The New York Times Building
New York, NY

IPD/BIM Thesis
Technical Report #2

Andres R. Perez
Structural Option – Team #3

Faculty Consultant: Dr. Andres Lepage
October 30th, 2009

TABLE OF CONTENTS

EXECUTIVE SUMMARY

In the second technical report of the New York Times Building, an evaluation of alternative floor systems was conducted using a typical 40'-0"x30'-0"exterior bay in the tower. In all, the design feasibility of four systems, including the existing composite steel beam system, were investigated. The system comparison was based upon but was not limited to: system weight, fire rating, vibration susceptibility, cost, structural depth, constructability, and architectural affects. Three alternative systems which were schematically designed and compared to the existing included:

- Castellated Composite Steel Beam
- Two-Way Flat Plate
- Two-Way Post-Tensioned Slab w/ Slab Bands

The composite castellated beam design was conducted in order to compare the existing composite beam system to another viable steel design solution. The schematic design resulted in the only system lighter that the existing floor structure of the New York Times Building. Being a proprietary system, a program provided by CMC Steel Products was utilized to perform the design calculations. The design resulted in 20.65" deep beams built-up from W14x22s and W14x26s and 18.07" deep girders built-up from W12x14s and W12x19s. Though this schematic design resulted in deeper members than the existing system, the overall plenum height is likely to be less than the existing composite beam system. This is due to other trades being able to utilize the opening in the castellated member. The composite castellated steel system was found to be viable design alternative and will be investigated further.

The schematic design of a two-way flat plate system resulted in a 14" slab which created greatest self weight of all four systems. The system was found to be very inefficient and uneconomical for the large bay size which is required as part of the architecture of the building. Due to this requirement, intermediate columns cannot be added in order to reduce the system span length. Therefore, the two-way flat plate system was not found to be a viable alternative to the existing floor framing system and will not be investigated further.

The two-way post-tensioned slab system with wide-shallow-beams was investigated due to the long span capabilities. The schematic design resulted in 33 tendons distributed uniformly through an 8" slab and 31 bonded tendons through the 16"x48" slab-beam section. Though the concrete system results in a higher system self weight than with the existing steel solution, the two-way post-tension system was determined to be a feasible floor design alternative based upon long span capabilities and a thin slab thickness. Therefore, the two-way post-tensioned slab with wide-shallow-beams will be investigated further.

INTRODUCTION

The New York Times Headquarters Building (NYTB) is home to the New York Times newsroom and offices, as well as several law firms, whose offices are leased through Forest City Ratner. In collaboration with FXFOWLE Architects, the intent of the Renzo Piano Workshop was to introduce a flagship structure which promoted sustainability, lightness, and transparency. The architectural façade reflects the ever-changing environment surrounding the building, an appropriate acknowledgement of the heart of New York City.

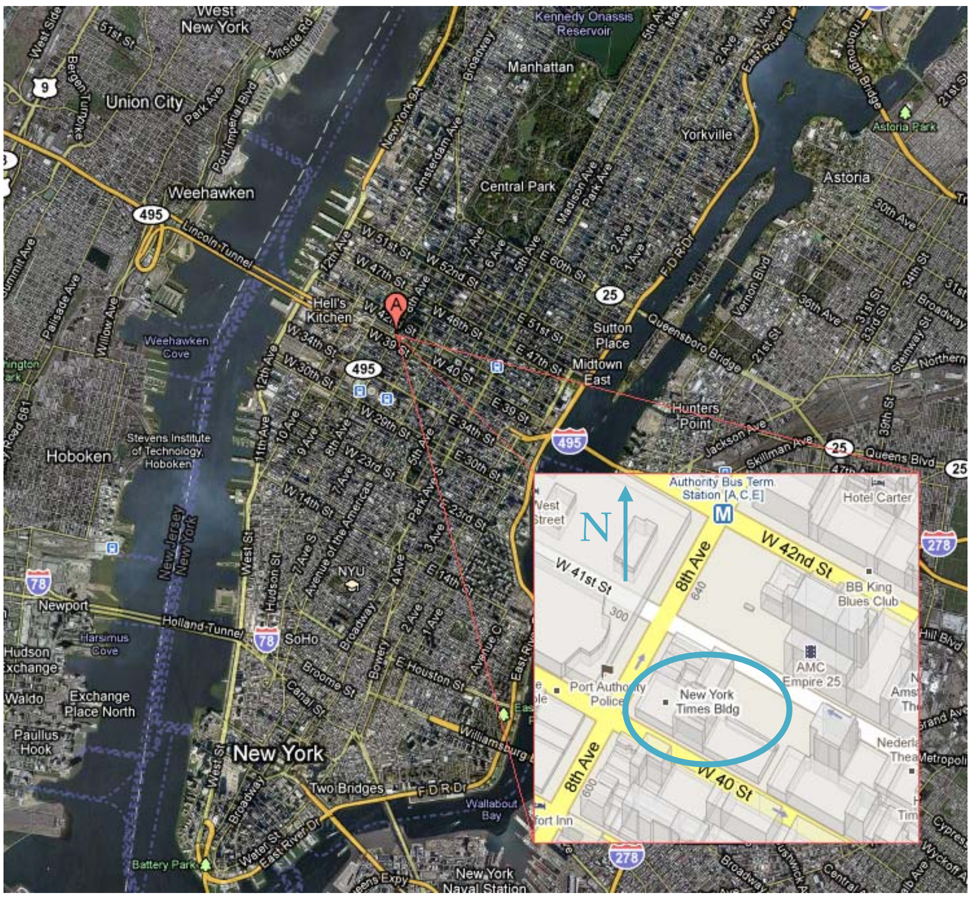


Figure 1: New York Times Building Location (Google Maps)

The 52 story, 1,500,000 square foot building rises 744 feet above Eighth Avenue between 40th and 41st Street creating a 200' x 400' footprint. The tower's 300 foot mast allows for the structure to top out at 1048 feet above ground level. The New York Times occupies the entire five-story podium of the structure, and the first 27 levels in the tower. The additional levels are the office spaces leased through Forest City Ratner. Story heights average approximately 13 feet 9 inches in the tower, lending a great view to the open office plans. At the mechanical floors on levels 28 and 51, however, the floor height is approximately 27 feet to accommodate equipment and two-story steel outriggers which link the perimeter columns to the braced framed core.

Structural System Description

Foundation

The foundation of the NYTB combines typical spread footings with caissons to achieve its maximum axial capacity. Below the building's 16-foot cellar, the tower and podium mostly bear on Medium/Hard rock with a bearing capacity of 80 ksf., Class 2-65 per the New York City Building Code. However, a core sample taken just before finalizing the site investigation report indicated that rock at the southeast corner of the tower only had a 16 ksf bearing capacity, Class 4-65. At the seven columns that fall within this area, indicated in red on Figure 2, 24-inch diameter concrete-filled steel caissons were used to replace the original foundation designs. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete.

Under the other 21 columns (indicated on Figure 2 in teal), spread footings with a compressive strength of 6,000 psi are used to support the loads. The areas depicted in blue represent the two cantilevered sections of the tower. The columns which fall in these areas do not directly transfer load to the ground which removes the need for footings at these locations.

The New York City Subway does pass the north and eastern sides of the New York Times Building. However, this is not a major site restriction since the transit system passes below Eighth Avenue and 41st Street and not directly beneath the structure. But, vibration effects on the foundation and building structure may have had an impact on the design.

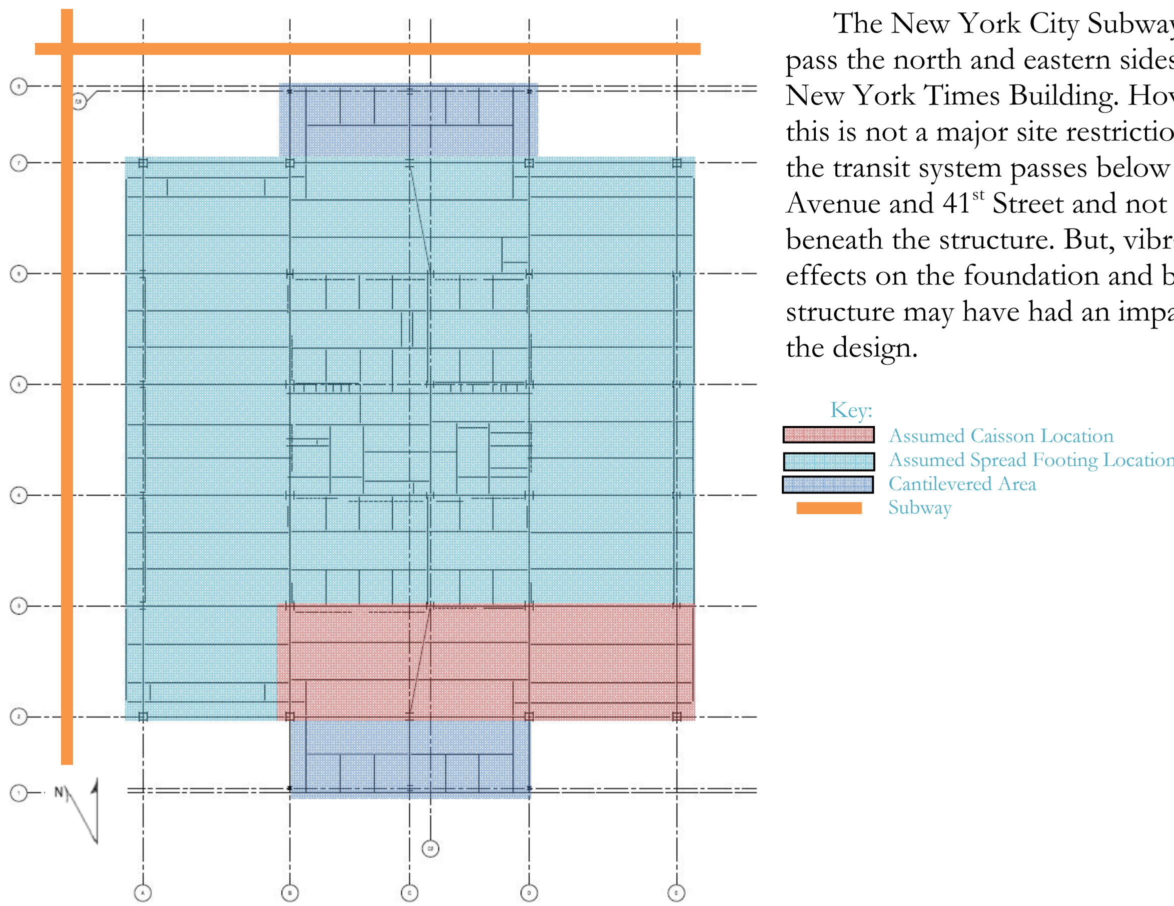


Figure 2: Foundation Locations

Columns

The 30" by 30" box columns (Figure 3) at the exterior notches of the tower consist of two 30 inch long flange plates and two web plates inset 3 inches from the exterior of the column on either side. Each web plate decreases in thickness from 7 inches as the column extends up the structure to account for the reduction in axial loads. Each flange plate decreases from 4 inches in thickness to relate to the architectural vision of the tower. Although the yield strength of the plates also varies with tower height, the strength was assumed to be a uniform 50 ksi for calculations. Interior columns are a combination of built-up sections and rolled shapes. Column locations stay consistent throughout the height of the building, and every column is engaged in the lateral system. Please refer to Figure 4 to view the column locations. Note that the unfilled boxes denote columns in the cantilevered area which do not extend to the ground.

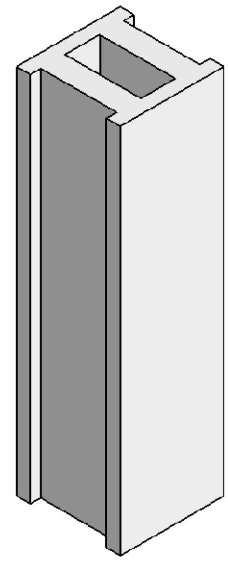


Figure 3: Box Column as Modeled in Revit Structure

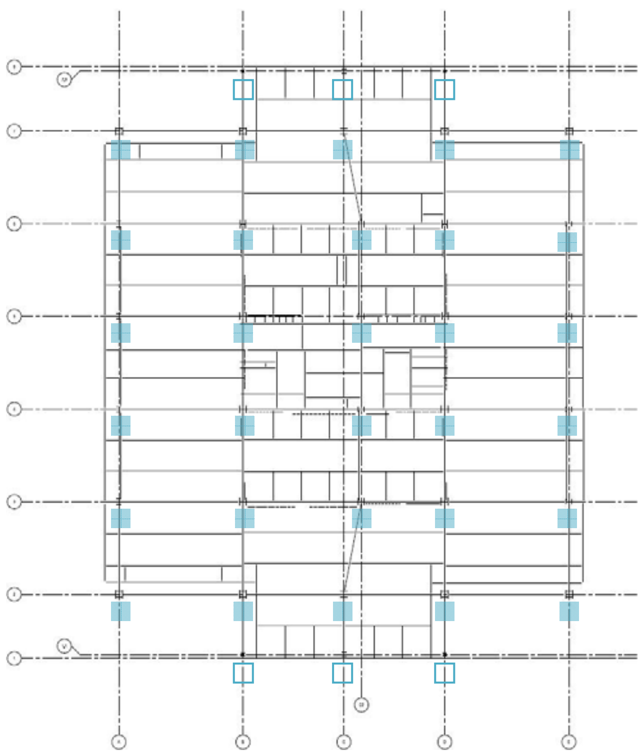


Figure 4: Tower Column Locations

Vierendeel Frame

A Vierendeel frame was used by Thornton Tomasetti as a combined solution at the 20 foot cantilever sections of the tower. Renzo Piano did not want columns obstructing the glass storefronts at the ground level, so these sections were cantilevered from the main structure. As a unique way to control deflections in the middle beams of the cantilevered section, the ladder-like moment frame engages all floors throughout the entire height of the tower. It connects to 28th and 52nd floor outriggers through the use of diagonal braces which effectively transfer loads from the frame to the core of the tower. Refer to Figure 9 on page 9 to view the brace location.

Lateral System

The main lateral load resisting system for the tower of the NYTB consists of a centralized steel braced frame core with outriggers on the two mechanical floors (Levels 28 and 51). The structural core consists of a combination of concentric and eccentric bracing which surrounds elevator shafts, MEP shafts, and stair wells. At this time, the member sizes of these braces have yet to be disclosed. The core configuration remains consistent from the ground level to the 27th floor as shown in Figure 5. But above the 28th floor, the low rise elevators were no longer required. In order to optimize the rentable space on the upper levels of the tower, the number of bracing lines in the North-South direction were reduced from two to one (Figure 6). Please refer to Figures 7 and 8 to view the typical core bracing configurations.

The outriggers on the mechanical floors consist of chevron braces (Figure 10) and single diagonal braces. The outrigger system was designed to increase the efficiency and redundancy of the tower by engaging the perimeter columns into the lateral system. Please refer to page 9 to view the framing plans and bracing elevations of the outrigger system.

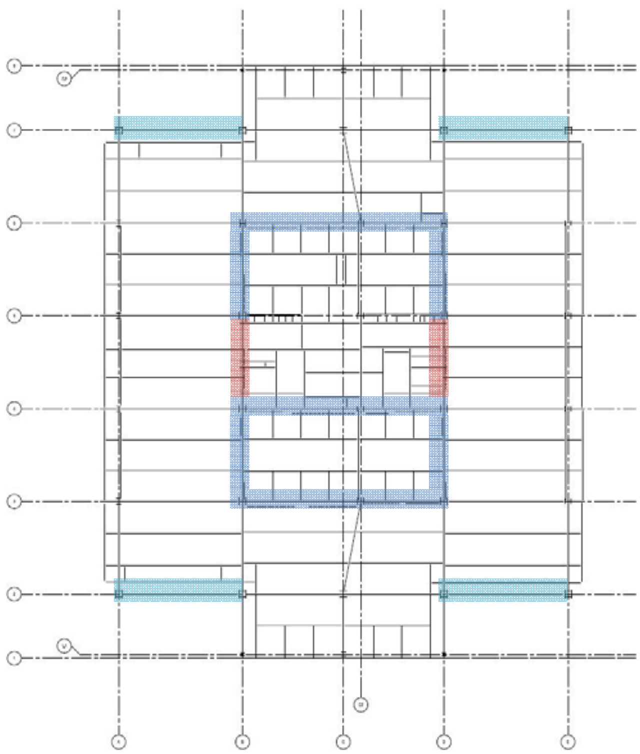


Figure 5: Typical Lateral System (Floors 1-27)

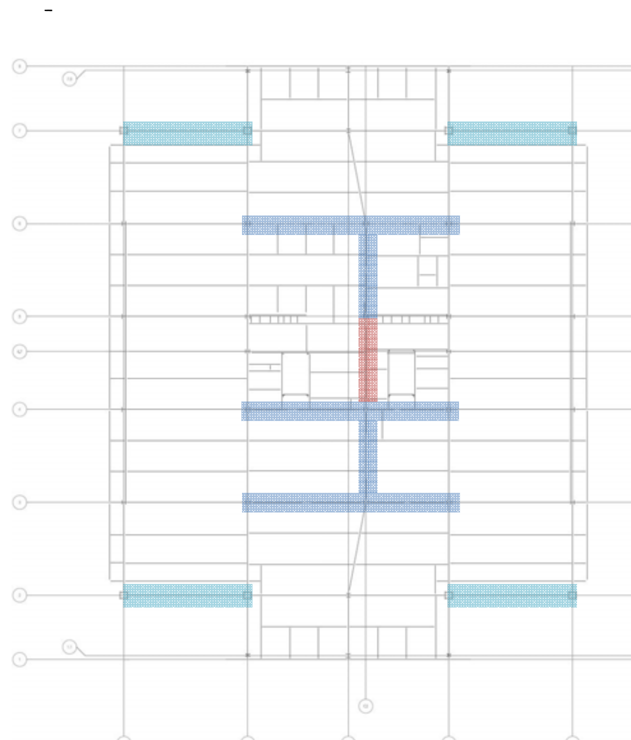


Figure 6: Typical Lateral System (Floors 29-50)

Key:
- Single Diagonal Bracing
- Pre-Tensioned Steel Rod X-Bracing
- Chevron & Eccentric Bracing

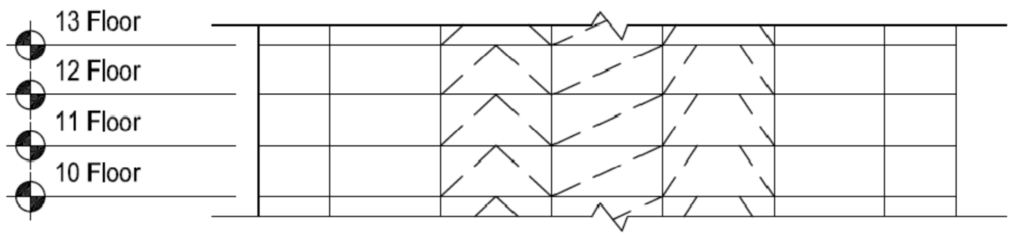


Figure 7: Typical Core N/S Core Bracing Elevation

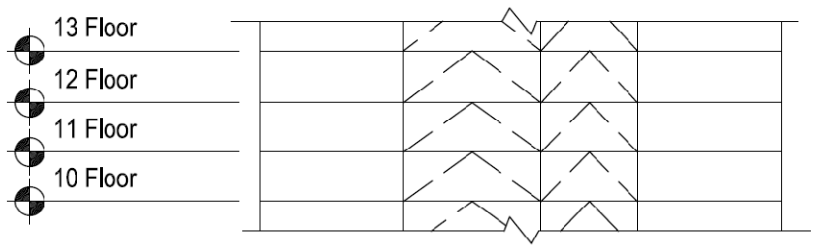


Figure 8: Typical Core E/W Core Bracing Elevation

During the design of the tower, the engineers at Thornton Tomasetti sized the members of the main lateral force resisting system merely for strength. In order to increase stiffness and meet deflection criterion, the structural engineers utilized the double story steel rod X-braces (original to Renzo Piano's exterior design) instead of increasing the member sizes of the main lateral force resisting system. These X-braces can be located on Figures 5 and 6 on the previous page. The high strength steel rods transition from 2.5" to 4" in diameter and were prestressed to 210 kips. This induced tensile load prevents the need for large compression members which would not conform to the architectural vision of the exterior.

Although the X-braces did reduce the need for an overall member size increase, the lateral system still did not completely conform to the deflection criterion. Therefore, some of the 30" by 30" base columns were designed as built-up solid sections which reduced the building drift caused by the building overturning moment. After combining these solid base columns and the X-braces with the main lateral force resisting system, the calculated deflection of the tower due to wind was L/450 with a 10 year return period and a building acceleration of less than 0.025g for non-hurricane winds.

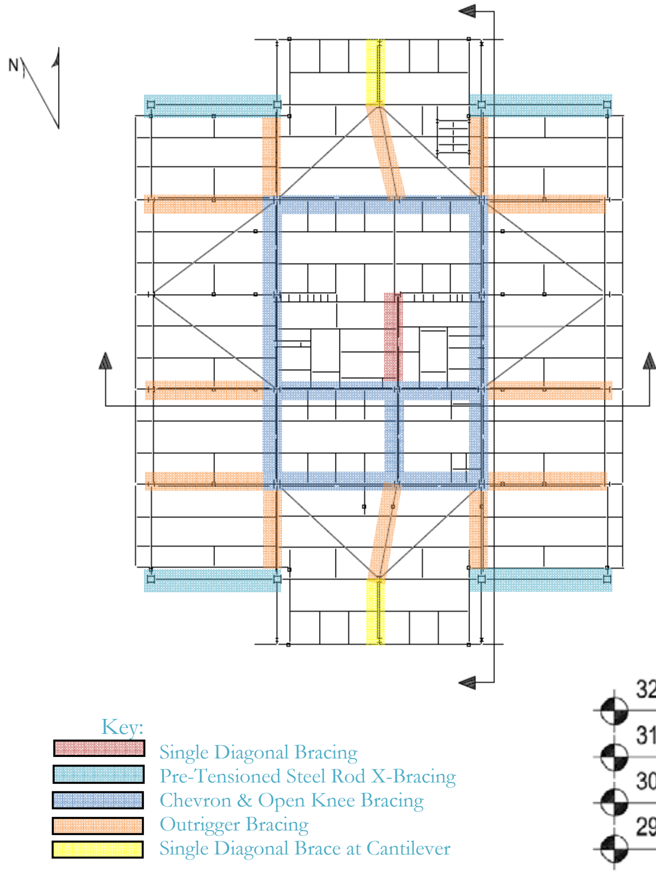


Figure 9: Mechanical Floor Framing Plan (Floors 28 & 51)

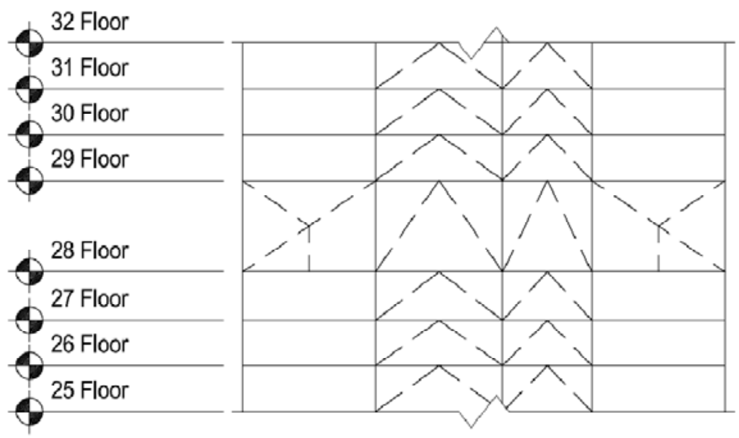


Figure 10: Typical E/W Outrigger Section (28th Floor)

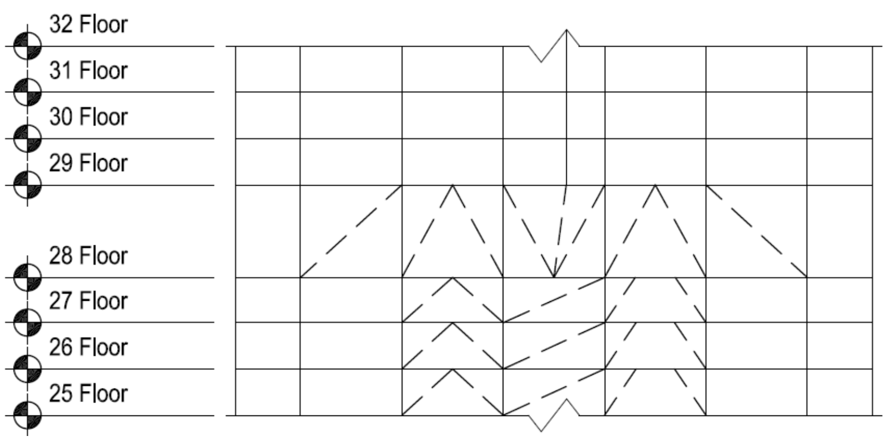


Figure 11: Typical N/S Outrigger Section (28th Floor)

Floor System

The existing floor structure of the NYTB is comprised of a composite steel beam system with typical bay dimensions of 30’-0”x40’-0”. These rectangular bays are configured into a cruciform shape around the perimeter of the core. This composite system was selected to reduce the self weight of the structural system which greatly affects member sizes in high rise buildings. By reducing member sizes, the structural system was able to conform to “transparency” desired by the architectural design. A more in depth discussion of existing floor framing system is presented later within the content of this report.

The remaining report evaluates and compares the existing composite beam floor system with three possible alternative floor systems. Please note that the designs with in this report are considered to merely be preliminary schematic designs used to determine the viability of each system. Those found to be feasible, have been noted and will taken into consideration for the expansion project proposal. Such items to be investigated are: system weight, fire rating, vibration susceptibility, cost, structural depth, constructability, architectural affects, and structural affects. In order to perform a proper comparison, all for systems were designed and evaluated using a typical perimeter bay, as shown in Figure 12 on the following page.

Gravity Loads

The gravity loads of the NYTB were the loads considered when designing the floor systems for comparison. The dead loads used for the evaluation were the calculated floor system self weights and the typical superimposed dead load of 25 psf. Also, the majority of the floor space within the tower, including the floor panel of interest, is allocated for office use. Therefore, the live load used in this comparison was 50 psf plus 20 psf for partitions. Also, please note that live loads were not reduced in order to be conservative.

Plan Not Disclosed Upon Owner's Request

Figure 12: Typical 40'-0"x30'-0" Exterior Bay

Design Codes and References

2006 International Building Code AISC – LRFD,

AISC – LRFD, Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318 – 08, Building Code Requirements for Structural Concrete, American Concrete Institute

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

PCA Time Saving Design Aids, Two-Way Post-Tensioned Design, Portland Cement Association

Aalami, B. O., (1993) "One-Way and Two-Way Post-Tensioned Floor Systems," PTI Technical Notes, Post-Tensioning Institute, Phoenix, AZ, Issue 3, October 1993.

Aalami, B. O., (2001) "Nonprestressed Bonded Reinforcement in Post-Tensioned Building Design," ADAPT Technical Publication, Redwood City, CA, Issue P2-01, February 2001.

Vulcraft Steel Roof & Floor Deck Catalog, Nucor – Vulcraft Group, 2008

Floor Vibration Testing and Analysis of SMARTBEAM FLOORS, Conducted by Structural Engineers, Inc. for the CMC Steel Group, October 2000.

Boyer, J. P. (1964), "Castellated Beams – New Developments," AISC Engineering Journal, July 1964.

Nilson, A. H., Darwin, D., Dolan, C. W., (2004) "Design of Concrete Structures, Thirteenth Edition," McGraw-Hill, New York, NY, 2004.

Deflection Criteria

Construction Dead Load deflection limitation for beams and girders – L/240

Live Load deflection limitation for beams and girders – L/360

Full Service Load deflection limitation for beams and girders – L/240

Concrete Systems to comply with provisions of ACI 318-08 Section 9.5.

Fire Protection

According to the 2006 IBC, the NYTB, based upon its building height and area, is classified as a Type 1A building. Floor construction, including supporting beams and joist, for this building type is required to meet a fire resistance rating of 2 hours. The following table shows the required clear cover in concrete slab systems in order to achieve this 2 hour rating.

STRUCTURAL PARTS TO BE PROTECTED	ITEM NUMBER	INSULATING MATERIAL USED	MINIMUM THICKNESS OF INSULATING MATERIAL FOR THE FOLLOWING FIRE-RESISTANCE PERIODS (inches)			
			4 hour	3 hour	2 hour	1 hour
3. Bonded pretensioned reinforcement in prestressed concrete[e]	3-1.1	Carbonate, lightweight, sand-lightweight and siliceous[f] aggregate concrete Beams or girders	4[g]	3[g]	2½	1½
		Solid slabs[h]		2	1½	1
4. Bonded or unbonded post-tensioned tendons in prestressed concrete[e, i]	4-1.1	Carbonate, lightweight, sand-lightweight and siliceous[i] aggregate concrete Unrestrained members: Solid slabs[h]	—	2	1½	—
		Beams and girders[j] 8″ wide		4½	2½	1¾
		greater than 12″ wide	3	2½	2	1½
	4-1.2	Carbonate, lightweight, sand-lightweight and siliceous aggregate Restrained members:[k] Solid slabs[h]	1¼	1	¾	—
		Beams and girders[j] 8″ wide	2½	2	1¾	—
		greater than 12″ wide	2	1¾	1½	—
7. Reinforcing and tie rods in floor and roof slabs[l]	7-1.1	Carbonate, lightweight and sand-lightweight aggregate concrete.	1	1	¾	¾
	7-1.2	Siliceous aggregate concrete.	1¼	1	1	¾

The structural steel within the floor systems must conform to the 2 hour fire rating as well. This rating can be reached through the application of spray on fire proofing, intumescent paint, or by enclosing structural members with in gypsum wall board. Please note that these fire protection systems were not analyzed for this evaluation.

EXISTING FLOOR FRAMING SYSTEM

Material Properties

Concrete:	5.5" slab (2.5" topping)
	f'_c= 4000 psi
Steel:	f_y= 50,000 psi
Reinforcement:	f_y= 60,000 psi
Metal Deck:	3 VLI 22 – 3 span

Loading

Self Weight:	59 psf
Live:	70 psf
Superimposed:	25 psf

Description

As mentioned previously, the structural engineers of the New York Times Building implemented a composite beam system (Figure 13) into the design of the floor framing structure. The typical bay size is 30'-0"x 40'-0" with 2 ½" normal weight concrete and 3" metal deck, typically spanning 10'-0" from W12x19 to W18x35 infill beams. These infill beams frame into W18x40 girders which in turn, transfer the floor loads to the various build-up columns throughout the structure.

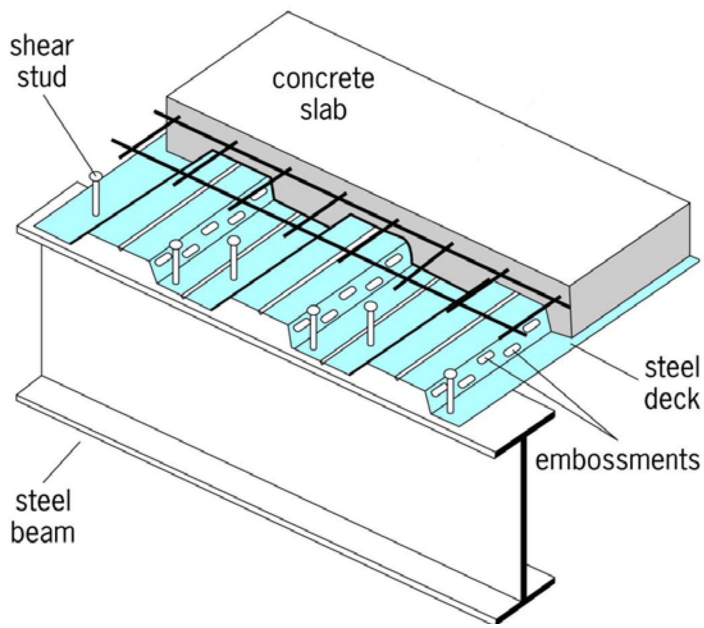


Figure 13: Typical Composite Beam Construction (Farlex)

The design verification calculations were conducted using the Vulcraft Steel Roof and Floor Deck Catalog as well as the AISC Steel Construction Manual. After conducting an analysis of the exterior floor panel of interest, it was found that the existing floor framing system is adequately designed to carry the applied gravity loads. To view the calculations supporting this analysis, please refer to Appendix B.

Advantages

This system has several advantages the first being speed of erection. By implementing the use of metal decking with a max unshored clear span of 10' – 11", no formwork nor shoring is needed during the construction process. This cuts back on erection time and reduces cost associated with construction labor. In addition, a steel system is fairly light weight in comparison to concrete. Also by taking advantage of a composited beam system, the member sizes are reduced from that of a non-composite system. This reduction in gravity loading is very advantageous to high rise design because it essentially reduces the loads of other gravity members, in particularly columns and foundations. But a low self weight is particularly beneficial when considering the cantilevered ends of the NYTB. The system allows for less self weight to be transferred back into the structure. In addition, the system allows for the versatility in the design of other building systems through the service plenum which will be created by a drop ceiling.

Disadvantages

Even though the system does reduce member sizes by taking advantage of composite beam construction, the system still results in a fairly deep floor system. In addition to the 5 ½" slab, the W18X40 girders add an additional 17.9" which totals to a 23.4" system depth. Also, steel beams and girders do not have inherent fire protection properties. This initiates the need for external fire protection, such as spray-on fire proofing or intumecent paint, in order to meet the required 2 hour fire rating. A steel system also presents the issue of longer lead ties in construction.

Feasibility

This floor system was found to be an excellent and efficient design for the New York Times Building. Due to the advantages of the light weight system along with the ease of construction the system will remain a viable solution for the floor system.

ALTERNATIVE FLOOR SYSTEMS

Plan Not Disclosed Upon Owner's Request

Figure 14: Typical 30'-0" x 40'-0" exterior bay used for alternative floor system designs

Figure 14 highlights the typical exterior panel used to design three alternative floor systems. The following were selected on the bases of finding alternative floor design solutions which can possibly be investigated further as part of the thesis proposal:

- Composite Castellated Steel Beams
- Two-Way Reinforced Concrete Flat Plate
- Two-Way Post-Tensioned Concrete Slab w/ Slab Bands

Composite Castellated Steel Beams

Material Properties

Concrete:	5.5" slab (2.5" topping)
	f'_c= 4000 psi
Steel:	f_y= 50,000 psi
Reinforcement:	f_y= 60,000 psi
Metal Deck:	3 VLI 22 – 3 span

Loading

Self Weight:	56 psf
Live:	70 psf
Superimposed:	25 psf

Description

The composite castellated beam system (Figure 15) was designed using the typical bay size of 30'-0" x 40'-0". The same composite deck (5 ½" slab with 3 VLI 22 metal deck) and beam spacing (10'-0") was used in order to yield the most comparable results to the existing composite beam system.

According to AISC, castellated beams and girders are proprietary and need to be designed using criterion established by the manufacture. Therefore, this schematic design was conducted the aid of a design program provided by CMC Steel Products. The calculations revealed that the lightest weight sections to be used were 20.65" beams built-up from W14x22s and W14x26s and 18.07" girders built-up from W12x14s and W12x19s. To review the calculations for this preliminary design, please see Appendix C.

Figure 15: Exposed Castellated Beam Construction (DJC.com)

Advantages

A composite castellated steel beam system has numerous benefits. Castellated beams can span greater distances than the conventional wide flange members with less steel creating a very efficient system. Also, by implementing a composite design as well as less steel being required, the castellated composite beam system proved to be the overall lightest floor framing system at 56 psf. As stated for the existing design, a smaller gravity loading is very advantageous to high rise design because it essentially reduces the loads of other gravity members. This is particularly advantageous for the cantilevered areas in the NYTB which require gravity loads to be transferred back to the interior structure. Although a castellated beam system does have a greater structural depth than when compared to a conventional wide flange system, other trades such as mechanical systems can be passed through the openings in the steel members which could possibly reduce the overall floor plenum height. Investigation conducted for CMC Steel Products also reviled that castellated beams are very good handling serviceability issues due to vibrations.

Disadvantages

As with the conventional composite beam system, the castellated steel members must be fire proofed in order to achieve the required 2 hour fire rating. Also when adding the slab thickness beam depth, the floor system introduces the largest over all structural depth of 26.1". This increase would require an increase in plenum space if the other trades are unable to utilize the openings in the castellated members. Thickening the plenum cannot be afforded in the NYTB due to the architectural impacts of decreasing the floor to ceiling heights. Also, castellated beams are proprietary which can be considered as a constraint in the design process. Lastly, castellated steel beams have a long lead time associated with them.

Feasibility

A castellated composite beam system is a very efficient floor system which is a viable alternative to the existing floor system of the New York Times Building. If no major architectural impacts are present after collaboration with the other disciplines of the BIM Thesis Team, a castellated composite floor system could be considered as option for the building proposal study.

Two-Way Flat Plate

Material Properties

Concrete:	14" slab (NWC)
	f'_c= 5000 psi
Reinforcement:	f_y= 60,000 psi
Column Sizes:	30"x30"

Loading

Self Weight:	175 psf
Live:	70 psf
Superimposed:	25 psf

Description

A two- way flat plate system (Figure 16) was designed for this evaluation using the typical exterior bay size of 30'-0" x 40'-0". The first step in the schematic design of this system was to determine the slab thickness required in order comply with deflection limitations provided by ACI 318-08 section 9.5. It was determined that a 14" slab was required to conform to these limitations.

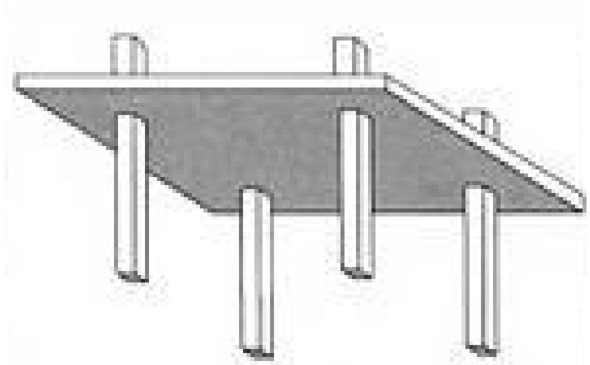


Figure 16: Two-Way Flat Plate (CRIS.org)

Though it was assumed that a flat plate designed with such a large bay would result in an inefficient, uneconomical floor system, the design was continued with the larger spans due to negative architectural affects that would result through the addition of intermediate columns. The system was schematically designed using the Direct Design Method found in ACI 318-08 section 13.6. To review the calculations and results for this preliminary design, please see Appendix D.

Advantages

The implementation of a two-way flat plate system as the floor design for the NYTB would have a few advantages. First of all, if the adequate clear cover is implemented into the design of the slab, a 2 hour fire rating can be achieved with in the floor system itself meaning no fire proofing is required. Also, in this particular design produced the thinnest overall floor structure of 14". This increase in the plenum space allows for the other trades to have the greatest flexibility for their designs. The use of a flat plate floor system is also advantageous for construction due to the shorter lead times associated with a concrete system. Lastly the mere mass of this two-way flat plate system makes it the least susceptible to vibration issues.

Disadvantages

Many disadvantages are associated with this two-way flat plate system. First off, the high self weight of 175 psf will affect the gravity and foundation systems the 62 story structure immensely. This self weight could be reduced through the addition of intermediate columns. But as mentioned previously, the addition of columns will negatively affect the New York Times Building's current architecture which utilizes the open area produced by the 40'-0"x30'-0" exterior bays. Using a flat plate system for the NYTB will require an entire redesign of the gravity system and lateral system from steel to concrete as well. Also, the additional weight of the system when designed with the existing column configuration produces the highest material cost of 31.5 dollars per square foot which is almost double the cost associated with the other alternative concrete system. Lastly, implementing a two-way flat plate system introduces the need for formwork and shoring. This presents the issue of an increase in labor cost and longer erection times.

Feasibility

Due to the increase in self weight and cost, a two-way flat plate floor system would be inefficient and uneconomical design solution for the New York Times Building. Also, the reduction of span length is not an option due to an architectural design constraint. After this evaluation, it is suggested that no further investigation be performed on this system.

Two-Way Post-Tensioned Slab w/ Slab Bands

Material Properties

Concrete:	8" slab w/ 14" drop panels
	16" Slab Beam
	f'_c= 5000 psi
	f'_{ci}= 3000 psi
Tendons:	Unbonded Tendons
	½" dia.7-Wire Strand
	f_{pu}= 270,000 psi
Reinforcement:	f_y= 60,000 psi
Column Sizes:	30"x30"

Loading

Self Weight:	113 psf
Live:	70 psf
Superimposed:	25 psf

Description

The third floor system to be evaluated as a possible alternative to the existing was a two-way post- tensioned concrete slab system (Figure 17). To create a comparable result, the system was designed using the typical exterior bay size of 30'-0" x 40'-0". In order to produce an efficient slab design while maintaining the existing column layout, a slab beam or wide shallow beam was utilized for the 40' span. This enabled the slab thickness to be determined using the shorter 30' span.

Figure 17: Two-Way PT Floor System (concreteconstruction.net)

This schematic design resulted in the slab thickness of 8" with (33) tendons uniformly distributed spanning the short direction and (31) tendons banded with in each wide-shallow-beam. Based upon technical information obtained from the Post-Tensioning Institute, the geometry of the slab beam was designed with a 48" width and 16" overall height. Minimum bonded reinforcement was also determined based upon the strength requirement of the floor system. Also, please note that the wide-shallow-beam was not used when checking punching shear. Although this assumption is unfeasible, it was made in order to be conservative for this schematic design. The 8" slab alone was unable to comply with the punching shear requirements. Therefore, 14" drop panels were required for the floor system. This assumption will have to be readdressed if further investigation of this system occurs. Please refer to Appendix E to review the schematic design and calculations for the two-way post-tensioned concrete floor system.

Advantages

A two-way post-tension concrete slab system was found to have many advantages associated with it. When a wide-shallow-beam is designed as part of the two-way slab, the system becomes very efficient at spanning long distances. Therefore, the architecture of the exterior bays in the NYTB would not change significantly if this system was implemented. Also, the thinner slab allows for more versatility and space in the plenum when compared to the existing floor system. As with the other concrete system, fire ratings are determined by clear cover in the concrete which removes the need for additional fire proofing. Also, the system does provide dampening to vibration affects.

Disadvantages

There are some disadvantages associated with a two-way post-tensioned system. Even though the material cost of the system is the lowest overall at 16.3 dollars per square foot, the cost associated to labor and construction is very high. Of all the four systems, the two-way flat plate is the most complex and labor intensive requiring the use of specialty trades. The system also has a slow erection time, not only due to forming and shoring, but due to the jacking process as well. Lastly being a concrete system, the gravity as well as the lateral system of the New York Times Building would have to be redesigned as concrete systems if selected to be investigated further.

Feasibility

The two-way post-tensioned slab with wide-shallow-beams is a very efficient floor system which should be considered as a viable alternative to the existing floor system of the New York Times Building. Though some disadvantages in terms of complexity are present, the system should still be considered feasible. If no major concerns are issued after collaboration with the other members of the BIM Thesis Team, a two-way post-tensioned slab system could be considered as option for the building proposal study.

SYSTEM COMPARISON

Criterion	Floor System Comparison - Typical Exterior Bay			
	Composite Steel Beam System (Existing)	Composite Castellated Steel Beam System	Two-Way Flat Plate	Two-Way PT Slab w/ Slab Beams
System Self Weight (psf)	59	56	175	113
Slab Depth (in)	5.5	5.5	14	8
Total Floor System Depth (in)	23.4	26.1	14	16
Constructability	Easy	Easy	Medium	Hard
Formwork Required	No	No	Yes	Yes
Lead Time	Long	Long	Short	Short
Gravity System Impact	N/A	No	Major	Yes
Lateral System Impact	N/A	No	Yes	Yes
Architectural Impact	N/A	Possible	Major	No
Vibration Control	Average	Above Average	Excellent	Above Average
Fire Rating (hr)	2	2	2	2
Fire Protection Required	Yes	Yes	No	No
System Cost ($/sf)*	28.7	27.3	31.5	16.3
Feasible	Yes	Yes	No	Yes
Additional Investigation	N/A	Yes	No	Yes

* System costs were determined using Cost Works by RSMeans. The prices shown here are only costs associated to materials. Labor was not a factor when calculating these values.

CONCLUSION

For the second technical report on the structural system of the New York Times Building, an evaluation was performed between the existing floor system and three alternatives. These alternatives were schematically designed using a typical 40'-0"x30'-0" bay on the exterior perimeter of the tower. During the evaluation, two criteria emerged as the main factors in determining the feasibility of the floors systems. System self weight, being the first factor, has an extreme effect high rise structures. A small increase in the self weight of a floor panel can translate into a large increase in the overall building weight. The second factor was that of architectural affects. The architectural design of New York Times Building tries to conform to the idea of structural transparency through the use of long spans which provide open areas and spaces.

After evaluating all the systems, two of the designs were found to be feasible as alternative designs for the New York Times Building. The first feasible alternative was the composite castellated steel beam system. Of all the systems, it was the only one to have a smaller self weight that the existing floor structure. Though the system would increase the overall floor structure thickness, the architectural effects could be negated if the other disciplines are able to utilize the openings in the castellated members which would reduce the overall floor plenum.

The second alternative would be the two-way post-tensioned concrete slab with wide-shallow-beams. Though the systems has a greater self weight than that of the existing system, its ability to span long distances, while still maintaining a small structural depth, allows for open areas and spaces as desired by the architect. However, it must be considered that this will require that the entire structural gravity and lateral systems must be redesigned as concrete systems.

A two-way flat plate system was schematically designed for the third comparison. It was determined that this system should disregarded as an alternative floor system design. This is due to the system's inability to span large distances without creating a heavy, inefficient design. System could have been considered as an alternative if more column lines were added throughout the structure. However, this would have not conformed to the architectural desire to have large open bays.

APPENDIX A – Typical Framing Plan

Plan Not Disclosed Upon Owner's Request

APPENDIX B – Existing Composite Steel Beam

After consulting the structural engineer and the general structural design criteria, it was determined that the metal decking for the existing 5 ½" composite slab was required to be 3" in depth, have a minimum yield strength of 40 ksi, and minimum thickness of 20 gage. The following is a table obtained from Vulcraft to find the permissible unshored clear span and superimposed live load of a typical metal deck type which would conform to this criterion.

(N=9.35) NORMAL WEIGHT CONCRETE (145 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0
5.00 (t=2.00) 45 PSF	3VLI22	9'-2	10'-7	11'-8	216	195	176	161	148	109	99	90	83	76	70	64	59	54	50
	3VLI20	10'-8	12'-11	13'-4	241	216	196	178	163	150	139	129	93	85	78	72	66	61	57
	3VLI19	12'-0	14'-4	14'-7	265	237	214	194	178	163	151	140	131	122	115	79	73	67	62
	3VLI18	12'-10	15'-1	15'-1	289	261	238	218	201	186	173	161	151	142	134	127	92	86	80
	3VLI16	13'-5	15'-7	15'-11	327	294	267	243	223	206	191	178	167	156	147	139	132	96	89
5.50 (t=2.50) 51 PSF	3VLI22	8'-9	9'-8	10'-11	247	222	201	184	137	124	113	103	94	87	80	73	67	62	57
	3VLI20	10'-1	12'-4	12'-9	275	247	223	203	186	171	159	116	106	97	89	82	76	70	65
	3VLI19	11'-4	13'-8	14'-2	302	270	244	222	203	186	172	160	149	107	98	90	83	77	71
	3VLI18	12'-5	14'-7	14'-7	330	298	271	248	229	212	197	184	173	162	153	112	105	98	92
	3VLI16	12'-9	14'-11	15'-5	373	335	304	277	255	235	218	203	190	178	168	159	117	109	102

The following is the framing plan for the floor panel of interest as designed by the structural engineer. Please note that the design shown on the following pages resulted in a different number of shear studs than depicted here in Figure 18. This difference was assumed to be minor and negligible for this report.

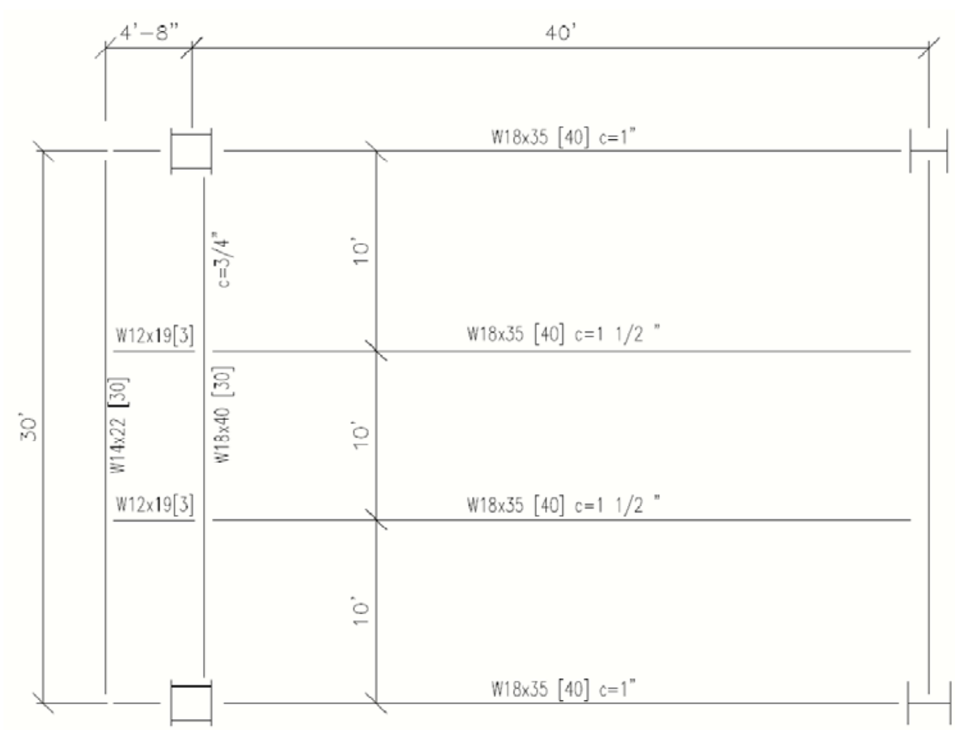


Figure 18: Existing Framing Plan

STAEDTLER® No. 937 811E Engineer's Computation Pad

Check Metal Decking

From Structural Design Criteria:

- 2 1/2" H.W. Conc. SLAB
- 3" Decking
- 40 ksi YIELD
- Min. 20 gage

From Vulcraft (3VLI, 145 pcf conc.)

Max. Unshored Clear Span

5 1/2" SLAB, 3 SPAN } 10'-11" = 10.92"

Max Clear Span Between Beams (use W 18x35)

℄ to ℄ = 10'-0"

$10' - \frac{6}{12} = 9.5'$ ✓ ok   Try 3VLI 22

Check Superimposed Live Load

Span = 9'-6", 5 1/2" SLAB } 124 psf

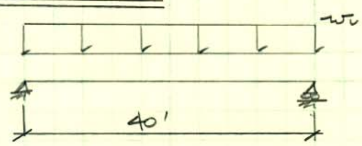
Typical Flr Loading

LL = 70 psf (office loading)
SDL = 25 psf

Total service LD = 95 psf ✓ ok

Use 3VLI 22 (Vulcraft)

BEAM CHECK



TRIB WIDTH = 10'-0"

W

SWDL = 52 psf(10') = 520 plf (SLAB)
SDL = 25 psf(10') = 250 plf (SUPERIMPOSED)
LL = 70 psf(10') = 700 plf

FACTORED LOADS

$w_u = 1.2(0.52 + 0.25) + 1.6(0.7) = 2.04$ KLF

Design

$M_u = \frac{(2.04)(40)^2}{8} = 408$ 'k

$V_u = (2.04)(\frac{40}{2}) = 40.8$ k

Assume: a = 1"

$Y_2 = 5\frac{1}{2} - \frac{1}{2}" = 5"$

TRY W18x35 $\quad \phi M_n = 435$ 'k @ BFL

beff
S = 9' ← CONTROLS $\quad \Sigma Q_n = 260^k$
$\frac{L}{4} = \frac{40}{4} = 10'$

$a = \frac{\Sigma Q_n}{0.85 f'_c \, b_{eff}} = \frac{260}{(0.85)(4)(9 \times 12)} = 0.71" < 1"$ ✓ ok

SHEAR STUDS

$\Sigma Q_n = 260^k$

$\# \text{ STUDS} = \frac{260^k}{17.2^k/\text{STUD}} = 16 \text{ STUDS} \times 2 = \underline{32 \text{ STUDS}}$

STAEDTLER® No. 937 811E Engineer's Computation Pad

TOTAL Δ $\left(\Delta \le \frac{l}{240}\right)$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(1.47)(40)^4(1728)}{384(29000)(1170)} = 2.5''$$

$I_{LB} = 1170 in^4$

$$\frac{l}{240} = \frac{(40)(12)}{240} = 2'' < \Delta = 2.5$$

However, Designed w/ 1½" CAMBER ∴ $\Delta = 2.5 - 1.5 = 1/2'' < 2''$ ✓ OK

Construction Δ $\left(\Delta \le \frac{l}{240}\right)$ $I = 510 in^4$

$$\Delta = \frac{(5)(0.52)(40)^4(1728)}{384(29000)(510)} = 2.03''$$

$\frac{l}{240} + 1\frac{1}{2}''$ CAMBER $= 3.5'' > \Delta$ ✓ OK

LL Δ $\left(\Delta \le \frac{l}{360}\right)$ $I_{LB} = 1170 in^4$

$$\Delta = \frac{(5)(0.7)(40)^4(1728)}{384(29000)(1170)} = 1.18'' = \frac{l}{404} < \frac{l}{360}$$ ✓ OK

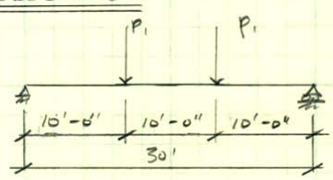
$\phi M_n = 435'k > 408'k$ ✓ OK

$\phi V_n = 159\,k > 40.8\,k$ ✓ OK

USE W 18 x 35 [32] c = 1 ½"

STAEDTLER® No. 937 811E Engineer's Computation Pad

GIRDER CHECK



TRIB WIDTH = 10'

P_1

CDL = 0.52 × 25 + 0.085 × 20 + 0.019 × 5 = 13.8 k
SDL = 0.25 × 25 = 6.3 k
LL = 0.7 × 25 = 17.5 k

FACTORED LOAD

$P_u = 1.2(13.8 + 6.3) + 1.6(17.5) = 52.12^k$

Design

$M_u = P_a = (52.12)(10') = 521.2\ 'k$

$V_u = 52.12^k$

Δ_{CDL} ($\ell/240$)

$$\Delta = \frac{P\ell^3}{28\ EI}$$

$$\frac{(30)(12)}{240} = \frac{(13.8)(30)^3(1728)}{(28)(29000)\ I_{req}} \qquad I_{req} = 528.6\ in^4$$

TRY W 18x40 $I = 612\ in^4$ Δ_{CDL} ✓ OK

STAEDTLER No. 937 811E Engineer's Computation Pad

ASSUME $a = 1''$ $Y_2 = 5\tfrac{1}{2}'' - 1''/2 = 5''$ $\phi M_n = 536'^k @ \#4$

$\Sigma Q_n = 351^k$ $b_{eff} \begin{cases} 10' \\ \frac{30'}{4} = 7.5' = 90'' \leftarrow \text{CONTROLS} \end{cases}$

$a = \frac{351}{(0.85)(4)(90)} = 1.14'' > 1''$ ∴ Assumption NG

ASSUME $a = 2''$ $Y_2 = 5\tfrac{1}{2}'' - 2''/2 = 4\tfrac{1}{2}''$ $\phi M_n = 523'^k @ \#4$

$a = \frac{351}{(0.85)(4)(90)} = 1.14'' < 2''$ ✓ OK

\# STUDS

$\Sigma Q_n = 351^k$

$\#\text{ STUDS} = \frac{351^k}{18.3^k/\text{STUD}} = 20 \times 2 = 40$ STUDS

CHECK TOTAL Δ $(\Delta \leq l/240)$ $I_{LB} = 1670\text{ in}^4$

$\Delta = \frac{(37.6)(30)^3(1728)}{(28)(29000)(1670)} = 1.29'' = \frac{l}{279} < \frac{l}{240}$ ✓ OK

CHECK LL Δ $(\Delta \leq l/360)$

$\Delta = \frac{(17.5)(30)^3(1728)}{(28)(29000)(1670)} = 0.6 = \frac{l}{598} < \frac{l}{360}$ ✓ OK

USE W18×40 [40]

$\phi M_n = 523'^k > 521.2'^k$ ✓ OK
$\phi V_n = 169^k > 52.12^k$ ✓ OK

EXISTING SYSTEM SW

CONC & DECK = 52 psf

$\text{FRAMING} = \frac{(35\text{ PLF})(40')(4) + (40\text{ PLF})(30')(2)}{(30')(40')} = 6.7\text{ psf}$

SW = 59 psf TOTAL

APPENDIX C – Composite Castellated Steel Beams

According to the AISC Steel Construction Manual, “castellated beams are currently designed and fabricated as a proprietary product” and are designed based on criterion put in place by the manufacture. (AISC, p.2-21) Therefore, a design program provided by CMC Steel Products was used to perform the schematic design of this alternative system.

Several assumptions were made when designing this proprietary system. The design of the composite slab was taken to be the same as that for the existing floor system (please refer to Appendix B). Also, the design program only enables one to design a composite castellated beam with metal deck running perpendicular to the member. Therefore, the design for the edge girder is not entirely correct but is assumed to be within reason for this preliminary design. However, this issue must be addressed if a composite castellated beam system is investigated further as a viable alternative floor system.

CASTELLATED BEAM INFORMATION

Job Name	40 Span Beams	
Beam Mark #	Beam	
Span	40.000	ft
Spac. Left	10.000	ft
Spac. Right	10.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	11.109	in
Duct W x H	6.250 in	10.874 in
Castellated Beam	CB21X22/26	
Root Beams (T/B)	W14X22	W14X26
d	13.74	13.91
bf	5	5.025
tf	0.335	0.42
tw	0.23	0.255

CASTELLATION PARAMETERS:

e	5.500	in
b	4.000	in
dt	3.500	in
S	19.000	in
dg	20.650	in
phi	59.623	deg
ho	13.650	in
wo	13.500	in

CMC Steel Products

LOADING INFORMATION

Uniform Distributed Loads

Live Load	700	plf	Pre-comp %	0%
Dead Load	250	plf	Pre-comp %	80%

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)	4500	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	150	stud ht. (in)	5
conc. above deck - tc (in)	2 1/2	studs per rib	1
rib height - hr (in)	3	composite %	100%
rib width - wr (in)	6	**Stud Spacing:**	
		N=22,Uniformly Dist.	

RESULTS

Failure Mode	Interaction	Status
Bending	**0.989**	<=1.0 OK!!
Web Post	**0.882**	<=1.0 OK!!
Shear	**0.819**	<=1.0 OK!!
Concrete	**0.283**	<=1.0 OK!!
Pre-Comp.	0.409	<=1.0 OK!!
Overall	**0.989**	**<=1.0 OK!!**
Pre-Composite Deflec.	0.904"	=L/531
Live Load Deflection	0.840"	=L/572

WARNINGS

EXPAND'D. SXN. PROP'S

Avg. wt.	24.0	plf
Anet	5.376	in^2
Agross	8.688	in^2
Ix net	499.73	in^4
Ix gross	552.07	in^4
Sx net	44.56	in^3
Sx gross	50.45	in^3
rx min	7.97	in
Iy	7.95	in^4
Sy	3.18	in^3

COMPOSITE SXN. PROP'S

n	7.44	
beffec.	120.00	in
Actr	40.315	in^2
N.A. ht.	23.13	In Deck
Itr	1655.97	in^4
Ieffec.	1655.97	in^3
Sxconc	548.05	in^3
Sxsteel	71.60	in^3

CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	1
Max. Bridging. Spacing (ft)	31

CASTELLATED BEAM INFORMATION

Job Name	Edge Girder	
Beam Mark #	Girder	
Span	30.000	ft
Spac. Left	5.000	ft
Spac. Right	20.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	9.825	in
Duct W x H	5.500 in	9.631 in
Castellated Beam	CB18X14/19	
Root Beams (T/B)	W12X14	W12X19
d	11.91	12.16
bf	3.97	4.005
tf	0.225	0.35
tw	0.2	0.235

CASTELLATION PARAMETERS:

e	5.000	in
b	3.500	in
dt	3.000	in
S	17.000	in
dg	18.070	in
phi	59.880	deg
ho	12.070	in
wo	12.000	in

CMC Steel Products

LOADING INFORMATION

Uniform Distributed Loads

Live Load	700	plf	Pre-comp %	0%
Dead Load	250	plf	Pre-comp %	100%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.60	10.00	100%	100%
P2	0.60	20.00	100%	100%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

EXPAND'D. SXN. PROP'S

Avg. wt.	16.5	plf
Anet	3.550	in^2
Agross	6.180	in^2
Ix net	245.23	in^4
Ix gross	278.92	in^4
Sx net	23.54	in^3
Sx gross	28.08	in^3
rx min	6.72	in
Iy	3.06	in^4
Sy	1.54	in^3

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	4500	stud dia. (in)	1/2"
conc. wt. - wc (pcf)	150	stud ht. (in)	5
conc. above deck - tc (in)	2 1/2	studs per rib	1
rib height - hr (in)	3	composite %	100%
rib width - wr (in)	6	Stud Spacing:	
		11 | 10 | 11	

COMPOSITE SXN. PROP'S

n	7.44	
beffec.	90.00	in
Actr	30.236	in^2
N.A. ht.	20.79	In Deck
Itr	944.58	in^4
Ieffec.	944.58	in^3
Sxconc	340.09	in^3
Sxsteel	45.43	in^3

RESULTS

Failure Mode	Interaction	Status
Bending	**0.918**	<=1.0 OK!!
Web Post	**0.904**	<=1.0 OK!!
Shear	**0.850**	<=1.0 OK!!
Concrete	**0.269**	<=1.0 OK!!
Pre-Comp.	0.642	<=1.0 OK!!
Overall	**0.918**	**<=1.0 OK!!**

Pre-Composite Deflec.	0.843"	=L/427
Live Load Deflection	0.466"	=L/773

WARNINGS

CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	1
Max. Bridging. Spacing (ft)	26

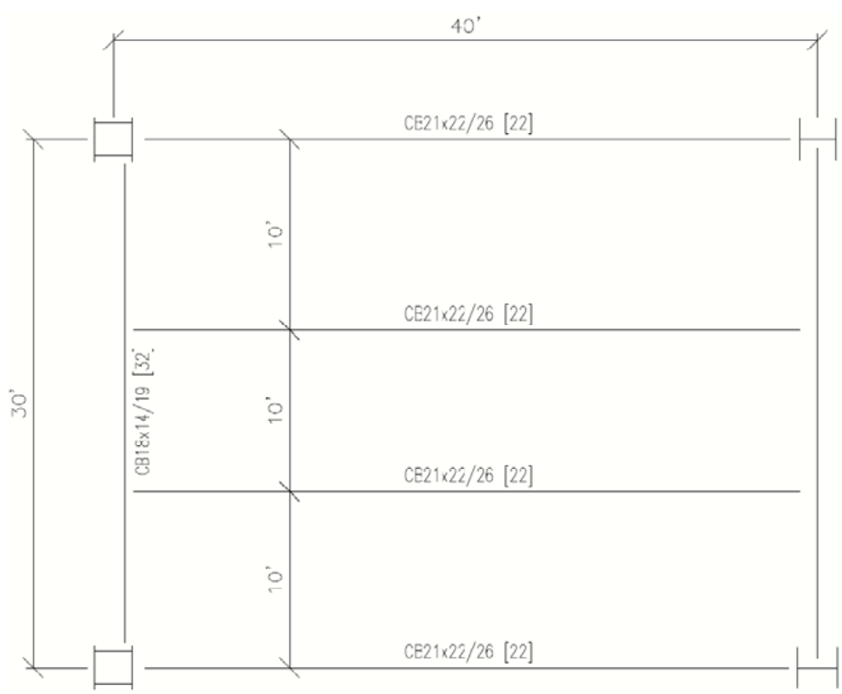


Figure 19: Castellated Beam Framing Plan

CASTELLATED BEAM System SW

Conc & Deck = 52 psf

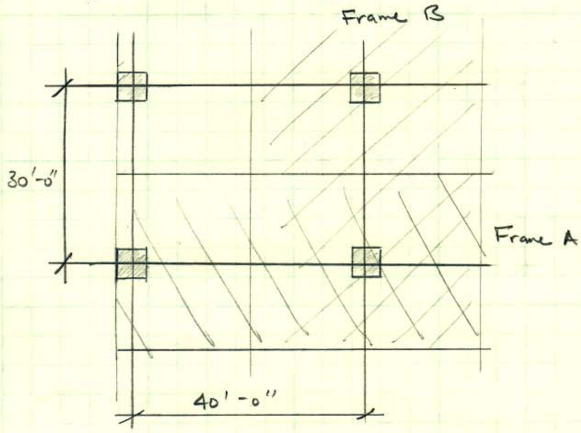
$$\text{Framing} = \frac{(24\ \text{plf})(40')(4) + (16.5\ \text{plf})(30')(2)}{(30')(40')} = 4.0\ \text{psf}$$

SW = 56 psf TOTAL

APPENDIX D – Two-Way Flat Plate

Two-Way Flat Plate

Typical Exterior Bay



Assume:
$f'_c = 5000\,psi$
$f_y = 60000\,psi$
$LL = 70\,PSF$
$SDL = 25\,PSF$
No Edge Beam
30" x 30" Cols

Determine Slab Thickness
– Start w/ no Drop panels

T9.5(c): $\frac{\ell_n}{33} = \frac{(40)(12) - 30}{33} = 13.63'' \longrightarrow$ Try 14"

$w_u = 1.2\left(\frac{14}{12}(150) + 25\right) + 1.6(70) = 240 + 112 = 352\,psf$

$V_u = w_u \cdot Area = 352\left(40' \times 30' - (2.5')^2\right) = 420.2\,k$

Check Punching Shear (Look @ Interior Col)

Assume #6 Bars

$d = 14'' - 0.75'' - 0.75'' = 12.5''$

$b_o = (30'' + 12.5'')(4) = 170''$

$\beta_c = \frac{30}{30} = 1$

$\alpha_s = 40$ For Int. Col

$$V_c = \begin{cases} 4\sqrt{f'_c}\, b_o d = 5\sqrt{4000}(170)(12.5) = 601\,k \longleftarrow \text{CONTROLS} \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2 + \frac{4}{1}\right)\sqrt{5000}(170)(12.5) = 901.5\,k \\ \left(\frac{\alpha_s}{\frac{b_o}{d}} + 2\right)\sqrt{f'_c}\, b_o d = \left[\frac{40}{(170/12.5)} + 2\right]\sqrt{5000}(170)(12.5) = 742.5\,k \end{cases}$$

$\phi V_c = 0.75(601) = 450.75\,k > V_u \therefore$ ok ✓

Use Direct Design Method to Calc. Moments

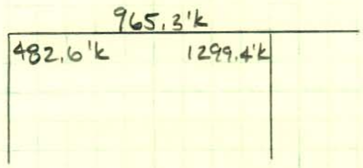
Total Factored Static Moment

Long Span – Frame A

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(352)(30)(40-2.5)^2}{8} = 1857 \text{ Ft-k}$$

Factored Moments § 13.6.3

Ext. Neg. = $0.26(M_o)$ = 482.6 'k
Pos. = $0.52(M_o)$ = 965.3 'k
Int. Neg. = $0.70(M_o)$ = 1299.4 'k



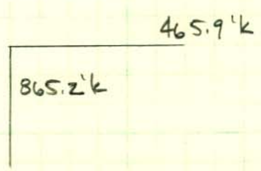
Short Span – Frame B

$$M_o = \frac{(352)(40)(30-2.5)^2}{8} = 1331 \text{ Ft-k}$$

Factored Moments § 13.6.3

Neg. = $0.65 M_o$ = (0.65)(1331) = 865.2 'k

Pos. = $0.35 M_o$ = (0.35)(1331) = 465.9 'k



Column Strip Moment § 13.6.4

$\alpha \frac{l_1}{l_2} = 0$ & $\beta_t = 0$ ∴ For Ext. Neg Moment → 1.0 Mult.

For Int. Neg Moment → 0.75 Mult.

For Pos. Moment → 0.6 Mult.

Long Span – Frame A

Ext. Neg M = 482.6'k → CS: 1.0(482.6) = 482.6'k
→ Ms: 0'k

Int. Neg. M = 1299.4'k → CS: (0.75)(1299.4) = 974.5'k
→ MS: (0.25)(1299.4) = 324.9'k

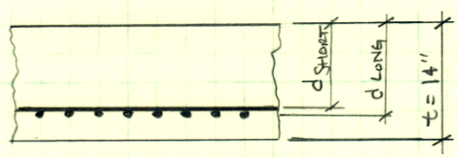
Pos. M = 963.3'k → CS: (0.6)(963.3) = 578'k
→ Ms: (0.4)(963.3) = 385.3'k

Short Span – Frame B

Neg. M = 865.2'k → CS: (0.75)(865.2) = 648.9'k
→ MS: (0.25)(865.2) = 465.9'k

Pos. M = 465.9'k → CS: (0.6)(465.9) = 279.5'k
→ Ms: (0.4)(465.9) = 186.4'k

Calc. of "d" Assuming #6 Bars



d_{long} = 14" – 0.75" – 0.5 (0.75") = 12.875"

d_{short} = 12.875" – 0.75" = 12.125"

Design of Slab Rein

Frame A

	Exterior				Interior	
Description	−Mcs	−Mms	+Mcs	+Mms	−Mcs	−Mms
Moment	−483	0	97.5	325	−578	−385
b	180"	180"	180"	180"	180"	180"
deff	12.875"	12.875"	12.875"	12.875"	12.875"	12.875"
$M_n = M_u/\phi$	−537	0	108.3	361	−642	−428
$R = M_n/d^2b \times 12000$	216	0	44	145	258	172
ρ	0.0037	—	0.0007	0.0025	0.0049	0.0034
$A_s = \rho bd$	8.57	—	1.62	5.79	10.2	7.88
$A_{smin} = 0.002tb$	5.04	5.04	5.04	5.04	5.04	5.04
$N = A_s/A_{s\,bar}$	19.4 → 20	11.4 → 12	11.4 → 12	13.2 → 14	23.2 → 24	17.9 → 18
$N_{min} = \frac{width}{2t}$	7	7	7	7	7	7

$\phi = 0.9$

Nilson T. A-5

#6 Bars

Frame B

Description	−Mcs	−Mms	+Mcs	+Mms
Moment	−649	−213	280	186
b	240"	240"	240"	240"
deff	12.125"	12.125"	12.125"	12.125"
$M_n = M_u/\phi$	−721	−237	311	207
$R = M_n/bd^2 \times 12000$	245	80	105	70
ρ	0.0042	0.0013	0.0018	0.0012
$A_s = \rho bd$	12.22	3.78	5.24	3.49
$A_{smin} = 0.002bd$	5.82	5.82	5.82	5.82
$N = A_s/A_{bar}$	27.7 → 28	13.2 → 14	14	14
$N_{min} = \frac{WIDTH}{2t}$	9	9	9	9

$\phi = 0.9$

Nilson T A-5

#6 Bars

Check d_{min}

$\rho_{max} = 0.0243$ (Nilson T. 4-2)

$$d_{min} = \sqrt{\frac{M_u \cdot 12000}{\phi \rho f_y b\left(1 - 0.59\rho \frac{f_y}{f'_c}\right)}}$$

Long Span

$$d_{min} = \sqrt{\frac{(975 \cdot 12000)}{(0.9)(0.0243)(60000)(180)\left[\left(1 - 0.59(0.0243)\left(\frac{60000}{5000}\right)\right)\right]}} = 7.7"$$

$d = 12.875" > 7.7"$ ✓ ok

Short Span

$$d_{min} = \sqrt{\frac{(649 \cdot 12000)}{(0.9)(0.0243)(60000)(240)\left[\left(1 - 0.59(0.0243)\left(\frac{60000}{5000}\right)\right)\right]}} = 5.6"$$

$d = 12.125 > 5.6"$ ✓ ok

Check Beam Shear

$V_u = 0.352\,(30 \times 20) = 211.2^k$

$\phi V_c = 0.75(2)\sqrt{5000}\,(30)(12)(12.5) = 477.3^k$

$\phi V_c > V_u$ ok ✓

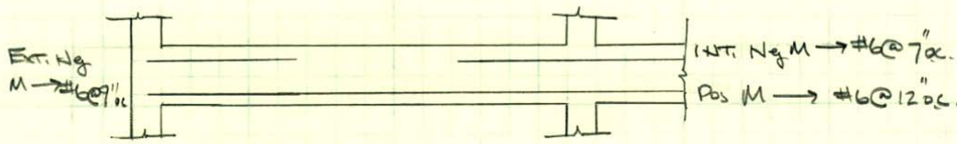
System WT

$SW = \frac{14}{12}(150) = 175$ psf

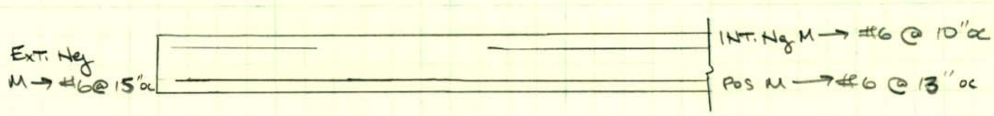
Design Summary

FRAME A

CS

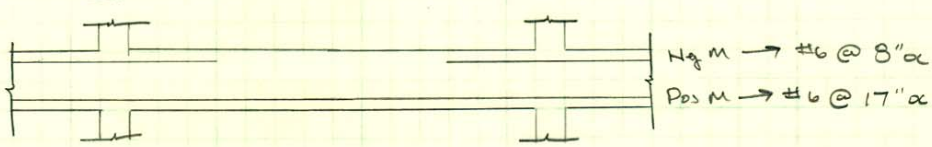


MS

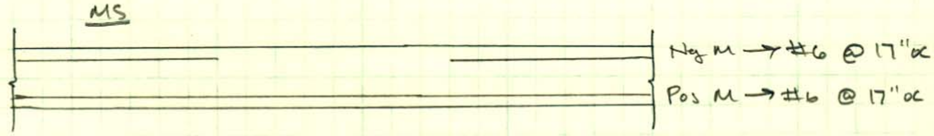


FRAME B

CS



MS



MAX SPACING = 18" or 3H = 42"

APPENDIX E – Two-Way Post-Tensioned Slab w/ Slab Bands

STAEDTLER® No. 937 811E Engineer's Computation Pad

POST-TENSIONED: TYPICAL EXTERIOR PANELS

ASSUME:

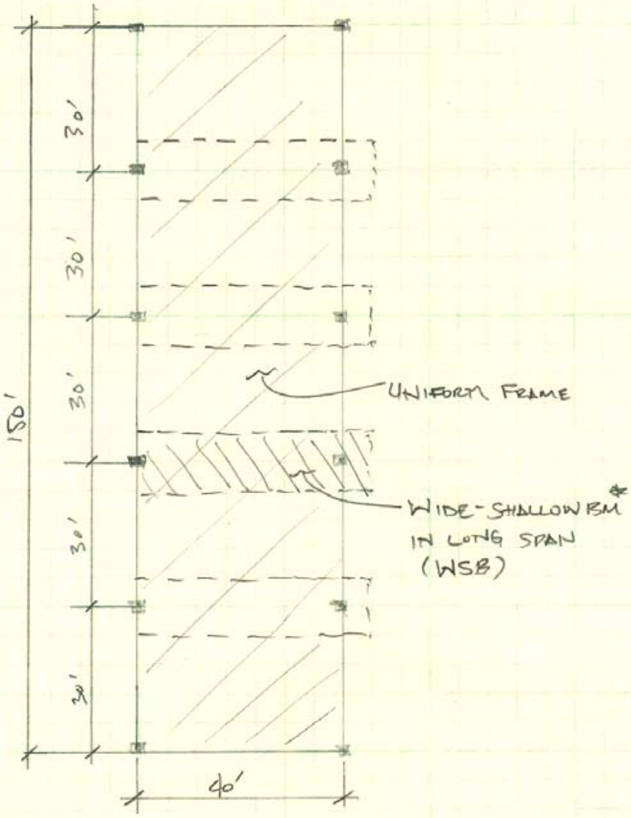
$f'_c = 5000$ psi
$f'_{ci} = 3000$ psi
$f_y = 6000$ psi

LL = 70 psf
SDL = 25 psf
2-HR FIRE RATING

30" x 30" COLS

1/2" Φ 7WIRE 270 ksi TENDONS
$f_{pu} = 270$ ksi
$f_{py} = 243$ ksi
$f_{pe} = 174$ ksi

$P_{eff} = (0.153)(174) = 26.6$ k/TEN



Determine Prelim SLAB t
BECAUSE USING WSB,
DETERMINE USING SHORT SPAN.

$L/h = 45$

$h = \frac{(30)(12)}{45} = \underline{8''}$

LOADING

DL = SW = $(\frac{8}{12})(150) = 100$ psf
SDL = 25 psf
LL = 70 psf

Section Prop

$A = (40)(12)(8) = 3840 \text{ in}^2$

$S = \frac{(40)(12)(8)^2}{6} = 5120 \text{ in}^3$

* ASSUME WSB IS 1-SPAN TO CORE

DESIGN PARAMETERS

– ALLOWABLE STRESSES

+ @ JACKING (ACI 18.4.1)

$f'_{ci} = 3500$ psi
Comp = $0.6(3500) = 2100$ psi
TENS = $3\sqrt{3500} = 177$ psi

+ @ SERVICE (ACI 18.4.2)

$f'_c = 5000$ psi
Comp = $0.45(5000) = 2250$ psi
TENS = $6\sqrt{5000} = 424$ psi

+ AVG. PRECOMP. LIMITS

$P/A = 125$ psi min
$= 300$ psi Max

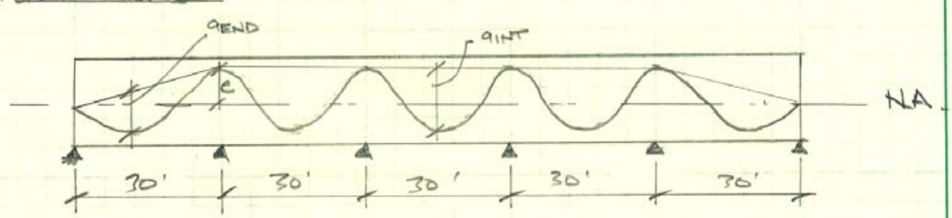
+ TARGET LOAD BALANCES

$0.75 w_{SW} = (0.75)(100) = 75$ psf

+ 2-HR FIRE RATING Req

RESTRAINED SLABS – 3/4" BOTTOM
UN-RESTRAINED SLABS – 1 1/2" BOTTOM
3/4" TOP

TENDON PROFILE



TENDON ORDINATE	TENDON (CG) LOCATION*
EXTERIOR SUPPORT	4.0"
INTERIOR SUPPORT – TOP	7.0"
INTERIOR SPAN – BOTTOM	1.0"
END SPAN – BOTTOM	1.75"

$q_{INT} = 7 - 1 = 6"$

$q_{END} = \frac{4+7}{2} - 1.75 = 3.75$

* CG ⟶ CENTER OF GRAVITY IS MEASURED FROM THE BOTTOM OF SLAB.

STAEDTLER® No. 937 811E Engineer's Computation Pad

PRESTRESS Force Required to BALANCE 75% of SW

$w_b = 0.75\, w_{SW} = 0.75(100)(40) = 3000 \text{ plf} = 3.0 \text{ k/ft}$

FORCE NEEDED to Counteract LOAD IN END BAY

$$P = \frac{w_b L^2}{8\, a_{end}} = \frac{(3.0)(30)^2}{8\left(\frac{3.75}{12}\right)} = 1080 \text{ k}$$

Check Precompression Allowance

$$\# \text{ TENDONS} = \frac{1080 \text{ k}}{26.6 \text{ k/TEN}} = 40.6 \rightarrow \text{USE } 40$$

$P_{ACT} = (40)(26.6) = 1064 \text{ k}$ $\quad w_b \text{ ADJ} = \frac{1064}{1080}(3.0) = 2.95 \text{ k/ft}$

Det. Actual Precompression Stresses

$$\frac{P_{ACT}}{A} = \frac{(1064)(1000)}{3840} = 277 \text{ psi} \quad < 700 \text{ psi} \checkmark \text{OK} \quad > 125 \text{ psi} \checkmark \text{OK}$$

Check Interior Span Force

$$P = \frac{w_b L^2}{8\, a_{int.}} = \frac{(3.0)(30)^2}{8\left(\frac{6}{12}\right)} = 675 < 1080 \text{ k}$$

$\therefore$ LESS THAN REQUIRED FOR INT. BAYS

ADJUST w_b (INT.)

$$w_b = \frac{1080}{675}(3.0) = 4.8 \text{ k/ft}$$

$w_D = (100 \text{ psf})(40) = 4000 \text{ plf} \rightarrow 4.0 \text{ k/ft} < 4.8 \text{ k/ft} \therefore \text{NG}$

Prestress Required to Balance 60% of SW

$w_b = 0.6 w_{SW} = (0.6)(100)(40) = 2400\,plf = 2.4\,k/ft$

Force Needed to counteract load in bay

$P = \frac{w_b L^2}{8 a_{end}} = \frac{(2.4)(30)^2}{8\left(\frac{3.75}{12}\right)} = 864\,k$

Check Precompression Allowance

$\#\,TENDON = \frac{864\,k}{26.6\,k/TEND} = 32.48 \rightarrow$ use 33

$P_{ACT} = (33)(26.6) = 877.8\,k \qquad w_b\,ADJ = \frac{877.8}{864}(2.4) = 2.44\,k/ft$

Det. Act. Precomp. Stress

$\frac{P_{ACT}}{A} = \frac{(877.8)\,1000}{3840} = 228.6\,psi$ $< 300\,psi$ ✓ OK
$> 125\,psi$ ✓ OK

Check Int. Span Force

$P = \frac{w_b L^2}{8 a_{int}} = \frac{(2.4)(30)^2}{8\left(\frac{6}{12}\right)} = 540\,k < 877.8\,k$

∴ Less Force Req. For Int. Bays

Adj w_b (Int.)

$w_b = \frac{877.8\,(2.4)}{540} = 3.9\,k/ft$

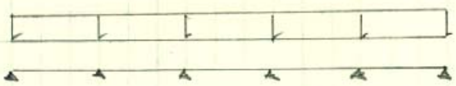
$w_D = 4.0\,k/ft > 3.9\,k/ft \rightarrow \frac{w_b}{w_D} = 98\%$ ✓ OK

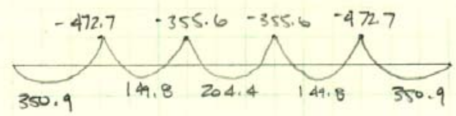
Effective Prestress Force - $P_{eff} = 878\,k$

SLAB STRESSES

DEAD LOAD MOMENTS

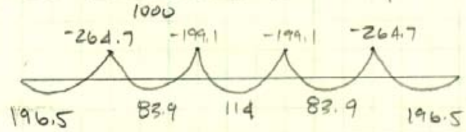
$$w_{DL} = \frac{(125\ psf)(40')}{1000} = 5\ k/ft$$





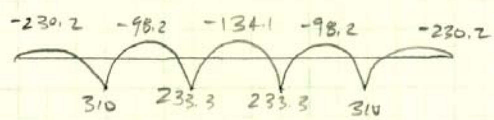
LIVE LOAD MOMENTS

$$w_{LL} = \frac{(70\ psf)(40')}{1000} = 2.8\ k/ft$$



TOTAL BALANCED MOMENT

$$w_b\ AVG = -\left[\frac{30((2)(2.44) + 3(3.92))}{5(30)}\right] = -3.3\ k/ft$$



Stage 1: Stresses Immediately after Jacking (DL+PT) (ACI 18.4.1)

MIDSPAN STRESSES:

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

$$f_{bot} = \left(\frac{+M_{DL} - M_{BAL}}{z}\right) - P/A$$

ENDS:

$$f_{top} = \frac{(-350.9 + 230.2)(12000)}{5120} - 228.6 = -512 \text{ psi comp}$$

$0.6f'_{ci} > 512$ ✓ OK

$$f_{bot} = \frac{(350.9 - 230.2)(12000)}{5120} - 228.6 = 55 \text{ psi tens}$$

$3\sqrt{f'_{ci}} > 55$ ✓ OK

INTERMEDIATE SPANS:

$$f_{top} = \frac{(-149.8 + 98.2)(12000)}{5120} - 228.6 = -350 \text{ psi comp}$$

$0.6f'_{ci} > 350$ ✓ OK

$$f_{BOT} = \frac{(149.8 - 98.2)(12000)}{5120} - 228.6 = -108 \text{ psi comp}$$

$0.6f'_{ci} > 108$ ✓ OK

CENTER SPAN

$$f_{top} = \frac{(-204.4 + 134.1)(12000)}{5120} - 228.6 = -393 \text{ psi comp}$$

$0.6f'_{ci} > 393$ ✓ OK

$$f_{BOT} = \frac{(204.4 - 134.1)(12000)}{5120} - 228.6 = -64 \text{ psi comp}$$

$0.6f'_{ci} > 64$ ✓ OK

Support Stresses

$$f_{TOP} = \frac{(M_{DL} - M_{BAL})}{S} - \frac{P}{A}$$

$$f_{BOT} = \frac{(-M_{DL} + M_{BAL})}{S} - \frac{P}{A}$$

1st Support

$$f_{TOP} = \frac{(472.7 - 310)(12000)}{5120} - 228.6 = 153 \text{ psi Tension}$$

$$3\sqrt{f'_{ci}} > 153 \quad \checkmark \text{ok}$$

$$f_{BOT} = \frac{(-472.7 + 310)(12000)}{5120} - 228.6 = -610 \text{ psi Comp}$$

$$0.6 f'_{ci} > 610 \quad \checkmark \text{ok}$$

2nd Support

$$f_{TOP} = \frac{(355.6 - 233.3)(12000)}{5120} - 228.6 = 58 \text{ psi Ten.}$$

$$3\sqrt{f'_{ci}} > 58 \quad \checkmark \text{ok}$$

$$f_{BOT} = \frac{(-355.6 + 233.3)(12000)}{5120} - 228.6 = -515 \text{ psi comp}$$

$$0.6 f'_{ci} > 515 \quad \checkmark \text{ok}$$

STAGE 2: STRESSES @ SERVICE (DL+LL+PT) (18.33 & 184.2)

MIDSPAN:

$$f_{TOP} = \left(\frac{-M_{DL} - M_{LL} + M_{BAL}}{S}\right) - \frac{P}{A}$$

$$f_{BOT} = \left(\frac{M_{DL} + M_{LL} - M_{BAL}}{S}\right) - \frac{P}{A}$$

END SPAN

$$f_{TOP} = \frac{(-350.9 - 196.5 + 230.2)(1200)}{(5120)} - 228.6 = -972 \text{ psi Comp}$$

$$0.45 f'_c > 972 \quad \text{OK}$$

$$f_{BOT} = \frac{(350.9 + 196.5 - 230.2)(1200)}{5120} - 228.6 = 515 \text{ psi Tens}$$

$$6\sqrt{f'_c} < 515 \therefore \quad \text{REIN. REQ.}$$

INTERMEDIATE SPAN

$$f_{TOP} = \frac{(-141.8 - 83.9 + 98.2)(12000)}{5120} - 228.6 = -546 \text{ psi Comp}$$

$$0.45 f'_c > 546 \quad \text{OK}$$

$$f_{BOT} = \frac{(141.8 + 83.9 - 98.2)(12000)}{5120} - 228.6 = 89 \text{ psi tens.}$$

$$6\sqrt{f'_c} > 89 \quad \text{OK}$$

CENTER SPANS

$$f_{TOP} = \frac{(-204.4 - 114 + 134.1)(12000)}{5120} - 228.6 = -661 \text{ psi Comp}$$

$$0.45 f'_c > 661 \quad \text{OK}$$

$$f_{BOT} \ \frac{(204.4 + 114 - 134.1)(12000)}{5120} - 228.6 = 203 \text{ psi TEN.}$$

$$6\sqrt{f'_c} > 203 \quad \text{OK}$$

Support Stresses

$$f_{TOP} = \frac{M_{DL} + M_{LL} - M_{BAL}}{S} - P/A$$

$$f_{BOT} = \frac{-M_{DL} - M_{LL} + M_{BAL}}{S} - P/A$$

1ST

$$f_{TOP} = \frac{(472.7 + 264.7 - 310)(12000)}{5120} - 288.6 = 773 \text{ psi TEN.}$$

$$6\sqrt{f'c} < 773 \therefore \quad \text{Rein. Req.}$$

$$f_{BOT} = \frac{(-472.7 - 264.7 + 310)(12000)}{5120} - 288.6 = -1230 \text{ psi Comp}$$

$$0.45 f'c > 1230 \quad \checkmark \text{ok}$$

2nd

$$f_{TOP} = \frac{(355.6 + 199.1 - 233.3)(12000)}{5120} - 288.6 = 525 \text{ psi Ten}$$

$$6\sqrt{f'c} < 525 \therefore \quad \text{Rein. Req.}$$

$$f_{BOT} = \frac{(-355.6 - 199.1 + 233.3)(12000)}{5120} - 288.6 = -935 \text{ psi comp}$$

$$0.45 f'c > 935 \quad \checkmark \text{ok}$$

STAEDTLER No. 937 811E Engineer's Computation Pad

ULTIMATE STRENGTH: LOOK @ PANEL OF INTEREST ONLY (CENTER)

CALC. PRIMARY PT MOM. $M_1 = Pe$

e @ Supports → e = 3" (NA OF TENDONS)

$M_1 = \frac{(878)(3)}{12} = 219.5\ 'k$

CALC. Secondary PT Moments

$M_{sec} = M_{BAL} - M_1$

Supports ON EITHER SIDE OF CENTER PANEL

$M_{sec} = 233.3 - 219.5 = 13.8 \rightarrow 14\ 'k$

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

@ Support

$M_u = 1.2(-356) + 1.6(-199) + 14 = -732\ 'k$

@ MID SPAN

$M_u = 1.2(204.4) + 1.6(114) + 14 = 441.7\ 'k$

DETERMINE MIN BONDED REIN. OF CENTER PANEL

Positive Mom.

$f_t = 203\ psi > 2\sqrt{f'c} = 141\ psi$ ∴ REIN REQ.

MIN Pos Rein Req

$y = \frac{f_t}{(f_t + f_c)} h = \frac{(203)(8)}{(203+661)} = 1.88\ in$

$N_c = \frac{M_{DL+LL}}{S}(0.5)(y)(l_2) = \frac{(205+114)(12)}{5120}(0.5)(1.88)(40)(12)$

$N_c = 337\ k$

$A_{s\,min} = \frac{N_c}{0.5 f_y} = \frac{337}{(0.5)(60)} = 11.24\ in^2$

No. 937 811E
STAEDTLER Engineer's Computation Pad

$$A_{s\,min} = \frac{11.24}{40} = 0.281\ in^2/ft$$

USE #4@ 8in o.c. BOTTOM = $0.29\ in^2/ft$

MIN LENGTH → 1/3 CLR SPAN & CENTERED IN POSITIVE MOMENT (ACI 18.9.4.1)

Neg. Mom. Region

$A_{s\,min} = 0.00075\ A_{cf}$ (18.9.3.3)

Supports:

CONTROLS

$A_{cf} = \max(8in)[(30')(40')](12) = 3840\ in^2$

$A_{s\,min} = 0.00075(3840) = 2.88\ in^2$

USE 10 #5 ($3.1\ in^2$) or Equiv.

MUST SPAN 1/6 Clear SPAN min ON EACH END SIDE OF SUPPORT; @ LEAST 4 BARS Req in EACH DIRECTION & SHALL NOT EXCEED 12" SPACING & WITHIN 1.5 h from FACE of Support (12") (18.9.4.2 & 18.9.3.3)

Check min Reinforcement is Sufficient for Ultimate STRENGTH

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

$$A_{ps} = 0.153(33) = 5.049\ in^2$$

$$f_{ps} = f_{se} + 10000 + \frac{f'_c b d}{300 A_{ps}} \qquad L/h = 40/8 = 5 > 35$$

$$f_{ps} = 174000 + 10000 + \left[\frac{(5000)(40)(12)d}{300(5.049)}\right] = 184000 + 1584.47\,d$$

$$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c\, b}$$

@ Supports

$d = 8 - 3/4'' - 1/4'' = 7''$

$f_{ps} = 184000 + 1584.47(7) = 195092$ psi

$a = \frac{(3.1)(60) + (5.049)(195)}{0.85(5)(40 \times 12)} = 0.57$

$\phi M_n = 0.9[(3.1)(60) + (5.049)(195)](7 - \frac{0.57}{2})(1/12)$

$= 590'^k < 732'^k \therefore$ Rein. for ULT. Controls

$732 = 0.9[(A_s)(60) + (5.049)(195)] \cdot [7 - (\frac{\frac{A_s(60) + (5.049)(195)}{2040}}{2})](\frac{1}{12})$

A_s req'd $= 8.08$ in^2

(14) #7 @ Interior Supports

@ Center Span

$d = 8'' - 1'' = 7''$

$f_{ps} = 184000 + 1584.17(7) = 195092$ psi

$a = \frac{(11.24)(60) + (5.049)(195)}{0.85(5)(40)(12)} = 0.81$

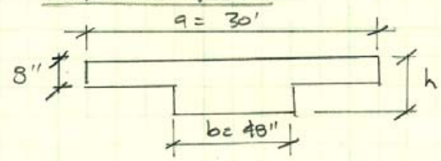
$\phi M_n = \frac{0.9}{12}[(11.24)(60) + (5.049)(195)](7 - \frac{0.81}{2})$

$= 820.5'^k > 441.7'^k$ Min Rein OK ✓

4 @ 8'' oc

Short SPAN

PROPORTION WSB



FROM PTI TECH. NOTE (AALAMI)

$h \leq 2t = 16$, $b \geq 3h = 24''$ ✓ OK

USE $h = 16''$

FOR CALC. USE EQ. UNIFORM SLAB Thickness (h_e)

$$h_e = \left[\frac{(t)(a) + b(h-t)}{a}\right] = \frac{[(8)(30)(12) + (48)(8)]}{(30)(12)} = 9.07''$$

LOADING

$SW = \left(\frac{9.07}{12}\right)(150) = 114$ psf

$SDL = 25$ psf

$LL = 70$ psf

Sect PROP

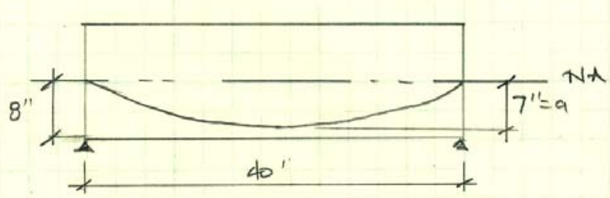
$A = (30)(12)(8) + (48)(8) = 3264 \text{ in}^2$

$S = \frac{(30)(12)(9.07)^2}{6} = 4933 \text{ in}^3$

TREAT AS SINGLE SPAN → Simplifying Assumption FOR this scematic Design. WILL NEED TO BE Addressed if SELECTED AS ALTERNATIVE.

TENDON PROFILE

Assume a @ 7''



Prestress Req to Balance 70% of SW

$w_b = 0.7(114)(30) = 2394 \text{ plf} \rightarrow 2.4 \text{ k/ft}$

Force Needed to Counteract Load

$$P = \frac{w_b L^2}{8a} = \frac{(2.4)(40)^2}{8(\frac{7}{12})} = 823^k$$

Check Precomp. Allowance

$$\# \text{ TENDONS} = \frac{823^k}{26.6^k/\text{TEN}} = 30.9 \rightarrow \text{USE } 31$$

$P_{ACT} = (31)(26.6) = 824.6^k \qquad w_b \text{ ADJ} = \frac{824.6}{823}(2.4) = 2.4^k/\text{ft}$

Determining Act Precomp. Stress

$$\frac{P_{ACT}}{A} = \frac{824.6}{3264}(1000) = 252.6 \text{ psi} < 300 \text{ psi} \checkmark \text{OK}$$

$> 120 \text{ psi}$ OK

$w_b = 2.4 \text{ k/ft}$

$w_D = (114)(30) = 3420 \text{ plf} \rightarrow 3.4 \text{ k/ft} > 2.4 \checkmark \text{OK}$

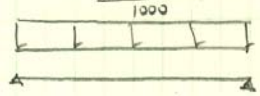
$\frac{w_b}{w_D} = 0.7$

$P_{eff} = 824.6k$

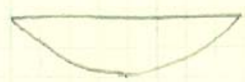
SLAB STRESSES

DEAD LOAD MOMENTS

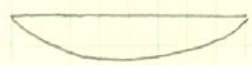
$w_{DL} = \frac{(114+25)(30)}{1000} = 3.9 \text{ k/Ft}$



$\frac{w l^2}{8} = \frac{3.9(30)^2}{8} = 439 \text{ k-Ft}$



LIVE LOAD MOMENTS

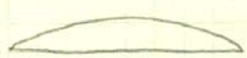


$w_{LL} = \frac{(70)(30)}{1000} = 2.1 \text{ k/Ft}$

$\frac{w l^2}{8} = \frac{(2.1)(30)^2}{8} = 236 \text{ k-Ft}$

TOTAL BALANCED MOMENT

$w_b = -2.4 \text{ k/Ft}$



$\frac{w l^2}{8} = \frac{(-2.4)(30)^2}{8} = -270 \text{ k-Ft}$

STAGE 1: STRESS IMMEDIATELY AFTER JACKING (DL+PT)

MIDSPAN

$$f_{TOP} = \frac{(-439+270)(12000)}{4933} - 252.6 = -663.7 \text{ psi comp}$$

$$0.6f'_{ci} > 663.7 \ \checkmark \text{ok}$$

$$f_{BOT} = \frac{(439-270)(12000)}{4933} - 252.6 = 158.5 \text{ psi Ten}$$

$$3\sqrt{f'_{ci}} > 158.5 \ \checkmark \text{ok}$$

Support Stresses

$$f_{TOP} = f_{BOT} = -\frac{P}{A} = -252.6 \text{ psi comp}$$

$$0.6f'_{ci} > 252.6 \ \checkmark \text{ok}$$

STAGE 2: STRESSES @ Service (DL+LL+PT)

MIDSPAN

$$f_{TOP} = \frac{(-439-236+270)(12000)}{4933} - 252.6 = -1237.8 \text{ psi comp}$$

$$0.45f'_c > 1237.8 \ \checkmark \text{ok}$$

$$f_{BOT} = \frac{(439+236-270)(12000)}{4933} - 252.6 = 732.6 \text{ psi Ten}$$

$$6\sqrt{f'_c} < 732.6 \ \therefore \ \text{Rein Req'd}$$

Support

-252.6 psi comp

$$0.45f'_c > 252.6 \ \text{ok} \ \checkmark$$

No. 937 811E
STAEDTLER
Engineer's Computation Pad

ULTAMATE STRENGTH (No Secondary Moments)

Load Combination

$M_u = 1.2 M_{DL} + 1.6 M_{LL} = 1.2(439) + 1.6(236) = 904.4$ k-ft

Determine Min Bonded Rein

Positive Moment

$f_t = 2309$ psi $> 2\sqrt{f'c} = 141$ psi. Rein Required

Min Pos. Rein Reg'd

$y = \frac{f_t}{f_t + f_c} h_c = \frac{(230.9)}{230.9 + 736.2}(9.07) = 2.2$ in

$N_c = \frac{439 + 236}{4933}(0.5)(2.2)(30)(144) = 650^k$

$A_{s\,min} = \frac{650}{(0.5)(60)} = 21.6 \text{ in}^2$

$A_{s\,min} = \frac{21.6}{30} = 0.72 \text{ in}^2/\text{ft}$

Use #6 @ 6" oc

Neg Rein Reg → see other span

Check min reinforcement is Suff. For ULT. STRENGTH

$A_{ps} = (0.153)(31) = 4.743 \text{ in}^2$ $\qquad L/h = 34/8 = 3.75 > 35$

$f_{ps} = 174000 + 10000 + \dfrac{(5000)(30)(12)d}{300(4.743)} = 184000 + 1265d$

$a = \dfrac{A_s f_y + A_{ps} f_{ps}}{0.85\, b f'c}$

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

@ Center Span

$d_{eff} = 9.07 - 1 = 8.07''$

$f_{ps} = 184000 + 1256(8.07) = 194123 \text{ psi}$

$a = \dfrac{(23.3)(60) + (4.743)(194)}{(0.85)(5)(30)(12)} = 1.51$

$\phi M_n = \dfrac{0.9}{12}\left[(23.2)(60) + 4.743(194)\right](9.07 - \frac{1.51}{2})$

$\phi M_n = 1442 \text{ ft-k} > 904.4'k$ ✓ ok

#6 @ 6 1/2" o.c.

No. 937 811E Engineer's Computation Pad STAEDTLER®

Check Punching Shear (TREAT LIKE NO WSB & 8" SLAB)

$w_u = 1.2(114 + 25) + 1.6(70) = 278.8$ psf

$V_u = w_u A = 0.279\,(30 \times 40 - (2.5)^2) = 333$ k

SAY $d_{eff} = 7.36"$

$b_o = (30 + 7.36)\,4 = 149.4$ in

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\,(149.4)(7.36) = 311$ k

$\phi V_c < V_u$ ∴ NEED DROP PANELS

$V_u \leq \phi V_c$

$333000 = 0.75(4)\sqrt{5000}\,[4(30+d)]\,d$

$d = 10"$ $\phi V_c = 339.4$ k

Need 11.25" DROP PANELS → USE 11 1/2" PANEL

Check BM SHEAR

@ PANEL ASSUME 10' PANEL

$V_u = 0.278(20 \times 30) = 160.2$ k

$\phi V_c = 0.75(2\sqrt{5000})(10 \times 12)(10.25) = 130.5$ k

$V_u > \phi V_c$ ∴ NG

INCREASE PANEL t

$V_c \leq \phi V_c$

$160.2 = 0.75(2)\sqrt{5000}\,(10 \times 12)\,d$

$d = 12.6"$ → SAY 12.75" USE 14" DROP PANEL

@ SLAB

$V_u = 0.278(30 \cdot 10) = 83.4$ k

$\phi V_c = 0.75(2)\sqrt{5000}\,(30 \cdot 12)(6.75) = 257.8$ k

$\phi V_c > V_u$ ✓OK

SYSTEM SW @ PANEL OF INTEREST

$$SW = \frac{\frac{8}{12}(150) \cdot 30 \cdot 40 + \frac{6}{12}(150)(10)(10) + \frac{8}{12}(150)(2)(40)}{(30)(40)} = 113\text{ psf}$$