The New York Times Building
New York, NY

IPD/BIM Thesis
Technical Report #2

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Structural Option

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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:
    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:

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_s (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.

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

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.

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.

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.
**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.

**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)
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.

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.

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 $-10^\circ F$ to $130^\circ F$. 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 $L/300$.

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](image)

**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 used 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.

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

<table>
<thead>
<tr>
<th></th>
<th>Existing Composite Steel</th>
<th>LW Composite Steel</th>
<th>One-Way Long Beams</th>
<th>One-Way Short Beams</th>
<th>Two-Way Post Tensioned Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Architectural Impact:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Framing impact</td>
<td>N/A</td>
<td>Minor</td>
<td>Major</td>
<td>Major</td>
<td>Yes</td>
</tr>
<tr>
<td>Slab Depth (in)</td>
<td>5.50</td>
<td>5.00</td>
<td>4.50</td>
<td>4.50</td>
<td>11.50</td>
</tr>
<tr>
<td>Total Depth (in)</td>
<td>23.40</td>
<td>22.50</td>
<td>28.00</td>
<td>33.00</td>
<td>11.50</td>
</tr>
<tr>
<td>Column sizes (in)</td>
<td>30x30</td>
<td>30x30</td>
<td>33x33</td>
<td>33x33</td>
<td>33x33</td>
</tr>
<tr>
<td>MEP System impact</td>
<td>N/A</td>
<td>Minor</td>
<td>Major</td>
<td>Major</td>
<td>Minor-Medium</td>
</tr>
</tbody>
</table>

| **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                         |

*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.
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:

<table>
<thead>
<tr>
<th>(N=9) NORMAL WEIGHT CONCRETE (145 PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Deck</strong></td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>5'</td>
</tr>
<tr>
<td>3'</td>
</tr>
<tr>
<td>2'</td>
</tr>
<tr>
<td>1'</td>
</tr>
</tbody>
</table>

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

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

**Material Properties:**
- Concrete: $f'_c = 4$ ksi
- Beam: $F_y = 50$ ksi
  - $F_u = 65$ ksi

**Loads:**
- Dead Loads:
  - Slab: 0.053 ksf
  - Beam Weight: 0.004 ksf
  - MEP/Ceiling: 0.027 ksf
- Live Loads:
  - Non-Reduced: 0.070 ksf

**Design Calculations:**
- 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

\[
w_u = 1.2D + 1.6L = 2.122 \text{ klf}
\]
\[
V_u = w_u l/2 = 42.440 \text{ k}
\]
\[
M_u = w_u l^2/8 = 424.400 \text{ ftk}
\]
b_{eff} = \min \text{ of } (\text{span}/8, 1/2 \text{ distance to adj. bm, dist. To edge of slab}) = 120.000 \text{ in}

Assume a = 1 \text{ in}
Y_2 = t_{slab} - a/2 = 5.000 \text{ in} \quad \text{Table 3-19}

**Check \( I_{req} \):**
\[
\Delta = l/240 + \text{camber} = 3.500 \text{ in}
\]
\[
I_{req} = 5w_{CDL}l^4/(384E\Delta) = 320.631 \text{ in}^4 < 510 \text{ in}^4 \quad \text{OK \quad 20}
\]

**Check member strength as un-shored:**
\[
w_{u}(\text{unshored}) = 1.2D + 1.6L = 0.998 \text{ klf} \quad \text{Table 3-19}
\]
\[
M_{u}(\text{unshored}) = w_{u}l^2/8 = 199.600 \text{ ftk} < 249 \text{ ftk} \quad \text{OK \quad 19}
\]
PNA location = 5
\[
\Sigma Q_{n} = 260 \text{ k} \quad \text{Table 3-19}
\]

**Check member strength:**
\[
\phi M_{n} = 435 \text{ ftk} > 424.400 \text{ ftk} \quad \text{OK \quad 19}
\]
\[
\phi V_{n} = 159 \text{ k} > 42.440 \text{ k} \quad \text{OK}
\]

**Check a:**
a = \Sigma Q_{n}/0.85f'c_{beff} = 0.637 \text{ in} < 1 \text{ in} \quad \text{OK}

**Check \( \Delta_{LL} \):**
\[
\Delta_{LL} = l/360 = 1.333 \text{ in} \quad I_{LB} = 1170 \text{ in}^4 \quad \text{Table 3-20}
\]
\[
\Delta_{LL} = 5w_{u}l^4/(384E_{ILB}) = 1.188 \text{ in} < 1.333 \text{ in} \quad \text{OK}
\]

**Check studs:**
\[
Q_{n} = 17.2 \text{ kips/stud} \quad \text{Table 3-21}
\]
\[
\# \text{ of studs} = \Sigma Q_{n}/Q_{n} = 15.116 \text{ use 16 studs/side}
\]
\[
\text{Total studs} = 32
\]

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:

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>$f'_c$</td>
<td>4 ksi</td>
</tr>
<tr>
<td>Beam</td>
<td>$f_y$</td>
<td>50 ksi</td>
</tr>
<tr>
<td></td>
<td>$f_u$</td>
<td>65 ksi</td>
</tr>
</tbody>
</table>

#### Loads:

**Dead Loads:**
- Slab: 0.053 ksf
- Beam Weight: 0.002 ksf
- MEP/Ceiling: 0.027 ksf

**Live Loads:**
- Non-Reduced: 0.070 ksf

**Load Calculations:**

- 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

\[
w_u = 1.2D + 1.6L = 2.103 \text{ klf}
\]
\[
V_u = w_u l/2 = 5.607 \text{ k}
\]
\[
M_u = w_u l^2/8 = 7.477 \text{ ftk}
\]

\[
b_{eff} = 16.000 \text{ in}
\]

Assume \( a = 1.5 \text{ in} \)

\[
Y_2 = t_{slab} - a/2 = 4.750 \text{ in}
\]

**Check \( I_{req} \):**

\[
\Delta = l/240 + \text{camber} = 0.267 \text{ in}
\]

\[
I_{req} = 5w_{CDL} l^4/(384E\Delta) = 1.292 \text{ in}^4 < 130 \text{ in}^4 \text{ OK}
\]

**Check member strength as un-shored:**

\[
w_{u(unshored)} = 1.2D + 1.6L = 0.979 \text{ klf}
\]
\[
M_{u(unshored)} = w_u l^2/8 = 3.480 \text{ ftk} < 92.6 \text{ ftk} \text{ OK}
\]

PNA location: 7

\[
\Sigma Q_a = 69.7 \text{ k}
\]
Check member strength:

\[
\phi M_n = 141.5 \text{ ftk} > 7.477 \text{ ftk \ OK Table 3-19}
\]

\[
\phi V_n = 85.7 \text{ k} > 5.607 \text{ k \ OK}
\]

Check a:

\[
a = \Sigma Q_n / 0.85f'c_{beff} = 1.281 \text{ in} < 1.5 \text{ in \ OK}
\]

Check \(\Delta_{LL}\):

\[
\Delta_{ul} = l/360 = 0.178 \text{ in} \quad I_{LB} = 261 \text{ in}^4 \text{ Table 3-20}
\]

\[
\Delta_{ul} = 5w_{ul}l^4/(384EIL_B) = 0.0017 \text{ in} < 0.178 \text{ in \ OK}
\]

Check studs:

\[
Q_n = 17.2 \text{ kips/stud Table 3-21}
\]

\[
\# \text{ of studs} = \Sigma Q_n / Q_n = 4.052 \use 5 \text{ studs/side}
\]

\[
\text{Total studs} = 10
\]

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 ($P_{d}$): 15,867 k
Const. dead load (unshored): 12,764 k
Const. dead load (unshored): 0.040 klf
Const. live load (unshored): 4,533 k

\[ P_{u} = 1.2D + 1.6L = 48.047 \text{ k} \]
\[ w_{u} = 1.2D + 1.6L = 0.048 \text{ klf} \]
\[ V_{u} = w_{u}l/2 + P_{u} = 48.767 \text{ k} \]
\[ M_{u} = w_{u}l^{2}/8 + P_{u}l/3 = 485.875 \text{ ftk} \]

Assume $a = 1.5$ in

\[ Y_{2} = t_{slab} - a/2 = 4.75 \text{ in} \]

Check $I_{req}$:

\[ \Delta = l/240 + \text{camber} = 2.250 \text{ in} \]
\[ I_{req} = 5w_{DL}l^{4}/(384E\Delta) + P_{DL}l^{3}/(28E\Delta) = 337.126 \text{ in}^{4} < 612 \text{ in}^{4} \text{ OK} \]
Check member strength as un-shored:

\[ P_{\text{ul}(\text{unshored})} = 1.2D + 1.6L = 22.570 \text{ k} \]
\[ w_{\text{ul}(\text{unshored})} = 1.2D + 1.6L = 0.048 \text{ klf} \]
\[ M_{\text{ul}(\text{unshored})} = w_{\text{ul}} l^2 / 8 + P_{\text{ul}} l / 3 = 231.101 \text{ ftk} \]
\[ \Sigma Q_n = 351 \text{ k} \]

Check member strength:

\[ \phi M_n = 516.5 \text{ ftk} > 485.875 \text{ ftk OK} \]
\[ \phi V_n = 169 \text{ k} > 48.767 \text{ k OK} \]

Check a:

\[ a = \Sigma Q_n / 0.85 f'c_{b\text{eff}} = 1.147 \text{ in} < 1.5 \text{ in OK} \]

Check \( \Delta_{LL} \):

\[ \Delta_{LL} = l / 360 = 1.000 \text{ in} \]
\[ I_{LB} = 1440 \text{ in}^4 \]
\[ \Delta_{LL} = P_{LL} l^3 / (28E_{ILB}) = 0.633 \text{ in} < 1 \text{ in OK} \]

Check studs:

\[ Q_n = 17.200 \text{ kips/stud} \]
\[ \# \text{ of studs} = \Sigma Q_n / Q_n = 20.407 \text{ use 21 studs/side} \]
\[ \text{Total studs} = 42 \]

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:

<table>
<thead>
<tr>
<th>Item</th>
<th>Number/bay</th>
<th>Weight (lbs)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>W18x35</td>
<td>3</td>
<td>1400</td>
<td>4200</td>
</tr>
<tr>
<td>W18x40</td>
<td>1</td>
<td>1200</td>
<td>1200</td>
</tr>
<tr>
<td>Composite System + Deck</td>
<td>1</td>
<td>63600</td>
<td>63600</td>
</tr>
<tr>
<td>Self weight (PSF)</td>
<td></td>
<td></td>
<td>57.50</td>
</tr>
</tbody>
</table>
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)

<table>
<thead>
<tr>
<th>Total Depth</th>
<th>SDB Type</th>
<th>SDB Max, Unshored</th>
<th>Clear Span</th>
<th>SDB Type</th>
<th>SDB Max, Unshored</th>
<th>Clear Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 in</td>
<td>Vulcraft 12-12</td>
<td>10</td>
<td>12</td>
<td>12</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>2 in</td>
<td>Vulcraft 12-12</td>
<td>10</td>
<td>12</td>
<td>12</td>
<td>10</td>
<td>12</td>
</tr>
</tbody>
</table>

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:
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: \( f'_{c} = 4 \text{ ksi} \)
- Beam: \( f_{y} = 50 \text{ ksi} \)
- \( f_{u} = 65 \text{ ksi} \)

- 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

\[
\begin{align*}
w_{u} &= 1.2D + 1.6L = 1.432 \text{ klf} \\
V_{u} &= w_{u}l/2 = 28.633 \text{ k} \\
M_{u} &= w_{u}l^{2}/8 = 286.332 \text{ ftk}
\end{align*}
\]
\[ b_{\text{eff}} = \text{min of } (\text{span}/8, 1/2 \text{ distance to adj. bm, dist. To edge of slab}) = 90.000 \text{ in} \]

Assume \( a = 1 \text{ in} \)

\[ Y_2 = t_{\text{slab}} - a/2 = 5.000 \text{ in} \]

**Check \( I_{\text{req}} \):**

\[ \Delta = l/240 + \text{camber} = 3.500 \text{ in} \]

\[ I_{\text{req}} = 5w_{\text{CDL}}l^4/(384E\Delta) = 164.884 \text{ in}^4 < 301 \text{ in}^4 \text{ OK} \]

**Check member strength as un-shored:**

\[ w_{u(\text{unshored})} = 1.2D + 1.6L = 0.589 \text{ klf} \]

\[ M_{u(\text{unshored})} = w_{u}l^2/8 = 117.732 \text{ ftk} < 166 \text{ ftk} \text{ OK} \]

PNA location = 4

\[ \Sigma Q_n = 242 \text{ k} \]

**Check member strength:**

\[ \phi M_n = 315 \text{ ftk} > 286.332 \text{ ftk} \text{ OK} \]

\[ \phi V_n = 106 \text{ k} > 28.633 \text{ k} \text{ OK} \]

**Check \( a \):**

\[ a = \Sigma Q_n/0.85f'c_{\text{eff}} = 0.791 \text{ in} < 1 \text{ in} \text{ OK} \]

**Check \( \Delta_{LL} \):**

\[ \Delta_{LL} = l/360 = 1.333 \text{ in} \]

\[ I_{LB} = 791 \text{ in}^4 \text{ Table 3-20} \]

\[ \Delta_{LL} = 5w_{u}l^4/(384E_{ILb}) = 1.318 \text{ in} < 1.333 \text{ in} \text{ OK} \]

**Check studs:**

\[ Q_n = 17.2 \text{ kips/stud} \text{ Table 3-21} \]

\[ \# \text{ of studs} = \Sigma Q_n/Q_n = 14.070 \text{ use 15 studs/side} \]

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

\( f'_c = 4 \text{ ksi} \)

Beam

\( F_y = 50 \text{ ksi} \)

\( F_u = 65 \text{ ksi} \)

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

\[ w_u = 1.2D + 1.6L = 1.900 \text{ klf} \]

\[ V_u = w_u l/2 = 5.068 \text{ k} \]

\[ M_u = w_u l^2/8 = 6.757 \text{ ftk} \]

\( b_{eff} = 16.000 \text{ in} \)

Assume \( a = 1.5 \text{ in} \)

\[ Y_2 = t_{slab} - a/2 = 4.750 \text{ in} \]

\[ \Delta = l/240 + \text{camber} = 0.267 \text{ in} \]

\[ I_{req} = 5w_{CDL} l^4/(384EI) = 0.895 \text{ in}^4 \] < 130 in\(^4\) OK Table 3-20

Check member strength as un-shored:

\[ w_{u(unshored)} = 1.2D + 1.6L = 0.776 \text{ klf} \]

\[ M_{u(unshored)} = w_u l^2/8 = 2.761 \text{ ftk} \]

\( \Sigma Q_n = 69.7 \text{ k} \)

PNA location = 7

\[ 92.600 \text{ ftk} \]

OK Table 3-19
Check member strength:

\[ \phi M_n = 141.5 \text{ ftk} \quad > \quad 6.757 \text{ ftk} \quad \text{OK} \quad \text{Table 3-19} \]

\[ \phi V_n = 85.7 \text{ k} \quad > \quad 5.068 \text{ k} \quad \text{OK} \]

Check a:

\[ a = \sum Q_n / 0.85 f'c_{beff} = 1.281 \text{ in} \quad < \quad 1.5 \text{ in} \quad \text{OK} \]

Check \( \Delta_{LL} \):

\[ \Delta_{UL} = l / 360 = 0.178 \text{ in} \quad I_{LB} = 261 \text{ in}^4 \quad \text{Table 3-20} \]

\[ \Delta_{UL} = 5wll^4 / (384EIL_b) = 0.0017 \text{ in} \quad < \quad 0.178 \text{ in} \quad \text{OK} \]

Check studs:

\[ Q_n = 17.2 \text{ kips/stud} \quad \text{Table 3-21} \]

\[ \# \text{ of studs} = \sum Q_n / Q_n = 4.052 \text{ use} \quad 5 \text{ studs/side} \]

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 ($P_d$): 11.595 k
Total dead load ($w_d$): 0.035 klf
Total live load($P_0$): 12.367 k

Const. dead load (unshored): 6.825 k
Const. dead load (unshored): 0.035 klf
Const. live load (unshored): 0.533 k

- $P_d = 1.2D + 1.6L = 33.701$ k
- $w_d = 1.2D + 1.6L = 0.042$ klf
- $V_d = w_d l/2 + P_d = 34.331$ k
- $M_d = w_d l^2/8 + P_d l/3 = 341.736$ ftk
- $b_{eff} = 90.000$ in

Assume $a = 1$ in

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

**Check $I_{req}$:**

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

Table 3-19

Table 3-20
Check member strength as un-shored:

\[
P_u(\text{unshored}) = 1.2D + 1.6L = 9.044 \text{ k}
\]

\[
w_u(\text{unshored}) = 1.2D + 1.6L = 0.042 \text{ klf}
\]

\[
M_u(\text{unshored}) = w_u l^2/8 + P_u l/3 = 95.163 \text{ ftk} < 249 \text{ ftk} \text{ OK}
\]

\[
PNA \text{ location} = 7
\]

\[
\Sigma Q_n = 129 \text{ k}
\]

Check member strength:

\[
\phi M_n = 363 \text{ ftk} > 341.736 \text{ ftk} \text{ OK}
\]

\[
\phi V_n = 159 \text{ k} > 34.331 \text{ k} \text{ OK}
\]

Check \(a\):

\[a = \Sigma Q_n/0.85f'c_{beff} = 0.422 \text{ in} < 1 \text{ in} \text{ OK}\]

Check \(\Delta L_L\):

\[
\Delta_L = l/360 = 1.000 \text{ in}
\]

\[
I_{LB} = 906 \text{ in}^4 \text{ Table 3-20}
\]

\[
\Delta_L = P_u l^3/(28E I_{LB}) = 0.784 \text{ in} < 1 \text{ in} \text{ OK}
\]

Check studs:

\[Q_n = 17.200 \text{ kips/stud} \text{ Table 3-21}\]

\[\# \text{ of studs} = \Sigma Q_n/Q_n = 7.500 \text{ use} 8 \text{ studs/side}\]

\[\text{Total studs} = 16\]

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

System Weight:

<table>
<thead>
<tr>
<th>Item</th>
<th>Number/bay</th>
<th>Weight (lbs)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>W16x26</td>
<td>4</td>
<td>1040</td>
<td>4160</td>
</tr>
<tr>
<td>W18x36</td>
<td>1</td>
<td>1080</td>
<td>1080</td>
</tr>
<tr>
<td>Composite System + Deck</td>
<td>1</td>
<td>43200</td>
<td>43200</td>
</tr>
<tr>
<td>Self weight (PSF)</td>
<td></td>
<td></td>
<td>40.37</td>
</tr>
</tbody>
</table>
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 ($l_0$): 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

Assumed Col 33 x 33

Slab Design:

- $w_{D, superimposed} = 0.025$ ksf
- $w_{D, slab\ contribution} = h x 150/12 = 0.056$ 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):
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New York, NY
Technical Report #2

M = \frac{wuh^2}{11} = -22.85 \text{ kIN}
M^+ = \frac{wuh^2}{16} = 15.71 \text{ kIN}
V_u = \frac{wuh}{2} = 1.05 \text{ k}

Assume # 4 bars for stirrups
Assume # 4 bars for flexure
Clear cover = 0.75 \text{ in}

d = h - cover - stirrup - 0.5d_{flexure} = 3.00 \text{ in}

A_s = 0.4 \text{ in}^2

Check A_{s,min}:
\begin{align*}
3\sqrt{f'_c bd / f_y} &= 0.114 \text{ in}^2 \\
200bd / f_y &= 0.120 \text{ in}^2 \\
A_{s,min} &= 0.120 \text{ in}^2 \quad < \quad 0.4 \quad \text{OK}
\end{align*}

Check A_{s,max}:
\begin{align*}
\rho_{max} &= 0.0206 \\
A_{s,max} = \rho bd &= 0.7431 \text{ in}^2 \quad > \quad 0.4 \quad \text{OK}
\end{align*}

Check A_{s,temp}:
\begin{align*}
A_{s,temp} = 0.0018bh &= 0.0972 \text{ in}^2 \quad < \quad 0.2 \quad \text{OK}
\end{align*}

Use A_{s,temp} = 0.2 \text{ in}^2 \quad @ \quad 18 \text{ in}

Determine M_n:
\begin{align*}
a &= A_s f_y / .85f'_c b &= 0.588 \text{ in} \\
\beta &= 0.850 \\
c &= a / \beta &= 0.692 \text{ in} \\
\epsilon_s &= 0.003(d-c) / c &= 0.0100 \text{ in/in} \\
\epsilon_y &= 60 / 29000 &= 0.0021 \text{ in/in} \quad < \quad 0.0100 \quad \text{OK} \\
\phi &= 0.9 \\
\phi M_n = \phi A_s f_y (d-a)/2 &= 58.45 \text{ kIN} \quad > \quad 22.85 \quad \text{OK}
\end{align*}

Maximum Number of Bars (Table A.7)
Max numbers of bars = 4 \quad > \quad 2 \quad \text{OK}

Minimum Number of Bars (Table A.8), for Crack Control
Min numbers of bars = 2 \quad < \quad 2 \quad \text{OK}
### Determine shear strength of beam without stirrups:

\[
\lambda = 1
\]

\[
V_c = 2 \lambda \varphi f'_c b_w d = 4.55 \text{k} > 1.05 \text{NO SHEAR REINF.}
\]

\[
\phi = 0.75
\]

\[
\phi V_n = 0.5 \phi V_c = 1.71 \text{k}
\]

### Beam Design:

- \( w_{D, \text{superimposed}} = 0.150 \text{ksf} \)
- \( w_{D, \text{slab contribution}} = hx150/12 = 0.056 \text{ksf} \)
- \( w_{D, \text{beam contribution}} = (h-t_{\text{lab}})xbx150/144 = 0.406 \text{kfl} \)
- \( w_{L,=} = 0.070 \text{ksf} \)

Analysis Trib width 10 ft = 120 in

\( w_u = 1.2D+1.6L = 4.083 \text{kfl} \)

### Moments (assume continuous interior span):

\[
M_- = w_u l_n^2 /11 = -6431.05 \text{kin}
\]

\[
M_+ = w_u l_n^2 /16 = 4421.35 \text{kin}
\]

\[
V_u = w_u l_n /2 = 77.57 \text{k}
\]

- Assume # 4 bars for stirrups
- Assume # 9 bars for flexure
- clear cover = 1.5 in
- \( b_s = 20 \text{in} \)
- \( d = h - \text{cover-stirrup-0.5d}_{\text{flexure}} = 21.44 \text{in} \)
- Number of bars = 7 9 & 0 0
- \( A_s = 7.00 \text{in}^2 \)

### Check If T-beam behavior occurs:

\[
h_f = 4.5 \text{in}
\]

\[
b_s + 16 h_f = 92 \text{in}
\]

\[
b_s + 2(0.5\text{clear distance}) = 100 \text{in}
\]

\[
.25\text{span length} = 120 \text{in}
\]

\[
b_{\text{eff, int}} = 92 \text{in}
\]

\[
M_{u,T\text{-Beam}} = \phi 0.85 f'_c bh_f(d-h_f/2) = 24,307 \text{kin} > 6,431 \text{NO T-BEAM}
\]

### Determine \( M_{u1} \) for \( \rho = \rho_{\text{max}} \):

\[
\rho_{\text{max}} = 0.85(f'_c/ f_c)\phi(0.003/(0.003+0.005)) = 0.0181
\]

\[
A_{s1} = \rho_{\text{max}} b d = 7.744 \text{in}^2
\]
a = \frac{A_s f_y}{0.85 f_{cy}} b = 6.833 \text{ in}^2

M_{sl} = 0.85 f_{cy} ab(d-a/2) = 8,374 \text{ kin} > 6,431 \text{ REINFORCED}

**Check A_{s,min}**:

\[ 3\sqrt{f_{cy} bd/f_y} = 3.156 \text{ in}^2 \]
\[ 200bd/f_y = 1.429 \text{ in}^2 \]
\[ A_{s,min} = 1.429 \text{ in}^2 < 7.00 \text{ OK} \]

**Check A_{s,max}**:

\[ \rho_{\text{max}} = 0.0206 \]
\[ A_{s,max} = \rho bd = 8.8506 \text{ in}^2 > 7.00 \text{ OK} \]

**Determine M_n**:

\[ a = \frac{A_s f_y}{0.85 f_{cy}} b = 6.176 \text{ in} \]
\[ \beta = 0.850 \]
\[ c = a/\beta = 7.266 \text{ in} \]
\[ \varepsilon_s = 0.003(d-c)/c = 0.0059 \text{ in/in} \]
\[ \varepsilon_y = 60/29000 = 0.0021 \text{ in/in} < 0.0059 \text{ OK} \]
\[ \phi = 0.9 \]
\[ \phi M_n = \phi A_s f_y (d-a/2) = 6,936 \text{ kin} > 6,431 \text{ OK} \]

**Maximum Number of Bars**:

\[ b_{\text{min}} = 2c + 2d_{tr} + nd_b + (n-1)4/3 = 19.88 < 20 \text{ OK} \]

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

Min numbers of bars = 3 < 7 \text{ OK}

**Determine shear strength of beam without stirrups**:

\[ \lambda = 1 \]
\[ V_c = 2 \lambda \sqrt{f_{cy} b w} d = 54.23 \text{ k} < 77.57 \text{ SHEAR REINF.} \]
\[ \phi = 0.75 \]
\[ \phi V_n = 0.5 \phi V_c = 20.34 \text{ k} \]

**Determine shear strength required by shear reinforcing**:

\[ V_u @ d = 70.27 \text{ k} \]
\[ V_s = V_u / \phi V_c = 39.47 \text{ k} \]
\[ V_s \leq 8 \sqrt{f_{cy} b w} d = 216.93 \text{ k} \text{ OK} \]

No reinforcing required at:

\[ 68.59 \text{ in} \]
**Determine maximum spacing of shear reinforcing:**

\[ V_s \leq 4 \sqrt{f'_c} \cdot b_w \cdot d = 108.47 \text{ k} \quad \text{OK} \]

\[ s = \frac{d}{2} \quad 10.72 \text{ in} \]

\[ s = 24 \quad 24 \text{ in} \]

\[ s_{\text{max}} = 10.72 \text{ in} \quad \text{use} \quad 10 \text{ in} \]

**Determine minimum shear reinforcement:**

\[ A_v = 0.75 \sqrt{f'_c} \cdot b_w \cdot s/f_y = 0.158 \text{ in}^2 \]

\[ A_v = 50 \cdot b_w \cdot s/f_y = 0.167 \text{ in}^2 \]

\[ A_{v,\text{min}} = 0.167 \text{ in}^2 \quad \text{use} \quad 4 \]

\[ A_{v,\text{used}} = 0.4 \text{ in}^3 \]

**Design Shear Reinforcement:**

\[ s = A_v f_y d/V_s = 13.04 \text{ in} \]

Use (2) # 4 stirrups: 1 @ 2", 7 @ 10 in each end

**Girder Design:**

- \( w_D, \text{superimposed} = 0.150 \text{ ksf} \)
- \( w_D, \text{slab contribution} = \frac{h \times 150}{12} = 0.056 \text{ ksf} \)
- \( w_D, \text{beam contribution} = \frac{(h-t_{\text{slab}}) \times 150}{144 \times 10 \text{ ft}} = 0.041 \text{ ksf} \)
- \( w_D, \text{girder contribution} = \frac{(h-t_{\text{slab}}) \times 150}{144} = 0.588 \text{ klf} \)
- \( w_L = 0.070 \text{ ksf} \)

- Analysis Trib width 30 ft = 360 in

\( w_u = 1.2D + 1.6L = 12.953 \text{ klf} \)

**Moments (assume continuous interior span):**

\( M^- = w_u l_a^2/11 = -10492.41 \text{ kin} \)

\( M^+ = w_u l_a^2/16 = 7213.53 \text{ kin} \)

\( V_u = w_u l_a/2 = 176.48 \text{ k} \)

Assume # 4 bars for stirrups

Assume # 9 bars & 10 bars for flexure

Clear cover = 1.5 in

\( b_n = 24 \text{ in} \)

\( d_c = \text{h-cover-stirrup-0.5d_{flexure}} = 25.38 \text{ in} \)

\( d = \text{h-cover-stirrup-1.5d_{flexure}} = 24.13 \text{ in} \)
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---

d' = cover + stirrup + 0.5d_{flexure} = 2.56 in
Number of bars = 5 9 & 5 10
A_s = 11.35 in²

Check If T-beam behavior occurs:

β = 0.85f'_c(b - bw) hf(hf - 2) = 27,989 kin > 10,492 NO T-BEAM

Asf = 0.85f'_c(b - bw) hf / fy = 16.83 in²
Mnf = 0.85f'_c(b - bw) hf(hf - 2) = 23,352 kin
Mnw = Mu / φ - Mn = (11,693) kin
φMnw = (10,278) kin

Determine Mn1 for ρ = ρ_{max}φ and compare to Mnw:

ρ_{max} = 0.85f'_c / fy = (0.003/(0.003 + 0.005)) = 0.0181
A_s1 = ρ_{max}bd = 10.458 in²
a = A_s1f_y / 0.85f'_c = 7.690 in²
β = 0.85
β = a / β = 9.047 in
M_{n1} = 0.85f'_cab(d - a / 2) = 13,510 kin > (11,693) SINGLY REINFORCED

Determine Mn2:

Mn2 = Mnw - Mn1 = (25,203) kin

Determine As2 assuming fs = fy:

As2 = Mn2 / fy(d - d') = -19.481 in²

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 in²

Required Steel:

As = Asf + Asw = 8 in²
A's = A'sw = (19) in²
Check $A_{s\text{,min}}$:

\[
\begin{align*}
3\sqrt{f'_{c}bd/f_{y}} &= 1.831 \text{ in}^2 \\
200bd/f_{y} &= 1.930 \text{ in}^2 \\
A_{s\text{,min}} &= 1.930 \text{ in}^2 < 11.35 \text{ OK}
\end{align*}
\]

Check $A_{s\text{,max}}$:

\[
\begin{align*}
\rho_{\text{max}} &= 0.0206 \\
A_{s\text{,max}} &= \rho bd = 11.9522 \text{ in}^2 > 11.35 \text{ OK}
\end{align*}
\]

Determine $M_n$:

\[
\begin{align*}
a &= A_{s}f_{y}/.85f'_{c}b &= 8.346 \text{ in} \\
\beta &= 0.850 \\
c &= a/\beta &= 9.818 \text{ in} \\
\varepsilon_{s} &= 0.003(d-c)/c &= 0.0048 \text{ in/in} \\
\varepsilon_{y} &= 60/29000 &= 0.0021 \text{ in/in} < 0.0048 \text{ OK} \\
\phi &= 0.88 \\
\phi M_{n} &= \phi A_{s}f_{y}(d-a/2) &= 11,943 \text{ kin} > 10,492 \text{ OK}
\end{align*}
\]

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:

\[
\begin{align*}
\lambda &= 1 \\
V_{c} &= 2\lambda \sqrt{f'_{c} b_{w} d} &= 73.24 \text{ k} < 176.48 \text{ SHEAR REINF.} \\
\phi &= 0.75 \\
\phi V_{n} &= 0.5\phi V_{c} &= 27.46 \text{ k}
\end{align*}
\]

Determine shear strength required by shear reinforcing:

\[
\begin{align*}
V_{u} @ d &= 150.44 \text{ k} \\
V_{s} &= V_{u}/\phi V_{c} &= 127.35 \text{ k} \\
V_{s} \leq 8 \sqrt{f'_{c} b_{w} d} &= 292.95 \text{ k} < 8 \sqrt{f'_{c} b_{w} d} \text{ OK}
\end{align*}
\]

No reinforcing required at: 95.65 in
Determine maximum spacing of shear reinforcing:

\[ V_s \leq 4 \sqrt{f'_c b_w d} = 146.48 \text{ k} \quad \text{OK} \]

\[ s = \frac{d}{2} = 12.06 \text{ in} \]
\[ s = 24 \text{ in} \]
\[ s_{\text{max}} = 12.06 \text{ in} \quad \text{use} \quad 12 \text{ in} \]

Determine minimum shear reinforcement:

\[ A_v = 0.75 \sqrt{f'_c b_w} \frac{s}{f_y} = 0.228 \text{ in}^2 \]
\[ A_v = 50 b_w \frac{s}{f_y} = 0.240 \text{ in}^2 \]
\[ A_{v,\text{min}} = 0.240 \text{ in}^2 \quad \text{use} \quad 4 \]
\[ A_{v,\text{used}} = 0.40 \text{ in}^3 \]

Design Shear Reinforcement:

\[ s = A_v f_y d/V_s = 4.55 \text{ in} \]

Use (2) # 4 stirrups: 1 @ 2", 22 @ 4 in each end

System Weight:

<table>
<thead>
<tr>
<th>Item</th>
<th>#/bay</th>
<th>Span (ft)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>3</td>
<td>38.00</td>
<td>48750 LBS</td>
</tr>
<tr>
<td>Girder</td>
<td>1</td>
<td>27.25</td>
<td>17625 LBS</td>
</tr>
<tr>
<td>Slab</td>
<td>1</td>
<td>67500</td>
<td>LBS</td>
</tr>
</tbody>
</table>

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  \( f'_{c} = 4000 \text{ psi} \)
Concrete in beams  \( f'_{c} = 4000 \text{ psi} \)
Reinforcement  \( f_{y} = 60000 \text{ psi} \)

Loads:

Superimposed Dead Loads:
- Ceiling: \( 0.005 \text{ ksf} \)
- MEP in raised floor system: \( 0.012 \text{ ksf} \)
- MEP in ceiling: \( 0.008 \text{ ksf} \)
- Total: \( 0.025 \text{ ksf} \)

Concrete self weight: \( 0.150 \text{ kcf} \)

Live Loads:
- Non-Reduced: \( 0.070 \text{ ksf} \)

Option 2:

Slab span \( (l_{0}) \): 10 ft
Beam span: 30 ft
Girder span: 40 ft

Preliminary \( h \):
- Slab: \( l/28 = 4.29 \text{ in} \) use 4.5 in
- Beam: \( l/21 = 0.00 \text{ in} \) use 18 in
- Girder: \( l/21 = 17.14 \text{ in} \) use 24 in

Assumed Col 33 x 33

Slab Design:

\( w_{D, \text{superimposed}} = 0.025 \text{ ksf} \)
\( w_{D, \text{slab contribution}} = h x 150/12 = 0.056 \text{ ksf} \)
\( w_{L} = 0.070 \text{ ksf} \)

Analysis 1 ft width, \( b = 12 \text{ in} \)
\( w_{u} = 1.2D + 1.6L = 0.210 \text{ klf} \)

Moments (assume continuous interior span):

\( M = w_{u} b_{n}^{2}/11 = -22.85 \text{ kln} \)
M+ = wuLa^2/16 = 15.71 k-in
V_u = wuLa/2 = 1.05 k

Assume # 4 bars for stirrups
Assume # 4 bars for flexure

Clear cover = 0.75 in
D = h-cover-stirrup-0.5dflexure = 3.00 in
A_s = 0.4

Check A_s,min:

\[ 3\sqrt{f'c'b'd}/f_y = 0.114 \text{ in}^2 \]
\[ 200bd/f_y = 0.120 \text{ in}^2 \]
\[ A_{s,min} = 0.120 \text{ in}^2 < 0.4 \text{ OK} \]

Check A_s,max:

\[ \rho_{max} = 0.0206 \]
\[ A_{s,max} = \rho bd = 0.7431 \text{ in}^2 > 0.4 \text{ OK} \]

Check A_s,temp:

\[ A_{s,temp} = 0.0018bh = 0.0972 \text{ in}^2 < 0.2 \text{ OK} \]

Use A_{s,temp} = 0.2 in^2 @ 18 in

Determine M_n:

\[ a=A_s f_y/.85f'c'b = 0.588 \text{ in} \]
\[ \beta = 0.850 \]
\[ c = a/\beta = 0.692 \text{ in} \]
\[ \varepsilon_s = 0.003(d-c)/c = 0.0100 \text{ in/in} \]
\[ \varepsilon_y = 60/29000 = 0.0021 \text{ in/in} < 0.0100 \text{ OK} \]
\[ \phi = 0.9 \]
\[ \phi M_n = \phi A_s f_y(d-a/2) = 58.45 \text{ k-in} > 22.85 \text{ OK} \]

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:
\[
\lambda = 1 \\
V_c = 2 \lambda V' c \ b_w \ d = 4.55 \text{ k} > 1.05 \\
\phi = 0.75 \\
\phi V_n = 0.5 \phi V_c = 1.71 \text{ k}
\]

**Beam Design:**

\[
w_D, \text{ superimposed} = 0.150 \text{ ksf} \\
w_D, \text{ slab contribution} = h \times 150/12 = 0.056 \text{ ksf} \\
w_D, \text{ beam contribution} = (h - t_{slab}) \times b \times 150/144 = 0.258 \text{ klf} \\
w_L = 0.070 \text{ ksf} \\
\text{Analysis 10 ft width, } b = 120 \text{ in} \\
w_u = 1.2D + 1.6L = 3.905 \text{ klf}
\]

**Moments (assume continuous interior span):**

\[
M = w_D L^2 / 11 = -3260.79 \text{ kin} \\
M^+ = w_D L^2 / 16 = 2241.80 \text{ kin} \\
V_u = w_u L / 2 = 54.02 \text{ k}
\]

Assume # 4 bars for stirrups
Assume # 9 bars & 0 bars for flexure

<table>
<thead>
<tr>
<th>clear cover</th>
<th>1.5 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>b_u</td>
<td>16 in</td>
</tr>
<tr>
<td>d= h-cover-stirrup-0.5d_{flexure}= 17.44 in</td>
<td></td>
</tr>
<tr>
<td>Number of bars</td>
<td>5 9 &amp; 0 0</td>
</tr>
<tr>
<td>A_s</td>
<td>5.00 in²</td>
</tr>
</tbody>
</table>

**Check If T-beam behavior occurs:**

\[
h_f = 4.5 \text{ in} \\
b_u + 16h_f = 88 \text{ in} \\
b_u + 2(.5\text{clear distance}) = 104 \text{ in} \\
.25\text{span length} = 90 \text{ in} \\
b_{eff,int} = 88 \text{ in} \\
M_{u,T-Beam} = \phi 0.85f'c \cdot bh(d-h/2) = 18,404 \text{ kin} > 3,261 \text{ NO T-BEAM}
\]

**Determine M_{stl} for } \rho = \rho_{max}$$;$$
\[
\rho_{max} = 0.85(f'c/f_y)\beta(0.003/(0.003+0.005)) = 0.0181 \\
A_{stl} = \rho_{max} bd = 5.039 \text{ in}²
\]
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\[ a = A_{st} f_c / 0.85 f_y = 5.558 \text{ in}^2 \]
\[ M_{nil} = 0.85 f_y d (d - a/2) = 4432 \text{ kin} > 3261 \text{ REINFORCED} \]

**Check \( A_{s, \text{min}} \):**

\[ 3 \sqrt{f_y bd / f_y} = 0.882 \text{ in}^2 \]
\[ 200bd / f_y = 0.930 \text{ in}^2 \]
\[ A_{s, \text{min}} = 0.930 \text{ in}^2 < 5.00 \text{ OK} \]

**Check \( A_{s, \text{max}} \):**

\[ \rho_{\text{max}} = 0.0206 \]
\[ A_{s, \text{max}} = \rho bd = 5.7594 \text{ in}^2 > 5.00 \text{ OK} \]

**Determine \( M_n \):**

\[ a = A_{s} f_c / 0.85 f_y = 5.515 \text{ in} \]
\[ \beta = 0.850 \]
\[ c = a / \beta = 6.488 \text{ in} \]
\[ g_s = 0.003 (d - c) / c = 0.0051 \text{ in/in} \]
\[ g_y = 60 / 29000 = 0.0021 \text{ in/in} < 0.0051 \text{ OK} \]
\[ \phi = 0.9 \]
\[ \phi M_n = \phi A_{s} f_c (d - a/2) = 3964 \text{ kin} > 3261 \text{ OK} \]

**Maximum Number of Bars:**

\[ b_{\text{min}} = 2c + 2d + nd + (n-1)4/3 = 14.96 < 16 \text{ OK} \]

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

Min numbers of bars = 3 < 5 \text{ OK} \]

**Determine shear strength of beam without stirrups:**

\[ \lambda = 1 \]
\[ V_c = 2 \lambda \sqrt{f_y b_w d} = 35.29 \text{ k} < 54.02 \text{ SHEAR REINF.} \]
\[ \phi = 0.75 \]
\[ \phi V_n = 0.5 \phi V_c = 13.23 \text{ k} \]

**Determine shear strength required by shear reinforcing:**

\[ V_u @ d = 48.34 \text{ k} \]
\[ V_s = V_u / \phi - V_c = 29.17 \text{ k} \]
\[ V_s \leq 8 \sqrt{f_y b_w d} = 141.16 \text{ k} \text{ OK} \]

No reinforcing required at:

\[ 57.55 \text{ in} \]
Determine maximum spacing of shear reinforcing:

\[ V_s \leq 4 \sqrt{f'c \cdot bw \cdot d} = 70.58 \text{ k} \quad \text{OK} \]
\[ s = \frac{d}{2} = 8.72 \text{ in} \]
\[ s = 24 = 24 \text{ in} \]
\[ s_{\text{max}} = 8.72 \text{ in} \quad \text{use 8 in} \]

Determine minimum shear reinforcement:

\[ A_v = 0.75 \sqrt{f'c \cdot bw} \cdot s/f_y = 0.101 \text{ in}^2 \]
\[ A_v = 50 \cdot bw \cdot s/f_y = 0.107 \text{ in}^2 \]
\[ A_{v,\text{min}} = 0.107 \text{ in}^2 \quad \text{use 4} \]
\[ A_{v,\text{used}} = 0.4 \text{ in}^3 \]

Design Shear Reinforcement:

\[ s = A_v f_y d/V_s = 14.35 \text{ in} \]

Use (2) # 4 stirrups: 1 @ 2", 8 @ 8 in each end

Girder Design:

\[ w_{D,\text{superimposed}} = 0.150 \text{ ksf} \]
\[ w_{D,\text{slab contribution}} = h x 150/12 = 0.056 \text{ ksf} \]
\[ w_{D,\text{beam contribution}} = (h-t_{\text{slab}}) b150/144x10\text{ft} = 0.026 \text{ ksf} \]
\[ w_{D,\text{girder contribution}} = (h-t_{\text{slab}}) b150/144 = 0.831 \text{ klf} \]
\[ w_L = 0.070 \text{ ksf} \]

Analysis Trib width 30 ft = 360 in
\[ w_o = 1.2D+1.6L = 12.713 \text{ klf} \]

Moments (assume continuous interior span):

\[ M = w_o h_o^2/11 = -19242.97 \text{ kin} \]
\[ M^+ = w_o h_o^2/16 = 13229.54 \text{ kin} \]
\[ V_u = w_o h_o/2 = 236.77 \text{ k} \]

Assume # 4 bars for stirrups
Assume # 9 bars & 10 bars for flexure

clear cover = 1.5 in
\[ b_w = 28 \text{ in} \]
\[ d_c = h\text{-cover-stirrup}-0.5d_{\text{flexure}} = 30.38 \text{ in} \]
\[ d = h\text{-cover-stirrup}-1.5d_{\text{flexure}} = 29.13 \text{ in} \]
d' = cover + stirrup + 0.5d_{lateral} = 2.56 in
Number of bars = 7 9 & 7 10
A_v = 15.89 in^2

Check If T-beam behavior occurs:

\[ h_f = 4.5 \text{ in} \]
\[ b_w + 16h_f = 100 \text{ in} \]
\[ b_w + 2(0.5\text{clear distance}) = 332 \text{ in} \]
\[ .25\text{span length} = 120 \text{ in} \]
\[ b_{eff, int} = 100 \text{ in} \]

\[ M_{u,T-beam} = \phi 0.85f'_c b h_f (d-h_f/2) = 37,753 \text{ kin} > 19,243 \text{ NO T-BEAM} \]
\[ A_{sf} = 0.85f'_c (b-b_w) h_f / f_y = 18.36 \text{ in}^2 \]
\[ M_{nf} = 0.85f'_c (b-b_w) h_f (d-h_f/2) = 30,983 \text{ kin} \]
\[ M_{nw} = M_u / \phi - M_{nf} = (9,601) \text{ kin} \]
\[ \phi M_{nw} = (8,424) \text{ kin} \]

Determine \( M_{nt1} \) for \( \rho = \rho_{max, \phi} \):
\[ \rho_{max, \phi} = 0.85(f'_c / f_y)\beta (0.003/(0.003+0.005)) = 0.0181 \]
\[ A_{nt} = \rho_{max, \phi} b d = 14.730 \text{ in}^2 \]
\[ a = A_{nt} f_y / 0.85f'_c b = 9.284 \text{ in}^2 \]
\[ \beta = 0.850 \]
\[ c = a / \beta = 10.922 \text{ in} \]

\[ M_{nt1} = 0.85f'_c a b (d-a/2) = 22,743 \text{ kin} > (9,601) \text{ SINGLY REINFORCED} \]

Determine \( M_{nt2} \):
\[ M_{nt2} = M_{nw} - M_{nt1} = 32,344 \text{ kin} \]

Determine \( A_{s2} \) assuming \( f_s = f_y \):
\[ A_{s2} = M_{nt2} / f_y (d-d') = -20.295 \text{ in}^2 \]

Find required \( A'sw \):
\[ f'_s = 0.003(c-d')E_s / c = 66588 \text{ psi} \]
\[ f'_s = 60000 \text{ psi} \]
\[ A'sw = A_{s2} (f_y / f'_s) = -20.29 \text{ in}^2 \]

Required Steel:
\[ A_s = A_{sf} + A_{sw} = 13 \text{ in}^2 \]
\[ A's = A'sw = (20) \text{ in}^2 \]
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Check $A_{s,\text{min}}$:

$$3\sqrt{f'_{c}bd/f_{y}} = 2.579 \text{ in}^2$$

$$200bd/f_{y} = 2.718 \text{ in}^2$$

$$A_{s,\text{min}} = 2.718 \text{ in}^2 < 15.89 \text{ OK}$$

Check $A_{s,\text{max}}$:

$$\rho_{\text{max}} = 0.0206$$

$$A_{s,\text{max}} = \rho_{\text{bd}} = 16.8343 \text{ in}^2 > 15.89 \text{ OK}$$

Determine $M_{n}$:

$$a = A_{s}f_{y}/0.85f'_{c}b = 10.015 \text{ in}$$

$$\beta = 0.850$$

$$c = a/\beta = 11.782 \text{ in}$$

$$\varepsilon_{s} = 0.003(d-c)/c = 0.0047 \text{ in/in}$$

$$\varepsilon_{y} = 60/29000 = 0.0021 \text{ in/in} < 0.0047 \text{ OK}$$

$$\phi = 0.88$$

$$\phi M_{n} = \phi A_{s}f_{y}(d-a/2) = 20,173 \text{ kin} > 19,243 \text{ OK}$$

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:

$$
\lambda = 1 \\
V_{c} = 2 \lambda \sqrt{f'_{c}bw}d = 103.15 \text{ k} < 236.77 \text{ SHEAR REINF.}$$

$$\phi = 0.75$$

$$\phi V_{n} = 0.5\phi V_{c} = 38.68 \text{ k}$$

Determine shear strength required by shear reinforcing:

$$V_{u} @ d = 205.92 \text{ k}$$

$$V_{s} = V_{u}/\phi V_{c} = 171.40 \text{ k}$$

$$V_{s} \leq 8 \sqrt{f'_{c}bw}d = 412.61 \text{ k}$$

No reinforcing required at: 126.13 in
Determine maximum spacing of shear reinforcing:

\[ V_s \leq 4 \sqrt{f'_c b_w d} = 206.31 \text{ k} \quad \text{OK} \]

\[ s = d/2 \quad 14.56 \text{ in} \]

\[ s = 24 \quad 24 \text{ in} \]

\[ s_{\text{max}} = 14.56 \text{ in} \quad \text{use} \quad 14 \text{ in} \]

Determine minimum shear reinforcement:

\[ \Lambda_v = 0.75 \sqrt{f'_c b_w s/f_y} = 0.310 \text{ in}^2 \]

\[ \Lambda_v = 50 b_w s/f_y = 0.327 \text{ in}^2 \]

\[ \Lambda_{v,\text{min}} = 0.327 \text{ in}^2 \quad \text{use} \quad 4 \]

\[ \Lambda_{v,\text{used}} = 0.40 \text{ in}^3 \]

Design Shear Reinforcement:

\[ s = \Lambda_v f_y d/V_s = 4.08 \text{ in} \]

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

System Weight:

<table>
<thead>
<tr>
<th>Item</th>
<th>#/bay</th>
<th>Span (ft)</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>4</td>
<td>27.67</td>
<td>31000 LBS</td>
</tr>
<tr>
<td>Girder</td>
<td>1</td>
<td>37.25</td>
<td>33250 LBS</td>
</tr>
<tr>
<td>Slab</td>
<td>1</td>
<td></td>
<td>67500 LBS</td>
</tr>
</tbody>
</table>

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:**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing Dead Load</td>
<td>Self weight</td>
<td></td>
</tr>
<tr>
<td>Superimposed Dead Load</td>
<td>25 psf</td>
<td></td>
</tr>
<tr>
<td>Live Load</td>
<td>70 psf</td>
<td></td>
</tr>
<tr>
<td>2 hour fire-rating</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Materials:**

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Normal weight</td>
<td>150 pcf</td>
</tr>
<tr>
<td></td>
<td>$f'_c$</td>
<td>5,000 psi</td>
</tr>
<tr>
<td></td>
<td>$f'_{ci}$</td>
<td>3,000 psi</td>
</tr>
<tr>
<td>Rebar</td>
<td>$f_y$</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Post Tension</td>
<td>Unbonded tendons</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$f_p$</td>
<td>0.153 in$^2$</td>
</tr>
<tr>
<td></td>
<td>$f_u$</td>
<td>270 ksi</td>
</tr>
<tr>
<td></td>
<td>Estimated prestress losses</td>
<td>15 ksi</td>
</tr>
<tr>
<td></td>
<td>$f_{se}$</td>
<td>0.7 $f_pu$, losses 174 kips/tendo</td>
</tr>
<tr>
<td></td>
<td>$P_{eff}$</td>
<td>26.62 n</td>
</tr>
</tbody>
</table>

**Determine Preliminary Slab Thickness:**

<table>
<thead>
<tr>
<th>Span Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long Span</td>
<td></td>
</tr>
<tr>
<td>$L_1$</td>
<td>30 ft</td>
</tr>
<tr>
<td>$L_2$</td>
<td>40 ft</td>
</tr>
<tr>
<td>$h = L_{longest}/45$</td>
<td>10.67 in</td>
</tr>
<tr>
<td>Preliminary slab thickness</td>
<td>11.50 in</td>
</tr>
</tbody>
</table>

**Loading:**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing Dead Load</td>
<td>self weight</td>
<td>143.75 psf</td>
</tr>
<tr>
<td>Superimposed Dead Load</td>
<td></td>
<td>25 psf</td>
</tr>
<tr>
<td>Live Load</td>
<td></td>
<td>70 psf</td>
</tr>
</tbody>
</table>

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

**Section Properties:**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A = bh = (360 \text{ in})t_{slab}$</td>
<td>4140 in$^2$</td>
</tr>
<tr>
<td>$S = bh^2/6 = (360 \text{ in})(t_{slab})^2/6$</td>
<td>7935 in$^3$</td>
</tr>
</tbody>
</table>
**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
- $a_{end} = 0.5 \times (1.5 \times t_{slab} - cover - 0.5 \times d_{tendon}) - cover - 0.5 \times d_{tendon} = 6.3750$ 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:

- $P = \frac{w_b L^2}{8a_{end}} = 974.12$ k

**Check Precompression Allowance:**

- $\#$ tendons $= 36.59$ use 36 tendon

Actual force for banded tendons:

- $P_{act} = \#$ tendons $\times P_{eff} = 958.4$ k
Adjust End Span Balanced Load:

\[ w_b = 2.546 \text{ klf} \]

Determine actual Precompression stress

\[ \frac{P_{\text{actual}}}{A} = \frac{P_{\text{actual}} \times 1000}{A} = 231.50 \text{ psi} > 125 \text{ psi min. OK} \]

\[ < 300 \text{ psi max. OK} \]

Check Interior Span Force:

\[ P = \frac{w_b L^2}{8a_{\text{int}}} = 653.68 \text{ k} < 958.4 \text{ k Required} \]

\[ w_b = \frac{P_{\text{act}}}{8 \times a_{\text{int}} / L^2} = 3.794 \text{ klf} \]

\[ w_b / w_{DL} = 0.880 \text{ OK} \]

Effective prestress force, \( P_{\text{eff}} = 958.4 \text{ k} \)

Check Slab Stresses:

Dead Load Moments:

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

Live Load Moments:

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

Total Balancing Moments:

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

Stage 1: Stresses Immediately after Jacking (DL + PT):
Midspan Stresses:
\[
\begin{align*}
 f_{\text{top}} &= (-MDL + MBAL) / S - P/A \\
 f_{\text{bot}} &= (+MDL - MBAL) / S - P/A
\end{align*}
\]
Interior Span:
\[
\begin{align*}
 f_{\text{top}} &= -346.00 < 1800 \text{ OK} \\
 f_{\text{bot}} &= -117.00 < 1800 \text{ OK}
\end{align*}
\]
End Span:
\[
\begin{align*}
 f_{\text{top}} &= -597.89 < 1800 \text{ OK} \\
 f_{\text{bot}} &= 134.90 < 164 \text{ OK}
\end{align*}
\]
Support Stresses:
\[
\begin{align*}
 f_{\text{top}} &= (+MDL - MBAL) / S - P/A \\
 f_{\text{bot}} &= (-MDL + MBAL) / S - P/A
\end{align*}
\]
\[
\begin{align*}
 f_{\text{top}} &= 226.50 > 164 \text{ NEED REINF.} \\
 f_{\text{bot}} &= -689.49 < 1800 \text{ OK}
\end{align*}
\]
Stage2: Stresses at Service Load (DL + LL + PT):
Midspan Stresses:
\[
\begin{align*}
 f_{\text{top}} &= (-MDL - MDL + MBAL) / S - P/A \\
 f_{\text{bot}} &= (+MDL + MDL - MBAL) / S - P/A
\end{align*}
\]
Interior Span:
\[
\begin{align*}
 f_{\text{top}} &= -473.03 < 1350 \text{ OK} \\
 f_{\text{bot}} &= 10.04 < 329 \text{ OK}
\end{align*}
\]
End Span:
\[
\begin{align*}
 f_{\text{top}} &= -1004.40 < 1350 \text{ OK} \\
 f_{\text{bot}} &= 541.41 > 329 \text{ NEED REINF.}
\end{align*}
\]
Support Stresses:
\[
\begin{align*}
 f_{\text{top}} &= (+MDL + MDL - MBAL) / S - P/A \\
 f_{\text{bot}} &= (-MDL - MDL + MBAL) / S - P/A
\end{align*}
\]
\[
\begin{align*}
 f_{\text{top}} &= 734.63 > 329 \text{ NEED REINF.} \\
 f_{\text{bot}} &= -1197.62 < 1350 \text{ OK}
\end{align*}
\]
Ultimate Strength:
\[
\begin{align*}
 M_{1} &= Pe = 379.36 \text{ kft} \\
 M_{\text{sec}} &= MBAL - M_{1} = 127.79 \text{ kft} \text{ @ interior supports}
\end{align*}
\]
\[ M_u = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{SEC} \]

- \( M_u = 1271.57 \text{ kft} \) @ midspan
- \( M_u = 1573.49 \text{ kft} \) @ support

**Minimum Bonded Reinforcement:**

**Positive Moment Region:**

**Interior Span:**
- \( f_t = 10.04 \text{ psi} \) < 141

**Exterior Span:**
- \( f_t = 541.41 \text{ psi} \) > 141

\[
y = \frac{f_t}{f_t + f_y} h = 4.03 \text{ in}
\]

\[
N_c = \frac{M_{DL} + M_{LL}}{S*0.5y*L_1} = 1005.19 \text{ kip}
\]

\[
A_{s,min} = \frac{N_c}{0.5f_y} = 33.51 \text{ in}^2
\]

Distribute reinforcement evenly across the width of the slab

\[
A_{s,min} = \frac{A_{s,min}}{L_1} = 1.117 \text{ in}^2/\text{ft}
\]

Use \#7 @ 6" O.C.

**Negative Moment Region:**

**Interior Supports:**

\[
A_{ef} = t_{slab} \times L \times 12 = 5520 \text{ in}^2
\]

\[
A_{s,min} = 0.00075 A_{ef} = 4.14 \text{ in}^2
\]

Use 14 #5 Top

\[
A_{ef} = t_{slab} \times L \times 12 = 4140 \text{ in}^2
\]

\[
A_{s,min} = 0.00075 A_{ef} = 3.105 \text{ in}^2
\]

Use 11 #5 Top

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:
\[ d_{\text{support}} = t_{\text{lab}} - \text{cover} - 0.5 \times d_{\text{tendon}} = 10.50 \text{ in} \]
\[ A_{ps} = 0.153 \text{ in}^2 \times \text{(number of tendons)} = 5.51 \text{ in}^2 \]
\[ f_{ps, \text{support}} = f_{se} + 10,000 + \frac{f'_{cbd}}{300A_{ps}} = 195,438 \text{ psi} \]
\[ a = \frac{(A_{sfy} + A_{ps}f_{ps})}{(0.85f'_{cb})} = 0.87 \text{ in} \]

\[ \phi M_n = \phi (A_{sfy} + A_{ps}f_{ps}) \left( d - a/2 \right) = 1,009 \text{ kft} < 1,573 \text{ kft} \]

**UTL. STGTH REINF. GOVERNS**

\[ A_{s, \text{reqd}} = 17.74 \text{ in}^2 \]

**Use #7 @ 12" O.C. Bottom at end span**

\[ A_s = 18.00 \text{ in}^2 \]

At Midspan:

\[ d_{\text{support}} = t_{\text{lab}} - \text{cover} - 0.5 \times d_{\text{tendon}} = 9.75 \text{ in} \]
\[ A_{ps} = 0.153 \text{ in}^2 \times \text{(number of tendons)} = 5.51 \text{ in}^2 \]
\[ f_{ps, \text{support}} = f_{se} + 10,000 + \frac{f'_{cbd}}{300A_{ps}} = 194,621 \text{ psi} \]
\[ a = \frac{(A_{sfy} + A_{ps}f_{ps})}{(0.85f'_{cb})} = 2.01 \text{ in} \]
\[ \phi M_n = \phi (A_{sfy} + A_{ps}f_{ps}) \left( d - a/2 \right) = 2,021 \text{ kft} > 1,272 \text{ kft} \]

**MIN. REINF. OK**

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

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

**Section Properties:**

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

**Design Parameters:**

Allowable stresses: Class U (ACI 18.3.3):

At time of jacking (ACI 18.4.1):

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

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

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

Average precompression limits:

\[ \frac{P}{A} = \begin{cases} 
125 & \text{psi min (ACI 18.12.4)} \\
300 & \text{psi max}
\end{cases} \]

Target load balances:

\[ \% = 0.6 \]
\[ \% \text{wDL} = 86 \text{ psf} \]

2 Hour Fire Rating:

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

Tendon Profile:

\[ a_{\text{int}} = t_{\text{slab}} - 2 \times \text{cover} - d_{\text{tendon}} = 9.50 \text{ in} \]
\[ a_{\text{end}} = 0.5 \times (1.5 \times t_{\text{slab}} - \text{cover} - 0.5 \times d_{\text{tendon}}) - \text{cover} - 0.5 \times d_{\text{tendon}} = 6.375 \text{ in} \]

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

\[ w_{b} = \% \text{wDL} = 3.450 \text{ klf} \]

Force needed to counteract load in end bay:

\[ P = \frac{w_{b}L^{2}}{8a_{\text{end}}} = 730.59 \text{ k} \]

Check Precompression Allowance:

\[ \# \text{ tendons} = 27.44 \text{ use 28 tendons} \]

Actual force for banded tendons

\[ P_{\text{act}} = \# \text{ tendons} \times P_{\text{eff}} = 745.4 \text{ k} \]

Adjust End Span Balanced Load:

\[ w_{b} = 3.520 \text{ klf} \]

Determine actual Precompression stress

\[ \frac{P_{\text{actual}}}{A} = \frac{P_{\text{actual}}}{A} = 135.04 \text{ psi} > 125 \text{ psi min. OK} \]
\[ < 300 \text{ psi max. OK} \]

Check Interior Span Force:

\[ P = \frac{w_{b}L^{2}}{8a_{\text{int}}} = 490.26 \text{ k} < 745.4 \text{ Less Force Required} \]
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\[ w_b = P_{act} x 8 x a_{int}/1^2 = 5.246 \text{ klf} \]
\[ w_b / w_{DL} = 0.912 \text{ OK} \]

Effective prestress force, \( P_{eff} \) = 745.4 k

Check Slab Stresses:

Dead Load Moments:

\[ w_{DL} = 6.750 \text{ klf} \]
\[ M_\text{e} = 607.50 \text{ t} \]
\[ M^\text{ex} = 486.00 \text{ t} \]
\[ M^\text{int} = 151.88 \text{ t} \]

Live Load Moments:

\[ w_{LL} = 2.800 \text{ klf} \]
\[ M_\text{e} = 252.00 \text{ t} \]
\[ M^\text{ex} = 201.60 \text{ t} \]
\[ M^\text{int} = 63.00 \text{ t} \]

Total Balancing Moments:

\[ w_b = 4.383 \text{ klf} \]
\[ M_\text{e} = 394.45 \text{ t} \]
\[ M^\text{ex} = 315.56 \text{ t} \]
\[ M^\text{int} = 98.61 \text{ t} \]

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

Midspan Stresses:

\[ f_{\text{top}} = (\text{-}M_{DL} + M_{BAL}) / S - P/A \]
\[ f_{\text{bot}} = (\text{+}M_{DL} - M_{BAL}) / S - P/A \]

Interior Span:

\[ f_{\text{top}} = 195.45 < 1800 \text{ OK} \]
\[ f_{\text{bot}} = -74.63 < 1800 \text{ OK} \]

End Span:
Structural Option

Dr. Andres Lepage

10/28/2009

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Support Stresses:

\[
\begin{align*}
\sigma_{\text{top}} &= \frac{+MDL-MBAL}{S} - \frac{P}{A} \\
\sigma_{\text{bot}} &= \frac{-MDL+MBAL}{S} - \frac{P}{A}
\end{align*}
\]

\[
\begin{align*}
\sigma_{\text{top}} &= 106.61 \, \text{ksi} < 164 \, \text{OK} \\
\sigma_{\text{bot}} &= -376.68 \, \text{ksi} < 1800 \, \text{OK}
\end{align*}
\]

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

Midspan Stresses:

\[
\begin{align*}
\sigma_{\text{top}} &= \frac{-MDL}{S} - \frac{MDL+MBAL}{S} - \frac{P}{A} \\
\sigma_{\text{bot}} &= \frac{+MDL}{S} - \frac{MDL-MBAL}{S} - \frac{P}{A}
\end{align*}
\]

Interior Span:

\[
\begin{align*}
\sigma_{\text{top}} &= -266.91 \, \text{ksi} < 1350 \, \text{OK} \\
\sigma_{\text{bot}} &= -3.17 \, \text{ksi} < 329 \, \text{OK}
\end{align*}
\]

End Span:

\[
\begin{align*}
\sigma_{\text{top}} &= -557.01 \, \text{ksi} < 1350 \, \text{OK} \\
\sigma_{\text{bot}} &= 286.94 \, \text{ksi} < 329 \, \text{OK}
\end{align*}
\]

Support Stresses:

\[
\begin{align*}
\sigma_{\text{top}} &= \frac{+MDL + MDL-MBAL}{S} - \frac{P}{A} \\
\sigma_{\text{bot}} &= \frac{-MDL - MDL+MBAL}{S} - \frac{P}{A}
\end{align*}
\]

End Span:

\[
\begin{align*}
\sigma_{\text{top}} &= 392.43 \, \text{ksi} > 329 \, \text{NEED REINF.} \\
\sigma_{\text{bot}} &= -662.51 \, \text{ksi} < 1350 \, \text{OK}
\end{align*}
\]

Ultimate Strength:

\[
\begin{align*}
M_1 &= \frac{P_e}{2} = 295.06 \, \text{kft} \\
M_{sec} &= MBAL - M_1 = 99.39 \, \text{kft @ interior supports} \\
M_u &= 1.2MDL + 1.6MLL + 1.0M_{sec} = 955.45 \, \text{kft @ midspan} \\
M_u &= 1.2MDL + 1.6MLL + 1.0M_{sec} = 1181.89 \, \text{kft @ support}
\end{align*}
\]

Minimum Bonded Reinforcement:

Positive Moment Region:

\[
\begin{align*}
\sigma_t &= -3.17 \, \text{ksi} < 141 \, \text{NO REINF. REQ.} \\
\sigma_t &= 286.94 \, \text{ksi} > 141 \, \text{NEED REINF.}
\end{align*}
\]

\[
y = \frac{f_t}{f_t + f_y} h = 3.91 \, \text{in}
\]
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Negative Moment Region:

Interior Supports:

\[
A_{ef} = t_{slab} \times L \times 12 = 5520 \text{ in}^2
\]

\[
A_{s,min} = 0.00075 A_{ef} = 4.14 \text{ in}^2
\]

Use 14 #5 Top

Exterior Supports:

\[
A_{ef} = t_{slab} \times L \times 12 = 5520 \text{ in}^2
\]

\[
A_{s,min} = 0.00075 A_{ef} = 4.14 \text{ in}^2
\]

Use 14 #5 Top

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:

\[
d_{supports} = t_{slab} - \text{cover} - 0.5 \times d_{tendon} = 10.50 \text{ in}
\]

\[
A_{ps} = 0.153 \text{ in}^2 \times (\text{number of tendons}) = 4.28 \text{ in}^2
\]

\[
f_{ps, supports} = f_{se} + 10,000 + (f'_{cd}d) / (300A_{ps}) = 203.6 \text{ psi}
\]

\[
a = (A_{sfy} + A_{ps}f_{ps}) / (0.85f'_{cd}b) = 0.56 \text{ in}
\]

\[
\phi M_n = \phi (A_{sfy} + A_{ps}f_{ps}) (d-a/2) = 868 \text{ kft} < 1,182 \text{ kft}
\]

UTL. STGTH REINF. GOVERNS

\[
A_{s,reqd} = 11.43 \text{ in}^2
\]

Use #7 @ 12" O.C. Bottom at end span

\[
A_s = 12.00 \text{ in}^2
\]
At Midspan:

\[ d_{\text{supports}} = t_{\text{slab}} - \text{cover} - 0.5 \times d_{\text{tendon}} = 9.75 \text{ in} \]
\[ A_p = 0.153 \text{ in}^2 \times (\text{number of tendons}) = 4.28 \text{ in}^2 \]
\[ f_{ps, \text{supports}} = f_{se} + 10,000 + \frac{(f'_{cbd})(300A_p)}{(f'_{bd})} = 202,207 \text{ psi} \]
\[ a = (A_{f'_{se}} + A_{psf_p}) / (0.85f'_{b}) = 1.14 \text{ in} \]
\[ \phi M_n = \phi (A_{sfy} + A_{psf_p}) (d-a/2) = 1,604 \text{ kft} \quad > \quad 955 \text{ kft} \]

**MIN. REINF. OK**

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

### Punching Shear:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing Dead Load</td>
<td>self weight = ( t_{\text{slab}}(150\text{pcf}) ) = 143.75 psf</td>
</tr>
<tr>
<td>Superimposed Dead Load</td>
<td>= 25 psf</td>
</tr>
<tr>
<td>Live Load</td>
<td>= 70 psf</td>
</tr>
<tr>
<td>( w_u = 1.2DL + 1.6LL )</td>
<td>= 314.5 psf</td>
</tr>
<tr>
<td>Area = A</td>
<td>= 1193.75 \text{ ft}^2</td>
</tr>
<tr>
<td>( V_u = w_uA )</td>
<td>= 375.43 k</td>
</tr>
<tr>
<td>( d_{\text{average}} )</td>
<td>= 10.125 in</td>
</tr>
<tr>
<td>( b_o )</td>
<td>= 160.5 in</td>
</tr>
<tr>
<td>( V_c = \text{min of:} )</td>
<td></td>
</tr>
<tr>
<td>( 4f'_{c}b_od )</td>
<td>= 459.64 k</td>
</tr>
<tr>
<td>( (2+4/\beta)f'_{c}b_od )</td>
<td>= 689.46 k</td>
</tr>
<tr>
<td>( (\alpha d/b_o + 2)f'_{c}b_od )</td>
<td>= 519.78 k</td>
</tr>
<tr>
<td>( V_c )</td>
<td>= 459.64 k</td>
</tr>
<tr>
<td>( \phi V_c )</td>
<td>= 344.73 k</td>
</tr>
</tbody>
</table>

\[ \text{MIN. REINF. OK} \]

### Size Drop Panel:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_u )</td>
<td>= ( \phi (f'_{c})^{1/2}b_od )</td>
</tr>
<tr>
<td>375.43</td>
<td>= 0.75x4x(f'_{c})(4(30+d))d</td>
</tr>
<tr>
<td>( d )</td>
<td>= 10.84 in</td>
</tr>
<tr>
<td>( \phi V_c )</td>
<td>= 375.65 k</td>
</tr>
<tr>
<td>\text{Need}</td>
<td>= 11.84 in</td>
</tr>
<tr>
<td>\text{use}</td>
<td>= 12 in</td>
</tr>
</tbody>
</table>

**OK**

### Beam Shear:

At Panel:
Area = \( A = 600 \text{ ft}^2 \)

\[ V_u = w_u A = 188.7 \text{ k} \]

assumed 14 ft drop panel

\[ b_w = 168 \text{ in} \]

\[ d = 11 \text{ in} \]

\[ \phi V_c = 0.75 \times 2 \sqrt{f'_{cb} w d} = 196.01 \text{ k} > 188.70 \text{ k OK} \]

At Slab:

Area = \( A = 390 \text{ ft}^2 \)

\[ V_u = w_u A = 122.66 \text{ k} \]

\[ b_w = 360 \text{ in} \]

\[ d = 10.84 \text{ in} \]

\[ \phi V_c = 0.75 \times 2 \sqrt{f'_{cb} w d} = 413.91 \text{ k} > 122.66 \text{ k OK} \]

**System Weight:**

<table>
<thead>
<tr>
<th>Item</th>
<th>#/bay</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>1</td>
<td>172500 lbs</td>
</tr>
<tr>
<td>Drop Panel</td>
<td>1</td>
<td>1225 lbs</td>
</tr>
</tbody>
</table>

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*