The New York Times Building

New York, NY

IPD/BIM Thesis
Technical Report #2

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Structural Option

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Table of Contents

EXECUTIVE SUMMARY

In this second technical report, alternative floor systems were investigated for the New York Times Building. A typical bay of 30'-0" x 40'-0" was analyzed for four floor systems, which includes the existing system. The four systems were then compared based on framing impacts, total structural depth, column sizes, constructability, fire rating and fireproofing, lead time, foundation and lateral system impact, structural weight, and relative cost. The existing floor system is a composite steel system with normal weight concrete and two infill beams. The additional three systems investigated include:

- Composite Steel Deck with Lightweight Concrete and Three Infill Beams
- One-Way Slab Concrete with two different beam layouts
- Two-Way Post-Tensioned Slab

The design of composite steel deck with lightweight concrete and three infill beams floor system resulted in a 5" thick lightweight composite deck with a structural depth of approximately 22 ½" . This system is lighter than the existing system, but may prove problematic in lateral and vibration analyses. In addition, the shallower members did not even add an inch more to a typical bay which would not give the mechanical, electrical, and plumbing systems much more room. In terms of relative cost, it was the second cheapest of the system with $30.07 per square foot. Further investigation into vibration and lateral system impacts is required to determine if this system is a viable option.

The one-way slab floor system resulted in two designs being investigated to determine which layout would yield a shallower depth. The beams spanning the 40'-0" direction resulted in a structural depth of 28" from top of slab to bottom of girder with 24x20 beam and 28x24 girders. Though the structural depth is shallower than that of the other option, its structural self weight is 25% greater than the other option where the beams span the 30'-0" direction. The second option where the beams span the 30'-0" direction resulted in a deeper structural depth of 33" from the top of the slab to the bottom of the girder with 20x28 beams and 33x28 girders. The cost of the long span beams versus the short span beams was $31.40 per square foot and $29.44 per square foot respectively. This aspect of this layout makes it the cheapest of all the systems analyzed. Even though this system is easily constructed, it is approximately two times heavier than the existing system and will result in foundation and lateral system changes.

The post-tensioned two-way slab was designed to limit the structural depth and weight of a concrete floor system, compared to the bulky one-way system designed before it. It was determined that a 11 ½" thick slab was needed to span the typical 40'-0" direction. In addition to the post-tensioned slab, ½" drop panels were necessary to prevent punching shear. The cost of the post-tensioned slab is $30.52 per square foot which is approximately the average cost of the four floor systems. At the interior supports, a substantial amount of reinforcement was required for ultimate strength. A post-tensioned system will be further investigated, due to its economical shallow depths and long span capabilities. However, the post-tensioned system is implemented it will affect the column grid due to concrete columns, and the foundations and lateral system due to a weight of approximately two and a half times greater than the existing floor system.

INTRODUCTION

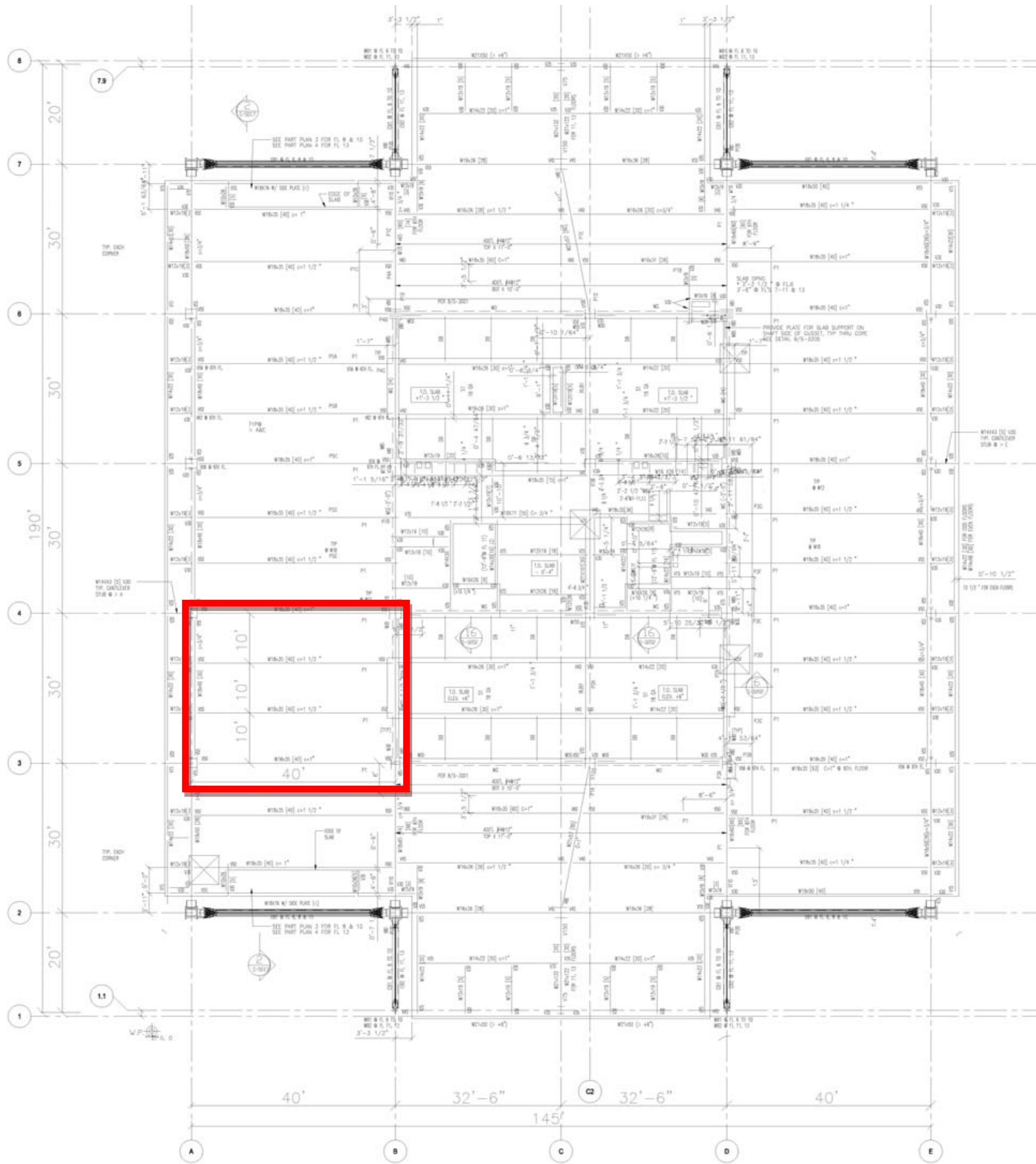


Figure 1: Typical Tower Framing Plan

The New York Times Headquarters Building is home to the New York Times newsroom and twenty six floors of Times offices, as well as several law firms whose offices are leased through Forest City Ratner. Designed by architect Renzo Piano in association with FFFOWLE Architects, it was intended to be a flagship building promoting 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.

The building rises fifty two stories with a height of 744 feet to the main roof. A 300 feet mast then extends up into the sky topping out at 1048 feet above Eighth Avenue between 40th and 41st Streets. The New York Times building totals 1.5 million square feet with the New York Times Company owning 800,000 square feet and Forest City Ratner Companies owning the other 700,000 square feet. It has one 16'-0" level below grade. The ground level contains a lobby, retail space and a glass-enclosed garden. The New York Times' newsroom occupies the entire five-story podium which is east of the tower structure. The tower ascends above the podium an additional forty eight stories. Story heights average approximately 13'-9" in the tower, lending a great view to the open office plans. At the mechanical floors on levels twenty eight and fifty one though, the floor height is approximately 27'-0" to accommodate equipment and two-story outriggers.

The steel structural system is comprised of composite floor beams and columns configured as shown in Figure 1, with lateral chevron braces in both the East-West and North-South directions in the core. Foundations are a combination of concrete spread footings and caissons to develop the required capacity. Many structural elements are also architectural details, including the exposed X bracing on the exterior of the structure and the built-up columns at the corner notches. Overall, the building exhibits ingenuity in design and construction.

The remainder of this report evaluates the existing floor framing system, as well as three alternative solutions. All designs are schematic, as the objective of this report is to study various floor systems that can be applied to the New York Times Building. Several variables are taken into account when comparing floor systems, such as framing layout, structural depths, fire protection, lead time, cost, and other structural impacts. All alternative floor systems will be designed and compared using a typical 30'-0" x 40'-0" interior bay, as seen boxed in red on Figure 1.

Design Codes and References

Design Codes

National Model Code:
- 1968 Building Code of the City of New York with latest supplements

Structural Standards:
- ASCE 7-98, Minimum Design Loads for Buildings and other Structures

Structural Design Codes:
- AISC – LRFD, Steel Construction Manual 2nd edition, American Institute of Steel Construction
- ACI 135-74 Manual of standard Practice for detailing Reinforced Concrete Structures
- ACI 318-99 American Concrete Institute Building Code Requirements for Reinforced Concrete
- ACI 530-95 Building Code Requirements for Masonry Structures
- National Building Code of Canada, 1995
- Uniform Building Code, 1997

Thesis Codes

National Model Code:
- 2006 International Building Code

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

Design Codes:
- AISC – LRFD, Steel Construction Manual 13th edition, American Institute of Steel Construction

Design Deflection Criteria

Lateral Deflections:
- Total building sway deflection for ten year wind loading is limited to H/450

Thermal Deflections:
- The shortening and elongating effects due to thermal fluctuations is designed to L/300.

Thesis Deflection Criteria

Gravity Deflections:
- Live load deflections for floor members are limited to L/360
- Total load deflections for floor members are limited to L/240

Lateral Deflections:
- Total building sway deflection for ten year wind loading is limited to H/450
- Allowable inter-story drift due to wind is H/400 to H/600 (ASCE 7-05 § CC.1.2)
- Building story sway deflection for seismic loading is limited to $0.015h_{sx}$ (ASCE 7-05 TABLE 12.12-1)

Thermal Deflections:
- The shortening and elongating effects due to thermal fluctuations is designed to L/300.

Material Strengths

Concrete:

Foundation Walls, Buttresses, S.O.G................Compressive strength of 4,000 psi, Normal Weight
Footings and Piers..Compressive strength of 5,950 psi, Normal Weight
Concrete on Metal Deck....................................Compressive strength of 4,000 psi, Normal Weight
Concrete Pads, Fill Slabs......................Compressive strength of 3,000 psi, Light Weight (115 PCF)
All Other Concrete..Compressive strength of 4,000 psi, Normal Weight
Reinforcing..ASTM A-615, Grade 60
Welded Wire Fabric..ASTM A185

Rock Anchor:

Dywidag Threadbars Anchors..ASTM A722, Grade 150 ksi
High Strength PVC Corrugated Sheathing..................................Compressive strength of 7,000 psi
Plates..ATSM A36

Structural Steel:

Rolled Shapes and Channels......................ASTM A572 or A992, Minimum yield strength of 50 ksi
Miscellaneous Angles..ASTM A36, Minimum yield strength of 36 ksi
"UAP" Channels........European Code EC3, Grade S-235JRG2, Minimum yield strength of 46 ksi
Tubes..ASTM A500, Grade B, Minimum yield strength of 42 ksi
Pipes..ASTM A500, Grade B, Minimum yield strength of 46 ksi
Plate Material used for Built-Up Members.............ASTM A572, Minimum yield strength of 50 ksi
Connections & Base Plate.......................ASTM A36 (36 ksi), A529 (42 ksi), A572 & A588 (50 ksi)
Diagonal & X-Braced Rods...ASTM A572, Grade 65

Metal Decking:

3" Composite Deck...........................ASTM A653 SQ, Grade 40, Minimum yield strength of 40 ksi
Headed Shear Studs ¾" ....ASTM A108, Type B

Connections:

Bolts...ASTM A325 or A490
Nuts..ASTM A563
Washers...ASTM A-F436
Anchor Bolts/ Rods...ASTM F-1554, Grade 55
Welding Electrodes E70XX..Tensile strength of 70 ksi

Masonry:

Mortar...Type M or S
Grout..Compressive strength of 3,000 psi
Concrete Masonry Units..Compressive strength of 3,000 psi
Reinforcing..ASTM A-615, Grade 60

Fire Protection and Fire Ratings

The fire rating of the existing structure is 2 hours. Therefore, to adequately compare fire ratings and fire protection of the existing floor system, the alternative systems must obtain a minimum rating of 2 hours if possible. All structural steel members must be protected against fire and must meet Underwriters Laboratories minimum requirement. Approximate weight and application of fire protection will be considered for the various methods such as cementitious fireproofing, sprayed fiber, intumescent paint, and gypsum board encasement. All reinforced concrete must be protected against fire and must meet ACI 318-08 minimum clear cover. For additional fire protection information for each system, see Appendix A: Existing Framing System, Appendix B: Lightweight Composite Framing System, Appendix C: One-Way Reinforced Concrete System, and Appendix D: Two-Way Post-Tensioned Concrete System.

Gravity Loads

The construction dead load for a typical floor system in this report includes the self weight of the floor system and a superimposed dead load of 25 psf for the ceiling, as well as mechanical, lighting and plumbing in the raised floor system and in the ceiling.

Live Load:		
Load Description	**ASCE 7-05 & NYC Bldg Code**	**Design Load**
Office:	50 psf	50+20 (for partitions) = 70 psf
Technology Floors:	100 psf	100 psf
Elevator Lobbies:	75 psf	75 psf
Corridors above First Floor:	80/75 psf	75 psf
All Other Lobbies & Corridors:	100 psf	100 psf
Exit Facilities:	100 psf	100 psf
Retail Areas:	100 psf	100 psf
Kitchen:	100 psf	150 psf
Cafeteria:	100 psf	100 psf
Auditorium (with fixed seats):	60 psf	100 psf
Light Storage Area:	125/100 psf	100 psf
Loading Dock:	250 psf	250 psf or actual weight whichever is greater
Mechanical Floors:	125 psf	150 psf or actual weight whichever is greater
Mechanical/Fan Rooms:	75 psf	75 psf or actual weight whichever is greater
Sidewalks	250 psf	600 psf
Roofs:	20 psf	30 psf + Drift
Roof Garden	100 psf	Not Specified

Table 1: Live Loads

The typical floor live load used in this report is 70 psf for office areas, 100 psf for the core and cafeteria floors, and 150 psf for mechanical floors. The typical floor system analyzed is in the office area with 70 psf. Live load reduction will not be considered in this report. For specific gravity loads of each system, see Appendix A: Existing Framing System, Appendix B: Alternative Composite Framing System, Appendix C: Alternative One-Way Concrete System, and Appendix D: Alternative Two-Way Concrete System.

Structural System Overview

Foundation

The foundation of the New York Times Headquarters 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 rock; Class 1-65 and 2-65 per the New York City Building Code, with a capacity of 20 - 40 ton per square foot. However, the rock at the southeast corner of the tower only had an 8 ton per square foot capacity; Class 4-65. Of the seven columns that fall within this area (indicated in Figure 2) 24-inch diameter concrete-filled steel caissons were used. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete.

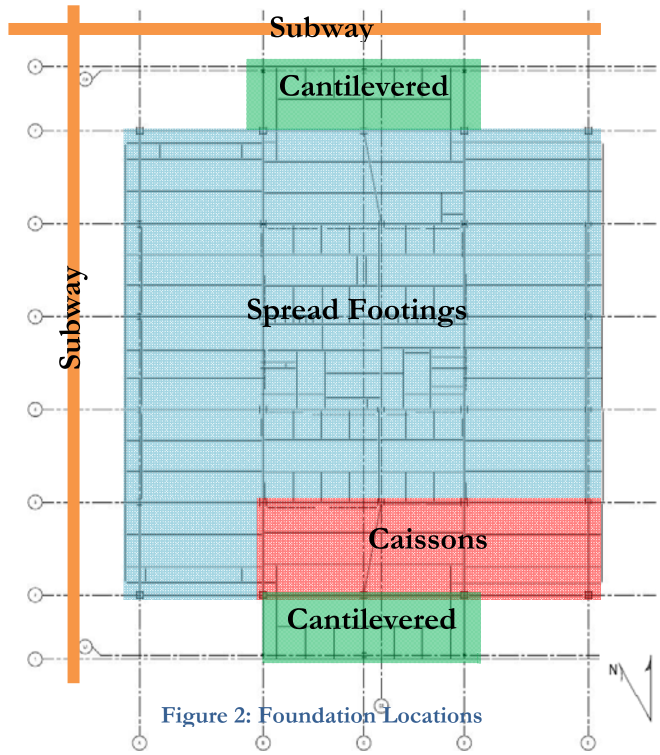


Figure 2: Foundation Locations

Under the other 21 columns (indicated on Figure 2) spread footings of unknown dimensions with a compressive strength of 6,000 psi are used to support the loads. The columns which fall in the cantilevered 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. Although, vibration effects on the foundation and building structure may have had an impact on the design.

Floor System

The floor system is a composite system with a typical bay size of 30'-0"x 40'-0" surrounding the 90'-0" x 65'-0" core. There are 60'-0" x 20'-0" cantilever bays on the north and south sides of the tower. The floor system is made up of 2 ½" normal weight concrete on 3" metal deck, typically spanning 10'-0" from W12x19 to W18x35 infill beams. The W12x19 and W18x35 beams span into W18x40 girders. The girders frame into the various built-up columns, box columns along the exterior and built-up non-box columns in the


Figure 3: 'Dog-leg' beam connection, courtesy of Thornton Tomasetti

core. Framing of the core consists of W12 and HSS shapes framing into W14 and W16 shapes which frame into W33 girders that frame into the core columns.

In the New York Times spaces, the structural slab is 16" below the finish floor and the spandrel panel, due to the raised floor system for the under floor mechanical systems. For all the exterior steel of the building to maintain a centerline at the center of the spandrel panel, a crooked connection or 'dog-leg' was used. See Figure 3 for an interior view of the 'dog-leg' connection during construction. The 'dog-leg' connection allows for the end of the beam to rise 10" before it leaves the interior of the building and penetrates the building envelope. Figure 4 shows the 'dog-leg' connection after the beam has penetrated the building envelope.


Figure 4: 'Dog-leg' penetrating building envelope

Columns

The 30"x30" box columns at the exterior notches (Figure 5) of the tower consist of two 30" long flange plates and two web plates inset 3" from the exterior of the column on either side. The two web plates of the welded box column vary from 7" thick at the ground floor to 1" thick at the fifty second floor. This is to account for the different steel areas needed for the higher forces at the bottom of the building. To maintain consistent proportions at all floors, a hierarchy of flange plate thicknesses was developed. At the ground floor, each flange plate is 4" thick and decreases to 2" thick at the fifty second floor. See Figure 6 for box column hierarchy. 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 perimeter column is engaged in the lateral system which will be described later.

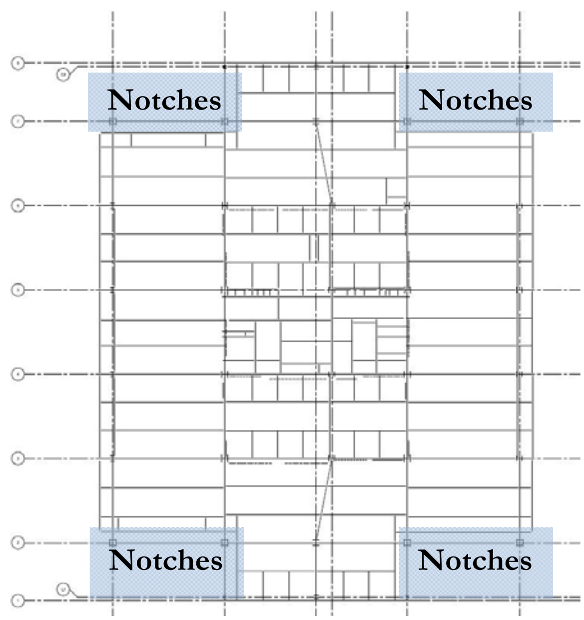


Figure 5: Typical Floor Plan with Column Notches

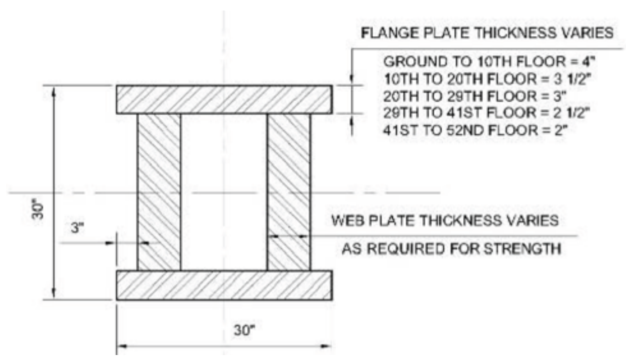


Figure 6: Box Column hierarchy, courtesy of Thornton Tomasetti

Vierendeel System

A Vierendeel system was used 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. The middle line of the cantilevered bays have beams moment connected to the columns thus creating the Vierendeel system and engaging every floor except at the outrigger levels. At the outrigger level; floor twenty eight and fifty two, large diagonal braces tie the middle line back to the core through the outrigger trusses. In extreme loading conditions, this provides a redundant load path. See Figure 7 for Vierendeel frame location. At the exterior beam lines of the cantilever, 2" diameter steel rods were connected from the columns to the ends of the beams to control deflection at every floor. This allowed the beams to be designed only for strength, thus avoiding bulky exterior members.

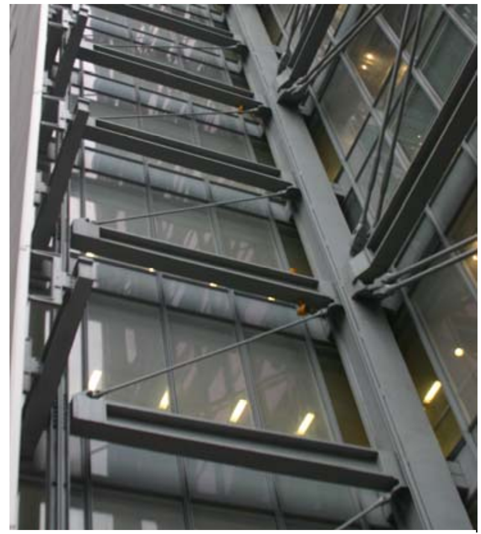
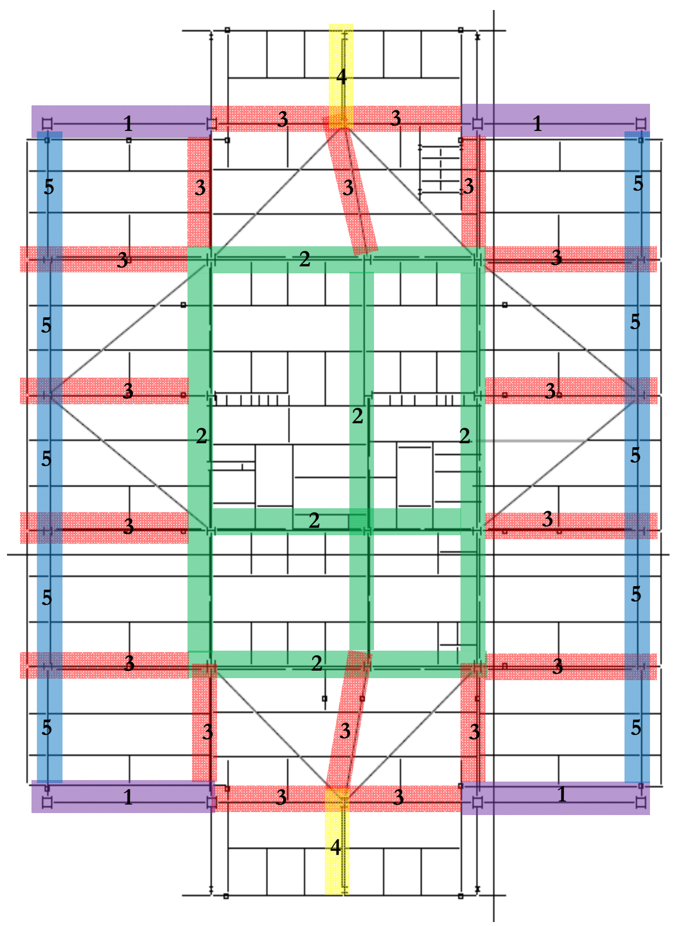


Figure 7: Cantilevered bays from exterior

Lateral System

The main lateral load resisting system for the tower of the New York Times Building consists of a centralized, steel braced frame core, with outriggers on the two mechanical floors (Levels twenty eight and fifty one). The structural core consists of concentric braces behind elevator shafts and eccentric braces at the elevator lobby entrances. At this time, the member sizes of these braces have yet to be disclosed, but the members were sized for strength. The core configuration remains consistent from the ground level to the twenty seventh floor as shown in Figure 8 and Figure 9. Above the twenty eighth floor, the low rise elevators were no longer required, and the number of bracing lines in the North-South direction was reduced from two to one.



Key:
Pre-Tensioned Steel Rod X-Bracing (1)
Chevron Core Bracing (2)
Outrigger Bracing (3)
Vierendeel System at Cantilever (4)
Thermal Trusses (5)

Figure 8: Mechanical Floor Framing Plan, Floor 28

Figure 9: Core bracing during construction

The outriggers on the mechanical floors engage all columns of the tower in the lateral system. The outriggers consist of single diagonal braces shown in Figure 8 and Figure 10. The outrigger system was designed to increase the stiffness of the tower by engaging the perimeter columns in the lateral system.

Figure 10: Outrigger bracing on mechanical floor

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 reduce lateral drift and acceleration, the structural engineers utilized the double story steel rod X-braces instead of increasing the member sizes of the main lateral force resisting system. These X-braces can be seen in Figure 8 on previous page and in Figure 11. The paired rods eliminate a center node and load sharing, in addition to eliminating eccentricities at the columns. 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 prevents the members from buckling and conforms 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's 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 11: Exposed exterior X-braced rods

Thermal differentials had to be considered due to interior steel members being maintained at room temperature and exposed steel members undergoing extreme temperature changes. Thornton Tomasetti designed the structure using a range of -10˚F to 130 ˚F. Due to the temperature deformation of the exterior columns and not the interior ones, differential deflection at upper floors exceeded L/100. To combat these thermal differentials, the outrigger trusses were utilized to even out the differential deflections. Thermal trusses were added along the east and west face at the twenty eighth and fifty first floors (Figure 12). These trusses provide bonus redundancy and limited deflection to L/300.

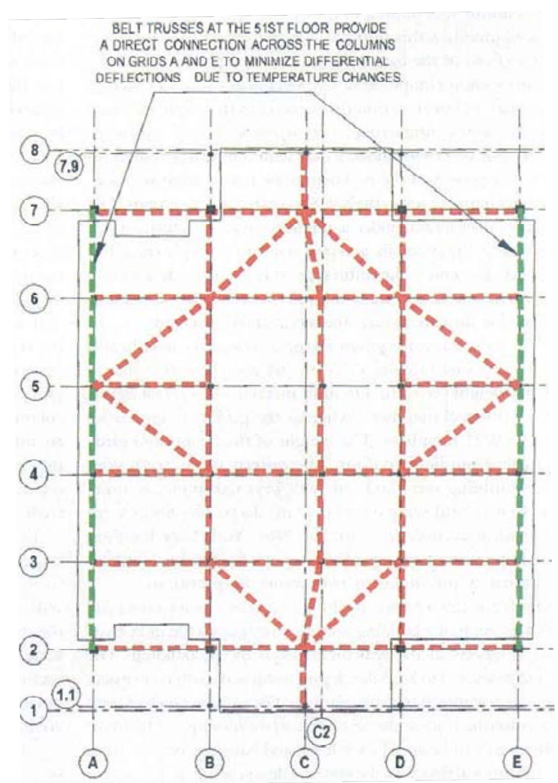


Figure 12: Thermal Truss, in green, located at the 28th and 51st floor, courtesy of Thornton Tomasetti

According to information obtained from the structural engineer, the podium of the New York Times Building was designed with a separate lateral system. Though information about the podium was not disclosed by the owner, an educated guess can be made about its lateral system. The podium contains the New York Times Newsroom and it can therefore be assumed that steel bracing, which would cut down on the usable floor space, would not be used. Also, the use of concrete shear walls would go against the architect's "transparent" and open plan layout of the building design. Therefore, it can be assumed that the lateral system of the podium, from the ground to sixth floor, is designed as a steel moment resisting frame.

EXISTING FLOOR FRAMING SYSTEM

The existing floor system is a composite system with a typical bay size of 30'-0"x 40'-0". 2 ½" normal weight concrete on 3" metal deck, typically span 10'-0" from infill beams. W12x19 and W18x35 beams span into a W18x40 girder. Figure 13 shows the typical bay analyzed for the existing floor framing system calculation. The calculations included in appendix A refer to the 13th edition of AISC for beam and girder design and Vulcraft for the steel deck design. The deck was checked to meet a 2 hour fire rating with spray on fireproofing and strength requirements. The beams were checked to meet strength requirements in addition to deflection requirements of L/240 for total load and L/360 for live load. It was also found that the calculated shear and flexural forces in the beams were fifteen percent less than Thornton Tomasetti's designed values. This is due to the fifteen percent increase Thornton Tomasetti added in to account for potential changes in office space and expansion of light MEP systems.

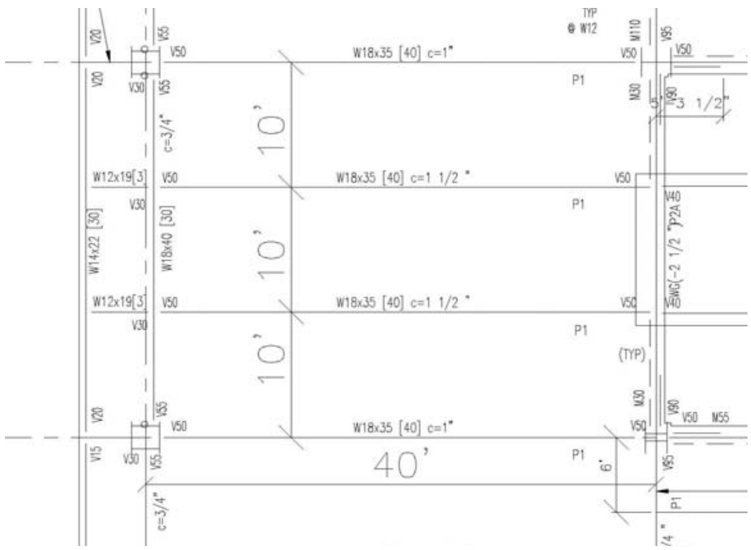


Figure 13: Typical Bay

Advantages and Disadvantages Analysis:

Composite steel is fairly simple to construct and combines the strength of concrete and steel, resulting in a system that is light and shallow. The current beams and girders are approximately 18" with a 5 ½" normal weight composite deck to make a combined structural depth of approximately 23 ½". The current system has a fire rating of 2 hours and structural weight of 57.5 psf including beams and girders. Currently, moment frames, bracings, outriggers, and built-up box columns are contributing to resisting lateral loads. This may have added additional cost to the system, where shear walls could have decreased cost, but at the time New York City labor union laws prevented concrete work occurring at the same time as steel work. Architecturally, this system allows for long open space without a really deep ceiling-to-floor sandwich. With the current column grid there are built up columns approximately every 30-40 feet. Though this allows for open space, the columns are big, bulky and conspicuous in the spaces where there are exposed; for example, in the lobby. The color of the fireproofing matches the window mullions, which exposes the viewer to look into the open air atrium if a bulky column is not obstructing their view. With the beams being 10 feet on center, it allows the mechanical systems to have room between the beams. However, due to the under floor air distribution system, the actual floor is 16" above the structural slab. In addition, to keep exterior beam line consistent a 'dog-leg' connection was designed for the end, which most likely added cost to the project.

Alternative Floor Framing System

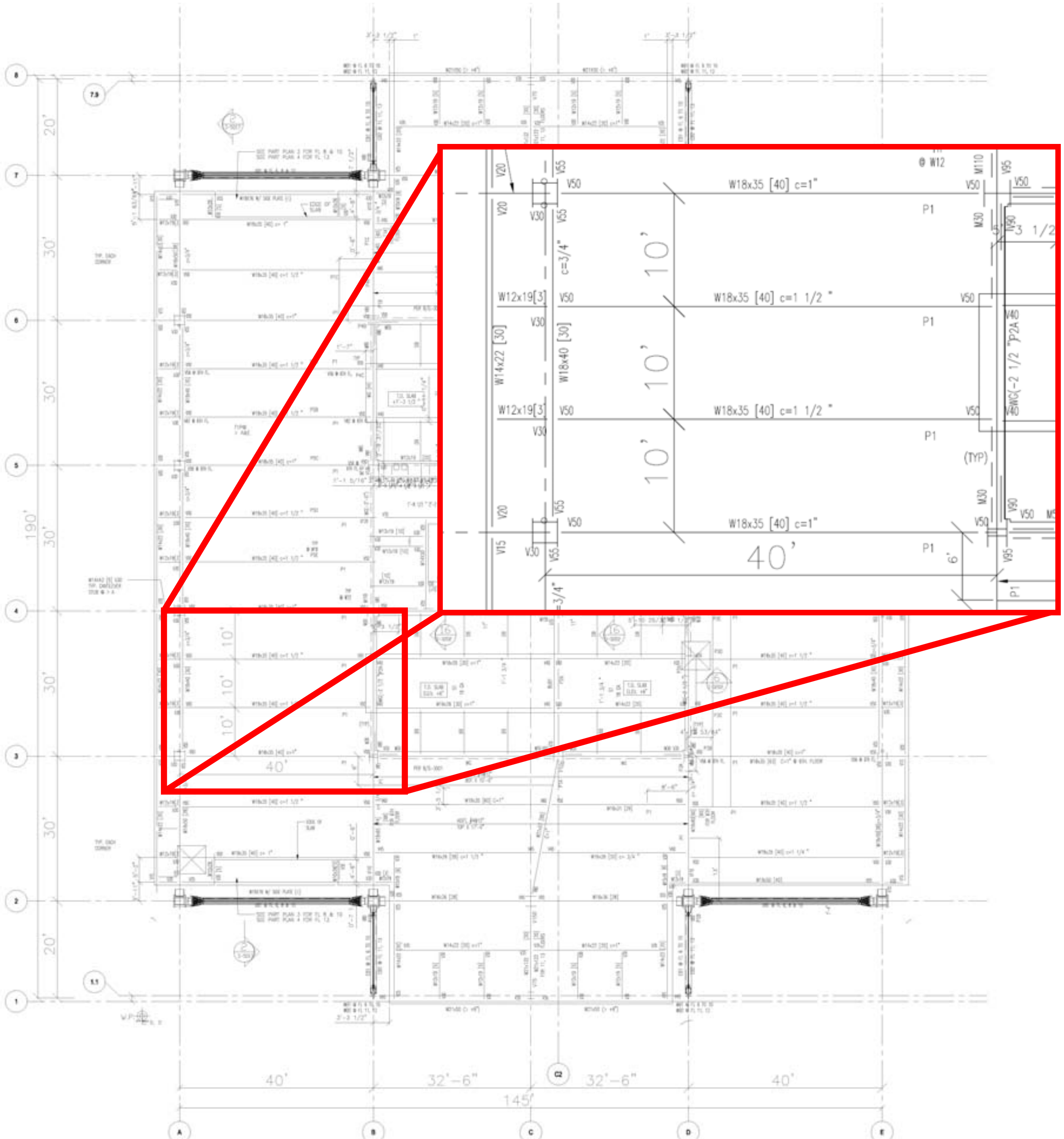


Figure 14: Typical 30'-0" x 40'-0" interior bay used

Three additional alternative floor systems were design and compared for the typical interior bay in Figure 14. The following floor systems were selected based on architectural, structural, and construction impacts on the existing building:

Lightweight Concrete Composite Deck on three infill beams
One-Way Reinforced Concrete Slab with two different beam layouts
Two-Way Post-Tensioned Concrete Slab

Composite Floor Framing System

For this alternative floor system the normal weight concrete was replaced with lightweight concrete and a 5 ½" metal deck was replaced with a 5" metal deck. The metal deck spans 6'-7" from infill beams. W12x19 and W16x26 beams span into a W18x35 girder. However, the W12x19s were used only to help size the girder for the bay. Figure 15 shows the typical bay analyzed for the alternative floor framing system calculation overlaid on the existing system. The calculation included in appendix B refers to the 13th edition of AISC for beam and girder design and Vulcraft for the steel deck design. The deck was checked to meet a 2 hour fire rating with spray on fireproofing and strength requirements. The beams were checked to meet strength requirements in addition to deflection requirements of L/240 for total load and L/360 for live load.

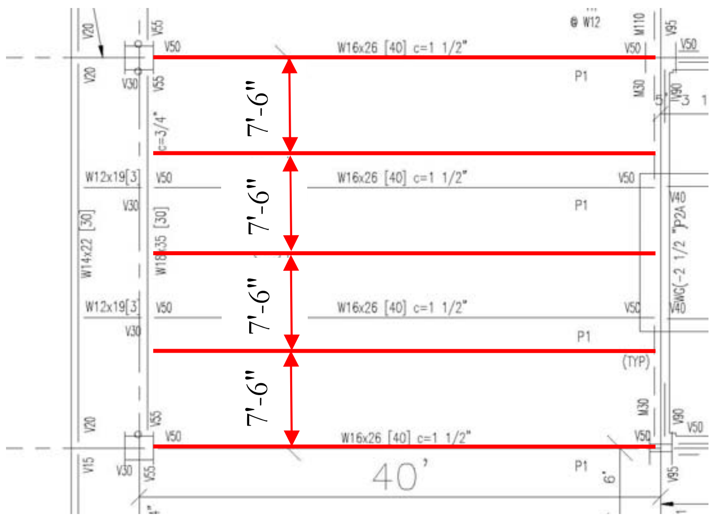


Figure 15: Alternative Composite System

Advantages and Disadvantages Analysis:

Utilizing composite action of the beams and concrete, the resulting system is lighter and shallower. The alternative beams and girder are approximately 16"-18"deep. Including the 5"deck the combine structural depth is approximately 21"-23". The current system has a fire rating of 2 hours and a structural weight of 40.37 psf including beams and girders. Vibration criteria was not evaluated for this report, however further investigation may indicate that the beams may need to be deeper or the slab may need to be thicker to prevent noticeable vibrations. The lightness of this system could affect the lateral system, causing additional bracing to resist lateral forces. For the foundation, the lightness could reduce the size. The addition of the extra beam would not affect the architecture of the space below or above, unless it is exposed or there are vibration issues. The thinner structural depth of the floor system is not so drastic as to change any heights of the mechanical systems in the ceiling or the under floor air distribution system. In addition, the column grid would not have to change, therefore keeping with Renzo Piano's open, light, and transparent feel for the building. Though the slab depth does not drastically change, it could possibly be cheaper to install, but on the other hand it could be slightly more expensive due to admixtures. The beams would need to be fireproofed, but since the current system already has composite beams with fireproofing, there might not be too much of a change, due to an addition beam with smaller depth. However, if cementitious fireproofing or sprayed fiber is replaced with intumescent paint, the cost will increase. Another construction issue that may arise is the coordination between the trades, because the additional beam may interfere with mechanical ductwork and electrical conduit.

One-Way Reinforced Concrete Floor System

This floor system uses a one-way reinforced concrete slab to transfer loads to concrete infill beams which frame into girders, which in turn transfer gravity loads to the columns. The typical interior bay size of 30'-0" x 40'-0" was used to design the floor system. For ease of calculation, it was assumed that this interior bay is continuous on both sides with same dimensions, therefore making slabs, beams, and girder typical. An additional assumption was that the slab and beams are cast integrally with 4000 psi concrete. The structural elements were designed to ACI 318-08 and meet flexural, shear, and deflection requirements stated by the code. PCA Column was use to approximate an appropriate column size to carry the gravity load with live load reduction. The columns assumed in the calculations are 33"x33" at the ground floor. A 2 hour fire rating was obtained by providing a minimum clear cover of ¾" for slabs and 1 ½" for beams and girders. See appendix C for design calculations.

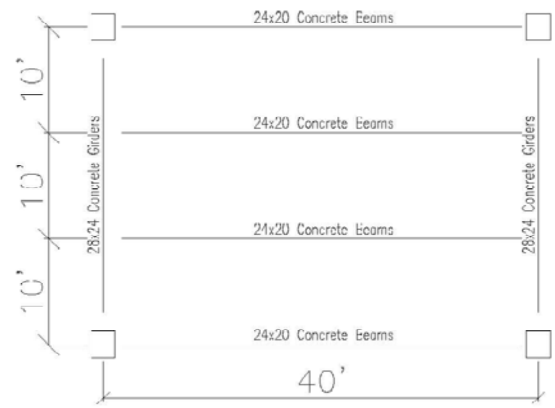


Figure 16:One-way Slab Option 1

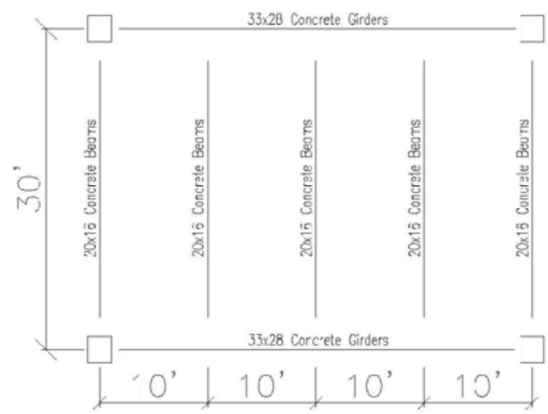


Figure 17: One-way Slab Option 2

Advantages and Disadvantages Analysis:
Two options were analyzed to compare sizes of beam within the one-way concrete slab alternative. Option one shown in Figure 16 has beams spanning the long direction, and option two shown in Figure 17 has beams spanning the short direction. Option one resulted in a 4 ½" thick slab with 24x20 beams and 28x24 girders. Option two resulted in a 4 ½" thick slab with 20x16 beams and 33x28 girders. (Refer to appendix C for reinforcing.) The self weight of the floor system is approximately 112 psf and 110 psf for option one and two respectively. This alternative results in the second heaviest floor system analyzed, which will alter the foundation design and the lateral system design. Attempting to keep the existing spans resulted in the sizes of the beam and girder stated above. This obviously changes the ceiling-to floor sandwich which can cause mechanical, electrical, and plumbing restriction and issues resulting in an even deeper ceiling-to floor sandwich. To decrease the depth of the beams, the spans should be shortened, which would affect the column sizes and placement, changing the space around them. One-way concrete systems are relatively easy to construct, due to less complex structural connections. Formwork will be required and will add an additional cost to the project, but concrete has a lesser lead time than steel. Relative costs of these options are $31.40 per square foot and $29.44 per square foot for one and two respectively. Option two is the cheaper alternative system analyzed, but will most likely result in the most drastic structural and architectural changes if implemented. The 2 hour fire rating is built into the concrete and therefore no additional cost for fireproofing will be needed.

Two-Way Post-Tensioned Floor System

This floor system uses a two-way post-tensioned slab. The typical interior bay size of 30'-0" x 40'-0" was analyzed using ACI 318-08 and resulted in a 11 ½" thick slab with (36) ½" diameter 270 ksi 7-wire strands in both directions. Minimum mild reinforcement was provided at midspan, while negative moment reinforcement at the supports was determined by strength requirements. An additional ½" was added for drop panels at the columns, which was required to meet punching shear requirements. A 2 hour fire rating was obtained by providing a 1 ½" clear cover at the bottom of the slab. (See appendix D for design and calculations.) For ease of calculation, it was assumed that this interior bay is continuous on both sides with the same dimensions.

Figure 18:Two-way post-tensioned floor system (www.wikipedia.com)

Advantages and Disadvantages Analysis:
A two-way post-tensioned floor system is efficient when spanning long distances and carrying heavy loads. This system has the smallest structural depth of all the floor systems analyzed in this report. This system is in keeping with Renzo Piano's open layout of the building. The thin ceiling-to-floor sandwich allows for more space for mechanical, electrical, and plumbing systems. The self weight of this alternative system is 144.8 psf. Though this system has the thinnest depth it is the heaviest of the alternative systems. This alternative system would also impact the foundations and lateral system due to the weight, causing the foundations and the lateral systems to be bigger in size. Construction for this system is difficult and requires an experienced construction team. Prior to construction, slab penetrations must be planned to avoid cutting post-tensioning strands. This system's relative cost is $30.52 per square foot. When compared to the lightweight composite steel system and option two of the one-way concrete system, the post-tension system is slightly more expensive. In addition, the post-tension system takes more time to construct, but with increased spans it has the ability to be more economical and efficient.

Conclusions

	Steel		Concrete		
	Existing Composite Steel	**LW Composite Steel**	**One-Way Long Beams**	**One-Way Short Beams**	**Two-Way Post Tensioned Slab**
Architectural Impact:					
Framing impact	N/A	Minor	Major	Major	Yes
Slab Depth (in)	5.50	5.00	4.50	4.50	11.50
Total Depth (in)	23.40	22.50	28.00	33.00	11.50
Column sizes (in)	30x30	30x30	33x33	33x33	33x33
MEP System impact	N/A	Minor	Major	Major	Minor-Medium
Construction Impact:					
Constructability	Medium	Medium	Easy	Easy	Hard
Fire Rating	2	2	2	2	2
Fireproofing	Yes	Yes	Built-in	Built-in	Built-in
Formwork	Minimal	Minimal	Yes	Yes	Yes
Lead Time	Medium	Medium	Short	Short	Short
Relative Cost*	$31.50/SF	$30.07/SF	$31.40/SF	$29.44/SF	$30.52/SF
Structural Impact:					
Foundation Impact	N/A	Possibly	Yes	Yes	Yes
Lateral System Impact	N/A	Possibly	Yes	Yes	Yes
Structural Weight (psf)	57.5	40.37	112	110	144.8
Vibrations	N/A	Additional Investigation Required			
Possible Alternative	N/A	Maybe	No	No	Yes

Table 2: Comparison of floor systems analyzed

* The system cost is a rough estimate using *RS Means Assemblies Cost Data* and *RS Means Facilities Construction Cost Data.*

After reviewing each floor system, it seems that the two way post-tensioned system is the most possible alternative to the current floor system, due to the system's relative cost, structural weight, structural depth, and impacts on mechanical, electrical, and plumbing systems. In regards to the construction process of post-tensioned system, experienced contractors with knowledge of post-tension construction methods and understanding are required to ensure proper construction. The column sizes for this system will be larger than the current system and it will possibly impact the architectural space and flow. Therefore, if this system is implemented, architectural attention must be maintained.

The 11 ½" slab of the post-tension system allows for a thinner ceiling-to-floor sandwich which in turn increases the height of the space below and allows for open space with the long spans. Vibrations and deflections can be reduced due to the balanced load that is produced by the ½" diameter 270 ksi 7-wire strands. However, at the interior supports, substantial amount of mild reinforcement was required for ultimate strength. No additional fireproofing is required.

The other two systems; composite steel framing with lightweight concrete and one-way slab with beams, had benefits which were outweighed by the disadvantages. The disadvantage for the one-way slab with beams system is the total structural depth increased 20%-40% of the existing system. The composite steel framing with lightweight concrete did not result in significant changes of structural depth and cost when compared to the existing system.

It has been determined that due to the information investigated through these sets of analyses that further investigation of the two-way post-tensioned slab system is required to determine feasibility of a possible proposal topic for AE Senior Thesis.

Appendix A: Existing Framing System

Checking the 5 1/2" Composite Deck

It was determined from Thornton Tomasetti's guidance and the architectural plans that the typical office bay metal decking chosen was a 20 gage, 3 inch deep deck with yield strength of 40 ksi, with 2.5 inches of concrete topping. The loading is as follows:

Superimposed Dead Loads:		
Ceiling	5	psf
MEP in raised floor system	12	psf
MEP in ceiling	8	psf
Fireproofing	2	psf
Total SIDL for Floor System Design:	**27**	**psf**
Typical Floor Live Loads:		
Office:	50	psf
Partitions:	20	psf
Total LL for Floor System Design:	**70**	**psf**
Total Superimposed Live Load for table	**97**	**psf**

Clear span = 10'-0" - 6" (thick of beam flange) = 9'-6"

The following table was taken from the 2001 Vulcraft catalog on page 48 for a 3 inch deep deck:

(N=9) 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"	3VLI22	7'-8	9'-7	9'-7	216	195	149	133	120	109	99	90	83	76	70	64	59	54	50
	3VLI21	8'-11	11'-3	11'-4	230	206	187	170	128	116	106	96	88	81	74	68	63	58	54
(t=2")	3VLI20	9'-6	11'-11	12'-4	241	216	196	178	163	150	111	101	93	85	78	72	66	61	57
	3VLI19	10'-8	13'-2	13'-7	265	237	214	194	178	163	151	140	102	94	86	79	73	67	62
	3VLI18	11'-8	14'-1	14'-6	289	261	238	218	201	186	173	161	151	142	106	98	92	86	80
44 PSF	3VLI17	12'-7	14'-11	15'-5	309	278	253	231	212	196	182	170	159	150	141	133	97	91	85
	3VLI16	13'-4	15'-8	15'-11	327	294	267	243	223	206	191	178	167	156	147	139	132	96	89
5 1/2"	3VLI22	7'-0	8'-9	8'-9	247	190	170	152	137	124	113	103	94	87	80	73	67	62	57
	3VLI21	8'-4	10'-4	10'-4	262	235	213	162	146	133	120	110	101	92	85	78	72	66	61
(t=2 1/2")	3VLI20	9'-0	11'-5	11'-9	275	247	223	203	186	140	127	116	106	97	89	82	76	70	65
	3VLI19	[illegible]	[illegible]	[illegible]	302	270	244	222	203	186	172	128	117	107	98	90	83	77	71
	3VLI18	11'-1	13'-5	13'-11	330	298	271	248	229	212	197	184	173	130	121	112	105	98	92
50 PSF	3VLI17	11'-11	14'-3	14'-9	352	317	288	263	242	224	208	194	182	171	128	119	111	104	97
	3VLI16	12'-8	15'-0	15'-5	373	335	304	277	255	235	218	203	190	178	168	159	117	109	102

Since 140 psf > 97 psf ∴ the deck capacity is **ok**

The following table was taken from the 2001 Vulcraft catalog on page 61 to check for fire protection:

Restrained Assembly Rating	Type of Protection	Concrete Thickness & Type (1)	U.L. Design No. (2,3,4)	Classified Deck Type: Fluted Deck	Classified Deck Type: Cellular Deck (5)	Unrestrained Beam Rating
2 Hr. (continued)	Sprayed Fiber	2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			861 *	12VLI,3VLI		1,1.5 Hr.
			870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW	864 *	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
			733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.

A 3VL20 with 2.5” of normal weight concrete meets the 2 hour fire rating there **ok**

Typical Beam (W18x35 [40] c=1.5")

Material Properties:

Concrete	$f'_c =$	4	ksi
Beam	$F_y =$	50	ksi
	$F_u =$	65	ksi

Spacing:	10	ft
Span:	40	ft

Loads:

Dead Loads:		
Slab:	0.053	ksf
Beam Weight:	0.004	ksf
MEP/Ceiling:	0.027	ksf
Live Loads:		
Non-Reduced:	0.070	ksf
Total dead load:	0.835	klf
Total live load:	0.700	klf
Const. dead load (unshored):	0.565	klf
Const. live load (unshored):	0.200	klf

$w_u=1.2D+1.6L=$	2.122	klf
$V_u=w_u l/2=$	42.440	k
$M_u=w_u l^2/8=$	424.400	ftk

b_{eff}= min of (span/8 , 1/2 distance to adj. bm, dist. To edge of slab)=	120.000	in					
Assume a=	1	in					
$Y_2=t_{slab}-a/2$=	5.000	in	Table 3-19				
Check I_{req}:							
$\Delta=l/240+camber$=	3.500	in					
$I_{req}=5w_{CDL}l^4/(384E\Delta)$=	320.631	in^4	<	510	in^4	OK	Table 3-20
Check member strength as un-shored:							
$w_{u(unshored)}=1.2D+1.6L$=	0.998	klf					
$M_{u(unshored)}=w_ul^2/8$=	199.600	ftk	<	249	ftk	OK	Table 3-19
PNA location=	5						
ΣQ_n=	260	k					Table 3-19
Check member strength:							
ϕM_n=	435	ftk	>	424.400	ftk	OK	Table 3-19
ϕV_n=	159	k	>	42.440	k	OK	
Check a:							
$a=\Sigma Qn/0.85f'_cb_{eff}$=	0.637	in	<	1	in	OK	
Check Δ_{LL}:							
$\Delta_{LL}=l/360$=	1.333	in	I_{LB}=	1170	in^4	Table 3-20	
$\Delta_{LL}=5w_{LL}l^4/(384EI_{LB})$=	1.188	in	<	1.333	in	OK	
Check studs:							
Q_n=	17.2	kips/stud		Table 3-21			
# of studs=$\Sigma Q_n/Q_n$=	15.116	use	16	studs/side			
Total studs=	32						

Since the number of shear studs actually used - 40 studs - is greater than 32 studs, therefore shear studs OK.

Typical Beam (W12x19 [3] c=0")

Material Properties:

Concrete	$f'_c =$	4	ksi
Beam	$F_y =$	50	ksi
	$F_u =$	65	ksi

Spacing:	10	ft
Span:	5.33	ft

Loads:

Dead Loads:		
Slab:	0.053	ksf
Beam Weight:	0.002	ksf
MEP/Ceiling:	0.027	ksf
Live Loads:		
Non-Reduced:	0.070	ksf
Total dead load:	0.819	klf
Total live load:	0.700	klf
Const. dead load (unshored):	0.549	klf
Const. live load (unshored):	0.200	klf
w_u=1.2D+1.6L=	2.103	klf
$V_u=w_u l/2$	5.607	k
$M_u=w_u l^2/8$	7.477	ftk
b_{eff}=	16.000	in
Assume a=	1.5	in
$Y_2=t_{slab}-a/2$=	4.750	in

Table 3-19

Check I_{req}:

Δ=l/240+camber=	0.267	in					
$I_{req}=5w_{CDL}l^4/(384E\Delta)$=	1.292	in^4	<	130	in^4	OK	Table 3-20

Check member strength as un-shored:

$w_{u(unshored)}$=1.2D+1.6L=	0.979	klf					
$M_{u(unshored)}=w_u l^2/8$=	3.480	ftk	<	92.6	ftk	OK	Table 3-19
PNA location=	7						
ΣQ_n=	69.7	k					Table 3-19

Check member strength:

ϕM_n=	141.5	ftk	>	7.477	ftk	OK	Table 3-19
ϕV_n=	85.7	k	>	5.607	k	OK	

Check a:

$a=\Sigma Qn/0.85f'_c b_{eff}$=	1.281	in	<	1.5	in	OK

Check Δ_{LL}:

$\Delta_{LL}=l/360$=	0.178	in	I_{LB}=	261	in^4	Table 3-20
$\Delta_{LL}=5w_{LL}l^4/(384EI_{LB})$=	0.0017	in	<	0.178	in	OK

Check studs:

Q_n=	17.2	kips/stud		Table 3-21
# of studs=$\Sigma Q_n/Q_n$=	4.052	use	5	studs/side
Total studs=	10			

Since the number of shear studs actually used - 3 studs - is less than 10 studs, it could be possible that there is a live reduction occurring in the area where the W12x19 are present, or the level of partial composite action is different, therefore shear studs OK.

Typical Girder (W18x40 [30] c=3/4")

Material Properties:

Concrete	f'_c =	4	ksi
Beam	F_y =	50	ksi
	F_u =	65	ksi

Span:	30.000	ft

Loads:

Dead Loads:		
P_{W18x35}:	16.700	k
P_{W12x19}:	2.184	k
Beam Weight:	0.040	klf
Live Loads:		
P_{W18x35}:	14.000	k
P_{W12x19}:	1.867	k
Total dead load (P_u):	18.884	k
Total dead load (w_u):	0.040	klf
Total live load(P_u):	15.867	k
Const. dead load (unshored):	12.764	k
Const. dead load (unshored):	0.040	klf
Const. live load (unshored):	4.533	k

P_u=1.2D+1.6L=	48.047	k	
w_u=1.2D+1.6L=	0.048	klf	
$V_u=w_u l/2+P_u$=	48.767	k	
$M_u=w_u l^2/8+P_u l/3$	485.875	ftk	
b_{eff}=	90	in	
Assume a=	1.5	in	
$Y_2=t_{slab}-a/2$=	4.75	in	Table 3-19

Check I_{req}:

Δ=l/240+camber=	2.250	in					
$I_{req}=5w_{DL}l^4/(384E\Delta)+P_{DL}l^3/(28E\Delta)$=	337.126	in^4	<	612	in^4	OK	Table 3-20

Check member strength as un-shored:

$P_{u(unshored)}$=1.2D+1.6L=	22.570	k					
$w_{u(unshored)}$=1.2D+1.6L=	0.048	klf					
$M_{u(unshored)}=w_u l^2/8+P_u l/3=$	231.101	ftk	<	294	ftk	OK	Table 3-19
PNA location=	4						
ΣQ_n=	351	k					Table 3-19

Check member strength:

ϕM_n=	516.5	ftk	>	485.875	ftk	OK
ϕV_n=	169	k	>	48.767	k	OK

Check a:

$a=\Sigma Qn/0.85f'_c b_{eff}$=	1.147	in	<	1.5	in	OK

Check Δ_{LL}:

Δ_{LL}=l/360=	1.000	in	I_{LB}=	1440	in^4	Table 3-20
$\Delta_{LL}=P_{LL}l^3/(28EI_{LB})$=	0.633	in	<	1	in	OK

Check studs:

Q_n=	17.200	kips/stud		Table 3-21
# of studs=$\Sigma Q_n/Q_n$=	20.407	use	21	studs/side
Total studs=	42			

Since the number of shear studs actually used - 30 studs - is less than 42 studs, it could be possible that there is a live reduction occurring in the area where the W12x19 are present, therefore affecting the W18x40 girder, or the level of partial composite action is different, therefore shear studs OK.

System Weight:

Item	Number/bay	Weight (lbs)	Total
W18x35	3	1400	4200
W18x40	1	1200	1200
Composite System + Deck	1	63600	63600
Self weight (PSF)=			57.50

Appendix B: Lightweight Composite Framing System

Checking the 5" Composite Deck

An alternative typical office bay metal decking chosen was a 20 gage, 3 inch deep deck with yield strength of 40 ksi, with 2 inch of concrete topping. The loading is as follows:

Superimposed Dead Loads:		
Ceiling	5	psf
MEP in raised floor system	12	psf
MEP in ceiling	8	psf
Fireproofing	2	psf
Total SIDL for Floor System Design:	**27**	**psf**
Typical Floor Live Loads:		
Office:	50	psf
Partitions:	20	psf
Total LL for Floor System Design:	**70**	**psf**
Total Superimposed Live Load for table	**97**	**psf**

Clear span = 7'-6"

The following table was taken from the 2001 Vulcraft catalog on page 49 for a 3 inch deep deck:

(N=14) LIGHTWEIGHT CONCRETE (110 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	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	14'-6	15'-0
	3VLI22	9'-1	11'-5	11'-5	141	127	115	83	75	67	60	54	49	45	40				
5"	3VLI21	9'-10	12'-4	12'-9	153	138	125	113	82	74	67	60	54	49	45	41			
	3VLI20	10'-6	13'-0	13'-5	163	147	133	121	110	102	72	65	59	54	49	44	40		
(t=2")	3VLI19	11'-10	14'-4	14'-10	185	166	150	136	124	114	105	97	68	62	57	52	47	43	
	3VLI18	13'-0	15'-4	15'-10	244	222	204	188	174	162	151	142	133	126	119	90	85	79	75
34 PSF	3VLI17	14'-0	16'-3	16'-6	262	238	218	201	185	172	161	150	141	133	126	119	113	85	80
	3VLI16	14'-5	16'-11	16'-11	277	254	234	217	202	189	177	166	157	149	141	134	127	99	94
	3VLI22	8'-5	10'-6	10'-6	161	121	107	95	85	77	69	62	56	51	46	42			
5 1/2"	3VLI21	9'-5	11'-10	12'-2	175	157	142	105	94	84	76	69	62	56	51	47	42		
	3VLI20	10'-0	12'-6	12'-11	186	167	151	138	126	91	82	74	67	61	56	51	46	42	
(t=2 1/2")	3VLI19	11'-3	13'-9	14'-3	211	189	171	155	142	130	120	86	78	71	65	59	54	49	45
	3VLI18	12'-4	14'-8	15'-2	278	253	232	214	198	184	172	161	152	118	110	103	97	91	85
39 PSF	3VLI17	13'-4	15'-7	16'-0	299	272	248	229	211	196	183	171	161	152	143	110	103	97	91
	3VLI16	14'-0	16'-5	16'-5	316	289	257	247	230	215	202	190	179	170	161	153	146	114	107

Since 163 psf > 97 psf ∴ the deck capacity is **ok**

The following table was taken from the 2001 Vulcraft catalog on page 61 to check for fire protection:

Restrained Assembly Rating	Type of Protection	Concrete Thickness & Type (1)	U.L. Design No. (2,3,4)	Classified Deck Type		Unrestrained Beam Rating
				Fluted Deck	Cellular Deck (5)	
2 Hr. (continued)	Sprayed Fiber	2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	822	[illegible]	[illegible]	[illegible]
			825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			861 *	12VLI,3VLI		1,1.5 Hr.
			870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	862 *	2VLI,3VLI		1 Hr.
		2 1/2" NW	864 *	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
			733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.
			840 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.

A 3VL20 with 2" of lightweight concrete meets the 2 hour fire rating there **ok**

Typical Beam (W16x26 [40] c=1.5")

Material Properties:

Concrete	$f'_c =$	4	ksi
Beam	$F_y =$	50	ksi
	$F_u =$	65	ksi

Spacing:	7.5	ft
Span:	40	ft

Loads:

Dead Loads:		
Slab:	0.036	ksf
Beam Weight:	0.0026	ksf
MEP/Ceiling:	0.027	ksf
Live Loads:		
Non-Reduced:	0.070	ksf
Total dead load:	0.493	klf
Total live load:	0.525	klf
Const. dead load (unshored):	0.291	klf
Const. live load (unshored):	0.150	klf

$w_u=1.2D+1.6L=$	1.432	klf
$V_u=w_u l/2=$	28.633	k
$M_u=w_u l^2/8=$	286.332	ftk

b_{eff}= min of (span/8 , 1/2 distance to adj. bm, dist. To edge of slab)=	90.000	in					
Assume a=	1	in					
$Y_2=t_{slab}-a/2$=	5.000	in					Table 3-19

Check I_{req}:

Δ=l/240+camber=	3.500	in					
$I_{req}=5w_{CDL}l^4/(384E\Delta)$=	164.884	in^4	<	301	in^4	OK	Table 3-20

Check member strength as un-shored:

$w_{u(unshored)}$=1.2D+1.6L=	0.589	klf					
$M_{u(unshored)}=w_ul^2/8$=	117.732	ftk	<	166	ftk	OK	Table 3-19
PNA location=	4						
ΣQ_n=	242	k					

Check member strength:

ϕM_n=	315	ftk	>	286.332	ftk	OK	Table 3-19
ϕV_n=	106	k	>	28.633	k	OK	

Check a:

$a=\Sigma Qn/0.85f'_cb_{eff}$=	0.791	in	<	1	in	OK

Check Δ_{LL}:

Δ_{LL}=l/360=	1.333	in	I_{LB}=	791	in^4	Table 3-20
$\Delta_{LL}=5w_{LL}l^4/(384EI_{LB})$=	1.318	in	<	1.333	in	OK

Check studs:

Q_n=	17.2	kips/stud		Table 3-21
# of studs=$\Sigma Q_n/Q_n$=	14.070	use	15	studs/side
Total studs=	30			

Since the number of shear studs used - 40 studs - is greater than 30 studs, therefore shear studs OK

Typical Beam (W12x19 [3] c=0")

Material Properties:

Concrete	f'_c =	4	ksi
Beam	F_y =	50	ksi
	F_u =	65	ksi

Spacing:	10	ft
Span:	5.33	ft

Loads:

Dead Loads:		
Slab:	0.036	ksf
Beam Weight:	0.002	ksf
MEP/Ceiling:	0.027	ksf
Live Loads:		
Non-Reduced:	0.070	ksf
Total dead load:	0.650	klf
Total live load:	0.700	klf
Const. dead load (unshored):	0.380	klf
Const. live load (unshored):	0.200	klf

w_u=1.2D+1.6L=	1.900	klf	
V_u=w_ul/2	5.068	k	
M_u=$w_u$$l^2$/8	6.757	ftk	
b_{eff}=	16.000	in	
Assume a=	1.5	in	
Y_2=t_{slab}-a/2=	4.750	in	Table 3-19

Check I_{req}:

Δ=l/240+camber=	0.267	in					
I_{req}=5$w_{CDL}$$l^4$/(384EΔ)=	0.895	in^4	<	130	in^4	OK	Table 3-20

Check member strength as un-shored:

$w_{u(unshored)}$=1.2D+1.6L=	0.776	klf					
$M_{u(unshored)}$=$w_u$$l^2$/8=	2.761	ftk	<	92.600	ftk	OK	Table 3-19
PNA location=	7						
ΣQ_n=	69.7	k					Table 3-19

Check member strength:

ϕM_n=	141.5	ftk	>	6.757	ftk	OK	Table 3-19
ϕV_n=	85.7	k	>	5.068	k	OK	

Check a:

$a=\Sigma Qn/0.85f'_c b_{eff}$=	1.281	in	<	1.5	in	OK

Check Δ_{LL}:

$\Delta_{LL}=l/360$=	0.178	in	I_{LB}=	261	in^4	Table 3-20
$\Delta_{LL}=5w_{LL}l^4/(384EI_{LB})$=	0.0017	in	<	0.178	in	OK

Check studs:

Q_n=	17.2	kips/stud		Table 3-21
# of studs=$\Sigma Q_n/Q_n$=	4.052	use	5	studs/side
Total studs=	10			

Since the number of shear studs used - 3 studs - is less than 10 studs, it could be possible that there is a live reduction occurring in the area where the W12x19 are present therefore shear studs OK

Please note: this beam was kept the same in order to relatively size the girder.

Typical Girder (W18x35 [30] c=3/4")

Material Properties:

Concrete	f'_c =	4	ksi
Beam	F_y =	50	ksi
	F_u =	65	ksi

Span:	30.000	ft

Loads:

Dead Loads:		
P_{W16x26}:	9.861	k
P_{W12x19}:	1.734	k
Beam Weight:	0.035	klf
Live Loads:		
P_{W16x26}:	10.500	k
P_{W12x19}:	1.867	k
Total dead load (P_u):	11.595	k
Total dead load (w_u):	0.035	klf
Total live load(P_u):	12.367	k
Const. dead load (unshored):	6.825	k
Const. dead load (unshored):	0.035	klf
Const. live load (unshored):	0.533	k
P_u=1.2D+1.6L=	33.701	k
w_u=1.2D+1.6L=	0.042	klf
$V_u=w_u l/2+P_u$=	34.331	k
$M_u=w_u l^2/8+P_u l/3$	341.736	ftk
b_{eff}=	90.000	in
Assume a=	1	in

$Y_2=t_{slab}-a/2$=	5	in	Table 3-19

Check I_{req}:

Δ=l/240+camber=	2.250	in					
$I_{req}=5w_{DL}l^4/(384E\Delta)+P_{DL}l^3/(28E\Delta)$=	184.076	in^4	<	510	in^4	OK	Table 3-20

Check member strength as un-shored:

$P_{u(unshored)}$=1.2D+1.6L=	9.044	k					
$w_{u(unshored)}$=1.2D+1.6L=	0.042	klf					
$M_{u(unshored)}=w_u l^2/8+P_u l/3$=	95.163	ftk	<	249	ftk	OK	Table 3-19
PNA location=	7						
ΣQ_n=	129	k					Table 3-19

Check member strength:

ϕM_n=	363	ftk	>	341.736	ftk	OK
ϕV_n=	159	k	>	34.331	k	OK

Check a:

$a=\Sigma Qn/0.85f'_c b_{eff}$=	0.422	in	<	1	in	OK

Check Δ_{LL}:

Δ_{LL}=l/360=	1.000	in	I_{LB}=	906	in^4	Table 3-20
$\Delta_{LL}=P_{LL}l^3/(28EI_{LB})$=	0.784	in	<	1	in	OK

Check studs:

Q_n=	17.200	kips/stud		Table 3-21
# of studs=$\Sigma Q_n/Q_n$=	7.500	use	8	studs/side
Total studs=	16			

Since the number of shear studs used, 30 studs, is greater than 16 studs, therefore shear studs OK

System Weight:

Item	Number/bay	Weight (lbs)	Total
W16x26	4	1040	4160
W18x36	1	1080	1080
Composite System + Deck	1	43200	43200
Self weight (PSF)=			40.37

Appendix C: One-Way Reinforced Concrete System

Option 1: One Way Slab Design
Material Properties:

Concrete in slab	f'_c =	4000	psi
Concrete in beams	f'_c =	4000	psi
Reinforcement	f_y =	60000	psi

Loads:

Superimposed Dead Loads:

Ceiling:	0.005	ksf
MEP in raised floor system:	0.012	ksf
MEP in ceiling:	0.008	ksf
Total:	0.025	ksf
Concrete self weight:	0.150	kcf

Live Loads:

Non-Reduced:	0.070	ksf

Option 1:

Slab span (l_n):	10	ft
Beam span:	40	ft
Girder span:	30	ft

Preliminary h:

h_{slab}:	1/28 =	4.29	in	use	4.5	in
h_{beam}:	1/21 =	0.00	in	use	24	in
h_{girder}:	1/21 =	22.86	in	use	18	in
Beam	24	x	20			
Girder	28	x	24			
Assumed Col	33	x	33			

Slab Design:

$w_{D,\ superimposed}$ =	0.025	ksf
$w_{D,\ slab\ contribution}$ = hx150/12 =	0.056	ksf
w_L =	0.070	ksf
Analysis 1 ft width, b =	12	in
w_u = 1.2D+1.6L=	0.210	klf

Moments (assume continuous interior span):

$M^- = w_u l_n^2/11$	-22.85	kin			
$M^+ = w_u l_n^2/16$	15.71	kin			
$V_u = w_u l_n/2$	1.05	k			
Assume #	4	bars	for stirrups		
Assume #	4	bars	for flexure		
clear cover=	0.75	in			
$d= h\text{-cover-stirrup-}0.5d_{flexure} =$	3.00	in			
$A_s =$	0.4	in^2			

Check $A_{s,min}$:

$3\sqrt{f'_c}bd/f_y =$	0.114	in^2			
$200bd/f_y =$	0.120	in^2			
$A_{s,min} =$	0.120	in^2	<	0.4	**OK**

Check $A_{s,max}$:

$\rho_{max} =$	0.0206				
$A_{s,max} = \rho bd =$	0.7431	in^2	>	0.4	**OK**

Check $A_{s,temp}$:

$A_{s,temp} = 0.0018bh =$	0.0972	in^2	<	0.2	**OK**
Use $A_{s,temp} =$	0.2	in^2	@	18	in

Determine M_n:

$a=A_s f_y/.85f'_c b =$	0.588	in			
$\beta =$	0.850				
$c = a/\beta =$	0.692	in			
$\varepsilon_s = 0.003(d-c)/c =$	0.0100	in/in			
$\varepsilon_y = 60/29000 =$	0.0021	in/in	<	0.0100	**OK**
$\phi =$	0.9				
$\phi M_n=\phi A_s f_y(d-a/2)=$	58.45	kin	>	22.85	**OK**

Maximum Number of Bars (Table A.7)

Max numbers of bars =	4	>	2	**OK**

Minimum Number of Bars (Table A.8), for Crack Control

Min numbers of bars =	2	<	2	**OK**

Determine shear strength of beam without stirrups:

λ =	1				
$V_c = 2\lambda\sqrt{f'_c}\, b_w d$ =	4.55	k	>	1.05	**NO SHEAR REINF.**
ϕ =	0.75				
$\phi V_n = 0.5\phi V_c$ =	1.71	k			

Beam Design:

$w_{D,\ superimposed}$ =	0.150	ksf
$w_{D,\ slab\ contribution}$ = hx150/12 =	0.056	ksf
$w_{D,\ beam\ contribution}$ = $(h-t_{slab})$xbx150/144 =	0.406	klf
w_L =	0.070	ksf
Analysis Trib width10 ft =	120	in
w_u = 1.2D+1.6L=	4.083	klf

Moments (assume continuous interior span):

$M^- = w_u l_n^2/11$	-6431.05	kin
$M^+ = w_u l_n^2/16$	4421.35	kin
$V_u = w_u l_n/2$	77.57	k

Assume #	4	bars	for stirrups			
Assume #	9	bars	&	0	bars	for flexure
clear cover=	1.5	in				
b_w=	20	in				
d= h-cover-stirrup-$0.5d_{flexure}$ =	21.44	in				
Number of bars =	7	9	&	0	0	
A_s =	7.00	in^2				

Check If T-beam behavior occurs:

h_f=	4.5	in			
b_w+16h_f=	92	in			
b_w+2(.5clear distance)=	100	in			
.25span length=	120	in			
$b_{eff,int}$ =	92	in			
$M_{u,T\text{-}Beam} = \phi\, 0.85 f'_c b h_f(d-h_f/2)$ =	24,307	kin	>	6,431	**NO T-BEAM**

Determine M_{n1} for $\rho = \rho_{max\phi}$:

$\rho_{max\phi} = 0.85(f'_c/f_y)\beta(0.003/(0.003+0.005))$=	0.0181	
$A_{s1} = \rho_{max\phi}\, bd$=	7.744	in^2

$a=A_{s1}f_y/.85f'_cb =$	6.833	in^2			
$M_{n1}=0.85f'_cab(d-a/2)=$	8,374	kin	>	6,431	**SINGLY REINFORCED**

Check $A_{s,min}$:

$3\sqrt{f'_c}bd/f_y =$	1.356	in^2			
$200bd/f_y =$	1.429	in^2			
$A_{s,min} =$	1.429	in^2	<	7.00	**OK**

Check $A_{s,max}$:

$\rho_{max} =$	0.0206				
$A_{s,max} = \rho bd =$	8.8506	in^2	>	7.00	**OK**

Determine M_n:

$a=A_sf_y/.85f'_cb =$	6.176	in			
$\beta =$	0.850				
$c = a/\beta =$	7.266	in			
$\varepsilon_s =0.003(d-c)/c =$	0.0059	in/in			
$\varepsilon_y =60/29000 =$	0.0021	in/in	<	0.0059	**OK**
$\phi =$	0.9				
$\phi M_n=\phi A_sf_y(d-a/2)=$	6,936	kin	>	6,431	**OK**

Maximum Number of Bars:

$b_{min} =2c_c+2d_{tr}+nd_b+(n-1)4/3 =$	19.88		<	20	**OK**

Minimum Number of Bars (Table A.8), for Crack Control

Min numbers of bars =	3		<	7	**OK**

Determine shear strength of beam without stirrups:

$\lambda =$	1				
$V_c = 2 \lambda \sqrt{f'_c}\, b_w\, d =$	54.23	k	<	77.57	**SHEAR REINF.**
$\phi =$	0.75				
$\phi V_n=0.5\phi V_c =$	20.34	k			

Determine shear strength required by shear reinforcing:

V_u @ d =	70.27	k			
$V_s = V_u/\phi - V_c =$	39.47	k			
$V_s \le 8 \sqrt{f'_c}\, b_w\, d =$	216.93	k		**OK**	
No reinforcing required at:	68.59	in			

Determine maximum spacing of shear reinforcing:

$V_s \leq 4\sqrt{f'_c}\, b_w\, d$ =	108.47	k		**OK**	
s= d/2	10.72	in			
s= 24	24	in			
s_{max}=	10.72	in	use	10	in

Determine minimum shear reinforcement:

$A_v = 0.75\sqrt{f'_c}\, b_w\, s/f_y$ =	0.158	in²		
$A_v = 50\, b_w\, s/f_y$ =	0.167	in²		
$A_{v,min}$ =	0.167	in²	use	4
$A_{v,used}$ =	0.4	in³		

Design Shear Reinforcement:

$s = A_v f_y d/V_s$ =	13.04	in			
Use (2) # 4 stirrups: 1 @ 2",	7	@	10	in	each end

Girder Design:

$w_{D,\ superimposed}$ =	0.150	ksf
$w_{D,\ slab\ contribution}$ = hx150/12 =	0.056	ksf
$w_{D,\ beam\ contribution}$ = $(h-t_{slab})$b150/144x10ft =	0.041	ksf
$w_{D,\ girder\ contribution}$ = $(h-t_{slab})$b150/144 =	0.588	klf
w_L =	0.070	ksf
Analysis Trib width 30 ft =	360	in
w_u = 1.2D+1.6L=	12.953	klf

Moments (assume continuous interior span):

$M^- = w_u l_n^2/11$	-10492.41	kin
$M^+ = w_u l_n^2/16$	7213.53	kin
$V_u = w_u l_n/2$	176.48	k

Assume #	4	bars	for stirrups			
Assume #	9	bars	&	10	bars	for flexure
clear cover=	1.5	in				
b_w=	24	in				
d_t= h-cover-stirrup-0.5$d_{flexure}$ =	25.38	in				
d= h-cover-stirrup-1.5$d_{flexure}$ =	24.13	in				

$d' = \text{cover}+\text{stirrup}+0.5d_{flexure}$ =	2.56	in			
Number of bars =	5	9	&	5	10
A_s =	11.35	in^2			

Check If T-beam behavior occurs:

h_f=	4.5	in			
b_w+16h_f=	96	in			
$b_w+2(.5\text{clear distance})$=	456	in			
.25span length=	90	in			
$b_{eff,int}$ =	90	in			
$M_{u,T\text{-}Beam} = \phi\, 0.85f'_c b h_f(d-h_f/2)$ =	27,989	kin	>	10,492	**NO T-BEAM**
$A_{sf}=0.85f'_c(b-b_w)h_f/f_y$=	16.83	in^2			
$M_{nf}=0.85f'_c(b-b_w)h_f(d-h_f/2)$=	23,352	kin			
$M_{nw}=M_u/\phi-M_{nf}$=	(11,693)	kin			
ϕM_{nw}=	(10,278)	kin			

Determine M_{n1} for $\rho = \rho_{max\phi}$ and compare to M_{nw}:

$\rho_{max\phi} = 0.85(f'_c/f_y)\beta(0.003/(0.003+0.005))$=	0.0181				
$A_{s1}= \rho_{max\phi}\, bd$=	10.458	in^2			
$a=A_{s1}f_y/.85f'_c b$ =	7.690	in^2			
β =	0.850				
$c = a/\beta$ =	9.047	in			
$M_{n1}=0.85f'_c ab(d-a/2)$=	13,510	kin	>	(11,693)	**SINGLY REINFORCED**

Determine M_{n2}:

Mn2 = Mnw −Mn1 =	(25,203)	kin

Determine As2 assuming fs = fy:

As2 =Mn2/fy(d-d')=	-19.481	in^2

Find required A'sw:

f's=0.003(c-d')Es/c =	62358	psi
f's=	60000	psi
A'sw=As2(fy/f's)=	-19.48	in^2

Required Steel:

As= Asf+Asw=	8	in^2
A's=A'w=	(19)	in^2

Check $A_{s,min}$:

$3\sqrt{f'_c}bd/f_y$ =	1.831	in^2			
$200bd/f_y$ =	1.930	in^2			
$A_{s,min}$ =	1.930	in^2	<	11.35	**OK**

Check $A_{s,max}$:

ρ_{max} =	0.0206				
$A_{s,max} = \rho bd$ =	11.9522	in^2	>	11.35	**OK**

Determine M_n:

$a=A_sf_y/.85f'_cb$ =	8.346	in			
β =	0.850				
$c = a/\beta$ =	9.818	in			
$\varepsilon_s = 0.003(d-c)/c$ =	0.0048	in/in			
$\varepsilon_y = 60/29000$ =	0.0021	in/in	<	0.0048	**OK**
ϕ =	0.88				
$\phi M_n=\phi A_sf_y(d-a/2)$=	11,943	kin	>	10,492	**OK**

Maximum Number of Bars (Table A.7)

Max numbers of bars =	9		>	5	**OK**
Max numbers of bars =	8		>	5	**OK**

Minimum Number of Bars (Table A.8), for Crack Control

Min numbers of bars =	3		<	9	**OK**
Min numbers of bars =	3		<	8	**OK**

Determine shear strength of beam without stirrups:

λ =	1				
$V_c = 2 \lambda \sqrt{f'_c}\, b_w\, d$ =	73.24	k	<	176.48	**SHEAR REINF.**
ϕ =	0.75				
$\phi V_n=0.5\phi V_c$ =	27.46	k			

Determine shear strength required by shear reinforcing:

V_u @ d =	150.44	k		
$V_s = V_u/\phi - V_c$ =	127.35	k		
$V_s \leq 8 \sqrt{f'_c}\, b_w\, d$ =	292.95	k		**OK**
No reinforcing required at:	95.65	in		

Determine maximum spacing of shear reinforcing:

$V_s \leq 4\sqrt{f'_c}\, b_w\, d =$	146.48	k		**OK**	
s= d/2	12.06	in			
s= 24	24	in			
$s_{max}=$	12.06	in	use	12	in

Determine minimum shear reinforcement:

$A_v = 0.75\sqrt{f'_c}\, b_w\, s/f_y =$	0.228	in²		
$A_v = 50\, b_w\, s/f_y =$	0.240	in²		
$A_{v,min} =$	0.240	in²	use	4
$A_{v,used} =$	0.40	in³		

Design Shear Reinforcement:

$s = A_v f_y d/V_s =$	4.55	in				
Use (2) # 4 stirrups: 1 @ 2",	22	@	4	in		each end

System Weight:

Item	#/bay	Span (ft)	Total	
Beams	3	38.00	48750	LBS
Girder	1	27.25	17625	LBS
Slab	1		67500	LBS
Self weight (PSF)=			112	PSF

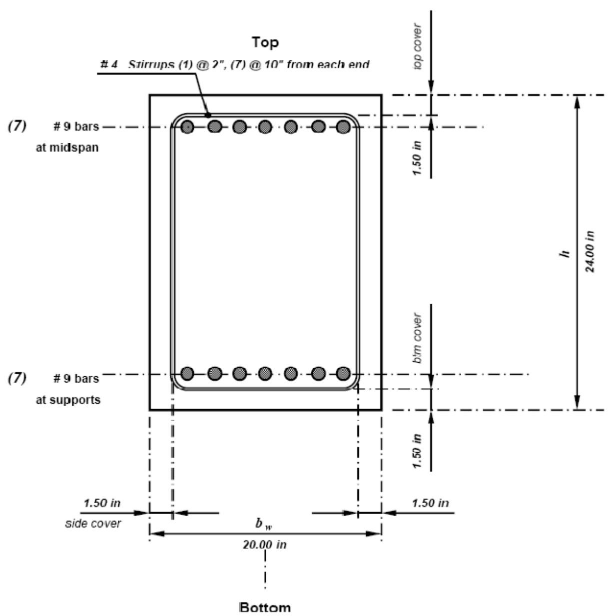


Figure 19: 24x20 Beam

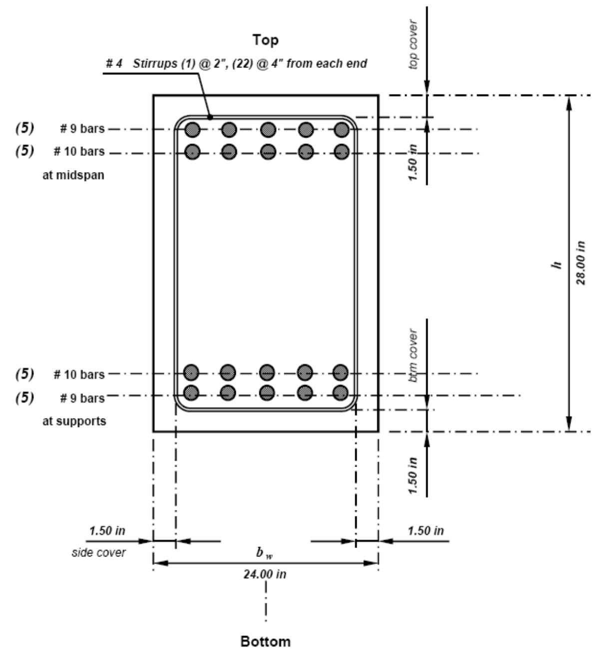


Figure 20: 28x24 Girder

Option 2: One Way Slab Design with Short Span Beams

Material Properties:

Concrete in slab	f'_c =	4000	psi
Concrete in beams	f'_c =	4000	psi
Reinforcement	f_y =	60000	psi

Loads:

Superimposed Dead Loads:

Ceiling:	0.005	ksf
MEP in raised floor system:	0.012	ksf
MEP in ceiling:	0.008	ksf
Total:	0.025	ksf
Concrete self weight:	0.150	kcf

Live Loads:

Non-Reduced:	0.070	ksf

Option 2:

Slab span (l_n):	10	ft
Beam span:	30	ft
Girder span:	40	ft

Preliminary h:

hslab:	l/28 =	4.29	in	use	4.5	in
hbeam:	l/21 =	0.00	in	use	18	in
hgirder:	l/21 =	17.14	in	use	24	in
beam	20	x	16			
Girder	33	x	28			
Assumed Col	33	x	33			

Slab Design:

$w_{D,\ superimposed}$ =	0.025	ksf
$w_{D,\ slab\ contribution}$ = hx150/12 =	0.056	ksf
w_L =	0.070	ksf
Analysis 1 ft width, b =	12	in
w_u = 1.2D+1.6L=	0.210	klf

Moments (assume continuous interior span):

$M^- = w_u l_n^2/11$	-22.85	kin

$M^+ = w_u l_n^2/16$	15.71	kin			
$V_u = w_u l_n/2$	1.05	k			
Assume #	4	bars	for stirrups		
Assume #	4	bars	for flexure		
clear cover=	0.75	in			
d= h-cover-stirrup-0.5$d_{flexure}$ =	3.00	in			
A_s =	0.4				

Check $A_{s,min}$:

$3\sqrt{f'_c}bd/f_y$ =	0.114	in²			
$200bd/f_y$ =	0.120	in²			
$A_{s,min}$ =	0.120	in²	<	0.4	**OK**

Check $A_{s,max}$:

ρ_{max} =	0.0206				
$A_{s,max} = \rho bd$ =	0.7431	in²	>	0.4	**OK**

Check $A_{s,temp}$:

$A_{s,temp} = 0.0018bh$ =	0.0972	in²	<	0.2	**OK**
Use $A_{s,temp}$ =	0.2	in²	@	18	in

Determine M_n:

$a = A_s f_y/.85f'_c b$ =	0.588	in			
β =	0.850				
$c = a/\beta$ =	0.692	in			
$\varepsilon_s = 0.003(d-c)/c$ =	0.0100	in/in			
$\varepsilon_y = 60/29000$ =	0.0021	in/in	<	0.0100	**OK**
ϕ =	0.9				
$\phi M_n = \phi A_s f_y(d-a/2)$=	58.45	kin	>	22.85	**OK**

Maximum Number of Bars (Table A.7)

Max numbers of bars =	4	>	2	**OK**

Minimum Number of Bars (Table A.8), for Crack Control

Min numbers of bars =	2	<	2	**OK**

Determine shear strength of beam without stirrups:

λ =	1				
$V_c = 2\lambda\sqrt{f'_c}\, b_w\, d$ =	4.55	k	>	1.05	**NO SHEAR REINF.**
ϕ =	0.75				
$\phi V_n = 0.5\phi V_c$ =	1.71	k			

Beam Design:

$w_{D,\ superimposed}$ =	0.150	ksf
$w_{D,\ slab\ contribution}$ = hx150/12 =	0.056	ksf
$w_{D,\ beam\ contribution}$ = $(h-t_{slab})$xbx150/144 =	0.258	klf
w_L =	0.070	ksf
Analysis 10 ft width, b =	120	in
w_u = 1.2D+1.6L=	3.905	klf

Moments (assume continuous interior span):

$M^- = w_u l_n^2/11$	-3260.79	kin					
$M^+ = w_u l_n^2/16$	2241.80	kin					
$V_u = w_u l_n/2$	54.02	k					
Assume #	4	bars	for stirrups				
Assume #	9	bars	&	0	bars		for flexure
clear cover=	1.5	in					
b_w=	16	in					
d= h-cover-stirrup-0.5$d_{flexure}$ =	17.44	in					
Number of bars =	5	9	&	0		0	
A_s =	5.00	in²					

Check If T-beam behavior occurs:

h_f=	4.5	in			
b_w+16h_f=	88	in			
b_w+2(.5clear distance)=	104	in			
.25span length=	90	in			
$b_{eff,int}$ =	88	in			
$M_{u,T\text{-}Beam} = \phi\, 0.85 f'_c b h_f (d-h_f/2)$ =	18,404	kin	>	3,261	**NO T-BEAM**

Determine M_{n1} for $\rho = \rho_{max\phi}$:

$\rho_{max\phi} = 0.85(f'_c/f_y)\beta(0.003/(0.003+0.005))$=	0.0181	
$A_{s1} = \rho_{max\phi}\, bd$=	5.039	in²

$a=A_{s1}f_y/.85f'_cb =$	5.558	in^2			
$M_{n1}=0.85f'_cab(d-a/2)=$	4,432	kin	>	3,261	**SINGLY REINFORCED**

Check $A_{s,min}$:

$3\sqrt{f'_c}bd/f_y =$	0.882	in^2			
$200bd/f_y =$	0.930	in^2			
$A_{s,min} =$	0.930	in^2	<	5.00	**OK**

Check $A_{s,max}$:

$\rho_{max} =$	0.0206				
$A_{s,max} = \rho bd =$	5.7594	in^2	>	5.00	**OK**

Determine M_n:

$a=A_sf_y/.85f'_cb =$	5.515	in			
$\beta =$	0.850				
$c = a/\beta =$	6.488	in			
$\varepsilon_s =0.003(d-c)/c =$	0.0051	in/in			
$\varepsilon_y =60/29000 =$	0.0021	in/in	<	0.0051	**OK**
$\phi =$	0.9				
$\phi M_n=\phi A_sf_y(d-a/2)=$	3,964	kin	>	3,261	**OK**

Maximum Number of Bars:

$b_{min} =2c_c+2d_{tr}+nd_b+(n-1)4/3 =$	14.96		<	16	**OK**

Minimum Number of Bars (Table A.8), for Crack Control

Min numbers of bars =	3		<	5	**OK**

Determine shear strength of beam without stirrups:

$\lambda =$	1				
$V_c = 2 \lambda \sqrt{f'_c}\, b_w\, d =$	35.29	k	<	54.02	**SHEAR REINF.**
$\phi =$	0.75				
$\phi V_n=0.5\phi V_c =$	13.23	k			

Determine shear strength required by shear reinforcing:

V_u @ d =	48.34	k			
$V_s = V_u/\phi - V_c =$	29.17	k			
$V_s \leq 8 \sqrt{f'_c}\, b_w\, d =$	141.16	k		**OK**	
No reinforcing required at:	57.55	in			

Determine maximum spacing of shear reinforcing:

$V_s \leq 4\sqrt{f'_c}\, b_w\, d$ =	70.58	k		**OK**	
s= d/2	8.72	in			
s= 24	24	in			
s_{max}=	8.72	in	use	8	in

Determine minimum shear reinforcement:

$A_v = 0.75\sqrt{f'_c}\, b_w\, s/f_y$ =	0.101	in^2		
$A_v = 50\, b_w\, s/f_y$ =	0.107	in^2		
$A_{v,min}$ =	0.107	in^2	use	4
$A_{v,used}$ =	0.4	in^3		

Design Shear Reinforcement:

$s = A_v f_y d/V_s$ =	14.35	in			
Use (2) # 4 stirrups: 1 @ 2",	8	@	8	in	each end

Girder Design:

$w_{D,\ superimposed}$ =	0.150	ksf
$w_{D,\ slab\ contribution}$ = hx150/12 =	0.056	ksf
$w_{D,\ beam\ contribution}$ = $(h-t_{slab})$b150/144x10ft =	0.026	ksf
$w_{D,\ girder\ contribution}$ = $(h-t_{slab})$b150/144 =	0.831	klf
w_L =	0.070	ksf
Analysis Trib width 30 ft =	360	in
w_u = 1.2D+1.6L=	12.713	klf

Moments (assume continuous interior span):

$M^- = w_u l_n^2/11$	-19242.97	kin
$M^+ = w_u l_n^2/16$	13229.54	kin
$V_u = w_u l_n/2$	236.77	k

Assume #	4	bars	for stirrups			
Assume #	9	bars	&	10	bars	for flexure
clear cover=	1.5	in				
b_w=	28	in				
d_t= h-cover-stirrup-0.5$d_{flexure}$ =	30.38	in				
d= h-cover-stirrup-1.5$d_{flexure}$ =	29.13	in				

$d' = \text{cover}+\text{stirrup}+0.5d_{flexure}$ =	2.56	in			
Number of bars =	7	9	&	7	10
A_s =	15.89	in^2			

Check If T-beam behavior occurs:

h_f=	4.5	in			
b_w+16h_f=	100	in			
$b_w+2(.5\text{clear distance})$=	332	in			
.25span length=	120	in			
$b_{eff,int}$ =	100	in			
$M_{u,T\text{-}Beam} = \phi\ 0.85f'_c b h_f(d-h_f/2)$ =	37,753	kin	>	19,243	**NO T-BEAM**
$A_{sf}=0.85f'_c(b-b_w)h_f/f_y$=	18.36	in^2			
$M_{nf}=0.85f'_c(b-b_w)h_f(d-h_f/2)$=	30,983	kin			
$M_{nw}=M_u/\phi-M_{nf}$=	(9,601)	kin			
ϕM_{nw}=	(8,424)	kin			

Determine M_{n1} for $\rho = \rho_{max\phi}$:

$\rho_{max\phi} = 0.85(f'_c/f_y)\beta(0.003/(0.003+0.005))$=	0.0181				
$A_{s1}= \rho_{max\phi}\, bd$=	14.730	in^2			
$a=A_{s1}f_y/.85f'_c b$ =	9.284	in^2			
β =	0.850				
$c = a/\beta$ =	10.922	in			
$M_{n1}=0.85f'_c ab(d-a/2)$=	22,743	kin	>	(9,601)	**SINGLY REINFORCED**

Determine M_{n2}:

Mn2 = Mnw −Mn1 =	(32,344)	kin

Determine As2 assuming fs = fy:

As2 =Mn2/fy(d-d')=	-20.295	in^2

Find required A'sw:

f's=0.003(c-d')Es/c =	66588	psi
f's=	60000	psi
A'sw=As2(fy/f's)=	-20.29	in^2

Required Steel:

As= Asf+Asw=	13	in^2
A's=A'w=	(20)	in^2

Check $A_{s,min}$:

$3\sqrt{f'_c}bd/f_y$ =	2.579	in^2			
$200bd/f_y$ =	2.718	in^2			
$A_{s,min}$ =	2.718	in^2	<	15.89	**OK**

Check $A_{s,max}$:

ρ_{max} =	0.0206				
$A_{s,max} = \rho bd$ =	16.8343	in^2	>	15.89	**OK**

Determine M_n:

$a=A_sf_y/.85f'_cb$ =	10.015	in			
β =	0.850				
$c = a/\beta$ =	11.782	in			
$\varepsilon_s = 0.003(d-c)/c$ =	0.0047	in/in			
$\varepsilon_y = 60/29000$ =	0.0021	in/in	<	0.0047	**OK**
ϕ =	0.88				
$\phi M_n=\phi A_sf_y(d-a/2)$=	20,173	kin	>	19,243	**OK**

Maximum Number of Bars (Table A.7)

Max numbers of bars =	10	>	7	**OK**
Max numbers of bars =	10	>	7	**OK**

Minimum Number of Bars (Table A.8), for Crack Control

Min numbers of bars =	3	<	9	**OK**
Min numbers of bars =	3	<	8	**OK**

Determine shear strength of beam without stirrups:

λ =	1				
$V_c = 2\lambda\sqrt{f'_c}\,b_w\,d$ =	103.15	k	<	236.77	**SHEAR REINF.**
ϕ =	0.75				
$\phi V_n=0.5\phi V_c$ =	38.68	k			

Determine shear strength required by shear reinforcing:

V_u @ d =	205.92	k	
$V_s = V_u/\phi - V_c$ =	171.40	k	
$V_s \leq 8\sqrt{f'_c}\,b_w\,d$ =	412.61	k	**OK**
No reinforcing required at:	126.13	in	

Determine maximum spacing of shear reinforcing:

$V_s \leq 4\sqrt{f'_c}\, b_w\, d =$	206.31	k		**OK**	
$s = d/2$	14.56	in			
$s = 24$	24	in			
$s_{max} =$	14.56	in	use	14	in

Determine minimum shear reinforcement:

$A_v = 0.75\sqrt{f'_c}\, b_w\, s/f_y =$	0.310	in²		
$A_v = 50\, b_w\, s/f_y =$	0.327	in²		
$A_{v,min} =$	0.327	in²	use	4
$A_{v,used} =$	0.40	in³		

Design Shear Reinforcement:

$s = A_v f_y d/V_s =$ 4.08 in

Use (2) # 4 stirrups: 1 @ 2", 31 @ 4 in each end

System Weight:

Item	#/bay	Span (ft)	Total	
Beams	4	27.67	31000	LBS
Girder	1	37.25	33250	LBS
Slab	1		67500	LBS
Self weight (PSF)=			110	PSF

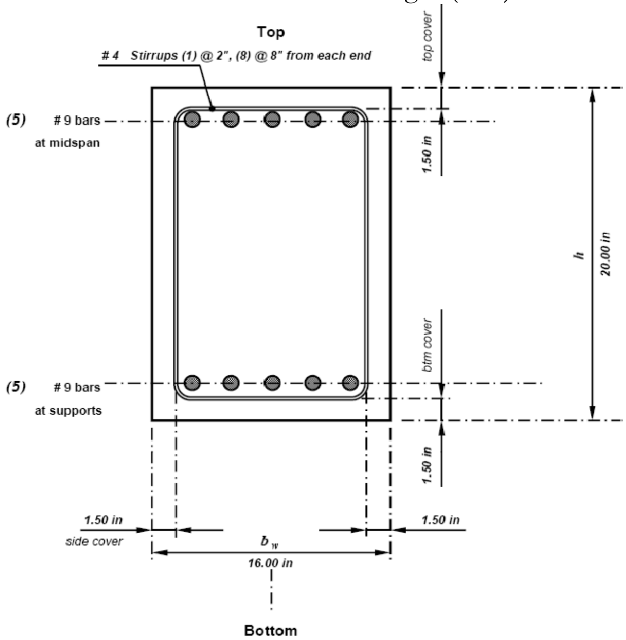


Figure 21: 20x16 Beam

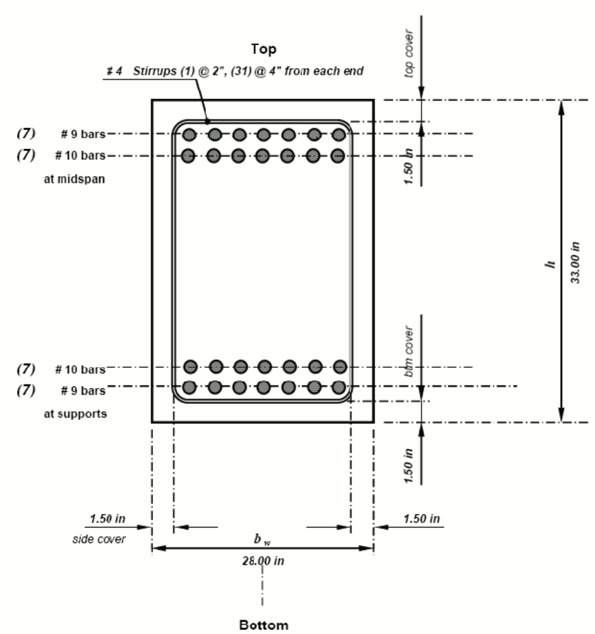


Figure 22: 33x28 Girder

APPENDIX D: TWO-WAY POST-TENSIONED CONCRETE SYSTEM

Two-Way Post-Tensioned Design

Loads:

Framing Dead Load	=	Self weight		
Superimposed Dead Load	=	25	psf	
Live Load	=	70	psf	
2 hour fire-rating				

Materials:

Concrete:	Normal weight			150	pcf	
		f'_c	=	5,000	psi	
		f'_{ci}	=	3,000	psi	
Rebar:		f_y	=	60,000	psi	
Post Tension:	Unbonded tendons					
	1/2"φ, 7-wire strands	A	=	0.153	in^2	
		f_{pu}	=	270	ksi	
	Estimated prestress losses		=	15	ksi	**ACI 18.6**
	$f_{se} = 0.7\ f_{pu}$- losses		=	174	ksi	**ACI 18.5.1**
	$Peff = A*f_{se}$		=	26.62	kips/tendon	

Determine Preliminary Slab Thickness:

Long Span:				
	L_1	=	30	ft
	L_2	=	40	ft
$h=L_{longest}/45$	h	=	10.67	in
preliminary slab thickness	h	=	11.50	in

Loading:

Framing Dead Load = self weight = t_{slab}(150pcf)	=	143.75	psf
Superimposed Dead Load	=	25	psf
Live Load	=	70	psf

Design of East West Frame (40 ft span 30ft width)

Section Properties:

$A = bh = (360\ in)t_{slab}$	=	4140	in^2
$S = bh^2/6 = (360\ in)(t_{slab})^2/6$	=	7935	in^3

Design Parameters:

Allowable stresses: Class U (ACI 18.3.3):

At time of jacking (ACI 18.4.1):

f'_{ci}	=	3,000	psi	
Compression = $0.60\ f'_{ci}$	=	1,800	psi	
Tension = $3\sqrt{f'_{ci}}$	=	164	psi	

At service loads (ACI 18.4.2(a) and 18.3.3):

f'_c	=	5,000	psi	
Compression = $0.45\ f'_c$	=	2,250	psi	
Tension = $6\sqrt{f'_c}$	=	424	psi	

Average precompression limits:

P/A	=	125	psi	min (ACI 18.12.4)
	=	300	psi	max

Target load balances:

%	=	0.6	
% w_{DL}	=	86	psf

2 Hour Fire Rating:

Restrained Slabs:	0.75	in	bottom
Unrestrained Slabs:	1.5	in	bottom
	0.75	in	top

Tendon Profile:

$a_{int} = t_{slab}$-2 x cover- d_{tendon}	=	9.50	in
a_{end} = 0.5 x (1.5 x t_{slab}-cover-0.5 x d_{tendon})-cover-0.5xd_{tendon}	=	6.3750	in

Pre-stress Force Required to Balance % of S.W.

$w_b = \%w_{DL}$	=	2.588	klf

Force needed to counteract load in end bay:

$P = w_b L^2 / 8a_{end}$	=	974.12	k

Check Precompression Allowance:

# tendons	=	36.59	use	36	tendons

Actual force for banded tendons

P_{act} = # tendons x P_{eff}	=	958.4	k

Adjust End Span Balanced Load:

w_b = 2.546 klf

Determine actual Precompression stress

P_{actual} /A = P_{actual} x 1000 / A = 231.50 psi > 125 psi min. **OK**

< 300 psi max. **OK**

Check Interior Span Force:

$P = w_b L^2 / 8a_{int}$ = 653.68 k < 958.4 **Less Force Required**

$w_b = P_{act} \times 8 \times a_{int}/l^2$ = 3.794 klf

w_b / w_{DL} = 0.880 **OK**

Effective prestress force, P_{eff} = 958.4 k

Check Slab Stresses:

Dead Load Moments:

w_{DL}	=	5.063	klf
M^-	=	810.00	kft
M^+_{ext}	=	648.00	kft
M^+_{int}	=	202.50	kft

Live Load Moments:

w_{LL}	=	2.100	klf
M^-	=	336.00	kft
M^+_{ext}	=	268.80	kft
M^+_{int}	=	84.00	kft

Total Balancing Moments:

w_b	=	3.170	klf
M^-	=	507.15	kft
M^+_{ext}	=	405.72	kft
M^+_{int}	=	126.79	kft

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

Midpsan Stresses:

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

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

Interior Span:

f_{top}	=	-346.00	<	1800	**OK**
f_{bot}	=	-117.00	<	1800	**OK**

End Span:

f_{top}	=	-597.89	<	1800	**OK**
f_{bot}	=	134.90	<	164	**OK**

Support Stresses:

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

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

f_{top}	=	226.50	>	164	**NEED REINF.**
f_{bot}	=	-689.49	<	1800	**OK**

Stage2: Stresses at Service Load (DL + LL + PT):

Midpsan Stresses:

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

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

Interior Span:

f_{top}	=	-473.03	<	1350	**OK**
f_{bot}	=	10.04	<	329	**OK**

End Span:

f_{top}	=	-1004.40	<	1350	**OK**
f_{bot}	=	541.41	>	329	**NEED REINF.**

Support Stresses:

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

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

f_{top}	=	734.63	>	329	**NEED REINF.**
f_{bot}	=	-1197.62	<	1350	**OK**

Ultimate Strength:

$M_1=Pe$	=	379.36	kft	
$M_{sec}=M_{BAL}-M_1$	=	127.79	kft	@ interior supports

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

M_u	=	1271.57	kft	@ midspan
M_u	=	1573.49	kft	@ support

Minimum Bonded Reinforcement:

Positive Moment Region:

Interior Span:	f_t	=	10.04	psi	< 141	**NO REINF. REQ.**
Exterior Span:	f_t	=	541.41	psi	> 141	**NEED REINF.**
	$y = f_t/(f_t+f_c)h$	=	4.03	in		
	$N_c = M_{DL+LL} / S^*.5^*y^*L_1$	=	1005.19	kips		
	$A_{s,min} = Nc/.5fy$	=	33.51	in^2		

Distribute reinforcement evenly across the width of the slab

$A_{s,min} = A_{s,min}/L_1$	=	1.117	in^2/ft
Use #7 @ 6" O.C. Bottom	=	1.2	in^2/ft

Negative Moment Region:

Interior Supports :

$A_{cf} = t_{slab}$ x L x12	=	5520	in^2
$A_{s,min} = 0.00075A_{cf}$	=	4.14	in^2
Use 14 #5 Top		4.34	in^2

Exterior Supports :

$A_{cf} = t_{slab}$ x L x12	=	4140	in^2
$A_{s,min} = 0.00075A_{cf}$	=	3.105	in^2
Use 11 #5 Top		3.41	in^2

Bars span minimum of 1/6 clear span each side of support
At least 4 bars in each direction

Max Bar Spacing = 12 in

Check if minimum reinforcement is sufficient for ultimate strength:

At Supports :

Interior Supports :

$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon}	=	10.50	in		
A_{ps} = 0.153 in^2 x (number of tendons)	=	5.51	in^2		
$f_{ps,\ supports} = f_{se} + 10{,}000 + (f'_c bd)/(300A_{ps})$	=	195,438	psi		
$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f'_c b)$	=	0.87	in		
$\phi Mn = \phi(Asfy + Apsfps)(d-a/2)$	=	1,009	kft	<	1,573 kft

UTL. STGTH REINF. GOVERNS

$A_{s,\ reqd}$ = 17.74 in^2

Use #7 @ 12" O.C. Bottom at end span A_s = 18.00 in^2

At Midspan:

$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon}	=	9.75	in		
A_{ps} = 0.153 in^2 x (number of tendons)	=	5.51	in^2		
$f_{ps,\ supports} = f_{se} + 10{,}000 + (f'_c bd)/(300A_{ps})$	=	194,621	psi		
$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f'_c b)$	=	2.01	in		
$\phi Mn = \phi(Asfy + Apsfps)(d-a/2)$	=	2,021	kft	>	1,272 kft

MIN. REINF. OK

Use #7 @ 6" O.C. Bottom at end spans

Design of North South Frame (30 ft span 40ft width)

Section Properties:

$A = bh = (360\text{ in})t_{slab}$	=	5520	in^2
$S = bh^2/6 = (360\text{ in})(t_{slab})^2/6$	=	10580	in^3

Design Parameters:

Allowable stresses: Class U (ACI 18.3.3):

At time of jacking (ACI 18.4.1):

f'_{ci}	=	3,000	psi
Compression = 0.60 f'_{ci}	=	1,800	psi
Tension = $3\sqrt{f'_{ci}}$	=	164	psi

At service loads (ACI 18.4.2(a) and 18.3.3):

f'_c	=	5,000	psi
Compression = 0.45 f'_c	=	2,250	psi

Tension = $6\sqrt{f'_c}$ = 424 psi

Average precompression limits:

P/A = 125 psi min (ACI 18.12.4)
= 300 psi max

Target load balances:

% = 0.6

% w_{DL} = 86 psf

2 Hour Fire Rating:

Restrained Slabs: 0.75 in bottom
Unrestrained Slabs: 1.5 in bottom
0.75 in top

Tendon Profile:

$a_{int} = t_{slab} - 2 \times cover - d_{tendon}$ = 9.50 in

$a_{end} = 0.5 \times (1.5 \times t_{slab} - cover - 0.5 \times d_{tendon}) - cover - 0.5 x d_{tendon}$ = 6.3750 in

Pre-stress Force Required to Balance % of S.W.

$w_b = \% w_{DL}$ = 3.450 klf

Force needed to counteract load in end bay:

$P = w_b L^2 / 8a_{end}$ = 730.59 k

Check Precompression Allowance:

tendons = 27.44 use 28 tendons

Actual force for banded tendons

P_{act} = # tendons x P_{eff} = 745.4 k

Adjust End Span Balanced Load:

w_b = 3.520 klf

Determine actual Precompression stress

$P_{actual} / A = P_{actual} \times 1000 / A$ = 135.04 psi > 125 psi min. **OK**
< 300 psi max. **OK**

Check Interior Span Force:

$P = w_b L^2 / 8a_{int}$ = 490.26 k < 745.4

Less Force Required

$w_b = P_{act} \times 8 \times a_{int}/l^2$	=	5.246	klf	
w_b / w_{DL}	=	0.912		**OK**

Effective prestress force, P_{eff}	**=**	**745.4**	**k**

Check Slab Stresses:

Dead Load Moments:

w_{DL}	=	6.750	klf
M^-	=	607.50	kft
M^+_{ext}	=	486.00	kft
M^+_{int}	=	151.88	kft

Live Load Moments:

w_{LL}	=	2.800	klf
M^-	=	252.00	kft
M^+_{ext}	=	201.60	kft
M^+_{int}	=	63.00	kft

Total Balancing Moments:

w_b	=	4.383	klf
M^-	=	394.45	kft
M^+_{ext}	=	315.56	kft
M^+_{int}	=	98.61	kft

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

Midpsan Stresses:

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

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

Interior Span:

f_{top}	=	-195.45	<	1800	**OK**
f_{bot}	=	-74.63	<	1800	**OK**

End Span:

-

f_{top} = 328.36 < 1800 **OK**

f_{bot} = 58.28 < 164 **OK**

Support Stresses:

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

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

f_{top} = 106.61 < 164 **OK**

-

f_{bot} = 376.68 < 1800 **OK**

Stage2: Stresses at Service Load (DL + LL + PT):

Midpsan Stresses:

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

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

Interior Span:

f_{top} = -266.91 < 1350 **OK**

f_{bot} = -3.17 < 329 **OK**

End Span:

f_{top} = -557.01 < 1350 **OK**

f_{bot} = 286.94 < 329 **OK**

Support Stresses:

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

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

f_{top} = 392.43 > 329 **NEED REINF.**

f_{bot} = -662.51 < 1350 **OK**

Ultimate Strength:

$M_1=Pe$ = 295.06 kft

$M_{sec}=M_{BAL}-M_1$ = 99.39 kft @ interior supports

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

M_u = 955.45 kft @ midspan

M_u = 1181.89 kft @ support

Minimum Bonded Reinforcement:

Positive Moment Region:

Interior Span: f_t = -3.17 psi < 141 **NO REINF. REQ.**

Exterior Span: f_t = 286.94 psi > 141 **NEED REINF.**

$y=f_t/(f_t+f_c)h$ = 3.91 in

$N_c = M_{DL+LL} / S{*}.5{*}y{*}L_1$ = 731.83 kips

$A_{s,min} = Nc/.5fy$ = 24.39 in²

Distribute reinforcement evenly across the width of the slab

$A_{s,min} = A_{s,min}/L_1$ = 0.610 in²/ft

Use #7 @ 10" O.C. Bottom = 0.72 in²/ft

Negative Moment Region:

Interior Supports :

$A_{cf} = t_{slab}$ x L x12 = 5520 in²

$A_{s,min} = 0.00075A_{cf}$ = 4.14 in²

Use 14 #5 Top 4.34 in²

Exterior Supports :

$A_{cf} = t_{slab}$ x L x12 = 5520 in²

$A_{s,min} = 0.00075A_{cf}$ = 4.140 in²

Use 14 #5 Top 4.34 in²

Bars span minimum of 1/6 clear span each side of support

At least 4 bars in each direction

Max Bar Spacing = 12 in

Check if minimum reinforcement is sufficient for ultimate strength:

At Supports:

$d_{supports} = t_{slab}$- cover- 0.5 x d_{tendon} = 10.50 in

A_{ps} = 0.153 in² x (number of tendons) = 4.28 in²

$f_{ps,\ supports} = f_{se} + 10{,}000 + (f'_c bd)/(300A_{ps})$ = 203,608 psi

$a = (A_s f_y + A_{ps} f_{ps}) / (0.85f'_c b)$ = 0.56 in

$\phi Mn = \phi(Asfy + Apsfps)(d-a/2)$ = 868 kft < 1,182 kft

UTL. STGTH REINF. GOVERNS

$A_{s,\ reqd}$ = 11.43 in²

Use #7 @ 12" O.C. Bottom at end span A_s = 12.00 in²

At Midspan:

$d_{supports} = t_{slab}$ - cover- 0.5 x d_{tendon}	=	9.75	in			
A_{ps} = 0.153 in^2 x (number of tendons)	=	4.28	in^2			
$f_{ps, supports} = f_{se}$ + 10,000 + $(f'_c bd)/(300A_{ps})$	=	202,207	psi			
$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f'_c b)$	=	1.14	in			
$\phi Mn = \phi(Asfy + Apsfps)(d-a/2)$	=	1,604	kft	>	955	kft
			MIN. REINF. OK			

Use #7 @ 10" O.C. Bottom at end spans

Punching Shear:

Framing Dead Load = self weight = t_{slab}(150pcf)	=	143.75	psf			
Superimposed Dead Load	=	25	psf			
Live Load	=	70	psf			
w_u=1.2DL+1.6LL	=	314.5	psf			
Area=A	=	1193.75	ft^2			
$V_u=w_u A$	=	375.43	k			
$d_{average}$	=	10.125	in			
b_o	=	160.5	in			
Vc=min of: $4\sqrt{f'_c} b_o d$	=	459.64	k			
$(2+4/\beta)\sqrt{f'_c} b_o d$	=	689.46	k			
$(\alpha d/b_o+2)\sqrt{f'_c} b_o d$	=	519.78	k			
V_c	=	459.64	k			
ϕV_c	=	344.73	k	<	375.43 k	

Size Drop Panel:

V_u	=	$\phi 4(f'_c)^{1/2} b_o d$			
375.43	=	$0.75 x 4 x \sqrt{f'_c}(4(30+d))d$			
d	=	10.84	in		
ϕV_c	=	375.65	k	>	375.43 k **OK**
	Need	11.84	in	thick drop panel	
	use	12	in		

Beam Shear:

At Panel:

Area=A	=	600	ft^2		
$V_u=w_uA$	=	188.7	k		
b_w	=	168	in	assumed 14 ft drop panel	
d	=	11	in		
$\phi V_c = 0.75x2\sqrt{f'_c}b_wd$	=	196.01	k	>	188.70 k **OK**

At Slab:

Area=A	=	390	ft^2		
$V_u=w_uA$	=	122.66	k		
b_w	=	360	in		
d	=	10.84	in		
$\phi V_c = 0.75x2\sqrt{f'_c}b_wd$	=	413.91	k	>	122.66k **OK**

System Weight:

Item	**#/bay**	**Total**	
Slab	1	172500	lbs
Drop Panel	1	1225	lbs
Self weight (PSF)=		144.77	PSF

Provide (36) 1/2" dia. 270 ksi 7-wire strand in each direction & each bay

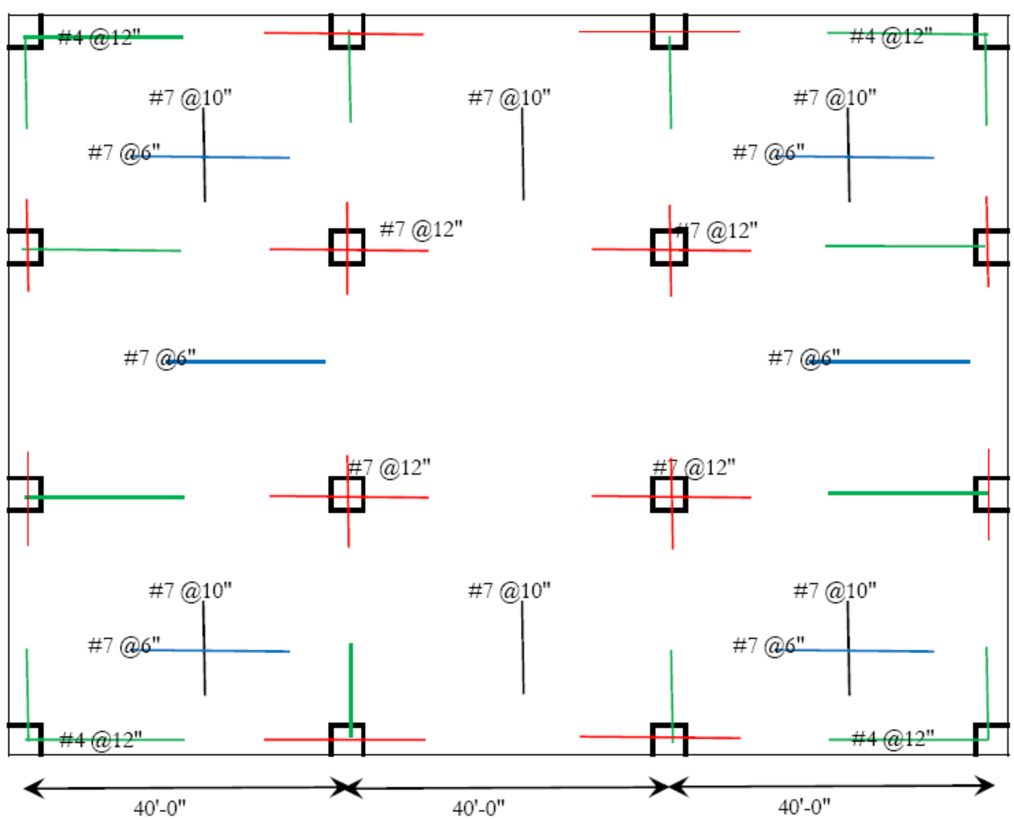


Figure 23: Post-Tensioned Slab Design