William W. Wilkins Professional Building

Columbus, Ohio

Technical Assignment 2

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Table of Contents

Executive Summary

The William W. Wilkins Professional Building is a 6 story, 112,000 sq. ft. medical office building located in Columbus, Ohio. Costing approximately $7.4 Million, it is essentially an addition to the Grant Riverside hospital across the street. These buildings are connected by a pedestrian bridge from the third floor. Enclosed by brick veneer, precast concrete and spandrel glass panels the exterior is non-load bearing. The structure is made up of steel beams acting compositely. Loads are transferred through girders to the W12 columns that carry the load down to caissons.

This report is a comparison of several alternate floor systems for the Wilkins building. Each system has advantages and disadvantages. The purpose of this report is to determine which, if any, of these systems warrant further investigation to be utilized instead of the existing system. The existing system consists of a 5½" slab acting compositely with steel beams.

The alternate systems investigated are:

1. One-way concrete slab with concrete beams
2. Two-way concrete flat plate
3. Post-tensioned concrete slab
4. Precast hollow core planks
5. Precast Double Tee planks
6. Open web steel joists

The floor system in the Wilkins building has a one-hour fire rating requirement. A common advantage in all of these systems is that the concrete slab will effectively meet this fire rating. In addition, many of these systems will increase the overall building weight. This will increase the bearing on the foundations requiring further investigation. Lateral systems used in resisting seismic forces will also have to be looked at.

Using a one-way concrete slab with concrete beams allows for a slightly shallower slab at 4½" instead of 5½". However, the loads placed on the girder require it to be 30", which is deeper than the W24 used in the existing system. Overall, this will result in a deeper system. This is not a major disadvantage as there are no height restrictions on the Wilkins building.

The two-way concrete flat plate uses simple formwork making it easy to construct. Roughly square bays make a two-way slab appropriate providing a shallower floor system than the existing composite system. Since height restrictions do not apply to the Wilkins building this is not a significant advantage.

The post-tensioned slab provides the shallowest floor system at 9". The higher rebar strengths allow for this shallower slab. Support for this system could be masonry bearing walls, steel beams or frames.

Hollow core spandeck with a 2" topping results in a total floor depth of 14". This system is easy to construct reducing labor expenses. The double-tee precast planks with a 2" topping require a 34" depth. Using double-tee planks will allow for greater floor spans opening up the floor space. As with the hollow core system, installation is quick and easy.

Open web steel joists have the advantage of openings for mechanical and electrical equipment. Due to their reduced strength capacity, however, a 22K7 is required. This is deeper then the existing beams. However, W24 girders can still be utilized. This will give the same overall floor depth.

After analyzing and comparing these various systems, I have concluded that with further research, four of the six alternatives could be considered. The open web steel joist system does not warrant further investigation as it has no real advantage over the existing system. The other option I ruled out was the one-way slab with concrete beams. This system results in a deeper ceiling cavity and a heavier building. Due to the higher labor costs associated with concrete, this system does not have enough advantages to compensate for the disadvantages.

Loads

	Existing loads(psf)	My loads(psf)	Source
Live:			ASCE 7-05
Office floor	50	50	
Corridors	80	80	
Lobbies & Stairs	100	100	
Library Stacks	150	150	
Roof		20	
Snow	25 + Drift	20	
Dead:			
Partitions	20	10	IBC
Metal Deck		2.4	Catalogues
Concrete		54	Catalogues
Beams	varies	varies	LRFD
Ceiling	5	2	
MEP		3	
Misc.	5	3.6	
Total:	30	75	
Roof Dead:	22 + steel	22 + steel	

Structural Systems

The William W. Wilkins Professional Building is a 6 story, 112,000 sq. ft. medical office building located in Columbus, Ohio. Costing approximately $7.4 Million, it is essentially an addition to the Grant Riverside hospital across the street. These buildings are connected by a pedestrian bridge from the third floor. Enclosed by brick veneer, precast concrete and spandrel glass panels the exterior is non-load bearing. The structure is made up of steel beams acting compositely. Loads are transferred through girders to the W12 columns that carry the load down to caissons.

Existing Floor system: Composite Steel Beams

The floor system in the Wilkins building is designed for composite behavior. Floor slabs consist of 3½″ normal weight concrete on 2″ 18-gage composite steel deck reinforced with W2.1xW2.1 welded wire fabric (WWF). Decking is welded to support steel. The slab on grade (SOG) varies slightly consisting of 4″ concrete on 6″ porous fill reinforced with W1.4xW1.4 WWF. A typical bay consists of W16x31 beams spanning 32′-4″ in the East-West direction framing into W24x55 girders spanning 30′-9″ in the North-South direction. ¾″ diameter by 4 ½″ long headed studs are spaced evenly across members to transfer loads. A typical bay is shown below in Figure 1.

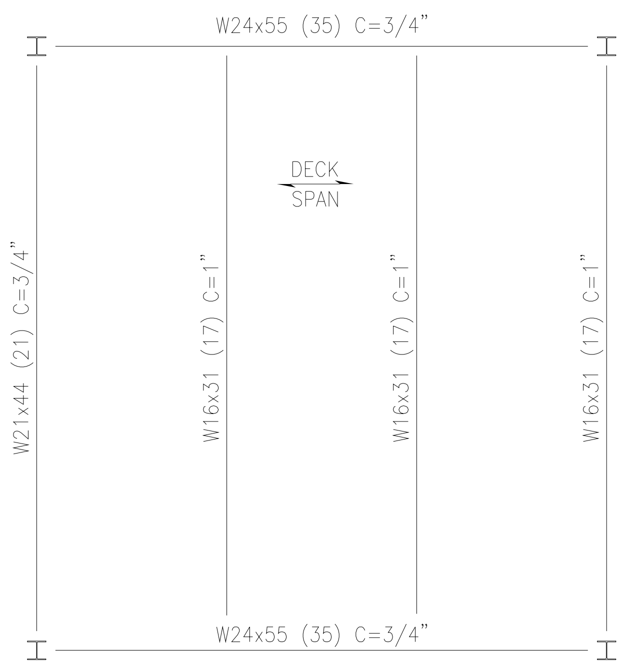


Figure 1: Existing Framing of Typical Bay

Alternate System 1: One-way Slab with Concrete Beams

Assuming concrete beams spaced at 10.25′, the same as the existing steel beams, I determined a 4½″ slab reinforced with #4 bars at 18″ o.c. would be required. Supporting the slab and distributing the loads to the girders are 12″x20″ beams with (6) #6 bars distributed in two layers. Carrying the loads to the columns would be 18″x30″ girders reinforced with (2) #9 and (4) #10 bars.

Reusable formwork will lower construction costs of this system. Fireproofing is an intrinsic property of concrete eliminating the need for additional fireproofing. The weight of the system will reduce the chances of vibration problems. In addition, there is room between beams for mechanical, electrical and plumbing placement in the ceiling cavity.

Disadvantages to this system include higher labor costs, larger concrete columns and an overall increase in depth of the floor system. This system also requires greater curing time to allow the beams to develop sufficient strength to support themselves before removing the forms. The increase in weight, which helps with vibration, will also increase the load placed on the foundations as well as increasing the seismic forces applied on the building. A typical bay is shown in Figure 2.

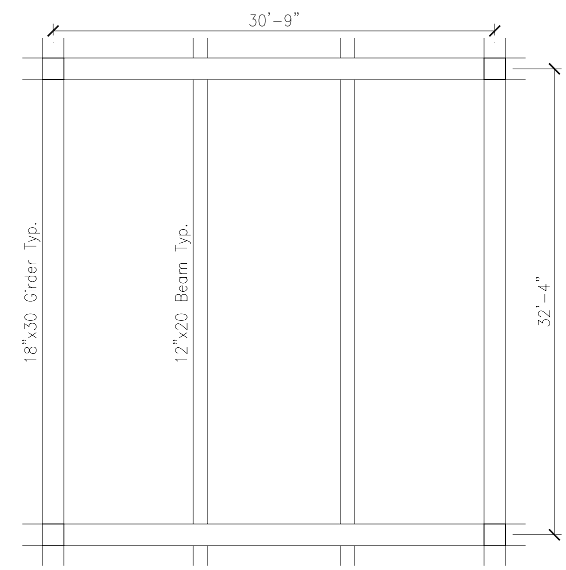


Figure 2: Typical Bay One-Way Slab with Concrete Beams

<u>Alternate System 2: Two-way Concrete Flat Plate</u>

The relatively square dimensions of the bays used in the Wilkins building lends itself to a two-way concrete system such as a flat plate. There are many advantages to the use of a flat plate system. The second shallowest ceiling cavity is obtained due to the absence of beams. This will allow room for mechanical, electrical and plumbing. The required one-hour fire rating is easily met with the 13" slab required for this bay size. The increased weight of this system will help reduce vibration in the building. One of the main advantages of the flat plate system is its economy. This is in part due to the simple formwork used. Moreover, the reinforcing steel layout is simple helping to reduce construction time, thus reducing labor costs.

Disadvantages associated with flat plates are once again the increased weight of the system. Construction costs will increase if larger foundations are required from the increased dead load of the system. Large concrete columns will be required, as opposed to the existing W12's. This will cut into the usable floor space.

Based on my calculations a 13" slab reinforced with #6 bars will be sufficient. The number of bars required varies. In the middle strip 11 or 12 bars is required. In the column strip anywhere from 12 to 30 bars is required. According to CRSI, columns of 35"-40" will be required for this span. A typical bay is shown below in Figure 3.

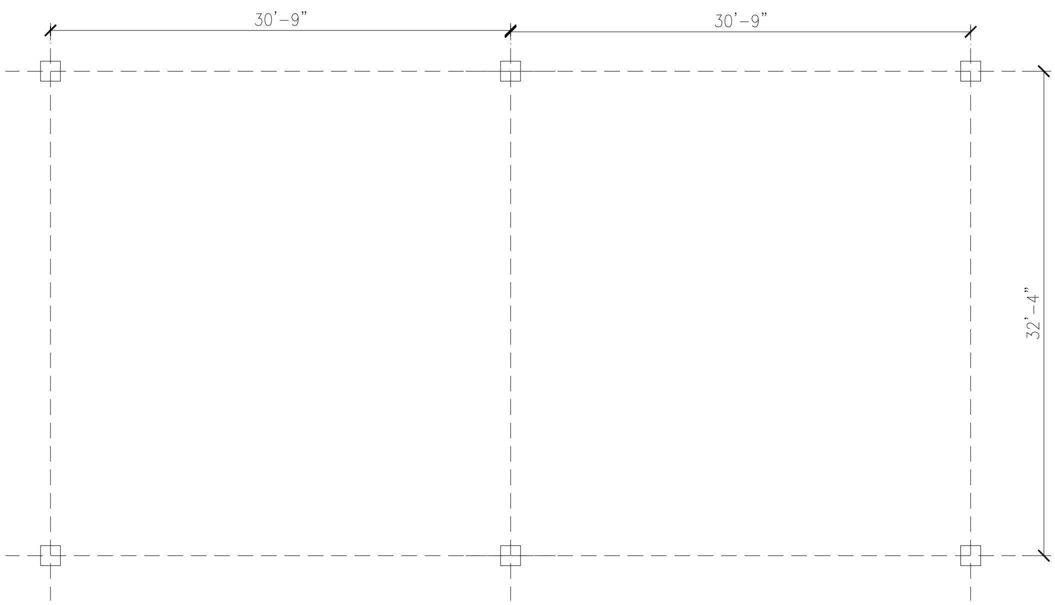


Figure 3: Flat Plate Plan

<u>Alternate System 3: Post-Tensioned Concrete Slab</u>

This system allowed for the shallowest ceiling cavity. With a slab depth of only 9", plenty of room is available for mechanical, electrical and plumbing (MEP) to be placed in the ceiling cavity. Even with the MEP equipment, the overall floor-to-floor height should be lower using post-tensioning than the existing system. However, height restrictions in the Columbus area are not an issue so this is not a significant factor. Advantages in using post-tensioning, besides the shallower slab, include built in fireproofing and additional weight to help reduce vibrations. As with alternatives 1 and 2, this additional weight may induce a need to redesign the foundations and/or lateral systems.

This system consists of a 9" slab reinforced with (12) strands at 20kip/strand of 270ksi steel. A typical bay is shown below in Figure 4.

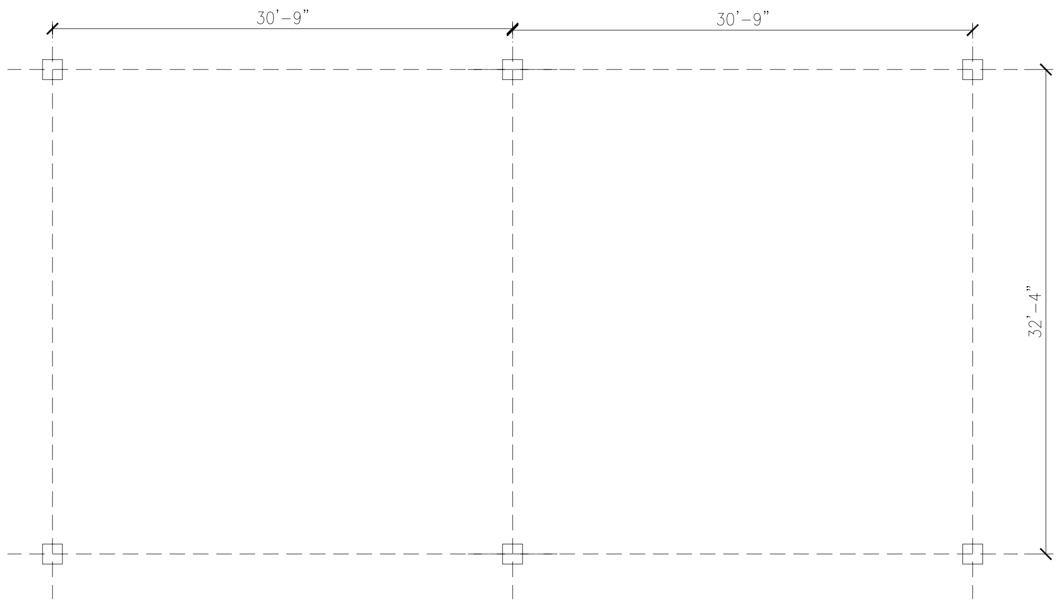


Figure 4: Post-tensioned Concrete Slab

Alternate System 4: Precast Hollow Core Planks

Using Nitterhouse Concrete Products, a 12"x4' spandeck with 2" cast-in-place topping, strand pattern 6, can carry a superimposed load of 140psf at 33'. The max span in the Wilkins building is 32'. With a superimposed load of 121psf in the lobby areas and lower in office areas, this will work with room to spare. The reinforcing used in these planks consists of ½" diameter, 270k, lo-relaxation strands. The support for precast planks could be girders spanning between columns or possibly masonry bearing walls.

In using these panels, it is assumed the column placement can be altered to 30'-9" x 32' bays. There are many advantages to precast panels. Hollow core planks offer room for mechanical, electrical and plumbing distribution. Even with the topping, the total depth of this system is only 14" making it a close third in overall depth. However, when you factor in the ability to run the mechanical, electrical and plumbing through the openings compared to running it below with the flat plate this becomes the second shallowest option.

As with the above systems, the extra weight will help with vibration while increasing foundation and lateral system sizes. Another advantage to this system is the quick installation on site. However, as with steel, a larger crane is needed to erect the panels and a longer lead time is required. A typical section is pictured below in Figure 5 as well as a typical bay in Figure 6.

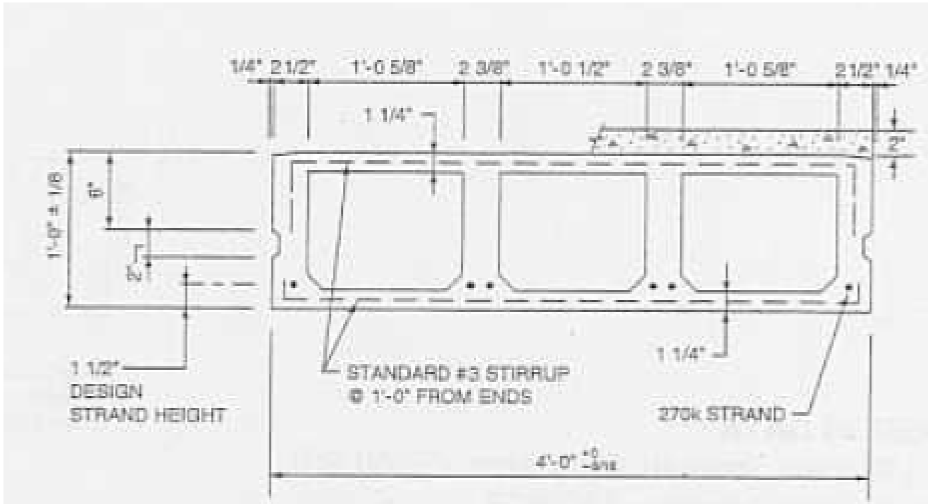


Figure 5: Prestressed Concrete Hollow Core Spandeck Section

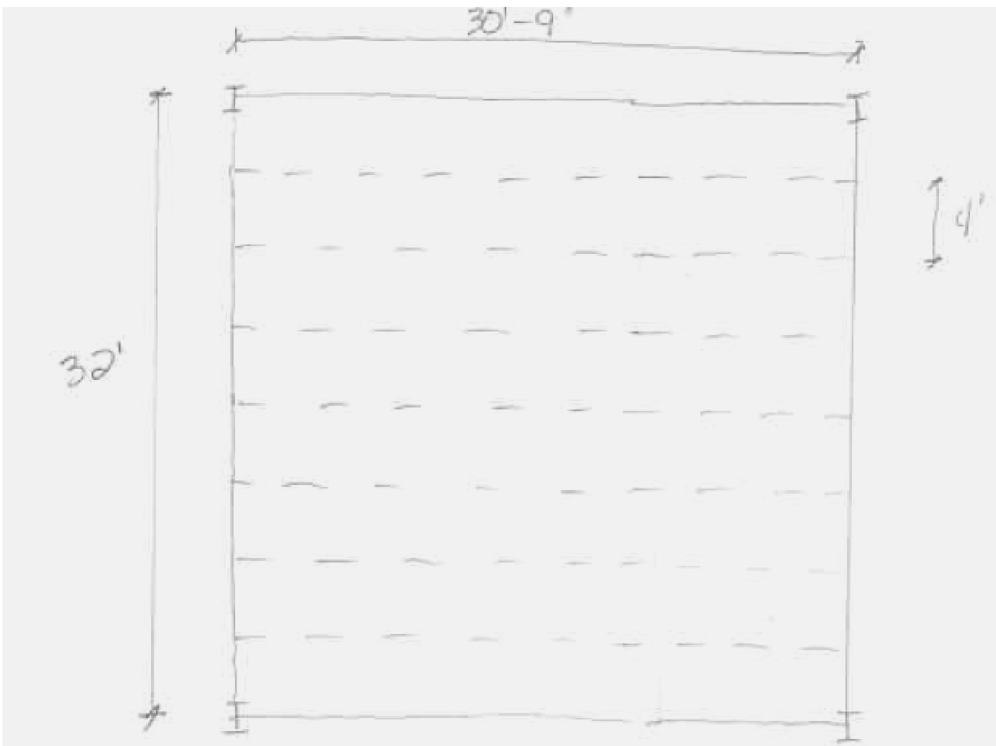


Figure 6: Hollow Core Spandeck Framing Plan

Alternate System 5: Precast Double-Tee Planks

Once again using Nitterhouse Concrete Products, a 32″x12′ double-tee with 2″ cast-in-place topping will sufficiently support the superimposed loads. Reinforcing consists of 0.6″ diameter, 270k, lo-relaxation strands. In picking plank members, it was again assumed the column placement could be altered. A typical bay is assumed to be 36′x46′-1½″. As with the hollow core planks supporting members could be girders or masonry bearing walls.

Altering the column placement in this way reduces the total number of columns used. This opens up the floor creating more useable space. The double-tee planks have the same advantages and disadvantages as the hollow core planks. However, the total depth is increased to 34″. This is deeper than the existing system of approximately 30″. The open floor plan offsets this slightly deeper ceiling cavity. As stated previously, there are no height restrictions on the Wilkins building; thus, a slight increase in overall building height will not be major deterrent. A typical section is exhibited below in Figure 7. A typical floor plan is shown below in Figure 8.

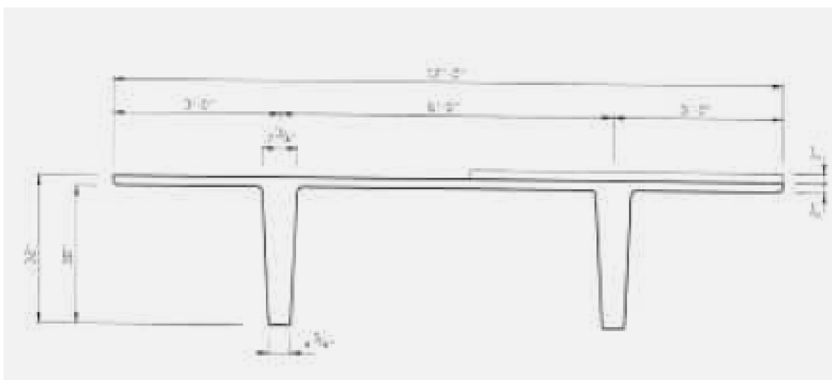


Figure 7: Prestressed Concrete Double Tee Section

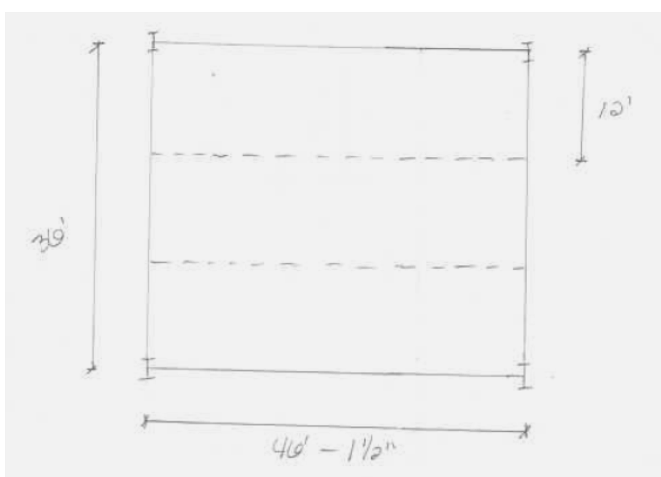


Figure 8: Double Tee Framing Plan

<u>Alternate System 6: Open Web Steel Joists</u>

One last system, open web steel joists, is evaluated here. Since the required fire rating is only one hour this is a viable option with the current slab. A typical framing plan is pictured below in Figure 9.

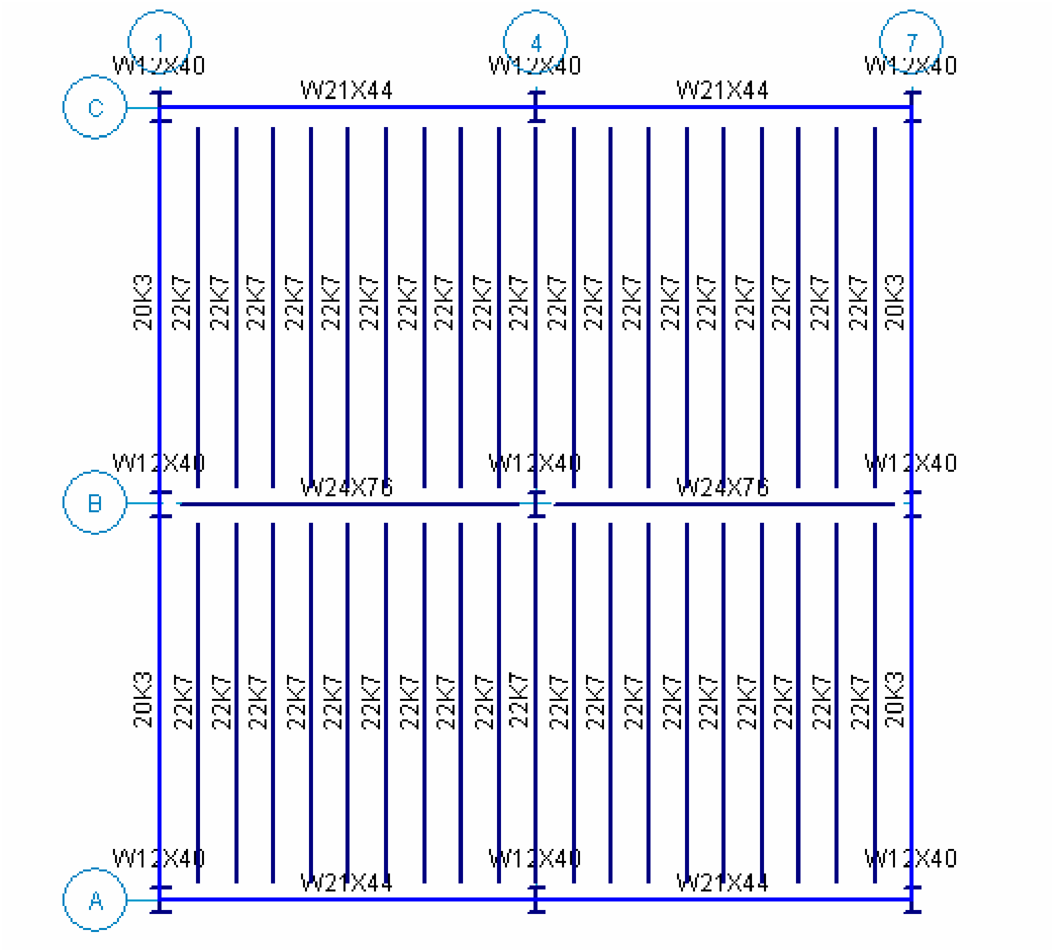


Figure 9: Open Web Steel Joist Framing Plan

As designed in RISAFloor, each 30’-9” span is split into 10 equal spaces resulting in a joist approximately every 3’. The weight of this system is equivalent to the existing composite system. Thus, seismic and foundation loads will not alter require a redesign of the lateral and foundation systems. Girder sizes are slightly larger as some strength is lost in going non-composite. There is plenty of space in the joist openings for mechanical, electrical and plumbing.

Since this is a lighter system, there may vibration problems that are not observed with the other alternatives. A slightly longer lead time is going to be required compared to some of the concrete systems.

Comparison

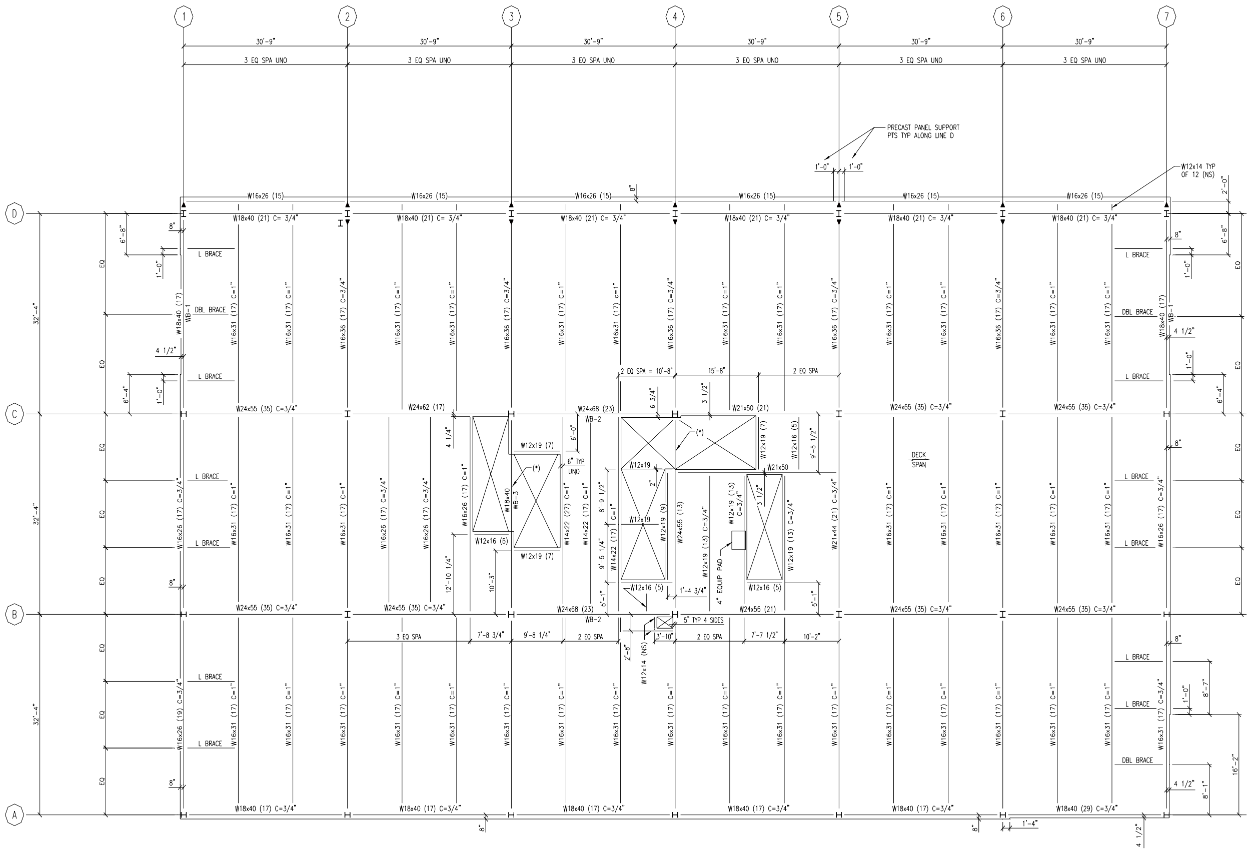
	Composite Steel Beams	One-Way Slab	Flat Plate	Post-Tensioned	Hollow Core	Double-Tee	Open Web Joists
Weight(psf)	59	93	163	113	102.5	83	58
Depth(in.)	29.5	34.5	13	9	14	34	29.5
Vibration	Maybe	No	No	No	No	No	Maybe
Constructability	Medium	Medium	Medium	Hard	Easy	Easy	Medium
Long Lead	Yes	No	No	No	Yes	Yes	Some
Formwork	No	Yes	Yes	Yes	No	No	No
Fire Rating	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Foundation Redesign	XXX	Yes	Yes	Yes	Yes	Probably	No
Viable Alternative	XXX	No	Yes	Yes	Yes	Yes	No

Conclusions

After investigating six floor systems in addition to the existing system several conclusions can be made. All of these systems are possible alternatives; however, some make more sense than do others. I feel open web steel joists can be ruled out. There are no real advantages to this system over the existing one. The equivalent weight makes for less work in a redesign. Nevertheless, construction process alone makes this option less appealing. Installing a member every 3’ will unnecessarily increase labor costs. The next least viable alternative would be the one-way slab with beams. There is an increase in weight and cost associated with this option. It does not offer any real advantage over the existing system. The other alternatives will most likely require a redesign of the foundation and lateral systems currently in place. However, the advantages of these other systems outweigh this disadvantage.

Appendix

A1. Typical Floor Plan



A2. One-Way Slab with Concrete Beams

Say clear span = 9.5' ⇒ b_w = 9"

$t_{min} = \frac{l}{28} = \frac{9.5(12)}{28} = 4.1 \approx 4.5"$

$w_D = \frac{4.5}{12}(150) + 21 = 77\,psf$

$w_u = 1.2(77) + 1.6(50) = 173\,psf$

Critical moment will be at exterior face of 1st interior support

$M_u = \frac{w_u l_n^2}{10}$

$= \frac{173(9.5)^2}{10} = 1.56^{'k}$

$\rho_{\phi=0.9} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.005}$

$= 0.85(0.80)\left(\frac{3500}{60000}\right)\frac{0.003}{0.008} = 0.0158 = 0.016$

$R = 800$

$M_u = \phi M_n = \phi R b d^2 \Rightarrow bd^2 = \frac{M_u}{\phi R} = \frac{1.56(12)}{0.9(800)} = 26$

$b = 12"$

$d = 1.5" < 4.5 - 1 = 3.5"$

use 3.5"

$A_s = \frac{M_u}{\phi f_y (d - a/2)}$

try $a = 1.0 \Rightarrow A_s = \frac{1.56(12)}{0.9(60)(3.5-0.5)} = 0.12$

$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{0.12(60)}{0.85(3.5)(12)} = 0.2$

$A_s = 0.102$

$a = 0.17$ ok

$A_{s,min} = \frac{200bd}{f_y} = 0.12$

$V_u = 1.15\left(\frac{173(9.5)}{2}\right) - 173\left(\frac{35}{72}\right) = 895 \text{ lbs}$

$\phi V_c = \phi 2\sqrt{f'_c}\, bd$

$= 0.75(2)\sqrt{3500}\,(12)(3.5)$
$= 3727 \text{ lbs}$

OK

Use #4 @ 18" o.c.

Design Beams

$w_u = 173 psf$

$b \le \frac{\ell}{4} = \frac{32.33(12)}{4} = 97"$

$\le \text{spacing} = 10.25(12) = 123"$

$\le b_w + 16h_f = 9 + 16(4.5) = 81"$ ← controls

typ. beam : pos $M_u = \frac{w_u \ell n^2}{16} = \frac{173(10.25)(32.33-1)^2}{16} = 108.6^{k}$

neg. $M_u = \frac{w_u \ell n^2}{11} = \frac{173(10.25)(32.33-1)^2}{11} = 158^{k}$

Design for 158^{k}

$b = 9"$
try $h = 16"$ ⇒ $d = 12.5$

Assume $a = h_f = 4.5"$

$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{158\ (12)}{0.9(60)(13.5 - 4.5/2)} = 3.12$

$A_{s\,min} = \frac{200}{f_y} b_w d = \frac{200}{60{,}000}(9)(13.5) = 0.41$ OK ← controlling case for $f'_c = 3500$ psi

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.12(60)}{0.85(3.5)(9)} = 7 > 4.5$ so in web

$A_{sf} = \frac{0.85 f'_c (b - b_w) h_f}{f_y} = \frac{0.85(3.5)(81-9)(4.5)}{60} = 16$

try $b = 12$
$h = 20$
$d = 17.5$

$A_s = \frac{158(12)}{0.9(60)(17.5 - 4.5/2)} = 2.29$

$a = 3.85 < 4.5$ ok

$A_s = 2.25$ ok
$a = 3.78$

add in self wt. $\frac{150}{144}(12)(15.5) = 0.19\ k/ft\ (1.2) = 0.23$

neg. $M_u = \frac{[1.73(10.25) + 0.23](31.33)^2}{11} = 178^{k\text{-}ft}$

$A_s = \frac{178(12)}{0.9(60)(17.5 - 4.5/2)} = 2.60$

$a = 4.37 < 4.5$ ok

$A_s = 2.58$

$A_{s\,min} = 0.7$ ok

Check tension controlled : $\frac{a}{\beta_1} = \frac{4.3}{0.85} = \frac{5.06}{d_t} = 0.326 < 0.375$ ok

Use (6) #6 $A_s = 2.64$

$b_{min} = 6(0.75) + 5(0.75) + 3 + 6/8 = 12"$ ok

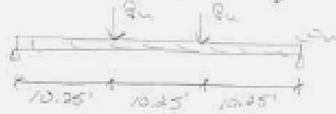
Beam: 12"x20" w/ (6) #6

Correction to above b_{min}, spacing between bars is 1″ thus place in two layers.

Design Girder

$w_u = 1.2(77)(1.5) + 1.6(50)(1.5) = 258.6$ plf

$P_u = \cancel{\phi}\left(\frac{1963(31.33)}{2\,\cancel{0}}\right) = 61.5^k$



$$M_u = \frac{0.259(29.75)^2}{8} + 61.5(10.25) = 659^{1k}$$

Same as Slab $\rho = 0.016 \Rightarrow R = 802$

$$bd^2 = \frac{M_u}{\phi R} = \frac{659}{0.9(802)} = 10956$$

$b \approx .5d$
$b = 17 \Rightarrow 18$
$d = 27$
$h = 30 \Rightarrow d = 27.5$

Self wt $= \frac{18(30)(150)}{144} = 0.56$ k/ft
$M_u = 721^{1k}$
$b = 17.5$ so ok w/ 18"

Assume $a = 7$

$A_s = 6.68$
$a = 7.48$
$A_s = 6.74$

try (2) #9 & (4) #10 $A_s = 7.08$

$a = 7.93$

$\phi M_n = 0.9(60)(7.08)(27.5 - 7.93/2) = 753^{1k} > 721^{1k}$ ok

$b_{min} = 2(2) + 4(1.27) + 3 + 4/8 + 5(1.27) = 17.18" < 18"$ ok

$A_{s,min} = \frac{200}{60000}(27.5)(18) = 1.65$ ok

Girder : 18" x 30" w/ (2) #9 & (4) #10

A3. Two-Way Concrete Flat plate

$\frac{l_2}{l_1} = \frac{30.75}{32.33} < 2$ ✓

$l_1 \geq \frac{2}{3}(30.75)$ ✓

- no column offsets, 3 continuous spans

$w_L \leq 2w_D$ ✓

Ok to use direct design method

$t_{min} = \frac{l_n}{33}$ int. $\frac{l_n}{30}$ ext.

$l_n = 32.33 - 1' = 31.33'$

$\frac{31.33(12)}{30} = 12.53 \cong 13"$ for slab w/ no ext. beams

$w_D = \frac{13}{12}(150) + 26 = 188.5\ psf$

$w_u = 1.2(188.5) + 1.6(50) = 306\ psf$

$M_{os} = \frac{wl^2}{8} = \frac{306(32.33)(30.75-1)^2}{8} = 1094.5^{\prime k}$

$M_{oL} = \frac{306(30.75)(32.33-1)^2}{8} = 1154.5^{\prime k}$

long span reinf. closer to top & bottom $\Rightarrow d = 13 - 1.5 = 11.5"$

Assume #6 bars

short span $d = 11.5 - 1 = 10.5"$

$b_s = 184.5$
$b_L = 194$

$A_{s,min} = 0.002bt$

$= 0.002(184.5)(13) = 4.8$

$= 0.002(194)(13) = 5.04$

Column Strip:

Long Direction

	End Span			Interior Span	
	End Neg.	Pos.	Int. Neg.	Pos.	Int. Neg.
Mu (ft.-k)	300.00	358.00	612.00	242.00	566.00
b (in.)	194.00	194.00	194.00	194.00	194.00
d (in.)	11.50	11.50	11.50	11.50	11.50
Mu/.9 (ft.-k)	333.33	397.78	680.00	268.89	628.89
R (psi)	155.91	186.05	318.05	125.76	294.14
ρ	0.00270	0.00319	0.00557	0.00214	0.00518
As (in^2)	6.02	7.12	12.43	4.77	11.56
Asmin (in2)	5.04	5.04	5.04	5.04	5.04
N	13.69	16.17	28.24	11.45	26.26
Nmin	7.46	7.46	7.46	7.46	7.46
Use	14.00	17.00	29.00	12.00	27.00

Short Direction:

	End Span			Interior Span	
	End Neg.	Pos.	Int. Neg.	Pos.	Int. Neg.
Mu (ft.-k)	285.00	339.00	580.00	230.00	536.00
b (in.)	184.50	184.50	184.50	184.50	184.50
d (in.)	10.50	10.50	10.50	10.50	10.50
Mu/.9 (ft.-k)	316.67	376.67	644.44	255.56	595.56
R (psi)	186.81	222.21	380.18	150.76	351.34
ρ	0.00320	0.00382	0.00673	0.00257	0.00618
As (in^2)	6.20	7.40	13.04	4.98	11.97
Asmin (in2)	4.80	4.80	4.80	4.80	4.80
N	14.09	16.82	29.63	11.32	27.21
Nmin	7.10	7.10	7.10	7.10	7.10
Use	15.00	17.00	30.00	12.00	28.00

Middle Strip:

Long Direction:

	End Span			Interior Span	
	End Neg.	Pos.	Int. Neg.	Pos.	Int. Neg.
Mu (ft.-k)	0.00	242.00	196.00	162.00	185.00
b (in.)	194.00	194.00	194.00	194.00	194.00
d (in.)	11.50	11.50	11.50	11.50	11.50
Mu/.9 (ft.-k)	0.00	268.89	217.78	180.00	205.56
R (psi)	0.00	125.76	101.86	84.19	96.14
ρ	Asmin con.	0.00214	Asmin con.	Asmin con.	Asmin con.
As (in^2)	Asmin con.	4.77	Asmin con.	Asmin con.	Asmin con.
Asmin (in2)	5.04	5.04	5.04	5.04	5.04
N	11.45	11.45	11.45	11.45	11.45
Nmin	7.46	7.46	7.46	7.46	7.46
Use	12.00	12.00	12.00	12.00	12.00

Short Direction:

	End Span			Interior Span	
	End Neg.	Pos.	Int. Neg.	Pos.	Int. Neg.
Mu (ft.-k)	0.00	230.00	186.00	153.00	175.00
b (in.)	184.50	184.50	184.50	184.50	184.50
d (in.)	10.50	10.50	10.50	10.50	10.50
Mu/.9 (ft.-k)	0.00	255.56	206.67	170.00	194.44
R (psi)	0.00	150.76	121.92	100.29	114.71
ρ	Asmin con.	0.00257	0.00207	Asmin con.	Asmin con.
As (in^2)	Asmin con.	4.98	4.01	Asmin con.	Asmin con.
Asmin (in2)	4.80	4.80	4.80	4.80	4.80
N	10.91	11.32	10.91	10.91	10.91
Nmin	7.10	7.10	7.10	7.10	7.10
Use	11.00	12.00	11.00	11.00	11.00

A4. Post-Tensioned Concrete Slab

Span 32'-4"

use span/depth ratio of 45

$$t = \frac{32.33(12)}{45} = 8.62 \approx 9\text{" slab}$$

Load to be balanced

$$\frac{9(150)}{12} = 112.5\,psf$$

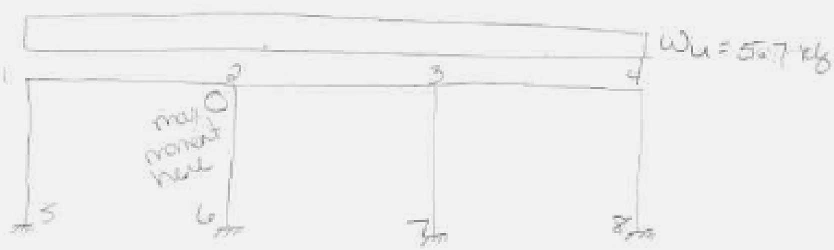
superimposed DL = 21 psf

LL = 50 psf for typical bay

Total = 183.5 ≈ 184 psf ↓

$w_{pe} = 0.9(112.5) = 101.25\,psf$ ↑

$w_{net} = 184 - 101.25 \approx 83\,psf$ ↓



$184(30.75) = 5.7\,k/ft$

$$\frac{wl^2}{12} = \frac{5.7(32.33)^2}{12} = 496^{'k}$$

Say Col. 16"x16"

$$\frac{C_{N1}}{l} = \frac{16}{30.75(12)} = 0.04$$

$$\frac{C_{N2}}{l_2} = \frac{16}{32.33(12)} = 0.04$$

$$I_s = \frac{30.75(48)(9)^3}{12} = 22417\text{ in}^4$$

at column $\Rightarrow \frac{1}{(1-c/l_2)^3} I_s = \frac{1}{(1-16/30.75(12))^3}(22417) = 24500\text{ in}^4$

$$k = \frac{4.03\,EI}{12 l_1} = \frac{4.03(22417)}{12(30.75)} E_c = 245 E_c$$

$CO = 0.502$

$H = 13.33$

$H_c = 12.58$

$H / H_c = 1.06$

$k_{ab} = 4.52$

$$k_c = \frac{kEI}{H} = \frac{4.52 E_c}{12(13.33)}\left(\frac{16^4}{12}\right) = 154 E_c$$

$$k_t = \frac{9 E_c C}{l_2 (1 - c/l_2)^3}$$

$$C = \left(1 - 0.63\left(\frac{9}{16}\right)\right)\frac{(9)^3(16)}{3} = 2510$$

$$k_t = \frac{9(2510)E_c}{369(0.958)^3} = 69.5 E_c$$

$$K_{ec} = \frac{\Sigma k_c \times \Sigma k_T}{\Sigma k_c + \Sigma k_T} = \frac{2(154) \times 2(69.5)}{2(154) + 2(69.5)} = 95.8 E_c \approx 96 E_c$$

Joint	1	2	
Member	1-2	2-1	2-3
DF	0.61	0.38	0.38
COF	0.502	0.502	0.502
FEM	-1000	1000	-1000
Dist	+610		
CO		+306	
Dist		-116	-116
CO	-58		
Dist	+35		
CO		+18	
Dist		-7	-7
CO	-3.5		
Dist	+2		
CO		+1	

Final Moments

2-1 = 1202

Real moment $= \frac{496}{1000}(1202)$

$= 596$

$$f_t = \frac{M}{S} - \frac{P_e}{A} - \frac{P_e e}{S}$$

$$P_e \geq \left(\frac{M}{S} - F_t\right)\left(\frac{e}{S} + \frac{1}{A}\right)^{-1}$$

$$F_t = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 0.53\ k/in^2$$

$$S = \frac{30.75(12)(9)^2}{6} = 4980\ in^3$$

$$A = 30.75(12)(9) = 277\ in^2$$

$$e = 2$$

$$P_e = \left(\frac{596(12)}{4980} - 0.53\right)\left(\frac{2}{4980} + \frac{1}{277}\right)^{-1}$$

$$= 226$$

$= 12$ strands @ $20\ ^k$/strand

$$f_b = -\frac{M}{S} - \frac{P_e}{A} + \frac{P_e e}{S}$$

$$= -\frac{596(12)}{4980} - \frac{226}{277} + \frac{226(2)}{4980}$$

$$= -2.16\ k/in^2$$

A5. Precast Hollow Core Planks

Assuming column locations can be changed look at precast concrete panels

LL ⇒ 80 corridors
50 offices
100 lobbies   psf

Superimposed DL = 21 psf

1hr fire rating

4' hollow core planks

alter bays to be 32' × 30'9"

for corridor area in typ. bay (controls general area)

$w = 80 + 21 = 101$ psf

12"x4' spandeck w/2" topping strand pattern 6 will work.

Lobby area ⇒ 121 psf

12"x4' spandeck w/2" topping strand pattern 6 will work

A6. Precast Double-Tee Planks

32" x 12' Double Tee

2" topping
max span Library area: 52' => 32-14.6DT
Corridor area: 70' => 32-18.6DT

3" topping
max span Library: 52' => 32-14.6DT
Corridor: 68' => 32-18.6DT

If change typ. bay to 36' x 46'-1½"

32"x12" Double Tee w/2" C.I.P. Topping

use 32-16.6DT for gc. area
32-8.6DT for Corridor areas
32-12.6DT for Library

Lobby areas will need to be specially designed due to loads and openings.