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Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

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Rachel Chicchi

Structural Option

Consultant: Dr. Thomas E. Boothby

October 24th, 2008

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TABLE OF CONTENTS

Rachel Chicchi
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IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

EXECUTIVE SUMMARY

The purpose of this report is to investigate flooring systems other than the two-way, flat plate slab system used in the IAC/InterActiveCorp Headquarters. Three alternative systems were studied:

1. Composite Steel
2. Two-Way Post-Tensioned Slab
3. Hollow Core Plank

In order to perform these analyses, an interior bay was selected from the fifth floor, which is the level just below the transfer slab. Figure 1, to the right, shows this bay. Because the bays within the IAC building are not uniform, a larger, somewhat typical frame was selected and simplified for ease with hand calculations. For instance, when computing sizes for the

(Full structural floor plan disclosed at owner's request)

Figure 1: 5th Level, displaying the analyzed bay

various floor systems, the bay was treated as though it was orthogonally shaped, rather than skewed. Despite this simplification, when designing each system, efforts were made to preserve the layout and material design assumptions of the initial system.

Sizes were determined for each of the systems through hand calculations. Deflection, stress limits, moment and shear capacity, and fireproofing dictated much of the design of the systems in order to comply with ACI 318-08 and the 13th edition of AISC for LRFD. After design was completed, comparisons were made with regard to key issues, such as cost, weight, depth, constructability, etc. It was determined that a post-tensioned system would be most ideal because of its versatility and reduced weight and depth. Composite steel would also be a viable option to consider. Precast hollow-core, on the other hand, would not be feasible because it requires a uniform layout which cannot be achieved with the IAC Headquarters without compromising the architecture.
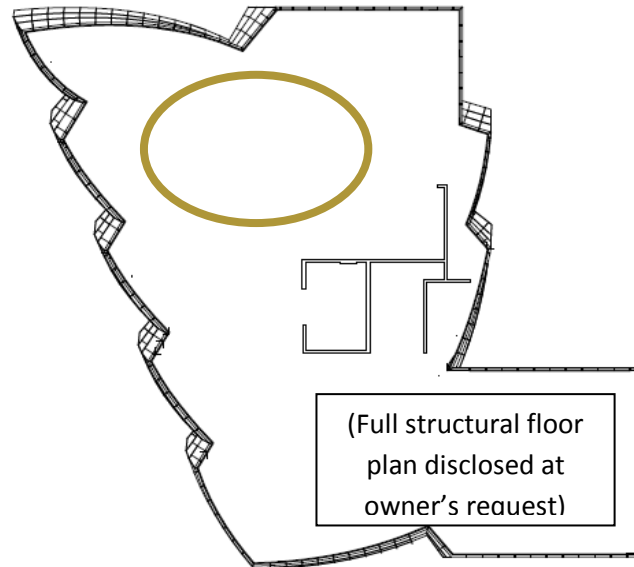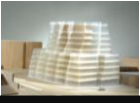
Rachel Chicchi
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IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

INTRODUCTION

Located along the Hudson River in the Chelsea neighborhood of Manhattan and outboard of the original Manhattan shoreline, the IAC/InterActiveCorp Headquarters stands out along the New York City skyline. This 11-story office building's unique design, which represents a ship at full sail, is credited to Gehry Partners of Los Angeles. This whimsical design became reality through the structural engineers on the project, DeSimone Consulting Engineers. Because of its unusual shape and gradual setbacks, it is no surprise that the structural system itself is not uniform. It is for this reason that the building was peer reviewed by the structural firm, Severud Associates, in order to verify the safety and practicality of the structural system.

The office was designed as an open-office layout and, therefore, it is difficult to determine exactly why columns were placed in seemingly random locations. Throughout this technical report and future research, it is important to determine whether columns can feasibly be rearranged within the grid to become more uniform. Yet, despite the strange configuration of columns, there does appear to be inherent grid lines, shown below in Figure 2, which may ultimately be used if redesigning the floor system for the entire building.

Another unusual aspect is that the office spaces seem to have been designed using a 60 psf live load, without accounting for the additional 20 psf that is typical for partitions. For the purposes of this report, a live load of only 60 psf was used so that a more accurate comparison could be made between the existing and proposed systems. In future analyses, an 80 psf live load will be used.
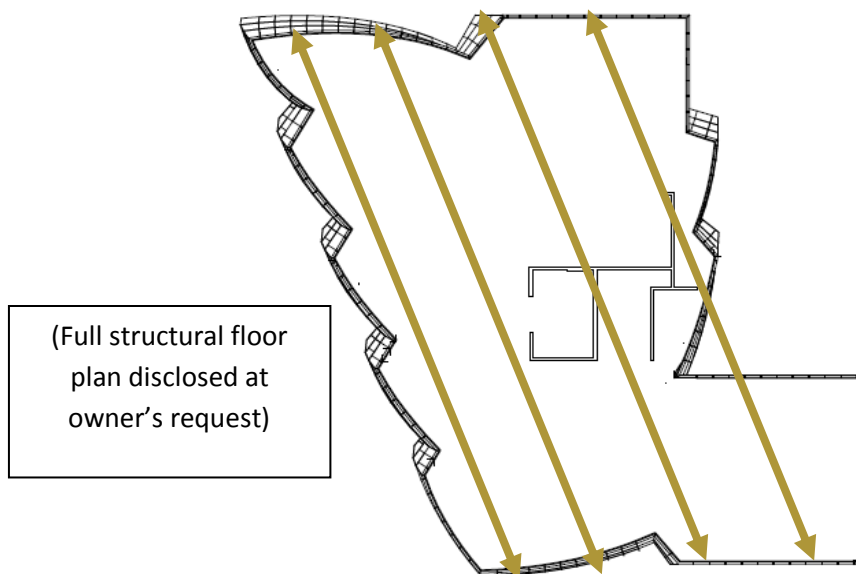

(Full structural floor plan disclosed at owner's request)

Figure 2: 5th Level, showing inherent column grid lines

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Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

EXISTING SYSTEM: FLAT PLATE TWO-WAY SLAB
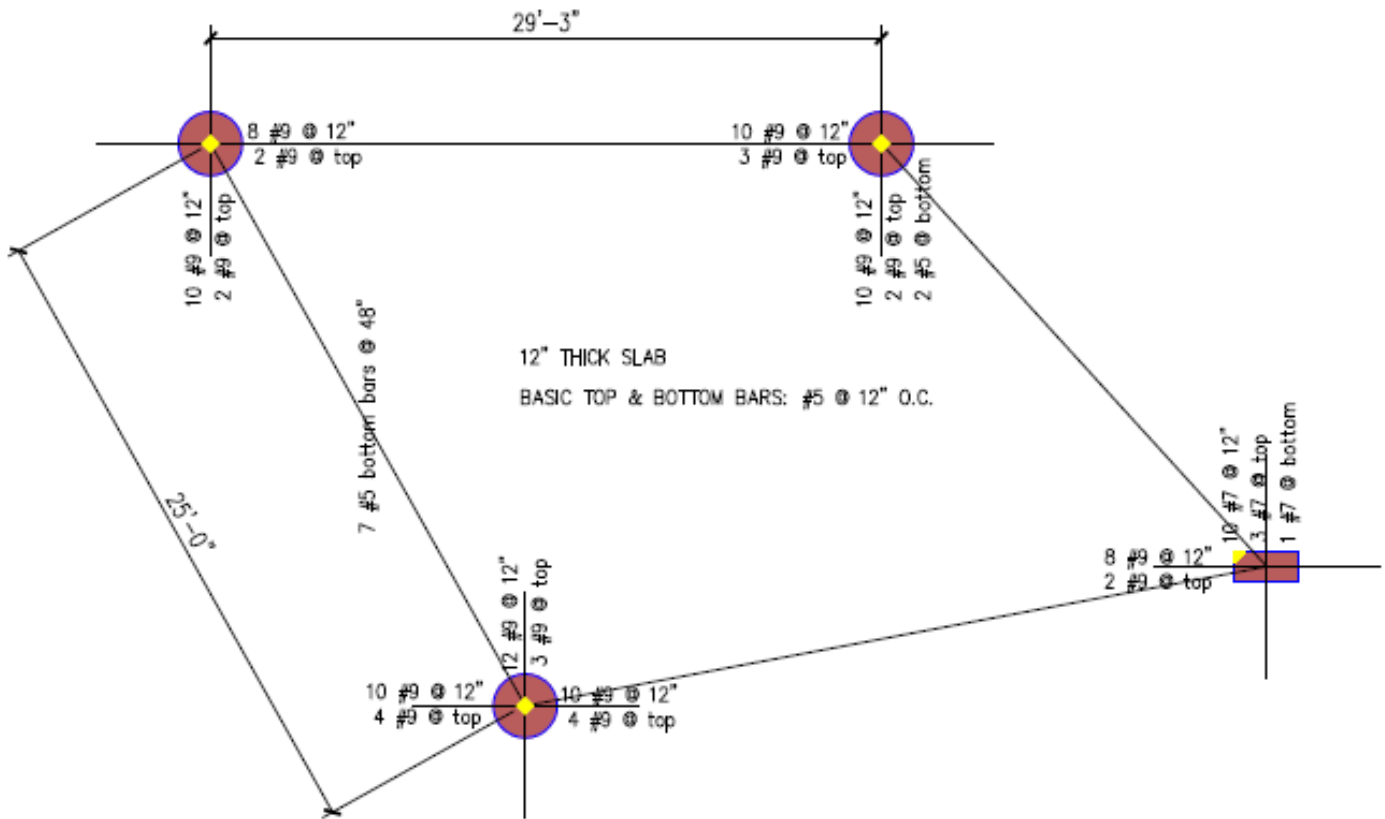
DESIGN CRITERIA

Slab Thickness= 12"	
f 'c= 5000 psi	
Normal Weight Concrete	
fy=60 ksi	
Self Weight= 150 psf	
Superimposed DL= 20 psf	
Live Load= 60 psf	
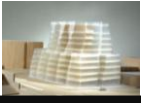
ADVANTAGES	DISADVANTAGES
- Exposed flat ceilings – good for coordination of trades	- Low stiffness (notable deflection)
- Locations of columns is relatively flexible	- Vulnerable to punching shear
- Low cost formwork	- Low shear capacity
- Fast/ Easy to form	

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IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

Floor System

The structure of IAC/InterActiveCorp Headquarters is a cast-in-place two-way concrete flat plate system. This type of system is primarily used in residential construction because it allows for ease of coordination between trades. More importantly, however, it allows the designer to place columns with relative ease in locations that would optimize the interior space. Despite the advantages of a flat plate system, it is, nevertheless, fairly unusual that this commercial building was designed by this method.

The slab thickness for the first through fifth floors is 12" with primarily #5 @ 12" o.c. top and bottom bars in the 5000 psi strength concrete. Additional top and bottom rebar is placed at the columns and midspans of the room where necessary. At the sixth floor, where the building is set back (leaving space for an outdoor terrace), the slab thickness is 24". The concrete strength at that level is 5000 psi as well, but the top and bottom reinforcing bars are typically #7 @ 12" o.c. It is at this location that the column layout changes much more radically. This thicker slab acts as a transfer diaphragm, which, in addition to supporting vertical live, dead, and snow loads, transfers lateral forces. Lateral forces, such as wind and seismic, can be transferred through the slab. Additionally, where columns are no longer stacked on top of each other, the slab must act as a transfer to carry loads from the upper columns to the lower ones. The seventh through roof levels have similar slab properties to the first through fifth floors, except that the upper floors have a slab thickness of 14". An unusual aspect of the slab reinforcing details is that unlike typical American Concrete Institute standard details which involves rotating rebar to match specific edge angles, the structural designers chose to design the reinforcing steel in the north-south and east-west orthogonal directions. This was done in an effort to improve the constructability of the building by eliminating the necessity to rotate rebar in various directions because of the unusual edge shape. Through the use of additional top and bottom bars in necessary locations and the overall uniformity of the bar layout, it seems that orienting the bars orthogonally is a plausible solution.

Though the building is primarily concrete, some steel shapes are used throughout to add additional stability. Steel hollow structural sections (HSS 12x4x1/2) act as elevator rail support posts on the ground floor and S8x18.4 shapes are used for the same purpose on the upper levels. Hollow structural sections are also used on the 11th floor as bracing.

Gravity System:

While the IAC building has a fairly uniform design amongst floors, all of the structural floor plans differ slightly because of the gradual building setback, including a more noticeable setback at the sixth floor. In order to accommodate this setback and allow for columns to be placed in desirable locations, most of the columns in the building's superstructure are sloped,

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

making the building tend to twist counter-clockwise under its own weight. This causes significant torsional rotation, which needed to be taken into consideration during the initial design process. In fact, a number of short-term and long-term studies were made through three-dimensional computer simulations to design the lateral system and predict curtain wall displacements.

The columns in the basement are primarily 28" in diameter for the perimeter columns and 34" to 38" in diameter for the interior columns. This range of column diameters is fairly consistent throughout the ground through fifth floors, but at the sixth floor the sizes are reduced to 20" to 24". Columns are typically spaced between 25 and 30 feet apart and all are specified with a strength of 5950 psi. The reason for this unusual column strength is because buildings constructed in New York City with strengths greater than 6000 psi must undergo more frequent test cylinders; therefore, by specifying a strength just under 6000 psi, less tests would be necessary.

Figure 3: Flat plate system during construction of the IAC Headquarters

At the sixth floor, the building setbacks become more distinct and, therefore, the columns begin to slope much more significantly in an effort to keep the columns along the perimeter and out of the way of the open office space. In addition a number of columns are displaced at the sixth floor level, resulting in column offsets up to 8'-0" long.

Figure 3, shown to the right, effectively displays the coordination of the flat plate slab and the circular columns along the perimeter.

Figure 4: Typical shear wall layout (4th floor)

Lateral System

The columns carry the gravity loads while the shear walls, that encase the elevator and stair core, carry the lateral forces. These shear walls tend to be between 12" and 14" thick. They are reinforced by #4's at 12" in the vertical and horizontal directions. This core, with numerous shear walls acting in each direction, works together with the reinforced slab to carry wind and seismic lateral loads. The shear walls typically span from the cellar level up to the roof. Figure 4, to the left, shows the basic layout for shear walls. In addition

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

to this shear wall core, the slab acts as a diaphragm in order to help distribute lateral loads. This is necessary because the shear wall core is so concentrated and would likely be ineffective without the contribution of the slab to distribute loads across the entire floor plan.

Foundation System

There is one below-grade basement level in the IAC building with a slab thickness of 24 inches. It was designed as a pressure slab in order to resist hydraulic uplift forces. A 48" thick structural mat supports the building core. This core mat is primarily reinforced at the top and bottom by #9's and #11's at 6" on center. In order to oppose lateral forces from the soil, the foundation wall is 18" thick with #4 bars primarily as reinforcement. All of the concrete in the foundation is 5000 psi concrete.
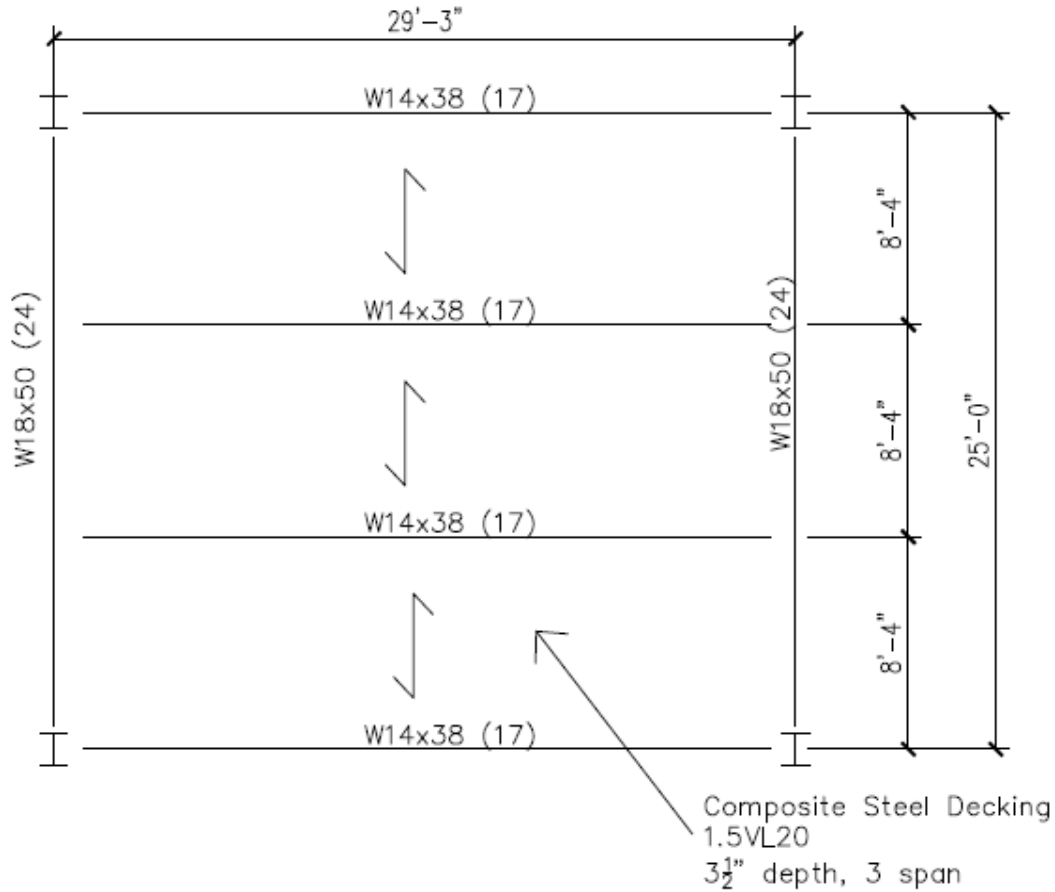
The gravity columns are supported on concrete-filled steel pipe piles (with a conical tip, as agreed upon with NYSDEC because of environmental sensitivity). These piles have a 175 ton capacity to provide the required axial capacity. There are also twenty-three 18" diameter caissons that end bear on the bedrock. Because the building is located below the 100-year flood elevation, much concern was taken with the waterproofing, as well as a hydraulic flood gate designed to seal the entrance ramp of the parking garage when needed. In addition, it was also contaminated from a ConEdison Manufactured Gas Plant facility previously on the site, so containment was very important.

Roof System

The roof is composed of 14" thick, 5000 psi concrete. Twenty-inch diameter columns support the roof along the perimeter, along with 14x14 inch posts intermittently positioned to support mechanical equipment. To provide additional reinforcement for the roof level, HSS 10X10X1/2" square tubes were used on the eleventh floor (mechanical mezzanine level) along the perimeter of the building. A fairly large window washing unit to service the entire building facade is located on the roof; however, information has not yet been found providing the unit's weight. A CMU wall and steel W-shapes are also used on the eleventh floor mechanical mezzanine level to support the mechanical equipment.

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

ALTERNATIVE #1: COMPOSITE STEEL

DESIGN CRITERIA
Slab Thickness= 3 ½"
Girder Depth= 18.0"
f′c= 5000 psi
Normal Weight Concrete
fy=60 ksi
Self Weight of slab= 54 psf
Superimposed DL= 20 psf
Live Load= 60 psf

ADVANTAGES	DISADVANTAGES
- 2-hour fire rating (with spray-on fireproofing)	- Heavy steel sections required
- Reduced weights/ shallower depths of members	- Steel beams add additional depth to system
- Basic, well-known form of construction (easy)	- Beams make coordination of trades more difficult
- Fast erection time	- Long lead time necessary
- Carries large live loads	

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

COMPOSITE STEEL DESIGN PROCESS

The sizing of members and the number of ¾" diameter shear studs, as labeled in the framing plan on the previous page, contribute to the composite action and were determined by hand calculations and referring to the 13th edition of AISC. In addition, the steel deck was designed using a deck catalog from Vulcraft Group, making sure that the deck chosen specified a 2-hour fire-rating. Beam and girder sizes were determined by taking into account deflection (complying with L/240 for total load and L/360 for live), as well as the moment and shear capacities of the members.

DESIGN CONSIDERATIONS

Structural:

Composite steel is a favorable framing system because it is fairly simple to construct and well-known.   In addition, from a structural design aspect, it combines the compressive strength of concrete with the tensile strength of steel, resulting in a system that is relatively light and shallow.

Vibration criteria was not evaluated for this report; however, further research may indicate that the beams may need to be deeper or the slab thicker to prevent noticeable vibrations.

Because the shear wall core of the existing IAC Headquarters is very concentrated, moment frames would likely be necessary in order to contribute to resisting lateral loads. This would add additional cost to the system; however, it seems necessary because the concentrated shear wall core would be unable to solely resist all of the lateral loads.

Architectural:

If a composite steel system is implemented in the IAC Headquarters, the columns would need to be changed to steel as well. Typical W-shape columns would be adequate. In addition, the column grid would need to be altered to make it more typical. Further research would be necessary to evaluate the feasibility of doing this; however, the composite system could work well if it was designed with basic changes to the 'inherent grid' mentioned previously and shown in the "Introduction" section of this report.

Construction:

While the slab is significantly thinner than other systems, the beams, add substantial depth to the system. Additionally, because the beams are exposed, they would likely need to be finished in order to be aesthetically pleasing to the clients. This not only adds additional costs, but makes it more difficult for the coordination of trades because beams and girders may interfere with mechanical ductwork and electrical conduit.

This system is relatively quick to construct because it does not require additional labor to install and remove formwork. Despite this advantage, the shapes must be rolled at the mill and transported to the site, so the lead time is longer than cast-in-place systems.

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

ALTERNATIVE #2: POST-TENSIONED TWO-WAY SLAB

29'-3"

(12) #4 @ top

(12) #4 @ top

(9) #6 @ top

(10) 1/2", 7-wire banded strands

(26) 1/2", 7-wire uniform strands

25'-0"

(9) #6 @ top

(12) #4 @ top

(12) #4 @ top

8" thick, 5000 psi concrete
Normal Weight
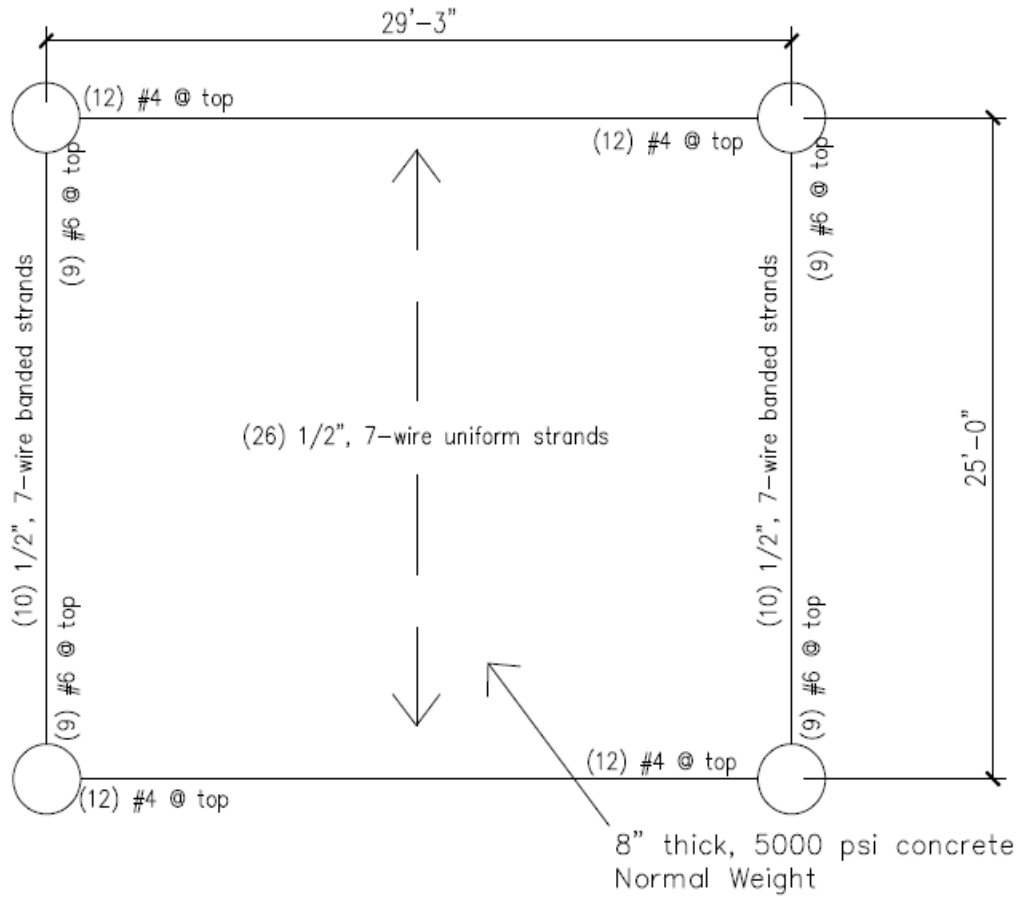
DESIGN CRITERIA
Slab Thickness= 8"
f 'c= 5000 psi
Normal Weight Concrete
Self Weight of slab= 100 psf
Superimposed DL= 20 psf
Live Load= 60 psf

ADVANTAGES	DISADVANTAGES
- Reduced floor depth (8")	- Formwork required
- Crack control and water-tightness	- Laying of tendons is labor intensive
- Deflection/vibration control	- Extra safety procedures required on the job site
- Large area slabs can be maintained without control joints	- Laborers in NYC not experienced with post-tensioned systems
- Longer spans possible	
- Increased speed of construction	
- Easy coordination of trades	
- Flexible design	
- 2-hour fire rating	
- Reduced mild-steel reinforcement	

POST-TENSIONED SLAB DESIGN

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

PROCESS

Hand calculations were conducted in order to design the system shown on the previous page. Moments were determined using the Direct Design Method and design criteria, such as stress limits, were checked according to ACI 318-08.

This system would implement banded ½" diameter strands along the columns in the North-South direction and uniform strands in the East-West direction. The reason for this decision was because the North-South direction is more linear, allowing the banded strand to run the length of the building more effectively than if it had to be bent back and forth through random placement of columns.

DESIGN CONSIDERATIONS

Structural:

While neither deflection nor vibrations were calculated for this system due to its complexity, post-tensioned systems are known to perform well under deflection. In addition, the slab thickness was determined based on the L/H=45 rule of thumb, concluding deflection should not be a problem in this case. With effective deflection and crack control and the liberty to place columns in various places in the plan without repercussion, the post-tensioned system appears to effectively control many of the issues that initially dictated the design of the IAC Headquarters.

In office buildings with a two-way, flat plate, post-tensioned system with spans between 25' and 35', it is suggested by design professionals that shear caps are integrated above the columns. This is a consideration for future implementation if use of post-tensioning is selected.

Like the flat plate system, the post-tensioned system should be able to contribute to the shear wall core by carrying some of the lateral loads.

Architectural:

The architecture of the IAC Headquarters would not need to be compromised by use of this system.

Construction:

The post-tensioned slab system appears to be a very good alternative to the existing system. The major problem, though, is that the construction industry in New York City is not experienced in post-tensioned construction. It is for this reason that a number of buildings in the city which would benefit from post-tensioned systems are actually constructed as flat plate or flat slab systems.

If the IAC building was evaluated independent of its location, it would benefit from a number of the advantages of a post-tensioned system. Similar to the existing flat plate system, the post-tensioned system displayed on the previous page has a flat, exposed concrete ceiling which would enable easy coordination of trades and would not require a finish. Additionally, because it is cast-in-place, it can be implemented in buildings with irregular geometries.

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2
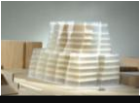
ALTERNATIVE #3: PRECAST HOLLOW CORE
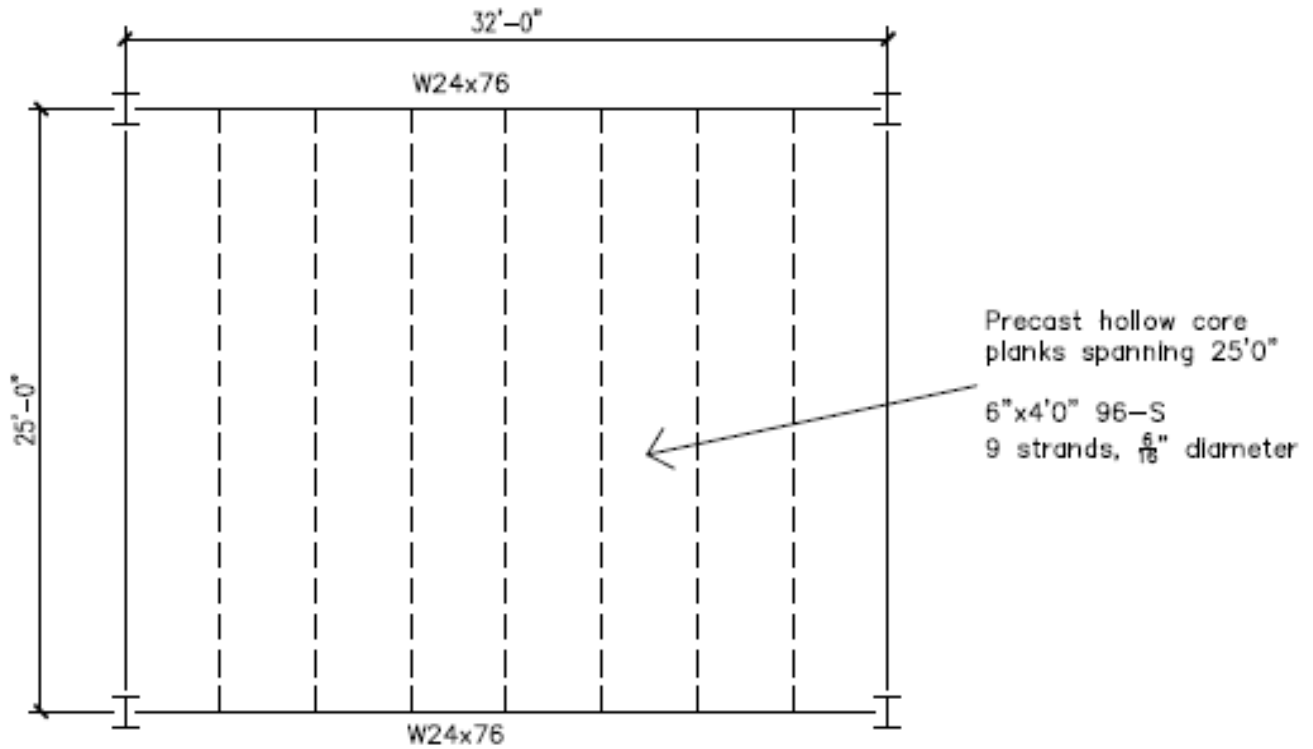
DESIGN CRITERIA
Slab Thickness= 6"
Girder Depth= 23.92"
f 'c= 5000 psi
Normal Weight Concrete
Self Weight of slab= 74 psf
Superimposed DL= 20 psf
Live Load= 60 psf

ADVANTAGES	DISADVANTAGES
- Fast and simple erection	- Long lead time
- 2-hour fire rating	- Unknown vibration effects
- Capable of carrying large loads	- Reconfiguration of column grid necessary
- Sustainable	- Irregular shapes will likely be more costly
- Thin slab system	- Irregular building makes using uniform panels impossible

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

HOLLOW-CORE PLANK DESIGN PROCESS

This system was designed by referring to load tables in the PCI Handbook. Once the planks were sized according to service load-carrying capacity, the steel girders supporting the planks were designed and evaluated for deflection and shear/moment capacity. The beams spanning parallel to the planks were not sized because they do not carry any significant loading. Nearly any typical W-shape would be adequate for that beam.

DESIGN CONSIDERATIONS

Structural:

Like the composite steel system, the precast system would likely need moment frame connections in order to help transfer the lateral loads.

Architectural:

The precast nature of this system causes a number of deterrents. Because the planks typically come in 4' increments, the columns would need to be moved and placed much more uniformly. This would ultimately compromise the architecture and functionality of the space.

Construction:

The precast hollow-core plank is an especially useful system because it is quick to construct. It uses normal-weight, high strength concrete that is very easy to install. In order to comply with fire-proofing requirements, the system on the previous page was designed using 2" of cast-in-place concrete topping. It also provides a flat, finished ceiling surface.

The footprint of the IAC Headquarters is certainly not uniform; thus, specialty planks would be necessary along the perimeter. This would cause considerable cost and labor increases, making it an unlikely system to consider for this building.

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

COMPARISON OF SYSTEMS

	System 1 Two-way Flat plate Slab (existing)	System 2 Composite Steel	System 3 Two-way Post-Tensioned Slab	System 4 Precast Hollow Core Slab on Steel
Cost	$20.22/sq ft	$29.28/sq ft	$17.18/sq ft	$33.02/sq ft
Structure Depth	12" slab	3 ½" slab 18" girder	8" slab	6" slab 23.92" girder
Structure Weight	150 psf	54 psf	100 psf	74 psf
Fireproofing	2 hr	2 hr (spray-on)	2 hr	2 hr
Effect on Column Grid	None	Little	None	Significant
Construction Difficulty	Medium	Easy	Medium/Hard	Easy
Lead Time	Short	Long	Short	Long
Further investigation?	Yes	Yes	Yes	No
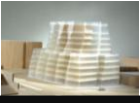
When comparing the aforementioned floor systems, a number of factors were considered in order to evaluate the effectiveness of each system in the IAC Headquarters. This criteria includes: cost, constructability, deflection, depth, fireproofing, foundation changes, layout changes, lead time, vibration, and weight. Based on the results of the preliminary analysis of the 4 floor systems, each of these factors was considered.

Cost
Using RS Means Assemblies 2009 data, a tentative cost for each of the systems was determined. The most expensive system is the hollow-core plank, which would only increase in cost in order to incorporate irregularly shaped precast planks. The cheapest system is the post-tensioned slab. While this system's strands typically cost two to three times that of regular steel, the reduction by thirty percent in the concrete slab significantly affects its overall cost. The cost of the composite steel seems much higher than what was anticipated. This could be due to the simplified assumptions in RS Means Assemblies. Nevertheless, in an effort to be consistent by gathering all data from the same source, the square footage cost for a composite system was not changed. In the future, it would be more accurate to gather actual costs from manufacturers in the New York City area.

Constructability
The steel composite system would be easy to construct because it is a very common system and does not require formwork, saving both time and money. However, because this building is located in New York City, where there are stringent laws requiring that no trades can work

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

above the steel workers, careful coordination of the sequencing of trades would be necessary. Precast hollow-core slab would also be very constructible; however, having a place to store the precast members may be difficult in the tight, urban area of the construction site. A post-tensioned system would not have been especially difficult to construct, except, as mentioned previously, laborers in NYC are not experienced with this type of construction. Lastly, though a flat plate system involves extra labor due to formwork and pouring, it can be easily constructed. In fact, it took only one week to complete two floors in the IAC Headquarters. The simplification of orienting the rebar orthogonally, which was mentioned previously helped to speed and ease the construction of the slabs.

Deflection/Vibration

Each of the floors systems was designed to meet the L/360 and L/240 serviceability requirements. Deflection will be a special concern at the sixth floor, where the columns are offset and the transfer slab exists. Deflection for post-tensioning is typically determined using computer programs. Because of inadequate knowledge with regards to modeling a post-tensioned system, deflection calculations were not performed. However, because of the balanced moment, it would likely perform very well for deflection.

Due to time constraints and limited knowledge, vibration calculations were not performed for this analysis. Vibration is affected by the mass and stiffness of the beam or slab; therefore, it is assumed that the more rigid and heavier floor systems (such as the concrete) would vibrate less.

Depth

Use of post-tensioning allows for the substantial reduction of the existing slab. While effectively limiting floor-to-floor heights in commercial buildings is not as imperative as in residential buildings, it remains an important consideration because it can ultimately affect cost, weight, deflection, etc. The composite steel slab is the shallowest, though the beams and girders are much deeper and would likely need to be finished with a ceiling, causing a large increase in the ceiling depth. The precast hollow-core planks are only 6" thick, but they are supported by quite deep W-shaped members. For these reasons, the post-tensioned system is the superior solution based on depth.

Fireproofing

Careful consideration was made in choosing floor systems with proper two-hour fireproofing. Because the normal-weight concrete slabs in the systems analyzed are greater than 4 ½", they would not require any additional fireproofing. The planks have an additional topping for fireproofing, but the composite system requires spray-on fireproofing. While spray-on fireproofing would require some additional cost and labor, it is not significant in dictating the flooring system that would be most appropriate.

Foundation Changes

The soil at the IAC Headquarters site is poor; therefore, changes in weight would ultimately affect the foundation design. For instance, the heavier concrete systems would likely need

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

more pile caps and deeper foundations than the lighter, composite steel system. This would prove to be very costly, because there were already a number of issues with the foundation when initially constructed with the flat plate design. From this standpoint, a composite steel or post-tensioned system would be preferable because of their lighter weight.

Layout Changes

The initial column layout of the building appears somewhat erratic. Designed as an open office layout, there does not seem to be any specific reason why the columns would need to remain in their exact locations. Future research into the architecture of the space is necessary. However, assuming that the column layout should not be changed, one of the great advantages of a post-tensioned system is that it allows columns to be placed in virtually any location. This flexibility of location is especially advantageous when considering the complex architectural shape of the building. The hollow-core plank system, on the other hand, must follow a much more stringent, uniform layout as it involves four foot increments. With proper design, composite steel and flat plate construction are both capable of performing effectively with various lengths and bay changes, though the composite steel would function best with more uniform bay layouts.

Lead time

Lead time is especially important in buildings that are fast-tracked. While this was not the case for the IAC Headquarters, it is always important for lowering labor costs. For the as-built, flat plate system, the floors were erected quickly; however, this involved little to no lead time since it was a cast-in-place system. Systems such as the precast hollow-core and composite steel would have a greater lead time than the concrete flat plate and post-tensioned systems. It is unknown at this time which system could ultimately be constructed the fastest, based on the region and the experience of the construction workers in the area.

Weight

This consideration is among one of the most important because it dictates a number of the other factors, especially cost, vibration, and foundation changes. Throughout the design of the systems, normal weight concrete was used in order to conform to the initial design assumptions. As mentioned in the 'Foundation Changes' section, the composite steel is the lowest in weight while the flat plate existing system weighs the most. The post-tensioned and precast plank systems both are medium weight, which is not surprising because their thicknesses are also average amongst the four systems.

Rachel Chicchi
Structural Option
Dr. Thomas E. Boothby

IAC/InterActiveCorp Headquarters
New York, NY
Technical Report #2

CONCLUSION
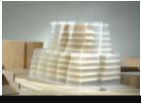
When evaluating the feasibility of flooring systems, it is important to consider a multitude of design factors. According to the structural engineers of the IAC Headquarters, it was designed as a flat plate system because of the ability to form concrete into different shapes when cast-in-place. This was crucial in allowing the building to curve and take on the flowing 'boat sail' look that was the design intent of the architect. One of the issues with this design seems to be especially prevalent at the sixth floor transfer level. Because flat plate slabs are not typically used as a transfer system, it is possible that long term creep of the transfer slab could occur, causing noticeable deflections in the floor.

After careful evaluation of the existing system and three additional alternatives, the post-tension system appears to be the preferred system, excluding the fact that the construction industry in New York City is not experienced with post-tensioning. This post-tensioned system would reduce the depth, weight and cost of the building. This reduction of weight could have played a considerate role in reducing the issues with the poor soil and foundations which plagued the contractors and owner during the initial stages of construction.

Composite steel also seems like a viable option for this system. The major deterrent for it, however, is that it is not as flexible as a post-tensioned system with its column layouts. Additionally, though not specifically calculated in this report, intuition suggests that the post-tensioned system would more effectively limit deflections and perhaps prevent long-term creep from causing serviceability issues.

Lastly, because of the strict uniformity necessary in a precast system, it does not seem to be a viable option for the IAC Headquarters.

In the future, an in-depth analysis of the post-tensioned system will need to be considered and the use of a computer program, such as RAM Concept, to design for the entire floor system would be necessary.

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Appendix A: Calculations

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COMPOSITE STEEL CALCS

Composite Steel

For a 25'0" x 29'3" frame
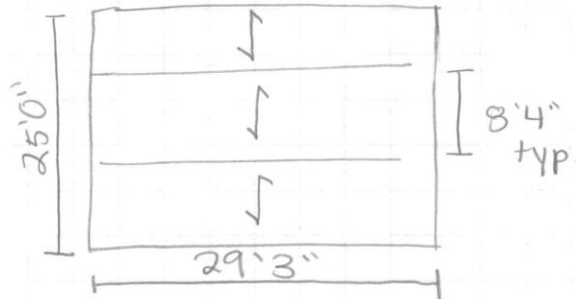
145 pcf concrete
$f'_c = 5000$
50 ksi steel
LL = 60 psf
SDL = 20 psf
Superimposed Load = 80 psf

Unshored construction, 3-span
2" thick concrete, 2hr unprotected deck
Use 1.5 VL20 (vulcraft, pg 48)
Wt of slab + deck = 33 psf

∴ Total DL = 53 psf
Live Load = 60 psf

Beam Design

$1.2D + 1.6L = 1.2(33+20) + 1.6(60) = 160 psf$

$M_u = \dfrac{160(8'4")(29.25^2)}{8000} = 143'^k$

Assume $a = 1$

$Y_2 = 6 - \frac{1}{2} = 5.5 in$, PNA = 6

Table 3-19 in steel manual
Choose W12 shape for most economical
$\phi M_p = 110$ for W12×22, $M_u = 193$
$\Sigma Q_n = 117$

$b_{eff} \leq \dfrac{29.25(12)}{4} = 88 in \leftarrow governs$

$\leq 8'4"(12) = 100 in$

$a = \dfrac{117}{.85(5)(88)} = .3128 < 1.0$ ok conservative

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(N=9.35) NORMAL WEIGHT CONCRETE (145 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)										
		1 SPAN	2 SPAN	3 SPAN	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0
3.50 (t=2.00) 33 PSF	1.5VL22	5'-10	7'-10	7'-10	314	279	230	206	186	169	154	141	130	120	111
	1.5VL20	7'-0	9'-4	9'-6	345	306	275	249	227	187	171	157	144	133	124
	1.5VL19	7'-11	10'-3	10'-8	372	330	296	268	244	224	186	171	157	145	134
	1.5VL18	8'-8	11'-0	11'-2	395	351	315	285	260	238	220	204	168	156	142
	1.5VL16	8'-10	11'-0	11'-4	397	353	316	286	261	239	221	205	169	156	145

Pg 48 of Vulcraft Decking Catalog

$DL = 53 + .022(29.25) = 53.6 \, psf$

Bare Steel Under Construction Loads

$M_u = 1.2 DL + 1.6 LL = \boxed{75.2^{k-ft}}$

$DL = \dfrac{53.6(8.33)(29.25^2)}{8000} = 47.7 \, k\text{-}ft \quad LL = 20(8.33) = 167 \, plf$

$\dfrac{.167(29.25^2)}{8} = 17.9^{ft}$

$75.2 < 110$

$V_u = \dfrac{53.6(8.3)(29.25)}{2} = 6.5^k < 96^k$

Construction Defl. Check:

$\dfrac{5(w_D + w_{constr.})(29.25^4)(1728)}{384(29000)(I_x)} = 2.23" > .98" \quad \text{Not OK}$

$_{.613} \qquad _{156}$

$\dfrac{5(.613)(29.25^4)(1728)}{384(29000)(I_x)} \le .98 \qquad \boxed{I_x \ge 355}$

$\Delta_{LL} = \dfrac{5}{384} \dfrac{(60)(8.33)(29.25^4)(1728)}{29000(I_{LB})} \le .98$

$_{1000}$

$\boxed{W14 \times 38}$ $\qquad \boxed{I_{LB} \ge 290 \, in^4}$

$\quad \hookrightarrow I_x = 385, \; I_{LB} = 803 \, in^4$

$\Sigma Q_n = 174^k$ $\qquad \dfrac{174}{21}(2) = \boxed{17 \text{ shear studs}}$

$\phi M_p = 231^k \checkmark$

$\phi V_n = 131^k \checkmark$

Girder Design

$DL = 53.6(8.3)\left(\dfrac{29.25}{2}\right)(2)$

$= 13.1^k + \text{self weight} = 14.2^k$

$\quad _{.038(29.25)(2)} \quad \tfrac{}{2}$

$LL = 60(8.3)(29.25) = 14.6^k$

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Total $\Delta \Rightarrow \dfrac{28.8(29.25)^3(1728)}{28(29000)\,I} \leq \dfrac{\ell}{240} = 1.25$

$$\boxed{I \geq 1227 \text{ in}^4}$$

LL $\Delta \Rightarrow \dfrac{14.6(29.25)^3(1728)}{28(29000)\,I} = \dfrac{\ell}{360} \Rightarrow I \geq 933 \text{ in}^4$

Constr. $\Delta \Rightarrow \dfrac{14.2(29.25^3)(1728)}{28(29000)\,I} = \dfrac{\ell}{240} = 1.25$
Dead

$$I \geq 605 \text{ in}^4$$

Try a
W18×40 $I_{LB} = 1260$
     $I_x = 612$
   $\phi M_p = 294$, $\sum Q_n = 210$

$\boxed{P_u = 41.5^K}$, $M_u = P_u(a) = 41.5(8.33) + \dfrac{.04(29.25)^2}{8}$

ϕM_p for W18×40 $< M_u$ $= \boxed{346 \text{ ft-K}}$
   Not OK

TRY a
W18×50 $\rightarrow I_{LB} = 1570$, $I_x = 712$ ✓
    $\phi M_p = 379 \geq 346$ ✓
    $\phi V_n = 192 \geq 41.5 + .05\left(\dfrac{29.25}{2}\right) = 42.2^K$ ✓
    $\sum Q_n = 245^K$

T of W18×50 = 15½ $\geq$ d of W14×38 = 14.1" ✓

$a = \dfrac{245}{.85(5)(75)} = .769 < 1.0$" ✓ $b_{eff} = 29.25(12) = 351$"
                                       ¼(25)(12) = 75"

$\dfrac{245(2)}{21} = \boxed{24 \text{ shear studs}}$

Cost Analysis

Using RS Means 2009, pg 416
For 25×30' bay, W shape, composite deck + slab
$22.30/sq ft (1.313)
            ↖ NYC location factor
$= \boxed{\$29.28 \text{ /sq ft}}$

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POST-TENSIONED TWO-WAY SLAB CALCS

Post Tension

29'3" × 25'0" frame, $f'_c = 5000$ psi

$f_y = 60$ ksi

$LL = 60$ psf, $SDL = 20$ psf

$f_{se} = .7(270) - 15 = 174$ ksi ← Assume 15 ksi prestress losses

$P_{eff} = Af_{se} = .153(174) = \underline{26.6^k}$ tendon

↖ Assuming ½"⌀, 7 wire strands, $A = .153$ in²

$f_{pu} = 270$ ksi

$\dfrac{\text{span}}{\text{depth}} = \dfrac{L}{45}$ $\dfrac{29.25(12)}{45} = 7.8"$ use $\boxed{8" \text{ slab}}$

DL due to self wt → $8(150)/12 = 100$ psf

$1.2(100+20)+1.6(60) = \boxed{240 \text{ psf}}$

$A = bh = 29.25(12)(8) = 2808$ in²

$S = \dfrac{bh^2}{6} = \dfrac{29.25(12)(8^2)}{6} = 3728$ in³

@ transfer: $f'_{ci} = 3000$ psi

compression: $.6(3000) = 1800$ psi

tension: $3\sqrt{3000} = 164$ psi

@ service loads: $f'_c = 5000$ psi

compression: $.45 f'_c = 2250$ psi

tension: $7.5\sqrt{f'_c} = 530$ psi

Average precompr. limits $P/A = 125$ min

(ACI 18.12.4) 300 max

Target Load Balances — $60-80\%$ self weight

Try 80% $.8 w_{self} = 80$ psf

Cover req't → 3/4" top & bottom

*Note: Chose not to reduce live loads in order to be conservative.

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In N-S direction, will use banded
tendons, along column strip.
In E-W direction, uniform tendons

E-W direction

Prestress Force to Balance 80% self weight

$$w_b = .8 \, w_{DL} = .8(100)(25) = 2 \, \text{k/ft}$$

$$P = \frac{w_B L^2}{8 \, a_{end}} = \frac{2(29.25^2)}{8(3.75/12)} = 684^k$$

$684^k / 26.6^k / \text{tendon} = 25.7 \quad \text{or} \quad 26 \, \text{tendons}$

$P_{actual} = 26(26.6) = 692^k$

Balanced Load:

$$w_b = \frac{692}{684}(2) = 2.02 \, \text{k/ft}$$

Actual Precompr. Stress

$P_{act} / \text{Area} = 692/2808 = \boxed{246 \, \text{psi}}$

$> 125 \, \text{min} \checkmark$

$\angle 300 \, \text{max} \checkmark$

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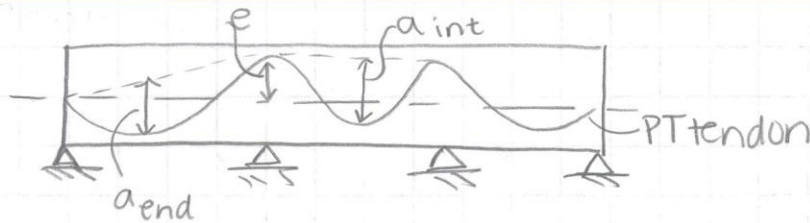
$$a_{int} = 7 - 1 = 6", \quad a_{end} = \frac{(4+7)}{2} - 1.75 = 3.75"$$

N-S direction

Prestress Force Req'd to Balance 80% selfweight

$$w_b = .8 w_{DL} = .8(100)(12.5) = 1000 \text{ lb/ft}$$

$$P = \frac{w_B L^2}{8 \, a_{end}} = \frac{1.0(25^2)}{8(3.75/12)} = 250^K$$

Calculating P for just column strip

$$250^K / 26.6^{K/tendon} = 9.4 \rightarrow 10 \text{ tendons}$$

Actual force for banded tendons

$$P_{actual} = 10(26.6) = 266^K$$

Balanced Load for end span:

$$w_b = \frac{266}{250}(1) = 1.06 \text{ K/ft}$$

Actual Precompr. Stress

$$P_{act} / Area = (266 / (12.5)(8)(1000)) = 222 \text{psi}$$
$$\qquad \qquad \qquad \qquad (12)$$

> 125 min ✓

Effective prestress force = $\boxed{222 \text{psi}}$ < 300 max ✓

For calculation of moments, use direct design method to determine.

(FRAME A → N-S direction)
(FRAME B → E-W direction)

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Stage 1: Stresses Immediately after Jacking
   Midspan Stresses

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

Using moments determined from Excel using DDM

$$P/A = 222 \text{ psi (Frame A)}$$
$$P/A \quad 246 \text{ psi (Frame B)}$$

Support Stresses

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

Stage 2: Stresses @ service load (DL + LL + PT)
   Midspan Stresses

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

Support Stresses

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

See excel spreadsheet for values
   determined for the various stresses

$$\text{FRAME A} - S = \frac{bh^2}{6} = 1600 \text{ in}^3 \quad \left(\frac{12.5 \times 12 \times 8^2}{6}\right)$$

$$\text{FRAME B} - S = \frac{bh^2}{6} = 3200 \text{ in}^3 \quad \left(\frac{300 \times 8^2}{6}\right)$$

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Finding Moments for Post-Tensioned Slab

l1	29.25	ft
l2	25	
Distance of column strip	6.25	(smaller of l1/4 or l2/4)
LL	60	psf
DL	120	psf
Load Comb.	240	psf
Slab Thickness	8	in
Column Diam	28	in
	34	in
Equiv. Sq	24.92	in
	30.26	in

Mo	wult*ln^2	
Frame A	221.689612	(DL Moments)
Frame B	267.901426	(DL Moments)
Frame A	110.844806	(LL Moments)
Frame B	133.950713	(LL Moments)
Frame A	-66.9489997	(Balancing Moments)
Frame B	-180.38696	(Balancing Moments)

FRAME A

Dead Load	3510	lb/ft
Live Loads	1755	lb/ft
Balancing	-1060	lb/ft

FRAME B

Dead Load	3000	lb/ft
Live Loads	1500	lb/ft
Balancing	-2020	lb/ft

FRAME A- DEAD LOAD MOMENTS

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	-57.64	Ext Neg	0.26	-57.64	0.00	0.00
	0.52	115.28	Pos	0.31	68.72	0.21	46.55
	0.70	-155.18	Int Neg	0.53	-117.50	0.17	-37.69
Int Span	0.35	77.59	Pos	0.21	46.55	0.14	31.04
	0.65	-144.10	Neg	0.49	-108.63	0.16	-35.47

* Factors for flat-plate slabs given in ACI Notes

FRAME A- LIVE LOAD MOMENTS

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	-28.8196	Ext Neg	0.26	-28.81964959	0	0
	0.52	-57.6393	Pos	0.31	34.36188989	0.21	23.27741
	0.7	-77.5914	Int Neg	0.53	-58.74774724	0.17	-18.8436
Int Span	0.35	-38.7957	Pos	0.21	23.27740928	0.14	15.51827
	0.65	-72.0491	Neg	0.49	-54.31395499	0.16	-17.7352

* Factors for flat-plate slabs given in ACI Notes

FRAME A- BALANCING MOMENTS

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	17.40674	Ext Neg	0.26	17.40673992	0	0
	0.52	-34.8135	Pos	0.31	-20.75418991	0.21	-14.0593
	0.7	46.8643	Int Neg	0.53	35.48296984	0.17	11.38133
Int Span	0.35	-23.4321	Pos	0.21	-14.05928994	0.14	-9.37286
	0.65	43.51685	Neg	0.49	32.80500985	0.16	10.71184

S	1600
P/A	222

STRESSES IMMEDIATELY AFTER JACKING	
MIDSPAN STRESSES	
-676.61 f(top)	ok
21.72 f(bott)	ok
SUPPORT STRESSES	
346.67 f(top)	ok
-790.67 f(bott)	ok
STRESSES @ SERVICE LOAD	
MIDSPAN STRESSES	
-222.05 f(top)	ok
-222.12 f(bott)	ok
SUPPORT STRESSES	
-221.88 f(top)	ok
-222.12 f(bott)	ok

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FRAME B- DEAD LOAD MOMENTS

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	-69.65	Ext Neg	0.26	-69.65	0.00	0.00
	0.52	139.31	Pos	0.31	83.05	0.21	56.26
	0.70	-187.53	Int Neg	0.53	-141.99	0.17	-45.54
Int Span	0.35	93.77	Pos	0.21	56.26	0.14	37.51
	0.65	-174.14	Neg	0.49	-131.27	0.16	-42.86

FRAME B- LIVE LOAD MOMENTS

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	-34.83	Ext Neg	0.26	-34.83	0.00	0.00
	0.52	69.65	Pos	0.31	41.52	0.21	28.13
	0.70	-93.77	Int Neg	0.53	-70.99	0.17	-22.77
Int Span	0.35	46.88	Pos	0.21	28.13	0.14	18.75
	0.65	-87.07	Neg	0.49	-65.64	0.16	-21.43

FRAME B- BALANCING MOMENTS

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	46.90	Ext Neg	0.26	46.90	0.00	0.00
	0.52	-93.80	Pos	0.31	-55.92	0.21	-37.88
	0.70	126.27	Int Neg	0.53	95.61	0.17	30.67
Int Span	0.35	63.14	Pos	0.21	-37.88	0.14	-25.25
	0.65	-117.25	Neg	0.49	88.39	0.16	28.86

S	3200	in3
P/A	246	psi

STRESSES IMMEDIATELY AFTER JACKING		
MIDSPAN STRESSES		
-360.863	f(top)	ok
-131.137	f(bott)	ok
SUPPORT STRESSES		
-32.6835	f(top)	ok
-459.317	f(bot)	ok
STRESSES @ SERVICE LOAD		
MIDSPAN STRESSES		
-246.024	f(top)	ok
-245.976	f(bott)	ok
SUPPORT STRESSES		
-245.955	f(top)	ok
-246.045	f(bott)	ok

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Ultimate Strength

$$M_1 = P \cdot e \qquad e = 3.0$$
$$= 266(3)/12 = 66.5^{ft-k} \ (N-S)$$
$$= 692(3)/12 = 173^{ft-k} \ (E-W)$$

$$M_{sec} = M_{bal} - M_1$$
$$= 32.81 - 66.5 = -33.7 \ (N-S)$$
$$= 117.3 - 173 = -55.7 \ (E-W)$$

$$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$$

@ midspan
$$M_u = 76.3^{k-ft} \ (N-S)$$
@ support $M_u = -273^{k-ft}$

@ midspan
$$M_u = 159.7^{k-ft} \ (E-W)$$
@ support $M_u = -404 \ k-ft$

Determine min. bonded reinforcement

Pos. Moment Region
Int Span = 15 psi $< 2\sqrt{f'_c} = 141$ psi
∴ No pos. reinf. required

Neg Moment Region
$A_{s\,min} = .00075 \, A_{cf}$
Int. Supports:
$A_{cf} = \max (8in)(29.25)(12) = 2808$
$A_{s\,min} = .00075 (2808) = 2.106 \ in^2$
11 - #4 top (2.20 in²)

Must span min of 1/6 clear span on each side
of support
At least 4 bars req'd in each direction
Place top bars w/in 1.5h away from face
of support = 12 in ↗
Max bar spacing = 12"

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Check min. reinf. if sufficient for ultimate strength

$$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$$
$$A_{ps} = .153 in^2 (\# tendons)$$

N-S direction

$$A_{ps} = .153(10) = 1.53 \ in^2 \qquad d = 8 - 3/4 - 1/4 = 7"$$
$$f_{ps} = f_{se} + 10000 + f'_c bd / 300 A_{ps}$$
$$174000 + 10000 + 5000(25 \cdot 12)(d)/300(1.53)$$
$$= 206876 \ psi$$
$$a = (A_s f_y + A_{ps} f_{ps})/.85 f'_c b$$
$$(2.2(60) + 1.53(206.876))/.85(5ksi)(25 \times 12) = .352$$
$$\emptyset M_n = .9[(2.2)(60) + 1.53(206.9)[7 - .35/2]/12$$

$$= 229^K \ \angle \ 273 \ \text{Reinf. governed by}$$
$$\frac{273(12)}{.9} = [A_s(60) + 1.53 \times 207](7 - \frac{A_s(60) + 1.53(207)}{.85(5)(25 \times 12)}) \ \text{ult strength req'ts}$$

E-W direction
$$A_{ps} = .153(26) = 3.98 \ in^2 \qquad \hookrightarrow A_s = 3.96 in^2$$
$$f_{ps} = 195759 \ psi \qquad = \boxed{(9) \#6 \ bars}$$
$$a = .611$$
$$\emptyset M_n = 457$$
$$\boxed{12 \ \#4 \ top \ @ \ int \ supports}$$

Summary

In N-S direction
@ neg. moment - 9 #6 bars
(10) 1/2" diameter, 7-wire strands
(Acting along column strips)

In E-W direction
@ neg moment - 12 #4 top bars
(26) 1/2" diameter, 7-wire strands
(Acting uniformly along 25' distance)

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Post Tension

Cost Analysis : Using RS Means 2009, pg 78

Prestressing Steel
$3.33 /lb

Cast in Place concrete
$$575 / yrd^3 \left(\frac{1 yd^3}{27 ft^3}\right)\left(\frac{8"}{12"/ft}\right) =$$

$14.20 / sq ft

$$3.33/ lb \, (655 lb) = \frac{\$2183}{(25)(29.25)} = \$2.98/sq\, ft$$

Strand weight = .52 lb/ft for ½" Ø strand

$$.\frac{52\, lb}{ft} \left(20(25) + 26(29.25)\right) = 655\, lb$$
↰ full lengths to be conservative

Total Cost =
$14.20 + $2.98 = $17.18/sq ft

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PRECAST HOLLOW-CORE PLANK CALCS

Hollow Core Plank

$SDL = 20 psf$
$LL = 60 psf$ $> 80 psf$ service load

$1.2(20) + 1.6(60) = 120 psf$ → When using PCI, use unfactored loads?
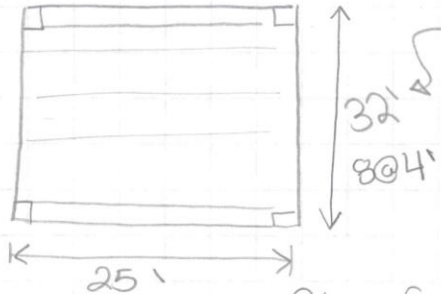
$32'$
$8@4'$ Normal @ 29', but adjust column grid to fit 4' increments
Reference: PCI Design Handbook @ a span of 25'0"
9 - 6/16" Ø strands (straight)
($6" \times 4'0"$ w/ 2" topping)

$25'$

2 hr. fire resistance rating w/ 2" topping

Safe Superimposed service load = 137 psf ⩾ 120

DL of 96-S Hollow Core Plank = 74 psf
w/ topping

$74 + SDL = 94 psf$, $LL = 60$
$\quad\quad\quad\quad\quad\quad\quad 20$
$1.2(94) + 1.6(60) = 209 psf$

Trib Width = $25'(209) = 5.23 K/ft$

$M_u = \dfrac{w_u \ell^2}{8} = \dfrac{5.23(32^2)}{8} = 669\ ft-k$

@ 32'0" →
$\Delta_{LL,max} = \dfrac{\ell}{360} = 1.07" = \dfrac{5(60)(25)(32)^4(1728)}{384(29000)I_{min}} \rightarrow I \geq 1140\ in^4$

$\Delta_{D+L} = \dfrac{\ell}{240} = 1.6" = \dfrac{5(60+94)(25)(32^4)(1728)}{384(29000)(I_{min})} \rightarrow I \geq 1958\ in^4$
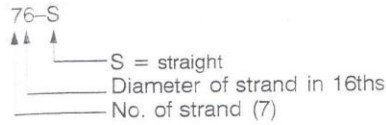
For W24x76
$\phi M_n = 750 \geq M_u = 669$
$I_{w24x76} = 2100 \geq 1958$ OK

Use W24x76 girders w/ hollow core plank ($6" \times 4'0"$ w/ 2" topping, 9?-S)

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Strand Pattern Designation

76-S

- S = straight
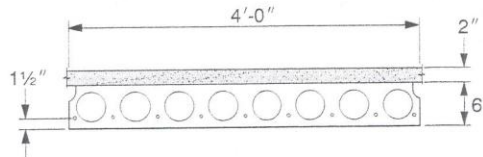- Diameter of strand in 16ths
- No. of strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
306 — Safe superimposed service load, psf
0.2 — Estimated camber at erection, in.
0.2 — Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete

4'-0"   2"
1½"   6"

$f'_c = 5,000$ psi
$f'_{ci} = 3,500$ psi

Section Properties

		Untopped		Topped	
A	=	187	in²	—	
I	=	763	in⁴	1,640	in⁴
y_b	=	3.00	in.	4.14	in.
y_t	=	3.00	in.	3.86	in.
S_b	=	254	in³	396	in³
S_t	=	254	in³	425	in³
b_w	=	16.00	in.	16.00	in.
wt	=	195	plf	295	plf
		49	psf	**74**	**psf**
V/S	=	1.73	in.		

4HC6
No Topping

Table of safe superimposed service load (psf) and cambers (in.)

Strand Designation Code	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	306	257	217	184	157	135	116	100	87	75	65	56	48	42	36	30			
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.4			
	0.2	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−1.0			
76-S	358	301	254	217	186	160	139	121	105	92	80	70	61	53	47	40	35		
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	−0.1	−0.3		
	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	−0.1	−0.3	−0.5	−0.7	−1.0		
96-S		384	326	279	240	208	182	159	140	123	109	97	86	76	67	60	53	46	41
		0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	−0.1
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.2	0.1	−0.1	−0.4	−0.6	−0.9
87-S		383	331	286	249	218	192	169	150	133	119	106	95	84	76	68	60	54	
		0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3	
		0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.5	0.4	0.2	0.0	−0.3	
97-S			364	317	277	243	214	189	168	150	134	120	107	96	87	78	70	62	
			0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	1.0	0.9	0.9	0.8	0.8	0.7	
			0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.0	1.0	0.9	0.8	0.6	0.4	0.2	

4HC6+2
2" Normal Weight Topping

Table of safe superimposed service load (psf) and cambers (in.)

Strand Designation Code	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	305	258	220	188	162	139	119	97	78	62	47	35					
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1					
	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9					
76-S	358	304	260	224	194	168	146	122	101	82	66	52	39				
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.2	−0.3	−0.5	−0.7	−0.9				
96-S		390	336	291	253	221	194	170	146	123	104	87	72	58	46	35	
		0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	
		0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	−0.1	0.3	−0.5	−0.7	−1.0	−1.4	

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For beams parallel to hollow core plank, no significant gravity loads. Would likely use a small W12 section.

Cost analysis:

Using RS Means for Precast Beam + Plank w/ 25×30' bay   2" topping

2009
Pg 78
of
Assemblies

$24.70 per sf. (material + installation)
− $11.30 (cost of precast T-beam in assembly)
+$ 11.75 (cost of W shape in 25×30 bay)

_____

$25.15 /sq ft (1.313) =   ↖ NYC location factor

$ 33.02 /sq ft

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FLAT PLATE SLAB CALCS

Flat Plate System

Cost Analysis:

Using RS means 2009, pg 67

$15.40 / sq ft (1.313) = ← NYC location factor

$$\boxed{\$20.22 / sq\ ft}$$

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Appendix B: Floor Plans

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Figure B-1: Cellar Floor Plan

(Full structural floor plans disclosed at owner's request)

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Figure B-2: 5th Floor Plan

(Full structural floor plans disclosed at owner's request)

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Figure B-3: 6th Floor Plan

(Full structural floor plans disclosed at owner's request)

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Figure B-4: 11th Floor Plan

(Full structural floor plans disclosed at owner's request)