

# Hampton Inn & Suites – National Harbor, MD



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**John Pillar**

Dr. Memari Thesis Advisor

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# Executive Summary

## Systems Overview

Hollow Core Plank (precast)  
Solid Flat Slab (precast)  
Post-Tensioned Concrete  
Composite Steel  
Open Web Steel Joists  
Existing System

## Intent

The purpose of this report is to investigate the feasibility of using other floor systems than the existing system. A list of advantages and disadvantages follow each analysis, and a final comparison is made at the end. Considerations include constructability, cost, weight, complexity, and depth, among others.

## Conclusions

The existing floor system proved to be the best solution. A 10-1/2" concrete slab with 2-1/2" drop panels at each column location provides unique advantages in this case that are unable to be matched closely by the other systems. Because the column spacing is irregular, framing members would be complicated and tedious. The formwork associated with pouring the concrete slab and drop panels is one significant disadvantage, but the poured concrete flows into any shape, beam or bay size.

A significant height disadvantage is associated with steel members as well. For typical bays analyzed in this building, minimum member depths reach nearly 30", where the maximum concrete depth does not exceed 13". With flat slabs, there is a lot of flexibility with mechanical ductwork and electrical conduit as opposed to steel.

All of these factors point to the existing system as the most economical, efficient and sensible floor system to implement for the Hampton Inn & Suites in National Harbor.

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## Overview

The Hampton Inn & Suites in National Harbor, Maryland, is an eleven story concrete flat slab structure with drop panels at each column. Columns are 12"x24" and the concrete strength is mostly 5000 psi above the third floor. The loading schedules provided show the typical floor loads to be 40 psf for guest rooms (with an additional 20 psf for partitions), 100 psf for hallways, and 125 psf for storage areas. To analyze the building, the bay between column lines D and F, 6 and 10 is selected. In this bay the loading is only for guests which means 40 psf plus an additional 20 psf for partition allowances.

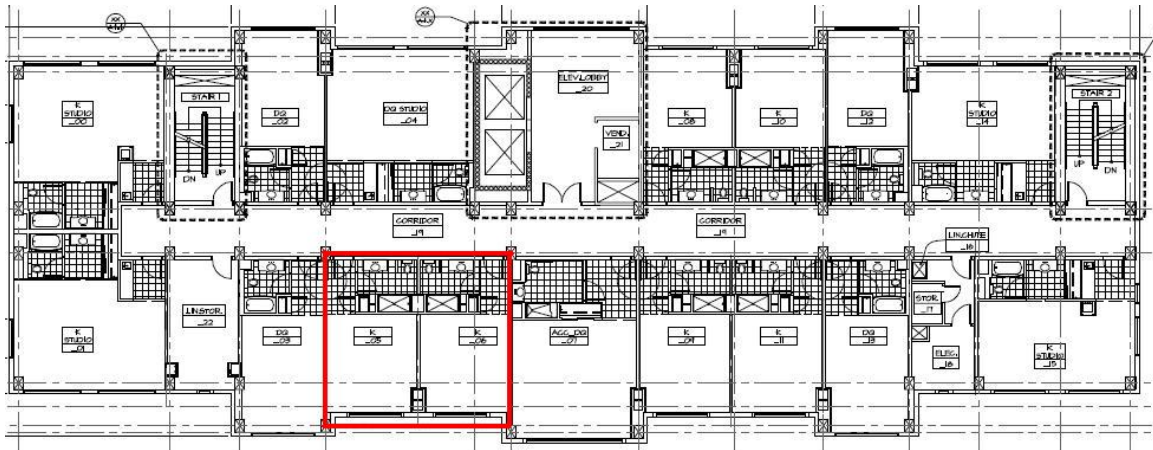
## Existing Conditions

### Floor Slabs

The floor slabs are usually 10-1/2" thick when not near columns. At each column there is a 2-1/2" drop panel to combine for a 13" slab thickness. A typical drop panel size is 5'-6"x6'-9" and accounts for 38 square feet. Steel reinforcing is laid out longitudinally and transversely on both the bottom and top. The slab reinforcing ranges from number 4 bars to number 6 bars spaced approximately 12 inches apart. Where not specified, number 5 bars spaced at 6" is the minimum required.

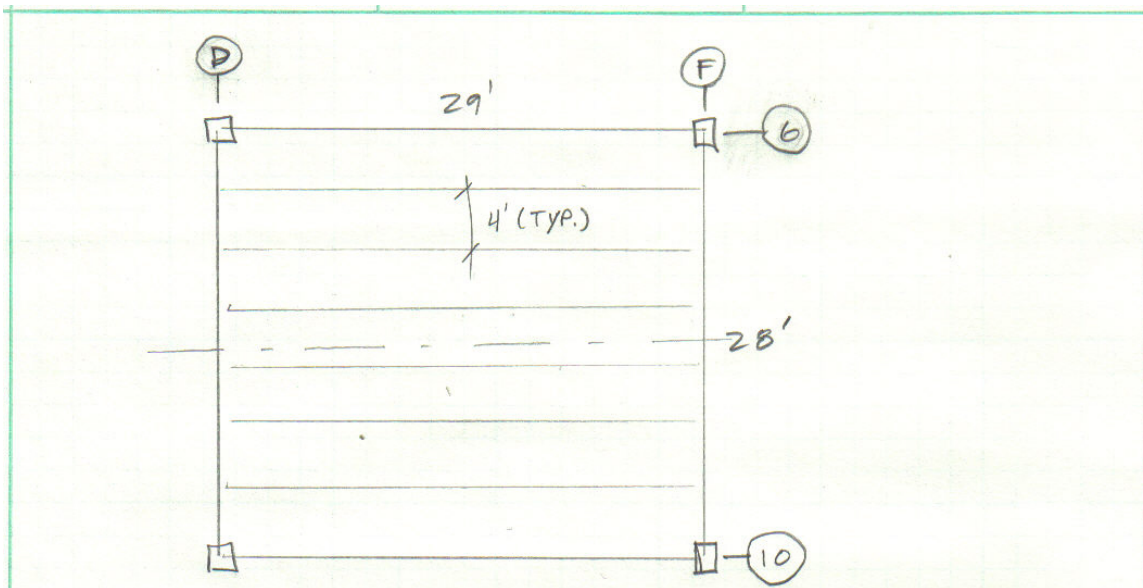
For slabs on level 3 and below, concrete strength is normal weight 6000 psi. Slabs resting on the fourth floor and up have a strength of 5000 psi. Minimum reinforcing protection for floor slabs is 3/4".

The slabs on this project are considered to act as two way slabs, meaning that they carry load in both lateral directions. The three largest bays have dimensions of 29'x26'-10". There are no beams spanning between columns in this case. In the largest bay, the drop panels cover roughly 6 feet of the span, or 20.7



## Alternate 1: Hollow Core Plank (precast)

In order to use precast shapes, the typical bays of a building should have some sort of regularity or repetition. Highly irregular column spacings, or non-orthogonal column lines dictate special attention. In this case, the bay has been modified by moving column D9 south one foot (onto column line 10) in order to make a 29'x28' bay. This is due to the fact that the planks come in increments of 4', which divides into 28.



From page 2-31 of the PCI Design Handbook 6<sup>th</sup> Edition, using a superimposed load of 40 (live) + 20 (partition) + 8 (misc) = 68, and a span of 29', products 4HC6 and 4HC8 are designed to work. Because the 4HC6 shape is only 6" thick vs. 8" for the 4HC8, the 6" plank is adequate for a preliminary size. It uses eight 7/16" 270k strands which are threaded straight. The precast piece is slightly cambered at 0.4" upon placement, but settles 0.7" upon time resulting in a deflection of  $L/497$ .

### Advantages

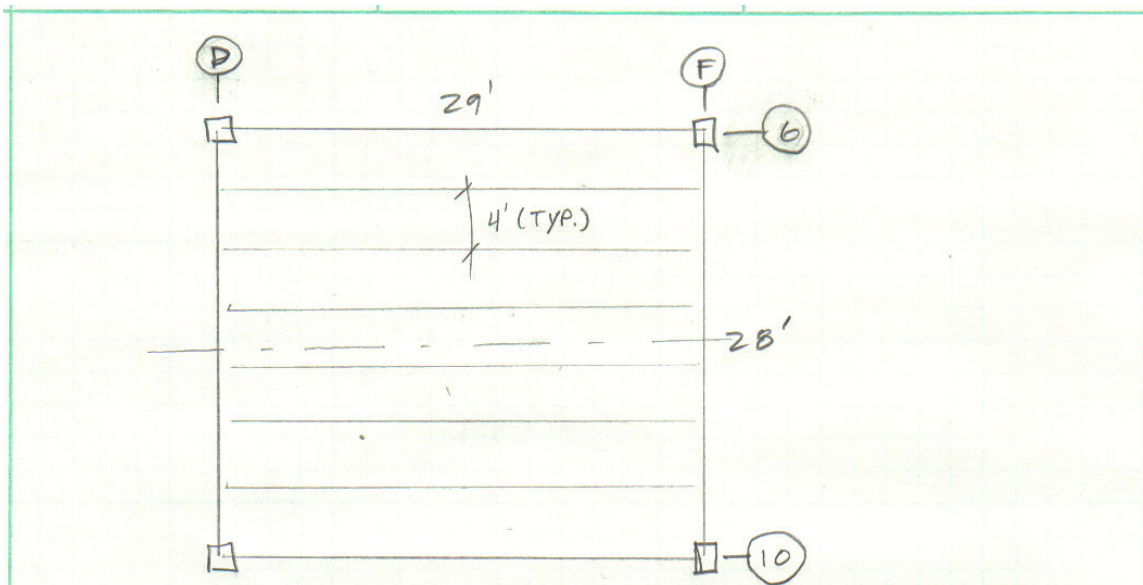
Precast planks have already been analyzed for shear, bending, and serviceability. This means that much less time is spent designing the performance of the floor system. They come directly from the manufacturer, often at strengths very close to the designed 28 day strength. Longer spans are achievable thanks to high strength tension strands, and a lighter section because it is hollow. A shallow floor section increases the amount of flexibility for mechanical and electrical conduit throughout the building. Also, no formwork is required to place the plank.

### Disadvantages

As mentioned previously, orthogonal and repetitive bays are where hollow core precast planks really shine. If the building, for some reason, does not have much redundancy, precast can become expensive. The building should be designed around the floor system to ensure maximum efficiency, and there isn't much flexibility of column spacings. Due to the high tension strands located within the plank, cutting holes in the floor system should be run by the engineer or plank manufacturer to avoid cutting the strands. Because the planks are hollow, increased vibration and sound transmission may occur.

### **Alternate 2: Flat Slab (precast)**

Also available from the PCI Design Handbook 6<sup>th</sup> Edition are flat slab design tables. Using the same bay as before, as well as the same loading, brings one to the FS8 assembly on page 2-41, as designated by PCI. The system is 8" thick with six 1/2" 270k tendons, which are threaded straight. A long term deflection of 0.3" results in a ratio of L/1160.



### Advantages

Because it is precast, the shapes again come in strips of 4'. The depth of the floor system is again a plus. It is deeper than the hollow core plank, though, and that is attributed to fewer and smaller tension strands. Most of the other

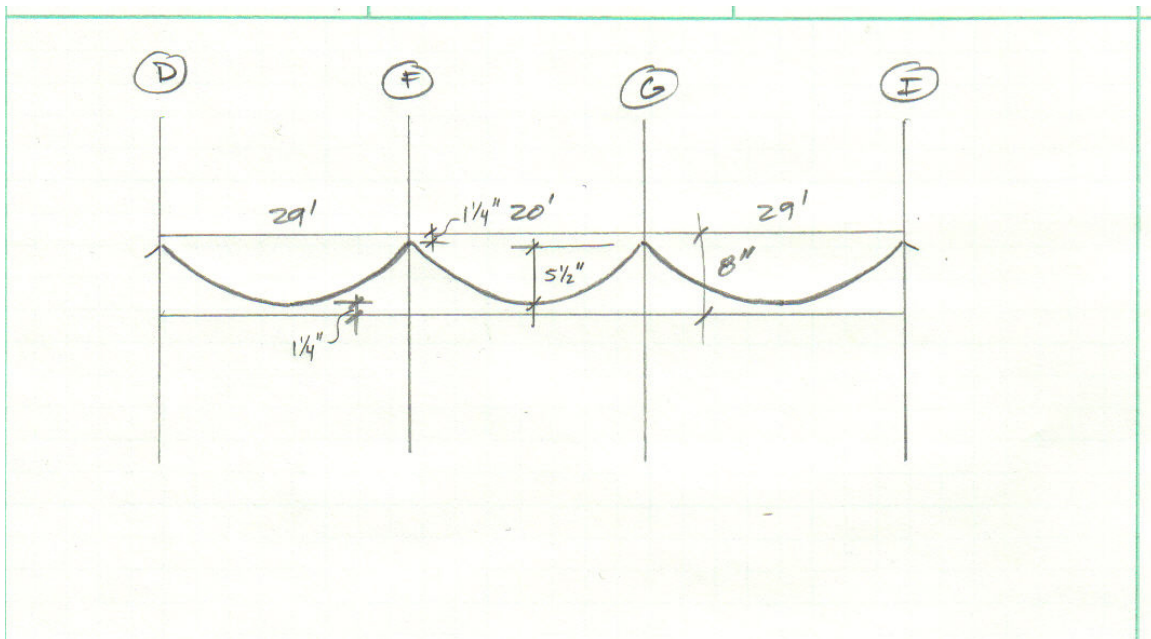
advantages are similar to precast hollow core plank members. A denser section may result in less vibration and sound transmission than hollow core plank.

### Disadvantages

Again, the building column lines must be essentially tailored to fit the precast shapes. Cutting holes in the slab result in the same risk as the hollow core plank system discussed earlier. Also, because the section is not hollow, heavier slabs will result from the same spans.

### **Alternate 3: Post-Tensioned Concrete**

Following examples from the Post-Tensioned Concrete Design Workbook, a preliminary system was developed. The slab thickness was determined to be 8" using a rule-of-thumb ratio of span to depth. Assuming the tendons are draped a total of 5-1/2" (1-1/4" cover on top and bottom for a two hour fire rating), the floor system is adequate to support the applied loads of 60 psf with 14 1/2" 270k tendons.



### Advantages

Post-tensioning has some nice features versus pre-tensioned sections as well as regularly reinforced sections. The engineer can easily control the geometry of the slab's tendons in post-tensioned concrete as opposed to pre-tensioned slabs. This enables the floor system to be designed for the application, instead of the

other way around. The high strength steel cables shift the bending diagram towards the compression side to enable the concrete to span longer distances than regular reinforcing. Camber can easily be introduced and controlled in a way that best suits the application.

#### Disadvantages

Post-tensioned concrete must be done so in a controlled manner. The procedure takes longer to complete than regular concrete placement because the concrete must cure before stress can be put into the tendons. Formwork is necessary to place the concrete, which means more construction costs and labor. Because the strands vary according to bay geometry, it is not always easy to locate the tendons when cutting holes in the slab. Unless bays are similar, each bay must be designed by the engineer ahead of time.

### **Alternate 4: Composite Steel**

A more conventional floor system for some large buildings is composite steel. In this case, the beams were run the long direction and the girders the short direction. Three spans in 28' meant that each beam saw 9'-4" of tributary width to support the floor. From the United Steel Deck catalog, a 20 gage 3" Lok-Floor deck (5-1/2" total slab depth) can span 9.35' at only one unbraced span. Because the loading in guest rooms was rather low, these beams were sized at a W14x22 with full composite action, achieved through 20 shear studs at 17.2k per stud. Deflection calculations were carried out and the beams were found to be satisfactory. The girder was sized in a similar fashion, and required a W18x40 shape with 32 shear studs.

#### Advantages

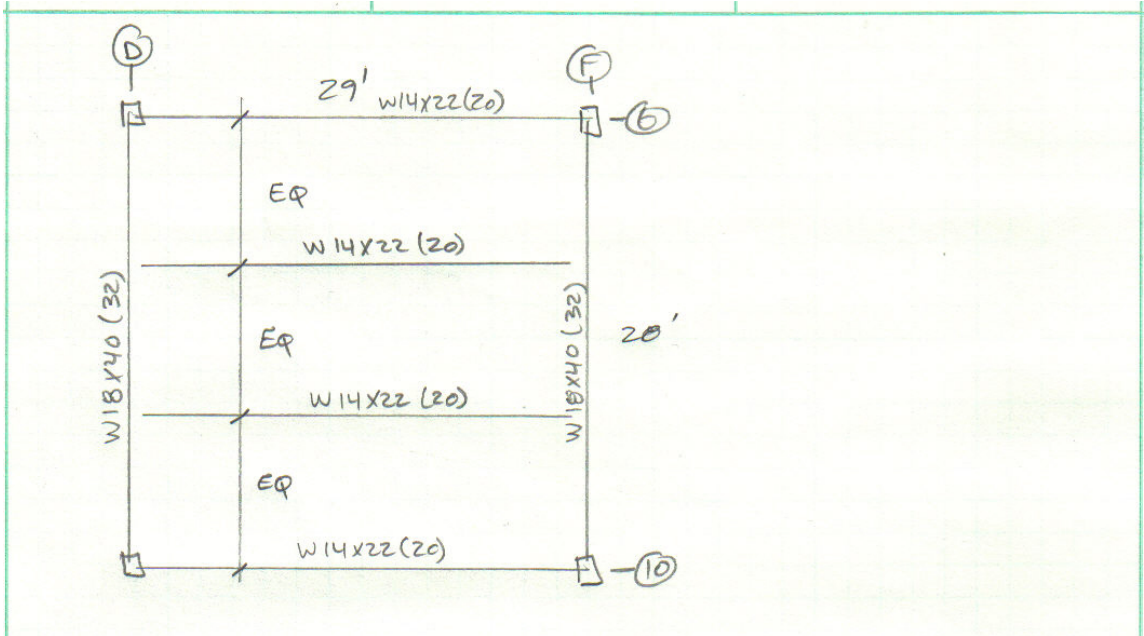
Because steel is so light compared to other floor systems, it is often the system of choice. With full composite action, the member depth can be reduced significantly. No shoring is required with steel as opposed to concrete, and it is easy and fast to erect. Since the seismic response is directly dependent on building mass, lighter skeletons could result in better seismic performance.

#### Disadvantages

Due to the fact that the market can fluctuate so dramatically, using steel can sometimes be a gamble. There is always a bit of uncertainty because of long lead times and mill tolerances. For longer spans, the depth of the member becomes a big issue. Engineer or contractor error could result in a higher potential risk than other systems such as two-way slab because there is not as

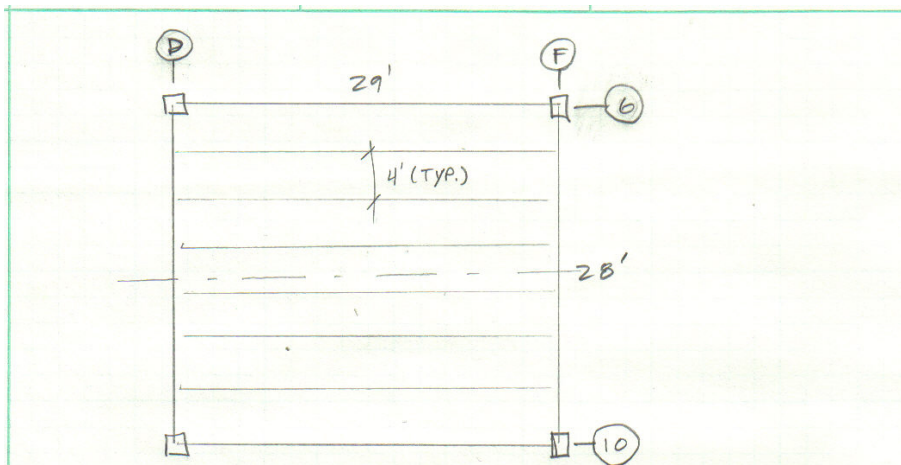


much redundancy, or redistribution of loads, available. Vibration within the floor system could arise because steel is more flexible and elastic than concrete.



### Alternate 5: Open Web Steel Joists

Possibly the lightest steel floor system available, open web steel joists dynamically show off the athleticism of steel when compared to concrete. From The New Columbia Joist Company joist catalog, an 18K9 joist is sufficient to carry the loads in this case at 4' spacing. Serviceability is already factored in, as the joist adheres to a limit of  $L/360$  deflection. To achieve a 2 hour fire rating, 2-1/2" of concrete must be above the deck. From the United Steel Deck catalog, the 3" Lok-Floor deck is again selected (total slab thickness of 5-1/2").



### Advantages

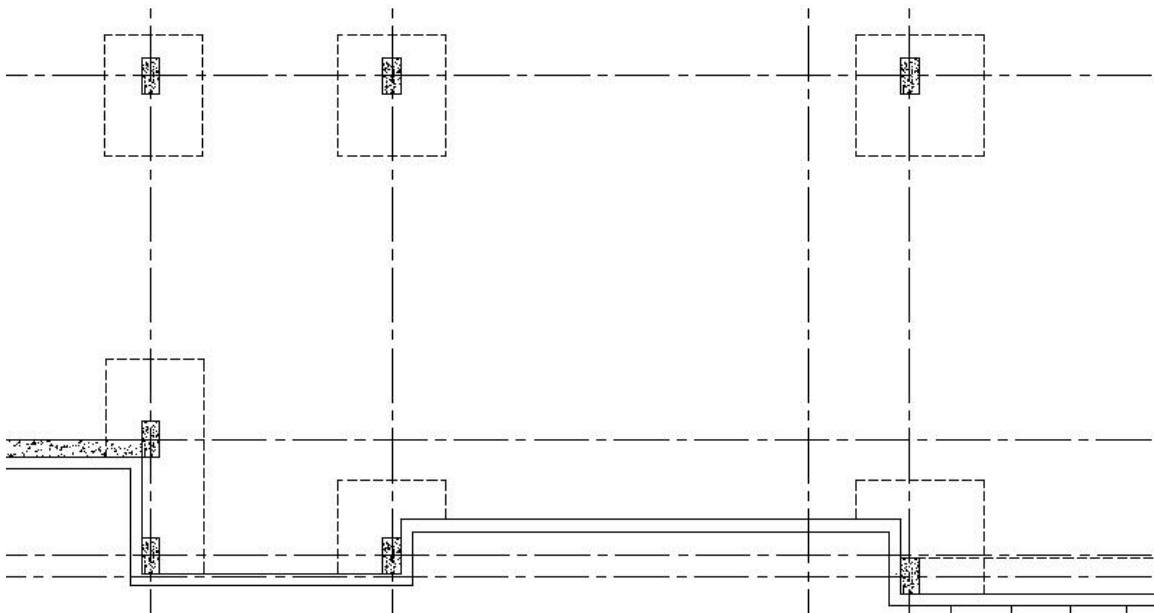
Because of the open webs, plenum space is found everywhere. There is very much flexibility with mechanical systems as well as electrical systems in terms of duct work and conduit. Seismic performance should be greater than concrete systems due to the significant weight reduction.

### Disadvantages

As with composite steel beams, vibration could play a major role in the performance of the structure. Because the joists are not as rigid as beams, they must be placed closer together. Moment connections are extremely expensive and impractical to make. Most of the time, joists are only designed to take load one way. Performance could alter drastically if they were to be loaded improperly or in anyway unforeseen.

### **Existing: Cast-in-Place Two Way Concrete Slab w/ Drop Panels**

The existing system, consisting of 10-1/2" slab and 2-1/2" drop panels, is adaptable to a large variety of bay sizes. Drop panels ensure that shear is adequately designed for, and the slab is reduced in thickness where shear is not a concern.



### Advantages

Two-way concrete systems are extremely redundant. That is, they can redistribute loads very well under unforeseen loading situations. Floor sections are usually not very deep and there is plenty of space for ductwork. Smaller columns can be used at drop panel locations because there is a larger shear area. Construction is simple and relatively fast, with little interruptions.

### Disadvantages

Most of the dependence of this system relies on the correct placement and usage of reinforcing steel. Through contractor error and mazes of reinforcement, it is easy to lose track of which bars go where, or accidentally kick over a rebar chair. Also, concrete strength is not a guarantee. When being mixed in the field or on site, there is no guarantee that the concrete will be mixed exactly as specified.

## Systems Analysis

System-->	Existing	Hollow Plank	Flat Slab	Post-Tension	Composite	Open Web Joists
Bay	28'x29'	30'x30'	25'x25'	28'x29'	30'x30'	30'x30'
Loading	Actual	75 psf	75 psf	Actual	75 psf	65 psf
Depth	13" total	6"	9-1/2"	8"	2' 5-1/2"	2' 5"
Cost	13.35 / sq. ft	<b>8.41</b> / sq. ft	13.35 / sq. ft	~13 / sq. ft	<b>15.10</b> / sq. ft	<b>15</b> / sq. ft
Constructability	Moderate	Easy	Easy	Difficult	Moderate	Moderate
Vibration	Little	Possible	Little	Little	Possible	Probable
Applicable	Yes	Yes	Yes	Yes	Yes	Yes
Feasible	<b>Yes</b>	No	No	<b>Yes</b>	<b>Yes</b>	No

## Conclusion

The best overall floor system that should be used in this case is the existing system of cast in place concrete with drop panels. This was mainly because of the irregular column spacing and building bays. Even though RS Means estimated the hollow plank assembly to be \$8.41 per square foot, the actual cost would be higher without the square 30' bays. Clearly, the cost would go up at a rate equal to the irregularity of the bays. The same can be said for the precast

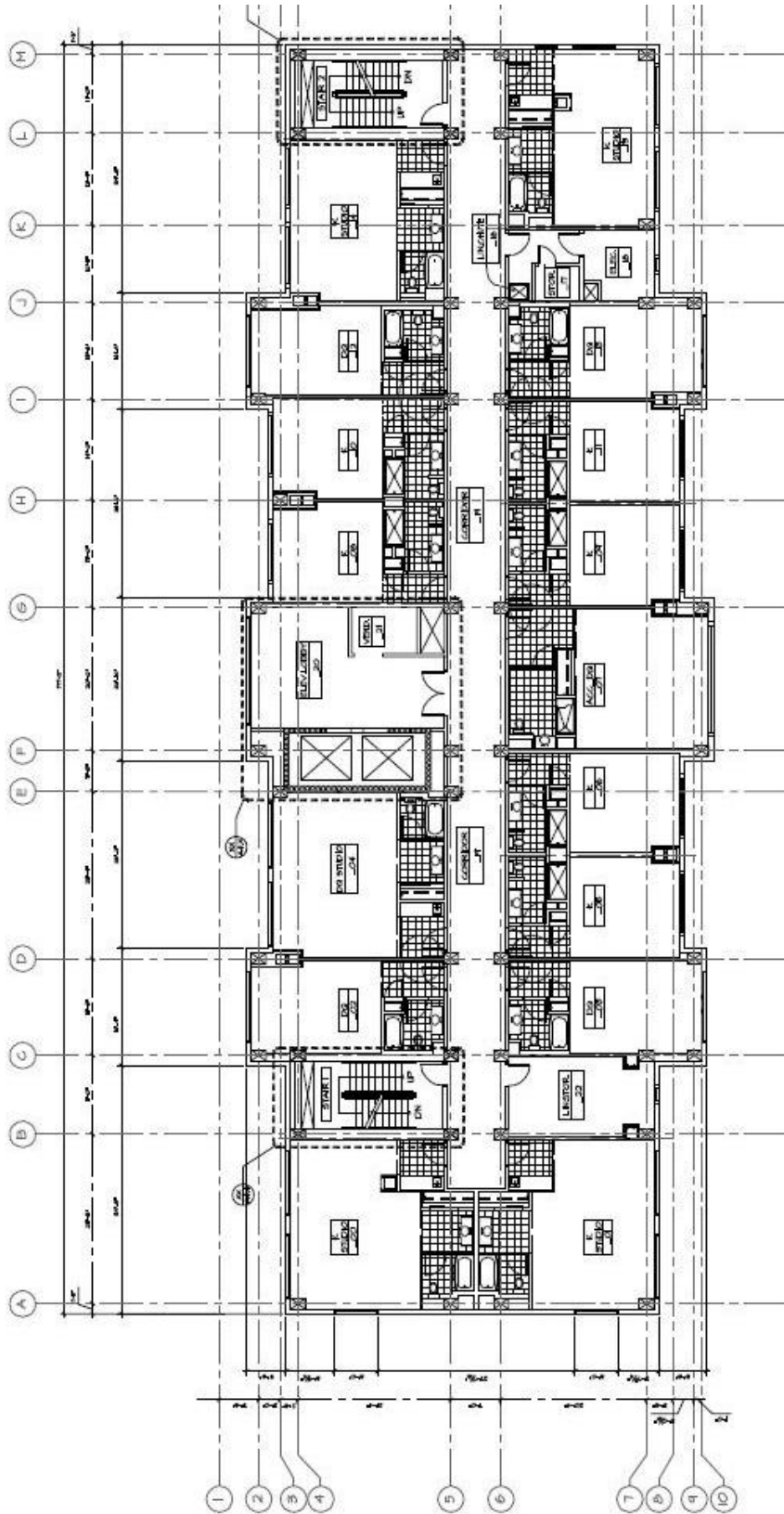
flab slab assembly. With cast in place concrete, bay size or variation does not matter nearly as much. There are no complex framing issues that arise. With hotels, the depth of the floor section is very important. If steel was used, for example, this would result in a 16" increase in deck thickness alone over the thickest concrete system. An eleven story structure could save around 15 feet in the height differential between the two systems. This could mean having a shorter building with increased seismic and wind performance, or adding another story altogether. An extra story of usable space would help offset the higher price for cast in place concrete. Fire protection is another good reason to use concrete here. Spray on fireproofing has to be applied to exposed steel w-shapes and open web joists to achieve a rating. Concrete has inherent fireproofing thanks to reinforcing cover requirements. The ease of construction is also a plus.

The most viable alternatives include post-tensioned slabs and composite steel. Post-tensioning would save additional height in the floor assembly, but it is more time consuming and complex than cast in place with standard reinforcing. The composite steel system would save weight but not height. This could result in a taller and less efficient building shape, but one that has seismic advantages that may yield smaller lateral members.

# Appendix A Calculations

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3RD THROUGH  
11TH FLOOR PLAN  
SCALE: 1/8" = 1'-0"

## Hollow core plank

### Guest room loading

40 psf live  
20 psf partition  
8 psf misc

68 psf      span = 29'

From PCI Design Handbook, 6<sup>th</sup> Ed, p. 2-31 & 2-32,

could use 4HC6 w/ 87-5 strands @ 75 psf  
4HC8 w/ 76-5 strands @ 75 psf

choose 4HC6 due to 6" thickness  
larger camber, however it is negligible at 0.4"

## Solid Flat slab

From PCI Design Handbook p. 2-41,

FS8 assembly @ 75 psf works  
68-5 strands @ 270k  
0.3" long term camber

Post-Tension

From Post-Tension concrete design workbook,

span/depth for continuous span = 45  $\therefore$  depth = 8"

$$8'' \text{ slab } \left(\frac{8}{12}\right)(150) = 100 \text{ lb/ft}^2$$

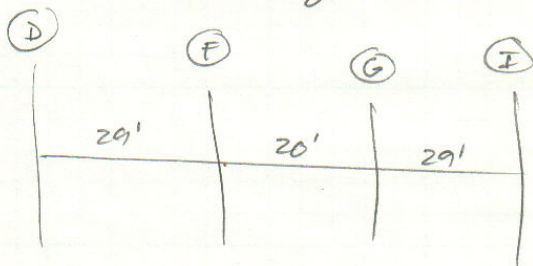
$$+ \frac{60 \text{ lb/ft}^2}{160 \text{ psf}}$$

$$W_{pre} = 0.9(100) = 90 \text{ upwards}$$

$$W_{net} = 160 - 90 = 70$$

For 2 hour cover, use 1" top & bottom

$$M_{pre} = \frac{90(29)^2}{8} = 9.46 \text{ ft}\cdot\text{k}$$

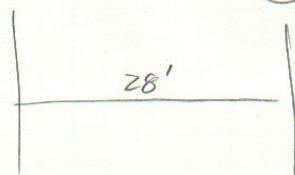


90	90	90	$W_{pre}$
9.46	4.5	9.46	$M_{pre}$
5.5	5.5	5.5	$a$
20.64	9.8	20.64	$F$

$$(20.64)(28) = 578 \text{ k long bay} \Rightarrow 2.14 \text{ in}^2 \text{ or } (14) \frac{1}{2}'' \phi \text{ tendons}$$

$$(9.8)(28) = 274.4 \text{ k short bay} \Rightarrow 1.06 \text{ in}^2 \text{ or } (7) \frac{1}{2}'' \phi \text{ tendons}$$

(6)



(10) Transverse Direction

$$M_{pre} = \frac{90(28)^2}{8} = 8.82 \text{ ft}\cdot\text{k}$$

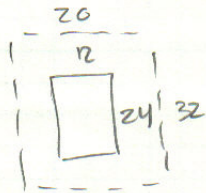
$$a = 4'' \quad F = 26.46 \text{ k}$$

$$(26.46)(29') = 767.3 \text{ k} \Rightarrow 2.84 \text{ in}^2$$

or (19)  $\frac{1}{2}'' \phi$  tendons



Shear @ columns



$$\frac{t}{2} = 4''$$

$$b' = 40 + 64 = 104''$$

$$d = 6.75''$$

$$V_c = 4 \sqrt{5000} (104) (6.75) = 178.5k$$

$$V_c = \left( \frac{30(6.75)}{104} + 2 \right) \sqrt{5000} (104) (6.75) = 195.9k \leftarrow \text{controls}$$

$$\phi V_c = 0.75(195.9) = 149k$$

$$\left( \frac{29}{2} + \frac{20}{2} \right) \left( \frac{28}{2} \right) = 343 \text{ ft}^2$$

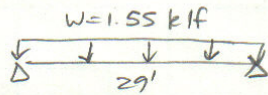
$$V_u = 1.2(100) + 1.6(60) = 216 \text{ lb/ft}^2 (343) = 74.1k < \phi V_c \therefore \text{OK}$$

Steel

Assume beams are 9'-4" o.c.

$$A_t = 270 \text{ ft}^2$$

From United Steel Deck, choose 3" Lok-Floor Deck w/ 2 1/2" top



$$p = 1.2(58) + 1.6(60) = 165.6 \text{ psf}$$

$$w = 165.6(9.33) = 1.55 \text{ k/f}$$

$$M_U = \frac{(1.55)(29)^2}{8} = 162.5 \text{ ft}\cdot\text{k}$$

assume  $a = 1"$ ,  $b_{eff} = 112"$

$$= \frac{29(12)}{4} = 87" \leftarrow \text{controls}$$

$$\lambda/2 = 5.5 - \frac{1}{2} = 5"$$

From LRFD Table 3-19, try W14x22 @ P.N.A T&F

$$\phi M_n = 209 > 162.5$$

$$\Sigma Q_n = 325 \text{ k}$$

$$a = \frac{325}{(0.85)(4000)(87)} = 1.098 > 1.0 \therefore \text{NG}$$

P.N.A @ B.F.L  $\Rightarrow \Sigma Q_n = 157$   $\phi M_n = 224 > 162.5$

$$a = \frac{157}{(0.85)(4000)(87)} = 0.53 \therefore \text{ok}$$

$$\lambda/2 = 5.5 - \frac{0.53}{2} = 5.235"$$

Assume 17.2k / stud

$$\frac{157}{17.2} \Rightarrow 9 \text{ studs / side}$$

Use W14x22 w/ 20 studs

$$\Delta_T = \frac{5(1.1)(29)^3(1728)}{384(29)(496)} = 1.21 < \frac{L}{240}$$

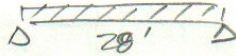
$$\Delta_L = \frac{5(0.56)(29)^3(1728)}{384(29)(496)} = 0.6 < \frac{L}{360}$$

$$\Delta_W = \frac{5(0.47)(29)^3(1728)}{384(29)(199)} = 1.29 < \frac{L}{240}$$

Deflection ok

$$\text{Girder } A_T = (29)(28) = 812 \text{ ft}^2$$

$$W = 165.6(29) = 4.8 \text{ k/ft}$$



$$N_U = \frac{4.8(28)^2}{8} = 470.4 \text{ ft} \cdot \text{k}$$

assume  $a = 1''$

$$b_{eff} = 348$$

$$\frac{28(12)}{4} = 84'' \leftarrow \text{controls}$$

$$y_2 = 5.5 - \frac{1}{2} = 5''$$

From LRFD Table, try W18X40

$$\phi M_n = 615 > 470.4 \quad \Sigma Q_N = 588 \text{ @ TOP PNA}$$

$$a = \frac{588}{.85(4000)(84)} = 2.05 > 1.0 \therefore \text{NG}$$

$$\text{PNA @ BFL} \quad \phi M_n = 491$$

$$\Sigma Q_N = 272$$

$$a = \frac{272}{.85(4000)(84)} = 0.95 < 1.0 \therefore \text{OK}$$

$$y_2 = 5.5 - \frac{0.95}{2} = 5.023$$

Assume 17.2 k/stud

$$\frac{272}{17.2} \Rightarrow 16 \text{ studs /side}$$

Use W18X40 w/ 32 studs

$$\Delta_T = \frac{5(3.2)(29)^4(1728)}{384(29)(1340)} = 1.31'' < \frac{l}{240}$$

$$\Delta_L = \frac{5(1.7)(29)^4(1728)}{384(29)(1340)} = 0.69'' < \frac{l}{360}$$

$$\Delta_W = \frac{5(1.46)(29)^4(1728)}{384(29)(612)} = 1.31 < \frac{l}{240}$$

Steel Joists

12K9 Joist @ 29'

254 lb. live load /ft meets 1/360

431 lb total /ft

$$d = 10''$$

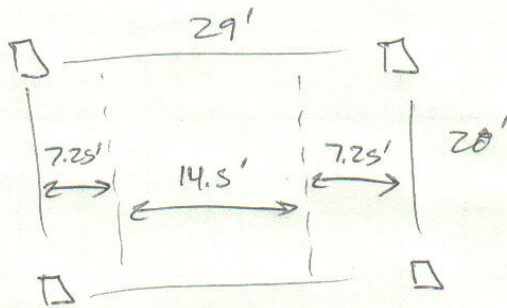
$$w = 10.2 \text{ lb/ft}$$

For 2 hour rating, exposed grid

2 1/2" conc. above deck

48" max spacing

## 2-way slab analysis



12x24 columns

$$f_c = 5 \text{ ksi}$$

$$f_y = 60 \text{ ksi}$$

Assume ~~10~~ 8" thickness

$$LL = 40 \text{ psf}$$

$$PL = 20 \text{ psf}$$

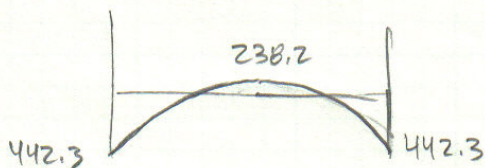
$$150(8/12) = 100 \text{ psf DL}$$

$$w = 1.2(100) + 1.6(80) = 248 \text{ lb/ft}^2$$

$$M_o = \frac{(1.25)(20)(20)^2}{8} = 680.5 \text{ ft}\cdot\text{k}$$

$$M_o^- = 442.3 \text{ ft}\cdot\text{k}$$

$$M_o^+ = 238.2 \text{ ft}\cdot\text{k}$$



+ Moment

60% to column strip

$$0.6(238) = 143 \text{ CS}$$

$$95 \text{ MS}$$

- Moment

75% to column strip

$$0.75(442.3) = 331.7 \text{ CS}$$

$$110.6 \text{ MS}$$

Negative Moment

Positive Moment

$$\frac{331.7}{14.5} = 22.9 \text{ CS}$$

$$\frac{143}{14.5} = 9.9 \text{ CS}$$

$$\frac{110.6}{14.5} = 7.6 \text{ MS}$$

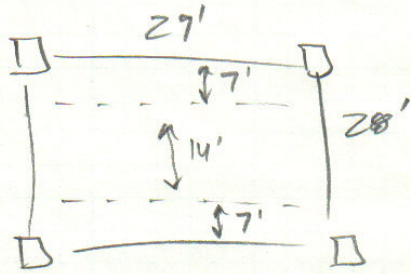
$$\frac{95}{14.5} = 6.55 \text{ MS}$$

MS

CS

MS			CS	
-M	+M		-M	+M
110.6	95	M <sub>u</sub>	331.7	143
174	174	b	174	174
6.9375	6.9375	d	6.9375	6.9375
123	105.6	M <sub>n</sub>	368.6	159
176.3	151.3	R	528	227.8
0.003	0.00255	P	0.00943	0.00387
3.621	3.07	A <sub>s</sub>	11.38	4.67
2.78	2.78	A <sub>smin</sub>	2.78	2.78
11.7	19.9	N	36.7	15.06
10.9	10.9	N <sub>min</sub>	10.9	10.9
11.7	10.9	N <sub>req</sub>	36.7	15.06
Use #5 @ 12"	Use #5 @ 16"		Use #5 @ 4"	Use #5 @ 10"

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$$M_o = \frac{(0.25)(29)(27)^2}{8} = 660 \text{ ft}\cdot\text{k}$$

$$M_o^- = 430 \text{ ft}\cdot\text{k}$$

$$M_o^+ = 230 \text{ ft}\cdot\text{k}$$

Positive

$$230(.6) = 138 \text{ CS}$$

$$92 \text{ MS}$$

Negative

$$430(.75) = 322.5 \text{ CS}$$

$$107.5 \text{ MS}$$

MS			CS	
-M	+M		-M	+M
107.5	92	M <sub>v</sub>	322.5	138
168	168	b	168	168
6.9375	6.9375	d	6.9375	6.9375
119.4	102.2	M <sub>n</sub>	358.3	153.3
177.2	151.7	R	531.8	227.5
0.0031	0.00251	P	0.00945	0.00378
3.61	2.93	A <sub>s</sub>	11.05	4.41
2.7	2.7	A <sub>smin</sub>	2.7	2.7
11.6	9.45	N	35.6	14.2
10.5	10.5	N <sub>min</sub>	10.5	10.5
11.6	10.5	N <sub>req</sub>	35.6	14.2
#5@12"	#5@16"		#5@4"	#5@12"

