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# Technical Report II

Simmons College School of Management, Boston, Ma

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## Executive Summary: Technical Report I

The Simmons College School of Management is a newly constructed five story educational facility located in Boston, Massachusetts. The building is 65,000 SF and sits on the south east corner of a five level below grade parking garage. Accommodations have been made in the original design for a future expansion of the building which would top out at a nine story building.

The below grade parking garage is a post tensioned concrete system with a slurry wall as the exterior foundation wall system. Interior columns are W14 shapes extend into the ground to form load bearing element foundations. At the plaza level provisions were made for the use of a crane in the construction of the above grade building. The five story building is steel with composite floors and primarily uses wide flange shapes. Lateral forces are resisted by a combination of braced frames and moment frames.

In this technical report, alternative floor framing options were investigated for the feasibility of potential design implementation. Each floor system was preliminarily designed for approximate member sizes to be expected if a full design were to be carried out. The systems were then evaluated for their advantages, disadvantages and impact on the total project design. Evaluation criteria include; system weight, structural depth, impact on the lateral system, serviceability requirements, and cost.

Four structural floor framing systems were evaluated as they applied to a typical 20'x40' bay in this technical report. The existing steel composite beam floor was included in the evaluation. The three alternative floor systems that were investigated for this report are as follows:

i. Reinforced Concrete Joists With Girders
ii. Reinforced Concrete Joists With Girders, Alternative Layout
iii. Two Way Post Tensioned Flat Plate, Double Bay Layout

Upon completing the evaluation of each system and comparing results, it was determined whether further investigation would be continued. The existing steel composite beam floor system remained as one of the floor systems that will be given continued consideration in the building. The reinforced concrete floor system with joists spanning in the short direction was also determined to be a viable option for floor framing in the building. The alternative reinforced concrete joist layout was not as effective for carrying the required loads when directly compared to the opposite layout and was consequently removed from further consideration. The two way flat plate post tensioned floor framing was also determined to be an unlikely floor system for the super structure of the building. However, the combination of post tensioned girders with the reinforced concrete floor framing is likely a more efficient framing system than the alternative designs presented in this report. Therefore, the PT system is noted as a alternative option that will be further investigated as a floor framing system.

## Introduction

The Simmons College School of Management is a newly completed five story educational facility to be located on the Simmons College campus in Boston, Massachusetts. The $63 million building which was completed in December of 2008 was designed by Cannon Design.

As part of the project a five level below grade parking structure was provided to replace the parking lot that previously occupied the site. This relocation of parking allowed for the creation of a new green space quad to serve the school.

When the building was completed it achieved the LEED Gold rating by the USGBC. The project received 40 LEED points which included recognition for significant reductions in water and energy usage.

The project includes design considerations for a future building expansion to be topped out at nine stories. This design parameter was considered from the beginning of the design process including the original geotechnical evaluation of the site.

## Alternative Floor System Investigation

This report evaluates alternative structural floor layouts for the superstructure of the Simmons College School of Management. Four structural floor systems are evaluated in a simulation of preliminary design for the potential of each system in the building. The existing composite steel beam floor system was evaluated as one of the four structural floor layouts. Three additional systems were then investigated as alternatives approaches to the building design.

i. Reinforced Concrete Joists with Girders
ii. Reinforced Concrete Joists with Girders, Alternative Layout
iii. Two Way Post Tensioned Concrete Flat Plate, Double Bay Layout

The floor plan of the above grade structure of the building has a varying structural plan from floor to floor and does not contain many areas with typical floor bay layouts. It was determined that the two adjacent 20’x40’ bays seen in Figures 1 and Figure 2 would be investigated for this report. The framing in this area is one of the larger spans of floor framing that would need to be addressed with the current column layout. Therefore would be most appropriate to evaluate each system in this critical area.

Figure 1 Bay Location

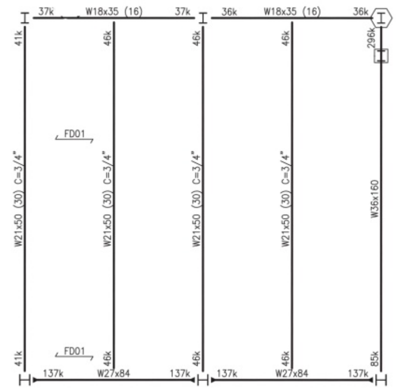



Figure 2 Typical Floor Bay

# Existing Structural Systems

## Foundations

The below grade parking structure was constructed by the top down method with the installation of a slurry wall and load bearing elements (LBE) prior to excavation. Slurry wall panels have varying widths ranging from 10'-0" to 25'-0" with the typical panel width being 24'-0". Penetration of the 10'-0" centerbite into marine sands on site ranges from 1'-0" to 43'-0" depending on the bearing capacity demands of the wall section. See Figure 5 for typical slurry wall panel elevation.

Load bearing elements are constructed with W14 columns from the garage embedded in concrete shafts. Depths of the concrete shafts are divided into four categories summarized in figure 3. W14 column embedment into the concrete shafts ranges from 16' to 27'. Typical shear studs are 4" long ¾" diameter and arranged in patterns of eight, ten, or 12 studs per foot seen in Figure 4. See Figure 6 for typical LBE configuration below the slab on grade.

| LBE INSTALLATION CRITERIA CATEGORIES | |
|---|---|
| CATEGORY 1 | MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT |
| CATEGORY 2 | MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT OR MINIMUM EMBEDMENT OF TWO (2) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF TEN (10) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSITS |
| CATEGORY 3 | MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSIT |
| CATEGORY 4 | MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF BEDROCK DEPOSIT |

Figure 3 Typical LBE Configuration

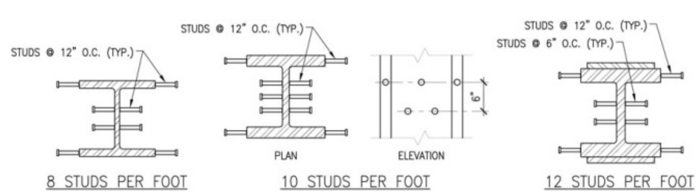



Figure 4 Typical LBE Configuration

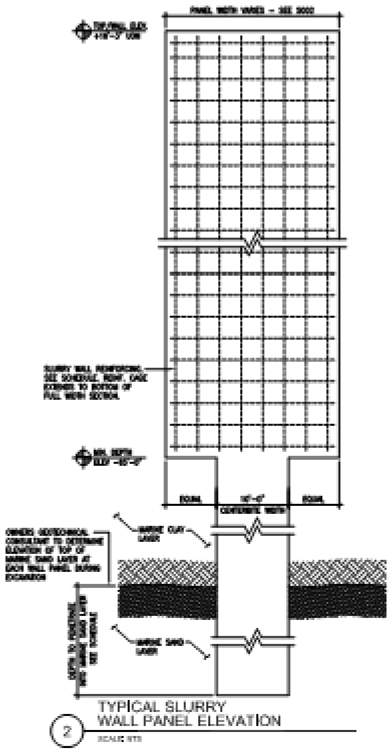



Figure 5 Slurry Wall Foundation Detail

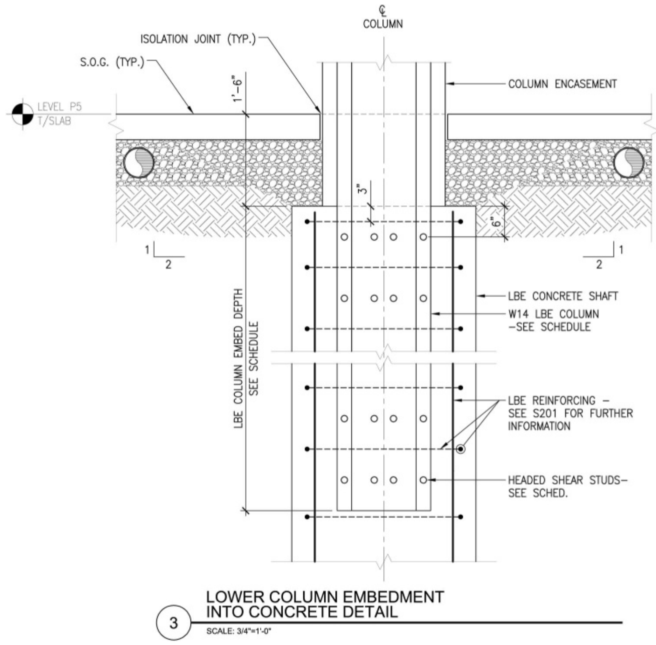



Figure 6 Load Bearing Element Foundation Detail

Beneath the area of the superstructure that is not located on top of the parking garage .365" thick, 10.75" diameter concrete filled steel pipe piles are used for foundations at column locations. Arrangements of piles include three, four, five, and eleven pile configurations.

## Floor Systems

Post tensioned concrete slabs are utilized for the typical floor system in the sub grade parking garage. Slab thickness in levels P1 through P4 is 14" with 6500 psi concrete. Bay sizes in the parking garage range from 36'x32' to 42'x49'.

Banded reinforcement spans in the north south direction of the parking garage plan with the typical bottom drape in each tendon meeting the minimum concrete cover at 1.75 inches. The typical force after all losses in these tendons is 1600 kips. Distributed reinforcement is placed in the east west direction at a maximum of 48 inches on center. At the column connections various patterns of stud rail arrangements and additional mild reinforcement are provided. For the lower four parking levels steel columns are encased in concrete to form a round 2'-8" diameter round column.

At the plaza and first floor level the structural floor system changes from post tensioned concrete to steel beams with composite floor slabs. In the main quad area typical bay sizes remain the same. Typical horizontal framing in this area ranges from W24x76 beams with 52 shear studs to W36x135 beams with 80 shear studs. Three inch deck with 9" of 3000psi concrete is typical for all horizontal surfaces at the main quad space. Plate girders are used to transfer load from superstructure columns above this level to the columns extending through the parking garage. All plate girders are 48 inches deep with weights from 330 to 849 lb/ft.

The use of steel beams with composite action is continued in the floor framing of the building above grade. See the framing the third floor in Figure 7 for a typical plan and framing layout.

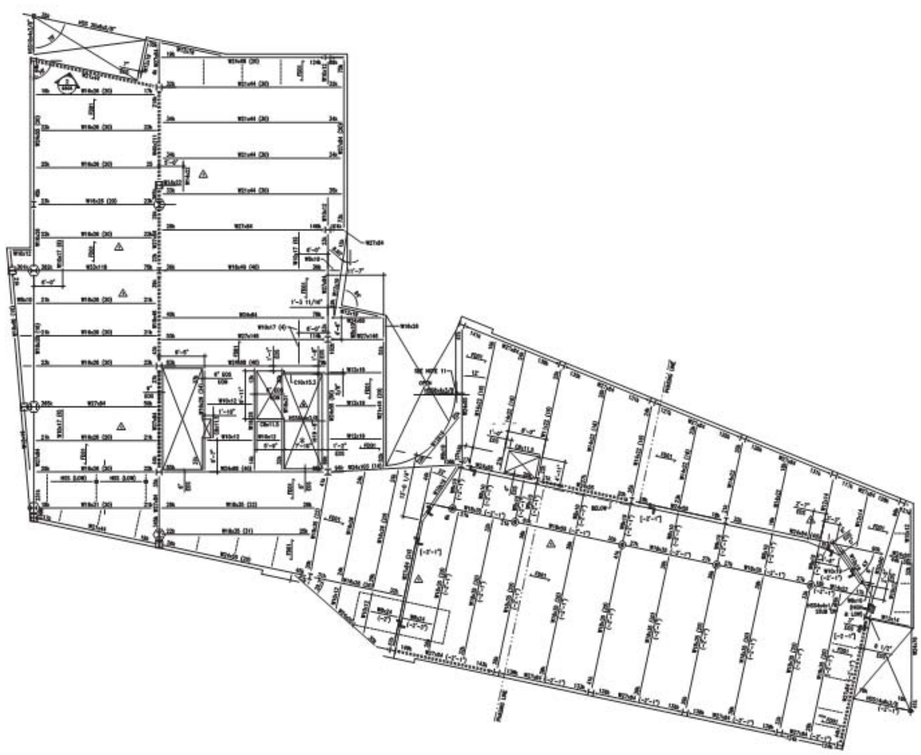


**Figure 7 Second Floor Framing Layout**

## Columns

Typical column sections for the superstructure of the Simmons College School of Management are wide flange sections with some usage of hollow structural steel (HSS) sections. Wide flange sections are all W14s with weights varying from 43 to 109 lb/ft. The most commonly used wide flange column is a W14X90. HSS sections are either HSS6x6 or HSS8x8. In addition to carrying gravity loads the majority of the columns participate in the lateral force resisting systems as part of either the moment frames or braced frames.

Once the building column loads have been transferred by the plate girders W14 column sections continue to carry the load through the parking garage. Weights vary from 159 to 398 lbs/ft. In two different locations W14x398 with side plates or W14x500 columns are used. Here all columns below the first parking garage level are encased in concrete to form a 2'-8" diameter round column.

## Lateral Systems

Two structural systems are used in the Simmons College School of Management to resist lateral forces applied to the building. In the north south direction of the building steel braced frames carry lateral loads. The lateral force resisting system in the east west direction is a combination of steel braced frames and steel moment frames. Locations of steel braced frames can be seen in Figure 8 and steel moment frames are noted in Figure 9. The number of steel braced frames used is reduced in the upper floors of the building. In some cases areas moment frames are used where braced frames are present on lower floors.

At all levels the concrete floor deck forms a ridged diaphragm which transfers lateral load to either the braced or moment frames. The amount of force that each lateral load resisting element receives is dependent on that element's relative stiffness in the system.

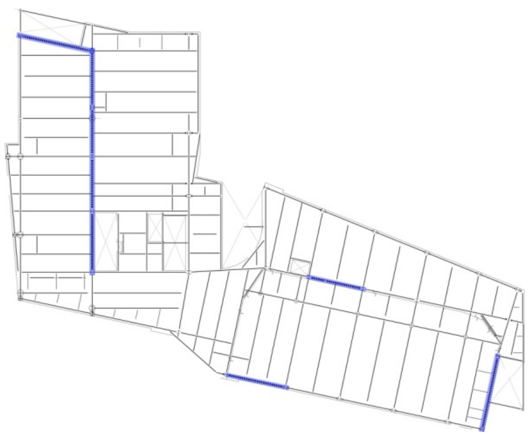


**Figure 8 Braced Frame Locations**

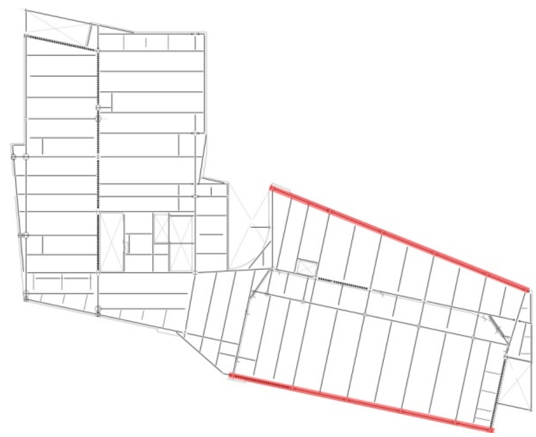


**Figure 9 Moment Frame Locations**

In the parking garage levels of the building, soil pressures generate lateral forces that need to be counteracted. Here the post tensioned floors provide the lateral bracing for the slurry walls. To ensure lateral stability during construction the parking garage was constructed in a top down method. Slurry walls and load bearing element columns were installed first with excavation and installation of the area beginning with the top slab.

## Supplementary Structural Systems

Two supplementary structural systems are used in the building in addition to the main load carrying elements. At the roof a braced frame screen is used to hide the penthouse and mechanical equipment. HSS sections are used for vertical and horizontal members while angles form the diagonal bracing.

In the parking garage reinforced concrete members are used to form the ramp access to all parking levels. Edge beams span the length of the length of the ramp with a 12 inch slab bridging the 21'-2" for the driving surface. Girders are 2'-7" deep and span below the slab at columns locations.

# Code Requirements

## Design Codes

| | |
|---|---|
| Building Code, Design Loads: | Massachusetts State Building Code CMR 780 6th Addition |
| Reinforced Concrete: | American Concrete Institute (ACI) 318 |
| Structural Steel: | American Institute of Steel Construction (AISC) |

## Substitute Codes for Thesis

| | |
|---|---|
| Building Code: | International Building Code (IBC) 2006 |
| Building Loads: | American Society of Civil Engineers (ASCE) 7-05 |
| Structural Steel: | American Institute of Steel Construction (AISC) 13th Edition 2005 |
| Reinforced Concrete: | American Concrete Institute (ACI) 318-08 |

# Materials

Concrete

| | |
|---|---|
| Footings | 3,000 psi |
| Foundation Walls | 4,000 psi |
| Grade Beams, Pile Caps | 4,000 psi |
| Concrete in pipe piles | 4,500 psi |
| Slab on Grade | 3,500 psi |
| Slab on metal deck | 3,000 psi (Normal and Light Weight) |
| All other concrete | 4,000 psi |
| Columns at P/T slab | 4,000 psi |
| Post Tensioned Concrete | 6,500 psi |
| Slurry Walls | 4,000 psi |

Reinforcing

| | |
|---|---|
| Mild Reinforcing Bars | ASTM A-615, Grade 60 |
| Welded Bars | ASTM A-706, Grade 60 |
| Welded Wire Fabric | ASTM A-185 |
| Steel Fibers | ASTM A-820 Type 1 |

Masonry

| | |
|---|---|
| Hollow Concrete Masonry Units | ASTM C90 Grade N, Type 1<br>F'm = 1900psi |
| Grout | ASTM C476<br>3,000 psi min. |
| Mortar | Type S - ASTM C270 |

Structural Steel

| | |
|---|---|
| Wide Flange Shapes, WT's | ASTM A-992 |
| Channels & Angles | ASTM A-36 |
| Pipe | ASTM A-53 Grade B |
| Pipe Piles | ASTM A252 Grade 3 |
| Tubular Shapes (Rect.) | ASTM A-500 Grade B |

| | |
|---|---|
| Base Plaets | ASTM A572 Grade B |
| All Other Steel Members | ASTM A-36 (Unless Otherwise noted) |
| High Strength bolts | ASTM A-325, or A-490 |
| Nuts and washers | Min. ¾" Diameter |
| Anchor rods | ASTM F1554 |
| Welding Electrode | E70XX |
| Metal Deck Welding Electrode | E60XX min. |
| Metal Deck | ASTM A653<br>Fy=33,000psi |

# Building Loads

## Dead Loads (all values in psf)

| | |
|---|---|
| FD01 | 43.2 |
| FD02 | 42.7 |
| FD03 | 69.0 |
| FD04 | 96.8 |
| PT floor slab | 175 |
| Structural Steel | Per AISC Manual |
| Green Roof | 100 |
| Superimposed Dead loads: | |
| MEP | 10 |
| Partitions | 20 |
| Finishes/Misc. | 5 |
| Curtain Wall | 10 |

## Live Loads (all values in psf)

| Space: | Design Value | ASCE 7-05 |
|---|---|---|
| Parking Floors | 50 | 40 |
| Plaza | 100<br>300 Construction | 100 |
| Exit Corridors | 100 | 100 |
| Stairs | 100 | 100 |
| Lobbies | 100 | 100 |
| Typical Floor | 50 | 50 (office load) |
| Corridors above 1$^{st}$ Floor | 80 | 80 |
| Roof Garden | 100 | 100 |
| Flat Roof | - | 20 |
| Mechanical Areas | 150 | |

# Alternative Floor Systems

## Existing Steel Composite Beams

The current floor system in the Simmons College School of Management uses steel beams with composite action. This framing system allows the steel framing to engage the floor plate and take advantage of the compressive strength of the concrete. Both girders and beams act as composite members. In these bays the floor is composed of 5 ½" light weight concrete on 2" metal deck. Shear studs provide the mechanical connection between the steel and concrete with the number used varying based on strength requirements. See Figures 10, 11, and 12 for typical layout and details of the steel composite beams.

One of the distinct advantages of this system is the ability to have varied floor framing and column layout. The geometry and architecture of the building necessitates changes in the column grid as well as the layout of floor framing members. The interface between the above grade steel building and the concrete parking deck below grade is handled with built up plate girders.

The steel framing system has several disadvantages when compared to the other systems evaluated in this report. In comparison this floor system has the largest total depth of structural members. Members also have to account for mechanical duct penetrations in several areas. Mechanical duct penetrations are necessary to maintain a consistent total ceiling to floor depth.

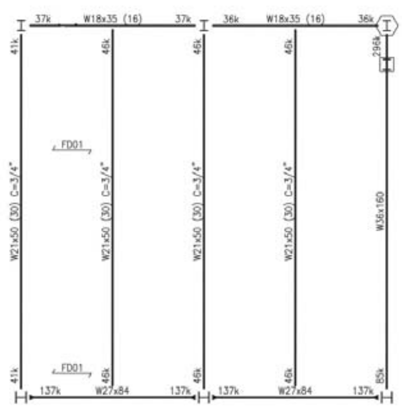



**Figure 10 Typical Steel Composite Beam Floor Bay**

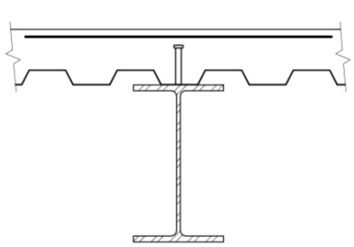


**Figure 11 Composite Action Girder Detail**

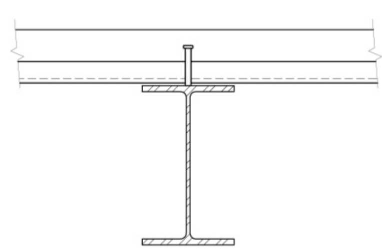


**Figure 12 Composite Action Beam Detail**

## Reinforced Concrete Joists with Girders

The first system that was investigated for the possible implementation into the building was a reinforced concrete floor system with repetitive joists spanning the short direction framing to girders which span the long direction. This system was chosen as one of the more efficient ways of constructing a concrete floor with a bay aspect ratio equal to two. Joists in this system span the 20' direction and were placed at a consistent interval with a 5' center to center spacing. Girders span perpendicular to the joists between the supporting columns. It is most economical in this system to match the bottom elevations of the joists and the girder to make the forming process as efficient as possible. Girder sizes are the most cost effective when the width is greater than or equal to the width of the supporting column. As a result of these constraints the system usually results in the design of wide shallow girders with the joist members sized to match the total height.

To perform the preliminary design, and excel spread sheet was developed utilizing the quadratic equation to determine required amounts of steel reinforcement. An iterative approach was then taken to determine desirable width and height dimensions for each member. Columns were first designed with preliminary loads and assumed moment affects which yielded an initial 20x20 size. Using this dimension the girders were designed to match the width of the column. With the system depth now determined, the joist members were sized to fit the system accordingly. Joists were estimated to be 6 inches wide with a total height of 17 inches. Girders were preliminarily sized at 20 inches wide and 17" from the top of slab to the bottom of the member. The layout and summary of members can be seen in Figures 13, 14, and 15 below. If necessary this system could be refined to have a lower total structural depth. To achieve this it would require the girders to become considerable wider to carry the building loads. The slab was conservatively determined to be 5" thick to meet the necessary strength and fire protection criteria. The girders of this system take advantage of t-beam action.

The one way joist framing system has several advantages when used as the structural floor system in a building. Comparatively this is estimated to have the lowest cost per square foot for any of the systems investigated in this report. The overall structural depth of the floor system is reduced from approximately 26" to 17." It is not likely that the reduction in structural depth would result in a reduced building height. Mechanical ducts that would have run through penetrations in the deeper beams and girders of the original floor framing would have to run beneath the joists and girders in this system.

Several disadvantages would accompany a switch to this system. Column sizes would be increased from the typical wide flange shapes to likely square columns that would occupy more space. The column size was estimated for just the loads that it would have to carry for the third floor and above. It is likely that sizes would increase for the lower floors. The lateral system would also have to change along with this floor system. In areas where braced frames were present a switch to shear walls would not affect the architecture of the building significantly. However, where moment frames are used to resist lateral forces, there would have to be additional study to the architectural effects of filling that space with a shear wall. Foundations for the building would need to account for a moderate increase in gravity load due to the increased material weight for the system.

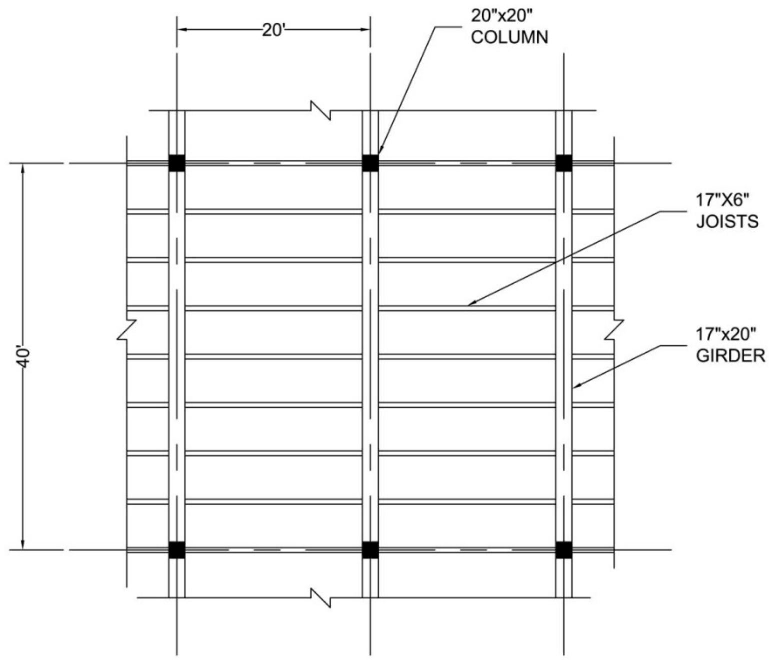



**Figure 13 Reinforced Concrete Joists with Girders**

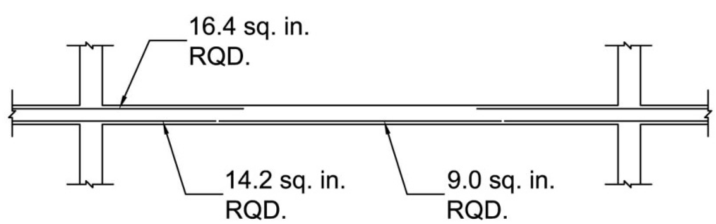



**Figure 14 Girder Required Steel Reinforcement, L = 40'**

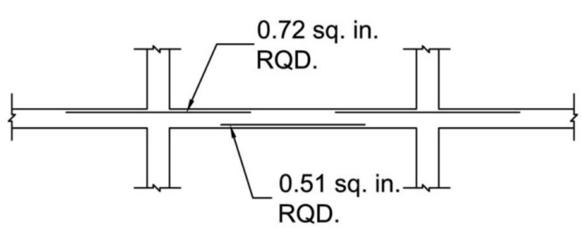



**Figure 15 Joist Required Steel Reinforcement, L = 20'**

## Reinforced Concrete Joists with Girders, Alternate Layout

The second alternative floor framing system investigated in this report uses a similar design methodology to the previously presented floor system. It was designed as a comparative study for the effectiveness of floor framing methods with repetitive concrete joists framing into girders. Due to the heavy reinforcement requirement for the girders in the previous layout, an alternate layout of a reinforced concrete system was investigated. This system uses joists spanning in the long direction of the 20x40 bay framing into girders which span in the shorter, 20' direction. To keep a close comparison of the systems 5' joist spacing was selected for this system to match that of the previous design.

In this design study it was discovered that the joists rather than the girders controlled the depth of the system. The moments induced by the longer span would have to be handled by either double reinforcement or increased depth. Eight inch wide joists with a total depth of 20 inches were required for the repetitive framing members. The girders in this system easily carried the loads from the joists with the column width and the joist depth constraining this member's dimensions. The resulting size was a 20 inch width and a 20 inch total depth of the girder. Figures 16, 17, and 18 below show the layout of members in this framing scheme.

It is most effective to directly compare this floor framing layout with the previously presented system. The layout with joists in the long direction yielded a deeper and heavier system than the previous design. Foundations and the lateral system would then have to resist increase gravity and seismic loads as a result of this alternative layout. Therefore, it was determined that this system was not as effective as the reinforced concrete floor with joists spanning the short direction.

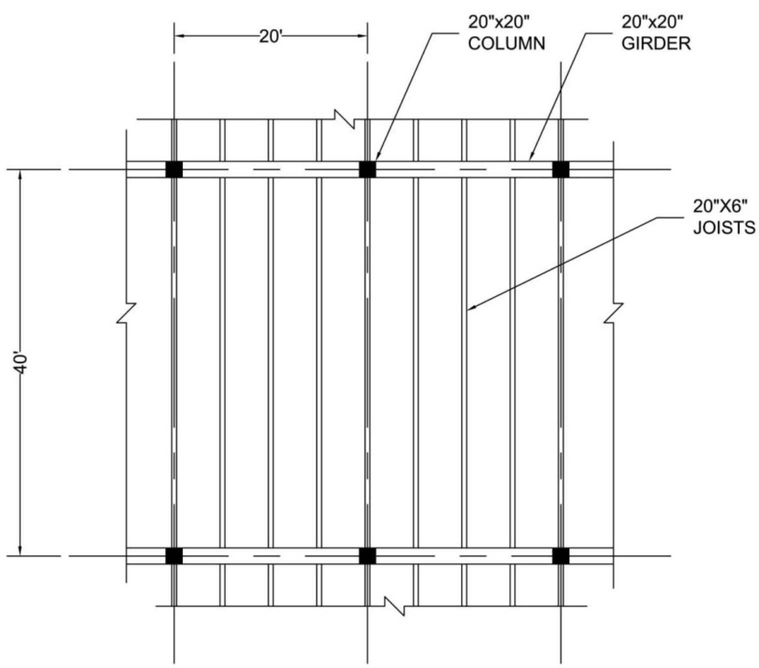



**Figure 16 Reinforced Concrete Joists with Girders, Alternative Layout**

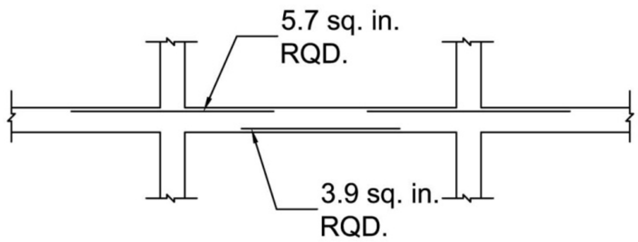



**Figure 17 Girder Required Steel Reinforcement, L = 20'**

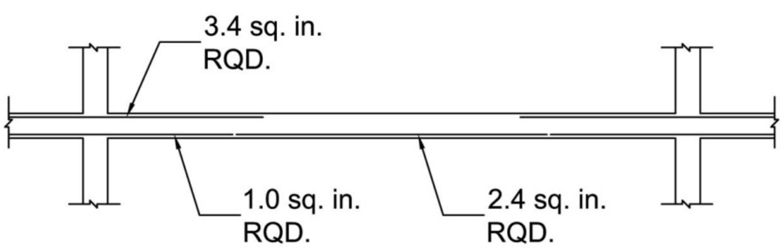



**Figure 18 Joist Required Steel Reinforcement, L = 40'**

## Two Way Post Tensioned Concrete Flat Plate

This system was investigated as a comparative example from the floor framing in the sub grade garage of the Simmons College School of Management. As discussed previously in the existing floor summary, bay sizes in the parking garage range from 36'x32' to 42'x49'. For initial design purposes, this system was applied to a combination of multiple existing bays. The previous three designs address a 20'x40' typical bay layout. The post tensioned two way flat plate system is more suited for a typical bay with approximately square dimensions. Two adjacent typical bays were combined to form a 40'x40' typical bay that would be spanned by this system. Multiple columns would be removed from the building as a result of this new layout.

At all parking garage levels a 14" thick concrete slab is used to span the typical bays. This preliminary size is taken to be the upper limit for what would be needed in the above grade structure. Sub grade floors not only resist gravity loads, but the lateral soil pressures exerted on the slurry walls as well. Moment capacity would then be altered as a result of the combined loading at the garage floor levels. It is likely then that the required depth of the floor would be less than the 14" depth assumed for this system. A typical layout is shown in Figures 19, 20, and 21.

A post tensioned floor system offers a variety of advantages to the building. This is the shallowest of the floor systems at the assumed 14" depth and it is likely that after full design is performed it would become thinner. Bay sizes are also increased, removing columns and allowing more freedom for varying layouts in each of the floor plans. The columns in the building are affected in a variety of ways as a result of this alternative framing option. By doubling the typical bay size, columns would now support twice as much area on each floor which would likely require an increased cross sectional area to carry the loads. This system is also the heaviest of the floor systems investigated in this report, weighing approximately 175 psf. This increase in building weight would significantly increase the seismic load that the lateral system would have to resist. Foundations would also have to account for the significant increase in gravity load. Additionally, problems are encountered with this system and the plan geometry of the Simmons College School of Management. It is typical to use a post tensioned floor system in a building that has rectangular bays and a rectangular plan. There would need to be additional study for an entire system layout of a post tensioned floor.

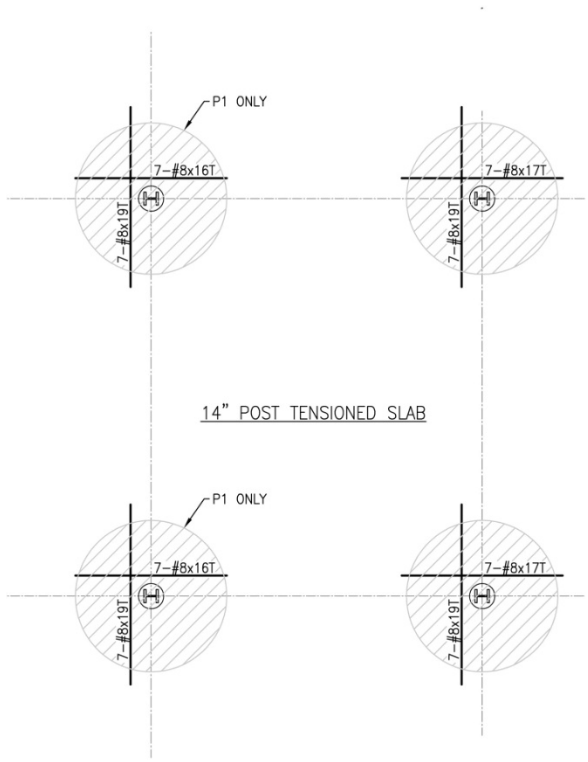



**Figure 19 Typical Post Tensioned Flat Plate 40'x40' Bay**

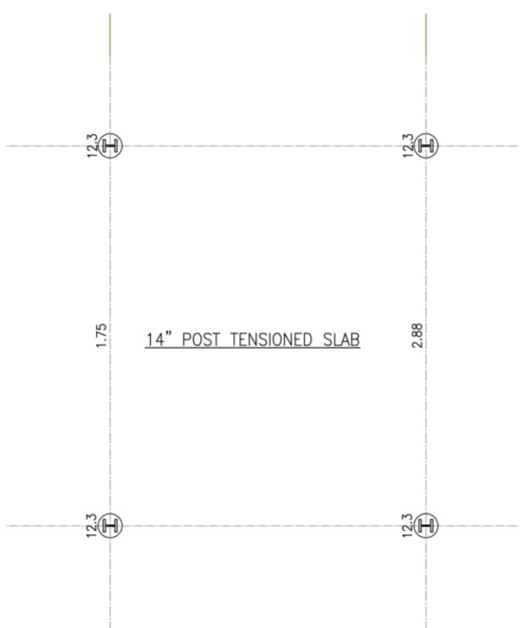



**Figure 20 Typical Banded Reinforcement**

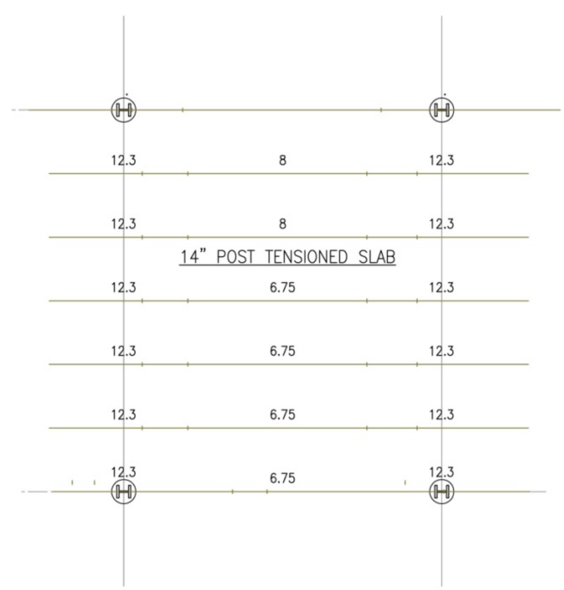



**Figure 21 Typical Distributed Reinforcement**

## Conclusion and Summary

| | Steel Composite | Reinforced Concrete 1 | Reinforced Concrete 2 | Post Tensioned |
|---|---|---|---|---|
| Weight (psf) | 85 | 93 | 97 | 175 |
| Depth (in) | 26 | 17 | 20 | </=14 |
| Column Size | W14x90 | 20x20 | 20x20 | >20x20 |
| Column Grid | - | Unchanged | Unchanged | Significant Changes |
| Lateral Forces | - | Moderate Increase | Moderate Increase | Significant Increase |
| Lateral System | Braced Frame | Shear Walls | Shear Walls | Shear Walls |
| Vibration | Satisfactory | Assumed Vibration Requirements Satisfied | | |
| Deflection | Satisfactory | Satisfactory | Satisfactory | Satisfactory |
| Material $/sf | 18.40 | 8.15 | 8.15 | 11.20 |
| Installation $/sf | 6.30 | 10.00 | 10.00 | 9.80 |
| Total $/sf | 24.70 | 18.15 | 18.15 | 21.00 |
| Continued Study | Yes | Yes | No | Yes |

The above table summarizes the design and construction factors and evaluation of each alternative floor system. For each new design the columns are a rough design for what would be required under the floor that was being investigated. Cost information was obtained from RS Means Assemblies 2009. Square foot costs presented here are unadjusted and should be used as rough estimates of the actual system costs. Serviceability requirements, including vibration was assumed to be satisfactory for the existing system. Additional study would need to be conducted to assess the alternative design floor systems for vibrations. At this point vibration requirements were assumed to be satisfactory due to the lack of a significantly lighter weight system.

The summary chart allows a direct comparison of each alternative floor framing system. It is clear that the reinforced concrete floor with joists spanning in the long direction is not a viable option to continue studying. The existing steel composite beam floor framing is still considered to be an effective floor system despite the highest cost per square foot. The reinforced concrete floor with joists spanning in the short direction presents a viable framing option and with continue to be considered as an alternative system.

The direct implementation of a two way post tensioned system is likely not a realistic option for this building. However there is a likely benefit to combining post tensioned girders with reinforced concrete joists. This would likely yield the shallowest and most material efficient floor framing system that could be used in this building. Therefore, the post tensioned system is noted for continued study for implementation in the Simmons College School of management.

# Appendix A: Typical Layout

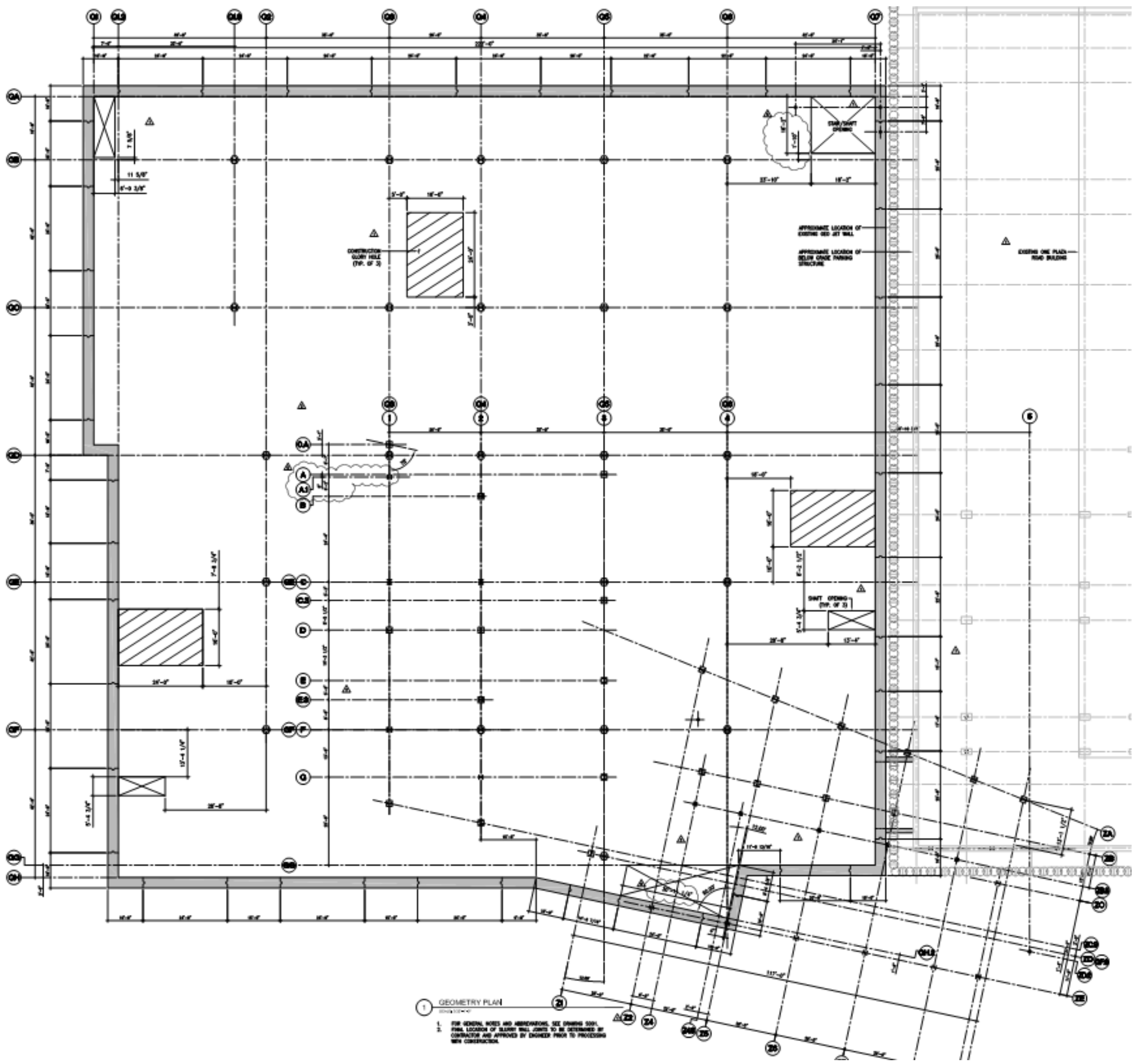



**Figure 22 Sub Grade Parking Garage Layout**

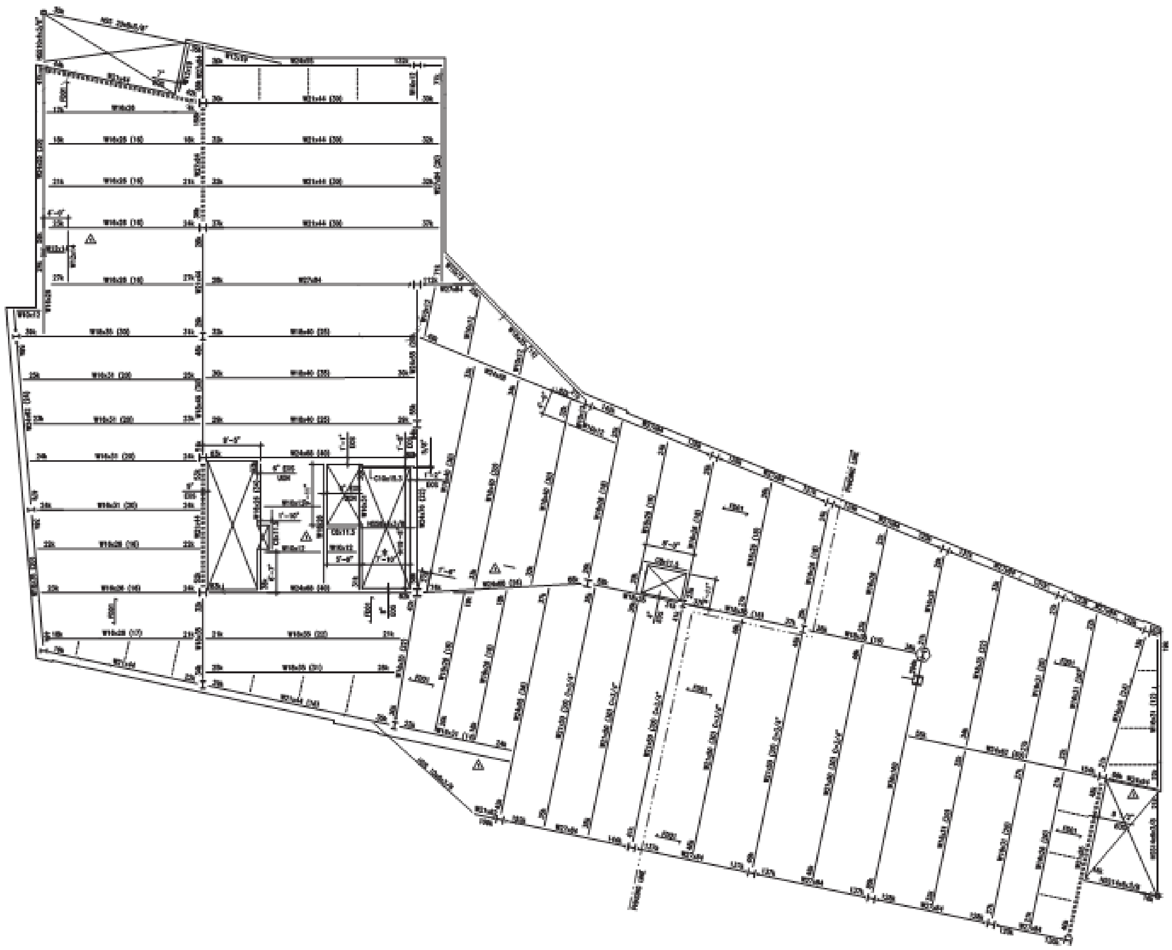
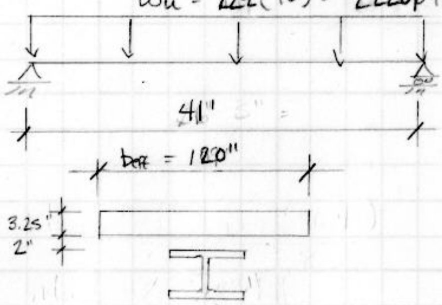


**Figure 23 Typical Above Grade Building Framing**

# Appendix B: Steel Composite Beams

SPOT CHECK: W21x50(30) C= 3/4" @ 3RD FL

$w_u = 222(10') = 2220\,plf = 2.22^{k/ft}$




Spacing = 10'
$f'_c$ = 3 Ksi
$f_y$ = 50 Ksi
$w_{LL}$ 80 psf ← use corridor LL
$w_{DL}$ 78.2 psf
2" Deck
No LL Reductions taken

$w_u = 1.2(78.2) + 1.6(80)$
$w_u = 222$ psf

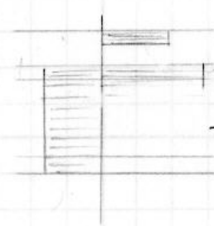
$M_u = \frac{w\ell^2}{8} = \frac{2.22(41^2)}{8}$
$M_u = 466^{1k}$

$b_{eff} \le \begin{cases} 10(12) = 120" \leftarrow \text{controls} \\ (41/4)(12) = 123" \end{cases}$

Start by assuming $a = 1"$
$\therefore Y_2 = 5.25 - \frac{1}{2} = 4.75"$

W16x40 PNA = 3 $\quad \Sigma Q_n = 378$

$a = \frac{378}{0.85(3)(120)} = 1.23$



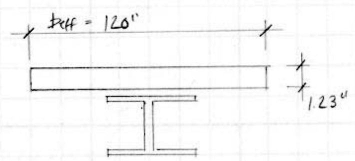
$C_c = 0.85(4)(1.23)(120) = 376^k$
$C_s = 2(50)(7)(0.306) = 214^k$
$T_s = 11.8(50) = 590^k$

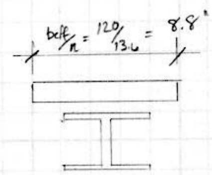
$M_n = 376(5.25 - 1.23/2) + 214(0.306/2) + 590(16/2) = 6430^{\prime k}$
$M_n = 535.8^{1k}$
$\phi M_n = 0.9(535.8) = 482^{1k} > 466^{1k} \therefore$ ok

Check Construction Loads
weight of conc. + deck : 43.2 psf
$w_{DL} = 43.2(10) + 40 = 472^{\#/ft}$
Assume LL $\quad w_u = 20\,psf(10) = 200\,plf$

$w_u = 1.2(472) + 1.6(200) = 886^{\#/ft}$
$M_u = \frac{0.886(41^2)}{8} = 186.2^{1k} < \phi M_p = 274^{1k}$

SPOT CHECK: W21 x 50 (30) C = 3/4" @ 3RD FL.




$$n = \frac{29000}{115^{1.5}\sqrt{3}} = 13.6$$

$$I_{st} = 612$$

$$I_T = \frac{8.8\,(1.23^3)}{12} = 1.36 \text{ in}^4$$

$$A_T = 8.8(1.23) = 10.82 \text{ in}^2$$

$$A_s = 11.8 \text{ in}^2$$

$$\bar{y} = \frac{11.8\,(16.0/2) + 10.82\,(16 + 5.25 - 1.23/2)}{(11.8) + 10.82} = 10.5"$$

$$I_{tr} = 612 + 11.8\,(8 - 10.5)^2 + 1.36 + 10.82\,(16 + 5.25 - 1.23/2 - 10.5)^2$$

$$I_{tr} = 1731 \text{ in}4$$

$$\Delta_{LL} = \frac{5}{384}\,\frac{w\ell^4}{EI} = \frac{5}{384}\,\frac{(0.782)(41^4)(1728)}{29000\,(1731)}$$

$$\Delta_{LL} = 0.99"$$

Limit: $\Delta_{max} = L/360 = 41(12)/360 = 1.37"$

$\Delta_{LL} < \Delta_{max}$ $\therefore$ OK

SPOT CHECK | W21X50 (30) C = 3/4" @ 3RD FL | 2

Check Construction Deflection

$$\Delta = \frac{5}{384} \frac{(0.412)(41)^4(1728)}{29000\ (612)} = 1.69$$

Construction Deflection Criteria

$$\min \begin{cases} L/360 = \frac{41(12)}{360} = 1.37'' \\ 1'' \leftarrow \text{controls} \end{cases}$$

$\therefore$ Camber 3/4"

$$\Delta = 1.69'' - 0.75 = 0.94'' < 1'' \quad \therefore \underline{OK}$$

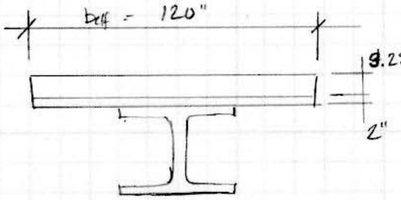
Number of Shear Studs

$$\Sigma Q_n = 378^k$$

$$\#\ \text{Studs} = 2\left(378/17.2\right) = 44$$

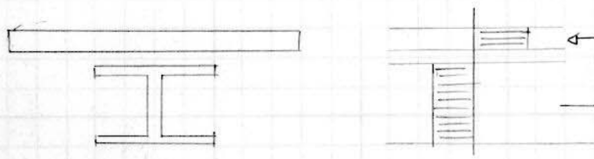
W16x40 [44] C = 3/4"

Member Capacity Check | W21x50 [30] c = 3/4" | @ 3rd Fl.




$\ell = 41'$
$\Sigma Q_n = 258^k$
$f'_c = 3$ ksi
$f_y = 50$ ksi

$$b_{eff} \le \begin{cases} S = 10(12) = 120'' \leftarrow \text{controls} \\ \ell/4 = 41(12)/4 = 123'' \end{cases}$$

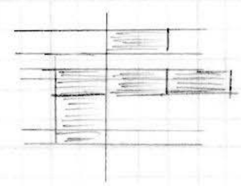


$\leftarrow C_c = 0.85 f'_c A_c = 994.5$

$\rightarrow T_s = A_s f_y = 735^k$

$C_c > \Sigma Q_n \quad \therefore \quad C_c = \Sigma Q_n = 258^k$

$$a = \frac{258}{0.85(3)(120)} = 0.843''$$



$$A_{sc} = \frac{735 - 235}{2(50)} = 5 \text{ in}^2$$

$$A_f = 6.53(0.535) = 3.50 \text{ in}^2$$

$$A_{wc} = \frac{5 - 3.5}{0.38} = 3.95''$$

$$M_n = 238\left(5.25 - 0.843/2\right) + 735\left(20.8/2\right) - 2(50)(3.5)\left(0.535/2\right) - 2(50)(1.5)\left(0.535 + (3.95/2)\right)$$

$M_n = 8323^{''-k} \Longrightarrow 693^{'k}$
$\phi M_n = 0.9(693) = 624^{'k}$

Allowable Load

$$M = \frac{w\ell^2}{8}$$

$w = 8M_u/\ell^2 = 8(624)/(41^2) = 2.97$ K/ft
$w_u = (2.97)(1000)/(10') = 297$ psf

Member Capacity Check | W21 x 50 [30] c = 3/4" @ 3RD FL

Construction Loads

$$\Delta_{max} = \begin{cases} l/360 = 41(12)/360 = 1.37" \\ 1" \leftarrow \text{Controls} \end{cases}$$
min

Camber = 3/4"

$\therefore \Delta_{max_{memb.}} = 1" + 3/4" = 1\,3/4"$

$$\Delta = \frac{5}{384}\frac{w\,l^4}{E\,I}$$

$$w_{CL} = \frac{384\Delta EI}{5\,l^4} = \frac{384(1.75)(29000)(984)}{5(41^4)(1728)}$$

$$w_{CL} = \underline{0.741}\ K/ft$$

$$w_{CL} = 74.1\ psf$$

Live Load Deflection

$n = \frac{29{,}000}{115^{1.5}\sqrt{3}} = 13.6$

$I_{st} = 984\ in^4$

$I_T = \frac{(120/13.6)(0.843^3)}{12} = 0.44$

$A_s = 14.7$

$A_T = 8.8(0.843) = 7.42\ in^2$

$\bar{y} = \frac{14.7(20.8/2) + 7.42(20.8 + 5.25 - 0.843/2)}{(14.7 + 7.42)} = 15.5$

$I_{tr} = 984 + 14.7(15.5 - 10.4)^2 + 0.44 + 7.42(20.8 + 5.25 - 0.843/2 - 15.5)^2$

$I_{tr} = 2128\ in^4$

$\Delta_{max} = L/360 = \frac{41(12)}{360} = 1.37"$

$1.37" = \frac{5}{384}\frac{w\,L^4}{EI}$

$w_{max_{LL}} = \frac{1.37(384)EI}{5\,L^4} = \frac{1.37(384)(29{,}000)(2128)}{5(41^4)(1728)}$

$w_{LL} = 1.33\ K/ft \Rightarrow 133\ psf.$

SPOT CHECK. | Column ZB-Z6 | W14x[illegible]

Loads (No Live Load Reduction)

Tributary Area

$A_T = (41/2)\,20 + (\tfrac{1}{2})(\tfrac{22}{2} + \tfrac{19}{2})(20) = 615 \text{ sf}$

Dead Loads

$P = (78.2)(615) = 48{,}093^{lbs} \Rightarrow 48.1^k$

Live Loads

$L_r = 20\,(615) = 12{,}300^{lbs} \Rightarrow 12.3^k$

$L = 80\,(615) = 49{,}200^{lbs} \Rightarrow 49.2^k$

Snow

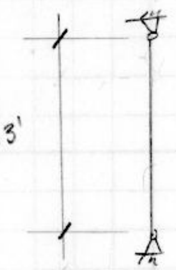
$S = 27.7\,(615) = 17{,}035 \Rightarrow 17^k$

Column Below Roof

$P_u = 1.2D + 1.6S$

$= 1.2(48.1) + 1.6(17) = 85^k$

13.3'



$KL_x = KL_y = 13.3'$

Check Capacity of W14x61

Weak Axis will Control

$\left(\frac{KL}{r}\right)_y = \frac{(13.3)(12)}{2.45} = 65.1$

A992 Steel

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113$

$KL/r = 65.1 < 113$

$F_{cr} = \left(0.658^{(F_y/F_e)}\right)F_y$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29000)}{(65.1)^2} = 67.5 \text{ ksi}$

$F_{cr} = \left(0.658^{(50/67.5)}\right)50 = 36.7$

$P_n = 36.7\,(17.9) = 657^k$

$\phi P_n = 0.9\,(657) = 591^k > 85^k \therefore \underline{ok}$

Find the least weight W14 to carry the load.

W14x43 still higher capacity than Demand

$\phi P_n = -\left(\frac{13.3 - 13}{14 - 13}\right)(345 - 319) + 345 = 337^k > 85^k \therefore \underline{ok}$

SPOT CHECK | Column ZB-Z6 | 2

Column Below 5th Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 1.2(48.1)(2) + 1.6(49.2) + 0.5(17)$

$P_u = 203^k$

$KL_y = KL_x = 13'$

Capacity W14x61

$\phi P_n = 599^k > 203^k \therefore$ ok

Least weight W14 → W14x43

$\phi P_n = 345^k > 203^k \therefore$ ok

Column Below 4th Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 1.2(48.1 \times 3) + 1.6(49.2 \times 2) + 0.5(17)$

$P_u = 339.1^k$

$KL_x = KL_y = 13'$

Capacity W14x61

$\phi P_n = 599 > 339.1^k \therefore$ ok

Least weight W14 → W14x43

$\phi P_n = 345 > 339.1^k \therefore$ ok

Column Below 3RD Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 1.2(48.1(4)) + 1.6(49.2 \times 3) + 0.5(17)$

$P_u = 476^k$

$KL_x = KL_y = 14.25'$

Capacity W14x82

$\phi P_n = 774 - \left(\frac{14.25 - 14}{15 - 14}\right)(774 - 736)$

$\phi P_n = 764.5^k > 476^k \therefore$ ok

Least weight W14 → W14x61

$\phi P_n = 572 - \left(\frac{14.25 - 14}{15 - 14}\right)(572 - 543)$

$\phi P_n = 565^k > 476^k \therefore$ ok

Column Below 2nd Floor

$P_u = 1.2D + 1.6L + 0.5S$

$P_u = 612^k$

$KL_x = KL_y = 15.6'$

Capacity W14x82

$\phi P_n = 736 - \left(\frac{0.25}{1}\right)(736 - 698)$

$\phi P_n = 726.5^k > 612^k \therefore$ ok

Least weight W14 → W14x74

$\phi P_n = 667 - \left(\frac{0.25}{1}\right)(667 - 632)$

$\phi P_n = 658 > 612 \therefore$ ok

# Appendix C: Reinforced Concrete Joists With Girders, Layout One

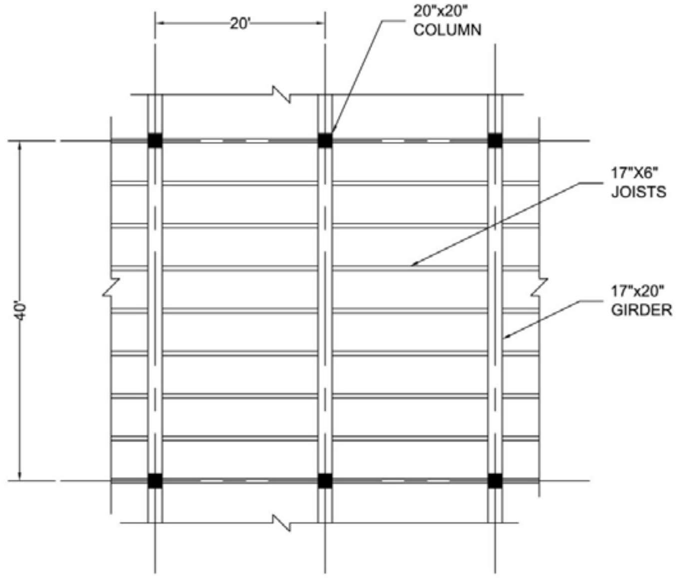



**Figure 24 Reinforced Concrete Joists with Girders**

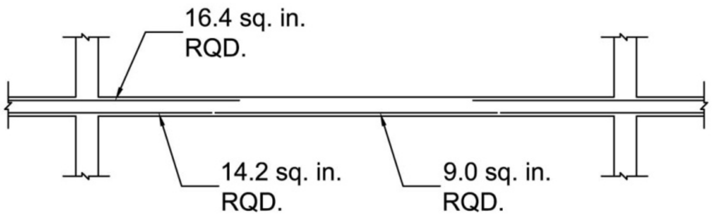



**Figure 25 Girder Required Steel Reinforcement, L = 40'**

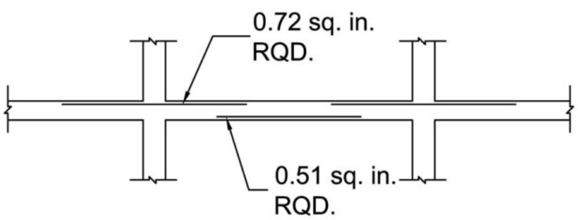



**Figure 26 Joist Required Steel Reinforcement, L = 20'**

Joist - Negative Moment Required Steel

| | | |
|---|---|---|
| wu | 1.33 | plf |
| Fy | 60000 | psi |
| F'c | 4000 | psi |
| b | 6 | in |
| d | 14.5 | in |
| Mu | 43.8 | ft-k |
| phi | 0.9 | |
| A | 79411.8 | |
| B | -783000.0 | |
| C | 525477.3 | |
| As | 9.136 | sq.in |
| | 0.724 | sq.in |
| rho | 0.0083 | in2/in2 |
| a | 2.13 | in |
| c | 2.51 | in |
| Et | 0.0144 | |
| Mu | 48.7 | ft-k |
| phiMu | 43.8 | ft-k |

Joists Positive Moment Required Steel

| | | |
|---|---|---|
| wu | 1.33 | plf |
| Fy | 60000 | psi |
| F'c | 4000 | psi |
| b | 6 | in |
| d | 14.5 | in |
| Mu | 31.3 | ft-k |
| phi | 0.9 | |
| A | 79411.8 | |
| B | -783000.0 | |
| C | 375341.0 | |
| As | 9.355 | sq.in |
| | 0.505 | sq.in |
| rho | 0.0058 | in2/in2 |
| a | 1.49 | in |
| c | 1.75 | in |
| Et | 0.0219 | |
| Mu | 34.8 | ft-k |
| phiMu | 31.3 | ft-k |

Girder Negative Moment Double Reinforced Design

| | | | |
|---|---|---|---|
| | wu | 5.62 | klf |
| | fy | 60000 | psi |
| | f'c | 4000 | psi |
| | b | 20 | in |
| | d | 14.5 | in |
| | d' | 2.5 | in |
| | Mu | 888.1 | ft-k |
| | rhomaxphi | 0.0181 | |
| | As1 | 5.24 | sq.in |
| | a | 4.622 | in |
| | c | 5.438 | in |
| | Mn1 | 319.2 | ft-k |
| | Mn2 | 667.6 | ft-k |
| | As2 | 11.13 | sq.in |
| | E's | 0.0016 | |
| | f's | 47.000 | ksi |
| Actual | F's | 47000 | psi |
| Required | A's | 14.20 | sq.in |
| Required | As | 16.36 | sq.in |

Analysis

| | | |
|---|---|---|
| C's | 667.566 | |
| c | 5.4375 | |
| a | 4.621875 | |
| E's | 0.001621 | |
| f's | 47000 | |
| Mn | 11841663 | in-lb |
| Mn | 986.8052 | ft-k |
| Et | 0.005 | |
| phi | 0.9 | |
| phiMn | 888.1247 | |

Girder Positive Moment Required Steel

| | |
|---|---|
| Fy | 60000 |
| F'c | 4000 |
| b | 112 |
| d | 14.5 |
| Mu | 7200000 |
| phi | 0.9 |
| A | 4254.201681 |
| B | -783000 |
| C | 7200000 |
| As | 174.345941 |
| | 9.707392293 |
| rho | 0.005977458 |
| a | 1.529526097 |
| c | 1.799442467 |
| Et | 0.021174154 |
| Mu | 8000000 |
| phiMu | 600 |

Deflection of Girder

| | | |
|---|---|---|
| f'c | 4000 | |
| h | 17 | in |
| hf | 5 | in |
| d | 14.5 | in |
| bw | 20 | in |
| b | 100 | in |
| spacing | 20 | ft |
| wdsup | 115 | psf |
| wd | 3.0 | klf |
| wl | 1.6 | klf |
| Positive Section | | |
| As | 9.7 | |
| Negative Section | | |
| As | 16.36 | in2 |
| A's | 14.2 | in2 |

MOMENTS

| | | | | |
|---|---|---|---|---|
| Positive | | | Negative | |
| M= | wln2/14 | | M= | wln2/10 |
| Md | 315.70071 | | Md | 441.981 |
| Ml | 167.9073 | | Ml | 235.0702 |
| Md+l | 483.60801 | | Md+l | 677.0512 |
| Msus | 366.0729 | | Msus | 512.5021 |

| Modulous of Rupture | |
|---|---|
| f'r | 474.34165 |
| Ec | 3644147.4 |
| n=Es/Ec | 8.0 |

Section Moment of Interia

Positive Moment Section

| | |
|---|---|
| yt | 11.743243 |
| Ig | 15637.883 |
| B | 1.2954666 |
| kd | 4.0219924 |
| Icr | 10643.549 |

Negative Moment Section

| | |
|---|---|
| r | 0.758901 |
| Ig | 8188.333 |
| B | 0.153619 |
| kd | 7.113011 |
| Icr | 11606.02 |

Effective Moment of Inertia

Positive Moment Section

| | |
|---|---|
| Mcr | 52.638065 |
| Mcr/Md | 0.1667341 |
| Ie d | 11476.275 |
| (Mcr/Msus)^3 | 0.002973 |
| Ie sus | 10658.397 |
| (Mcr/Md+l)^3 | 0.0012895 |
| (Ie)d+l | 10649.989 |

Negative Moment Section

| | |
|---|---|
| Mcr | 426.5012 |
| Mcr/Md | 0.964976 |
| Ie d | 8308.033 |
| (Mcr/Msus)^3 | 0.576334 |
| Ie sus | 9636.291 |
| (Mcr/Md+l)^3 | 0.249975 |
| (Ie)d+l | 10751.68 |

Average Inertia Values

| | |
|---|---|
| Ie d | 11001.039 |
| Ie sus | 10505.081 |
| (Ie)d+l | 10665.243 |

Short Term Deflections

| | | | | |
|---|---|---|---|---|
| Mo | 846.31402 | | | |
| K | 0.85 | | | |
| Deltai d | 1.6968668 | L/180 | 2.666667 | OK |
| | 1.7701701 | | | |
| Deltai sus | 2.1185992 | | | |
| | 2.149519 | | | |
| Deltai d+l | 2.801028 | | | |
| | 2.7970218 | | | |
| | | | | |
| Delta l | 1.1041612 | L/360 | 1.333333 | OK |
| | 1.0268517 | | | |

Column Sizing - Square Column, Interior

$P_u = 1.2D + 1.6L$

$P_u = [1.2(115 \times 20 \times 40) + 1.6(80 \times 20 \times 40)] \div 1000$

$P_u = 213^K \leftarrow$ Per Floor

Total Load - 3RD, 4TH, 5TH, Roof

$P_u = 213(4) = 852^K$

$M_u = 250'^K$ (ASSUMED)

$e = \frac{M_u}{P_u} = \frac{250(12)}{852} = 3.5''$

Assume $d' = 2.5''$

$\gamma = \frac{h - 2d'}{h}$

| $h$ | $\gamma$ | $e/h$ |
|---|---|---|
| 20 | 0.750 | 0.175 |
| 22 | 0.773 | 0.159 |
| 24 | 0.792 | 0.146 |

Use figure A9.b, $\gamma = 0.75$, $e/h \cong 0.16$, $\rho_g = 0.03$

$\Rightarrow \frac{\phi P_n}{bh} = 2.3$, $bh = \frac{\phi P_n}{2.3} = \frac{852}{2.3} = 370.4$

$b = h = 19.2''$ try 20" sq. Column

size assumed for preliminary design

# Appendix D: Reinforced Concrete Joists With Girders, Layout Two

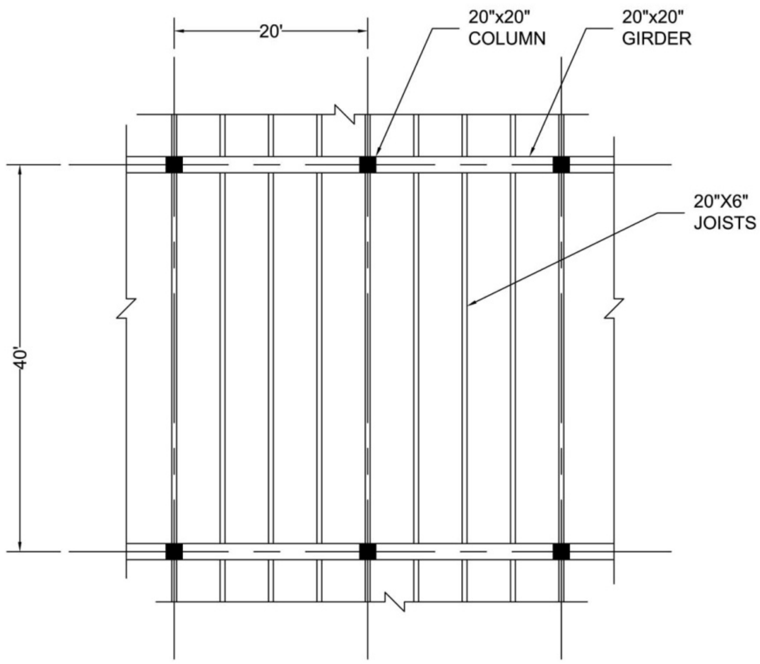



**Figure 27 Reinforced Concrete Joists with Girders, Alternative Layout**

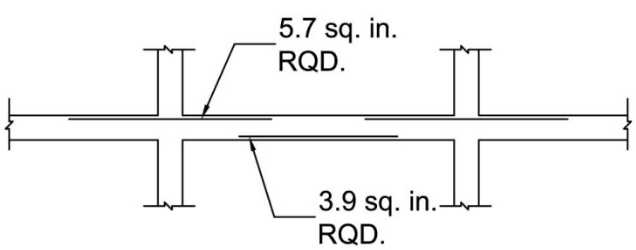



**Figure 28 Girder Required Steel Reinforcement, L = 20'**

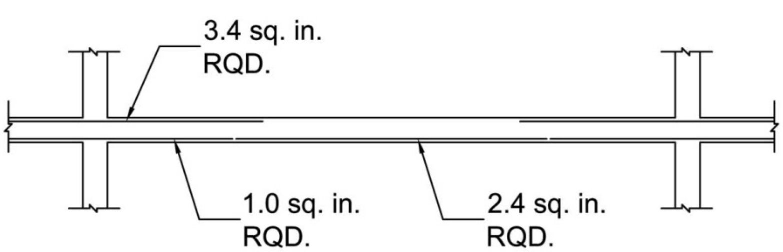



**Figure 29 Joist Required Steel Reinforcement, L = 40'**

Girder Negative Moment Required Steel

| | | |
|---|---|---|
| wu | 11 | klf |
| Fy | 60000 | psi |
| F'c | 4000 | psi |
| b | 20 | in |
| d | 17.5 | in |
| Mu | 386.5 | ft-k |
| phi | 0.9 | |
| A | 23823.5 | |
| B | -945000.0 | |
| C | 4638333.3 | |
| As | 33.928 | sq.in |
| | 5.738 | sq.in |
| rho | 0.0164 | in2/in2 |
| a | 5.06 | in |
| c | 5.96 | in |
| Et | 0.0058 | |
| Mu | 429.5 | ft-k |
| phiMu | 386.5 | ft-k |

Girder Postive Moment Required Steel

| | | |
|---|---|---|
| wu | 11 | klf |
| Fy | 60000 | psi |
| F'c | 4000 | psi |
| b | 20 | in |
| d | 17.5 | in |
| Mu | 276.1 | ft-k |
| phi | 0.9 | |
| A | 23823.5 | |
| B | -945000.0 | |
| C | 3313095.2 | |
| As | 35.780 | sq.in |
| | 3.887 | sq.in |
| rho | 0.0111 | in2/in2 |
| a | 3.43 | in |
| c | 4.03 | in |
| Et | 0.0100 | |
| Mu | 306.8 | ft-k |
| phiMu | 276.1 | ft-k |

Joist Negative Moment Requred steel - Double Reinforcement

| | | | |
|---|---|---|---|
| | wu | 1.33 | klf |
| | fy | 60000 | psi |
| | f'c | 4000 | psi |
| | b | 8 | in |
| | d | 17.5 | in |
| | d' | 2.5 | in |
| | Mu | 224.8 | ft-k |
| | rhomaxphi | 0.0181 | |
| | As1 | 2.53 | sq.in |
| | a | 5.578 | in |
| | c | 6.563 | in |
| | Mn1 | 186.0 | ft-k |
| | Mn2 | 63.8 | ft-k |
| | As2 | 0.85 | sq.in |
| | E's | 0.0019 | |
| | f's | 53.857 | ksi |
| Actual | F's | 53857.14 | psi |
| Required | A's | 0.95 | sq.in |
| Required | As | 3.38 | sq.in |

Analysis

| | | |
|---|---|---|
| C's | 51.04331 | |
| c | 6.5625 | |
| a | 5.578125 | |
| E's | 0.001857 | |
| f's | 53857.14 | |
| Mn | 2997667 | in-lb |
| Mn | 249.8056 | ft-k |
| Et | 0.005 | |
| phi | 0.9 | |
| phiMn | 224.825 | |

Joist Postive Moment Reinforcement

| | | |
|---|---|---|
| wu | 1.33 | klf |
| Fy | 60000 | psi |
| F'c | 4000 | psi |
| b | 8 | in |
| d | 17.5 | in |
| Mu | 160.6 | ft-k |
| phi | 0.9 | |
| A | 59558.8 | |
| B | -945000.0 | |
| C | 1927071.4 | |
| As | 13.463 | sq.in |
| | 2.403 | sq.in |
| rho | 0.0172 | in2/in2 |
| a | 5.30 | in |
| c | 6.24 | in |
| Et | 0.0054 | |
| Mu | 178.4 | ft-k |
| phiMu | 160.6 | ft-k |

# Appendix E: Two Way Post Tensioned Concrete Flat Plate

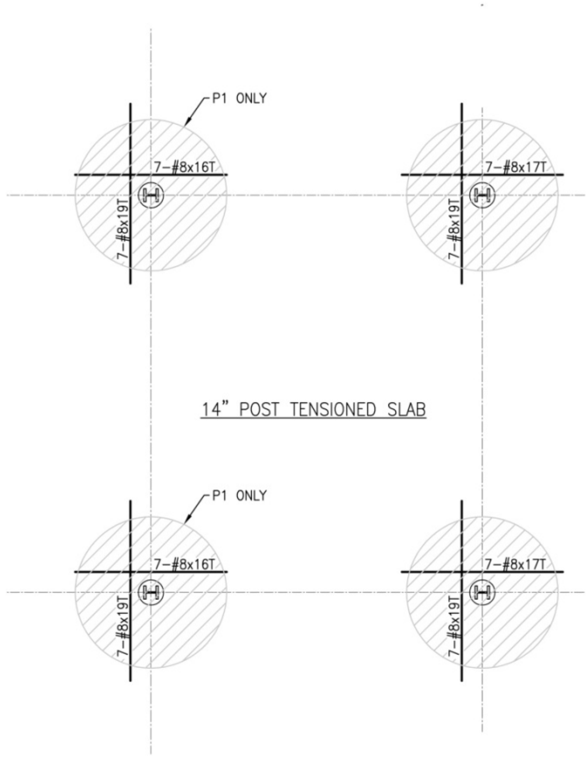



**Figure 30 Typical Post Tensioned Flat Plate 40'x40' Bay**

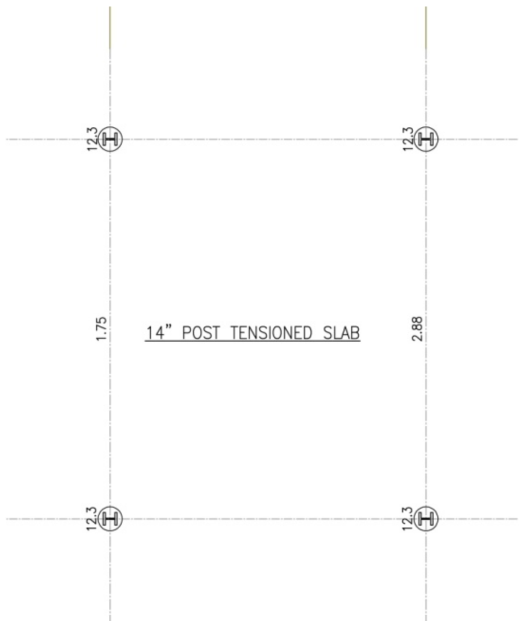



**Figure 20 Typical Banded Reinforcement**

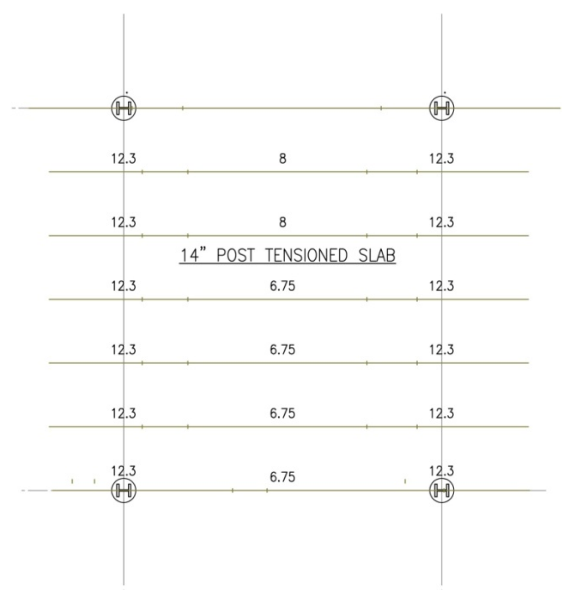



**Figure 21 Typical Distributed Reinforcement**

# Appendix F: Cost Estimates

Cost Data
RS Means 2009 - Assemblies

| System | Material | Installation | Total | Reference |
|---|---|---|---|---|
| Existing Stl. w/ 5½" Conc. on 2" Deck | 18.40 | 6.30 | 24.70 | pg. 97 B1010 256 4500 |
| Cast in Place Multi Span Joists | 8.15 | 10.00 | 18.15 | pg. 69 B1010 226 6400 |
| Cast in Place Multi Span Joist | 8.15 | 10.00 | 18.15 | pg. 69 B1010 226 6400 |
| Post Tension Fl. (Assume similar to Cast in Place 40x40) | 11.20 | 9.80 | 21.00 | pg. 67 B1010 223 |