St. Vincent Mercy Medical Center Heart Pavilion

Toledo, Ohio

Technical Report II

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TABLE OF CONTENTS

Table of Contents 2

Executive Summary 3

Introduction: St. Vincent Mercy Medical Center Heart Pavilion 4

Alternate Floor Systems 5

Code and Design Requirements 6

Materials 7

Gravity and Lateral Loads 8

Existing Structural Description 9

Existing Composite Steel Floor System 11

Composite Cellular Beam System 13

Two-Way Flat Slab System 15

Two-Way Post-Tensioned System 18

System Comparison 20

Conclusion 21

Appendix A: Building Layout 22

Appendix B: Existing Floor System and Member Spot Checks 25

Appendix C: Composite Cellular Beam Analysis 32

Appendix D: Two-Way Flat Slab Analysis 38

Appendix E: Two-Way Post-Tensioned Slab Analysis 46

EXECUTIVE SUMMARY

St. Vincent Mercy Medical Center Heart Pavilion is a four story hospital that provides diagnostics, surgery, and patient care. It was constructed for St. Vincent's Mercy Medical Center Campus, established in 1855, in downtown Toledo, Ohio.

The facility is approximately 144,000 square feet and reaches a height of 57'5" above grade with a typical floor to floor height of approximately 14 feet. A typical interior bay is 30 feet by 35 feet and is comprised of composite steel with a concrete slab on deck. The lateral system utilizes steel moment frames due to limited floor space. Drilled caissons and spread footings make up the foundation system. The ground floor is a reinforced slab on grade with grade beams between caissons to transfer wall load into the foundation.

In this second technical report, alternate floor systems are investigated through the preliminary design of a typical interior bay. Three alternative floor systems and the existing floor system are compared with respect to cost, constructability, serviceability, architecture, and fire protection. Conclusions are then drawn about the overall efficiency of each design, and whether it is still a viable solution for an alternative floor system within St. Vincent Mercy Medical Center Heart Pavilion.

The existing floor system is made up of composite steel framing and normal weight concrete. The alternative floor systems chosen for analysis are as follows:

- Composite Cellular Beam Framing
- Two-Way Flat Slab
- Two-Way Post Tensioned Slab

Upon completion of schematic design of each system and cost comparison, it was determined that the two best solutions for alternative floor systems are composite cellular beam framing and a two-way post tensioned slab. Both systems will reduce overall floor thickness and are very economical with respect to cost. The composite cellular beam system actually eliminates quite a few existing columns as these beams perform most efficiently under long spans. The two-way post tensioned slab employs the existing column grid and only requires small shear caps. However, all three systems qualify for further investigation as an alternative floor system for St. Vincent Mercy Medical Center Heart Pavilion.

INTRODUCTION: ST. VINCENT MERCY MEDICAL CENTER HEART PAVILION

St. Vincent's Heart Pavilion is one of the seven hospitals that comprise Mercy Health Partners. As Toledo's first and only facility for the treatment of vascular disease, St. Vincent's Heart Pavilion has become a staple within the community. St. Vincent's Mercy Medical Center Campus is now able to take a leadership role in providing education to its students as well as saving lives through the treatment of vascular disease.

Modernization is emphasized through the façade of St. Vincent Mercy Medical Center Heart Pavilion. As one approaches the building from the North, a beautiful curtain wall composed of curved aluminum and spandrel glass is seen, thus adding great verticality to the building. As the eye gazes along the façade, stone bands and brick veneer promote horizontal progression to an attractive vertical component of stairs wrapped in stone veneer and spandrel glass. The eye is then led to the pedestrian bridge, connecting the Heart Pavilion to a parking garage, which shows off its structure through exposed chevron bracing.

The structure of the Heart Pavilion is comprised of a composite steel floor system that utilizes steel moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The ground floor is a reinforced slab on grade with grade beams between caissons to transfer wall load into the foundation.

The purpose of Technical Report II is to explore alternate floor systems in efforts to discover other systems that will also meet the needs of the Heart Pavilion. Upon completion of this report, conclusions will be drawn on the viability of the existing system and the alternate systems studied with respect to cost, constructability, serviceability, architecture, and fire protection.

ALTERNATE FLOOR SYSTEMS

Alternative floor systems were analyzed for St. Vincent Mercy Medical Center in a desire to explore other options and draw conclusions on viability. Initial design intent of the alternate systems was to minimize floor thickness with the idea of adding another floor to this facility, while keeping serviceability requirements as a top priority. However, the addition of another floor is not a viable opportunity as new zoning regulations limit the maximum height of buildings within this district to 50 feet. Permits for this facility were issued before these zoning regulations took effect, therefore previous zoning requirements were followed, allowing the facility to rise to 57'5". The systems that are analyzed within this report are listed in the order they are discussed:

- Composite Cellular Beam System
- Two-Way Flat Slab System
- Two-Way Post Tensioned System

Various references were used in order to carry out the preliminary design of these systems:

- AISC Specification for Structural Steel Buildings, 13th Edition
- ACI 318-08 Building Code and Commentary
- CMC Steel Products Cellular Beam Design Guide
- PCA Post Tensioned Slab Design Guide
- RS Means Assemblies Cost Data, 2008 Edition
- RS Means Square Foot Cost Data, 2007 Edition

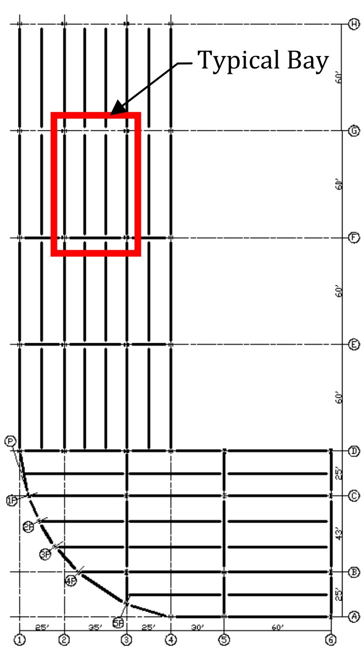


Figure 1: *Alternative Column Layout*

The two concrete systems discussed within this report were analyzed using the existing column grid. A typical interior bay within this grid is 35' by 30'. A typical floor plan showing the existing column grid is provided in Appendix A for further reference. However, the existing column grid would not be very efficient to use for cellular beam design. Cellular beams perform the best when the infill beams span a far distance, while the girders span a much shorter distance. As a result, the existing column grid was modified by eliminating some column lines, creating a typical interior bay size of 35' by 60' as seen in figure 1. Please refer to Appendix A to compare the existing floor plan with the alternative floor plan.

CODE AND DESIGN REQUIREMENTS

Various references were used by the Engineer of Record in order to carry out the structural design of St. Vincent Mercy Medical Center Heart Pavilion:

Codes and References

- The 2002 International Building Code as amended by the State of Ohio
- The Building Code Requirements for Structural Concrete (ACI 318-02), American Concrete Institute
- Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings —Load and Resistance Factor Design, Third Edition, American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), American Society of Civil Engineers

Serviceability requirements for St. Vincent Mercy Medical Center Heart Pavilion used by the Engineer of Record are as follows:

Deflection Criteria

Floor Deflection:

L/240 Total Load

L/360 Live Load

L/600 Curtain Wall Load

L/1666 Impact Load on Elevator Support Beams

Lateral Deflection:

H/500 Total Allowable Wind Drift

H/400 Total Story Wind Drift

$0.015h_{sx}$ Total Allowable Seismic Drift

MATERIALS

Multiple materials were used for the construction of St. Vincent Mercy Medical Center Heart Pavilion. The details of these materials are listed as follows:

Concrete

Foundations	f'_c = 3000 psi
Walls	f'_c = 3000 psi
Slabs	f'_c = 3500 psi
Grade Beams	f'_c = 4000 psi

Reinforcing Steel

Reinforcing Bar	A.S.T.M. A-615 GRADE 60
Tie Wire	A.S.T.M. A-82
Welded Wire Fabric	A.S.T.M. A-185

Structural Steel

Wide Flange	A.S.T.M. A992
Angle, Plate, Channel	A.S.T.M. A36
Connection Bolts	A.S.T.M. A325
Anchor Bolts	A.S.T.M. A307 OR A36
Square/Rectangle (HSS)	A.S.T.M. A500, GRADE B
Round (HSS)	A.S.T.M. A500, GRADE B

Metal Deck and Shear Studs

Composite Floor	2” 20. GA.
Roof Deck	1 ½” 22 GA.
Shear Studs	¾” x 5 ½”

GRAVITY AND LATERAL LOADS

Loading conditions are a very important consideration for the design of any structure. The dead load conditions assumed by the engineer of record at the time of design and live load conditions obtained from ASCE 7-02 are provided for reference:

Dead Loads

Concrete	150 PCF
Steel	490 PCF
Partitions	20 PSF
MEP	10 PSF
Windows & Framing	10 PSF
Finishes & Miscellaneous	5 PSF
Roof	20 PSF

Live Loads

First Floor Corridors	100 PSF
Lobbies	100 PSF
Loading Dock	100 PSF
Penthouse Floor	100 PSF
Corridors above First Floor	80 PSF
Patient Rooms	60 PSF
Operating rooms	60 PSF
Bridge Floor	60 PSF
Roof	20 PSF
Snow Drift (Low End)	16.8 PSF
Snow Drift (High End)	61.9 PSF

**Please reference Technical Report I for snow drift calculations.*

EXISTING STRUCTURAL DESCRIPTION

Foundation System

The foundation system is made up of 80 drilled caissons and 6 spread footings that support the entrance lobby. The caisson caps are a uniform size of 4'x4'x3' thick. Between caissons are grade beams, varying in depth from 2' to 4' depending on the location, which transfer façade and wall load to the foundation system. The ground (main) floor rests on a 6" concrete slab reinforced with W/4x4-W4.0x4.0 welded wire fabric.

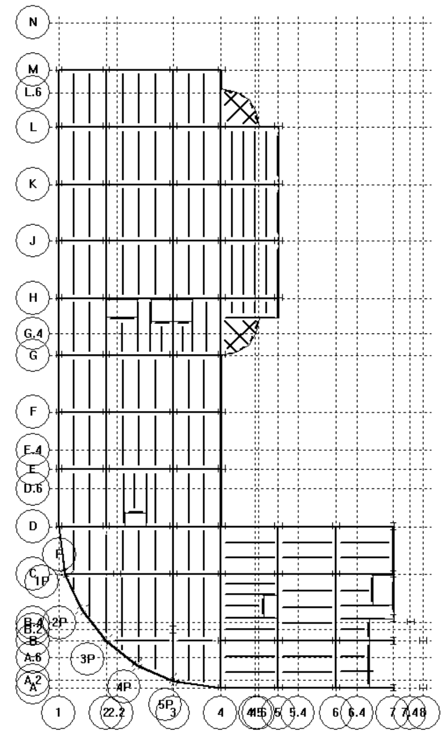


Figure 2: *Typical Floor Layout*

Columns

The columns used in St. Vincent Mercy Medical Center Heart Pavilion range from W10x119's to W12x210's, depending on their location within the building. While these sizes may seem large based purely on gravity, each column must resist induced moment since all columns are part of a moment frame. Pipe columns are used to support the roof for the main entrance lobby and the emergency vestibule canopy. All of the main building columns are spliced at the 2nd-3rd floor. Base plates range in thickness from 1" to 2 ¼" depending on which columns they are supporting. Each base plate utilizes a standard 4 bolt connection using either ¾" A325 or 1 ¼" A325 bolts.

Lateral System

At the time of design, braced frames were thought to be architecturally incompatible with this floor plan. As a result, steel moment frames were used for the lateral load resisting system. Please reference figure 2 to view the typical floor layout. Please reference figure 3, located on the following page, indicating the lateral system in red. Please reference Appendix A for a larger view of all floor plans.

The moment frames are connected in two different fashions as seen in figures 4 and 5 on the following page. The beam to column web moment connection is comprised of flange plates that are fillet welded to the column web and flange. The beam flanges are full-penetration welded to these plates. The beam to column flange moment connection utilizes double angles connecting the beam to the column flange, where the column flange is then full penetration welded to the beam flange.

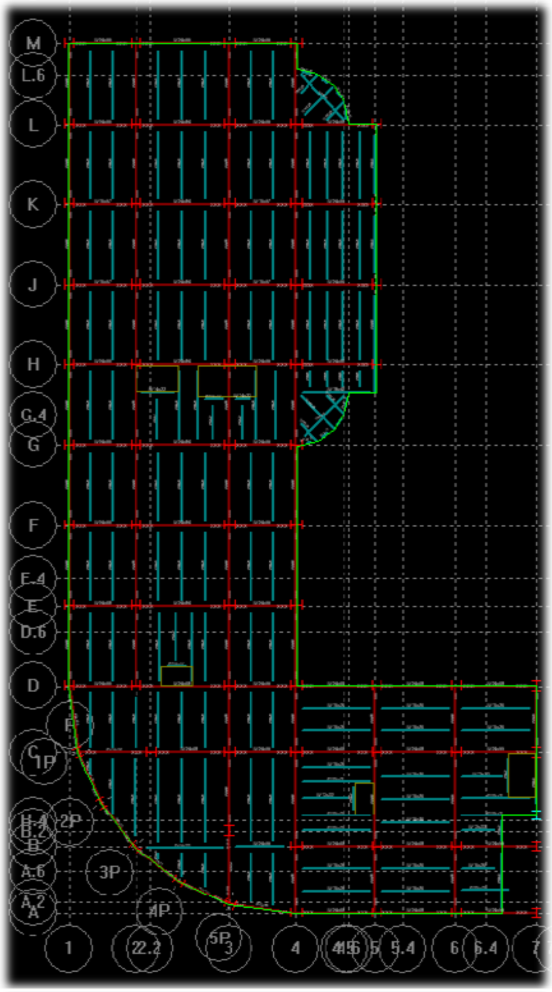


Figure 3: *Typical Floor Plan Indicating Lateral System*

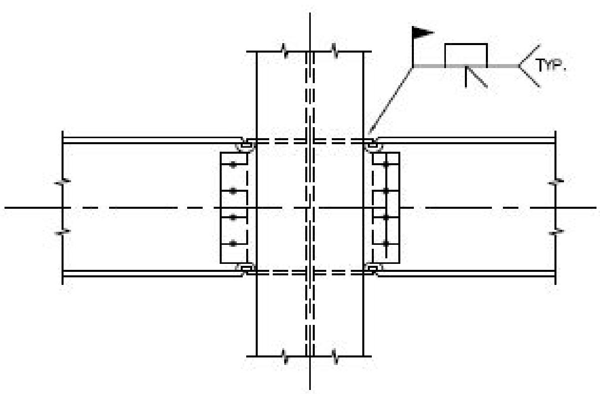


Figure 4: *Beam to Column Web Connection*

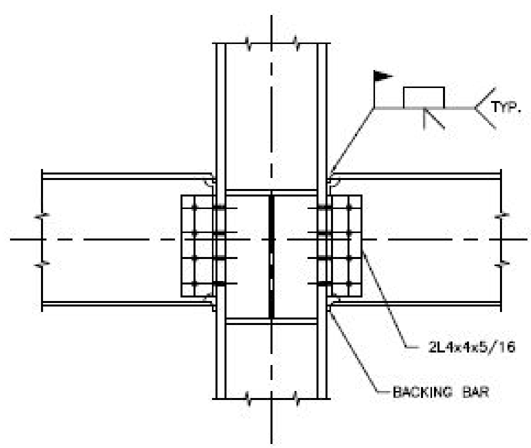


Figure 5: *Beam to Column Flange Connection*

EXISTING COMPOSITE STEEL FLOOR SYSTEM

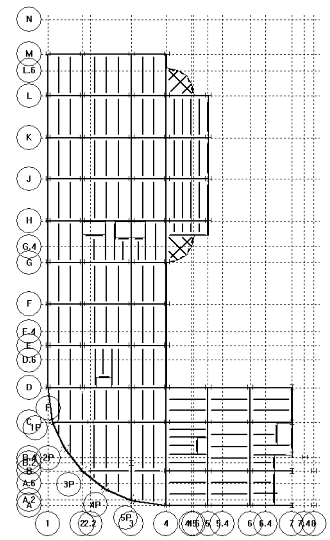


Figure 6: *Typical Floor Layout*

St. Vincent Mercy Medical Center Heart Pavilion's typical floor system is made up of composite steel framing and normal weight concrete, creating a total floor thickness of 6½". Please refer to figure 7 on the following page to view the existing composite steel floor system design. Composite action is created by the use of 2" 20 gauge steel deck with 5½" long, ¾" diameter shear studs evenly spaced over the length of each beam. Even though a composite system is used, the girders are actually non-composite. In order to avoid coping of the infill beams, the girders are placed 2" higher than the beams on a typical floor and 1½" higher on the roof, as seen in figure 8 on the following page. This system saved money and fabrication time which resulted in faster steel erection. Strength requirements are met by approximately 4.2% while deflection criteria are met by approximately 26%. Please reference Appendix B for detailed calculations checking member validity and deflection criteria of the existing floor system.

Pro-Con Analysis:

The floor plan of St. Vincent's Heart Pavilion does not allow for a large amount of columns interrupting spaces such as patient rooms or surgery space. As a result, large bays of 25' by 30' and 35' by 30' are essential for this facility. Composite steel framing is very useful for projects where this aspect is vital. The decking and concrete slab combination achieves an adequate fire rating of two hours with the provided fire proofing. The large steel members minimize deflection and vibration, both of which are critical in the serviceability of this building. In addition to these benefits, steel erection is a much faster process than placing concrete columns and floor slabs. Also, formwork and shoring is not required for this type of system which will also speed up construction.

While the benefits of composite steel construction seem very enticing, there is also a downside to using this system. First, deep members are required for the girders as a result of utilizing large spans. On top of these 24" deep sections is 4½ " of concrete and 2" of metal decking, creating a total floor thickness of 30½ ". This may not seem very thick, however, mechanical equipment is placed underneath the structural steel, creating a floor to floor height of approximately 14 feet.

Overall, this system does handle the structural demand and meets the architectural requirements for this project.

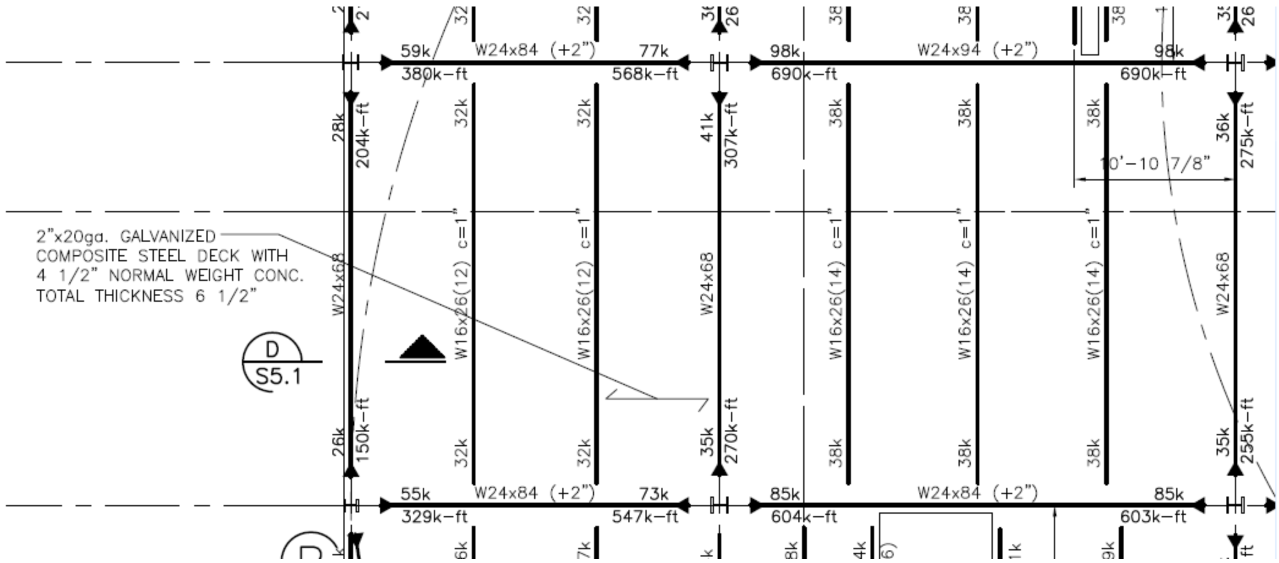


Figure 7: *Existing Composite Floor System*

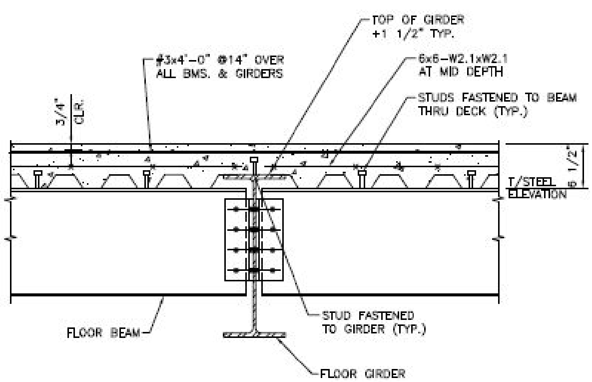


Figure 8: *Detail of Existing Composite Floor System*

COMPOSITE CELLULAR BEAM SYSTEM

Originally developed during steel shortages in the United States and Europe, castellation is a fabrication method used to "expand" a standard shape. This process results in a beam that is 50% deeper and 50% stronger than the original, without adding weight. There are two types of these beams: castellated beams (hexagonal openings) and cellular beams (round openings). Cellular beams were selected to analyze for a prospective alternative floor system as this allows duct work to be run through the openings in an easier manner. Figure 11 is provided on the following page showing the design of a typical interior bay using a detailed spreadsheet provided by CMC Steel Products. Please see Appendix B to view spreadsheet input and commentary regarding failure modes that this software incorporates.

Pro-Con Analysis:

Performing most efficiently under long spans, cellular beams seem to be the perfect fit for an alternative floor system. In addition to long span benefits, vibration characteristics are improved due to increased stiffness in the floor as these members are 50% deeper than standard beams. As seen in figure 10, mechanical equipment can be run through the openings in the beams, reducing the overall floor to floor height. Cellular beams can be painted, galvanized, or fireproofed up to a 3 hour rating for floor assemblies and a 1½ hour rating for roof assemblies. Also, foundation requirements can be significantly reduced due to having a lighter structure. Time and money is saved on fabrication and erection as this system requires fewer pieces. Moment frames are typically used for lateral resistance with this type of floor system, however, braced frames using standard steel shapes is another option. In addition to these many advantages, a potential of 8-13 LEED points can be awarded for the following categories: building reuse, resource reuse, recycled content, and local materials.

Figure 9: *Cellular Beam System*
Photo courtesy of www.fabsec.co.uk

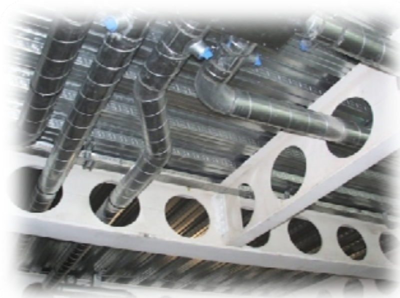


Figure 10: *Cellular Beam System Integrating Mechanical Equipment*
Photo courtesy of CMC Steel Products

The main drawback with this system is that integrating mechanical equipment to run through the cellular openings can be a very tedious process. This is especially true when considering how often the structure of the project changes due to architectural revisions during the design phase. Possible adjustments or alterations may be required by the mechanical engineer in order to ensure the equipment is laid out to precisely pass through these openings.

With all of these facts taken into consideration, the advantages of cellular beams outweigh the disadvantages. This floor system qualifies for further investigation for use within St. Vincent's Heart Pavilion.

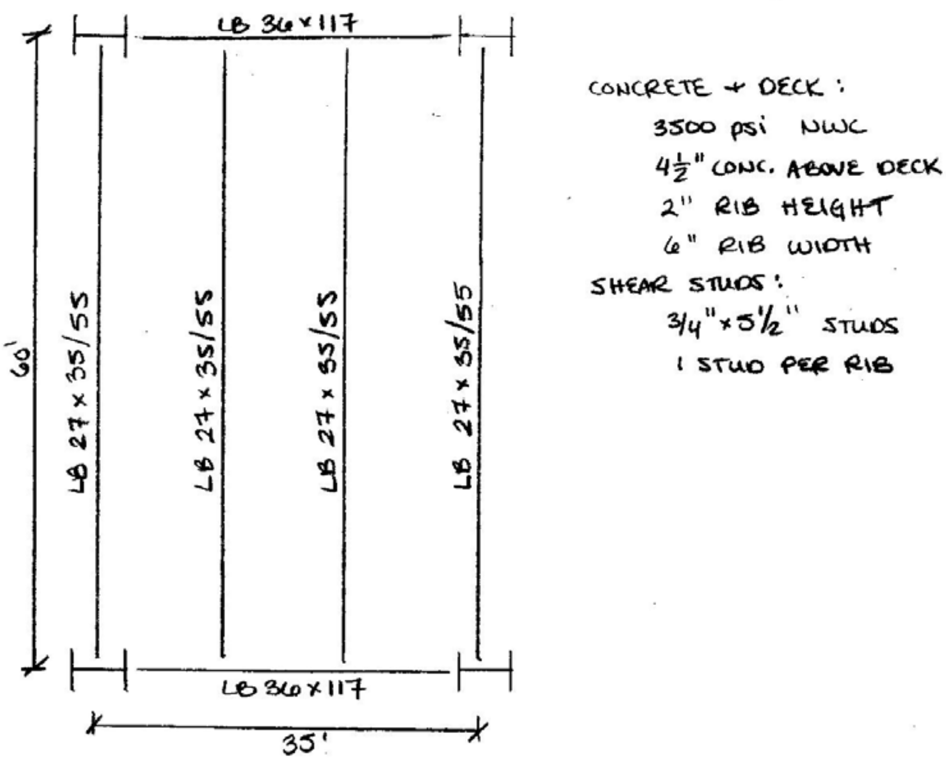


Figure 11: *Interior Bay of Cellular Beam Design*

TWO-WAY FLAT SLAB SYSTEM

In efforts to reduce slab thickness, steel reinforcing, and column sections that a conventional concrete floor system would require, a two-way flat slab system was chosen for analysis. A floor thickness of 11½" was obtained by using this system, still meeting the required fire rating of 2 hours. Figures 14 and 15 are provided on the following pages to show the design of a typical interior bay and rebar layout for this system. Please refer to Appendix C for detailed calculations on the design of a typical interior bay.

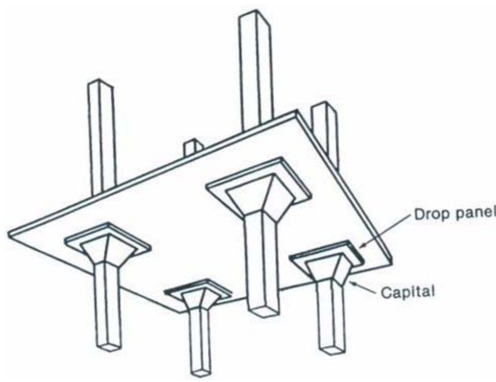


Figure 12: *Two-Way Flat Slab with Drop Panels*
Photo courtesy of stommel.tamu.edu

Pro-Con Analysis:

This floor system was chosen as a possible alternative as it can greatly reduce floor thickness. The thickness of the floor between drop panels utilizing this system is approximately one third the depth of the existing composite steel system. Column capitals and drop panels allow smaller column sections to be used. In addition, the use of column capitals and drop panels reduce the amount of steel reinforcement and concrete needed to achieve the same strength within the slab.

However, multiple challenges could arise with integrating the mechanical equipment and ceiling design into the slab thickness increase around drop panels. The additional weight of this floor system may also have a significant impact on the foundation system. In addition to this, construction of a two-way flat slab is also a challenge as a lot of formwork is required to build the column capitals and drop panels. This extra formwork would increase labor costs.

Figure 13: *Two-Way Flat Slab with Drop Panels*
Photo courtesy of www.esdep.org

Upon review of this floor system, it still seems as though it is a viable option for an alternative floor system. Construction would not be the simplest procedure; however, the overall cost to install this system is efficient, as seen in the comparison chart located in the "System Comparison" section of this report. This floor system does qualify for further investigation for use within St. Vincent's Heart Pavilion.

FINAL DESIGN: $t_{SLAB} = 9.5''$ $DIAMETER_{COL\ CAP} = 6.5'$

$t_{DROP} = 2.5''$ $WIDTH_{DROP} = 11.75'$

NOTE: ALL REBAR #7 BARS

QUANTITY OF BARS IN A SECTION INDICATED IN CIRCLE

$f'_c = 5000$ psi

ALL COLUMNS 18"x18"

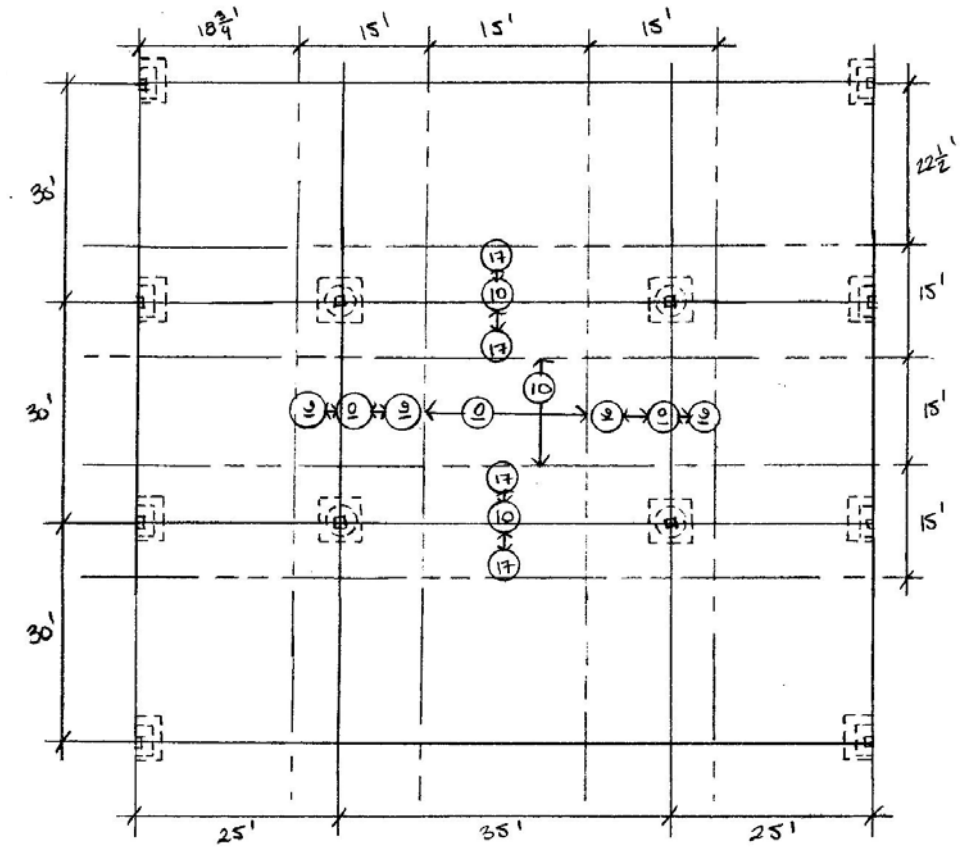


Figure 14: *Interior Bay of Two Way Flat Slab Design*

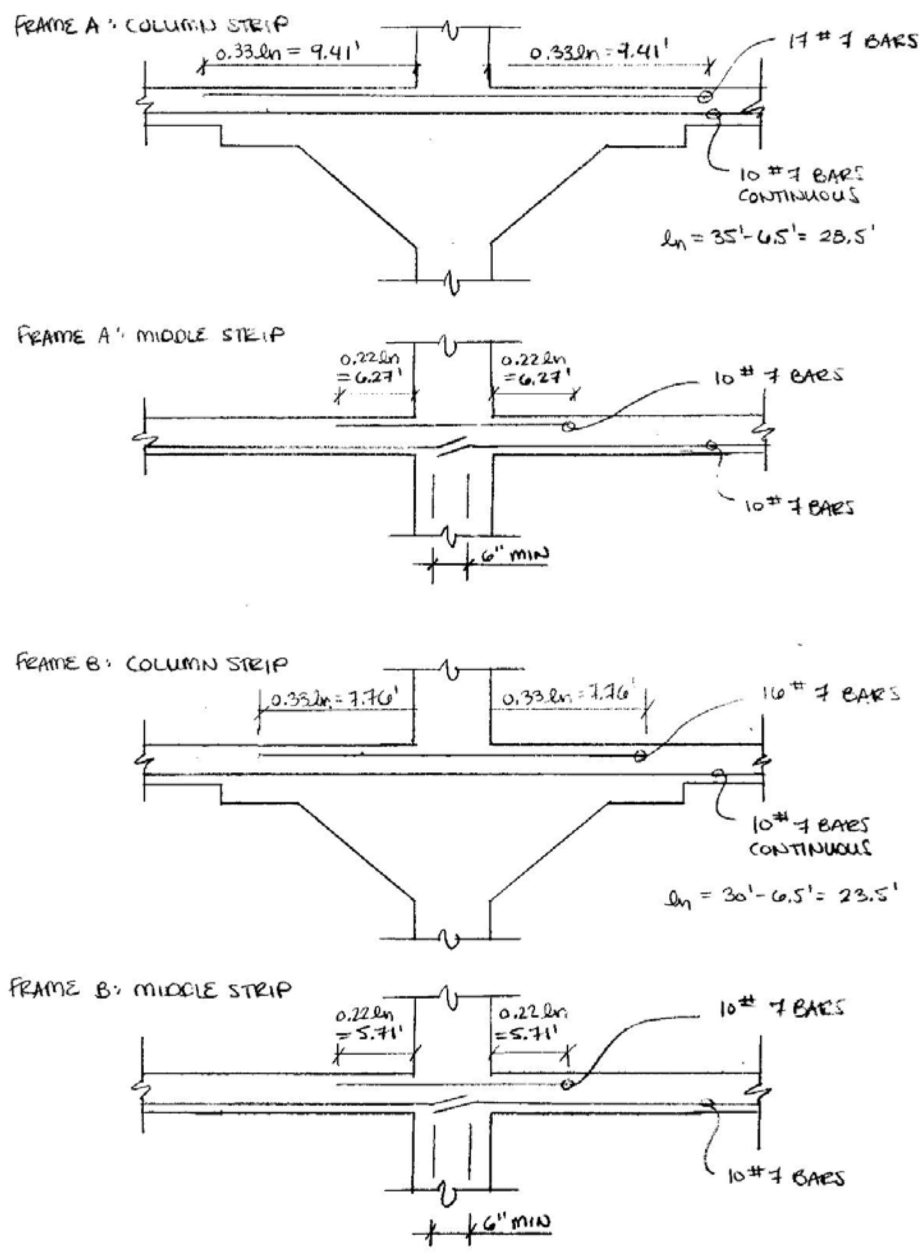


Figure 15: *Rebar Layout of Two Way Flat Slab Design*

TWO-WAY POST TENSIONED SLAB SYSTEM

Pre-stressed concrete is used because it forces concrete to overcome its one weakness- tension. A two-way post tensioned slab was chosen to further analyze this type of system and its many benefits. The main intent in choosing this system was to create thin floor thickness and avoid the use of column capitals and drop panels. A floor thickness of 9½" was obtained by using this system, still meeting the required fire rating of 2 hours. The post tensioned system is very useful when long spans are required by the floor plan. Figure 18 is provided on the following page to show the design of a typical interior bay. Please refer to Appendix D for detailed calculations of the design for a typical interior bay.

Pro-Con Analysis:

Post tensioned design is very useful as it can achieve large spans while maintaining a thin slab thickness. The large rectangular bays created by this system allow for patient rooms and corridor space to be free of intruding columns. Adding an additional floor would be very attainable while using this system as the floor thickness would be reduced from 30" to 9½". A fire rating of 2 hours would also be provided by this system.

Figure 16: *Typical Tendon Layout Photo courtesy of www.tucsonazrealestateblog.com*

However, there are a few negative aspects of this type of design. The level of construction for this system is extremely difficult as a very experienced and knowledgeable team of workers is required. Extra safety procedures are also required on site as the pre-stressed tendons are essentially like rubber bands holding an immense amount of tension that could seriously injure someone if they snap. For this reason, supervision of the post-tensioning process is required. In addition, adding openings in the floor system after it is in place is not an option due to the risk of severing a tendon.

Figure 17: *Detail of Tendon Anchor Photo courtesy of www.tucsonazrealestateblog.com*

After taking all these factors into consideration, post-tensioning still seems like a viable option. Structural requirements are easily met while architectural requirements are exceeded due to creating a much thinner floor thickness while maintaining large bays with few columns. This floor system qualifies for further investigation for use within St. Vincent's Heart Pavilion.

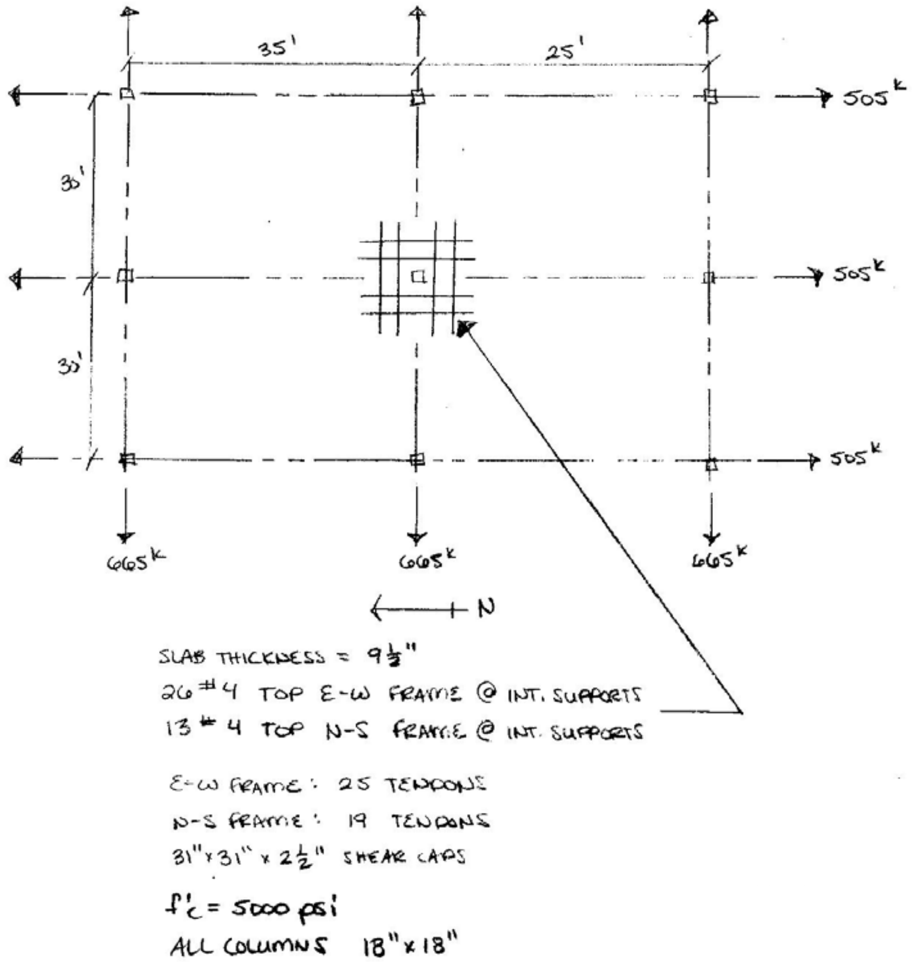


Figure 18: *Interior Bay of Post Tensioned Design*

SYSTEM COMPARISON

	Floor Systems			
	Existing Composite Steel	Composite Cellular Beams	Two-Way Flat Slab	Two-Way Post Tensioned Slab
System Weight (psf)	78	74	148	119
Slab Depth (in)	6.5	6.5	9.5	9.5
Total Depth (in)	30	36	11.875	9.5
Column Size	W12x170	*Little Difference	18"x18"	18"x18"
Effect on Column Grid	-	Little	None	None
Lead Time	Medium	Medium	Short	Short
Formwork	No	No	Yes	Yes
Construction Difficulty	Medium	Medium	Difficult	Difficult
Impact on Foundation	N/A	Little	Moderate	Little
Fireproofing	Yes	Yes	No	No
Fire Rating	Satisfied	Satisfied	Satisfied	Satisfied
Vibration	Satisfactory	Additional Study Required		
Material Cost Per S.F.	$23.50	$10.00	$10.25	$7.63
Labor Cost Per S.F.	$8.45	$6.00	$9.65	$8.30
Total Cost Per S.F.	$31.95	$16.00	$19.90	$15.93
Viable Alternative	-	Yes	No	Yes
Additional Study	-	Yes	No	Yes

Figure 19: *System Comparison*

**Columns for the Cellular Beam System were not designed; however they would not differ greatly from those provided by the current floor system.*

** System weight for the Cellular Beam System was determined for half of the 60' by 35' bay to ensure weights for all systems are based on the same dimensions.*

**RS Means data available upon request.*

After completing a side by side comparison of each schematic design, it is seen that the three alternative systems chosen for analysis are very economical. The cellular beam system is actually lighter than the existing composite steel system and is the most economical choice from a cost and construction standpoint. The two-way flat slab system seems like a more economical choice, however, the money saved with respect to material and labor would be spent on increased foundation costs due to extra structure weight. In addition to this, the formwork and labor involved with actually constructing this system is not very practical. The two-way post tensioned slab system provides a lighter structure than the two-way flat slab system and is more economical.

CONCLUSION

Technical Report II examines alternative floor systems in efforts to discover a system that is a viable option for use within St. Vincent Mercy Medical Center Heart Pavilion. All systems were chosen with reduction of floor thickness as the main priority. The cellular beam system and two-way post tensioned slab are the more feasible options for an alternate floor system based on the analysis done within this report.

A composite cellular beam system is the best steel solution for an alternative floor system. This system performs most efficiently at even larger bay sizes than the existing 35' by 30' bay; therefore quite a few existing columns were able to be eliminated. In addition, the overall floor thickness will be reduced as mechanical equipment can be run through the openings within the cellular beams and girders. Serviceability is also improved with the use of this system as vibration requirements will be surpassed due to member sizes being 50% stiffer than standard steel shapes. Also, a potential of 8-13 LEED points can be awarded for the following categories: building reuse, resource reuse, recycled content, and local materials.

A two-way flat slab is still a viable alternative floor system based on the schematic design and cost data within this technical report. The floor thickness is reduced from the current 30½" to 12", including drop panels. The overall cost of this system is efficient even though labor costs would be increased due to the extra formwork that is required for the installation of this system. However, the money saved on material and labor may be spent on increased foundation costs due to added structure weight.

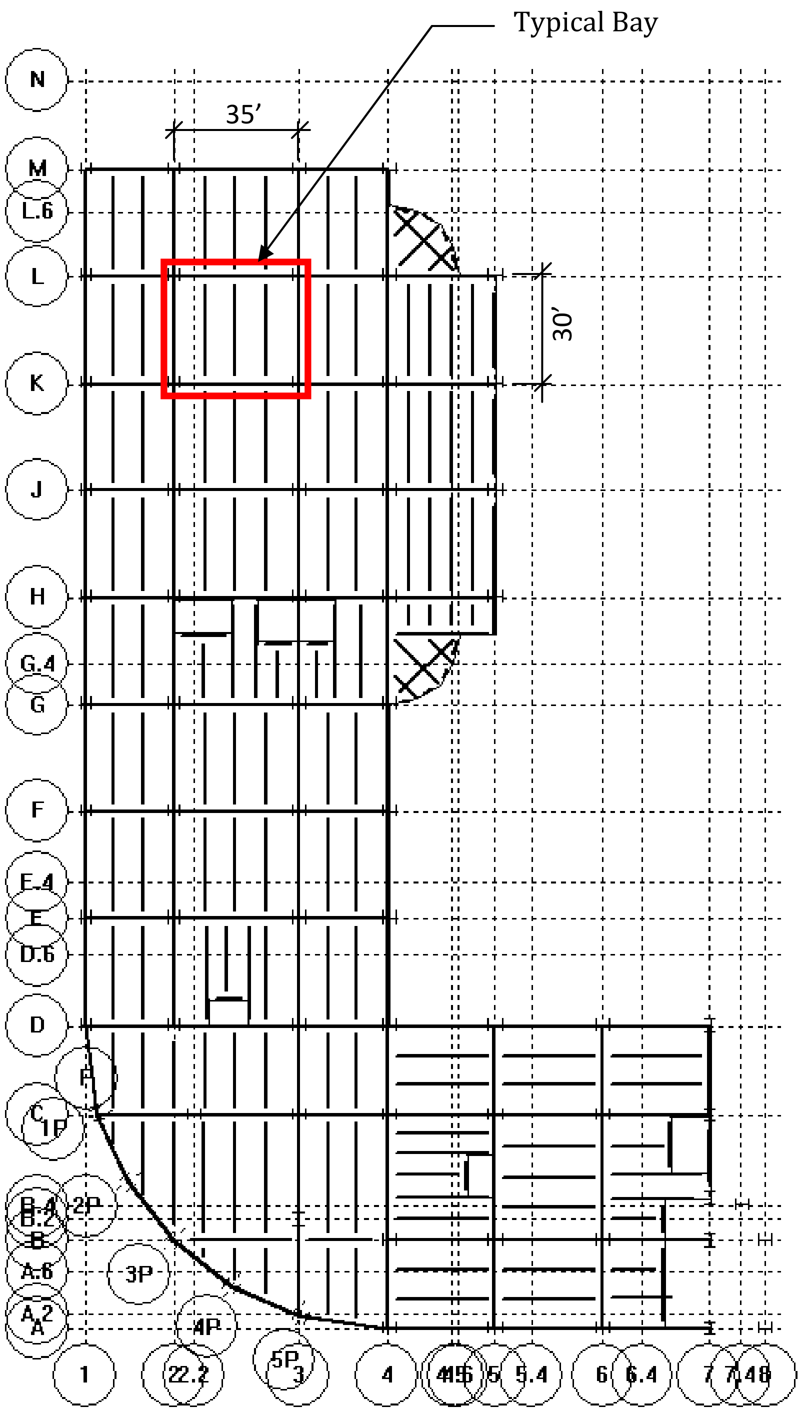
Two-way post tensioning is the best concrete solution for an alternative floor system. Floor thickness is vastly reduced to 9½" while only 2½" shear caps are needed as opposed to column capitals and drop panels. Little formwork will be required to construct this system when compared to the flat slab system. In addition, post tensioning is very economical with respect to cost.

All design values used and procedures carried out were done in accordance with applicable codes. Please refer to the appendices for further review of detailed notes, figures, or tables regarding this matter. Questions should be directed to Kristen M. Lechner via email: kml5016@psu.edu.

APPENDIX A: BUILDING LAYOUT

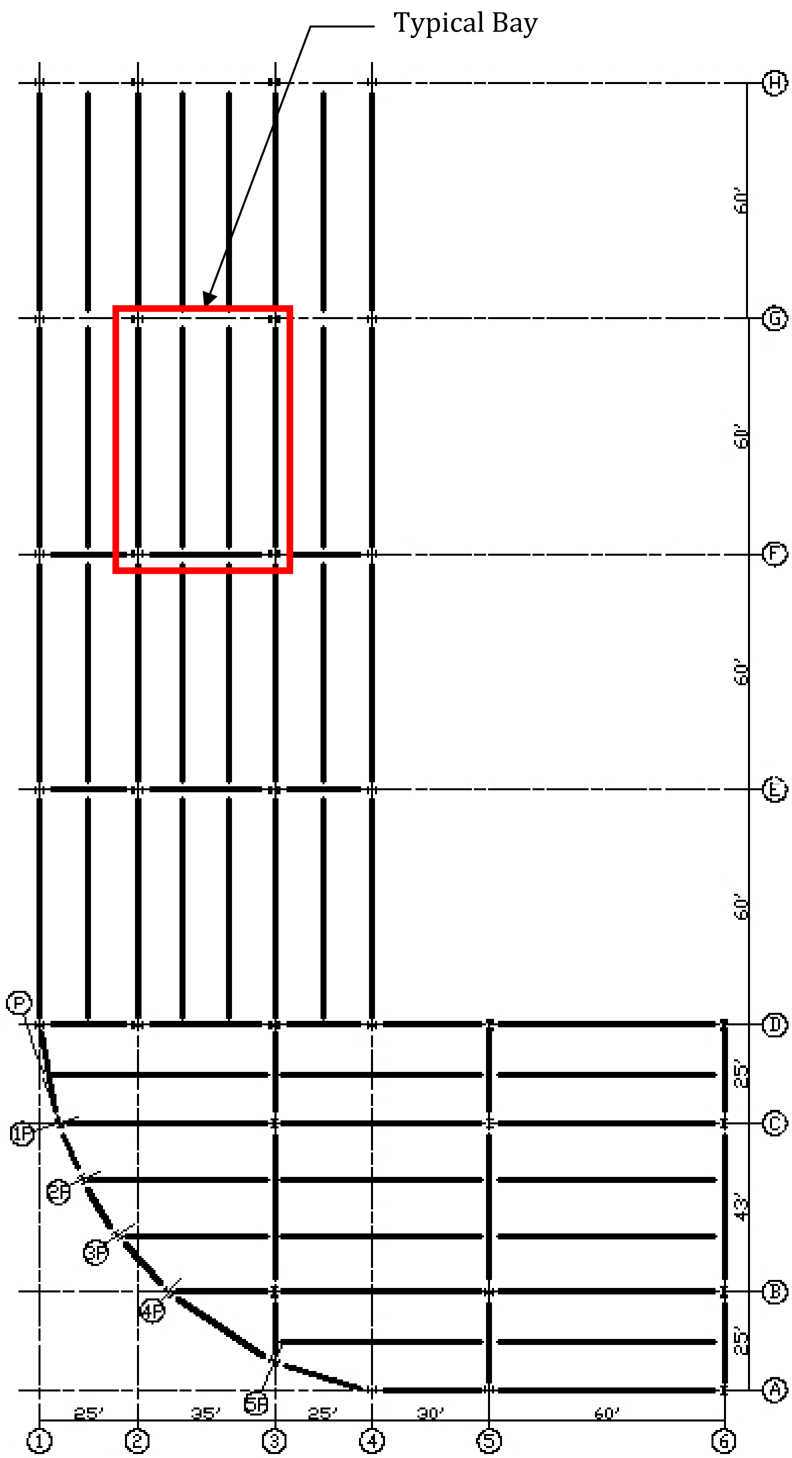
Photos courtesy of Ruby + Associates

Existing Floor Layout

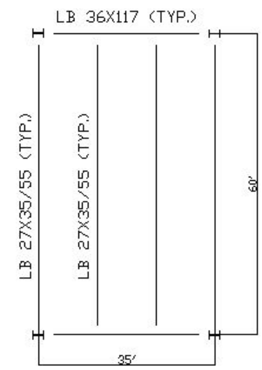


Second Floor Plan

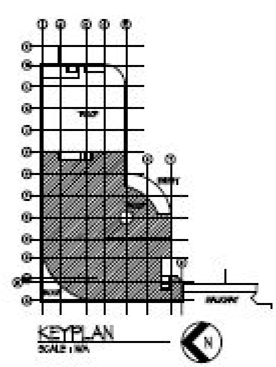
Alternative Floor Layout



Alternate Second Floor Plan



Typical Interior Bay



Provided for orientation only, does not show revised column lines

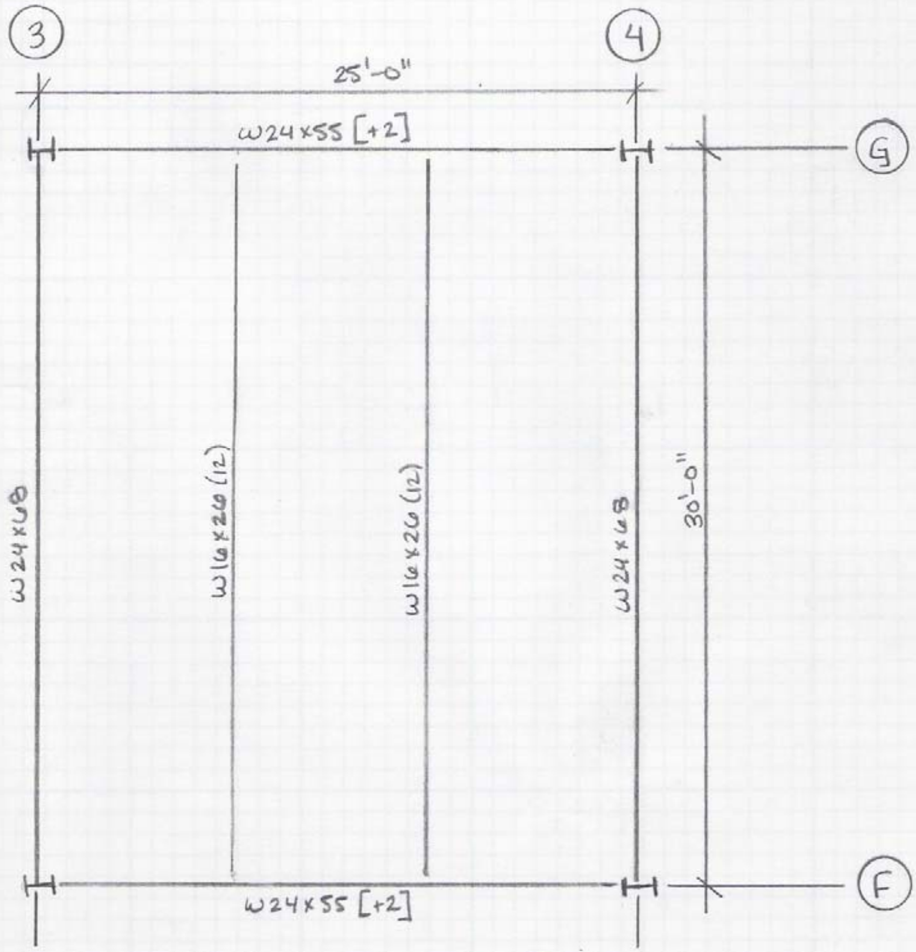
APPENDIX B: EXISTING FLOOR SYSTEM AND MEMBER SPOT CHECKS

Photo courtesy of www.secapp.com

Floor System and Member Spot Checks

Composite Floor System

TYPICAL INTERIOR BAY : 2ND FLOOR



LOADS :

LIVE: ASSUME 100 PSF

DEAD: 110 PSF [SEE APPENDIX C: SEISMIC ANALYSIS]

FLOOR SYSTEM:

$4\frac{1}{2}$" N.W.C.

2" x 20 GA DECK

LOAD COMBINATION:

1.2D + 1.6L

Composite Floor System

SPOT CHECK BEAM

FACTORED LOAD: $1.2D + 1.6L$

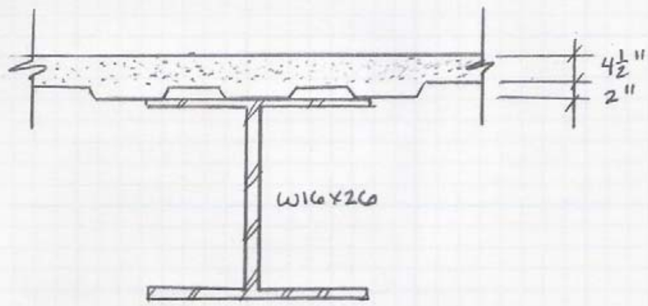
$w_u = 1.2(110\text{psf}) + 1.6(100\text{psf}) = 292\text{psf}$

TRIB. WIDTH = 8.33'

$w_u = 292\text{psf}\,(8.33')/1000 = 2.43\text{ klf}$

$$M_u = \frac{w_u l^2}{8} = \frac{(2.43\text{ klf})(30')^2}{8} = 273^{1k}$$

$$b_{eff} = \min \begin{cases} \text{SPACING} = 8.33' \times 12 = 100'' \\ \dfrac{\text{SPAN}}{4} = \dfrac{30(12)}{4} = 90'' \leftarrow \text{CONTROLS} \end{cases}$$



CHECK FOR DEFLECTION UNDER CONSTRUCTION LOADS:

$$\Delta_{CONSTR.} = \frac{5 w_{CONC} l^4}{384 EI}$$

$w_{CONC} = 150\text{pcf}\,(4.5''/12) = 56.3\text{psf}$

$w_{CONC} = 56.3\text{psf}\,(8.33') = 469\text{plf} = 0.469\text{ klf}$

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

$$I_{req} = \frac{5 w_{CONC} l^4}{384 \Delta_{CONSTR} E} = \frac{5(0.469)(30)^4(1728)}{384(1)(29000)} = 295\text{ in}^4$$

$I_{W16x26} = 301\text{ in}^4 > 295\text{ in}^4$ ✓ OK

CHECK BENDING FOR CONSTRUCTION LOADING:

$w_{CONC} = 0.469\text{ klf}$

$w_{LIVE} = 20\text{psf}(8.33') = 0.167\text{ klf}$

$w_u = 1.2(0.469) + 1.6(0.167) = 0.83\text{ klf}$

$M_u = w_u l^2/8 = 0.83(30)^2/8 = 93.4^{1k}$

$\phi M_{n\,W16x26} = 166^{1k} > 93.4^{1k}$

[COMPARE M_u WITH ϕM_n FOR W16x26 FROM Z_x TABLE B/C SYSTEM NOT COMPOSITE UNTIL CONSTRUCTION IS COMPLETE]

Floor System and Member Spot Checks

Composite Floor System

FROM TABLE 3-19:

ASSUME $\Sigma Q_n = 145^k$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{145^k}{0.85(3.5)(90)} = 0.542''$$

$$Y2 = 6.5'' - a/2 = 6.5'' - 0.542''/2 = 6.23''$$ [ROUND ↓ TO 6'' TO BE CONSERVATIVE]

USING TABLE 3-19:

W16x26 $Y2 = 6''$ $\Sigma Q_n = 145^k$ @ PNA #6

$\phi M_n = 285^{1k} > M_u = 273^{1k}$

CHECK NUMBER OF SHEAR STUDS:

TABLE 3-21:

$\left.\begin{array}{l}\text{SHEAR STUD DIAM} = 3/4'' \text{ ; 1 STUD/RIB} \\ \text{DECK PERPENDICULAR} \\ f'_c = 3000 \text{ KSI (CONSERVATIVE)}\end{array}\right\} Q_n = 17.2^k$

$$\text{\# STUDS REQ'D} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{145}{17.2} \times 2 = 16.9 \rightarrow \text{17 STUDS REQ'D}$$

\# STUDS PROVIDED = 30 [STUDS PLACED @ 12'' O.C. OVER LENGTH OF BM]

\# STUDS PROVIDED > \# STUDS REQ'D ✓ OK

CHECK DEFLECTION:

TABLE 3-20:

$Y2 = 6'' \Rightarrow I_{LB} = 705 \text{ in}^4$

$$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.833)(30)^4}{384(29000)(705)} = 0.74''$$

$$w_u = 100 \text{ psf}(8.33')/1000 = 0.833 \text{ klf}$$

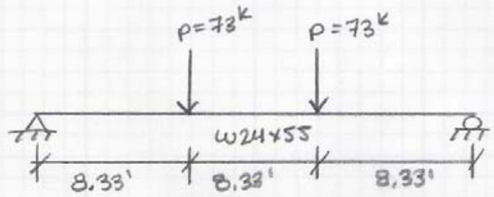
$$\Delta_{ALLOW} = l/360 = 30(12)/360 = 1''$$

$0.74'' < 1''$ ✓ OK

Floor System and Member Spot Checks

Girder

SPOT CHECK GIRDER



$w_{u,DL} = 1.2(110psf)(8.33')/1000 = 1.10\,klf$

$P_{DL} = \frac{w_{u,DL}\,l}{2} = \frac{(1.10)(30)}{2} = 16.5^k$

$w_{u,LL} = 1.6(100psf)(8.33')/1000 = 1.33\,klf$

$P_{LL} = \frac{w_{u,LL}\,l}{2} = \frac{(1.33)(30)}{2} = 20.0^k$

TOTAL P ON GIRDER $= (16.5^k + 20.0^k) \times 2 = 73^k$ ← BMS FRAME IN ON EACH SIDE

$M_{MAX} = P(a) = 73^k(8.33') = 608'^k$

ASSUME $Y2 = 4"$ ⟹ REQUIRING PNA # 7 (TABLE 3-19)

$\Sigma Q_n = 203^k$

$b_{eff} = \min \begin{cases} \text{SPACING} = 30' = 360" \\ \frac{\text{SPAN}}{4} = \frac{25}{4} = 6.25' = 75" \end{cases}$ ← CONTROLS

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{203^k}{0.85(3.5)(75)} = 0.91"$

GIRDERS PLACED 2" ABOVE BEAMS ∴ DEPTH = $4\frac{1}{2}"$ NOT $6\frac{1}{2}"$

$Y2 = 4\frac{1}{2} - a/2 = 4\frac{1}{2} - 0.91/2 = 4.04"$ [ROUND ↓ TO 4" TO BE CONSERVATIVE]

USING TABLE 3-19:

$Y2 = 4"$ $\quad \Sigma Q_n = 203^k$ $\quad$ W24X55 $\quad$ PNA # 7

$\phi M_n = 705'^k > M_{MAX} = 608'^k$ ✓ OK

CHECK DEFLECTION:

$I_{LB} = 2160\,in^4$ (TABLE 3-20)

$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.833)(25)^4(1728)}{384(29000)(2160)} = 0.12"$

$w_u = 100psf(8.33')/1000 = 0.833\,klf$

$\Delta_{ALLOW} = l/360 = 25(12)/360 = 0.83"$

$0.12" < 0.83"$ ✓ OK

Floor System and Member Spot Checks

Columns

Floor	Tributary Area (ft^2)	Dead Load (psf)	Live Load (psf)	Influence Area (ft^2)	Reduction Factor (>=0.4)	Live Load (k)	Dead Load (k)	Load Combination	Load at Floor (k)	Accumulated Load (k)
Roof	900	45	100	3600	-	90.0	40.6	$1.2D + 0.5L_r$	93.7	93.7
3	900	112	100	3600	0.500	45.0	101.2	1.2D + 1.6L	193.4	287.1
2	900	110	100	3600	0.500	45.0	98.6	1.2D + 1.6L	190.3	477.4
1	900	123	100	3600	0.500	45.0	111.1	1.2D + 1.6L	205.3	682.6
Main	900	103	100	3600	0.500	45.0	92.7	1.2D + 1.6L	183.2	865.9

Accumulated Load on Columns

COLUMN SPOT CHECK : E3

SEE SPREADSHEET FOR LOADS

FLOOR 4 (ROOF) : $P_u = 93.7^k$

W12x170 ; $h = 14.5'$

$A_g = 50.0 \text{ in}^2$

$I_x = 1650 \text{ in}^4$ $\quad$ $I_y = 517 \text{ in}^4$

$r_x = 5.74 \text{ in}$ $\quad$ $r_y = 3.22 \text{ in}$

$$\frac{KL}{r_x} = \frac{14.5(12)}{5.74} = 30.3 \qquad \frac{KL}{r_y} = \frac{14.5(12)}{3.22} = 54.0 \leftarrow \text{CONTROLS}$$

$$\frac{KL}{r} \le 4.71\sqrt{E/F_y} = 4.71\sqrt{29000/50} = 113$$

$54.0 < 113$ $\therefore$ INELASTIC BEHAVIOR

$$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y = \left[0.658^{50/98.1}\right](50) = 40.4 \text{ ksi}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29000)}{(54)^2} = 98.1 \text{ ksi}$$

$$\phi P_n = \phi F_{cr} A_g = 0.9(40.4)(50.0) = 1818^k$$

$$P_u = 93.7^k \ll \phi P_n = 1818^k$$

Floor System and Member Spot Checks

Columns

CHECK W/ TABLE 4-22:

$\frac{KL}{r} = 54 \qquad \phi F_{cr} = 36.4\,ksi$

$\phi F_{cr} = 0.9(40.4) = 36.4\,ksi$ ✓ METHOD BY HAND CHECKS

CHECK W/ TABLE 4-1:

$KL = 14.5' \qquad W12 \times 170$

$\phi P_n = 1815^k \approx 1818^k$ ✓ METHOD BY HAND CHECKS

*NOTE: TABLE 4-1 SHALL BE USED FOR REMAINING COLUMN CHECKS AS IT IS BASED ON METHOD BY HAND SHOWN ABOVE.

COMMENTS: COLUMN SIZES ARE VERY LARGE WHILE CONSIDERING GRAVITY LOADS ALONE, HOWEVER, EVERY COLUMN IS PART OF A MOMENT CONNECTION HENCE RECEIVES LARGE INDUCED MOMENTS

FLOOR 3: $P_u = 287^k$

$W12 \times 170 \; ; h = 14' = KL$

TABLE 4-1:

$\phi P_n = 1840^k > P_u = 287^k$ ✓ OK

FLOOR 2: $P_u = 477^k$

$W12 \times 170 \; ; h = 14' = KL$

TABLE 4-1:

$\phi P_n = 1840^k > P_u = 477^k$ ✓ OK

FLOOR 1: $P_u = 683^k$

$W12 \times 170 \; ; h = 15' = KL$

TABLE 4-1:

$\phi P_n = 1790^k > P_u = 683^k$ ✓ OK

OBSERVATIONS:

COLUMNS ARE SO LARGE DUE TO THE MOMENT THEY MUST RESIST FROM LATERAL FORCES

-End of Section-

APPENDIX C: COMPOSITE CELLULAR BEAM ANALYSIS

Photos courtesy of CMC Steel Products

Theory of Castellated and Cellular Beams: Commentary on Design Methods

The design theory for castellated beams is solely based on *Design of Welded Structures* by Omer Blodgett and additional research investigating the behavior of web post buckling by Dr. Richard Redwood. The design theory for cellular beams has been developed by the Steel Construction Institute of the United Kingdom. It is difficult to unify these procedures as many different parties have been involved. In efforts to bring all these theories together, a design guide is currently being written by AISC. Since this is not yet available, software provided by CMC Steel Products was used for analysis within this technical report.

The software developed by CMC Steel Products will design a single member based on bay size and loading conditions. The first step in designing cellular beams is to find the overall bending moment and shear force at each opening and web post caused by the applied loads. This overall bending moment and shear force are referred to as global forces. These global forces create localized forces acting in the top and bottom tees, the web posts, and the full section. The components of the beam are then checked with respect to the following limit states: vierendeel bending, web post buckling, vertical and horizontal shear, lateral torsional buckling, and deflection.

When bending moment acting on the entire beam causes tension or compression forces within the top and bottom tees of the castellated or cellular beam, vierendeel bending occurs. This is known as the primary force. Bending moment within the top and bottom tees is created due to shear force acting on the entire beam that passes through the openings within the beam. This is known as the secondary force. As the values of the bending moment and shear force change along the length of the beam, it is required to check the interaction at each opening in the beam. The stresses caused by the shear and bending moment acting on the beam create are additive. Therefore, the most efficient use for castellated and cellular beams is when the maximum shear and moment occur away from each other. A good example of this type of situation is a simply supported beam with a uniformly distributed load: maximum shear is seen at the ends of the beam while maximum bending moment occurs at the middle. This explains why castellated and cellular beams perform most efficiently when they span large distances.

Due to the many web openings within castellated and cellular beams, horizontal and vertical shear is a more serious issue than in standard steel shapes. The main reason that this issue is magnified is because both forces must be resisted by the net section of the shape.

When a horizontal shear force passes through the web posts of a castellated or cellular beam, the web posts can buckle. Web openings for castellated beams and cellular beams are shaped differently, therefore the web posts are also shaped differently. As a result, each require a different set of equations to calculate the maximum horizontal shear force the beam can take before the web posts will buckle. For castellated beams, equations for calculating the buckling capacity were obtained through destructive testing done by Dr. Redwood at McGill University. For cellular beams, buckling capacity equations were obtained by the Steel Construction Institute of the United Kingdom.

Lateral torsional buckling, flange local bucking, and tension flange yielding for castellated and cellular beams are addressing in Chapter F of the AISC Steel Manual. The main difference between castellated/cellular beams and standard steel shapes is that the web cannot contribute to the stability of the member as a result of the many web openings along its length.

Deflection is not likely to control the design as castellated and cellular beams have a higher span to depth ratio than standard steel shapes. Although shear deformations around web openings cause additional deflection, it is not typically significant unless the span is very short or very heavy concentrated loads are applied.

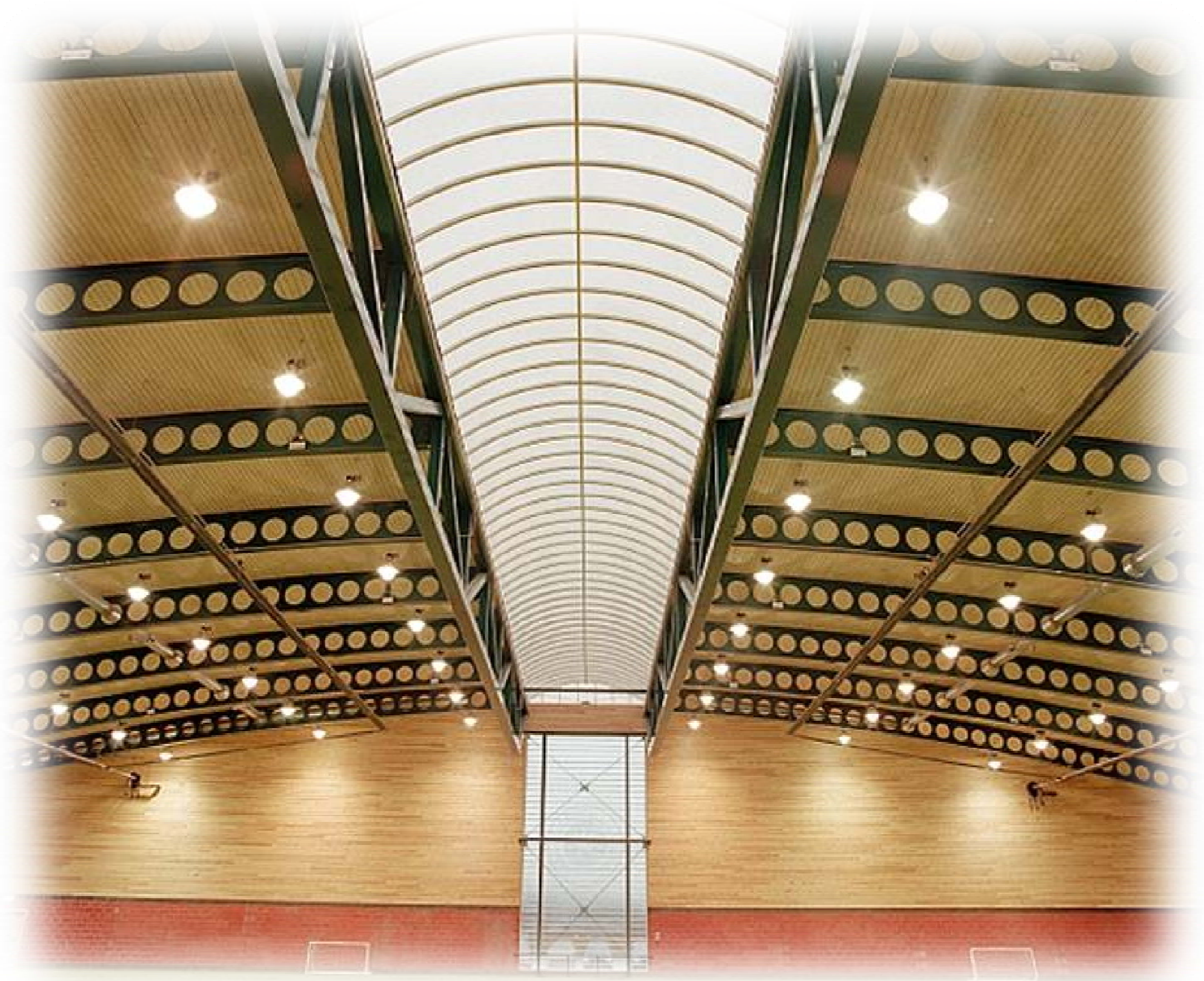
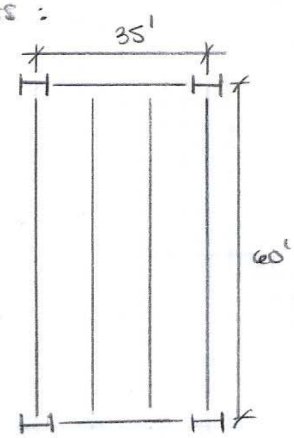


Photo courtesy of CMC Steel Products

Input for Composite Cellular Beam Software

COMPOSITE CELLULAR BEAM DESIGN

INPUT FOR SOFTWARE COURTESY OF CMC STEEL PRODUCTS :



CELLULAR BM : SPACING = 11.67'

SPAN = 60'

LL = 60psf (11.67') = 700plf

DL = 30psf (11.67') = 350plf

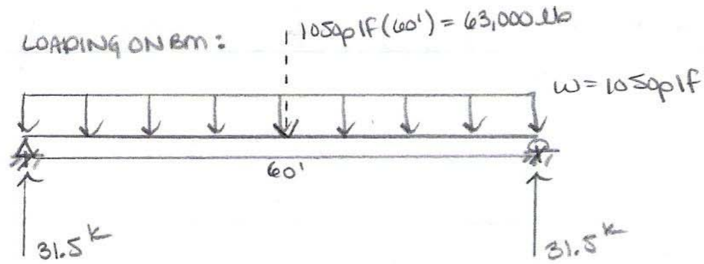
SOFTWARE INCLUDES SELF WT. ONCE BM SIZE IS CHOSEN

DO NOT FACTOR LOADS — SOFTWARE DESIGNS IN ASD

CELLULAR GIRDER : SPACING = 60'

SPAN = 35'

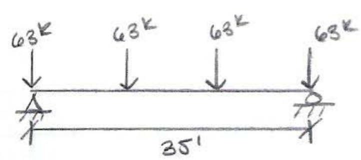
LOADING ON BM :



LOADING ON GIRDER :

$31.5^k \times 2 = 63^k$

BMS FRAME INTO GIRDER ON EA. SIDE



$$\% DL = \frac{(350\ plf)(60')/2}{(1050\ plf)(60')/2} = 33\%$$

% PRE-COMPOSITE = % DL = 33% [WILL NOT HAVE LL ON STRUCTURE UNTIL SYSTEM IS COMPOSITE]

Composite Cellular Beam Design

CELLULAR BEAM INFORMATION		
Job Name	TEST	
Beam Mark #	LB1	
Span	60.000	ft
Spac. Left	11.670	ft
Spac. Right	11.670	ft
Mat. Strength-Fy	50	ksi
Cellular Beam	LB27X35/55	
Root Beams (T/B)	W18X35	W18X55
d	17.7	18.11
bf	6	7.53
tf	0.425	0.63
tw	0.3	0.39

CELLULAR PARAMETERS:		
Min. Hole Diameter	13.60	in
Max. Hole Diameter	23.75	in
STD Hole Diameter Do	17.75	in
STD Hole Spacing S	25.750	in
Web Post Width "e"	8.000	in
S / Do	1.45	
Gross Depth "dg"	25.83	in
dg / Do	1.455	
Cutting Loss	0.953	
dt top	3.936	in
dt bot	4.141	in

LOADING INFORMATION				
Uniform Distributed Loads				
Live Load	700	plf	Pre-comp %	0%
Dead Load	350	plf	Pre-comp %	85%
Concentrated Point Loads				
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION			
Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	3500	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	150	stud ht. (in)	5 1/2
conc. above deck - tc (in)	4 1/2	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	6	STUD SPACING:	
		N=42,Uniformly Dist.	

RESULTS		
Failure Mode	Interaction	Status
Bending	0.942	OK
Web Post	0.876	OK
Shear	0.605	OK
Concrete	0.400	OK
Pre-Comp.	0.692	OK
Overall	0.942	OK

WARNINGS

DEFLECTION		
Pre-Composite Deflection	2.868	=L/251
Live Load Deflection	1.716	=L/419

EXPAND'D. SXN. PROP'S		
Avg. wt.	45.00	plf
Anet	9.79	in^2
Agross	15.92	in^2
Ix net	1357	in^4
Ix gross	1542	in^4
Ix critical	1405	in^4
Min Sx net	85	in^3
Min Sx gross	103	in^3
Min Sx critical	88	in^3
rx min	9.84	in
Iy net	30	in^4
Sy net	10.03	in^3

COMPOSITE SXN. PROP'S		
n	8.438	
beffec.	140.040	in
Actr	74.686	in^2
N.A. ht.	27.610	In Deck
Itr	5251	in^4
Ieffec.	5251	in^3
Sxconc	1112.986	in^3
Sxsteel	190.175	in^3

CONSTRUCTION BRIDGING	
End Connection type	Double clip
Min. No. Of Bridging Rows	1
Max. Bridging Spacing (ft)	40

Stnd " Do " & " S " | To Help Sheet | Find Lightest Cellular Beam

CMC Steel Products

Composite Cellular Girder Design

CELLULAR BEAM INFORMATION		
Job Name	TEST	
Beam Mark #	LB1	
Span	35.000	ft
Spac. Left	60.000	ft
Spac. Right	60.000	ft
Mat. Strength-Fy	50	ksi
Cellular Beam	LB36X117	
Root Beams (T/B)	W24X117	W24X117
d	24.26	24.26
bf	12.8	12.8
tf	0.85	0.85
tw	0.55	0.55

CELLULAR PARAMETERS:		
Min. Hole Diameter	18.42	in
Max. Hole Diameter	32.25	in
NS Hole Diameter Do	24.75	in
NS Hole Spacing S	33.000	in
Web Post Width "e"	8.250	in
S / Do	1.33	
Gross Depth "dg"	35.93	in
dg / Do	1.452	
Cutting Loss	0.708	
dt top	5.589	in
dt bot	5.589	in

LOADING INFORMATION				
Uniform Distributed Loads				
Live Load	0	plf	Pre-comp %	0%
Dead Load	0	plf	Pre-comp %	85%
Concentrated Point Loads				
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	63.00	11.67	33%	33%
P2	63.00	23.34	33%	33%
P3	63.00	0.00	33%	33%
P4	63.00	35.00	33%	33%

COMPOSITE INFORMATION			
Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	3500	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	150	stud ht. (in)	5 1/2
conc. above deck - tc (in)	4 1/2	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	6	STUD SPACING:	
		N=114, Spacing Req'd	

RESULTS		
Failure Mode	Interaction	Status
Bending	0.999	OK
Web Post	0.860	OK
Shear	0.575	OK
Concrete	0.379	OK
Pre-Comp.	0.528	OK
Overall	0.999	OK

WARNINGS

DEFLECTION		
Pre-Composite Deflection	0.292	=L/1438
Live Load Deflection	0.258	=L/1626

EXPAND'D. SXN. PROP'S		
Avg. wt.	117.00	plf
Anet	27.05	in^2
Agross	40.66	in^2
Ix net	7860	in^4
Ix gross	8555	in^4
Ix critical	8060	in^4
Min Sx net	438	in^3
Min Sx gross	476	in^3
Min Sx critical	449	in^3
rx min	14.50	in
Iy net	297	in^4
Sy net	46.44	in^3

COMPOSITE SXN. PROP'S		
n	8.438	
beffec.	105.000	in
Actr	55.998	in^2
N.A. ht.	32.684	In Steel
Itr	17476	in^4
Ieffec.	17476	in^3
Sxconc	1793.612	in^3
Sxsteel	534.691	in^3

CONSTRUCTION BRIDGING	
End Connection type	Double clip
Min. No. Of Bridging Rows	0
Max. Bridging Spacing (ft)	62

Stnd " Do " & " S " | To Help Sheet | Find Lightest Cellular Beam

CMC Steel Products

-End of Section-

APPENDIX D: TWO-WAY FLAT SLAB ANALYSIS

Photos courtesy of loftsboston.com
and excelsiorlofts.com

TWO-WAY FLAT SLAB

ASSUMPTIONS: $f'_c = 5000psi$ $f_y = 60{,}000 psi$ ASSUME 18"x18" COLS

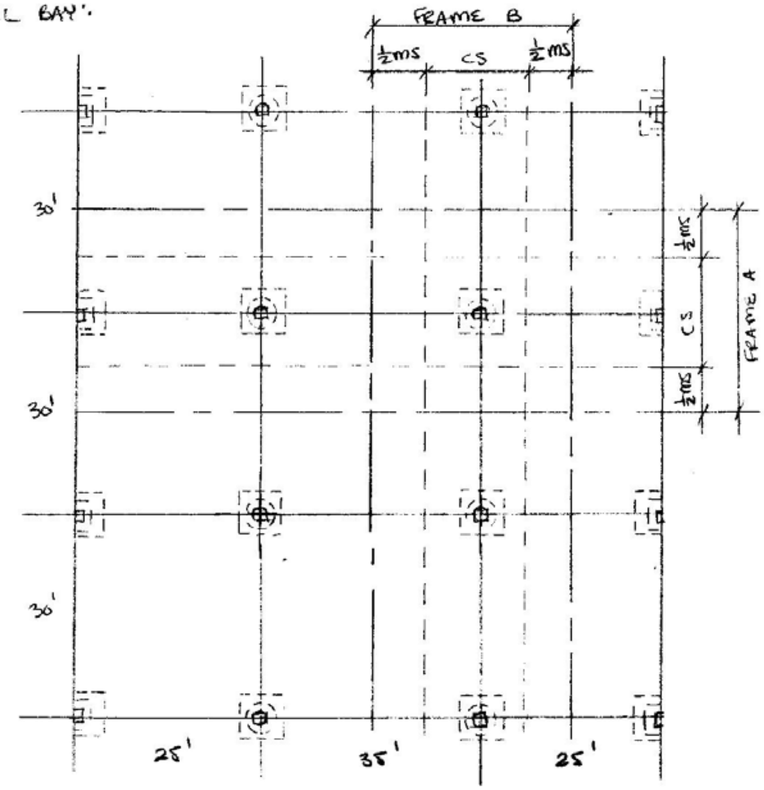
$w_c = 150 pcf$ STORY HT. = 14'

LIVE LOAD = 60 psf (HOSPITAL)

PARTITIONS = 20psf

SUPER IMPOSED DL = 10 psf

TYPICAL BAY:



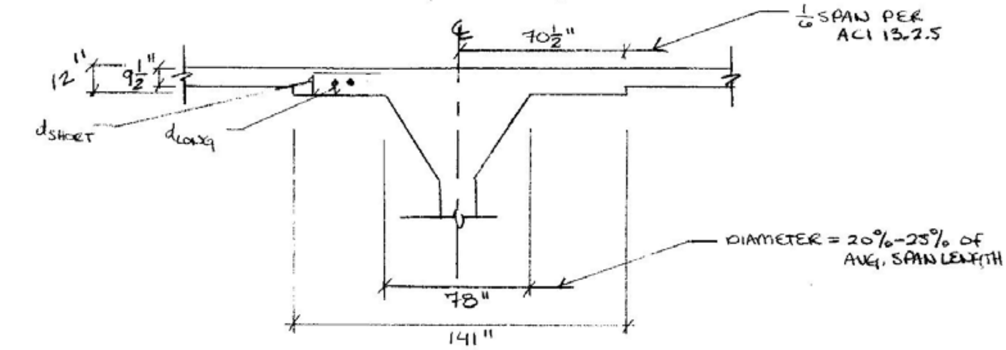
MINIMUM SLAB THK BASED ON DEFLECTION: (ACI 9.5.3.3 TABLE 9.5(C))

$t_{min} = \ell n/36$ [MUST SIZE DROP PANEL + COL CAPITOLS TO OBTAIN ℓn]

ℓn = FACE TO FACE OF COL CAPITOLS

TABLE 9.5C GIVES THICKNESS LIMITATIONS BASED UPON DEFLECTION CRITERIA, THEREFORE DEFLECTION IS OK BY INSPECTION.

SIZE COL CAPITOLS + DROP PANELS: (ACI 13.2.5)



$\frac{1}{6}$ SPAN = $\frac{1}{6}(35 \times 12) = 70''$ USE 70.5" SO TOTAL DROP WIDTH = 11.75' = 141"

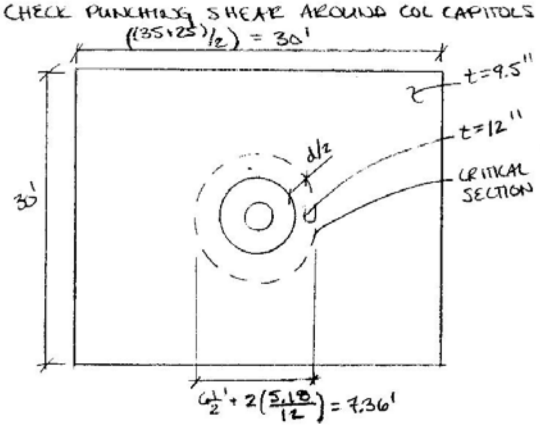
DIAMETER OF COL CAPITOL = $0.20\left(\frac{30+35}{2}\right) = 6.5' = 78''$

l_n = FACE TO FACE OF COL CAPITOLS = $35' - 6.5' = 28.5' = 342''$

$t_{SLAB} = l_n/36 = 342/36 = 9.5''$

$t_{DROP} = 1/4\ t_{SLAB} = 1/4(9.5'') = 2.375''$ ∴ USE $2\frac{1}{2}''$ } 12" THK @ DROP PANEL

CHECK PUNCHING SHEAR AROUND COL CAPITOLS



$d_{LONG} = 12'' - 3/4'' - 1/2(0.875'')$
$= 10.8''$

$d_{SHORT} = 10.8'' - 0.875'' = 9.9''$

$d_{AVG} = \frac{(10.8+9.9)}{2} = 10.35''$

$d_{AVG}/2 = 10.35/2 = 5.18''$

PERIMETER, b_o

$b_o = \pi(7.36 \times 12) = 277''$

$b_o/d = 277/10.25 = 26.8$

$\beta_c = 1.0$ FOR CIRC. COLS

$\alpha_s = 40$ FOR INT. COLS

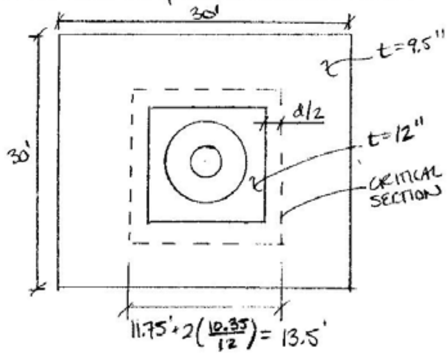
$w_u = 1.2\left(9.5/12\,(150) + 30\right) + 1.6(60) = 275\,psf$

$V_u = 0.275\left(30 \times 30 - \pi(7.36)^2/4\right) = 236^k$ = SHEAR PRESENT

$$V_c = \min\begin{cases} 4\sqrt{f'_c}\,b_o d = 4\sqrt{5000}\,(277)(10.25)/1000 = 811^k \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\,b_o d = (2+4)\sqrt{5000}\,(277)(10.25)/1000 = 1216^k \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\,b_o d = \left(\frac{40}{27.0} + 2\right)\sqrt{5000}\,(277)(10.35)/1000 = 706^k \Leftarrow \text{CONTROLS} \end{cases}$$

$\phi V_c = 0.75(706^k) = 530^k > V_u = 236^k$ ✓ OK

CHECK PUNCHING SHEAR AROUND DROPS:



$d_{AVG} = 10.35''$

PERIMETER, b_o

$b_o = 13.5(4)(12) = 648''$

$b_o/d = 648/10.35 = 62.6$

$V_u = 0.275\text{ ksf}(30\times30 - 13.5(13.5)) = 197^k$

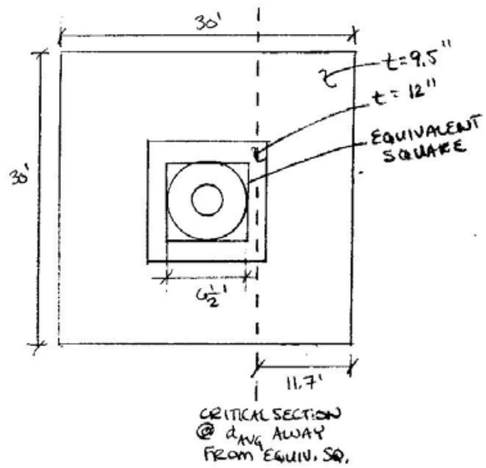
$\beta_c = 1.0$ FOR CIRC. COLS

$\alpha_s = 40$ FOR INT. COLS

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}(648)(10.35)/1000 = 1897^k \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = (2+4)(\sqrt{5000})(648)(10.35)/1000 = 2845^k \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{62.6} + 2\right)(\sqrt{5000})(648)(10.35)/1000 = 1252^k \Longleftarrow \text{CONTROLS} \end{cases}$$

$\phi V_c = 0.75(1252) = 939^k > V_u = 197^k$ ✓ OK

CHECK WIDE BM ACTION:



$d_{AVG} = 10.35''$

DIAM. OF EQUIV. SQ = a^2

$a^2 = \frac{\pi(6\frac{1}{2})^2}{4} = 33.2'$

$a = 5.76'$

CRITICAL SECTION —

$15' - \frac{5.76'}{2} - \frac{10.35}{2(12)} = 11.7'$

$V_u = 0.275\text{ksf}(11.7')(30') = 96.5^k$

$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{5000}(30\times12)(10.35-2.5)/1000 = 400^k$

$\phi V_c = 0.75(400^k) = 300^k > V_u = 96.5^k$ ✓ OK

FIND MOMENTS AT CRITICAL SECTIONS:

FRAME A: $M_o = \frac{1}{8} w_u \ell_2 \ell_1 \left(1 - \frac{2c}{3\ell_1}\right)^2$

$w_u = 1.2\left[\frac{9.5}{12}(150)+30\right] + 1.6(60) = 0.275 \text{ ksf}$

$M_o = \frac{1}{8}(0.275)(30)(35)^2\left(1 - \frac{2(6.5)}{3(35)}\right)^2 = 970^{1k}$

FRAME B: $M_o = \frac{1}{8}(0.275)(35)(30)^2\left(1 - \frac{2(6.5)}{3(30)}\right)^2 = 793^{1k}$

MOMENT DISTRIBUTION: (ACI 13.6.3.2) NO EDGE BMS

MOMENT	FRAME A	FRAME B	
M^-	631^{1k}	515^{1k}	$0.65M_o$
M^+	339^{1k}	278^{1k}	$0.35M_o$

ACI 13.6.4: NO EDGE BMS

$\alpha \ell_2/\ell_1 = 0$

75% M^- TO CS 25% M^- TO MS

60% M^+ TO CS 40% M^+ TO MS

SUMMARY OF MOMENTS:

FRAME A: TOTAL WIDTH = 30'; CS = 15'; MS = 15'

TOTAL MOMENT	-631	+339	-631
CS SLAB	-473	+203	-473
MS SLAB	-158	+136	-158

FRAME B: TOTAL WIDTH = 30'; CS = 15'; MS = 15'

TOTAL MOMENT	-515	+278	-515
CS SLAB	-386	+167	-386
MS SLAB	-129	+111	-129

DETERMINE REINFORCING:

		FRAME A				FRAME B			
		CS		MS		CS		MS	
ITEM	DESCRIPTION	m^-	m^+	m^-	m^+	m^-	m^+	m^-	m^+
1	m_n (ft-k)	−473	+203	−158	+136	−386	+167	−129	+111
2	SLAB WIDTH, b (in)	180	180	180	180	180	180	180	180
3	EFF. DEPTH, d (in)	10.8	8.3	8.3	8.3	9.9	7.4	7.4	7.4
4	$m_u = m_n/\phi = m_n/0.9$	−526	+226	−176	+151	−429	+186	−143	+123
5	$m_n \times 12/b$ (k·in/in)	−31.5	+13.5	−10.5	+9.06	−25.7	+11.1	−8.60	+7.40
6	$R = m_u/(bd^2)$	301	219	170	146	292	226	157	150
7	ρ (INTERPOLATION TABLE A-5a NDO TEXTBOOK)	0.0052	0.0038	0.0029	0.0025	0.0051	0.0039	0.0027	0.0026
8	$A_s = \rho bd$ (in²)	10.1	5.68	4.33	3.74	9.09	5.19	3.60	3.46
9	$As_{min} = 0.0018bt$	3.71	3.08	3.08	3.08	3.71	3.08	3.08	3.08
10	N = LARGER $A_s/0.60$	17	10	8	7	16	9	6	6
11	N = $\frac{\text{WIDTH OF STRIP}}{2t}$	10	10	10	10	10	10	10	10

EFFECTIVE DEPTHS:

FRAME A CS — $d = d_{LONG} = 10.8$ (FOR m^-)

$d = 9.5'' - 0.75'' - \frac{1}{2}(0.875'') = 8.3''$ (FOR m^+)

FRAME A MS — $d = 8.3''$ (FOR m^- AND m^+)

FRAME B CS — $d = 10.7'' - 0.875'' = 9.9''$ (FOR m^-)

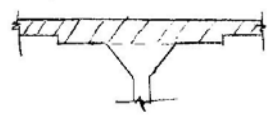
$d = 8.3'' - 0.875'' = 7.4''$ (FOR m^+)

FRAME B MS — $d = 7.4''$ (FOR m^- AND m^+)

As_{min} — OBTAINING bt:

FOR CS NEG MOMENTS IN BOTH FRAMES,

$bt = 11.75'(12)(12') + 9.5(15 \times 12 - 11.75(12)) = 2063\ in^2$



FOR ALL OTHER MOMENTS,

$bt = 15 \times 12 \times 9.5 = 1710\ in^2$

-End of Section-

FINAL DESIGN: $t_{SLAB} = 9.5''$ $DIAMETER_{COL\ CAP} = 6.5'$

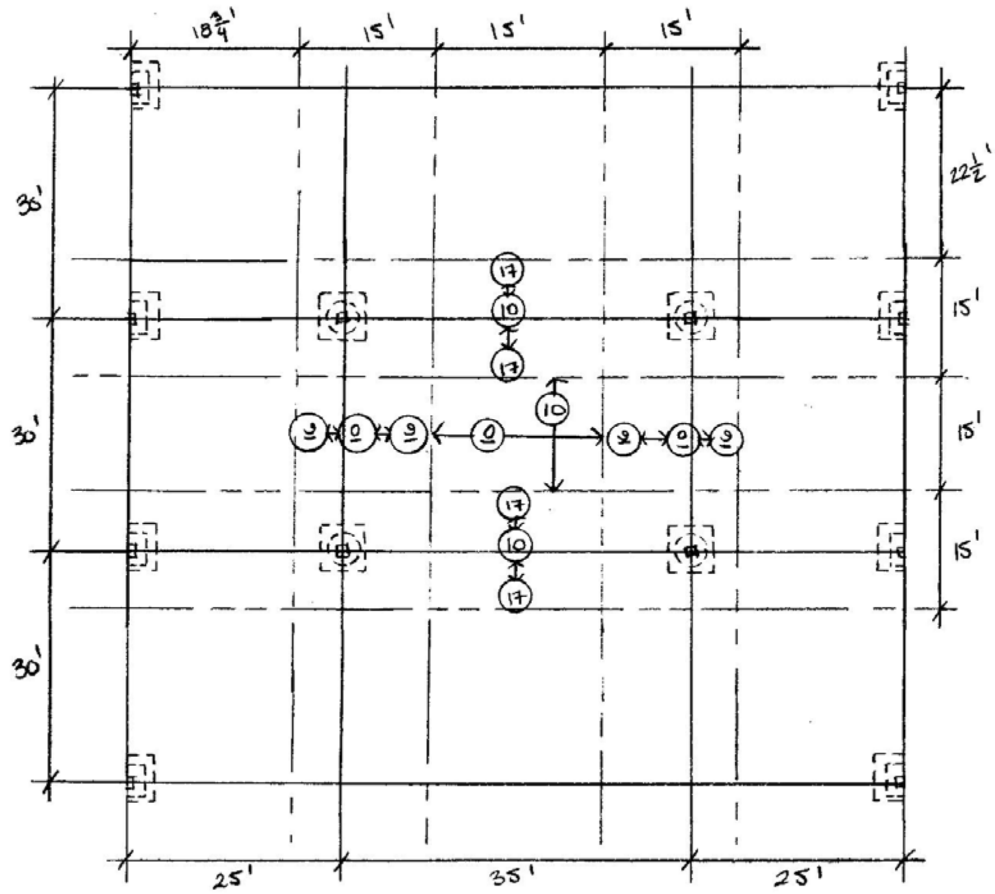
$t_{DROP} = 2.5''$ $WIDTH_{DROP} = 11.75'$

NOTE: ALL REBAR #7 BARS

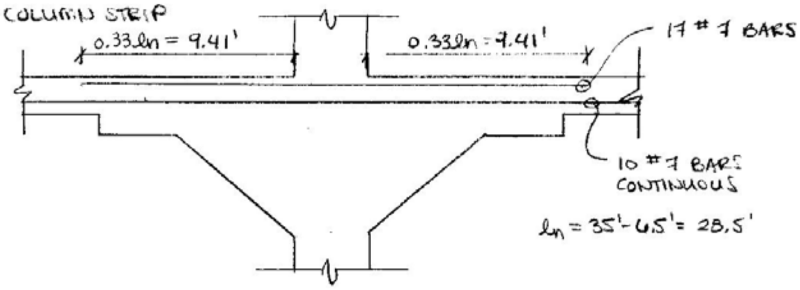
QUANTITY OF BARS IN A SECTION INDICATED IN CIRCLE

$f'_c = 5000$ psi

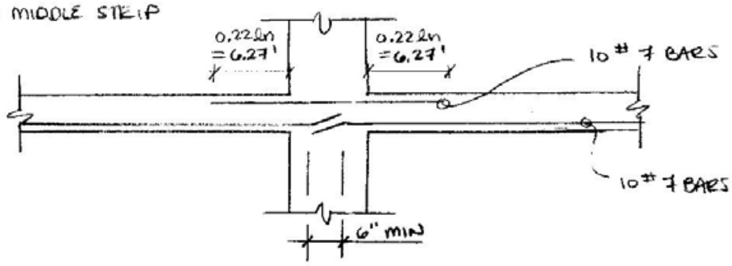
ALL COLUMNS 18"x18"



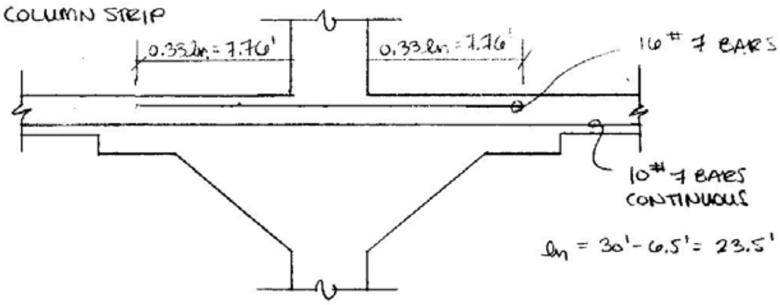
FRAME A: COLUMN STRIP



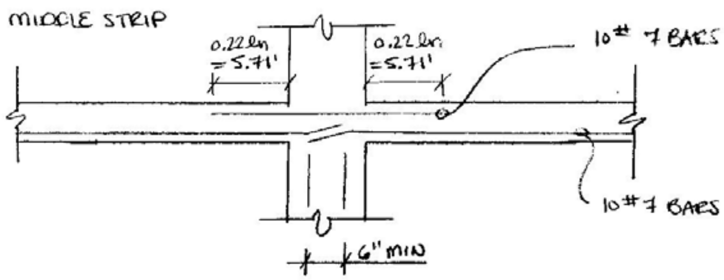
FRAME A: MIDDLE STRIP



FRAME B: COLUMN STRIP



FRAME B: MIDDLE STRIP



-End of Section-

APPENDIX E: TWO-WAY POST TENSIONED SLAB ANALYSIS

Photos courtesy of adaptsoft.com,
suncoast-pt.com, and
imgs.ebuild.com

TWO-WAY POST TENSIONED SLAB

ASSUMPTIONS: $f'_c = 5000$ psi $f_y = 60{,}000$ psi ASSUME 18"×18" COLS

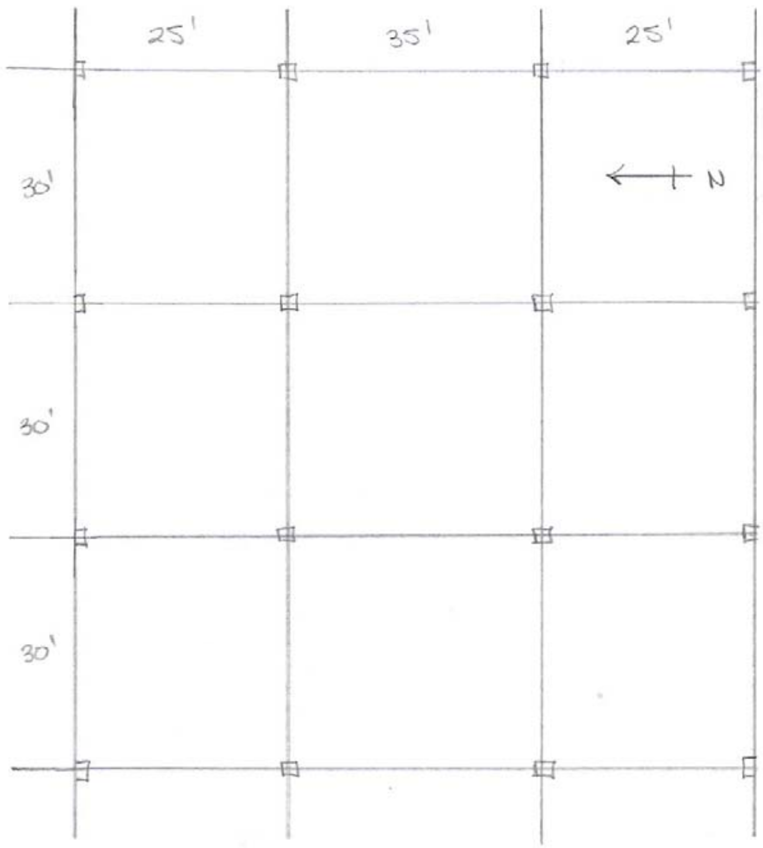
$w_c = 150$ pcf STORY HT = 14'

LIVE LOAD = 60 psf (HOSPITAL)

PARTITIONS = 20 psf

SUPER IMPOSED DL = 10 psf

TYPICAL BAY:



ADDITIONAL MATERIALS:

$f'_{ci} = 3000$ psi

UNBONDED TENDONS — ½"ϕ, 7-WIRE STRANDS $A = 0.153 in^2$ $f_{pu} = 270$ ksi

ESTIMATED PRESTRESSED LOSSES = 15 ksi

$f_{se} = 0.7(270ksi) - 15ksi = 174 ksi$

$P_{eff} = A(f_{se}) = 0.153in^2(174ksi) = 26.6\ ^k/TENDON$

DETERMINE PRELIMINARY SLAB THK:

$L/h = 45$ SPAN = 35'

$h = (35 \times 12)/45 = 9.33''$ ∴ TRY 9.5" SLAB

SELF WT. = $9.5''(150pcf)/12 = 119\ psf$

LOADING:

DL = 119psf   SUPERIMPOSED = 10psf   LL = 60psf

LL REDUCTION: $LL = L_o\sqrt{0.25 + 15/\sqrt{A_I}}$

TRIB AREA = $\left(\frac{25+35}{2}\right)(30) = 900ft^2$

INFLUENCE AREA = $4(900ft^2) = 3600ft^2$

$LL = L_o\sqrt{0.25 + 15/\sqrt{3600}} = 0.707$

$LL = 0.707(60psf) = 42.4\ psf$

DESIGN OF E-W INTERIOR FRAME:

USING EQUIVALENT FRAME METHOD, ACI 13.7 (EXCLUDING 13.7.7.4-5)

TOTAL BAY WIDTH = 30'

IGNORE COL STIFFNESS FOR SIMPLICITY

NO PATTERN LOADING REQ'D SINCE $LL/DL < 3/4$ (ACI 13.7.6.2)

CALCULATE SECTION PROPERTIES:

TWO-WAY SLAB MUST BE DESIGNED AS CLASS U (ACI 18.3.3)

$A = bh = (30 \times 12)(9.5) = 3420\ in^2$

$S = bh^2/6 = (30 \times 12)(9.5)^2/6 = 5415\ in^3$

DESIGN PARAMETERS:

ALLOWABLE STRESSES (CLASS U) —

AT TIME OF JACKING (ACI 18.4.1)

$f'_{ci} = 3000psi$

$C = 0.60f'_{ci} = 0.6(3000psi) = 1800\ psi$

$T = 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164\ psi$

AT SERVICE LOADS (ACI 18.4.2 + 18.3.3)

$f'_c = 5000\ psi$

$C = 0.45f'_c = 0.45(5000psi) = 2250\ psi$

$T = 6\sqrt{f'_c} = 6\sqrt{5000} = 424\ psi$

AVERAGE COMPRESSION LIMITS (ACI 18.12.4) —

$P/A = 125$ psi MIN

$= 300$ psi MAX

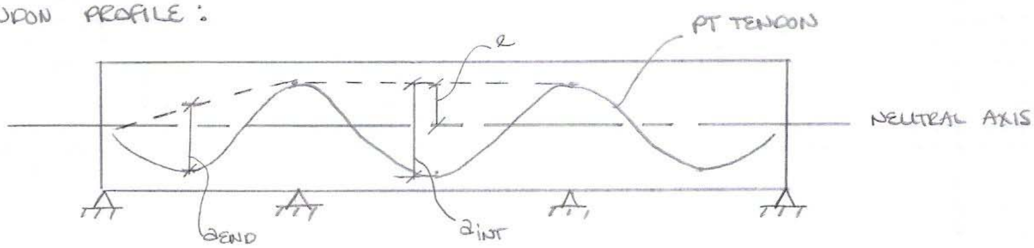
TARGET LOAD BALANCES = 75%

$0.75(w_{DL}) = 0.75(119\text{ psf}) = 89.3\text{ psf}$

COVER REQUIREMENTS — 2 HOUR FIRE RATING

RESTRAINED SLABS – 3/4" BOTTOM

TENDON PROFILE:



TENDON ORDINATE	TENDON (CG) LOCATION	
EXT. SUPPORT ANCHOR	4.75"	t/2
INT. SUPPORT – TOP	8.5"	t−1"
INT. SPAN – BOTT.	1.0"	
END SPAN – BOTT.	1.75"	

$a_{INT} = 8.5'' - 1.0'' = 7.5''$

$a_{END} = (4.75+8.5)/2 - 1.75'' = 4.875''$

PRESTRESSED FORCE REQ'D TO BALANCE 75% SW DL:

$w_b = 0.75\, w_{DL} = 89.3\text{ psf}\,(30\text{ft}) = 2.679\text{ klf}$

$P = \dfrac{w_b L^2}{8\, a_{INT}} = \dfrac{2.679\,(35)^2}{8\,(7.5)/12} = 656^k$ = FORCE IN TENDONS TO COUNTERACT LOAD IN INTERIOR BAY

CHECK PRECOMPRESSION ALLOWANCE:

DETERMINE # TENDONS TO ACHIEVE 656^k

$\#\text{ TENDONS} = 656^k / 26.6^k/\text{TENDON} = 25$ TENDONS

ACTUAL FORCE FOR BANDED TENDONS —

$P_{ACTUAL} = (25)(26.6^k/\text{TENDON}) = 665^k$

ADJUST UNBALANCED LOAD —

$w_b = 665/656\,(2.679) = 2.716\text{ klf}$

DET. ACTUAL PRECOMPR. STRESS —

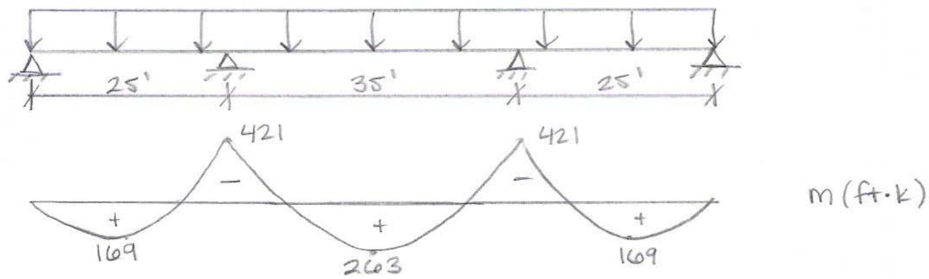
$P_{ACTUAL}/A = \dfrac{665(1000)}{9.5 \times 30 \times 12} = 194\text{ psi}$ > 125 psi MIN
< 300 psi MAX ✓OK

$\boxed{\text{EFFECTIVE PRESTRESS FORCE} = 665^k \text{ FOR E-W FRAME}}$

CHECK SLAB STRESSES:

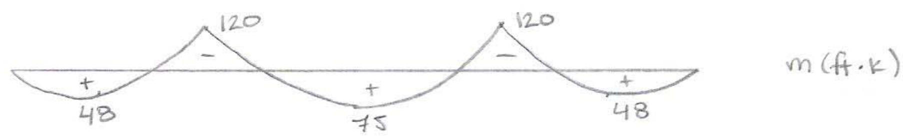
DEAD LOAD MOMENTS —

$w_d = (30\,ft)(119psf + 10psf + 20psf) = 4.47\,klf$



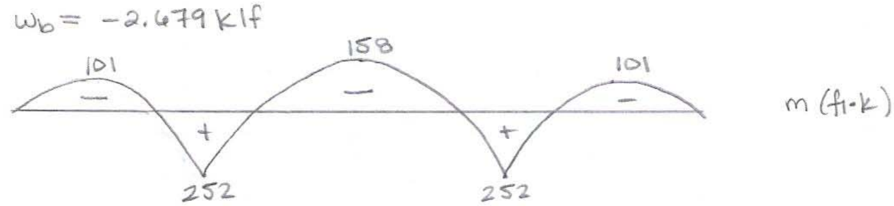
LIVE LOAD MOMENTS —

$w_{LL} = (30ft)(42.4psf) = 1.27\,klf$



TOTAL BALANCED MOMENTS —

$w_b = -2.679\,klf$



MOMENT DIAGRAMS OBTAINED USING SAP 2000 —
MODELS AVAILABLE UPON REQUEST

STAGE 1: STRESSES IMMED. AFTER JACKING (DL + PT)

MIDSPAN

$f_{TOP} = (-M_{DL} + M_b)/S - P/A$

$= 12000(-263 + 158)/5415 - 194 = -427\,psi < 0.6f'_{ci} = 1800psi$

$f_{BOT} = (+M_{DL} - M_b)/S - P/A$

$= 12000(263 - 158)/5415 - 194 = 38.7psi < 3\sqrt{f'_{ci}} = 164psi$ ✓OK

SUPPORT

$f_{TOP} = (+M_{DL} - M_b)/S - P/A$

$= 12000(421 - 252)/5415 - 194 = 180\,psi < 6\sqrt{f'_{ci}} = 329psi$

$f_{BOT} = (-M_{DL} + M_b)/S - P/A$

$= 12000(-421 + 252)/5415 - 194 = -569\,psi < 0.6f'_{ci} = 1800psi$ ✓OK

STAGE 2: STRESSES @ SERVICE LOAD (DL+LL+PT)

MIDSPAN

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

$= 12000(-263-75+158)/5415 - 194 = -399\text{ psi} < 0.45f'_c = 2250\text{ psi}$

$f_{BOT} = (+M_{DL} + M_{LL} - M_b)/S - P/A$ ✓OK

$= 12000(263+75-158)/5415 - 194 = 205\text{ psi} < 6\sqrt{f'_c} = 424\text{ psi}$

SUPPORT

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

$= 12000(421+120-252)/5415 - 194 = 446\text{ psi} < 7.5\sqrt{f'_c} = 530\text{ psi}$

$f_{BOT} = (-M_{DL} - M_{LL} + M_b)/S - P/A$ ✓OK

$= 12000(-421-120+252)/5415 - 194 = -834\text{ psi} < 0.45f'_c = 2250\text{ psi}$

ULTIMATE STRENGTH:

DET. FACTORED MOMENTS — $e = a_{INT}/2 = 7.5''/2 = 3.75''$

$M_1 = Pe = 665^k(3.75'')/12 = 208^{1k}$ (PRIMARY PT MOMENT)

$M_{SEC} = M_b - M_1 = 252^{1k} - 208^{1k} = 44^{1k}$

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

$M_{u,MIDSPAN} = 1.2(169) + 1.6(48) + 1.0(22) = 302^{1k}$

$M_{u,SUPPORT} = 1.2(-421) + 1.6(-120) + 1.0(44) = -653^{1k}$

DET. MIN BONDED REINF.:

POSITIVE MOMENT REGION —

INT. SPAN: $f_T = 15\text{ psi} < 2\sqrt{f'_c} = 2\sqrt{5000} = 141\text{ psi}$

∴ NO REINF. NEEDED PER ACI 18.9.3.1

NEGATIVE MOMENT REGION —

INT. SUPPORTS: $A_{cf} = 9.5''\left(\frac{35+25}{2}\right) \times 12 = 3420\text{ in}^2$

$A_{s,min} = 0.00075A_{cf} = 0.00075(3420) = 2.6\text{ in}^2$

13 #4 TOP (2.6 in^2)

CHECK MIN REQ'D:

$\phi M_n = \phi(A_s f_y + A_{ps} f_{ps})(d - a/2)$ $d = 9.5'' - 3/4'' - 1/4'' = 8.5''$

$A_{ps} = 0.153\text{ in}^2$ (25 TENDONS) $= 3.825\text{ in}^2$

$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c bd}{300A_{ps}} = 184{,}000 + \frac{5000(30\times12)(8.5)}{300(3.825)} = 197\text{ ksi}$

$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85f'_c b} = \frac{2.6(60) + 3.825(197)}{0.85(5)(30\times12)} = 0.595''$

$\phi M_n = 0.9(2.6(60) + 3.825(197))(8.5 - 0.595/2) = 560^{1k} < 653^{1k}$

∴ REINF. FOR ULTIMATE STRENGTH CONTROLS

$653(12) = 0.9(A_{s,REQ}(60) + 3.825(197))(8.5 - 0.595/2) \Longrightarrow A_{s,REQ} = 5.13\text{ in}^2$

∴ USE 26 #4 TOP @ INT SUPPORTS E-W FRAME

DESIGN OF N-S INTERIOR FRAME :

$A = bh = (30\times12)(9.5) = 3420\,in^2$

$S = bh^2/6 = (30\times12)(9.5)^2/6 = 5415\,in^3$

$w_b = 0.75 w_{DL} = 0.75\,(119\,psf)(30') = 2.679\,klf$

$$P = \frac{w_b L^2}{8\, a_{INT}} = \frac{(2.679)(30)^2}{8\,(7.5)/12} = 482^k$$

$\#\ TENDONS = 482^k / 26.6^k/_{TENDON} = 19\ TENDONS$

$P_{ACTUAL} = 19\,(26.6^k/_{TENDON}) = 505^k$

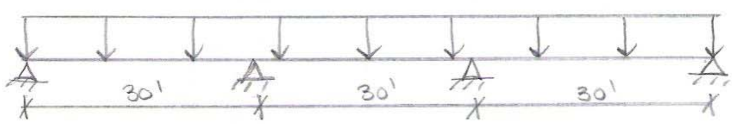
$$w_b = \frac{505}{482}(2.679\,klf) = 2.81\,klf$$

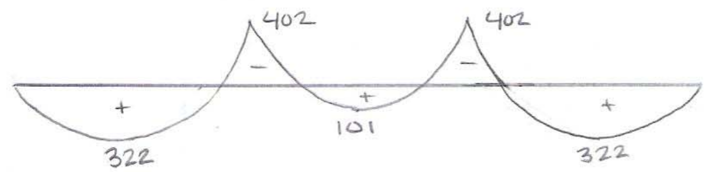
$$P/A = \frac{505\,(1000)}{9.5\times30\times12} = 148\,psi \quad \begin{array}{l} > 125\,psi\ MIN \\ < 300\,psi\ MAX \end{array} \quad \checkmark_{OK}$$

$$\boxed{\text{EFFECTIVE PRESTRESS FORCE} = 505^k \text{ FOR N-S FRAME}}$$

DEAD LOAD MOMENTS —

$w_{DL} = (30ft)(119\,psf + 10\,psf + 20\,psf) = 4.47\,klf$

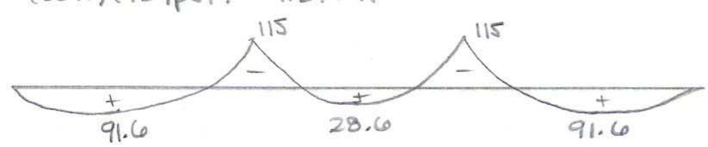




m (ft·k)

LIVE LOAD MOMENTS —

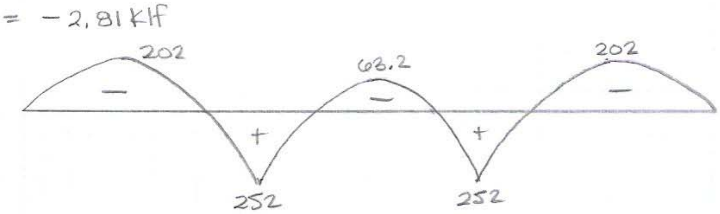
$w_{LL} = (30ft)(42.4\,psf) = 1.27\,klf$



m (ft·k)

TOTAL BALANCED MOMENTS —

$w_b = -2.81\,klf$



m (ft-k)

STAGE 1: STRESSES IMMED. AFTER JACKING

MIDSPAN
$$f_{TOP} = 12000(-101+63.2)/5415 - 148 = -232\text{ psi} < 0.6f'_{ci} = 1800\text{ psi}$$
$$f_{BOT} = 12000(101-63.2)/5415 - 148 = -64.2\text{ psi} < 0.6f'_{ci} = 1800\text{ psi}$$

✓OK

SUPPORT
$$f_{TOP} = 12000(402-252)/5415 - 148 = 184\text{ psi} < 6\sqrt{f'_{ci}} = 329\text{ psi}$$
$$f_{BOT} = 12000(-402+252/5415 - 148 = -480\text{ psi} < 0.6 f'_c = 1800\text{ psi}$$

STAGE 2: STRESSES @ SERVICE LOADS

MIDSPAN
$$f_{TOP} = 12000(-101-28.6+63.2)/5415 - 148 = -295\text{ psi} < 0.45f'_c = 2250\text{ psi}$$
$$f_{BOT} = 12000(101+28.6-63.2)/5415 - 148 = -1.0\text{ psi} < 0.45f'_c = 2250\text{ psi}$$

✓OK

SUPPORT
$$f_{TOP} = 12000(402+115-252)/5415 - 148 = 439\text{ psi} < 7.5\sqrt{f'_c} = 530\text{ psi}$$
$$f_{BOT} = 12000(-402-115+252)/5415 - 148 = -735\text{ psi} < 0.45f'_c = 2250\text{ psi}$$

ULTIMATE STRENGTH:

$$M_1 = Pe = 505^k(3.75")/12 = 157^{1k}$$
$$M_{sec} = M_b - M_1 = 252 - 157 = 95^{1k}$$
$$M_{u,MIDSPAN} = 1.2(101) + 1.6(28.6) + 1.0(63.2) = 230^{1k}$$
$$M_{u,SUPPORT} = 1.2(-402) + 1.6(-115) + 1.0(+239) = -414^{1k}$$

REINFORCEMENT:

NO REINF. NEEDED @ POSITIVE MOMENT

NEGATIVE MOMENT:

$$A_{cf} = 9.5\left(\frac{30+30}{2}\right) \times 12 = 3420\text{ in}^2$$
$$A_{s\,min} = 0.00075(3420) = 2.6\text{ in}^2$$

13 #4 TOP (2.6 in²)

CHECK MIN REQ'D:

$$A_{ps} = 0.153(19\text{ TENDONS}) = 2.907\text{ in}^2 \qquad d = 8.5"$$
$$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c bd}{300 A_{ps}} = 184{,}000 + \frac{5000(30\times12)(8.5)}{300(2.907)} = 202\text{ ksi}$$
$$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c b} = \frac{2.6(60) + 2.907(202)}{0.85(5)(30\times12)} = 0.49"$$
$$\phi M_n = 0.9(2.6(60) + 2.907(202))(8.5 - 0.49/2) = 460^{1k} > 414^{1k}$$

∴ MIN REINF. OK

∴ USE 13 #4 TOP @ INT. SUPPORT N-S FRAME

CHECK PUNCHING SHEAR :

ASSUME 18" x 18" COL

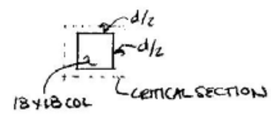
$d = 9.5" - 3/4" - 1/2(0.5") = 8.5"$

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d & = 4\sqrt{5000}\,(106)(8.5)/1000 = 255^k \Leftarrow \text{CONTROLS} \\ \left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d & = \left(2+\frac{4}{1}\right)\sqrt{5000}\,(106)(8.5)/1000 = 382^k \\ \left(\frac{\alpha_s}{b_o/d}+2\right)\sqrt{f'_c}\, b_o d & = \left(\frac{40}{12.5}+2\right)\sqrt{5000}\,(106)(8.5) = 331^k \end{cases}$$

$\alpha_s = 40$ FOR INT. COL.

$b_o = (18+8.5)4 = 106"$

$b_o/d = 106/8.5 = 12.5"$



$DL = 1.2\left[(9.5" \times 150\,pcf)/12 + 20\,psf + 10\,psf\right] = 179\,psf$

$LL = 1.6\,(60\,psf) = 96\,psf$

$\}\ 275\,psf$

$A_T = 30'\left(\frac{30'+35'}{2}\right) = 900\,ft^2$

$V_u = 0.275\,ksf\left(900 - \frac{18\times18}{144}\right) = 247^k$

$\phi V_n = 0.75(255^k) = 191^k$

$V_u = 247^k > \phi V_n = 191^k$ ∴ NEED SHEAR CAPS

$247(1000)/0.75 = 4\sqrt{5000}\; b_o\,(9.5") \Longrightarrow b_o = 123"$

$123"/4 = 18 + d \Longrightarrow d = 12.75"$

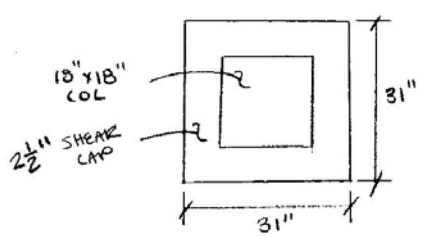
USE $d = 13"$

∴ SHEAR CAP EXTENDS $6\frac{1}{2}"$ FROM COL FACE ON ALL SIDES ; USE 31" x 31" SHEAR CAP

PER ACI 13.2.5,

$t_{CAP} \geq 1/4\, t_{SLAB}$

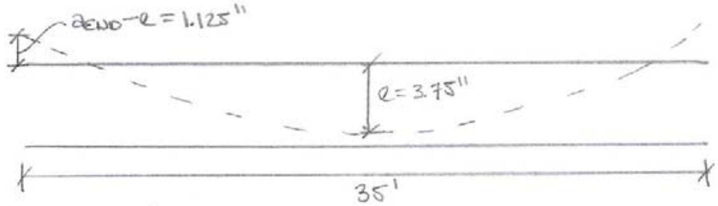
$t_{CAP} = 1/4\,(9.5") = 2.375"$ ∴ USE $2\frac{1}{2}"$



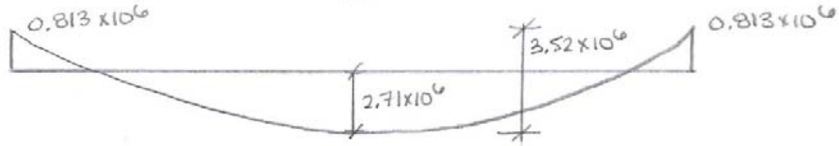
CHECK DEFLECTION: (FOR E-W FRAME)

PRESTRESSED FORCE BEFORE LOSSES $= P_i = 0.7(270\text{ ksi})(0.153\text{in}^2)(25\text{ TENDONS}) = 723^k$

PRESTRESSED FORCE AFTER LOSSES $= P_e = 665^k$



$\frac{35 \times 12}{2} = 210''$



$$\Delta_{pi} = \frac{-(3.52\times10^6 \times 210 \times 2/3 \times 210 \times 5/8) + (0.813\times10^6 \times 210 \times 105)}{(4\times10^6\text{ psi})(25{,}721\text{ in}^4)} = -0.45''$$

$$E_c = 57000\sqrt{f'_c} = 57000\sqrt{5000} = 4\times10^6\text{ psi}$$

$$I_s = \frac{(30\times12)(9.5)^3}{12} = 25{,}721\text{ in}^4$$

NEGATIVE SIGN ON Δ_{pi} INDICATES UPWARD DEFLECTION DUE TO INITIAL PRESTRESS ALONE

$$\Delta_0 = \frac{5w\ell^4}{384EI} = \frac{5(119\text{psf}\times30)(35)^4(1728)}{384(4\times10^6)(25{,}721)} = +1.17''$$

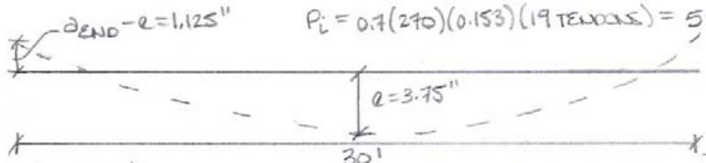
$$\Delta_{NET} = \Delta_{pi} + \Delta_0 = -0.45'' + 1.17'' = +0.72''$$

$$0.72'' = \frac{35(12)}{x} \Rightarrow x = 583$$

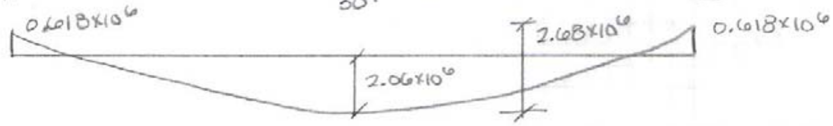
$\therefore 0.72'' = L/583$ ✓ OK

TENDONS RUNNING IN N-S FRAME WILL ALSO CAUSE UPWARD DEFLECTION—

$P_i = 0.7(270)(0.153)(19\text{ TENDONS}) = 549^k$



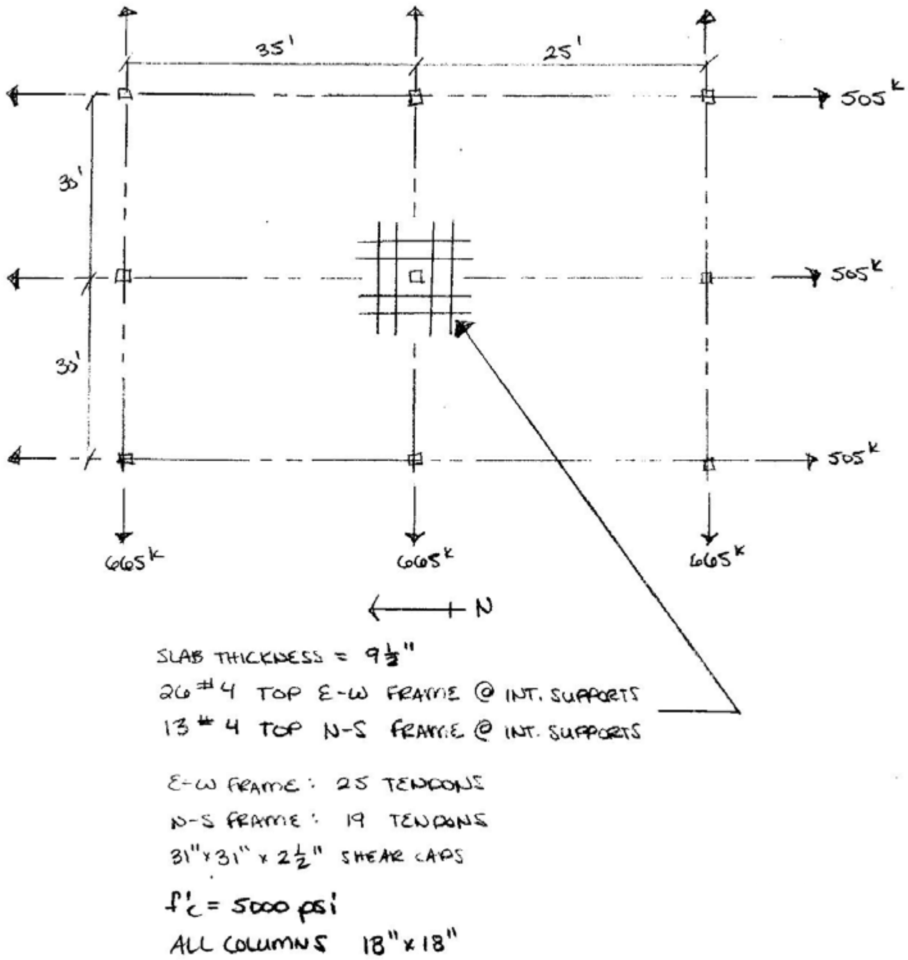
$\frac{30\times12}{2} = 180''$



$$\Delta_{pi} = \frac{-(2.68\times10^6 \times 180 \times 2/3 \times 180 \times 5/8) + (0.618\times10^6 \times 180 \times 90)}{(4\times10^6)(25{,}721\text{ in}^4)} = -0.25''$$

TOTAL NET DEFLECTION: $\Delta_{NET} = -0.25'' + (-0.45'') + 1.17'' = 0.47''$

$0.47'' = \frac{30(12)}{x} \Rightarrow x = 767$ $\quad 0.47'' = L/767$ ✓ OK



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