Structural Technical Report 2
(Alternate Framing Systems)

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Executive Summary

The Christina Landing Apartment Tower is a 22 story apartment building located just outside center city Wilmington, DE. The tower provides 250,000 square feet of floor space. The structure is a predominately cast-in-place concrete building. Its floors are supported by a two way flat slab system. The typical floor system also incorporates small areas of reinforced concrete and post-tensioned beams to aid the lateral force resisting system. The floors are supported by square and round concrete columns. Lateral forces induced on the building are resisted by a box of four shear walls. All columns and shear walls rest on a foundation system of H-piles and pile caps. Typical floor loads are 130psf dead load and 40psf live load.

For this report four alternate floor system designs were analyzed in addition to the existing design. The existing design is an 8" flat slab with top and bottom reinforcing. This system was found to be sufficient for the applied loads. The first alternate designed was a 7.5" flat plate with 3.5" drop panel. This system saves concrete, however it has less constructability than the existing slab. Second, a steel frame with a 4" composite deck was analyzed over 4 bays using RAM structural system and hand calculations. Typical beam sizes were W14x22s and typical girders were W18x35s. This system is not practical for the existing column grid because of span lengths and constructability issues. The third system analyzed was steel joists with a 3.5" metal deck. This system was also analyzed using RAM and checked with the New Columbia Joist manual. Similarly to the steel alternate this system is unlikely due to the floor layout. Lastly a 7" post-tensioned slab was designed for the floor system. This system was found to be sufficient with 82k of pre-stressing force per foot for a typical bay.

After evaluation of each system it seems that both the steel with composite deck system and the joist with metal deck system are unlikely candidates for a redesign because of the unusual column layout and spans. For these systems fabrication and erection of steel would be difficult. Both flat plate with drop panels and the post-tensioned slab are more likely to be favorable systems for a redesign. Both systems provide potential significant material savings but also require a higher constructability cost.

Building Introduction

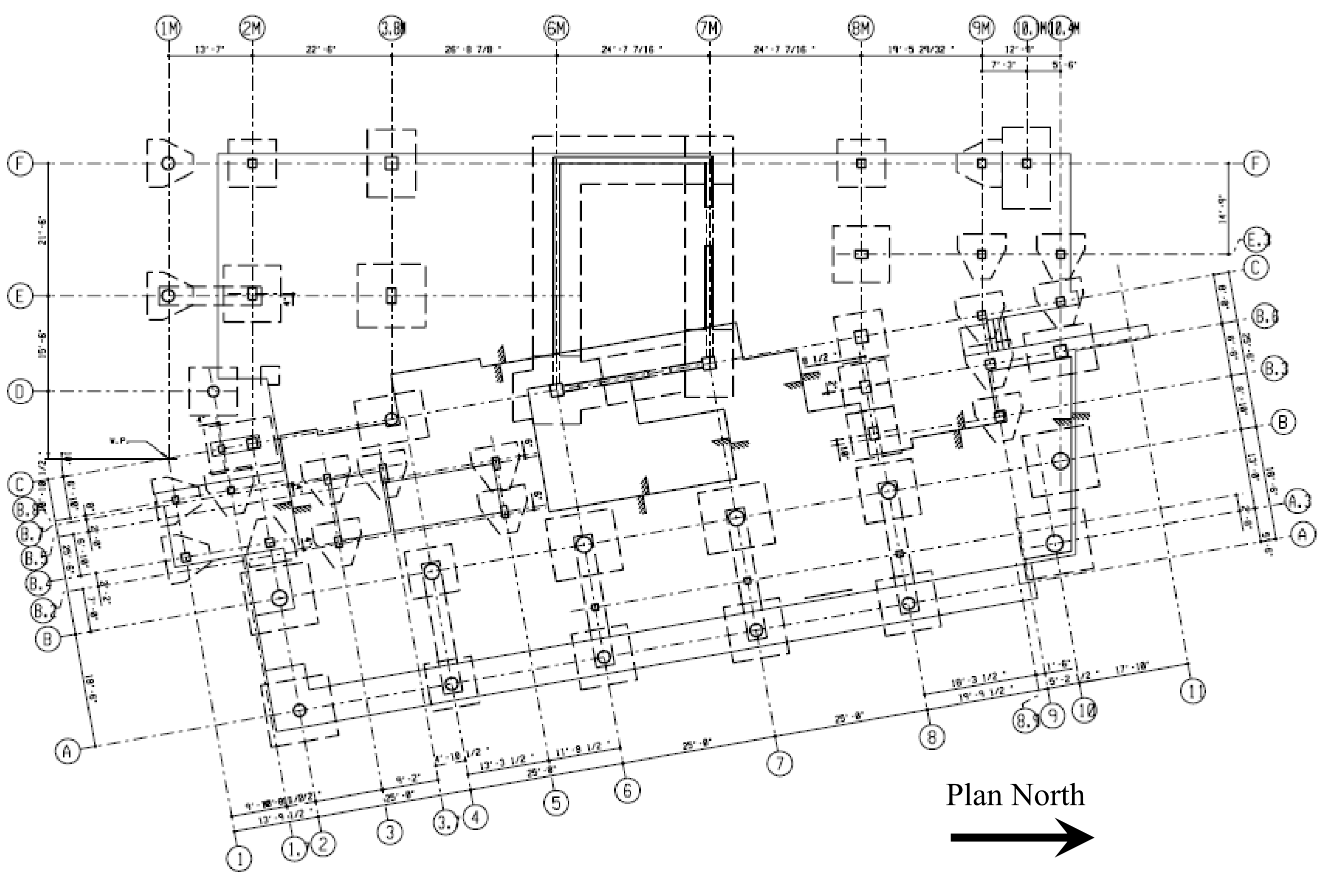
The Christina Landing Apartment Tower is a 22 story apartment building located just outside center city Wilmington, DE. The tower provides 250,000 square feet of floor space and its footprint covers approximately 12,000 square feet. The typical floor to floor height (floors 3-20) is 10 feet, while the common spaces on the first and second floors and the penthouses on the 21st and 22nd floor have 12 foot floor heights. The total building height is 230'. The structure is a predominately cast-in-place concrete building. Its floors are supported by a two way flat slab system. Spans between columns are on average approximately 20 to 25 feet. Negating the bays that contain slab openings the typical panel ratios range from 1:1 to 1:1.5 (see page 5 for framing plan). The typical floor system also incorporates small areas of reinforced concrete beams and post-tensioned beams in the plan-northeast and southeast corners to aid the lateral force resisting system. The floors are supported by square and round concrete columns. Column sizes for typical bays are 2' square or 2' round columns. For columns that surround slab openings and support smaller spans, sizes range down to 12"x12". Columns sizes seldom vary from floor to floor (see page 29 for column schedule). Lateral forces induced on the building are resisted by a box of four shear walls located in the center of the west wall. Because of the large torsional force created by this eccentricity of the center of rigidity the regions of post-tensioned framing are used to provide extra stiffness. All columns and shear walls rest on a foundation system of H-piles and pile caps. Concrete strengths differ throughout the structure, ranging from 4000psi to 8000psi (see page 4 for concrete strength schedule.)

The loads used for this design are as follows:

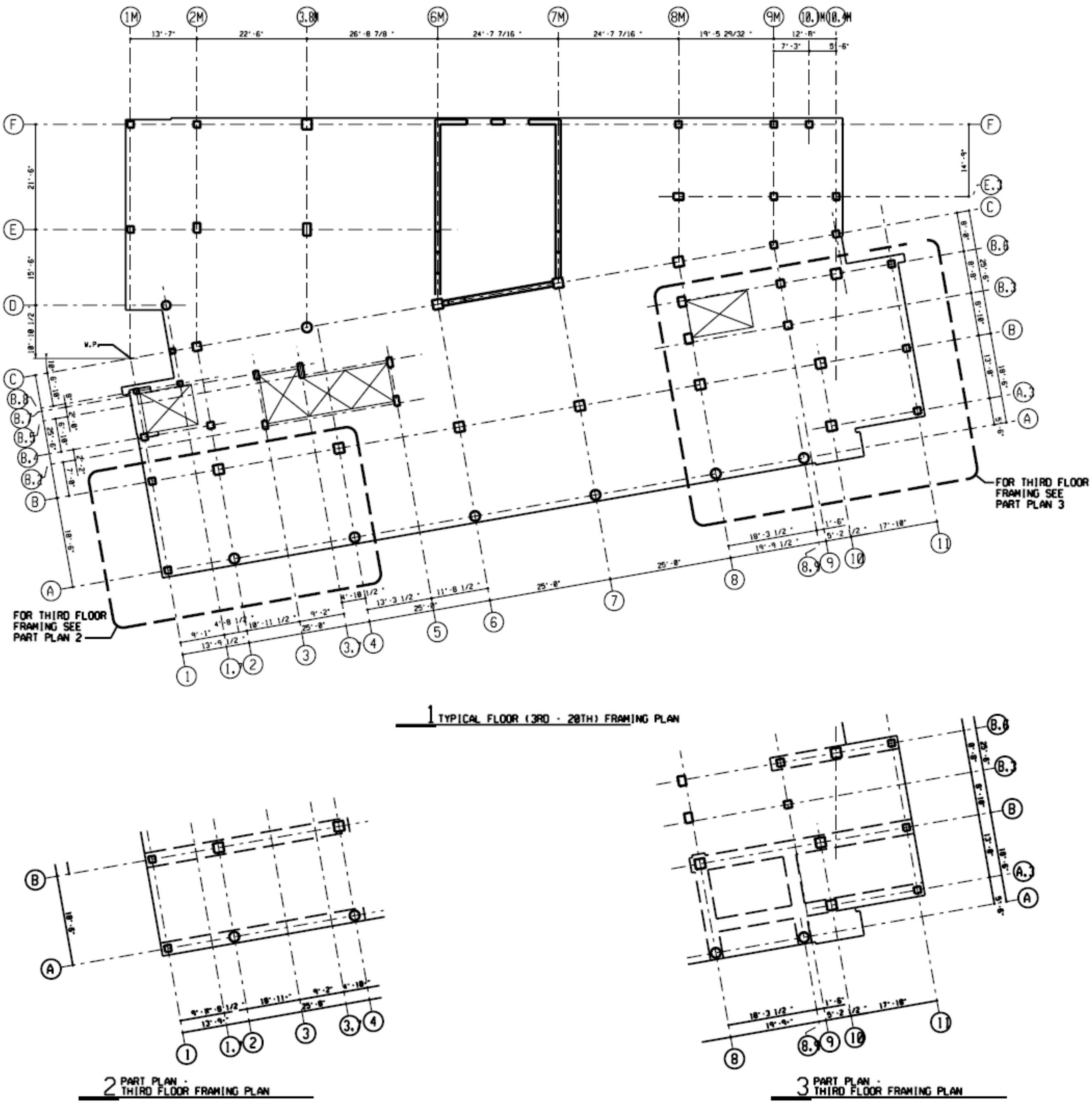
Self Weight Slab =	100psf
Partitions =	20psf
Miscellaneous Dead Load =	10psf
Live Load =	40psf

Concrete Strength Schedule	
Element	28 Day Cylinder Strength (psi)
Pile Caps	4,000
Slabs 5th Floor and Above	4,500
Slabs Below 5th Floor	5,600
Columns 5th Floor and Above	5,000
Columns Below 5th Floor	8,000
Exterior Slabs and Paving	5,000
Shear Walls	5,000
Topping Fills	4,000

1st Floor Framing Plan

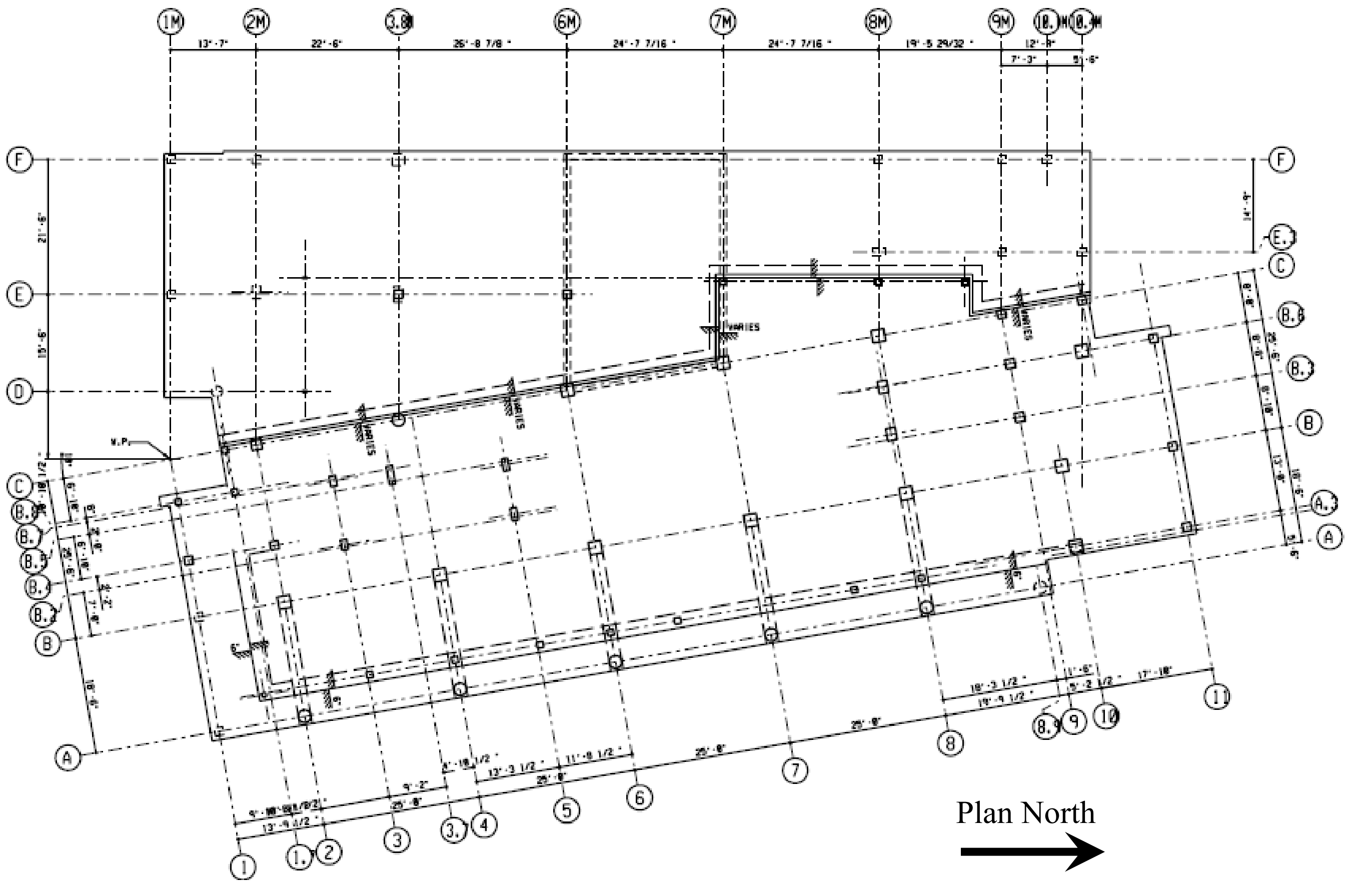


Typical Framing Plan



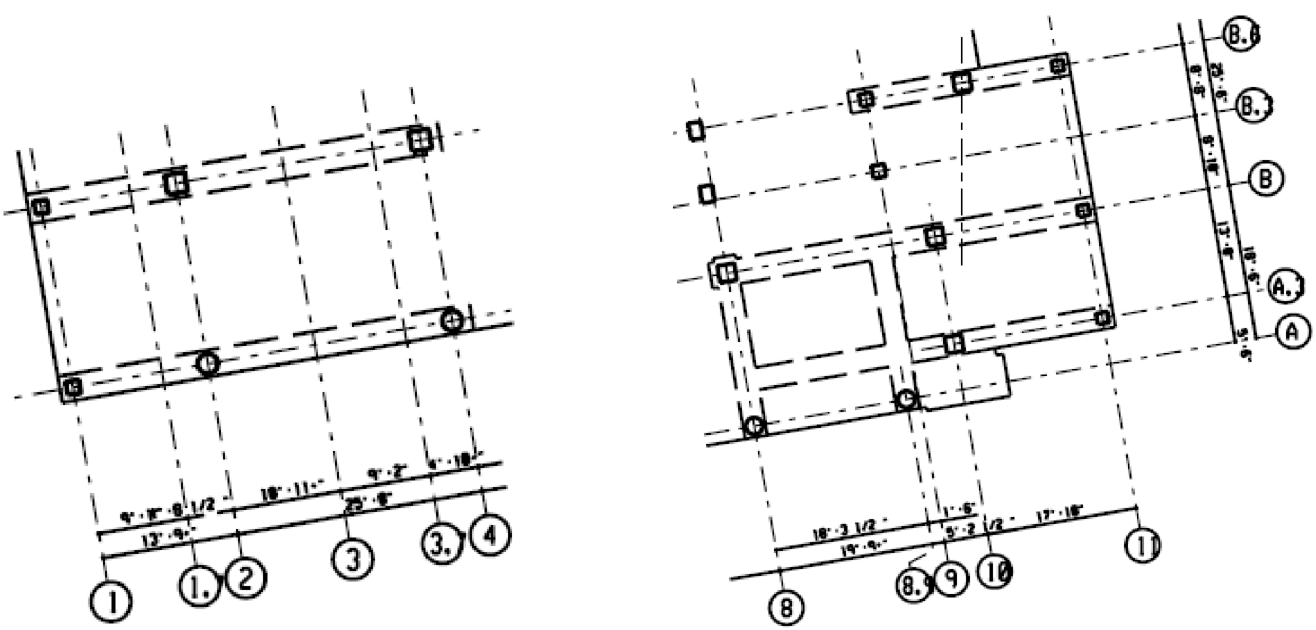
Plan North →

21st Floor Framing Plan



Existing Slab and Framing System

All the floors in the building have the same two way flat slab system, including the roof and the ground floor. Spans between columns are on average approximately 20 to 25 feet. Negating the bays that contain slab openings the typical panel ratios range from 1:1 to 1:1.5 (see page 4 for framing plan). The typical floor system also incorporates small areas of reinforced concrete beams and post-tensioned beams in the plan-northeast and southeast corners to aid the lateral force resisting system. The slab its self is an 8" slab with #6 bars at 10" on center, each way in the top and #4 bars at 10" on center, each way in the bottom. The strength of the concrete in the floor system is 5,600psi from the ground floor to the fifth floor and 4,500psi above the fifth floor. The post-tensioned members in the corners of the floors are 36"x 60" and vary in tensioning force. The members in the north-south direction have a higher force for the fact that they are longer spans and this is the principal direction in which the lateral force resisting system needs extra stiffness (see diagram below for clarification).



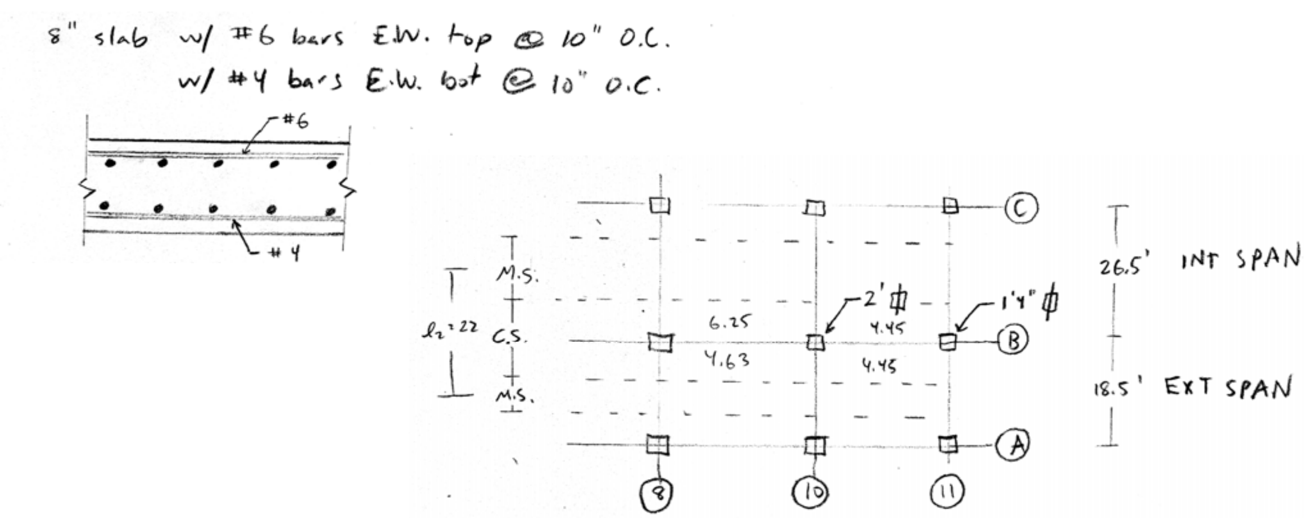
Plan North

Existing Conditions (Flat Slab)

The existing system is an 8" flat slab with with #6 bars at 10" on center, each way in the top and #4 bars at 10" on center, each way in the bottom. First I checked the minimum slab thickness according to ACI 318-05 and found that the minimum slab is $(l_n/33)$ or 8.3". I note this as an area of concern that needs further investigation. For the system check I analyzed the column strip and middle strip along column line B between column lines 8 and 10. This includes one interior and one exterior span (see diagram below). After finding all moments in these spans I determined the controlling positive and negative moments, of -14.3ft-k/ft and 6.2ft-k/ft respectively.

Results:
For the maximum negative moment of -14.3ft-k/ft I found #6 bars at 10" on center to be slightly insufficient yielding a ΦM_n of 13.7ft-k/ft, a difference of 0.6ft-k/ft. After equivalent frame analysis including all element stiffnesses the system will most likely yield a smaller maximum moment at the support making the 13.7ft-k/ft satisfactory. For the maximum positive moment of 6.2ft-k/ft I found that #4 bars at 10" on center were sufficient yielding a ΦM_n of 6.8ft-k/ft. See pages 18-20 in the appendix for further assumptions and calculations.

Conclusions:
This system was chosen by the engineers of record for good reason. It suits the existing layout quite well. However further investigation is needed into the minimum slab thickness requirement.



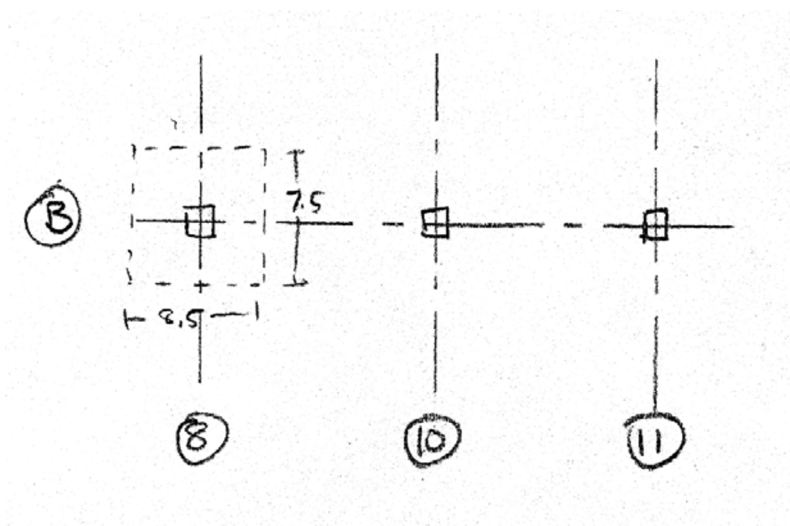
Flat Plat w/ Drop Panels

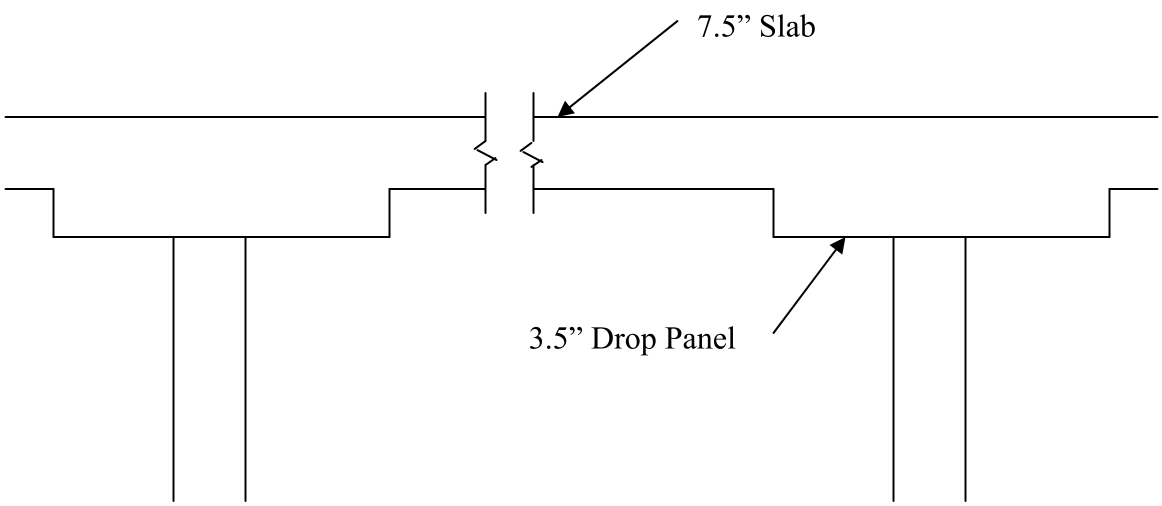
The first alternate system I explored was a flat plat slab similar to the existing conditions but incorporating drop panels at the columns (see following page for schematic). For the original design an 8” slab was used, however according to my calculations based on ACI 318-05 the minimum slab thickness that should have been used for an interior panel with out drop panels is (l_n/33) or 8.3”. While this value was above the minimum allowable thickness, the flexural strength was sufficient for the applied moments. I took these facts into consideration when designing the slab with drop panels. According to ACI 318-05 the minimum slab thickness with drop panels on an interior span is (l_n/36) or 7.67”. Instead of using an 8” slab and not altering the design I chose to try a 7.5” slab. For the controlling spans analyzed, the minimum drop panel depth was found to be 1.75”. I chose to use 3.5” drop panels. Using 3.5” drop panels aids in constructability because the laborers can simply use a 2x4 to frame the drop. The sizes of the drop panels would vary in this design depending on the spans surrounding the column. For the panel analyzed I found the size to be 7.5’x8.5’. The advantage of using a drop panel system as opposed to a flat slab is to save concrete by using a thinner slab while increasing the moment capacity at the supports. For my analysis I found that the concrete save was slightly greater than the extra concrete used for the drop panel, however I conservatively assumed the loads for the existing conditions to be the same for this system. See pages 21-22 in the appendix for further assumptions and calculations.

Results:
The drop panels system with a 7.5” slab and 3.5” drop panels was able to hold the applied moments. Reinforcing used was #5 bars at 9” on center each way in the top and #4 bars at 10” on center each way in the bottom.

Conclusions and Recommendation
The advantage of saving concrete using this method is very realistic, I found that approximately 8 cubic yards of concrete would be saved per bay. After totaling these savings over the whole building they would be significant. The drop panel’s ability to resist higher support moments also saves in the size and amount of reinforcing needed. The biggest disadvantage to this system however is the loss of some constructability. A purely flat slab requires much less formwork material, and labor. Without doing a full

estimation of cost it is hard to say but I would presume based on the cost of labor vs. material cost that the savings in concrete material would be insignificant when compared to the increase in labor. The lateral system would be stiffened by the addition of drop panels which would increase the stiffness of the joints. The equivalent frame method would be useful in determining how much extra stiffness could be expected. The foundation system would be virtually unaffected by the addition of drop panels.





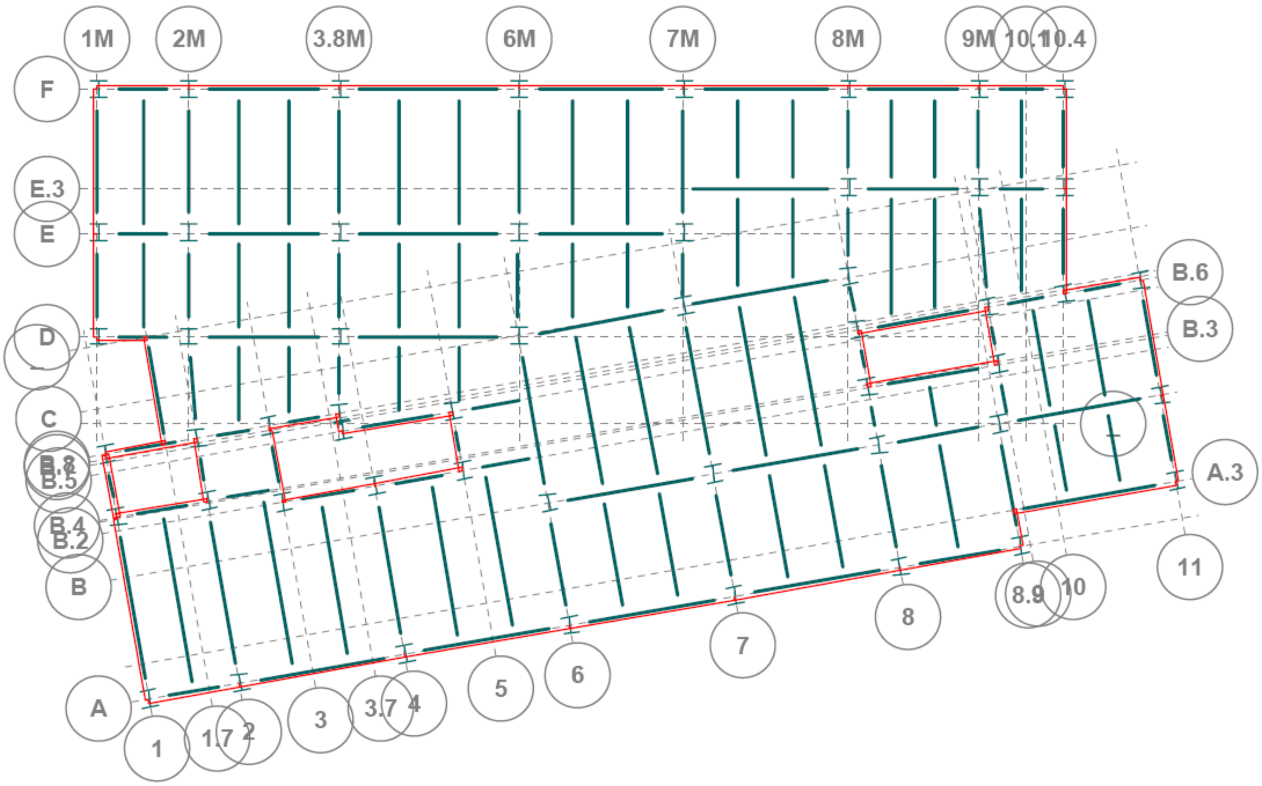
Steel with Composite Deck

For my second alternate system I used steel with composite deck. Using the existing architectural plans I laid out a steel frame for the building. I was able to get a plan which would maximize constructability for the building's unusual column layout without infringing on the existing architecture (see plan below). For my analysis I looked at the four bays enclosed by column lines 6, 8, A, and C (see schematic below). Using the loads given I used the Vulcraft deck manual and found that a 1.5" deep flute with 2.5" of concrete on top would be suitable for my spans. After deciding the decking I used RAM structural software to obtain an initial design (see design below). Using this design as a basis I checked the sizes of one beam and one girder with hand calculations. See pages 23-25 in the appendix for further assumptions and calculations.

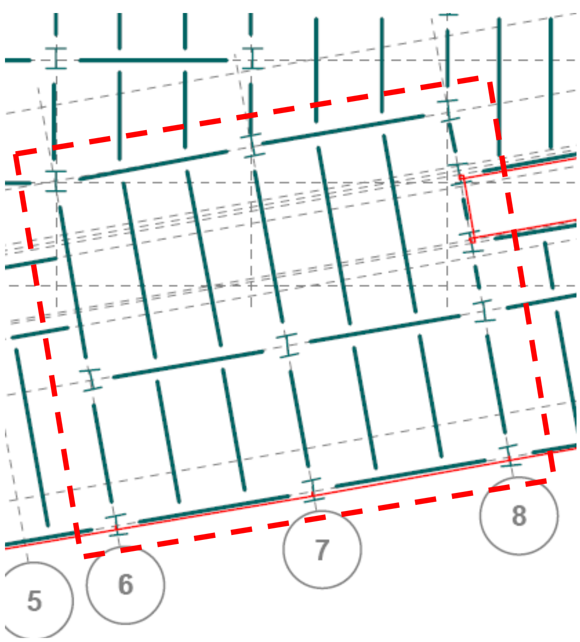
Results:
I found the W14x22 beam sufficient to carry the 112ft-k applied moment. It has a ΦM_n of 167ft-k with partially composite action using 8 shear studs, and 232ft-k assuming fully composite action. Therefore the RAM design is conservative. For the girder I found the W18x35 using 13 shear studs to be sufficient to carry the 243ft-k applied moment. I obtained a ΦM_n of 320ft-k with partially composite action using 14 shear studs. The difference between the number of shear studs is most likely because I rounded up before doubling the amount of studs needed in my calculations.

Conclusions:
This systems advantage is savings in cost of labor and speed of construction. Without having to lay all the reinforcing and pour all the structural members significant time and cost savings can be achieved. Another advantage is the building would be much lighter which could impact the foundation needed. In this system it may be possible to use spread footings as opposed to piles. This system also has several major disadvantages. First, due to the shape and layout of the floor plan it does not lend its self well to easy fabrication, and erection. Many of the members join columns and girders at angles making for tricky connections. This can add significant cost and construction time. Another major disadvantage is the increased sandwich depth due to the framing members. Existing floor to floor heights are 10' and the new structure would definitely have to increase this in order to work. This would result in an overall increase in building height which would

increase wind load on the building and potentially exceed height limitations. For this system the lateral system would most be changed to some kind of braced frame or moment frame.



Framing Plan



Bays Analyzed

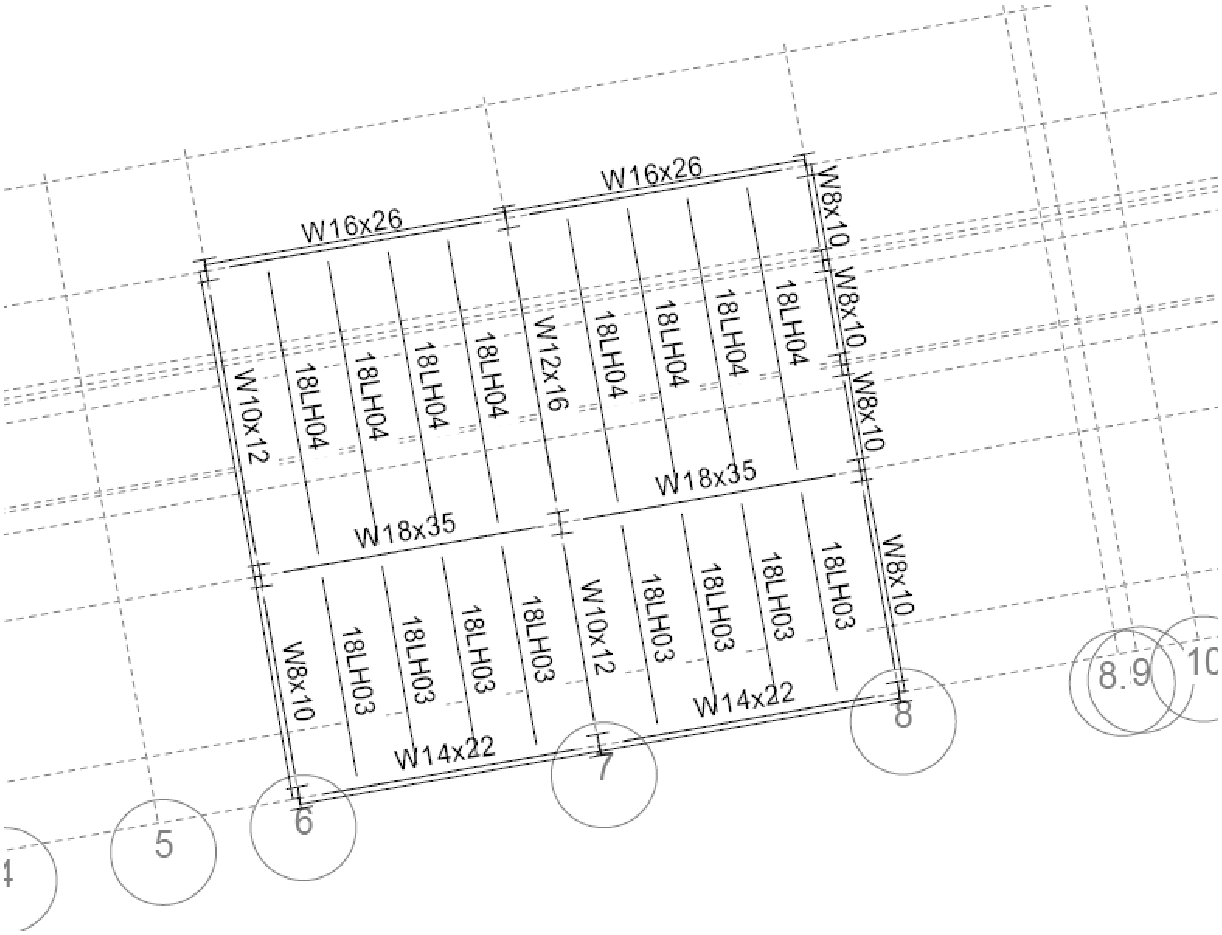
Beam and Girder Spot Checked

Steel Joist with Metal Deck

For my third alternate system I used steel joists with metal deck. For the 25' span I spaced the joist at 5' on center. Using the Vulcraft steel deck manual I found that a 1.5" flute with 2" of concrete on top was acceptable for the spans analyzed. Using this information I plugged the loads in RAM structural system and analyzed the joists (see RAM output below). I then checked the RAM output using the New Columbia Joist manual to see if the joists designed were acceptable to support the applied loads.

Results:
All joists designed were acceptable to hold the applied loads according to the New Columbia Joist manual.

Conclusions:
This systems advantage is savings in cost of labor and speed of construction. Without having to lay all the reinforcing and pour all the structural members significant time and cost savings can be achieved. Another advantage is the building would be much lighter which could impact the foundation needed. In this system it may be possible to use spread footings as opposed to piles. This system also has several major disadvantages. First, due to the shape and layout of the floor plan it does not lend its self well to easy fabrication, and erection. Many of the members join columns and girders at angles making for tricky connections. This can add significant cost and construction time. Another major disadvantage is the increased sandwich depth due to the framing members. Existing floor to floor heights are 10' and the new structure would definitely have to increase this in order to work. This would result in an overall increase in building height which would increase wind load on the building and potentially exceed height limitations. For this system the lateral system would most be changed to some kind of braced frame or moment frame.



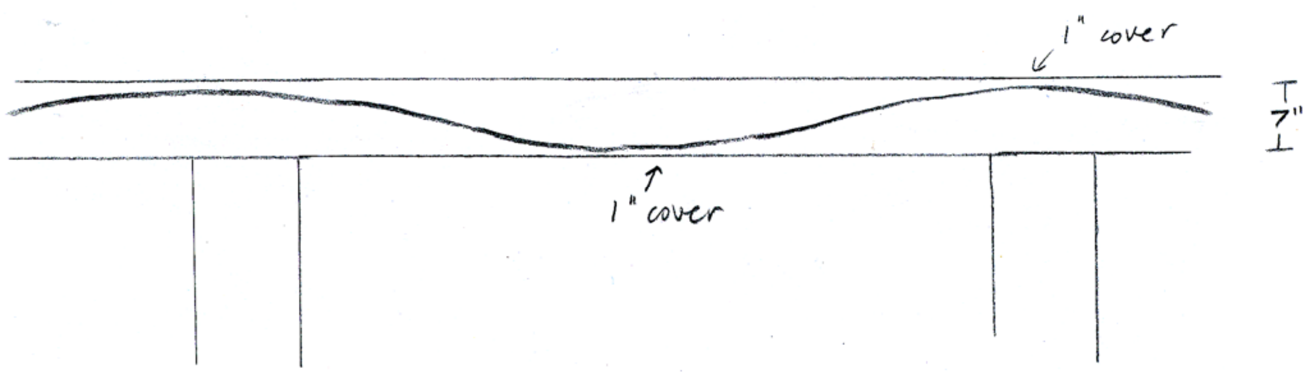
<u>Ram Output</u>

Post-Tensioned Slab

For my last system I explored a post-tensioned slab. Using a span/depth ratio of 45 I found the minimum slab depth to be approximately 7". The minimum pre-stressing force was found to be 82k per foot, using maximum tension of an uncracked concrete section as a parameter. This also assumed 1" of concrete cover for the tendons. In order to achieve 82k of force per foot two and a half .6" diameter bars would be needed.

Results:
The system outlined was found sufficient to carry the applied moments across the floor system. See pages 26-28 of the appendix for further assumptions and calculations.

Conclusions:
The primary advantage of this system is the savings associated with the cost of concrete material. 1" of concrete is save over the area of the entire floor, when totaled over the entire building this is approximately 750 cubic yards of concrete. Assuming the cost of concrete is 100 dollars per cubic yard this yields a material savings of $75,000. The disadvantages to this system are the additional care that needs to be taken during the construction process to set and stress the tendons, as well as the extra equipment that is needed. The equipment needed for stressing the tendons will already have some presence on site because the building contains areas of post-tensioned beams, however more jacks will be needed if the floor systems are entirely post-tenstioned.



Comparison Chart

System	Depth	Weight	Advantages	Disadvantages
Existing Flat Slab	8”	100	Ease of construction Low Labor Cost	High Labor Cost
Flat Plate with Drop Panels	7.5” w/ 3.5” drops	98	Less Material Cost	Higher Labor Cost
Steel with Composite Deck	24”	52	Lighter System Faster Construction Less Labor Cost	Unusual Layout Difficult to Construct and Fabricate Higher Structure
Joists with Metal Deck	24”	??	Lighter System Faster Construction Less Labor Cost	Unusual Layout Difficult to Construct and Fabricate Higher Structure
Post-Tensioned Slab	7”	88	Less Material	More Involved Construction Process

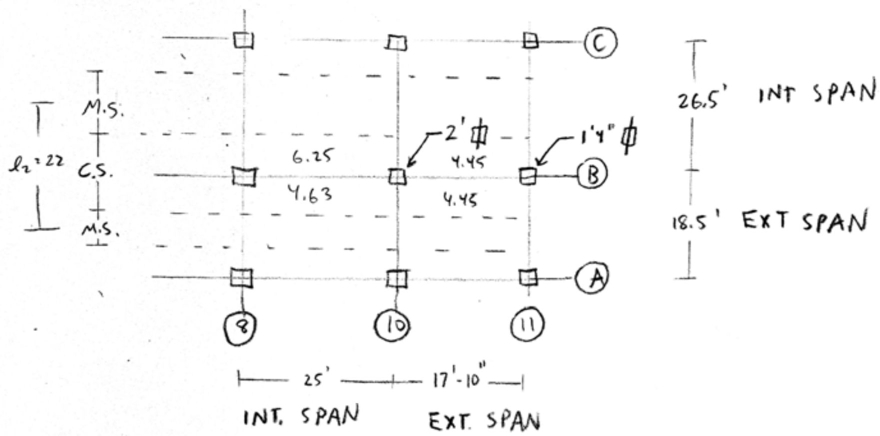
Conclusion

After comparison of the four alternate systems chosen two seem viable alternates while two systems should be ruled out as potential redesigns. In both steel with composite deck and joist with metal deck the disadvantages outweigh the advantages. For the building’s column layout this system isn’t practical. Typically steel framed building have longer spans and a more regular bay size for ease of construction and fabrication. Also the floor to floor height would need to be increased for these systems. Both flat plate with drop panels, and post-tensioned slab systems require further investigation. For existing flat slab the concrete saved is significant when totaled over the building; however the additional labor involved will increase the cost. This alternative will require a further investigation into material cost vs. labor cost. The most likely system to be used is a post-tensioned slab. In this alternate 1” of concrete is saved over the whole floor area, which when summed over the entire building is a huge amount of material savings. The downside to this process is the extra amount of work and care put into the construction process. For this building it does not seem to much of a stretch to use post-tensioned slabs because the building contains areas of post-tensioned beams so the equipment is in use on site anyway.

Appendix

EXISTING CONDITIONS DIRECT DESIGN METHOD

8" slab w/ #6 bars E.W. top @ 10" O.C.
w/ #4 bars E.W. bot @ 10" O.C.



LOADING

S.W. SLAB $= 150\left(\frac{8}{12}\right) = 100$ psf

MISC DL = 10 psf

PARTITION = 20 psf

LIVE LOAD = 40 psf

$w_u = 1.2(130) + 1.6(40) = 220$ psf

$$M_o = \frac{.22(22)(23)^2}{8} = 320.0 \text{ ft}\cdot\text{k} \quad \underline{\text{INT SPAN}}$$

$$M_o = \frac{.22(22)(16.167)^2}{8} = 158.1 \text{ ft}\cdot\text{k} \quad \underline{\text{EXT SPAN}}$$

INT SPAN	support 65%	C.S. 75%	156	10.9	14.3
	208 ft·k	M.S. 25%	52	11.1	4.7
	midspan 35%	C.S. 60%	67.2	10.9	6.2
	112 ft·k	M.S. 40%	44.8	11.1	4.0
EXT SPAN	int support 70%	C.S. 75%	83.0	8.9	9.3
	110.7 ft·k	M.S. 25%	27.7	13.1	2.1
	ext support 26%	C.S. 100%	47.4	8.9	5.3
	47.4 ft·k	M.S. 0%	0	13.1	0
	midspan 52%	C.S. 60%	47.5	8.9	5.3
	79.1 ft·k	M.S. 40%	31.6	13.1	2.4

INT SPAN SUPPORT C.S. controls for neg. moment.

1 #6 $A_s = .44$

A_s per foot $= .44\left(\frac{12}{10}\right) = .528$

$d = 8 - .75 - .75 - \frac{.75}{2} = 6.125$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.528(60)}{.85(4.5)(12)} = .690$$

$$M_n = .528(60)\left(6.125 - \frac{.69}{2}\right) = 15.3 \text{ ft·k}$$

$$\phi M_n = .9(15.3) = 13.7 \text{ ft·k} \leq 14.3 \text{ ft·k}$$ NO GOOD

BUT CLOSE

after equivalent frame analysis and including all element stiffnesses moment will most likely decrease making this 13.7 ft·k satisfactory.

INT. SPAN MIDSPAN C.S. controls for pos. moment

1 #4 $A_s = .2$

A_s per foot = $.2\left(\frac{12}{10}\right) = .24 \text{ in}^2$

$d = 8.0 - .75 - .5 - \frac{.5}{2} = 6.5 \text{ in}$

$a = \frac{.24(60)}{.85(4.5)(12)} = .314$

$M_n = .24(60)\left(6.5 - \frac{.314}{2}\right) = 91.3 \text{ in}\cdot k = 7.6 \text{ ft}\cdot k$

$\phi M_n = .9(7.6) = 6.8 \text{ ft}\cdot k \geq 6.2 \text{ ft}\cdot k$ OK

FLAT PLATE w/ DROP PANELS

Table 9.5(c)

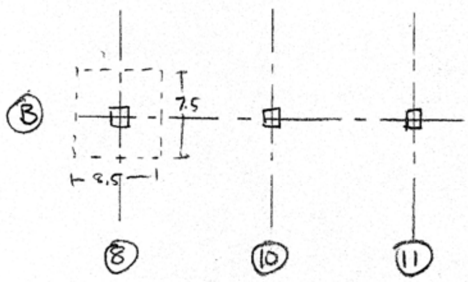
fy: 60 ksi

int panels w/o drop panels

$\ell_n/33 = \frac{(25-2)(12)}{33} = 8.3''$ actual design uses 8" slab further investigation needed

int panels w/ drop panels

$\ell_n/36 = \frac{(25-2)(12)}{36} = 7.67''$ by inspection of existing systems relationship to min slab thickness try 7.5" slab.



min drop panel thickness $= \frac{1}{4} d_s$

$= \frac{1}{4}(7) = 1.75$

use d.p. depth = 3.5" for ease of construction

min d.p. size $= \frac{1}{6}$ span each side

E.X. for col. B-8

$\frac{1}{6}(25) + \frac{1}{6}(25)$ by $\frac{1}{6}(26.5) + \frac{1}{6}(18.5)$

8.33' by 7.5'

use 8.5' x 7.5'

LOAD DIFFERENCES B/W EXISTING AND W/ DROP PANELS

1 DROP PANEL 7.5(8.5)(144)(3.5) = 33415

19.3 cy conc.

.5" conc over 1 panel

.5(25)(26.5)(144) = 47700

27.6 cy conc.

because most panels are smaller than this assume loads are approx equal.

use loads and moments from existing conditions

$\phi M_n = 14.3$

$M_n = \frac{14.3}{.9} = 15.9$

find min A_s to yield 15.9

try $A_s = .2$

$a = \frac{.2(60)}{.85(4.5)(12)} = .261$

$d = 7 + 3.5 - .75 - .5 - \frac{.5}{2} = 9"$

$M_n = .2(60)\left(9 - \frac{.261}{2}\right) = 106.4 \text{ in}\cdot\text{k} = 8.9 \text{ ft}\cdot\text{k}$

try #5 bars @ 9" O.C.

$A_s = .41$

$a = \frac{.41(60)}{.85(4.5)(12)} = .536$

$d = 10.5 - \frac{5}{8} - .75 - \frac{5/8}{2} = 8.81$

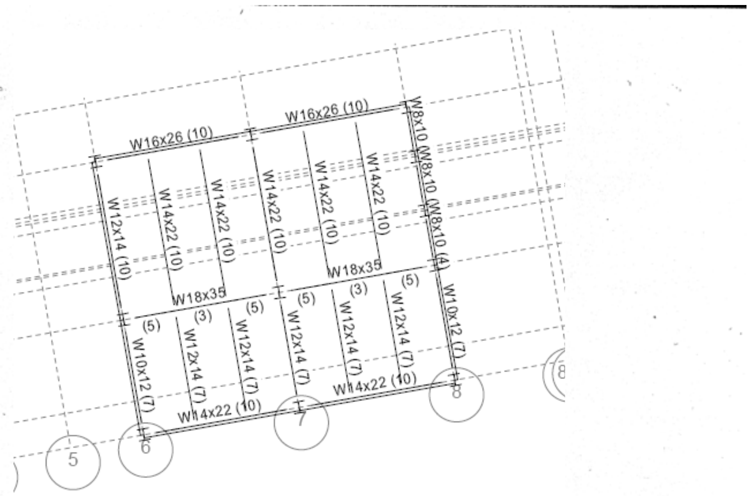
$M_n = .41(60)\left(8.81 - \frac{.536}{2}\right) = 210.1 \text{ in}\cdot\text{k} = 17.5 \text{ ft}\cdot\text{k}$

$17.5 \geq 15.9 \text{ ft}\cdot\text{k}$ ok

for bottom bars use same as existing #4 bars @ 10" O.C.

STEEL W/ COMPOSITE DECK

RAM OUTPUT →



check W14×22 beam 26'6" long w/ 10 shear studs

floor loads

decking 44 psf

live load 40 psf

misc. D.L. 10 psf

partition 20 psf

use Vulcraft 1.5VLR

1.5" flute
2.5" conc. > 4" total depth

$w_u = 1.2\,(74) + 1.6\,(40) = .153$ ksf

trib width = 8.33'

Span = 26.5'

$M = \dfrac{.153\,(8.33)(26.5)^2}{8} = 111.9$ ft·k

$b_{eff} = \min \begin{cases} 8.33(12) = 100 \\ \dfrac{26.5\,(12)}{4} = 80 \leftarrow \text{controls} \end{cases}$

W14×22

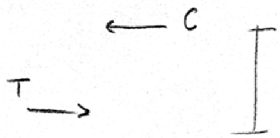
A = 6.49 t_w = .23 b_f = 5 d = 13.7 t_f = .335

$C_{conc.} = .85\,(3)(80)(2.5) = 510$ k

$T_{stl} = 6.49\,(50) = 324.5$ k

T_{SH} controls

$C_c = 324.5$



$4 + \frac{13.7}{2} - \frac{1.59}{2} = 10.1$

$324.5 = .85(80)(3)(a)$

$a = 1.59$

$\phi M_n = .85(324.5)(10.1) = 2786 \text{ in}\cdot\text{k} = 232 \text{ ft}\cdot\text{k} \geq 111.9 \text{ ft}\cdot\text{k}$

(assumes fully composit

Account for shear studs (not fully composite)

assume $a = 1.0$

$Y2 = 4 - .5 = 3.5$

try $\Sigma Q_n = 81.1$

$\frac{81.1}{.85(3)(80)} = .40 \leq$ assumed (a) OK

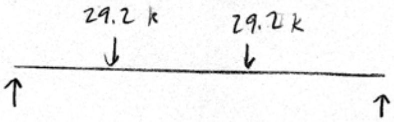
$\frac{81.1}{21} = 3.9$ use 4 per side = 8 shear studs

$\phi M_n = 167 \text{ ft}\cdot\text{k} > 112 \text{ ft}\cdot\text{k}$ OK

I found 8 shear studs, RAM uses 10 OK

STEEL W/ COMPOSITE DECK

Girder check



D.L. $= .153(8.33)\left[\left(\frac{26.5}{2}\right)+\left(\frac{18.5}{2}\right)\right] = 28.7$

S.W. beams $= .026\left(\frac{26.5}{2}\right) + .016\left(\frac{18.5}{2}\right) = .4$

TOTAL point loads $= 29.2$

$M = 29.2\,(8.33) = 243.2$ ft-k

check W18x35 w/ 13 shear studs

$b_{eff} = \min \left| \begin{array}{l} 18.5(12) = 222 \\ \frac{25(12)}{4} = 75 \leftarrow \text{controls} \end{array} \right.$

$\phi_b M_p = 249$

try $a = 1''$ $\quad Y2 = 3.5$

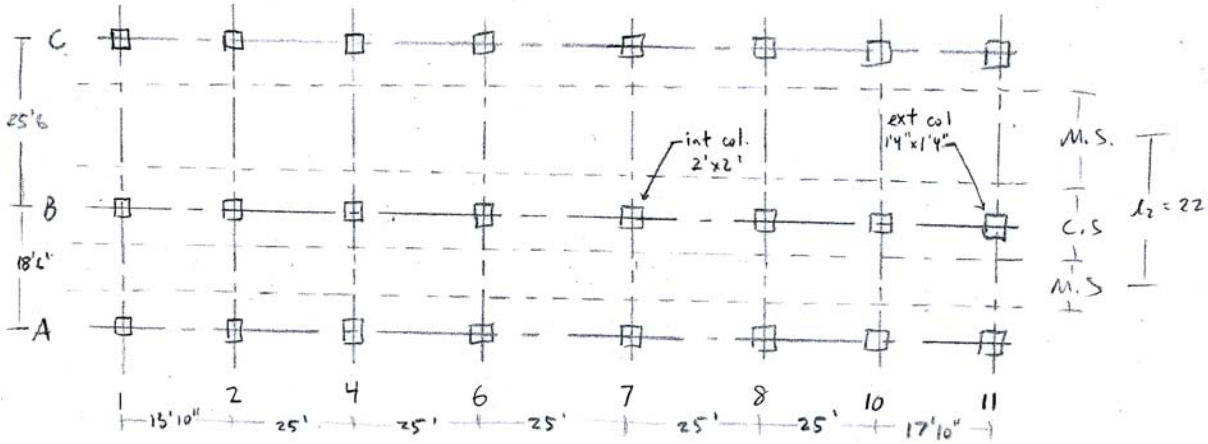
$\Sigma Q_n = 129 \qquad a = \frac{129}{.85(3)(75)} = .67 < 1''$ OK $\qquad \phi M_n = 320 > 243$ ft-k OK

$\frac{129}{21} = 6.14 \Rightarrow$ use 7 both sides $= 14$ shear studs

RAM uses 13 difference negligable OK

PRESTRESS ANALYSIS

TO SIMPLIFY ANALYSIS ASSUME FOLLOWING LAYOUT
(FOR ACTUAL LAYOUT SEE PLAN)



FIND MAX POS. AND NEG. MOMENTS IN FRAME B

FOR CONTINUOUS FLOOR APPOX 2-WAY FLAT SLAB
SPAN / DEPTH RATIO = 45

$45 = \frac{25(12)}{d}$ Min $d = 6.67$ in

TRY 7" SLAB

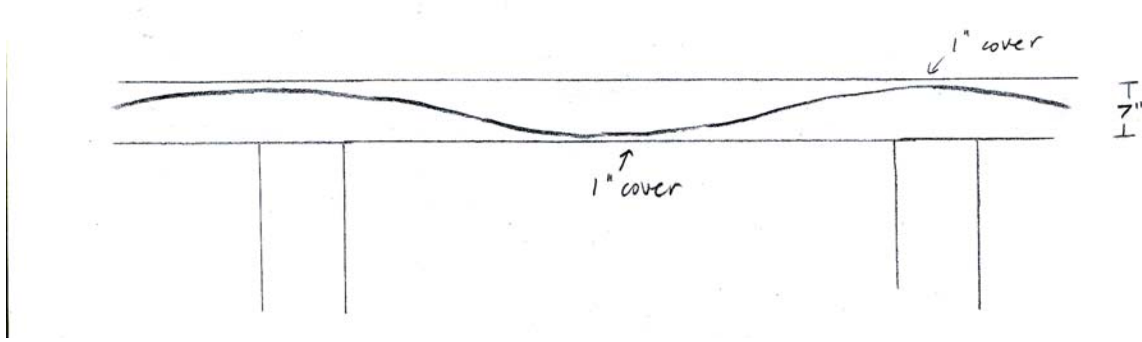
DEAD LOAD = $150\left(\frac{7}{12}\right) = 87.5$ psf

MISC DL = 25 psf

L.L. TYP. = 40 psf

TOTAL LOAD = 87.5 + 25 + 40 = 162.5 psf

- 13.7.6.2 use full load on all spans. (loading pattern not known)
- use 1" cover for prestressing steel



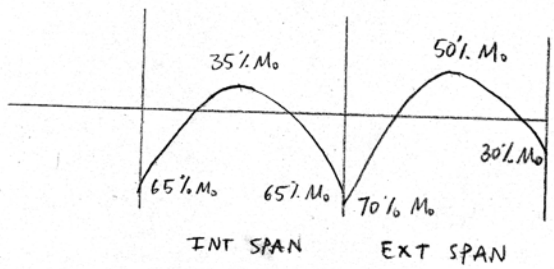
MOMENT using Direct Design Method

int. span

$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{.1625\,(22)(23)^2}{8} = 236.4 \text{ ft}\cdot\text{k}$$

larger of 2 exterior spans

$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{.1625\,(22)(17.833 - 1 - .667)^2}{8} = 116.8 \text{ ft}\cdot\text{k}$$



INT SPAN

support moment $.65(236.4) = -153.7$ ft·k ← controlling neg. moment

midspan moment $.35(236.4) = 82.7$ ft·k ← controlling pos moment

EXT SPAN

ext support moment $.3(116.8) = -35.0$ ft·k

int support moment $.7(116.8) = -81.8$ ft·k

midspan moment $.5(116.8) = 58.4$ ft·k

use class U post-tensioned system

$f_{max\,t} = 7.5\sqrt{f'c} = 7.5\sqrt{4000} = 474.3$ psi

$f_{max\,c} = .6\,f'c = .6(4000) = 2400$ psi

$$f_{top} = -\frac{M}{S} - \frac{P}{A} + \frac{Pe}{S}$$

$$f_{bot} = \frac{M}{S} - \frac{P}{A} - \frac{Pe}{S}$$

$$S = \frac{bd^2}{6} = \frac{22(12)(7)^2}{6} = 2156 \text{ in}^3$$

$A = 22(12)(7) = 1848 \text{ in}^2$

$e = \frac{7}{2} - 1'' = 2.5$

cover

Moments to use for determining the prestressing force

$.9(87.5\,psf) = 78.8\,psf$

$$M_o = \frac{.0788(22)(23)^2}{8} = 114.6\ ft\cdot k$$

$$P_{top} = \left(f_{top} + \frac{M}{S}\right)\left(-\frac{1}{A} + \frac{e}{S}\right)^{-1} = \left(.474 + \frac{114.6(12)}{2156}\right)\left(-\frac{1}{1848} + \frac{2.5}{2156}\right)^{-1} = 1797.9$$

$$P_{bot} = \left(f_{bot} - \frac{M}{S}\right)\left(-\frac{1}{A} - \frac{e}{S}\right)^{-1} = \left(-2.4 + \frac{114.6(12)}{2156}\right)\left(-\frac{1}{1848} - \frac{2.5}{2156}\right)^{-1} = 1036$$

max force per tendon

.6" dia bar

$270\,ksi\,(.6)(.216) = 35\,k$

losses

Total number of tendons needed

$$\frac{1798}{35} = 51.4$$

tendon spacing

$$\frac{22'(12)}{51.4} = 5.14'' \Rightarrow \text{space tendons } 5''\ O.C.$$

Ultimate strength

controlling pos moment $= 82.7\ ft\cdot k$ or $3.76\ ft\cdot k$ per foot

controlling neg. moment $= -153.7\ ft\cdot k$ or $6.99\ ft\cdot k$ per foot

check w/ only area of prestressing tendons

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.216(2.4)(270)}{.85(4.5)(12)} = 3.05$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) = .216(2.4)(270)\left(5.7 - \frac{3.05}{2}\right) = 584.4\ ft\cdot k/ft$$

COLUMN SCHEDULE

COLUMN MARK / ELEVATION	A-1, B-1	A-2	A-4,6,7,8	A-8,9	A.3-3,4,5, 6,6.5,7.5, 8, E.1-7M	A.3-6,7,8	A.3-10	A.3-11, B-11, B.6-11	B-2	B-4,6, 7,8	B-10	B-10.4M	B.2-3, B.7-3	B.2-5, B.5-5,	B.3-8, B.6-8	B.3-9, B.6-9	B.4-1.2, C-9M,10.4M	B.5&7-3.7	B.6-10.4M C-8M, F-3.8M	B.8-1,1.7, C-1.7	C-2M	C-3.8M	C-6M	C-7M	D-1.7	E-1M, F-1M	E-2M, E.3-8M	E-3.8M	E.3-9M, 10.4M, F-9M,10.1M	F-2M,8M
UPPER ROOF																														
T/SLAB EL. - SEE PLAN 22nd FLOOR		24 DIA. 8 - #10	24 DIA. 6 - #10		12x12 4 - #8		24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 6 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 B-6.9 ONLY 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11							
T/SLAB EL. - SEE PLAN 21st FLOOR & LOW ROOF		24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 6 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11					18x30 16 - #10		
T/SLAB EL. - SEE PLAN 20th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 6 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 18th & 19th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 6 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 16th & 17th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 6 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 14th & 15th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 6 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 12th & 13th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 10 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 10th & 11th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 6 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 14 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 8th & 9th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 12 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 18 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 6 - #9	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 6th & 7th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 12 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 18 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 10 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 12 - #10	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 5th FLOOR	16x16 4 - #8	24 DIA. 8 - #10	24 DIA. 12 - #10	24 DIA. 6 - #8			24x24 8 - #8	16x16 4 - #8	24x24 8 - #11	24x24 18 - #11	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 10 - #10	12x36 6 - #8	24x24 6 - #8	12x12 6 - #10	20x20 4 - #9	24 DIA. 12 - #10	24x24 12 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 8 - #10	16x24 8 - #10	18x30 16 - #10	16x16 4 - #8	20x20 6 - #9
T/SLAB EL. - SEE PLAN 4th FLOOR	16x16 8 - #8	24 DIA. 14 - #11	24 DIA. 6 - #10	24 DIA. 12 - #11			24x24 10 - #11	16x16 8 - #8	24x24 20 - #11	24x24 6 - #10	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 4 - #9	20x20 4 - #9	24 DIA. 8 - # 10	24x24 18 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 4 - #8	16x24 8 - #10	18x30 14 - #10	16x16 4 - #8	20x20 4 - #9
T/SLAB EL. - SEE PLAN 3rd FLOOR	16x16 8 - #8	24 DIA. 14 - #11	24 DIA. 6 - #10	24 DIA. 12 - #11			24x24 10 - #11	16x16 8 - #8	24x24 20 - #11	24x24 6 - #10	24x24 10 - #10		12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 4 - #9	20x20 4 - #9	24 DIA. 8 - # 10	24x24 18 - #11	24x24 12 - #11	21 DIA. 6 - #8	16x16 4 - #8	16x24 8 - #10	18x30 14 - #10	16x16 4 - #8	20x20 4 - #9
T/SLAB EL. - SEE PLAN 2nd FLOOR			24 DIA. 6 - #10				32 DIA. 12 - #10					32 DIA. 8 - #10	12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #8	12x12 4 - #9	20x20 4 - #9	24 DIA. 8 - # 10	24x24 18 - #11	24x24 12 - #11	21 DIA. 6 - #8	24 DIA. 6 - #8	16x24 8 - #10	18x30 14 - #10	16x16 4 - #8	20x20 4 - #9
T/SLAB EL. - SEE PLAN 1st FLOOR		24 DIA. 14 - #11	24 DIA. 6 - #10			12x12 4 - #8	32 DIA. 12 - #10		30 DIA. 8 - #10	30 DIA. 8 - #10		32 DIA. 8 - #10	12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #9	12x12 4 - #9	20x20 4 - #9	24 DIA. 8 - # 10	24x24 18 - #11	24x24 12 - #11	21 DIA. 6 - #8	24 DIA. 6 - #8	16x24 8 - #10	18x30 14 - #10	16x16 4 - #8	20x20 4 - #9
T/SLAB EL. - SEE PLAN T/PILECAP OR GRADE BEAM			24 DIA. 6 - #10			12x12 4 - #8	32 DIA. 12 - #10					32 DIA. 8 - #10	12x20 4 - #8	12x24 4 - #8	24x18 6 - #9	16x16 4 - #8	16x16 4 - #10	12x36 6 - #8	24x24 6 - #9	12x12 4 - #9	20x20 4 - #9	24 DIA. 8 - # 10	24x24 18 - #11	24x24 12 - #11	21 DIA. 6 - #8	24 DIA. 6 - #8	16x24 8 - #10	18x30 14 - #10	16x16 4 - #8	20x20 4 - #9

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