100 Eleventh Avenue

New York, New York

Tyler Graybill | Structural Option
Consultant: Professor Thomas Boothby

Wednesday, April 7th 2010

Final Report

100 Eleventh Avenue
New York, New York

Tyler E. Graybill
Structural Option

The Building
Location: Manhattan's West Chelsea District
Function: Residential
Size: 170,000 sf
No. of Stories: 23 (1 below grade)
Dates of Construction: September 2007 through early 2010

The Players
Owner: Cape Advisor Inc. in partnership with
Alf Naman Real Estate Advisors
Design Architect: Ateliers Jean Nouvel
Executive Architect: Beyer Blinder Belle
Construction Manager: Gotham Construction
Structural Engineer: DeSimone Consulting Engineers
MEP Engineer: AKF Engineers
Geotech Engineer: Langan Engineering

The Architecture
- Ultra-luxury condominium building with 55 units
- 6000 sf of street-level retail
- Curved facade composed of 1650 uniquely sized and oriented windows
- Second facade offset 16' towards street on lower 6 floors creating unique "hanging garden" space

The Construction
- Curtain wall anchored to slab by Halfen channels
- Slab curve achieved through use of rubber transition piece embedded in formwork
- Curved slab edge detailed with straight rebar with one bend to save costs

The Structure
- Cast-in-place concrete with two-way flat plate floor system
- Typical slab (6ksi) thickens from 9" to 18.5" on curved edge to support facade loads and 35' spans
- Cantilevered balconies supported by cantilevered columns and posttensioned Dywidag bars
- Columns transfer at 19th floor via 18" transfer slab and at 3rd floor by up to 5-feet deep transfer beams
- Lateral system comprised of concrete core shear walls and 7 "long" columns up to 25' in length
- Secant wall system used in lieu of foundation walls
- 36" pressure slab spans between piles and caissons

The MEP System
- Conventional heat pump loop system with 800 GPM cooling tower on roof and 2000 MBtu/hr boiler in cellar
- Perimeter radiant floor panels at facade
- Units serviced by 120/208V 1-phase electric panels

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Last, but certainly not least, a special thanks is due all the friends and family who provided much needed love and encouragement throughout this year.

Executive Summary

The following report is the result of a year-long study conducted of 100 Eleventh Avenue and alternate designs for portions of its structural system. 100 Eleventh Avenue is a 22-story, 148,000 sf residential building located in Manhattan's West Chelsea District, containing 6,000 sf of street-level retail space in addition to its 55 condominium units. Its defining feature is its facade, a panelized curtainwall system consisting of 1650 windows, each a different size and uniquely oriented in space. The building's superstructure is cast-in-place concrete, with a two-way flat plate floor system. Lateral loads are resisted by core shear walls and seven long columns.

Alternate designs for two aspects of 100 Eleventh Avenue's structural system were developed. The first of these was the redesign of the building's perimeter slab strip. Due to spans as long as 34 feet and the addition of the glass facade load, the slab was thickened from 9" to 18.5" at this portion of the floor to limit deflections to 1", a requirement given by the facade consultant. The redesign successfully reduced the slab thickness to the 9" thickness found throughout the majority of the floor by post-tensioning this slab strip in one direction with 16 ½" Ø 7-wire strands. Due to site restrictions and architectural restraints, only Floors 7 through 21 can be efficiently post-tensioned. Through a construction management study, it was determined that this post-tensioned redesign reduces the building weight by 5.2%, reduces the cost of the superstructure by $180,000, and will require 18 additional days to construct. Thus, this design is a very viable option that improves the interior space while reducing the cost of the structure.

The second aspect studied was an alternate design for the 19th level transfer system. The current design transfers the load carried by three columns via an 18.5" slab reinforced by #10's @ 6" o.c. each way and on both top and bottom of slab. In an effort to reduce the material usage and cost, an alternate system of (5) conventionally-reinforced transfer beams was developed. However, the loads and spans were such that deflection limitations and shear/torsion reinforcement requirements could not be met without violating strict floor-to-ceiling heights. Additional criteria rendered this alternate design unsatisfactory, including a worsened exposed soffit appearance and an insignificant reduction in cost of $15,000.

In addition to the described structural system alternate designs, a breadth study of shading strategies used in 100 Eleventh Avenue was conducted. The implementation of exterior shading was studied as a more effective solution to stopping unwanted direct solar gain from penetrating the glass facade. By extending the facade mullions outward a distance of 3'-2" at every level, the amount of sunlight entering the south-facing windows in the summer would be significantly reduced, while still admitting desirable solar gains in the winter. Despite the increased performance of this proposed shading strategy, however, the impact on the building's aesthetics would likely be too drastic, rendering this an unfavorable solution.

Introduction to 100 Eleventh Avenue

100 Eleventh Avenue is a 22-story, 170,000 sf condominium building located in Manhattan's Chelsea District, a neighborhood adjacent to the Hudson River which 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, including the IAC 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 facade, 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 facade 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 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

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".

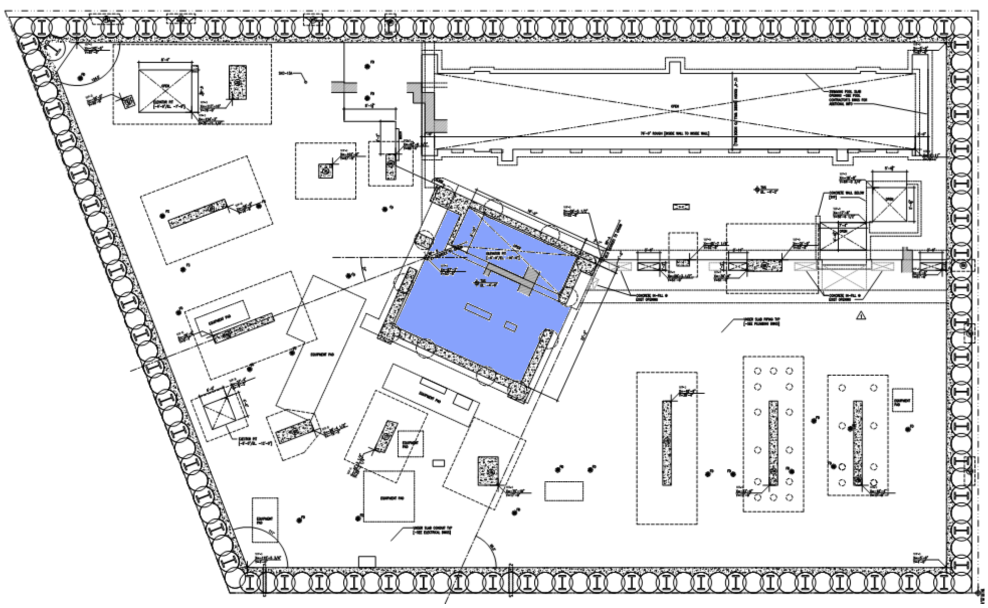


Figure 3: Cellar plan with core denoted

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.

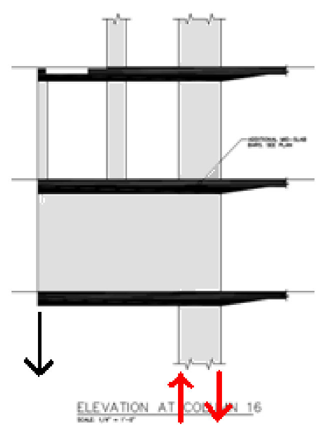


Figure 4: Cantilevered column creating tension in piles

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 accommodating 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 used where necessary, likely due to atypical loads and spans. 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 clear spans as long as 34 feet, the slab thickens from 9" to 18.5" along the curved perimeter portion of the building. For appearances, the slab gradually increases in thickness over a distance of 5'-0", as seen in Figure 6 & 7, rather than undergoing an abrupt increase.

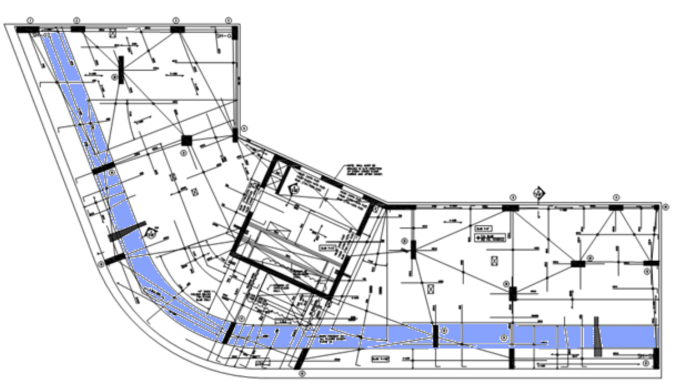


Figure 6: Typical plan with slab thickness transition area highlighted

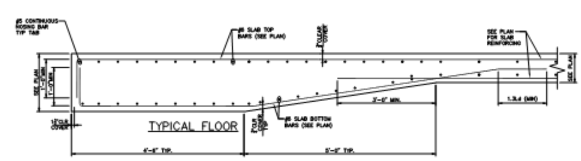


Figure 7: Detail of thickened slab at curved edge

On the lower six floors, balconies begin to cantilever out towards the second street facade. An example of this is shown in Figure 8, where the balcony on the 6th floor 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 facade and tree loads, three post-tensioned high-strength Dywidag bars were used, highlighted in green.

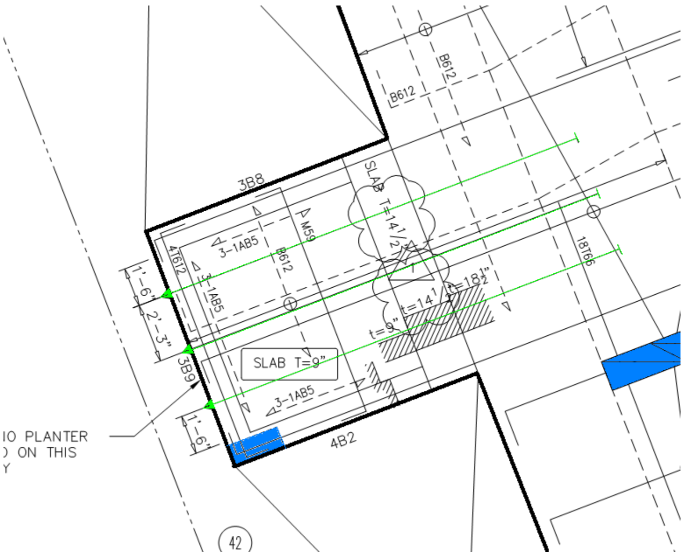


Figure 8: Cantilevered balcony utilizing post-tensioning

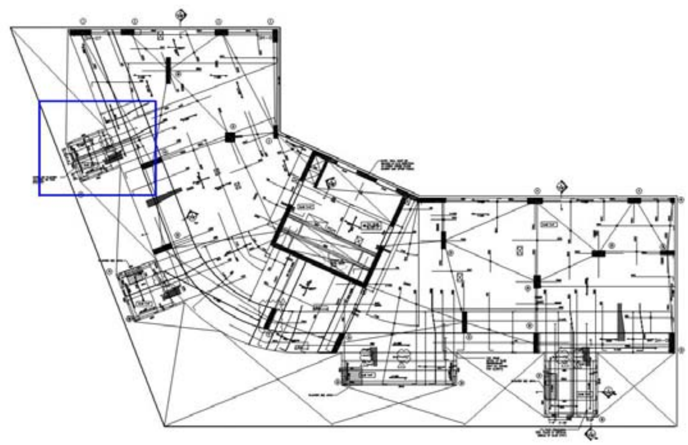


Figure 9: 6th Floor Plan

On the 19th floor, the building sets back 13 feet on the east side, and several columns transfer, as shown in Figure 10. The gravity forces carried by these columns are transferred via an 18.5" thick transfer slab, reinforced with #10 @6" E.W. on both top and bottom of slab.

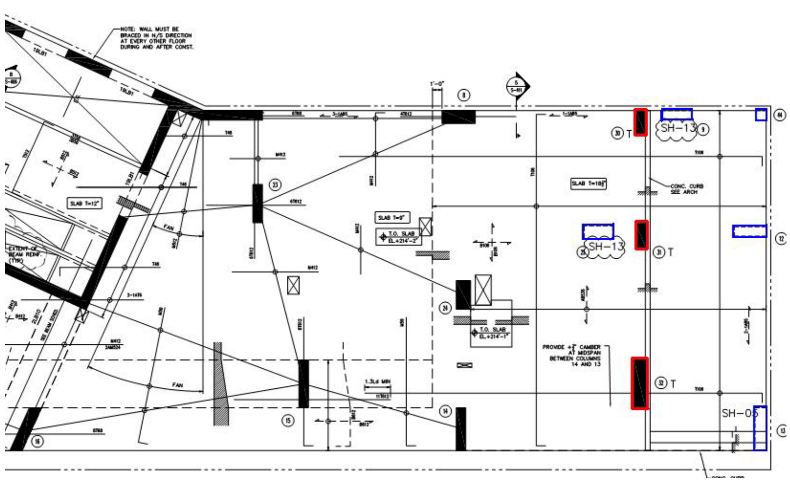


Figure 10: 19th floor transfer slab with red denoting terminated columns from above and blue denoting new column locations on the 18th level below

Columns

Concrete strength for columns supporting the cellar level through the 9th level is 8 ksi; those supporting the 10th through the roof have 7 ksi concrete. 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'

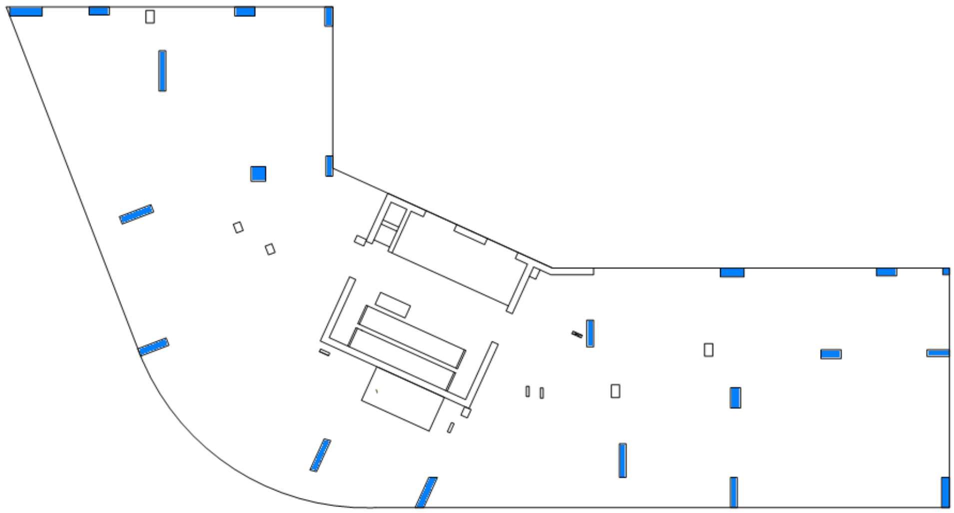


Figure 11: Typical floor column layout

exist. Column sizes range widely throughout a single floor, as well as from floor to floor. The 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, seen in Figure 15. On these floors, intermittent boxes protrude out from the inner facade to meet the outer street facade, which is offset 16' towards the street. On the second level, six of the long columns transfer the balcony system loads by cantilevering outwards 18' to 28', allowing for the column-free space between the double facade system at street level, shown in Figure 1 above. Figure 13 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 tops of the columns into the main portion of the slab.

Figure 12: Photo showing portion of cantilevered balcony system

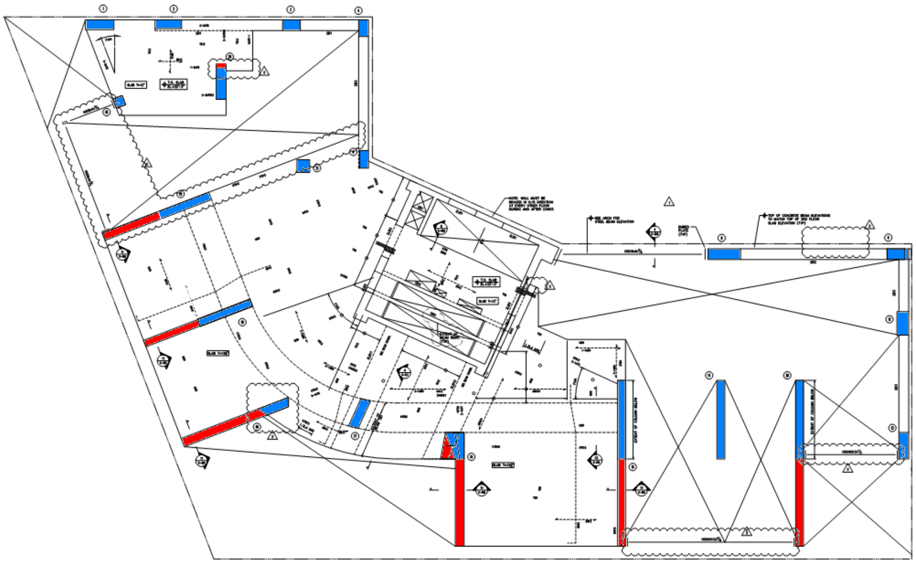


Figure 13: 2nd Floor column layout

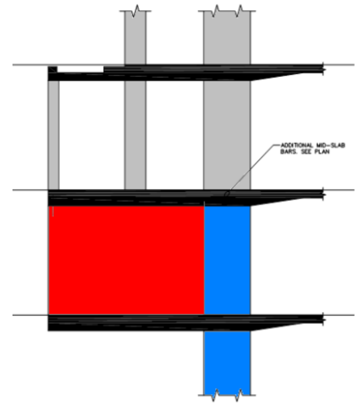


Figure 14: Cantilevered Column Elevation

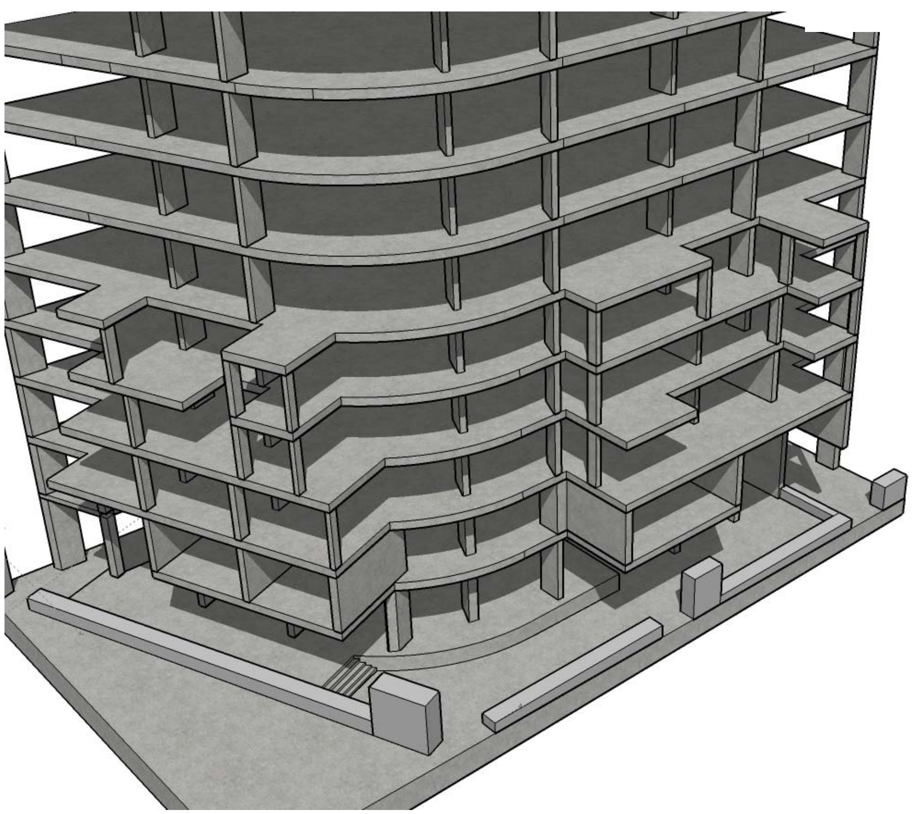


Figure 15: Model showing complicated balcony system

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 16 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.

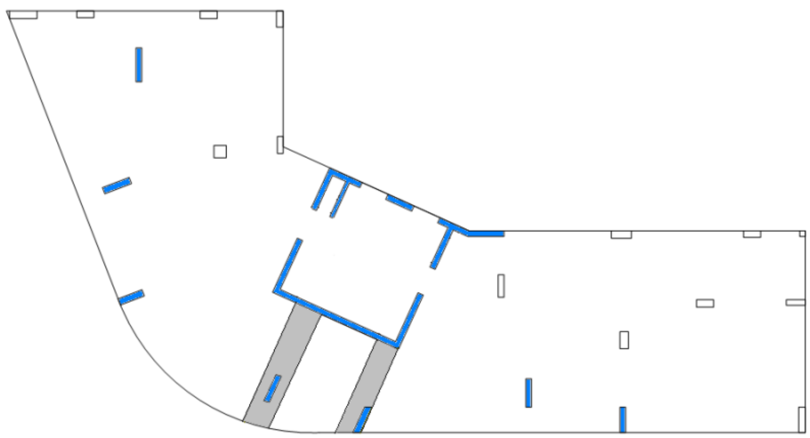


Figure 16: Lateral system with link beams denoted

Structural System Redesign – Proposal

The intent of this report is to investigate two aspects of 100 Eleventh Avenue's structure that may benefit from an alternate system design. These two aspects are the thickened perimeter slab and the 19th floor transfer system. The advantages and disadvantages of the existing designs are looked at in the *Problem Statement* section below. The following *Proposed Solution* section explains the alternate designs that will be investigated.

Throughout the design of these alternate systems, the following will serve as the design requirements that need to be satisfied:

- **Strength (e.g. flexure, shear, torsion)**
- **Service (e.g. deflections)**
- **Architecture (e.g. floor-to-floor heights, column locations)**

The specifics of these requirements will be elaborated on in each system's respective section. Once complete, the designs' ultimate success will be based upon criteria such as material savings, cost, architectural impacts, and impacts on the construction schedule.

Problem Statement

The floor system of 100 Eleventh Avenue must be designed to resist gravity forces due to live load, superimposed dead loads such as partitions and mechanical equipment, and the self weight of the structure. A reinforced 9"-thick concrete slab is sufficient throughout the majority of each level, where typical clear spans range from 18' to 23'. On the street-facing perimeter, however, the concrete slab must span lengths of up to 34', while supporting an additional 500 plf load from the panelized facade system. Additionally, the two-way floor system is weaker along its edge due to the lack of stiffening edge beams. To accommodate this, the slab thickens from 9" to 18.5" at the perimeter. This solution provides for a practical means of strengthening the slab along the perimeter, yet has several negative effects, such as increased weight and increased material usage. Perhaps more importantly, the interior architecture of these high-end units is negatively affected as a result of decreased floor-to-ceiling heights at the building perimeter and an unappealing appearance (partially compensated for by gradually increasing the slab thickness over a distance of 5' rather than undergoing an abrupt increase).

Additionally, on the 19th level a 13-foot offset on the building's east side requires several columns to shift as they move from the 19th to 18th level. In the existing design, the gravity loads carried by the terminated columns on the 19th level are transferred to the columns below via the slab. Much like at the perimeter, the slab at this portion of the building is thickened to 18.5" in order to transfer these forces. In addition to an increased thickness, this transfer slab is heavily reinforced, with #10 @ 6" each way on both top and bottom of slab. While this transfer system requires minimal formwork, it uses substantial material quantity and is a very heavy solution.

Figure 17: View of transition from 9" slab to 18.5" slab

Figure 18: Cantilevered slab as part of balcony system

Figure 19: Heavily-reinforced 19th floor transfer slab

Proposed Solution

Post-tensioning the slab perimeter in one direction will be investigated as an alternative to the existing solution as a means of resisting the increased loads and spans found at the building perimeter. Based on results from Technical Report #2, it is possible to reduce the perimeter slab thickness to 10" using prestressed steel. Further design concepts will be explored with the ultimate goal of reducing the slab thickness at the perimeter to the 9" used throughout the rest of the floor. The portions of the perimeter slab that extend outwards as part of the balcony system will be analyzed to ensure that the thinned slab, in combination with the existing post-tensioned Dywidag bars, will still provide sufficient strength and deflection control.

On the 19th floor, an alternative transfer system composed of post-tensioned beams will be used in lieu of the heavy transfer slab. To preserve floor-to-ceiling heights, the maximum depth of these beams will be the existing slab thickness of 18.5". See Figure 20 for an early schematic sketch showing potential orientations of transfer beams. Because the columns do not lie in a grid, any orientation of a beam supported by two columns will likely have significant torsional forces that will need to be designed for. The ultimate goal of this redesign is to significantly reduce the cost and material usage of the transfer system without affecting the architecture or lengthening the construction schedule significantly.

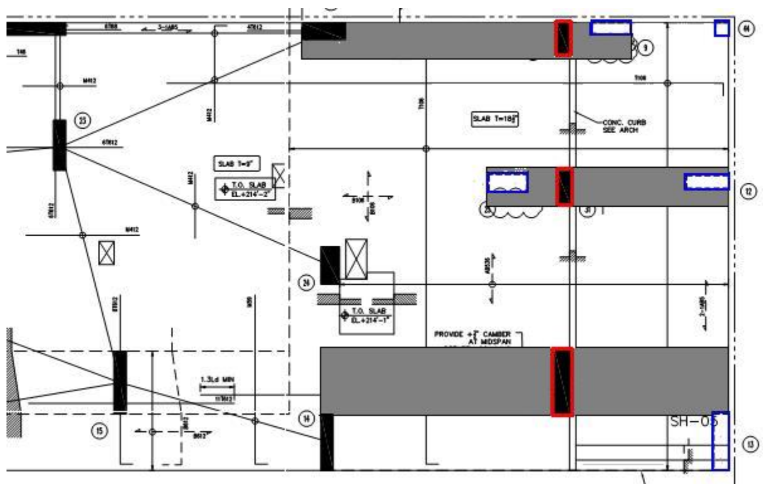


Figure 20: Schematic beam layout on 19th Floor with red denoting terminated columns and blue denoting new column locations on 18th level below

The redesign of the slab perimeter/balcony as well as the 19th floor column transfer system will be compared to the existing design using criteria such as material and labor savings, weight, improvements to interior space, and construction feasibility.

M.A.E. Resources:

Due to the irregularity of the building's shape and the lack of any regular column grid, structural analysis software will be relied upon to accurately analyze 100 Eleventh Avenue's floor system. RAM Concept, a 3D finite element method analysis program for elevated slabs was chosen as this software. Because of the importance of understanding how a computer program produces results, concepts learned in AE 597A: *Computer Modeling* will be drawn upon to learn, use, and understand the analysis software. These concepts include the behavior of truss, beam, frame, and grid elements, and the interpretation of computer analysis results. In particular, understanding the theory of finite element analysis and how best to mesh a structural element proved to be very valuable in using this FEM software. The use of this program is intended to fulfill the MAE requirement for the senior thesis capstone project.

Design Criteria

Codes & Design Standards Used

Existing design for 100 Eleventh Avenue's structural system utilized the following codes and standards:

- **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***

For the purposes of the senior thesis capstone project, the following codes and standards were used in all system redesigns:

- **ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures***
- **ACI 318-08, *Building Code Requirements for Structural Concrete***

Floor System Gravity Loads

Tables 1 & 2 below tabulate the loads assumed to act on the floor systems for their redesign. Live loads were taken from ASCE 7-05.

Floor System Loads	
Description	**Load**
Normal-Weight Concrete	150 pcf
Superimposed Dead	52 psf
Live Load	40 psf
Glass Curtainwall	500 lb/ft
Masonry Curtainwall	750 lb/ft
Planter	4500 lb
Balcony Live Load (exterior)*	60 psf

*NYCBC requires exterior balconies to carry 150% of live load on adjoining occupied area, but not more than 100 psf

Table 1

Superimposed Dead Load		
Item	**pcf**	**psf**
MEP	**-**	**10**
Partitions	**-**	**18**
LWC leveling slab (2")	**115**	**20**
Epoxy Terrazzo (3/8")	**-**	**4**
Total		**52**

Table 2

Slab Perimeter Redesign

Understanding the Existing Design

Before starting design of the post-tensioned slab perimeter, the existing design was studied in order to more fully understand the floor system and to determine what exactly required the 18.5" slab thickness. Preliminary hand calculations were performed by treating a portion of the 9.5'-wide thickened slab strip as an equivalent frame. Moments at mid-span and supports were generated via moment distribution, with results ranging from 1.5 ft-k per foot-width of slab to 48 ft-k per foot-width of slab. Using the common flexural design formula of

$$Mu \leq \phi\rho f_y bd^2 * (1 - 0.59\rho \frac{f_y}{f'_c})$$

and substituting for ρ the maximum ratio that still allows for a tension-controlled member, a minimum depth, *d*, of 9.4" is required. Thus, a 12"-thick slab would be sufficient to satisfy strength requirements for this preliminary approximation, and one could surmise that a 9"-thickness would suffice if the stiffening effects of the rest of the structure on the perimeter were included.

To determine how deflection limitations shaped the existing design, it was necessary to utilize a computer program, as hand calculations treating the perimeter strip as an isolated equivalent frame would ignore the significant stiffening effects of the rest of the attached structure. Therefore, a typical floor was modeled without the slab thickness increase in RAM Concept, a finite element method (FEM) analysis program for elevated slabs. This program was chosen because other programs which utilize the more traditional Equivalent Frame Method are difficult to use on a building such as 100 Eleventh Avenue, with little to no regularity in its column grid. By developing a finite element model of the entire slab, Concept can predict the elastic behavior of the slab much more accurately than frame models. RAM Concept's deflection results are shown below for both the existing design and without the thickness increase.

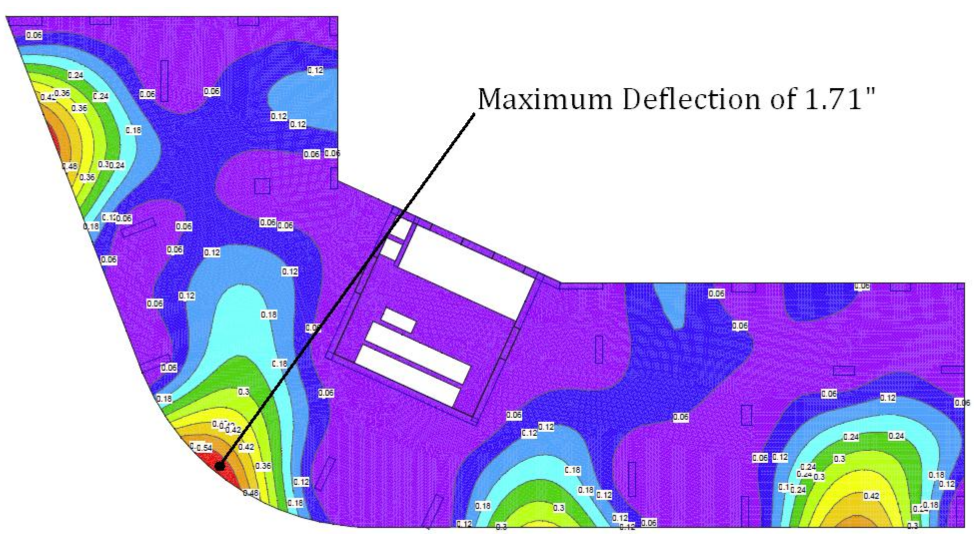


Figure 21: Typical Floor without Thickened Perimeter - Deflection Plan

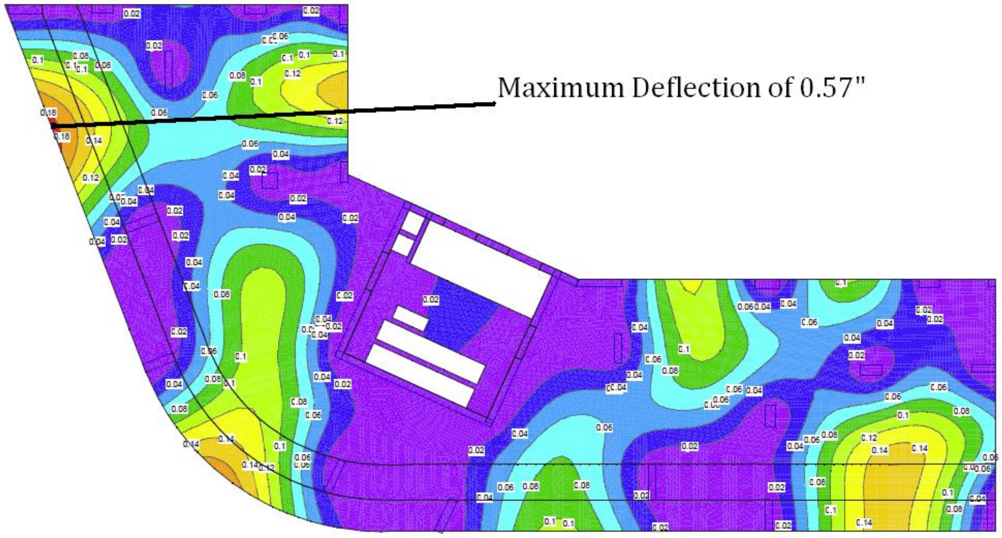


Figure 12: Typical Floor (Existing) - Deflection Plan

100 Eleventh Avenue's structural engineers were limited by the facade consultant to a 1" deflection for any slab edge supporting the glass curtain wall. From the above deflection plots, it is obvious that without the thickened slab perimeter, deflection limitations are not met, which is most likely the reason for the thickened perimeter. Also notice that, with the help of the colored contours, it is evident that the most significant displacements occur at the four long spans along the curved slab edge. Details on how these maximum displacements were derived are found on Page 27 in the slab perimeter redesign section. With this information now in hand, the post-tensioned design can be carried out, keeping in mind that the design will likely be governed by deflection limitations.

Post-tensioned Perimeter Design – Typical Floor

DESIGN PROCEDURE

As discussed in previous sections, 100 Eleventh Avenue's floor plans vary from one floor to the next. Floors 7 through 16 are identical and the layout is shown below in Figure 23. Floors 17 through 21 vary slightly but the area of interest – the slab perimeter strip – remains largely unchanged, allowing for a single design that will apply to Floors 7 and higher. Balconies begin to extrude from the slab perimeter on Floors 6 and lower, an example of which is shown in Figures 8 & 9 above, requiring each of these levels to be looked at separately.

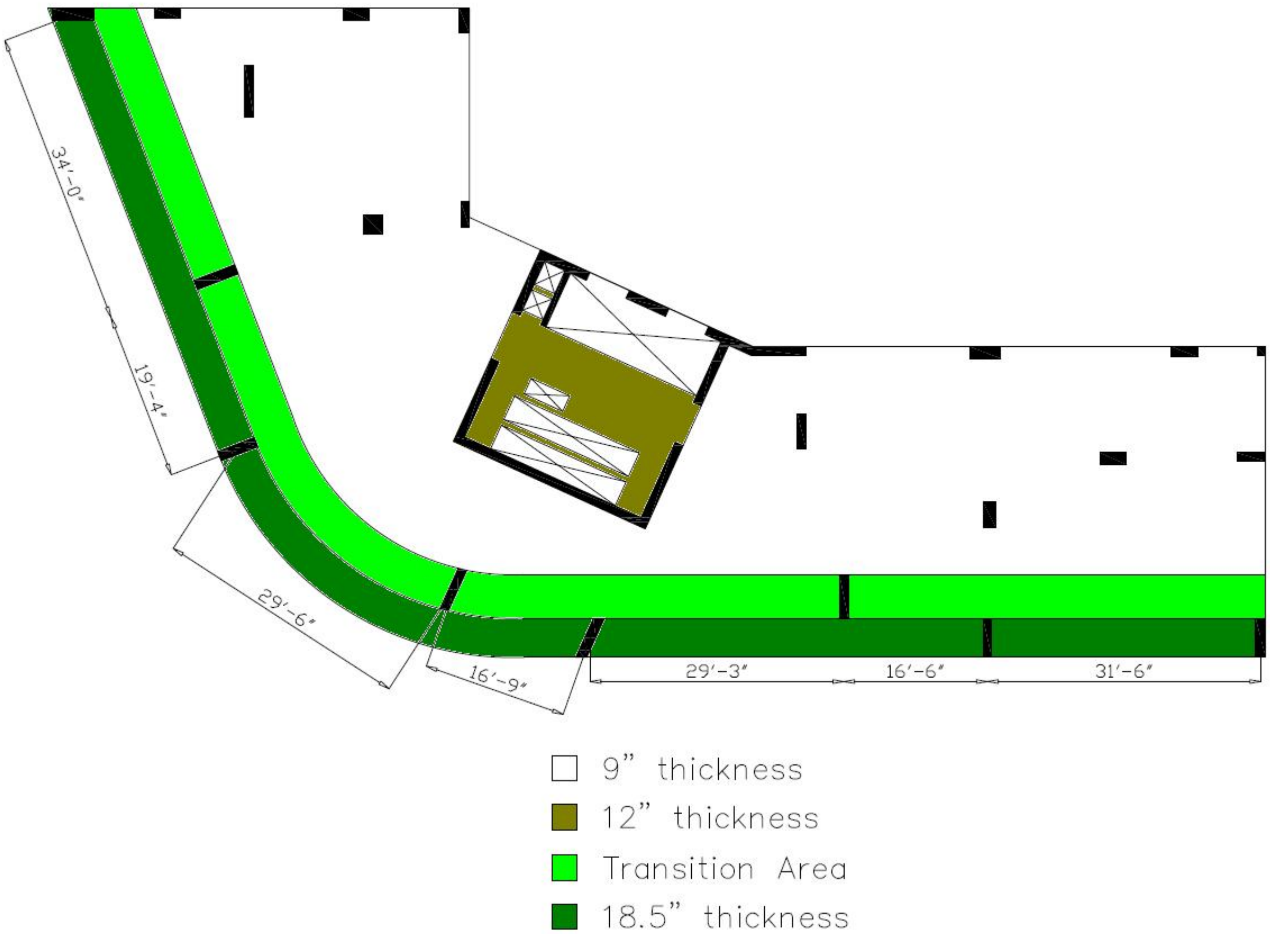


Figure 23: Typical Floor

Figure 24: Detail of thickened slab at curved edge

The typical-floor post-tensioned design was looked at first. In order to analyze and design the perimeter strip and in spite of the significant curve, the 9.5' thickened edge portion was assumed to act as a single equivalent frame spanning from column to column along the entire perimeter, as is shown in Figure 25. Consideration was given to treating the perimeter strip as two orthogonal equivalent frames that intersect at the building's largest point of curvature. However, this would ignore the slab's continuity that is found at the building's interior curve and treat it as two end spans, resulting in much higher moments than truly exist. After consulting with an industry member experienced in post-tensioning, it was confirmed that treating the entire strip as a single frame is a valid and common design assumption,

so long as the curved tendons inherent desire to straighten and "push in" towards the building's core upon stressing is countered by fastening them to the slab via hairpins. More on this issue is found in a later section.

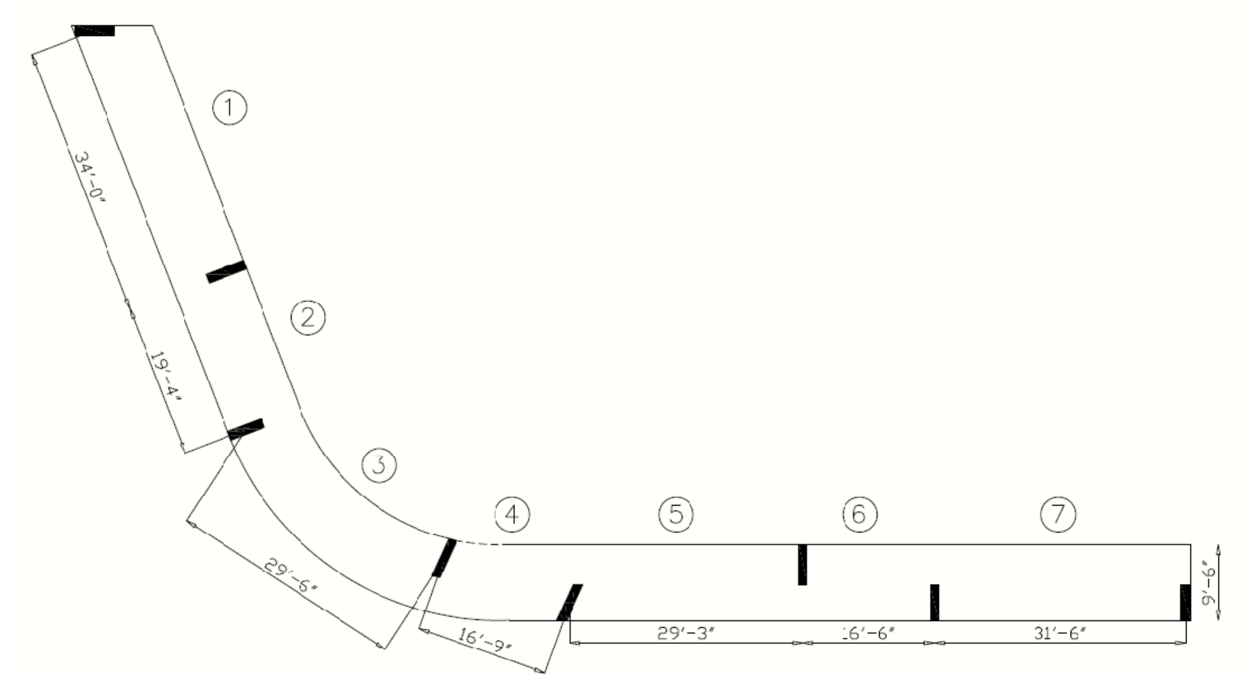


Figure 25: Slab Perimeter Equivalent Frame in its Actual Configuration

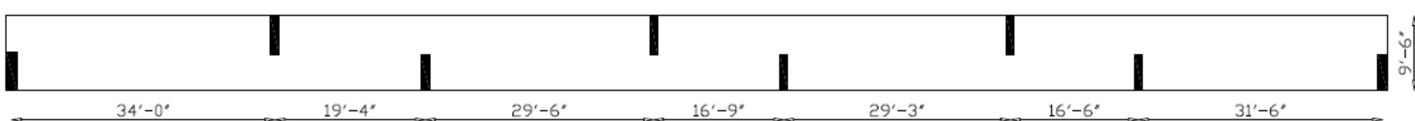


Figure 26: Slab Perimeter Equivalent Frame "Straightened" for Analysis and Design

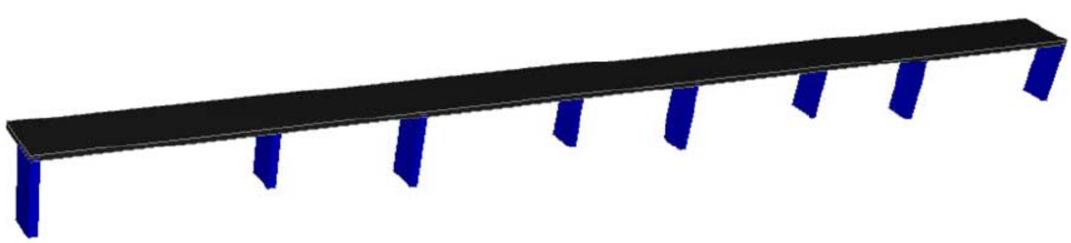


Figure 27: Equivalent Frame Modeled in RAM Concept

In order to generate a preliminary design, this 9.5' wide strip was entered in RAM Concept's Strip Wizard, a utility that quickly initiates a post-tensioned design based on a minimum prestress of 125 psi and a user-specified minimum balanced load. Not only did this provide a preliminary number of tendons to be used in design, but also provided a frame simple enough for the computer solution's results to be checked by hand. Due to the complexity of the existing floor's actual geometry, any assumptions made to enable analysis by hand would produce significant deviations from the floor's true behavior. Thus RAM Concept's design procedures were checked against this simplified equivalent frame.

85% of the dead load was entered as a minimum balanced load. Other relevant design parameters are found in Table 3. These parameters are used throughout the post-tensioning redesign on this structure. One important parameter worth noting is the classification of the post-tensioning strip as Class U: $f_t \leq 7.5\sqrt{f'_c}$. Keeping concrete cross sections from exceeding the modulus of rupture ($f_r=7.5\sqrt{f'_c}$), which is commonplace in post-tensioned design, allows uncracked behavior to be assumed, significantly decreasing calculated deflections. ACI 318-08 18.3.3 however, instructs the use of Class U: $f_t \leq 6\sqrt{f'_c}$ with all prestressed two-way slab systems. Because only one strip in a single direction is to be prestressed, it was decided that the proposed design is not classified as a prestressed two-way slab system, and thus stresses up to $7.5\sqrt{f'_c}$ will be allowed.

Post-tensioning Design Parameters	
System	Unbonded mono-strand
Tendons	1/2"Ø 7-wire strand
f_{pu}	270 ksi
f_{se}	175 ksi
Prestress Loss	14 ksi
P_{eff}	26.7 k
Classification	Class U: $f_t \leq 7.5\sqrt{f'_c}$
f'_{ci}	3000 psi
f'_c	6000 psi

Table 3

The resulting preliminary design (21 tendons) was checked by hand, ignoring column stiffness, and found to satisfy all serviceability and strength requirements. These calculations can be found on Page 70 of Appendix B.

The program's initial design was then entered in Concept as part of the entire floor. As was expected, the strip behaved differently as part of a larger structure and the number of tendons and their profile points were adjusted accordingly.

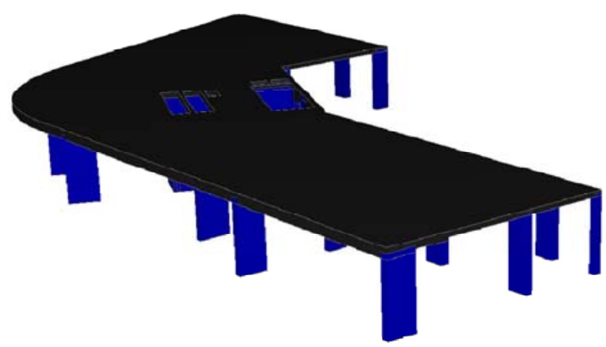


Figure 28: Typical Floor Perspective Modeled in RAM Concept

At this point, it was important to adequately understand the process RAM Concept uses to analyze a structure. Because RAM Concept utilizes the finite element method in place of the equivalent frame method, high peak moment and stress concentrations are often produced which are inappropriate for design. Thus, RAM Concept utilizes "design strips" to link finite element analysis with concrete code rules which allow the averaging

or "smearing" of these peak moments and shears across a designated width. This makes the drawing of design strips in RAM Concept very important. As shown in Figure 29, the design strips were defined to model the curved slab perimeter as a single equivalent frame, following the initial behavioral assumption discussed earlier.

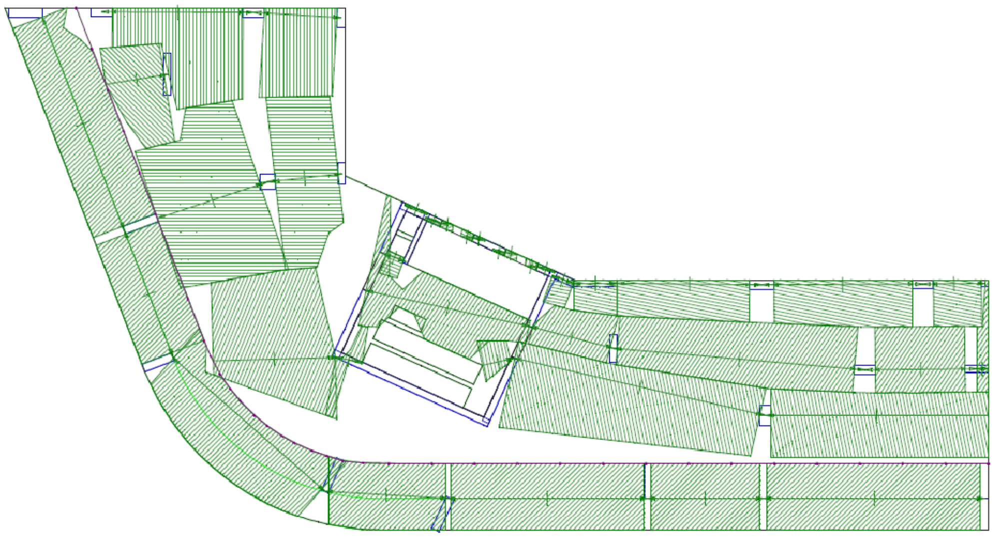


Figure 29: Horizontal Design Strips Used in RAM Concept

The loads entered into RAM Concept and used in hand calculations are summarized in Table 4 below. Curtainwall loads were applied as line loads at the very edge of slab, in hopes of replicating actual behavior and accurately modeling deflection problems along the curved perimeter.

Floor System Loads	
Description	**Load**
Superimposed Dead	52 psf
Live	40 psf
Glass Curtainwall	500 lb/ft
Masonry Curtainwall	750 lb/ft

Table 4

DESIGN RESULTS

By utilizing RAM Concept as a design tool to quickly vary the number of tendons and tendon profile points, a final design for the typical floor that satisfies strength and serviceability requirements was arrived at. The design is shown graphically in Figure 30, found on the following page. Table 5 also tabulates much of the relevant span and support information relevant to post-tensioning design. Numbered spans were identified in Figure 25 above.

Post-tensioning Design Summary							
Item	Spans						
	1	2	3	4	5	6	7
Span (ft)	34'-0	19'-4"	29'-6"	16'-9"	29'-3"	16'-6"	31'-6"
# of Tendons	16	16	16	16	16	16	16
P/A (psi)	416	416	416	416	416	416	416
Balanced Load (k/ft)*	1.4	1.3	2.3	1.2	1.4	1.3	1.5
% Dead Load Balanced	58%	48%	77%	51%	58%	50%	65%
Midspan Total Deflection	0.75"	0.09"	0.71"	0.09"	0.56"	0.09"	0.69"
Lesser of 1" & L/480	0.85"	0.48"	0.74"	0.42"	0.73"	0.41"	0.79"
Midspan Initial Service Stresses**							
f_{top} (psi)	-148	-212	96	-175	-69	-214	-116
f_{bot} (psi)	-473	-235	-422	-160	-292	-251	-494
Midspan Service Stresses**							
f_{top} (psi)	-580	-262	-310	-142	-332	-200	-620
f_{bot} (psi)	-73	-72	82	-145	79	-224	81

Item	Supports							
	1	2	3	4	5	6	7	8
Column #	13	14	15	16	17	18	19	1
Support Initial Service Stresses**								
f_{top} (psi)	-802	-203	-491	-375	-348	-228	-383	-651
f_{bot} (psi)	-368	-119	32	2	22	-85	-78	-229
Support Service Stresses**								
f_{top} (psi)	-281	401	248	250	399	375	329	-55
f_{bot} (psi)	-621	-678	-472	-695	-579	-611	-650	-666

* Average balanced load = 1.5 k/ft

**Negative values denote compression

Table 5: Design Summary

Allowable Stresses (psi)		
	At Stress Transfer	After Losses
Compression	$0.6(f'_{ci}) = 1800$	$0.45(f'_c) = 2700$
Tension	$3\sqrt{f'_{ci}} = 164$	$7.5\sqrt{f'_c} = 581$
Bonded Reinf. Req'd.	$f_t \geq 2\sqrt{f'_c} = 155$	

Table 6: Allowable Stresses per ACI 318-08 at Service Loads

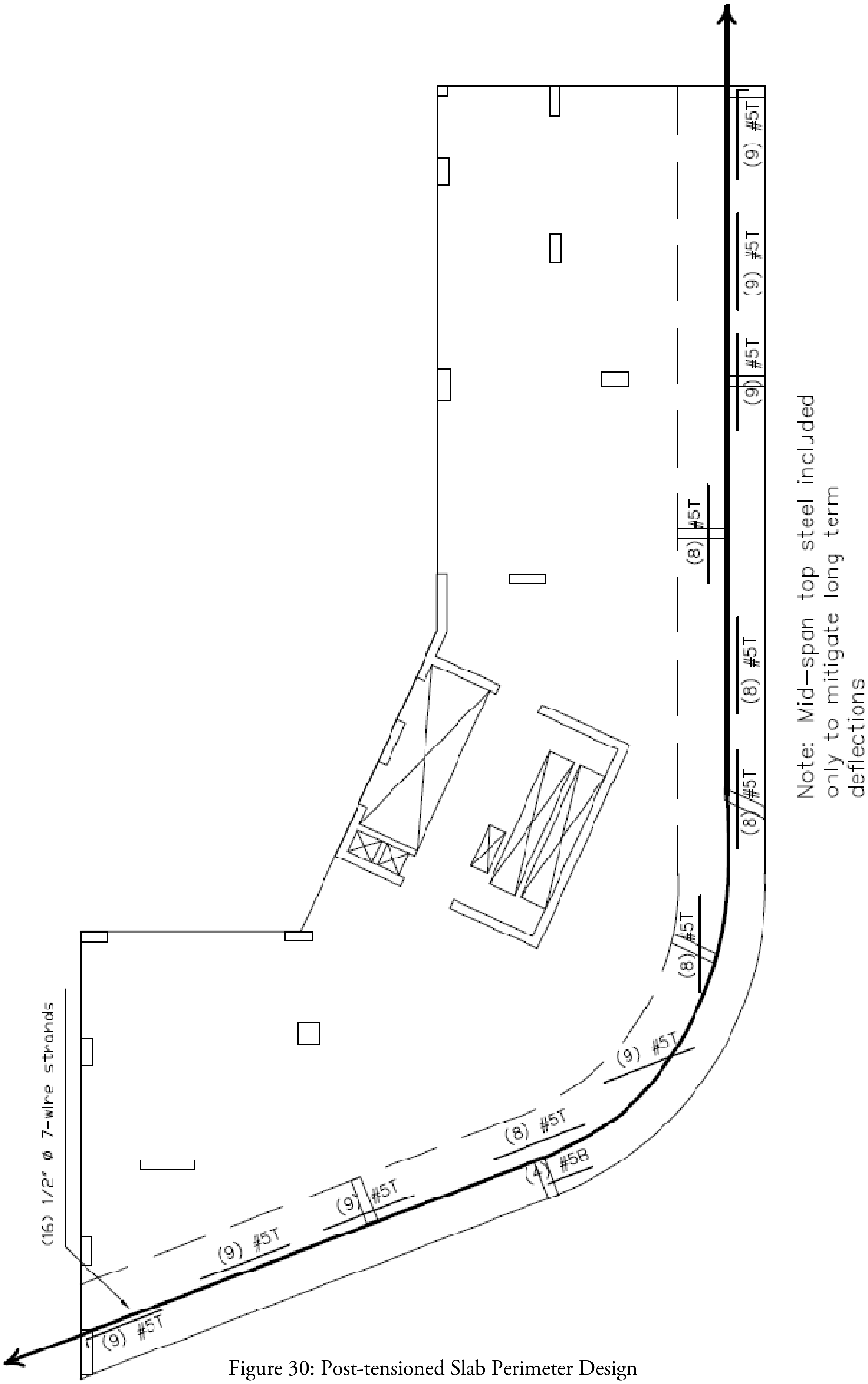


Figure 30: Post-tensioned Slab Perimeter Design

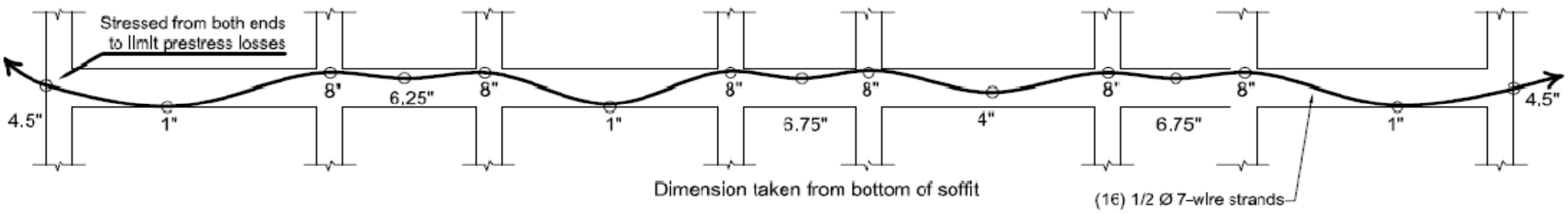


Figure 31: Cross-section showing PT tendon drape

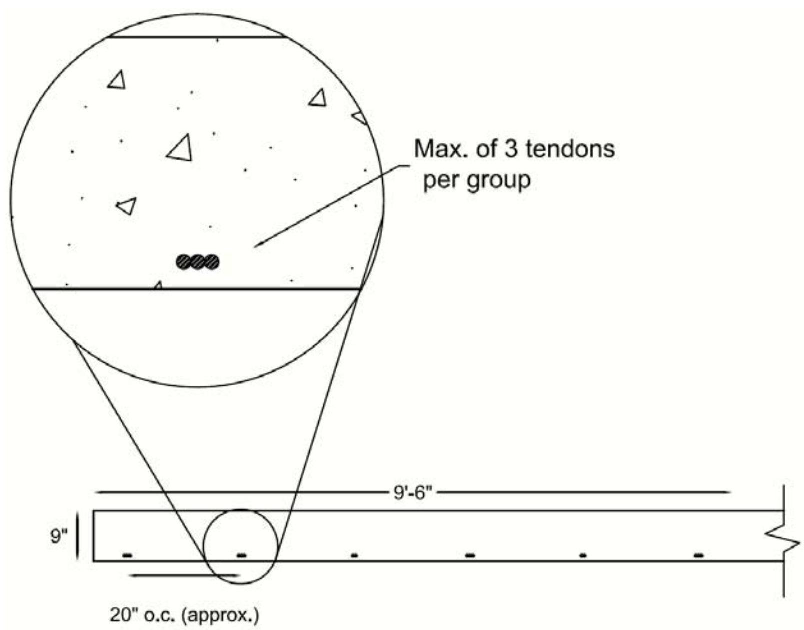


Figure 32: General Grouping of Tendons

Using the unbonded mono-strand system common in most post-tensioned building applications, the design uses 16 ½" Ø 7-wire strands. For the design to be acceptable, several requirements were to be met. First, ACI 318-08 mandates flexural stresses on concrete sections not exceed the allowable stresses listed in Table 6 for service loads. RAM Concept conveniently outputs these stresses, which are listed in Table 5. At prestress transfer, the only loads present are those from the post-tensioning and self-weight of the structure.

It is worth noting that midspan stresses do not exceed $2\sqrt{f'_c}$ and as a result, no bonded reinforcement is required at midspans. Ultimate strength is also sufficient without the need for additional bonded reinforcement. Due to the susceptibility to creep and long-term deflections, however, additional midspan top steel was included. Also worth commenting on is the fact that midspan bottom service stresses for Span 1 are in compression while shorter spans with approximately equal balanced loads are experiencing tension. This is because RAM Concept's finite element analysis accounts for all loads that will make their way to the perimeter strip, whether or not they are within the "drawn" design strip. In other words, these shorter spans with higher stresses have larger tributary areas and thus more load.

Like any other concrete member resisting flexural loads, the post-tensioned perimeter must also satisfy ultimate strength requirements as well as minimum reinforcement requirements. Treating the strip as a simplified equivalent frame, the redesign has been verified by hand to have sufficient ultimate strength. A summary of the applicable code requirements from ACI 318-08 can be found in Table 7.

Applicable Code Requirements (ACI 318-08)			
Allowable Stresses (psi)			
	At Stress Transfer	After Losses	
Compression	$0.6(f'_{ci})$	$0.45(f'_c)$	18.4.1
Tension	$3\sqrt{f'_{ci}}$	$7.5\sqrt{f'_{ci}}$	18.3.3 & 18.4.1
Minimum Bonded Reinforcement			
At midspan where $f_t \geq 2\sqrt{f'_c}$	$A_s \geq N_c/(0.5f_y)$		18.9.3.1
At (-) moment area over columns	$A_s \geq 0.00075A_{cf}$		18.9.3.3
Average Effective Prestress			
Minimum	125 psi		18.12.4
Maximum	300 psi		*

*Reccommendation

Table 7: Applicable Code Requirements

As was predicted by studying the existing design, deflection limitations controlled the post-tensioning design. Because of the susceptibility of the longer spans along the curved perimeter to "sag" under the curtain wall load, as well as the glass curtain wall's sensitivity to slab deflections, significant attention was paid to limiting deflections.

In the existing design, slab edge deflection was limited to 1" for any edge supporting the glass facade panels, as per the curtain wall consultant. This limitation was again followed for the post-tensioned redesign. In addition, the net deflection occurring after application of the self-weight and balanced loading (i.e. deflection resulting from all superimposed dead loads including the curtain wall, and live load) will be limited to L/480. This is a more conservative limit than what is prescribed in ACI 318-08 Table 9.5(b), where "*that part of the total deflection occurring after attachment of nonstructural elements*" shall be limited to L/480 for nonstructural elements that will be damaged by large deflections. This would allow the deflection due to the curtain wall itself to be ignored; however, because so much of the immediate deflection is due to the curtain wall, this will conservatively be included as part of the deflection effecting non-structural elements in order to avoid any glass panel problems resulting from too much deflection during the actual curtain wall installment.

Table 8 below presents a summary of the maximum deflections occurring in 1) a design without a thickened *or* post-tensioned slab perimeter, 2) the existing design with the thickened slab perimeter and 3) the post-tensioned redesign. As can be observed, both the existing design and the post-tensioned design meet the deflection limitations of 1" and L/480, as they should. Not surprisingly, the 9" slab perimeter has a total deflection greater than 1" and a deflection of L/220 occurring upon attachment of structural elements.

To attain realistic approximations of a complicated geometric floor system such as 100 Eleventh Avenue, RAM Concept's deflection contour plans were utilized. RAM Concept analyzes the concrete floor with a linear elastic analysis and the program's deflection contour plots are representative of this. Thus, it does not consider cracking and/or creep. As a result of this, RAM Concept cannot be accurately used for long-term displacements (where creep plays a significant role) but will be suitable for immediate displacements because concrete stresses were limited to the modulus of rupture and thus can

Stage	Maximum Deflection (in.)		
	9" Slab	Existing Design	PT Design
Immediate Δ (D_{sw})	0.24	0.11	-0.03
Immediate Δ (D_{total})	0.48	0.17	0.22
Immediate Δ (L)	0.09	0.03	0.06
Long Term Δ (D+0.5L)	1.05	0.37	0.5
Δ_{total}	1.62	0.57	0.78
Lesser of 1" & L/480	0.74"	0.85"	0.85"
L/x	L/220	L/720	L/520
Critical Δ*	1.38	0.46	0.81
Slab edge deflection limited to 1", per glass curtainwall consultant			

*Critical Δ taken to be that part of total deflection occurring *upon* attachment of nonstructural elements

Table 8: Maximum Deflections Present in Three Configurations

be considered uncracked. This is the primary benefit post-tensioning offers as a design strategy in this building – by treating concrete sections as uncracked, deflections can be significantly reduced while keeping the slab relatively thin. The immediate deflections calculated by RAM Concept were checked by hand (Page 74, Appendix B) and confirmed to be realistic.

Long-Term Deflection Load Combination in RAM Concept	
Loading	Load Factor
Self-Dead Loading	3.35
Balance Loading	3.35
Other Dead Loading	3.35
Live Loading	2.18

Table 9

Though RAM Concept's contour plots do not consider creep, it does provide a "Long Term Deflection" load combination where the effects of creep and shrinkage are estimated by applying the load factors listed in Table 9. Because the origin of these factors is unknown and the load factors seem excessively conservative, all long-term deflection calculations were done by hand based on the immediate deflections calculated by elastic analysis through RAM Concept. Long term deflections were calculated by multiplying all sustained loads by the long-term effect multiplier, λ_Δ, (ACI 318-08 9.5.2.5), assuming a duration of five years or more and no top steel. Thus, λ_Δ=2. The total deflection was then calculated using the following formula:

$$\Delta_{total} = \lambda(\Delta_{i,dead} + \Delta_{i,sustained\ live}) + \Delta_{i,dead} + \Delta_{i,sustained\ live}$$

Sustained loads were considered to be all dead loads and 50% of the live load.

Designing to the aforementioned deflection limits required a significantly higher balanced load than was necessary for ultimate strength. Thus, at service level stresses, four of the seven spans are in compression at their bottom. High balanced loads such as this can sometimes create excessive midspan top tensile stresses at initial stages, where much of the load designed for is yet to be applied. This is not an issue with this design, where the largest tensile stress at initial service stresses is below the allowable stress of $3\sqrt{f'_{ci}}$.

Because the post-tensioned redesign reduced the slab thickness from 18.5" to 9", punching shear needed to be checked again. To verify that punching shear was not an issue along the slab perimeter, Columns 1 & 13 were checked by hand. These columns were chosen to be the worst case scenarios, due to their location at slab corners, which leaves them with roughly half of the shear resistance provided by the surrounding slab of an interior or even edge column. These calculations can be found Page 75 of Appendix B. Taking into account the direct shear imparted by gravity forces as well as the additional shear created by the transfer of moment from slab to column, the slab at these columns was shown to have sufficient punching shear resistance.

Post-tensioned Perimeter Design – Lower Floors

With a design finalized for the typical floor of Levels 7 through 21, attention was turned to the more atypical floors of 2 through 6. As mentioned previously, a post-tensioned perimeter design for these levels is complicated by the various balconies that protrude from the thickened slab portion. If post-tensioning were to be implemented on these levels, the balconies would need to be checked to determine that a new slab thickness of 9" will be sufficient for strength and deflection requirements. After considering two key issues particular to these levels, however, it was determined that post-tensioning the perimeter slab edge of Floors 2 through 6 simply is not feasible.

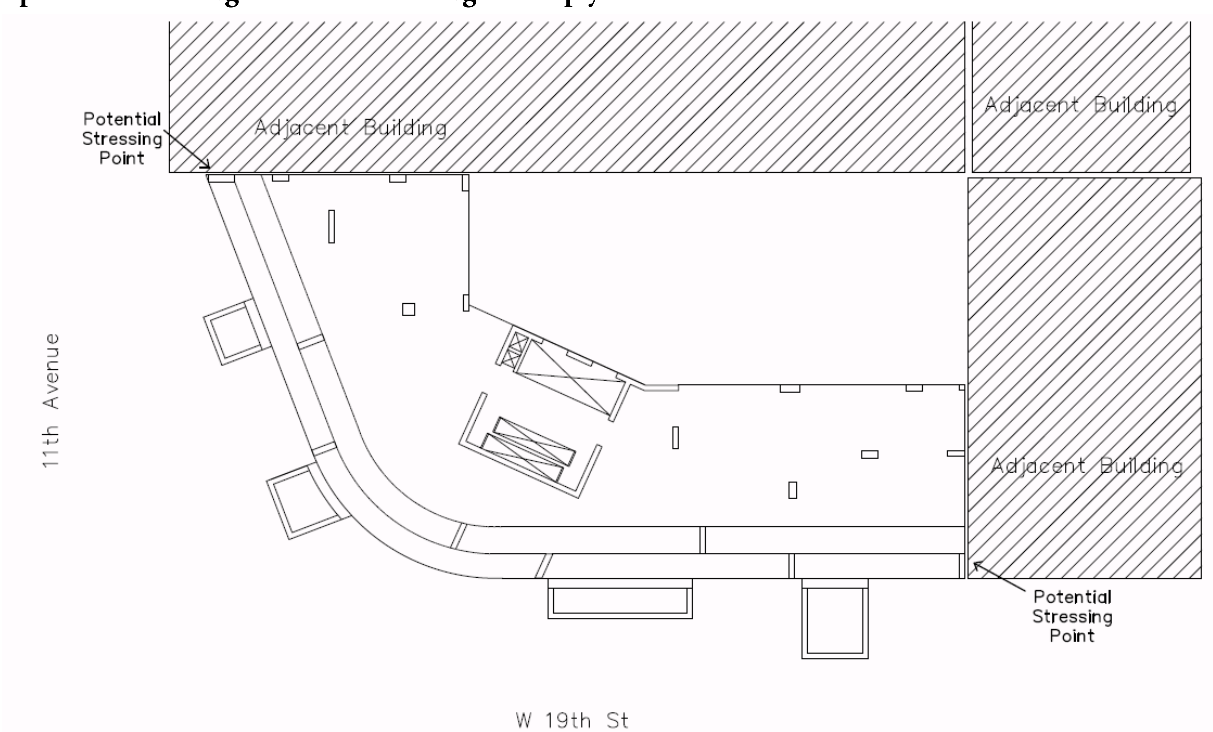


Figure 33: 6th Floor Plan Showing Neighboring Buildings and Potential Stressing Locations

Firstly, the close proximity of neighboring buildings at these levels leaves little to no room for a tensioning jack. As seen in Figure 33, adjacent buildings are located at both potential stressing points, with only 6" to12" of clear space – not enough room for a tensioning jack to tension the high-strength tendons. One solution would be to offset the corner columns several feet in from the edge, which would allow space for the tendons to be tensioned. Afterwards the remaining few feet of slab would be poured up to the adjacent building's perimeter. However, this would require manipulating the interior space design, which was not an option on this particular project. The tallest of the neighboring buildings reaches only to 100 Eleventh Avenue's sixth level, leaving all stories above this level unaffected by surrounding structures.

The second issue is a result of the architectural sub-flooring requirements of the balconies. Many of the balconies have a fluid-applied waterproofing assembly with concrete pavers as a flooring system. This thicker flooring is to be flush with the interior spaces, which is accomplished through the use of slab depressions which thin the floor to 9" while keeping the soffit continuous, as is shown in the balcony cross-section detail in Figure 36.

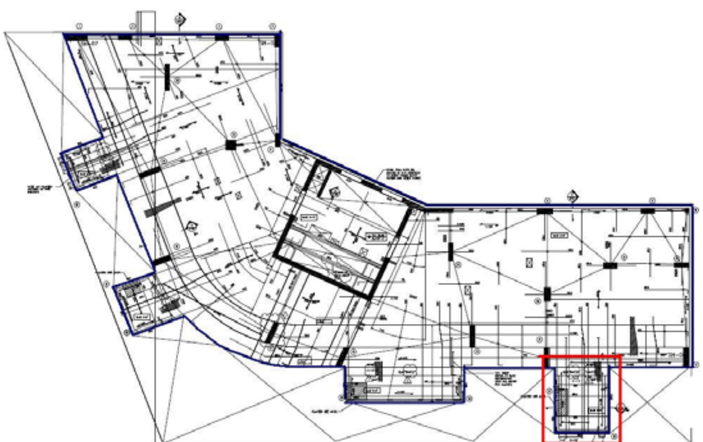


Figure 34: 6th Floor Plan with Balcony Denoted

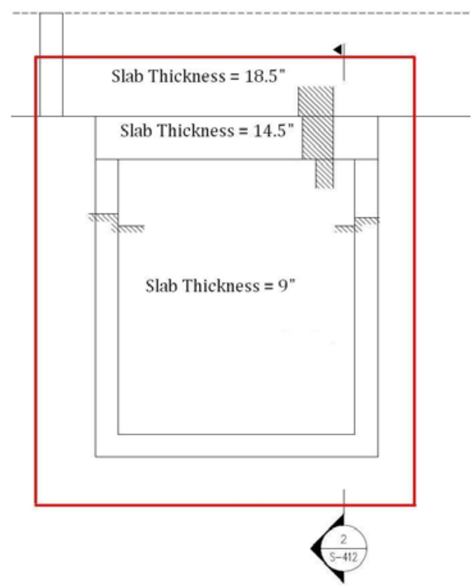


Figure 35: Close Up of Balcony

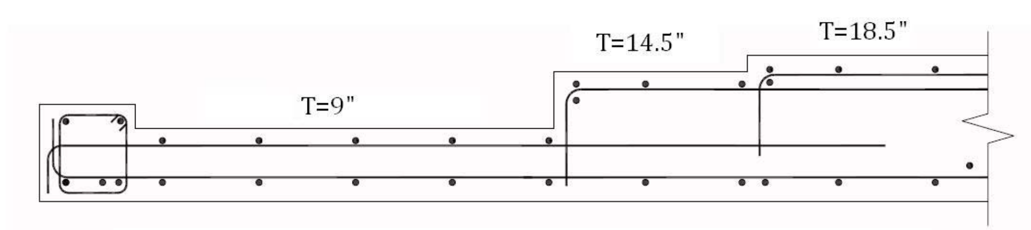


Figure 36: Existing Balcony Cross Section

Figure 37 shows where the 9" post-tensioned slab would be located with respect to the original design. In this configuration, the balcony subflooring cannot be accommodated. Creating slab drops at the balcony location to allow the balcony subflooring to sit below the 9" soffit, as shown in Figure 37, would accommodate the flooring issue, but would create slab drops along the post-tensioned tendon layout, something very unfavorable to post-tensioned design. Because 1" Ø post-tensioned bars were required in the original design to control deflections, post-tensioned tendons will almost certainly be required along the balcony spans to limit deflections.

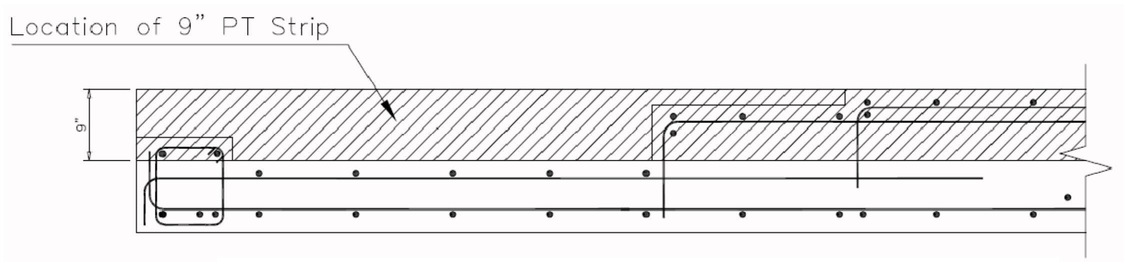


Figure 37: Existing Balcony Cross Section with 9" PT Slab Superimposed

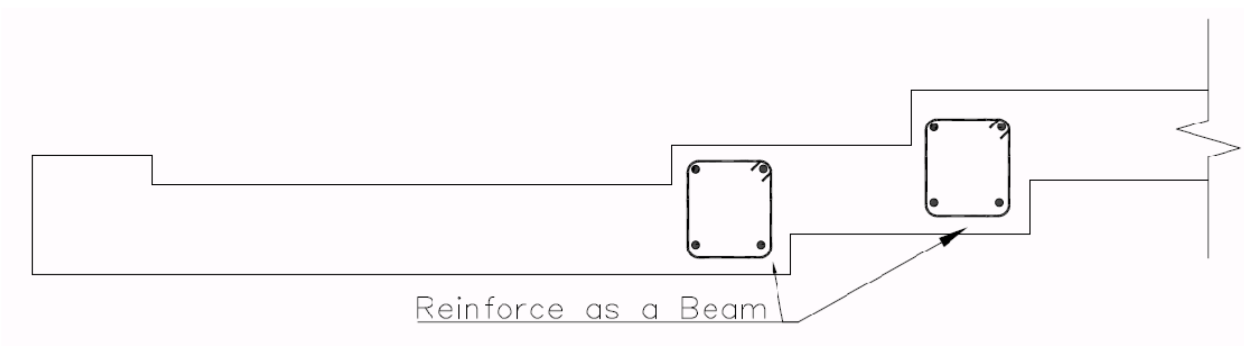


Figure 38: Alternative PT Balcony Cross Section to Accommodate Flooring

The combination of these two complications involving the post-tensioning of the lower 6 floors dictates that it is not a feasible design solution. If only one of the issues was present, perhaps a viable solution could be arrived at, but developing solutions that circumvent both will almost certainly be unrealistic, costly, and/or time-consuming, essentially defeating the purpose of the post-tensioned design of creating a more *efficient* floor system. With this in mind, it was decided that the existing 18.5" thick perimeter slab design is the better design for Levels 2 through 6.

Other Considerations

Several aspects concerning the above post-tensioned redesign and typical post-tensioned design strategies require more discussion.

Firstly, it is important that the prestressed tendons used in the perimeter slab are stressed from both ends of the structure. Tendons stressed from only one end and used in configurations longer than 100 ft begin to experience substantial losses in prestress due to friction along the tendon. 100 Eleventh Avenue's perimeter tendon layout is 177 ft. Stressing the tendons from both ends helps ensure that the force in the tendon is relatively the same from end to end.

Secondly, to meet the deflection limitation described above, significant prestressing forces were required such that the average effective prestress (P/A) reached 416 psi, significantly more than the recommended value of 300 psi for two-way slab systems. It's important to note, however, that this value assumes the entire floor system experiences this prestress. In the case of this design, however, only the 9.5 ft strip is prestressed, so that, should a problem due to these forces present itself, the prestressing force will be capable of dissipating out towards the mildly-reinforced concrete slab.

Thirdly, because of the significant curve of the tendon layout, hairpins will be used to fasten the tendons to the concrete. Because any object linear in shape will innately try to form a straight line when tensioned, the curved tendons in the post-tensioned redesign for 100 Eleventh Avenue will attempt to "straighten" the building itself out, resulting in the exertion of forces shown schematically in Figure 40 below. This can also be thought to act as a harped tendon, which will exert a balanced load on the building in the direction shown in Figure 40. The purpose of the hairpins is to resist these forces by fastening the tendons to the slab.

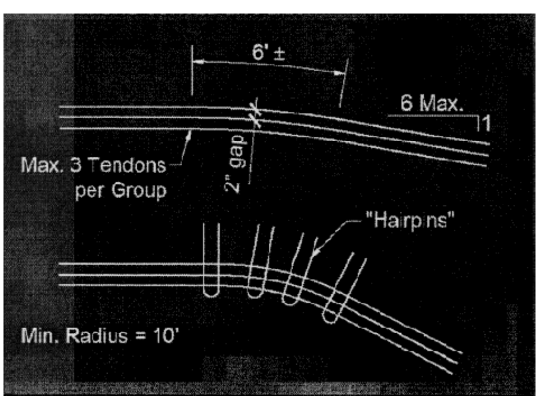


Figure 39: Sample Detail Showing Typical Hairpin Use

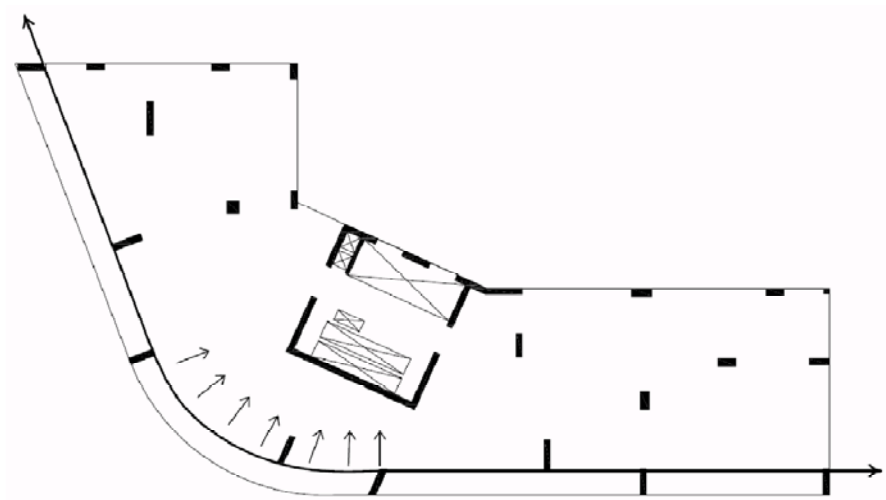


Figure 40: Plan showing tendency of curved tendons to try to "straighten" out when tensioned

19th Floor Transfer System Redesign

Background

As described in previous sections, a transfer system is necessary on the 19th level to accommodate a building setback. Columns 30, 31, and 32 support Levels 20, 21, and the roof and terminate at the 19th level as the building's perimeter shifts 12'-8" to the east. Figures 41 and 42 below show the existing 19th level plan and a section elevation showing a transferring column, respectively. The current design calls for an 18.5" slab to transfer the column forces, reinforced with #10 bars at 6" o.c. in both directions on both top and bottom of the slab.

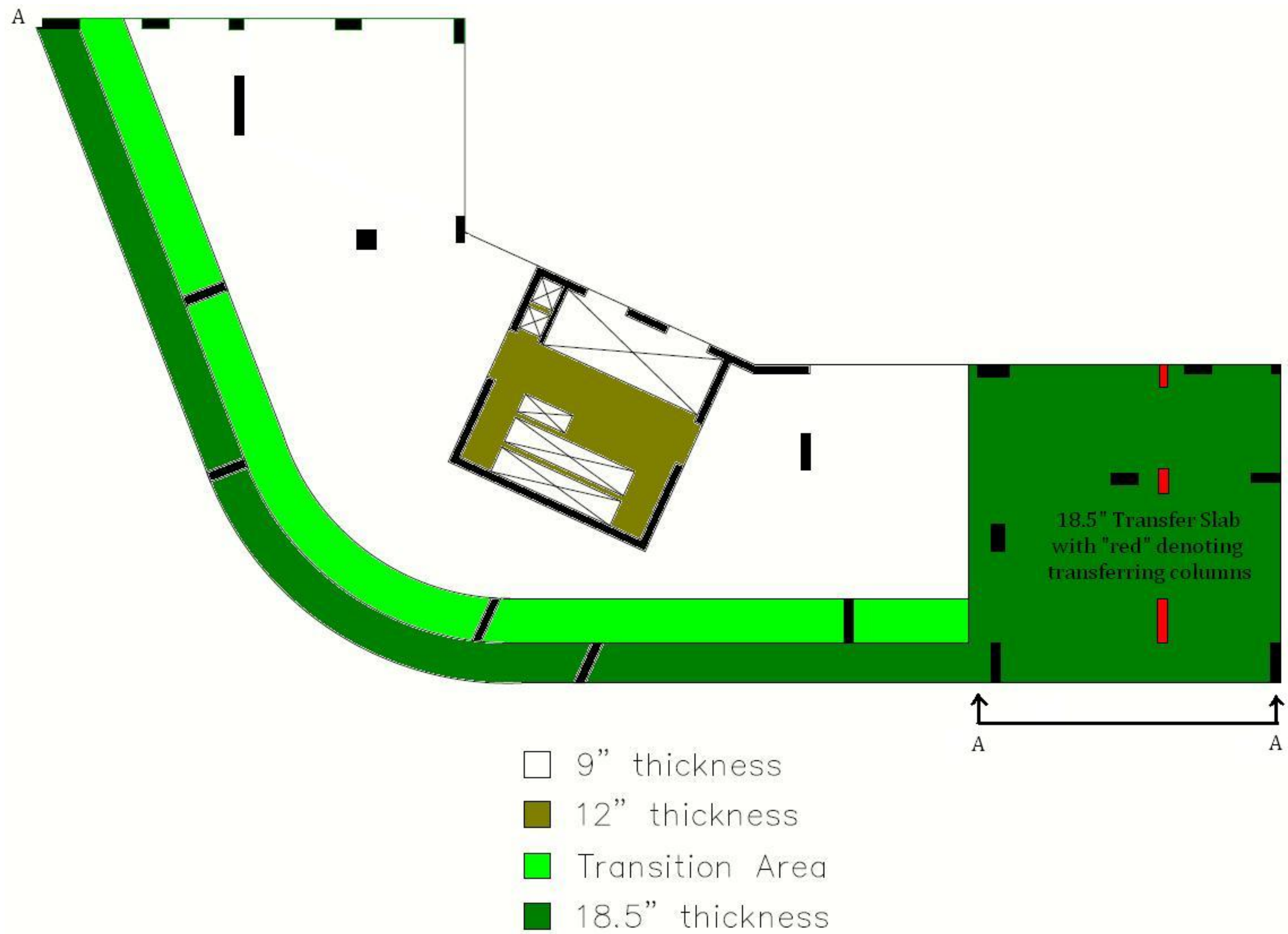


Figure 41: Existing 19th Level Design

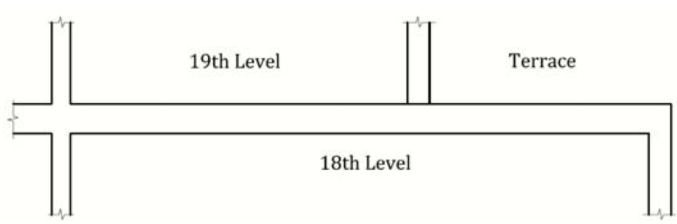
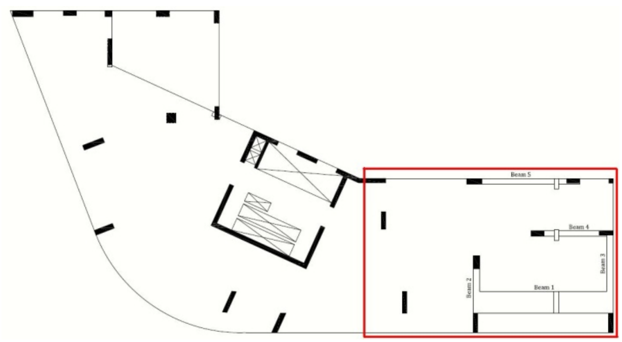
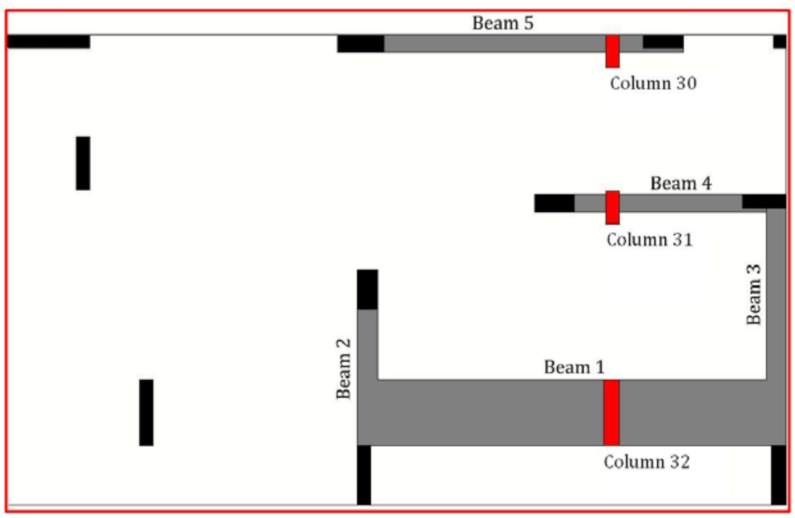


Figure 42: Section A-A showing Column 32 transfer

The proposed transfer system redesign utilizes beams spanning between columns to transfer loads, thereby eliminating the need for such a thick, heavily-reinforced slab. Figure 6 above is a depiction of preliminary beam locations from very early in the design process. It was quickly decided, however, that far too much torsion would be developed in the southern-most beam, with its centroid nearly 5 feet from that of its supports. This required a modification of the beam layout to eliminate these torsional forces, and the final layout was arrived at, as shown in Figure 43 below.



(a)



(b)

Figure 43: (a) Full Floor Plan for Reference
(b) Final Design Beam Layout

During the design of the five transfer beams, it was discovered that the use of post-tensioned tendons in these members is not feasible. For example, Column 32 imposes a point load of 290 kip, to be transferred by Beam 1 to its two support columns. Using a harped tendon profile with maximum drape, balancing just 75% of the column load would require 103 ½" Ø 7-wire strands, an unrealistic quantity for a beam of this size. The allowable stresses for prestressed concrete, even then would almost certainly be exceeded. Ignoring these requirements, one *could* add enough mild-steel so that the beam meets ultimate strength requirements. However, because stresses will exceed the modulus of rupture, the

concrete section must be treated as cracked, which is no different than if the member were not prestressed at all. The tendons will act no differently than the mild-steel reinforcement in resisting factored loads. For these reasons, the beams will be designed as conventionally reinforced concrete members. The reason that this can be done in spite of the fact that post-tensioning is not feasible is that reinforced concrete design allows for stresses to exceed the modulus of rupture, so long as the cross section is treated as cracked.

In designing the five beams that make up the redesigned transfer system, five major design requirements had to be met. They are as follows:

- **Flexure**
- **Shear**
- **Torsion**
- **Deflection**
- **Architecture**

Flexure and shear requirements are typical of nearly all beams and require no further explanation. The torsion induced on Beam 3 and Beam 5 by Beam 1 and Column 30, respectively, needs to be designed for through the use of additional transverse reinforcement. Architectural requirements are equally important on this project. These include the limitation of beam depth to 18.5" - the depth of the current transfer slab design. Increasing the depth would negatively impact the space of the residential units. In addition, effort will be made to keep the beams' widths flush with their supporting columns.

Design Results

The results of the beam design are shown below in Figures 44-48. Each beam was designed by hand to meet the five parameters just discussed. A column load takedown was performed for columns 30, 31, and 32, with the results shown in Table 10. The complete column takedown can be found in Appendix C. Because Beams 2, 3, 4, and 5 frame directly into columns that are very stiff in the direction of the beam, support conditions were assumed to be fixed. Beam 1 is the exception to this, as it frames into Beams 2 & 3. Using ACI 318-08 8.3 coefficients, support conditions were approximated as somewhere between fixed and pinned, as Beams 2 & 3 will provide some torsional restraint to rotation. Based on these support conditions, design moments were linearly interpolated between that for fixed end conditions and pinned end conditions. See Appendix D for these calculations. Table 11 below summarizes some general information for each beam.

Column Load Takedown Results	
Column	Load (k)
30	122
31	165
32	290

Table 10

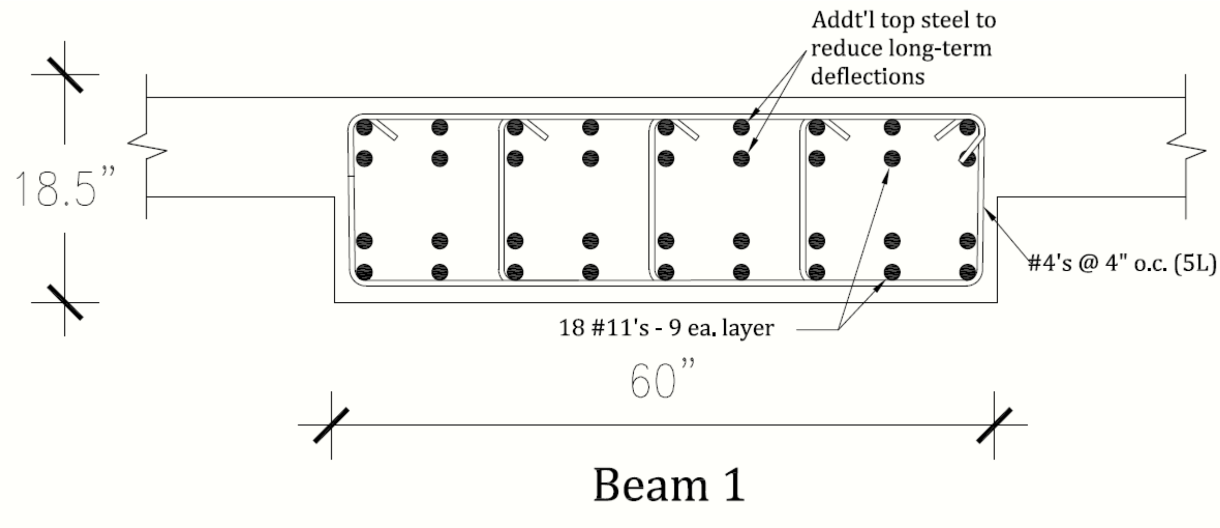


Figure 44

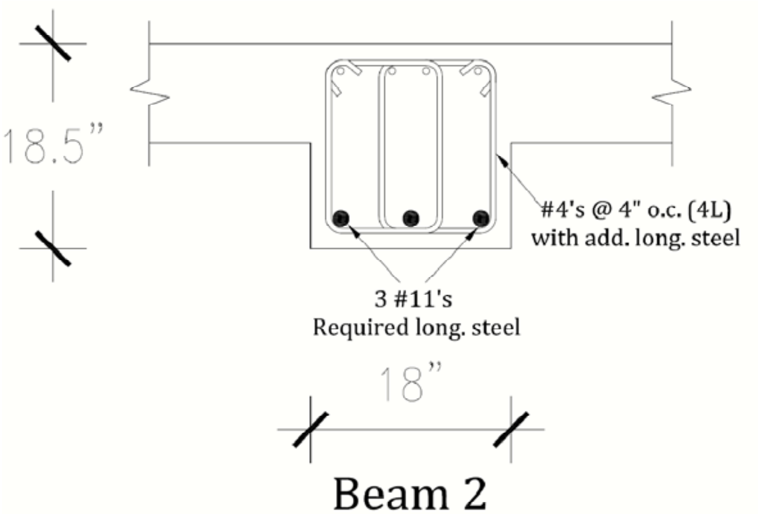


Figure 45

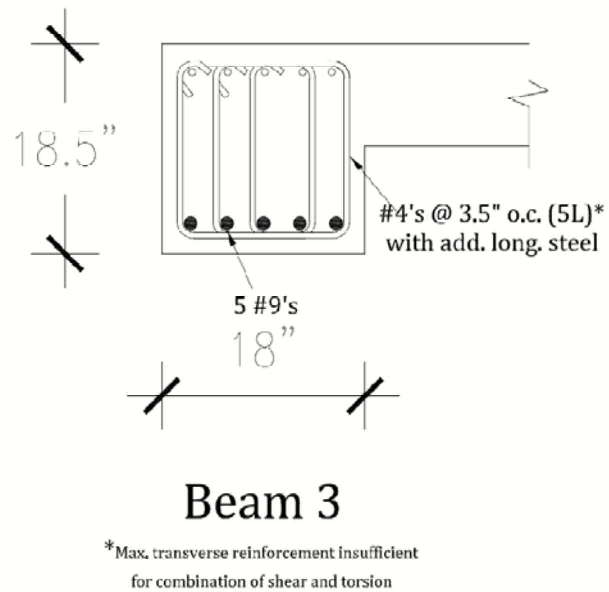


Figure 46

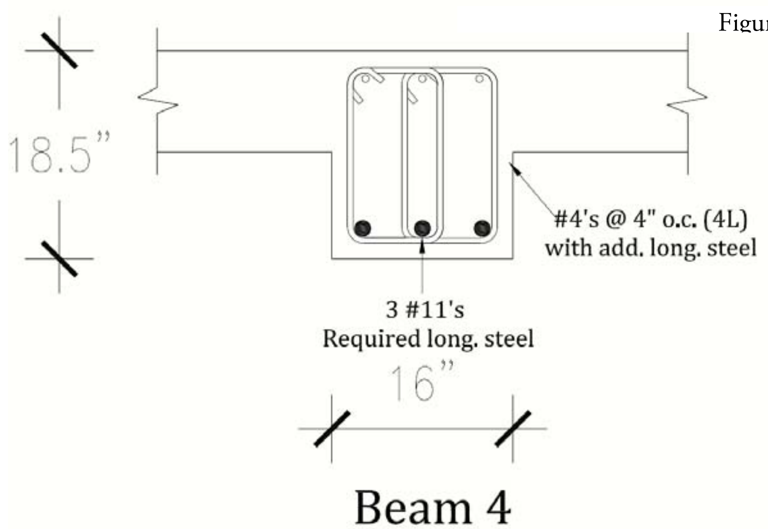


Figure 47

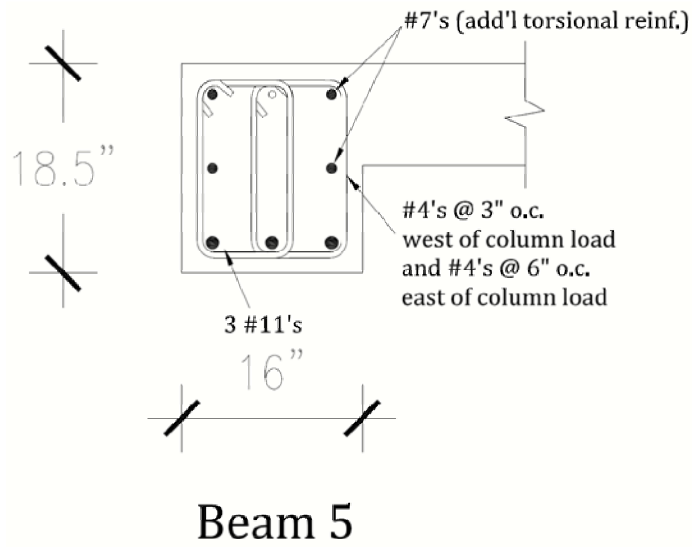


Figure 48

Beam Design Summary									
Beam ID	Span	Point Load Location	M_u (ft-k)	ϕM_n (ft-k)	V_u (k)	ϕV_n (k)	T_u (ft-k)	ϕT_n (ft-k)	All design paramteres satisfied?
1	31'-6"	0.40L	+1651	1775*	251	263	-	-	No
2	14'-0"	0.34L	+221	301	152	175	-	-	Yes
3	20'-9"	0.23L	+335	335*	224	**	37	**	No
4	16'-0"	0.28L	+232	297	152	171	-	-	Yes
5	23'-0"	0.17L	-234	297	138	138	71	71	Yes

*Uses effective flange width as beam width in flexural strength calculation

** ACI 318-08 Eq. 11-18 not met (i.e. section not large enough for combined shear and torsion)

Table 11: Beam Design Summary

Discussion of Design Results

As previously mentioned, each beam was designed to satisfy flexure, shear, torsion, deflection, and architectural requirements. Beams 2, 4, and 5 were able to satisfy all requirements without issue. All strength requirements were met, deflection limitations were not exceeded, and each beam's width was limited to the largest column support width and beam depth limited to the existing system depth of 18.5".

Torsion was also considered in Beams 3 and 5 for two reasons. Torsion was assumed to be present when a point load or column support acted at an eccentricity to the beams centroid. This is the case in Beam 3, where the beam width is 4" greater than that of its support and Beam 5, where Column 30 acts at an eccentricity of 7" from the beam's centroid. These beams are also located at the slab edge, which significantly reduces the member's torsional stiffness. The torque, or torsional moment, was approximated by simply multiplying the force or support reaction by the distance between its line of action and the beam's centroid.

Unlike Beams 2, 4 & 5, Beams 1 & 3 were unable to be designed to meet all five parameters. These members are discussed further below.

BEAM 1

Beam 1's width of 60" is flush with its supporting columns and is 18.5" deep. This is essentially the maximum section that was able to be designed and still meet the architectural requirements outlined above. The section is sufficient for all strength requirements (flexure, shear, and torsion) but falls short in deflection limitations. The design engineers were limited to elastic deflections of L/360 and total deflections of L/240. In addition, deflection occurring *after* attachment of nonstructural elements was checked to ensure it was below the code recommendation of L/480. Beam 1 satisfied all deflection requirements except $\Delta_{TOTAL} \leq$ L/240. Even with the addition of significant top steel reinforcement, Δ_{TOTAL}= 2.74" > $\Delta_{ALLOWABLE}$ = 1.58". However, the most significant deflection limitation – that which occurs after attachment of nonstructural elements – was limited to Δ_{LL} = 0.75", compared to $\Delta_{LL,ALLOWED}$ = 0.79".

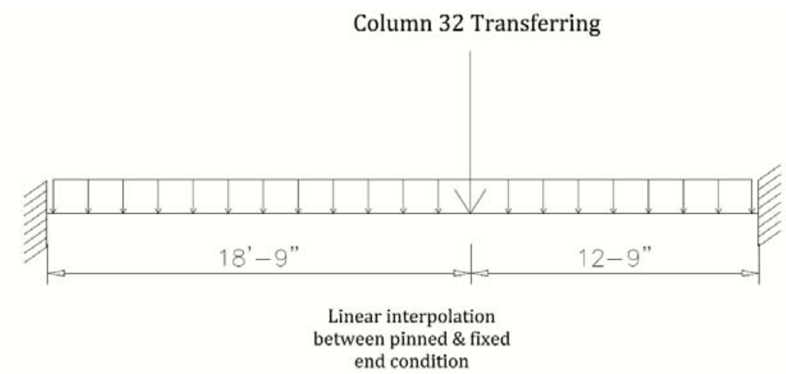


Figure 49: Simplified Model Used in Design of Beam 1

The combination of a large span (31.5') and heavy concentrated load near mid-span (290 kips at x=0.6L) is the reason for the unsatisfactory deflections. The most obvious remedy would be to deepen the member to take advantage of the cubic relation of depth with respect to moment of inertia. This, however, is not an option due to the strict floor-to-ceiling height restrictions.

BEAM 3

Beam 3 is directly related to Beam 1 in that it, together with Beam 2, supports Beam 1 and distributes its load to columns. Beam 2 has two characteristics that make its design very difficult. The first is the position of the point load (0.23L) acting on it. Because it is so close to one support, the majority of the shear is concentrated on this side. Using the support column width of 14" does not provide enough shear area, as the shear resistance required of steel reinforcement exceeds the maximum allowed value of $8\sqrt{(f'_c)}b_w d$. Thus, the width of the beam was increased from 14" to 18".

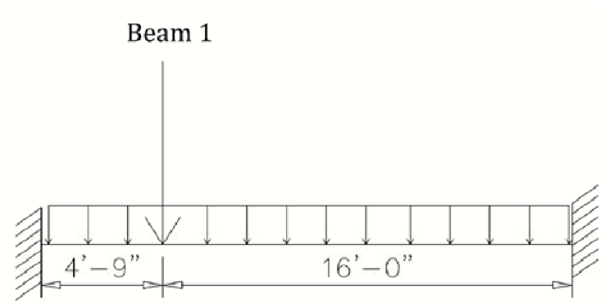


Figure 50: Simplified Model Used in Design of Beam 3

With this modification, a torsional moment was induced in Beam 3 due to its eccentricity with the column support. The torque is such that Eq. 11-18, shown below, from ACI 318-08 11.5.3.1 is not satisfied, which means the section is not large enough for the combined forces of shear and torsion.

$$\text{For Solid Sections: } \left\{ \left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^{\ 2}}\right)^2 \right\}^2 \leq \phi \left(\frac{V_c}{b_w d} + 8\sqrt{f_i}\ \right) \quad \text{ACI Eq. 11-19}$$

Because increasing the depth is not an option, widening the beam is the only way to increase the beam cross section, which will only increase the reaction eccentricity, the induced torque, and the need for a larger section.

Integration with Post-tensioned Slab Perimeter

Because a portion of the transfer system overlaps with the post-tensioned slab perimeter design, one must address how these two systems work together. For the 19th level, the post-tensioned tendons will not span the east-most perimeter span, as shown in Figure 51 below. A "dead end" will be located at Column 14, forcing the tendons to be stressed from only one end. This will result in increased losses due to friction. The post-tensioning system for this level will have to be designed with these increased losses in mind. This will not be explored in this report, as the result will likely vary only slightly from the typical floor design.

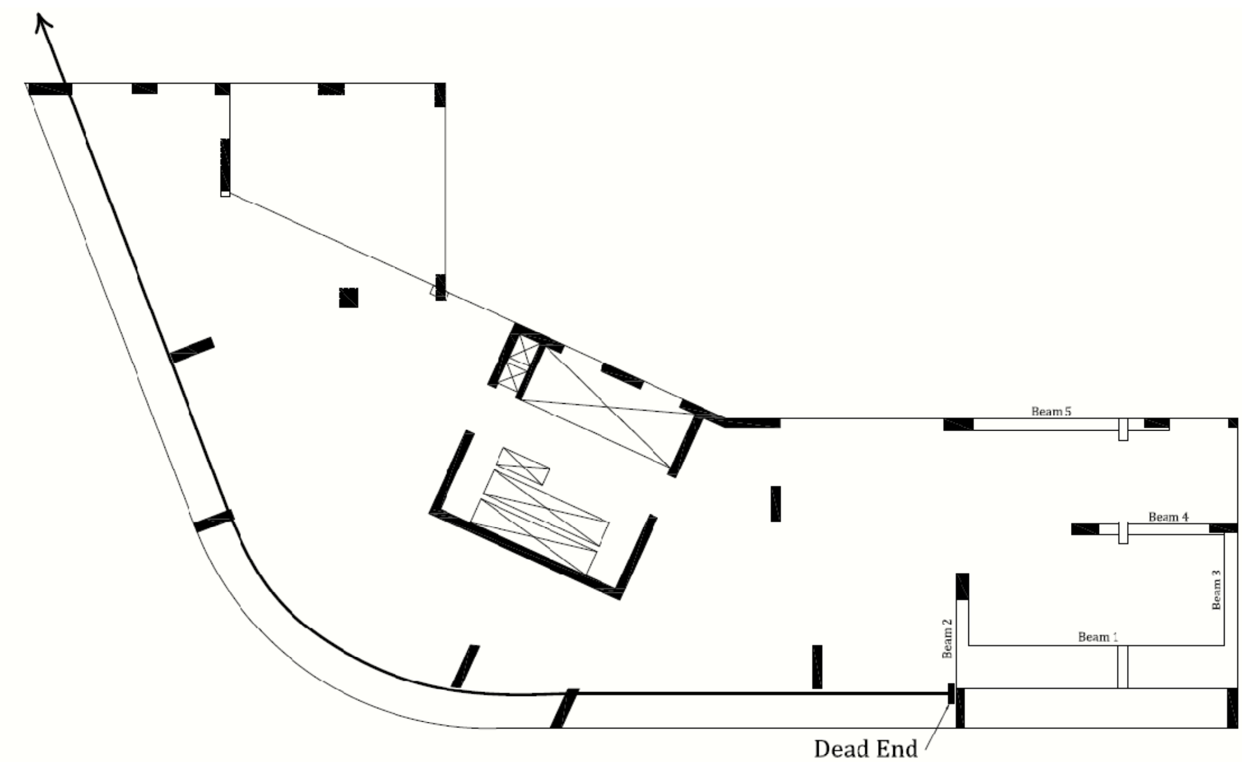


Figure 51: PT design shown together with redesigned transfer system

The end span can go without post-tensioning on this level because of the presence of Beam 1. Beam 1 was designed to support the 4.5'-wide, 9"-thick slab portion between it and the slab edge. The 18.5" thick Beam 1 will support the end span in nearly the same fashion as the 18.5"-thick slab perimeter did in the existing design. Should this 9" slab portion require additional stiffness, a fraction of the 16 prestressed strands can continue on through the end span and be anchored at the corner of the building.

The 9"-thick interior slab spanning between the transfer beams can be reinforced much like the remainder of the building, as the slab spans distances similar to those found in the rest of the structure. The design of this 9" slab spanning between the transfer beams was not carried out, as the intent of this portion of the report was to redesign the transfer system, and there is little question that this portion of the slab can easily resist the dead and live area loads imposed on it.

Construction Management Breadth

With alternate designs for both the perimeter slab strip and transfer system complete, a construction management study will be conducted. The purpose of this study is to determine the impact the alternate designs will have on 100 Eleventh Avenue's *project cost* and *project schedule.* These impacts are vitally important to determining the success of the alternate systems, as a design's success is closely related to the cost and time required to carry it out. Thus, the impact on cost and schedule will be analyzed for both alternate systems.

***RS Means Building Construction Cost Data 2009* was used to determine material, labor, and equipment costs, as well as daily output for typical crews. The 2009 publication was used because this is the year in which the superstructure of 100 Eleventh Avenue was erected. Information from the project's structural consultants was also used in comparing construction time required. In order to accurately compare systems, both the existing and alternate designs were analyzed using the same resources.**

Slab Perimeter Redesign

Impact on Project Cost

In order to determine the impact on cost, a steel and concrete take off was performed for both the existing and alternate design. The results are shown in Table 12, with detailed calculations found in Appendix E. As can be seen, significant material savings have resulted in the alternate design, with the savings multiplied over 15 levels.

Perimeter Strip Material Take off			
Item	**Concrete (cy)**	**Steel (ton)**	**Prestressed Steel (ton)**
Existing Design	1229	47	0
Alternate Design	696	16	11
Material Savings	+533	+31	-11
Total Weight Reduction	2197 kips		
Structure is 5.2% lighter*			

*Compared to structure weight of 41,852 k calculated in Technical Report 1

Table 12

As mentioned above, RS Means was utilized in determining the material, labor, and equipment costs associated with the material quantities determined above. It is important to note that only the materials and tasks affected by the redesign (concrete, steel reinforcement, prestressed tendons) were looked at. Items such as floor finishing and concrete formwork will have very little differences and thus will not affect the project cost and schedule. The resulting cost study is broken down for both the existing design and post-tensioned design in Tables 13 & 14, respectively. More detailed calculations can again be found in Appendix E.

Existing Design						
Item	Quantity	Costs ($)				
		Material	Labor	Equipment	Total	Total*
Steel Reinforcement (tons)	46.8	77220	22932	-	100152	121680
Cast-in-place Concrete (cy)	1229	171691	33798	16346	221835	257353
Total	-	248911	56730	16346	321987	379033
Adjusted for Location	-	$ 264,095	$ 93,377	$ 16,346	$ 373,817	$ 495,396

*Includes O&P

Table 13: Existing Design Cost Breakdown

Post-tensioned Slab Perimeter Design						
Item	Quantity	Costs ($)				
		Material	Labor	Equipment	Total	Total*
Steel Reinforcement (tons)	16.4	27060	8036	-	35096	42640
Cast-in-place Concrete (cy)	696	97231	15660	7586	120478	138434
Prestressing Steel (lb)	22794	14132	27353	456	41941	60860
Total	-	138423	51049	8042	197515	241934
Adjusted for Location	-	$ 146,867	$ 84,026	$ 8,042	$ 238,936	$ 316,208

*Includes O&P

Table 14: PT Design Cost Breakdown

As is shown in the above tables, the post-tensioned redesign results in a total savings of nearly $180,000, when overhead and profit are included. The reduction in concrete and steel results in less material costs and less labor required to install it. A portion of the monetary savings is counteracted by the increased cost of the post-tensioned tendons, which require over $60,000 to purchase and install.

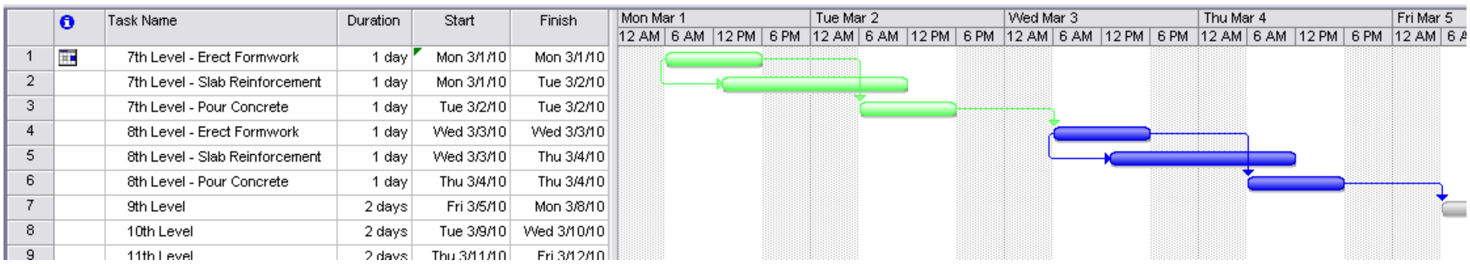
A very important observation can be taken from this cost breakdown. Post-tensioned buildings are very uncommon in New York City and as a result, few contractors have this expertise. There are two main reasons for this. Firstly, post-tensioning requires additional labor for installation and the come-back tensioning of the tendons. This becomes very costly in New York City where labor is of the most expensive in the nation. This is reflected in RS Means, where installation of PT tendons is nearly five times the cost per unit as mild-steel reinforcing. Secondly, post-tensioning has significant value in decreasing a building's weight, which reduces foundation costs. Most buildings in NYC bear on shallow bedrock, so the decrease in building weight is not that important.

Despite the increased labor costs of post-tensioned tendons in the floor system of 100 Eleventh Avenue, the savings resulting from reducing the concrete and mild-steel reinforcement required are enough to reduce the total cost of the superstructure.

IMPACT ON PROJECT SCHEDULE

Equally important as the cost of a building is the time required for construction. The introduction of post-tensioning into 100 Eleventh Avenue will have a significant impact on the construction schedule. The purpose of this analysis is to determine just how the schedule is impacted. Because the post-tensioned design only involves Floors 7 through 21, the lower levels remain the same and can be ignored in this study.

The construction of the existing superstructure was very rapid. Typical floors (Levels 7 through 21) were erected in 2-day cycles. This is shown graphically in Figure 52.



	Task Name	Duration	Start	Finish
1	7th Level - Erect Formwork	1 day	Mon 3/1/10	Mon 3/1/10
2	7th Level - Slab Reinforcement	1 day	Mon 3/1/10	Tue 3/2/10
3	7th Level - Pour Concrete	1 day	Tue 3/2/10	Tue 3/2/10
4	8th Level - Erect Formwork	1 day	Wed 3/3/10	Wed 3/3/10
5	8th Level - Slab Reinforcement	1 day	Wed 3/3/10	Thu 3/4/10
6	8th Level - Pour Concrete	1 day	Thu 3/4/10	Thu 3/4/10
7	9th Level	2 days	Fri 3/5/10	Mon 3/8/10
8	10th Level	2 days	Tue 3/9/10	Wed 3/10/10
9	11th Level	2 days	Thu 3/11/10	Fri 3/12/10

Figure 52: Existing Design Construction Sequence of Levels 7 & 8

For illustrative purposes, assume Level 7 was to begin construction on Day 1. Carpenters would begin building the formwork and would finish by day's end. A crew of lathers would begin placing rebar at mid-day of Day 1. On Day 2, concrete will begin to be poured where reinforcing is in place. The lathers will finish at mid-day of Day 2 and the concrete floor will be entirely poured by the end of Day 2. With the floor entirely shored and formwork still in place, carpenters will begin the cycle again the following day by forming Level 8 above Level 7. Formwork will be stripped from Level 7 and used on Level 9, requiring two sets of forms for the 2-day cycle. This process repeats itself until the roof level is reached, where three days are required for construction.

Carrying this process out, a total construction time of 30 days is required for erection of Levels 7 through 21.

Post-tensioning the perimeter slab will require an additional crew experienced with post-tensioning techniques to place the tendons and stress the tendons once the concrete has reached a strength of f'_{ci} = 3000 psi. According to RS Means, a crew of four (one foreman and three laborers) can place 1200 lb of prestressed steel in a day. The post-tensioned design calls for 1500 lb per floor. Thus, it is reasonable to assume that in New York City's rapid construction pace, the perimeter strip tendons can be placed in a day's time. The construction sequence will also adjust slightly because the tendons will stretch the entire perimeter of the building. For instance, unlike the mild-steel reinforcement, the tendons cannot be placed until all the formwork has been built. Likewise, concrete pouring cannot commence until all the tendons have been placed. This results in a 3-day cycle, which is shown graphically in Figure 53 below.

		Task Name	Duration	Start	Finish	Predecessors	Re
1		7th Level - Erect Formwork	1 day	Mon 3/1/10	Mon 3/1/10		
2		7th Level - Slab Reinforcement	1 day	Mon 3/1/10	Tue 3/2/10	1SS+0.5 days	
3		7th Level - Tendon Placement	1 day	Tue 3/2/10	Tue 3/2/10	1	
4		7th Level - Pour Concrete	1 day	Wed 3/3/10	Wed 3/3/10	3	
5		7th Level - Stress Tendons	1 day	Mon 3/8/10	Mon 3/8/10	4FS+2 days	
6		8th Level - Erect Formwork	1 day	Thu 3/4/10	Thu 3/4/10	4	
7		8th Level - Slab Reinforcement	1 day	Thu 3/4/10	Fri 3/5/10	6SS+0.5 days	
8		8th Level - Tendon Placement	1 day	Fri 3/5/10	Fri 3/5/10	6	
9		8th Level - Pour Concrete	1 day	Mon 3/8/10	Mon 3/8/10	8,6	
10		8th Level - Stresse Tendons	1 day	Thu 3/11/10	Thu 3/11/10	9FS+2 days	
11		9th Level	3 days	Tue 3/9/10	Thu 3/11/10	9	

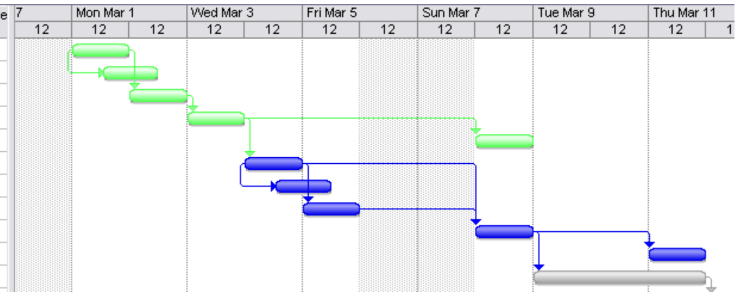


Figure 53: PT Design Construction Sequence of Levels 7 & 8

Again, for illustrative purposes, assume Level 7 was to begin construction on Day 1. Carpenters would begin erecting formwork and would finish at day's end. Lathers would begin placing mild-steel reinforcement at mid-day of Day 1 and finish at mid-day of Day 2. Tendons will be placed beginning on Day 2, after completion of the formwork. This will take approximately one day, allowing concrete to be poured on Day 3. On Day 4, with Level 7 fully shored and formwork still in place, formwork can begin to be erected for Level 8.

The tendons still need to be stressed, which cannot take place until the concrete has reached f'_{ci} = 3000 psi, the initial strength used in all PT calculations. The following equation was used to determine that the time required for concrete strength to reach $0.5f'_c$ = 3000 psi is two days.

$$\text{For Type III cement:} \quad f'_{c(t)} = f'_{c,(28\ days)} \left(\frac{t}{2.3+0.92t}\right) \quad \text{ACI Committee 209}$$

Thus, a minimum of two days after completion of a level, the tendons can be stressed. Once they are stressed, the forms and shoring can be removed. The slab will perform satisfactorily at this point because it has been designed for an initial stage where concrete is not at full strength and a service stage where all loads are applied and concrete is at full strength.

If this process is extrapolated through Level 21, the total construction time required for these levels is 48 days, compared to the 30 days required for the existing design.

Transfer System Redesign

Impact on Project Cost

As with the perimeter slab design, a material take off was performed for both the existing and alternate design. The results are shown in Table 15. As can be seen, the amount of concrete and steel has been reduced in the alternate design. However, the introduction of beams into the system requires significantly more formwork and labor, which will both increase cost and lengthen the schedule. To what extent the cost and schedule are affected will ultimately determine if the design is satisfactory.

Transfer System Material Take off			
Item	Concrete (cy)	Steel (ton)	Beam Formwork (sfca)
Existing Design	71	11	0
Alternate Design	42	6.5	370
Material Savings	+29	+4.5	-370
Total Weight Reduction	127 kips		
Transfer System is 40% lighter / Structure is 0.3% lighter*			

*Compared to structure weight of 41,852 k calculated in Technical Report 1

Table 15

By once again utilizing RS Means for material, labor, and equipment costs, the total cost for both the existing and alternate system was determined. The results are broken down by material in Tables 16 & 17 below.

Existing Design						
Item	Quantity	Costs ($)				
		Material	Labor	Equipment	Total	Total*
Slab Reinforcement - #4 to #7 (tons)	0.5	825	245	-	1070	1300
Slab Reinforcement - #8 to #18 (tons)	10.7	17655	3124	-	20779	22802
Cast-in-place Concrete (cy)	71	9919	1953	944	12816	14867
Total	-	28399	5322	944	34665	38969
Adjusted for Location	-	$ 30,131	$ 8,760	$ 944	$ 39,835	$ 50,933

*Includes O&P

Table 16: Existing Transfer System Cost Breakdown

Transfer Beam Design						
Item	Quantity	Costs ($)				
		Material	Labor	Equipment	Total	Total*
Slab Reinforcement - #4 to #7 (tons)	1.6	2640	784	-	3424	4160
Slab Reinforcement - #8 to #18 (tons)	0	0	0	-	0	0
Slab Cast-in-place Concrete (cy)	27.3	3814	614	298	4726	5064
Beam Formwork - Exterior (sfca)	109	108	649	-	756	1123
Beam Formwork - Interior (sfca)	261	300	1263	-	1563	2297
Beam Reinforcing - #3 to #7 (tons)	1.3	2015	1157	-	3172	4095
Beam Reinforcing - #8 to #18 (tons)	3.6	5580	1908	-	7488	9270
Beam Cast-in-place Concrete (cy)	7.7	1076	424	204	1703	2047
Total	-	15533	6799	502	22833	28056
Adjusted for Location	-	$ 16,480	$ 11,190	$ 502	$ 28,172	$ 36,669

*Includes O&P

Table 17: Alternate Transfer System Cost Breakdown

As can be seen, a monetary savings of approximately $15,000 results. This is likely a trivial amount and will be addressed later. Material costs were nearly cut in half, but the cost involved with labor not surprisingly increased.

IMPACT ON PROJECT SCHEDULE

Because little information was attainable concerning the time required for construction of the existing 19th floor transfer system, RS Means was relied upon for comparable typical crew output. The relevant values used are shown in Table 18.

Construction Output from RS Means Building Construction Cost Data, 2009		
Material	Unit	Output/Day
Forms in Place, Beam	sfca	377
Forms in Place, Flat Plate	sfca	560
Reinforcing, Beam	ton	2.7
Reinforcing, Slab, #4-#7 and higher	ton	2.9
Reinforcing, Slab, #7 and higher	ton	4.9
Concrete, Slab, 6-10" thick, Crane & Bucket	cy	110
Placing concrete, Slab, over 10" thick, Crane & Bucket	cy	90
Placing concrete, Beam, Crane & Bucket	cy	45

Table 18

Using these values along with the material quantity determined in the system take offs, an accurate idea of the additional time required to construct the beam transfer system can be arrived at.

For the existing design, the heavily reinforced transfer slab will require 2 days for lathers to place the rebar and another day to pour the concrete, resulting in a total of 3 days for the existing transfer system design.

The alternate transfer system design requires one additional day to build the beam formwork, 2.5 days to place the slab and beam reinforcement, and 1 day to pour the concrete, resulting in a total of approximately 4.5 days. Calculations showing how these figures were arrived at can be found in Appendix E.

Therefore, using RS Means as a guide to typical crew output, implementing beams into the 19th floor transfer system in order to reduce the thickness of the slab and the amount of rebar requires 1 ½ additional days to construct.

Alternate System Conclusions

The ultimate quality of a design is a complicated function of many items, ranging from the most obvious (whether or not strength and service requirements are met) to more management-oriented concepts (such as cost and construction time). This section attempts to use these and other criteria to determine whether or not the redesign is a satisfactory alternative to the existing design. An overall summary of these comparisons is tabulated in Table 11.

Alternate System Conclusions						
Design	Strength, Service, and Architectural Req'ts Met?	Interior Appearance	Monetary Savings	Material Savings	Weight Reduction	Impact on Schedule
PT Perimeter Slab Strip	Yes	Improved	$180,000	Significant	2197 k	+18 days
Transfer Beam System	No	Worsened	$15,000	Insignificant	127 k	+1.5 days

Table 6

PT Perimeter Slab Strip

Strength, Service and Architectural Requirements

The goal of implementing post-tensioning into the perimeter slab strip was to reduce the 18.5" slab thickness at the perimeter to the 9" thickness found elsewhere. This was accomplished while still meeting all strength and deflection requirements. Slab deflections were the primary reason for the thickened slab in the existing design. The prestressed tendons aided in the 9"-thick slab deflections by keeping section stresses from exceeding the modulus of rupture and thus allowing the slab to be treated as uncracked.

Interior Appearance

Perhaps the most significant improvement in the post-tensioned design is keeping the slap perimeter thickness constant throughout the floor. Not only does this allow for higher floor-to-ceiling heights at the interior space's perimeter, but the smooth, uninterrupted floor soffit creates a much more pleasing appearance. This improvement alone is enough to designate the redesign as successful, so long as the cost and schedule are not negatively impacted.

Savings

The use of PT tendons significantly reduces the amount of concrete and mild-steel reinforcement required. The high cost of using post-tensioning is more than made up for by the material and labor savings of concrete and reinforcing, resulting in a cheaper structure. Using RS Means, the monetary savings are approximated as $180,000 (a 7% reduction in system cost). In addition, the weight of the superstructure is reduced by 2197 k – over 5% of the total existing structure weight. This has the potential to reduce foundation costs and seismic-induced forces.

Schedule

Implementing post-tensioning into 100 Eleventh Avenue extends the superstructure schedule by approximately 18 days. While undesirable, this is likely insignificant in comparison to the entire project schedule.

Conclusion

By using the above categories as gauges, post-tensioning 100 Eleventh Avenue's perimeter slab is deemed an appropriate alternative design. Not only does it allow for a constant slab thickness throughout the majority of the floor, but does so while saving money and reducing building weight.

Transfer Beam System

Strength, Service and Architectural Requirements

As previously discussed, in designing the five transfer beams that make up the system, requirements from three general categories (strength, serviceability, and architectural) were to be met. It was discovered that all three of these requirements could not be satisfied on each structural member. Beam 1's span and loading is such that deflection requirements cannot be met without adding depth to the member, which would violate the architectural requirements of floor-to-ceiling height. Also, the combination of shear and torsion on Beam 3 also required the deepening of the member – creating a clash between strength and architectural requirements.

Interior Appearance

Assuming the transfer beam design could have been accomplished, the appearance of the soffit has worsened. The interior design is such that the underside of the slab will be exposed and used as the ceiling. The ridges and valleys created by the beam system takes away from the clean, uninterrupted look that the 18.5" transfer slab accomplished.

Savings

The transfer beam design reduced the concrete and steel required and subsequently reduced the cost by $15,000. On a project such as this, however, such savings pale in comparison to the overall building budget. A weight reduction of 0.3% of the total was also deemed insignificant.

Schedule

The additional formwork needed to build the transfer beams requires 1.5 additional days to construct.

Conclusion

Because the strength, service, and architectural requirements could not be met, this redesign is an unsatisfactory alternate. Even if these requirements were able to be satisfied, the worsened visual appearance of the soffit combined with insignificant material and cost savings would again point towards this being a poor alternate design.

Shading Breadth

100 Eleventh Avenue's defining feature is clearly its glittering glass facade of hundreds of irregularly shaped windows which reflects fragments of sky and the surrounding city outwards and allows for magnificent, unobstructed views from within. A very important issue to be dealt with when such a curtain wall is in use is the regulation of the sunlight entering the interior. By not regulating the amount of penetrating sunlight, unwanted heat gains can occur during cooling periods, inducing significant, costly loads on the mechanical equipment. In addition, sunlight can cause visual discomfort in the form of glares off reflective surfaces or the bright intensity of the sun itself.

Figure 54: Interior of residential unit showing current shading devices

Figure 55: Product Image of Roller Shades from Lutron

The current design calls for Lutron solar shades to be used along the perimeter curtain wall, as seen in Figure 54 above. These shades allow occupants to vary the amount of penetrating sunlight with the touch of a button.

Interior shading devices such as these (as opposed to exterior shading devices that intercept the intense rays of the sun *before* they pass through a building's transparent envelope) have a significant disadvantage in reducing the amount of heat entering a space. It is estimated in *Mechanical & Electrical Equipment for Buildings, 10th Edition* that effective external shading rejects about 80% of solar energy, whereas internal shading absorbs and reradiates 80% of it. A large reason for this is that external shading can be quickly cooled off by a gentle breeze, but internal shading tends to act as part of a heat trap which radiates heat, creating discomfort for those in the adjacent spaces.

A way in which external shading devices can be implemented into 100 Eleventh Avenue without drastically changing the facade's appearance is to extend the mullions of the panelized windows outward. Figures 56 and 57 show that an uninterrupted mullion is found at the bottom and top of each mega-panel. This will serve as an ideal mullion to "stretch" outwards to intercept the sun.

Figure 56: A single "mega-panel" unit

Figure 57: View of panelized facade from south-west corner

During the summer, the sun is at its highest altitude. Because of this, the most effective shading device for south-facing windows is a horizontal overhang. The extended mullions will serve as this horizontal overhang. The advantage of this technique is that when shading is desired, the sun is at a high altitude, which favors shading. During the winter months when solar heating is desirable, the sun is at its lowest altitude, allowing for its rays to pass beneath the overhang and penetrate the interior spaces.

In order to determine the extent to which the mullion should be extended, the solar data shown in Table 12 below was utilized. For simplicity, only the summer and winter solstices at a solar time of 12:00 (noon) were considered, in order to cover the two seasonal extremes. In addition, because the summer sun is most intense on south-facing surfaces, the south-facing potion of the facade was the focus of this study.

Solar Data for 40° N Latitude @ 12:00 (noon)			
New York City, NY: 40° 47 ' N Latitude			
Date	Description	Altitude	Azimuth
June 21	Summer Solstice	73.5°	0
Dec 21	Winter Solstice	26.6°	0
March 21	Spring Equinox	50°	0
Sept 21	Autumn Equinox	50°	0

Table 7

Figures 58 and 59 show the results of the analysis. The mullion length was designed to block all direct sunlight from entering a south-facing window at solar noon. With this design goal in place and a solar altitude of 73.5°, the mullion length was determined to be 3'-2".

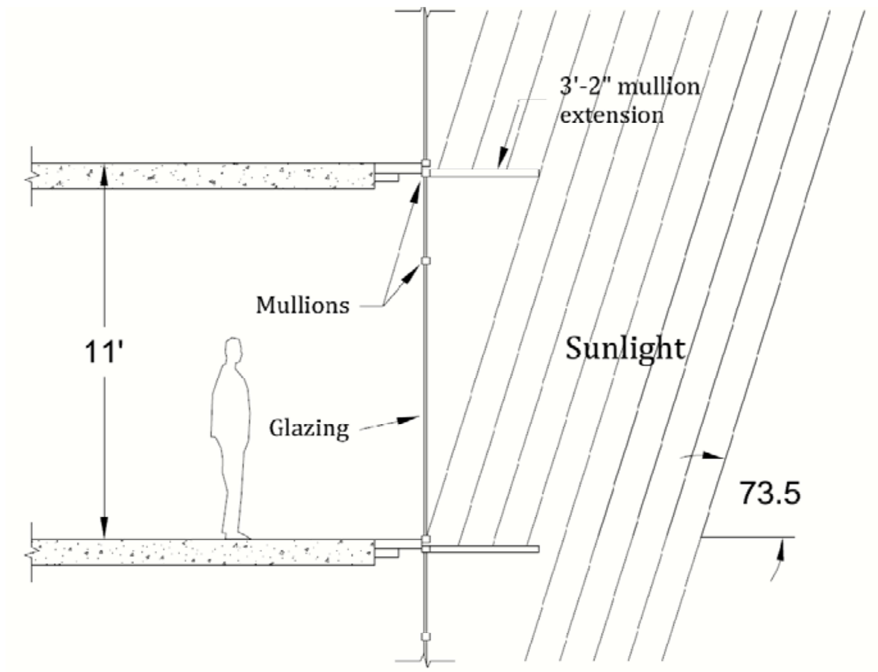


Figure 58: Summer Solstice Shading

Figure 59 shows graphically how the lower solar altitude in the winter promotes desirable solar heating in spite of the horizontal overhang. It's important to keep in mind that the interior solar shades are still available for occupant use, should the winter sun create thermal or visual discomfort.

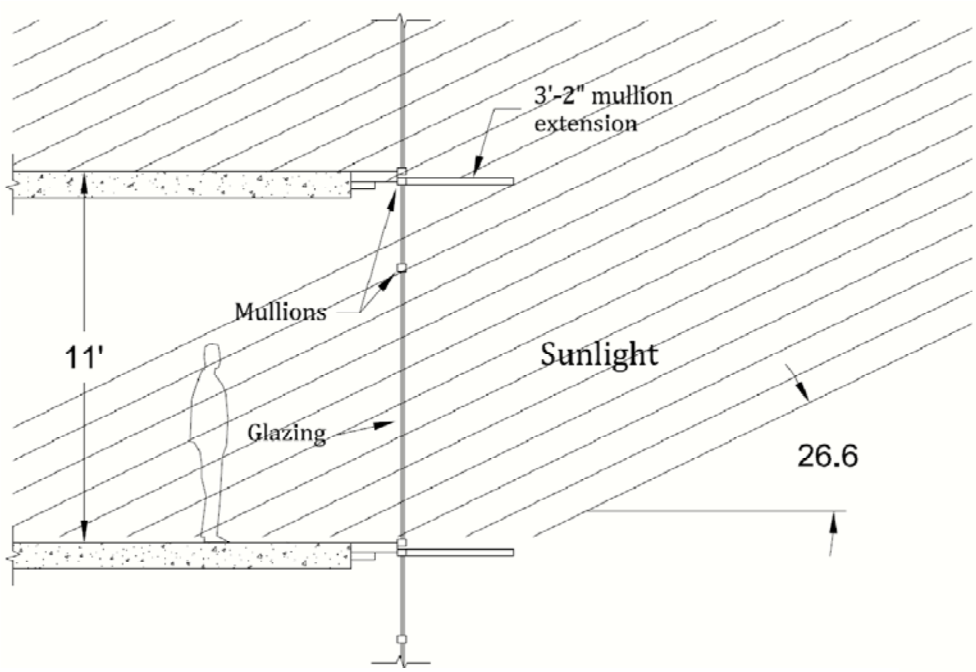


Figure 59: Winter Solstice Shading

The primary concern in altering 100 Eleventh Avenue's facade is the effect on its architecture. Extending the mullions approximately 3 feet will have a noticeable effect on its appearance and needs to be considered for this to be a feasible idea.

From any viewpoint a significant distance from the building such as that shown in Figure 60, the effect on the appearance will likely be minimal. The extended mullions will create much more defined horizontal lines between levels, potentially creating an interesting pattern. Nearby views of the building from ground level such as that shown in Figure 61 however, will likely be significantly altered. Due to the sharp viewing angle, the extended mullions will be very evident, creating a disrupted facade appearance. Views from inside will also be restricted, as building occupants will no longer be able to look directly down or up. This change in how the facade performs creates a very serious hurdle for the acceptance of this shading technique by the architect, as 100 Eleventh Avenue was designed with the idea of it being a "vision machine", providing sweeping views of downtown New York and the Hudson River.

Figure 60: View from a distance

This horizontal shading technique will significantly reduce the amount of solar radiation penetrating the interior spaces during the summer months on the south side of the building. However, during the evening hours, the sun shines primarily on the west side of 100 Eleventh Avenue's facade, but at a much lower altitude. At these times, the horizontal overhang is much less effective for the same reason that solar radiation in the winter can penetrate the envelope. In this case, vertical fins would be an effective shading technique; however, these would almost certainly create a significant divergence from how the architect envisioned the building facade. For this reason, it was deemed a poor solution and no further investigation of vertical fins was carried out.

Figure 61: View from street level

Thus, while integrating horizontal overhangs into 100 Eleventh Avenue's facade has the potential to significantly reduce cooling loads in the summer months, the effects on the intended architecture may be undesirable – enough so to render the extension of mullions an unsatisfactory design.

Overall Summary and Conclusions

Post-tensioning 100 Eleventh Avenue's perimeter slab strip seems, by all accounts, to be a very successful design alternative. The system takes advantage of the entire concrete section in resisting flexural loads by keeping the concrete slab from cracking. This limits deflections (the driving force behind the 18.5" thickness found in the existing design) while keeping the slab relatively thin. The redesign results in higher floor-to-ceiling heights at the interior space's perimeter and a more aesthetically-pleasing soffit appearance, while reducing the overall system cost by $180,000 and reducing the building weight by over 5%. The only negative impact is on the construction schedule, which would be lengthened by 18 days with the implementation of post-tensioning. However, this is likely insignificant when compared to the entire project's schedule and the resulting improvements.

The result of the 19th floor transfer system redesign was quite the opposite of the slab perimeter redesign. The spans and loadings are such that deflection limitations and shear/torsion reinforcement requirements cannot be met without violating strict floor-to-ceiling heights. In addition, the material and monetary savings are not as significant as initially predicted, particularly when compared to the overall project material usage and cost. Finally, it is the author's belief that the exposed soffit appearance would be worsened by the deep ridges and valleys created by the transfer beams.

In addition to the above structural studies, the use of exterior shading devices was also looked at as a breadth study. The exterior shading would be provided by extending the curtain wall mullions found at each slab level outwards a distance of 3'-2". This dimension was arrived at by designing the overhang to intercept all direct sunlight penetrating the south-facing windows at solar noon on the summer solstice. While shading in this manner would be much more effective than the existing interior, user-controlled shades, the negative effect on the facade's performance – both its appearance from outside and the views it provides the building's occupants – will likely relegate this design as unsatisfactory.

Resources

American Concrete Institute. (2002). *Building code requirements for structural concrete (ACI 318-02) and Commentary.* Michigan: ACI.

American Concrete Institute. (2008). *Building code requirements for structural concrete (ACI 318-08) and Commentary.* Michigan: ACI.

American Society of Civil Engineers. (2005). *ASCE 7-05: Minimum design loads for building and other structures.* Virginia: ASCE.

Lutron Shading Solutions, http://www.lutron.com/lutron-shades/default.html.

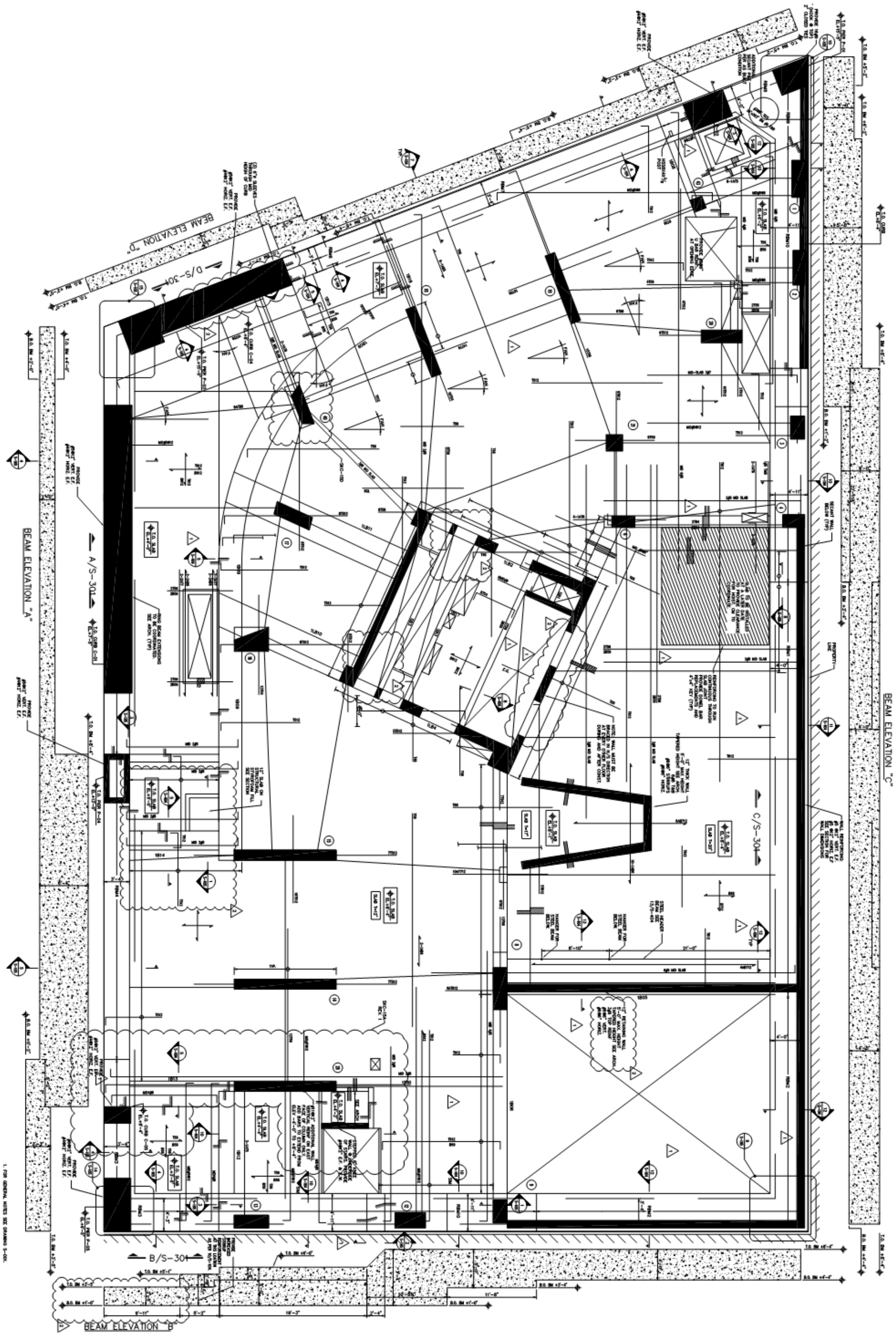
Portland Cement Association. (2005). *Time Saving Design Aids: Two-Way Post-Tensioned Design.* Retrieved February 1, 2010 from http://www.cement.org/buildings/Timesaving-2WayPost-IA.pdf

Stein, B., Reynolds, J., Grondzik, W., Kwok, A. (2006). *Mechanical and electrical equipment for buildings* (10th Edition). New Jersey: John Wiley & Sons, Inc.

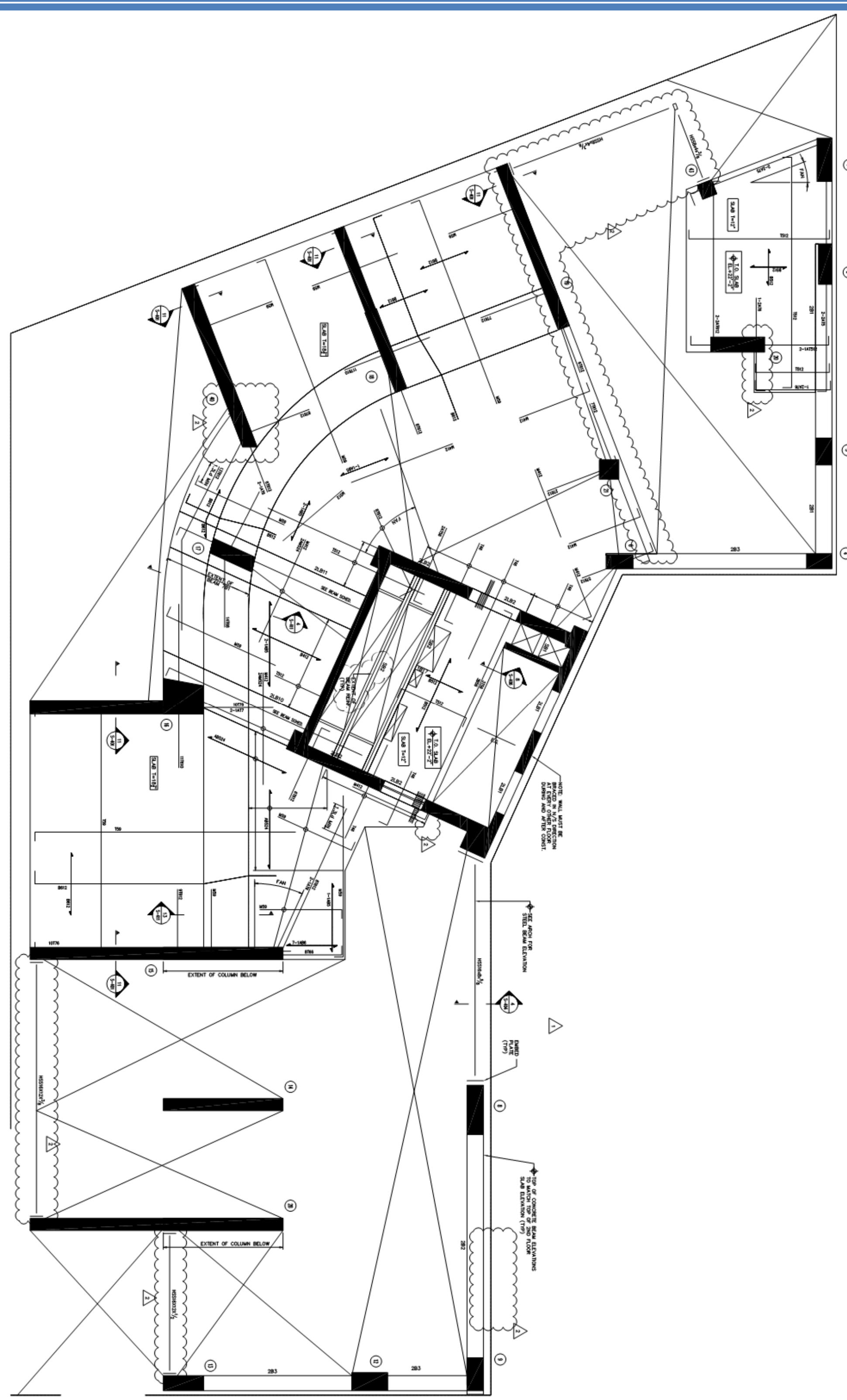
Wight, J., MacGregor, J. (2009). *Reinforced concrete: Mechanics and design* (5th Edition). New Jersey: Pearson Prentice Hall.

APPENDIX A

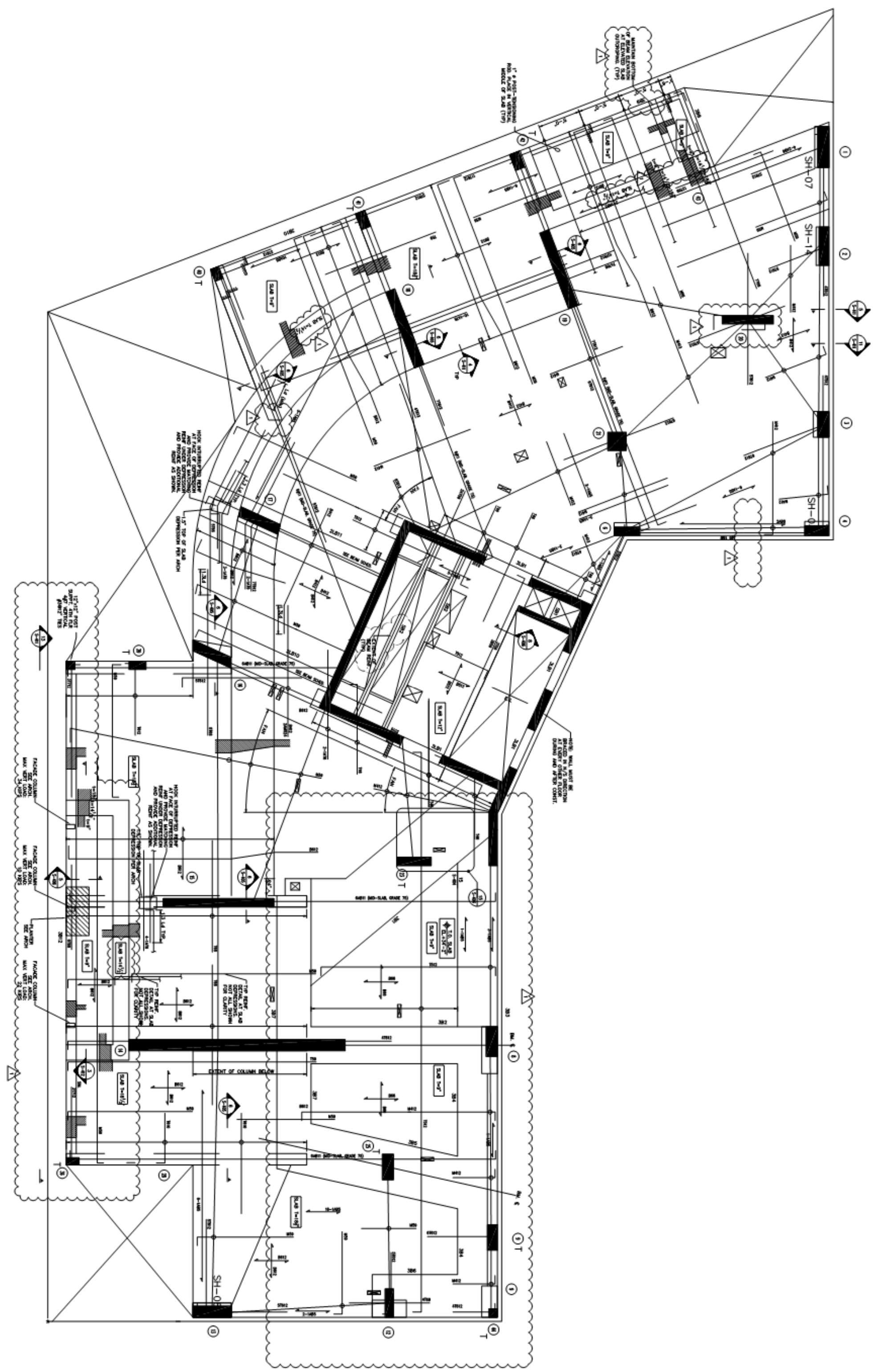
BUILDING PLANS



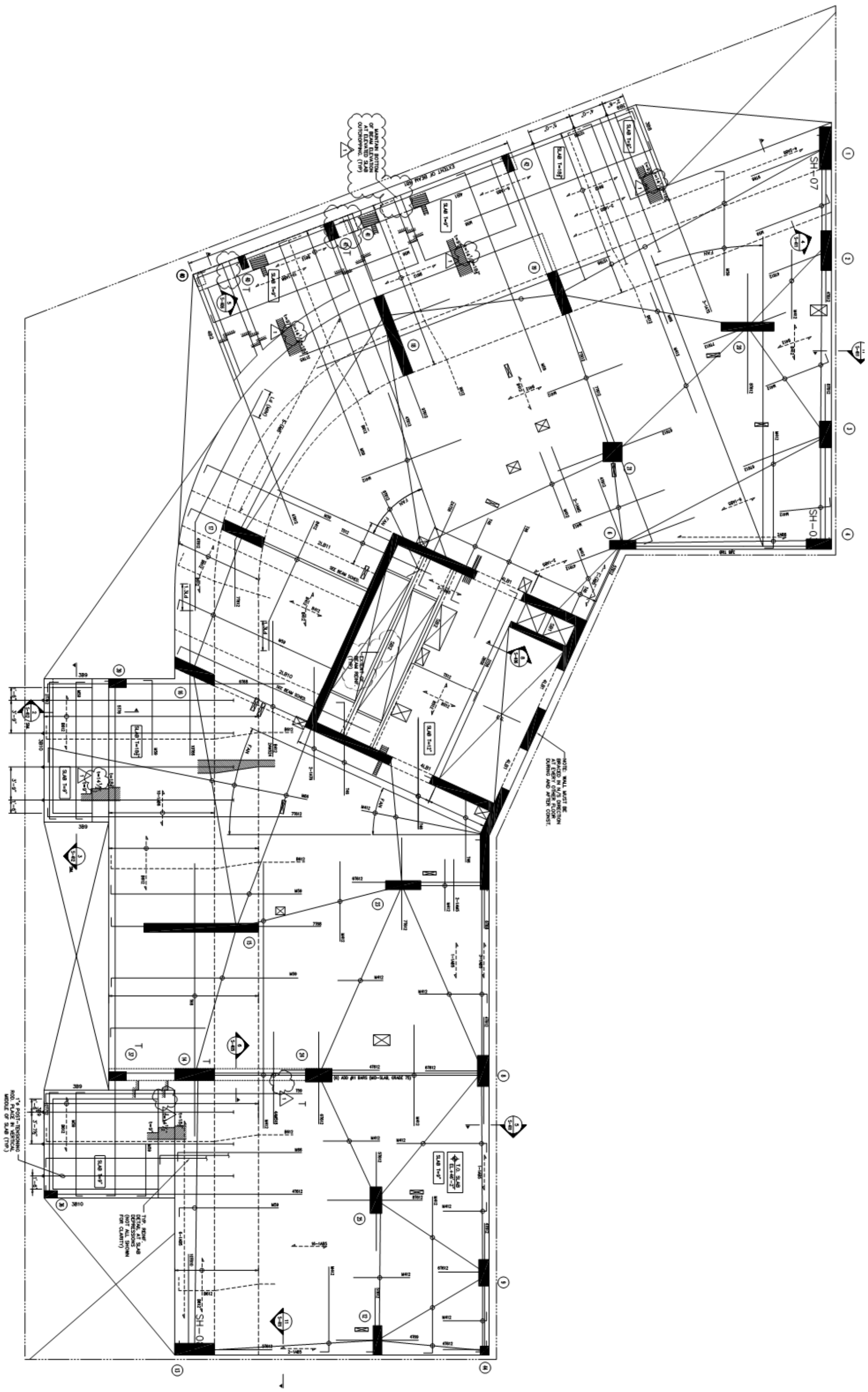
Ground Floor Plan



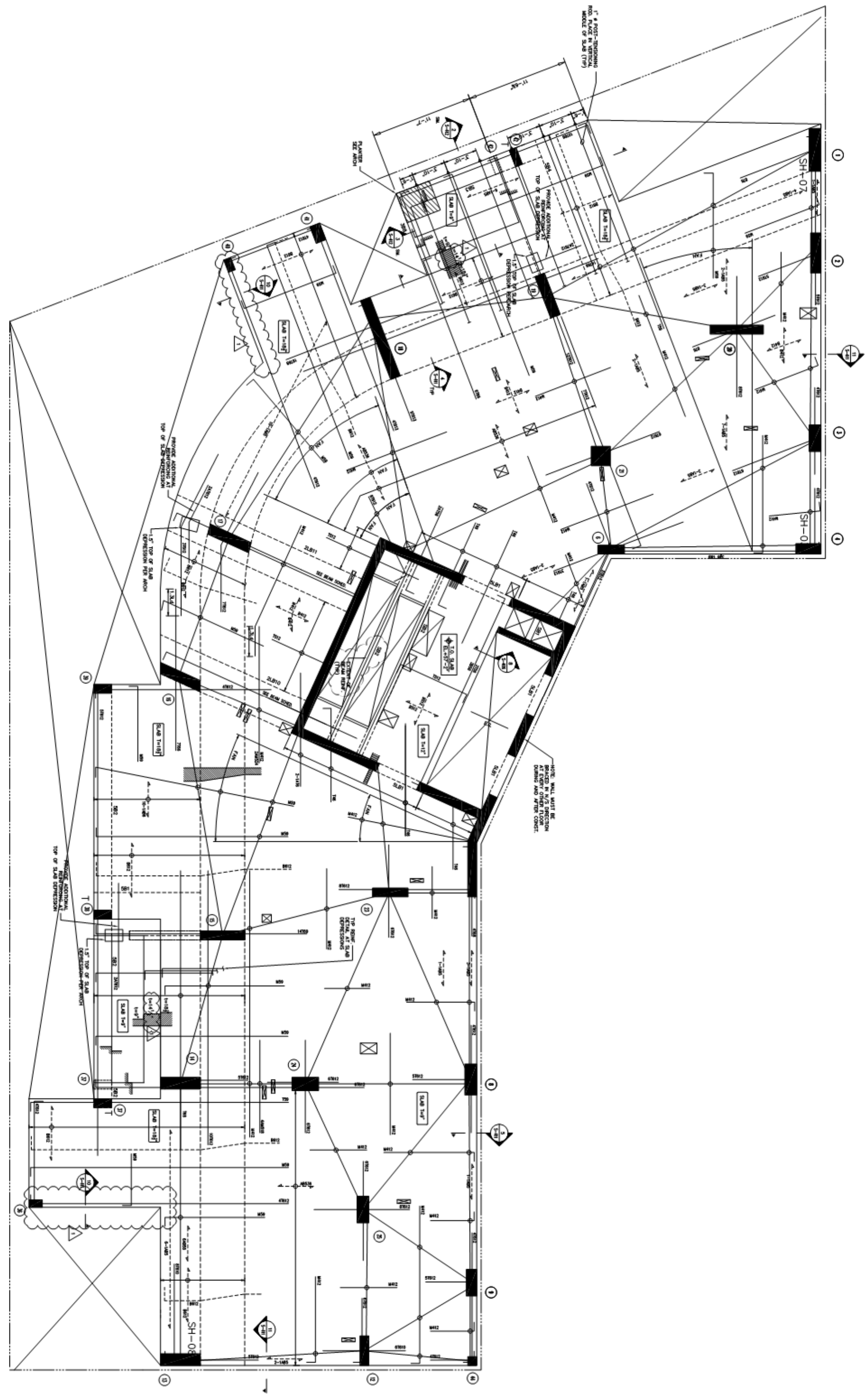
2nd Floor Plan



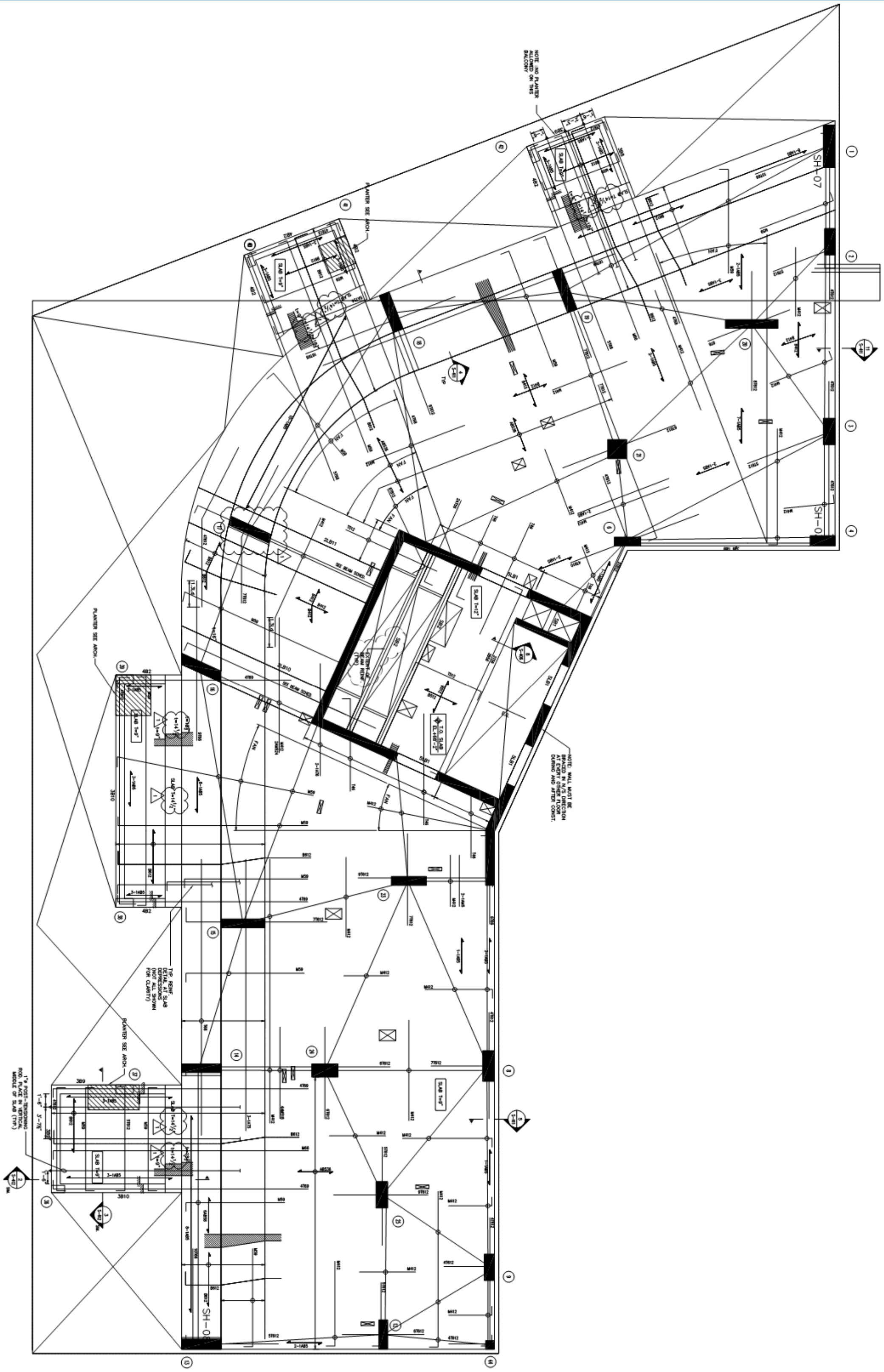
3rd Floor Plan



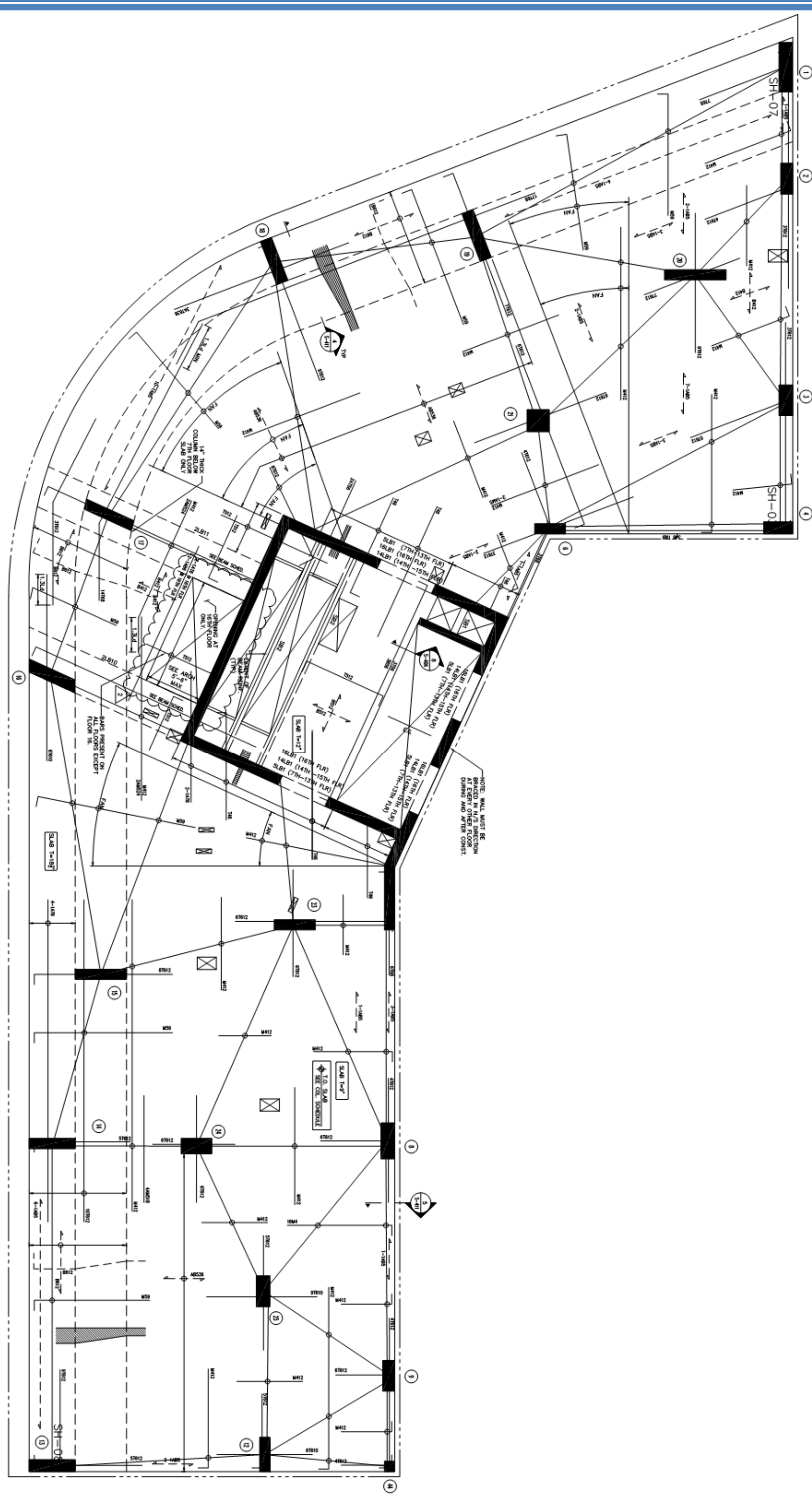
4th Floor Plan



5th Floor Plan



6th Floor Plan



7th-16th Floor Plan
(17th-Roof Plans differ from typical plan only slightly)

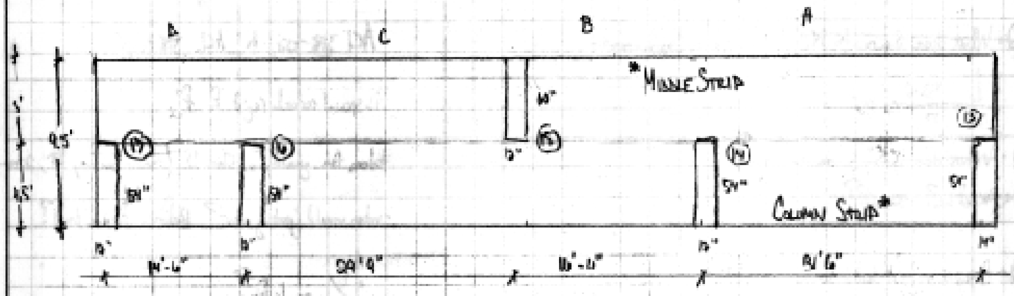
APPENDIX B

POST-TENSIONED PERIMETER SLAB REDESIGN CALC'S

1

Calculation of Moments in Slab by Equivalent Frame Method – horizontal spans

9"-slab



* Assume 5' strip acts as middle strip, as it will be much less stiff b/c only 1 column is located there

$w_L = 40 psf$ $f'_c = 1000$

$w_D = 104 psf$ $f_y = 40{,}000$

· assume all panels loaded by live load Floor height = 11'

Table 9.5(c) → $\frac{l_n}{33} \times 1.1 = \frac{30.5 \cdot 12}{33} \times 1.1 = 12.2" \Rightarrow$ Use 12.25"

no edge beam

$w_u = 1.2(104) + 1.6(40) = 261 psf$

A) Column Stiffnesses

$K_{col} = \frac{4E_cI_c}{L-2t}$ $L-2t = 11' - 2(12.25) = 107.5$

(13) $I_c = \frac{1}{12} \cdot 54" \cdot 14^3 = 12348 in^4$ (14) $I_c = 7776 in^4 = (16) = (13)$ (15) $I_c = 8640 in^4$

$K_{col} = 459E_c$ $K_{col} = 289E_c$ $K_{col} = 321E_c$

B) Torsional Stiffness

$K_t = \frac{9EC}{l_2\left(1-\frac{c_2}{l_2}\right)^3}$

(13) $C = \left(1 - 0.63 \cdot \frac{12.25}{14}\right)\left(\frac{12.25^3 \cdot 14}{3}\right) = 3850$ (14) $C = \left(1 - 0.63 \cdot \frac{12.25}{12}\right)\left(\frac{12.25^3 \cdot 12}{3}\right) = 2624 = (16) = (15)$

$K_t = \frac{9E(3850)}{9.5' \cdot 12 \cdot \left(1 - \frac{54}{9.5 \cdot 12}\right)} = 578E_{cs}$ $K_t = \frac{9E(2624)}{9.5 \cdot 12 \cdot \left(1 - \frac{54}{9.5 \cdot 12}\right)} = 394E_{cs}$

2

(15) $C = 2624$

$$K_c = \frac{9E(2624)}{9.5\cdot 12\left(1 - \frac{62}{9.5\cdot 12}\right)} = 437E_{cs}$$

Equivalent Column Stiffness:

(13) $$\frac{1}{\frac{1}{2\cdot 459E_c} + \frac{1}{578E_{cs}}} = \frac{1}{\frac{1}{2\cdot 459\cdot 1.17E_{cs}} + \frac{1}{578E_{cs}}} = 376E_c$$

(14) $$\frac{1}{\frac{1}{2\cdot 289\cdot 1.17E_c} + \frac{1}{394E_{cs}}} = 250E_c = (16) = (17)$$

(15) $$\frac{1}{\frac{1}{2\cdot 321E_c\cdot 1.17} + \frac{1}{437E_c}} = 274E_c$$

c) SLAB STIFFNESS

$$K_s^A = \frac{4E_c(17463)}{(31.5\cdot 12) - \frac{13''}{2}} \qquad I_s = \frac{1}{12}\cdot 9.5\cdot 12\cdot 12.25^3 = 17463$$

$$K_s^A = 188E_c$$

$$K_s^B = \frac{4E_c(17463)}{(16.5\cdot 12) - \frac{12}{2}} = 364E_c$$

$$K_s^C = \frac{4E_c(17463)}{(29.75\cdot 12) - \frac{12}{2}} = 199E_c$$

$$K_s^D = \frac{4E_c(17463)}{(14.5\cdot 12) - 6} = 416E_c$$

MOMENT DISTRIBUTION

$$DF_{13\text{-}14} = \frac{188}{188 + 376} = 0.333$$

$$DF_{14\text{-}13} = \frac{188}{188 + 364 + 250} = 0.23$$

$$DF_{14\text{-}15} = \frac{364}{364 + 188 + 250} = 0.45$$

$$DF_{15\text{-}14} = \frac{364}{364 + 199 + 276} = 0.43$$

$$DF_{15\text{-}16} = \frac{199}{199 + 364 + 276} = 0.24$$

$$DF_{16\text{-}15} = \frac{199}{199 + 416 + 250} = 0.23$$

$$DF_{16\text{-}17} = \frac{416}{416 + 199 + 250} = 0.48$$

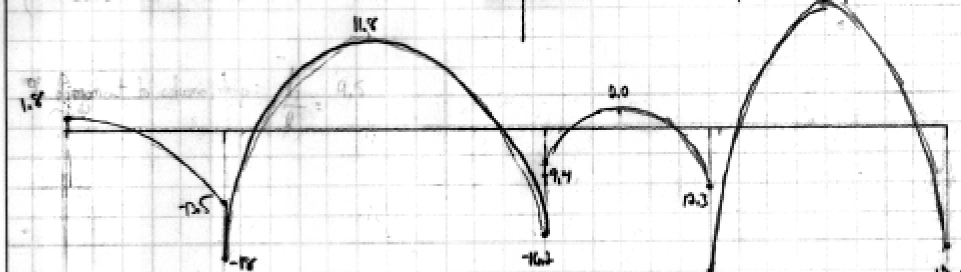
$$DF_{17\text{-}16} = \frac{416}{416 + 250} = 0.62$$

3

$FEM = \frac{w_u \cdot l_2 \cdot l_n^2}{12}$

$= \frac{k\text{-in}}{ft\text{-width}}$

	17		16			15			14		13
DF	0.62		0.48	0.33		0.24	0.43		0.45	0.33	0.33
	-4.57		4.57	-19.3		19.3	-5.92		5.92	-21.6	21.6
	3.83	→	1.92							-3.6 ←	-7.1
	8.4	←	16.8	3.1 →		1.6	4.4	←	8.7	4.4 →	2.2
				-2.4	←	-4.7	-8.3	→	-4.2		
	0.6	←	-1.2	-0.6	→	+0.3	-1.0	←	-1.9	-1.0 →	+0.5
	-0.3	→	-0.1	0.3		-0.3	-0.6			0 ←	-0.6
			-0.1								
	1.76		13.5	-18.5		16.2	-9.42		12.3	-19.8	18.0



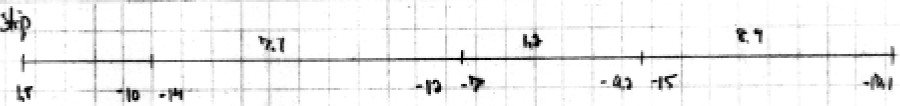
$\frac{l_2}{l_1} : \frac{45}{x} = 0.66$ 0.32 0.58 0.30

% Moment to Column Strip: 100

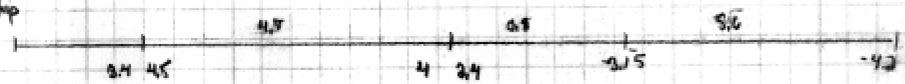
100%

Exterior (-) : 100%
All other (-) : 75%
All positive : 60%

Column Strip



Middle Strip



4

Corrected Moment Distribution

DF	0.52		0.48	0.22		0.24	0.43		0.45	0.23		0.33
	-13		13	-56		56	-17		17	-63		63
	8	→	4							-11	←	-21
	9	←	19	9	→	5	13	←	26	13	→	7
				-7	←	-14	-25	→	-12			
	2	←	3	2	→	1	2	←	5	3	→	2
	-7	→	-4			-1	-1			-2	←	-3
	1	←	2	1					1			
	0	(1.5)	57	-51	(33)	47	-28	(6.6)	37	-60	(47)	48
CS	0	(1.0)	38	38	(21)	35	21	(4)	28	45	(28)	45

5

0.2049 0.24 0.2?

Check that 12.25" is sufficiently thick for strength requirements

$$d^2 = \frac{M_u}{\phi b \rho f_y \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)}$$

$$d^2 = \frac{\cancel{748}\ \text{ft}\cdot\text{k} \cdot 12000}{0.9(12")(0.0239)(60{,}000\text{psi})\left(1 - 0.59 \cdot 0.0239 \cdot \frac{60}{6}\right)}$$

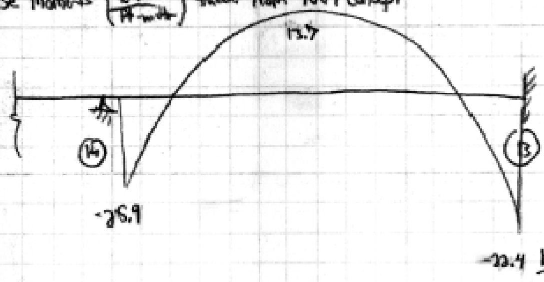
$d \geq 9.4"$

This set of calculations verified that the moments produced by RAM Concept were sensible. These moments were then used to determine the minimum slab thickness capable of resisting the loads. Because only a 10" slab would suffice, deflections were determined to be the controlling criteria for the 18.5" perimeter slab.

1

Sample Deflection Calc - Span (13)(14)

- use moments $\left(\frac{k \cdot ft}{ft\text{-width}}\right)$ taken from RAM Concept



(Moments due to service loads)

$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{6000} = 581 \text{ psi}$

$M_{cr} = \frac{f_r I_{ut}}{y_t}$ $\quad I_{ut} = \frac{1}{12}(12'')(12.25'')^3 = 1838 \text{ in}^4$ (ignoring reinforcement)

$M_{cr} = \frac{581 \cdot 1838}{6.125''} = 14.5 \text{ k}$

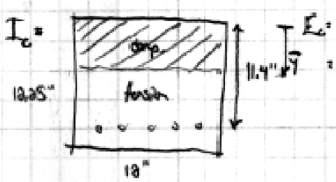
Dead load moments = -22.5, +11.8, -19.4

$I_{e,\text{middle}} \Rightarrow$ Use I_g

$I_{e,\text{right end}} = \left(\frac{14.5}{19.4}\right)^3 \cdot 1838 + \left(1 - \left(\frac{14.5}{19.4}\right)^3\right) 1383 = 1573$

$I_{e,\text{left end}} = \left(\frac{14.5}{22.5}\right)^3 \cdot 1838 + \left(1 - \left(\frac{14.5}{22.5}\right)^3\right) 1383 = 1506$

$I_{cr} =$



$E_c = 57\sqrt{6000} = 4415$

$n = \frac{29000}{4415} = 6.6$

$A_s = 0.0239(b \cdot d) = 0.0239(12 \cdot 11.4) = 3.3 \text{ in}^2$

$n \cdot A_s = 21.6$

$(\bar{y})(12'')(6'') = (21.6)(11.4 - \bar{y})$

$\bar{y} = 4.9''$ $\quad 11.4 - 4.9 = 6.5''$

$I_{cr} = \frac{1}{12} \cdot 12 \cdot 4.9^3 + 12 \cdot 4.9 \cdot \left(\frac{4.9}{2}\right)^2 + (21.6)(6.5)^2 = 1383$

$I_e = 0.5(1838) + 0.25(1573 + 1506) = 1689 \text{ in}^4$

$(\Delta_i)_{DL} = \frac{5(0.5 + 0.164)(30.5)^4}{384(4415)(1689)} \cdot 1728 = 1.7''$

2

$(\Delta_i)_{DL+LL}$:

$I_{e,middle} = I_g$

$I_{e,right} = 1506$

$I_{e,left} = \left(\frac{14.5}{25.9}\right)^3 1838 + \left(1 - \left(\frac{14.8}{25.9}\right)^3\right) 1363 = 1463$

$I_e = 0.5(1838) + 0.25(1506 + 1463)$

$= 1661\ in^4$

$(\Delta_i)_{DL+LL} = \frac{5(0.5 + 0.201)(30.5)^4}{384(4415)(1661)} \cdot 1728 = 1.86''$

$(\Delta_{long})_{DL+LL} = \lambda \Delta_i = 2.0(1.86'') = 3.72''$

$\lambda = \frac{1}{1 + 50\rho'} = \frac{2.0}{1 + 50 \cdot 0} = 2.0$

$\rho' = 0$

Thus, total Δ after attachment of nonstructural elements: $(\Delta_{long})_{DL+LL} - (\Delta_i)_{DL} = 3.72'' - 1.7'' = 2.0''$

Deflection limitation $= \frac{l}{480} = \frac{30.5 \cdot 12}{480} = 0.76''$

Using the minimum slab thickness specified by ACI 318-08 for the spans present, these calculations verified that deflections were the driving force for the 18.5”-thick slab perimeter.

Hand Calculation (Approximation) of Single Span PT Design

1

- treat 9.5' strip as equivalent frame (ignoring column stiffness for simplicity) - Note, this may change results significantly from RAM Concept
- Facade load applied as a linear load down center of frame, after initial prestress force loss

Post Tensioning

$\frac{1}{2}$" ø 7 wire strands, $A = 0.153 \text{ in}^2$

Unbonded, $f_{pu} = 270$ ksi

Assume 14 ksi of prestress loss

$f_{se} = 0.7(270) - 14 = 175$ ksi

$P_{eff} = 0.153(175) = 26.7$ k/tendon

Design calls for 16 tendons in strip, with tendon profile as detailed in sketch above

Loading

$D_s = (150 \text{ pcf})(\frac{9}{12}) = 112.5$ psf

$D_{SI} = 50$ psf

Facade load $= 500$ lb/ft

Live $= 40$ psf

Section Properties: $A = bh = (9.5 \cdot 12)(9) = 1026 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(9.5 \cdot 12)(9)^2}{6} = 1539 \text{ in}^3$

Precompression Stress

$\frac{P_{actual}}{A} = \frac{(26.7 \cdot 16)(1000)}{1026} = 416 \text{ psi} > 125 \text{ psi minimum}$

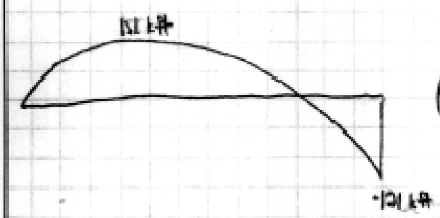
2

Balanced Load

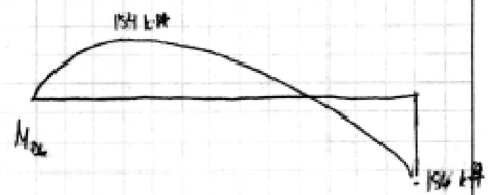
$w_b = \frac{8aP}{L^2} = \frac{8(5.25)(26.7 \cdot 1.6)}{34^2} = 1.29 \text{ k/ft}$ $\qquad \frac{w_b}{w_D} = \frac{1.29}{2.07} = 62\%$ (compared to 67% in RAM Concept)

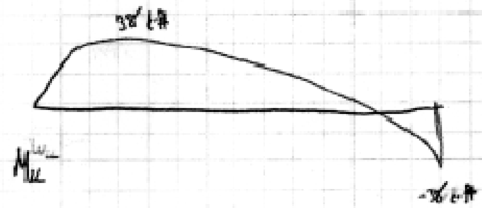
$a = \frac{8+4.5}{2} - 1.0 = 5.25$

M_{bal}



(From SAP)

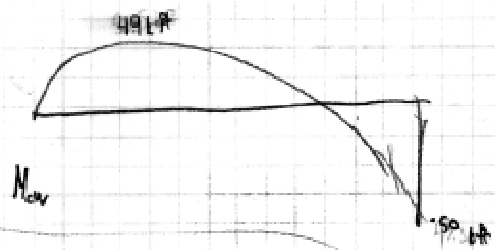




$w_{DL} = \frac{(10.5+50)(9.5)}{1000} = 1.56 \text{ k/ft}$

$w_{LL} = 0.38 \text{ k/ft}$

$w_{DL\,(wind)} = 0.5 \text{ k/ft}$



Check Slab Stresses

Allowable Stresses: @ jacking, $f'_{ci} = 3000$ psi

compression $\leq 0.6(3000) = 1800$

tension $\leq 3\sqrt{3000} = 164$

@ service, $f'_c = 6000$ psi

compression $\leq 0.45(6000) = 2700$

* tension $\leq 7.5\sqrt{6000} = 581$

* Note: Because floor is predominantly a mild steel reinforced two-way system, flexural members are use Class U: $f_t \leq 7.5\sqrt{f'_c}$

3

Stage 1: @ jacking

Midspan

$f_{top} = \frac{(-154+131)(12000)}{1539} - 416 = -595\ psi$

$f_{bot} = \frac{(154-131)(12000)}{1539} - 416 = -237\ psi$

Support

$f_{top} = -143\ psi$

$f_{bot} = -688\ psi$

$\therefore$ Ok (all < 1800 psi)

Stage 2: Service Loads

Midspan

$f_{top} = \frac{(-154+131-38-49)(12000)}{1539} - 416 = -1273$

$f_{bot} = \frac{(154-131+38+49)(12000)}{1539} - 416 = +442$

Support

$f_{top} = -1375$

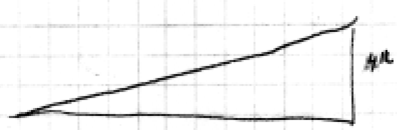
$f_{bot} = +543$

$\therefore$ Ok $\rightarrow$ all compression < 2700 psi
" tension < 581 psi

Ultimate Strength

$M_1 = Pe$

$M_{1\,@support} = (427)(3.5) = 1495\ in\text{-}k = 125^{k}$

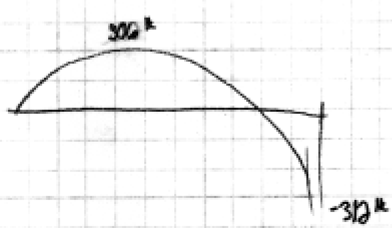


$M_{sec} = M_{bal} - M_1 = 121 - 125 = 4^{k}$

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

$M_{mid} = 1.2(154+49) + 1.6(38) + 2 = 306^{k}$

$M_{supp} = 1.2(156+56) + 1.6(38) + 4 = 312^{k}$



4

Minimum Bonded Reinforcement

$f_{t,max}$ for (+) moment region $\leq 2\sqrt{f'_c}$

$f_{tot} = 440 > 2\sqrt{f'_c} \Rightarrow y = \frac{442}{(442+1273)} \times 9" = 2.32"$; $N_c = \frac{(154^k + 38^k + 49^k)(2)}{1529 \text{ in}^2} \times 0.5 \times 2.32 \times 9.5' \cdot 2" = 548^k$; $A_s = \frac{548^k}{0.5 \times 60} = 8.27 \text{ in}^2$

$\rightarrow$ 27 #5s (8.37 in^2)

At support : $A_{s,min} = 0.00075 A_{cf} = 0.00075(9.5 \cdot 12 \cdot 9) = 0.77 \text{ in}^2 \rightarrow$ 3 #5's (0.93 in^2)

Flexural Strength

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

$A_{ps} = 0.153 \cdot 16 = 2.4 \text{ in}^2$

$f_{ps} = f_{se} + 10{,}000 + (f'_c b d_p) / (300 A_{ps})$

$= 175000 + 10{,}000 + \frac{6000 \cdot 9.5 \cdot 12 \, d}{300(2.4)} = 185{,}000 + 950d$

$a = \frac{A_{ps} f_y + A_{ps} f_{ps}}{0.85 f'_c b}$

Midspan

$d_{ps} = 9' - \frac{3}{4}" - \frac{1}{4}" = 8"$

$f_{ps} = 192{,}600$

$a = \frac{(2.4)(192600) + (8.37)(60{,}000)}{0.85 \cdot 6000 \cdot 9.5 \cdot 12} = 1.66"$

$\phi M_n = 0.9(2.4 \cdot 192.6 + 8.37 \cdot 60)(8 - \frac{1.66}{2}) = 515^k > M_u = 306^{1k}$

$\therefore$ ok

*Note: Ultimate strength ignoring bonded reinforcement is 264^{k}, which is 86% of M_u. Therefore, the reduced stresses that considering strip as part of larger model results in will make for a sufficient design.

Support

$a = \frac{(2.4 \cdot 192600 + 0.93 \cdot 60000)}{0.85 \cdot 6000 \cdot 9.5 \cdot 12} = 0.89"$

$\phi M_n = 0.9(2.4 \cdot 192.6 + 0.93 \cdot 60)(8 - \frac{0.89}{2}) = 873^{1k} < M_u$

$\therefore$ No Good

Increase bonded reinforcement to achieve satisfactory design.
(Concept calls for 9 #5's)

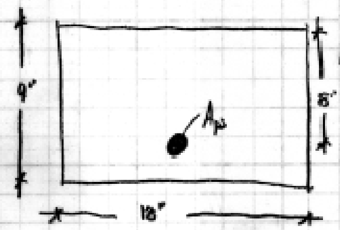
CONCEPT DESIGN CONFIRMED TO BE A VALID DESIGN

*Note: When PT strip modeled as part of entire floor, stresses are decreased significantly, resulting in much less bonded reinforcement in actual design

5

Check Deflection

* Assumption: Along outside edge, deflection is primarily one-way in action, allowing deflections to be closely approximated by treating unit width as a beam



$A_{ps} = 0.153 \text{ in}^2 \times \frac{16}{8.5} = 0.240 \text{ in}^2$

$E_s = 29000$

$E_c = 4415$

$n = 6.57$

$(n-1)A_s = 5.6(0.240) = 1.5 \text{ in}^2$

$I_{ut} = \frac{1}{12} \cdot 12 \cdot 9^3 + 1.50 \cdot 3^2 = 745 \text{ in}^4$

$\Delta_{immediate} = \frac{5wl^4}{384EI_g} = \frac{5(-0.023)(34-1')^4}{384(4415)(745)} \times 1728 = -0.19''$ (w/o cw load) (compared to $-0.16''$ in RAM Concept)

$w_i = (112.5\text{psf} \times 1') - \left(\frac{1.39}{35} \cdot \frac{18}{8.5}\right) \times 1000 = -23 \text{ lb/ft} = -0.023 \text{ k/ft}$

$(\Delta_i)_{total} = \frac{5(0.121)(33)^4}{384 \cdot 4415 \cdot 745} = 0.98''$ → (compared to $0.70''$ in RAM Concept)

$w_{total} = \frac{52 + 40 + 112.5}{1000} + \frac{0.5}{9.5} - \frac{1.39}{9.5} = 0.121 \text{ k/ft}$

DL+SDL+LL / wall load / balanced load

Long Term Loading → $\lambda = 2.0$

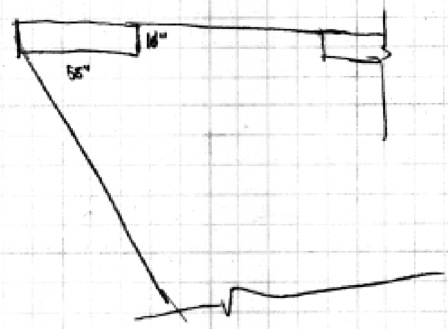
$\Delta_{LT} = 2.0\Delta_{i,service} + \Delta_{i,service} = 3 \cdot \Delta_i = 3(0.98'') = 2.94''$

(compared to $2.14''$ in RAM Concept)

* Note: these values change very much when modeled as part of entire floor

These calculations verified that the results produced through RAM Concept (treating the perimeter slab as a single equivalent frame) were indeed satisfactory. Deflections were also verified to be comparable to those approximated by hand.

Punching Shear Check : Column # 1 — 1



$V_c = \left(2 + \frac{4}{3.63}\right)\sqrt{6000}\,(62.4)(8) = 120^k$ governs

$\beta_c = \frac{58}{16} = 3.63$

$b_o = 58 \times \underbrace{0.8}_{} + 16' = 62.4$

estimate of dimension reduction

Because of angled edge of slab, cannot use entire 58" dimension for shear resistance

$A_T = 146sf$

$A_{shear} = 146sf - \frac{(58 \cdot 0.8 + 4)(16+4)}{144} = 139sf$

trib length for curtainwall (glass) = 17'-4" = 16.7'

" " for masonry wall = 3.5-4" = 3.2'

$V_u = 139sf\left[1.2(52+112.5) + 1.6(40)\right] + 1.2 \cdot 16.7 \cdot 500 + 1.2 \cdot 3.2 \cdot 350$

$V_u = 36^k + 13^k = 49^k$ ⟶ [According to ACI ... $V_u = ...$

Unbalanced Moment: (only critical direction of long span will be checked)

$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{16+4}{58 \cdot 0.8 + 4}}} = 0.70$

$M_o = \frac{(3.08)(33)^2}{8} = 419^{'k} \longrightarrow 0.3M_o = 126^{'k} \Longrightarrow (1-0.70)(126) = 38^{'k}$ to be balanced by shear at column-slab interface

$q_u = 1.2\left[(52+112.5)(9.5) + 500\right] + 1.6 \cdot 40 \cdot 9.5 = 3.08 k/ft$

2

$\underline{J_c}$

$b_1 = 16" + 4" = 20"$

$b_2 = 58" \times 0.8 + 4" = 50"$

$\bar{x} = \dfrac{20 \cdot \frac{20}{2}}{20+50} = 2.86"$

$J_c = 32,000$

$V_u = 49 + \dfrac{(38^{ft-k})(12)(2.86")}{32,000} \times 58 \times 9 = 49 + 21 = 70k$

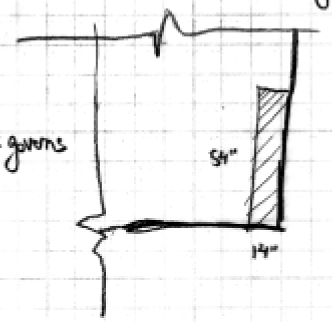
$V_u < \phi V_n = 0.75(120^k) = 90^k \therefore \underline{ok}$

Punching Shear Check: Column #13 1

$V_c = 4\sqrt{f'_c}\, b_o d = 310 b_o d$

$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = 235 b_o d \Longleftarrow$ governs

$\beta_c = \frac{54}{14} = 3.86$



$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d = 337 b_o d$

$d = 8"$

$b_o = 54 + 14 = 68"$

$V_c = \frac{235(68)(8)}{1000} = 128^k$

$A_T = 206\,sf$

$A_{shear} = 206\,sf - \frac{(54+4")(14+4")}{144} = 199\,sf$

trib. length for curtainwall load: $16.25' - \frac{(14+4)}{12} = 14.75'$ (glass facade)

& $12.33' - \frac{(54+4)}{12} = 7.5'$ (masonry wall)

$V_u = (199sf)(52 + 112.5) \times 1.2 + (199sf)(40psf) \times 1.6 + (14.75')(500 lb/ft) \times 1.2 + (7.5')(750 lb/ft) \times 1.2$

$V_u = 39.3^k + 12.7^k + 8.6^k + 6.8^k$

$V_u = 67.4^k$

$\phi V_n = 0.75(128) = 96^k > V_u = 67.4^k \therefore$ ok

$\boxed{\text{ACI 13.5.3.2}}$

Still need to account for unbalanced moment contributing to punching shear...

$\gamma_f = \frac{1}{1 + \left(\frac{2}{3}\right)\sqrt{b_1/b_2}} = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{14+4"}{54+4}}} = 73\%$ additional shear resisted by flexural steel in slab

$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(3.08)(30.5)^2}{8} = 358 \rightarrow 0.3M_o = 107$ k-ft transferred to column-slab connection

$\boxed{\text{ACI 13.6.3.6}}$

$q_u = 1.2\left[(52+112.5)(9.5)+500\right] + 1.6 \cdot 40 \cdot 9.5 = 3.08\ k/ft$

$(1-0.73)(107) = 29^k$ to be transferred by shear

2

$$J_c = 2\left(\frac{b_1 d^3}{12}\right) + 2\left(\frac{d b_1^3}{12}\right) + 2(b_1 d)\left(\frac{b_1}{2} - \bar{x}\right)^2 + (b_2 d)\bar{x}^2$$

$b_1 = 14 + 4 = 18"$ $\quad d = 8"$

$b_2 = 54 + 4 = 58"$ $\quad \bar{x} = \dfrac{18 \cdot \frac{18}{2}}{18 + 58} = 2.13"$

$J_c = 25000$

$$V_u = 67.4 + \frac{(29^{ik})(12)(2.13")}{25000\ in^4} \times 54 \times 9 = 67.4 + 14.4$$

$V_u = 82^{k} < \phi V_n = 96^k \therefore ok$

Other Direction: (N-S)

$$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{58}{18}}} = 0.46$$

$$M_o = \frac{(5.0)(18)^2}{8} = 203^{ik} \rightarrow 0.3M_o = 61^{ik}$$

$$q_u = 1.2\left[(52 + 10.5)(18.67) + 750\right] + 1.6 \cdot 40 \cdot 18.67 = 5.0 k/ft$$

$(1 - 0.46)(61) = 33^{ik}$ to be transferred by shear

J_c:

$b_1 = 58"$ $\quad d = 8"$

$b_2 = 18"$ $\quad \bar{x} = \dfrac{58 \cdot \frac{58}{2}}{58 + 18} = 22"$

$J_c = 380{,}000$

$V_u = 67.4 + 11.1 = 79^{k} < \phi V_n = 96^k \therefore ok$

Deflection Calculations 1

$$\Delta_{total} = \lambda\left(\Delta_{i,dead} + \Delta_{i,\substack{sustained\\live}}\right) + \Delta_{i,dead} + \Delta_{i,\substack{sustained\\load}}$$

* unless otherwise calculated, λ is conservatively assumed to be 2

$$\Delta_{sustained} = \Delta_{i,dead} + \Delta_{i,\substack{sustained\\live}}$$

* Sustained live load is assumed to be, 50% of total live load
* Immediate, uncracked, elastic deflections will be taken from RAM Concept

SPAN 1

$\Delta_{i,dead} = 0.22''$

$\Delta_{i,live} = 0.06''$

$\Delta_{total} = 3 \times 0.22 + 2(.5 \times 0.06) + 0.06 = 0.78''$

$\frac{L}{480} = \frac{34 \cdot 12}{480} = 0.85''$

Span 2

$\Delta_{i,dead} = 0.03''$

$\Delta_{i,live} = 0$

$\Delta_{total} = 0.09''$

$\frac{L}{480} = 0.48''$

Span 3

$\Delta_{i,dead} = 0.19''$

$\Delta_{i,live} = 0.09''$

$\Delta_{total} = 0.75''$

$\frac{L}{480} = 0.74''$

2

Span 4

$\Delta_{i,dead} = 0.03''$

$\Delta_{i,live} = 0$

$\Delta_{total} = 0.09''$

$\frac{L}{480} = 0.42''$

Span 5

$\Delta_{i,dead} = 0.16''$

$\Delta_{i,live} = 0.05''$

$\Delta_{total} = 0.58''$

$\frac{L}{480} = 0.73''$

Span 6

$\Delta_{i,dead} = 0.03''$

$\Delta_{i,live} = 0''$

$\Delta_{total} = 0.09''$

$\frac{L}{480} = 0.41''$

Span 7

$\Delta_{i,dead} = 0.20''$

$\Delta_{i,live} = 0.06''$

$\Delta_{total} = 0.72''$

$\frac{L}{480} = 0.79''$

APPENDIX C

19TH FLOOR EXISTING TRANSFER SYSTEM CALC'S

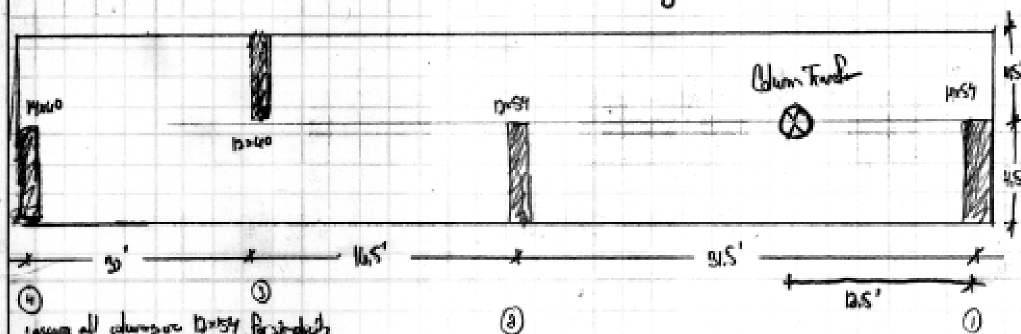
Column 30 Load Takedown (kips)																
Statistics								Loads								Load Combination
Column Below Level	Floor Height	A_t Resid.	A_t Balc.	Column Area	Ext. Wall Height (ft)	Ext. Wall Perimeter(ft)	Balc. Wall Perimter (ft)	Self-Weight (k)	Dead (psf)	Ext. Wall Load (psf)	Ext. Balc. Load (lb/ft)	Cooling Tower Dead* (k)	Live Resid. (psf)	Live Balc. (psf)	Snow (psf)	$1.2D+1.6L+0.5(L_r$ or S)
Roof	15.08	73		2.5	11.5	16.2		6	180	90	586	17.8	40	60	20	69
21	12	73		2.5	13.5	16.2		5	164	90	586	-	40	60	0	117
20	12	73		2.5	12	16.2		5	164	90	586	-	40	60	0	163
*Includes Cooling Tower Enclosure & Structure								Scale Load Based on Max. Capacity of Existing Desig**n: Pu=0.75*(Calculated Load) =								122

Column 31 Load Takedown (kips)																
Statistics								Loads								Load Combination
Column Below Level	Floor Height	A_t Resid.	A_t Balc.	Column Area	Ext. Wall Height (ft)	Ext. Wall Perimeter(ft)	Balc. Wall Perimter (ft)	Self Weight (k)	Dead (psf)	Ext. Wall Load (psf)	Ext. Balc. Load (lb/ft)	Cooling Tower Dead* (k)	Live Resid. (psf)	Live Balc. (psf)	Snow (psf)	$1.2D+1.6L+0.5(L_r$ or S)
Roof	15.08	128	18	2.5	11.5	11	10.5	6	180	90	586	13.8	40	60	20	87
21	12	128	18	2.5	13.5	11	10.5	5	164	90	586	-	40	60	0	155
20	12	128	18	2.5	12	11	10.5	5	164	90	586	-	40	60	0	220
*Includes Cooling Tower Enclosure & Structure								Scale Load Based on Max. Capacity of Existing Design**: Pu=0.75*(Calculated Load) =								165

Column 32 Load Takedown (kips)																
Statistics								Loads								Load Combination
Column Below Level	Floor Height	A_t Resid.	A_t Balc.	Column Area	Ext. Wall Height (ft)	Ext. Wall Perimeter(ft)	Balc. Wall Perimter (ft)	Self-Weight (k)	Dead (psf)	Ext. Wall Load (psf)	Ext. Balc. Load (lb/ft)	Cooling Tower Dead* (k)	Live Resid. (psf)	Live Balc. (psf)	Snow (psf)	$1.2D+1.6L+0.5(L_r$ or S)
Roof	15.08	150	131	5.83	11.5	22	15.5	13	224	41.7	586	13.8	40	60	20	156
21	12	150	131	5.83	13.5	22	15.5	10	164	41.7	586	-	40	60	0	272
20	12	150	131	5.83	12	22	15.5	10	164	41.7	586	-	40	60	0	386
*Includes Cooling Tower Enclosure & Structure								Scale Load Based on Max. Capacity of Existing Design**: Pu=0.75*(Calculated Load) =								290

**Existing design analyzed to find maximum force from Column 32 able to be resisted. After finding this strength to be signficantly less than the takedown column load calculated above, all calculated loads were factored to attain the appropriate column load for redesign

Hand Check of Existing 19th Floor Transfer Slab - Equivalent Frame



• assume all columns are 12x54, for simplicity

$w_L = 40 psf$

$w_{topping} = 52 psf$

• assume 18.5" thickness $\rightarrow w_D = 231 psf$

C.W. = 500 lb/ft

Column Transfer = 420^k (factored already ; load+live+tors)

$w_u = 1.2(52 \cdot 9 + 231 \cdot 9 + 500) + 1.6(40 \cdot 9') = 4.2 k/ft$

$P_u = 420^k$

Column Stiffness: $K_{col} = \frac{4E_c I_c}{L - 2t} = \frac{4E_c(7776)}{12 \cdot 11 - 2 \cdot 18.5} = 327 E_c$

$I_c = \frac{1}{12} \cdot 54 \cdot 12^3 = 7776 in^4$

Torsional Stiffness: $K_t = \frac{9EC}{l_2(1 - \frac{c_2}{l_2})^3} = \frac{9E(728)}{12 \cdot 9' \cdot (1 - \frac{54}{9 \cdot 12})} = 121E$

$C = (1 - 0.63 \cdot \frac{18.5}{12})(\frac{18.5^3 \cdot 12}{3}) = 728$

Equivalent Column Stiffness:

$\frac{1}{2 \cdot 327} + \frac{1}{121} = 0.009 \Rightarrow K = 102 k/in$

Slab Stiffness: $I_s = \frac{1}{12} \cdot 9 \cdot 12 \cdot 18.5^3 = 57,000$

$K_1 = \frac{4E_c \cdot 57000}{31.5 \cdot 12 - \frac{12}{2}} = 613$

$K_2 = \frac{4E_c \cdot 57000}{16.5 \cdot 12 - \frac{12}{2}} = 1188$ $K_3 = \frac{4E_c \cdot 57000}{30 \cdot 12 - \frac{12}{2}} = 644$

$DF_{12} = \frac{613}{613 + 102} = 0.86$

$DF_{21} = \frac{613}{613 + 102 + 1188} = 0.32$

$DF_{23} = \frac{1188}{1188 + 613 + 102} = 0.62$

$DF_{32} = \frac{1188}{1188 + 644 + 102} = 0.61$

$DF_{34} = \frac{644}{644 + 1188 + 102} = 0.33$

$DF_{43} = \frac{644}{644 + 102} = 0.86$

2

0.86	0.33	0.61	0.62	0.32	0.86
-315	315	-95	95	-347	347
271	136			-149	-298
-58	-117	-217	249	128	64
		134	-108		
50	25			-38	-55
	-49	-91	84	44	
-24		42	-46		22
				-9	-18
20	10				
	-19	-32	34	18	
-8		17	-16		9
				-4	-8
7	4				
	-7	-13	12	6	

SEE SPREADSHEET FOR FINAL SOLUTION

$$FEM = \frac{w_u \cdot 9' \cdot l_n^2}{12} = \frac{(4.2\,k/ft)(9')\,l_n^2}{12} = 0.35\,l_n^2$$

Span 1 $\rightarrow 0.35(31.5)^2 = 347$ k#

Span 2 $\rightarrow 0.35(16.5)^2 = 95$ k#

Span 3 $\rightarrow 0.35(30)^2 = 315$ k#

Span 3 also has column transfer (point load):

$$FEM:\ \text{Left side} = M_1 = \frac{Pab^2}{l^2} = \frac{P(19')(12.5')^2}{31.5^2} = 2.99P$$

$$\text{Right side} = M_2 = \frac{Pa^2b}{l^2} = \frac{P(19')^2(12.5)}{31.5^2} = 4.55P$$

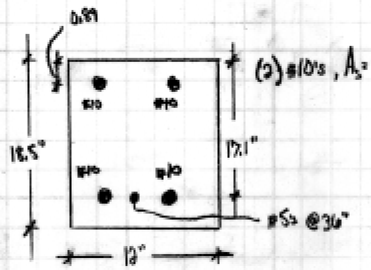
$$M_{a,\,at\,point} = \frac{2Pa^2b^2}{l^3} = \frac{2P(19)^2(12.5)^2}{31.5^3} = 3.6P$$

$$M_{midspan} = \frac{Pb^2}{l^3}(3a+b)x - \frac{Pab^2}{l^2} = \frac{P(12.5)^2}{31.5^3}(3 \cdot 19 + 12.5)(15.75) - \frac{P(19)(12.5)^2}{31.5^2} = 2.48P$$

$$M_{mid\text{-}span\,(simple\,beam)} = \frac{Pbx}{l} = \frac{P \cdot 12.5(15.75)}{31.5} = 6.25P$$

3

Design Strength of 18.5" thick perimeter strip - 19th Floor w/ column transfer at end span



(2) #10's, $A_s = 2(1.27 in^2) = 2.54"$ (only bottom steel)

$A_{s,total} = 2.54" + 0.31 \times \frac{1}{3} = 2.64"$

$\phi M_n = 0.9\left(2.64 \cdot 60\left(17.1 - \frac{2.59}{2}\right)\right)$

$\phi M_n = 2253^{"k} = 188^{'k}$

$a = \frac{A_s f_y}{0.85 \cdot b \cdot f'_c} = \frac{(2.64 in^2)(60 ksi)}{0.85(12 in)(6 ksi)} = 2.59"$

$c = \frac{2.59}{0.75} = 3.45" \rightarrow \varepsilon_s = \frac{0.003}{3.45}(17.1 - 3.45) = 0.01 > \varepsilon_y \therefore \phi = 0.9$

Include top steel in strength:

$a = \frac{A_s f_y - A'_s f_y}{0.85 f'_c \cdot b} = \frac{(2.64 \cdot 60000) - (2.54 \cdot 60000)}{0.85 \cdot 6000 \cdot 12} = 0.10"$

$c = \frac{0.10}{0.75} = 0.13$

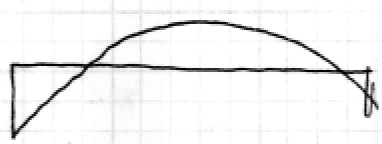
$M_n = A'_s f_y (d - d') + 0.85 f'_c ba\left(d - \frac{a}{2}\right)$

$= (2.54)(60)(17.1 - 0.89) + 0.85(6)(12)(0.10)\left(17.1 - \frac{0.10}{2}\right)$

$= 2575^{"k} = 214^{'k} \rightarrow 0.9(214) = 193^{'k}$

Ignore effects of top steel $\rightarrow \phi M_n = 188^{'k}$ / ft width of slab

Simplifying 9' strip down to equivalent frame and using $P_{col} = 426^k$ from estimated takedown, calculated moments in 31.5' span are as follows:



$M_{left} = -201$

$M_{midspan} = 232$

$M_{right} = 43$

This is greater than calculated capacity of $\underline{188^{'k} = \phi M_n}$ $\rightarrow \therefore$ column load must be too large

Column load of around 325^k gives $M_u = 185^{'k} < \phi M_n \therefore ok \therefore$ Use $P_{col} = 325^k = 75\%$ of calc. load

Support	4		3			2			1
DF	0.86		0.33	0.61		0.62	0.32		0.86
FEM	-315		315	-95		95	-347		347
	271		135				-149		-298
			-117	-217		249	128		
	-59			124		-108			64
	50		25				-23		-55
			-49	-91		84	44		
	-25			42		-46			22
	21		11				-9		-19
			-17	-32		34	18		
	-9			17		-16			9
	7		4				-4		-8
			-7	-13		12	6		
	-3			6		-5			3
	3		1				-1		-3
			-3	-5		5	2		
	-1			2		-2			1
	1		1				-1		-1
			-1	-2		2	1		
	-58		298	-262		302	-340		62

M_u (k-ft/ft width of slab)	-6	33	33	-29	-15	34	-38	36	7

	Support
	Midspan

Moment Calculation for distributed loads

P_{col}= 426

Support	4		3			2			1
DF	0.86		0.33	0.61		0.62	0.32		0.86
FEM	0		0	0		0	-1274		1938
	0		0				-833		-1667
			0	0		1306	674		
	0			653		0			337
	0		0				-145		-290
			-216	-398		90	46		
	-108			45		-199			23
	93		46				-10		-20
			-30	-56		130	67		
	-15			65		-28			33
	13		6				-14		-29
			-24	-44		26	14		
	-12			13		-22			7
	10		5				-3		-6
			-6	-11		15	8		
	-3			8		-6			4
	3		1				-2		-3
			-3	-5		4	2		
	-19.27		-219.00	269.58		1317.65	-1469.79		328.00
M_u (k-ft/ft width of slab)	-2	64	-24	30	-42	146	-163	196	36

	Support
	Midspan

Moment Calculation using just point load from transferring column

Essentially, these calculations were used to accurately determine the loads carried by the columns to be transferred. For reasons unknown, the column load takedown produced a load that was much too high. The column loads from the column load takedowns were adjusted to equal the maximum moments able to be resisted by the existing transfer system design.

APPENDIX D

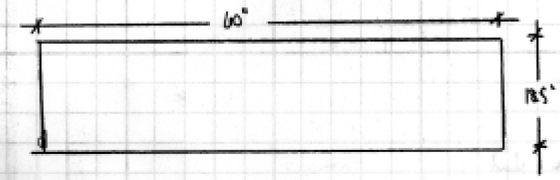
19TH FLOOR TRANSFER SYSTEM REDESIGN CALC'S

Beam 1 Design

Beam Design 1

Determine max. strength I can get out of beam that is 18.5" deep

Max. P.T. precompression = 500 psi (R. Apple)

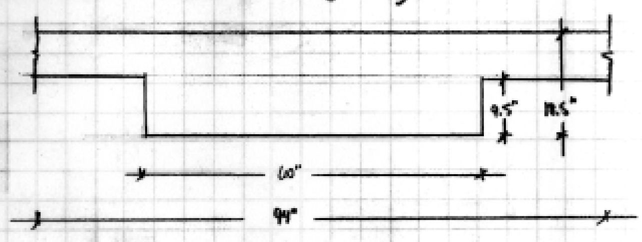


$b_{eff} \leq b_w + 16h_f = 60 + 16\cdot 9 = 204"$

$\leq b_w + 2(\tfrac{1}{2} \text{ clear dist.}) = 60 + 2\cdot(\tfrac{142}{2}) = 202"$

$\leq \tfrac{1}{4} \text{ span length} = \tfrac{1}{4}(31.5\cdot 12) = 94"$ ⇐ governs

ACI 8.12



$A = 60\cdot 18.5 + 34\cdot 9 = 1110\ in^2 + 306\ in^2 = 1416\ in^2$

$(500\,psi)\times(1416\,in^2) = 708{,}000\,lb = 708^k$ of allowable prestress

Assume $f_{ps} = 200\,ksi$ & $d = 18.5 - 0.75 - \tfrac{1}{2}(2") = 16.75"$ → assumed depth of anchorage system (multi-tendon anchorage)

Assume each tendon is $\tfrac{1}{2}"\,ø$ 7-wire strand → $P_{eff} = 26.7^k$

$A_{ps} = 0.153\,in^2$

$\dfrac{708^k}{26.7^k/tendon} = 27$ tendons → $27\cdot 26.7 = 721^k$

Mild-steel reinforcement: ρ_{max} for $ø = 0.9$ & $f'_c = 6000 = 0.0239$

$A_{s,max} = 0.0239\times(60\,in \times 16.75") = 24\,in^2$

$A_{ps} = 27\times 0.153 = 4.1\,in^2$

$A_{s,mild} = 19.9\,in^2$ @ $d = 18.5 - 0.75 - \tfrac{1}{2}(1") = 17.25"$

2

$a = \frac{A_s f_y}{0.85 \cdot b f'_c} = \frac{(4.1 \cdot 500) + (19.9 \cdot 60)}{0.85 \cdot 94" \cdot b} = 4.20" < 9"$ ∴ no t-beam behavior confirmed

↳ assumed no t-beam behavior

$M_n = \underbrace{4.1 \cdot 200 \left(16.75 - \frac{4.20}{2}\right)}_{\text{prestressed}} + \underbrace{19.9 \cdot 60 \left(17.25 - \frac{4.20}{2}\right)}_{\text{mild}} = 12013^{\text{in-k}} + 18089^{\text{in-k}} = 30{,}102^{\text{in-k}} = 2510^{\text{ft-k}}$

$\phi M_n = 0.9(2510) = 2258^{\text{ft-k}}$

M_u calculation

For 31.5' span with Column 33 transferring:

Assume that the beams this beam frames into provide minimal torsional restraint so that exterior support condition approaches that defined in ACI 318-08 8.3.3 for when support is spandrel beam of $M_u = \frac{w_u l_n^2}{24}$

Thus, midspan $= \frac{w_u l_n^2}{8} - \frac{w_u l_n^2}{24}$

$= \frac{w_u l_n^2}{12}$

This approximation is 50% of the way between fixed & pinned end conditions for end moments

" " " 30% " " " " " " " " " mid span moments

Apply these ratios to moments generated by point load

$w_u = 1.2(52 \cdot 10' + 112.5 \cdot 11' + 235 \cdot 5') + 1.6(40 \cdot 16') + 1.2(500) = 3893 + 1024 + 600 = 5.5\ k/ft$

(dead) (live) (CW)

$\frac{9"}{12} \times 150 = 112.5$ psf

$\frac{18.5}{12} \times 150 = 231$ psf

$\frac{w_u l_n^2}{12} = \frac{(5.5k/ft)(30.25ft)^2}{12} = 419\ k\text{-}ft \rightarrow \frac{419}{2258} = 18.6\%$ of capacity

$\frac{w_u l_n^2}{24} = \frac{(5.5k/ft)(30.25)^2}{24} = 210^{\text{ft-k}}$

$M_{u,12'}$ = (interpolate) $= \frac{(419 - -210)}{15.25} \times (12') + -210 = 285^{\text{ft-k}} \rightarrow \times 1.10$ (for parabolic shape adjustment) $= 313^{\text{ft-k}}$

3

Now determine moment induced by column transfer load:

If pinned-pinned, moment at midspan = $\frac{Pbx}{l} = \frac{290\cdot12\cdot15.25}{30.5} = 1740^{1k}$

" " " " " load location = $\frac{290\cdot18.5\cdot12}{30.5} = 2111^{1k}$

If fixed-fixed, moment at midspan = $\frac{Pb^2}{l^3}(3a+b)x - \frac{Pab^2}{l^2} = \frac{290\cdot12^2}{30.5^3}(3\cdot18.5+12)\times15.25 - \frac{290\cdot18.5\cdot12^2}{30.5^2} = 685^{1k}$

" " " " " loading: $\frac{2Pa^2b^2}{l^3} = \frac{2\cdot290\cdot18.5^2\cdot12^2}{30.5^3} = 1067^{1k}$

If pinned-pinned, moment at left end = 0

" " " " right " = 0

If fixed-fixed, moment at left end = $\frac{Pab^2}{l^2} = \frac{290\cdot18.5\cdot12^2}{30.5^2} = 830^{1k}$

" " " " " right " = $\frac{Pa^2b}{l^2} = \frac{290\cdot18.5^2\cdot12}{30.5^2} = 1280^{1k}$

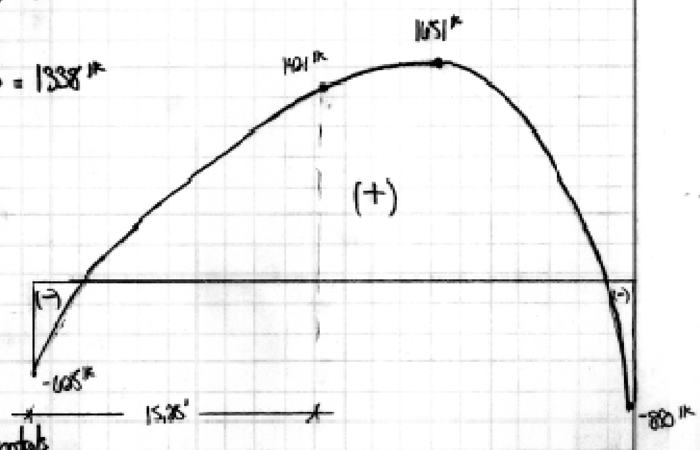
Using approximate support conditions used in distributed load calculations:

$M^-_{left} = (830-0)\times50\% = 415^{1k}$

$M^-_{right} = (1280-0)\times50\% = 640^{1k}$

$M^+_{midspan} = (1740-685)\times30\% + 685 = 1002^{1k}$

$M^+_{loading} = (2111-1067)\times30\% + 1067 = 1338^{1k}$



Total Moments

$M^-_{left} = -210 + -415 = -625^{1k}$

$M^-_{right} = -210 + -640 = -850^{1k}$

$M^+_{midspan} = 419 + 1002 = 1421^{1k}$

$M^+_{loading} = 313 + 1338 = 1651^{1k}$ ⟸ controls

$\phi M_{n,max} = 2258^{1k} > M_u = 1651^{1k}$ ∴ Design can be accommodated by wide-shallow transfer beam

4

As it turns out, PT does very little for this beam → Design beam as R/C

Using same beam dimensions:

$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$ $\quad d = 16.5 - 1.5 - 0.5 - \frac{1.41}{2} = 15.8''$

First check that T-beam behavior occurs:

$M_{u_{max}} = \phi \cdot 0.85 \cdot f'_c \cdot b \cdot h_f \left(d - h_f/2\right)$

$= 0.9 \cdot 0.85 \cdot 6 \cdot 94 \cdot 9 \left(15.8 - 9/2\right)$

$= 43879^{ik} = 3657^{k} > M_u$ → No T-Beam behavior and $b = 94''$

$12 \cdot 1651 = \rho \cdot 60 \cdot 94 \cdot 15.8^2 \left(1 - 0.59 \rho \cdot \frac{60}{6}\right)$

$19812 = 1407970 \rho - 8307021 \rho^2$

$\rho = 0.0155 < \rho_{max\,\phi} = 0.0239 \therefore ok$

$A_s = 0.0239 (94 \times 15.8) = 35 in^2$ → 23 #11s, too many for single layer

Double layer $d_{eff} = \frac{15.8'' + 13''}{2} = 14.4''$

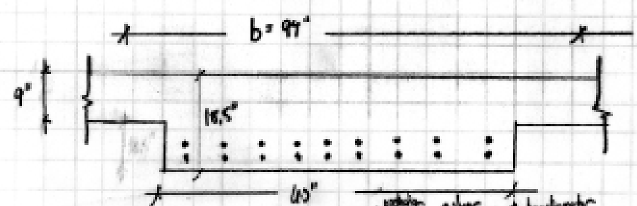
$d_{2nd\ layer} = 15.8'' - 1.41'' - 1.41'' = 13.0''$
↓
clear dist

$19812 = \rho \cdot 60 \cdot 94 \cdot 14.4^2 \left(1 - 0.59 \rho \cdot \frac{60}{6}\right)$

$\rho = 0.0191 < 0.0239 \therefore ok$

$A_s = 0.0191 \times (94 \times 14.4) = 25.9 in^2$ → 17 #11 bars → use 9 #11s on each layer, $A_s = 28.1 in^2$

$\rho = \frac{28.1}{94 \times 14.4} = 0.0208 < 0.0239 \therefore OK$



protection, p.shear, bar diameter

Clear spacing btw bars: $\frac{(60'' - 2 \cdot 1.5'' - 2 \cdot 0.5'' - 1.41'')}{8\ spaces} = 6.8''$

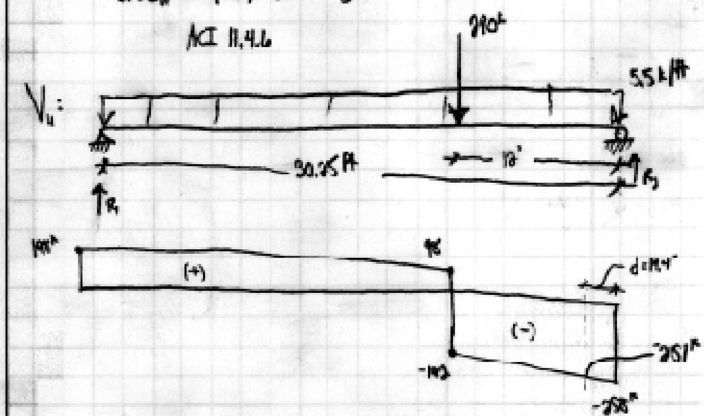
5

Shear Reinforcement

$h = 13.5'' < 24''$
$< 2.5 h_f = 2.5(9'') = 22.5''$
$< 0.5 b_w = 0.5(60'') = 30''$

ACI 11.4.6

$A_{v,min}$ provided only when $V_u > 2\sqrt{f'_c} \cdot b_w d = 2\sqrt{6000} \times 60'' \cdot 14.4'' = 134^k$



$R_1 = \frac{5.5 \cdot 30.25}{2} + 290\left(\frac{12}{30.25}\right) = 198^k$

$R_2 = \frac{5.5 \cdot 30.25}{2} + 290\left(\frac{30.25-12}{30.25}\right) = 258^k$

for $V_u = 251^k$: $V_s = \frac{V_u}{\phi} - V_c = \frac{251}{0.75} - 134 = 201^k < 8\sqrt{f'_c}\, b_w d = 4(134) = 536^k$ ∴ ok

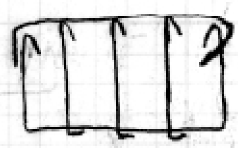
max spacing: $V_s < 4\sqrt{f'_c}\, b_w d = 268^k$, so $s_{max} = \min \begin{cases} d/2 = \frac{14.4}{2} = 7.2'' \leftarrow \text{use } 6'' \text{ o.c.} \\ 24'' \end{cases}$

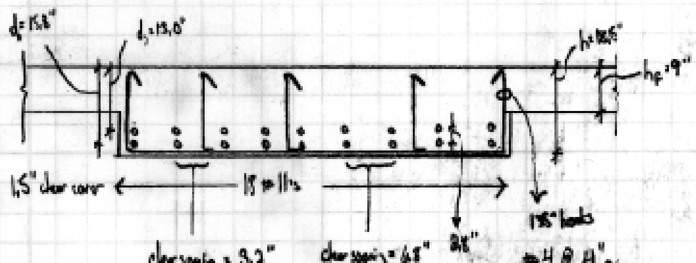
min. shear reinf. $= A_{v,min} = \max \begin{cases} 0.75\sqrt{f'_c}\, b_w s / f_{yt} = 0.75\sqrt{\frac{6000}{1000}} \cdot 60 \cdot 6'' / 60 = 0.35 \text{ in}^2 \leftarrow \\ 50 b_w s / f_{yt} = 50(60)(6)/60000 = 0.3 \end{cases}$

min. reinforcement is #4 stirrups @ 6" o.c. ($A_v = 2\text{legs} \times 0.20 = 0.40 \text{ in}^2$)

$s = \frac{A_v f_{yt} d}{V_s} = \frac{(0.40)(60)(14.4)}{201} = 1.72''$ → not practical

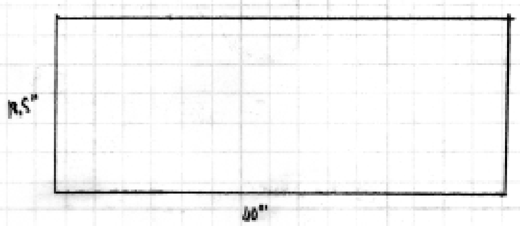
Try 5 legs → $s = \frac{(5 \cdot 0.20)(60)(14.4)}{201} = 4.29''$ → use #4 @ 4" o.c. 5 legs





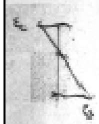
6

Check that negative moment section works :



$\phi M_n = A_s f_y (d - \frac{a}{2})$ $\quad a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60)(31.2)}{0.85 \cdot 6 \cdot 60} = 6.12" \longrightarrow c = \frac{6.12}{0.75} = 8.16"$

$\phi M_n = ^{0.9}(31.2)(60)(14.4 - \frac{6.12}{2}) = 19105^{\text{ink}} = 1592^{\text{1k}} > M_u = 850^k \therefore \text{OK}$



$\varepsilon_s \cdot (d-c) = \frac{\varepsilon}{c}$

$\varepsilon_s = \frac{0.003(d-c)}{c} = \frac{0.003(18.5 - 8.16)}{8.16} = 0.00381 > 0.002 \therefore$ not compression controlled

$< 0.005 \therefore \phi \neq 0.9$

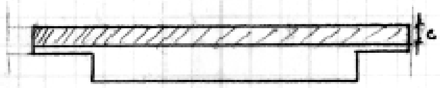
$\phi = 0.65 + (\varepsilon_t - 0.002)(\frac{250}{3}) = 0.65 + (0.00381 - 0.002)(\frac{250}{3}) = 0.72$

$\phi M_n = 0.72(31.2)(60)(14.4 - \frac{6.12}{2}) = 1274^{\text{1k}} > M_u = 850^k \therefore \text{OK}$

$\therefore$ Will work at negative moment region (w/o t-beam)

Check deflection

Assume section completely cracked $\therefore I = I_{cr}$



$n = 10,\ nA_s = 10(31.2) = 312 in^2$

calc $k = -\rho n + \sqrt{(\rho n)^2 + 2\rho n} = -(0.0179)(10) + \sqrt{(10 \cdot 0.0179)^2 + 2(10)(0.0179)} = 0.446$

$\rho = \frac{31.2}{44 \cdot 18.5} = 0.0179$

$c = kd = 0.446(14.4) = 6.24"$

$I_{cr} = \frac{(44 in)(6.24 in)^3}{12} + (44 \cdot 6.24)(6.24 - \frac{6.24}{2})^2 + (312 in^2)(14.4 - 6.24)^2$

$I_{cr} = 1903 + 5710 + 20,774 = 28,387 in^4$

Δ = (conservatively assume pin-pinned connection)

$\Delta = \frac{5}{384}$

$w = \underbrace{52 psf \cdot 16}_{\text{SDL}} + \underbrace{112.5 \cdot 11 + 232 \cdot 5}_{\text{self wt}} + \underbrace{40 \cdot 16}_{\text{LL}} + \underbrace{500}_{\text{curtain wall}} = 4.41 k/ft$ — $P = 883$

$w_{DL} = 112.5 \cdot 11 + 232 \cdot 5 + 52 \cdot 16 + 500 = 3.73 k/ft$ — $P = 190$

$w_{LL} = 40 \cdot 16 = 0.640 k/ft$ — $P = 31$

$w_{sustained} = 3.73 + 0.5 \cdot 0.64 = 4.05 k/ft$ — $P = 207$

$P = \phi \cdot \xi = (6.8)^2 \cdot 30$

$47^2 \cdot 4 / 207 - 1493 + \frac{4^2}{4} / 30$

17

Δ Due to distributed load

$\Delta_{DL,\,immediate} = \frac{3/2}{384} \cdot \frac{3.73 \cdot 30.5^4}{(4415)(28,387)} \cdot 1728 = 0.17''$

$\Delta_{LL+DL} = \Delta_{total\ immed.} = 0.17\left(\frac{3.73+0.64}{3.73}\right) = 0.20''$

$\Delta_{LL} = 0.20 - 0.17 = 0.03''$

$\Delta_{sust,\,imm} = 0.17\left(\frac{4.05}{3.73}\right) = 0.18''$

$\Delta_{long} = 2.0(0.18) = 0.36''$

$\Delta_{total} = \Delta_{long} + \Delta_{LL+DL} = 0.36'' + 0.18'' = 0.54''$

$\frac{3/2}{384}$ is 50% of the way btw fixed-fixed & pinned-pinned, which is thought to be an appropriate approximation for this problem

Δ due to point load

$\Delta_{column\ load\ midspan}$ (Using $\Delta_x = \frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx)$)

$\Delta = \frac{P(12 \cdot 12)^2(15.25 \cdot 12)^2}{6(4415)(28387)(30.5 \cdot 12)^3}(3 \cdot 15.25 \cdot 12 - 3 \cdot 15.25 \cdot 15.25 \cdot 144 - 12 \cdot 12 \cdot 15.25 \cdot 12)$

$= P \times 0.00183$ (fixed-fixed) $\Delta = P \times 0.0087$ (pinned-pinned)

Approximate support conditions as $\Delta = 0.0035P$ (50% pinned/50% fixed)

$\Delta_{DL,\,immediate} = 0.0035(192) = 0.672$

$\Delta_{LL+DL} = 0.0035(223) = 0.78''$

$\Delta_{LL} = 0.11''$

$\Delta_{sus} = 0.0035(208) = 0.73''$

$\Delta_{long} = 2.0(0.73) = 1.46''$

$\Delta_{total} = 0.73 + 1.46 = 2.2''$

Limits:

(from design engineers)

$\Delta_{tot} \le \frac{L}{240}$

$\Delta_{immediate} \le \frac{L}{360}$ (elastic)

(from ACI 318-08, Table 9.5(b))

Δ occurring after attachment of nonstructural elements $\le \frac{L}{480}$

$\Delta_{tot} = 2.74'' > \frac{L}{240} = \frac{31.5 \cdot 12}{240} = 1.58$ ∴ No Good → Use topsteel to limit long term deflections, use 18 #11's ($A_s = 28.1$)

$\lambda = \frac{2.0}{1+50\left(\frac{28.1}{165 \cdot 44}\right)} = 1.11$

$\Delta_{long} = 1.11(0.36 + 0.73) = 1.21''$

$\Delta_{tot} = 1.21 + 0.36 + 0.73 = 2.3'' > 1.58''$ ∴ NG

Total deflection limitations cannot be met w/ current beam dimensions; Solution will be to increase depth of beam, thus increasing I_{cr}. However, this is not possible w/ architectural req'ts and will not be investigated

$\Delta_{immed} = 0.872'' < \frac{L}{360} = \frac{31.5 \cdot 12}{360} = 1.05''$ ∴ Ok

Because superstructure is in place for some time before nonstructural elements are attached, a portion of the creep has already occurred: ξ for 3 months $= 1.0$

$\lambda_{3\,months} = 1.11\left(\frac{1}{2}\right) = 0.55$; $\Delta = \Delta_{long} - 0.55\Delta_{sus} + \Delta_{LL} = 1.21 - 0.55(1.04) + 0.14 = 0.75'' < \frac{L}{480} = 0.79$ ∴ Ok

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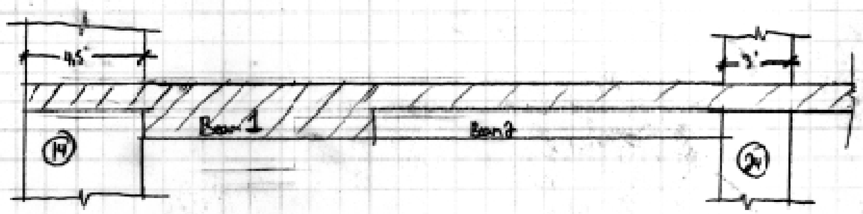
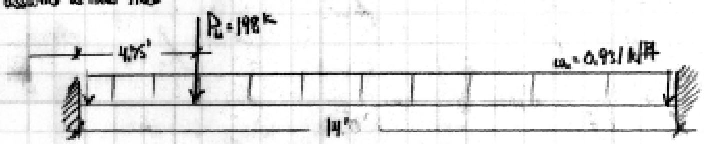
Beam 2 Design

Beam 2 Design 1



Because this beam frames directly into columns and $l_n = 10'4''$ and long columns are much stiffer than beam, (ACI 318-05 5.3.3)

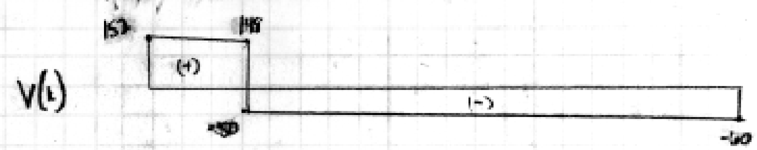
Beam can be assumed as fixed-fixed



$w_u = 1.2(53 \cdot 2.5' + 112.5 \cdot 2.5' + 231 \cdot 1') + 1.6(40 \cdot 2.5') = 771 + 160 = 931$ lb/ft $= 0.931$ k/ft

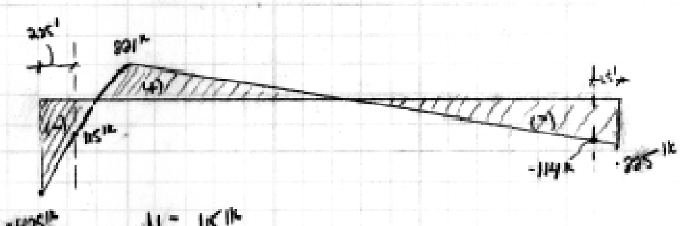
$R_1 = \frac{0.931 \cdot 14'}{2} + \frac{198 \cdot 9.25^2}{14^3}(3 \cdot 4.75 + 9.25) = 6.5 + 145 = 152^k$

$R_2 = 6.52 + \frac{198 \cdot 4.75^2}{14^3}(4.75 + 3 \cdot 9.25) = 6.5 + 53 = 60^k$



$M_1 = \frac{w_u l^2}{12} + \frac{P a b^2}{l^2} = \frac{0.931 \cdot 14^2}{12} + \frac{198 \cdot 4.75 \cdot 9.25^2}{14^2} = 15.2 + 410 = 425$ 'k

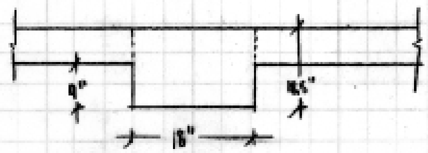
$M_2 = \frac{w_u l^2}{12} + \frac{P a^2 b}{l^2} = 15.2 + 210 = 225^k$



$M_u^- = 115$ 'k

$M_u^+ = 221$ 'k

7



$b_{eff} \le b_w + 16h_f = 12 + 16 \cdot 18.5 = 308"$

$\le b_w + 2(\tfrac{1}{2}\text{ clear}) = 12 + (2.5 \cdot 12)12 = 186"$

$\le \tfrac{1}{4}\text{ span length} = \tfrac{1}{4}(10.3 \cdot 12) = 30.9" \Leftarrow$

$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$ $d = 18.5 - 1.5 - 0.5 - \frac{1.41}{2} = 15.8"$

M_u small enough that no t-beam behavior occurs @ $b = 30"$ → For maximum moment (-), no t-beam assumed b/c it's a negative moment

$12 \times 221 = \rho \cdot 60 \cdot 12 \cdot 15.8^2 \left(1 - 0.59 \rho \cdot \frac{60}{6}\right)$ (w/o t-beam)

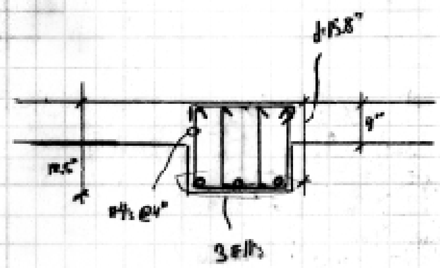
$\rho = 0.0163$

$A_s = 0.0142(13 \times 15.8) = 3.09 \text{ in}^2 \rightarrow (2)\#11, A_s = 3.12 \text{ in}^2$

$\rho_{max,cl} = 0.0239 > \rho \therefore \phi = 0.9$

$\rho_{min} = \frac{3\sqrt{6000}}{60000} = 0.0039 \ge \frac{200}{f_y} = 0.0033$

$\rho_{min} < \rho \therefore ok$



min. clear spacing = 1.44" (thru longitudinal bar & shear reinf.)

* added extra long. bar so that shear reinf. configuration worked better

$\rho = 0.0165 < \rho_{max}$

<u>Shear</u> (12" does not work, use 18" width)

$V_u = 152^k$ $\phi V_c = 2\sqrt{f'_c}\, b_w \cdot d = 2\sqrt{6000}\,(18)(15.8) = 44^k$

$V_s = \frac{152}{0.75} - 44 = 158 < 8\sqrt{f'_c}\, b_w d = 4(44) = 176 \therefore ok$

max spacing : $V > 4\sqrt{f'_c}\, b_w d = 88^k$, so $s_{max} = \min \begin{cases} d/4 = 4" \leftarrow \text{use } 4" \\ 12" \end{cases}$

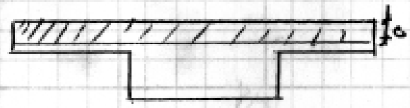
min. reinf. $= A_{v,min} = \max \begin{cases} 0.75\sqrt{f'_c}\, b_w s / f_{yt} = 0.75\frac{\sqrt{6000}}{1000} \cdot \frac{18 \cdot 4}{60} = 0.07 \text{ in}^2 \\ 50 b_w s / f_{yt} = 50 \cdot 18 \cdot 4 / 60^k = 0.06 \text{ in}^2 \end{cases}$

min. reinforcement is #4's @4", 2 legs, $A_v = 0.4 \text{ in}^2/4"$

$s = \frac{A_v f_{yt} \cdot d}{V_s} = \frac{(0.40)(60)(15.8)}{158} = 2.4" < 4" \therefore$ not practical → Using 4 legs, s = 4.8" → Use #4 @4" o.c. 4 legs

3

Check Deflection



$n = 10$, $nA_s = 10(2 \cdot 1.56) = 31.2\ in^2$ (calc'd w/ only the 2 necessary longitudinal bars)

$(30")(c)\left(\frac{c}{2}\right) = (31.2)(15.8 - c)$

$15c^2 + 31.2c - 493 = 0$

$c = 4.79 < 9"$ ∴ c w/in flange

$I_{cr} = \frac{1}{12}(30in \cdot 4.79^3) + (30 \cdot 4.79)\left(\frac{4.79}{2}\right)^2 + (46.8)(15.8 - 4.79)^2$

$= 275 + 824 + 5673 = 6772\ in^4$

Distributed load is less than 5% of total (distributed load + point load); therefore, only Δ due to point load will be investigated

$P_{DL} = 122^k$
$P_{LL} = 20^k$
$P_{sus} = 133$
} unfactored

*Note: l_n was used for deflections

$\Delta_{max} = \frac{2P_a \cdot b^2}{3EI(3a+b)^2} = \frac{2(122)(25')^3(7.8)^2}{3(4415)(6772)(3 \cdot 25 + 7.8)^2} \times 1728 = 0.02$ (Dead Load)

$a = 25'$
$b = 7.8'$

$\Delta_{DL,i} = 0.02"$

$\Delta_{LL,i} \approx 0"$

$\Delta_{sus} = 0.022"$

$\Delta_{long} = 2.0(0.022) = 0.044"$

$\Delta_{total} = 0.066" = 0.07"$

$\Delta_{tot} = 0.07" < \frac{L}{240} = \frac{10.3 \cdot 12}{240} = 0.5"$ ∴ ok

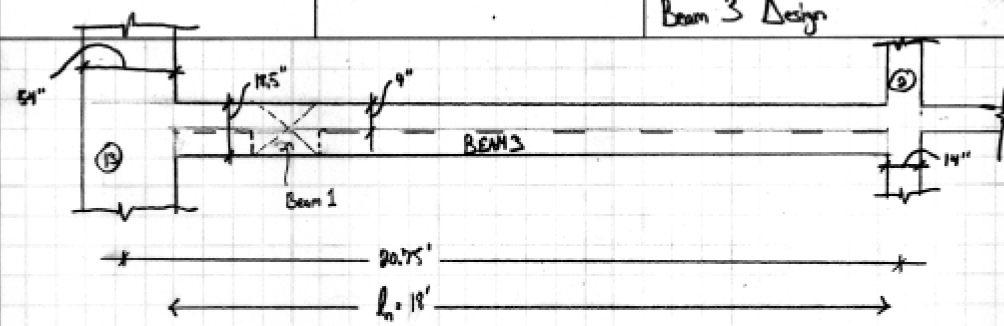
$\Delta_{imm} = 0.02" < \frac{L}{360} = \frac{10.3 \cdot 12}{360} = 0.34"$ ∴ ok

$\Delta_{long} + \Delta_{LL} = 0.044" + 0" = 0.044" < \frac{L}{480} = 0.26"$ ∴ ok

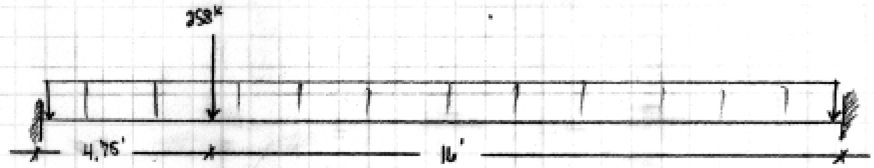
Deflection Limitations met ✓

Beam 3 Design

Beam 3 Design 1



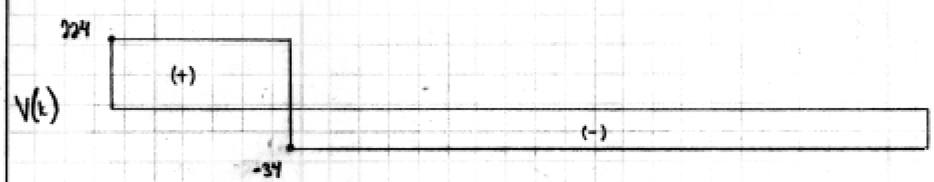
• assume columns are stiff enough to assume fixed connections



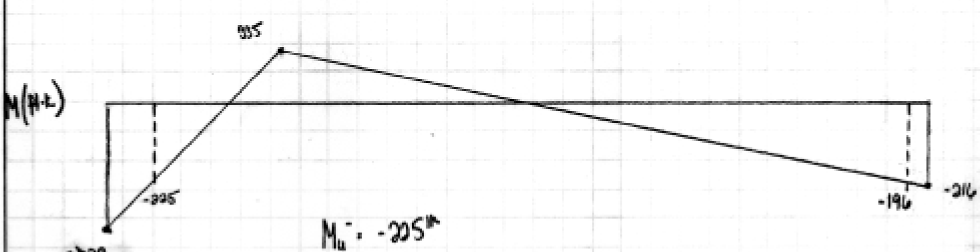
$w_{selfwt} = (150\,pcf)(18.5-9)(14)\cdot\frac{1}{144} = 139$ lb/ft $= 0.139$ k/ft $\Rightarrow$ insignificant when compared to 258^k point load which is $\frac{258}{258+0.139\cdot 20.75} = 99\%$ Pload

w_{selfwt} will be ignored

$R_1 = \frac{Pb^2}{l^3}(3a+b) = \frac{258\cdot 16^2}{20.75^3}(3\cdot 4.75+16) = 224$ $\quad R_2 = \frac{258\cdot 4.75^2}{20.75^3}(4.75+3\cdot 16) = 34$



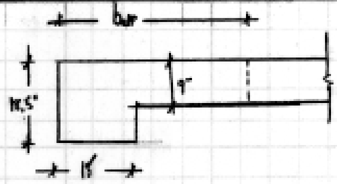
$M_1 = \frac{Pab^2}{l^3} = \frac{258\cdot 4.75\cdot 16^2}{20.75^3} = 729^{ft\text{-}k}$ $\quad M_2 = \frac{258\cdot 4.75^2\cdot 16}{20.75^3} = 216$



$M_u^- = -225^{ft\text{-}k}$

$M_u^+ = 335^{ft\text{-}k}$

2



$b_{eff} \leq 18 + 6 \cdot 9" = 72"$

$\leq 18" + \frac{1}{3}(11.5' \cdot 12) = 87"$

$\leq 18" + \frac{1}{12} \cdot 14' \cdot 12 = 32" \Leftarrow$ governs

Negative moment (no t-beam behavior):

$\frac{12 \cdot 225}{0.9} = \rho \cdot 60 \cdot 14 \cdot 15.8^2 (1 - 0.59 \rho \frac{60}{6}) \times 0.9$

$\rho \geq 0.012 < \rho_{max} = 0.0239 \therefore OK$

$A_s = 0.011(18 \cdot 15.8) = 3.41 in^2 \Rightarrow$ Try 4 #11s, $A_s = 6.24$

$\rho = 0.0219 < \rho_{max} \therefore OK$

Check Positive Moment:

$a = \frac{6.24 \cdot 60}{0.85 \cdot 32 \cdot 6} = 2.29" < 9" \therefore$ no t-beam behavior

$\phi M_n = 0.9(6.24)(60)(15.8 - \frac{2.29}{2}) = 411^k > M_u = 335^k \therefore OK$

* Note: Using 5 #9 ($A_s = 5 in^2$) gives $\phi M_n = 333 \approx M_u \therefore OK$
May work better for shear reinf. attachment

Shear

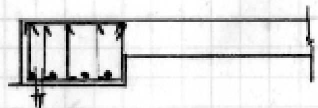
$V_u = 224^k$ $\phi V_c = 2\sqrt{6000}(18)(15.8) = 44^k$

$V_s = \frac{224}{0.75} - 44 = 255^k > 8\sqrt{f'_c} = 176^k \therefore NG \Rightarrow$ Increasing beam width could solve this, but this would induce more torsion into the beam, requiring additional transverse reinf., which will not fit

Max spacing: min $\begin{cases} d/4 = 4" \\ 12" \end{cases}$

min reinf. = $A_{v,min}$ = #4s @ 4", 2 legs, $A_v = 0.40 in^2/4"$ (for Beam 2)

$s = \frac{A_v \cdot f_{yt} \cdot d}{V_s} = \frac{(5 \cdot 0.2)(60)(15.8)}{255} = 3.72" \Rightarrow$ Use 3.5" o.c.



$s_{min} = 1.57" \therefore OK$

Only solution is to further increase beam width, which could create excessive torsion at support - needs to be further investigated

See next page for verification that beam cannot resist combination of shear & torsion

3

$T_u = 2'' \times 224^k = 448^{''k}$

Is section large enough for combined shear & torsion?

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u \cdot P_h}{1.7 A_{oh}^2}\right)^2} \leq \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$$

$V_u = 224$

$P_h = 14.5 \cdot 2 + 15 \cdot 2 = 59^{in}$

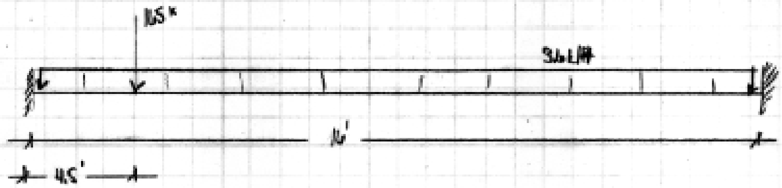
$A_{oh} = 14.5 \cdot 15 = 218\ in^2$

$$\sqrt{\left(\frac{224}{18 \cdot 15.8}\right)^2 + \left(\frac{448 \cdot 59}{1.7 \cdot 218}\right)^2} \overset{?}{\leq} 0.75\left(\frac{44}{18 \cdot 15.8} + \frac{8\sqrt{6000}}{1000}\right)$$

$0.901 \leq 0.58$ ∴ No Good ⇒ the shear & torsion in this beam is just to great to be resisted by transverse reinforcement

Beam 4 Design

Beam 4 Design 1



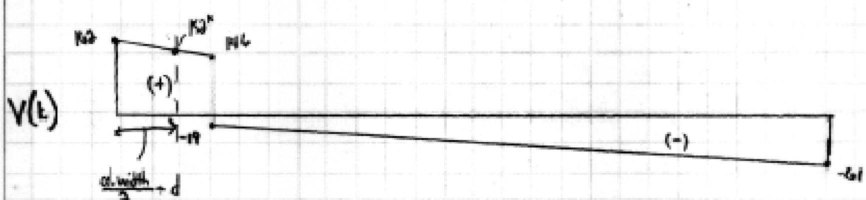
Because this beam frames directly into very stiff columns, it is assumed fixed-fixed

$w_u = 1.2(52\cdot13.75 + 10.5\cdot13.75) + 1.6(40\cdot13.75) = 3.6$ k/ft

$b_{trib} = 13.75$

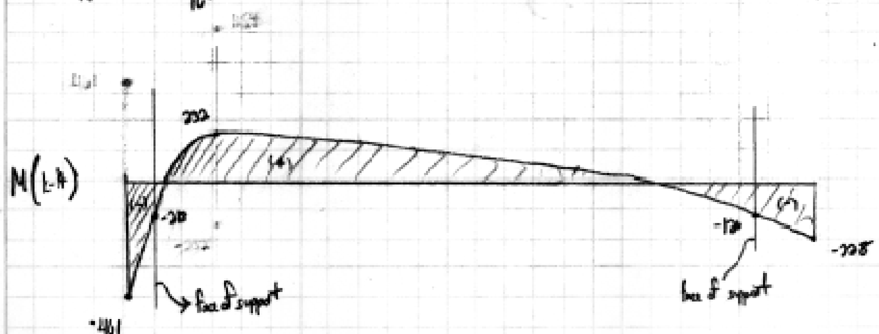
$$R_1 = \frac{3.6\cdot16}{2} + \frac{165\cdot11.5^2}{16^3}(3\cdot4.5+11.5) = 29 + 133 = 162^k$$

$$R_2 = \frac{3.6\cdot16}{2} + \frac{165\cdot4.5^2}{16^3}(4.5+3\cdot11.5) = 29 + 32 = 61^k$$



$$M_1 = \frac{3.6\cdot16^2}{12} + \frac{165\cdot4.5\cdot11.5^2}{16^2} = 77 + 384 = 461^{k}$$

$$M_2 = \frac{3.6\cdot16^2}{12} + \frac{165\cdot4.5^2\cdot11.5}{16^2} = 77 + 151 = 228^{k}$$



$M_u^- = -318$ $\quad V_u = 152^k$

$M_u^+ = 232$

2

$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$

$\frac{252 \cdot 12}{0.9} = \rho \cdot 60 \cdot 16 \cdot 15.8^2 \left(1 - 0.59 \rho \frac{60}{6}\right)$

$\rho = 0.0141 < \rho_{max} \therefore OK$

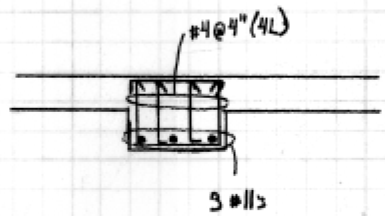
$A_s = 0.0141(15.8 \cdot 16) = 3.56 \rightarrow 3$ #11's, $A_s = 4.68 \text{ in}^2$

<u>Shear</u>

$V_u = 152^k \quad V_c = 2\sqrt{6000}\,(16)(15.8) = 39^k$

$V_s = \frac{152}{0.75} - 39 = 164^k \leq 8\sqrt{f'_c}\, b_w d = 156^k$

max spacing $= V > 4\sqrt{f'_c}\, b_w d \;\; s_{max} = \min \begin{cases} d/4 = 4'' \Leftarrow \\ 12'' \end{cases}$



$s = \frac{(0.80)(60)(15.8)}{164} = 4.62'' \rightarrow$ use $4''$ spacing (4 legs)

• min spacing btw bars $= 1.58'' > d_b \therefore OK$

<u>Deflection</u>

$nA_s = 10(4.68) = 46.8 \text{ in}^2 \qquad b_{eff} = b_w + 16 h_f = 16 + 16 \cdot 9 =$

$= b_w + 2(\tfrac{1}{2} d_{clr}) = 16 + 57 + 96 =$

$= \tfrac{1}{4}$ span length $= \tfrac{1}{4} \cdot 16 \cdot 12 = 48'' \Leftarrow$

$(48'')(c)\left(\frac{c}{2}\right) = (46.8)(15.8 - c)$

$24c^2 + 46.8c - 739 = 0$

$c = 5.0'' < h_f \therefore c$ is w/in flange

$I_{cr} = \frac{1}{12}(48)(5)^3 + (48)(5)\left(\frac{5}{2}\right)^2 + (46.8)(15.8 - 5)^2 = 500 + 1500 + 5459 = 7459 \text{ in}^4$

$\Delta_{dist.\,load} = \frac{wl^4}{384EI} = \frac{w(16)^4}{384(4415)(7459)} \times 1728 = 0.008955w$

$\Delta_{point\,load} = \frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx) = \frac{P(4.5)^2(8)^2}{6(4415)(7459)(16)^3}(3 \cdot 4.5 \cdot 16 - 3 \cdot 4.5 \cdot 8 - 11.5 \cdot 8) \times 1728 = 0.0008891P$

3

Use factored loads for a quick check of what I believe will not be a problem:

$\Delta_i = 0.008955(3.0) + 0.0002891(105) = 0.0325$

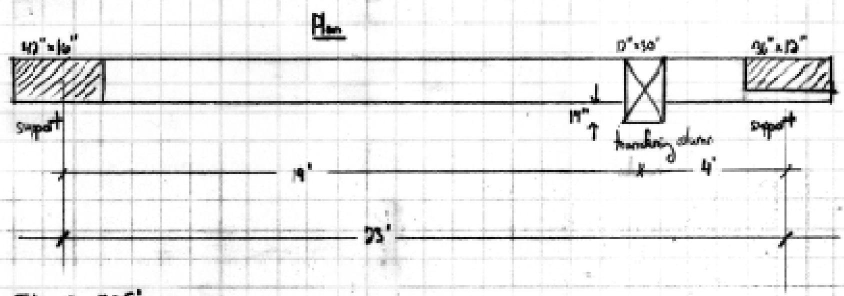
$\Delta_{lt_2} + \Delta_i = 3(0.0325) = 0.10''$

Even conservatively using factored loads, and considering all loads to be sustained produces a total deflection of only $0.10''$ ∴ Deflections will not be a problem

Beam 5 Design

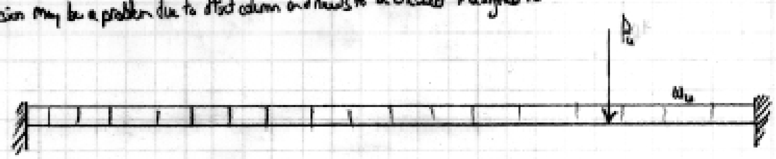
Beam 5 Design 1

Plan



Trib width = 5.75'

- Due to the large stiffness of the columns the beam frames into, supports assumed as fixed
- Torsion may be a problem due to offset column and needs to be checked & designed for



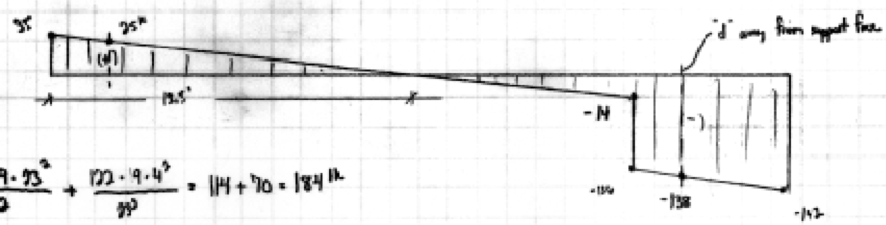
$$w_u = 1.2\left[\underset{\text{SDL}}{5.75' \cdot 52} + \underset{\text{DL}}{\left(5.75 - \tfrac{16'}{12}\right)(125)} + \underset{\text{self wt}}{\tfrac{16}{12}(231)} + \underset{\text{wall loading}}{750}\right] + 1.6\left[\underset{\text{LL}}{5.75 \cdot 40}\right] = 2225 + 368 = 2.59^k$$

$P_u = 122^k$

$$R_1 = \frac{2.59 \cdot 23}{2} + \frac{122 \cdot 4^2}{23^3}(2 \cdot 19 + 4) = 30 + 5 = 35^k$$

$$R_2 = \frac{2.59 \cdot 23}{2} + \frac{122 \cdot 19^2}{23^3}(19 + 3 \cdot 4) = 30 + 112 = 142^k$$

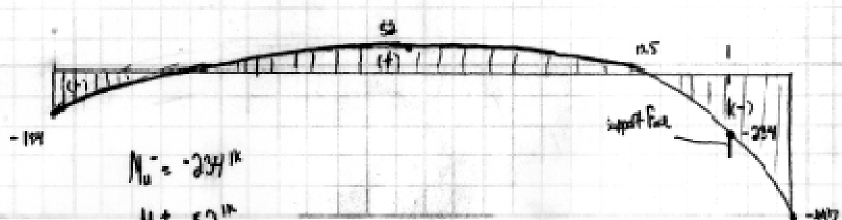
V(k)



$$M_1 = \frac{2.59 \cdot 23^2}{12} + \frac{122 \cdot 19 \cdot 4^2}{23^2} = 114 + 70 = 184^{1k}$$

$$M_2 = \frac{2.59 \cdot 23^2}{12} + \frac{122 \cdot 19^2 \cdot 4}{23^2} = 114 + 333 = 447^{1k}$$

M(ft-k)



$M_u^- = -234^{1k}$

$M_u^+ = 52^{1k}$

2

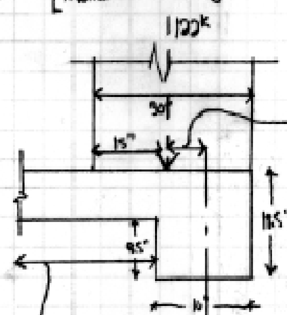
$$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$$

$$\frac{234 \cdot 12}{0.9} \cdot \frac{1}{16 \cdot 15.8^2 \cdot 60} = \left(1 - 0.59 \rho \cdot 10\right)\rho$$

$$\rho \geq 0.0141 < \rho_{max} = 0.0339 \therefore \text{ok}$$

$$A_s = 0.0141\left(16 \cdot 15.8\right) = 3.56 \text{ in}^2 \Rightarrow 3 \#11\text{s}, A_s = 4.68 \text{ in}^2$$

[Available steel for longitudinal torsional reinf. = 4.68 − 3.56 = 1.12 in²]



e = 7" ⇒ T_u can be approximated as $P_u \cdot e$ = (122k)(7") = 854 in-k

effective overhang $\begin{cases} \leq h - h_f = 18.5 - 9 = 9.5" \Leftarrow \\ \leq 4h_f = 4 \cdot 9 = 36" \end{cases}$

Should Torsion Be Considered?

Is $T_u \geq \phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right)$?

$$A_{cp} = (16 \cdot 18.5) + (9.5 \cdot 9) = 382 \text{ in}^2$$

$$P_{cp} = 18.5 + 16 + 9.5 + 9.5 + 9 + 9.5 + 16 = 88 \text{ in}$$

$$\phi = 0.75$$

$$T_u = 854 \text{ in-k} > \frac{0.75\sqrt{6000}}{1000}\left(\frac{382^2}{88}\right) = 96 \text{ in-k}$$

Torsion must be considered

Is section large enough for combined shear & torsion?

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \leq \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$$

$$V_u = 138^k$$

$$P_h = 12.5 \cdot 2 + 15 \cdot 2 = 55 \text{ in} \qquad V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{6000} \cdot 16 \cdot 15.8 = 39^k$$

$$A_{oh} = 0.85(12.5 \cdot 15) = 187 \text{ in}^2$$

$$\sqrt{\left(\frac{138^k}{(16 \text{ in})(15.8 \text{ in})}\right)^2 + \left(\frac{854 \text{ in-k} \cdot 55 \text{ in}}{1.7 \cdot 187.3}\right)^2} \overset{?}{\leq} 0.75\left(\frac{39^k}{16 \cdot 15.8} + \frac{8\sqrt{6000}}{1000}\right)$$

$$0.55 \leq 0.58 \therefore \text{ok}$$

3

Req'd Transverse Reinf. For Shear

$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = 0.75 \cdot 2\sqrt{6000}\,(16)(13.8) = 39^k$

$V_u = 138 > \phi V_c$ ∴ need shear reinf.

$V_s = \frac{138^k}{0.75} - 39 = 145^k < 8\sqrt{f'_c}\, b_w d = 156^k$

$\frac{A_v}{s} = \frac{V_s}{f_{yt} \cdot d} = \frac{145}{60 \cdot 16.8} = 0.15\ in^2/in$

" " " Torsion

$T_n = \frac{T_u}{\phi} = \frac{71^{ft k}}{0.75} = 95^{ft k}$

$\frac{A_t}{s} = \frac{T_n}{2A_o f_{yt} \cot\theta} = \frac{95 \cdot 12000}{2 \cdot 159 \cdot 60^{ksi} \cot 45} = 0.0597\ in^2/in$

$A_o = 0.85 A_{oh} = 0.85(187) = 159\ in^2$

Transverse Reinf. for Combined Shear & Torsion

$\frac{A_v}{s} + \frac{2A_t}{s} = 0.15 + 2(0.0597) = 0.269\ in^2/in$

For #4, Area of 2 legs = 0.40 in²/in
" " " " 4 legs = 0.80 in²/in

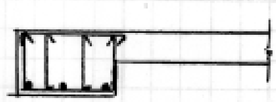
Req'd s = $\frac{0.80}{0.269} = 3.0"$ ⇒ use s = 3" ⇒ Very close, however clear spacing btw transverse bars is (3" − 0.5") = 2.5" > d_b ∴ Ok

At all other parts of beam (lower V), shear reinf. only req'd where $V_u > \phi V_c = 0.75 \times 39^k = 29^k$ which is only at last 4.5'

$\frac{A_v}{s} + \frac{2A_t}{s} = 0 + 2(0.0597) = 0.119$

Req'd s = $\frac{0.80}{0.119} = 6.7"$ ⇒ use 6"

⇒ For shear & torsion transverse reinf., use #4 @ 6" (4L) to left of column load & #4 @ 3" (4L) to right of column load



Required Longitudinal Torsional Reinf.

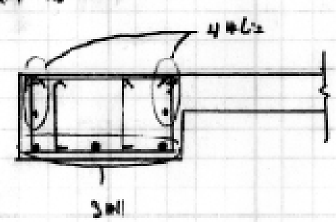
Add. $A_\ell = \frac{A_t}{s} P_h \left(\frac{f_{yv}}{f_{y\ell}}\right) \cot^2\theta = 0.0597 (55 in)(1)\left(\frac{1}{\tan 45}\right)^2 = 3.28 in^2$

Min $A_\ell = \frac{5\sqrt{f'_c} \cdot A_{cp}}{f_{y\ell}} - \frac{A_t}{s} P_h \frac{f_{yv}}{f_{y\ell}} = \frac{5\sqrt{6000} \cdot 383}{60000} - 0.0597 \cdot 55 \cdot 1 = (a\ negative) \Rightarrow \therefore ok$

Dist. additional longitudinal reinforcing around perimeter @ spacing $\leq 12"$

$\frac{1}{3} A_L = \frac{1}{3}(3.28) = 1.09\ in^2$ — top
bottom
middle

2 #7 = $A_s = 1.20\ in^2$



5

Check Deflection

$b_{eff} = b_w + \frac{1}{12}\,\text{span} = 16 + \frac{1}{12} \cdot 23 \cdot 12 = 39''$

$n \cdot A_s = 10(4.68) = 46.8$

Find c: $(39'')(c)\left(\frac{c}{2}\right) = (46.8)(15.8 - c)$

$19.5c^2 + 46.8c - 739 = 0$

$c = 5.1''$

$I_{cr} = \frac{1}{12}(39)(5.1)^3 + (39)(5.1)\left(\frac{5.1}{2}\right)^2 + (46.8)(15.8 - 5.1)^2 = 7083\ \text{in}^4$

$\Delta_{dist\ load} = \frac{wl^4}{384EI} = \frac{w(23)^4}{384(4415)(7083)} \times 1728 = 0.0403\,w$

$\Delta_{point\ load} = \frac{Pb^2x^2}{6EI\,l^3}(3al - 3ax - bx) = \frac{P(4.5)^2(11.5)^2}{6(4415)(7083)(23)^3} \times 1728 \times (3 \cdot 19 \cdot 23 - 3 \cdot 19 \cdot 11.5 - 4 \cdot 11.5) = 0.000976\,P$

@ mid-span

↓

$\Delta_{max,\ point\ load}$ @ $x = 14.3'$ so

$\Delta_{mid\text{-}span\ point\ load}$ is close enough

If factored loads are not a problem in deflection, then deflections are not a problem

$\Delta_i = 0.0403(2.59) + 0.000976(122) = 0.10'' + 0.119'' = 0.22''$

Conservatively apply $\lambda_\infty = 2.0$ to this $\Delta_i \Rightarrow \Delta_{LT} = 2.0(0.22) = 0.44''$

$\Delta_{total}\ (\text{conservatively}) = 0.44'' + 0.22'' = 0.68'' < \frac{L}{240} = \frac{(23 \cdot 12)}{240} = 1.15''$ $\therefore$ Deflections will not be a problem

APPENDIX E

BREADTH STUDY CALC'S

Perimeter Slab Material Takeoff

Perimeter Slab 1

Material Savings - Concrete

Existing → 812 sf of 18.5" × $\frac{18.5}{12}$ = 1252 cf = 46 cy

868 sf of transition slab × $\left(\frac{18.5+9}{2}\right) \times \frac{1}{12}$ = 995 cf = 37 cy

15 stories → 15 × (46+37) = 1245 cy

Take out transfer system → 175 × $\left(\frac{18.5+9}{2}\right)\frac{1}{12}$ + 157 × $\frac{18.5}{12}$ = 443 cf = 16.4 cy

Total = 1245 − 16 = 1229 cy

New PT Design → (812+868)$\left(\frac{9}{12}\right)$ = 1260 cf = 47 cy

15 stories → 15 × 47 = 705 cy

Take out transfer system → (175+157) × $\frac{9}{12}$ = 249 cf = 9.2 cy

Total = 705 − 9 = 696 cy

Difference in concrete = 1229 − 696 = 533 cy

Δweight = 150 $\frac{lb}{ft^3}$ × 533 y^3 × 27 = 2159 k

Material Savings - Steel

Existing Design

S[illegible]

Top Steel: 9 #6 - 10'
10 #6 - 35'
4 #6 - 35'
6 #6 - 20'
3 #6 - 16'
14 #6 - 26'
17 #6 - 40'
5 #6 - 15'
7 #6 - 18.5'
6 #4 - 7'

Bottom Steel: 6 #5 - 32.5'
10 #5 - 32.25'
4 #5 - 32'

Base Bott. Mat of 6 @ 6" → 10 #6's along the 185' perimeter

646' of #5s

3760' of #6's

42' of #4's

Weight = (3760 ft × 1.502 lb/ft) +
(42 ft × 0.668 lb/ft) +
(646 ft × 1.043 lb/ft) = 6349 lb of steel / floor
(14th floor → 4786 lb)

2

New PT design

9·10' + 9·32.5' + 9·25.5 + 8·14 + 8·27 + 8·14.9 + 8·5' + 8×15.3 + 9·28 + 4·9.5 + 9·17 + 9·33 + 1·9

= 2136 ft of #5's

Weight = 2136 ft × 1.043 lb/ft = 2228 lb of steel / level
(1616 on 19th level)

PT tendons
(185')(16·0.520 lb/ft) = 1539 lb / level
(1298 lb on 19th level)

Savings → (6349 − 2228) × 14 + 4786 − 1616 = 60,864 lb of standard reinforcing steel

or 35% of existing usage

(65% lighter)

PT tendons

Total weight saved is 61k + 2159k − 22.5k = 2197k lighter

(Calculated building weight from Tech 2 = 41,852k)

BUILDING IS 5.2% lighter

Transfer System Material Takeoff

Material Savings - Transfer Slab 1

Existing Design

A = 1,248 sf

Concrete: $\left(\frac{18.5 \text{ in}}{12}\right)(1248 \text{ ft}^2) \times \frac{1 \text{ cy}}{27 \text{ ft}^3} = 71 \text{ cy}$ (288,000 lb)

Steel: #10's @ 6" E.W. top & bottom ⇒ $\frac{4 \text{ ft of steel}}{1 \text{ sf area}}$ ⇒ 4992 ft of #10's ⇒ (4992 ft)(4.303 lb/ft) = 21,481 lb

26 add'l #5's: (36 ft)(26 bars) = 936 ft ⇒ (936 ft)(1.043 lb/ft) = 976 lb

976 + 21,481 lb = $\boxed{\text{22,460 lb of steel}}$

Transfer Beam Design

total area = 1,248 sf

area of beams = 204 sf

Concrete: $(1{,}248 \text{ sf})\left(\frac{9"}{12}\right) + \frac{(204 \text{ sf})(18.5"-9")}{12} \times \frac{1 \text{ cy}}{27 \text{ ft}^3} = 42 \text{ cy}$ (170,000 lb)

Steel:

Beam Reinforcing - Beam 1: (32.5 ft)(36) = 1170 ft #11

(16.34/4") × (32.5 ft) × 3 = 1589 ft #4

Beam 2: (17.8 ft)(2) = 36 ft #11

(8 ft/4") × (17.8 ft) = 427 ft #4

Beam 3: (23.5 ft)(5) = 118' #4

(9.3 ft/3.5") × (23.5 ft) = 749' #4

Beam 4: (19 ft)(3) = 57 ft #11

(7.33 ft/4") × (19 ft) = 418 ft #4

Beam 5: (26.2 ft)(3) = 79 ft #11

(26.2 ft)(4) = 105 ft #5

& (7.33 ft/3)(25) + (7.33 ft/6")(17.33) = 320 ft #4

Total = 1342 ft of #11

3510 ft of #4 ⇒ 9689 lb

105 ft of #5

7

9" slab reinforcing: Typical floor reinforcing w/ typical floor spans are: column strip - #4@12" EW, bottom; #6 @12" EW, top $\Big\}$ (0.668·2 + 1.502·2) = 4.34 lb/sf

mid strip - #4@12" EW, bottom; #5@12" one way, top $\Big\}$ (0.668·2 + 1.043·1) = 2.34 lb/sf

- Assuming that the 9" slab supported by the 18.5" beams is reinforced similarly to the typical floors (which have similar spans) $\Rightarrow$ (1,248 sf - 264 sf)$\underbrace{(0.6 \times 2.34 + 0.4 \times 4.34)}_{\text{assume approximately 40\% of floor is column strip}}$ = 3,090 lb

Total Steel $\rightarrow$ 9,689 lb + 3,090 lb = <u>12,880 lb</u>

Concrete Savings: 71 cy - 42 cy = 29 cy $\Rightarrow$ 117,450 lb

Steel Savings: 22,460 lb - 12,880 lb = 9,580 lb

Transfer system is 127^k lighter $\Rightarrow$ 0.3% of building weight

Perimeter Slab Cost Estimate

RS Means - Perimeter Slab Estimate 1

Post-tensioning the 9.5'-wide perimeter will require an additional crew for this step on each floor. The small changes in forming requirements likely will not require additional labor or material. Thus, the only difference will be found in the reinforcing steel layout, post-tensioned tendons, and pouring of concrete. All other tasks (e.g. floor finishing, accessories) will remain unchanged, in terms of the cost of material and labor per unit material.

From RS Means Building Construction Cost Data 2009*:

				MAT	Labor	EQT	Total	Total w/ O&P	
Reinforcing Steel, Elevated Slabs, #4-#7	4 Rodm	2.90 tons/day		1,050	490		2140	2600	p59
Post-tensioning Tendons, Ungrouted single strand, 25 kip	Crew C4: 1 Rodman Foreman, 3 Rodmen (outside), 3 stressing equipment	1200 lb/day		0.62	1.20	0.02	1.84	2.47	p61
Cast-in-place Concrete	6000 psi		127/cy				127	139	p64
• For high early strength add 10%	C7	110 cy/day							
" placing concrete, Elevated slabs 6-10" thick, w/ crane & bucket				22.50	16.90	33.40	46		p.64
" " over 10"		90/day		27.50	13.30	40.80	56.50		

includes 4-time use formwork

Apply 10% multiplier for Project Overhead to total cost

City Cost Indexes

		MAT	INST	TOT
NY, NY:	Concrete Forming -	103	144.5	139.3
"	Reinf. -	106.3	140.3	140.9
	Cast-in-place concrete -	108.5	148.8	134.7
Total:		106.1	114.6	130.7

* 2009 will be used, as this is the time frame in which the superstructure was built

Transfer System Cost Estimate

RS Means - Transfer System Estimate

The major difference between the existing design and proposed redesign will be the difference in concrete and steel material and the increased forming costs for the beams.

From RS Means Building Construction Cost Data 2009 and in addition to that already referenced for post-tensioned slab redesign:

Forms In Place, Beams & Girders								
Exterior Spandrel 18" wide 4 use	C2	315 sfca/day	0.99	5.95	0	6.94	10.30	p 42
Interior Beam Job-built, 12" wide 4 use		377 sfca/day	1.40	4.95	0	6.35	9.25	"
" " 24" " "		395 /day	0.90	4.73	0	5.63	8.35	p 43
Forms In Place, Elevated Slab								
Flat Plate, job-built, 15' high 4 use		560 sf/day	1.47	3.39	0	4.86	6.75	p 44
Reinforcing								
Beams & Girders, #3 to #7		2.90 ton/day	1550	530	0	2080	2575	p 59
Placing Cast in place concrete Beams w/ crane & bucket small beams		45 cy/day	55	55	94.50	8.50	113	p.64
Reinforcing Steel, Elevated Slabs #3 to #7		[illegible] ton/day	1150	892	0	–	[illegible]	

Transfer System: Impact on Schedule

Transfer System - Impact on Schedule 1

Existing Design

- 0.5 tons reinf. #4-#7 ÷ 2.9 = 0.17 days - 2 hrs
- 10.7 tons reinf. #8-#18 ÷ 4.9 = 2.2 days

$\}$ 2.37 days

- 71 cy concrete ÷ 90 = 0.8 days → 1 day

2.37 days + 0.8 days = 3.17 days

with the rate of construction in NYC, this can likely be truncated to 3 days ⇒ 2 days laying reinforcement + a day placing concrete.

*Note: Forming of slab will be ignored, as this will be nearly identical in both designs; only additional beam formwork will be considered

- PT requires 2nd crew
- Can build above a floor before stressing takes place
- tendon placement will require an additional day for labor
- comment on why PT isn't used
- comment on cost assumptions

Talk about what Dan did on electrical/existing schedule

Talk about RS Means - 3000 sf/day using PT tendons

Tendons require 2 days for HS concrete to reach f'_{ci}

Alternate Design

- ✓ 1.6 tons slab reinf. #4-#7 ÷ 2.9 = 0.6 days
- 87.3 cy slab ÷ 110 = 0.25 days
- ✓ 370 sfca beam formwork ÷ 377 = 1.0 days
- ✓ 5.0 tons beam reinf. ÷ 2.7 = 2.0 days
- beam concrete 7.7 cy ÷ 75 = 0.2

1 day of formwork
2.5 days of reinf. placement
1 day of placing concrete

$\}$ ≃ 4.5 days