35 WEST 21ST STREET NEW YORK, NY

Technical Report
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Structural Option
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Executive summary

The goal of this report is to study and evaluate different floor systems that could possibly yield a more efficient building design for 35 West 21st Street, and compare these systems to the existing floor. Four floor systems including the existing were analyzed for gravity loads to determine the feasibility of each one. The cost of each different floor structure was also taken into account to determine economic feasibility.

The existing structure is a two way flat plate concrete system reinforced with mild steel of yield stress 60,000 psi. The first alternative solution analyzed was a two way flat plat post tensioned concrete floor. The second alternative considered was a composite concrete slab with wide flange steel beams and girders. Lastly, a system of precast hollow core plank on steel girders was evaluated. This report contains discussions that compare each of the floor systems to the existing two way flat plate concrete floor, as well as images and calculations to verify the information argued in this report.

After a careful and detailed analysis was completed of all the floor systems, it was determined that a post tensioned flat plate two way slab is the most practicable replacement for the two way flat plate mild steel reinforced floor of the existing structure. To achieve this, however, the columns must be realigned to form a regular column grid. This will ultimately affect the plan of the building; therefore, the architecture will have to be rearranged as well. The composite steel beam and slab system failed to be reasonable since the very small W shapes were able to carry the gravity loads, there was not much difference between construction loads and serviceability loads, and the depth of the ceiling cavity increased. For the precast plank system, a deeper ceiling cavity is inevitably true due to the fact that steel beams need to transfer the plank forces into columns. A reduction of the slab thickness as well as ceiling cavity depth could change the building significantly. It was found that by introducing individual HVAC units for every apartment, the need for a ceiling cavity is eradicated because there will not be a central HVAC system. By decreasing the slab thickness and removing the ceiling cavity, there exists the potential for adding another floor, but still staying within building height limitations. This Concept will be analyzed in further detail in later reports.

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Introduction

35 West 21st Street is shaped by the surrounding buildings and its site. With adjacent 4-12 story buildings, the plan takes on a T-shape to maximize the footprint. The stem of the T-shape is an eight story residential tower facing the north, while the top of the T-shape is a fifteen story residential tower facing the south with retail space at grade. Over 162,000 sq. feet of residential and retail space are provided.

35 West 21st Street is located in the Flatiron District within the Ladies' Mile Historic District. The area is zoned as C6-4A which allows for commercial, light manufacturing, and residential construction. The predominant historical requirements of Ladies' Mile consist of street walls a minimum of 60 feet tall that are in character with the surrounding area. Therefore, the building is has a classic stone facade with infill glass windows.

The columns of superstructure are continuous from the foundation to the top of the building with no transfers throughout the building. The columns are arranged in a semi regular pattern where most bays are rectangular in plan. The arrangement of columns allows for open residential and retail floor plans while a two way flat plate concrete floor system allows for 8' high ceilings while maintaining a typical 9'-8" floor to floor height. The top residential units have large personal balconies which overlook the surrounding city and allow for a spacious outdoor room in crowded New York City.

Structural Systems Overview

Floor System

35 West 21st Street is a typical reinforced concrete residential structure. The floor system is a two way flat plate slab without drop panels or beams. Typical residential floors are 8 inches thick with typical reinforcement of #5 deformed bars at 12 inches on center bottom bars (each way) and #5 deformed bars at 12 inches on center Middle Strip top bars. Column Strip top bars vary according to span lengths which range from 13' to 18'. In areas of high shear, slab supports also have stud rails to help prevent punch through shear. Typical columns are gravity only, and run the entire height of the building without transfers. On the fifteenth floor, columns lining the exterior balconies are transferred to the 14" slab and then transferred to nearby columns that go down to the foundations. Typical columns are 16"x18" with 8-#7 longitudinal bars and #3 ties at 12 inches on center. Minimum concrete compressive strength is 5 ksi for slabs above ground, and 5.95 ksi for columns. The slab also provides a two hour fire rating.

Basement

The basement floor is a slab on grade reinforced with 6" WWF 6x6 – W2.0xW2.0. Typical slab on grade thickness is 6".

Roof system

The roof slab is 12 inches thick with typical reinforcing like that on all the residential floors. Cooling towers sit on dunnage that consists of 16"x16" concrete piers and galvanized W10x33 steel beams. The remaining mechanical equipment including elevator machines are housed in the bulkhead which consists of shear wall 16 and three transfer columns. Shear The concrete piers and columns are transferred through the 12" slab and into columns below that continue to the foundation.

Lateral System

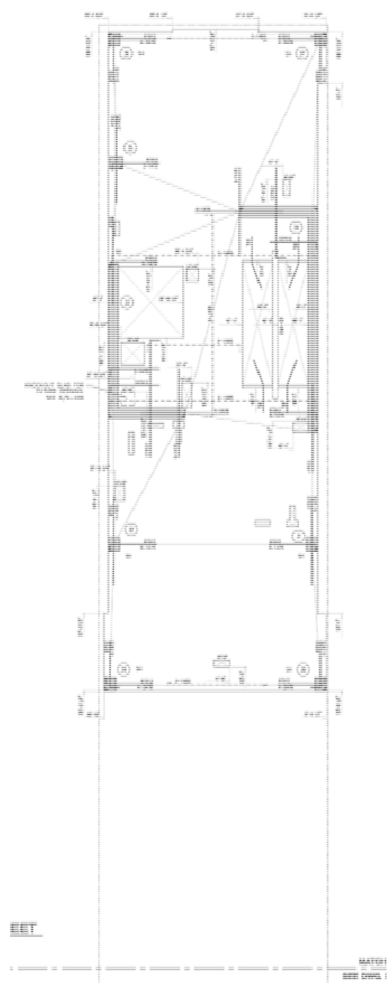
The lateral system of 35 West 21st Street is comprised of shear walls in both the North-South and East-West directions of the building. The two towers of the building are built integrally with each other through the two way slab on the basement, ground and second floor. However, at the second floor, the 15 story south tower steps back to allow for an outdoor courtyard, thus breaking the connection between the two towers. Because the connection of the two towers only exists on the first two floors, the towers' lateral systems were designed separately from each other. The assumption that the two buildings act separately and thus do not transfer any torsional moment between the two lateral systems will be investigated more closely in following technical reports. As for this technical report, it is assumed that the two buildings act separately. Typical shear walls are 1'-0" wide and longitudinal reinforcement ranges from #10 at 12" on center at the base of the shear walls to #4 at 12" on center at the top of the building. Horizontal shear reinforcement typically consists of #4 at 12" on center closed loop bars.

Foundation

The foundation system consists of spread footings for typical concrete columns and large mat foundations for shear walls. On the east side of the building, 240 ton caissons spread loads from the footings to the bedrock below. The caissons are at a minimum drilled 9'-0" into bedrock and are typically 12 inches in diameter.

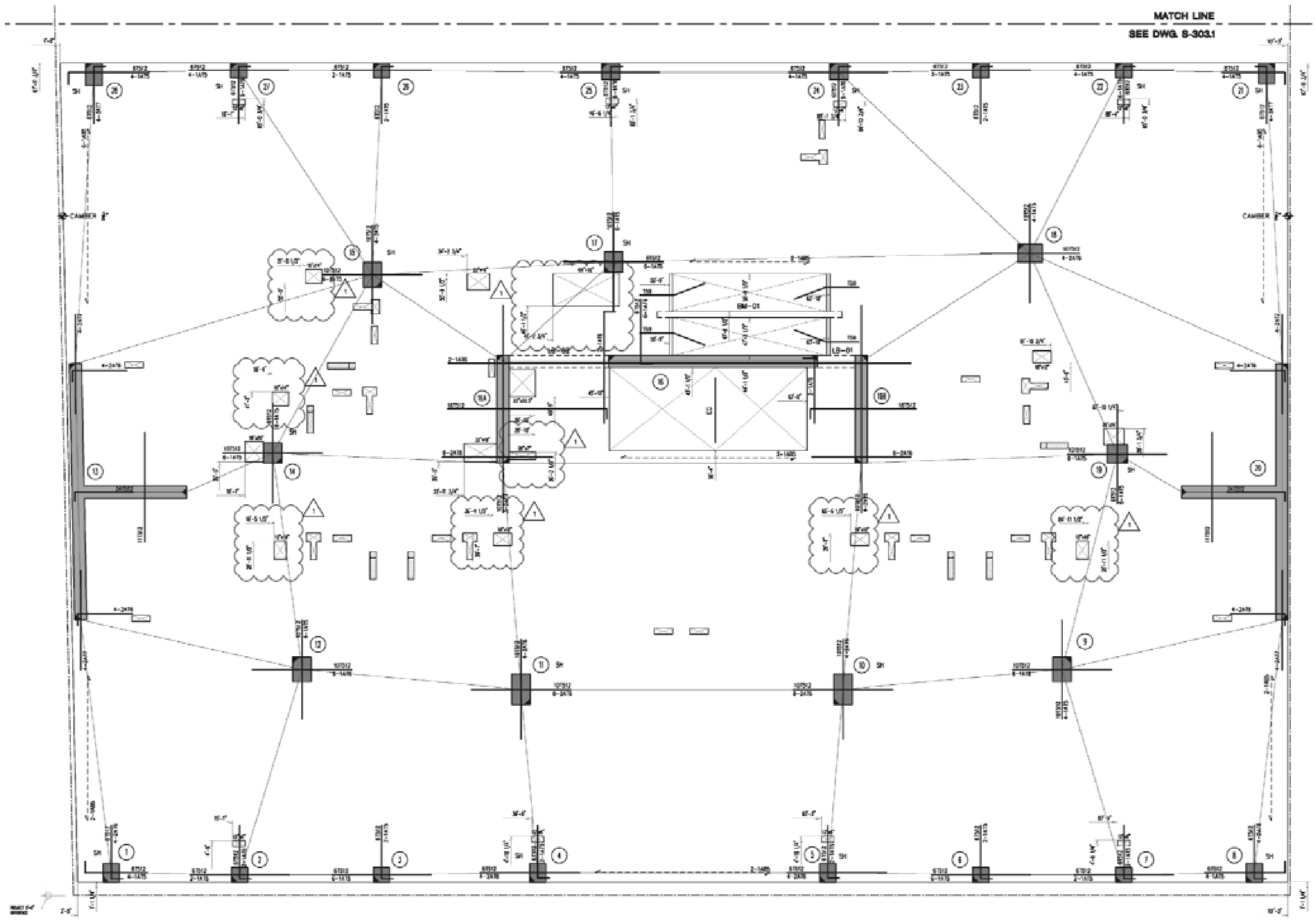
Existing Floor Framing Plan

Typicall framing plan (level 3-14). Most columns follow the same pattern the whole way through the building. All residential floors except for the 15th floor follow the same floor plan. On the 15th floor the building steps back to allow for large balcony space on the top floor.



8 Story Tower

Due to the small width of the tower, columns are able to be placed only at the exterior walls of the structure. This allows for completely open spaces, however the two way slab system does not meet the limitations of ACI 318-05 Chapter 13 for the direct design method.



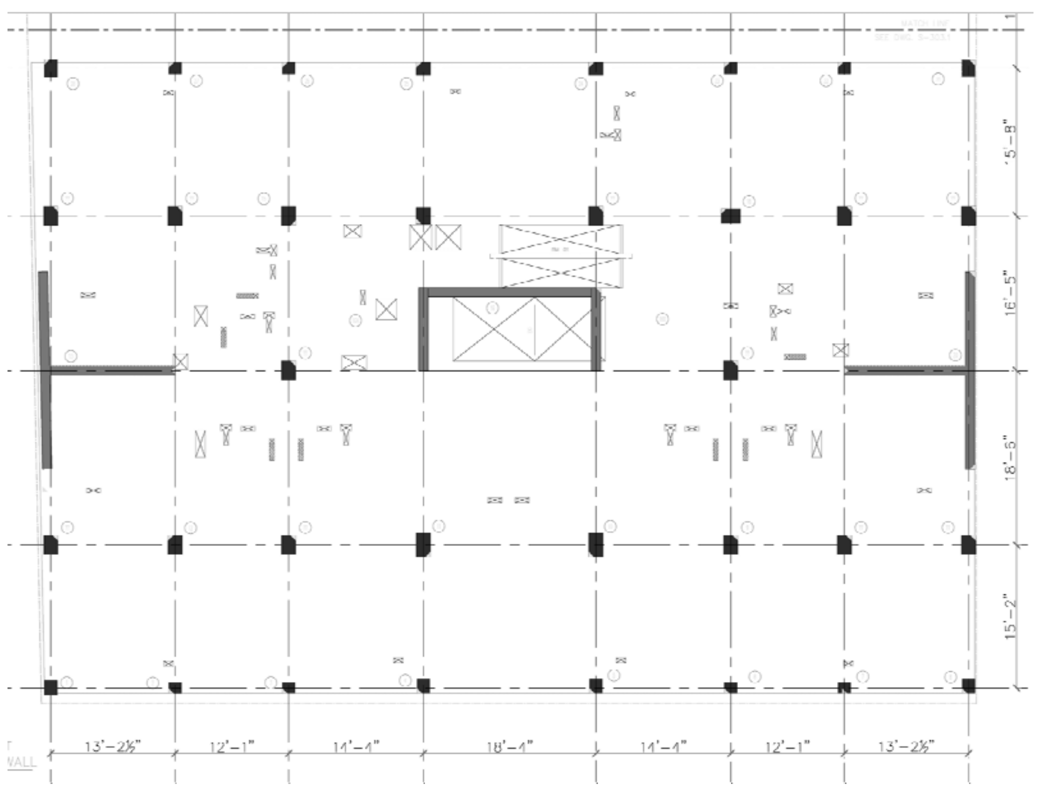
15 story tower

Bays do not follow a typical pattern or spacing, however most bays have a semi-regular shape that meets the design requirements of ACI 318-05 Chapter 13 for the direct design method. Columns are spaced to optimize both the retail and residential areas.

Two-Way Post Tensioned Flat Plate

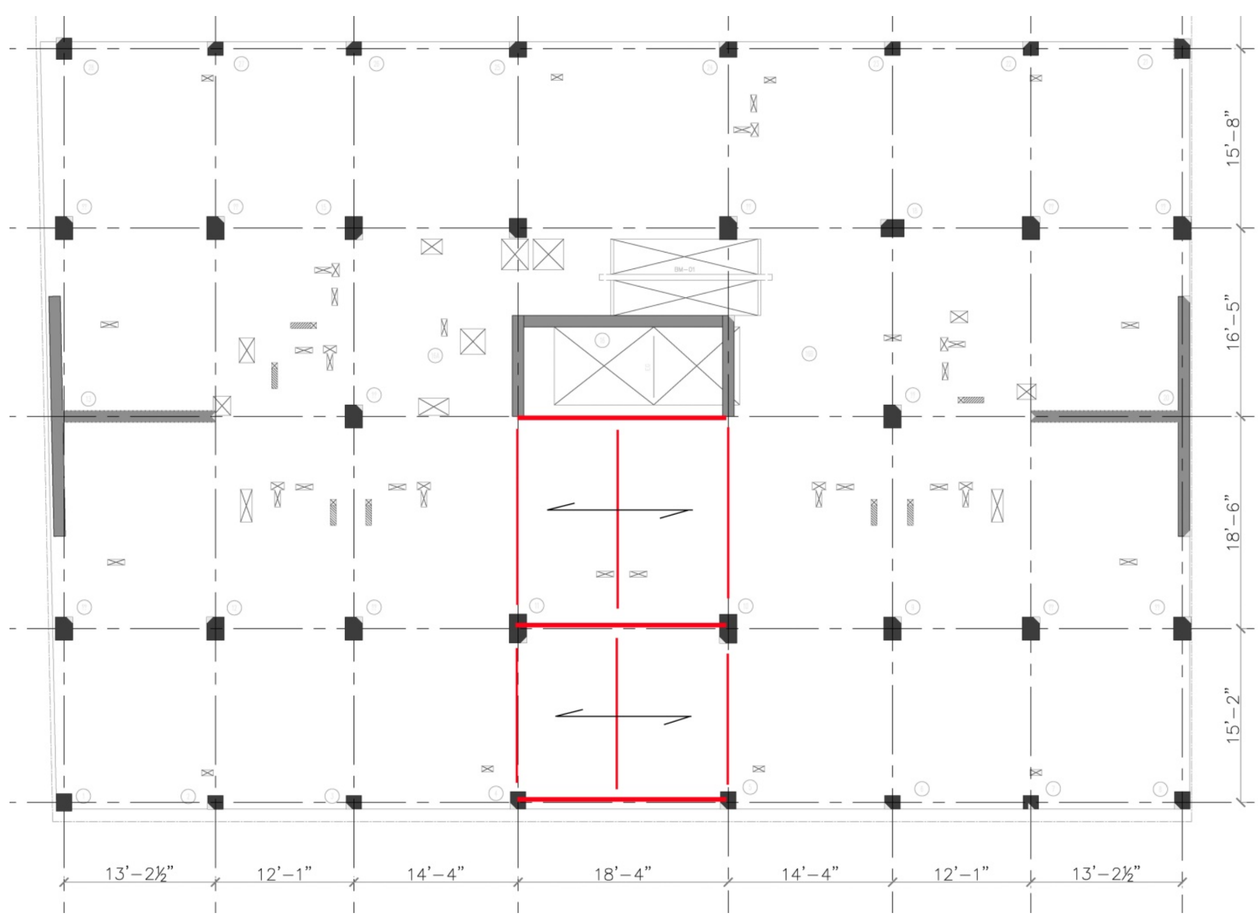
In order to make post tensioning more feasible, the column layout of the building needed to be changed into a regular type of pattern as shown below. Initial calculations of the post tensioned system indicate that a 6in. thick slab will work with a prestressing force of about 160 kips for each equivalent frame to balance the dead load. However, deflections were around 10 times less than allowable, and shear strength of the slab is about double that required. This suggests that it is possible to make the slab thinner, perhaps 5in. or less thick.

If the slab were to be 5 in. thick per floor, the building would have an extra (3”x15 stories) = 45” to play with in regards to the height. If the ceiling cavity is removed, an extra (12”x15 stories) = 15’ will be available to add an extra floor. In total, 18’-9” will be available, which could possibly yield an extra two stories of usable residential space. Although the cost of post tensioning the slab is greater than that of a mild steel reinforced two-way slab, an extra two floors of usable, rentable space will easily offset that initial cost.



Composite Steel and Concrete Floor

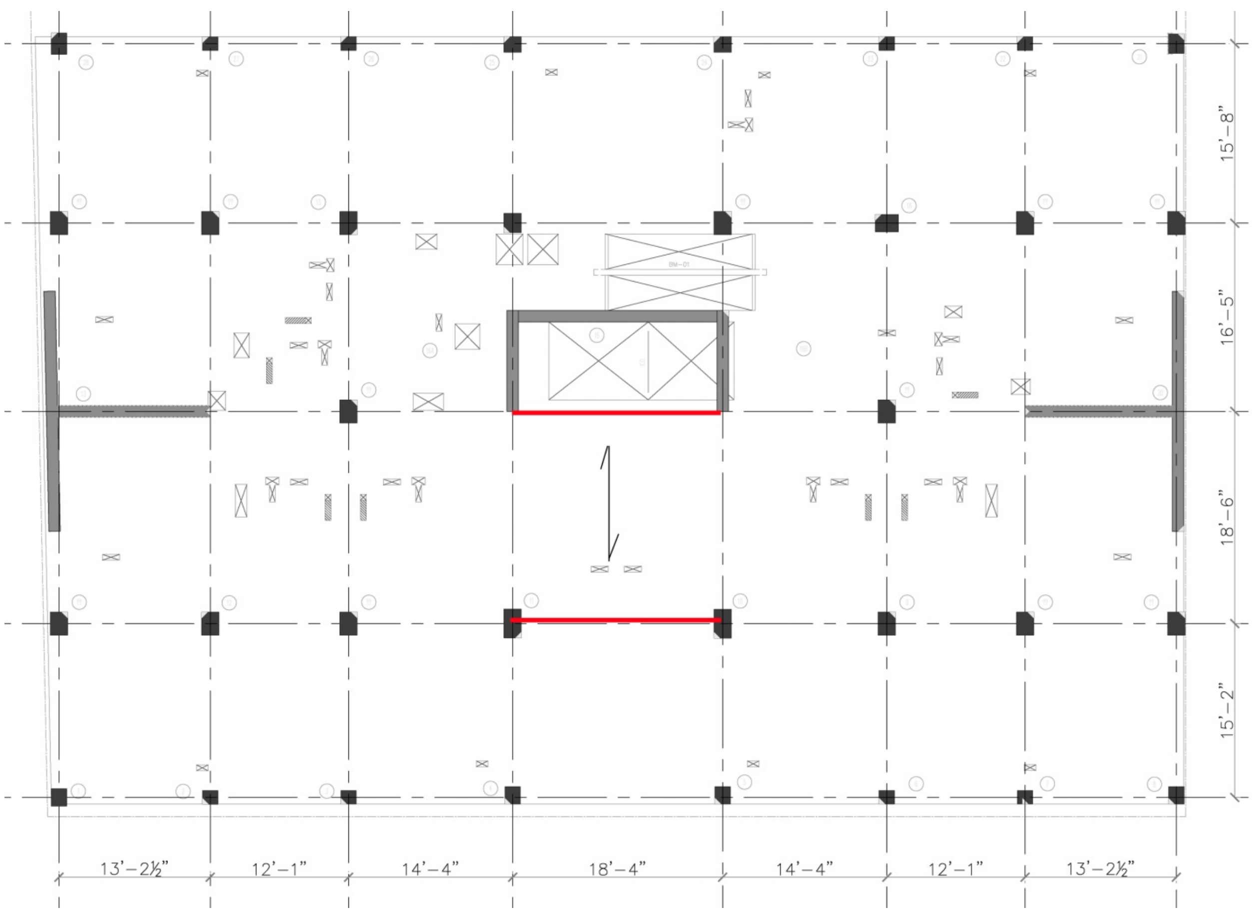
From the beginning, the feasibility of a composite steel and concrete slab system was highly unlikely due to the increase in floor framing depth. In order to prove this hypothesis an analysis was done on the largest bay possible to actually determine the viability. It was quickly found that due to the relatively small spans and light loads of the structure, a W section as small as a W12X22 could be used for girders and a W12X16 for intermediate beams without composite action between the beams and slab. Because there are not many sections smaller than those listed above, the cost of adding shear studs would most likely be greater than the cost of using the W12X22 and W12X16. With this system, the ceiling cavity would be about 12" + 5" (slab thickness) = 17". This is a significant increase from the current floor thickness and would not allow for the addition of another floor. The bays used for the analysis are shown below. (The red lines represent the beams and girders. Although represented by filled rectangles, columns are wide flange members.)



Precast Hollow Core Concrete Plank

For almost the same reasons, the precast hollow core concrete plant on wide flange beams is easily determined to be unfeasible. The floor depth due to 4" hollow core plank with a 2" concrete topping and approximately a 12" supporting beam becomes 18". This is slightly worse than that of the composite slab and beam system discussed above. Another floor could not be achieved within the same building height limitations. However, there is one up side to this form of construction.

Erection time of the floor system would decrease significantly, allowing for a faster schedule which would enable the owner to get tenants into the apartments sooner. This would allow the owner to begin collecting revenue at an earlier stage. However, allowing tenants in sooner, will not outweigh the benefits of adding two floors. The bay analyzed for hollow core concrete plank is shown below. (The red lines indicate the supporting beams. Although represented by filled rectangles, columns are wide flange members.)



Conclusion

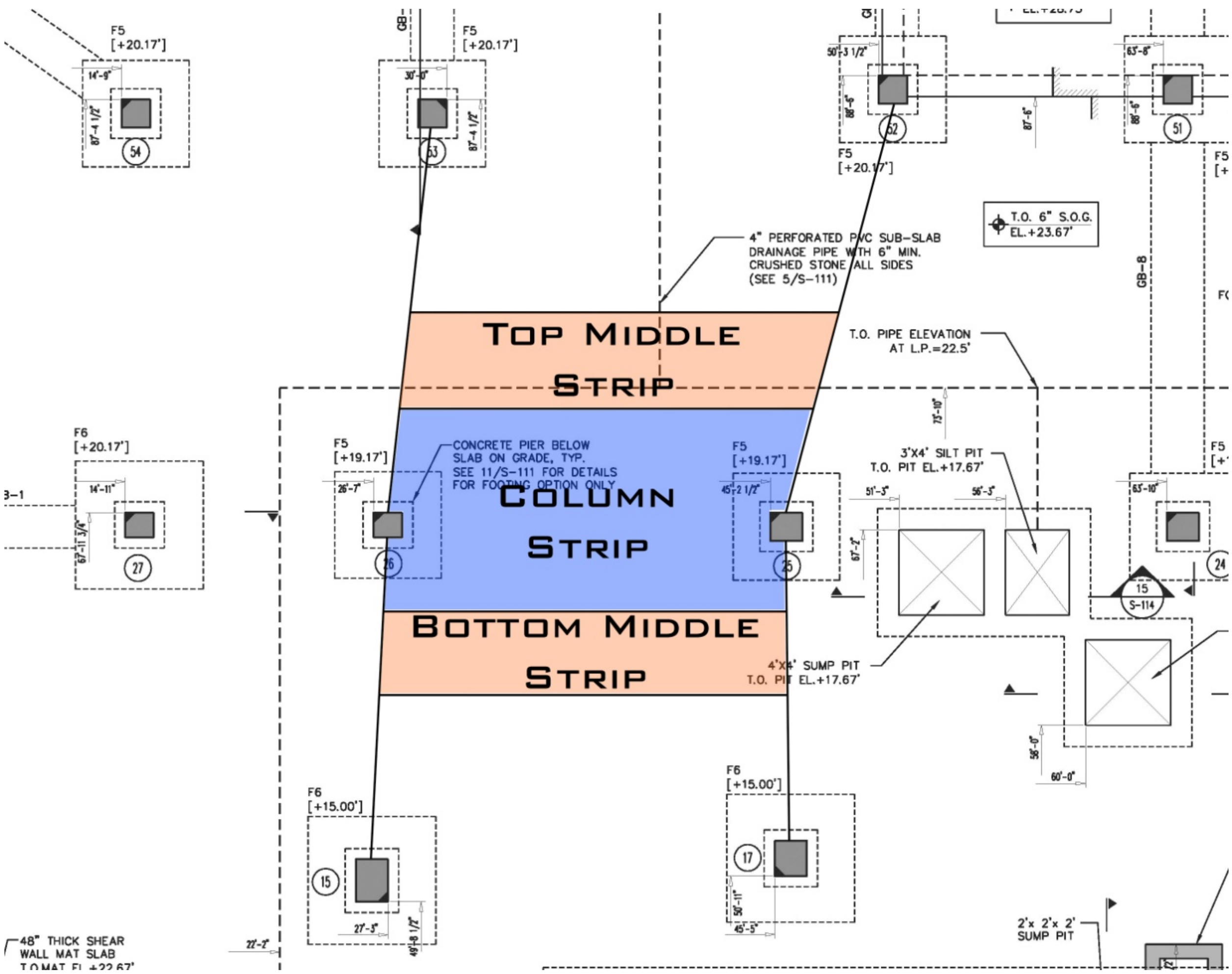
The analysis clearly shows that a two-way post tensioned flat plate floor system is the most feasible and economical alternative floor plan. Based on system strength, deflection, cost and thickness, the post tensioned floor system along with the modification of the HVAC system can yield another two rentable floors without building above the height restrictions. This will ultimately lead to higher monthly revenue obtained by the owner. The other systems analyzed may have the benefit of easier constructability, however the increased floor depth will not allow for additional floors while maintaining the same building height.

The application of a post tensioned slab needs to be further analyzed to determine if a thinner slab can be used to gain room for extra stories. The application of individual HVAC units for each apartment also needs to be researched to determine its feasibility. Although post tensioning and using individual HVAC units, may cost more up front, the addition of two extra floors will quickly overcome those costs by adding more apartments which will yield more revenue.

Appendix A – Calculations

Existing Two-Way Flat Plate Concrete

Bay Used for Direct Design Method (ACI 318-05 Chapter 13.)



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DIRECT DESIGN METHOD PER ACI 318-05 13.6

13.6.1 → LIMITATIONS

- MIN OF 3 CNTS. SPANS IN EACH DIRECTION ✓
- REGULAR PANELS WITH $\ell_1/\ell_2 \leq 2$ ✓
- SUCCESSIVE SPANS DO NOT DIFFER BY MORE THAN 1/3 THE LONGER SPAN ✓
- OFFSET OF COLUMNS BY A MAX OF 10%. ✓
- GRAVITY ONLY AN L ≤ 2D ✓

COL 15

CENTER @ (26'-6", 50'-8½")

COL 17

CENTER @ (46'-2", 51'-9")

COL 26

CENTER @ (27'-3", 68'-6¾")

SIZE = 16"x14"

COL 25

CENTER @ (45'-11½", 68'-7¾")

SIZE = 18"x16"

COL 53

CENTER @ (30'-8", 86'-8½")

COL 52

CENTER @ (50'-11½", 87'-10")

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$\ell_1 = (45'\text{-}11\tfrac{1}{2}") - (27'\text{-}3")$

$= 18'\text{-}8\tfrac{1}{2}"$

$\ell_n = (18'\text{-}8\tfrac{1}{2}") - \tfrac{1}{2}(18") - \tfrac{1}{2}(16")$

$= 17'\text{-}3\tfrac{1}{2}"$

$$\ell_2 = \frac{(87'\text{-}10") - (50'\text{-}8\tfrac{1}{2}")}{2}$$

$= 19'\text{-}3\tfrac{1}{4}"$

DEAD LOADS:

CONC. $= (150 \text{ PCF})(10/12 \text{ FT}) = 125$ PSF

MEP/PARTITIONS $= 15$ PSF

LIVE LOADS:

RETAIL/LOBBY $= 100$ PSF

$q_u = 1.2D + 1.6L$

$= 1.2(140) + 1.6(100)$

$= 328$ PSF

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 13.6.2.2$$

$$= \frac{(0.328)(19.27)(17.29)^2}{8}$$

$= 237$ ft-k

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NEGATIVE FACTORED MOMENT
& POSITIVE FACTORED MOMENT . . . . ACI 318-05 13.6.3.2

$M^- = (0.65)(237\ \text{ft-k})$
$= 154\ \text{ft-k}$

$M^+ = (0.35)(237\ \text{ft-k})$
$= 83\ \text{ft-k}$

COLUMN STRIP MOMENTS . . . . ACI 318-05 13.6.4.1

$\frac{\ell_1}{\ell_2} = \frac{18'-8\frac{1}{2}''}{19'-3\frac{1}{4}''} = 0.97$

$M^-_{\text{COL STRIP}} = (0.75)(154)$
$= 116\ \text{ft-k}$

$M^+_{\text{COL STRIP}} = (0.6)(83)$
$= 50\ \text{ft-k}$

MIDDLE STRIP MOMENTS . . . . ACI 318-05 13.6.6.1

$M^-_{\text{MID STRIP}} = (0.25)(154)$
$= 39\ \text{ft-k}$

$M^+_{\text{MID STRIP}} = (0.4)(83)$
$= 34\ \text{ft-k}$

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SLAB STRENGTH: ($f'_c = 5$ ksi, $f_y = 60$ ksi)

COLUMN STRIP:

$b = 115.6''$

$h = 10''$

$d = 10 - \underset{\text{CVR}}{0.75} - \underset{2\text{-}\#5 \text{ BARS}}{2(0.625)}$

NEGATIVE MOMENT REINF.

10-#5 AND 12-#6

$A_s = 10(0.31) + 12(0.44)$

$= 8.38 \text{ in}^2$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{(8.38)(60)}{(0.85)(5)(115.6)} = 1.02 \text{ in}$

$c = \dfrac{1.02}{\beta_1} = \dfrac{1.02}{0.8} = 1.28 \text{ in}$

$\dfrac{\varepsilon_y}{d-c} = \dfrac{\varepsilon_c}{c}$

$\Rightarrow \varepsilon_y = \dfrac{0.003}{1.28}(8 - 1.28) = 0.0158$

$\varepsilon_y > 0.005 \Longrightarrow$ TENSION CONTROLLED AND $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= (0.9)(8.38)(60)(8 - \frac{1.02}{2})$

$= 3389$ in-k

$\phi M_n = 282$ ft-k

$\Rightarrow \boxed{\phi M_n > M^-_{\text{COL STRIP}} \quad \therefore \underline{\text{ok}}}$

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POSITIVE MOMENT REINF.

#5 @ 12"

$\frac{115.6}{12} = 10$ BARS

$A_s = 10(0.31) = 3.1 \text{ in}^2$

$a = \frac{(3.1)(60)}{(0.85)(5)(115.6)} = 0.38$

$c = \frac{0.38}{0.8} = 0.48$

$\varepsilon_y = \frac{0.003}{0.48}(8 - 0.48)$

$= 0.047$

$\varepsilon_y > 0.005 \Rightarrow$ TENSION CONTROLLED AND $\phi = 0.9$

$\phi M_n = (3.1)(60)(8 - \frac{0.38}{2})$

$= 1452$ in-k

$\phi M_n = 121$ ft-k

$\Rightarrow \boxed{\phi M_n > M^+_{\text{COL STRIP}} \therefore \underline{\text{OK}}}$

MIDDLE STRIP:

b = 115.6"
h = 10"
d = 8"
f'c = 5ksi
fy = 60 KSI

NEGATIVE MOMENT REINF.

5 @ 12

$A_s = 9$ BARS

$A_s = 9(0.31) = 2.79 \text{ in}^2$

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$$a = \frac{(2.79)(60)}{(0.85)(5)(115.6)} = 0.34 \text{ in}$$

$$c = \frac{0.34}{0.8} = 0.43$$

$$\varepsilon_y = \frac{0.003}{0.43}(8 - 0.43) = 0.0528$$

$\varepsilon_y > 0.005 \Rightarrow$ TENSION CONTROLLED AND $\phi = 0.9$

$$\phi M_n = 0.9(2.79)(60)\left(8 - \frac{0.34}{2}\right)$$

$$= 1179 \text{ in-k}$$

$$\phi M_n = 98 \text{ ft-k}$$

$$\Rightarrow \boxed{\phi M_n > M^-_{\text{MID STRIP}} \therefore \underline{OK}}$$

POSITIVE MOMENT REINF.

SAME AS NEGATIVE MOMENT REINF.

$$\Rightarrow \phi M_n = 98 \text{ ft-k}$$

$$\boxed{\phi M_n > M^+_{\text{MID STRIP}} \therefore \underline{OK}}$$

STRENGTH OF COLUMN STRIP AND MIDDLE STRIP SELECTED EXCEEDS REQUIRED STRENGTH PER ACI 318-05 CHAPTER 13.

SLAB IS OK IN FLEXURE

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SLAB DEFLECTION:

LONGEST SPAN IS BTWN COL 11 & COL

$\ell_n = 24'\text{-}9\tfrac{1}{2}"$

MIN SLAB THICKNESS PER ACI 318-05 13.1.4

$t > 5\text{ in} \Rightarrow$ ok 9.5.3.2

$t > \dfrac{\ell_n}{36}$ .... TABLE 9.5(c)

$t > \dfrac{297.5}{36} = 8.26\text{ in} \Rightarrow$ ok

SLAB THICKNESS EXCEEDS THAT REQUIRED BY 9.5.3 ∴ DEFLECTIONS NEED NOT BE CALCULATED.

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SHEAR PER ACI 318-05 CHAPTER 13

↳ PROVISIONS OF CHAPTER 11 SHALL APPLY

TWO WAY ACTION (PUNCHING SHEAR) :

COL 25

18" X 16"

$d = 8"$

$\Rightarrow b_o = (16 + 8/2)2 + (18 + 8/2)2$ ...... ACI 318-05 11.12.1.2

$= 84"$

$V_c = $ MIN of:

- $\left(2+\frac{4}{\beta}\right)\sqrt{f'_c}\, b_o d$ 11.12.2.1
- $\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d$
- $4\sqrt{f'_c}\, b_o d$

$\beta = \frac{18}{16} = 1.125$

$\alpha_s = 40$ FOR INT. COLUMNS

$\left(2+\frac{4}{\beta}\right)\sqrt{f'_c}\, b_o d = \left(2+\frac{4}{1.125}\right)\sqrt{5000}(84)(8)$

$= 263{,}986^{lbs} = 263.98$ KIPS

$\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d = \left(\frac{(40)(8)}{84}+2\right)\sqrt{5000}(84)(8)$

$= 276{,}054^{lbs} = 276.05^{KIPS}$

$4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}(84)(8)$

$= 190{,}070^{lbs} = 190.07^{KIPS}$ ← GOVERNS

$V_c = 190.07$ KIPS

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TRIB AREA FOR COL 25

$$A_{TRIB} = \left(\frac{18'-7\frac{1}{2}"}{2} + \frac{18'-8\frac{1}{2}"}{2}\right)\left(\frac{20'-0"}{2} + \frac{15'-6"}{2}\right) - \left(\frac{22 \times 20}{144}\right)$$

$A_{TRIB} = 328.3\ FT^2$

$q_u = 328\ PSF$

$V_u = (328.3)(328)$
$= 107{,}683\ ^{lbs}$
$= 107.69\ ^{KIPS}$

$\phi V_c = (0.75)(190.07) = 142.55\ ^{KIPS}$

$\Rightarrow \boxed{\phi V_c > V_u \therefore \underline{ok}}$

BEAM ACTION SHEAR PER ACI 318-05 CHAPT 11:

$b_w = 115.6" \times 2 = 231\ in$

$V_c = 2\sqrt{f'_c}\, b_w d$ 11.3.1.1
$= 2\sqrt{5000}(231)(8)$
$= 261{,}346\ ^{lbs} = 261.34\ ^{KIPS}$

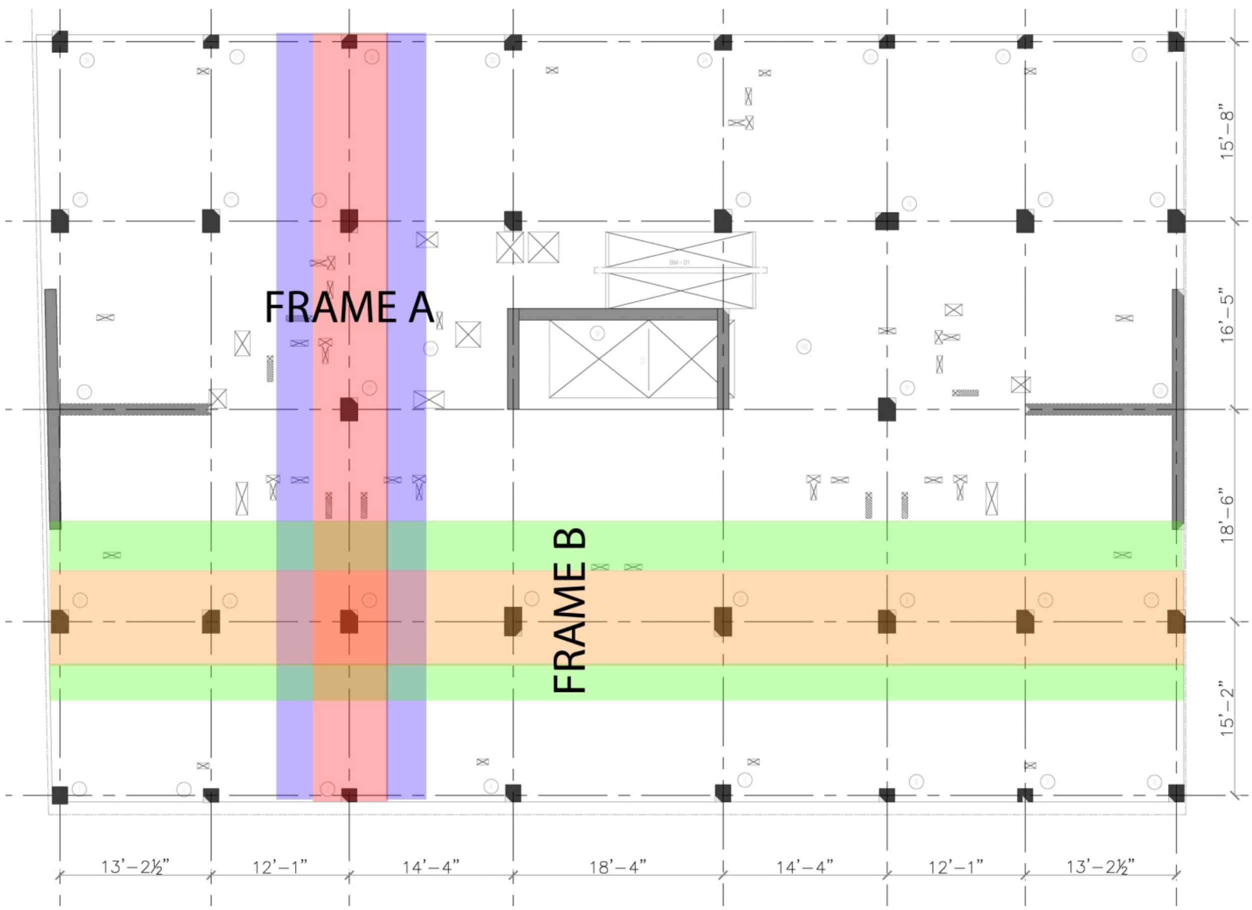
$\phi V_c = (0.75)(261.34)$
$= 196\ ^{KIPS}$

$\Rightarrow \boxed{\phi V_c > V_u \therefore \underline{ok}}$

Appendix B – Calculations

Post Tensioned Two-Way Flat Plate

The frames that were analyzed are shown below. The equivalent frame method was used to determine design moments.



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POST TENSIONED 2-WAY CONC. FLAT PLATE

SLAB THICKNESS:

$$\ell/40 = \frac{(18.5)(12)}{40} = 5.55$$

USE 6" SLAB

$\Rightarrow W_{SLAB} = 75$ PSF

ASSUME:

- NORMAL WT. CONC.
- $f'_c = 5000$ PSI → SLABS & 5.95 KSI → COLUMNS
- $f'_{ci} = 4000$ PSI
- $E_c = 57000\sqrt{5000} = 4000$ KSI → SLAB
- UNBONDED 6" ϕ STRANDS w/ AREA = 0.217 in²
- $f_{pu} = 270$ KSI
- $f_{pe} = 160$ KSI
- COVER TO CENTER OF STRAND = 1.25 in
- ALL COLUMNS ARE 20" x 20"
- $E_c = 57000\sqrt{5950} = 4400$ KSI → COLUMNS
- FLOOR TO FLOOR HEIGHT = 9.67 ft

2

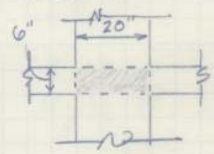
ELEMENT STIFFNESSES

COLUMN:

$$I_c = \frac{(20)(20)^3}{12} = 13{,}333 \text{ in}^4$$

$$K_c = \frac{4E_cI_c}{L-t} = \frac{4(4400\text{ ksi})(13{,}333\text{ in}^4)}{116\text{ in} - 6\text{ in}} = 2133280 \text{ k-in}$$

$$C_c = \sum\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3} = \left(1 - 0.63\frac{6}{20}\right)\frac{(6^3)(20)}{3} = 1168 \text{ in}^4$$



$$K_T = \frac{9E_cC_c}{\ell_2\left(1-\frac{c_2}{\ell_2}\right)^3} = \frac{9(4000\text{ ksi})(1168\text{ in}^4)}{(18.5)(12\text{in})\left(1-\frac{20}{(18.5)(12)}\right)^3} = 251418 \text{ k-in}$$

$$\frac{1}{K_{EQ,COL}} = \frac{1}{2K_c} + \frac{1}{2K_T} = \frac{1}{2(2133280)} + \frac{1}{2(251418)}$$

$$K_{EQ,COL} = 449{,}822 \text{ k-in}$$

SLAB:

FRAME A WIDTH $= \frac{1}{2}(145") + \frac{1}{2}(172") = 158.5"$

$$I_{s,A} = \frac{(158.5)(6)^3}{12} = 2{,}853 \text{ in}^4$$

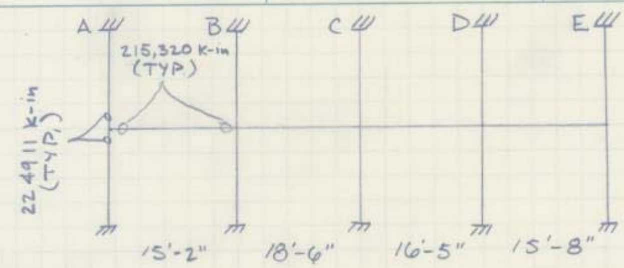
$$K_{s,A} = \frac{4E_cI_{s,A}}{\ell_1 - \frac{c_1}{2}} = \frac{4(4000\text{ ksi})(2853\text{ in}^4)}{(18.5)(12\text{in}) - \left(\frac{20\text{ in}}{2}\right)} = 215{,}320 \text{ k-in}$$

FRAME B WIDTH $= \frac{1}{2}(222") + \frac{1}{2}(182") = 202 \text{ in}$

$$I_{s,B} = \frac{(202)(6)^3}{12} = 3{,}636 \text{ in}^4$$

$$K_{s,B} = \frac{4E_cI_{s,B}}{\ell_1 - \frac{c_1}{2}} = \frac{4(4000\text{ ksi})(3636\text{ in}^4)}{(18.5)(12\text{ in}) - \left(\frac{20\text{ in}}{2}\right)} = 274{,}415 \text{ k-in}$$

3



FRAME A STIFFNESS

(FRAME B STIFFNESS SIMILAR)

DISTRIBUTION FACTORS:

FRAME A

$$DF_{AB} = DF_{ED} = \frac{215320}{449822 + 215320} = 0.324$$

$$DF_{BA} = DF_{BC} = DF_{CB} = DF_{CD} = DF_{DC} = DF_{DE}$$

$$= \frac{215320}{449822 + (2)(215320)} = 0.245$$

FIXED END MOMENTS:

FRAME A:

$$w_u = 1.2(15\text{ PSF} + 75\text{ PSF}) + 1.6(40\text{ PSF}) = 172\text{ PSF}$$

$$FEM_{AB} = \frac{(172)(13.21)(15.17 - \frac{20}{12})^2}{12} = 35\text{ ft-k}$$

$$FEM_{BC} = \frac{(172)(13.21)(18.5 - \frac{20}{12})^2}{12} = 54\text{ ft-k}$$

$$FEM_{CD} = \frac{(172)(13.21)(16.42 - \frac{20}{12})^2}{12} = 42\text{ ft-k}$$

$$FEM_{DE} = \frac{(172)(13.21)(15.67 - \frac{20}{12})^2}{12} = 38\text{ ft-k}$$

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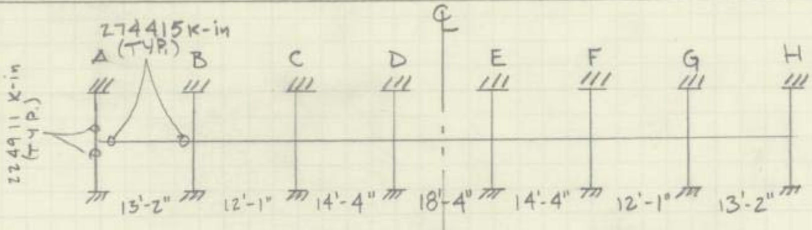
MOMENT DISTRIBUTION
FRAME A

DF	0.324	0.245	0.245	0.245	0.245	0.245	0.245	0.324
FEM	-35	+35	-54	+54	-42	+42	-38	+38
DIST	+11.34	+4.66	+4.66	-2.94	-2.94	-0.98	-0.98	-12.31
CO	+2.33	+5.67	-1.47	+2.33	-0.49	-1.47	-6.16	-0.49
DIST	-0.75	-1.03	-1.03	-0.45	-0.45	+1.87	+1.87	+0.16
CO	-0.52	-0.38	-0.23	-0.52	+0.94	-0.23	+0.08	+0.94
DIST	+0.17	+0.15	+0.15	-0.10	-0.10	+0.04	+0.04	-0.30
CO	+0.08	+0.09	-0.05	+0.09	+0.02	-0.05	-0.15	+0.02
MOMENT (K-FT)	-22.35	+44.16	-51.97	+52.41	-45.02	+41.18	-43.3	+26.02



FRAME B STIFFNESS

DISTRIBUTION FACTORS

FRAME B:

$$DF_{AB,HG} = \frac{274415}{449822 + 274415} = 0.379$$

$$DF_{BA,BC,CB,CD\ldots} = \frac{274415}{449822 + 2(274415)} = 0.275$$

FEM's

$$FEM_{AB} = \frac{(172)(16.83)(13.17 - \frac{20}{12})^2}{12} = 32\ \text{ft-k}$$

5

$$FEM_{BC} = \frac{(172)(16.83)(12.08 - \frac{20}{12})^2}{12} = 27 \text{ ft-k}$$

$$FEM_{CD} = \frac{(172)(16.83)(14.33 - \frac{20}{12})^2}{12} = 39 \text{ ft-k}$$

$$FEM_{DE} = \frac{(172)(16.83)(18.33 - \frac{20}{12})^2}{12} = 67 \text{ ft-k}$$

MOMENT DISTRIBUTION
FRAME B

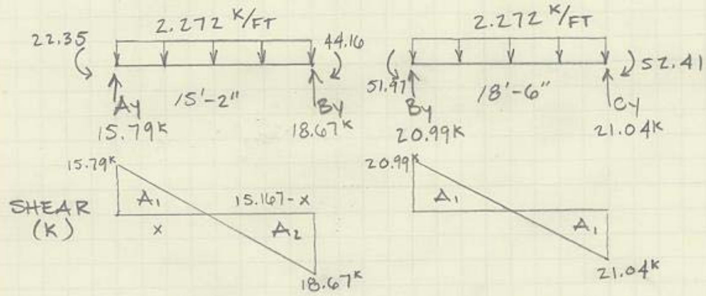
								℄
	0.379	0.275	0.275	0.275	0.275	0.275	0.275	0.275
FEM	-32	+32	-27	+27	-39	+39	-67	+67
DIST	+12.13	-1.38	-1.38	+3.3	+3.3	+7.7	+7.7	
CO	-0.69	+6.07	+1.65	-0.69	+3.85	+1.65	-1.65	
DIST	+0.26	-2.12	-2.12	-0.87	-0.87	0	0	
CO	-1.06	+0.13	-0.44	-1.06	0	-0.44	+0.44	
DIST	+0.4	+0.09	+0.09	+0.29	+0.29	0	0	
CO	+0.05	+0.2	+0.15	+0.05	0	+0.15	-0.15	
MOMENT (ft-K)	-20.91	+34.81	-29.05	+28.02	-32.43	+48.06	-60.66	

6

MOMENT DIAGRAMS

FRAME A:

* LOADS AND MOMENTS ARE FACTORED



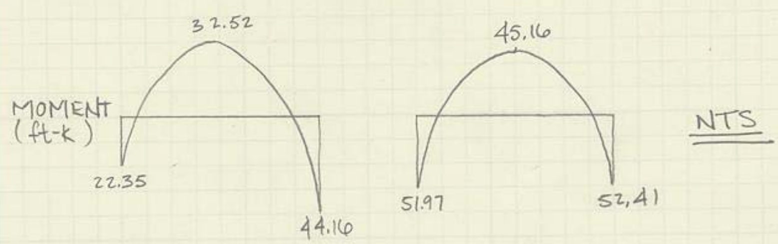
$$\frac{15.79}{x} = \frac{18.67}{15.167 - x}$$

$$x = 6.95^{K}$$

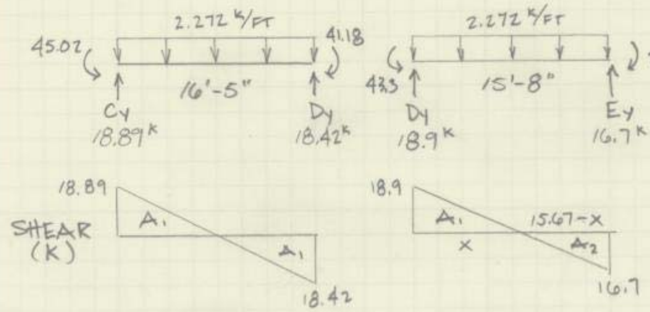
$$A_1 = \tfrac{1}{2}(6.95)(15.79) = 54.87$$

$$A_2 = \tfrac{1}{2}(8.22)(18.67) = 76.73$$

$$A_1 = \tfrac{1}{2}(9.25)(21) = 97.13 \text{ ft-k}$$



7



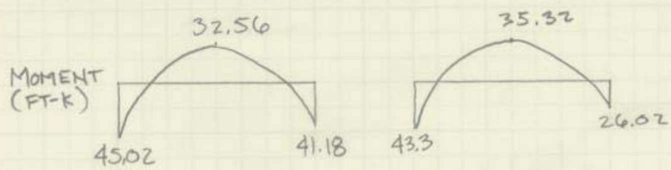
$A_1 = \frac{1}{2}(8.21)(18.9)$
$= 77.58$

$\frac{x}{18.9} = \frac{15.67-x}{16.7}$

$x = 8.32$

$A_1 = \frac{1}{2}(8.32)(18.9) = 78.62$

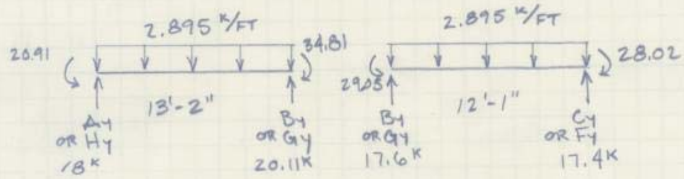
$A_2 = \frac{1}{2}(7.35)(16.7) = 61.37$

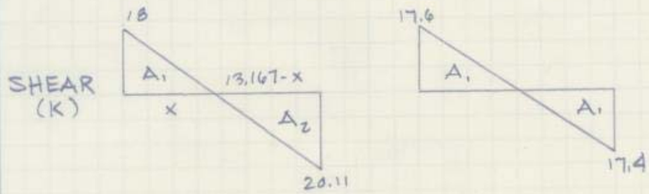


8

MOMENT DIAGRAMS

FRAME B : (FACTORED LOADS & MOMENTS)





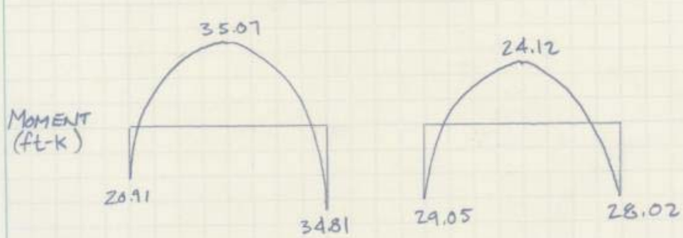
$$\frac{x}{18} = \frac{13.167 - x}{20.11}$$

$A_1 = \frac{1}{2}(6.042)(17.6) = 53.17$

$x = 6.22$

$$A_1 = \frac{1}{2}(18)(6.22) = 55.98$$

$$A_2 = \frac{1}{2}(20.11)(6.95) = 69.88$$



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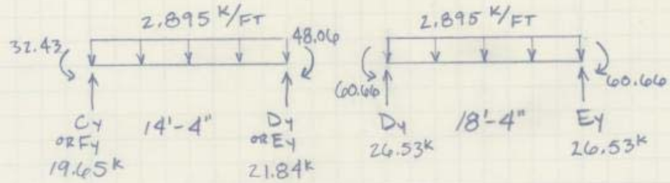
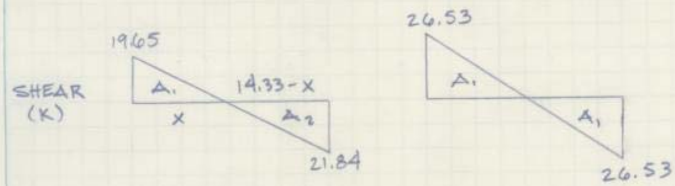




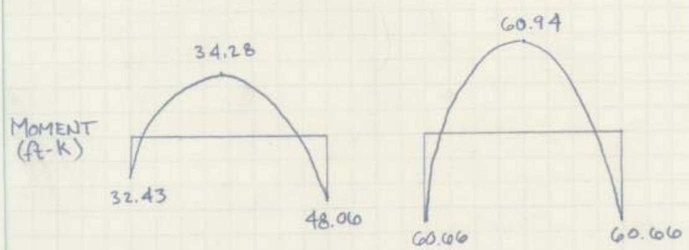
$$\frac{x}{19.65} = \frac{14.33-x}{21.84}$$

$$x = 6.79$$

$$A_1 = \tfrac{1}{2}(6.79)(19.65) = 66.71$$

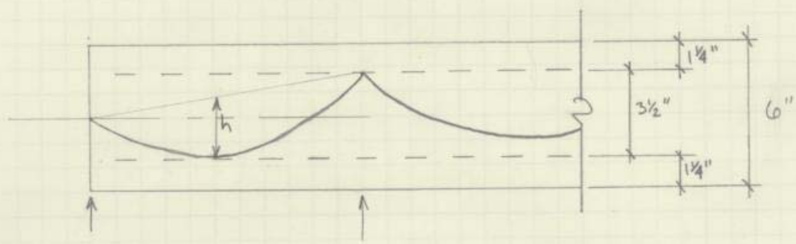
$$A_2 = \tfrac{1}{2}(7.54)(21.84) = 82.34$$

$$A_1 = \tfrac{1}{2}(9.167)(26.53) = 121.6$$



10

DESIGN TENDON LAYOUT



- ASSUME LOAD TO BE BALANCED IS TOTAL DEAD LOAD

FRAME A

$w_b = w_D = (15 \text{ PSF} + 75 \text{ PSF})(13.21 \text{ ft})/1000$

$= 1.189 \text{ K/FT}$

11

$$h = \frac{3\tfrac{1}{2}"}{2} + \frac{3\tfrac{1}{2}"}{5} = 2.45" \quad \text{(APPROXIMATION)}$$

REQUIRED PRESTRESSING FORCE:

$$F = \frac{w_b l^2}{8h} = \frac{(1.189)(15.17)^2}{8\left(\frac{2.45}{12}\right)} = 167 \text{ KIPS}$$

REQUIRED ECCENTRICITIES FOR OTHER SPANS

SPAN BC

$$h = \frac{w_b l^2}{8F} = \frac{(1.189)(18.5)^2}{8(167)} = 3.65"$$

SPAN CD

$$h = \frac{(1.189)(16.42)^2}{8(167)} = 2.88"$$

SPAN DE

$$h = \frac{(1.189)(15.66)^2}{8(167)} = 2.62 \text{ in}$$

$$\text{\# OF STRANDS} = \frac{167}{(0.217)(160)} \cong 5 \text{ STRANDS}$$

PLACE 75% IN COL STRIP ⟶ 3 STRANDS

$$\text{COL STRIP WIDTH} = \frac{145}{4} + \frac{172}{4} = 79.25"$$

$$\sigma_{\text{COL STRIP}} = \frac{(3)(0.217)(160)}{(79.25)(6)} \times 1000 = 219 \text{ psi}$$

MIDDLE STRIP WIDTH = 79.25"

$$\sigma_{\text{MID STRIP}} = \frac{2(0.217)(160)}{(79.25)(6)} \times 1000 = 146 \text{ PSI}$$

$$\text{AVG. PRESTRESS} = \frac{146 + 219}{2} = 183 \text{ PSI}$$

WITH IN RECOMENDED RANGE
∴ OK

12

SERVICE STRESSES:

$Z_{COL\ STRIP} = Z_{MID\ STRIP} = \frac{bh^2}{6} = \frac{(79.25)(6)^2}{6} = 475.5\ in^3$

SPAN AB (COL STRIP)

$M_{SERVICE} = \frac{75+15+40}{172} M_{FACTORED} = 0.756 M_{FACTORED}$

MAX $M_{NEGATIVE} = (0.756)(44.16)(12000) = 400620$ in-lb

MAX $M^+ = (0.756)(32.52)(12000) = 295021$ in-lb

FOR M^-: (ASSUME 75% GOES TO COL STRIP)

$\sigma_{top} = \frac{M}{Z}(0.75) - \sigma_c = \frac{400620}{475.5}(0.75) - 219$

$= 413$ psi

$\sigma_{ALLOW} = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530$ psi $\therefore$ ok

$\sigma_{bot} = 632 + 219 = 851$ psi

$\sigma_{ALLOW} = 0.45 f'_c = 0.45(5000) = 2250$ psi $\therefore$ ok

FOR M^+:

$\sigma_{top} = \frac{295021}{475.5}(0.75) + 219 = 684$ psi < 2250 $\therefore$ ok

$\sigma_{bot} = 465 - 219 = 246$ psi < 530 psi $\therefore$ ok

SPAN BC (COL STRIP)

$M^- = (52.41)(0.756)(12000) = 475464$ in-lb

$M^+ = (45.16)(0.756)(12000) = 409692$ in-lb

↳ OK BY INSPECTION

FOR M^-:

$\sigma_{top} = \frac{475464}{475.5}(0.75) - 219 = 531$ psi $\therefore$ ok

$\sigma_{bot} = 750 + 219 = 969$ psi $\therefore$ ok

13

SPAN CD

$M_{MAX} = (45.02)(0.756)(12000) = 408421$ in-lb

↳ OK BY INSPECTION

SPAN DE

MAX = 43.3

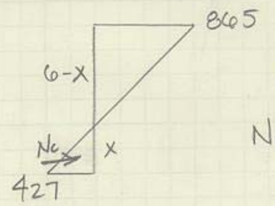
↳ OK BY INSPECTION

* BY INSPECTION, MIDDLE STRIP MEETS SERVICE STRESS CRITERIA.

$M^{+}_{MAX} = (0.756)(45.16)(12000) = 409692$ lb-in

$\sigma_{bot,MAX} = \frac{409692}{475.5}(0.75) - 219 = 427$ psi

$\sigma_{top} = 646 + 219 = 865$ psi



$\frac{x}{427} = \frac{6-x}{865} \Rightarrow x = 1.98$

$N_c = \frac{1}{2}(1.98)(427)(79.25)$
$= 33501$ lb

MIN BONDED REINF. AT BOTTOM:

$A_{s,MIN} = \frac{N_c}{0.5 f_y} = \frac{33501}{\frac{1}{2}(60000)} = 1.12 \text{ in}^2$

USE 6-#4 BARS → $A_s = 1.2 \text{ in}^2$

MIN BONDED REINF. AT TOP:

$A_{s,MIN} = 0.00075(6)(18.5)(12) = 1.00 \text{ in}^2$

USE 5-#4 BARS → $A_s = 1.00 \text{ in}^2$

14

ULTIMATE FLEXURAL STRENGTH

ASSUME $f_{ps} = f_{pe} + 7.5 = 160 + 7.5 = 167.5$ KSI

$A_{ps} = 5(0.217) = 1.085\ in^2$

$A_s = 1.2\ in^2$

$d_p = d_s = d_e = 4.75\ in$

$f_y = 60$ KSI

$$a = \frac{A_{ps}f_{ps} + A_s f_y}{(0.85)f'_c\, b} = \frac{(1.085)(167.5) + (1.2)(60)}{(0.85)(5)(158.5)}$$

$a = 0.377$

$$c = \frac{a}{\beta_1} = \frac{0.377}{0.8} = 0.471\ in^2$$

$$\varepsilon_s = \frac{0.003}{0.471}(4.75 - 0.471) = 0.0273 > 0.005$$

$\Rightarrow \phi = 0.9$

$$\phi M_n = (0.9)(A_{ps}f_{ps} + A_s f_y)(d - \tfrac{a}{2})$$
$$= (0.9)[(1.085)(167.5) + (1.2)(60)](4.75 - \tfrac{0.377}{2}) \times \tfrac{1}{12}$$
$$= 86.8 \text{ ft-K}$$

$\Rightarrow \phi M_n > M_u \therefore$ OK

PUNCHING SHEAR:

$$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$$

$b_o = 4(20 + 4.75) = 99$

$\alpha_s = 40$ (INT. COL)

$$V_c = \left(\frac{(40)(4.75)}{99} + 2\right)\sqrt{5000}\,(99)(4.75) \times \frac{1}{1000}$$
$$= 130.32 \text{ KIPS}$$

15

$\phi V_c = 0.75(130.32) = 97.74^k$

$V_u = (172\ PSF)\left[\frac{18.5 + 15.17}{2} \times \frac{18.33 + 14.33}{2}\right] \times \frac{1}{1000}$

$V_u = 47.3^k$

$\phi V_c > V_u \therefore ok$

DEFLECTION :

$\Delta_{LL} = K \frac{\omega \ell^4}{E_c h^3}$

ASSUME $K = 0.11$

$\Delta_{LL} = 0.11 \frac{(40)(12 \times 18.33)^4}{144(4)(10^6)(6^3)} = 0.08\ in$

$\frac{\ell}{360} = \frac{(12)(18.33)}{360} = 0.61\ in$

$\Rightarrow \Delta_{LL} < \frac{\ell}{360} \therefore ok$

16

FRAME B:

REQ'D PRESTRESSING FORCE:

$w_b = (15+75)(16.83)\frac{1}{1000} = 1.515\ ^{k}/_{FT}$

MIN PRESTRESSING FORCE:

$M_{MAX} = (0.756)(60.94)(12000) = 552848\ \text{lb-in}$

$\sigma_{ALLOW,TENSION} = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530\ \text{psi}$

$Z = \frac{(101)(6)^2}{6} = 606\ in^3$

$\frac{M}{Z} - \sigma_{ps} = \sigma_{ALLOW}$

$\sigma_{ps} = \frac{552848}{606} \times 0.75 - 530 = 154\ \text{psi}$

$F_{ps} = (154)(101)(6) \times \frac{1}{1000} = 93.3^{k}$

$F_{REQD} = \frac{w_b l^2}{8h} = \frac{(1.515)(13.17)^2}{8\left(\frac{2.45}{12}\right)} = 161^{k}$ $\therefore$ OK

ECCENTRICITIES FOR OTHER SPANS

SPAN BC & FG

$h = \frac{w_b l^2}{8F} = \frac{(1.515)(12.083)^2}{8(161)} = 2.06\ in$

SPAN CD & EF

$h = \frac{(1.515)(14.33)^2}{8(161)} = 2.9\ in$

SPAN DE

$h = \frac{(1.515)(18.33)^2}{8(161)} = 4.74\ in$

17

$\# \text{ OF STRANDS} = \frac{161}{(0.217)(160)} \cong 5 \text{ STRANDS}$

PLACE 3 IN COL STRIP & 1 IN EACH ½ MID STRIP

$\sigma_{ps,\,COL\,STRIP} = \frac{(3)(0.217)(160)}{(101)(6)} \times 1000 = 172 \text{ PSI}$

$\sigma_{ps,\,MID\,STRIP} = \frac{(2)(0.217)(160)}{(101)(6)} \times 1000 = 115 \text{ PSI}$

$\sigma_{ps,\,AVG.} = \frac{115 + 172}{2} = 144 \text{ PSI}$ ⇒ WITHIN RECOMENDED RANGE ∴ OK

SERVICE STRESSES:

$M^-_{MAX} = (60.66)(0.756)(12000) = 550308 \text{ in-lb}$

$M^+_{MAX} = (60.94)(0.756)(12000) = 552848 \text{ in-lb}$

FOR M^-:

$\sigma_{top} = \frac{M}{S} - \sigma_{ps} = \frac{550308}{606}(0.75) - 172 = 509 \text{ psi}$

$\sigma_{ALLOW} = 530 \text{ psi} > \sigma_{top}$ ∴ OK

$\sigma_{bot} = \frac{M}{S} + \sigma_{ps} = 681 + 172 = 853 \text{ psi}$

$\sigma_{ALLOW} = 0.45f'_c = 0.45(5000) = 2250 \text{ psi}$

⇒ $\sigma_{ALLOW} > \sigma_{bot}$ ∴ OK

FOR M^+:

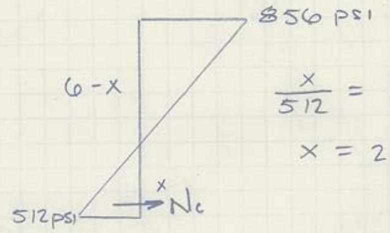
$\sigma_{top} = \frac{552848}{606} \times 0.75 + 172 = 856 \text{ psi}$ ∴ ok

$\sigma_{bot} = 684 - 172 = 512 \text{ psi}$ ∴ ok

* BY INSPECTION, MID STRIP MEETS SERVICE STRESS CRITERIA

MIN BONDED REINF

BOTTOM



$$\frac{x}{512} = \frac{6-x}{856}$$

$$x = 2.246$$

$$N_c = \tfrac{1}{2}(2.246)(512)(101) = 58073^{\text{lb}}$$

MIN BONDED BOTTOM REINF:

$$A_{s,MIN} = \frac{N_c}{0.5 f_y} = \frac{58073}{\tfrac{1}{2}(60000)} = 1.94 \text{ in}^2$$

USE 10-#4 BARS $\rightarrow A_s = 2.0 \text{ in}^2$

MIN BONDED TOP REINF:

$$A_{s,MIN} = 0.00075(6)(18.33)(12) = 1.00 \text{ in}^2$$

USE 5-#4 BARS $\rightarrow A_s = 1.00 \text{ in}^2$

19

ULTIMATE FLEXURAL STRENGTH

Assume $f_{ps} = f_{pe} + 7.5 = 167.5^{K}$

$A_{ps} = 5(0.217) = 1.085 \text{ in}^2$

$A_s = 1.00 \text{ in}^2$

$d_p = d_s = d_e = 4.75 \text{ in}$

$f_y = 60 \text{ KSI}$

$$a = \frac{A_{ps} f_{ps} + A_s f_y}{0.85 f'_c b} = \frac{(1.085)(167.5) + (1.00)(60)}{(0.85)(5)(202)}$$

$$= 0.282 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{0.282}{0.8} = 0.353 \text{ in}$$

$$\varepsilon_s = \frac{0.003}{0.353}(4.75 - 0.353) = 0.0374$$

$$\varepsilon_s > 0.005 \Rightarrow \phi = 0.9$$

$$\phi M_n = \phi(A_{ps} f_{ps} + A_s f_y)(d - \tfrac{a}{2})$$

$$= (0.9)[(1.085)(167.5) + (1.0)(60)](4.75 - \tfrac{0.282}{2})$$

$$= 1003 \text{ K-in}$$

$\phi M_n = 83.6 \text{ ft-K}$

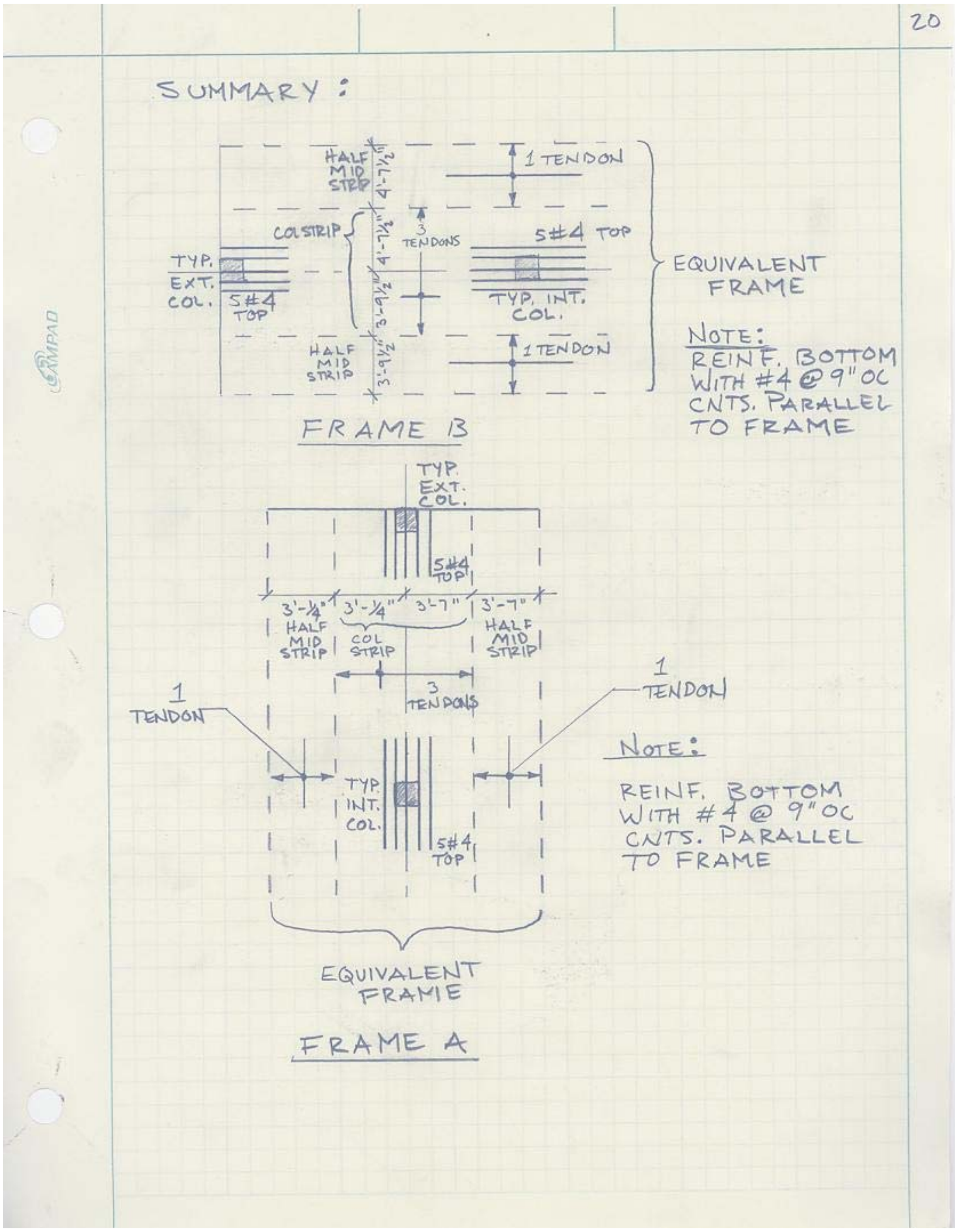
$\Rightarrow \phi M_n > M_u \therefore$ OK

PUNCHING SHEAR OK (FROM PREVIOUS CALC) (ON PG. 14)

DEFLECTION:

$$\Delta_{LL} = K \frac{w \ell^4}{E_c h^3} = \frac{(0.11)(40)(12 \times 18.5)^4}{144(4)(10^6)(6^3)} = 0.09 \text{ in}$$

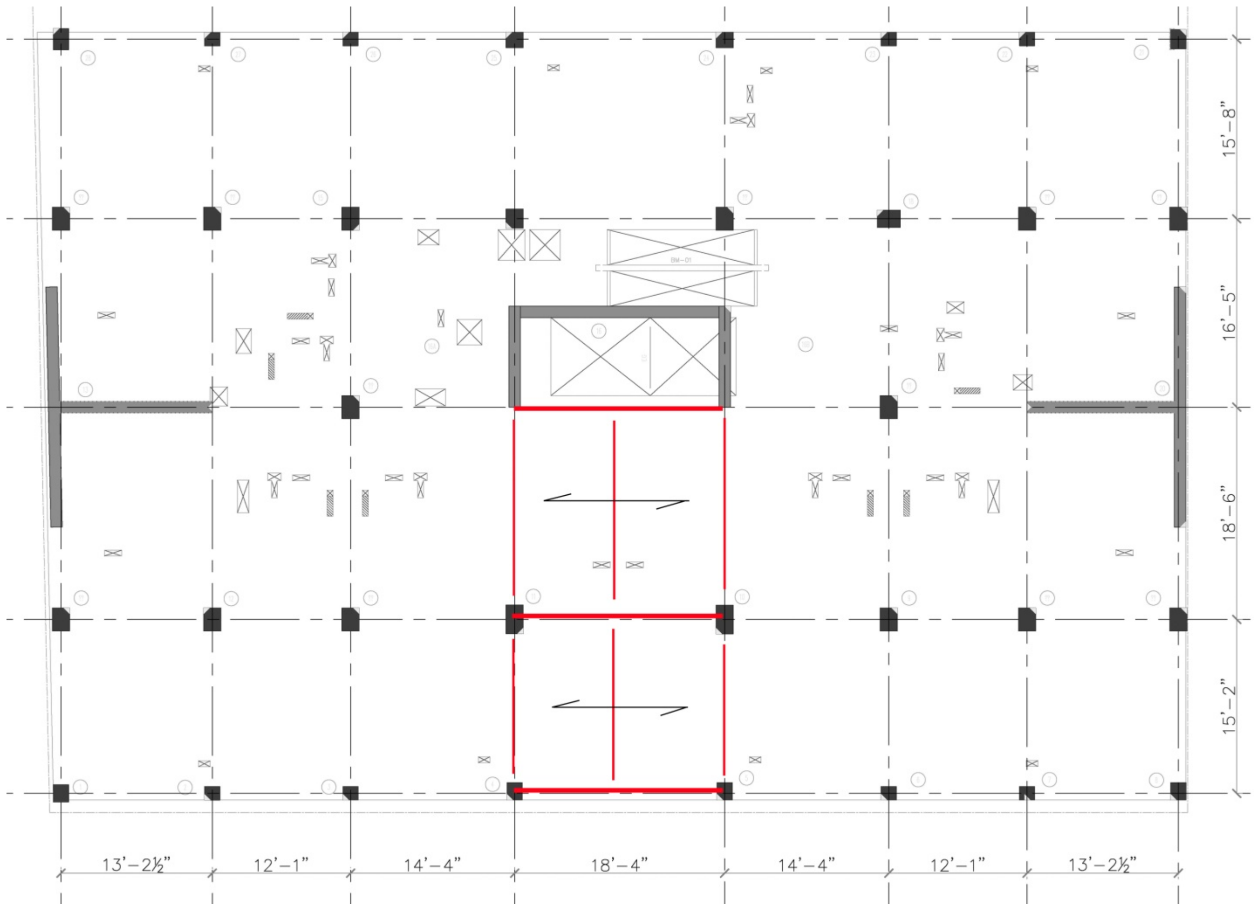
$$\frac{\ell}{360} = \frac{(12)(18.5)}{360} = 0.62 \text{ in} > \Delta_{LL} \therefore \text{OK}$$


20
SUMMARY:
HALF MID STRIP
4'-7½"
1 TENDON
COL STRIP
4'-7½"
3 TENDONS
5#4 TOP
TYP. EXT. COL.
5#4 TOP
3'-9½"
TYP. INT. COL.
EQUIVALENT FRAME
HALF MID STRIP
3'-9½"
1 TENDON
NOTE: REINF. BOTTOM WITH #4 @ 9" OC CNTS. PARALLEL TO FRAME
FRAME B
TYP. EXT. COL.
5#4 TOP
3'-¼"
3'-¼"
3'-7"
3'-7"
HALF MID STRIP
COL STRIP
HALF MID STRIP
1 TENDON
3 TENDONS
1 TENDON
NOTE:
REINF. BOTTOM WITH #4 @ 9" OC CNTS. PARALLEL TO FRAME
TYP. INT. COL.
5#4 TOP
EQUIVALENT FRAME
FRAME A

Appendix C – Calculations

Composite Steel Beam and Slab

The frames analyzed are shown below using Red to outline the Girders and Spandrel beams. The direction of the deck span is also shown.



1

COMPOSITE STEEL & CONC. FLOOR SYSTEM

CHOOSE DECK

TRY 18 GAGE 1.5" LOK-FLOOR FROM UNITED STEEL DECK

MAX UNSHORED SPAN = 9.36 ft (3 SPANS)

$\phi M_{nf} = 55.7$ in-k

$w_u = 1.2(\underbrace{15}_{\text{MEP \& PARTITION}} + \underbrace{5}_{\text{BM}} + \underbrace{39}_{\text{SLAB}}) + 1.6(40) = 135$ PSF

$M_u = \dfrac{(135)(1)(9.36)^2}{8} \times \dfrac{12}{1000} = 17.75$ in-k

$\phi M_{nf} > M_u \therefore$ OK

DESIGN TYP. WIDE FLANGE BM

LOADS:

SLAB = 39(9.36) = 365 PLF

CONST. LIVE = 10(9.36) = 94 PLF

SELF = 30 PLF (ASSUMED)

LIVE = 40(9.36) = 375 PLF

CONST. DEFLECTION GOVERNS

$$\Delta \leq \begin{cases} \ell/360 = \dfrac{(18.5)(12)}{360} = 0.62 \text{ in} \\ 1.0 \text{ in} \end{cases}$$

$w = 0.365 + 0.094 + 0.03 = 0.489$ k/ft

$$\Delta = \frac{5w\ell^4}{384EI}$$

$$\Rightarrow \frac{5(0.489)(18.5)^4}{384(29000)I_{bm}}(1728) \leq 0.62$$

$I_{bm} \geq 72$ in^4

2

REQUIRED CONST. STRENGTH

$w_u = 1.2(15+5+39) + 1.6(10) = 87 \text{ PSF}$

$M_u = \dfrac{(87)(9.36)(18.5)^2}{8} \times \dfrac{1}{1000} = 35 \text{ ft-k}$

$I_{W12\times16} = 103 \text{ in}^4 > I_{REQD} = 72 \text{ in}^4$

$\phi M_{p\,W12\times16} = 75.4 \text{ ft-k} > M_u = 35 \text{ ft-k}$

CHECK ULTIMATE STRENGTH:

$w_u = 1.2(15+5+39) + 1.6(40) = 135 \text{ PSF}$

$M_u = \dfrac{(135)(9.36)(18.5)^2}{8} \times \dfrac{1}{1000} = 54 \text{ ft-k}$

$\phi M_{p\,W12\times16} = 75.4 \text{ ft-k} > M_u = 54 \text{ ft-k}$

$\Rightarrow$ NO NEED TO MAKE BMS COMPOSITE

CHECK PLAUSIBILITY OF MAKING GIRDERS COMPOSITE:

- ASSUME GIRDERS HAVE POINT LOAD AT CENTER

GIRDER SPAN = 18.33 ft

$w_u = 135 \text{ PSF}$

$\text{POINT LOAD} = \left(\dfrac{18.33\text{ft}}{2}\right)\left(\dfrac{18.5\text{ft}}{2} + \dfrac{15.167\text{ft}}{2}\right)(135 \text{ PSF})$

$= 20.83^{K}$

$M_u = \dfrac{(20.83^{K})(18.33\text{ft})}{4} = 96 \text{ ft-k}$

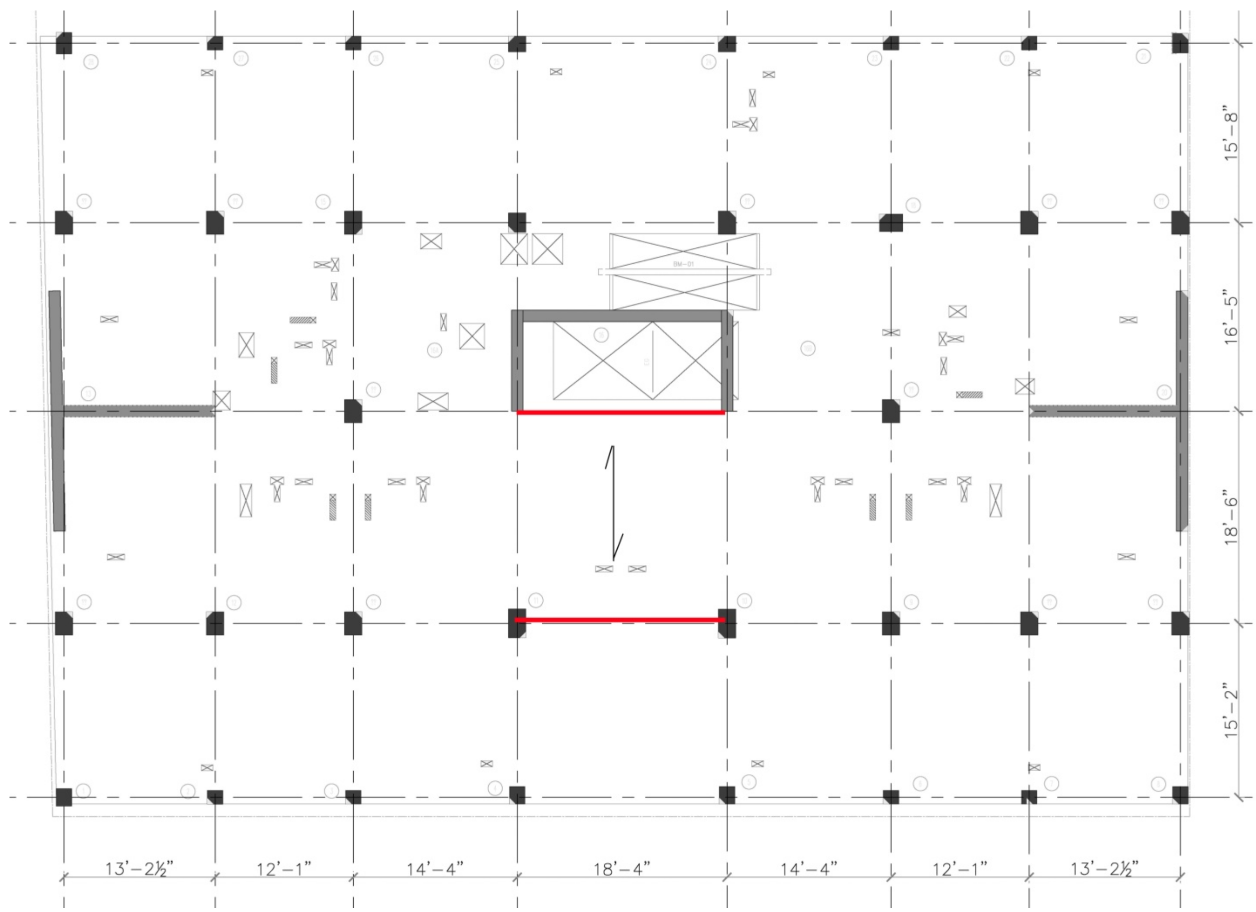
$W12\times22 \rightarrow \phi M_p = 110 \text{ ft-k}$

* BEAMS ARE ALREADY SMALL, NO NEED TO MAKE USE OF COMPOSITE ACTION.

Appendix D – Calculations

Precast Hollow Core Concrete Plank

The bay used to analyze the feasibility of hollow core concrete plank is shown below. Red illustrates supporting beams. The span direction of the planks is also shown



PRESTRESSED CONCRETE HOLLOW CORE PLANK

MAX SPAN = 18'-6"

USE 6" x 4'-0" WITH 2" TOPPING

STRAND PATTERN → 4 - ½" ϕ

⇒ SAFE LOAD = 141 PSF

$w_u = 1.2(15+5) + 1.6(40) = 88$ PSF < 141

$\therefore$ ok

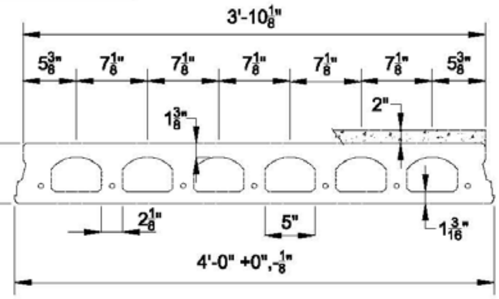
Prestressed Concrete
6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2} Precast S_{bc} = 370 in.^{3}
I_c = 1519 in.^{4} Topping S_{tc} = 551 in.^{3}
Y_{bc} = 4.10 in. Precast S_{tc} = 799 in.^{3}
Y_{tc} = 1.90 in. Wt.= 195 PLF
Wt.= 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 67.5 k-ft
   7-1/2"Ø, 270K = 104.2 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
4 - 1/2"ø	LOAD (PSF)	227	187	360	306	268	229	194	165	141	120	102	86	73	61	50				
7 - 1/2"ø	LOAD (PSF)	367	305	495	455	418	387	340	312	275	243	215	189	167	147	130	114	97	83	70

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

05/14/07

6F2.0T