100 Eleventh Avenue

New York, New York

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Technical Report II

Table of Contents

Executive Summary

The Pro-Con Structural Study of Alternate Floor Systems consists of a comparison of 100 Eleventh Avenue's existing structural floor system and three potential alternatives. The intent is to determine if any feasible alternative system exists. A 32' x 22.5' exterior bay was used for the analysis. Criteria such as system cost, depth, weight, and constructability were looked at.

The four floor systems analyzed in the report are as follows:

- **Two-way Flat Plate**
- **Composite Steel**
- **Post-tensioned Two-way Flat Plate**
- **Precast Hollow-Core Plank on Steel Beams**

The existing system consists of 9" and 18.5" thick slab with a bottom reinforcing mat of #4 @ 12" E.W. Middle strip bars are also #4 @ 12" (top) and column strip bars are #5 @ 12" (top). The composite steel alternative was designed using AISC *Steel Construction Manual, 13th Edition* and Vulcraft Group's Deck Catalog. The preliminary design resulted in W12x16 members @ 7.5' o.c. supporting a 1.5VL20 composite deck and a 4.5" thick concrete slab. These beams spans between a W12x50 girder and a W12x30 girder. For the post-tensioned design, only a column strip was analyzed in each direction. The result was (14) ½"Ø 7-wire tendons within an 8.5" slab in the N-S direction and (17) tendons within a 10" slab in the E-W direction. Bonded reinforcing bars were added as necessary. For the precast hollow-core system, a 6" plank with 2" of topping slab and (6) 3/8"Ø pre-stressed strands was selected from the *PCI Industry Handbook, 6th Edition.* W18x50 girders were designed to support the concrete planks.

Upon comparison, only the cast-in-place systems were deemed feasible, as the irregular column layout and column offsets create very expensive and complicated systems when steel and precast concrete systems are used. The post-tensioned two-way flat plate system was deemed the only feasible alternative. The pre-stressed tendons decreased the slab thickness from 9" to 8.5" in the typical slab and from 18.5" to 10" along the curved edge. This would result in a lighter structure and higher ceilings, while keeping the building height the same. Upon performing a rough cost estimate, the systems were determined to be similar in cost. The main deterrent to implementing the post-tensioned floor system is New York City contractor's unfamiliarity with pre-stressed construction.

With these preliminary comparisons complete, the post-tensioned two-way flat plate system is worth future investigation as an alternative floor system.

Introduction to 100 Eleventh Avenue

100 Eleventh Avenue is a 22-story, 170,000 sf ultra-luxury condominium building located in Manhattan's Chelsea District, a neighborhood next to the Hudson River that is quickly gaining in popularity within the city. 100 Eleventh Avenue will join several other recently completed projects that have helped in revitalizing the area, such as IAC's headquarters designed by architect Frank Gehry, and the High Line, a former elevated rail line running through the area that has been converted into an elevated park.

Dubbed a "vision machine" by its Pritzker Prize-winning architect Jean Nouvel, 100 Eleventh Avenue's defining feature is its façade, a panelized curtainwall system consisting of 1650 windows, each a different size and uniquely oriented in space. Light reflecting off the randomly-oriented windows limits views into the building while still allowing occupants spectacular floor-to-ceiling views of both New York City and the Hudson River. In addition, the lower six floors are enclosed by a second façade offset 16 feet towards the street. As seen in Figure 1 below, the space between the two facades is filled with intricate steel framing and cantilevered walls, columns, and balconies. Trees are suspended in air at varying heights, creating a "hanging garden" and a unique atrium space.

The building's structural system is cast-in-place concrete – common for residential buildings in the city. The ground level contains 6000 sf of retail space, as well as an elevated garden space for the residents, which spans over a junior Olympic-sized pool. Levels 2 through 21 house the residential units, with the penthouse making up the 21st floor, containing an extensive private roof terrace.

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Figure 1: Space within double facade

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Figure 2: View from Westside Highway

Existing Structural System Summary

Foundations

100 Eleventh Avenue is located on a man-made portion of Manhattan Island. Therefore, the shallow bedrock typical of much of the island is not present, and the use of piles and drilled caissons is necessary to effectively transfer vertical and horizontal loads to the earth. 127 piles at 150 ton capacity transfer column loads to the ground. Thirteen of these are detailed to provide a 50 kip tension capacity, as several cantilevered columns may, under certain loading conditions, induce tension in the piles, as seen in Figure 4. In addition, 12 large-diameter caissons are located at the structure's shear wall core, ranging in capacity from 600-1500 ton and providing at least 50 kip in lateral capacity. At the cellar level, a 20" thick mat foundation ties the piles together, while resisting the upward soil pressure. At the building's core, this mat slab thickens to 36".

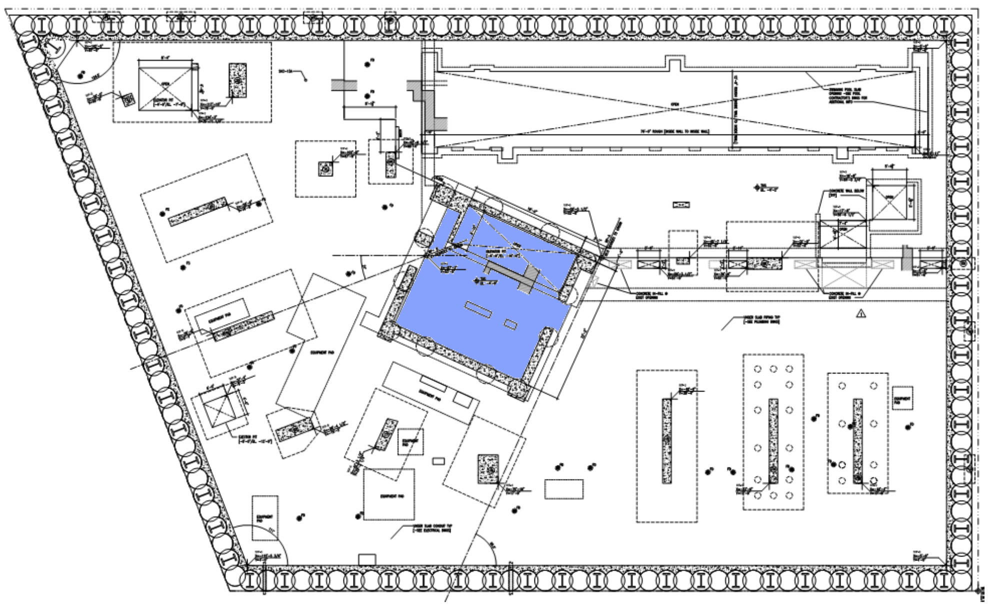


Figure 3: Cellar plan with core denoted

In order to eliminate the cost of underpinning the adjacent structures during excavation, a concrete secant wall system was used instead of traditional foundation walls. As seen in Figure 3, the secant piles are driven around the entire perimeter and resist the lateral soil pressures. The secant wall is braced at its top by the 12" ground floor slab. At all slab steps on the ground floor, torsion beams were used to resist torsion created by the lateral forces from the secant wall.

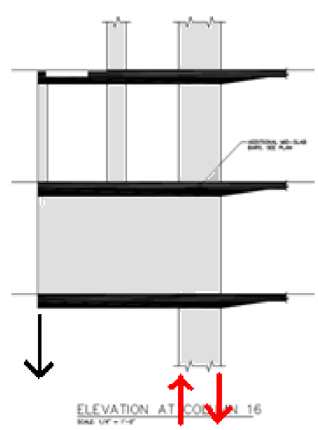


Figure 4: Cantilevered column creating tension in piles

Gravity System

Floor System

100 Eleventh Avenue has a cast-in-place two-way concrete flat-plate floor system. This type of system is common for residential buildings in New York City due to the ease of accommodation of column offsets, the minimal floor system thickness, and the sound isolation properties of concrete.

Figure 5: Superstructure

The typical floor is comprised of 9" thick, 5,950 psi concrete reinforced with a basic bottom reinforcing mat of #4 @ 12" E.W. Middle strip bars are also #4 @ 12" unless otherwise noted. Column strip bars are primarily #6 @12". Additional top and bottom bars are added where necessary, likely due to longer spans and varying loads. The slab thickness increases to 12" at the elevator core, where the bottom reinforcing steel is #5 @12" E.W. While no standard span exists, most slab spans range from 18'-23'. Due to increased loads from the curtainwall as well as spans as long as 34 feet, the slab thickens from 9" to 18.5" along the curved portion of the building. Due to aesthetics, the slab gradually increases in thickness over a distance of 5'-0", as seen in Figure 6, rather than an abrupt increase.

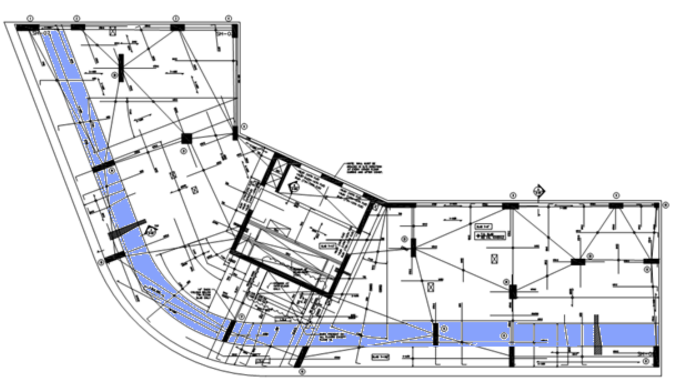


Figure 6: Typical plan with slab thickness transition area highlighted

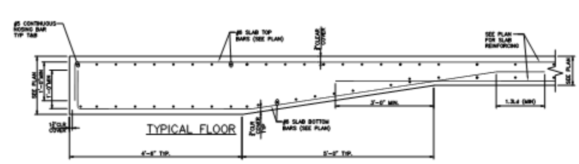


Figure 7: Detail of thickened slab at curved edge

As seen from the typical structural plan, Figure 7, floor reinforcing along the curve is detailed as straight bars with a single bend, thereby avoiding the additional costs and installation difficulties involved with curved bars. Slab reinforcing was detailed radially throughout the floor to match the building's three distinct geometric axis.

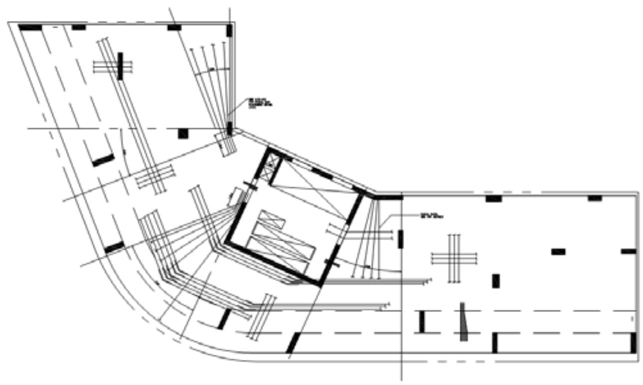


Figure 8: Slab reinforcing schematic layout

On the lower six floors, balconies begin to cantilever out towards the second street façade. An example of this is shown in Figure 11, where the balcony extends 9'-10" from the building. Notice that, due to architectural constraints, the balcony has only one corner supported by a column below. To resolve excessive deflection caused by the façade and tree loads, three post-tensioned high-strength Dywidag bars were used, highlighted in green.

Figure 9: Cantilevered balcony utilizing post-tensioning

Columns

Column strength for columns supporting the cellar level through the 9th level are 8 ksi; those supporting the 10th through the roof are 7 ksi. As evidenced by the typical floor plan, no regular grid exists. Spans typically range from 18'-23', except on the curved edge portion, where spans of up to 34' exist. Column

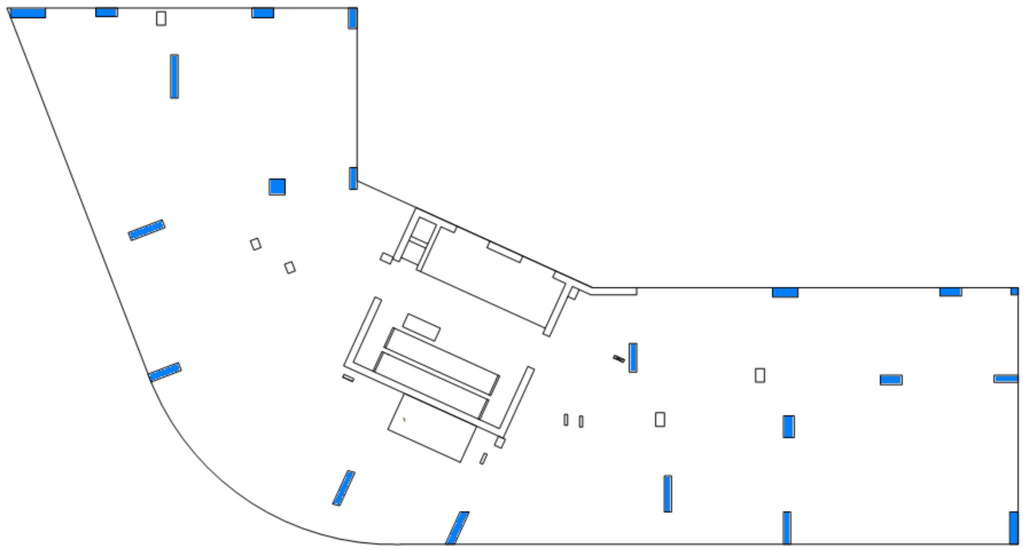


Figure 10: Typical floor column layout

sizes range widely throughout a single floor, as well as from floor to floor. The vast majority are 12"-16" wide and 3-4 times as long, resulting in many "long" columns. This allows the columns to be placed within the walls separating individual units. Also, seven of these long columns were designed as part of the lateral system. More discussion on this can be found in the lateral system summary.

On the lower six floors of the building, these seven long columns also serve as support for the complex balcony system that defines the lower floors. On these floors, intermittent boxes "poke" out from the inner façade to meet the outer street façade, which is offset 16' towards the street. On the second level, several of these outstretched balconies are supported by cantilevered columns ranging in length from 18' to 28'. Figure 14 shows the columns supporting the 3rd level, with red denoting the cantilevered portion of the columns. Due to significant tensile forces at the tops of these cantilevered columns, additional reinforcement of six mid-slab #11 Grade 75 bars tie the top of the columns into the main portion of the slab.

Figure 11: Photo showing portion of cantilevered balcony system

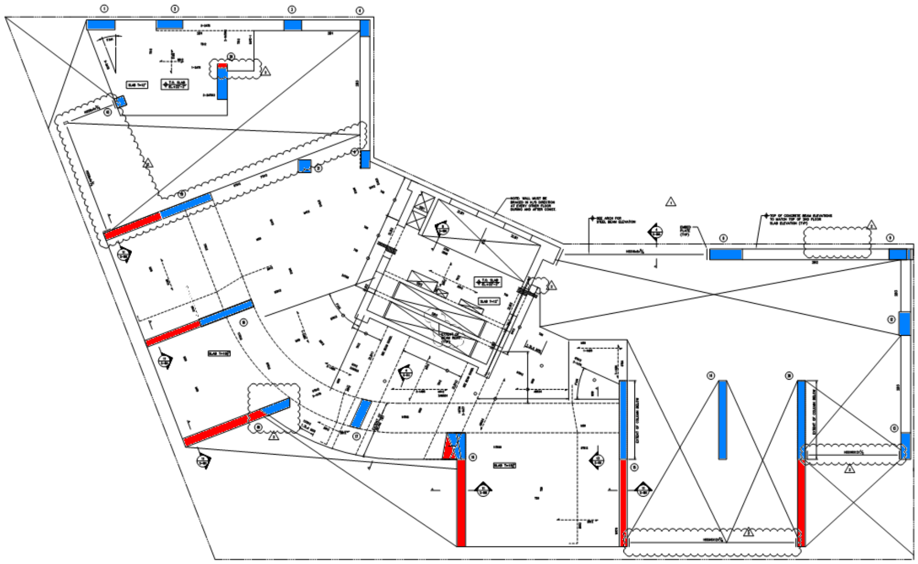


Figure 12: 2nd Floor column layout

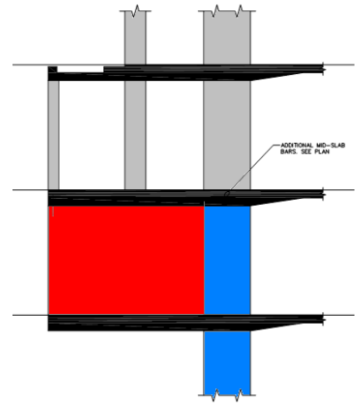


Figure 13: Cantilevered Column Elevation

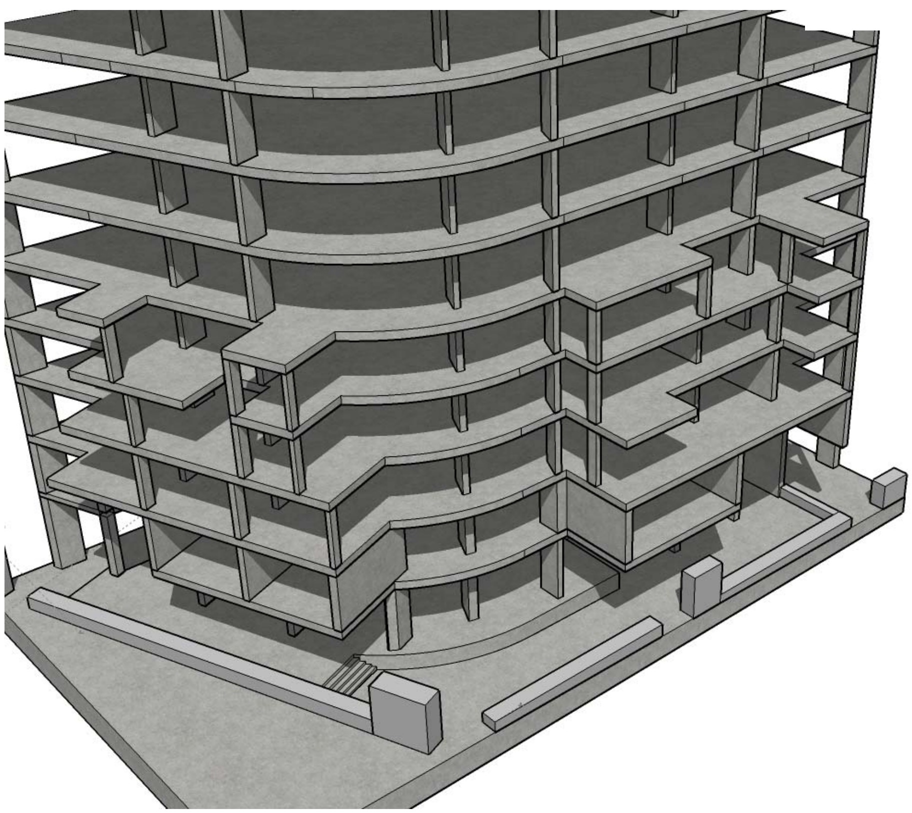


Figure 14: Model showing complicated balcony system

Lateral System

100 Eleventh Avenue's main lateral force resisting system is comprised of concrete shear walls located at the building elevator core, in combination with seven "long" columns, as shown in Figure 17 below. Because architectural constraints restricted the use of shear walls to the relatively small elevator core, the seismic loading necessitated that these seven columns also be designed to resist lateral forces. Two of these columns are connected to the main core via in-slab outrigger beams for additional stiffness. These 4' wide beams are reinforced with 11 #7 bars on both the top and bottom. The diaphragm connects the remaining columns to the building core. As lateral force is imposed on the building, the rigid floor distributes the forces to both the columns and shear walls, which in turn transfer the loads to the ground. The shear walls are typically 12" thick with #11 @12" E.F. vertically (Grade 75) and #6 @9" E.F. horizontally.

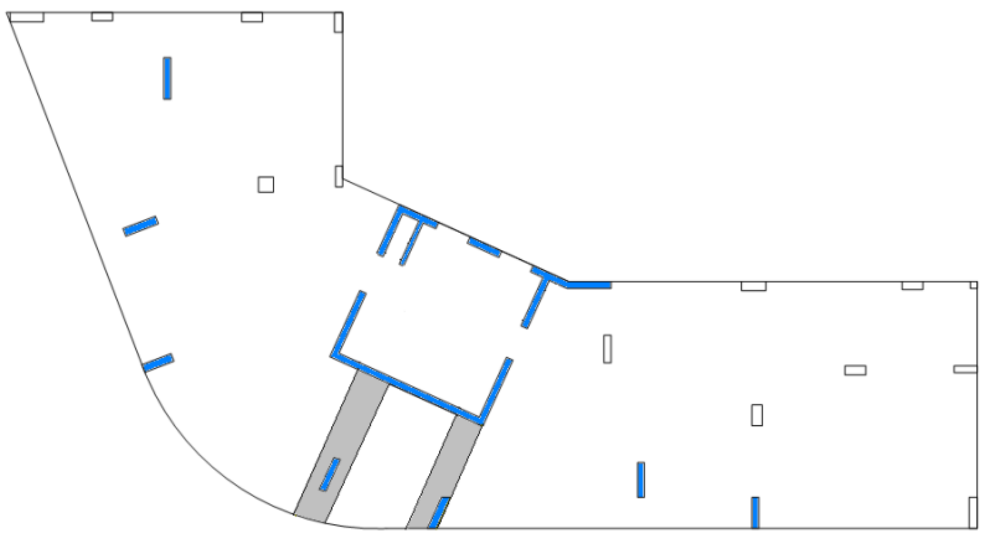


Figure 15: Lateral system with link beams denoted

Codes, Design Standards, & References

Used in original design

1968 New York City Building Code

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

ACI 318-99, Building Code Requirements for Structural Concrete

Used in thesis analysis & design

ASCE 7-05 Minimum Design Loads for Buildings and Other Structures

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary, 2008 Edition

Steel Construction Manual, American Institute of Steel Construction, 13th Edition

PCI Industry Handbook, 6th Edition

RS Means Assemblies Cost Data 2009

RS Means Facilities Construction Data 2009

Material Summary

Concrete	f'_c (ksi)
Foundations	5
Slabs	5.95
Columns supporting:	
- Cellar through 9th	8
- 9th through Roof	7
Shear Walls supporting:	
- Cellar through 9th	8
- 9th through Roof	7

Table 1

Reinforcement

- All #11 bars to be Grade 75 steel
- Vertical reinforcement in shear walls to be Grade 75
- Select column reinforcement to be Grade 75
- Remaining reinforcement is ASTM A615, Grade 60

Building Loads

Gravity Loads

Gravity Loads			
Description	**NYC Building Code**	**Design Load**	**ASCE 7-05 Load**
Typical Dead Load			
Normal-Weight Concrete	150 pcf		
Light-Weight Concrete	115 pcf		
Epoxy Terrazzo (3/8")	4 psf		
Superimposed Dead Load			
Partition	18 psf	18 psf	-
MEP	10 psf	10 psf	-
Live Load			
Residential	40 psf	40 psf	40 psf
Corridors	100 psf	100 psf	100 psf
Lobby	100 psf	100 psf	100 psf (1st Floor)**
Assembly	100 psf	100 psf	100 psf
Equipment Rooms	75 psf	75 psf	-
Balconies (exterior)*	60 psf	60 psf	100 psf
Additional Loads			
Planter	4.500 lb		
Curtainwall	500 plf		
* NYCBC requires exterior balconies to carry 150% of live load on adjoining occupied area, but not more than 100 psf			
** All remaining floors same as occupancy served			

Table 2

Curtainwall Load

The double façade system is connected to the concrete slab on levels 1 through 6 via Halfen channel anchors. Therefore, the weight of this complex curtainwall will need to be factored into the dead load of the structure. The structural engineers on the project assumed a 500 plf loading in their design. Once the individual façade reactions were received from the façade consultant, the initial design was checked and found to be sufficient. The 500 plf façade load will be used for initial computations.

Floor Systems

As previously stated, the intent of the floor systems comparison is to compare three alternative floor systems with the existing floor system, addressing items such as cost, least depth, self-weight, and constructability.

For all four systems, a single portion of the floor was chosen to be compared. Effort was made to redesign a portion of the floor representative of the typical floor as well as the thickened portion along the curved perimeter. In order to effectively redesign the floor in the alternative systems, several modifications and simplifications to the floor plan were necessary. They are graphically shown below.

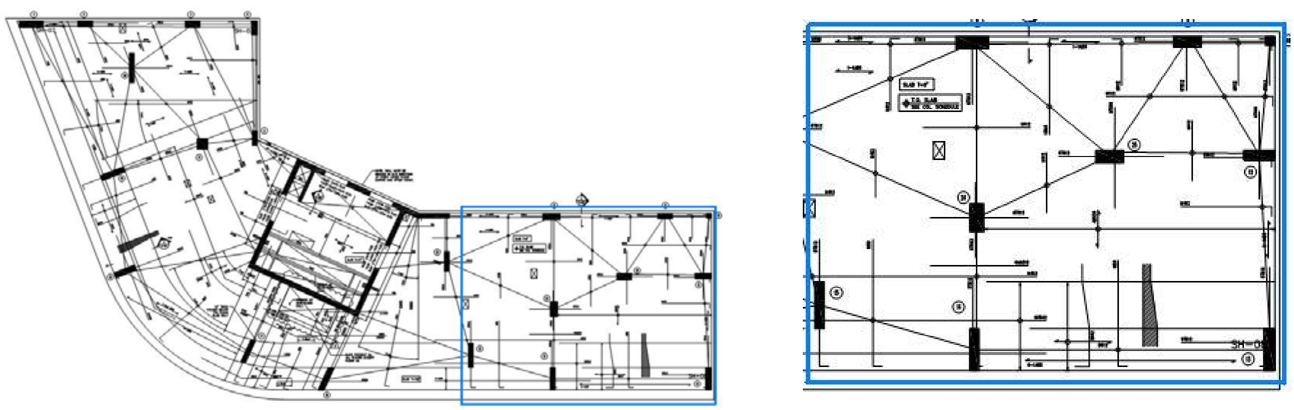


Figure 16: Existing Floor Plan

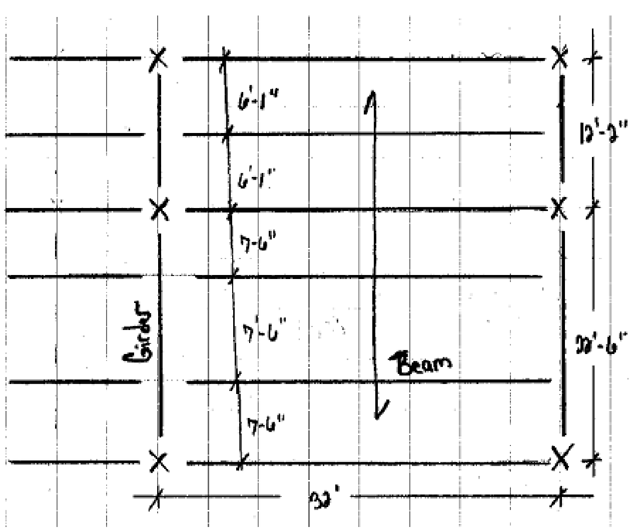


Figure 17: Plan used for Composite Steel design (identical column grid used for Hollow-core Plank design)

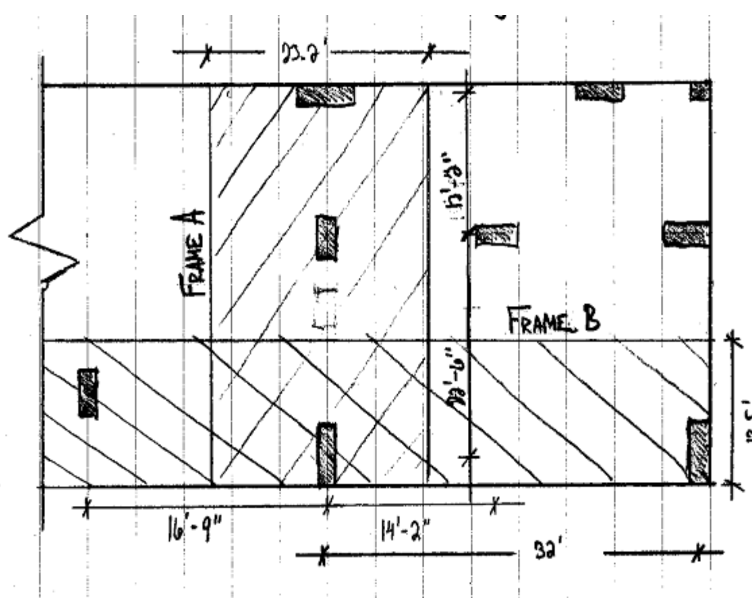


Figure 18: Plan used for Post-tensioned Flat Plate design

TWO-WAY FLAT PLATE SYSTEM – EXISTING

Description

The existing floor system for 100 Eleventh Avenue is a cast-in-place two-way concrete flat plate system. The typical slab thickness is 9", with a basic bottom reinforcing mat of #4 @ 12" E.W. Middle strip bars are also #4 @ 12" (top) unless otherwise noted. Column strip bars are primarily #6 @ 12" (top). Additional top and bottom bars are added where necessary. While no standard span exists, most slab spans range from 18'-23'.

A significant departure from this occurs along the curved perimeter of the building, where spans stretch to as long as 34 feet and imposed loads from the heavy curtain wall are supported. Here, the slab thickens to 18.5" along the entire perimeter, as seen in Figures 19 and 20 below.

A simplified spot check was performed using the Direct Design Method outlined in ACI 318. It was concluded that the 9" slab thickness is governed by deflection requirements, as much of the reinforcing provided is the minimum steel as required by code.

Advantages

The advantages to using a two-way flat plate system for a New York City residential building are numerous. The most important of these is the ease in which columns can be located according to the strict architectural requirements of a residential floor plan. No regular bay is necessary, and shifting columns from floor to floor is easily accommodated. Additionally, this system allows for minimal floor system thickness, another important facet of residential design. Construction is aided by the relative simplicity of the forming, as is evidenced by the two-day cycles accomplished on the regular floors. Because it can be formed into nearly limitless shapes, concrete is also the ideal building material for curved structures, such as 100 Eleventh Avenue. The use of concrete also aids in sound isolation between living units, and the smooth slab allows for the use of an exposed ceiling. Several other advantages to the flat plate system are the local contractors' familiarity with its construction and concrete's inherent fireproofing characteristics.

Disadvantages

Two-way flat plate systems, while ideal for moderate spans, are less efficient in longer spans. This is evident along the curved portion of 100 Eleventh Avenue, where the slab thickens to 18.5" to accommodate spans of up to 34'. This thickness could be reduced by using an alternative system. Punching shear, which typically governs the thickness of two-way flat plate slabs, is not an issue, as the large perimeters of the rectangular columns increase the slab area resisting shear.

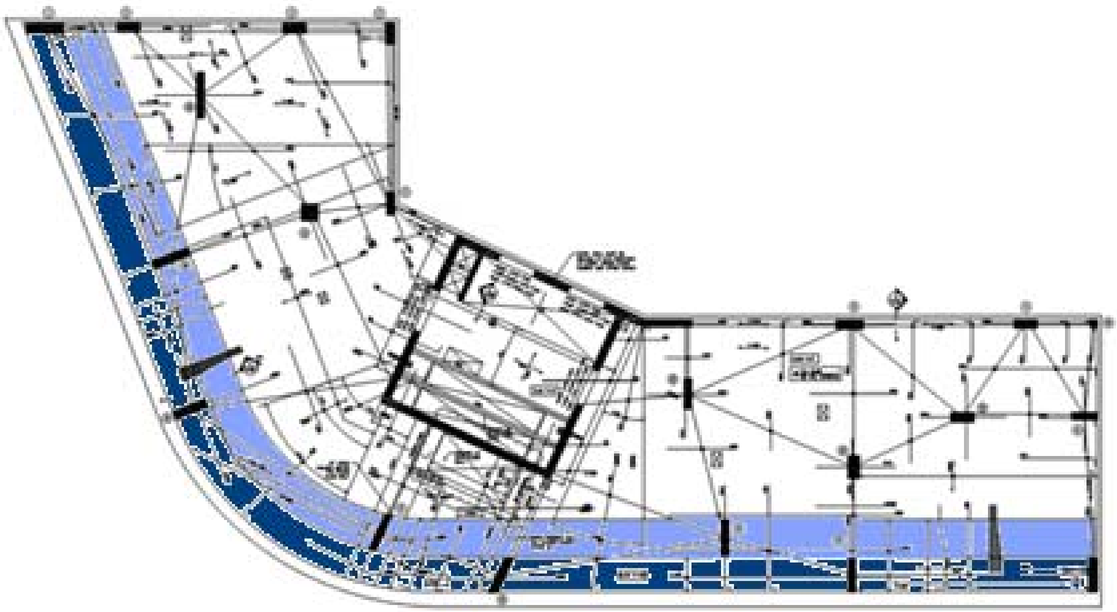


Figure 19: Typical Plan with thickened slab highlighted

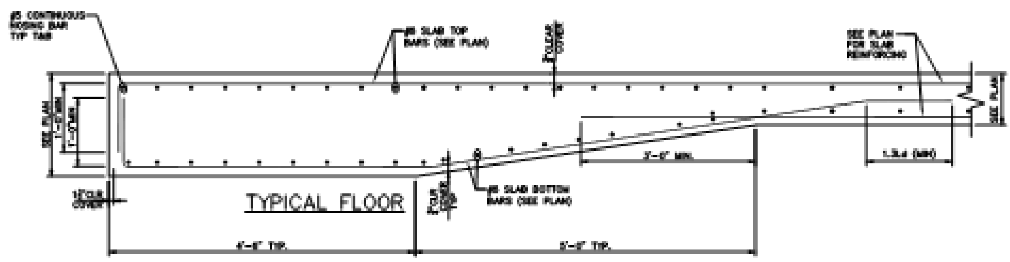


Figure 20: Detail of thickened slab at curved edge

COMPOSITE STEEL SYSTEM: ALTERNATIVE #1

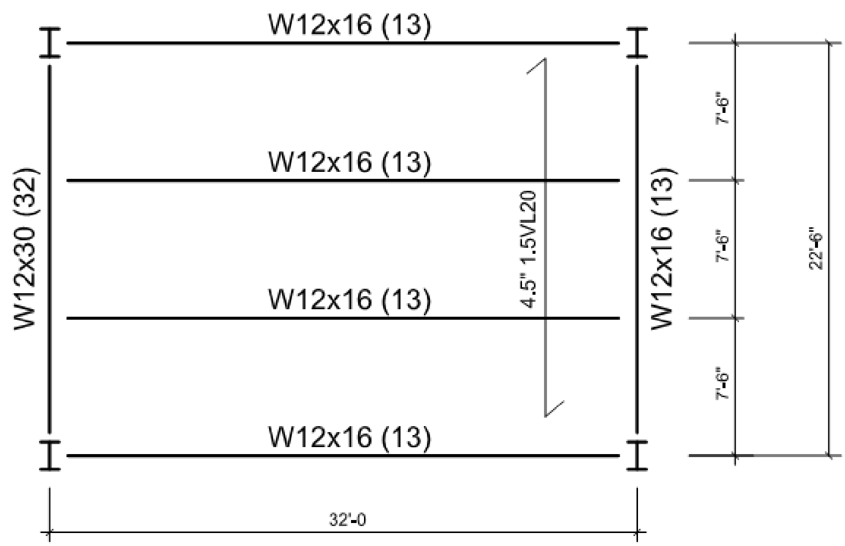


Figure 21: Composite Steel Layout

Description

This composite steel floor was designed using the modified bay shown above. One column was moved several feet plan north and two columns were removed entirely, allowing for a more orthogonal bay. Members were sized using AISC Steel Construction Manual. A 3-span deck was chosen from Vulcraft Group's deck catalog. Supporting calculations can be found in Appendix A.

Advantages

Composite steel's main advantage is its high strength-to-weight ratio, allowing it to span long distances. Whereas the slab in the existing design needed to more than double its thickness (from 9" to 18.5") for the 34' spans along the building perimeter, a W12 was sufficient for the steel beam. As can be seen in Figures 16 and 17, several columns were able to be removed, potentially allowing for more floor plan flexibility. Other general advantages include a lighter structure, which would significantly reduce the forces transferred to the foundation system, and the lack of need for formwork or shoring, as the chosen deck is capable of spanning an unshored distance of 9'-4".

Disadvantages

Steel framing is most efficient when regular, orthogonal bays are in use. 100 Eleventh Avenue's floor plans contain no such bays, as column layout is very irregular. In order to implement this system efficiently, many of the columns would require relocation, which would adversely affect the architecture. In residential buildings such as 100 Eleventh Avenue, column locations are almost entirely dependent on the space's architecture, leaving column relocation a poor option.

In addition, with the exception of the area around the building's perimeter, the composite steel floor system thickness of 16.5" is significantly thicker than the existing flat plate system thickness of 9". This is a very important matter within residential buildings, because a lower floor system thickness can allow for increased floor to ceiling heights, as well as the addition of floors while keeping the building height constant, both of which are very desirable in the eyes of a developer.

Also of concern is the interaction between the lateral system and floor system. Currently, concrete columns serve as part of the lateral system, meaning several moment frames would be required in the steel system, as steel columns would replace the existing concrete columns. Should concrete shear walls remain part of the lateral system, complications could also arise from the concrete-to-steel connections at the lateral system interface.

Further disadvantages include the need for additional fireproofing and the increased erection time, compared to the two-day cycle achieved on much of 100 Eleventh Avenue's present design.

POST-TENSIONED TWO-WAY FLAT PLATE SYSTEM: ALTERNATIVE #2

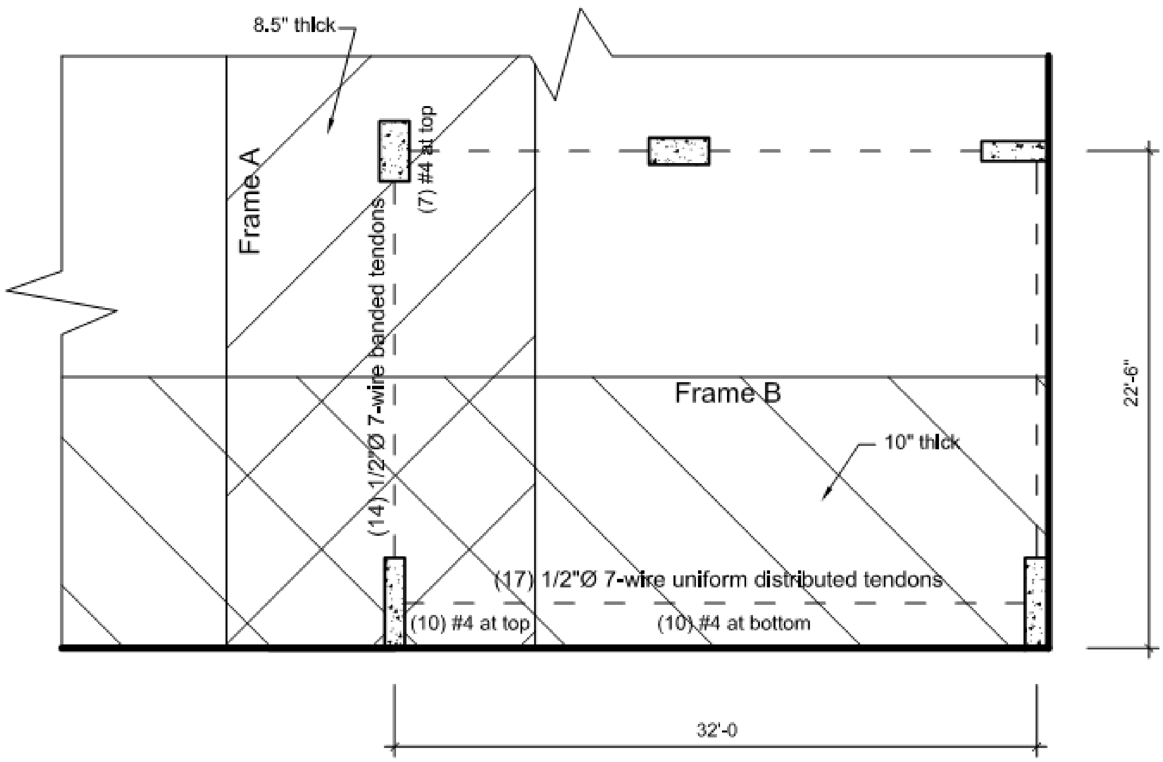


Figure 22: Post-tensioned Two-Way Flat Plate Layout

Description

In order to analyze a post-tensioned two-way flat plate system, a frame in each direction was chosen to be designed. Frame A is meant to represent the typical spans found in the building. Frame B was analyzed to determine if the thickened 18.5" existing slab could be reduced with the addition of post-tensioned tendons. A PCA design example was followed to conduct the design, as well as ACI 318-08. The tendons in the N-S direction, which contains the shorter spans, were chosen to be banded.

As can be seen in the above graphic, the design resulted in 14 tendons in the N-S direction and 17 tendons in the E-W direction. The N-S and E-W directions required a slab thickness of 8.5" and 10", respectively. Deflections were not considered in calculations and will be dealt with in future reports.

Advantages

The most evident advantage of this system is the reduced slab thickness, particularly along the curved perimeter. The addition of post-tensioned tendons to the floor system allows the slab thickness to be reduced from 18.5" to 10" along said perimeter, allowing for a more uniform ceiling profile, decreased building material, and decreased building weight. The majority of the floor slab thickness is ½" thinner. It is feasible that this thickness could be reduced further with additional post-tensioning, resulting in larger floor-to-ceiling heights.

This system also has many of the same advantages of the two-way flat plate system, including relative ease of concrete placement, fireproofing, and an attractive ceiling.

Disadvantages

Perhaps the most significant disadvantage of this system is the expertise needed in its construction. Specialty contractors will likely need to be brought into the project, as post-tensioned systems are not as common as flat plate systems. The more general construction-related problems of this system are similar to those in a flat plate system, and include the need for shoring and formwork.

Another disadvantage particular to this individual building is the lack of a regular column grid. Typically, tendons are banded in one direction and uniformly distributed in another. The banded direction typically spans between columns in the direction of the most regular column grid. Due to the lack of a regular grid, the placement of the banded tendons may be difficult. Also, the curved profile of the building necessitates that the tendons follow multiple geometric axes, much like the bonded reinforcing in the existing design. This will likely complicate the design and construction of the floor system. Another matter further complicating the tendon placement is the varying span lengths, which require the tendon drape to be increased or decreased in order to generate the same equivalent loads from span to span.

PRECAST HOLLOW-CORE SYSTEM: ALTERNATIVE #3

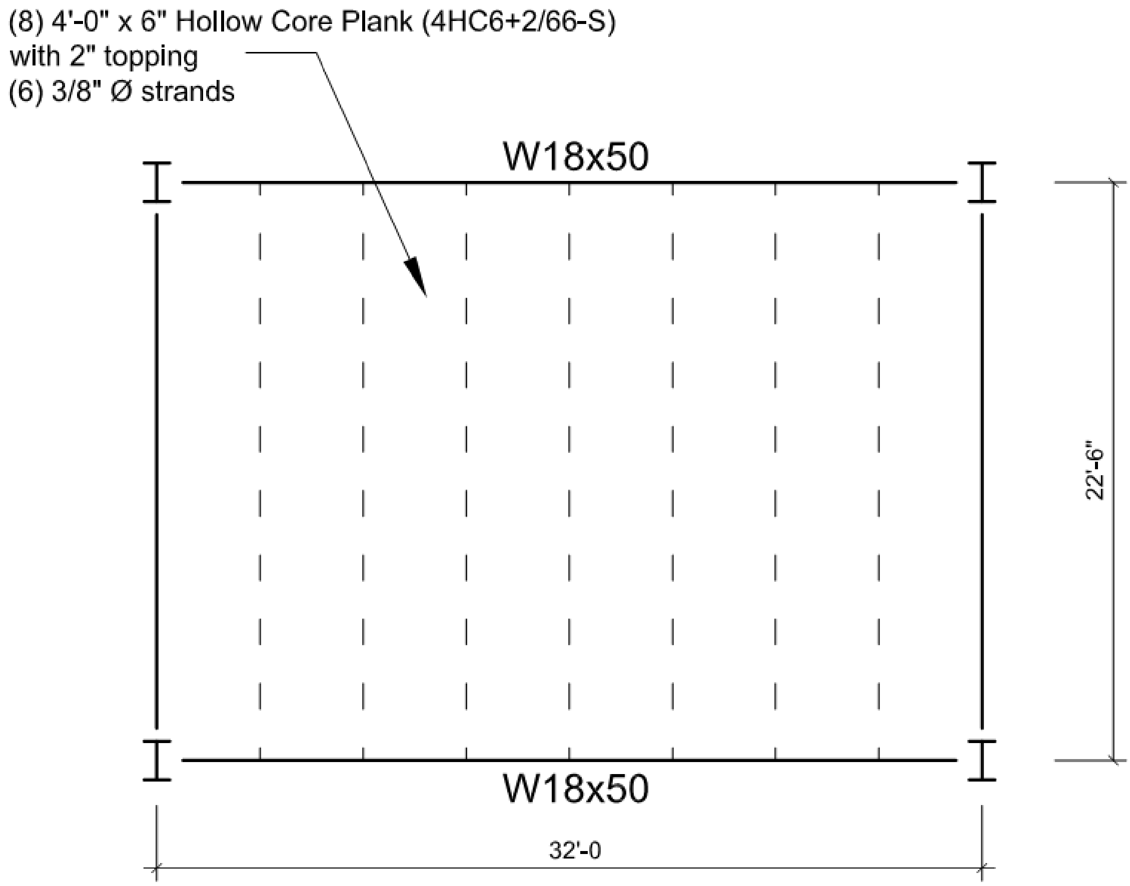


Figure 23: Precast Hollow-core Layout

Description

Using the same modified grid used in the composite steel alternative design, precast hollow-core planks were chosen from *PCI Industry Handbook, 6th Edition* based on a service load of 72 psf and a span of 22'-6". The shortest spanning direction was chosen for the planks.

Advantages

Precast hollow-core planks are capable of spanning long distances with a relatively small depth. Because the member is constructed off-site, construction is simple and quick. In addition, no additional fire proofing is necessary.

Disadvantages

The disadvantages of using this system largely outweigh the advantages. Much like steel, the most significant problem is the irregular column layout. Plank systems are advantageous with regular, repeating bays spaced in multiples of the plank width (4-feet in this case). Use of this system in 100 Eleventh Avenue would require significant relocation of columns. Specialty planks would also be required along the curved perimeter of the building.

Another significant disadvantage is the increased floor thickness. The 6" plank is topped with 2" of concrete and rests on a W18x50, resulting in a total system thickness of 26". It is worth noting that this thickness could be decreased by implementing custom systems in which the concrete plank frames into the side of the girder, rather than bearing on its top.

The problems resulting from the interaction between the steel girders/columns and the concrete lateral system remain the same as those outlined in the composite steel system. Additional disadvantages include long lead times and significant increases in cost.

System Comparison

Comparison Criteria	Two-way Flat Plate (Existing)	Composite Steel	Post-tensioned Two-way Flat Plate	Precast Hollow-core Plank
Cost/sf*	$19.90	$25.10	$19.00	$32.30
System Depth**	9" / 18.5"	16.5"	8.5" / 10"	26"
System Weight	125 psf / 231 psf	51 psf	106 psf / 125 psf	53 psf
Effect on Column Grid	N/A	Significant	Minimal	Signficant
Construction Difficulty	Moderate	Simple	Moderate/Difficult	Simple
Lead Time	Short	Long	Short	Long
Additional Fireproofing	No	Yes	No	No
Increased Vibration	N/A	Yes	No	Possible
Further Investigation	Yes	No	Yes	No

*System cost is a rough estimate taken from 2009 RS Means

**In concrete systems, depth for both typical slab and the portion along the curved perimter are listed

Table 3

Conclusions

Upon comparison of the criteria listed in Table 3 above, as well as the advantages and disadvantages discussed in the preceding sections, a post-tensioned two-way flat plate floor system is the most feasible alternative to the existing two-way flat plate design.

It is evident that the criteria governing the most suitable floor system for 100 Eleventh Avenue is the accommodation of the existing architecture. Because of the need to locate columns in very specific locations, with no regular spans or spacing, the use of either a composite steel or precast hollow-core plank system becomes very unrealistic. The lack of a regular grid and the need for transfer members makes the use of these systems very expensive and difficult to construct, almost ruling them out by this criteria alone. It's important to note that the already higher cost/sf of these two systems listed in Table 3 is based on a floor plan with regular bays and will only increase in price when implemented into 100 Eleventh Avenue's irregular layout. Also, the curved portion of the structure is much more easily formed by cast-in-place concrete than its more modular steel and precast counterparts. Further decreasing their

feasibility are the increased floor depth and lead time of the composite steel and precast plank systems. Therefore, these systems are impractical and will not investigated further.

The remaining cast-in-place options (one existing and one alternative) are both a form of two-way flat plate systems, and thus share many of the same advantages and disadvantages. When compared to the composite steel and concrete plank systems, both are less expensive, allow for a thinner profile, require little lead time, are significantly heavier, and have little to no effect on the column grid. However, when compared to each other, several differences stand out. The pre-stressed tendons in the post-tensioned two-way flat plate system allow for a system depth that is ½" thinner on the majority of the floor and 12.5" thinner on the exterior curved portion. This could result in higher floor-to-ceiling heights without increasing the overall building height. In addition, the weight of the post-tensioned system is lessened significantly, reducing the forces transferred to the foundation system, an important improvement considering the poor quality of the site's soil. In spite of these advantages, the rough cost estimate of the post-tensioned system is relatively equal to that of the non pre-stressed system.

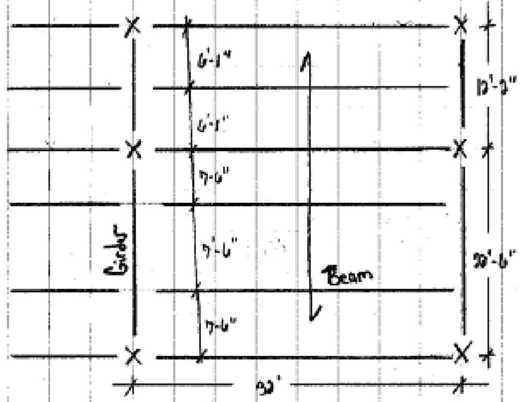
The main deterrent to using post-tensioning in this building is New York City contractor's unfamiliarity with pre-stressed construction. This would most likely increase the cost of its construction. Also, the layout of the pre-stressing tendons would be complicated by the building's curve. Because the tendons cannot follow the exterior curve, they will have to follow several linear geometric axes in order to effectively post-tension all parts of the floor.

As a result of this comparison, the post-tensioned two-way flat plate system described is worth future investigation as an alternative floor system. Because the slab along the curved perimeter of the building resulted in a much more drastic decrease in floor thickness as a result of the post-tensioning, it will also be worthwhile to investigate the effectiveness of post-tensioning only this portion of the floor system.

APPENDIX A

COMPOSITE STEEL

Alternate Floor System #1: Composite Steel



$f'_c = 5.95$ ksi
$f_y = 50$ ksi f_y (reinforcing) = 60ksi
$w_L = 40$ psf
$w_{SDL} = 32$ psf
Use 3" conc. above Vulcraft 1.5 VL20 deck (p. 48 Vulcraft Deck Catalog)
$w_{DL} = 45$ psf
$w_u = 1.2(32+45) + 1.6(40) = 156.4$ psf

Size Beams

$M = \frac{wl^2}{8} = \frac{(0.1564\text{ ksf})(7.5\text{ft})(32)^2}{8} = 150^{k}$

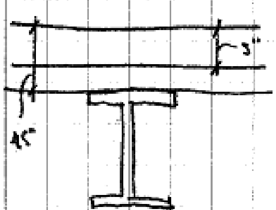


Table 3-19, AISC Manual
Assume $a = 1" \rightarrow Y2 = 4.5 - \frac{1}{2} = 4.0"$

Beam selfweight: $\frac{\left(\frac{16}{1000}\right)(32)^2}{8} = 2.05^{k}$
$150^{k} + 2^{k} = 152^{k} < \phi M_n \therefore$ OK

Choosing from W12's $\rightarrow$ W12 × 16, $\phi M = 158$, PNA @ 4, $\Sigma Q_n = 156$ k

$b_{eff} = 7.5' \Leftarrow$
$= \frac{32'}{4} = 8'$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{156}{0.85(5.95)(7.5 \cdot 12)} = 0.343 < 1" \therefore$ Assumption was conservative

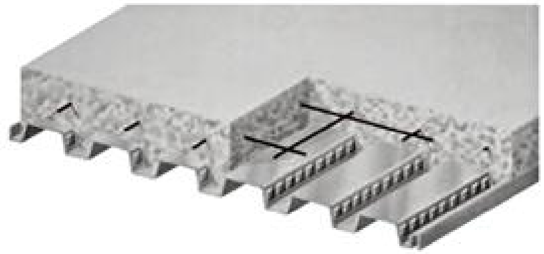
Assuming shear studs carry $12.0^{k} \rightarrow \frac{156^{k}}{12.0} = 13$ shear studs

Check beam under construction loads

$w_D = 45$ psf
Assume construction live load of $w_{LL} = 15$ psf
$w_u = 1.2(45) + 1.6(15) = 78$ psf
$M = \frac{(0.078)(7.5)(32)^2}{8} = 74.9^{k} < \phi M_p = 75.4^{k}$

No LTB because decking still provides bracing

VULCRAFT



SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd³ / 100 ft²	Theo. Concrete Volume ft³ / ft²	Recommended Welded Wire Fabric
3 1/2	0.78	0.211	6x6 - W1.4xW1.4
4	0.94	0.253	6x6 - W1.4xW1.4
4 1/2	1.09	0.294	6x6 - W1.4xW1.4
4 3/4	1.17	0.315	6x6 - W1.4xW1.4
5	1.24	0.336	6x6 - W2.1xW2.1
5 1/2	1.40	0.378	6x6 - W2.1xW2.1
5 3/4	1.48	0.398	6x6 - W2.1xW2.1
6	1.55	0.419	6x6 - W2.1xW2.1

(N=14.15) LIGHTWEIGHT CONCRETE (110 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	10'-6	11'-0	11'-6	12'-0
	1.5VL22	6'-4	8'-5	8'-6	278	247	222	185	167	152	139	124	105	89	76	66	57	50	44
3.50	1.5VL20	7'-8	9'-7	9'-11	305	271	243	220	201	184	154	135	114	97	83	72	62	54	48
(t=2.00)	1.5VL19	8'-8	10'-7	11'-0	329	292	262	237	216	198	173	145	122	104	89	77	67	58	51
26 PSF	1.5VL18	9'-6	11'-4	11'-9	350	311	279	252	230	211	184	153	129	110	94	81	71	62	54
	1.5VL16	9'-8	11'-5	11'-10	352	312	280	253	231	212	195	171	144	122	105	91	79	69	61
	1.5VL22	6'-0	8'-1	8'-1	324	288	258	215	194	177	161	148	136	126	113	98	85	75	66
4.5	1.5VL20	7'-3	9'-7	9'-9	355	315	283	256	233	195	178	164	151	140	123	106	92	81	71
(t=2.50)	1.5VL19	8'-2	10'-7	10'-11	382	339	304	275	251	230	212	178	164	152	131	113	99	86	76
30 PSF	1.5VL18	8'-11	11'-4	11'-5	400	360	323	292	266	244	225	209	175	162	139	120	104	91	80
	1.5VL16	9'-1	11'-4	11'-8	400	360	323	292	266	244	225	209	195	162	151	134	116	102	90
	1.5VL22	5'-9	7'-8	7'-8	372	330	275	246	223	202	185	170	156	145	134	125	116	106	93
4.50	1.5VL20	6'-11	9'-2	9'-4	400	361	324	293	246	223	204	188	173	160	149	139	129	114	101
(t=3.00)	1.5VL19	7'-9	10'-1	10'-5	400	388	348	315	287	264	221	203	188	174	162	151	140	122	107
35 PSF	1.5VL18	8'-6	10'-10	11'-0	400	400	369	334	305	279	258	239	200	186	173	161	147	129	114
	1.5VL16	8'-7	10'-10	11'-2	400	400	369	334	304	279	257	239	199	185	172	160	150	140	126
	1.5VL22	5'-7	7'-7	7'-7	396	352	293	263	237	216	197	181	167	154	143	133	124	115	108
4.75	1.5VL20	6'-9	9'-0	9'-1	400	385	345	312	262	238	218	200	184	171	159	148	138	129	118
(t=3.25)	1.5VL19	7'-7	9'-11	10'-3	400	400	371	336	306	281	235	216	200	185	172	160	150	140	126
37 PSF	1.5VL18	8'-3	10'-7	10'-9	400	400	393	356	324	298	274	231	213	198	184	171	160	150	133
	1.5VL16	8'-5	10'-7	11'-0	400	400	392	355	324	297	274	230	212	197	183	171	159	149	140
	1.5VL22	5'-6	7'-6	7'-5	400	374	311	279	252	229	209	192	177	164	152	141	131	123	115
5.00	1.5VL20	6'-7	8'-10	8'-11	400	400	367	332	278	253	231	212	196	181	168	157	146	137	128
(t=3.50)	1.5VL19	7'-5	9'-9	10'-1	400	400	394	356	325	273	250	230	212	197	183	170	159	149	140
39 PSF	1.5VL18	8'-1	10'-5	10'-7	400	400	400	378	344	316	291	245	226	210	195	182	170	159	149
	1.5VL16	8'-3	10'-5	10'-9	400	400	400	377	343	315	291	244	225	209	194	181	169	159	149
	1.5VL22	5'-2	7'-0	7'-0	400	400	367	329	297	270	247	227	209	193	179	166	155	145	135
5.75	1.5VL20	6'-2	8'-4	8'-5	400	400	400	362	327	298	272	250	231	214	199	185	172	161	151
(t=4.25)	1.5VL19	7'-0	9'-2	9'-6	400	400	400	400	383	322	295	271	250	232	215	201	187	175	165
46 PSF	1.5VL18	7'-7	9'-10	10'-0	400	400	400	400	400	372	314	289	267	247	230	214	200	188	176
	1.5VL16	7'-9	9'-10	10'-2	400	400	400	400	400	371	312	287	265	246	229	213	199	187	175

Notes: 1. Minimum exterior bearing length required is 1.50 inches. Minimum interior bearing length required is 3.00 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.

COMPOSITE

Check LL Deflection

$\Delta_{LL} \leq \frac{L}{360} = \frac{32 \cdot 12}{360} = 1.07''$

$\Delta = \frac{5}{384}\frac{wL^4}{EI} = \frac{5(0.04 ksf)(7.5 ft)(32 ft)^4 1728}{384(29000)(291)} = 0.839 < 1.07 \therefore ok$

$I_{LB} = 291$ (Table 3-20, AISC Steel Manual)

Exterior Beam has additional curtain wall load of 500 plf

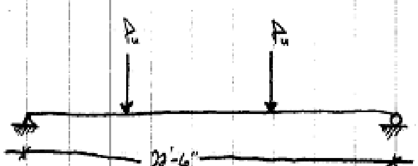
$w_u = 1.2(32 psf + 45 psf)(\frac{7.5}{2}) + 1.2(500 lb/ft) + 1.6(40)(\frac{7.5}{2}) = 1.19 k/ft$

$M_u = \frac{wl^2}{8} = \frac{(1.19)(32)^2}{8} = 152^{ik} < \phi M_n = 158^{ik} \therefore OK$

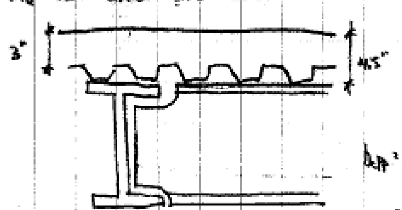
Beam remains a W12x16

Size Girder

$P_u = (156.4 psf)(7.5 ft)(32')/1000 = 37.5^k$



$M_u = Pa = 37.5^k \times 7.5' = 281^{ik}$



(Table 3-19, AISC Manual)

Assume $a = 1'' \rightarrow Y2 = 4.5'' - \frac{1}{2} = 4.0''$

Use W12x26, $\phi M_n = 290$, PNA @ TFL, $\phi M_p = 140^{ik}$, $\Sigma Q_n = 382^k$

$b_{eff} = 32'$
or
$= \frac{22.5}{4} = 5.625' \Leftarrow$

$a = \frac{382}{0.85(5.95)(5.025 \cdot 12)} = 1.11'' >$ assumed value $\therefore$ No Good

Assume $a = 2'' \rightarrow Y2 = 4.5'' - 1'' = 3.5''$

Use W12x30, $\phi M_n = 300^{ik}$, PNA @ 2, $\phi M_p = 162^{ik}$, $\Sigma Q_n = 308^k$

$a = \frac{308}{0.85(5.95)(5.025 \cdot 12)} = 1.08'' <$ assumed value of $a = 2'' \therefore ok$

Self Weight: $\frac{wl^2}{8} = \frac{(0.030)(22.5)^2}{8} = 1.90^{ik}$

$M_u = 283^{ik} < \phi M_n \therefore ok$

Check girder under construction load

$P_{u,const} = (0.078 \text{ ksf})(7.5)(32) = 18.7^k$

$M_{u,const} = Pa + \frac{wl^2}{8} = (18.7^k)(7.5) + \frac{(0.03)(22.5)^2}{8} \cdot 1.2 = 143^{k} < \phi M_p = 162^k \therefore ok$

# of shear studs

$\frac{382^k}{12} = 32$ studs

Δ_{LL}

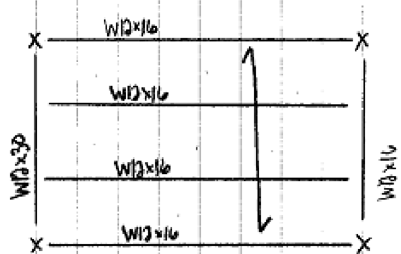
$\Delta_{LL} \le \frac{L}{360} = \frac{22.5 \cdot 12}{360} = 0.75$

$\Delta = \frac{PA}{24EI}(3l^2 - 4a^2) = \frac{(9.6)(7.5)(3 \cdot 22.5^2 - 4 \cdot 7.5^2)(1728)}{24(29000)(613)} = 0.377'' < 0.75'' \therefore ok$

$P_{live} = 0.04 \text{ psf} \times 7.5 \times 32' = 9.6^k$

$I_{18} = 613$

Exterior Girder would get half the load $\longrightarrow M_u = \frac{281}{2} = 141^{'k}$
Assume W12x16 chosen for beams ($\phi M_n = 158^{'k}$) will work here



Weight Calc's

W12x16: 16 lb/ft ÷ 7.5' = 2.13 lb/ft

W12x30: 30 lb/ft ÷ 16' = 1.88 lb/ft

W12x16 (N-S): 16 lb/ft ÷ 16' = 1.0 lb/ft

concrete & deck: 45 psf (Vulcraft)

shear studs: approximately 75 shear studs go to this panel; assume 1 lb of steel/stud $\rightarrow$ (75 studs × 1 lb/stud)/(32' × 22.5) = 1.04 lb/ft

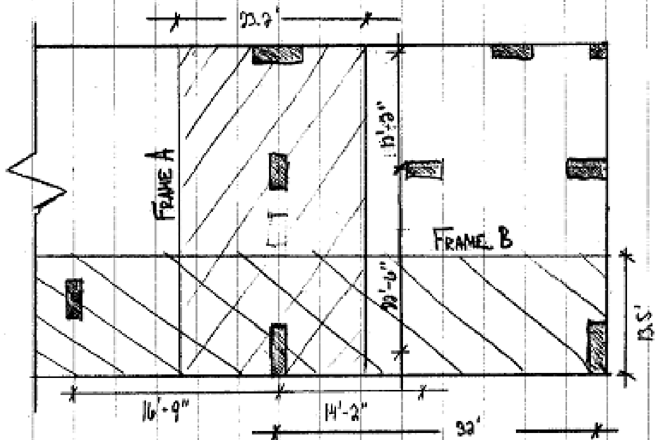
TOTAL = (2.13 + 1.88 + 1.0) + 45 + 1.04 = 51 psf

APPENDIX B

POST-TENSIONED TWO-WAY FLAT PLATE

Alternative Floor System #2: Two-Way Post-Tensioned Concrete

Use same simplified bay developed in composite steel calc's, for preliminary sizing



Loads

Framing DL = self weight

Superimposed Dead Load = 32 psf (partitions, M/E, misc)

Live Load = 40 psf

Materials

Concrete: Normal Weight 150 pcf

$f'_c = 5{,}950$ psi

$f'_{ci} = 3{,}600$ psi (assumed)

Rebar: $f_y = 60{,}000$ psi

PT: unbonded tendons

½" ø, 7-wire strands, $A = 0.153\ in^2$

$f_{pu} = 270$ ksi

Estimated prestress losses = 15 ksi (ACI 18.6)

$f_{se} = 0.7(270\ ksi) - 15\ ksi = 174\ ksi$ (ACI 18.5.1)

$P_{eff} = A * f_{se} = (0.153)(174\ ksi) = 26.6$ kips/tendon

Determine Preliminary Slab Thickness

Start with $L/h = 45$

longest span = 32' (E-W direction)

$h = (32')(12)/45$

$= 8.53" \rightarrow$ Try 8.5" preliminary slab thickness

Loading

$DL = (8.5")(150 pcf) = 106 psf$

$SDL = 32 psf$

$LL_o = 40 psf$

Design of N-S Interior Frame

Assumptions

- Frames are representative of all floor spans in given direction
- maximum moments at midspans & supports
- No live load reduction
- Column stiffness ignored
- No pattern loading since $\frac{L}{D L} < \frac{3}{4}$ (ACI 13.7.6)
- total bay width between centerlines $= \frac{3}{4}(16'-4") + \frac{3}{4}(14'-2") = 23.2'$ [3/4 instead of 1/2 used in an attempt to replicate the actual frame A_T]

Section Properties: Class U (ACI 18.3.3)

$A = b \cdot h = (23.2 \cdot 12)(8.5) = 2366 in^2$

$S = bh^2/6 = (23.2 \cdot 12)(8.5)^2/6 = 3352 in^3$

Allowable Stresses:

At time of jacking: $f'_{ci} = 3,600 psi$

(ACI 18.4.1) compression $= 0.60 f'_{ci} = 0.60(3600) = 2160$

tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3600} = 180$

At service loads: $f'_c = 5950 psi$

(ACI 18.4.2(a) 18.3.3) compression $= 0.45 f'_c = 0.45(5950) = 2678 psi$

tension $= 6\sqrt{f'_c} = 6\sqrt{5950} = 463 psi$

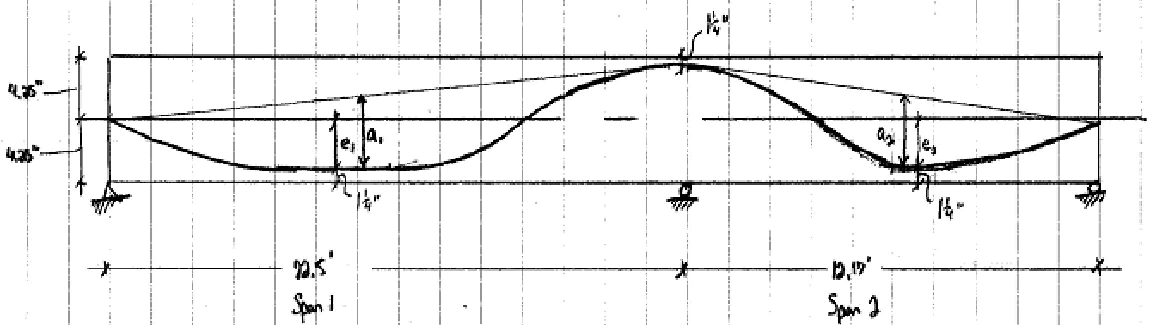
Average Precompression Limits: $P/A = 125 psi$ min.

$= 300 psi$ max.

Target Load balances:

- balance self weight & SDL = 32 psf
- assume N-S direction (short direction) takes 70% of DL = 0.70(106+32) = 97 psf (30% by long direction)

Cover requirements: Assume $1\frac{1}{4}$" to centroid of tendon for 2-hour fire rating



$e_1 = 4.25 - 1.25 = 3"$

$a_1 = \text{drape } 1 = 3" + 3"/2 = 4.5"$

$e_2 = 4.25 - 1.25 = 3"$

$a_2 = \text{drape } 2 = 3" + 3"/2 = 4.5"$

$$\text{Effective prestress} = P_{e_{a,1}} = \frac{w_b L^2}{8a} = \frac{(97\text{ psf})(23.24)(22.5)^2}{8(4.5)} = 379{,}755 \text{ lb}$$

$$\# \text{ of tendons} = \frac{380^k}{26.6\text{ k/tendon}} = 14.3 \text{ tendons} \rightarrow \text{Use } 14 \text{ tendons}$$

$$P_{actual} = 14(26.6) = 372.4^k$$

$$\text{Balanced Load} = w_b = \left(\frac{372.4}{380}\right)(97\text{ psf}) = 95 \text{ psf}$$

$$\text{For smaller span: } w_b = \frac{P_{ea_2} \cdot 8 \cdot a}{L^2} = \frac{(372.4)(8)(4.5)}{(12.17)^2} = 7.54 \text{ k/ft} \rightarrow 325 \text{ psf}$$

→ * Note: much too large → assume drape will be adjusted to allow for equal w_b on each span.

$P_{eff} = 372.4^k$

Stage 1: Stresses Immediately After Jacking

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

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

Span 1

$f_{top} = (-134 + 92.4)/3352 - 157 = -306 < 2160 \therefore ok$

$f_{bot} = (134 - 92.4)/3352 - 157 = +99.3 < 190 \therefore ok$

Span 2

$P_{top} = (-7.5 + 5.10)/3352 - 157 = -165 < 2160 \therefore ok$

$P_{bot} = (7.5 - 5.16)/3352 - 157 = -148 < 2160 \therefore ok$

Support

$P_{top} = (151 - 104)/3352 - 157 = 11.3 < 18.0 \therefore ok$

$P_{bot} = (-151 + 104)/3352 - 157 = -325 < 2160 \therefore ok$

Stage 2: Stresses at service load

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

$P_{bot} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

Span 1

$P_{top} = (-134 - 38.9 + 92.4)/3352 - 157 = -445 < 2678$
$P_{bot} = (134 + 38.9 - 92.4)/3352 - 157 = 131 < 463$
} ok

Span 2

$P_{top} = (-7.5 - 2.17 + 5.16)/3352 - 157 = -173 < 2678$
$P_{bot} = (7.5 + 2.17 - 5.16)/3352 - 157 = -141 < 2678$
} ok

Support

$P_{top} = (151 + 43.7 - 103.8)/3352 - 157 = 168 < 463$
$P_{bot} = (-151 - 43.7 + 103.8)/3352 - 157 = -485 < 2678$
} ok

All stresses are within permissible ok limits

Ultimate Strength (By inspection, span 1 is critical over span 2)

Primary PT Moments

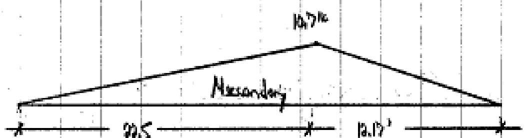
$M_1 = P \cdot e$

$e = 0$ in at ext. support

$e = 3.0$ in @ support

$M_1 = (372.4)(3/12) = 93.1$ k

$M_{sec} = M_{bal} - M_1 = 103.8 - 93.1 = 10.7$ k



$M_u = 1.2(134) + 1.6(38.9) + 1.0(5.35) = 228$ k
midspan

$M_{u_{support}} = 1.2(-151.5) + 1.6(-43.2) + 1.0(+10.7) = -240$ k

Determine min. bonded reinforcement

(+) Span 1: $f_t = 131 < 2\sqrt{f'_c} = 2\sqrt{5950} = 154$ psi $\rightarrow$ No positive reinf. required (ACI 18.9.3.1)

– Negative Moment Reinforcement (at support) $\rightarrow$ $A_{s,min} = 0.00075 A_{cf}$

$A_{cf} = \max 8.5\text{in} \left[\left(\frac{16.75 + 14.13}{2}\right), \left(\frac{22.5 + 12.0}{2}\right) \right] \times 12$

$= 1795 \text{ in}^2$

$A_{s,min} = 0.00075(1795) = 1.35 \text{ in}^2 \longrightarrow 7 \#4, A_s = 1.4 \text{ in}^2$

Max spacing is 12" o.c.

Check Ultimate Strength

$M_n = (A_s f_y + A_{ps} f_{ps})(d - \frac{a}{2})$

d = effective depth

$A_{ps} = 0.153 \text{ in}^2 \text{ (14 tendons)} = 2.14 \text{ in}^2$

$f_{ps} = f_{se} + 10000 + \frac{(f'_c b d)}{(300 A_{ps})}$ for slabs w/ $\frac{L}{h} > 35$

$= 174{,}000 + 10{,}000 + \frac{(5950 \cdot 23.2 \cdot 12)\, d}{300 \cdot 2.14} = 184000 + 2580\, d$

$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f'_c b)$

At supports:

$d = 8.5" - 1.25" = 7.25"$

$f_{ps} = 184{,}000 + 2580(7.25) = 202{,}705$

$$a = \frac{(2.14 \cdot 202{,}705) + (1.4 \times 60{,}000)}{(0.85)(5950)(23.2 \cdot 12)} = 0.368$$

$$\phi M_n = 0.9\,(1.4 \times 60 + 2.14 \times 202.705)\left(7.25 - \frac{0.368}{2}\right) = 274^{\text{'k}} > M_u = 240^{\text{'k}} \therefore \text{ok}$$

At midspan

$d = 7.25"$

$f_{ps} = 202.7$

$$a = \frac{(2.14 \cdot 202{,}705)}{(0.85)(5950)(23.2 \times 12)} = 0.309 \therefore \text{ok}$$

$$\phi M_n = 0.9\,(2.14 \cdot 202{,}705)\left(7.25 - \frac{0.309}{2}\right) = 230^{\text{'k}} > M_u = 228^{\text{'k}} \therefore \text{ok}$$

Design of E-W Exterior Frame

Note: This exterior frame has an additional loading of 500 plf (curtain wall). This load will be assumed to be resisted by entire exterior frame.

$w_{curtain\,wall} = \frac{(500\ plf)}{13.5'} = 37.0\ psf$

Section Properties: Class U

$A = b \cdot h = (13.5 \times 12)(10) = 1620\ in^2$

$S = bh^2/6 = (13.5 \cdot 12)(10)^2/6 = 2700\ in^3$

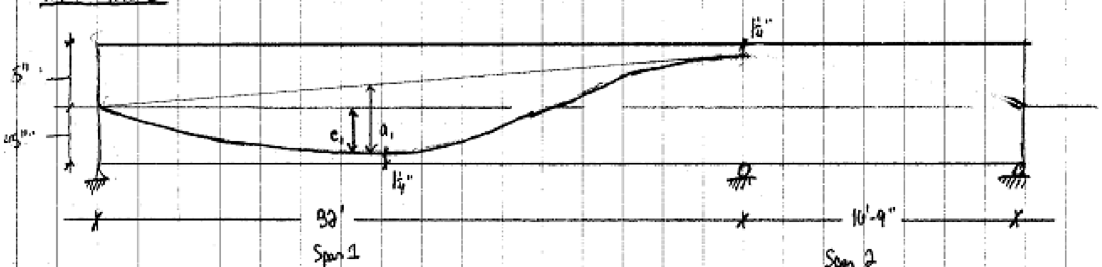
Allowable Stresses remain the same as in N-S direction

Self weight = 10/12 · 150 = 125 psf

Target Load Balances

· assume E-W (long direction) takes 70% of (DL + curtain wall load) = 0.75 (125 + 30 + 37) = 146 psf

Try 10" thickness



$e_1 = 4.5 - 1.25 = 3.75"$

$a_1 = \text{drape } 1 = 3.75" + \frac{3.75}{2} = 5.63"$

Effective Prestress $= P_{e_1} = \frac{w_b L^2}{8a} = \frac{(146)(32)^2(13.42)}{8(5.63/12)} = 445^k$

$\#\text{ of tendons} = \frac{445^k}{26.6\ k/\text{tendon}} = 16.7 \rightarrow 17\ \text{tendons}$

$P_{actual} = 17(26.6) = 452^k$

Balanced Load $= w_b = \left(\frac{452}{445}\right)(146) = 148\ psf \rightarrow 1.99\ k/ft$

Span 2

Assume drape will be adjusted to attain similar w_{bal} with $P_{actual} = 452$

$a = \frac{w_b L^2}{8P} = \frac{(146)(10.75)^2(13.42)}{8(452)} = 1.82"$

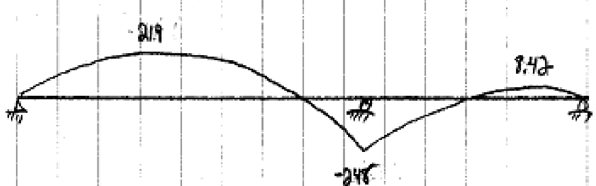
Drape to do this is $a = 1.82"$

Precompression Stress: $\frac{P_{actual}}{A} = \frac{(452{,}000)}{1020} = 379$ psi $\left.\begin{array}{l} < 300 \text{ psi max} \\ > 125 \text{ psi min} \end{array}\right\}$ $\therefore$ Ok

<u>Check Slab Stress</u>

Dead Load Moments

$w_{DL} = (125+32+37)(13.42)/1000 = 2.60$ k/ft



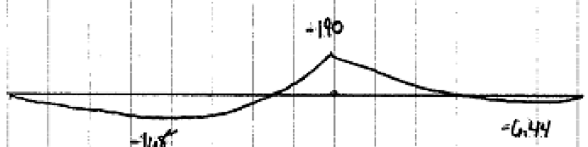
Live Load Moments

$w_{LL} = (40)(13.42)/1000 = 0.537$ k/ft

From SAP analysis

Total Balancing Moment

$w_{bal} = 1.99$ k/ft



Stage I: Stresses Immediately After Jacking

Span 1 is critical span (Span 2 not checked)

Midspan

$f_{top} = (-219 + 168)/2700 - 279 = -505 \leq 2160 \therefore ok$

$f_{bot} = (219 - 168)/2700 - 279 = -252.3 < 2160 \therefore ok$

Support

$f_{top} = (248 - 190)/2700 - 279 = -21.2$

$f_{bot} = (-248 + 190)/2700 - 279 = -537$

$< 2160 \therefore ok$

Stage II: Stresses at Service Load

Midspan

$f_{top} = (-219 - 48.2 + 168)/2700 - 279 = -707 < -2678 \therefore ok$

$f_{bot} = (219 + 48.2 - 168)/2700 - 279 = 149 < 463 \therefore ok$

Support

$f_{top} = (248 + 51.3 - 190)/2700 - 279 = 206.8 < 463 \therefore ok$

$f_{bot} = (-248 - 51.3 + 190)/2700 - 279 = -765 < 2678 \therefore ok$

All stresses are within permissible code limits

Ultimate Strength (By inspection, span 1 is critical)

Primary PT Moments

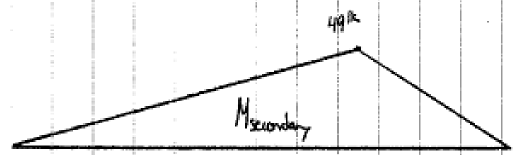
$M_1 = Pe$

$e = 6$ @ ext. support

$e = 3.75''$ @ int. support

$M_1 = (452)(3.75/12) = 141^{k}$

$M_{sec} = M_{bal} - M_1 = 190 - 141^{k} = 49^{k}$



$M_u = 1.2(219) + 1.6(45.2) + 1.0(49) = 384^{k}$
midspan

$M_{u,support} = 1.2(-248) + 1.6(-51.3) + 1.0(49) = -331^{k}$

Determine Minimum Bonded Reinforcement

(+) span: $f_t = 149\,psi < 2\sqrt{f'_c} = 154\,psi$ → No positive reinf. req'd

(−) support: $A_{s,min} = 0.00075\,A_{cf}$

$A_{cf} = \max\ 10in\ [(13.5),(20.5)] \times 12 = 2700$

$A_{s,min} = 0.00075(2700) = 2.03\,in^2$ ⟶ 10 #4s, $A_s = 2.0\,in^2$

Max spacing = 12''

Check Ultimate Strength

$M_n = (A_s f_y + A_{ps} f_{ps})(d - \frac{a}{2})$

$A_{ps} = 0.153(17) = 2.60\,in^2$

$f_{ps} = 174{,}000 + 10{,}000 + \dfrac{(5930 \cdot 13.40 \cdot 12)\,d}{300 \cdot 2.60} = 184{,}000 + 1208\,d$

$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 \cdot f'_c\, b)$

At supports

$d = 10'' - 1.25 = 8.75''$

$f_{ps} = 184000 + 1228\,(8.75) = 194{,}745$

$$a = \frac{(2.60 \cdot 194{,}745) + (2 \cdot 60000)}{(0.85 \cdot 5950 \cdot 13.40 \cdot 12)} = 0.769 \text{ in.}$$

$$\phi M_n = 0.9\left(\frac{2.6 \cdot 194745 + 2 \cdot 60000}{12000}\right)\left(8.75 - \frac{0.769}{2}\right) = 393^{k} > M_u = 331 \therefore ok$$

At Midspan

$$a = \frac{(2.60 \cdot 194{,}745)}{(0.85 \cdot 5950 \cdot 13.42 \cdot 12)} = 0.622 \text{ in.}$$

$$\phi M_n = 0.9\left(\frac{2.6 \cdot 194745}{12000}\right)\left(8.75 - \frac{0.622}{2}\right) = 320^{k} > M_u = 384^{k} \therefore NG \rightarrow \text{need bonded reinf.}$$

Use min. reinf. specified in negative moment @ support $\longrightarrow$ 10 #4's, $A_s = 2.0 \text{ in}^2$

Then $\phi M_n = 393^{k} > M_u = 384^{k} \therefore ok$

APPENDIX C

PRECAST HOLLOW-CORE PLANK

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
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

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

$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

		Untopped		Topped	
A	=	187	in.²	283	in.²
I	=	763	in.⁴	1,640	in.⁴
y_b	=	3.00	in.	4.14	in.
y_t	=	3.00	in.	3.86	in.
S_b	=	254	in.³	396	in.³
S_t	=	254	in.³	425	in.³
wt	=	195	plf	295	plf
DL	=	49	psf	74	psf
V/S	=	1.73	in.		

4HC6

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

Strand Designation Code	Span, ft																				
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	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	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	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
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	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
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		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		
		0.2	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	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			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	0.4	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	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			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
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

PCI Design Handbook/Sixth Edition
First Printing/CD-ROM Edition

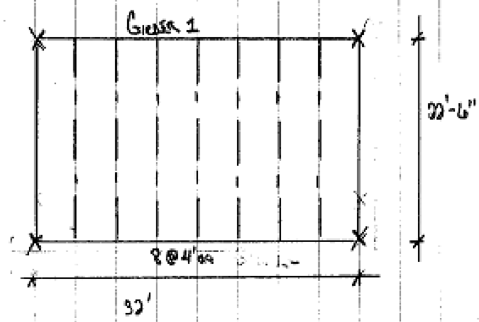
2–31

Alternate Floor System #3: Hollow Core Floor Slab

Using same simplified bay as that used in composite steel design:

LL = 40 psf
SDL = 32 psf

w_o = 40 + 32 = 72 psf
- Span is 22.5'
- no adjustment needed in selected bay to fit 4' module size



From PCI Industry Handbook ⟶ 6" Hollow-Core plank w/2" topping, (6) 3/8 ø straight strands, 66-S
Safe Superimposed service load of 85 psf @ 23' span > 72 psf ∴ ok
DL = 49 psf

8 @ 4' o.c.

Size Girder

Trib Width of Girder 1: $w_t = \frac{22'6"}{2} + \frac{12'2"}{2} = 17.3'$

w_{DL} = 49 psf
w_{SDL} = 32 psf
w_{LL} = 40 psf

$w_u = 1.2(49+32) + 1.6(40) = 161$ psf

$W_u = (161\text{ psf})(17.3')/1000 = 2.79$ k/ft

$M_u = \frac{w_u l^2}{8} = \frac{(2.79)(32)^2}{8} = 357^{\text{'k}}$ Assume braced against lateral movement

W21×44, $\phi M_p = 358^{\text{'k}}$

S.W. → $M_u = \frac{(1.2 \cdot 0.044)(32)^2}{8} = 6.76$

$M_u = 364^{\text{'k}}$ ⟶ W18×50, $\phi M_p = 379^{\text{'k}} > 364$ ∴ ok

Deflection

$I_x = 800 \text{ in}^4$

$\Delta_{LL} \leq \frac{L}{360} = \frac{32 \cdot 12}{360} = 1.07''$

$\Delta_{LL} = \frac{5}{384} \frac{wL^4}{EI} = \frac{5}{384} \frac{(0.04 \text{ ksf} \cdot 17.3)(32 \text{ ft})^4 (1728)}{(29000)(800)} = 0.704 \text{ in.}$

$\Delta_{LL} < 1.07''$ ∴ ok

Use W 18×50 girder w/ hollow core plank (4HC6 +2/66-S)

System Weight

plank: 49 psf

girder: $1.5 \times 50 \text{ plf} \div 22.5' = 3.33 \text{ psf}$

parallel beams: assume 1.0 psf

Total = $49 + 3.33 + 1.0 = 53 \text{ psf}$

APPENDIX D

SYSTEM COSTS

Floor Systems Cost

From RS Means Assemblies Cost Data, 2009

New York, New York: Location Cost Factor

	Mat	Inst	Total	
Shell superstructure	102.1	164.6	130.7	(p500)

Two-way Flat Plate p117

25×25 bay size, 9" slab thickness → 7.15 + 8.05 = \$15.20/sf → × 1.307 = **\$19.9/sf**

Composite Beam, Deck & Slab p97

25×30 bay size, 1'-11-½" → 14 + 5.20 = \$19.20/sf

19.20/sf × 1.307 = **\$25.1/sf**

RS Means "Facilities" Construction Cost Data, 2009

Post-tensioned Two-way Flat Plate

Ungraded Strand, 300k, 100' span → \$1.51/lb p78

Cast-in-place Concrete, flatplate, 35' span → \$483.40/cy

Strands: (1.51 \$/lb)(0.520 lb/ft)(2·14 tendons · 22.5' + 2 · 17 tendons · 32')

= \$1349 → \$1349/(32' × 22.5') = \$1.87/sf

Concrete: (\$483.40/cy)$\left(\frac{8.5 \div 12}{3}\right)$ = \$114 /sq yd ⟹ \$114/sy ÷ 9 sf/sy

= \$12.67/sf

Total = 12.67 + 1.87 = \$14.54/sf × 130.7 = **\$19.0/sf**

Precast Plank

Precast Beam & Plank w/ 2" topping

25×30 bay \$24.70/sf

(* assume that the cost of precast t-beams and wide flange are similar)

24.70 × 1.307 = **\$32.3/sf**