1830)} = 0.94'' < 1''$ ✓

$\Delta max = l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{Pl^3}{28EI} = \frac{43.9(30)^3(12)^3}{28(29000)(1830)} = 1.38'' < 1.5''$ ✓

• Exterior Girder

$\Delta max = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{21.9(30)^3(12)^3}{28(29000)(1330)} = 0.95'' < 1''$ ✓

$\Delta max = l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{Pl^3}{28EI} = \frac{30.5(30)^3(12)^3}{28(29000)(1330)} = 1.32'' < 1.5''$ ✓

• "Cantilever" Girder

$\Delta max = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{15(30)^3(12)^3}{28(29000)(510)} = 1.69'' > 1''$ No!

Try a W21×50

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{15(30)^3(12)^3}{28(29000)(984)} = 0.88'' < 1''$ ✓

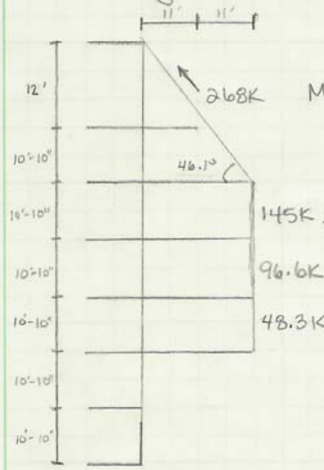
$\Delta max = l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{Pl^3}{28EI} = \frac{15.9(30)^3(12)^3}{28(29000)(984)} = 0.93'' < 1.5''$ ✓

Composite Steel Deck with Non-Composite Beams

Alternative Slab 1	Composite Steel Deck	Non-Composite Beams

Design Steel Hangers for Cantilever



Material: Round HSS, A500
$F_y = 42$ KSI
$F_u = 58$ Ksi

Tensile Loading:
3rd Floor − (23.4 K) 2 girders per rod + 1.5 K
4th Floor − (23.4 K) 2 girders per rod + 1.5 K
5th Floor − (23.4 K) 2 girders per rod + 1.5 K
6th Floor − (23.4 K) 2 girders per rod + 1.5 K
7th Floor − will carry itself. to the column

Total tensile force in top rod
$P_u = 268$ K

Steel Construction Manual − Table 5-7

HSS 6.875 × 0.500

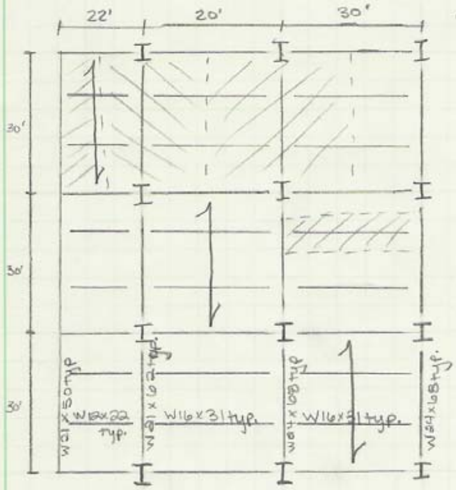
$A_g = 9.36$ in² $\qquad \phi P_n$ yielding $= 354$ K $> P_u = 268$ K ✓
$A_e = 0.75 A_g = 7.02$ in² $\qquad \phi P_n$ rupture $= 305$ K $> P_u = 268$ K ✓

If this alternative system gets used, the columns should be redesigned in steel and the hanger connections should be checked against the $A_e = 7.02$ in²

APPENDIX C – Composite Steel Deck with Composite Beams

Composite Steel Deck with Composite Beams

Alternative Slab 2 | Composite Steel Deck | Composite Beams



10' beam spacing
Assume fully laterally braced

- Volcraft Steel Roof + Floor Deck Catalog
  Loading: 80 psf + 20 psf partitions
  Dead load of Structure
  $w_u = 1.6(100) = 160$ psf

Materials: $f'c = 3000$ psi
$f_y = 50$ ksi (beams + girders)

2" metal deck, Volcraft
18 gauge

5¼" concrete (3¼" above rib)
LWC - dead weight of deck + concrete = 42 psf

recommended welded wire fabric 6×6 - W1.4 × W1.4

Superimposed live load
@ 10' clearspan = 176 psf > 160 psf ✓

Design the Beams

- Interior Beam

Tributary width: worst case 10'

$L_r = 100(0.25 + 15/\sqrt{2(30)(10)}) = 86.2$ psf

$w_u = 1.6(86.2) + 1.2(42) = 188$ psf

$w = 188(10) = 1.88$ K/ft

$M_n = \frac{wl^2}{8} = \frac{1.88(30)^2}{8} = 212$ ft-K $\qquad V_n = \frac{wl}{2} = \frac{1.88(30)}{2} = 28.2$ K

Steel Construction Manual - Table 3-19

assume $a = 1.5"$

$Y_2 = 3\frac{1}{4}" - a/2 = 3\frac{1}{4} - 1.5/2 = 2.5"$

W12×26 PNA location 3 $\phi M_n = 225$ ft-K $> M_n = 212$ ft-K ✓
$\Sigma Q_n = 259$ K

$a = \frac{259}{0.85(3)(90)} = 1.13" < 1.5"$ ✓

Table 3-21
3/4" stud, perpendicular deck, 1 stud weak, 3ksi LWC = 17.2K

# of studs = 259K/17.2K = 16 studs per side = 32 studs

Composite Steel Deck with Composite Beams

Alternative Slab 2 | Composite Steel Deck | Composite Beams

Table 3-19

W14×26 PNA location 6 $\phi M_n = 220$ ft-K $> M_n = 212$ ft-K ✓
$\Sigma Q_n = 135$ K

$a = \frac{135}{0.85(3)(66)} = 0.80'' < 1.5''$ ✓

# of studs $= \frac{135}{17.2} = 8$ studs per side $= 16$ studs

- Exterior "Cantilevered" Beam
  tributary width ≑ 10'
  $w = 1.6(100) + 1.2(42) = 210$ psf
  $w = 210(10) = 2.10$ K/ft

$M_n = \frac{wl^2}{8} = \frac{2.10(22)^2}{8} = 127$ ft-K $V_n = \frac{wl}{2} = \frac{2.10(22)}{2} = 23.1$ K

Steel Construction Manual – Table 3-19
assume $a = 1.5''$
$Y_2 = 3'/4'' - 9/2 = 3'/4'' - 1.5/2 = 2.5''$

W12×19 PNA location 7 $\phi M_n = 130$ ft-K $> M_n = 127$ ft-K ✓
$\Sigma Q_n = 69.7$K

$a = \frac{69.7}{0.85(3)(66)} = 0.41'' < 1.5''$ ✓

Table 3-21
$Q_n = 17.2$K

# of studs $= 69.7/17.2 = 5$ studs per side $= 10$ studs

Composite Steel Deck with Composite Beams

Alternative Slab2 | Composite Steel Deck | Composite Beams

Check Deflections in Beams

- Interior Beam (worst case span)

Table 3-20
$I = 424 \text{ in}^4$

$\Delta_{max} = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1)(30)^4(12)^3}{384(29000)(424)} = 1.48'' > 1''$ No!

$I_{req} = 629 \text{ in}^4$

Try W16×31 PNA location 6 $\phi M_n = 294 \text{ ft-K} > M_n = 212 \text{ ft-K}$
$\Sigma Q_n = 164 \text{ K}$

$a = \frac{164}{0.85(3)(66)} = 0.97'' < 1.5''$ ✓

# of studs $= 164/17.2 = 10$ studs per side
$= 20$ studs

$\Delta_{LL} = \frac{5(1)(30)^4(12)^3}{384(29000)(638)} = 0.98'' < 1''$ ✓

$\Delta_{max} = l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{5(1.42)(30)^4(12)^3}{384(29000)(638)} = 1.40'' < 1.5''$ ✓

- Exterior "Cantilevered" Beam

Table 3-20
$I = 212 \text{ in}^4$

$\Delta_{max} = l/360 = 22(12)(360) = 0.73''$

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1)(22)^4(12)^3}{384(29000)(212)} = 0.86'' > 0.73''$ No!

$I_{req} = 249 \text{ in}^2$

Try W12×22 PNA location 7 $\phi M_n = 153 \text{ ft-K} > M_n = 127 \text{ ft-K}$
$\Sigma Q_n = 81.0$

$a = \frac{81.0}{0.85(3)(66)} = 0.48'' < 1.5''$ ✓

# of studs $= 81/17.2 = 5$ studs per side
$= 10$ studs

$\Delta_{LL} = \frac{5(1)(22)^4(12)^3}{384(29000)(253)} = 0.72'' < 0.73''$ ✓

$\Delta_{max} = l/240 = 22(12)/240 = 1.1''$

$\Delta_{D+L} = \frac{5(1.42)(22)^4(12)^3}{384(29000)(253)} = 1.02'' < 1.1''$ ✓

Composite Steel Deck with Composite Beams

Alternative Slab 2 | Composite Steel Deck | Composite Beams

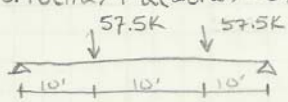
Design the Girders

Interior Girder

Tributary width: worst case 30'

beam self-weight: 31 plf (30) = 093

0.93(1.2) + 2(28.2) = 57.5K



$V_n = 57.5K$

$M_n = Pa = 57.5(10) = 575$ ft-K

Steel Construction Manual - Table 3-2

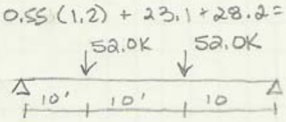
W21x68 $\phi M_n = 600$ ft-K > $M_n = 575$ ft-K ✓ $\phi V_n = 273K > V_n = 57.5K$ ✓

Exterior Girder

Tributary width: 11' + 10'

beam self-weight: 31 plf (10) + 22 plf (11) = 0.552K

0.55 (1.2) + 23.1 + 28.2 = 52.0K



$V_n = 52.0K$

$M_n = Pa = 52.0(10) = 520$ K

Steel Construction Manual - Table 3-2

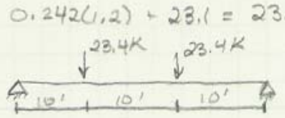
W21x62 $\phi M_n = 540$ ft-K > $M_n = 520K$ ✓ $\phi V_n = 252K > V_n = 52.0K$ ✓

"Cantilevered" Girder

Tributary width: 11'

beam self-weight: 22 plf (11) = 0.242 K

0.242(1.2) + 23.1 = 23.4K



$V_n = 23.4K$

$M_n = Pa = 23.4(10) = 234$ ft-K

Steel Construction Manual - Table 3-2

W18x35 $\phi M_n = 249$ ft-K > $M_n = 234$ ft-K ✓ $\phi V_n = 154K > V_n = 23.4K$ ✓

Composite Steel Deck with Composite Beams

Alternative Slab 2 | Composite Steel Deck | Composite Beams

Check Deflections in Girders

Interior Girder (worst case 30')

$\Delta_{max} = \ell/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{P\ell^3}{28EI} = \frac{30(30)^3(12)^3}{28(29000)(1480)} = 1.16'' > 1''$ No!

Try a W24x68

$\Delta_{LL} = \frac{P\ell^3}{28EI} = \frac{30(30)^3(12)^3}{28(29000)(1830)} = 0.94'' < 1''$ ✓

$\Delta_{max} = \ell/240 = 30(12)/(240) = 1.5''$

$\Delta_{D+L} = \frac{P\ell^3}{28EI} = \frac{43.9(30)^3(12)^3}{28(29000)(1830)} = 1.38'' < 1.5''$ ✓

Exterior Girder

$\Delta_{max} = \ell/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{P\ell^3}{28EI} = \frac{21.9(30)^3(12)^3}{28(29000)(1330)} = 0.95'' < 1''$ ✓

$\Delta_{max} = \ell/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{P\ell^3}{28EI} = \frac{30.5(30)^3(12)^3}{28(29000)(1330)} = 1.32'' < 1.5''$ ✓

"Cantilevered" Girder

$\Delta_{max} = \ell/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{P\ell^3}{8.8EI} = \frac{15(30)^3(12)^3}{28(29000)(510)} = 1.69'' > 1''$ No!

Try W21x50

$\Delta_{LL} = \frac{P\ell^3}{28EI} = \frac{15(30)^3(12)^3}{28(29000)(984)} = 0.88'' < 1''$ ✓

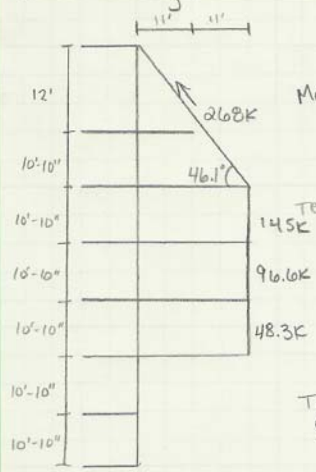
$\Delta_{max} = \ell/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{P\ell^3}{28EI} = \frac{15.9(30)^3(12)^3}{28(29000)(984)} = 0.93'' < 1.5''$ ✓

Composite Steel Deck with Composite Beams

Alternative Slab 2 | Composite Steel Deck | Composite Beams

Design Steel Hangers for Cantilevers



Material: Round HSS, A500
$F_y = 42ksi$
$F_u = 58ksi$

Tensile Loading:
3rd Floor – (23.4K) 2 girders per rod + 1.5K
4th Floor – (23.4K) 2 girders per rod + 1.5K
5th Floor – (23.4K) 2 girders per rod + 1.5K
6th Floor – (23.4K) 2 girders per rod + 1.5K
7th Floor – will carry itself to the column

Total tensile force in top rod
$P_u = 268K$

Steel Construction Manual – Table 5-7

HSS6.875 × 0.500

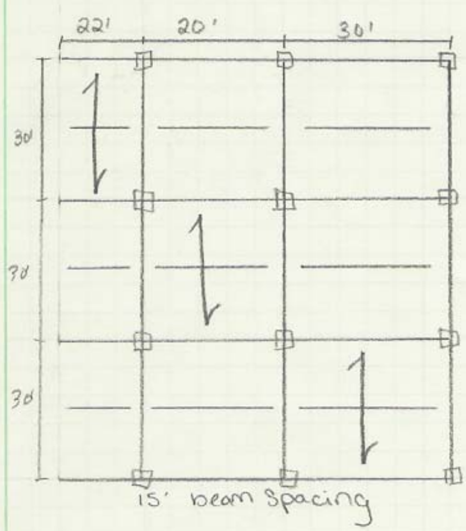
$A_g = 9.36in^2$ | ϕP_n yielding $= 354K > P_u = 268K$ ✓
$A_e = 0.75A_g = 7.02in^2$ | ϕP_n rupture $= 305K > P_u = 268K$ ✓

If this alternative system gets used, the columns should be redesigned in steel and the hanger connections should be checked against the $A_e = 7.02in^2$

APPENDIX D – Pre-Cast Hollow Core Slab on Pre-Cast Beams

Pre-Cast Hollow Core Slab on Pre-Cast Beams

Alternative Slab3 | Precast Hollow Core Slab | Pre-Cast Beams



Loading: 80 psf + 20 psf partitions
Dead load of Structure
$w_u = 1.6(100) = 160$ psf

Materials: Pre-Cast Hollow Core Slab
Pre-Cast Beams

Design the Slab

PCI Handbook p. 2-31
4'-0" × 6" w/ 2" NWC Topping
Strand designation 66-S
15' span, 285 psf service load

Use Manufacterer in Milwaukee, Wisconsin - Spancrete Machinery Corp.
p. 2-36

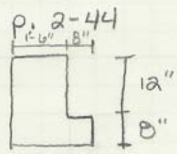
4'-0" × 6" $A = 189 in^2$
$y_b = 4.19 in$
$I = 1,760 in^4$
$w_t = 71$ psf

need 7.5 for the 30' spans
5 for the 20' spans
5.5 for the 22' spans

Design the Edge Beams

$w_u = 1.2(71 \text{ psf})(7.5') + (160 \text{ psf})(7.5') = 759$ plf

p. 2-44



26LB20 Designation
158-S No. Strand
30' span
2480 plf Service load
442 plf weight

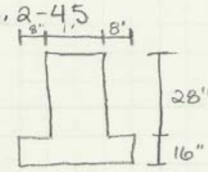
Pre-Cast Hollow Core Slab on Pre-Cast Beams

Alternative Slab3 | Pre-Cast Hollow Core Slab | Pre-Cast Beams

Design the Interior Beams

$w_u = 1.2(71\ psf)(30') + (160\ psf)(30') = 7356\ plf$

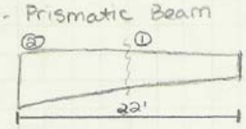
p. 2-45



28IT40 Designation
198-S No. Strand
30' span
7440 plf Service Load
817 plf weight

Design the "Cantilevered" Beams

Non-Prismatic Beam



$w_u = 1.2(71\ psf)(30') + (160\ psf)(30') = 7356\ plf$

$Mu_1 = \frac{w\ell^2}{2} = \frac{(7356)(11)^2}{2} = 439\ ft\text{-}K$

$Mu_2 = \frac{w\ell^2}{2} = \frac{7356(22)^2}{2} = 1754\ ft\text{-}K$

• Estimate size ①

$20M_u = bd^2$
$20(439) = 24d^2$
$d = 19.1''$

• Estimate size ②

$20M_u = bd^2$
$20(1754) = 24d^2$
$d = 38.2''$

not considered a deep beam $21' = \ell n > 4h = 4(40) = 13.3'$

• Compute Asreq ①

$A_s = \frac{M_u}{4d} = \frac{439}{4(19.1)} = 5.75\ in^2$ (6)#9

• Compute As req ②

$A_s = \frac{M_u}{4d} = \frac{1754}{4(38.2)} = 11.5\ in^2$ (12)#9

Assume $\varepsilon_s > \varepsilon_y$

$a = \frac{A_s f_y}{f'_c \beta_1 b} = \frac{6(60)}{6(0.75)(24)} = 3.33''$

$a = \frac{A_s f_y}{f'_c \beta_1 b} = \frac{12(60)}{6(0.75)(24)} = 6.67''$

$c = a/\beta_1 = 3.33/0.75 = 4.44''$

$c = a/\beta_1 = 6.67/0.75 = 8.89''$

$\varepsilon_s = \frac{\varepsilon_u(d-c)}{c} = \frac{0.003(19.1-4.44)}{4.44} = 0.00991$

$\phi = 0.9$

$\varepsilon_s = \frac{0.003(38.2-8.89)}{8.89} = 0.00989$

$\phi = 0.9$

Pre-Cast Hollow Core Slab on Pre-Cast Beams

Alternative Slab 3 | Pre-Cast Hollow Core Slab | Pre-Cast Beams

1,128 ϕ

• ϕM_n at ①

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.9(6)(60)(19.1 - 3.33/2)$
$= 471\ ft\text{-}K > M_u = 439\ ft\text{-}K$ ✓

• ϕM_n at ②

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.9(12)(60)(38.2 - 6.67/2)$
$= 1883\ ft\text{-}K > M_u = 1754\ ft\text{-}K$ ✓

• $A_{s,min}$ ①

$A_{s,min} = \begin{cases} \dfrac{3\sqrt{f'c}\,bd}{f_y} = \dfrac{3\sqrt{6000}(24)(19.1)}{60000} = 1.78 in^2 \leftarrow \\ \dfrac{200bd}{f_y} = \dfrac{200(24)(19.1)}{60000} = 1.53 in^2 \end{cases}$

$A_{s,min} < A_s$ ✓

• $A_{s,min}$ ②

$A_{s,min} = \begin{cases} \dfrac{3\sqrt{6000}(24)(38.2)}{60000} = 3.55 in^2 \leftarrow \\ \dfrac{200(24)(38.2)}{60000} = 3.06 in^2 \end{cases}$

$A_{s,min} < A_s$ ✓

• $A_{s,max}$ ①

• $A_{s,max}$ ②

$\rho_{max} = 0.85\beta_1 (f'c/f_y)(\epsilon_u/(\epsilon_u + 0.004))$
$= 0.85(0.75)(6/60)(0.003/0.007)$
$= 0.02732$

$A_{s,max} = \rho_{max} bd$
$= 0.02732(24)(19.1)$
$= 12.5 in^2$

$A_s < A_{s,max}$ ✓

$A_{s,max} = \rho_{max} bd$
$= 0.02732(24)(38.2)$
$= 25.0 in^2$

$A_s < A_{s,max}$ ✓

use (12) #9 bars at the top of the beam

• Shear Reinforcing

$V_c = 2\lambda\sqrt{f'c}\,b_w d = 2(1)\sqrt{6000}(24)(38.2) = 142K$ @ column
$= 2(1)\sqrt{6000}(24)(19.1) = 71.0\ K$ @ midspan

$\phi V_n = 0.5(0.75)(142) = 53.3K$ @ column
$= 0.5(0.75)(71.0) = 26.6K$ @ midspan

$V_u = 159\ K$ @ column
$= 79.7$ @ midspan
} need shear reinforcing throughout the entire beam

Pre-Cast Hollow Core Slab on Pre-Cast Beams

Alternative Slab 3 | Pre-Cast Hollow Core Slab | Pre-Cast Beams

$V_s = V_u/\phi - V_c = 159/0.75 - 53.3 = 159K \leq 8\sqrt{6000}(24)(38.2) = 568K$ ✓

$V_s \leq 4\sqrt{6000}(24)(38.2) = 284K$ ✓

$S_{max} = \begin{cases} d/2 = 38.2/2 = 19.1'' \leftarrow \\ 24'' \end{cases}$ Use 19″

$A_{v\,min} = \begin{cases} 0.75\sqrt{6000}(24)(19)/60000 = 0.44\ in^2 \leftarrow \\ 50(24)(19)/60000 = 0.38\ in^2 \end{cases}$

Use #3 stirrups @ 19″ (4 legs $A_v = 0.44 in^2$)

$S = A_v f_{yt} d / V_s = 0.44(60)(38.2)/159 = 6.34''$ Use 6″

Use (4) #3 stirrups : 1 @ 2″, 41 @ 6″

Pre-Cast Hollow Core Slab on Pre-Cast Beams

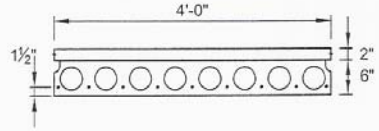
Strand Pattern Designation
76-S
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

		Untopped		Topped	
A	=	187	in.^{2}	283	in.^{2}
I	=	763	in.^{4}	1,640	in.^{4}
y_b	=	3.00	in.	4.14	in.
y_t	=	3.00	in.	3.86	in.
S_b	=	254	in.^{3}	396	in.^{3}
S_t	=	254	in.^{3}	425	in.^{3}
wt	=	195	plf	295	plf
DL	=	49	psf	74	psf
V/S	=	1.73	in.		

4HC6

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	Span, ft																				
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.3	−0.5	−0.7	−0.9	−1.2	−1.5	−1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	−0.1	−0.3	−0.4	−0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.7	−0.9	−1.2	−1.6	−2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	−0.1	−0.3	−0.6	−0.9	−1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	−0.3	−0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	−0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	−0.0	−0.1	−0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2	−1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	−0.1	−0.3	−0.5	−0.7	−1.0	−1.4	−1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	−0.1	−0.3	−0.5	−0.8	−1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	−0.2	−0.5	−0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

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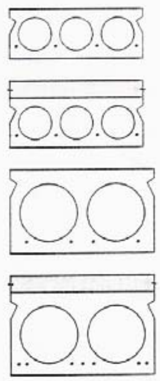
Pre-Cast Hollow Core Slab on Pre-Cast Beams

HOLLOW-CORE SLABS

Figure 2.5.3 Section Properties – Normal Weight Concrete **Flexicore**

Trade Name: Flexicore ®
Licensing Organization: The Flexicore Co. Inc., Dayton, Ohio



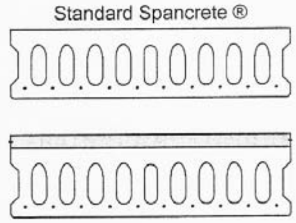
Section	Untopped				With 2 in. topping		
width x depth	A in.^{2}	y_b in.	I in.^{4}	wt psf	y_b in.	I in.^{4}	wt psf
1'-4" x 6"	55	3.00	243	43	4.23	523	68
2'-0" x 6"	86	3.00	366	45	4.20	793	70
1'-4" x 8"	73	4.00	560	57	5.26	1,028	82
2'-0" x 8"	110	4.00	843	57	5.26	1,547	82
1'-8" x 10"	98	5.00	1,254	61	6.43	2,109	86
2'-0" x 10"	138	5.00	1,587	72	6.27	2,651	97
2'-0" x 12"	141	6.00	2,595	73	7.46	4,049	98

Note: All sections are not available from all producers. Check availability with local manufacturers.

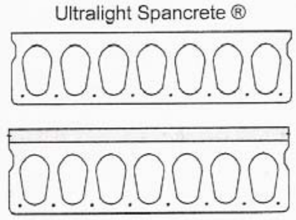
Figure 2.5.4 Section Properties – Normal Weight Concrete **Spancrete**

Trade Name: Spancrete ®
Licensing Organization: Spancrete Machinery Corp., Milwaukee, Wisconsin

Standard Spancrete ®



Ultralight Spancrete ®



Section	Untopped				With 2 in. topping		
width x depth	A in.^{2}	y_b in.	I in.^{4}	wt psf	y_b in.	I in.^{4}	wt psf
4'-0" x 4"	138	2.00	238	34	3.14	739	59
4'-0" x 6"	189	2.93	762	46	4.19	1,760	71
4'-0" x 8"	258	3.98	1,806	63	5.22	3,443	88
4'-0" x 10"	312	5.16	3,484	76	6.41	5,787	101
4'-0" x 12"	355	6.28	5,784	86	7.58	8,904	111
4'-0" x 15"	370	7.87	9,765	90	9.39	14,351	115

4'-0" x 8"	246	4.17	1,730	60	5.41	3,230	85
4'-0" x 10"	277	5.22	3,178	67	6.58	5,376	92
4'-0" x 12"	316	6.22	5,311	77	7.66	8,410	102

Note: Spancrete is also available in 40 in. and 96 in. widths. All sections are not available from all producers. Check availability with local manufacturers.

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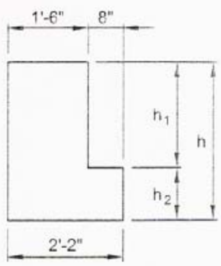
Pre-Cast Hollow Core Slab on Pre-Cast Beams

L-BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Section Properties								
Designation	h in.	h_1/h_2 in./in.	A in.^{2}	I in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}	wt plf
26LB20	20	12/8	424	14,298	9.09	1,573	1,311	442
26LB24	24	12/12	528	24,716	10.91	2,265	1,888	550
26LB28	28	16/12	600	39,241	12.72	3,085	2,568	625
26LB32	32	20/12	672	58,533	14.57	4,017	3,358	700
26LB36	36	24/12	744	83,176	16.45	5,056	4,255	775
26LB40	40	24/16	848	114,381	18.19	6,288	5,244	883
26LB44	44	28/16	920	152,104	20.05	7,586	6,351	958
26LB48	48	32/16	992	197,159	21.94	8,986	7,566	1,033
26LB52	52	36/16	1,064	250,126	23.83	10,496	8,879	1,108
26LB56	56	40/16	1,136	311,586	25.75	12,100	10,300	1,183
26LB60	60	44/16	1,208	382,118	27.67	13,810	11,819	1,258

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key

9672 – Safe superimposed service load, plf.
0.4 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Designation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
26LB20	158-S	2.67 2.67	9672 0.4 0.2	7563 0.5 0.3	6054 0.6 0.3	4938 0.7 0.4	4089 0.8 0.4	3428 1.0 0.5	2903 1.1 0.5	2480 1.2 0.6	2134 1.4 0.6	1847 1.5 0.7	1607 1.6 0.7	1403 1.7 0.7	1230 1.8 0.7	1080 1.9 0.7	950 1.9 0.6			
26LB24	158-S	2.67 2.67			9165 0.5 0.2	7493 0.5 0.2	6221 0.6 0.2	5231 0.7 0.2	4445 0.8 0.3	3811 0.9 0.3	3293 1.0 0.3	2863 1.1 0.3	2503 1.2 0.3	2198 1.3 0.3	1938 1.3 0.3	1714 1.4 0.2	1520 1.5 0.2	1350 1.5 0.1	1202 1.5 0.1	1070 1.5 0.0
26LB28	188-S	3.33 3.33					8437 0.6 0.2	7170 0.6 0.2	6056 0.7 0.3	5207 0.8 0.3	4511 0.9 0.3	3935 1.0 0.3	3452 1.1 0.3	3043 1.2 0.3	2694 1.3 0.3	2394 1.3 0.3	2134 1.4 0.3	1907 1.5 0.3	1707 1.5 0.2	1532 1.6 0.2
26LB32	218-S	4.00 4.00						9265 0.6 0.2	7906 0.7 0.3	6809 0.7 0.3	5912 0.8 0.3	5169 0.9 0.3	4545 1.0 0.4	4018 1.1 0.4	3568 1.2 0.4	3180 1.2 0.4	2844 1.3 0.4	2551 1.4 0.4	2294 1.5 0.3	2067 1.5 0.3
26LB36	248-S	4.50 4.50								8722 0.7 0.3	7585 0.8 0.3	6643 0.9 0.3	5854 0.9 0.4	5186 1.0 0.4	4615 1.1 0.4	4125 1.2 0.4	3699 1.3 0.4	3328 1.3 0.4	3002 1.4 0.4	2715 1.5 0.4
26LB40	278-S	5.11 5.11									9372 0.7 0.3	8216 0.8 0.3	7246 0.9 0.3	6426 0.9 0.4	5726 1.0 0.4	5123 1.1 0.4	4601 1.2 0.4	4145 1.2 0.4	3745 1.3 0.4	3392 1.4 0.4
26LB44	288-S	5.29 5.29											8992 0.8 0.3	7986 0.8 0.3	7127 0.9 0.3	6388 1.0 0.3	5748 1.0 0.3	5189 1.1 0.3	4698 1.2 0.3	4266 1.2 0.3
26LB48	328-S	5.75 5.75												9635 0.8 0.3	8609 0.9 0.4	7726 1.0 0.4	6961 1.0 0.4	6294 1.1 0.4	5708 1.2 0.4	5191 1.3 0.4
26LB52	358-S	6.29 6.29														9137 0.9 0.4	8241 1.0 0.4	7459 1.1 0.4	6773 1.1 0.4	6167 1.2 0.5
26LB56	378-S	7.00 7.00															9539 0.9 0.4	8641 1.0 0.4	7853 1.1 0.4	7158 1.1 0.4
26LB60	388-S	7.68 7.68																9904 0.9 0.3	9008 0.9 0.3	8217 1.0 0.3

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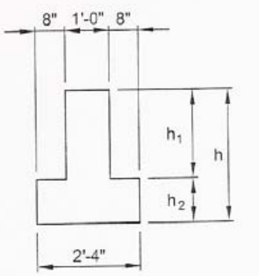
Pre-Cast Hollow Core Slab on Pre-Cast Beams

INVERTED TEE BEAMS



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Normal Weight Concrete

Section Properties								
Designation	h in.	h_1/h_2 in./in.	A $in.^2$	I $in.^4$	y_b in.	S_b $in.^3$	S_t $in.^3$	wt plf
28IT20	20	12/8	368	11,688	7.91	1,478	967	383
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	500
28IT28	28	16/12	528	32,076	11.09	2,892	1,897	550
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	600
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	650
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	767
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	817
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	867
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,017

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key
6511 – Safe superimposed service load, plf.
0.2 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	98-S	2.44 2.44	6511 0.2 0.1	5076 0.3 0.1	4049 0.4 0.1	3289 0.4 0.1	2711 0.5 0.1	2262 0.5 0.1	1905 0.6 0.0	1617 0.7 0.0	1381 0.7 0.0	1186 0.7 0.0	1022 0.8 –0.1							
28IT24	188-S	2.73 2.73	9612 0.2 0.1	7504 0.3 0.1	5997 0.3 0.1	4882 0.4 0.1	4034 0.4 0.1	3374 0.5 0.1	2850 0.6 0.1	2427 0.6 0.1	2081 0.7 0.1	1795 0.7 0.1	1555 0.7 0.0	1351 0.8 0.0	1178 0.8 –0.1	1029 0.8 –0.2				
28IT28	138-S	3.08 3.08			8353 0.3 0.1	6822 0.3 0.1	5657 0.4 0.1	4750 0.5 0.1	4031 0.5 0.1	3451 0.6 0.1	2976 0.6 0.1	2582 0.7 0.1	2252 0.7 0.1	1973 0.8 0.1	1735 0.8 0.0	1530 0.8 0.0	1352 0.9 –0.1	1197 0.8 –0.2	1061 0.8 –0.2	
28IT32	158-S	3.47 3.47				9049 0.3 0.1	7521 0.4 0.1	5333 0.4 0.1	5389 0.5 0.1	4628 0.5 0.1	4006 0.6 0.1	3490 0.6 0.1	3057 0.7 0.1	2691 0.7 0.1	2379 0.8 0.1	2110 0.8 0.1	1876 0.9 0.0	1673 0.9 0.0	1495 0.9 0.0	1337 0.9 –0.1
28IT36	168-S	3.50 3.50					9832 0.3 0.1	8295 0.4 0.1	7075 0.4 0.1	6092 0.5 0.1	5287 0.5 0.1	4619 0.6 0.1	4060 0.6 0.1	3587 0.7 0.1	3183 0.7 0.1	2835 0.8 0.1	2534 0.8 0.0	2271 0.9 0.0	2040 0.9 0.0	1836 0.9 –0.1
28IT40	198-S	4.21 4.21							8638 0.4 0.1	7440 0.5 0.1	6460 0.5 0.1	5647 0.6 0.1	4966 0.6 0.1	4390 0.7 0.1	3898 0.7 0.1	3474 0.8 0.1	3107 0.8 0.1	2787 0.8 0.1	2506 0.9 0.1	2258 0.9 0.1
28IT44	208-S	4.40 4.40								9186 0.4 0.1	7989 0.5 0.1	6997 0.5 0.1	6165 0.6 0.1	5462 0.6 0.1	4861 0.7 0.1	4344 0.7 0.1	3896 0.7 0.1	3505 0.8 0.1	3162 0.8 0.1	2859 0.8 0.0
28IT48	228-S	4.55 4.55									9719 0.4 0.1	8525 0.5 0.1	7523 0.5 0.1	6676 0.6 0.1	5953 0.6 0.1	5330 0.7 0.1	4791 0.7 0.1	4320 0.8 0.1	3907 0.8 0.1	3542 0.9 0.1
28IT52	248-S	5.17 5.17										9987 0.5 0.1	8823 0.5 0.1	7838 0.6 0.1	6998 0.6 0.1	6274 0.6 0.1	5647 0.7 0.1	4100 0.7 0.1	4619 0.8 0.1	4196 0.8 0.1
28IT56	268-S	5.23 5.23												9307 0.5 0.2	8319 0.6 0.2	7469 0.6 0.2	6731 0.7 0.2	6088 0.7 0.2	5524 0.8 0.2	5026 0.8 0.2
28IT60	288-S	5.57 5.57													9645 0.6 0.2	8668 0.6 0.2	7820 0.7 0.2	7081 0.7 0.2	6432 0.8 0.2	5859 0.8 0.2

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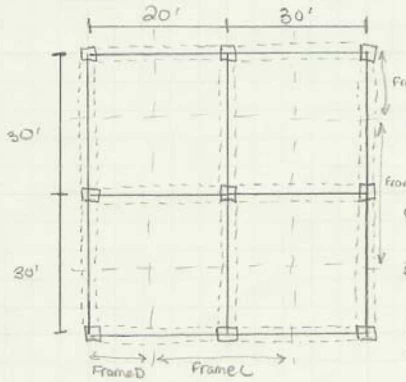
APPENDIX E – Two-Way Reinforced Concrete Slab

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-way RC Slab | Concrete Beams



Loading: 80psf + 20 psf partitions
Dead Load of Structure

Materials: $f'c = 6000psi$
$f_y = 60ksi$ (rebar)

Columns: 24" x 24"

Beams: 24" x 20"

Direct Design Method
Take l_1 as 30' and l_2 as 30'

Use the Cantilever Design from Slab 3

Determine Slab Thickness

$h = \ell n/33 = 28(12)/33 = 10.2$ use 10.5"

• Determine α

Interior Beam

$b_e = b_w + 2h_w = 24 + 2(9.5) = 43" < b_w + 8t = 108"$ ✓

$b_e/b_w = 1.79$

$t/h = 10.5/20 = 0.525$

$K = 1.3379$

$$I_b = 1.3379\left(\frac{24(20)^3}{12}\right) = 21406 in^4$$

$$I_s = \frac{\ell_2 t^3}{12} = \frac{30(12)(10.5)^3}{12} = 34729 in^4$$

$$\alpha = \frac{21406}{34729} = 0.616$$

Edge Beam

$b_e = b_w + h_w = 24 + 9.5 = 33.5" < b_w + 4t = 66"$

$b_e/b_w = 1.40$

$t/h = 10.5/20 = 0.525$

$K = 1.2448$

$$I_b = 1.2448\left(\frac{24(20)^3}{12}\right) = 19917 in^4$$

$$I_s = \frac{\ell_2 t^3}{12} = \frac{15(12)(10.5)^3}{12} = 17364 in^4$$

$$\alpha = \frac{19917}{17364} = 1.15$$

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-way RC Slab | Concrete Beams

• Check the thickness

$$\alpha_m \text{ Panel } 1 = \frac{1.15+1.15+0.616+0.616}{4} = 0.883$$

$$\alpha_m \text{ Panel } 2 = \frac{1.15+0.616+0.616+0.616}{4} = 0.750$$

$$\alpha_m \text{ Panel } 3 = \frac{0.616+0.616+0.616+0.616}{4} = 0.616$$

$$\alpha_m \text{ Panel } 4 = \frac{1.15+0.616+0.616+0.616}{4} = 0.750$$

$$t_{min} = \frac{l_n(0.8 + f_y/200{,}000)}{36+5\beta(\alpha_m - 0.2)} \qquad \beta = l_n/s_n = \left(\frac{30-2}{30-2}\right) = 1$$

$$= \frac{30(0.8 + 60000/200{,}000)}{36+5(1)(0.616-0.2)}$$

$$= 10.4'' < 10.5'' \text{ assumed } \checkmark$$

Deflection controlled through table 9.5(c) ✓

Distribute the Moments

$$w_u = 1.2(10.5/12 \cdot 150) + 1.6(100) = 318 \text{ psf}$$

Frame A $M_o = 1/8(0.318)(30)(30-2)^2 = 935 \text{ ft-K}$

Frame B $M_o = 1/8(0.318)(15)(30-2)^2 = 467 \text{ ft-K}$

Frame C $M_o = 1/8(0.318)(30)(30-2)^2 = 935 \text{ ft-K}$

Frame D $M_o = 1/8(0.318)(15)(30-2)^2 = 467 \text{ ft-K}$

Interior Span

Negative Factored Moment	0.65
Positive Factored Moment	0.35

Exterior Span

Interior Negative Factored Moment	0.70
Positive Factored Moment	0.57
Exterior Negative Factor Moment	0.16

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-way RC Slab | Concrete Beams

Frame A

-150	+533	-655	-608	+327

Frame B

-74.7	+266	-327	-304	+163

Frame C

-150	+533	-655	-608	+327

Frame D

-74.7	+266	-327	-304	+163

Item	Description	Frame A	Frame B	Frame C	Frame D
1	Total width	360"	180"	360"	180"
2	CS width	180"	90"	180"	90"
3	MS width	2@90"	1@90"	2@90"	1@90"
4	Torsional Constant	19189	15523	19189	15523
5	Slab $I_s = \ell t^3/12$	34729	34729	34729	34729
6	$\beta_t = C/2I_s$	0.28	0.22	0.28	0.22
7	$\alpha_1 = I_b/I_s$	0.62	1.15	0.62	1.15
8	ℓ_2/ℓ_1	1	1	1	1
9	$\alpha \ell_2/\ell_1$	0.62	1.15	0.62	1.15
10	Exterior -M to CS	97.2%	97.8	97.2%	97.8
11	+M to CS	69.3%	75%	69.3%	75%
12	Interior -M to CS	75%	75%	75%	75%

Frame A

Total Moment	-150	+533	-655	-608	+327
% in CS	97.2%	69.3%	75%	75%	69.3%
M in CS	-146	+369	-499	-456	+227
M in MS	-4	+164	-156	-152	+100

Frame B

Total Moment	-74.7	+266	-327	-304	+163
% in CS	97.8%	75%	75%	75%	75%
M in CS	-73.0	+200	-245	-228	+122
M in MS	-1.7	+66	-82	-76	+41

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-way RC Slab | Concrete Beams

Frame C

Total Moment	-150	+533	-655	-608	+327
% in CS	97.2%	69.3%	75%	75%	69.3%
M in CS	-146	+369	-499	-456	+227
M in MS	-4	+164	-156	-152	+100

Frame D

Total Moment	-74.7	+266	-327	-304	+163
% in CS	97.8%	75%	75%	75%	75%
M in CS	-73.0	+200	-245	-228	+122
M in MS	-1.7	+66	-82	-76	+41

Design Frame A and Frame C Reinforcing

• Design Reinforcing in CS

Assume # 8 bars, 3/4" clear cover

$d_{short} = 10.5 - 3/4 - 1/2(1.00) = 9.25"$

$d_{long} = 9.25 - 1.00 = 8.25$

Item	Description	ext. span M_{ext}^-	ext. span M^+	ext. span M_{int}^-	int. span M^-	int. span M^+
1	M in CS	-150	+533	-655	-608	+327
2	CS width	180"	180"	180"	180"	180"
3	effective depth	8.25"	8.25"	8.25"	8.25"	8.25"
4	$M_n = M_u/\phi$	-167	+592	-728	-676	+363
5	$R = M_u/\phi bd^2$	163	580	713	662	356
6	ρ from table A5a	0.0028	0.0103	0.0129	0.0119	0.0062
7	$A_s = \rho bd$	4.16	15.3	19.2	17.7	9.21
8	$A_{smin} = 0.002bt$	3.78	3.78	3.78	3.78	3.78
9	$n = \frac{\text{larger of 7 or 8}}{0.79}$	6	20	25	23	12
10	$n_{min} = \frac{\text{width of strip}}{2t}$	9	9	9	9	9

• check d_{min}

$f'_c = 6000$ psi $f_y = 60{,}000$ psi

$\rho_{max} = 0.0273$

$M_u = \phi \rho f_y b d^2 (1 - 0.59\, \rho f_y / f'_c)$

$$d^2 = \frac{M_u}{0.9(0.0273)(60000)(180)(1-0.59\; 0.0273(60000)/6000)} = \frac{M_u}{222615}$$

$$d_{min} = \sqrt{\frac{655(12000)}{222615}} = 5.94" < 8.25" \checkmark$$

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-way RC Slab | Concrete Beams

- Design Reinforcing in MS
  Assume # 5 bars, 3/4" clear cover

$d_{short} = 10.5 - 0.75 - \frac{1}{2}(0.625) = 9.44''$
$d_{long} = 9.44'' - 0.625'' = 8.82''$

Item	Description	ext. span M_{ext}^-	ext. span M^+	ext. span M_{int}^-	int. span M^-	int. span M^+
1	M in MS	-4	+164	-156	-152	+100
2	width of MS	180"	180"	180"	180"	180"
3	effective depth	8.82"	8.82"	8.82"	8.82"	8.82"
4	$M_n = M_u/\phi$	-4.44	+182	-173	-169	+111
5	$R = M_u/\phi bd^2$	4	156	149	145	95
6	ρ from Table A5a	0.00007	0.0026	0.0025	0.0024	0.0016
7	$A_s = \rho bd$	1.11	4.13	3.97	3.81	2.54
8	$A_{s min} = 0.002bd$	3.18	3.18	3.18	3.18	3.18
9	$n = \frac{\text{larger of 7 or 8}}{0.31}$	11	14	13	13	11
10	$n_{min} = \frac{\text{width of strip}}{2t}$	9	9	9	9	9

- check d_{min}

$d^2 = \frac{M_u}{222615}$ $\quad d_{min} = \sqrt{\frac{164(12000)}{222615}} = 2.97'' < 8.82''$ ✓

Design Frame B and Frame D Reinforcing

- Design Reinforcing in CS
  Assume #8 bars, 3/4" clear cover

$d_{short} = 9.25''$
$d_{long} = 8.25''$

Item	Description	ext. span M_{ext}^-	ext. span M^+	ext. span M_{int}^-	int. span M^-	int. span M^+
1	M in CS	-73.0	+200	-245	-228	+122
2	CS width	90"	90"	90"	90"	90"
3	effective depth	8.25"	8.25"	8.25"	8.25"	8.25"
4	$M_n = M_u/\phi$	-81.1	+222	-272	-253	+136
5	$R = M_u/\phi bd^2$	159	435	533	496	267
6	ρ from table A5a	0.0027	0.0076	0.0094	0.0087	0.0046
7	$A_s = \rho bd$	2.00	5.64	6.98	6.46	3.42
8	$A_{s min} = 0.002bt$	1.89	1.89	1.89	1.89	1.89
9	$n = \frac{\text{larger of 7 or 8}}{0.79}$	3	8	9	9	5
10	$n_{min} = \frac{\text{width of strip}}{2t}$	5	5	5	5	5

- check d_{min}

$d^2 = \frac{M_u}{222615}$ $\quad d_{min} = \sqrt{\frac{245(12000)}{222615}} = 3.63'' < 8.25''$ ✓

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-way RC Slab | Concrete Beams

- Design Reinforcing in MS
  Assume #5 bars, 3/4" clear cover

$d_{short} = 9.82''$
$d_{long} = 8.82''$

Item	Description	ext. span M_{ext}^-	ext. span M^+	ext. span M_{int}^-	int. span M^-	int. span M^+
1	M in MS	-1.7	+66	-82	-76	+41
2	width of MS	90"	90"	90"	90"	90"
3	effective depth	8.82"	8.82"	8.82"	8.82"	8.82"
4	$M_n = M_u/\phi$	-1.9	+73	-91	-84	+46
5	$R = M_u/\phi b d^2$	3	126	156	145	78
6	ρ from Table A5a	0.00005	0.0021	0.0026	0.0024	0.0013
7	$A_s = \rho b d$	0.040	1.67	2.06	1.91	1.03
8	$A_{s\,min} = 0.002bd$	1.59	1.59	1.59	1.59	1.59
9	$n = \frac{\text{larger of 7 or 8}}{0.31}$	6	6	7	7	6
10	$n_{min} = \frac{\text{width of strip}}{2t}$	5	5	5	5	5

- check d_{min}

$$d^2 = \frac{M_u}{222615} \qquad d_{min} = \sqrt{\frac{82(12000)}{222615}} = 2.10'' < 8.82'' \checkmark$$

Check Shear Capacity

- check Slab
  $w_u = 1.2(10.5/12 \cdot 150) + 1.6(100) = 318\text{ psf}$
  critical section at $d = 8.25''$ from face of the beam

$$V_n = (0.318)(1)(15 - 8.25/12 - 3/2) = 4.23K$$
$$\phi V_c = \phi 2\sqrt{f'c}\, bd = 0.75(2)\sqrt{6000}(12)(8.25)/1000 = 11.5K > V_n = 4.23K \checkmark$$

- check punching shear
  critical section at $d/2$ from column
  $d = \frac{8.25 + 9.25}{2} = 8.75''$

$V_u = (0.318)(30 \cdot 30 - (2.73 \cdot 2.73)) = 284K$
$b_o = 10.92'$
$b_o/d = 14.976 \qquad B_c = b_\ell/b_s = 1 \quad \alpha_s = 40$

$V_c = 4\sqrt{f'c}\, b_o d = 4\sqrt{6000}(131.04)(8.75) = 355K \leftarrow$
$V_c = (2 + 4/B_c)\sqrt{f'c}\, b_o d = (2 + 4/1)\sqrt{f'c}(131.04)(8.75) = 533K$
$V_c = (\alpha_s/b_o/d + 2)\sqrt{f'c}\, b_o d = (40/14.976 + 2)\sqrt{6000}(131.04)(8.75) = 415K$

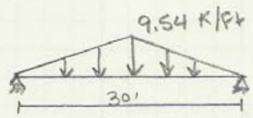
$\phi V_c = 0.75(355) = 266K < V_n = 284K$
Need Shear rails around the colums

Two-Way Reinforced Concrete Slab

Alternative Slab 4	2-way RC Slab	Concrete Beams

Design Beam Reinforcing

- Interior Beam



$M_u = \frac{wl}{6} = \frac{9.54(30)}{6} = 47.7$ ft-K
572.4 K-in

$M_{u,TBeam} = \phi\, 0.85 f'_c\, b\, h_f (d - h_f/2)$
$= 0.9(0.85)(6)(24)(10.5)(17 - 10.5/2)$
$= 13591$ K-in $> M_u$ ∴ no T-beam behavior

$A_s = \frac{M_u}{4d} = \frac{47.7}{4(17)} = 0.701\ in^2$ use $2(\#9) = 2\ in^2$

Assume $\epsilon_s > \epsilon_y$

$a = \frac{A_s f_y}{f'_c \beta_1 b} = \frac{2(60)}{(6)(0.75)(24)} = 1.11''$

$c = a/\beta_1 = 1.11/0.75 = 1.48''$

$\epsilon_s = \frac{\epsilon_u (d-c)}{c} = \frac{0.003(17-1.48)}{1.48} = 0.0315 > \epsilon_y$ ✓

$\phi = 0.9$ $\epsilon_s > 0.005$

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(2)(60)(17 - 1.11/2) = 1776$ K-in
$= 148$ K-ft $> M_u = 47.7$ K-ft ✓

$$A_{s\,min} = \begin{cases} \frac{3\sqrt{f'_c}\, bd}{f_y} = \frac{3\sqrt{6000}(24)(17)}{60000} = 1.58\ in^2 \\ \frac{200\, bd}{f_y} = \frac{200(24)(17)}{60000} = 1.36\ in^2 \end{cases}$$

$A_s > A_{s\,min}$ ✓

$\rho_{max} = 0.85\beta_1 (f'_c/f_y)(\epsilon_u/(\epsilon_u + 0.004)) = 0.85(0.75)(6/60)\left(\frac{0.003}{0.007}\right) = 0.0273$

$A_{s\,max} = \rho_{max}\, bd = 0.0273(24)(17) = 11.1\ in^2$

$A_s < A_{s\,max}$ ✓

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-Way RC Slab | Concrete Beams

Shear Reinforcement

$V_c = 2\lambda\sqrt{f'c}\, b_w d = 2(1)\sqrt{6000}(24)(17) = 63.2K$

$\phi V_n = 0.5(0.75)(63.2) = 23.7K$

$V_s = \frac{V_u}{\phi} - V_c = \frac{71.6}{0.75} - 63.2 = 32.3K < 8\sqrt{6000}(24)(17) = 253K \checkmark$

V_u at $d \approx 71.6K$

$V_s \leq 4\sqrt{f'c}\, bwd = 4\sqrt{6000}(24)(17) = 126K \checkmark$

$S_{max} = \begin{cases} d/2 = 17/2 = 8.5'' \leftarrow \text{use } 8'' \\ 24'' \end{cases}$

$A_v = \max\begin{cases} 0.75\sqrt{f'c}\, b_w s/f_{yt} = 0.75\sqrt{6000}(24)(8)/60000 = 0.19 in^2 \leftarrow \\ 50 b_w s/f_{yt} = 50(24)(8)/60000 = 0.16 in^2 \end{cases}$

Use #3 stirrups at 8"
($A_v = 2$ legs $\bullet$ $0.11 in^2/leg = 0.22 in^2$)

$S = A_v f_{yt} d / V_s = 0.22(60)(17)/32.3 = 6.95''$

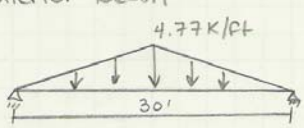
Use (2) #3 @ 6" at member ends, starting 2" from face of support

$\phi V_n = \phi V_c + \phi V_s = 0.75\left(63.2 + \frac{0.22(60)(17)}{8}\right) = 68.4K$

not much use

(2) #3 stirrups: (1) @ 2", (34) @ 5" each end

• Exterior Beam



$M_u = \frac{wl}{6} = \frac{4.77(30)}{6} = 23.9$ K-ft
286 K-in

$M_{u,T\text{-beam}} = \phi\, 0.85 f'c\, b h_f (d - h_f/2)$
$= 0.9(0.85)(6)(24)(10.5)(17 - 10.5/2)$
$= 13591$ K-in $> M_u$ ∴ no T-Beam behavior

$A_s = \frac{M_u}{4d} = \frac{23.9}{4(17)} = 0.35 in^2$ use (2) #9 $= 2 in^2$

Two-Way Reinforced Concrete Slab

Alternative Slab 4 | 2-Way RC Slab | Concrete Beams

Assume $\epsilon_s > \epsilon_y$

$$a = \frac{A_s f_y}{f'_c \beta_1 b} = \frac{2(60)}{6(0.75)(24)} = 1.11''$$

$$c = a/\beta_1 = 1.11/0.75 = 1.48''$$

$$\epsilon_s = \frac{\epsilon_u (d-c)}{c} = \frac{0.003(17-1.48)}{1.48} = 0.0315 > \epsilon_y \checkmark$$

$\phi = 0.9$ $\quad \epsilon_s > 0.005$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(2)(60)(17 - 1.11/2) = 1776 \text{ K-in}$$
$$= 148 \text{ K-ft} > M_u = 47.7 \text{ K-ft} \checkmark$$

$$A_{s,min} = \begin{cases} \dfrac{3\sqrt{f'_c}\, bd}{f_y} = \dfrac{3\sqrt{6000}(24)(17)}{60000} = 1.58 \text{ in}^2 \\ \dfrac{200\, bd}{f_y} \quad \dfrac{200(24)(17)}{6000} = 1.36 \text{ in}^2 \end{cases}$$

$A_s > A_{s,min} \checkmark$

$$\rho_{max} = 0.85\beta_1 (f'_c/f_y)(\epsilon_u/(\epsilon_u + 0.004)) = 0.85(0.75)(6/60)\left(\frac{0.003}{0.007}\right) = 0.0273$$

$$A_{s,max} = \rho_{max}\, bd = 0.0273(24)(17) = 11.1 \text{ in}^2$$

$A_s < A_{s,max} \checkmark$

Shear Reinforcement

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{6000}(24)(17) = 63.2 \text{K}$$
$$\phi V_n = 0.5(0.75)(63.2) = 23.7 \text{ K}$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{35.8}{0.75} - 63.2 = -15.5 \text{ K} < 8\sqrt{6000}(24)(17) = 253\text{K} \checkmark$$

V_u at $d \approx 35.8$K

$$V_s \le 4\sqrt{f'_c}\, b_w d = 4\sqrt{6000}(24)(17) = 126 \text{K} \checkmark$$

$$S_{max} = \begin{cases} d/2 = 17/2 = 8.5'' \leftarrow \\ 24'' \end{cases}$$

Use 8"

$A_{v,min} = 0.19 \text{in}^2$ $\quad$ Use #3 stirrups ($A_v = 0.22 \text{in}^2$)

$s = A_v f_{yt}\, d / V_s = 0.22(60)(17)/15.5 = 14.5''$

(2) #3 stirrups: (1) @ 2", (21) @ 8" each end

Two-Way Reinforced Concrete Slab

Alternate Slab 4 | 2-way RC Slab | Concrete Beams

Reinforcing Layout