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MCPHS Peer Review

Dillon T. O’Toole

Worcester Polytechnic Institute

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MCPHS Peer Review

A Major Qualifying Project

Submitted to the Faculty

Of the

WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Bachelor of Science

By

Dillon O’Toole

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Submitted To:

Professor Tahar El-Korchi

Professor Edward J. Swierz
Abstract

The objective of this MQP was to review the structural design of the superstructure of an office and academic building recently built for the Massachusetts College of Pharmacy and Health Studies. The design included post tensioned concrete slabs, as well as reinforced concrete columns, shear walls and a mat foundation. The design was reviewed using load resistance factor design, in accordance with the Commonwealth of Massachusetts state building code, and accounted for both gravity and lateral loads. The flooring system was also redesigned as an alternative steel design.
Acknowledgements

First and foremost I would like to thank my advisors for the project, Professor Edward J. Swierz and Professor Tahar El-Korchi for their feedback, guidance and support. I would also like to thank Souza, True and Partners Inc. and KSID for allowing me to access their design plans and architectural drawings.
Capstone Design Statement

The capstone design requirement of this project was met by proposing a redesign of the existing floor plan with structural steel and a concrete slab on metal decking instead of post tensioned concrete. WPI faculty and students would be interested in this comparison because it showcases the different shape and weight constraints and structural behavior brought about by differing building materials. Construction using steel can have a significant effect on the characteristics of a building in comparison to concrete. This project addressed the following realistic constraints: economic, constructability, social and political.

Economic

The cost of building a structure hinges upon the cost of building materials, as well as labor and other such expenses. Redesigning the flooring system with steel members as well as a concrete deck changes the weight of the building in a large way, but is not necessarily financially advantageous. The rising price of metals in the current economic climate also has a hand to play in the pricing of any proposed steel project.

Constructability

Constructability plays a large part in the selection of building materials and design. A scheme may be perfectly designed to bear the proper loads, but if the shaping is too exotic it cannot be created in a real world application. Concrete typically needs a formwork created beforehand in the shape of the desired member. The proposed redesign is advantageous for multiple reasons. Steel members are premade and shipped to a job site, eliminating the need for formwork to be made. The steel decking used for the floor slabs also serve as a mold for the concrete to be poured. The concrete slab proposed in the steel redesign also requires no post tensioning, so that step is eliminated from the construction schedule.
Social

The material used also has an effect on the social and current labor markets in the surrounding community. Any construction project has a positive boon to the surrounding community. In the New England area there is a strong steel labor force as well as concrete, so there is a positive effect with both material choices. Construction projects in general also provide a positive social impact to the community through increasing the amount of jobs and sales in the surrounding area.

Political

Political issues can arise for multiple reasons during construction. These issues are usually based on the effect construction has on the surrounding area, and not on the materials used in the construction phase. Because of this, a redesign would not have much if any effect on the political issues that arose from the building of this new structure.
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Introduction

Throughout the design process, there are many decisions and calculations that affect the accuracy and viability of the project. The risk of inaccuracies and the high cost of such errors makes peer reviews a necessary and vital part of the design and permit process. These reviews may entail redesigning major aspects of the building to compensate for erroneous aspects, and more importantly to check that the engineer of record used the proper design processes and Code mandates. Calculation accuracy and design integrity are paramount to a safe and functional structure.

The Massachusetts College of Pharmacy and Health Sciences (MCPHS) is an accredited institution that is headquartered in Boston, MA. MCPHS is a graduate school that focuses on the medical field. The Worcester campus hosts the accelerated pharmacy programs as well as physician assistance studies. The Worcester campus of MCPHS is twelve years old, having been added in 2000. This expansion is necessary to accommodate an ever growing influx of students. MCPHS recently started building a new six story, reinforced concrete building for office, laboratory and lecture hall purposes. This building is being constructed at 10 Lincoln square, Worcester, MA.

This project centered on a peer review of the essential aspects of the superstructure. This report should help WPI academia understand more about the design aspects chosen for this building. The data needed to review this structure, such as the permit and construction set of drawings design and architectural drawings was provided by the construction management firm in charge of the stage two renovations Souza, True and Partners. Additional code and design information was provided by the American Concrete Institute’s ACI 318-08 manual and the American institute for Steel Construction’s

1 Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. Farmington Hills, MI: American Concrete Institute, 2008. Print.
steel design manual\textsuperscript{2}. The structural design and analysis knowledge learned from courses taken at WPI's department of Civil and Environmental Engineering were utilized for the completion of this project as well.

Background

To complete this project, two major objectives were established. The first objective was to review the design of the MCPHS superstructure to check for the accuracy and validity of the design. The second was to redesign the flooring systems using structural steel as opposed to concrete. Before these two objectives could be achieved, information such as loading and usage, the original design and the applicable building codes were also needed. Once these items were obtained, the design could be reviewed, and found to be either adequate and ready for construction, or recommendations could be made that would make the building feasible and code compliant. In order to understand this project completely, this chapter on the background of the project is presented.

Academic and Office Building

The Massachusetts College of Pharmacy and Health studies recently needed to add a new building to its Worcester campus to accommodate larger demand for the college’s programs. The Worcester campus hosts the accelerated pharmacy programs as well as physician assistance studies and other graduate studies. This upgrade of the campus should act as a boon to both the institution and surrounding areas of Worcester.

Location

MCPHS’s new building is located on the same plot of land as another MCPHS building. The location is 10 Lincoln Square, in Worcester MA. This building is part of an effort to add more capacity to MCPHS’s Worcester campus. The building can be seen at its Lincoln Square location in figure 1 below. This building should also allow for expansion of the accelerated programs offered in the Worcester campus.
Description

The MCPHS building is a six story cast-in-place concrete building located on 10 Lincoln square in downtown Worcester. The flooring system consists of post tensioned slabs and spandrel beams along the perimeter of the building. All prestressing steel is 7 wires, half inch diameter steel with an area of 0.153 square inches and an ultimate strength of 41.3 kips. The flooring system uses 5,000 psi concrete.

The second floor framing system can be seen in Figure 3. The shear walls for the structure are reinforced concrete, as are the columns. The column specifications can be found in Figure 6. The shear walls
provide resistance for the lateral load demand. These members also are 5,000 psi concrete. The roofing is a pan joist and girder framing system. The foundation is a 3 foot thick, conventionally reinforced concrete mat foundation of 4,000 psi concrete. The building will hold a lecture hall on the main floor, as well as a large lobby, and laboratory space. The second floor will house office and administrative areas. The upper three floors will be used for similar programmatic needs. The sixth floor has a higher loading than the lower floors, so the slab is thicker. The façade is brick on the first floor, and glass from the second to the sixth, except for the shear walls.

Concrete

Modern concrete is made up of multiple ingredients at a specific ratio that determines the physical properties of the intended mix. Concrete is comprised of Portland cement, fine and coarse aggregates, water and admixtures. These four ingredients can be combined in any number of different ratios, which change the physical properties of the concrete. The strength of concrete can range anywhere from 3,000 pounds per square inch, to 16,000 pounds per square inch and higher in some markets.

Concrete is an excellent building material for anything that is under a compressive loading. Another positive aspect of concrete is its fluid state, because concrete can be poured into any shape imaginable, as long as the aggregate can fit and a proper mold is created. Concrete is relatively weak in tension however. In order to combat this, steel is placed within the concrete members to take the tensile load forces while the concrete handles compression. There are two different ways to achieve this balance of steel and concrete in modern concrete structures, Reinforced concrete, and Prestressed concrete.
Reinforced Concrete

Reinforced concrete is any concrete that has reinforcing bars to supply the required tensile strength for the concrete member. This technique is relatively low tech and has been in existence longer than prestressing. In reinforced concrete, the reinforcing cage is typically placed first, and then concrete is poured, creating a structure. Once the concrete is poured, it is left to cure for roughly a month, in order to achieve a useful strength. The reinforcement bars used in reinforced concrete are typically 60,000 psi yield strength bars, and are available in diameter increments of 1/8 of an inch. The largest bar that can typically be found is a #18 bar, which has a nominal diameter of 2 ⅛ inches.

In the MCPHS structure, most members are reinforced concrete. The columns, shear walls and foundation are all made from reinforced concrete, whereas the floor framing is prestressed concrete.

Prestressed Concrete

Prestressed concrete is the other modern use of concrete in structural applications. Prestressed concrete makes use of wire strands of steel that are banded into steel tendons. These tendons are stressed either before or after the concrete is added, and that timing of the stressing is the key difference between prestressed and post tensioned concrete. For slab applications, the strands are usually placed in a parabolic shape through the slab, in a shape that is similar to the bending moment diagram of the given slab.

Pretensioning involves the strands being tensioned before the concrete is placed. Once the concrete is sufficiently hardened, the anchors holding the strands are released, and the resulting force adds compressive stress to the concrete. Because a strong anchoring point is needed, Pretensioning is usually done at a plant, and then the piece is shipped to a job site.
Post tensioning involves placing an unstressed tendon, and then the concrete is poured. Once the concrete hardens to certain strength, typically 3,000 psi, the steel tendons are tensioned. It is critical that the concrete reaches a strength that can support the tensioning, or else the tensioning anchorage will fail. This approach is advantageous when a large section is being cast, such as the floor systems in the MCPHS building. There are some extra losses of prestressed strength with this approach due to friction and other prestress losses.

Figure 2: Post tensioning cables laid before a concrete pour
**Structural Steel**

Structural steel provides certain advantages that concrete does not. Firstly, steel comes in standard industry sizes, unlike concrete that has to be molded to a certain shape. Steel can be purchased in many different shapes, but the shape used for the beams and girders in this report were wide flanged, or W beams. Since all the beams are manufactured to certain specifications, most properties can be easily found in the AISC manual. Structural steel can be quick to erect if the shipments from fabricators are timed well, and doesn’t require the lengthy curing time of concrete.

**Loading**

When designing a structure, a critical design value needed is the loading that the structure will undergo. The loading of any building can be broken into three groups: live loads, dead loads and environmental loads. Live loads are any loads that are not static, such as occupants. Dead loads can be defined as permanent loads that are typically part of the building itself. Environmental loads are any loads that occur from the environment, such as earthquake and snow loads.

Minimum loading conditions can be found through the Massachusetts State Building Code, or MSBC. Though the MSBC covers most loads, any loads that aren’t covered can be found in the ASCE 7 standards. ASCE 7 is a reference material offered by the American Society of Civil Engineers in which minimum load provisions can be found. The only loads not given from MSBC and ASCE 7 are the self-load, which is calculated from the materials, used. These loads are factored with different values in order to add a factor of safety against limit states.

There are two different design approaches, LRFD and ASD, and each has their own load combinations and factors. For this project LRFD, or load resistance factor design, was used. There are multiple load combinations that can be used in different situations depending on what loads are acting on the member. For example, a floor system would have a higher load from a combination that has a
higher factor for dead and live loads, so they are to be used. Once the proper loads and load combination are found, the design of members could be performed.

Prestressed Floor system

According to the design drawings provided by the construction firm building the MCPHS structure, the flooring slabs were 90 feet by 93 feet overall and designed as post tensioned concrete slabs. The flooring was designed as a flat plate floor system, using techniques learned from CE 4017, prestressed concrete design. The strength of the concrete was given as 5,000 psi, and the values of the steel were found from design drawings.

The losses due to prestressing were found using lump sum losses, which were found in table 3.1 of Edward Nawy’s text book “Prestressed Concrete” \(^3\). Lump sum losses do not take loss due to friction into account, so that value had to be calculated independently and added to the lump sum losses. These losses were subtracted from the strength in prestressed reinforcement at nominal strength to find the stress after prestressing.

The trial thickness was found as the product of length of the slab divided by 45. Once a trial slab thickness was found, the loads acting on the slabs were calculated. The load combination chosen for slab design was 1.2*dead load + 1.6*live load. This combination was chosen for all six floors, as it had the largest load. The 2\(^{nd}\), 3\(^{rd}\), 4\(^{th}\) And 5\(^{th}\) floors are subjected to the same loading, so they were uniformly designed. The 6\(^{th}\) floor was subject to a higher loading then those floors below it.

The next step performed was identifying the tendon profile. The strands per bay were found to be 31 stands per bay. The strands were reverse parabolic with high points at the columns, and low points in between the columns. The net loads were then calculated.

Figure 3: tendon layout for the 2nd through 5th floor slabs
Figure 4: east-west and north-south tendon profile for the 2nd floor respectfully. The tendon reaches a high point at the columns, and a low point at the midpoint of the slabs. The values are the depths of the tendons at columns and midpoints. The above tendon profile is uniformly distributed while the lower profile is banded.

The equivalent Frame characteristics were the next step in the design of the slab. The torsional stiffness of the slab at the column line, $K_c$ and $K_t$ were found, and the equivalent column stiffness $K_{eq}$ was found using those two values. This process was repeated for interior columns. Once both exterior and interior columns were done, slab stiffness was found. These values were important because the distribution factors for moment distribution were found using them with the formula $DF = K_s/\sum K$.

The Design service load Moments and stresses were computed next. Fixed end moments, or FEM, were found for both the interior and interior spans, using the $W_{net}$ from load balancing that was computed previously. The moment distribution of net load moments was calculated using the moment distribution method of analysis. That analysis method involves the fixed end moment being distributed and then carried over. This process was used multiple times to find moment distributions. In this instance, maximum net moment and midspan $f$, were found. These values were found at the support as well as at the midspan.
The fixed end moments were calculated for interior and exterior spans. The distribution factors found previously were used again, and the same moment distribution was performed. Once the final $M_{bal}$ was found, secondary moments were calculated for the spans. Secondary moments were found by subtracting $M_1$, which was load multiplied by the eccentricity, from the $M_{bal}$ found previously. A factored FEM$_u$ was also found.

The FEM$_u$ found were then subjected to moment distribution to find the final $M_u$. These moments were then used to calculate the design moments $M_u$. These $M_u$ values were found for all locations on the slab. The shear at each of the locations on the slab was also found. The shear was calculated as $W_uL/2 \pm$ the moment at the second column minus the moment at the first column divided by the length of the bay. Once the shear was found, required $M_n$ was found as the centerline moment minus the secondary moment. A table of the shears found for both the 2nd-5th floors and the 6th floor can be found below. The first letter given references the column at which the slab section is starting at, and the second letter references the column the slab is heading toward. Slab section AB starts at the first column, which is the north most, and continues toward the next column, column b.
Once the moments were found, the non prestressed reinforcement steel needed to be designed. Once the area of steel was found, area per foot was calculated. Once a trial steel amount was found, the available moments had to be checked against the moment loads found earlier. If the available load capacity was found to be higher, no additional reinforcing steel was needed. This calculation was performed for both midsection of spans and interior support sections.

Once these values were completed, a reinforcement summary was written. The reinforcement summary details the amount of steel, both reinforced and prestressed, that would be needed for the design. Banding in the column regions would also need to be taken into consideration. The steel was then banded around the column. These calculations were performed twice: Once for the 2\textsuperscript{nd} to 5\textsuperscript{th} floors, and once for the 6\textsuperscript{th} floor, which has different loading demands than the lower floors.

In order to check for strength, the nominal moment at both the positive and negative moment areas were compared to the ultimate moments. The ultimate moment is calculated as $WL^2/8$. The moments also need to be checked versus the ACI 318 allowable stresses. Negative moment stresses with the addition of nonprestressed reinforcement is limited to $6 \sqrt{f'c}$, and positive moment stresses is limited to $2\sqrt{f'c}$.

<table>
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<th>Factored Shear</th>
<th>Ms</th>
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<tbody>
<tr>
<td>ab</td>
<td>3649.8</td>
</tr>
<tr>
<td>ba</td>
<td>4855.2</td>
</tr>
<tr>
<td>bc</td>
<td>3240.0</td>
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</tbody>
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<table>
<thead>
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<th>Factored Shear</th>
<th>Ms</th>
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</thead>
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<td>4120.9</td>
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<tr>
<td>ba</td>
<td>5392.1</td>
</tr>
<tr>
<td>bc</td>
<td>3624.0</td>
</tr>
</tbody>
</table>
The flooring system also contains spandrel beams at the perimeter of the post tensioned slab. These beams are reinforced concrete. The beams are 5,000 psi and 18 by 24 inch rectangular beams. The first step in the beam design was to find the factored load and moment and shear demand.

Once the load and moment values were calculated, the capacity of the section was checked. The maximum nominal moment is calculated first, and knowing \(bd^2\), one can calculate \(\rho\) and determine the required steel.

Once the compression and tension steel is found, the available moment capacity is found. If the moment capacity is higher than the moment found in the beginning of the design, no further check for moment capacity is needed.

The design needed to be checked for shear capacity as well. The shear strength of the concrete \(V_c\) is calculated and compared to the maximum shear \(Vu\). If one half of the shear strength of the concrete is less than the ultimate shear, shear steel is needed. Two number 4 stirrups were placed with a spacing of 8 inches. Once the beam has been designed for shear and moment, deflection needs to be checked. If the deflection is less than the limit set by ACI 318, the beam is sufficiently designed.
Columns

The columns were the next members to be reviewed. The columns used in this design were not prestressed, but reinforced concrete. There were five different column designs used in this structure, according to the design drawings provided. Only five were used for constructability reasons. For the peer review, I checked three representative columns. These were one interior column, one column on the corner of the slabs, and one located at the end of a slab in the middle of the wall, not at either corner. These three different columns were sufficient to check the accuracy of the column design.
Column concrete strength was listed as 5,000 psi. Since columns support the entire building load above it, the first and second floor columns would have the highest loading, while columns on the 5th floor would be subject to much lower loading.
The first step towards review was finding the tributary area and loading. This was done simply by calculating the loads for each of the floors above the second, as well as the weight of the upper floor columns as well. The tributary area was found to be 31.5 X 30 feet for interior columns. With the loading found, the next step was to find the gross area of the columns, which was found using the formula $A_g = \frac{P_u}{\Phi \alpha \cdot (0.85f'c + pg \cdot (F_y - 0.85f'c))}$.

The columns had a $K$ value of 1.0. The story height was also found to be less than 12 times $h$, so slenderness could be neglected. The area of steel was found next. The next steel that needed to be found was the ties. There are two different types of tie used in column design: lateral ties and spiral ties. For the columns in this design, lateral ties were used. The tie spacing cannot exceed the least column dimension, 48 tie bar diameters or 16 longitudinal bar diameters, whichever value is lowest. There is also moment in the column, which means there will be shear that will also act on the stirrups.

Once the column size and steel are designed, P-M diagrams needed to be created. P-M diagrams are created using 5 points on the load vs. moment graph. P-M diagrams also display the eccentricity at these points.

The first point of an interaction diagram is found when you set $F_s$ equal to $F_y$. $\epsilon_u$ is assumed to be equal to .003 in this case. $\epsilon_y$ is found as the product of $F_u/E_s$, or the ultimate strength divided by the modulus of elasticity for the steel. The depth, $d$, is found as the length of a side of the column minus the cover minus the diameter of the steel tie and finally subtracting half the diameter of the reinforcing bar. $C$ can be found once these values have been found. Once the value of $C_b$ or $C$ balanced, has been found, $C$ is found. $C$ is equal to $.85f'c \cdot a \cdot h$. $C_b$ is the distance in the compression block where balanced failure happens. The nominal load $P_n$ and nominal moment $M_n$ are found after and plotted on the interaction diagram. The eccentricity is also marked on the interaction diagram, typically as a dashed line leading to the point.
The next point on the diagram is found in a very similar fashion. The only difference is a smaller value is chosen in place of \( C_b \). This means that the value is in the tension failure area of the interaction diagram. The same formulas are followed after this, and a new point can be added to the interaction diagram.

The third point on the diagram is found through a similar procedure. This time around, a \( C \) value higher than \( C_b \) was chosen, to see how the column reacted to compression failure. Since the column is made of concrete, it can be assumed that the values associated with compression failure would be higher than tension failure.

The fourth point is found when \( C \) is infinite. When \( C \) is infinite, the eccentricity will be 0. Under this condition, there will be no moment, only a load. This point is found on the Y axis of interaction diagrams. The final point is a pure moment load.

When an edge or side column is reviewed as opposed to an interior column, the tributary area will be lower. This means that the load becomes lower. This is counterbalanced however, with a much higher eccentricity. Another factor to take into account is constructability. It is much easier for a construction crew to create the same size column multiple times, instead of many different size columns. This helps account for the uniform nature of the columns found in this design.
Shear Walls

A shear wall is a member of a structure that is designed for the lateral load demands on the structure. These loads are typically wind and earthquakes. Similar to the columns in this structure, the shear walls are made of reinforced and not prestressed concrete. These walls work as a large cantilever beam jutting from the base of the building. This structure has three shear walls, one near the stairs, one near the elevator, and one in the north east corner of the structure. The review of shear walls was completed while using a journal article titled "Time Saving Design Aids for Reinforced Concrete."4

Because shear walls are subject to more diverse loading, a new load combination was needed. Once the new load combination was found, the loads due to wind and earthquake needed to be calculated. The shear walls were a foot thick and 5,000 psi concrete. A new floor starts every 10.5 feet.

The shear load on the shear walls was first checked at the first floor. This floor was subject to the largest loading and therefore would have the highest demand for a shear wall. First, the total shear was calculated. Then the nominal shear, $\phi V_c$ was found. Because the ultimate shear is higher than the nominal shear strength of the concrete, horizontal reinforcing steel was needed. The amount of required horizontal shear reinforcement is decided using table 6.5 from the guide referenced above. The required vertical reinforcement is then found. For the first floor, #6 bars at a spacing of 12 inches were calculated.

The second floor calculations were completed using the same methodology. First the horizontal shear reinforcement was found, followed by the vertical. The upper level floors were apparently given the same bar spacing as well, for the sake of constructability and simplicity. Though this simplifies the construction, it is a more costly option.

Shear walls resist overturning moment as well as shear forces. The overturning moment essentially is the force that attempts to destabilize the structure by lifting and overturning it. Shear walls need to be designed to withstand the moment that loading offers without overturning. In order to design the shear walls for overturning, moment load and axial load are calculated for each floor. The moment strength was then checked based on the required vertical shear reinforcement. The area of the steel, \( \omega \), \( \alpha \) and \( c/lw \) were found in order to calculate the available moment. Once the available moment, \( \phi M_n \) was found, it was compared to the ultimate moment. If the available moment was larger than the ultimate, no more reinforcement was needed for moment. This process was repeated for each floor.
**Structural Steel Flooring System**

In order to satisfy the capstone design requirement, the floor framing were redesigned as structural steel instead of post tensioned concrete. Structural steel offers advantages over concrete. The steel layout chosen is a beam and girder system with a metal deck and concrete poured onto the decking. The decking provides a mold for the concrete, and shear studs are welded to the deck to facilitate composite action. The beams and girders were both wide flanged, also referred to as W shape beams.

The steel scheme started like its concrete counterpart, with loads being calculated. Due to the smaller depth of the concrete, the dead load is much less for the steel scheme. The Scheme consists of 30 foot beams and 31.5 foot girders on the outer spans and 30 foot girders in the inner span. The beams were spaced at 10 foot intervals.
Figure 6: Beam and girder layout for the 5th and 6th floor respectfully. All beams are 30 feet long and are connected to girders on both ends. The numbers in parenthesis on the beams are the number of shear studs needed. Steel decking and 5 inch concrete slabs are to be placed atop this girder and beam layout.

The first calculation was a plastic capacity calculation for the beam. The ultimate moment due to loading was found initially. Then the moment was divided by $\phi_f$ to find the value of $Z_x$. Once $Z_x$ was found, a trial size for the beams could be selected. This selection was from table 3.2 from the AISC steel
construction manual. Once the trial beam size was found, a new dead load was calculated taking self-weight of the beam into account. The beam choice was checked by comparing the new allowable moment to the new ultimate moment due to loading. This process needed to be repeated until the allowable moment was higher than the moment due to loading.

Once a beam size is selected, it needs to be checked against deflection. The deflection of a beam when only dead load is considered has to be below length over 360, whichever is smaller. This ensures the flooring won’t be so slanted it causes issues. In order to calculate deflection, the moment of inertia $I$ needed to be obtained from table 3.3 of the AISC manual. The beam chosen for the design scheme was found to be sufficient, but if it was not a new beam would be selected for a higher moment of inertia from table 3.3. After that deflection case was tested, a second case involving full dead load and half live load was tested. Since this had a higher load, the allowable deflections were higher as well. The new limits used were length divided by 240 or 1.5 inches, whichever was smaller.

After the beam was checked for deflection limit state, it was time to find a girder size. The method for finding a girder size is the same as for beams. The girders will naturally be larger than beams, because girders have to support the flooring and the beams, as well as self-weight. The final beam and girder sizings for the 6th and 5th floors can be found in the table below.

Once the girder and beam sizes were established, the decking and concrete flooring were designed. The depth of the concrete slab was designed as 5 inches. The concrete was 4000 psi concrete. The shear studs along the decking were ¾ inch diameter studs.

A composite beam was calculated next. In order to find a new beam, the value of $B_e$ was found. $B_e$ is twice the value of the lesser of length over 8 and beam spacing over two. In this case $B_e$ was found to be 90 inches. The next value found was $Y_2$. $Y_2$ is the thickness of the concrete minus $a/2$. With these values, a new beam can be chosen from table 3-19 of the AISC steel manual. Once a new beam is
chosen, a and Y2 need to be recalculated. The shear capacity $\sum Q_n$ was then found for the beam, and with the shear capacity the moment capacity could be calculated. When compared to the moment load, it was found to be inadequate, so a new beam had to be chosen. The new beam was chosen for its $\sum Q_n$ value, so it was sufficient.

Figure 7: Steel decking before a concrete pour

After these calculations the deflection of the new beam needed to be checked. The process was the same as detailed earlier. Once the deflection is checked the shear studs needed to be calculated. The capacity per stud was calculated. Once the limiting stud capacity was found, the shear capacity was divided by the capacity per stud to find the number of studs needed.

The design needed to be checked for unshored construction loads next. This check makes sure the design won’t be too weak to support the forces construction will put on it. The loads taken into account were the weight of wet concrete, and the beam weight and construction live load. The construction live load was assumed to be roughly 20 psf. A load and moment were calculated for these weights, and they were tested against the allowable moment for the beam found in table 3.2. Deflection during unshored construction was also calculated.

The next step in the process was to design for full composite. Composite is the term for the concrete slab and steel decking and beams working as one member. The principles behind can be compared to the principles behind the positioning of prestressing tendons. The steel is located on the bottom of the slab to handle tension forces, while the concrete handles compression forces. Full composite is typically less advantageous then its partial counterpart, as it is more cost effective than full composite, due to less materials and less welding. Partial composite is similar to full composite, but uses less shear studs to accomplish the same uses. Both full and partial composite is more cost effective than a non-composite design.

The first step in full composite calculations is calculating a new shear capacity $\Sigma Q_n$. The capacity per shear stud remains the same, so the number of studs does not change dramatically. In order to check if shear composite is possible, the value $a$ needed to be calculated again. As long as “a “ was not larger than the thickness of the concrete, full composite was possible.

The next step in the design was to check how much composite capacity was needed. This is checked by using table 3-19. This table shows the moment values at different levels of the partial neutral axis. The further down the steel the partial neutral axis is, the more steel is in tension. Once the partial neutral axis was located, a deflection check was performed. Once the deflection was found to be sufficient, the number of shear studs needed was calculated. The capacity per stud remained the same,
but the shear capacity $\sum Q$, was dramatically lower, allowing for fewer shear studs to be needed. The shear studs for the 6th floor, for example, went from 21 to 7 studs. Once the shear studs were found, the beam design was completed. The girders were designed for the decking slab next. The girders were designed as described previously.

Once the interior beams and girders were designed, spandrel beams and girders were designed. The spandrel beams and girders were designed separately, because they have higher loading due to the shear walls. The beams and girders were designed in the same way as the interior beams and girders, but they had half the tributary area and a much higher dead load. The final floor design layouts can be found in figure 6 above.
Results and Conclusions

The Massachusetts College of Pharmacy and Health Sciences peer review reviews a six story post tensioned concrete building currently being erected in Worcester, Massachusetts. Souza, True and Partners is the structural engineering firm responsible for the design of the building. The building was designed for office space, lecture halls, and laboratories. This post tensioned structure was reviewed for slab, column, and shear wall design. The design checked for both gravity and lateral loading. The loads were all supplied by the design plans. All designs were performed according to Load and resistance factor design, as well as the American Concrete Institute manual ACI 318 08, and American institute of steel construction CAPS manual.

The floor slabs were the first aspects reviewed. The 2\textsuperscript{nd} through 5\textsuperscript{th} floor slabs were uniform, and exposed to the same loading. The 6\textsuperscript{th} floor had a higher live load and the first floor had higher loading as well. Due to this uniform loading, only the 6\textsuperscript{th} and 5\textsuperscript{th} floor slabs were reviewed. The slabs were found to be satisfactorily designed, and adequately resistant to both shear and moment.

The next aspect reviewed was the columns. These columns were comprised of reinforced concrete, as opposed to the post tensioned concrete used in other aspects of the building. The building only had 5 different configurations of columns, and only three were used frequently. One column type was used for interior columns, one for corner columns, and one for center exterior columns. The difference in columns can be attributed to a difference in the loading and tributary area. Though columns on the third floor had higher loads than columns on the fourth floor, the same sizes were used. This was mainly for ease of construction. Interaction diagrams were created for the columns.

The lateral loading for the building was handled by shear walls. These shear walls are effectively large cantilever beams, that take wind and earthquake loads with minimum displacement. These shear walls supply the rigidity for the structure.
An alternate design for the floor slabs using structural steel beams and girders was completed as well. The floor deck itself was a metal decking with a 5 inch thick concrete slab. The floor slab was checked for unshored construction, as well as full and partial composite action. The slab design was checked for deflection as well. The dead loading for this scheme was much lower than the post tensioned slab. This is due to the high weight of concrete as opposed to steel.

The superstructure peer review of MCPHS was a culmination of the design courses taken over the past few years. The building materials used included post tensioned concrete, reinforced concrete, and steel design. All aspects that were reviewed were found to be satisfactory. In some cases, such as the columns in the upper levels, the building is over designed for ease of construction and repeatability. Though not all aspects were checked, a sufficient amount of the design was investigated to assume that the engineer of record was competent and correct in his design process.
Appendix

2nd Floor slab design excel sheet

2nd floor Floor slab.
Givens
Concrete Steel Dimensions
F'c 5000 PSI As 0.153 in^2 93 X 90
F'ci 3333.33 PSI Es, Eps 0 Psi
Fy 60000 PSI Fpu 270000

Lump Sum Losses 33000 Psi
Friction is not included
Assume a y of roughly 9 inches, because it cannot be found without knowing slab thickness.
K=
prestress loss due to friction= 52447.5
total prestress loss= 85448
fps=.7*fpu 189000 Psi
Fpe 103553 Psi
Fpy 240000 Psi

Max fc due to stresses=.45f'c 2250

Trial slab thickness
h= (height X 12)/2*(1/45)
h= 8 inches
Ac= 96 in^2 per square foot

Loads
Corridors 80 psf
Elevator Machine Rooms 150 psf
Ceilings 5 psf
DLS
Service 10 psf
Ceilings 5 psf
Self-Weight 96.66 psf

Total DL 111.66
Total LL 85
Total 196.66
Load

\[ W_u = 1.2 \text{dead} + 1.6 \text{live} \]

\[ W_u = 270 \text{ Psf} \]
\[ L_n = 31 \]
\[ L (e-w) = 30 \text{ Ft} \]
\[ P_e \text{ per strand} = 15843 \text{ lb} \]

Assume a Fc of 170

\[ F = 16320 \]
\[ P_e \text{ per strand} = 15843 \]
\[ F_e = F \times L = 489600 \]

Number of strands = \( F_e / P_e \) = 30.9022 strands

\[ F_e = F \times P_e \times \text{number of strands} = 491149 \]
\[ F = F_e \times L = 16371.65 \]
\[ f_c = F / A_c = 170.54 \]
\[ f_c \text{ allow} = 141.42 \]

Definitely not acceptable \( f_c \), so a new thickness needs to be picked.

Try a thickness of 10"

\[ \text{new } h = 10 \text{ inches} \]
\[ A_c = 120 \text{ in}^2 \]

New Fc = 136.43

Assumed starting tendon depth = 5 in.
Assumed midpoint tendon depth = 9 in.
Assumed tendon highpoint depth = 1 in.
\[ a_1 = a_3 = 6 \]
outside spans

\[ W_{bal} = 8F_a / L_n^2 = 68.14 \text{ psf} \]
\[ w_{net} = W_w - W_{bal} = 128.52 \]

Interior Span
\[ a_2 = \text{low point} - \text{high point} = 8 \text{ inches} \]

\[ W_{bal} = 8F_a / L^2 = 97.02 \]
\[ w_{net} = W_w - W_{bal} = 99.65 \]

Equivalent frame characteristics

\[ K_s = 4E_c I_c / (L_n - 2h) \]
where \( L_n = 180 \)

all columns are 24x24
\[ c_1 = c_2 = 24 \text{ inches} \]
\[ I_c = 27648 \]
Assume \( ec/es = 1 \)
total \( K_c = 4E_c I_c / (L_n - 2h) = 691.2 \)
From equation 9.10b \( C = (1 - 0.63x/y)(x^3y/3) \)

\( x = 10 \) inches \( y = 24 \) inches

\( C = 5900 \)

torsional stiffness of the slab at the column line

\( K_t = \text{sum}(9EcsC/L^2(1-C^2/c1)) \)

\( K_t = 2177.02 \)

\( K_{ec} = (1/K_c + 1/K_t)^{-1} \)

\( K_{ec} = 524.63 \)

Slab stiffness

\( K_s = 4EcIc/(Ln - C1/2) \) for interior Columns

307.2

\( K_s = 4EcIc/(Ln - C1/2) \) For exterior Columns

317.7931

DF for \( a = K_s/\text{sum}(K) \)

DF for outer joint A slab \( 0.36 \)

DF for left joint B slab \( 0.267 \)

DF for right joint B slab \( 0.267 \)

DF for left joint C slab \( 0.270 \)

**Work Load Check**

Fixed end moment for exterior spans

\( FEM = WL^2/12 \)

Length for exterior spans \( 31.5 \)

\( FEM = 127526.4 \)

Fixed end moment for interior spans

\( FEM = WL^2/12 \)

Length for interior spans \( 30 \)

\( FEM = 89684.5 \)

\( COF = 0.5 \)

\[
\begin{array}{ccc}
\text{A} & \text{B} & \text{C} \\
0.26721 & 0.26721 & 0.26721 \\
0.364662 & 8 & 8 \\
0.5 & 0.5 & 0.5 \\
127.526 & 89.6845 & 127.526 \\
10^3 \text{in-lb} & 4 & 3 & 89.684532 & 4 \\
\end{array}
\]

\( \text{DIST} \)

34.0773 34.0773

23.9653 24.188185

12.0940 11.982652

\( \text{CO} \)

6.21335261 3 7 3.231757

\( \text{DIST} \)

-70.197053 178.642

-81.0451

41
*10^3 per ft

Vc is equal to the lowest value of the next three equations according to figure 9-11

\[ Vc = (2+4/\beta) \sqrt{f'c} b \]  
\[ Vc^b = 1 \]  
\[ Vc = (\alpha D/b_0 + 2) \sqrt{f'c} b \]  
\[ \beta = 1 \]  
\[ b_0 = 30 \]  
\[ d = 5 \]  
\[ \alpha = 40 \]  
\[ \text{interior} \]  
\[ Vc = 4 \sqrt{f'c} b \]  
\[ \text{exterior} \]  
\[ Vc = 42426.4 \]

Tensile strength at support

\[ M_{net} = M_{net} - Vc/b \]  
\[ M_{net} = 56054.9175 \]  
\[ b = 12 \text{ inches} \]  
\[ \text{height} = 10 \text{ inches} \]  
\[ S = bh^2/6 \]  
\[ S = 200 \]  
\[ ft = P/A-M/s = 143.84 \]  
\[ \text{Allowable ft} = 6 \sqrt{f'c} \]  
\[ 424.26 \]

Tensile Strength at midpoint

\[ M_{net,max} = WL^2/8 - FEM \]  
\[ M_{net,max} = 53481 \]  
\[ \text{midspan} \]  
\[ ft = 130.97 \]

Design Moments Mu

\[ FEM_{bal} = W_{bal} L^2/12 \]  
\[ \text{Span AB or CD} \]  
\[ FEM_{bal} = 67616.1 \]  
\[ \text{Span BC} \]  
\[ FEM_{bal} = 87315.47 \]

\[
\begin{array}{ccc|ccc|ccc}
A & B & C \\
\hline
0.26721 & 0.26721 & 0.26721 & 0.26721 & 0.26721 & 0.26721 & 0.26721 & 0.26721 & 0.26721 \\
0.364662 & 8 & 8 & 0.269703 & 8 & 0.26721 & 0.26721 & 0.26721 & 0.26721 \\
0.5 & 0.5 & 0.5 & 0.5 & 0.5 & 0.5 & 0.5 & 0.5 & 0.5 \\
67.6161081 & 1 & 7 & 87.315468 & 1 & 7 & 87.315468 & 1 & 7 \\
67.6161081 & 1 & 7 & 87.315468 & 1 & 7 & 87.315468 & 1 & 7 \\
\end{array}
\]

\[
\begin{array}{ccc}
A & B & C \\
\hline
D & 18.0682 & 23.3322 & 18.0682 \\
24.6570253 & 3 & 5 & 23.549241 & 3 \\
12.3285 & 11.7746 & 11.666124 \\
\end{array}
\]
\[
\begin{array}{ccc}
\text{DIST} & 3.29439824 & 3.14638843 \\
\end{array}
\]

Final Mnet *10^3 per ft
-37.219367 94.71844 -78.9042

Span AB
\( e = 0 \)
Mbal = 37219.36
Ms = 37219.36
FEMu = WuL^2/12
FEMu = 267907.5

Span BA
\( e = 4 \)
M1 = PeE
M1 = 65486.6
Mbal = 94718.45
Ms = 29231.85

Span BC
\( e = 4 \)
M1 = 65486.6
Mbal = 78904.2
Ms = 13417.6
FEMu = 243000

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
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</thead>
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<tr>
<td>DF</td>
<td>0.364662</td>
<td>0.26721</td>
</tr>
<tr>
<td>COF</td>
<td>0.5</td>
<td>8</td>
</tr>
<tr>
<td>FEM</td>
<td>267.9075</td>
<td>0.269703</td>
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</table>

FEM X10^3 in-lb
71.5896 64.9339 71.5896

DIST 97.6956852 6 3 65.537822 6
48.8478 32.7689

CO 35.7948287 4 1 32.466965
13.0530 8.75643

DIST 13.0530139 1 7 8.7564369

Final Mu *10^3 per ft
-147.47 375.292 -219.591
Vab=WuL/2-(Mu@b-Mu@a)/2
4252.5 - 602.7
Vab= 3649.79633

C= 24
Centerline Mu=Mu-Ms
110250.6

Req. column face Mu
81052.26

Req. Mn= 90058.0697

Joint B (BA) moment
Vba= 4855.20367

C= 24
Mu= 346060.14

Required column face Mu
307218.5

Required Mn
341353.9

Joint B (BC)

Vbc=w*24/2
3240
Mu= 206173.79

Req. Mu 180253.79

Req. Mn 200281.99

Factored Shear Ms
ab 3649.79 37219.3
ba 4855.20367 29231.8
bc 3240 13417.6

Maximum positive moment Span AB

x=Vab/Wu

12 ft.

Max positive Mu=VabX-WuX^2/2-Mu+ms
Mu= 66709.45

Req. positive Mn= 74121.6

Maximum positive moment span BC

Mu=Vbc*Ln/2-(Wu-L/2)^2*(L/4)
Mu= 317700

Mn= 353000
Flexural Strength Mn

As=0.00075HLn

\[ 2.79 \text{ inches}^2 \]

try #4

bars area= 0.196

14.21 needs 15 bars at .3 inches a piece

As= 2.95

30 Ft panel 30 ft
As per foot= 0.098 in^2

Pp=Aps/bd

0.00146

fps=Fpe+F'c/300Pp+10,000psi

124937.7

Fps=fps*Aps*strand number/L

19752.65

Fs=60,000*As/ft

5890.5

total force F/ft= 25643

a=AsFy+ApsFps/(.85*f'c*b)

0.502806

6 inches

Bars and tendons should be placed at 12'-'1', or 11 inch depth

11 inches

available Mn= 275627
required Mn= 200282
so no more moment strength is needed

a=Apsfps/.85f'cb

0.11

available -Mn=Apsfps(d-a/2)

negative Mn= 216138.4

This is less than the required positive Mn, so it is unsatisfactory

try adding a #5 bar

As= 0.31

Asfy= 18407.8

a= 0.75

available +Mn

405487.9

satisfactory

Add a number 5 bar at the bottom fiber, and number 4 bars at the top fiber

Width of column strip

180 inches

assume 70% of strands are banded
.7*31  
21.7 
22 banded, 9 in the middle strip  
22  
9  
column strip moment factor =  0.71  
Middle strip moment factor=  0.29  
Max total -M at column face B=  178642
Max total +M at midspan = 73080.9
6th floor slab design excel spreadsheet

6th floor Floor slab.

Givens

Concrete Steel Dimensions

\[ F'c \] 5000 PSI As 0.153 in^2 93 X 90

\[ F'ci \] 3333.3 PSI Es, Eps 285000000 Psi

\[ Fy \] 60000 PSI Fpu 270000

Lump Sum Losses 33000 Psi

Friction is not included

Assume a y of roughly 9 inches, because it cannot be found without knowing slab thickness.

\[ K = \frac{2.4 \mu}{\alpha} \]

\[ \text{prestress loss due to friction} = 52447.5 \]

\[ \text{total prestress loss} = 85448 \]

\[ \text{fps} = 0.7 \text{fpu} \]

\[ Fpe = 103553 \] Psi

\[ Fpy = 240000 \] Psi

Max fc due to stresses = 0.45f\( c \)

2250

Trial slab thickness

\[ h = \frac{\text{height} \times 12}{2} \times \frac{1}{45} \]

Use 93, because it will give the higher, and therefore more likely value

\[ h = 8 \text{ inches} \]

\[ Ac = 96 \text{ in}^2 \text{ per square foot} \]

Loads

6th floor live load 100 psf

Elevator Machine Rooms 150 psf

Ceilings 5 psf

DLS Service 10 psf

Ceilings 5 psf

Self-Weight 96.6 psf

Total DL 111.6

Total LL 105

Total Load 216.6

\[ Wu = 1.2 \text{dead} + 1.6 \text{ live} \]

\[ Wu = 302 \] Psf

\[ Ln = 31 \]
L (e-w)= 30 Ft
Pe per strand= 15843.53 lb
Assume a Fc of 170
unit F= 16320
Pe per strand= 15843.53
Fe=F*L 489600
Number of strands = Fe/Pe 30.9 31 strands
Pe=Fe=Pe* number of strands 491149.5
F=Fe*L 16371.65
fc=F/Ac 170.54
fc allow= 141.42
Definitely not acceptable fc, so a new thickness needs to be picked.
Try a thickness of 10"
new h= 12 inches
Ac= 144 in^2
New Fc= 113.69
Assumed starting tendon depth 6 in.
Assumed midpoint tendon depth 11 in.
Assumed tendon highpoint depth 1 in.
a1=a3= 7.5
outside spans
Wbal=8Fa/Ln^2 85.18 psf
wnet=Ww-Wbal 131.49
Interior Span
a2=low point - high point 10 inches
Wbal=8Fa/L^2 113.57
wnet=Ww-Wbal 103.09
Equivalent frame characteristics
Ks=4Eclc/(Ln-2h) where Ln=lu= 180 inches
ls=bh^3 51840
all columns are 24x24 c1=c2= 24 inches
lc= 27648
Assume ec/es=1
total Kc=4Eclc/(Ln-2h)
708.92308

From equation 9.10b C=(1-.63x/y)(x^3y/3
x= 12 inches y= 24 inches
C= 9469.44
torsional stiffness of the slab at the column line
Kt=sum(9EcsC/L2(1-C2/c1))
Kt = 3494.1
Kec = (1/Kc + 1/Kt)^-1
Kec = 589.35
Slab stiffness
Ks = 4EcIc/(Ln-C1/2) for interior Columns
307.2
Ks = 4EcIc/(Ln-C1/2) for exterior Columns
317.7931
DF for a = Ks/sum(K)
DF for outer joint A slab = 0.33
DF for left joint B slab = 0.25
DF for right joint B slab = 0.25
FDF for left joint C slab = 0.25

Work Load Check

Fixed end moment for exterior spans
FEM = WL^2/12
FEM = 130467.36
Length for exterior spans = 31.5
Fixed end moment for interior spans
FEM = WL^2/12
FEM = 92783.66
Length for interior spans = 30
COF = 0.5

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF</td>
<td>0.3386459</td>
<td>0.252976</td>
<td>0.252976</td>
</tr>
<tr>
<td>COF</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
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<tr>
<td>FEM</td>
<td>130.467365</td>
<td>130.4674</td>
<td>92.78366</td>
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<tr>
<td>X10^3 in-lb</td>
<td>92.783661</td>
<td>130.4674</td>
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DIST 44.1822382 33.00517 23.47208 23.678639 33.00517
CO 16.502586 22.09112 11.83932 11.736041
DIST 5.58853309 5.588533 2.995069 2.9950691

Final Mnet *10^3 per ft
-75.371074 179.9751 -84.146

Vc is equal to the lowest value of the next three equations

Vc = (2+4/beta)sqrt(f'c)bod
Vc = bo = 30
Vc = interior

Vc = (alphasD/bo +2)sqrt(f'c)bod
d = 6
Vc = alpha = 40
Vc = exterior

Vc = 4sqrt(f'c)bod
Vc = 50911.69

Slab concrete tensile stress at support
Tensile strength at support

\[
M_{net} = M_{net} - \frac{Vc}{3} \\
M_{net} = 58400.51 \\
b = 12 \text{ inches} \\
h = 288 \\
ft = -\frac{P}{A-M/s} = 89.09 \\
\text{Allowable ft} = 6\sqrt{f'(c)} = 424.26
\]

Tensile Strength at midpoint

\[
M_{net,\text{max}} = WL^2/8 - FEM \\
M_{net,\text{max}} = 55029.524 \\
\text{midspan} \\
ft = 77.38
\]

Design Moments Mu

\[
FEM_{bal} = W_{bal}L^2/12 \\
\text{Span AB or CD} \\
FEM_{bal} = 84520.1351
\]

Span BC

\[
FEM_{bal} = 102216.339
\]

<table>
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</thead>
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<td>0.252976</td>
</tr>
<tr>
<td>COF</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>FEMX10^3 in-lb</td>
<td>84.5201351</td>
<td>84.52014</td>
<td>102.2163</td>
</tr>
<tr>
<td>DIST</td>
<td>28.6223972</td>
<td>21.3816</td>
<td>25.85833</td>
</tr>
<tr>
<td>CO</td>
<td>10.6908023</td>
<td>14.3112</td>
<td>13.04294</td>
</tr>
<tr>
<td>DIST</td>
<td>3.62039635</td>
<td>3.620396</td>
<td>3.299557</td>
</tr>
<tr>
<td>Final Mnet*10^3 per ft</td>
<td>-48.827332</td>
<td>116.5925</td>
<td>-92.7005</td>
</tr>
</tbody>
</table>

Span AB

\[
e = 0 \\
M_{bal} = 48827.33 \\
Ms = 48827.33 \\
FEMu = WU^2/12 \\
FEMu = 299659.5 \\
\text{Span BA} \\
e = 5
M1=PeE
M1= 81858.25
Mbal= 116592.54
Ms= 34734.29
Span BC e= 5
M1= 81858.2513
Mbal= 92700.51
Ms= 10842.2587
FEMu= 271800

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<tr>
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<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>FEM</td>
<td>299.6595</td>
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</tr>
</tbody>
</table>

X10^3 in-lb

| DIST | 101.478461 | 75.8068   | 68.759   | 69.364087 | 75.8068 |
| CO   | 37.9033996 | 50.73923  | 34.68204 | 34.379501 |
| DIST | 12.8358309 | 12.83583  | 8.773741 | 8.7737406 |

Final Mu *10^3 per ft

| -173.11347 | 413.3697 | -246.497 |

Vab=WuL/2-(Mu@b-Mu@a)/2

\[ 4756.5 \quad - \quad 635.6 \]

Vab= 4120.9

c= 24

Centerline Mu=Mu-Ms
124286.14

Req. column face Mu
91318.926

req. Mn= 101465.473

Joint B (BA) moment
Vba= 5392.1

c= 24

Mu= 378635.4

Required column face Mu
335498.62

Required Mn
372776.24
Joint B (BC)

\[ V_{bc} = \frac{w \times 24}{2} \]

3624

\[ \mu = 235654.524 \]
\[ \text{Req. } \mu = 206662.524 \]
\[ \text{Req. } \mu n = 229625.026 \]

Factored Shear

<table>
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<tr>
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<tbody>
<tr>
<td>ab</td>
<td>4120.90151</td>
</tr>
<tr>
<td>ba</td>
<td>5392.09849</td>
</tr>
<tr>
<td>bc</td>
<td>3624</td>
</tr>
</tbody>
</table>

Maximum positive moment Span AB

\[ x = \frac{V_{ab}}{W_u} \]

12 ft.

Max positive \( \mu = V_{ab}X - W_uX^2/2 - \mu - +ms \)

\[ \mu = 82505.0366 \]
\[ \text{Req. positive } \mu n = 91672.26 \]

Maximum positive moment span BC

\[ \mu = V_{bc} \times \frac{L_n}{2} - (W_u - L/2) \times (L/4) \]

\[ \mu = 359820 \]
\[ \mu n = 399800 \]

Flexural Strength \( \mu n \)

\[ A_s = 0.00075H L_n \]

3.348 inches ^2

try #4

bars area = 0.196
17.051224 needs 18 bars at .3 inches a piece

\[ A_s = 3.53 \]

30 Ft panel 30 ft

As per foot = 0.118 in ^2

\[ P_p = A_{ps}/b_d \]

0.0012

\[ f_{ps} = F_{pe} + F'c/300P_p + 10,000\text{psi} \]

127467.74

\[ F_{ps} = f_{ps} \times A_{ps} \times \text{strand number}/L \]

20152.65

\[ F_s = 60,000 \times A_s/ft \]

7068.5835

\[ \text{total force } F/ft = 27221.23 \]

\[ a = A_s F_y + A_{ps} F_{ps}/(0.85 * f'c * b) \]

0.53 inches
Bars and tendons should be placed at 12'-1', or 11 inch depth

available Mn= 292168.9
required Mn= 229625
so no more moment strength is needed

\[ a = \frac{A_{psfps}}{0.85f'c_b} \]
\[ 0.14 \]

available -Mn = A_{psfps}(d-a/2)

negative Mn= 220282.58

This is less than the required positive Mn, so it is unsatisfactory

try adding a #5 bar

\[ A_s = 0.31 \]
\[ A_{sfy} = 18407.7 \]
\[ a = 0.75608666 \]

available +Mn 409587.11

satisfactory

Add a number 5 bar at the bottom fiber, and number 4 bars at the top fiber

Width of column strip 180 inches

assume 70% of strands are banded

\[ 0.7 \times 31 = 21.7 \]

22 banded, 9 in the middle strip

\[ 22 - 9 \]

column strip moment factor = 0.709677

Middle strip moment factor = 0.290323

Max total -M at column face B = 179975.1

Max total +M at midspan = 73626.19
Second Floor Interior column Excel Spreadsheet

<table>
<thead>
<tr>
<th>Column Design</th>
<th>2nd floor interior</th>
</tr>
</thead>
<tbody>
<tr>
<td>length</td>
<td>10.5 Ft</td>
</tr>
<tr>
<td>f’c</td>
<td>5000 psi(Assumed)</td>
</tr>
<tr>
<td>F’ci</td>
<td>3333.333 PSI</td>
</tr>
<tr>
<td>Fy</td>
<td>60000 PSI</td>
</tr>
<tr>
<td>fps</td>
<td>189000 Psi</td>
</tr>
<tr>
<td>As</td>
<td>0.153 in^2</td>
</tr>
<tr>
<td>Phi=</td>
<td>2.85E+08 Psi</td>
</tr>
<tr>
<td>alpha=</td>
<td>270000</td>
</tr>
<tr>
<td>pg=</td>
<td>60000</td>
</tr>
</tbody>
</table>

Effective area of support

- 31 ft by 30 ft: 930 ft^2
- Roof load= 466.3 psf
- 6th floor load= 358 psf
- 6th floor column weight= 6300 lbs
- 5th floor load= 296 psf
- Column weight= 6300 lbs
- 4th floor load= 296 psf
- Column weight= 6300 lbs
- 3rd floor load= 296 psf
- Column weight= 6300 lbs

\[ \text{total } Pu = 1617639 \text{ lbs} \]
\[ = 1617.6 \text{ kips} \]

Trial size
\[ Ag = \frac{Pu}{\Phi \times \alpha \times (0.85f’c + pg(Fy - 0.85f’c))} \]
\[ Ag = 525.26 \text{ in}^2 \]
\[ 22.9 \]
\[ 576 \]

assume a trial size of 24 X 24, or 576 in^2
\[ Ag = 576 \text{ in}^2 \]

trial h= 24
Short or slender
\[ l/h = 5.25 \]
short column
assume a fixed fixed column, so K=1.0

\[ > \text{story height, so slenderness can be neglected. (according to PDC handout)} \]

12h= 288
As = pg * area of the column  
17.28 in^2
1.44

diameter of steel = 1.41
Nominal Area = 1.56

use 12 #11 bars
As = 18.72

Check using \[ \Phi P_n = 0.8 * \Phi \left[ 0.85 * f'c (A_g - A_{st}) + f_y A_{st} \right] \]
\[ 0.8 * 0.65 * (0.85 * 5000 * (576 - 18.72) + (60 * 18.72)) \]
\[ \Phi P_n = 1815.6 \text{ Kips} > 1617.6 \text{ so the section checks} \]

use #4 lateral ties.
diameter ties = 0.5 in
16 long. Diameters
22.56
48 tie bar diameters
24

Least Column dimension
25
tie spacing cannot exceed 22.56 in.

Clear spacing
7.885
use cross ties, because spacing is greater than 6 inches

<table>
<thead>
<tr>
<th>P-M diagrams</th>
<th>(Fs=Fy)</th>
</tr>
</thead>
<tbody>
<tr>
<td>assume ( \varepsilon_u = 0.003 )</td>
<td>( \varepsilon_y = \frac{F_u}{E_s} = 0.0021053 )</td>
</tr>
</tbody>
</table>

\[ d = h - \text{cover - diameter of tie-1/2} \text{ diameter of reinforcing bar} \]
\[ d' = 22.795 \text{ in} \]
\[ C_b = 13.395 \text{ in} \]

depth a = 0.85 * Cb  
11.4 in
f's = \( \varepsilon_u * E_s (C_b - d') / C_b \)  
77.8 \( \leq \) 60 ksi

C = 0.85 * f'c * a * h  
1161347 lbs  
As = 4
1161.347 kips  
A's = 4

\[ P_n = 0.85 * f'c * a * b + A_s f_s - A's f's \]
\[ P_n = 1161.347 \]

\[ M_n = P_n * e = 0.85 * f'c * a * b (H/2 - a/2) + a' s f_s (h/2 - d') + A_s f_s (d-h/2) \]
12506.36 kip inches
1042.196  kip feet

\[
e = 10.76 \text{ Inches}
\]

Choose a C smaller than the previous C

\[
Cb = 5 \text{ In}
\]

\[
depth a = 0.85 \times Cb \quad 4.25 \text{ in}
\]

\[
f's = \varepsilon u * E_s (C_b - d') / C_b
\]

\[
65 \quad \leq \quad 60 \text{ ksi}
\]

\[
C = 0.85 f'c * a * h
\]

\[
94828.13 \text{ lbs} \quad \text{As} = 3
\]

\[
94.82 \text{ kips} \quad \text{A's} = 3
\]

\[
P_n = 0.85 f'c * a * b + A's F_s - A's F's
\]

\[
P_n = 433.5
\]

\[
M_n = P_n * e = 0.85 f'c * a * b (H / 2 - a / 2) + a's f's (h / 2 - d') + A's F_s (d / 2 - h / 2)
\]

\[
920.81 \text{ kip inches}
\]

\[
76.73 \text{ kip feet}
\]

\[
e = 2.124135 \text{ Inches}
\]

Choose a C larger than the previous C

\[
Cb = 24 \text{ In}
\]

\[
depth a = 0.85 \times Cb \quad 20.4 \text{ in}
\]

\[
f_s = \varepsilon u * E_s (d' - C) / C
\]

\[
-4.29281
\]

\[
4.292812
\]

\[
f's = \varepsilon u * E_s (C_b - d') / C_b
\]

\[
56.9875 \quad \leq \quad 60 \text{ ksi}
\]

\[
C = 0.85 f'c * a * h
\]

\[
2080800 \text{ lbs} \quad \text{As} = 3
\]

\[
2080.8 \text{ kips} \quad \text{A's} = 3
\]

\[
P_n = 0.85 f'c * a * b + A's F_s - A's F's
\]

\[
P_n = 2291.579
\]

\[
M_n = P_n * e = 0.85 f'c * a * b (H / 2 - a / 2) + a's f's (h / 2 - d') + A's F_s (d / 2 - h / 2)
\]

\[
6686.895 \text{ kip inches}
\]

\[
557.2413 \text{ kip feet}
\]

\[
e = 2.91803 \text{ Inches}
\]

Set C to infinite and e=0

\[
F_s = 60 \text{ ksi}
\]

\[
P_n = 0.85 f'c * a * b + A's F_s
\]

\[
P_n = 3168
\]
# Second Floor Exterior Middle Column Excel Spreadsheet

Column Design
- 2nd floor exterior centered column
- **length**: 10.5 ft
- **f'c**: 5000 psi (assumed)

<table>
<thead>
<tr>
<th><strong>As</strong></th>
<th><strong>0.153 in^2</strong></th>
<th><strong>Phi</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>3333.33</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **F'ci**: 3 psi
- **Es, Eps**: 2.85E+08 psi
- **alpha**:  
- **Fy**: 60000 psi
- **Fpu**: 270000 psi
- **fps=.7*fpu**: 189000 psi

Effective area of support
- Assume a value of 500 pounds per linear foot per floor for outside façade
- **17.5 ft by 30 ft**: 525 ft^2
- **Roof load**: 466.3 psf
- **6th floor load**: 358 psf
- **6th floor column weight**: 2275 lbs
- **5th floor load**: 296 psf
- **Column weight**: 6300 lbs
- **4th floor load**: 296 psf
- **Column weight**: 6300 lbs
- **3rd floor load**: 296 psf
- **Column weight**: 6300 lbs

**total Pu**
- 980132.5 lbs
- 980.1325 kips

**Trial size**
- **Ag=Pu/Φ*α*(.85f'c+pg(Fy-.85f'c))**
- 318.3 in^2
- 17.8
- 324

Assume a trial size of 24 X 24, or 576 in^2

- **Ag**: 324 in^2
- **trial h**: 18
- **Short or slender**: 7
- **short column**: Assume a fixed fixed column, so K=1.0
12h = 216 > story height, so slenderness can be neglected. (according to PDC handout)

\[ \text{As} = \text{pg} \times \text{area of the column} \]

\[ 9.72 \text{ in}^2 \]

\[ 0.81 \]

diameter of steel = 1.27

Nominal Area = 1.27

use 8 #10 bars

\[ \text{As} = 10.16 \]

Check using \( \Phi P_n = 0.8 \Phi \{0.65 \times 0.85 \times f'(Ag - Ast) + fyAst\} \)

\[ 0.8 \times 0.65 \times (0.85 \times 5000 \times (576 - 18.72) + (60 \times 18.72)) \]

\[ \Phi P_n = 1010.5 \text{ Kips} > 980.1325 \] so the section checks

use #3 lateral ties.

\[ \text{diameter ties} = 0.375 \text{ in} \]

16 long. Diameters

20.32

48 tie bar
diameters

18

Least Column dimension

18
tie spacing cannot exceed 18 in.

Clear spacing

8.095 in

use cross ties, because spacing is greater than 6 inches

**P-M diagrams \((F_s=F_y)\)**

 assume \( \varepsilon_u = 0.003 \)

\[ \varepsilon_y = \frac{F_u}{E} = 0.002105 \]

\[ s = 3 \]

d = h - cover - diameter of tie - 1/2 diameter of reinforcing bar

d' = 16.99 in

Cb = 9.98 in

\[ \text{depth } a = 0.85 \times Cb = 8.486242 \text{ in} \]

\[ f's = \varepsilon u \times E \times (C_b - d') / C_b \]

\[ 25.4673 \leq 60 \text{ ksi} \]

C = 0.85f'c*a*h

\[ 649197 \text{ lbs} \]

\[ \text{As} = 4 \]
649.2  kips       A's=4
Pn=.85*f'c*a*b+AsFs-A'sF's
Pn=  545.6
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
3596.5  kip inches
299.7  kip feet
e=  6.59  Inches

Choose a C smaller than the previous C
Cb= 8  In

depth a=.85*Cb  6.8  in
f's= εu*Es(Cb-d')/Cb
10.58  ≤  60  ksi
C=.85f'c*a*h
202300  lbs
202.3  kips

Choose a C larger than the previous C
Cb= 18  In

depth a=.85*Cb  15.3  in
fs= εu*Es(d'-C)/C
-4.7975  4.7975
f's= εu*Es(Cb-d')/Cb
36.6333  ≤  60  ksi
C=.85f'c*a*h
117045
0  lbs
1170.45  kips

60
193.5 kip feet
e= 1.83492 inches

Set C to infinite and e=0

\[
\begin{align*}
FS &= 60 \text{ ksi} \\
PN &= 0.85f'c*a*b + AsFs \\
PN &= 1857
\end{align*}
\]
### 2nd Floor Edge

<table>
<thead>
<tr>
<th>Column Design</th>
<th>2nd floor edge</th>
</tr>
</thead>
<tbody>
<tr>
<td>length</td>
<td>10.5 ft</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>5000 psi(Assumed)</td>
</tr>
</tbody>
</table>

**As** $= 0.153 \text{in}^2$, **$\Phi$** $= 0.65$

- $f'_{ci} = 3333.33 \text{PSI}$
- $E_s, E_p = 2.85 \times 10^8 \text{Psi}$
- $\alpha = 0.8$
- $F_y = 60000 \text{PSI}$
- $F_{pu} = 270000 \text{PSI}$
- $\beta = 1.2$
- $f_{ps} = 0.7 \times F_{pu} = 189000 \text{Psi}$

#### Effective area of support

- 17.5 ft by 16.75 ft $= 293.125 \text{ft}^2$
- Roof load $= 466.3 \text{psf}$
- 6th floor load $= 358 \text{psf}$
- 6th floor column weight $= 2275 \text{lbs}$
- 5th floor load $= 296 \text{psf}$
- Column weight $= 6300 \text{lbs}$
- 4th floor load $= 296 \text{psf}$
- Column weight $= 6300 \text{lbs}$
- 3rd floor load $= 296 \text{psf}$
- Column weight $= 6300 \text{lbs}$

**Total Pu** $= 591592.4 \text{lbs}$

**591.5924 kips**

#### Trial size

$$Ag = \frac{Pu}{\Phi \times \alpha \times (0.85f'_c + pg(F_y - 0.85f'_c))}$$

- $Ag = 3 \text{in}^2$, $13.85$, $196$
- Assume a trial size of 24 X 24, or 576 in$^2$
- Trial $h = 14$
- Short or slender
- $l/h = 9$
- Short column
assume a fixed fixed column, so $K=1.0$

$12h=168 >$ story height, so slenderness can be neglected. (according to PDC handout)

$A_s = pg \cdot \text{area of the column}$

$5.88 \text{ in}^2$

$0.49$

diameter of steel = 1.41

Nominal Area = 1.56

use 4 #4 bars

$A_s = 6.24$

Check using $\Phi_{Pn} = 0.8 \cdot \Phi [0.85 \cdot f'_c (A_g - A_{st}) + f_y A_{st}]$

$\Phi_{Pn} = 614.056 \text{ Kips} > 591.5929$ so the section checks

use #3 lateral ties.

diameter ties = 0.375 in

16 long. Diameters

22.56

48 tie bar diameters

18

Least Column dimension

14

tie spacing cannot exceed 14 in.

Clear spacing

7.885

use cross ties, because spacing is greater than 6 inches

<table>
<thead>
<tr>
<th>P-M diagrams</th>
<th>(Fs=Fy)</th>
</tr>
</thead>
<tbody>
<tr>
<td>assume $\varepsilon_u =$</td>
<td>0.003</td>
</tr>
<tr>
<td>$\varepsilon_y = Fu/E$</td>
<td>0.002105</td>
</tr>
<tr>
<td>$s =$</td>
<td>3</td>
</tr>
</tbody>
</table>

$d = h - \text{cover - diameter of tie - } 1/2 \text{ diameter of reinforcing bar}$

$d' =$ 12.92 in

$C_b =$ 7.59215 in

$\text{depth } a = 0.85 \cdot C_b =$ 2 in

$f's = \varepsilon_u \cdot E_s (C_b - d') / C_b$

$39.2 \leq 60 \text{ ksi}$

$C = 0.85 f'_c a \cdot h$
As = 4

\[ P_n = 0.85 f'c a b + A's F's \]

\[ P_n = 185.416 \]

\[ M_n = P_n e = 0.85 f'c a b (H/2 - a/2) + a's f's (h/2 - d') + A's F(d - h/2) \]

\[ 1304.31 \text{ kip inches} \]

\[ 108.692 \text{ kip feet} \]

Choose a \( C \) smaller than the previous \( C \)

\[ C_b = 5 \text{ in} \]

\[ \text{depth } a = 0.85 Cb = 4.25 \text{ in} \]

\[ f's = \varepsilon u E_s (C_b - d')/C_b \]

\[ C = 0.85 f'c a h = 162562 \text{ lbs} \]

\[ 162.562 \text{ kips} \]

\[ P_n = 0.85 f'c a b + A's F's \]

\[ P_n = -75.061 \]

\[ M_n = P_n e = 0.85 f'c a b (H/2 - a/2) + a's f's (h/2 - d') + A's F(d - h/2) \]

\[ 447.234 \text{ kip inches} \]

\[ 37.2695 \text{ kip feet} \]

\[ e = 5.95829 \text{ inches} \]

Choose a \( C \) larger than the previous \( C \)

\[ C_b = 14 \text{ in} \]

\[ \text{depth } a = 0.85 Cb = 11.9 \text{ in} \]

\[ f's = \varepsilon u E_s (d' - C)/C \]

\[ 6.59571 \]

\[ 6.59571 \]

\[ 4 \]
\[ f's = \epsilon u \times E \times (C_b - d')/C_b \]

\[ 12.5 \leq 60 \text{ ksi} \]

\[ C = 0.85 f'^c a h \]

\[ 708050 \text{ lbs} \quad \text{As} = 3 \]

\[ 708.05 \text{ kips} \quad \text{A's} = 3 \]

\[ P_n = 0.85 f'^c a b + A_s F_s - A's F's \]

\[ 719.887 \]

\[ P_n = 1 \]

\[ M_n = P_n e = 0.85 f'^c a b (H/2 - a/2) + a's f's (h/2 - d') + A_s F_s (d-h/2) \]

\[ 760.708 \text{ kip inches} \]

\[ 63.3923 \text{ kip feet} \]

\[ e = 1.05673 \text{ inches} \]

Set C to infinite and e=0

\[ F_s = 60 \text{ ksi} \]

\[ P_n = 0.85 f'^c a b + A_s F_s \]

\[ P_n = 1073 \]

Since the moment and Pn are negative for one value, a larger column is needed

---

**Column Design**

2nd floor edge

<table>
<thead>
<tr>
<th>Length</th>
<th>10.5 ft</th>
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<tbody>
<tr>
<td>f'c</td>
<td>5000 psi (Assumed)</td>
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<td>As</td>
<td>0.153 in^2</td>
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<tr>
<td>Phi</td>
<td>0.65</td>
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<td>2.85E+08 Psi</td>
</tr>
<tr>
<td>alpha</td>
<td>0.8</td>
</tr>
<tr>
<td>Fy</td>
<td>60000 PSI</td>
</tr>
<tr>
<td>Fpu</td>
<td>270000</td>
</tr>
<tr>
<td>pg</td>
<td>0.03</td>
</tr>
<tr>
<td>fps=.7*fpu</td>
<td>189000 Psi</td>
</tr>
<tr>
<td>beta</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Effective area of support

Assume a value of 500 pounds per linear foot per floor for outside façade

<table>
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<tr>
<th>17.5 ft by 16.75 ft</th>
<th>293.125 ft^2</th>
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<tbody>
<tr>
<td>Roof load=</td>
<td>466.3 psf</td>
</tr>
<tr>
<td>6th floor load=</td>
<td>358 psf</td>
</tr>
<tr>
<td>6th floor column weight=</td>
<td>2275 lbs</td>
</tr>
</tbody>
</table>

65
5th floor load = 296 psf
Column weight = 6300 lbs

4th floor load = 296 psf
Column weight = 6300 lbs

3rd floor load = 296 psf
Column weight = 6300 lbs

Total
Pu = 591592.9 lbs
591.5929 kips

Trial size
Ag = Pu / Φ * α * (0.85f'c + pg(Fy - 0.85f'c))

Ag = 192.093 in^2

Assume a trial size of 24 X 24, or 576 in^2
Ag = 576 in^2

Trial h = 24
Short or slender
l/h = 5.25
short column
assume a fixed fixed column, so K = 1.0

12h = 288 > story height, so slenderness can be neglected. (according to PDC handout)

As = pg * area of the column
17.28 in^2
1.44

Diameter of steel = 1.41
Nominal Area = 1.56

Use 4 #4 bars
As = 6.24

Check using ΦPn = 0.8 * Φ [0.85 * f'c(Ag - Ast) + fyAst]
0.8 * 0.65 * 0.85 * 5000 *(576 - 18.72) + (60 * 18.72)
ΦPn = 1453 Kips > 591.5929 so the section checks

Use #3 lateral ties.
Diameter ties = 0.375 in
16 long. Diameters
22.56
48 tie bar
diameters
18
Least Column dimension
14
tie spacing cannot exceed 14 in.

Clear spacing
7.885
use cross ties, because spacing is greater than 6 inches

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<tr>
<td>$\varepsilon_y =$</td>
<td>$\frac{F_u}{E}$</td>
</tr>
<tr>
<td>$s =$</td>
<td>3</td>
</tr>
</tbody>
</table>

d=$h$-cover-diameter of tie -$1/2$ diameter of reinforcing bar
d'=$22.92$ in
$13.4684$
$C_b =$ | 5 in |

depth $a =$ | $0.85 * C_b$ | 11.4 in |
f's = $\varepsilon_u * E_s (C_b - d')/C_b$
| 78.6439 |
| 8 | $\leq$ | 60 ksi |
C = $0.85 * f'_c * a * h$
| 116771 |
| 5 lbs | As=4 |
| 1167.71 |
| 5 kips | A's=4 |
Pn = $0.85 * f'_c * a * b + A_s F'_s$
| 1167.71 |
Pn = 5
Mn = Pn* $e =$ $0.85 * f'_c * a * b (H/2 - a/2) + a'_s f'_s (h/2 - d') + A_s F_s (d - h/2)$
| 9949.27 |
| 1 kip inches |
| 829.105 |
| 9 kip feet |
| 8.5 |
e = 1 inches

Choose a C smaller than the previous C

$C_b =$ | 5 in |
depth \( a = 0.85 \times C_b \) \( \quad 4.25 \) in

\( f's = \varepsilon u \times E_s (C_b - d') / C_b \)

\[
67.032 \leq 60 \text{ ksi}
\]

\( C = 0.85 \times f'c \times a \times h \)

\[
94,828.1 \times 3 \quad \text{lbs} \quad \text{As} = 3
\]

\[
94,8281 \times 3 \quad \text{kips} \quad \text{A's} = 3
\]

\( P_n = 0.85 \times f'c \times a \times b + A_s F_s - A's F's \)

\( P_n = 553.38 \)

\( M_n = P_n \times e = 0.85 \times f'c \times a \times b (H/2 - a/2) + a's F_s (h/2 - d') + A_s F_s (d - h/2) \)

\[
2600.8 \quad \text{kip inches}
\]

\[
216.734 \quad \text{kip feet}
\]

\[
4.69986
\]

\( e = 7 \) inches

Choose a C larger than the previous C

\( C_b = 24 \) in

depth \( a = 0.85 \times C_b \) \( \quad 20.4 \) in

\( f's = \varepsilon u \times E_s (d' - C) / C \)

\[
-3.8475
\]

\[
3.8475
\]

\( f's = \varepsilon u \times E_s (C_b - d') / C_b \)

\[
57.3 \leq 60 \text{ ksi}
\]

\( C = 0.85 \times f'c \times a \times h \)

\[
208080 \quad \text{lbs} \quad \text{As} = 3
\]

\[
2080.8 \quad \text{kips} \quad \text{A's} = 3
\]

\( P_n = 0.85 \times f'c \times a \times b + A_s F_s - A's F's \)

\( P_n = 2187.7 \)

\( M_n = P_n \times e = 0.85 \times f'c \times a \times b (H/2 - a/2) + a's F_s (h/2 - d') + A_s F_s (d - h/2) \)

\[
5212.98 \quad \text{kip inches}
\]

\[
434.415 \quad \text{kip feet}
\]

\[
2.3
\]

\( e = 3 \) inches

Set C to infinite and \( e = 0 \)
Shear Wall Excel Spreadsheet

Shear wall design

https://engineering.purdue.edu/~frosch/


<table>
<thead>
<tr>
<th>Givens</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>f'c</td>
<td>5000</td>
<td>psi</td>
</tr>
<tr>
<td>Ss</td>
<td>0.24</td>
<td>Sds</td>
</tr>
<tr>
<td>S1</td>
<td>0.067</td>
<td>Sd1</td>
</tr>
<tr>
<td>R</td>
<td>1.5</td>
<td>at existing</td>
</tr>
<tr>
<td>Cd</td>
<td>1.5</td>
<td>at existing</td>
</tr>
</tbody>
</table>

- High hazard occupancy
- Site class D
- Importance factor: 1.25
- Over strength factor: 2.5
- fy of steel: 60,000 psi

Reinforced concrete shear walls

Tributary area for each shear wall: 945 square feet

Floor live load

- Floor 2-5: 80 psf
- Floor 1, 6: 100 psf

Snow load: 43 psf

Floor dead load

- 1st, 6th floor: 150 psf + 15 = 165 psf
- 2-5th floor: 125 psf + 15 = 140 psf

Earthquake loads

- 8704.8 1st floor
- 37497.6 2nd
- 56246.4 3rd
- 74995.2 4th
- 93744 5th
- 132580.8 6th

Area per floor

- 30 X 10.5
Earthquake load is much greater than wind, so the load combo used will be
1.2D ± 1.0E + 0.5L + 0.2S

<table>
<thead>
<tr>
<th>Floor</th>
<th>Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>251.1918</td>
</tr>
<tr>
<td>Second</td>
<td>242.1846</td>
</tr>
<tr>
<td>Third</td>
<td>260.9334</td>
</tr>
<tr>
<td>Fourth</td>
<td>279.6822</td>
</tr>
<tr>
<td>Fifth</td>
<td>298.431</td>
</tr>
<tr>
<td>6th</td>
<td>375.0678</td>
</tr>
</tbody>
</table>

Check shear strength in 1st story
Vu=1.3*total shear
2219.73804 kips
from table 6
\( \Phi V_c = 30 \times 12.4 \)
762.6 kips
\( \Phi V_s = Av F_y d / s \)
number 6 bars
Av= 0.441786
Fy = 60
d= 9.6
s= 12
\( \Phi V_s = 16.9646 \)
from table 5, use number 6 bars at a spacing of 12 inches

Check shear at 2nd story
Vu=1.3* the floor loads except the 1st
1893.1887
still higher than Vc, so number 6 shear bars should be added at a spacing of 12 inches

Shear at 3rd floor
Vu=
1578.34872
as above, use number 6 bars at a spacing of 12 inches, both horizontally and vertically

Shear at 4th floor
Vu=
1239.1353

Use #6 bars at a spacing of 12 inches for each floor
Dead load and moment in the first floor
Pu = 1020600 1020.6 kips
Mu = 403768.8 403.7688 kips

2nd floor
Pu = 824040 824.04 kips
Mu = 395064 395.064 ft. Kips

3rd floor
Pu = 627480 627.48
Mu = 357566.4 357.5664

Check moment strength based on required vertical reinforcement for shear
Ast = 13.25359 in^2
ω = 0.052594
α = 0.084375
\( \frac{c}{lw} = 0.174441 \)
ϕMn = 169993.8 in. kips
\( \frac{\phiMn}{14166.15} = 11.87 \) ft-kips
moment is sufficient

2nd floor moment strength
Ast = 13.25359
ω = 0.052594
α = 0.068125
\( \frac{c}{lw} = 0.153745 \)
ϕMn = 76313.12
\( \frac{\phiMn}{6359.427} = 11.90 \) ft-kips
Moment is sufficient

3rd floor moment strength
Ast = 13.25359
ω = 0.052594
α = 0.051875
\( \frac{c}{lw} = 0.133049 \)
ϕMn = 78179.41
\( \frac{\phiMn}{6514.95} = 11.93 \) ft-kips
Moment is sufficient

The moment values are quite higher than needed, so the wall is fine for moment and axial loads
5th Floor Steel Beam and Girder Excel Spreadsheet

Scheme 1

<table>
<thead>
<tr>
<th>DL</th>
<th>Slab</th>
<th>Services</th>
<th>Ceiling</th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>754.16 lbs/ft</td>
<td>60.41667</td>
<td>10</td>
<td>5</td>
<td>30</td>
<td>10</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LL</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>525</td>
<td>70</td>
</tr>
</tbody>
</table>

lps/ft

**PLASTIC CAPACITY CALCULATIONS**

Load Combinations

\[ U = 1.2D + 1.6LL \]

\[ U = 1745 \text{ lbs/ft} \]

\[ \text{Mu} = wL^2/8 \]

\[ \text{Mu} = 196312.5 \]

\[ \text{Mu} = 196.3 \text{ ft/k} \]

Zx required

\[ Zx = \text{Mu}/\Phi_{fy} \]

\[ Zx = 52.35 \text{ in}^3 \]

Off this we pick a W 18X35

with:

- weight per foot 35 lbs/ft
- Zx 66.5 in^3

New DL calculations

\[ \text{New DL} = \text{original DL} + \text{weight of beam} \]

\[ \text{New DL} = 789.16 \text{ lbs/ft} \]

Live load is the same

New U

\[ U = 1.2D + 1.6LL \]

\[ U = 1787 \text{ lbs/ft} \]

\[ \text{Mu} = wL^2/8 \]

\[ \text{Mu} = 201037.5 \]

\[ \text{Mu} = 201 \text{ ft/k} \]
\[ \Phi_{M_p} = \Phi Z_x F_y \]
\[ \Phi_{M_p} = 249.375 \]

the beam is sufficient

**DEFLECTION CHECK**

Case 1 LL only

Beam W 18X35

Deflection must be less than \( L/360 \) or 1" whichever is smaller

Limit = \( L/360 \)

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 Inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

\[ \Delta = \frac{5wL^4}{384EI_x} \]

\[ \Delta = 510 \text{ in}^4 \text{ from Table 3.3} \]

\[ \Delta = 0.32 \text{ Inch} \]

So we are okay

Case II Full DL + Half Live

Limit = \( L/240 \)

<table>
<thead>
<tr>
<th>Limit</th>
<th>1.5 Inch</th>
</tr>
</thead>
</table>

\[ \Delta = \frac{5wL^4}{384EI_x} \]

\[ \Delta = 1.25 \text{ Inch} \]

So the current beam is sufficient by Deflection

**31.5' GIRDER**

<table>
<thead>
<tr>
<th>DL superimposed</th>
<th>Slab</th>
<th>Services</th>
<th>Ceiling</th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>75.416</td>
<td>60.41667</td>
<td>10</td>
<td>5</td>
<td>31.5</td>
<td>30</td>
</tr>
</tbody>
</table>
**PLASTIC CAPACITY CALCULATIONS**

**Load Combinations**

$U = 1.2D + 1.6LL$

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>$U$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6201 lbs/ft</td>
</tr>
</tbody>
</table>

$Mu = wL^2/8$

<table>
<thead>
<tr>
<th>$Mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>769117.8</td>
</tr>
</tbody>
</table>

$Zx = Mu/\Phi_{fy}$

<table>
<thead>
<tr>
<th>$Zx$</th>
</tr>
</thead>
<tbody>
<tr>
<td>205.1 in$^3$</td>
</tr>
</tbody>
</table>

Off this we pick a W 24X84 girder with:

- weight per foot: 84 lbs/ft
- $Zx$: 224 in$^3$

**DEFLECTION CHECK**

Case 1 LL only

Girder 33X130

Deflection must be less than $L/360$ or 1" whichever is smaller

<table>
<thead>
<tr>
<th>Limit</th>
<th>1.05 Inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

$\Delta = 5wL^4/384EI_x$

<table>
<thead>
<tr>
<th>$\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3 Inch</td>
</tr>
</tbody>
</table>

$I_x$: 2370 in$^4$ from Table 3.3
So we are okay

Case II Full DL + Half Live
Limit=L/240
Limit 1.575 Inch
\[ \Delta = \frac{5wL^4}{384EIx} \]
\[ \Delta \quad 1.067663 \text{ Inch} \]

So the current girder is sufficient by Deflection

<table>
<thead>
<tr>
<th>FINAL beam and girder size</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Beam</td>
<td>18X35</td>
</tr>
<tr>
<td>Typical Girder</td>
<td>24X84</td>
</tr>
</tbody>
</table>
### 6th Floor Steel Beam and Girder Excel Spreadsheet

**Scheme 1**

<table>
<thead>
<tr>
<th>DL</th>
<th>Slab</th>
<th>Services</th>
<th>Ceiling</th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>754.1667</td>
<td>60.41667</td>
<td>10</td>
<td>5</td>
<td>30</td>
<td>10</td>
</tr>
</tbody>
</table>

**Load Combinations**

- **U** = 1.2D + 1.6LL
  - **U** = 2105 lbs/ft

- **μ** = wL^2 / 8
  - **μ** = 236812.5
  - **μ** = 236.8 ft/k

- **Z_x** required:
  - **Z_x** = μ / Φ_fy
    - **Z_x** = 63.15 in^3

- **New DL calculations**
  - **New DL** = original DL + weight of beam
    - **New DL** = 789.16 lbs/ft

- Live load is the same

- **New μ**
  - **μ** = wL^2 / 8
    - **μ** = 241537.5
Mu 241.5 ft/k

\[ \Phi M_p = \Phi Z_x * F_y \]
\[ \Phi M_p = 249.375 \]

the beam is sufficient

**DEFLECTION CHECK**

Case 1 LL only

Beam  W 18X35

Deflection must be less than L/360 or 1" whichever is smaller

Limit = L/360

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 Inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

\[ \Delta = \frac{5wL^4}{384EI_x} \]
\[ \Delta = 0.46 \text{ Inch} \]

from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit = L/240

Limit  1.5 Inch

\[ \Delta = \frac{5wL^4}{384EI_x} \]
\[ \Delta = 1.391417 \text{ Inch} \]

So the current beam is sufficient by Deflection

**31.5' GIRDER**

<table>
<thead>
<tr>
<th>DL superimposed</th>
<th>Slab</th>
<th>Services</th>
<th>Ceiling</th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>75.416</td>
<td>60.416</td>
<td>10</td>
<td>5</td>
<td>31.5</td>
<td>30</td>
</tr>
</tbody>
</table>
LL  Load  3000  100
lbs/ft
Girder load Approx.
Wdl=(DLsp+(Beam weight/beams pacing))*Girder spacing
WDL  2367.5

PLASTIC CAPACITY CALCULATIONS
Load Combinations
U=1.2D+1.6LL

<table>
<thead>
<tr>
<th>U</th>
<th>7641</th>
<th>lbs/ft</th>
</tr>
</thead>
</table>

Mu=wl^2/8

<table>
<thead>
<tr>
<th>Mu</th>
<th>947722.8</th>
</tr>
</thead>
</table>

Mu  947.7 ft/k

Zx required
Zx =Mu/Φfy

<table>
<thead>
<tr>
<th>Zx</th>
<th>252.7 in^3</th>
</tr>
</thead>
</table>

Off this we pick a W 30X90 girder
with :
weight per foot  90 lbs/ft
Zx  283 in^3

DEFLECTION CHECK
Case 1 LL only

Girder  W 30X90

Deflection must be less than L/360 or 1" whichever is smaller
Limit=L/360

<table>
<thead>
<tr>
<th>Limit</th>
<th>1.05 inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

\[ \Delta=\frac{5wl^4}{384EIx} \]

<table>
<thead>
<tr>
<th>(\Delta)</th>
<th>0.32 inch</th>
</tr>
</thead>
</table>

from Table 3.3
So we are okay

Case II Full DL + Half Live
Limit=L/240
Limit 1.575 inch
\[ \Delta = \frac{5wL^4}{384EIx} \]
\[ \Delta = 0.79 \text{ inch} \]

So the current girder is sufficient by Deflection

<table>
<thead>
<tr>
<th>FINAL beam and girder size</th>
<th>W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Beam</td>
<td>18X35</td>
</tr>
<tr>
<td>W</td>
<td></td>
</tr>
<tr>
<td>Typical Girder</td>
<td>30X90</td>
</tr>
</tbody>
</table>
## Scheme 1: 5" Slab

### GIVENS:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>t concrete</td>
<td>5 inches</td>
</tr>
<tr>
<td>Beam Span</td>
<td>30 ft</td>
</tr>
<tr>
<td>Beam Spacing</td>
<td>10 ft</td>
</tr>
<tr>
<td>F’c</td>
<td>4 ksi</td>
</tr>
<tr>
<td>Fy</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Es</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Fu</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Stud Dia.</td>
<td>0.75</td>
</tr>
<tr>
<td>Stud As.</td>
<td>0.44 inches^2</td>
</tr>
<tr>
<td>Beam Span</td>
<td>30 ft</td>
</tr>
<tr>
<td>Fy</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Fu</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Stud Dia.</td>
<td>0.75</td>
</tr>
<tr>
<td>Stud As.</td>
<td>0.44 inches^2</td>
</tr>
</tbody>
</table>

### Super Imposed Dead Load

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total DL Unfactored</td>
<td>754.16 plf</td>
</tr>
<tr>
<td>Concrete</td>
<td>60.4167 psf</td>
</tr>
<tr>
<td>Services</td>
<td>10 psf</td>
</tr>
<tr>
<td>ceilings</td>
<td>5 psf</td>
</tr>
<tr>
<td>Type Load Unit</td>
<td></td>
</tr>
</tbody>
</table>

### Load Combinations

\[ U = 1.2D + 1.6LL \]

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>U</td>
<td>2025 plf</td>
</tr>
</tbody>
</table>

\[ Mu = wL^2/8 \]

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu</td>
<td>227812.5</td>
</tr>
<tr>
<td>Mu</td>
<td>227.8 ft*k</td>
</tr>
</tbody>
</table>

### Picking a Preliminary Beam Size

Assuming \( a = 1 \)

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Be/2 ≤ L/8 ≤ beams spacing/2</td>
<td></td>
</tr>
<tr>
<td>L/8</td>
<td>3.75</td>
</tr>
<tr>
<td>beams spacing/2</td>
<td>5</td>
</tr>
<tr>
<td>Be/2</td>
<td>3.75</td>
</tr>
<tr>
<td>Be</td>
<td>7.5 ft</td>
</tr>
<tr>
<td>Be</td>
<td>90 inches</td>
</tr>
</tbody>
</table>

\[ Y2 = t \text{ concrete} - a/2 \]

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y2</td>
<td>4.5</td>
</tr>
</tbody>
</table>
From table 3-19

<table>
<thead>
<tr>
<th>Picked</th>
<th>W 12X22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>12.3</td>
</tr>
<tr>
<td>Weight</td>
<td>22</td>
</tr>
<tr>
<td>Area</td>
<td>6.48</td>
</tr>
<tr>
<td>Steel</td>
<td>6.48</td>
</tr>
</tbody>
</table>

a Calculation

\[ a = \frac{F_y \cdot A_s}{(0.85 \cdot F'_c \cdot B_e)} \]

\[ a = 1.0 \text{ inches} \]

New \( Y_2 = 4.5 \)
Section Still
Okay

\[ e = \frac{1}{2} (t \text{ concrete}) + \text{depth of beam/2} \]

\[ e = 8.65 \text{ inches} \]

\( \Sigma Q_n \) Calculation

\[ \Sigma Q_n = A_s \cdot F_y \]

\[ \Sigma Q_n = 324 \text{ k} \]

Capacity Calculation

\[ \Phi M_n = e \cdot \Sigma Q_n \cdot \Phi \]

\[ \Phi M_n = 2522.34 \text{ ft} \cdot \text{k} \]

Section is inadequate

Pick a new beam

W 12X30

| Depth    | 12.3 |
| Weight   | 30   |
| Area     | 8.79 |
| Steel    | 8.79 |

a Calculation

\[ a = \frac{F_y \cdot A_s}{(0.85 \cdot F'_c \cdot B_e)} \]

\[ a = 1.4 \text{ inches} \]

New \( Y_2 = 4.3 \)
\[ \sum Q_n = A_s \cdot F_y \]
\[ \sum Q_n = 439.5 \text{ k} \]

\[ \Phi M_n = e \cdot \sum Q_n \cdot \Phi \]
\[ \Phi M_n = 3421.5075 \]
\[ \Phi M_n = 285.1 \text{ ft*k} \]

Beam is now Adequate

Limit = 1 \text{ inches}
Max Limit = 1 \text{ inch}
Usable Limit = 1 \text{ inch}

Deflection Calculations
\[ \Delta = \frac{5wL^4}{384EI_x} \]
\[ \Delta = 0.29 \text{ inch} \]

Design of Shear Studs

Stud Capacity = \( 0.5 \cdot \text{Area steel stud} \cdot \sqrt{F'_c \cdot E_c} \leq \text{Area steel stud} \cdot F_u \cdot R_g \cdot R_p \)

Stud Capacity = 26.1 ≤ 21.5

\begin{align*}
R_g & = 1 \\
R_p & = 0.75
\end{align*}

\[ E_c = w^{1.5} \cdot \sqrt{F'_c} \]
\[ E_c = 3492 \text{ ksi} \]

\[ w = 145 \text{ pcf} \]

So limiting stud capacity = 21.5 \text{ k}
20.4 Studs
21 Studs

Construction Capacity

Unshored Construction

<table>
<thead>
<tr>
<th>Beam Weight</th>
<th>Construction DL</th>
<th>Wet Concrete</th>
<th>Construction LL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Weight</td>
<td>30 plf</td>
<td>60.416 psf</td>
<td>20 psf</td>
</tr>
</tbody>
</table>

804.16 plf
Load Combinations

\[ U = 1.2D + 1.6LL \]

\[ U \]
\[ 1322.66 \text{ lbs/ft} \]

\[ \text{Mu=}wL^2/8 \]
\[ \text{Mu} \]
\[ 148800 \]
\[ 148.8 \text{ ft*k} \]

For this Beam
\[ \Phi M_p = 162 \text{ ft*k} \]
from table 3.2

Check for Deflection Serviceability

Live Load Deflection
Limit=\( L/360 \)
\[ \text{Limit=} \]
\[ 1 \text{ inches} \]
Max Limit
\[ 1 \text{ inch} \]

Usable Limit
\[ 1 \text{ inch} \]

Deflection Calculations
\[ \Delta=5wL^4/384EI_x \]
\[ \Delta \]
\[ 0.35 \text{ inch} \]
from Table 3-20

Required \( I_x \) calculations

Design shear anchors for full composite
\[ \Sigma Q_n \text{ Calculation} \]
\[ \Sigma Q_n=As*F_y \]
\[ \Sigma Q_n = 439.5 \text{ k} \]

Stud Capacity is the Same

\# of Studs=\( Q_n/\text{Stud Capacity} \)
\# of Studs
\[ 20.40665636 \]

Boom
\[ 21 \text{ Studs} \]

Check if full composite is possible
\[ a = \frac{F_y A_s}{0.85 F'_c b_e} \]

\[
\begin{array}{l}
a = 1.4 \text{ inches} \\
t_{\text{concrete}} = 5 \text{ inches}
\end{array}
\]

So still okay with slab depth

---

### Needed Composite Capacity

<table>
<thead>
<tr>
<th>Super Imposed Dead Load</th>
<th>Concrete</th>
<th>Services</th>
<th>Ceilings</th>
<th>Beam</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total DL Unfactored</td>
<td>784.16 plf</td>
<td>60.4167 psf</td>
<td>10 psf</td>
<td>5 psf</td>
<td>30 plf</td>
</tr>
<tr>
<td>Total LL Unfactored</td>
<td>700 plf</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Load Combinations

\[ U = 1.2D + 1.6LL \] Governs

\[
\begin{array}{l}
U = 2061 \text{ lbs/ft}
\end{array}
\]

\[ M_u = wL^2/8 \]

\[
\begin{array}{l}
M_u = 231862.5
\end{array}
\]

\[
\begin{array}{l}
M_u = 231.8 \text{ ft*k}
\end{array}
\]

### a Calculation

\[ a = \frac{F_y A_s}{0.85 F'_c b_e} \]

\[
\begin{array}{l}
a = 1.436 \text{ inches} \\
\text{New YZ} = 4.281 \text{ inches}
\end{array}
\]

Using Table 3-19, in the W 12X30 column

We need an \( M_u \) of 231.8 ft*k

So a PNA location of : 6

will be sufficient for our needs

At location 7

\[
\begin{array}{l}
\Phi M_n = 238 \text{ ft*k} \\
\Sigma Q_n = 131 \text{ k}
\end{array}
\]

---

### Check for Deflection Serviceability

Live Load Deflection

Limit=\( L/360 \)
Limit = 1 inches
Max Limit 1 inch
Usable Limit 1 inch

Deflection Calculations

$\Delta = \frac{5wL^4}{384EI_x}$

Lower Bound $I_{LB}$

<table>
<thead>
<tr>
<th>$\Delta$</th>
<th>0.486 inch</th>
</tr>
</thead>
</table>

Beam is sufficient

# of Studs Calculation

Stud Capacity is the Same

# of Studs = $\frac{Qn}{Stud Capacity}$

# of Studs = 6.08253011

Boom 7 Studs

Final Beam size

W12X30 (14)

3/4 inch studs
5 inch Slab and Steel decking for 6th floor Excel Spreadsheet

**Scheme 1: 5" Slab**

**GIVENS:**

| t concrete | 5 inches |
| Beam Span | 30 ft |
| Beam Spacing | 10 ft |
| F’c | 4 ksi |
| Fy | 50 ksi |
| Es | 29000 ksi |
| Fu | 65 ksi |
| Stud Dia. | 0.75 |
| Stud As. | 0.441786467 inches^2 |

**Super Imposed Dead Load**

| Total DL Unfactored | 754.1666667 plf |
| Total LL Unfactored | 1000 plf |

**Concrete Services ceilings Type**

| Concrete | 60.41667 psf |
| Services | 10 psf |
| ceilings | 5 psf |

**Load Combinations**

U=1.2D+1.6LL

| U | 2505 plf |

Mu=wL^2/8

| Mu | 281812.5 |
| Mu | 281.8125 ft*k |

**Picking a Preliminary Beam Size**

Assuming a=1

| 1 |

**Be Calculations**

Be/2≤L/8≤beam spacing/2

| L/8 | 3.75 |
| beam spacing/2 | 5 |
| Be/2 | 3.75 |
| Be | 7.5 ft |
| Be | 90 inches |

Y2=t concrete-a/2

| Y2 | 4.5 |
From table 3-19

<table>
<thead>
<tr>
<th>Picked</th>
<th>W 12X22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>12.3</td>
</tr>
<tr>
<td>Weight</td>
<td>22</td>
</tr>
</tbody>
</table>

\[a = \frac{F_y A_s}{0.85 F'_c B_e}\]

\[a = 1.058824\] inches

New Y2 = 4.470588
Section Still Okay

\[e = \frac{1}{2} (t_{\text{concrete}}) + \text{depth of beam}/2\]

\[e = 8.65\] inches

\[\Sigma Q_n\] Calculation
\[\Sigma Q_n = A_s F_y\]

\[\Sigma Q_n = 324\] k

\[\Phi M_n = e \Sigma Q_n \Phi\]
\[\Phi M_n = 2522.34\] ft*k

Section is inadequate

Pick a new beam

**W 12X30**

<table>
<thead>
<tr>
<th>Depth</th>
<th>12.3</th>
<th>inches</th>
<th>Area Steel</th>
<th>8.79 in^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>30</td>
<td>lbs/ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[a = \frac{F_y A_s}{0.85 F'_c B_e}\]

\[a = 1.436275\] inches

New Y2 = 4.281863

\[\Sigma Q_n\] Calculation
\[\Sigma Q_n = A_s F_y\]

\[\Sigma Q_n = 439.5\] k
\[ \Phi Mn = e^* \Sigma Qn \* \Phi \]
\[ \Phi Mn = 3421.5075 \]
\[ \Phi Mn = 285.125625 \text{ ft}^2 \text{k} \]

**Beam is now Adequate**

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Limit</td>
<td>1 inch</td>
</tr>
</tbody>
</table>

**Usable Limit**

| 1 inch             |

**Deflection Calculations**

\[ \Delta = \frac{5wl^4}{384EIx} \]
\[ \Delta = 0.426355682 \text{ inch} \]

**Design of Shear Studs**

Stud Capacity = \(0.5 \* \text{Area steel stud} \* \sqrt{F'c \* Ec}) \leq \text{Area steel stud} \* Fu \* Rg \* Rp

**Stud Capacity**

\[ 26.10678591 \leq 21.53709 \]

<table>
<thead>
<tr>
<th>Rg</th>
<th>1 for solid slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rp</td>
<td>0.75 for solid slab</td>
</tr>
</tbody>
</table>

**Ec** = \(w^{1.5} \* \sqrt{F'c})

\[ Ec = 3492.062428 \text{ ksi} \]

**So limiting stud capacity**

\[ 21.53709 \text{ k} \]

- 20.40666 Studs
- 21 Studs

**Construction Capacity**

**Unshored Construction**

<table>
<thead>
<tr>
<th>Beam Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction DL</td>
</tr>
<tr>
<td>Construction LL</td>
</tr>
<tr>
<td>804.1666667 plf</td>
</tr>
</tbody>
</table>

**Load Combinations**
U=1.2D+1.6LL

<table>
<thead>
<tr>
<th>U</th>
<th>1322.666667 lbs/ft</th>
</tr>
</thead>
</table>

Mu=wL^2/8

<table>
<thead>
<tr>
<th>Mu</th>
<th>148800</th>
</tr>
</thead>
</table>

For this Beam

<table>
<thead>
<tr>
<th>ΦMp</th>
<th>162 ft*k</th>
</tr>
</thead>
</table>

from table 3.2

Check for Deflection Serviceability

Live Load Deflection

Limit=L/360

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 inches</th>
</tr>
</thead>
</table>

Max Limit

<table>
<thead>
<tr>
<th>Max Limit</th>
<th>1 inch</th>
</tr>
</thead>
</table>

Usable Limit

<table>
<thead>
<tr>
<th>Usable Limit</th>
<th>1 inch</th>
</tr>
</thead>
</table>

Deflection Calculations

Δ=5wL^4/384EIx

<table>
<thead>
<tr>
<th>Δ</th>
<th>0.350956358 inch</th>
</tr>
</thead>
</table>

Lower Bound I

<table>
<thead>
<tr>
<th>I_{LB}</th>
<th>1440 in^4</th>
</tr>
</thead>
</table>

from Table 3-20

Required Ix calculations

Design shear anchors for full composite

ΣQn Calculation

ΣQn=As*Fy

<table>
<thead>
<tr>
<th>ΣQn</th>
<th>439.5 k</th>
</tr>
</thead>
</table>

Stud Capacity is the Same

# of Studs=Qn/Stud Capacity

<table>
<thead>
<tr>
<th># of Studs</th>
<th>20.40665636</th>
</tr>
</thead>
</table>

Boom

<table>
<thead>
<tr>
<th>Boom</th>
<th>21 Studs</th>
</tr>
</thead>
</table>

Check if full composite is possible

a=Fy*AS/.85*F'c*be

<table>
<thead>
<tr>
<th>a</th>
<th>1.43627451 inches</th>
</tr>
</thead>
</table>

89
t concrete = 5 inches
So still okay with slab depth

### Needed Composite Capacity

<table>
<thead>
<tr>
<th>Super Imposed Dead Load</th>
<th>Concrete Services</th>
<th>Ceilings</th>
<th>Beam</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total DL Unfactored</td>
<td>784.1666667 plf</td>
<td>60.41667 psf</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5</td>
<td>30</td>
</tr>
<tr>
<td>Total LL Unfactored</td>
<td>700 plf</td>
<td>60.41667 psf</td>
<td>10</td>
<td>5</td>
</tr>
</tbody>
</table>

#### Load Combinations

\[ U = 1.2D + 1.6LL \]

\[ U = 2061 \text{ lbs/ft} \]

\[ \text{Mu} = wL^2/8 \]

\[ \text{Mu} = 231862.5 \]

\[ \text{Mu} = 231.8625 \text{ ft*k} \]

a Calculation

\[ a = \frac{Fy*As}{(0.85*F'c*Be)} \]

\[ a = 1.43627451 \text{ inches} \]

\[ \text{New Y2} = 4.281862745 \text{ inches} \]

Using Table 3-19, in the W 12X30 column

We need an MU of 231.8 ft*k

So a PNA location of: 6

will be sufficient for our needs

At location 7

\[ \Phi M_n = 238 \text{ ft*k} \]

\[ \Sigma Q_n = 131 \text{ k} \]

### Check for Deflection Serviceability

Live Load Deflection

\[ \text{Limit} = L/360 \]

\[ \text{Limit} = 1 \text{ inches} \]

Max Limit

\[ \text{Max Limit} = 1 \text{ inch} \]
Usable Limit: 1 inch

**Deflection Calculations**

\[ \Delta = \frac{5wL^4}{384EI} \]

\[ \Delta = 0.486630302 \text{ inch} \]

Beam is sufficient

**# of Studs Calculation**

Stud Capacity is the Same

\[ \# \text{ of Studs} = \frac{Qn}{\text{Stud Capacity}} \]

<table>
<thead>
<tr>
<th># of Studs</th>
<th>Studs</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.08253011</td>
<td>7</td>
</tr>
</tbody>
</table>

Boom: 7 Studs

**Final Beam size**

W12X30 (14)

3/4 inch studs
Fifth floor Girder Design Excel Spreadsheet

<table>
<thead>
<tr>
<th>Scheme</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>1581.25 lbs/ft</td>
</tr>
<tr>
<td>Slab</td>
<td>60.41667</td>
</tr>
<tr>
<td>Services</td>
<td>10</td>
</tr>
<tr>
<td>Ceiling</td>
<td>5</td>
</tr>
<tr>
<td>Beam</td>
<td></td>
</tr>
<tr>
<td>DL</td>
<td>30</td>
</tr>
<tr>
<td>Wall</td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>30</td>
</tr>
<tr>
<td>Spacing</td>
<td>15</td>
</tr>
</tbody>
</table>

| LL     | 525 lbs/ft |
| Load   | 70 |

**PLASTIC CAPACITY CALCULATIONS**

Load Combinations

\[ U = 1.2D + 1.6LL \]

\[ U = 2737.5 \text{ lbs/ft} \]

\[ Mu = wL^2/8 \]

\[ Mu = 307968.8 \]

\[ Mu = 307.9688 \text{ ft/k} \]

\[ Z_x \text{ required} \]

\[ Z_x = Mu/\Phi_fy \]

\[ Z_x = 82.125 \text{ in}^3 \]

Off this we pick a W 21X44 with:

- weight per foot: 44 lbs/ft
- \( Z_x = 95.4 \text{ in}^3 \)

New DL calculations

\[ \text{New DL} = \text{original DL} + \text{weight of beam} \]

\[ \text{New DL} = 1625.25 \text{ lbs/ft} \]

Live load is the same

New U

\[ U = 1.2D + 1.6LL \]

\[ U = 2790.3 \text{ lbs/ft} \]

\[ Mu = wL^2/8 \]
\( \Phi M_p = \Phi Z_x \times F_y \)
\( \Phi M_p = 357.75 \)

the beam is sufficient

**DEFLECTION CHECK**

Case 1 LL only

Beam \ W 18X35

Deflection must be less than \( \frac{L}{360} \) or 1" whichever is smaller

Limit = \( \frac{L}{360} \)

Limit

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

\[ \Delta = \frac{5wL^4}{384EI_x} \]

<table>
<thead>
<tr>
<th>( \Delta )</th>
<th>0.195924 inch</th>
</tr>
</thead>
</table>

from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit = \( \frac{L}{240} \)

Limit

<table>
<thead>
<tr>
<th>Limit</th>
<th>1.5 inch</th>
</tr>
</thead>
</table>

\[ \Delta = \frac{5wL^4}{384EI_x} \]

<table>
<thead>
<tr>
<th>( \Delta )</th>
<th>1.37613 inch</th>
</tr>
</thead>
</table>

So the current girder is sufficient by Deflection
Sixth Floor Girder Design Excel Spreadsheet

Scheme 1

<table>
<thead>
<tr>
<th>DL</th>
<th>Slab</th>
<th>Services</th>
<th>Ceiling</th>
<th>Beam DL</th>
<th>Wall</th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1581.25</td>
<td>60.41667</td>
<td>10</td>
<td>5</td>
<td>30</td>
<td></td>
<td>30</td>
<td>15</td>
</tr>
</tbody>
</table>

**LL**

<table>
<thead>
<tr>
<th>Load</th>
<th>750</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>lbs/ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PLASTIC CAPACITY CALCULATIONS**

**Load Combinations**

\[ U = 1.2D + 1.6LL \]

\[ U = 3097.5 \text{ lbs/ft} \]

\[ Mu = wL^2/8 \]

\[ Mu = 348468.8 \]

\[ Mu = 348.4688 \text{ ft/k} \]

**Zx required**

\[ Zx = Mu/\Phi fy \]

\[ Zx = 92.925 \text{ in}^3 \]

Off this we pick a W 21X44 with:

- weight per foot: 44 lbs/ft
- Zx: 95.4 in^3

**New DL calculations**

New DL = original DL + weight of beam

\[ \text{New DL} = 1625.25 \text{ lbs/ft} \]

Live load is the same

**New U**

\[ U = 1.2D + 1.6LL \]

\[ U = 3150.3 \text{ lbs/ft} \]

\[ Mu = wL^2/8 \]
Mu  354408.8
Mu  354.4088  ft/k

ΦMp=ΦZx*Fy
ΦMp  357.75

the beam is sufficient

**DEFLECTION CHECK**

Case 1 LL only

Beam  W 18X35

Deflection must be less than L/360 or 1" whichever is smaller

Limit=L/360

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

\[ \Delta = \frac{5wL^4}{384EI_x} \]

<table>
<thead>
<tr>
<th>Ix</th>
<th>842 in^4</th>
</tr>
</thead>
</table>

\[ \Delta = 0.279891 \text{ inch} \]  from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit=L/240

<table>
<thead>
<tr>
<th>Limit</th>
<th>1.5 inch</th>
</tr>
</thead>
</table>

\[ \Delta = \frac{5wL^4}{384EI_x} \]

<table>
<thead>
<tr>
<th>\Delta</th>
<th>1.460097 inch</th>
</tr>
</thead>
</table>

So the current girder is sufficient by Deflection
Fifth Floor Spandrel Beam Excel Spreadsheet

**Scheme 1: 5" Slab**

**GIVENS:**

| t concrete | 5 inches | F'c | 4 ksi | Es | 29000 ksi |
| Beam Span  | 30 ft    | Fy  | 50 ksi | Fu | 65 ksi    |
| Beam Spacing | 10 ft | Stud Dia. | 0.75 |
| Stud As. | 0.441786467 inches^2 |

**Super Imposed Dead Load**

| Total DL Unfactored | 1069.166667 plf |
| Concrete Services ceilings | 60.41667 psf 10 psf 5 psf |
| Type load Unit | shear walls 315 |

| Total LL Unfactored | 700 plf |

Load Combinations

\[ U = 1.2D + 1.6LL \]

Governs

| U | 2403 plf |

\[ \text{Mu} = wL^2/8 \]

| Mu | 270337.5 |

| Mu | 270.3375 ft*k |

**Picking a Preliminary Beam Size**

Assuming a=1

| 1 |

**Be Calculations**

\[ \text{Be} = 90 \text{ inches} \]

\[ Y2 = t \text{ concrete} - a/2 \]

| Y2 | 4.5 |
From Table 3-19

<table>
<thead>
<tr>
<th>Picked</th>
<th>W 18X40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>17.9</td>
</tr>
<tr>
<td>Weight</td>
<td>40</td>
</tr>
<tr>
<td>Area</td>
<td>11.8</td>
</tr>
<tr>
<td>Steel</td>
<td>in²</td>
</tr>
</tbody>
</table>

From Table 3-19

<table>
<thead>
<tr>
<th>Picked</th>
<th>W 18X40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>17.9</td>
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<tr>
<td>Weight</td>
<td>40</td>
</tr>
<tr>
<td>Area</td>
<td>11.8</td>
</tr>
<tr>
<td>Steel</td>
<td>in²</td>
</tr>
</tbody>
</table>

a Calculation

\[ a = \frac{F_y A_s}{(0.85 F'_c B_e)} \]

\[ a = 1.928105 \text{ inches} \]

New Y2 = 4.035948
Section Still Okay

E Calculation

\[ e = \frac{1}{2} (t_{\text{concrete}}) + \text{depth of beam}/2 \]

\[ e = 11.45 \text{ inches} \]

\[ \Sigma Q_n = 590 \text{ k} \]

Capacity Calculation

\[ \Phi M_n = e \cdot \Sigma Q_n \cdot \Phi \]

\[ \Phi M_n = 6079.95 \]

\[ \Phi M_n = 506.6625 \text{ ft*k} \]

Limit = 1 inch
Max Limit = 1 inch
Usable Limit = 1 inch

Deflection Calculations

\[ \Delta = \frac{5wL^4}{384EI_x} \]

\[ \Delta = 0.13747306 \text{ inch} \]

\[ I_{LB} = 1600 \text{ in}^4 \]

Design of Shear Studs

Stud Capacity = 0.5 \cdot \text{Area steel stud} \cdot \sqrt{F'_c E_c} \leq \text{Area steel stud} \cdot F_u \cdot R_g \cdot R_p

Stud Capacity = 26.10678591 ≤ 21.53709

R_g = 1 for solid
Ec = w^1.5 * sqrt(F'c)

Ec = 3492.062428 ksi

w = 145 pcf

So limiting stud capacity = 21.53709 k

27.3946 Studs

28 Studs

Construction Capacity

Unshored Construction

Beam Weight

<table>
<thead>
<tr>
<th>Construction DL</th>
<th>30 plf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction LL</td>
<td>60.4166667 psf</td>
</tr>
<tr>
<td>Wet Concrete</td>
<td>804.166667 plf</td>
</tr>
<tr>
<td>Construction LL</td>
<td>20 psf</td>
</tr>
</tbody>
</table>

Load Combinations

U = 1.2D + 1.6LL

Governs

U = 1322.666667 lbs/ft

Mu = wL^2/8

Mu = 148800

Mu = 148.8 ft*k

For this Beam

\( \Phi \sigma_p = 289 \text{ ft}\cdot\text{k} \)

from table 3.2

Check for Deflection Serviceability

Live Load Deflection

Limit = L/360

Limit = 1 inches

Max Limit = 1 inch

Usable Limit = 1 inch

Deflection Calculations
\[ \Delta = \frac{5wL^4}{384EI} \]

| Lower Bound I |  
|-----------------|---------------------|
| \( I_{LB} \) | 1600 \( \text{in}^4 \) 
| Source | from Table 3-20 |

Required \( Ix \) calculations

Design shear anchors for full composite

\[ \Sigma Q_n = A_s \times F_y \]

| \( \Sigma Q_n \) | 590 \( \text{k} \) |

Stud Capacity is the same

\# of Studs = \( Q_n / \text{Stud Capacity} \)

| \# of Studs | 27.39460126 |
| Boom | 28 Studs |

Check if full composite is possible

\[ a = F_y \times A_s / 0.85 \times F_c \times b_E \]

| \( a \) | 1.928104575 \( \text{inches} \) |
| \( t_{\text{concrete}} \) | 5 \( \text{inches} \) |

So still okay with slab depth

Needed Composite Capacity

<table>
<thead>
<tr>
<th>Super Imposed Dead Load</th>
<th>Concrete Services</th>
<th>Ceilings</th>
<th>Beam</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total DL Unfactored</td>
<td>1131.16667 ( \text{plf} )</td>
<td>60.41667 ( \text{psf} )</td>
<td>10 ( \text{psf} )</td>
<td>5 ( \text{psf} )</td>
</tr>
<tr>
<td>Total LL Unfactored</td>
<td>700 ( \text{plf} )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Load Combinations

\[ U = 1.2D + 1.6LL \]

| \( U \) | 2477.4 \( \text{lbs/ft} \) |

\[ \text{Mu} = \frac{wL^2}{8} \]

| \( \text{Mu} \) | 2787.075 |
| \( \text{Mu} \) | 278.7075 \( \text{ft} \times \text{k} \) |
a Calculation

\[ a = \frac{F_y A_s}{0.85 F'_c B_e} \]

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>1.928104575 inches</td>
</tr>
<tr>
<td>New Y2</td>
<td>4.035947712 inches</td>
</tr>
</tbody>
</table>

Using Table 3-19, in the W 18X40 column

We need an MU of 278.70 ft*k

So a PNA location of : 7

will be sufficient for our needs

At location 7

<table>
<thead>
<tr>
<th>( \Phi M_n )</th>
<th>412 ft*k</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Sigma Q_n )</td>
<td>148 k</td>
</tr>
</tbody>
</table>

Check for Deflection Serviceability

Live Load Deflection

Limit = L/360

\[ \text{Limit} = 1 \text{ inches} \]

Max Limit

1 inch

Usable Limit

1 inch

Deflection Calculations

\[ \Delta = \frac{5wL^4}{384EI_x} \]

\[ \Delta = 0.13747306 \text{ inch} \]

Beam is sufficient

# of Studs Calculation

Stud Capacity is the Same

\[ # \text{ of Studs} = \frac{Q_n}{\text{Stud Capacity}} \]

\[ # \text{ of Studs} = 6.871866078 \]

7 Studs

Final Beam size

W18X40(14)

3/4 inch studs
Sixth Floor Spandrel Beam Excel Spreadsheet

GIVENS:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>t concrete</td>
<td>5</td>
</tr>
<tr>
<td>Beam Span</td>
<td>30 ft</td>
</tr>
<tr>
<td>Beam Spacing</td>
<td>10 ft</td>
</tr>
<tr>
<td>F'c</td>
<td>4</td>
</tr>
<tr>
<td>Fy</td>
<td>50</td>
</tr>
<tr>
<td>Es</td>
<td>29000</td>
</tr>
<tr>
<td>Fu</td>
<td>65</td>
</tr>
<tr>
<td>Stud Dia.</td>
<td>0.75</td>
</tr>
<tr>
<td>Stud As.</td>
<td>0.44178646 inches^2</td>
</tr>
<tr>
<td>Beam Span</td>
<td>30 ft</td>
</tr>
<tr>
<td>Stud Dia.</td>
<td>0.75</td>
</tr>
<tr>
<td>Stud As.</td>
<td>0.44178646 inches^2</td>
</tr>
</tbody>
</table>

Super Imposed Dead Load

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>1069.16666</th>
<th>60.4166</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total DL Unfactored</td>
<td>7</td>
<td>plf</td>
<td>psf</td>
</tr>
<tr>
<td>Total LL Unfactored</td>
<td>1000</td>
<td>plf</td>
<td>psf</td>
</tr>
</tbody>
</table>

Load Combinations

U=1.2D+1.6LL

U 2883 plf

Mu=wl^2/8

Mu 324337.5

Mu 324.3375 ft*k

Picking a Preliminary Beam Size

Assuming a=1

Be Calculations

Be/2≤L/8≤beamspaceing/2

L/8 3.75
beam spacing/2 5
Be/2 3.75
Be 7.5 ft
Be 90 inches

Y2=tcement-a/2

Y2 4.5
From Table 3-19

<table>
<thead>
<tr>
<th>Picked</th>
<th>W 18X46</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>18.1 inches</td>
</tr>
<tr>
<td>Weight</td>
<td>46 lbs/ft</td>
</tr>
</tbody>
</table>

Area Steel: 13.5 in^2

**Calculation**

\[ a = \frac{F_y \cdot A_s}{(0.85 \cdot F'_c \cdot B_e)} \]

- \( a = 2.20588 \) inches
- New \( Y_2 = 3.89705 \) inches
- Section Still: Okay

**Calculation**

\[ e = \frac{1}{2}(t_{\text{concrete}}) + \text{depth of beam}/2 \]

- \( e = 11.55 \) inches

**\( \Sigma Q_n \) Calculation**

\[ \Sigma Q_n = A_s \cdot F_y \]

- \( \Sigma Q_n = 675 \) k

**Capacity Calculation**

\[ \Phi M_n = e \cdot \Sigma Q_n \cdot \Phi \]

- \( \Phi M_n = 7016.625 \) ft*k
- \( \Phi M_n = 584.71875 \) ft*k

**Beam is now Adequate**

- Limit: 1 inch
- Max Limit: 1 inch
- Usable Limit: 1 inch

**Deflection Calculations**

\[ \Delta = \frac{5wL^4}{384EI_x} \]

- Lower Bound I
  - \( I_{LB} = 712 \) in^4
  - \( \Delta = 0.44132603 \) inches
- \( \Delta = 6 \) inches

**Design of Shear Studs**
Stud Capacity = \(0.5 \times \text{Area steel stud} \times \sqrt{F'c \times Ec}\) ≤ \(\text{Area steel stud} \times Fu \times Rg \times Rp\)

\[
\begin{align*}
\text{Stud Capacity} &= \frac{26.1067859}{21.5370} \\
Rg &= 1 \quad \text{for solid slab} \\
Rp &= 0.75 \quad \text{for solid slab}
\end{align*}
\]

\(Ec = w^{1.5} \times \sqrt{F'c}\)

\[
\begin{align*}
Ec &= 3492.06242 \\
\text{w} &= 145 \text{ pcf}
\end{align*}
\]

So limiting stud capacity = \(21.5370\) k

31.3412 k

8 Studs

43 Studs

**Construction Capacity**

**Unshored Construction**

<table>
<thead>
<tr>
<th>Beam Weight</th>
<th>Construction DL</th>
<th>30 plf</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Wet Concrete</th>
<th>Construction LL</th>
</tr>
</thead>
<tbody>
<tr>
<td>60.4166666</td>
<td>7 psf</td>
</tr>
<tr>
<td>804.1666667</td>
<td>20 psf</td>
</tr>
</tbody>
</table>

**Load Combinations**

\(U = 1.2D + 1.6LL\)

\[
\begin{align*}
U &= 1322.66666 \\
\text{U} &= 7 \text{ lbs/ft}
\end{align*}
\]

\(Mu = wL^2/8\)

\[
\begin{align*}
\text{Mu} &= 148800 \\
\text{Mu} &= 148.8 \text{ ft*k}
\end{align*}
\]

For this Beam \(\Phi M_p = 162 \text{ ft*k}\) from table 3.2
Check for Deflection Serviceability

Live Load Deflection
Limit = L/360

Limit = 1 inch
Max Limit = 1 inch

Usable Limit = 1 inch

Deflection Calculations

\[ \Delta = \frac{5wL^4}{384EIx} \]

\[ \Delta \text{ from Table 3-20} = \frac{0.70979937}{5} \text{ inch} \]

Required Ix calculations

Design shear anchors for full composite

\[ \Sigma Qn = A_s F_y \]

\[ \Sigma Qn = 675 \text{ k} \]

Stud Capacity is the Same

# of Studs = \( \frac{Qn}{\text{Stud Capacity}} \)

# of Studs = 31.3412811
Boom = 32 Studs

Check if full composite is possible

\[ a = F_y A_s \cdot 0.85 \cdot F'c \cdot b \cdot E \]

\[ a = 2.20588235 \text{ inches} \]

t concrete = 5 inches

So still okay with slab depth

Needed Composite Capacity

<table>
<thead>
<tr>
<th>Super Imposed Dead Load</th>
<th>Concrete Services</th>
<th>Ceilings</th>
<th>Beam</th>
<th>Type Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total DL Unfactored</td>
<td>60.4166</td>
<td>10</td>
<td>5</td>
<td>62</td>
</tr>
</tbody>
</table>
Load Combinations

\[ U = 1.2D + 1.6LL \]  
Governs

\[ U = 2477.4 \text{ lbs/ft} \]

\[ \mu = wL^2/8 \]

\[ \mu = 278707.5 \]

\[ \mu = 278.7075 \text{ ft}*k \]

a Calculation

\[ a = \frac{F_y*As}{(0.85*F'_c*B_e)} \]

\[ a = 2.20588235 \]  
3 inches

\[ 3.89705882 \]  
4 inches

New Y2 =

Using Table 3-19, in the W 18X46 column

We need an \( \mu \) of 278.70 ft\(*k\)

So a PNA location of

\[ : 7 \]

will be sufficient for our needs

At location 7

\[ \Phi Mn \]  
475 ft\(*k\)

\[ \Sigma Qn \]  
169 k

Check for Deflection Serviceability

Live Load Deflection

Limit = L/360

Limit = 1 inch

Max Limit = 1 inch

Usable Limit = 1 inch

Deflection Calculations
\[ \Delta = \frac{5wL^4}{384EIx} \]

<table>
<thead>
<tr>
<th>( \Delta )</th>
<th>Lower Bound I</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.18799734</td>
<td>1170 in(^4)</td>
</tr>
</tbody>
</table>

Beam is sufficient

\[
\text{Stud Capacity is the Same}
\]

\[
\# \text{ of Studs} = \frac{Qn}{\text{Stud Capacity}}
\]

<table>
<thead>
<tr>
<th># of Studs</th>
<th>7.84692815</th>
</tr>
</thead>
<tbody>
<tr>
<td># of Studs</td>
<td>7</td>
</tr>
<tr>
<td>Boom</td>
<td>8 Studs</td>
</tr>
</tbody>
</table>

Final Beam size

W18X46 (16)

3/4 inch studs
Fifth Floor Spandrel Girder Excel Spreadsheet

<table>
<thead>
<tr>
<th>Scheme</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>2121.25 lbs/ft</td>
</tr>
<tr>
<td>Slab</td>
<td>60.41667</td>
</tr>
<tr>
<td>Services</td>
<td>10</td>
</tr>
<tr>
<td>Ceiling</td>
<td>5</td>
</tr>
<tr>
<td>Beam DL</td>
<td>45</td>
</tr>
<tr>
<td>Wall</td>
<td>315</td>
</tr>
<tr>
<td>Length</td>
<td>30</td>
</tr>
<tr>
<td>Spacing</td>
<td>15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LL</th>
<th>Load</th>
<th>525 lbs/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>70</td>
<td></td>
</tr>
</tbody>
</table>

**PLASTIC CAPACITY CALCULATIONS**

Load Combinations

\[ U = 1.2D + 1.6LL \]

- **U** = 3385.5 lbs/ft
- **Mu** = \( wL^2/8 \)
  - **Mu** = 380868.8 lbs/ft
  - **Mu** = 380.8688 ft/k

**Zx required**

\[ Zx = \frac{Mu}{\Phi fy} \]

- **Zx** = 101.565 in^3

Off this we pick a W 21X55

- **weight per foot** = 55 lbs/ft
- **Zx** = 126 in^3

**New DL calculations**

- **New DL** = original DL + weight of beam
  - **New DL** = 2176.25 lbs/ft

Live load is the same

- **New U**
  - **U** = 3451.5 lbs/ft
  - **Mu** = \( wL^2/8 \)
Mu 388293.8

Mu 388.2938 ft/k

ΦMp=ΦZx*Fy

ΦMp 472.5

the beam is sufficient

DEFLECTION CHECK

Case 1 LL only

Beam W 18X35

Deflection must be less than L/360 or 1" whichever is smaller

Limit=L/360

Limit 1 inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx

Ix 1140 in^4

Δ 0.144708 inch

from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit=L/240

Limit 1.5 inch

Δ=5wL^4/384EIx

Δ 1.314091 inch

So the current girder is sufficient by Deflection
Sixth Floor Spandrel Girder Excel Spreadsheet

## Scheme 1

<table>
<thead>
<tr>
<th>DL</th>
<th>Slab</th>
<th>Services</th>
<th>Ceiling</th>
<th>Beam DL</th>
<th>Wall</th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2121.25 lbs/ft</td>
<td>60.41667</td>
<td>10</td>
<td>5</td>
<td>45</td>
<td>315</td>
<td>30</td>
<td>15</td>
</tr>
</tbody>
</table>

## Load

<table>
<thead>
<tr>
<th>LL</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000 lbs/ft</td>
<td>100</td>
</tr>
</tbody>
</table>

### PLASTIC CAPACITY CALCULATIONS

**Load Combinations**

\[ U = 1.2D + 1.6LL \]

\[ U = 4145.5 \text{ lbs/ft} \]

\[ Mu = wL^2/8 \]

\[ Mu = 466368.8 \text{ ft/k} \]

\[ Zx = Mu/\Phi fy \]

\[ Zx = 124.365 \text{ in}^3 \]

Off this we pick a W 24X55 with:

- weight per foot: 55 lbs/ft
- \( Zx = 134 \text{ in}^3 \)

**New DL calculations**

\[ \text{New DL} = \text{original DL} + \text{weight of beam} \]

\[ \text{New DL} = 2176.25 \text{ lbs/ft} \]

Live load is the same

**New U**

\[ U = 4211.5 \text{ lbs/ft} \]

\[ Mu = wL^2/8 \]
Mu 473793.8
Mu 473.7938 ft/k

Φ Mp = Φ Zx * Fy
Φ Mp 502.5

the beam is sufficient

**DEFLECTION CHECK**

Case 1 LL only

Beam W 24X55

Deflection must be less than L/360 or 1" whichever is smaller

Limit = L/360

<table>
<thead>
<tr>
<th>Limit</th>
<th>1 inch</th>
</tr>
</thead>
</table>

So the deflection limit is the maximum, 1"

Δ = 5wL^4/384EIx

<table>
<thead>
<tr>
<th>Δ</th>
<th>0.232759 inch</th>
</tr>
</thead>
</table>

from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit = L/240

<table>
<thead>
<tr>
<th>Limit</th>
<th>1.5 inch</th>
</tr>
</thead>
</table>

Δ = 5wL^4/384EIx

<table>
<thead>
<tr>
<th>Δ</th>
<th>1.220237 inch</th>
</tr>
</thead>
</table>

So the current girder is sufficient by Deflection