Executive Summary

This report investigates the effectiveness of using an alternative floor framing system than the existing. The existing framing system was designed and compared to four additional framing systems for cost, total thickness and constructability. The existing floor system consists of an 8” flat slab with 8” drop panels at column locations. The four alternative systems are two variations of concrete and two of steel and are noted in the report respectively, concrete flat plate, post tensioning, composite steel beam, and open web steel joists.

The two steel systems were analyzed using RAM structural analysis program and the steel composite was also checked by hand calculations displayed in Appendix D. The deck and load calculations of the steel joists are called out in Appendix E. As for the two concrete systems, the flat plate system examines the constructability compared to the how much more concrete this system will require over the flat slab with drop panels. The post tension is an alternative attempting to achieve a system considerably less thick than the existing. The hand calculations for the flat plate and post tension are in Appendix B and C respectively.

The findings through this intense analysis show two systems to be more dominate over the others. The concrete post tensioning and steel composite beams both achieve two goals, thinner slab and constructability, and warrant the continued research to see which system will be most rewarding in the end. The post tensioning was designed to be a 7 ½” slab but through more research and a better understanding of the system, a thinner slab can be achieved. The composite beam is less labor intensive and lowers the dead load self weight approximately 40 psf less than the post tensioning.

Design Introduction

The Executive Tower consists of normal office loads for the DC area codes and will be designed according to these criteria. Normal office live and dead loads will apply to existing and alternative design systems. The height restriction is enforced for the Executive Tower will be taken under consideration. The Maximum height is 130 feet, Executive Tower tops out at 129'-6". This leaves no room for the possibility of making floor heights taller, so each floor will be designed to fit a 2'-6" floor thickness or lesser criteria. The bays being examined for this study is incase by column lines G, F, 7, and 8 (Bay 2) over looking New York Ave and a second, just for analyzing the steel alternative systems, at columns lines A, B, 5, and 4 (Bay 1)representing the northwest corner adjacent H St and the existing church. Both are represented below in Figures 2.1 and 2.2.

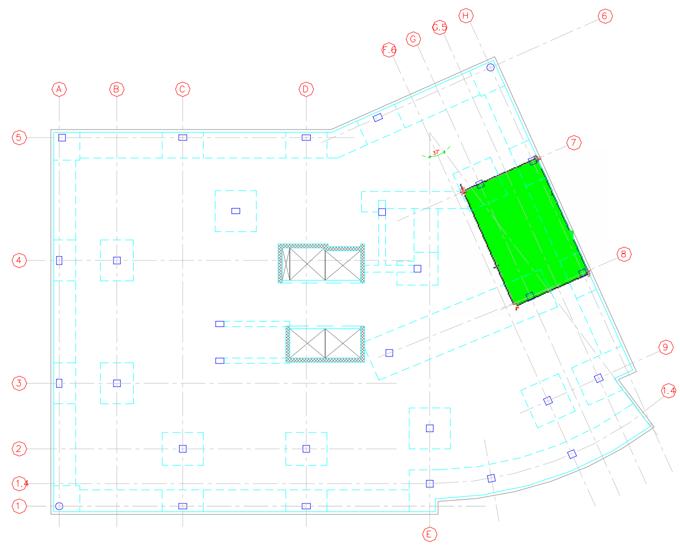


Figure 2.1 - shows Bay 2 that is being used to analysis the existing system, the 2-Way Flat Plate and the 2-Way Post Tension. Bay size is 30'x14'

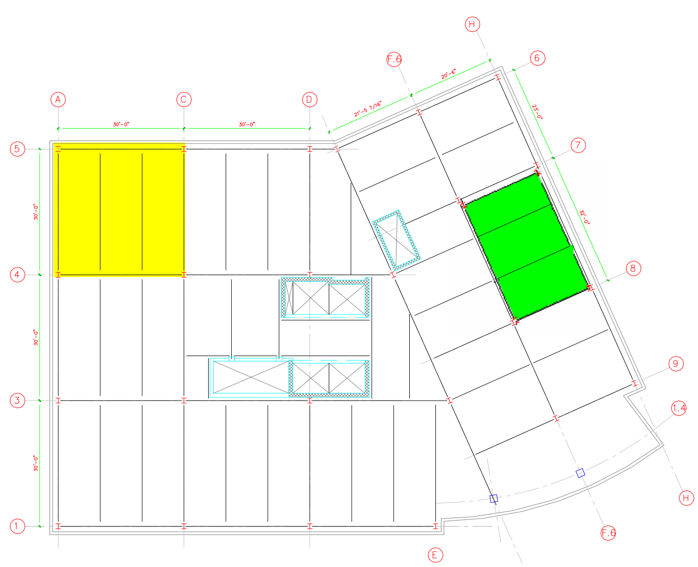


Figure 2.2 – shows Bays 1 & 2 that are being used to analysis the Steel Composite and Open Web Steel Joists alternative systems. Bay 1 (yellow) is 30'x30' and Bay 2 (green) is 32'x20'-6". The beam spacing differs from the two systems and is explained in the respective reports.

Loads

Live Loads

Listed are all the Loads required in the designing of Executive Tower in accordance with ASCE7-02. However, for this study of the designed bay will consists of only office space.

• **Office + partitions**	**80 + 20 = 100 psf**
• Lobby	100 psf
• Mechanical	150 psf
• terrace (Viewing Area)	100 psf
• Roof	30 psf
• Corridor	100 psf
• Corridor above 1st floor	80 psf
• Parking	40 psf
• Stairs	100 psf

Dead Loads

The design bay being considered for this assignment was arbitrary picked for the 3rd floor. Floors 1-6 adjacent the existing church is a self supporting CMU block wall and will not be distributed into the existing or alternative design problems. The calculations for the north façade at floor 3 and the floor are listed below.

<u>3rd Floor Framing</u>

8” reinforced Concrete slab	100 psf
Sprinklers	5 psf
MEP ducts	5 psf
Finishes	10 psf
	120 psf

<u>3rd Floor Curtain Wall</u>

Tributary area (height) = 9’-0”

6” Precast Panel (trib = 50”)	(150 pcf)*(6”x45”) = 280 pLf
½” Glass	(160 pcf)*(1/2”x108”) = 60 pLf
	340 pLf

Existing Floor System

2-way Flat slab with drop panels

The Executive Towers is an 11 story office building with a 12th floor as the penthouse supplying space to the mechanical rooms and offices for the building engineer. Three levels of parking are located underground and are accessible from New York Ave. Executive Tower is layout into a typical open plan office spaces with partitions. The building is primarily cast in place concrete with minimal precast for architectural details on the façade. The building takes on an irregular footprint the adjacent church (top of drawing) and the three avenues on the remaining faces. Due to the fact Executive Tower rest on an unusual footprint, the columns layout takes no typical form as can be seen in Figure 1.1 located below.

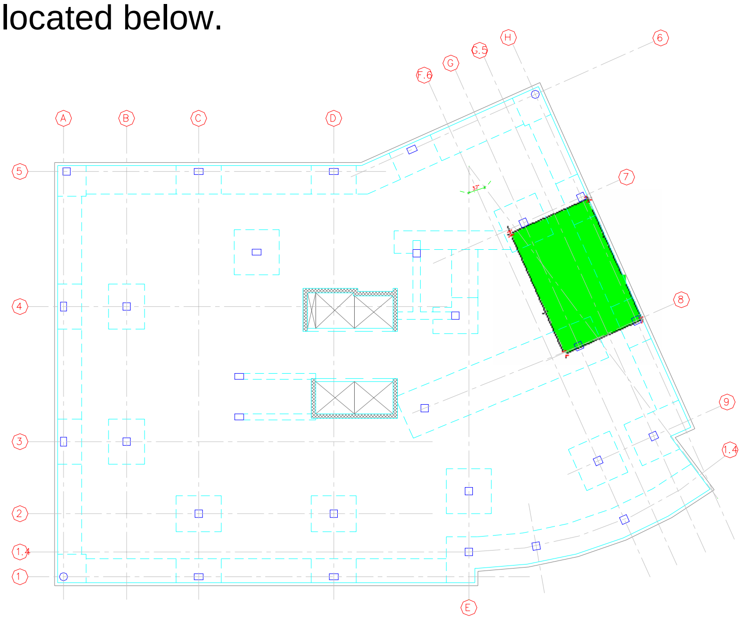


Figure 1.1 – AutoCAD Drawing from 3rd floor framing plan. All floors are typical.

From this design criteria, the concrete framing structure was formed into a two-way 8" concrete flat slab with 10'x8'x0'-8" drop panels at column locations. An additional 4'-0'x3-1/2" continuous thickened slab was placed at the perimeter of the building supporting the glass and precast curtain wall façade.

The layout of reinforcing steel is placed as a general #4 bars at 12" O.C. for the entire flooring system and then additional bars are placed according to the flexural strength required.

Alternative Floor Framing Systems

Concrete Flat Plate

For one of the alternative systems, a concrete flat plate, similar to a concrete flat slab with drop panels, allows the construction of buildings with irregular positioned columns. With this system the column locations and thus the façade can remain unchanged. As seen in Figure 1.1, this analysis of a flat plate system looks closely at Bay 2 on the right exterior panel seen below in Figure 5.1.

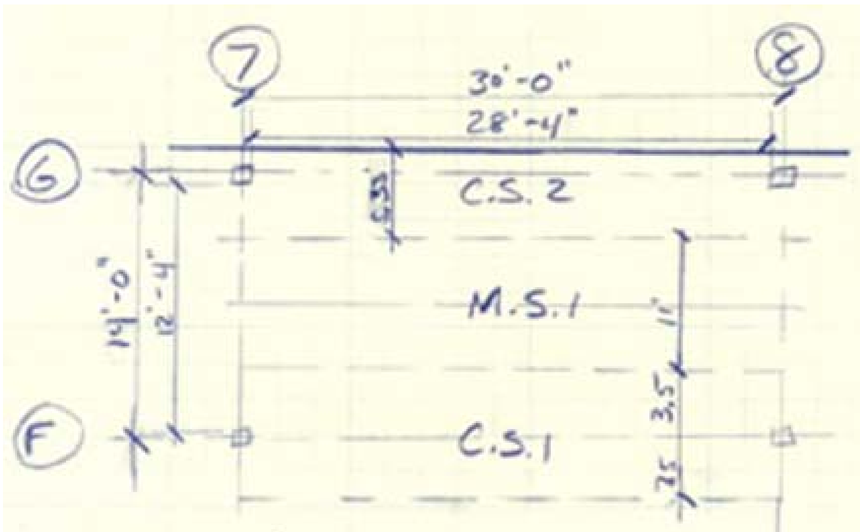


Figure 5.1 – Bay 2 being analyzed for the concrete flat plate alternative system

From the analysis of this system, it was found that the controlling slab depth was 11". This added weight will cause a negative affect through the entire structure requiring it to carry approximately 38 psf more than the existing 8" slab. The added weight will result in higher costs to purchase concrete for the whole building and requires thicker and more steel reinforcing bars. The concrete Mat foundation will have to be increased to satisfy these new loads.

The Concrete Flat Plate will however be easier to construct since it will not require extra to form the drop panels. This will help with construction time and costs but this will not be sufficient to out-weigh the materials costs.

2-Way Post Tensioning

A two-way post tensioning was studied to attempt to reach a floor depth considerably less than the current system. The original flat slab with drop panels uses an 8" slab with 8" drop panels. The typical Bay 2 was used to study this system and its advantages it would have on the structural as a whole. The calculations for all the post tensioning can be seen in Appendix C. Bay 2 was used for this investigation as seen below in Figure 6.1.

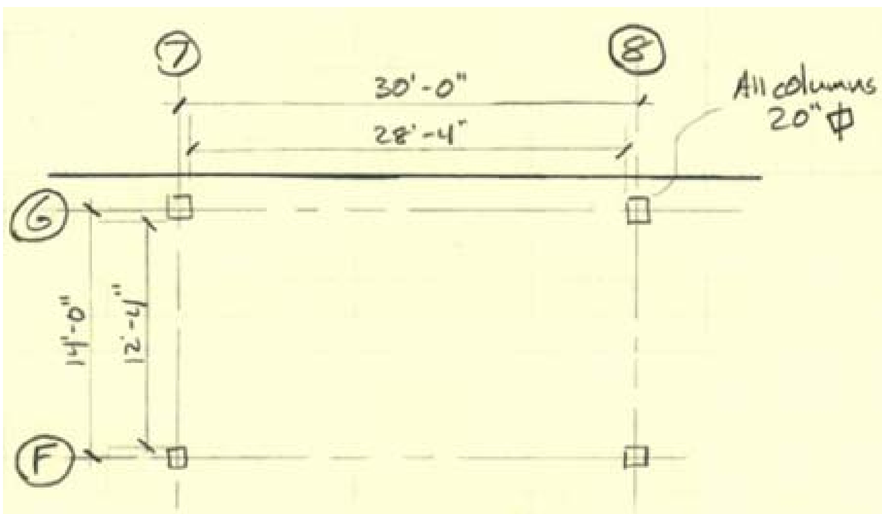


Figure 6.1 – Bay 2 used to analyze the post tensioning alternative system

After investigating the post tensioning system, it was discovered that a minimum thickness of 7 ½" could be achieved. This is only a ½" difference compared to the flat slab but this is also excluding the 8" drop panels at all column locations creating a total dead load self weight of 94 psf approximately 25 psf less than the flat slab if including the drop panels in with the slab self weight. It was calculated that the slab will require 14 tendons spread evenly across the midspan to midspan tensioned to 24.8 kips per strand.

The amount of concrete removed when using post tensioning will not be a deciding factor. Post tension is an expensive process that will need and will need to drastically adjust the slab thickness to be worth it. However after further study of a post tension system using an equivalent frame method and other advanced studies, a slab thickness less than 7 ½" can be achieved.

Composite Steel Beams

A steel frame with composite beams and composite decks were decided to be investigated for this assignment. When laying out the bays to Executive Tower's footprint, two distinctive bays were incorporated. It might have been possible to make one typical bay for the whole building but doing so would disrupt the architecture of the building's façade by moving around exterior columns. It was decided to frame the steel beams and girders according to the exterior column layout for this reason and can be seen in Figure 7.1.

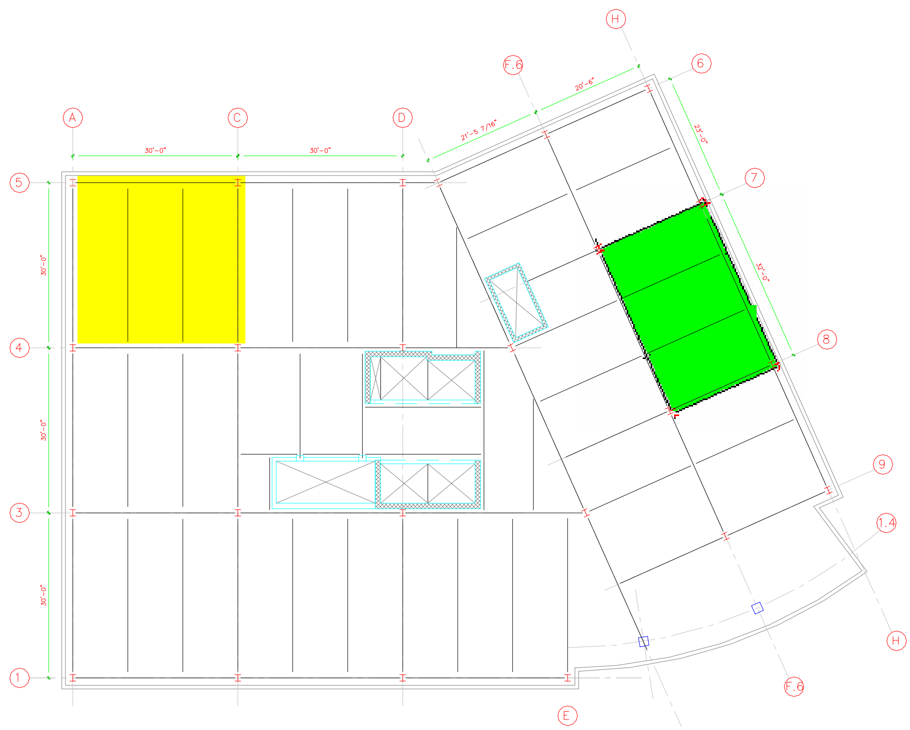


Figure 7.1 – shows the two typical bays used in the analysis of the steel composite system. Bay 1 is 30'x30' and is highlighted yellow. Bay 2 is 32'x 20'-6" and is highlighted green.

For this analysis of a steel composite system, the framing system was developed in AutoCAD and sized using RAM structural analysis. The sections computed by RAM were used as starting points to solve for the beams and girders by hand calculations which can be found in Appendix D. The two separate bays used can be seen on the next page in Figures 8.1 & 8.2. An initial slab thickness was chosen to be 4.5" with a 1.5 VLR22 Vulcraft deck due to the exposed slab load. As a result, the slab is 3.5" thinner than the current flat slab reducing the total dead load per by 42 psf per floor. This significant drop in dead load will recycle all the way down through the columns and into the foundation. The decreased loading in the foundation would result in a decreased thickness the Mat Foundation and after an intense analysis of the soils and loading might prove that a Mat Foundation is not required.

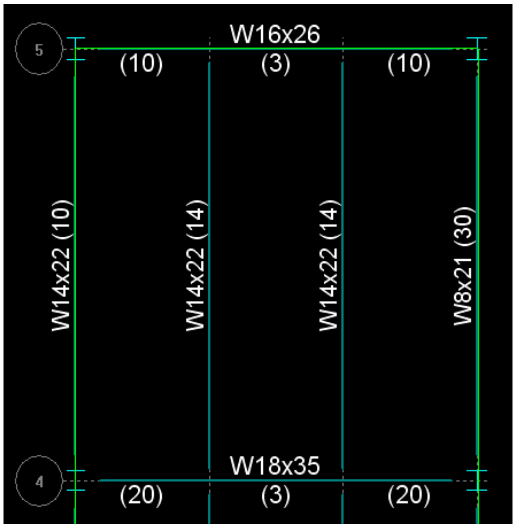


Figure 8.1 – results from the RAM analysis of the composite beam alternative system for Bay 1

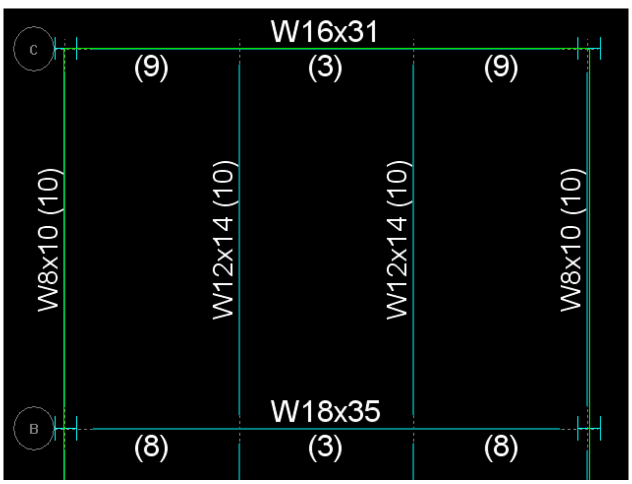


Figure 8.2 – results from RAM analysis of composite beam alternative system for Bay 2

The deepest section solved for was G2 (App. B, p.7) resulting in a W21x44 girder supporting to four beams and its respective deck. The total depth of this section including the deck comes to 22.4". This number rest with in the limitations set in the beginning of the report and leaves approximately 7.6" of workable space.

Open Web Steel Joists

The open web steel joists were decided to be investigated based on the fact that the slab thickness can be made decisively less than the flat slabs 8" thickness. This allows the floor dead load to be considerable less. Just like the composite steel, the steel joist framing plan and bays analyzed are displayed below in Figure 9.1. The hand calculations to determine the deck used and loading to apply to the frame are presented in Appendix E. The calculations for joist and beam sizes were computed using RAM and are recorded in the report.

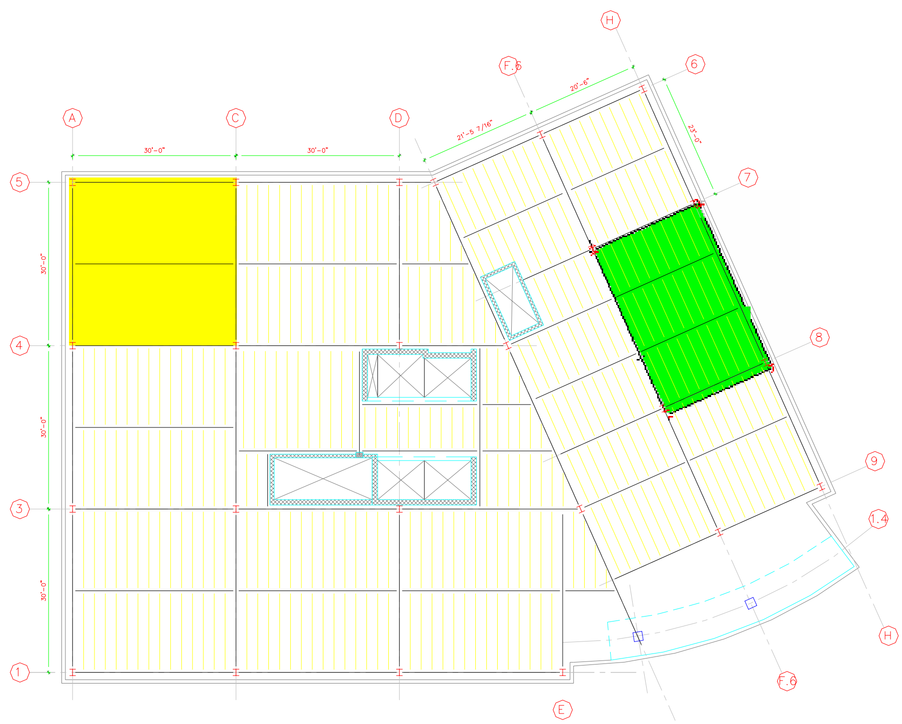


Figure 9.1 – shows the framing plan if using an open web steel joist system. Bay 1 (yellow) is 30'x30' and Bay 2 (green) is 32'x20'-6".

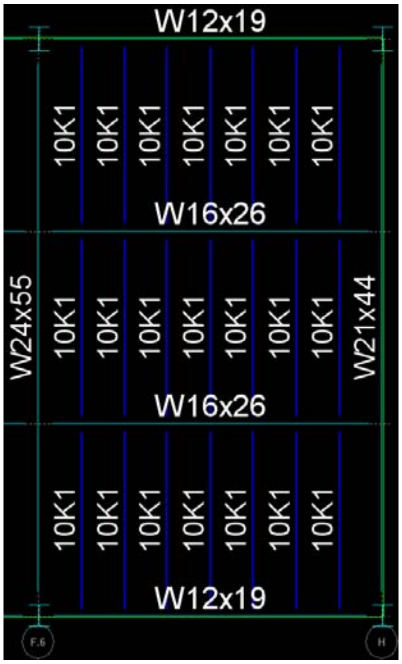


Figure 10.1 – RAM results for steel joist system at Bay 1. The bay is 32'x20'-6" with beams spaced at 10'-4" O.C. and joists spaced at 2'-6" O.C.

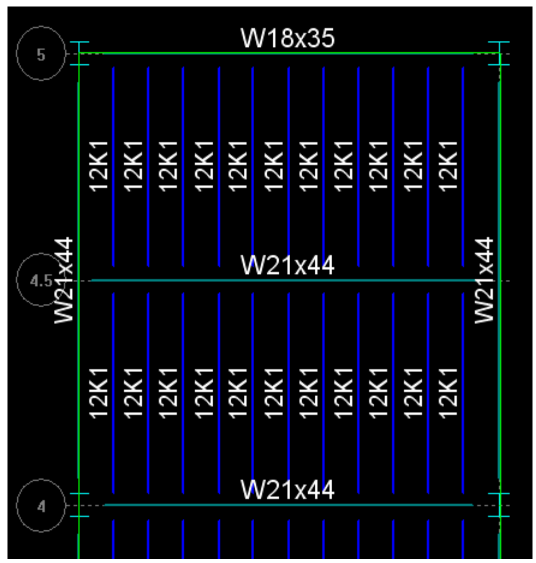


Figure 10.2 – RAM results for steel joist system at Bay 2. The bay is 30'x30' with a beam at the midspan and joists spaced at 2'-6" O.C.

Through the RAM analysis and hand calculations, it was found the most efficient deck and span was a 0.6C Vulcraft deck spanning 2'-6" O.C. with a 2" deck. The total depth of the framing system is 23" thick leaving 7" of open space to be used to pass MEP duct work under. The reduced slab thickness will benefit the design of the columns and foundation used and ultimately the costs. However, this will not out weigh the cost to prefabricate all the joists required for a twelve story building. This system has a considerably less dead weight and can used to redesign the foundation system to not as much concrete.

systems	depth	constructability	pros	cons	Further research?
flat slab	8"	3	thin concrete slab, and thin floor depths	have to form the drop panels, and more reinforcements than post tensioning	
flat plate	11"	4	flat plate are convenient to construct only having to use a continuous plywood form	slab thickness almost a full foot deep creating a large unwanted dead load criteria	no
post tension	7.5"	1	slab thickness reduces materials costs	expensive for post tensioning, slab thick didn't reduce to out weigh the time and cost to post tension	yes
composite beam	22.4"	2	compared to the steel joist, this will work compositely with the slab the beams to be slightly thin	depth of large girder sections will get in way of MEP ducts and may increase floor depths	yes
steel joists	23"	5	easy to place and construct, open webs allow dust work to run through joists	cost to order and prefab all the joists	no

Conclusion

This report has analyzed the probability of using four alternative framing systems to be used in place of the existing structure. All the systems have been design to fit the 2'-6" floor thickness criteria as best as possible. The major goal in this report was developing a system that would have a thinner section than the existing. This will drastically cut down on the amount and weight of the concrete being used which will reduce the materials costs. In the steel systems if used, rather than using the CMU block shear walls lateral system, it would be feasible to construct a braced frame system around the exterior of the elevator core. This will make trying in the lateral system with the framing easier to construct. The shear walls for the concrete systems can remain but for the flat plate, the shear wall will have to be reinvestigated due to the added dead weight of the system to insure they are still affiant.

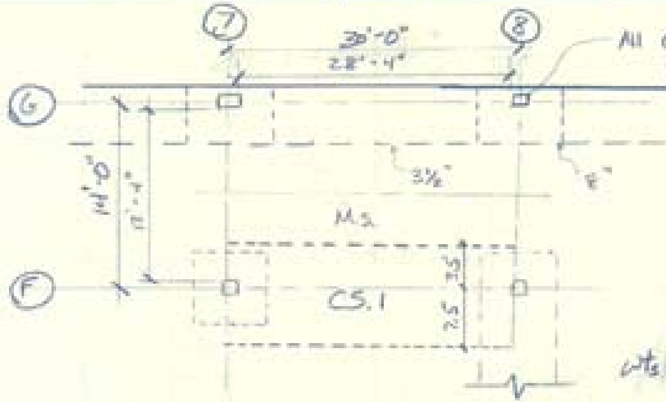
The flat plate system required a slab thickness of 11" across the critical sections. The reinforcement required to support this much added dead weight creates an area of steel in the vicinity 7 square inches and a continuous #5 bar placed every 12" O.C. rather than the standard #4 at 12" O.C. This system proves to be too costly to perform any further studies on.

The post tension system allows the floor depth to be slightly thinner than the one in place. However, during calculating the checks for the current system, the minimum slab thickness according to ACI should have been 9 ½" but the slab is 8". Through more research, a slab thickness considerably less than 7 ½" can be acquired.

A composite steel frame will prove useful in studying more. Having the slab take part of the load will directly affect the beams and girders section sizes. The bays examined for this assignment did all stay under the 2'-6" requirement but allowed only approximately 7" to work with. This could still be useful studying further assuming the height restrictions on Executive Tower were lifted.

It was found the steel joists could be used effectively at saving money on non composite decks and beams and the joists can allow MEP duct to pass freely under the 2" concrete deck. The open web joists were best for constructing as quickly as possible but since time is not critical here, the costs to have everything prefabricated would make this system not worth continued research.

Appendix A



$f'_c = 4000psi$
$f_y = 60ksi$
$h = 8"$
typical reinf.
#4@12" U.O.N.

$w_{s/} = \frac{8"}{12}(150pcf)$
$= 100 psf$
$w_u = 1.2(110) + 1.6(100)$
$w_u = 292 psf$

Designing F + G slab + reinforcement

C.S.1 — 11'

$M_o = \frac{w l_2 l_n^2}{8} = \frac{(.292 ksf)(22')(28'-4")^2}{8}$

$= 644.5^{'k}$

				A-k	ft	ft-k/ft	
644.5'k	Int support 65%	418.9'k	C.S. 75%	314.2	11'	28.6	-5.72 = 22.68 ①
			M.S. 25%	104.7	11'	9.5	-5.72 = 3.78 ②
	Int mid span 35%	225.6'k	C.S. 60%	135.4	11'	12.3	-5.72 = 6.58 ③
			M.S. 40%	90.2	11'	8.2	-5.72 = 2.48 ④

Designing extra reinf. to satify remaining flexural moments

test
#4@12"
$A_s = .20 in^2/ft$

$d = 8.0in - .75in - .5/2 = 6.5 in$

$a = \frac{(.20 in^2/ft)(60ksi)}{.85(4ksi)(12"/ft)} = .294"$

$\phi M_n = .9(.10 in^2/ft)(60ks)(6.5 - \frac{.294}{2})$

$= 68.6^{"k} \Rightarrow 5.72^{'k}$

int Sup C.S. = $249.48^{'k}$
M.S. = $41.58^{'k}$
mid C.S. = $72.38^{'k}$
M.S. = $27.28^{'k}$

① solve like a wide beam

$M_u = 249.5'^k$

$b = 11'(12") = 132"$

assume

$a = .9d = 5.85$

$A_s = \frac{M_u}{\phi f_y a} = \frac{249.5(\frac{12"}{1'})}{.9(60)(5.85)} = 9.48 in^2$

Try 13#8 $A_s = 10.27 in^2$

$d = 8 - .75 - .5 - .5 = 6"$

$a = \frac{A_s f_y}{.85(f'_c)(b)} = \frac{(10.27 in^2)(60)}{.85(4)(132)} = 1.37"$

$\phi M_n = \phi(10.27)(60)(6 - \frac{1.37}{2})$

$= 2947.6 \Rightarrow 245.6'^k < 249.5'^k$ probably close enough but use one extra bar

#4 @ 12" + 14#8
spaced evenly across 11 ft

② $M_u = 41.58$

assume

$a = .85d = 5.1"$ instead of .9

$A_s = \frac{M_u}{\phi f_y a} = \frac{41.58(\frac{12"}{1'})}{.9(60)(5.1)} = 1.81 in^2$

$a = \frac{2.48(60)}{.85(4)(132)}$

$a = .33"$

Try 8#5
$A_s = 2.48$

$d = 8 - .75 - .5 - .3125$
$d = 6.44$

$\phi M_n = \phi(2.48)(60)(6.44 - \frac{.33}{2})$

$\phi M_n = 840.3"^k \Rightarrow 70.0'^k > 41.6'^k$ ok

#4@12" + 8#5
spaced evenly across 11 ft

(3) $M_u = 72.38^{'k}$

assume

$$A_s = \frac{M_u}{\phi f_y .85d} = \frac{72.4(\frac{12}{1})}{.9(60)(.85)(7)} = 2.10\,in^2$$

Try 6#6
$A_s = 2.64\,in^2$

$$a = \frac{2.64(60)}{.85(4)(32)} = .35$$

$d = 8 - .75 - .5 - .375 = 6.375''$

$$\phi M_n = \phi(2.64)(60)(6.375 - \tfrac{.35}{2})$$

$$\phi M_n = 883.9^{''k} \Rightarrow 73.6^{'k} > 72.38^{'k} \text{ ok}$$

$\boxed{\text{USE \#4@12'' + 6\#6 spaced across 11 ft}}$

(4) $M_u = 27.28^{'k}$

assume

$$A_s = \frac{M_u}{\phi f_y .85d} = \frac{27.26(\frac{12''}{1})}{.9(60)(.85)(8)} = .89\,in^2$$

Try 3#5
$A_s = .93\,in^2$

$$a = \frac{(.93)(60)}{.85(4ks)(32)} = .124''$$

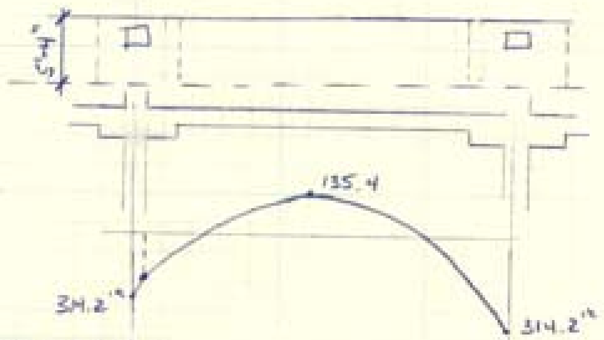
$d = 8 - .75 - .5 - .325$
$= 6.43$

$$\phi M_n = \phi(.93)(60)(6.43 - \tfrac{.124}{2})$$

$$\phi M_n = 319.8 \Rightarrow 26.6 < 27.3^{'k} \text{ use extra bar}$$

$\boxed{\text{use \#4@12'' + 4\#5 spaced across 11 ft}}$

design edge beam



$135.4 + 314.2 = 449.6$

$449.6 = \frac{1}{2} V \left(\frac{30}{2}\right)$

$V = 59.9^k$

$M = \frac{59.9}{2}(x^2)^? - 314.2$

$M_u = -293.4'^k$

Top reinf. $(-M_u)$

$M_u = 293.4$

assume

$A_s = \frac{M_u}{\phi f_y .85 d} = \frac{2934}{.9(60)(.85)(6")} = 4.79 \text{ in}^2$

due to drop panel

8" slab + 8" panel

Try 7 #8

$A_s = 5.53 \text{ in}^2$

$d = 16 - .75 - .5 - .5 = 14.25$

$b = 5.33'$

$a = \frac{5.53(60)}{.85(4)(63.96)}$

$a = 1.53"$

$\phi M_n = \phi(5.53)(60)(14.25 - \frac{1.53}{2})$

$= 4026.9'' ^k \Longrightarrow 335.6'^k > 293.4'^k$ ok

$\boxed{\text{use } 7 \# 8}$

bottom reinf. $(+M_u)$

$M_u = 135.4'^k$

no drop panels

assume

$$A_s = \frac{M_u}{\phi f_y .85d} = \frac{135.4(\frac{12"}{1'})}{.9(60)(.85)(8")} = 4.42 in^2$$

Try 6 #8

$A_s = 4.74$

$$a = \frac{4.74(60)}{.85(4)(63.96")} = 1.31"$$

$d = 8" - .75 - .5 - .5 = 6.25"$

$\phi M_n = \phi(4.74 in^2)(60)(6.25 - \frac{1.31}{2})$

$\phi M_n = 1432.1"^k \longrightarrow 119.3'^k < 135.4'^k$ no good

Try 8 #8

$A_s = 6.32$

$$a = \frac{6.32(60)}{.85(4)(63.96)} = 1.74"$$

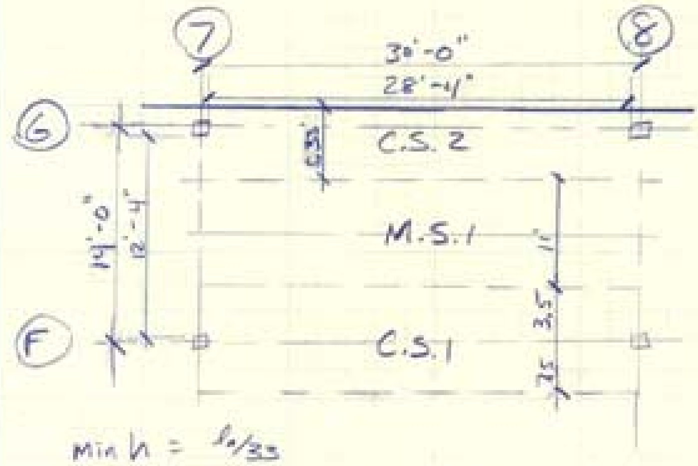
$\phi M_n = \phi(6.32)(60)(6.25 - \frac{1.74}{2})$

$\phi M_n = 1836.1"^k \longrightarrow 153.0'^k > 135.4'^k$ ok

$\boxed{\text{USE 8 \#8}}$

Appendix B

Alternative 1 | Appendix B | two-way Flat Plate | 1/4



$f'_c = 4000$ psi
$f_y = 60$ ksi

typical reinf.
#4 @ 12"

Min $h = \ell_n/33$

$h = \dfrac{30(12")}{33} = 10.9" \Longrightarrow 11"$.

Slab wt $= \dfrac{11"}{12"}(150\text{ pcf}) = 137.5$ psf

$w = 1.2(137.5+10) + 1.6(100)$

$w = 337$ psf

$M_o = \dfrac{w \ell_2 \ell_n^2}{8} = \dfrac{(337)(22)(28.33)^2}{8} = 743.8^{k'}$

				ft-k	ft	ft-k/ft	
743.8"	int. Sup. 65%	483.47	C.S. .75	362.6	11'	33.0	$-12.8^{k'} = 20.2^{k'/ft}$ ①
			M.S. .25	120.9	11'	11.0	
	int. mid. 35%	260.3	C.S. .60	156.2	11'	14.2	$-12.8^{k} = 1.4^{k/ft}$ ②
			M.S. .40	104.1	11'	9.5	

test
#4@12"
$A_s = .20\text{ in}^2/\text{ft}$

$d = 11" - .75 - .5 - .25 = 9.5"$

$a = \dfrac{(.20)(60)}{.85(4\text{ ksi})(12"/1')} = .294"$

$\phi M_n = \phi(.20\text{ in}^2/\text{ft})(60\text{ ksi})(9.5 - \frac{.294}{2})$

$= 8.4^{k/ft}$

try #5@12" $\quad d = 9.437$ $\quad a = \dfrac{.31(60)}{.85(4)(12)} = .46$

$A_s = .31\text{ in}^2/\text{ft}$

$\phi M_n = \phi(.31)(60)(9.44 - \frac{.46}{2})$

$\phi M_n = 12.8^{k}$

① $M_u = 20.2(11') = 222.2^{'k}$

$b = 132''$

assume

$$A_s = \frac{M_u}{.9 f_y .85 d} = \frac{222.2}{.9(60)(.85)(11)} = 5.28 in^2$$

Try 8#8

$A_s = 6.32$ $d = 11 - .75 - .5 - .6 = 9.25$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(6.32)(60)}{.85(4)(132)} = .84''$$

$\phi M_n = \phi(6.32)(60)(11 - \frac{.84}{2})$

$\phi M_n = 3610.7^{''k} \Longrightarrow 300.9^{'k} > 222.2$ ok

$\boxed{\text{USE \#5@12'' + 8\#8 spaced across 11'}}$

② $M_u = (1.4^{k/ft})(11') = 15.4^{'k}$

assume

$$A_s = \frac{M_u}{.9 f_y .85 d} = \frac{15.4(12'')}{.9(60)(.85)(11)} = .366$$

Try 4#3

$A_s = .44 in^2$ $d = 11 - .75 - .5 - .1875 = 9.56''$

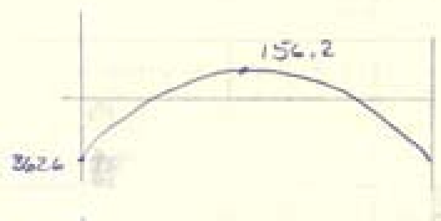
$$a = \frac{.44(60)}{.85(4)(132)} = .059''$$

$\phi M_n = \phi(.44)(60)(9.56 - \frac{.059}{2})$

$\phi M_n = 226.4^{''k} \Longrightarrow 18^{'k} > 15.4^{'k}$ ok

$\boxed{\text{USE \#5@12'' + 4\#3 spaced across 11'}}$

for CS2



Top reinf

$M_u = 293.4^{k}$

assume $A_s = \frac{M_u}{\phi f_y .85d} = \frac{293.4(12")}{.9(60)(.85)(11)} = 6.97 in^2$

Try 9#8 $A_s = 7.11 in^2$ $d = 11" - .75 - .5 - .5" = 9.25$

$b = 5.33$ $a = \frac{(7.11)(60)}{.85(4)(63.96)} = 1.96"$

$\phi M_n = \phi(7.11)(60)(9.25 - \frac{1.96}{2})$

$\phi M_n = 3175.2^{"k} \Longrightarrow 264.6^{'k} < 362.6$ no good

try 12#8 $A_s = 9.48 in^2$ $a = \frac{(9.48)(60)}{.85(4)(63.96)} = 2.61$

$\phi M_n = \phi(9.48)(60)(9.25 - \frac{2.61}{2})$

$\phi M_n = 4067.2^{"k} \Longrightarrow 338.9^{k} < 362.6$ no good

try 14#8 $A_s = 11.08 in^2$ $a = \frac{(11.08)(60)}{.85(4)(63.96)} = 3.06"$

$\phi M_n = \phi(11.08)(60)(9.25 - \frac{3.06}{2})$

$\phi M_n = 4619.0^{"k} \Longrightarrow 384.9^{'k} > 362.6^{'k}$ ok

use 14#8

bottom reinf.

$M_u = 156.2$

assume $A_s = \frac{M_u}{\phi f_y \, .75d} = \frac{156.2\,(\frac{12}{1})}{.9(60)(.75)(11)} = 4.21\,in^2$

try 6 #8
$A_s = 4.74\,in^2$ $\qquad d = 11 - .75 - 1.0 = 9.25''$

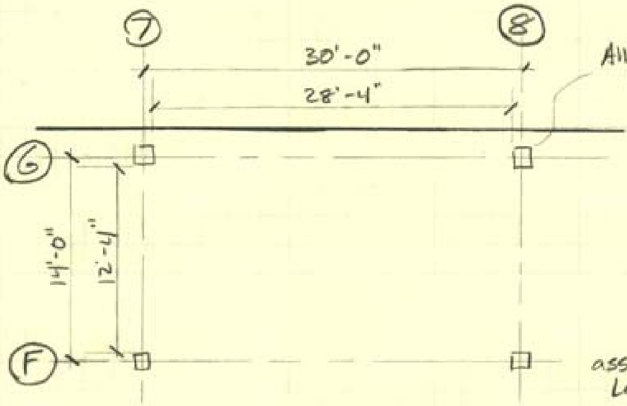
$a = \frac{(4.74)(60)}{.85(4)(23.96)} = 1.31''$

$\phi M_n = \phi(4.74)(60)(9.25 - \frac{1.31}{2})$

$\phi M_n = 2200.0^{ik} \Longrightarrow 183.3^{'k} > 156^{'k}$ ok

$\boxed{\text{USE } 6\ \#8}$

Appendix C



All columns 20" ϕ

$f'_c = 4000$ psi
$f_y = 60$ ksi

PT: $\frac{1}{2}$" ϕ, 7-wire strands
$A = .153\,in^2$
$f_{pu} = 270$ ksi

$P_e = (.60)(270)(.153)$ — assumed Losses

$P_e = 24.8^k$

Loads

slab DL = self wt
superimposed DL = 10 psf
LL = 100 psf

$h = \frac{L}{45}$

$h = \frac{28.33(12)}{45} \approx 7.5"$

Slab DL = self wt = $\frac{7.5"}{12"}(150\ pcf) = 93.75$ psf

Loading
DL = 93.75 psf
SIDL = 10.0 psf

$LL_o = 100$ psf

$A_T = 420\,ft^2 > 400\,ft^2$ ok use LL reduction

$$LL = LL_o\left(.25 + \frac{15}{\sqrt{1(420)}}\right)$$

$LL = 98.0$ psf

Design to Class U

$A = bh = (14')(12''/1')(7.5'') = 1260 in^2$

$S = \frac{bh^2}{6} = \frac{(14')(\frac{12''}{1'})(7.5')^2}{6} = 1575 in^3$

Set Parameters

$f'_c = 4000$ psi

Comp. $= .60 f'_c = .6(4000) = 2400$ psi

Tension $= 3\sqrt{f'_c} = 3\sqrt{4000} = 190$ psi

Service Loads

Comp $= .45 f'_c = .45(4000) = 1800$ psi

Tension $= 6\sqrt{f'_c} = 6\sqrt{4000} = 379$ psi

Target load balance

$.65 w_{DL} = .65(93.75) = 60.9$ psf

$w_b = 60.9(14') = 852.6$ plf

$a_{end} = \frac{6.5 + 3.75}{2} - 1.75 = 3.375''$

$a_{int} = 6.5 - 1.0 = 5.5''$

$P = \frac{w_b L^2}{8 a_{end}} = \frac{(852.6)(30)^2}{8(3.375/12)} \Longrightarrow 341.0^k$

Check Precomp All.

$\#$ tendons $= \frac{(341)^k}{24.8^k} = 13.75$ USE 14

$P_{act} = (14\ t)(24.8) = 347.2^k$

$w_b = \frac{347.2}{341.0}(852.6) = 868.1$ plf

$P_{act}/A = \frac{(347.2^k)(\frac{1000\#}{1^k})}{1260 in^2} = 275.5$ psi check interior span force

$$P = \frac{(.868\ klf)(30)^2}{8(6(\frac{1'}{12}))} = 195.3^k$$

check slab stresses

Dead Load

$$w_{DL} = \frac{(93.75)(14')}{1000} = 1.31\ klf$$

$$+M = \frac{(1.31 klf)(30^2)}{24} = 49.1^{'k}$$

$$-M = \frac{(1.31\ klf)(30^2)}{12} = 98.3^{'k}$$

Live load

$$w_{LL} = \frac{(98.0)(14')}{1000} = 1.37\ klf$$

$$+M = \frac{(1.37 klf)(30^2)}{24} = 51.4^{'k}$$

$$-M = \frac{(1.37\ klf)(30)^2}{12} = 102.7^{'k}$$

total balancing moment

$$w_b = -.868\ klf$$

$$+M = \frac{(-.868)(30^2)}{24} = -32.5^{'k}$$

$$-M = \frac{(-.868)(30^2)}{12} = -65.1^{'k}$$

stresses immediately after jacking (DL + PT)

mid span stresses

$f_{top} = (-M_{DL} + M_{bal})/S - P/A$

$f_{bott} = (+M_{DL} - M_{bal})/S - P/A$

$$f_{top} = \frac{(49^{'k} - 32.5^{'k})(\frac{12''}{1'})(\frac{1000^{\#}}{1^k})}{1575 in^3} - 275.5 psi$$

$= -149.8$ psi comp < 2400 psi ok

$$f_{bot} = \frac{(-49 + 32.5)(\frac{12''}{1'})(\frac{1000^{\#}}{1^k})}{1575 in^2} - 275.5 psi$$

$= -401.2$ psi < 2400 psi ok

Support stresses

$$f_{top} = \frac{(98.3 - 65.1)(\frac{12''}{1'})(\frac{1000^{\#}}{1^k})}{1575} - 275.5$$

$= -22.5$ psi < 2400 psi ok

$$f_{bot} = \frac{(-98.3 + 65.1)(\frac{12''}{1'})(\frac{1000^{\#}}{1^k})}{1575} - 275.5$$

$= -528.4$ psi < 2400 psi ok

Stresses at service loads (DL+LL+PT)

midspan stresses

Int span

$$f_{top} = \frac{(49.1 + 51.4 - 32.5)\left(\frac{12''}{1'}\right)\left(\frac{1000^{\#}}{1^k}\right)}{1575\,in^3} - 275.5\,psi$$

$$= 242.6\,psi < 1800\,psi \quad \underline{ok}$$

$$f_{bot} = \frac{(-49.1 - 51.4 + 32.5)\left(\frac{12''}{1'}\right)\left(\frac{1000^{\#}}{1^k}\right)}{1575\,in^3} - 275.5\,psi$$

$$= 793.6\,psi < 1800\,psi \quad \underline{ok}$$

support stresses

$$f_{top} = \frac{(98.3 + 102.7 - 65.1)\left(\frac{12''}{1'}\right)\left(\frac{1000^{\#}}{1^k}\right)}{1575\,in^3} - 275.5$$

$$= 759.9\,psi < 1800\,psi \quad \underline{ok}$$

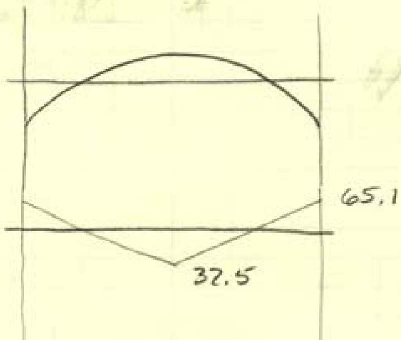
$$f_{bott} = \frac{(-98.3 - 102.7 + 65.1)\left(\frac{12''}{1'}\right)\left(\frac{1000^{\#}}{1^k}\right)}{1575\,in^3} - 275.5$$

$$= 1310.9\,psi < 1800\,psi \quad \underline{ok}$$

Ultimate Strength

$M_1 = P \times e$

$= (341^k)(2.75'')(\frac{1'}{12''}) = 78.1^{1k}$



midspan $M_u = 1.2(49.1) + 1.6(51.4) + 1.0(-32.5)$

$M_u = 108.66^{1k}$

support $M_u = 1.2(-98.3) + 1.6(-102.7) + 1.0(65.1)$

$M_u = -217.2^{1k}$

midspan

$M_u = +108.66$

assume $A_s = \frac{M_u}{\phi f_y .75d} = \frac{108.66(12'')}{.9(60)(.75)(7.5)} = \frac{4.29\ in^2}{14} = .306\ in^2/ft$

Try #5 @ 12"

$A_s = .31\ in^2/ft$

$a = \frac{A_s f_y}{.85(f'_c)(b)} = \frac{.31(60)}{.85(4)(12'')} = .45''$

$d = 7.5 - .5 - .3125 = 6.69$

$\phi M_n = \phi(.31)(60)(6.69 - \frac{.45}{2})$

$\phi M_n = 108.2^{1k/ft} \Rightarrow 9.01^{1k/ft}$

$= (9.01)(14') = 126.1^{1k} > 108.6$ ok

USE #5@12"

support

$M_u = -217.2^{'k}$

assume $A_s = \dfrac{M_u}{\phi f_y(.7)(d)} = \dfrac{217.2(12)}{.9(60)(.7)(7.5)} = \dfrac{9.19 \text{ in}^2}{14'} = .66 \text{ in}^2/\text{ft}$

Try #8@12"
$A_s = .79 \text{ in}^2/\text{ft}$

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{(.79)(60)}{.85(4)(12")} = 1.16$

$d = 7.5 - .5 - .5 = 6.5$

$\phi M_n = \phi(.79)(60)(6.5 - \frac{1.16}{2})$

$\phi M_n = 252.5^{\text{in-k}} \Rightarrow (21.04^{'k/ft})(14)$

$= 294.6^{'k} > 217.2^{'k}$ ok

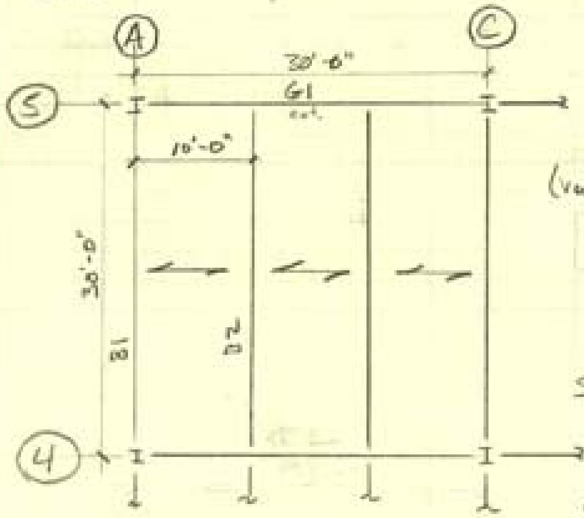
$\boxed{\begin{array}{l}\text{USE \#5@12" bottom} \\ \text{\#8@12" top} \\ \text{+ 14 PT tendons across 14'}\end{array}}$

Appendix D

Alternative 3 | Appendix D | Steel w/ composit deck 1/7

Design Check Steel composite Framing system
1) find appropriate decking
2) Design beams for Bays 1 + 2
3) Design composite Girders for Bays 1+2

Bay 1



Live load = 80 + 20 psf (partitions)
= 100 psf

critical span for both Bays
use L = 11'-0"
(Vulcraft Floor Decks - pg 44)

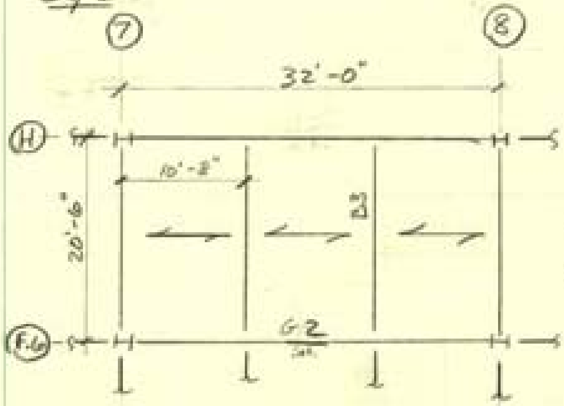
Using a Vulcraft
1.5 VLR22
4½ normal wt concrete

Sup. LL = 124 psf > 100 psf ok

equiv. cubic feet
$.329 \frac{ft^3}{ft^2}$

deck wt = 1.78 psf ⟹ 2 psf

Bay 2



Dead Load
concrete $150 pcf (.329 \frac{ft^3}{ft^2}) = 49.35 psf$
sprinklers 5 psf
MEP 5 psf
~60 psf

$\omega_u = 1.2(60 psf) + 1.6(100 psf)$
$= 232 psf$

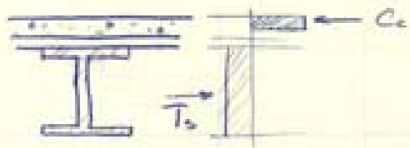
Bay 2 Design

B3 int beam Trib Area =

Try W10x12
$A = 3.54\ in^2$
$d = 9.87\ in$

$b_{eff} = 10.66'\left(\frac{12"}{1'}\right) = 127.9"$

$= \frac{20.5'}{4}\left(\frac{12"}{1'}\right) = 61.5"$ ← controls



$C_c = .85(4ksi)(61.5")(3") = 627.3^k$

$T_s = 3.54(50) = 177^k$ ← controls

$177 = .85(61.5)(4ksi)\,a$
$a = .85$

$\phi M_n = \phi(177^k)(14.37 - \frac{.85}{2} - \frac{9.87}{2})$

$= 1375.5^{ik} \Longrightarrow 112.1^{k} < 130.8^{k}$ no good

$w = .232\ ksf(10.67') + 1.2\left(\frac{12}{1000}\right) = 2.49\ klf$

$M_u = \frac{(2.49\ klf)(20.5)^2}{8} = 130.8^{ik}$ $\frac{113}{3.54} = \frac{130.8}{\phi A}$

$A \approx 4.13\ in^2$

Try W10x15
$A = 4.41\ in^2$
$d = 9.99\ in$

$C_c = 627.3^k$
$T_s = 4.41(50) = 220.5^k$

$220.5 = .85(61.5")(4ks)\,a$
$a = 1.05"$

$\phi M_n = \phi(220.5^k)(14.49 - \frac{9.99}{2} - \frac{1.05}{2})$

$= 1681.2 \Longrightarrow 140.1^{ik} > 130.8^{ik}$ ok

USE W10x15

Bay 1 Design

$f'_c = 4000\,psi$
$f_y = 50\,ksi$

B1 interior beam $\quad$ Trib A = $(30')(10') = 300\,ft^2$

Try W14x22 (LRFD table 1-1)

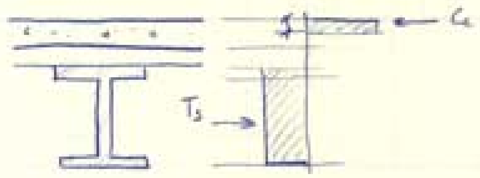
Assume Fully Composite

$A = 6.29\,in^2$
$t_f = .335\,in$
$b_f = 5.00\,in$
$t_w = .230\,in$
$d = 13.7\,in$

$b_{eff} = 10'(\frac{12''}{1}) = 120''$
$\quad\;\; = \dfrac{30'(\frac{12''}{1})}{4} = 90''$ ← controls

$C_c = .85(4\,ksi)(90'')(3'') = 918^k$

$T_s = (6.29\,in^2)(50\,ksi) = 314.5^k$



$314.5 = .85(90'')(4\,ksi)\,a$

$a = 1.03''$

$\phi M_n = \phi(314.5)(18.2 - \frac{1.03}{2} - \frac{13.7}{2}) = 2896.5^{''k} \rightarrow 241.4^{'k}$

$W = (232\,psf)(10') \Rightarrow 2.32\,klf$

$M_u = \dfrac{(2.32\,klf)(30')^2}{8} = 261^{'k}$

$\phi M_n = 241.4^{'k} > 261^{'k}$ no good

$\dfrac{241.4}{6.29^2} = \dfrac{261}{A^2}$

$A \sim 6.54\,in^2$

try W14x26 (LRFD tbl 1-1)

$A = 7.69$
$d = 13.9$

W14 x 26 cont'

$C_c = 918^k$

$T_s = (7.69 in^2)(50 ksi) = 384.5^k$

$384.5 = .85(90)(4 ksi)a$

$a = 1.26 in$

$\phi M_n = \phi(384.5^k)(18.4 - \frac{1.26}{2} - \frac{13.9}{2}) = 3536.2 \rightarrow 294.7'^k$

$\phi M_n = 294.7'^k > 261'^k$ ok

req'd shear studs

$\Sigma Q = 384.5^k$

$\frac{3}{4}\phi$ studs $q = 26.1^k$ $n = \frac{384.5}{26.1} = 14.7$

(tab 5-13)

use 15 studs

Bay 1

B2 exterior beam Trib Area = $(30')(10')/2 = 150 ft^2$

$W = (232 psf)(5') \Rightarrow 1.16$ curtain wall $DL = .340 klf$

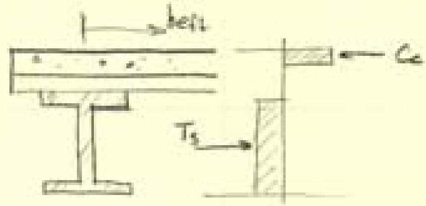
$W_T = 1.16 + 1.2(.340) = 1.568 klf$

unfactored

$M_u = \frac{wl^2}{8} = \frac{(1.568 klf)(30')^2}{8} = 176.4$

try W14x22
$A = 6.29 in^2$
$d = 13.7 in$

$b_{eff} = \frac{10'}{2}(\frac{12''}{1'}) = 60 in$
$= \frac{30}{8}(\frac{12''}{1'}) = 45 in$ ← controls



$C_c = .85(4ksi)(45'')(3'') = 459^k$

$T_s = (6.29 in^2)(50ksi) = 314.5^k$ ← controls

$314.5 = .85(4ksi)(45'')a$

$a = 2.05''$

$\phi M_n = \phi(314.5)(18.2 - \frac{2.05}{2} - \frac{13.7}{2})$

$= 2760.13^{ik} \Rightarrow 230.01 > 176.4$ ok but trying a smaller section

Try W12x19
$A = 5.57 in^2$
$d = 12.2 in$

$C_c = 459$

$T_s = (5.57 in^2)(50) = 278.5$ ← controls

$278.5 = .85(4ksi)(45'')a$

$a = 1.82 in$

$\phi M_n = \phi(278.5^k)(16.7 - \frac{12.2}{2} - \frac{1.82}{2})$

$= 2293.9^{ik} \Rightarrow 191.2^{k} > 176.4$ ok

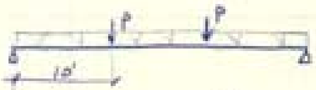
USE W12x19

Bay 1

G1 no curtain wall load

$P = \frac{232(10')}{2}(30') + \frac{26(30)}{2}$

$P = 35.2^k$



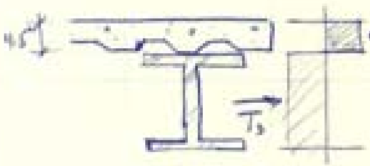
$M_u = PL + \frac{wl^2}{8}$

$M_u = 35.2^k(10) + \frac{(.026^{k/ft})(30')^2}{8} = 352.1'^k$

Try W16×26
$A = 7.68\,in^2$
$d = 15.7\,in$

$b_{eff} = 30'\left(\frac{12"}{1'}\right) = 360"$

$= \frac{30'}{8}\left(\frac{12"}{1'}\right) = 45"$ ← controls



$C_c = .85(4ksi)(45")(4.5") = 688.5^k$

$T_s = (7.68\,in^2)(50ksi) = 384^k$

$384 = .85(4)(45")a$

$a = 2.51"$

$\phi M_n = \phi(384^k)(20.2 - \frac{15.7}{2} - \frac{2.51}{2})$

$= 3621.4"^k \Rightarrow 301.8^k < 352.1^k$ no good

$\frac{301}{7.68^2} = \frac{352.1}{\# A^2}$

$A \sim 8.31\,in^2$

Try W16×31
$A = 9.13\,in^2$
$d = 15.9\,in$

$C_c = 688.5$

$T_s = (9.13\,in^2)(50\,ksi) = 456.5^k$

$456.5 = .85(4ksi)(45")a$

$a = 2.98"$

$\phi M_n = \phi(456.5)(20.4 - \frac{15.9}{2} - \frac{2.98}{2})$

$= 4252.8"^k \Rightarrow 354.4 > 352.1$ ok

Bay 2
G2 interior span

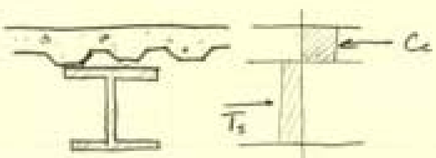
$2P = 2\left[\frac{(.232 \text{ksf})}{2}(10.67')(20.5') + \frac{.035(20.5)}{2}\right]$

$2P = 51.5^{k}$

$M_u = 51.5^{k}(10.67') + \frac{(.035 \text{ klf})(32')}{8} = 549.6^{'k}$

Try W16x40
$A = 11.8 \text{ in}^2$
$d = 17.9 \text{ in}$

$b_{eff} = 42\left(\frac{12''}{1}\right) = 504''$
$= \frac{32}{4}\left(\frac{12''}{1}\right) = 96''$ ← controls



$C_c = .85(4 \text{ksi})(96'')(4.5'') = 1468.8^{k}$

$T_s = (11.8 \text{ in}^2)(50 \text{ksi}) = 590^{k}$

$590 = .85(4 \text{ksi})(96'')a$

$a = 1.81''$

$\phi M_n = \phi(590^{k})\left(22.4'' - \frac{17.9''}{2} - \frac{1.81''}{2}\right)$

$= 6291.3^{''k} \Rightarrow 524.3 < 549.6^{'k}$ no good

Try W21x44
$A = 13.0 \text{ in}^2$
$d = 20.7 \text{ in}$

$C_c = 1468.8^{k}$

$T_s = (13.0)(50 \text{ksi}) = 650^{k}$

$650 = .85(4 \text{ksi})(96)a$

$a = 1.99''$

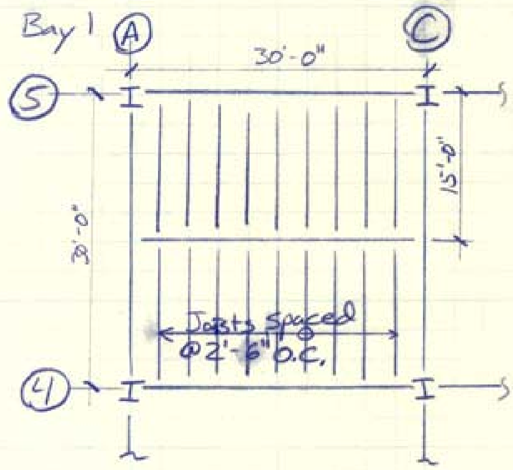
$\phi M_n = \phi(650^{k})\left(25.2 - \frac{20.7}{2} - \frac{1.99}{2}\right)$

$= 7654.9^{''k} \Rightarrow 637.9^{'k} > 549.6$ ok

Appendix E

Design open web stee Joists

Bay 1



$LL = 80 + 20$ psf

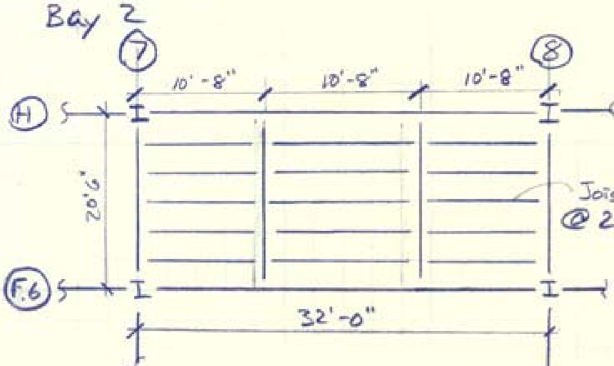
$= 100$ psf

critical joist spacing
2'-6" O.C.

(from Valcraft pg 20)

USE 0.6C 2" slab
@ 2'-6" O.C. spacing
(assume 3 span condition)
$\Sigma LL = 124 > 100$ psf ok
reinf. = 6"x6"-W1.4 x W1.4

Bay 2



Self wt of slab + deck — .079 $\frac{cf}{sf}$

$DL_s = .079(145\,pcf)(2") + 1\,psf = 23.9$ psf
↖ deck

$w_u = 1.2(23.9 + 10) + 1.6(100)$

$= 200.7$ psf