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MCPHS Peer Review

Dillon T. O'Toole
Worcester Polytechnic Institute

MCPHS Peer Review

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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 head quartered 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[1] and the American institute for Steel Construction's

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[1] *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. Farmington Hills, MI: American Concrete Institute, 2008. Print.

steel design manual[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.

[2] *Steel Construction Manual*. Chicago, IL: American Institute of Steel Construction, 2007. Print.

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.

Figure 1: Street view of the west elevation of the MCPHS building during construction.

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 ¼ 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 2nd, 3rd, 4th And 5th floors are subjected to the same loading, so they were uniformly designed. The 6th floor was subject to a higher loading then those floors below it.

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[3] Nawy, Edward G. *Prestressed Concrete: A Fundamental Approach*. Upper Saddle River, NJ: Prentice Hall, 2000. Print.

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.

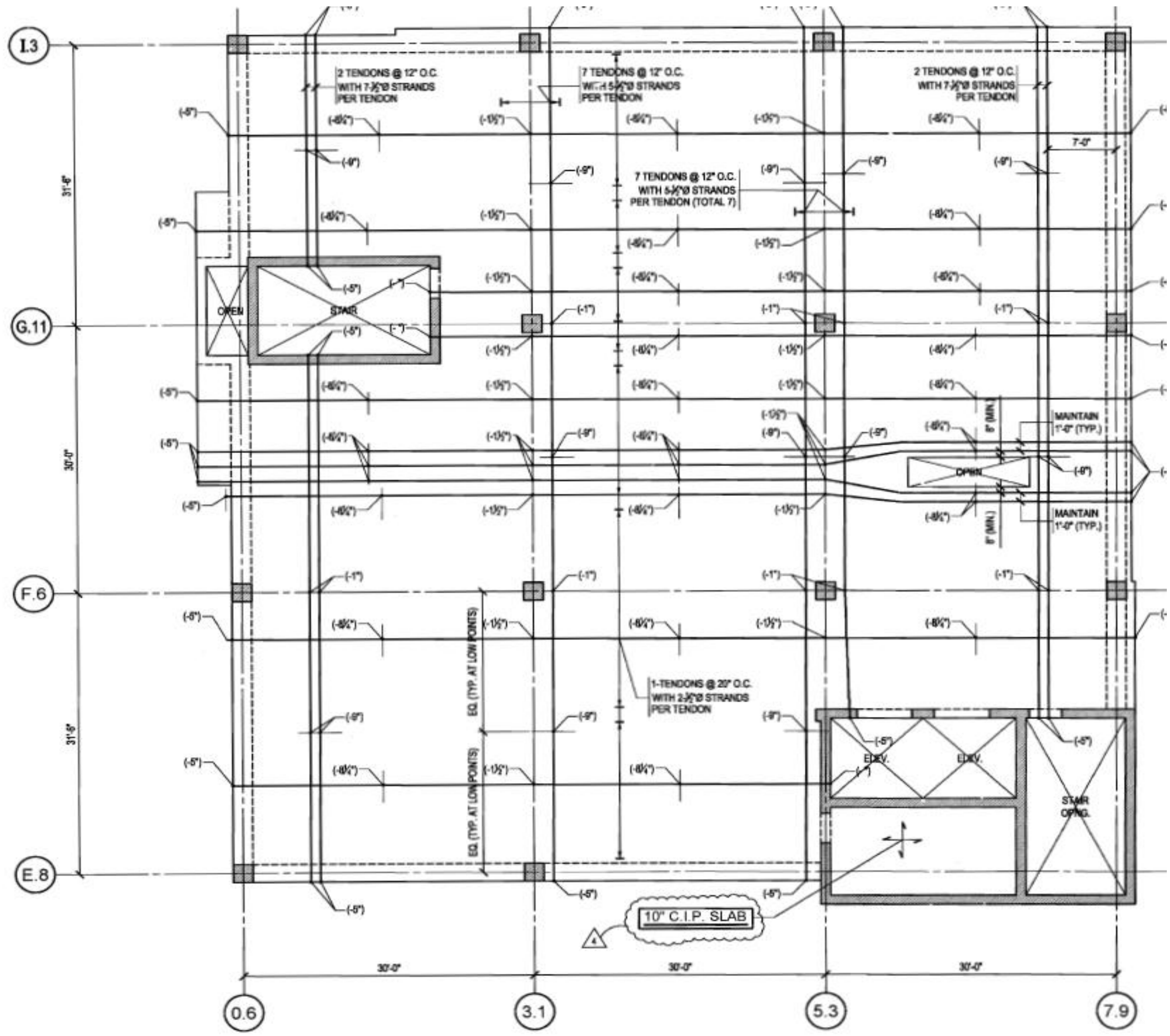

Figure 3: tendon layout for the 2nd through 5th floor slabs

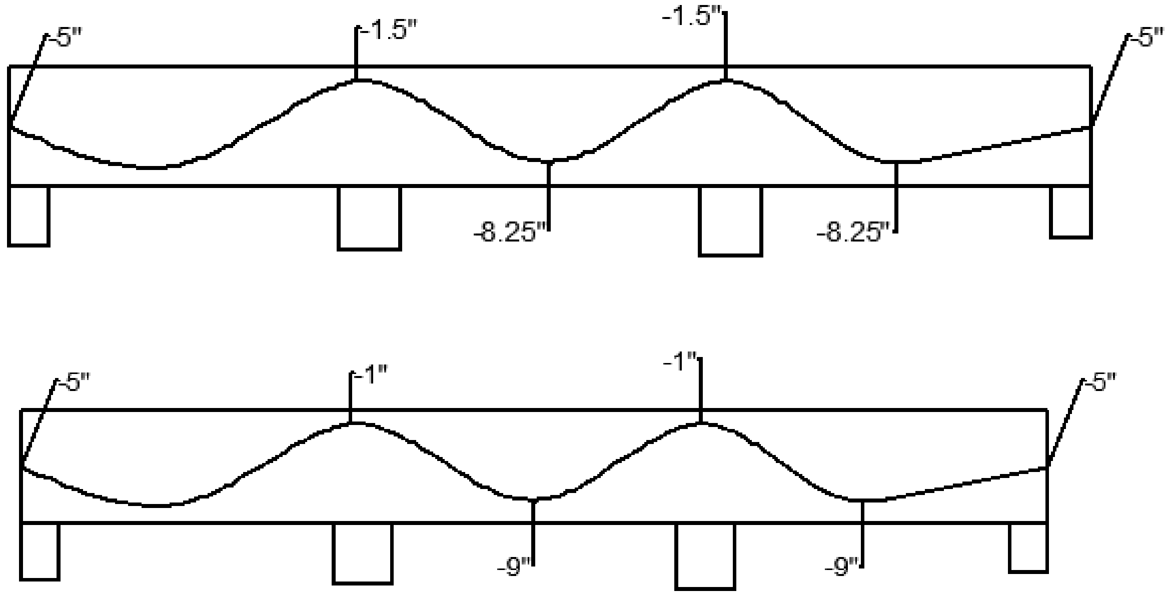

Figure 4: east-west and north-south tendon profile for the 2nd floor respectfully. The tendon reaches a high point at the columns, and a low point at the midpoint of the slabs. The values are the depths of the tendons at columns and midpoints. The above tendon profile is uniformly distributed while the 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_{ec} 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_t were found. These values were found at the support as well as at the midspan.

Table 1: Example Moment Distribution Calculations

A		B		C	
DF	0.36	0.27	0.27	0.27	0.27
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	127.5	127.5	89.7	89.7	127.5
DIST	46.5	34.1	24.0	24.2	34.1
CO	17.0	23.3	12.1	12.0	
DIST	6.2	6.2	3.2	3.2	
Final Mnet *10^3 per ft	-70.2	178.6	-81.0		

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 Mn 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.

Table 2: factored shear (lbs per foot) and secondary moments (ft-lb) for the fifth floor

Factored Shear			Ms
ab	3649.8		37219.4
ba	4855.2		29231.9
bc	3240.0		13417.6

Table 3: factored shear (lbs per foot) and secondary moments (ft-lb) for the sixth floor

Factored Shear			Ms
ab	4120.9		48827.3
ba	5392.1		34734.3
bc	3624.0		10842.3

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^{nd} to 5^{th} floors, and once for the 6^{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 √f'c, and positive moment stresses is limited to 2√f'c.

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.

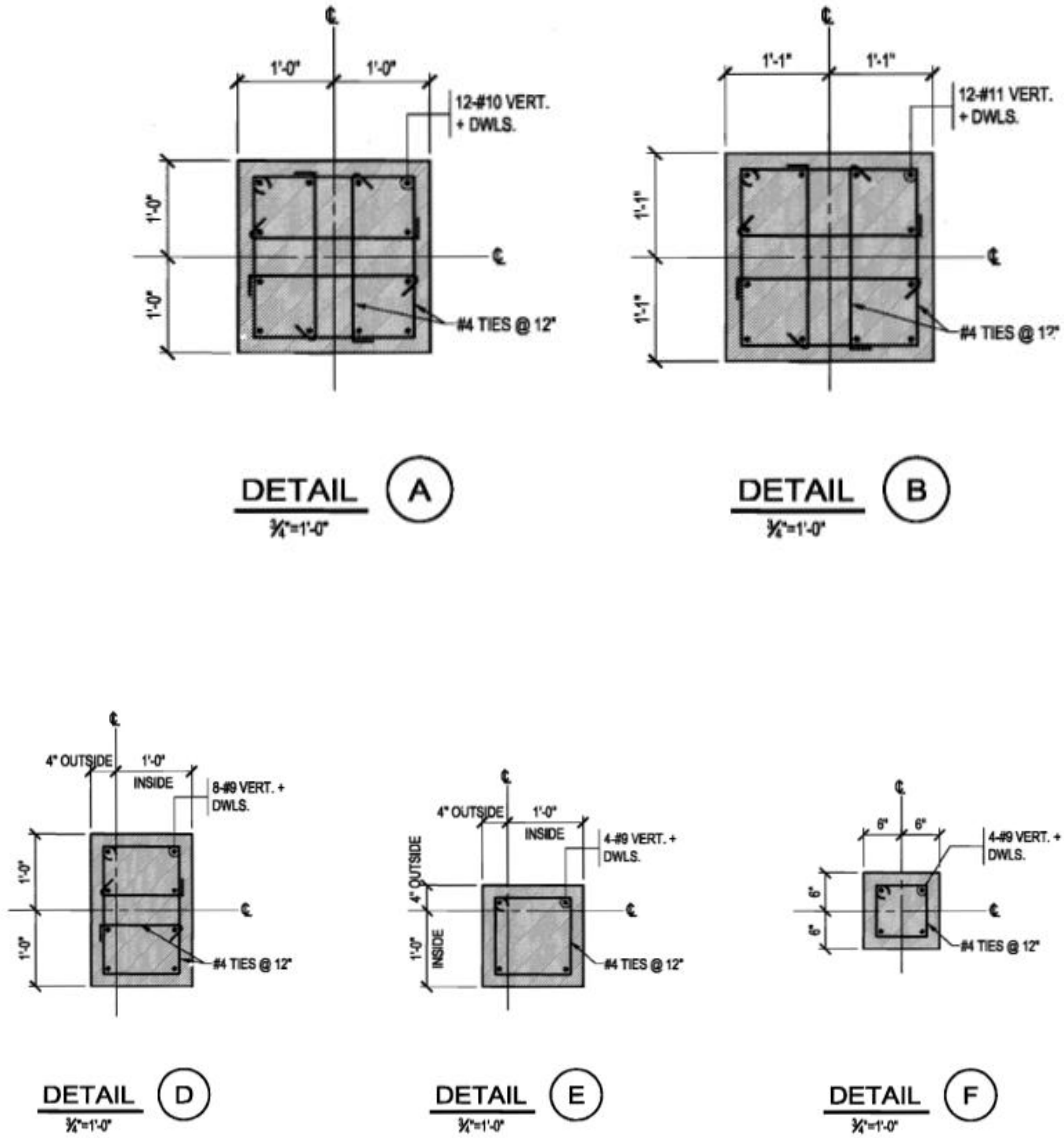

Figure 5: Column specifications

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 Ag=Pu/Φ*α*(.85f'c+pg (Fy-.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. ϵ_U is assumed to be equal to .003 in this case. ϵ_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 $.85f'_c*a*h$. 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, ϕV_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.

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[4] Fanella, David A. "Time Saving Design Aids for Reinforced Concrete." *Structural Engineer* (2001): 42-47. Web. <https://engineering.purdue.edu/~frosch/CE576/Time%20Saving%20Design%20Tips/Time%20Saving-Columns&Walls.pdf>.

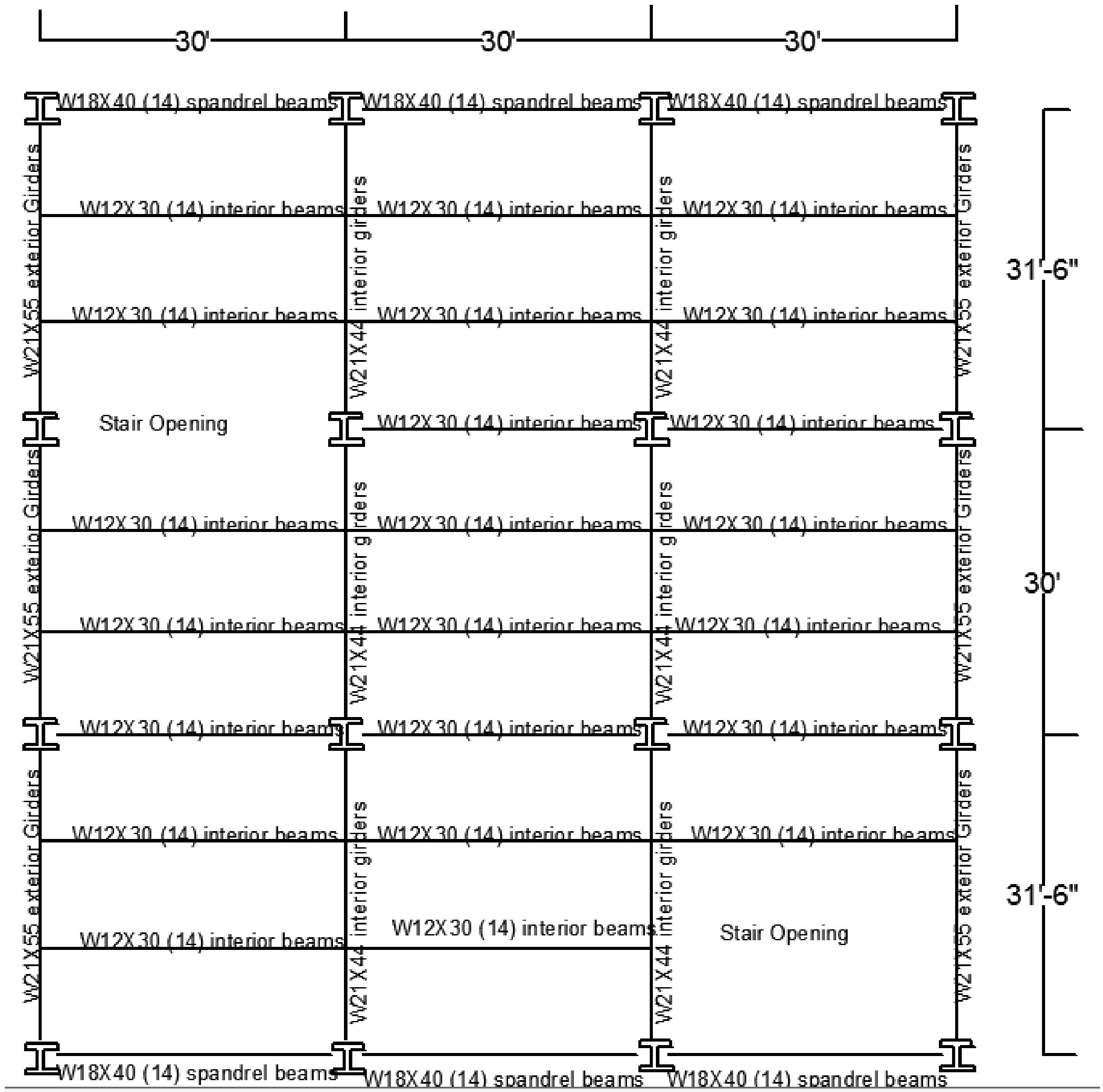
The second floor calculations were completed using the same methodology. First the horizontal shear reinforcement was found, followed by the vertical. The upper level floors were apparently given the same bar spacing as well, for the sake of constructability and simplicity. Though this simplifies the 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, ω, α and c/lw were found in order to calculate the available moment. Once the available moment, ϕMn 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.


30'
30'
30'
W18X40 (14) spandrel beams
W18X40 (14) spandrel beams
W18X40 (14) spandrel beams
W21X55 exterior Girders
W21X44 interior girders
W12X30 (14) interior beams
Stair Opening
31'-6"
30'
31'-6"

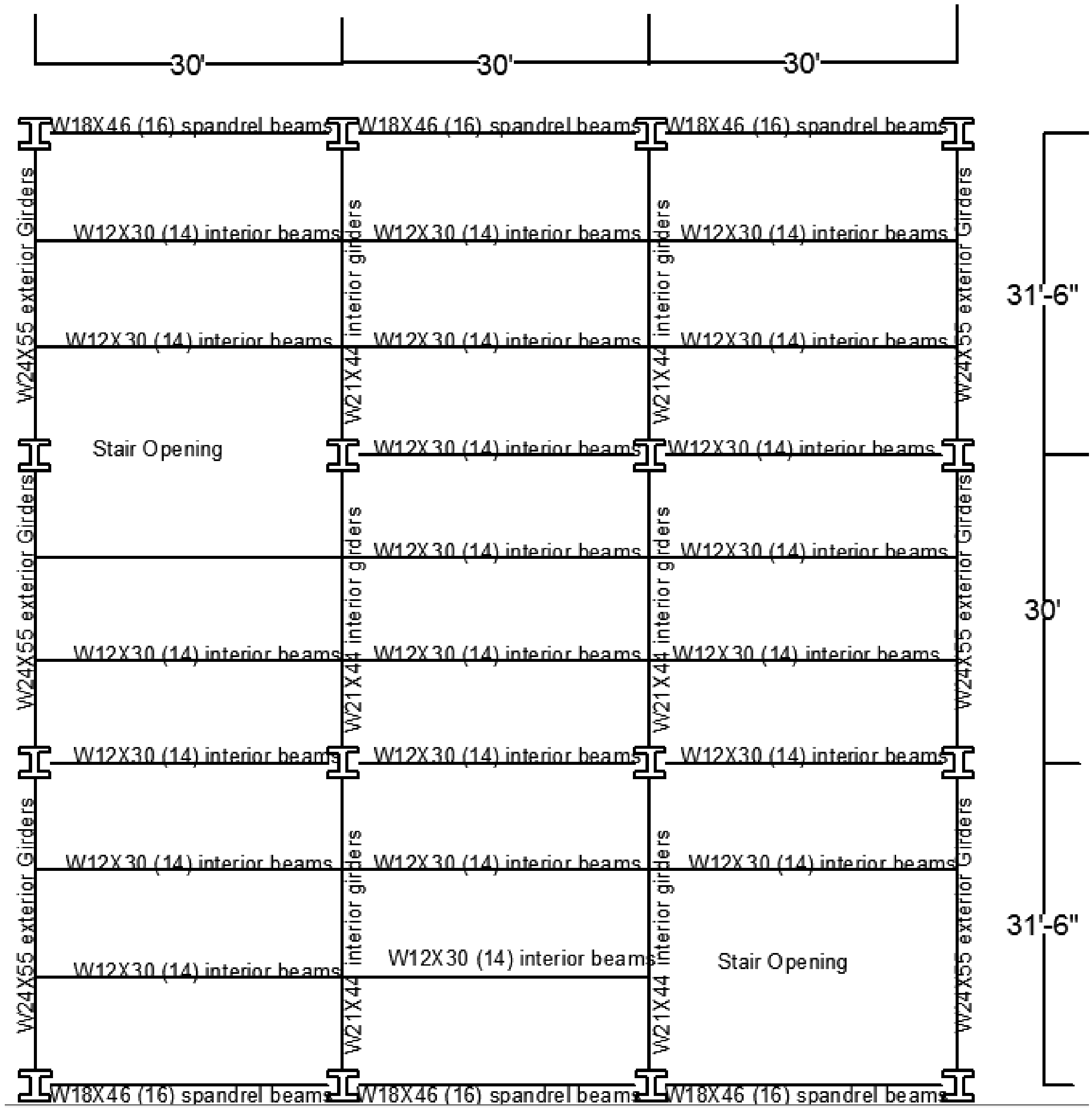


Figure 6: Beam and girder layout for the 5th and 6th floor respectfully. All beams are 30 feet long and are connected to girders on both ends. The numbers in parenthesis on the beams are the number of shear studs needed. Steel decking and 5 inch concrete slabs are to be placed atop this girder and beam 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 ϕf_y 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_x 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, which ever 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 Y2. Y2 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[5]

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.

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[5] "Construction Work: DECKING INSTALLATION." *Construction Work*. Web. 18 Apr. 2012. <http://www.teachconstruction.org/2011/05/decking-instalation.html>.

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 $\sum Q_n$. 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 $\sum Q_n$ 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 2nd through 5th floor slabs were uniform, and exposed to the same loading. The 6th floor had a higher live load and the first floor had higher loading as well. Due to this uniform loading, only the 6th and 5th 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
				28500000			
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.

x=30	alpha=8y/x	2.4	mu= .1	K= .00125
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

Wu=1.2dead + 1.6 live				
Wu=	270	Psf		
Ln=	31			
L (e-w)=	30	Ft		
Pe per strand=		15843 lb		
Assume a Fc of 170				
unit F=	16320			
Pe per strand=		15843		
Fe=F*L	489600			
Number of strands = Fe/Pe			30.9022	31 strands
Pe=Fe=Pe* number of strands			491149	
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=	10	Inches		
Ac=	120	in^2		
New Fc=	136.43			
Assumed starting tendon depth				5 in.
Assumed midpoint tendon depth				9 in.
Assumed tendon highpoint depth				1 in.
a1=a3=	6			
outside spans				
Wbal=8Fa/Ln^2		68.14 psf		
wnet=Ww-Wbal		128.52		
Interior Span				
a2=low point - high point				
8 inches				
Wbal=8Fa/L^2		97.02		
wnet=Ww-Wbal		99.65		

Equivalent frame characteristics

Ks=4EcIc/(Ln-2h) where Ln=lu= 180

all columns are 24x24 c1=c2= 24 inches

Ic= 27648

Assume ec/es=1

total Kc=4EcIc/(Ln-2h)

691.2

From equation 9.10b C=(1-.63x/y)(x^3y/3

x= 10 Inches y= 24 inches

C= 5900

torsional stiffness of the slab at the column line

Kt=sum(9EcsC/L2(1-C2/c1))

Kt= 2177.02

Kec=(1/Kc+1/Kt)^-1

Kec= 524.63

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	0.36
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

	A	B		C	
DF	0.364662	0.267218	0.267218	0.269703	0.267218
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	127.526392	127.5264	89.68453	89.684532	127.5264
DIST	46.5040294	34.07732	23.9653	24.188185	34.07732
CO	17.0386621	23.25201	12.09409	11.982652	
DIST	6.21335261	6.213353	3.231757	3.231757	
Final Mnet	-70.197053	178.6424	-81.0451		

*10^3 per ft

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

				according to figure
Vc=(2+4/beta)sqrt(f'c)bod		beta=	1	9-11
Vc=		bo=30	30	
Vc=(alphasD/bo +2)sqrt(f'c)bod		d=	5	
Vc=		alpha=	40	interior
Vc=4sqrt(f'c)bod			30	exterior
Vc=	42426.4			

Tensile strength at support					
Mnet=Mnet-Vc/3			b=	12	inches
Mnet=	56054.9175		height=	10	inches
S=bh^2/6	S=	200			
ft=-P/A-M/s=		143.84			
Allowable ft=6sqrt(f'c)		424.26			
Tensile Strength at midpoint					
Mnet,max=WL^2/8-FEM					
53481					
midspan ft	130.97				

Design Moments Mu

FEMbal=Wbal*L^2/12

Span AB or CD

FEMbal= 67616.1

Span BC

FEMbal= 87315.47

	A	B		C	
DF	0.364662	0.267218	0.267218	0.269703	0.267218
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	67.6161081	67.61611	87.31547	87.315468	67.61611
DIST	24.6570253	18.06823	23.33225	23.549241	18.06823
CO	9.03411441	12.32851	11.77462	11.666124	

DIST	3.29439824	3.294398	3.146388	3.1463884
Final Mnet *10^3 per ft	-37.219367	94.71845	-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

	A	B		C	
DF	0.364662	0.267218	0.267218	0.269703	0.267218
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	267.9075	267.9075	243	243	267.9075
DIST	97.6956852	71.58966	64.93393	65.537822	71.58966
CO	35.7948287	48.84784	32.76891	32.466965	
DIST	13.0530139	13.05301	8.756437	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

1

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)*(L/4)

Mu= 317700

Mn= 353000

Flexural Strength Mn
As=.00075HLn

2.79	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.

x=30 alpha=8y/x 2.4 mu= .1 K= .00125

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

h= 8 Inches

Ac= 96 in^2 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=1.2dead + 1.6 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=4EcIc/(Ln-2h)			where Ln=lu=	180 inches		Is=bh^3	51840
all columns are 24x24			c1=c2=		24	inches	
Ic=	27648						
Assume ec/es=1							
total Kc=4EcIc/(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 Length for exterior spans 31.5

FEM= 130467.36

Fixed end moment for interior spans

FEM=WL^2/12 Length for interior spans 30

FEM= 92783.66

COF = 0.5

	A		B		C
DF	0.3386459	0.252976	0.252976	0.2552027	0.252976
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	130.467365	130.4674	92.78366	92.783661	130.4674
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	beta=	1	according to figure 9-11
Vc=	bo=30	30	
Vc=(alphasD/bo +2)sqrt(f'c)bod	d=	6	
Vc=	alpha=	40	interior
Vc=4sqrt(f'c)bod		30	exterior
Vc= 50911.69			

Slab concrete tensile stress at support

Tensile strength at support					
Mnet=Mnet-Vc/3			b=	12	inches
Mnet=	58400.51		height=	12	inches
S=bh^2/6	S=	288			
ft=-P/A-M/s=		89.09			
Allowable ft=6sqrt(f'c)		424.26			

Tensile Strength at midpoint

Mnet,max=WL^2/8-FEM

55029.524

midspan ft	77.38

Design Moments Mu

FEMbal=Wbal*L^2/12

Span AB or CD

FEMbal= 84520.1351

Span BC

FEMbal= 102216.339

	A		B		C
DF	0.3386459	0.252976	0.252976	0.2552027	0.252976
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	84.5201351	84.52014	102.2163	102.21634	84.52014
DIST	28.6223972	21.3816	25.85833	26.085883	21.3816
CO	10.6908023	14.3112	13.04294	12.929164	
DIST	3.62039635	3.620396	3.299557	3.2995572	
Final Mnet *10^3 per ft	-48.827332	116.5925	-92.7005		

Span AB	
e=	0
Mbal=	48827.33
Ms=	48827.33
FEMu=WuL^2/12	
FEMu=	299659.5
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

	A		B		C
DF	0.3386459	0.252976	0.252976	0.2552027	0.252976
COF	0.5	0.5	0.5	0.5	0.5
FEM X10^3 in-lb	299.6595	299.6595	271.8	271.8	299.6595
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	-	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)

Vbc=w*24/2

3624

Mu=	235654.524		
Req. Mu	206662.524		
Req. Mn	229625.026		

Factored Shear			Ms
ab	4120.90151		48827.33
ba	5392.09849		34734.29
bc	3624		10842.26

Maximum positive moment Span AB

x=Vab/Wu

12 ft.

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

Mu= 82505.0366

Req. positive Mn= 91672.26

Maximum positive moment span BC

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

Mu= 359820

Mn= 399800

Flexural Strength Mn

As=.00075HLn

3.348 inches ^2

try #4

bars area= 0.196

17.051224 needs 18 bars at .3 inches a piece

As= 3.53

30 Ft panel 30 ft

As per foot= 0.118 in^2

Pp=Aps/bd

0.0012

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

127467.74

Fps=fps*Aps*strand number/L

20152.65

Fs=60,000*As/ft

7068.5835

total force F/ft= 27221.23

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

0.53 inches

Bars and tendons should be placed at 12'-1', or 11 inch depth			11	inches	
available Mn=	292168.9				
required Mn=	229625				
so no more moment strength is needed					
a=Apsfps/.85f'cb	0.14				
available -Mn=Apsfps(d-a/2)					
negative Mn=					
220282.58					
This is less than the required positive Mn, so it is unsatisfactory					
try adding a #5 bar					
As=	0.31				
Asfy=	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				53114.29	19121.14
.7*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

Column Design		2nd floor interior				
length	10.5	Ft				
f'c	5000	psi(Assumed)				
			As	0.153	in^2	Phi=
F'ci	3333.333	PSI	Es, Eps	2.85E+08	Psi	alpha=
Fy	60000	PSI	Fpu	270000		pg=
fps=.7*fpu		189000	Psi			beta=

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
total Pu=	1617639	lbs
	1617.6	kips

Trial size

Ag=Pu/Φ*α*(.85f'c+pg(Fy-.85f'c)

Ag=	525.26	in^2	22.9	576

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 12 #11 bars
As= 18.72
Check using ΦPn=.8*Φ[.85*f'c(Ag-Ast)+fyAst]
.8*.65*(.85*5000*(576-18.72)+(60*18.72))
ΦPn= 1815.6 Kips > 1617.6 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

P-M diagrams	(Fs=Fy)	
assume εu=		0.003
εy=Fu/Es =		0.0021053

d=h-cover-diameter of tie-1/2 diameter of reinforcing bar
d'= 22.795 In
Cb= 13.395 In

depth a=.85*Cb 11.4 in
f's= εu*Es(Cb-d')/Cb
77.8 ≤ 60 ksi
C=.85f'c*a*h
1161347 lbs As=4
1161.347 kips A's=4
Pn=.85*f'c*a*b+AsFs-A'sF's
Pn= 1161.347
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
12506.36 kip inches

1042.196	kip feet			
e=		10.76	Inches	

Choose a C smaller than the previous C

Cb=		5	In	
depth a=.85*Cb			4.25	in
f's= εu*Es(Cb-d')/Cb				
65		≤	60	ksi
C=.85f'c*a*h				
94828.13	lbs			As=3
94.82	kips			A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's				
Pn=		433.5		
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)				
920.81	kip inches			
76.73	kip feet			
e=	2.124135	Inches		

Choose a C larger than the previous C

Cb=		24	In	
depth a=.85*Cb			20.4	in
fs= εu*Es(d'-C)/C				
-4.29281				
4.292812				
f's= εu*Es(Cb-d')/Cb				
56.9875		≤	60	ksi
C=.85f'c*a*h				
2080800	lbs			As=3
2080.8	kips			A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's				
Pn=	2291.579			
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)				
6686.895	kip inches			
557.2413	kip feet			
e=		2.91803	Inches	

Set C to infinite and e=0

Fs		=	60	ksi
Pn=.85*f'c*a*b+AsFs				
Pn=		3168		

Second Floor Exterior Middle Column Excel Spreadsheet

Column Design		2nd floor exterior centered column					
length	10.5	Ft					
f'c	5000	psi(Assumed)					
			As		0.153	in^2	Phi=
	3333.33						
F'ci	3	PSI	Es, Eps		2.85E+08	Psi	alpha=
Fy	60000	PSI	Fpu		270000		pg=
fps=.7*fpu		189000	Psi				beta=
Effective area of support							
17.5 ft by 30 ft		525	ft^2	assume a value of 500 pounds per linear foot per floor for outside façade			
Roof load=		466.3	psf		60000		(16.75)*500*4 floors
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)							
Ag=	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							
l/h=	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)

As= pg*area of the column
9.72 in^2
0.81
diameter of steel= 1.27
Nominal Area= 1.27
use 8 #10 bars
As= 10.16
Check using ΦPn=.8*Φ[.85*f'c(Ag-Ast)+fyAst]
.8*.65*(.85*5000*(576-18.72)+(60*18.72))
ΦPn= 1010.5 Kips > 980.1325 so the section checks
use #3 lateral ties.
diameter ties= 0.375 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	(Fs=Fy)
assume εu=	0.003
εy=Fu/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

8.486242
depth a=.85*Cb 3 in
f's= εu*Es(Cb-d')/Cb
25.4673 ≤ 60 ksi
C=.85f'c*a*h
649197 lbs 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 As=3
202.3 kips A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's
Pn= 520.2
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
2510.9 kip inches
209.2 kip feet
e= 4.82689 Inches

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 As=3
1170.45 kips A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's
1265.95
Pn= 8
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
2323 kip inches

193.5	kip feet		
e=	1.83492	Inches	

Set C to infinite and e=0			
Fs	=	60	ksi
Pn=.85*f'c*a*b+AsFs			
Pn=	1857		

2nd Floor Edge

Column Design		2nd floor edge					
length	10.5	ft					
f'c	5000	psi(Assumed)					
			As	0.153	in^2	Phi=	0.65
	3333.33						
F'ci	3	PSI	Es, Eps	2.85E+08	Psi	alpha=	0.8
Fy	60000	PSI	Fpu	270000		pg=	0.03
fps=.7*fpu		189000	Psi			beta=	1.2

Effective area of support

17.5 ft by 16.75 ft	293.125	ft^2	assume a value of 500 pounds per linear foot per floor for outside façade		
Roof load=	466.3	psf	68500		(17.5+16.75)*500*4 floors
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=	591592.4	lbs
	591.5924	kips

Trial size

Ag=Pu/Φ*α*(.85f'c+pg(Fy-.85f'c)

	192.094			
Ag=	3	in^2	13.85	196

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

Ag=	196	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)

As= pg*area of the column
5.88 in^2
0.49
diameter of steel= 1.41
Nominal Area= 1.56
use 4 #4 bars
As= 6.24
Check using ΦPn=.8*Φ[.85*f'c(Ag-Ast)+fyAst]
.8*.65*(.85*5000*(576-18.72)+(60*18.72))
ΦPn= 614.056 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

P-M diagrams	(Fs=Fy)
assume εu=	0.003
εy=Fu/E	0.002105
s =	3

d=h-cover-diameter of tie-1/2 diameter of reinforcing bar
d'= 12.92 in
Cb= 7.59215 in

depth a=.85*Cb 6.4533402 in
f's= εu*Es(Cb-d')/Cb
39.2 ≤ 60 ksi
C=.85f'c*a*h

383973.
7 lbs As=4
383.973
7 kips A's=4
Pn=.85*f'c*a*b+AsFs-A'sF's
185.416
Pn= 5
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
1304.31
4 kip inches
108.692
9 kip feet
e= 7 inches

Choose a C smaller than the previous C
Cb= 5 in

depth a=.85*Cb 4.25 in
f's= εu*Es(Cb-
d')/Cb
-
103.968 ≤ 60 ksi
C=.85f'c*a*h
162562.
5 lbs As=3
162.562
5 kips A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's
Pn= -75.061
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
-
447.234 kip inches
-
37.2695 kip feet
e= 5.95829 inches

Choose a C larger than the previous C
Cb= 14 in

depth a=.85*Cb 11.9 in
fs= εu*Es(d'-C)/C
-
6.59571
6.59571
4

f's= εu*Es(Cb-d')/Cb

12.5 ≤ 60 ksi

C=.85f'c*a*h

708050 lbs As=3

708.05 kips A's=3

Pn=.85*f'c*a*b+AsFs-A'sF's

719.887

Pn= 1

Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)

760.708 kip inches

63.3923 kip feet

e= 1.05673 inches

Set C to infinite and e=0

Fs = 60 ksi

Pn=.85*f'c*a*b+AsFs

Pn= 1073

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

Column Design		2nd floor edge					
length	10.5	ft					
f'c	5000	psi(Assumed)					
			As	0.153	in^2	Phi=	0.65
	3333.33						
F'ci	3	PSI	Es, Eps	2.85E+08	Psi	alpha=	0.8
Fy	60000	PSI	Fpu	270000		pg=	0.03
fps=.7*fpu		189000	Psi			beta=	1.2

Effective area of support

17.5 ft by 16.75 ft	293.125	ft^2	assume a value of 500 pounds per linear foot per floor for outside façade	
Roof load=	466.3	psf	68500	(17.5+16.75)*500*4 floors
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	591592.9	
Pu=	4	lbs
	591.5929	
	4	kips

Trial size

Ag=Pu/Φ*α*(.85f'c+pg(Fy-.85f'c)

			13.859	
Ag=	192.093	in^2	1	576

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=.8*Φ[.85*f'c(Ag-Ast)+fyAst]

.8*.65*(.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

P-M diagrams	(Fs=Fy)
assume εu=	0.003
εy=Fu/E	0.002105
s =	3

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

d'=	22.92	in
	13.4684	
Cb=	5	in

depth a=.85*Cb	11.4	in

f's= εu*Es(Cb-d')/Cb

78.6439 8	≤	60	ksi

C=.85f'c*a*h

116771 5	lbs	As=4
1167.71 5	kips	A's=4

Pn=.85*f'c*a*b+AsFs-A'sF's

Pn=	1167.71 5

Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)

9949.27 1	kip inches
829.105 9	kip feet

e=	8.5 1	inches

Choose a C smaller than the previous C		
Cb=	5	in

depth a=.85*Cb 4.25 in
f's= εu*Es(Cb-d')/Cb
67.032 ≤ 60 ksi
C=.85f'c*a*h
94828.13 lbs As=3
94.82813 kips A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's
Pn= 553.38
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
2600.8 kip inches
216.734 kip feet
e= 4.699867 inches

Choose a C larger than the previous C
Cb= 24 in

depth a=.85*Cb 20.4 in
fs= εu*Es(d'-C)/C
-3.8475
3.8475
f's= εu*Es(Cb-d')/Cb
57.3 ≤ 60 ksi
C=.85f'c*a*h
208080 lbs As=3
2080.8 kips A's=3
Pn=.85*f'c*a*b+AsFs-A'sF's
Pn= 2187.705
Mn=Pn*e=.85*f'c*a*b(H/2-a/2)+a'sf's(h/2-d')+AsFs(d-h/2)
5212.98 kip inches
434.415 kip feet
e= 2.33 inches

Set C to infinite and e=0

Shear Wall Excel Spreadsheet

Shear wall design
https://engineering.purdue.edu/~frosch/
CE576/Time%20Saving%20Design%20Tips/Time%20Saving-Columns&Walls.pdf

Givens				
f'c	5000	psi		
Ss	0.24	Sds	0.256	g
S1	0.067	Sd1	0.107	g
R	1.5	at existing	5	at addition
Cd	1.5	at existing	4.5	at addition
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	
2-5th floor				
	125	psf		
+	15		140	
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		

	315	sq ft

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

First floor load	251.1918	kips
second	242.1846	kips
third	260.9334	kips
fourth	279.6822	kips
fifth	298.431	kips
6th	375.0678	kips

Check shear strength in 1st story
1707.4908 kips
Vu=1.3*total shear
2219.73804 kips
from table 6
ΦVc=30*12.4
762.6 kips
ΦVs=AvFyd/s
number 6 bars

Av=	0.441786
Fy	60
d=	9.6
s=	12
ΦVs	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	foot kips
2nd floor			
Pu=	824040	824.04	kips
Mu=	395064	395.064	ft. Kips
3rd floor		0	
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	
c/lw=	0.174441	
ϕMn=	169993.8	in. kips
	14166.15	ft-kips

moment is sufficient

2nd floor moment strength

Ast=	13.25359
ω =	0.052594
α=	0.068125
c/lw=	0.153745
ϕMn=	76313.12
	6359.427

Moment is sufficient

3rd floor moment strength

Ast=	13.25359
ω =	0.052594
α=	0.051875
c/lw=	0.133049
ϕMn=	78179.41
	6514.95

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

DL	Slab	Services	Ceiling	Length	30
754.16	60.41667	10	5	Spacing	10
lbs/ft					

LL	Load
525	70
lbs/ft	

30' BEAM

PLASTIC CAPACITY CALCULATIONS

Load Combinations

U=1.2D+1.6LL

U	1745	lbs/ft

Mu=wL^2/8

Mu 196312.5

Mu	196.3	ft/k

Zx required

Zx =Mu/Φfy

Zx	52.35	in^3

Off this we pick a W 18X35

with :

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

New DL calculations

New DL=original DL + weight of beam

New DL	789.16	lbs/ft

Live load is the same

New U

U=1.2D+1.6LL

U	1787	lbs/ft

Mu=wL^2/8

Mu 201037.5

Mu	201	ft/k

ΦMp=ΦZx*Fy
ΦMp 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

Limit	1	Inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx			Ix	510	in^4
Δ	0.32	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	1.25	Inch

So the current beam is sufficient by Deflection

31.5' GIRDER

DL superimposed					
	Slab	Services	Ceiling	Length	31.5
75.416	60.41667	10	5	Spacing	30

LL	Load
2100	70

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

U	6201	lbs/ft

Mu=wL^2/8

Mu 769117.8

Mu	769.1	ft/k

Zx required

Zx =Mu/Φfy

Zx	205.1	in^3

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 W 33X130

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

Limit=L/360

Limit	1.05	Inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx

Δ	0.3	Inch

Ix 2370 in^4

from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit=L/240

Limit	1.575	Inch

Δ=5wL^4/384EIx

Δ	1.067663	Inch

So the current girder is sufficient by Deflection

FINAL beam and girder size	
Typical Beam	W 18X35
Typical Girder	W 24X84

6th Floor Steel Beam and Girder Excel Spreadsheet

Scheme 1

DL	Slab	Services	Ceiling	Length	30
754.1667	60.41667	10	5	Spacing	10
lbs/ft					

LL	Load
750	
1,000???	100
lbs/ft	

30' BEAM

PLASTIC CAPACITY CALCULATIONS

Load Combinations

U=1.2D+1.6LL

U	2105	lbs/ft

Mu=wL^2/8

Mu 236812.5

Mu	236.8	ft/k

Zx required

Zx =Mu/Φfy

Zx	63.15	in^3

Off this we pick a W 18X35

with :

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

New DL calculations

New DL=original DL + weight of beam

New DL	789.16	lbs/ft

Live load is the same

New U

U=1.2D+1.6LL

U	2147	lbs/ft

Mu=wL^2/8

Mu 241537.5

Mu	241.5	ft/k

ΦMp=ΦZx*Fy

ΦMp 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

Limit	1	Inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx			Ix	510	in^4
Δ	0.46	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	1.391417	Inch

So the current beam is sufficient by Deflection

31.5' GIRDER

DL superimposed					
	Slab	Services	Ceiling	Length	31.5
75.416	60.416	10	5	Spacing	30

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

U	7641	lbs/ft

Mu=wL^2/8

Mu 947722.8

Mu	947.7	ft/k

Zx required

Zx =Mu/Φfy

Zx	252.7	in^3

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

Limit	1.05	inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx

Δ	0.32	inch

Ix 3610 in^4

from Table 3.3

So we are okay

Case II Full DL + Half Live

Limit=L/240

Limit	1.575	inch
Δ=5wL^4/384EIx		
Δ	0.79	inch

So the current girder is sufficient by Deflection

FINAL beam and girder size	
Typical Beam	W 18X35
Typical Girder	W 30X90

5 inch Slab and Steel Decking for 5th floor 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.	0.44	inches^2			

Super Imposed Dead Load

			Concrete	Services	ceilings	Type
Total DL Unfactored	754.16	plf	60.41667	10	5	Load
			psf	psf	psf	Unit
Total LL Unfactored	700	plf				

plf= pounds per linear foot

Load Combinations

U=1.2D+1.6LL Governs

U	2025	plf

Mu=wL^2/8

Mu	227812.5	
Mu	227.8	ft*k

Picking a Preliminary Beam Size

Assuming a=1 1

Be Calculations

Be/2≤L/8≤beamspacing/2

L/8	3.75	
beams pacing/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	Picked	W 12X22			
Depth	12.3	inches	Area Steel	6.48	in^2
Weight	22	lbs/ft			

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.0	inches
New Y2=	4.5	
Section Still Okay		

e Calculation

e=1/2(t concrete)+depth of beam/2

e=	8.65	inches

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	324	k

Capacity Calculation

ΦMn=e*ΣQn*Φ

ΦMn=	2522.34	
ΦMn=	210.195	ft*k

Section is inadequate

Pick a new beam

W 12X30

Depth	12.3	inches	Area Steel	8.79	in^2
Weight	30	lbs/ft			

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.4	inches
New Y2=	4.3	

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	439.5	k

ΦMn=e*ΣQn*Φ

ΦMn= 3421.5075

ΦMn=	285.1	ft*k

Beam is now Adequate

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

Deflection Calculations

Δ=5wL^4/384EIx

Δ	0.29	inch

Lower Bound I

I_{LB}	737	in^4

from Table 3-20

Design of Shear Studs

Stud Capacity=.5*Area steel stud*sqrt(F'c*Ec)≤Area steel stud*Fu*Rg*Rp

Stud Capacity= 26.1 ≤ 21.5

Rg	1	for solid slab
Rp	0.75	for solid slab

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

Ec=	3492	ksi
w	145	pcf

So limiting stud capacity=	21.5	k
	20.4	Studs
	21	Studs

Construction Capacity

Unshored Construction

	Beam Weight				
Construction DL	30	plf			
	Wet Concrete		Construction LL		
Construction LL	60.416	psf	20	psf	
804.16 plf					

Load Combinations		
U=1.2D+1.6LL	Governs	
U	1322.66	lbs/ft

Mu=wL^2/8		
Mu	148800	
Mu	148.8	ft*k

For this Beam	ΦMp=	162	ft*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					
Δ=5wL^4/384EIx			Lower Bound I	I_{LB} 1440	in^4
Δ	0.35	inch		from Table 3-20	

Required Ix calculations

Design shear anchors for full composite		
ΣQn Calculation		
ΣQn=As*Fy		
ΣQn=	439.5	k
Stud Capacity is the Same		
# of Studs=Qn/Stud Capacity		
# of Studs	20.40665636	
Boom	21	Studs

Check if full composite is possible

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

a=	1.4	inches
t concrete=	5	inches

So still okay with slab depth

Needed Composite Capacity

Super Imposed Dead Load

			Concrete	Services	Ceilings	Beam	Type
Total DL Unfactored	784.16	plf	60.41667	10	5	30	Load
			psf	psf	psf	plf	Unit
Total LL Unfactored	700	plf					

Load Combinations

U=1.2D+1.6LL Governs

U	2061	lbs/ft

Mu=wL^2/8

Mu 231862.5

Mu	231.8	ft*k

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.436	inches
New Y2=	4.281	inches

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

We need an MU of231.8 ft*k

So a PNA location of :	6

will be sufficient for our needs

At location 7

ΦMn	238	ft*k
ΣQn	131	k

Check for Deflection Serviceability

Live Load Deflection

Limit=L/360

	Limit=	1	inches
Max Limit		1	inch

Usable Limit	1	inch

Deflection Calculations

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

Δ	0.486	inch

Beam is sufficient

Lower Bound I

I_{LB}	452	in^4

from Table 3-20

# of Studs Calculation

Stud Capacity is the Same

# of Studs=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	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

			Concrete	Services	ceilings	Type
Total DL Unfactored	754.1666667	plf	60.41667	10	5	Load
			psf	psf	psf	Unit
Total LL Unfactored	1000	plf				

plf= pounds per linear foot

Load Combinations

U=1.2D+1.6LL Governs

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≤beamspacing/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	Picked	W 12X22			
Depth	12.3	inches	Area Steel	6.48	in^2
Weight	22	lbs/ft			

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.058824	inches

New Y2= 4.470588

Section Still Okay

e Calculation

e=1/2(t concrete)+depth of beam/2

e=	8.65	inches

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	324	k

Capacity Calculation

ΦMn=e*ΣQn*Φ

ΦMn= 2522.34

ΦMn=	210.195	ft*k

Section is inadequate

Pick a new beam

W 12X30

Depth	12.3	inches	Area Steel	8.79	in^2
Weight	30	lbs/ft			

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.436275	inches

New Y2= 4.281863

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	439.5	k

ΦMn=e*ΣQn*Φ

ΦMn= 3421.5075

ΦMn=	285.125625	ft*k

Beam is now Adequate

Limit=	1	inches
Max Limit	1	inch

Usable Limit	1	inch

Deflection Calculations

Δ=5wL^4/384EIx

Δ	0.426355682	inch

Lower Bound I

I_{LB}	737	in^4

from Table 3-20

Design of Shear Studs

Stud Capacity=.5*Area steel stud*sqrt(F'c*Ec)≤Area steel stud*Fu*Rg*Rp

Stud Capacity= 26.10678591 ≤ 21.53709

Rg	1	for solid slab
Rp	0.75	for solid slab

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

Ec=	3492.062428	ksi

w	145	pcf

So limiting stud capacity=	21.53709	k
	20.40666	Studs
	21	Studs

Construction Capacity

Unshored Construction

	Beam Weight	
Construction DL	30	plf

Wet Concrete		Construction LL	
60.41666667	psf	20	psf

Construction LL
804.1666667 plf

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

ΦMp=	162	ft*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

Δ=5wL^4/384EIx

Δ	0.350956358	inch

Lower Bound I

I_{LB}	1440	in^4

from Table 3-20

Required Ix calculations

Design shear anchors for full composite

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	439.5	k

Stud Capacity is the Same

# of Studs=Qn/Stud Capacity

# of Studs 20.40665636

Boom	21	Studs

Check if full composite is possible

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

a=	1.43627451	inches

t concrete=	5	inches

So still okay with slab depth

Needed Composite Capacity

Super Imposed Dead Load

			Concrete	Services	Ceilings	Beam	Type
Total DL Unfactored	784.1666667	plf	60.41667	10	5	30	Load
			psf	psf	psf	plf	Unit
Total LL Unfactored	700	plf					

Load Combinations

U=1.2D+1.6LL Governs

U	2061	lbs/ft

Mu=wL^2/8

Mu	231862.5	
Mu	231.8625	ft*k

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.43627451	inches
New Y2=	4.281862745	inches

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

We need an MU of231.8 ft*k

So a PNA location of :	6

will be sufficient for our needs

At location 7

ΦMn	238	ft*k
ΣQn	131	k

Check for Deflection Serviceability

Live Load Deflection

Limit=L/360

Limit=	1	inches
Max Limit	1	inch

Usable Limit	1	inch

Deflection Calculations

Δ=5wL^4/384EIx

Δ	0.486630302	inch

Beam is sufficient

Lower Bound I

I_{LB}	452	in^4

from Table 3-20

# of Studs Calculation

Stud Capacity is the Same

# of Studs=Qn/Stud Capacity

# of Studs	6.08253011	
Boom	7	Studs

Final Beam size
W12X30 (14)
3/4 inch studs

Fifth floor Girder Design Excel Spreadsheet

Scheme
1

DL	Slab	Services	Ceiling	Beam DL	Wall	Length	30
1581.25	60.41667	10	5	30		Spacing	15

lbs/ft

LL	Load
525	70

lbs/ft

31.5" girder

PLASTIC CAPACITY CALCULATIONS

Load Combinations

U=1.2D+1.6LL

U 2737.5 lbs/ft

Mu=wL^2/8

Mu 307968.8

Mu 307.9688 ft/k

Zx required

Zx =Mu/Φfy

Zx 82.125 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

New DL 1625.25 lbs/ft

Live load is the same

New U

U=1.2D+1.6LL

U 2790.3 lbs/ft

Mu=wL^2/8

Mu	313908.8	
Mu	313.9088	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

Limit 1 inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx			Ix	842	in^4
Δ	0.195924	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 1.37613 inch

So the current girder is sufficient by Deflection

Sixth Floor Girder Design Excel Spreadsheet

Scheme
1

DL	Slab	Services	Ceiling	Beam DL	Wall	Length	30
1581.25	60.41667	10	5	30		Spacing	15
lbs/ft							

LL	Load
750	100
lbs/ft	

31.5" girder spandrel

PLASTIC CAPACITY CALCULATIONS

Load Combinations

U=1.2D+1.6LL

U	3097.5	lbs/ft

Mu=wL^2/2/8

Mu	348468.8	
Mu	348.4688	ft/k

Zx required

Zx =Mu/Φfy

Zx	92.925	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

New DL	1625.25	lbs/ft

Live load is the same

New U

U=1.2D+1.6LL

U	3150.3	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

Limit 1 inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx

Δ 0.279891 inch

Ix 842 in^4

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 1.460097 inch

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

			Concrete	Services	ceilings	Type	shear walls
Total DL Unfactored	1069.166667	plf	60.41667	10	5	Load	315
			psf	psf	psf	Unit	
Total LL Unfactored	700	plf					

plf= pounds per linear foot

Load Combinations

U=1.2D+1.6LL	Governs	
U	2403	plf
Mu=wL^2/8		
Mu	270337.5	
Mu	270.3375	ft*k

Picking a Preliminary Beam Size

Assuming a=1 1

Be Calculations

Be/2≤L/8≤beamspacing/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	Picked	W 18X40			
Depth	17.9	inches	Area Steel	11.8	in^2
Weight	40	lbs/ft			

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.928105	inches

New Y2= 4.035948

Section Still Okay

e Calculation

e=1/2(t concrete)+depth of beam/2

e=	11.45	inches

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	590	k

Capacity Calculation

ΦMn=e*ΣQn*Φ

ΦMn= 6079.95

ΦMn=	506.6625	ft*k

Limit=	1	inches
Max Limit	1	inch

Usable Limit	1	inch

Deflection Calculations

Δ=5wL^4/384EIx

Lower Bound I

I_{LB}	1600	in^4

from Table 3-20

Δ	0.13747306	inch

Design of Shear Studs

Stud Capacity=.5*Area steel stud*sqrt(F'c*Ec)≤Area steel stud*Fu*Rg*Rp

Stud Capacity= 26.10678591 ≤ 21.53709

Rg	1	for solid

Rp	0.75	slab for solid slab

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			
Construction DL	30	plf		
	Wet Concrete		Construction LL	
Construction LL 804.1666667 plf	60.41666667	psf	20	psf

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	ΦMp=	289	ft*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

Δ=5wL^4/384EIx			Lower Bound I		I_{LB}	1600 in^4
Δ	0.315860722	inch				from Table 3-20

Required Ix calculations

Design shear anchors for full composite

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	590	k

Stud Capacity is the Same

# of Studs=Qn/Stud Capacity

# of Studs	27.39460126	
Boom	28	Studs

Check if full composite is possible

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

a=	1.928104575	inches
t concrete=	5	inches

So still okay with slab depth

Needed Composite Capacity

Super Imposed Dead Load			Concrete	Services	Ceilings	Beam	Type
Total DL Unfactored	1131.166667	plf	60.41667	10	5	62	Load
			psf	psf	psf	plf	Unit
Total LL Unfactored	700	plf					

Load Combinations

U=1.2D+1.6LL Governs

U	2477.4	lbs/ft

Mu=wL^2/8

Mu	278707.5	
Mu	278.7075	ft*k

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	1.928104575	inches
New Y2=	4.035947712	inches

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

We need an MU of278.70 ft*k

So a PNA location of :	7

will be sufficient for our needs

At location 7

ΦMn	412	ft*k
ΣQn	148	k

Check for Deflection Serviceability

Live Load Deflection

Limit=L/360

Limit=	1	inches
Max Limit	1	inch

Usable Limit	1	inch

Deflection Calculations

Δ=5wL^4/384EIx

Δ	0.13747306	inch

Lower Bound I

I_{LB}	1600	in^4

from Table 3-20

Beam is sufficient

# of Studs Calculation

Stud Capacity is the Same

# of Studs=Qn/Stud Capacity

# of Studs	6.871866078	
Boom	7	Studs

Final Beam size
W18X40(14)
3/4 inch studs

Sixth Floor Spandrel Beam Excel Spreadsheet

GIVENS:

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

Super Imposed Dead Load

			Concrete	Services	ceilings	Type	shear walls
Total DL Unfactored	1069.166667	plf	60.41667	10	5	Load	315
			psf	psf	psf	Unit	
Total LL Unfactored	1000	plf					
			plf=	pounds per linear foot			

Load Combinations

U=1.2D+1.6LL	Governs	
U	2883	plf

Mu=wL^2/8		
Mu	324337.5	
Mu	324.3375	ft*k

Picking a Preliminary Beam Size

Assuming a=1	1

Be Calculations

Be/2≤L/8≤beamspacing/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	Picked	W 18X46			
Depth	18.1	inches	Area Steel	13.5	in^2
Weight	46	lbs/ft			

a Calculation

a=Fy*As/(.85*F'c*Be)

	2.20588	
a=	2	inches
	3.89705	
New Y2=	9	
Section Still Okay		

e Calculation

e=1/2(t concrete)+depth of beam/2

e=	11.55	inches

ΣQn Calculation

ΣQn=As*Fy

ΣQn=	675	k

Capacity Calculation

ΦMn=e*ΣQn*Φ

ΦMn= 7016.625

ΦMn=	584.71875	ft*k

Beam is now Adequate

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

Deflection Calculations

Δ=5wL^4/384EIx

	0.44132603	
Δ	6	inch

Lower Bound I

I_{LB}	712	in^4

from Table 3-20

Design of Shear Studs

Stud Capacity=.5*Area steel stud*sqrt(F'c*Ec)≤Area steel stud*Fu*Rg*Rp

Stud Capacity=	26.1067859 1	≤	21.5370 9

Rg	1	for solid slab
Rp	0.75	for solid slab

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

Ec=	3492.06242 8	ksi

w	145	pcf

So limiting stud capacity=	21.5370 9	k
	31.3412 8	Studs
	43	Studs

Construction Capacity

Unshored Construction

	Beam Weight			
Construction DL	30	plf		
	Wet Concrete		Construction LL	
Construction LL	60.4166666 7	psf	20	psf
804.1666667 plf				

Load Combinations

U=1.2D+1.6LL Governs

U	1322.66666 7	lbs/ft

Mu=wL^2/8

Mu	148800	
Mu	148.8	ft*k

For this Beam	ΦMp=	162	ft*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				
Δ=5wL^4/384EIx			Lower Bound I	I_{LB} 712 in^4
	0.70979937			
Δ	5	inch		from Table 3-20

Required Ix calculations

Design shear anchors for full composite		
ΣQn Calculation		
ΣQn=As*Fy		
ΣQn=	675	k

Stud Capacity is the Same		
# of Studs=Qn/Stud Capacity		
# of Studs	31.3412811	
Boom	32	Studs

Check if full composite is possible		
a=Fy*AS/.85*F'c*bE		
	2.20588235	
a=	3	inches
t concrete=	5	inches
So still okay with slab depth		

Needed Composite Capacity							
Super Imposed Dead Load			Concrete	Services	Ceilings	Beam	Type
Total DL Unfactored	1131.16666	plf	60.4166	10	5	62	Load

	7	7				
		psf	psf	psf	plf	Unit

Total LL Unfactored	700	plf

Load Combinations

U=1.2D+1.6LL Governs

U	2477.4	lbs/ft

Mu=wL^2/8

Mu	278707.5	
Mu	278.7075	ft*k

a Calculation

a=Fy*As/(.85*F'c*Be)

a=	2.205882353	inches
New Y2=	3.897058824	inches

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

We need an MU of278.70 ft*k

So a PNA location of :	7

will be sufficient for our needs

At location 7

ΦMn	475	ft*k
ΣQn	169	k

Check for Deflection Serviceability

Live Load Deflection

Limit=L/360

Limit=	1	inches
Max Limit	1	inch

Usable Limit	1	inch

Deflection Calculations

Δ=5wL^4/384EIx			Lower Bound I		I_{LB}	1170 in^4
	0.18799734					
Δ	7	inch				from Table 3-20
Beam is sufficient						

# of Studs Calculation

Stud Capacity is the Same

# of Studs=Qn/Stud Capacity		
	7.84692815	
# of Studs	7	
Boom	8	Studs

Final Beam size
W18X46 (16)
3/4 inch studs

Fifth Floor Spandrel Girder Excel Spreadsheet

Scheme 1

DL	Slab	Services	Ceiling	Beam DL	Wall	Length	30
2121.25	60.41667	10	5	45	315	Spacing	15
lbs/ft							

LL	Load
525	70
lbs/ft	

31.5" girder spandrel

PLASTIC CAPACITY CALCULATIONS

Load Combinations

U=1.2D+1.6LL

U 3385.5 lbs/ft

Mu=wL^2/8

Mu 380868.8

Mu 380.8688 ft/k

Zx required

Zx =Mu/Φfy

Zx 101.565 in^3

Off this we pick a W 21X55

with :

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=1.2D+1.6LL

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

				Beam			
DL	Slab	Services	Ceiling	DL	Wall	Length	30
2121.25	60.41667	10	5	45	315	Spacing	15
lbs/ft							

LL	Load
1000	100
lbs/ft	

31.5" girder spandrel

PLASTIC CAPACITY CALCULATIONS

Load Combinations

U=1.2D+1.6LL

U	4145.5	lbs/ft

Mu=wL^2/8

Mu 466368.8

Mu	466.3688	ft/k

Zx required

Zx =Mu/Φfy

Zx	124.365	in^3

Off this we pick a W 24X55

with :

weight per foot	55	lbs/ft
Zx	134	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=1.2D+1.6LL

U	4211.5	lbs/ft

Mu=wL^2/8

Mu	473793.8	
Mu	473.7938	ft/k

ΦMp=ΦZx*Fy

ΦMp 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

Limit	1	inch

So the deflection limit is the maximum, 1"

Δ=5wL^4/384EIx			Ix	1350	in^4
Δ	0.232759	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	1.220237	inch

So the current girder is sufficient by Deflection