Northside Piers – Brooklyn, NY
Structural System Redesign

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AE 482 – 5th Year Senior Thesis
Pennsylvania State University
Structural Option

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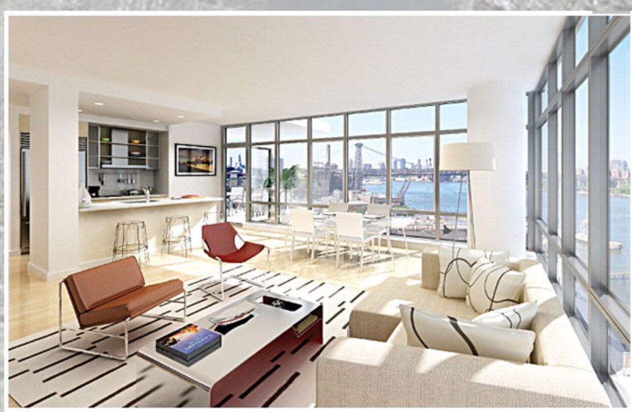
North Side Piers

Brooklyn, NY

Building Information
Type - Condominiums
Stories - 29 (above grade)
Size - Approximately 250,000 sq ft
Construction - May 2006 - Early 2008
Delivery Method - Construction Management

Project Team
Owner – Toll Brothers, City Living
Architect – FXFowle Architects
Construction – Kreisler Borg Florman
Structural – McLaren Engineering Group
Mechanical – Consentini Associates

Architecture
Building features a tower of condominium units with a glass curtain system which allows for floor to ceiling views of the New York City skyline. The second floor is where many of the public spaces are including the gym, yoga room, sauna, massage room, and kid's play room. The building also contains four townhouses which blend it into the surrounding community.



Structural System
- Typically 8" 6000 psi two-way reinforced concrete floor slabs
- 8000 psi columns along perimeter of building with few interior columns
- Concrete shear walls around the central stair core to resist lateral loads
- Foundation of 200 ton piles at ten feet below grade with perimeter grade beams

Mechanical System
- Condo units serviced by heat pumps
- Two rooftop HVAC units supply air to the corridors
- Self-contained air conditioners service the storage areas, lobby, and gym
- Two 500 ton cooling towers reject heat

Lighting/Electrical
- Power distributed on 480, 208, 120/208 volt circuits
- 500 kVA transformer for elevator circuits
- Lighting is primarily compact fluorescents

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www.engr.psu.edu/ae/thesis/portfolios/2008/jte132/

Structural Option

Acknowledgements

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A special thanks to all of my friends and family for all of their support throughout my entire college career.

Finally, thanks to all of my fellow students and the Class of '08.

Executive Summary

This report evaluates some potential changes to the structural system of Northside Piers including switching the existing mild reinforced floor slab to a post-tensioned system as well as considering alternative shear wall layouts.

An alternative post-tensioned system was designed for the current gravity loads. Two of the typical floor plans that are repeated throughout the height of the building were redesigned. The new system will consist of a 7” thick slab with ½” unbonded tendons. The tendons will be banded in the North-South direction and uniformly spaced in the East-West direction. This new system will have better control over long-term deflections with an expected decrease of 30% in total deflection. It will perform just as well in terms of sound transmission as the original design and will cost approximately $2/SF less. In addition, the 1” saved on slab height can used for a reduction in building height of 30” which would result in a savings of $36,000 in cladding cost.

Five alternative shear wall layouts were analyzed which determined that a layout with an additional wall off of the central core that goes up 11 stories of the building is the most efficient. This layout should save 5% of the cost of the original layout due to the reduction in required concrete and rebar. It was also found that adding 3” to the depth of the link beams will reduce the torsional deflection by 12%. This is important because the largest acceleration issues in the building come from the torsional deflection of the building.

The schedules for the alternative shear wall layout and post-tensioned slab were determined to be relatively unadjusted due to their similar nature to the original construction processes.

Finally it was determined that the risers used for exhaust in the building should not be adjusted in size. Due to pressure losses in the ducts, a reduction in size of 33% will result in a higher operating cost. An increase in size of 50% will save some in operating cost, but will result in a higher initial cost. When comparing the lifetime costs of these alternatives to today’s dollars, the original duct size is the cheapest. While reducing the size of the ducts will result in fewer conflicts with the slab reinforcement and penetrations, the additional expense is not worth it.

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Introduction

This report is the conclusion to the capstone project for the Architectural Engineering program at The Pennsylvania State University. The project involves taking a newly constructed building and spending an entire year reviewing the existing systems and developing potential alternatives for the building.

The Fall Semester involved studying in detail all of the existing conditions for the building. This included examining the architecture, structural system, lighting/electrical systems, and mechanical systems. Then, three technical reports were created that involved studying specific elements of the student's specialty in more detail. For a structural student, this consisted of determining all of the loadings for the building and confirming the capacities of the existing structure. It also included studying four alternative floor systems to determine which were feasible alternatives. The Fall Semester ended by creating a proposal for the Spring Semester redesign.

The contents of this report include all of the studies done during the Spring Semester. These studies consisted of two analyses within the structural discipline and two breadth studies of different disciplines. The first structural analysis involved determining whether or not a post-tensioned slab would be more efficient than the existing mild reinforced slab. The second structural analysis considered alternative layouts for the shear walls in order to improve serviceability and decrease cost. The breadth studies consisted of looking at cost and schedule implications, as well as deciding if alternative ductwork riser sizes should be used.

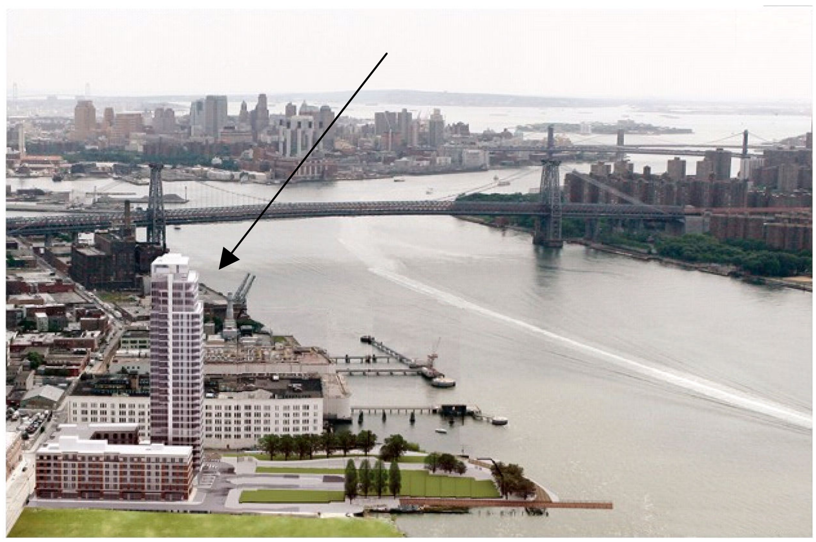
It is important to note that while great efforts have been taken to provide accurate and complete information within this report, there is still potential for errors in the calculations and designs. Any modification or changes related to the original building designs are solely my interpretations. Differences may be due to alternate assumptions, code references, and/or requirements.

The goal of this project is not solely to decide what could have been done by the original engineers, but rather it is for educational purposes. There will be some studies and checks included that are not completely necessary for the building, but are rather performed for the sake of exercise. Likewise, there will be some checks that are not included because the experience is tedious and not very personally benefiting. A more exhaustive approach should be used if this were to be a real design.

Building Background

Architecture:

Northside Piers is a building currently being constructed on 164 Kent Ave. in the Brooklyn, New York area. It is a 29-story condominium tower built directly off of the East River across from Manhattan Island. It is going to be the first of three residential towers to be built on the site. The building is taking advantage of a recent change in zoning that now allows residential properties to be built in that area, so it is the first of its kind.



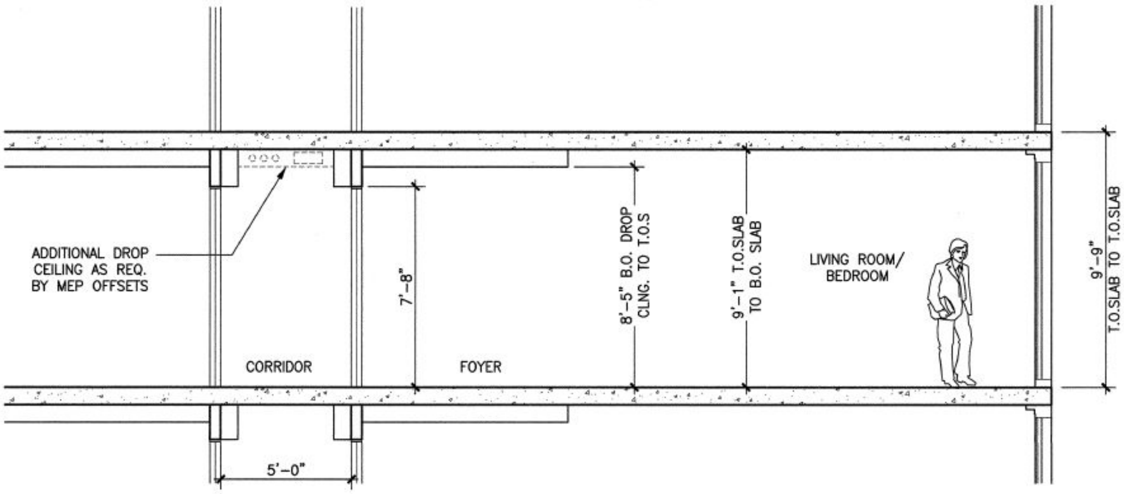
The building features a glass cladding system that allows for floor to ceiling windows for uninhibited views of New York City. The ground floor contains the lobby, which leads to the central elevator shaft that services the building. The second floor is where many of the public spaces will be including a fitness room, yoga room, sauna, media room, and children's playroom. The other twenty-seven levels are dedicated to the private condominium units. The mechanical equipment is located in the cellar, ground floor, and on the roof. The typical floor size is approximately 7,500 square feet.

Structural System:

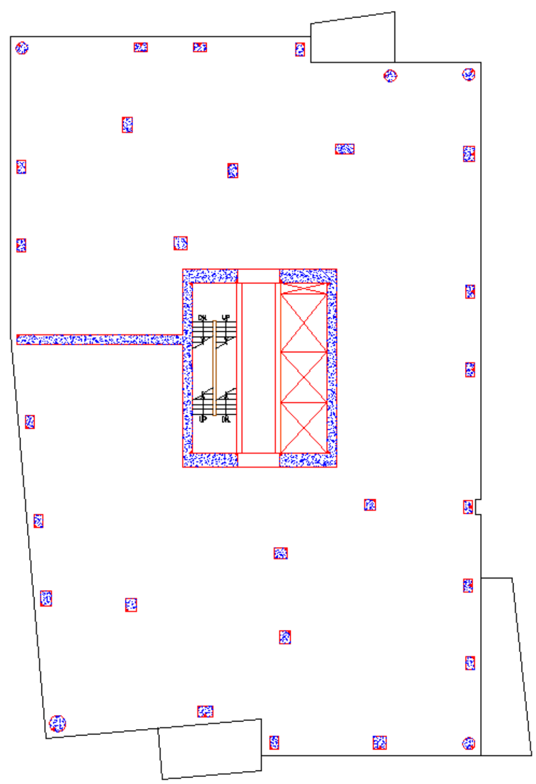
Floor System

The typical floor system consists of an 8" thick two-way flat plate slab system. Slabs consist of 6,000 psi concrete with #5 reinforcing bars spaced 12" o/c on the top and bottom of the slab going both directions. Additional reinforcement is placed at the columns as needed.

Additional beams are only introduced a few times in the building. These are used to transfer column loads when the building's setbacks force the column grid to change.



The ceiling finishes are attached either directly to the underside of the slab or there is an 8" drop that is used for MEP. The floor-to-floor height is 9'-9" leaving very limited space for any additional structure. Any additional depth would need to be added to the overall building height.



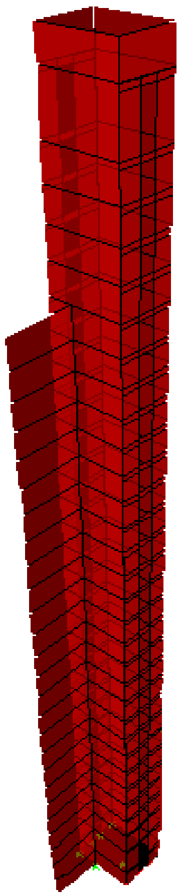
Columns

The columns in the building do not follow a regular grid because they are adjusted to fit in better with the floor plans. Most of the columns are located around the perimeter of the building with an average spacing of 15 feet. There are also a few columns located on the interior to break up the large bays. Most of the columns are rectangular and are hidden behind walls, but the exposed ones are circular. Columns consist of 8,000 psi concrete with usually 8 reinforcing bars along their edge varying in size from #7-#11. The bars are held in place with ties. A typical plan is shown to the right.

Lateral Resisting System

Lateral forces are carried in this building by the central core, which can be seen below. It consists of concrete shear walls surrounding the elevator shaft and stairwell on all four sides.

The walls are 1 ½ foot thick in the North-South direction and 2 feet thick in the East-West direction, and they extend from the foundation to the top of the building. The concrete strength is 8000 psi until the 14th level where it decreases to 6000 psi. The reinforcing is typically #5-#7 at 12 in. o/c. on both faces of the walls.



The walls in the East-West direction contain penetrations at every level to accommodate for doorways. The wall is still continuous due to a 2 feet deep coupling beam reinforced with #9 and #10 bars at the top and bottom.

There is one additional shear wall in the East-West direction that extends off of the building core. It starts at the foundation of the building and goes all the way to the 25th floor.

Foundation

The columns and shear walls sit on top of a foundation of 200-ton piles that are located ten feet below grade. Grade beams run along the perimeter of the building. The highest concentration of piles is directly underneath the central core of the building in order to transfer the high moments to the ground below.

Depth Analysis – Alternative Slab System

Introduction:

The first study performed involved looking at the current floor system and trying to come up with an alternative that could be more efficient and cost effective. The building has two major column grid layouts that are used repetitively in the building. The first goes from the Cellar to the 25th floor of the building with only a minor variation to that scheme on the Ground and 2nd floor. The other grid is used on the 26th floor to the Roof. These are the two layouts that are going to be studied in detail. The structural plans can be seen in the appendix.

When considering an alternative to the current slab system depth became one of the major factors that made several options poor solutions. The way the building is now, the ceilings above the living rooms and bedrooms have the finishes placed directly on the slab, meaning there is no plenum there. For a system with beams or drop panels to work the structure would have to go down into the spaces below or there would need to be a plenum over the entire building. One of the highlights of this building architecturally is its glass cladding system, which allowing for floor-to-ceiling views. Any structure that would interrupt that would not be a viable solution, so a flat-plate system is the best choice.

Given the building's situation a post-tensioned system is the best possible alternative to the current system. The additional compression on the concrete allows for the slab to be used more efficiently which should result in less mild reinforcement being required. It also gives the potential for a reduction in slab depth, which would result in savings in concrete. A post-tensioned slab also performs better in terms of deflection due to the load balancing effects the tendons give.

The design of the new post-tensioned slab will be done using the program RAM Concept. This program works using a 3D finite element mesh, so it requires the input of the entire slab as opposed to just a specific strip like PCA Slab does. This is necessary for this design given the irregular column locations. The program will design the slab for all of the standards required by ACI 318-05.

In order to come up with initial values as well as check the program's results, two spreadsheets were created. The spreadsheets check all of the stresses and strengths for one-way and two-way slabs. Both spreadsheets were needed because the behavior of having banded tendons in one direction and uniformly distributed tendons in the other direction falls somewhere between that of one-way and two-way slabs.

The relevant gravity loads are show below. The post-tensioned concrete system will be designed to the standards of ACI 318-05 using the loads taken from the New York City Code as well as from manufacturers. The loads in this analysis are the same values that were used in the original design.

Gravity Loads Summary

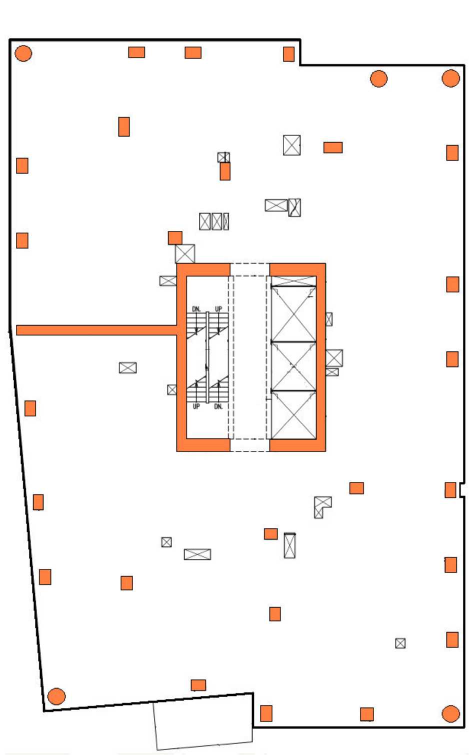
	Live Load*	Superimposed Dead Load
Multifamily Dwellings	40 psf	30 psf
Balconies (150% of serviced area)	60 psf	15 psf
Equipment Rooms (Pumps, Boilers, Tanks, etc)	150 psf	15 psf
Light Storage Areas	100 psf	50 psf
Lobby/Public Spaces	100 psf	40 psf
Offices	50 psf	30 psf
1st Floor Elevator Lobbies	100 psf	40 psf

*Live Loads May Be Reduced

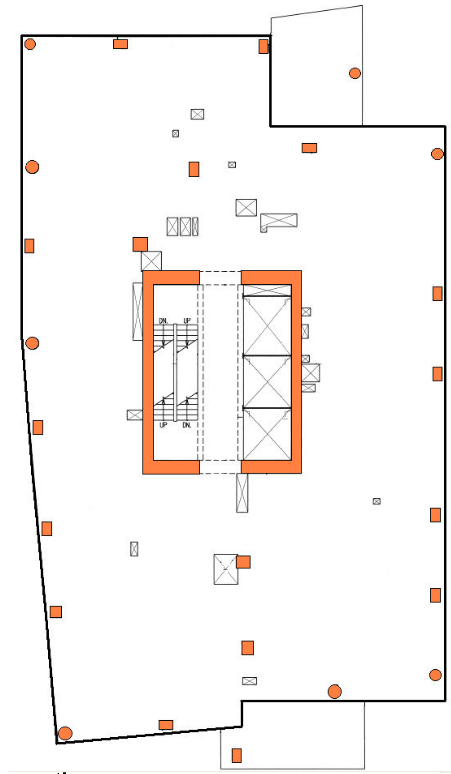
	Dead Load
Concrete	150 pcf
Glass Cladding	8 psf

Current Design – Two-Way Mild Reinforced Flat Plate:

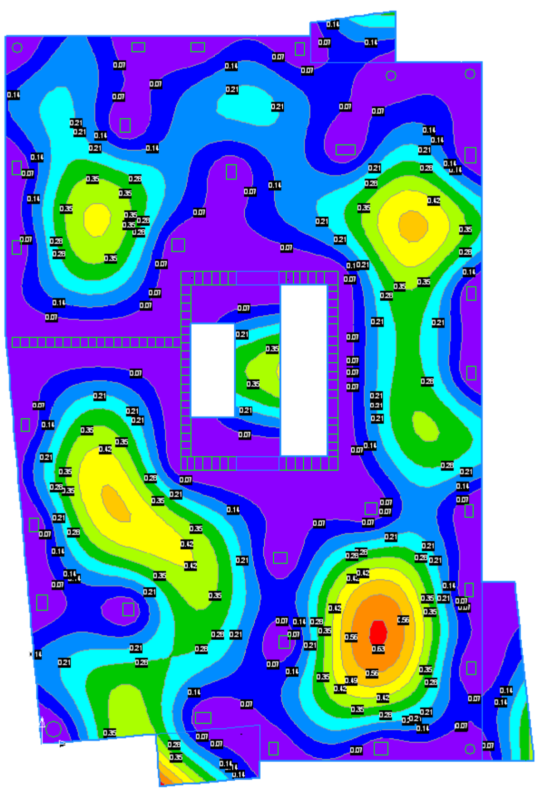
The layouts for the two typical floors studied are shown on the next page along with their long-term deflection plans. The slabs are 8" thick with #5 bars @ 12"o/c on the top and bottom, except where additional reinforcement is required which is labeled on the plan. It is indicated as 3AT512 which would mean (3) #5 bars @ 12"o/c at Top of slab. The reinforcing plans can be found in the appendix.



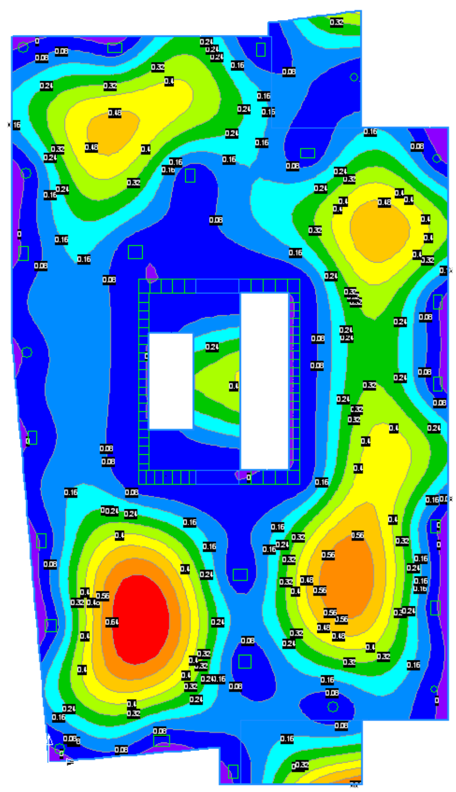
3rd-25th Typical Slab Layout



26th-Roof Typical Slab Layout



3rd-25th Long-Term Deflection



26th-Roof Long-Term Deflection

The long-term deflection plans were determined by taking 3 times the dead load short-term deflection plus 2 times the live load deflection. These multipliers were determined by 9.5.2.5 of the ACI code which states that any loads that are on the structure for 5 years or more shall have an *additional* deflection from creep equal to 2 times the short-term deflection. All of the dead load was treated as loading the structure for more than 5 years, giving a multiplier of 3, and half of the live load was treated as loading the structure for more than 5 years, giving a multiplier of 2.

The maximum long-term deflection for the 3^{rd}-25^{th} floor plan is 0.67" and for the 26^{th}-Roof floor plan is 0.74". The ACI code allows for a deflection of L/480 for floor constructions attached to elements likely to be damaged by large deflections. Since the finish is attached directly to the slab, this standard must be met. The bays where these deflections occur have spans of about 30', which corresponds to allowable deflections of 0.75". The slab meets this requirement, but not by much.

The punching shear for the slab was checked in RAM Concept using a depth of slab of 6.75" (1.25" to center of reinforcing). The percentage of load vs. capacity is shown in the table below. The exterior columns are numbered starting from the bottom left corner then going around counterclockwise. The interior columns are numbered from bottom to top. You can see that the highest values reached are 62% for the 3^{rd}-25^{th} slab and 68% for the 26^{th}-Roof slab. This indicates that punching shear is not a significant issue for the design of the floor system.

Punching Shear Check (% Capacity)

3^{rd}-25th Original											
Ext. Col. #	1*	2	3	4	5*	6	7	8	9	10	11
	46%	43%	25%	42%	34%	43%	45%	39%	52%	57%	44%
	12*	13	14	15	16	17*	18	19	20	21	22
	17%	28%	33%	32%	23%	46%	43%	41%	40%	49%	22%
Int. Col. #	1	2	3	4	5	6	7	8			
	50%	56%	37%	50%	42%	62%	42%	49%			
26^{th}-Roof Original											
Ext. Col. #	1*	2	3	4	5*	6	7	8	9	10*	
	49%	49%	37%	51%	38%	46%	61%	53%	59%	66%	
	11	12	13	14*	15	16	17	18	19	20	
	41%	40%	61%	45%	63%	32%	37%	26%	40%	65%	
Int. Col. #	1	2	3	4	5						
	47%	66%	40%	68%	50%						

* Indicates Corner Column

Alternate Design – Two-Way Post-Tensioned Flat Plate:

When considering potential tendon plans, the column locations were the most important factor. Since the floor plans were already designed to fit with a specific column grid, it would be best to try to maintain this as much as possible to prevent conflicts with the columns and the floor plans. Since the banded tendons need to go directly over the columns and can only be bent a certain amount, the best place to put them would be over straight column runs. Also, the banded tendons act like girders so they should have the shorter span lengths.

After attempting several layouts in Ram Concept, it was found that the tendon plans shown on the following pages yielded the best results. For the 26th-Roof plans, it was decided that the banded tendons should run N-S and the uniform tendons should run E-W. This layout led to an easy plan to construct with fairly uniform spans.

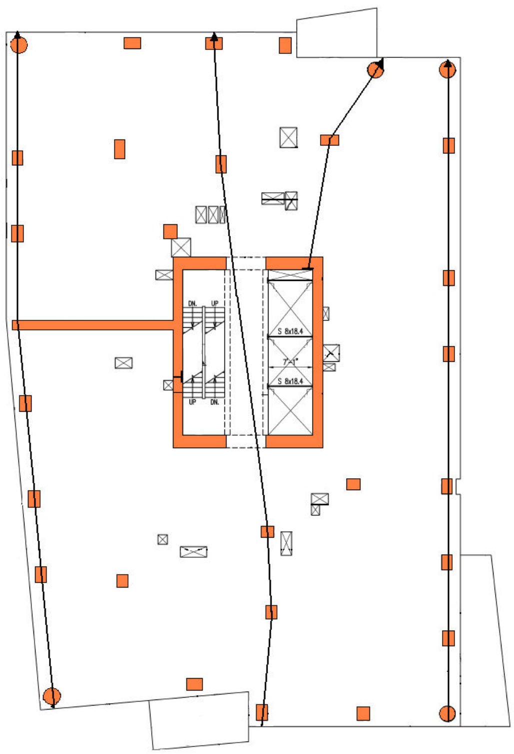
For the 3rd-25th floors, it was decided that the banded tendons should run N-S just like the 26th-Roof plans. Another option that strongly considered was having the banded tendons run E-W on only the Northern half of the building and N-S on the Southern half of the building. This was considered because it works well with the column scheme, but this idea was eventually declined in order to make the layout more standard and speed up construction. Also, this layout would require many anchors in the plan in order to deal with the changing direction of tendons.

In order to implement the change of slab system, the column scheme had to be adjusted some. On the 3rd-25th floors an interior column was deleted because it was found to be unnecessary for the current layout. On the 26th-Roof plans, an additional column was required on the North side of the building on the exterior for the banded tendons. It will require a transfer girder on the 26th floor, but one already exists there, so it will only need to be extended several feet. The new column fits in appropriately with the existing floor plans.

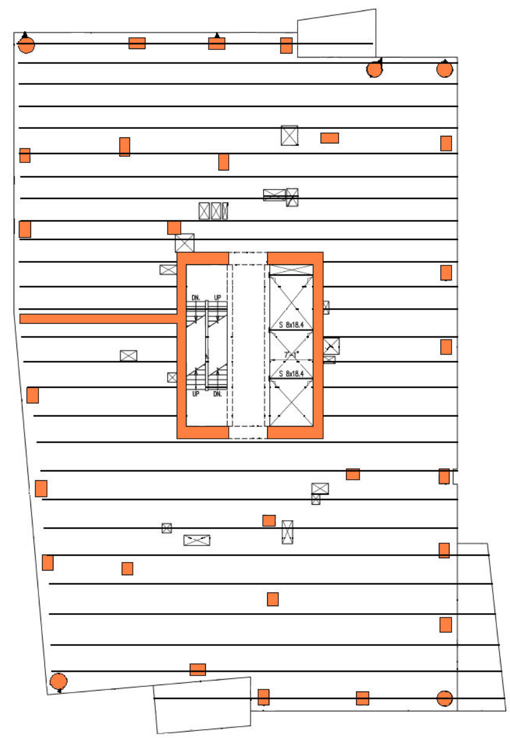
Beyond tendon layout, there were also some other factors considered in the design. Slabs of different thicknesses (8", 7", 6") were analyzed considering deflections, punching shear, and reinforcement costs, and the 7" slab gave the best overall results. The tendons used were ½" unbonded tendons because they don't require as large of ducts as a bonded system, and the additional strength of a bonded system was unnecessary. The uniform tendon profiles used a distance of 1.25" from the top and bottom of the slab in order to provide adequate cover (3/4" required for interior slabs). The banded tendon profiles used a distances of 1.25" from the bottom of the slab and 2" from the top of the slab (in order to prevent conflicts with the uniform tendons).

The final considered factor was the load balancing percentages. In the case of a 7" thick slab, the dead load from the slab contributes to about 75% of the total dead load. Since these loads are more predictable than the superimposed loads, it was decided not to exceed this value. Then, through running the analysis several times and trying values in the spreadsheets, it was decided to balance about 40% of the dead load in the uniform direction and 35% of the dead load in the banded direction. This resulted in a maximum pre-compression (F/A) stress of 320psi. **The actual reinforcing plans along with tendon profiles for the PT slab can be found in the appendix** as well as the spreadsheet examples.

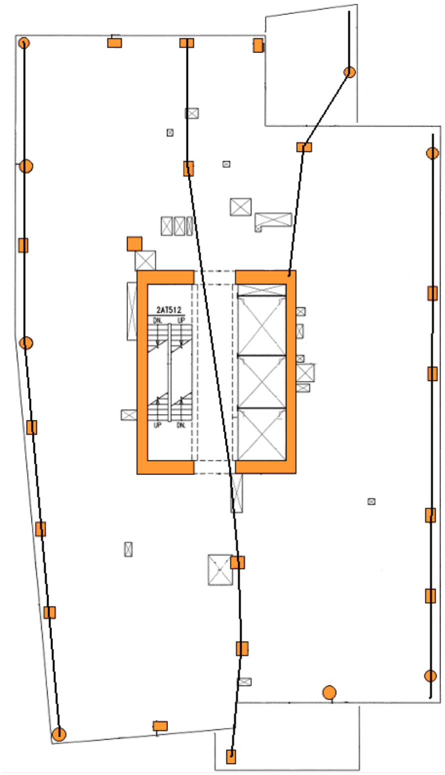
In order to create a uniform reinforcement plan, it was decided to place the mild reinforcement at the bottom of the slab as #4's @ 24" o/c going both directions. The top reinforcement is then placed at each column. The plans indicate how much reinforcement is placed at each column.



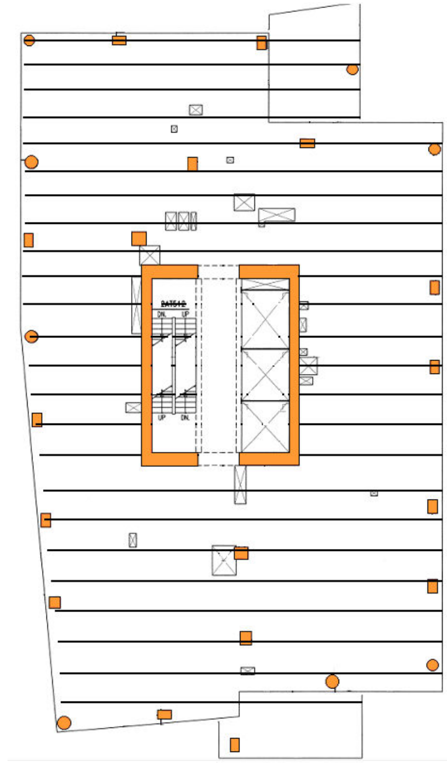
3rd-25th Banded Tendons Plan



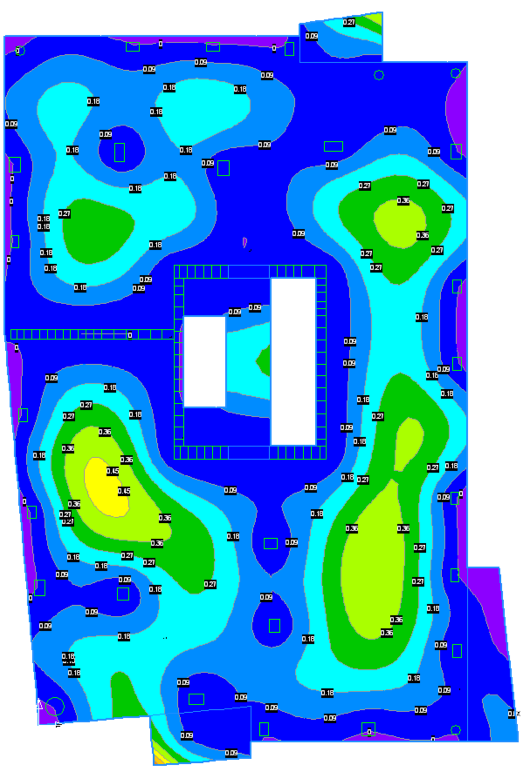
3rd-25th Uniform Tendons Plan



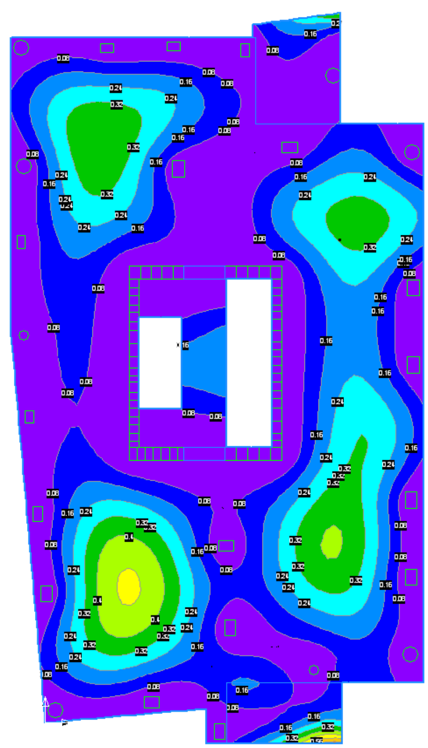
26th-Roof Banded Tendons Plan



26th-Roof Uniform Tendons Plan



3rd-25th Long-Term Deflection



26th-Roof Long-Term Deflection

The long-term deflection plans are shown above. The maximum deflections were found to be 0.48" for the 3rd-25th floor plan and 0.49" for the 26th-Roof plans.

The percentage of load vs. capacity for punching shear is shown in the table below. Due to the thinner slab size, punching shear became a slightly larger issue for the post-tensioned system, but not too much larger because the thinner slab also decreased the total dead load by 10%. With the new column sizes, the highest values reached are 63% for the 3rd-25th slab and 75% for the 26th-Roof slab. The existing column sizes still work for the 7" thick slab.

Punching Shear Check (% Capacity)											
3rd-25th Post-Tensioned											
Ext. Col. #	1*	2	3	4	5*	6	7	8	9	10	11
	50%	45%	39%	32%	44%	48%	54%	63%	54%	56%	42%
	12*	13	14	15	16	17*	18	19	20	21	22
	18%	43%	30%	47%	34%	46%	45%	42%	45%	57%	33%
Int. Col. #	1	2	3	4	5	6	7	8			
	58%	51%	51%	-	51%	54%	51%	37%			
26th-Roof Post-Tensioned											
Ext. Col. #	1*	2	3	4	5*	6	7	8	9	10*	
	43%	42%	41%	50%	45%	47%	63%	54%	60%	65%	
	11	12	13	14*	15	16	17	18	19	20	
	30%	40%	43%	43%	71%	38%	36%	27%	45%	64%	
Int. Col. #	1	2	3	4	5						
	56%	54%	45%	75%	62%						

* Indicates Corner Column

Comparison/Conclusions:

When deciding whether or not to switch from the original system of mild reinforcement to the new system of post-tensioning, several factors must be discussed. Firstly, the slab must meet certain serviceability criteria. Too much long-term deflection can lead to issues such as cracking of finishes. If this problem occurs, it will become a huge problem because the building is going to have dozens of owners through the sale of condominiums. This should definitely be avoided. The post-tensioning system has much more potential to prevent these deflections problems. The maximum deflections of the PT system were found to be 0.48" and

0.49", while the maximum deflections for the original system were found to be 0.67" and 0.74". That's a decrease of about 30% in deflection. Both designs, though, do at least meet the standards of ACI 318-05 for deflection.

When considering fire-rating issues, it was found that the 7" slab still meets the required code. In order to get a 2 hr fire-rating, IBC Table 720.1(3) states that only 2-½" of concrete in the slab is required. This means that decreasing the slab to 7" does not result in a lower fire-rating, and this is not an factor.

Another potential serviceability factor that must be considered is sound transmission, which can be very important for residential buildings. Since the post-tensioned slab was decreased in size from 8" to 7", it has the potential to transmit more sound than the original design. A study was found that determined the STC (Sound Transmission Class) Rating of a 6" slab to be 55. The STC Rating corresponds to the number of decibels that a sound level will drop by passing through it. It was also found that most objects follow the "Mass Law" which states that for every doubling of mass the STC Rating will increase by 5. After plotting that pattern starting with the 6" slab's STC Rating of 55, the equation for the STC Rating was determined to be $y=7.21\ln(x) + 42$. This means that a 7" slab will have an STC Rating of 56 and an 8" slab will have an STC Rating of 57. A single decibel is considered imperceptible to human hearing. This means that sound transmission is not a factor between the two designs.

Probably the most important consideration for which design to choose is cost. Cost however will come in several different forms, including cost of material, cost of construction, and time of construction. The cost of materials and cost of construction can be estimated using the values from RS Means. The totals from the estimate are shown below.

Floor Slab Cost Comparison

	Original (3rd-25th)		PT (3rd-25th)		Original (26-Roof)		PT (26-Roof)	
	Amt.	**Cost**	**Amt.**	**Cost**	**Amt.**	**Cost**	**Amt.**	**Cost**
Concrete (CY)	185.2	$25,558	164	$22,632	153.1	$21,128	134	$18,492
Post-Tensioning (lbs)	0	$0	4,273	$6,367	0	$0	2,921	$4,352
Formwork (SFCA)	7,589	$33,088	7,589	$33,088	6,199	$27,028	6,199	$27,028
Formwork Edge (LF)	360	$839	360	$839	346	$806	346	$806
Mild Steel Reinforcing (ton)	17.85	$23,919	2.8	$3,752	14.94	$20,020	2.3	$3,082
Total		$83,404		$66,678		$68,982		$53,760
Cost/SF		$10.99		$8.79		$11.13		$8.67

It was determined that the cost of the post-tensioned system in terms of labor and materials is significantly cheaper than the mild reinforced system. This is due to the savings in concrete as well as the savings in reinforcing. The savings were larger than expected based on the initial analysis done in RAM Concept. When comparing the minimum requirements of materials for each system, the PT slabs were about

$7,000 cheaper than the mild reinforced slabs. But these minimum requirements still had to be converted into constructible designs with more uniform reinforcement layouts. This change meant the mild reinforcement increased in both designs. A uniform top and bottom reinforcement mat was used in the original design, while only a uniform bottom mat was required in the PT design. This led to the original design being more over reinforced than the PT design. This explains the additional cost that wasn't expected initially. The PT designs ended up being about 20% cheaper than the mild reinforced designs.

The costs of other components must be also be considered. The columns and foundations will be affected by the reduction of gravity load due to the reduction of slab thickness. Since the foundation and some of the columns are tension controlled due to the high moments, this reduction in load may result in more rebar being required. While this should be examined in more detail, the differences in load are not large enough to expect significant changes that will alter the result of this analysis. Also, due to the non-uniform column layout, each column is controlled on an individual basis and many are already oversized in order to keep sizes more uniform.

The final element of cost includes the schedule. When looking at values based on RSMeans, it was found that both designs take close to the same amount of time to construct. This will be discussed more in the CM Breadth section, but for now it can be noted that the schedule was relatively unaffected, and this is not a significant factor for selecting which system to use.

In addition to the cost savings listed above, there is potential to save on the cost of cladding. The reduction of 1" in the slab size can be translated into a reduction of overall building height of 30". This will result in a potential savings of about $36,000.

Based on the analysis performed and all of the factors discussed above, it can be determined that the post-tensioned design is a better option. It performs better in terms of long-term deflection with an approximate decrease in deflection of 30%. Its sound transmission level and fire rating were determined to not be factors. The cost estimate found the post-tensioned slab to be about 20% cheaper than the mild reinforced designs, and finally it was found that the schedule was relatively unaffected.

Depth Analysis – Modified Lateral System

Introduction:

The next major study involved looking at the current lateral system of shear walls and trying to determine if there was any way to optimize them and make them more efficient and cost effective. Another goal for the redesign involves improving the torsional resistance of the shear walls. This is an important criterion because it is known that the building did not initially meet the torsional acceleration standards when the building was first tested by a wind tunnel test, so additional slab thickness was added to the top floor in order to act as a pendulum.

The shear walls will be analyzed and designed using ETABS. The designs will meet the standards of ACI318-05. The ETABS design results will be verified using a spreadsheet to check the shear walls strengths. They will also be compared to the values used in the original design to make sure the model is acting correctly. In order to save time in the analyzing process, only the shear walls will be modeled. If 30 stories of slab were required to be meshed and included in the analysis, the results would take several hours to be calculated. Instead, the gravity loads on the shear walls will be added manually with their values coming from the RAM Concept models done previously.

The wind loads used in the design will be taken from the results of the wind tunnel test. These are the values that were used in the original design and in order to get consistent results they should be used again. The wind tunnel test provided static loads to be applied at each floor including Wind-X, Wind-Y, and Torsion. It also provided 24 recommended combinations of the loads. These values can be seen below the lateral loads summary.

The seismic loads were calculated using ASCE7-05. It was found that the wind loads clearly control the design. The calculated loads are shown below.

Lateral Loads Summary					
		Wind			Seismic
Floor	Floor Height (ft)	East-West Direction (kip)	North-South Direction (kip)	Torsion (ft-kip)	Force (kip)
Building Top	318	2	1	20	0
BULKHEAD	315	14	9	118	4
EMR FLOOR	304	27	18	353	20
ROOF	294	43	36	939	45
29	282	49	41	999	31
28	272	45	38	952	28
27	261	42	36	931	26
26	251	47	40	1,290	23
25	240	46	38	1,250	22
24	231	45	36	1,220	20
23	221	45	36	1,180	18
22	211	45	35	1,140	16
21	201	45	34	1,100	14
20	192	45	34	1,050	12
19	182	43	32	9,980	11
18	172	41	30	938	9
17	162	40	29	880	8
16	153	38	27	820	7
15	143	38	26	760	6
14	132	36	25	691	5
13	123	35	23	637	4
12	113	34	21	560	3
11	103	33	20	506	3
10	93	32	18	449	2
9	83	32	17	396	2
8	74	30	17	394	1
7	64	28	18	393	1
6	54	26	19	402	1
5	45	24	19	427	0
4	35	21	15	520	0
3	25	19	12	674	0
2	14	33	7	607	0
LOBBY	0	21	2	132	0
Total Base Shear (kip)		1,140	810	0	340
Total Base Moment (kip-ft)		194,000	149,000	23,700	80,500

Wind Load Combinations

Load Case	X	Y	T
1	0.60	0.60	0.25
2	0.40	1.00	0.30
3	0.40	0.40	0.90
4	0.60	0.60	-0.25
5	0.40	1.00	-0.30
6	0.40	0.40	-1.00
7	0.60	-0.40	0.30
8	0.40	-0.75	0.30
9	0.40	-0.40	0.90
10	0.60	-0.40	-0.25
11	0.40	-0.75	-0.25
12	0.40	-0.40	-1.00
13	-1.00	0.40	0.25
14	-0.40	1.00	0.30
15	-0.40	0.40	0.90
16	-1.00	0.45	-0.25
17	-0.40	1.00	-0.30
18	-0.40	0.45	-1.00
19	-1.00	-0.45	0.25
20	-0.55	-0.75	0.30
21	-0.40	-0.45	0.90
22	-1.00	-0.45	-0.25
23	-0.55	-0.75	-0.25
24	-0.40	-0.45	-1.00

The design load cases recommended by ACI318-05 are:

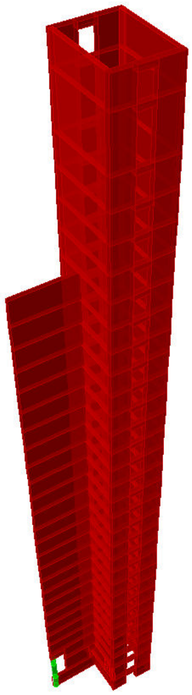
1.4D
1.2D + 1.6L + 0.5S
1.2D + 1.6S + (1.0L or 0.8W)
1.2D + 1.6W + 1.0L + 0.5S
1.2D + 1.0E + 1.0L + 0.2S
0.9D + 1.6W
0.9D + 1.0E

All of the combinations that included wind loads were inputted as the 24 different combinations recommended by the wind tunnel test.

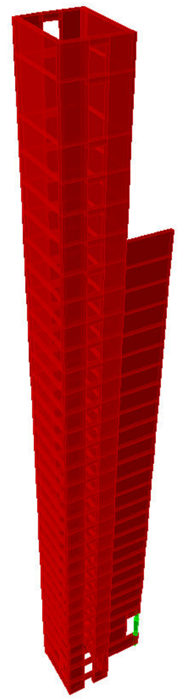
The shear walls will be designed to meet certain serviceability Issues. These include meeting the standard for story drift of H/400 as well as keeping the acceleration under the recommended values of 15 milli-g linear acceleration and 3 milli-rad/sec torsional acceleration. Since the accelerations cannot be accurately predicted without a wind tunnel test, the goal will be to meet or improve the deflections of the current design, which should correspond with an equivalent or improved acceleration value.

Current Shear Wall Design:

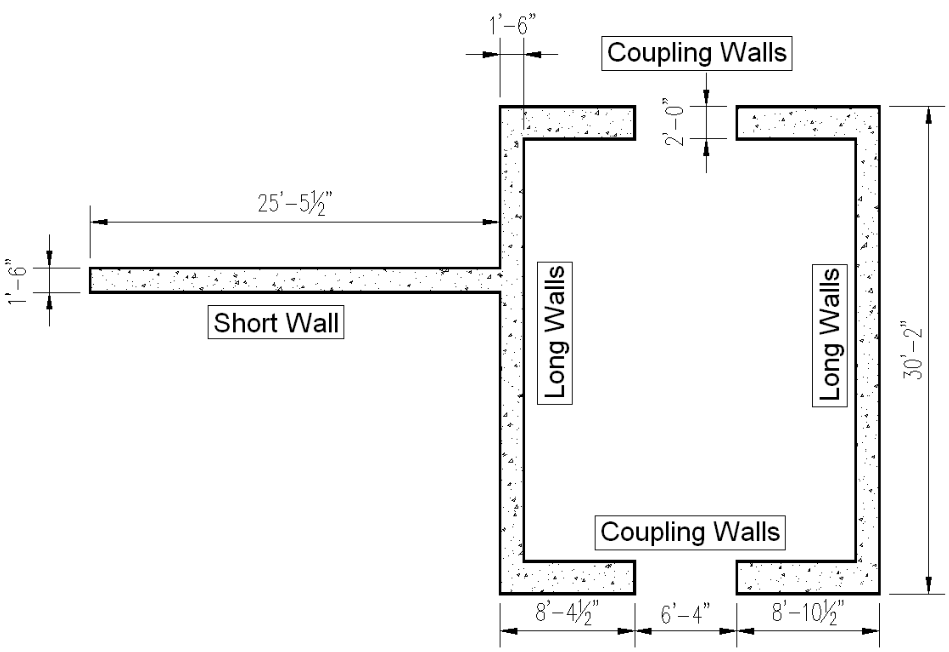
The layout of the original shear walls can be seen below. Note the wall labels on the plan because they will be referenced in the following pages.



Original Shear Walls
Viewed From South West



Original Shear Walls
Viewed From North East



Original Shear Walls Plan

The existing shear walls consist of a central core connected with a 2-foot deep link beam above the door openings. There is an additional wall coming off the core that only goes 25 floors up the building. It should also be noted that the concrete strength drops from 8,000 psi to 6,000 psi at the 14^{th} floor.

The shear walls were analyzed and designed using ETABS as discussed previously. The amount of rebar required matched closely to the values used in the original design. It was found that on some floors, however, there was slightly more rebar required than was used in the original design. This variance may be due to some additional dead loads that were not added to the model such as the weight of pumps, cooling towers, and other mechanical equipment. Since the walls tend to be tension controlled, missing some dead load would result in a larger amount of rebar required. The values only vary by about single bar sizes so they are close enough for this analysis. **The new rebar requirements from the ETABS model will be used for comparison values in order to keep everything consistent.**

The deflections calculated are listed on the following page. It was found that the shear walls had a max deflection of 3.80" in the Y-direction and 3.32" in the X-direction. These values correspond with a building deflection of H/1004 and H/1149, which is significantly better than the recommended value of H/400. This value is exceeded in order to keep acceleration values within acceptable limits. It is also important because the glass cladding system has low tolerances to deflections. The max story drift was H/444, but this occurred on one of the mechanical levels. The highest story drift for a floor with cladding was found to be H/759, which again exceeds standards by a significant amount.

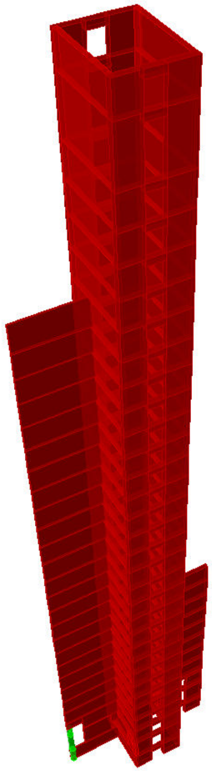
ETABS determined the center of rigidity to be approximately 38" East and 27" North of the recommended point of application for the wind loads. This will creates some torsional irregularity, but not to an excessive amount. For example, on the 26^{th} floor where the wind loads are near their maximum, the amount of torsion from irregularities will be 233 ft-kip. The amount of torsion from the wind is expected to be 8,035 ft-kip, so it can be seen that the torsion from wind is much more significant than that from torsional irregularity.

The plans for the rebar are shown on the following page. The values listed are for bars on each side of the wall spaced at 12" o/c. The sizes of the bars were chosen based on the percent longitudinal reinforcing values taken from ETABS as well as the in^2/ft shear reinforcing values. They were also selected trying to minimize the number of times that the bars would have to be spliced, so it was decided that every bar should be at least 3 stories high.

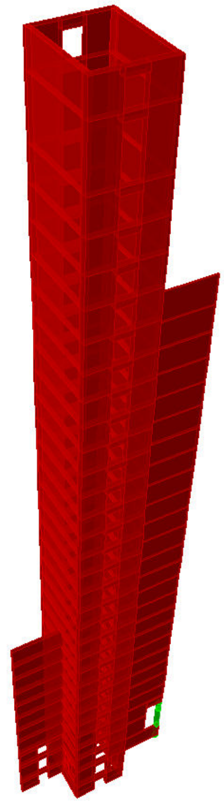
Original Wind Deflections					Original Rebar Plans		
Floor	**Floor Height (ft)**	**Story X Deflection (in)**	**Story Y Deflection (in)**	**Story Torsional Deflection (milli-rad)**	**Short Wall**	**Long Walls**	**Coupled Walls**
Building Top	318	0.115	0.151	0.010			
EMR FLOOR	304	0.210	0.271	0.020		# 5	# 6
ROOF	294	0.130	0.166	0.020		"	"
29	282	0.126	0.158	0.020		"	"
28	272	0.128	0.158	0.030		"	"
27	261	0.129	0.151	0.030		"	"
26	251	0.128	0.152	0.040		"	"
25	240	0.118	0.146	0.040	# 5	"	"
24	231	0.118	0.145	0.050	"	"	"
23	221	0.118	0.145	0.050	"	"	"
22	211	0.118	0.143	0.060	"	"	"
21	201	0.118	0.142	0.070	"	"	"
20	192	0.117	0.140	0.080	"	"	"
19	182	0.116	0.137	0.100	"	"	"
18	172	0.115	0.135	0.110	"	"	"
17	162	0.114	0.131	0.120	"	"	"
16	153	0.112	0.128	0.120	"	"	# 7
15	143	0.118	0.133	0.140	"	"	"
14	132	0.106	0.119	0.120	"	"	"
13	123	0.103	0.114	0.120	"	# 6	"
12	113	0.100	0.109	0.120	"	"	"
11	103	0.097	0.104	0.130	"	"	"
10	93	0.093	0.098	0.130	#7	"	# 9
9	83	0.089	0.091	0.120	"	# 8	"
8	74	0.084	0.084	0.130	"	"	"
7	64	0.079	0.077	0.130	"	"	"
6	54	0.073	0.069	0.120	"	"	"
5	45	0.067	0.060	0.120	#9	"	# 10
4	35	0.060	0.049	0.120	"	# 10	"
3	25	0.059	0.044	0.130	"	"	"
2	14	0.054	0.041	0.160	# 10	"	# 11
LOBBY	0	0.011	0.011	0.060	"	"	"
BASEMENT	-10				"	"	"
Max Story Deflection		H/571	H/444				
Total Deflection		3.32”	3.80”	2.82 milli-rad			
Total Deflection		H/1149	H/1004				
				Shear Rebar	# 5	# 7	# 6
				Total Rebar (ft^3)	77	286	150
				Total Concrete (ft^3)	9,750	27,825	24,110
				Total Formwork (ft^2)	13,000	37,100	12,055

Alternate Shear Wall Design:

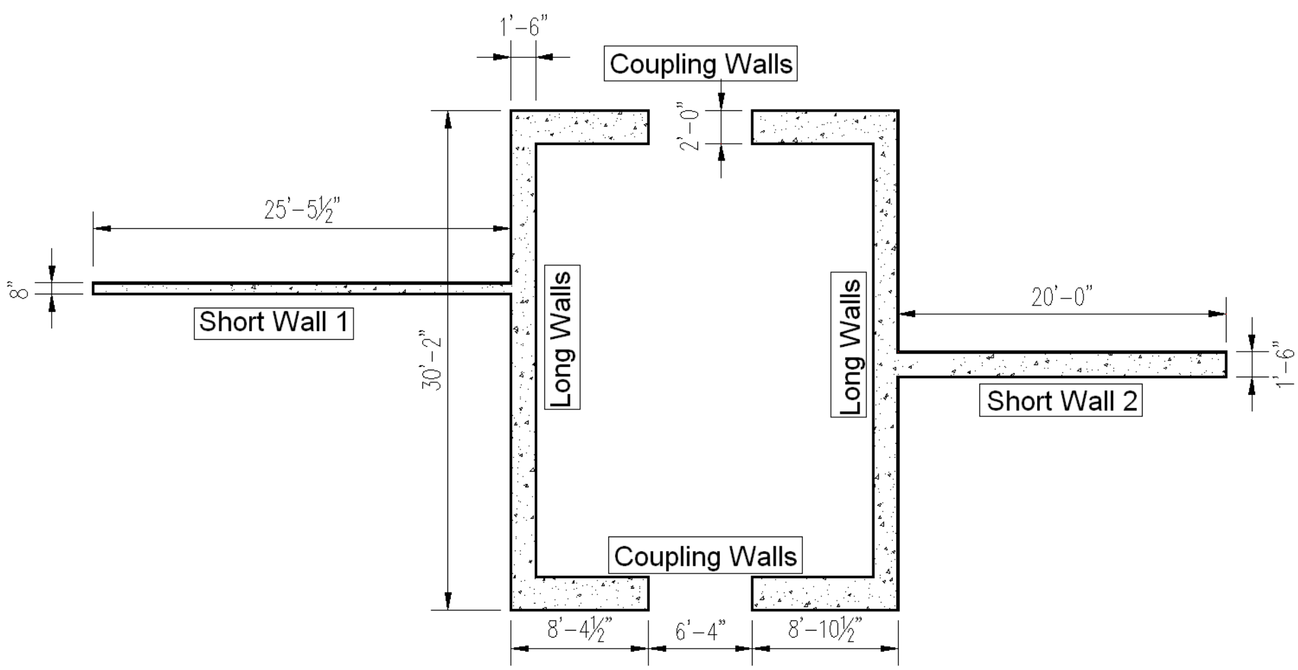
The layout of the new shear walls can be seen below.



New Shear Walls
Viewed From South West



New Shear Walls
Viewed From North East

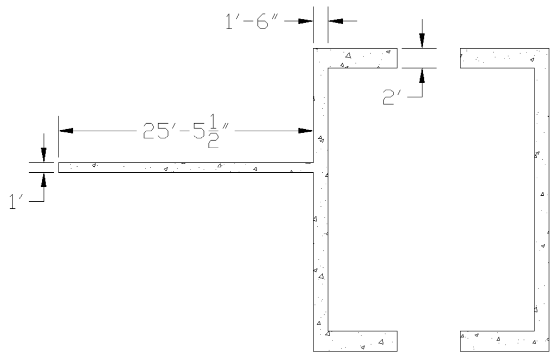


New Shear Walls Plan

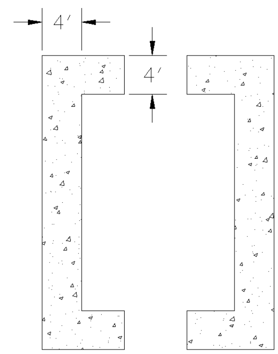
Several different layouts were modeled and analyzed in ETABS in order to come up with the most efficient alternate design. These studies included looking at modifying the original wall widths while deepening the link beam depth (Modified Original), a core only design (Core Only), adding an additional wall on the East side of the core (Additional Wall), shortening the original wall that comes from the core (Shortened Wall), and removing the original wall and adding the new wall on the opposite side (Opposite Wall Only). (Shortening a wall is not typically an effective redesign, but there was some loss in stiffness in the original layout due to a penetration in that wall at the lower levels. The shortened wall avoids that penetration.)

All of these designs were modeled and sized to meet the deflection results from the original design. The plans for these layouts are shown below.

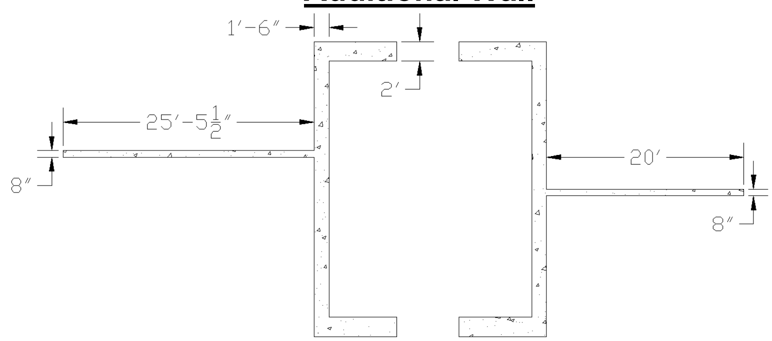
Modified Original



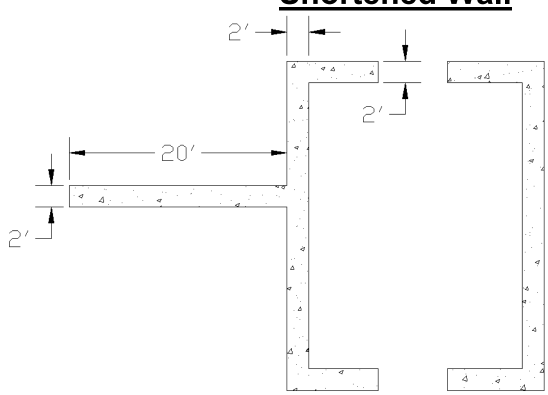
Core Only



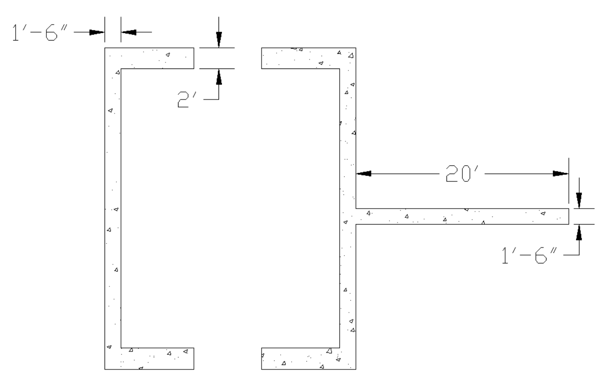
Additional Wall



Shortened Wall



Opposite Wall Only



A cost estimate was then produced by adding all the concrete, formwork, and rebar for the shear wall layouts at the ground level. The summary of these findings is listed below. It was found that the layout that produced the best results was the “Additional Wall” layout, so this was the layout that was studied in more detail.

Alternate Layouts Initial Estimate

	X Deflection (in)	Y Deflection (in)	Torsional Deflection (milli-rad)	Estimate Cost	Rank	Price Difference
Original	3.32	3.80	2.82	$24,463	4	$1,882
Modified Original	3.70	3.79	1.99	$23,555	2	$975
Core Only	3.29	2.08	1.54	$32,492	6	$9,911
Additional Wall	**3.37**	**3.82**	**2.83**	**$22,580**	**1**	**$0**
Shortened Wall	3.64	3.49	2.67	$27,661	5	$5,081
Opposite Wall Only	3.85	3.85	2.82	$24,070	3	$1,490

Another piece of information discovered from the initial studies was how much adding to the link beam depth helped the building in terms of torsion. The “Modified Original” layout included deepening the link beam by 6”, which resulted in a torsional deflection of 70% of the original value. The link beam is currently 24” deep which puts the bottom of it at 7’-9” above the floor. The IBC allows for a minimum exit ceiling height of 7’-6” above the floor, so that only allows for 3” of finishes to be added to the beam. However, the building is to be designed to the New York City Code which allows for a minimum exit ceiling height of 7’-0” as long as the ceiling is that height for less than 25% of the total area, and it doesn’t block exit signs. This means that there is a potential to increase the link beam to a depth of 30” with still 3” to spare. It was decided to increase the depth of the link beam to 27”, which would provide a significant increase in torsional resistance while not invading into the ceiling space too much.

The deflections for the final shear wall layout calculated are listed on the following page. It was found that the shear walls had a max deflection of 3.81” in the Y-direction and 3.13” in the X-direction. The deflection in the Y-direction remained the same while the deflection in the X-direction decreased to 94% of its original value. A decrease in the X-direction deflection is important because the building is longer in the North-South direction. This means that accelerations from torsion are greatest in the far North and South sides of the building, which will be additive to the accelerations from the X-direction. The story drift values remained nearly the same.

The deflection from torsion was found to be 2.49 milli-rad, which is 88% of the original value of 2.82 milli-rad. This decrease should cut down on the torsional acceleration of the building to an extent. The exact amount will be unknown, though, until a wind tunnel test is performed.

The Center of Rigidity for the new layout improved slightly by moving about 5” South which is closer to the point of application for wind forces, but this again is not a significant issue.

The plans for the rebar are shown on the following page. The values listed are for bars on each side of the wall spaced at 12” o/c, except for the “Short Wall 2”. It was found to be more cost effective to use only a single curtain of rebar. This will allow for the reinforcing to be decreased to a value closer to the minimum reinforcing percentage required for the wall without having to increase the bar spacing.

New Wind Deflections / New Rebar Plans

Floor	Floor Height (ft)	Story X Deflection (in)	Story Y Deflection (in)	Story Torsional Deflection (milli-rad)	Short Wall 1	Short Wall 2	Long Walls	Coupled Walls
Building Top	318	0.116	0.151	0.000				
EMR FLOOR	304	0.212	0.272	0.020			# 5	# 6
ROOF	294	0.131	0.166	0.010			"	"
29	282	0.127	0.159	0.020			"	"
28	272	0.128	0.159	0.020			"	"
27	261	0.129	0.155	0.030			"	"
26	251	0.129	0.155	0.030			"	"
25	240	0.119	0.146	0.040	# 5		"	"
24	231	0.119	0.146	0.040	"		"	"
23	221	0.119	0.145	0.040	"		"	"
22	211	0.118	0.143	0.050	"		"	"
21	201	0.118	0.142	0.060	"		"	"
20	192	0.117	0.140	0.070	"		"	"
19	182	0.115	0.137	0.090	"		"	"
18	172	0.114	0.135	0.100	"		"	"
17	162	0.111	0.132	0.100	"		"	"
16	153	0.108	0.128	0.110	"		"	# 7
15	143	0.112	0.133	0.120	"		"	"
14	132	0.099	0.119	0.110	# 7		"	"
13	123	0.094	0.114	0.110	"		"	"
12	113	0.088	0.111	0.110	"		"	"
11	103	0.077	0.105	0.110	"	# 6	"	"
10	93	0.072	0.097	0.110	"	"	"	#9
9	83	0.069	0.091	0.110	"	# 8	"	"
8	74	0.066	0.084	0.110	"	"	"	"
7	64	0.062	0.076	0.110	"	# 11	"	"
6	54	0.057	0.068	0.120	"	"	"	"
5	45	0.052	0.059	0.100	# 9	"	"	#10
4	35	0.046	0.049	0.110	"	"	"	"
3	25	0.048	0.042	0.120	"	# 14	"	"
2	14	0.048	0.041	0.150	# 11	"	# 7	# 11
LOBBY	0	0.012	0.011	0.060	"	"	"	"
					"	"	"	"
Max Story Deflection		L/566	L/441					
Total Deflection		3.13"	3.81"	2.49 milli-rad				
Total Deflection		L/1218	L/1002					
				Shear Rebar	# 5	# 5	# 7	# 6
				Total Rebar (ft^3)	43	59	179	150
				Total Concrete (ft^3)	4,333	3,390	27,825	24,290
				Total Formwork (ft^2)	13,000	4,520	37,100	12,055

* All walls are reinforced on both sides except Short Wall 1

Comparison/Conclusions:

There are two main factors that will determine which system should be chosen. These include serviceability and cost. In terms of serviceability, the new shear walls were designed to at least meet the same standards as the original design. The new walls did have 94% of the x-deflection of the original design. In terms of torsion, the new walls had 88% of the torsional deflection of the original design. Both of these decreases should lead to less accelerations in the building.

In order to have a better understanding of cost, an estimate for the materials and labor was made based on the unit costs of the rebar, rebar splices, concrete, and formwork. The final values are listed below.

Shear Wall Cost Comparison				
	Original Design		New Design	
	Amt.	**Cost**	**Amt.**	**Cost**
Total Rebar (ton)	126	$182,248	106	$151,369
Total Concrete (CY)	2183	$334,858	2113	$328,170
Total Formwork (SFCA)	62155	$121,202	66675	$130,016
Total		$638,308		$609,555

The new design performed better in terms of cost for a number of reasons. The shear walls can be compared to a cantilever beam. If you were able to have a varying stiffness for the beam, the most efficient design would be to have the stiffness decrease with distance from the support. This is because any rotation that happens to the beam gets integrated and affects the deflection throughout the entire beam. A comparison of the deflection of a constant stiffness vs. a varying stiffness is located in the appendix. It was found that a beam with a stiffness that varies by x^2 will have a deflection of 66.7% of the deflection of a beam with constant stiffness.

The new design allows for the stiffness of the wall to drop off at more points than in the original design. The stiffness drops when the “Short Wall 2” ends, when the concrete strength drops, and when the “Short Wall 1” ends. The stiffness of the gross sections can be seen on the chart below. The calculations can be found in the appendix.

The decrease in required total stiffness as well as the added depth to the shear walls by the addition of “Short Wall 2” led to savings in the amount of concrete required in order to reach the same deflection amounts as the original design.

Shear Wall Stiffnesses

Floor	Floor Height (ft)	Original Stiffness (10^{10} k-ft^2)	New Stiffness (10^{10}k-ft^2)	Floor	Floor Height (ft)	Original Stiffness (10^{10} k-ft^2)	New Stiffness (10^{10}k-ft^2)
Building Top	318					2.23	2.22
EMR FLOOR	304	0.89	0.89	15	143	2.23	2.22
ROOF	294	0.89	0.89	14	132	2.23	2.22
29	282	0.89	0.89	13	123	2.57	2.71
28	272	0.89	0.89	12	113	2.57	2.71
27	261	0.89	0.89	11	103	2.57	2.71
26	251	0.89	0.89	10	93	2.57	3.44
25	240	0.89	0.89	9	83	2.57	3.44
24	231	2.23	2.22	8	74	2.57	3.44
23	221	2.23	2.22	7	64	2.57	3.44
22	211	2.23	2.22	6	54	2.57	3.44
21	201	2.23	2.22	5	45	2.57	3.44
20	192	2.23	2.22	4	35	2.57	3.44
19	182	2.23	2.22	3	25	2.57	3.44
18	172	2.23	2.22	2	14	2.57	3.44
17	162	2.23	2.22	LOBBY	0	2.57	3.44
16	153	2.23	2.22	BASEMENT	-10	2.57	3.44

The new design was also less expensive because the shape at the lower levels is more balanced. In the original design, both of the “Long Walls” were designed to contain the exact same amount of reinforcing. In actuality, the “Long Wall” on the east side had much more tension that the one on the west side. This was because the entire core acts compositely and the “Long Wall” on the east side is at the extreme end. By adding the “Short Wall 2”, the stresses on the “Long Walls” became much closer to the same, which saved significantly on rebar costs.

The costs of other components must be also be considered. The columns will be affected by the new shear wall stiffnesses. Since the columns are formed directly into the floor, they deflect as much as the floor drifts. Most of the moment that they experience is due to these drifts. The new stiffer shear walls should result in some reduction of moment in the columns. It is possible that some columns may be bumped to lower sizes due to this, but the difference isn’t enough to expect the majority of columns to shrink.

It is important to remember that there are additional costs beyond just the cost of material and labor. The time of construction will play a major role in how much the actually costs the owner. The longer it takes to construct, the longer the owner will have to wait to finalize all of the condominium sales, so additional money will be spent in the form of interest on loans. When looking at the estimated time of construction based on RSMeans, it was found that the total time to construct the shear walls is approximately the same. The only real difference is how long certain floors take to construct. The lower levels on the new layout take longer to construct because there is an additional wall. The middle levels, however, take less time to construct because the “Short Wall 1” is about half the size of the original and requires about half the amount of rebar. This results in the total construction time being about the same, making schedule not an issue.

Based on the analysis performed and all of the factors discussed above, it can be determined that the new shear wall layout is a better option. It performs better in terms of deflection with an X-deflection of 94% of the original and a torsional deflection of 88% of the original. The cost estimate found the new shear wall layout to be about 5% cheaper than the original layout, and finally it was found that the schedule was relatively unaffected.

Breadth Study – Construction Cost and Schedule

Introduction:

All of the recommended changes to the floor slab and shear walls will also require changes in the cost and time the building takes to be constructed, as well as create new constructability issues. These topics must be discussed in order to gain a better understanding of all the effects these changes will have.

All values for time and schedule are coming from RSMeans Building Cost Data. For an accurate total cost, the final values should be multiplied by 1.281 in order to adjust for being built in Brooklyn, New York. This increase is because the labor cost is Brooklyn is 159.1% of the national average. Values do not need to be adjusted for inflation because the project is still in the construction phase.

Floor Slab:

The floor slabs for the building are being changed from a mild reinforcing flat plate slab to a post-tensioned slab. A breakdown of the cost and construction time is shown below.

Detailed Floor Estimates

Post-Tensioned Slab (3rd-25th Floors) (7589 SF)

	Amount	Unit	Unit Cost	Daily Output	Total	Time
Concrete Mix and Placing	164	CY	138	110	$22,632	1.49
Post-Tensioning Cost	4273	Lbs	1.49	1275	$6,367	3.35
Formwork Costs	7589	SF	4.36	560	$33,088	13.55
Formwork Edge Cost	360	LF	2.33	500	$839	0.72
Mild Steel Reinforcing Costs	2.8	Tons	1340	2.9	$3,752	0.97
				Total	$66,678	20.08
				Cost/SF	$8.79	

Mild Reinforced Slab (3rd-25th Floors) (7589 SF)

	Amount	Unit	Unit Cost	Daily Output	Total	Time
Concrete Mix and Placing	185.2	CY	138	110	$25,558	1.68
Formwork Cost	7589	SF	4.36	560	$33,088	13.55
Formwork Edge Cost	360	LF	2.33	500	$839	0.72
Mild Steel Reinforcing Costs	17.85	Tons	1340	2.9	$23,919	6.16
				Total	$83,403	22.11
				Cost/SF	$10.99	

Post-Tensioned Slab (26th-Roof) (6199 SF)						
	Amount	**Unit**	**Unit Cost**	**Daily Output**	**Total**	**Time**
Concrete Mix and Placing	134	CY	138	110	$18,492	1.22
Post-Tensioning Cost	2921	Lbs	1.49	1275	$4,352	2.29
Formwork Costs	6199	SF	4.36	560	$27,028	11.07
Formwork Edge Cost	346	LF	2.33	500	$806	0.69
Mild Steel Reinforcing Costs	2.3	Tons	1340	2.9	$3,082	0.79
				Total	$53,760	16.06
				Cost/SF	$8.67	

Mild Reinforced (26th-Roof) (6199 SF)						
	Amount	**Unit**	**Unit Cost**	**Daily Output**	**Total**	**Time**
Concrete Mix and Placing	153.1	CY	138	110	$21,128	1.39
Formwork Costs	6199	SF	4.36	560	$27,028	11.07
Formwork Edge Cost	346	LF	2.33	500	$806	0.69
Mild Steel Reinforcing Costs	14.94	Tons	1340	2.9	$20,020	5.15
				Total	$68,981	18.31
				Cost/SF	$11.13	

It can be seen from the summary above that the major savings in the post-tensioned design come from the reduction in mild reinforcement cost ($15,725 and $15,221). There is also savings from the reduction in concrete ($2,926 and $2,636). The additional expense comes from the post-tensioning tendons ($6,367 and $4,352). This results in a total of savings of $16,725 for the 3rd-25th Floor and $15,221 for the 26th- Roof. Adjusting these savings for Brooklyn, New York gives a savings of $21,425 and $19,498.

The reason for the large savings in mild reinforcement is because not all of the reinforcement is required for strength. The original slab is reinforced with a top and bottom mat in both direction of #5 bars @ 12"o/c. This high level of reinforcement is only required at certain points in the slab, but in order to keep the plans more uniform for easy construction, the reinforcement is continuously placed. The post-tensioned system also required uniform mild reinforcement, but only a bottom mat of #4 bars @ 24"o/c. This resulted in less of the slab being over reinforced and thus a reduction in mild reinforcement cost.

Besides the cost of materials and labor, the construction time must be considered. The major difference in construction process from the original design and the new design is the requirement of post-tensioned tendons. Besides this, the systems are essentially the same and will require the same process. When adding up the expected time of construction for the two systems, it was found that the post-tensioned system would be able to be constructed slightly faster than the original system. Again, this reduction is due to less mild reinforcement being required than in the original system.

Another factor that must be discussed from this is the potential to reduce the overall building height. Because the slab is being reduced by 7”, there is the potential to decrease each floor by 1”, which would result in a decrease in building height of 30”. The cost savings from this comes from the reduction in required cladding. An estimate from RSMeans says that the cost of cladding is $40/SF. This 30” reduction in height results in a savings of 900 SF of cladding which is a savings of $36,000. There is also savings in duct length, but that only came to a total of about $2,500. The savings from the height reduction will have to be decided by the owner whether or not he wants to take advantage of it. Since it was already decided to add 3” to the link beams in the shear wall, taking this reduction would result in the ceiling height under these beams being at a height of 7’-4”. This still meets code, but it is 4” lower than originally designed.

Shear Walls:

The shear walls for the building are still going to remain the same kind of system, so few changes to the construction method and schedule are required. A breakdown of the cost and construction time is shown below. The details of the estimate can be found in the appendix.

Shear Wall Estimate		
Original Shear Walls		
Item	**Total Cost**	**Construction Time (Days)**
Concrete	$334,858	20.8
Formwork	$121,202	49.3
Rebar	$182,248	40.0
Total	$638,308	110.1
New Shear Walls		
Item	**Total Cost**	**Construction Time (Days)**
Concrete	$328,170	20.4
Formwork	$130,016	52.9
Rebar	$150,459	33.3
Total	$608,645	106.5

It can be seen from the summary above that the major savings in the new design come from the reduction in concrete ($6,688) and rebar costs ($31,789), but there is the additional expense of formwork ($8,814), which leaves a savings of $29,663. Adjusting this for Brooklyn, New York gives a savings of $37,998.

Beyond cost, the constructability and schedule issues must be discussed. The construction time is not constant throughout the height of the building due to less walls being required toward the top. A breakdown of the construction time at different points in the construction process is shown below.

Wall Schedule

Original Shear Walls (Base Floor)

Amount	Time
Placing Concrete	0.68
Formwork	2.01
Mild Rebar	1.15
Total	**3.83**

Original Shear Walls (12th Floor)

Amount	Time
Placing Concrete	0.68
Formwork	2.01
Mild Rebar	0.67
Total	**3.35**

New Shear Walls (Base Floor)

Amount	Time
Placing Concrete	0.72
Formwork	2.33
Mild Rebar	1.49
Total	**4.54**

New Shear Walls (12th Floor)

Amount	Time
Placing Concrete	0.61
Formwork	2.01
Mild Rebar	0.58
Total	**3.20**

Original Shear Walls (Roof)

Amount	Time
Placing Concrete	0.54
Formwork	1.60
Mild Rebar	0.21
Total	**2.35**

New Shear Walls (Roof)

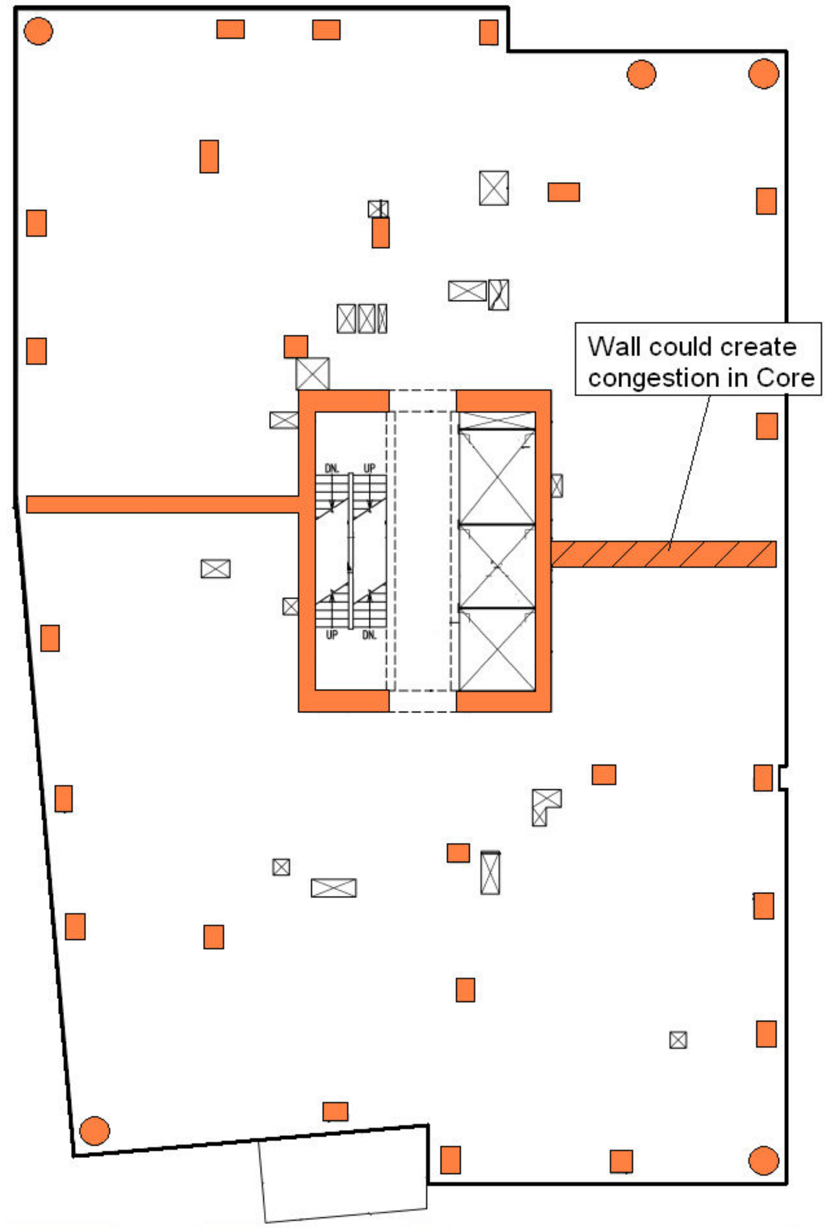
Amount	Time
Placing Concrete	0.54
Formwork	1.60
Mild Rebar	0.21
Total	**2.35**

The chart above shows that the new shear walls will take longer to construct at the base of the building than the original shear walls. On the middle floors, though, the new design will take less time. Finally, the top of the building will take the same amount of time.

It is important to construct the lower floors quickly in order to allow the other trades to come in and begin working. The difference in time, though, is not enough to make a significant difference. The most important thing is that they complete the entire structure as quick as possible and both options require about the same total time (3 days saved on new design).

A potential issue with the new layout is congestion. On the lower levels with the new wall added, the only passage from the North to South sides of the building will be through the central core of the building. Before the entire East side of the building was available to transport large material and equipment. This will be an issue that would need to be discussed with the contractor in order to determine how much of problem this would be.

Typical Floor Plan (Ground-11th Floor)



Breadth Study – Mechanical Risers

Introduction:

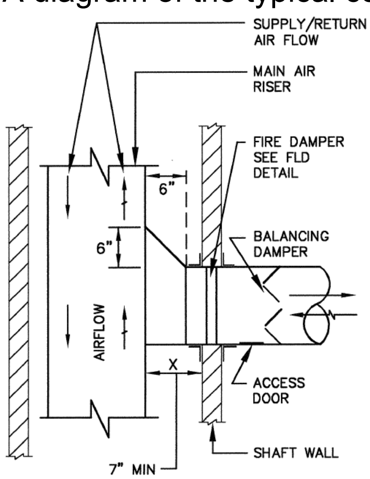
The engineering trade that has the most spatial conflicts with the structural design of buildings is the HVAC system. Ductwork is constantly penetrating slabs and competing for space with beams. This makes it important to have a good understanding of what goes in to deciding what size to make ducts and how flexible that size is. That is why it was decided to study the typical exhaust riser in the building to see how much duct size really matters.

In terms of the proposed redesign of the slab, the duct size is very important because all of the tendons that intersect with openings from the risers will need to be bent around them. The conflicts with openings are going to be more critical than in the original design of mild reinforcement, therefore it will be determined how much more expensive it will be to have the duct size decreased.

Existing System:

Currently there are 21 duct risers that run almost the entire height of the building. The major risers include 11 toilet exhaust risers, 7 dryer exhaust risers, 1 kitchen exhaust riser, and 1 trash room exhaust riser. They remove 50 CFM, 160 CFM, 120 CFM, and 100 CFM per floor respectively. There is also one conditioned air supply that supplies 250 CFM per floor to the elevator lobbies. The sizes of the exhaust ducts are all fairly similar starting with an initial size of around 14x26, then decreasing in size about every 5 floors, and finally ending in a size around 8x6.

A diagram of the typical converging ducts at each floor is shown below.



The air is going to be forced through the risers by fans placed on the roof of the building. The most common type is Model SWB by Greenheck, which is a Centrifugal Utility Fan.

Riser Redesign:

The typical duct that was studied in more detail was the trash room exhaust. It was decided to redesign the ducts with 2/3 of the equivalent area of the original system and also 3/2 of the equivalent area of the original system. Looking at these two options should give a good understanding of how much duct size affects the required fan size.

The total pressure loss in the duct was determined by converting the rectangular ducts into their equivalent circular sizes from the ASHRAE Handbook. The velocity was then determined at each level. The pressure losses were than calculated by multiplying the loss coefficient by the velocity pressure. The loss coefficients came from the book HVAC Systems Duct Design. The loss from friction in the ducts came from the ASHRAE Handbook.

The pressure losses that were added included friction in ducts, converging ducts, transition losses, and elbow losses. Once the final pressure was known for the existing system, this became the target for the redesigned systems. With the new static pressures known, new fans were chosen that would meet the design requirements.

The calculations are show on the following pages. The redesigned duct systems are listed with their new duct sizes as well as their new fan's RPM and static pressure.

Original Trash Room Exhaust Pressure Losses

Fan Details	CFM	Ext. Stat.	rpm	model	BHP	Motor HP
	2800	2.0	1588	SWB-216-20	1.39	2

Branch Details	Exit Duct Size	Eq. Size (in)	Eq. Area (ft^2)	CFM
	8x6	7.5	0.31	100

Pressure Loss Calcs.

Section	Size (in)	Length (ft)	CFM	Eq. Size (in)	Eq. Area (ft^2)	Velocity (ft/min.)	DeltaP/100ft (in. w.g.)	Duct Loss (in. w.g.)	Qb/Qc	Converging Loss (in. w.g.)	Ao/A1	Transition Loss (in. w.g.)	Elbow Loss (in. w.g.)
Roof	14x26	18	2800	20.6	2.31	1210	0.095	0.0171	0.000	0.0136			
29	14x26	11	2700	20.6	2.31	1167	0.09	0.0099	0.000	0.0126			
28	14x26	10.5	2600	20.6	2.31	1123	0.086	0.0090	0.000	0.0116			
27	14x26	10.5	2500	20.6	2.31	1080	0.081	0.0085	0.000	0.0107			
26	14x26	10.5	2400	20.6	2.31	1037	0.077	0.0081	0.000	0.0099			
25	14x26	10.5	2300	20.6	2.31	994	0.072	0.0076	0.000	0.0090			
24	14x26	9.75	2200	20.6	2.31	951	0.068	0.0066	0.000	0.0111			
Duct Trans											1.16	0.0140	
23	14X22	9.75	2100	19.1	1.99	1055	0.095	0.0093	0.000	0.0101			
22	14X22	9.75	2000	19.1	1.99	1005	0.088	0.0086	0.000	0.0091			
21	14X22	9.75	1900	19.1	1.99	955	0.08	0.0078	0.000	0.0082			
20	14X22	9.75	1800	19.1	1.99	905	0.073	0.0071	0.000	0.0073			
19	14X22	9.75	1700	19.1	1.99	854	0.065	0.0063	0.000	0.0065			
18	14X22	9.75	1600	19.1	1.99	804	0.058	0.0057	0.000	0.0057			
17	14X22	9.75	1500	19.1	1.99	754	0.05	0.0049	0.000	0.0093			
Duct Trans											1.37	0.0201	
16	14X16	9.75	1400	16.3	1.45	966	0.08	0.0078	0.000	0.0080			
15	14X16	9.75	1300	16.3	1.45	897	0.071	0.0069	0.000	0.0068			
14	14X16	10.5	1200	16.3	1.45	828	0.062	0.0065	0.000	0.0057			
13	14X16	9.75	1100	16.3	1.45	759	0.053	0.0052	0.000	0.0048			
12	14X16	9.75	1000	16.3	1.45	690	0.044	0.0043	0.000	0.0038			
11	14X16	9.75	900	16.3	1.45	621	0.035	0.0034	0.000	0.0078			
Duct Trans											1.60	0.0218	
10	14X10	9.75	800	12.9	0.91	881	0.09	0.0088	0.000	0.0070			
9	14X10	9.75	700	12.9	0.91	771	0.075	0.0073	0.000	0.0057			
8	14X10	9.75	600	12.9	0.91	661	0.059	0.0058	0.001	0.0044			
7	14X10	9.75	500	12.9	0.91	551	0.044	0.0043	0.001	0.0033			
6	14X10	9.75	400	12.9	0.91	441	0.028	0.0027	0.001	0.0078			
Duct Trans											1.73	0.0135	
5	10X8	9.75	300	9.8	0.52	573	0.06	0.0059	0.001	0.0064			
Duct Trans											1.36	0.0112	
4	10X6	9.75	200	8.4	0.38	520	0.06	0.0059	0.002	0.0022			
3	10X6	11	100	8.4	0.38	260	0.017	0.0019					0.0051
							Total	0.1930		0.2083		0.0806	0.0051
							Total Loss	0.4869					
							Final Pressure	1.5131					

50% Larger Trash Room Exhaust Pressure Losses

Fan Details	CFM	Ext. Stat.	rpm	model	BHP	Motor HP
	2800	1.69	1509	SWB-216-20	1.22	2

Branch Details	Exit Duct Size	Eq. Size (in)	Eq. Area (ft^2)	CFM
	8x6	7.5	0.31	100

Pressure Loss Calcs.

Section	Size (in)	Length (ft)	CFM	Eq. Size (in)	Eq. Area (ft^2)	Velocity (ft/min.)	DeltaP/100ft (in. w.g.)	Duct Loss (in. w.g.)	Qb/Qc	Converging Loss (in. w.g.)	Ao/A1	Transition Loss (in. w.g.)	Elbow Loss (in. w.g.)
Roof	18x30	18	2800	25.2	3.46	808	0.034	0.0061	0.036	0.0061			
29	18x30	11	2700	25.2	3.46	780	0.032	0.0035	0.037	0.0040			
28	18x30	10.5	2600	27.5	4.12	630	0.03	0.0032	0.038	0.0037			
27	18x30	10.5	2500	27.5	4.12	606	0.028	0.0029	0.040	0.0034			
26	18x30	10.5	2400	27.5	4.12	582	0.025	0.0026	0.042	0.0031			
25	18x30	10.5	2300	27.5	4.12	558	0.023	0.0024	0.043	0.0028			
24	18x30	9.75	2200	27.5	4.12	533	0.021	0.0020	0.045	0.0048			
Duct Trans											1.37	0.0100	
23	18x26	9.75	2100	23.5	3.01	697	0.034	0.0033	0.048	0.0044			
22	18x26	9.75	2000	23.5	3.01	664	0.031	0.0030	0.050	0.0040			
21	18x26	9.75	1900	23.5	3.01	631	0.028	0.0027	0.053	0.0036			
20	18x26	9.75	1800	23.5	3.01	598	0.025	0.0024	0.056	0.0032			
19	18x26	9.75	1700	23.5	3.01	564	0.022	0.0021	0.059	0.0028			
18	18x26	9.75	1600	23.5	3.01	531	0.019	0.0019	0.063	0.0025			
17	18x26	9.75	1500	23.5	3.01	498	0.016	0.0016	0.067	0.0044			
Duct Trans											1.42	0.0099	
16	18x18	9.75	1400	19.7	2.12	661	0.04	0.0039	0.071	0.0038			
15	18x18	9.75	1300	19.7	2.12	614	0.034	0.0033	0.077	0.0032			
14	18x18	10.5	1200	19.7	2.12	567	0.028	0.0029	0.083	0.0027			
13	18x18	9.75	1100	19.7	2.12	520	0.023	0.0022	0.091	0.0022			
12	18x18	9.75	1000	19.7	2.12	472	0.017	0.0017	0.100	0.0018			
11	18x18	9.75	900	19.7	2.12	425	0.011	0.0011	0.111	0.0033			
Duct Trans											1.52	0.0088	
10	18x12	9.75	800	16	1.40	573	0.031	0.0030	0.125	0.0030			
9	18x12	9.75	700	16	1.40	501	0.023	0.0022	0.143	0.0024			
8	18x12	9.75	600	16	1.40	430	0.016	0.0016	0.167	0.0018			
7	18x12	9.75	500	16	1.40	358	0.008	0.0008	0.200	0.0014			
6	18x12	9.75	400	16	1.40	286	0	0.0000	0.250	0.0036			
Duct Trans											1.81	0.0063	
5	12x10	9.75	300	11.9	0.77	388	0	0.0000	0.333	0.0035			
Duct Trans											1.47	0.0068	
4	10x8	9.75	200	9.8	0.52	382	0	0.0000	0.500	0.0012			
3	10x8	11	100	9.8	0.52	191	0	0.0000					0.0027
							Total	0.0626		0.0864		0.0418	0.0027
							Total Loss	0.1936					
							Final Pressure	1.4964					

33% Smaller Trash Room Exhaust Pressure Losses

Fan Details	CFM	Ext. Stat.	rpm	model	BHP	Motor HP
	2800	2.61	1747	SWB-216-20	1.89	2

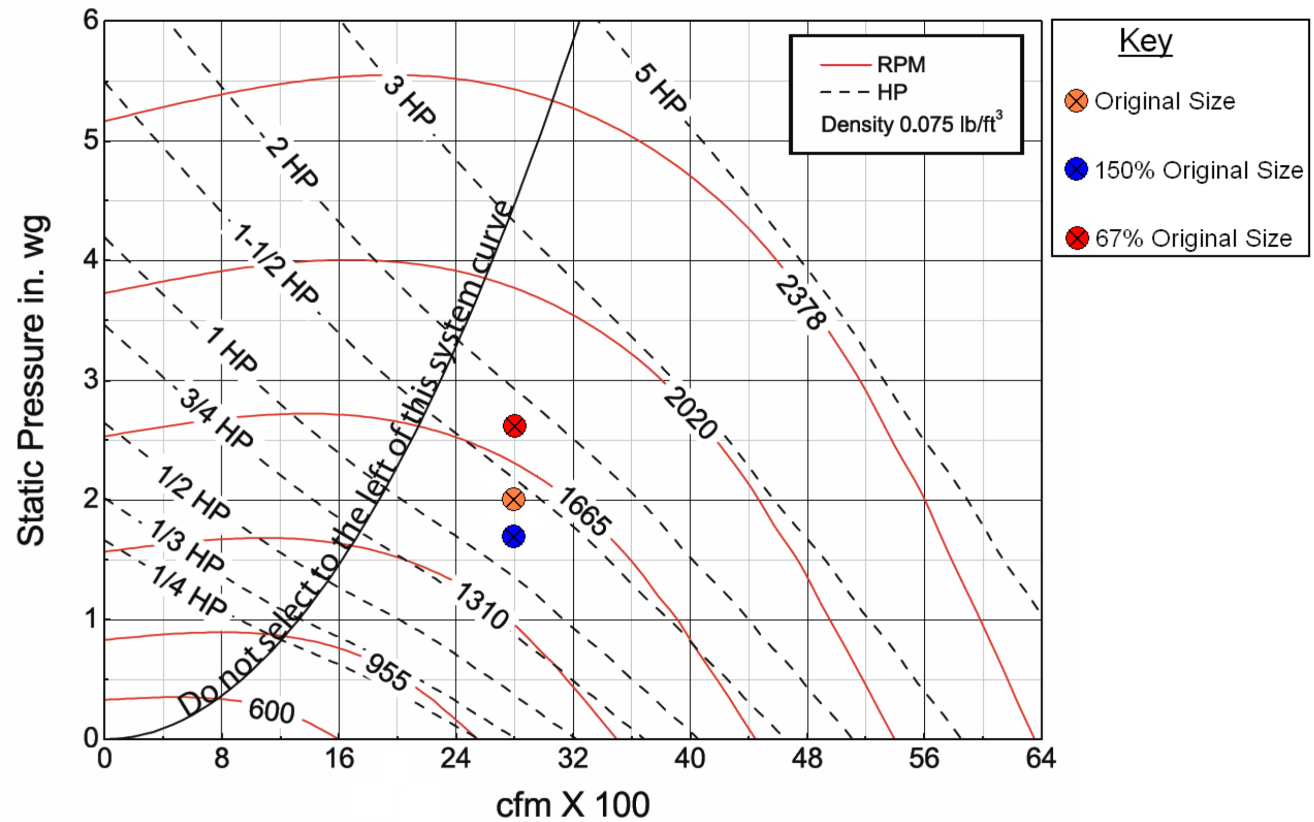
Branch Details	Exit Duct Size	Eq. Size (in)	Eq. Area (ft^2)	CFM
	8x6	7.5	0.31	100

Pressure Loss Calcs.

Section	Size (in)	Length (ft)	CFM	Eq. Size (in)	Eq. Area (ft^2)	Velocity (ft/min.)	DeltaP/100ft (in. w.g.)	Duct Loss (in. w.g.)	Qb/Qc	Converging Loss (in. w.g.)	Ao/A1	Transition Loss (in. w.g.)	Elbow Loss (in. w.g.)
Roof	12x22	18	2800	17.6	1.69	1657	0.23	0.0414	0.036	0.0255			
29	12x22	11	2700	17.6	1.69	1598	0.21	0.0231	0.037	0.0236			
28	12x22	10.5	2600	17.6	1.69	1539	0.2	0.0210	0.038	0.0218			
27	12x22	10.5	2500	17.6	1.69	1480	0.18	0.0189	0.040	0.0201			
26	12x22	10.5	2400	17.6	1.69	1421	0.16	0.0168	0.042	0.0185			
25	12x22	10.5	2300	17.6	1.69	1361	0.15	0.0158	0.043	0.0169			
24	12x22	9.75	2200	17.6	1.69	1302	0.13	0.0127	0.045	0.0226			
Duct Trans											1.21	0.0337	
23	12x18	9.75	2100	16	1.40	1504	0.21	0.0205	0.048	0.0205			
22	12x18	9.75	2000	16	1.40	1432	0.2	0.0195	0.050	0.0185			
21	12x18	9.75	1900	16	1.40	1361	0.18	0.0176	0.053	0.0166			
20	12x18	9.75	1800	16	1.40	1289	0.17	0.0166	0.056	0.0148			
19	12x18	9.75	1700	16	1.40	1218	0.15	0.0146	0.059	0.0131			
18	12x18	9.75	1600	16	1.40	1146	0.14	0.0137	0.063	0.0115			
17	12x18	9.75	1500	16	1.40	1074	0.12	0.0117	0.067	0.0223			
Duct Trans											1.49	0.0538	
16	12x12	9.75	1400	13.1	0.94	1496	0.35	0.0341	0.071	0.0192			
15	12x12	9.75	1300	13.1	0.94	1389	0.31	0.0302	0.077	0.0164			
14	12x12	10.5	1200	13.1	0.94	1282	0.26	0.0273	0.083	0.0138			
13	12x12	9.75	1100	13.1	0.94	1175	0.22	0.0215	0.091	0.0114			
12	12x12	9.75	1000	13.1	0.94	1068	0.17	0.0166	0.100	0.0092			
11	12x12	9.75	900	13.1	0.94	962	0.13	0.0127	0.111	0.0164			
Duct Trans											1.50	0.0437	
10	12x8	9.75	800	10.7	0.62	1281	0.28	0.0273	0.125	0.0149			
9	12x8	9.75	700	10.7	0.62	1121	0.23	0.0224	0.143	0.0121			
8	12x8	9.75	600	10.7	0.62	961	0.17	0.0166	0.167	0.0092			
7	12x8	9.75	500	10.7	0.62	801	0.12	0.0117	0.200	0.0069			
6	12x8	9.75	400	10.7	0.62	641	0.069	0.0067	0.250	0.0120			
Duct Trans											1.48	0.0186	
5	8x8	9.75	300	8.8	0.42	710	0.1	0.0098	0.333	0.0101			
Duct Trans											1.38	0.0180	
4	8x6	9.75	200	7.5	0.31	652	0.11	0.0107	0.500	0.0035			
3	8x6	11	100	7.5	0.31	326	0.03	0.0033					0.0079
							Total	0.5146		0.4213		0.1679	0.0079
							Total Loss	1.1117					
							Final Pressure	1.4983					

Conclusions:

It was found that the total pressure losses for the original size, the 50% larger size, and the 33% smaller sizes were 0.49in.w.g., 0.19in.w.g, and 1.11in.w.g., respectively. In order to end up with the same final pressures, new static pressures were chosen so after all the losses they will result in the same value. The static pressures and CFM values are plotted on the specifications graph for the fan below.



The required RPMs and Horsepowers were then determined by interpolating from values given in the fans specification tables. It was found that the original size, the 50% larger size, and the 33% smaller sizes required functioning HPs of 1.39, 1.22, and 1.89, respectively.

The original fan model selected has the capability to handle 2 HP. This allows for added flexibility in static pressure if needed. The 67% of the original size ducts would probably require the 3 HP engine in order to keep the flexibility that the original design had. The 150% of the original size ducts will still do fine with the 2 HP engine.

If you convert the required operating horsepowers for each of the duct sizes into KW-hrs, you can calculate the total cost per year for an electricity cost of $0.13/KW-hr.

Operating Expenses

Original Ducts: 1.39BHP = 1.04KW = 25.0 KW-hrs / day = $1,180 / year
50% Larger Ducts: 1.22BHP = 0.91KW = 21.8 KW-hrs / day = $1,036 / year
33% Smaller Ducts: 1.89BHP = 1.41KW = 33.8 KW-hrs / day = $1,605 / day

In order to make a decision about duct size the initial cost must be known as well. The $/lb for ductwork estimated from RSMeans is $5.92. The initial costs are summarized below. The details of the estimate can be found in the appendix.

Initial Expenses

Original Ducts: 1,859 lbs = $11,005
50% Larger Ducts: 2,249 lbs = $13,316
33% Smaller Ducts: 1,553 lbs = $9,152

Finally, in order to compare the ducts lifetime costs, the operating expenses must be converted into present day dollars. Using compound interest tables for 8%, the present cost for an annual cost is given as 12.5 * Annual Cost. The total costs are summarized below.

Lifetime Expense (Today's Dollars)

Original Ducts: $25,755
50% Larger Ducts: $26,266
33% Smaller Ducts: $29,215

Looking at the lifetime expenses shows that the original duct sizes end up costing the building the least amount. The 50% larger ducts do not make any sense to use because they are more expensive, take up more space, and will cause the most conflicts with the slab reinforcement. The 33% smaller ducts will cost $3,460 more over the lifetime expense. This amount is too much to be economical. It would only save 8 in^2/floor, which is not worth it. The original ducts should stay the same.

Summary of Results & Conclusions

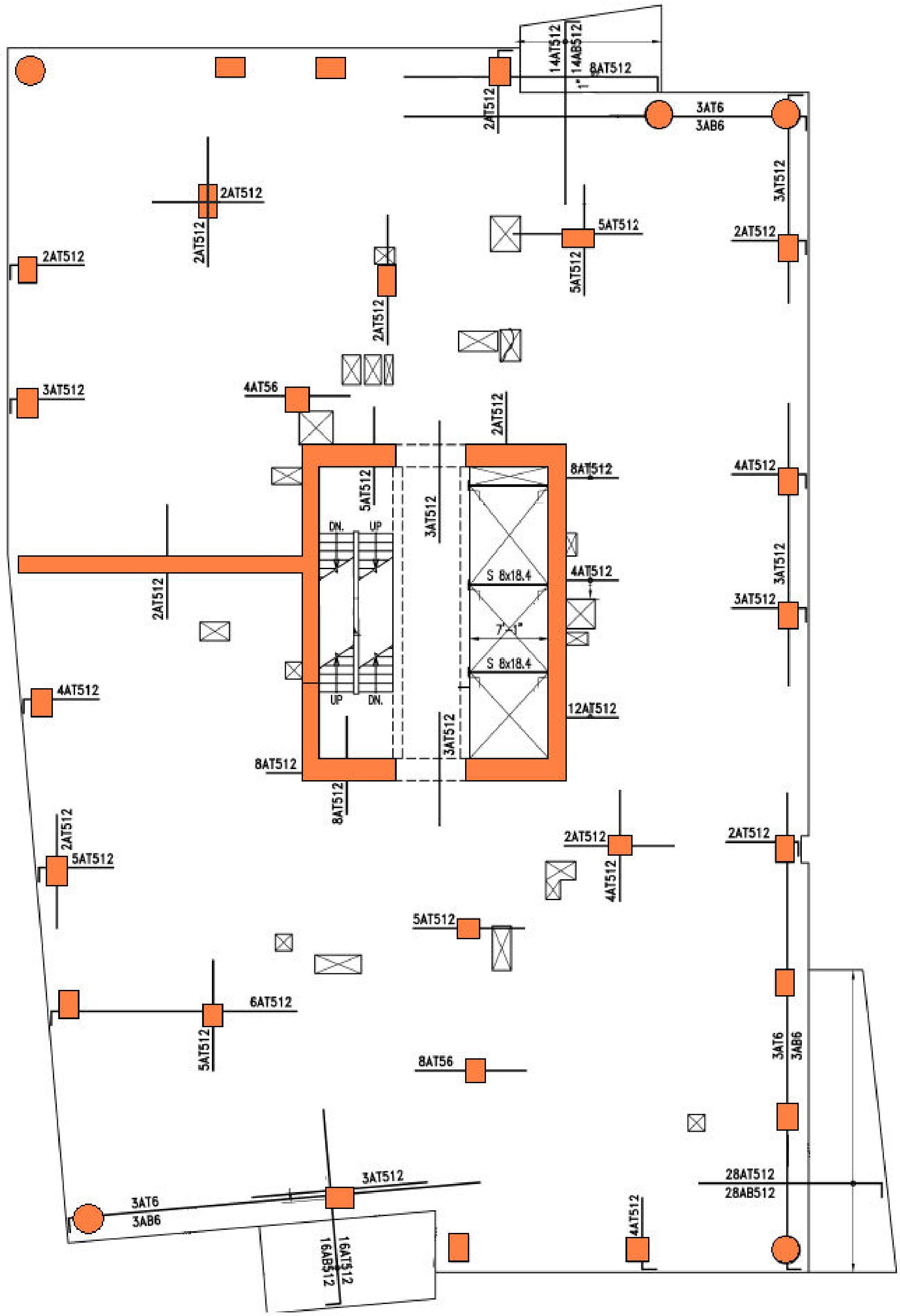
The studies performed on Northside Piers yielded several important results. The key results are listed below.

1. Switching the floor system from a mild reinforced slab into a post-tensioned slab will create better control over long-term deflections with an expected decrease of 30% in total deflection.
2. In terms of sound transmission, the 7" post-tensioned slab will perform almost as well as the 8" mild reinforced slab with an almost imperceptible difference.
3. The post-tensioned slab will cost approximately $2/SF less than the mild reinforced slab. The major savings coming from reduced amounts of mild reinforcement being required with some additional savings from the reduced slab thickness.
4. A shear wall layout with an additional wall at the lower levels is estimated to save approximately 5% of the cost of the original layout. The major savings comes from the reduction in rebar due to a more balanced layout. There is also savings in concrete due to the added depth at the base as well as moving the increased stiffness towards the base reduces the total required stiffness.
5. Adding 3" to the depth of the link beams reduces the torsional deflection by 12%. The ceiling height will be reduced to 7'-5" where the beams are located, which still meets New York City Code which has a minimum of 7'-0".
6. The schedules for the alternative shear wall layout and post-tensioned slab will be relatively unadjusted due to their similar nature to the original construction processes.
7. A potential savings of $36,000 can be made if the story heights are reduced by 1". This is possible to do while keeping the same overall ceiling height due to the thinner slab. The ceiling underneath the link beam would be at a height of 7'-4" which is still acceptable by the code, but it is 4" lower than originally designed.
8. The risers used for exhaust in the building should not be adjusted in size. A reduction in size of 33% will result in a higher operating cost and an increase in size of 50% will result in a higher initial cost. When comparing these costs to today's dollars the original duct size is the cheapest. While reducing the size of the ducts will result in fewer conflicts with the reinforcement and penetrations, the additional expense is not worth it.

This project was certainly an important capstone to my education at Penn State. It was the most real experience that I could get at a university, and I am sure that I will be much more prepared now when I begin to work in the industry.

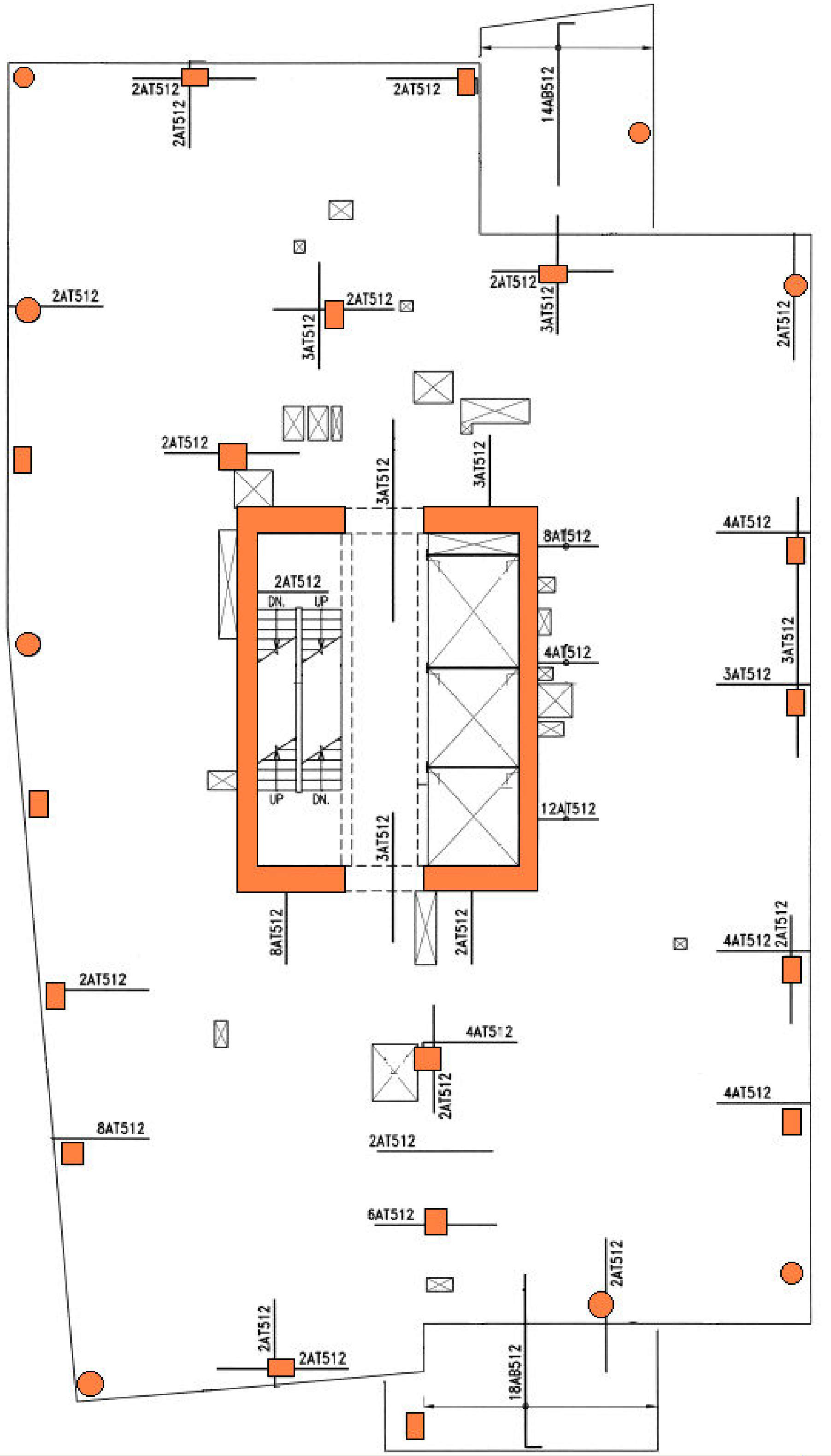
Appendix

3rd-25th Floor Original Rebar Plan:



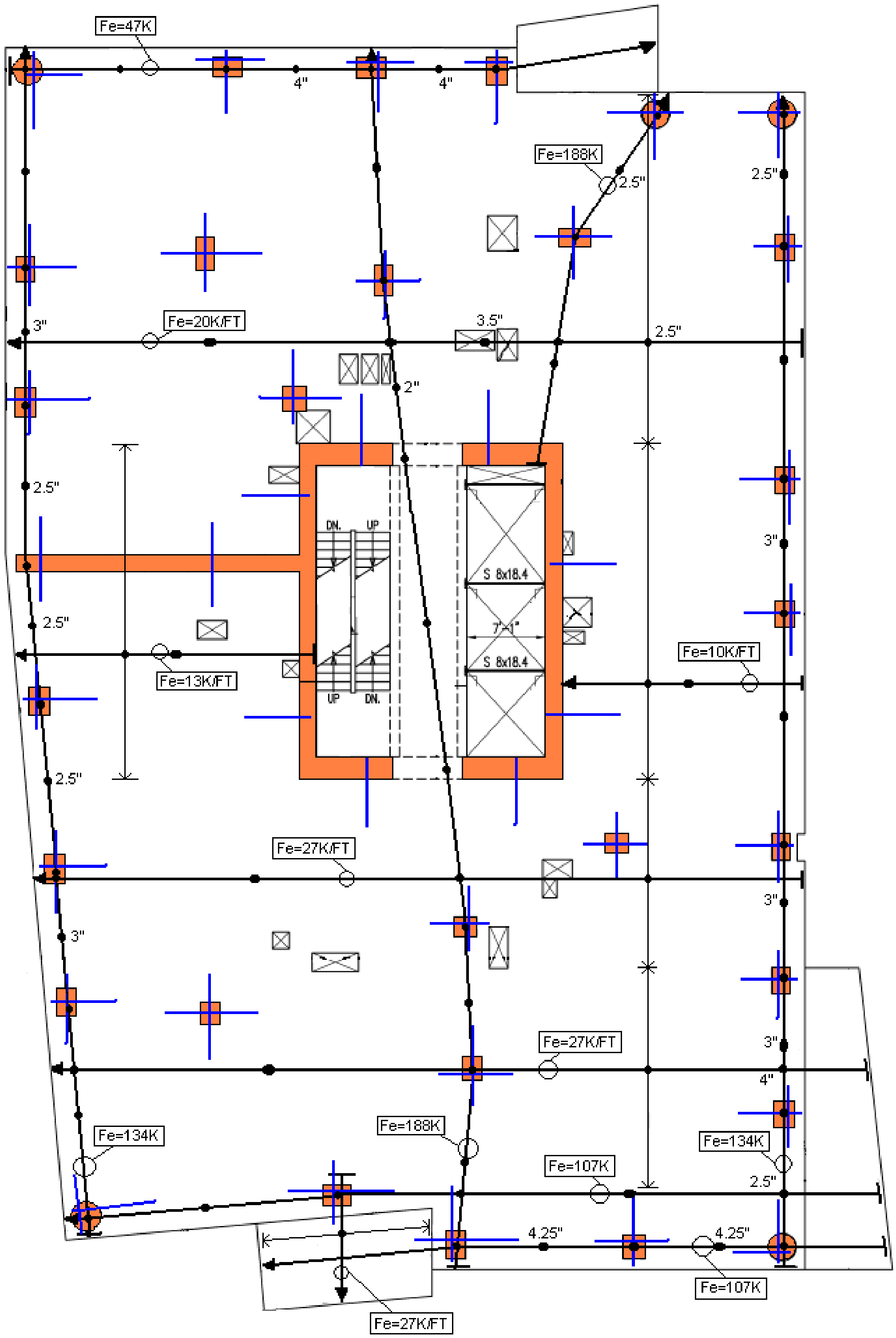
- 8" Slab with #5's @ 12" o/c on Top and Bottom going both ways. Additional bars added as indicated by plan (8AT512 = (8) #5's @ 12" o/c AT TOP).

26th-Roof Original Rebar Plan:



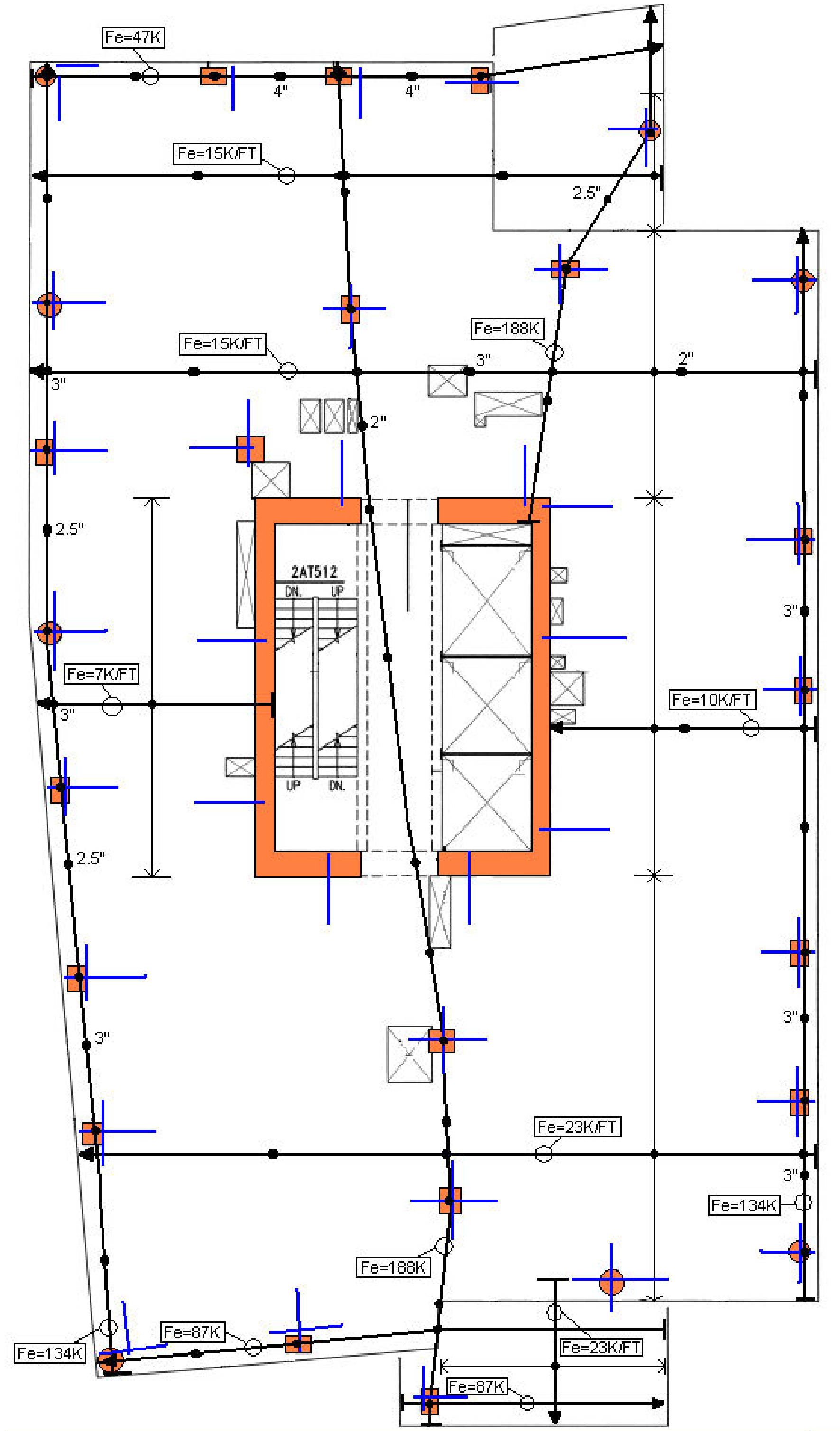
- 8” Slab with #5’s @ 12” o/c on Top and Bottom going both ways. Additional bars added as indicated by plan (8AT512 = (8) #5’s @ 12” o/c AT TOP).

3rd-25th Floor Post-Tensioned Plan:



- 7" Slab with #4's @ 24"o/c on Bottom going both ways. Blue lines signal for (5) #5's AT Top for the ext. columns and walls, as well as for (6) #5's AT Top for the interior columns. Tendons will be ½" unbonded. Tendon profiles are 1.25" and 5.75" for uniform tendons and 1.25" and 5" for banded tendons, except where noted.

26th-Roof Floor Post-Tensioned Plan:



- 7” Slab with #4’s @ 24”o/c on Bottom going both ways. Blue lines signal for (5) #5’s AT Top for the ext. columns and walls, as well as for (6) #5’s AT Top for the interior columns. Tendons will be ½” unbonded. Tendon profiles are 1.25” and 5.75” for uniform tendons and 1.25” and 5” for banded tendons, except where noted.

One-Way Post-Tensioned Slab Example:

One-Way Slab, Two Equal Spans

Span	25	ft
Thickness	7	in
Cover	1.25	in
Eccentricity	3.375	in
Dead Load	87.5	psf
Super Dead Load	30	psf
Live Load	40	psf
Losses	15	%
Load Balancing Percentage (Dead+Super)	75	%
Pe	24479.167	lb/ft width
Pi	28799	lb/ft width
Balancing Load at Transfer	104	psf upward
Total Load at Transfer	14	psf downward
P/A	291	psi

Concrete Section Properties:

A	84	in^2
S	98	in^2

Number of Strands:

fpi	175	ksi
Ap	0.165	in^2/ft width
	1.08	tendons/ft
rho p	0.002385	

Strength of Section (no mild reinf):

fpu	270	ksi	
fpy	243	ksi	
gamma	0.28	(pfy/pfu greater than 0.9)	
beta 1	0.75		
fps	254	ksi	
a	1.02	in	
phi Mn	197	in.-kip/ft	
wu	205	plf	
Mu	192.1875	in-kip/ft	OK

At Transfer Conditions:

Midspan Moment	6480	in-lb/ft width
Support Moment	-12960	in-lb/ft width
Midspan Stress (top)	-409	psi/ft width
Midspan Stress (bottom)	-277	psi/ft width
Support Stress (top)	-211	psi/ft width
Support Stress (bottom)	-475	psi/ft width
Ends Stress	-343	psi/ft width

After Losses Full Service:

Midspan Moment	32520	in-lb/ft width
Support Moment	-65039	in-lb/ft width
Midspan Stress (top)	-623	psi/ft width
Midspan Stress (bottom)	40	psi/ft width
Support Stress (top)	372	psi/ft width
Support Stress (bottom)	-955	psi/ft width
Ends Stress	-291	psi/ft width

Stress Limits (Class U):

f'ci	3000			
Initial Compression	-1800	OK	-475	
Initial Tension	164	OK	-211	
f'c	4000			
Service Compression	-1800	OK	-955	can be 1/3 larger if transient load
Service Tension	474	OK	372	

Two-Way Post-Tensioned Slab Example:

Two-Way Slab, Two Equal Spans

lx/ly 0.50 ly/lx 2.00

X-Span	15	ft		
Y-Span	30	ft		
Thickness	7	in		
Cover	1.25	in		
Eccentricity	3.375	in		
Dead Load	87.5	psf		
Super Dead Load	30	psf		
Live Load	40	psf		
Losses	12	%		
			psf	plf
X-Load Balancing	55	%	64.63	969.375
Y-Load Balancing	40	%	47	188
X-Pe	6.5	kip/ft width		
X-Pi	7.3	kip/ft width		
Y-Pe	18.8	kip/ft width		
Y-Pi	21.4	kip/ft width		
Balancing Load at Transfer	126.8	psf upward		
Total Load at Transfer	-9	psf downward		
Balancing Load at Service	112	psf upward		
Total Load at Service	6	psf downward		

Midspan Moment Coefficient (X)	0.050	
Support Moment Coefficient (X)	0.085	
Midspan Moment Coefficient (Y)	0.009	
Support Moment Coefficient (Y)	0.015	

At Transfer Conditions (X-Direction): | **At Transfer Conditions (Y-Direction):**

	X-Direction		Y-Direction	
Midspan Moment	-1.3	kip-in/ft width	-0.9	kip-in/ft width
Support Moment	2.1	kip-in/ft width	1.5	kip-in/ft width
Initial Compression	87	psi	254	psi
Midspan Stress (top)	-75	psi	-245	psi
Midspan Stress (bottom)	-100	psi	-264	psi
Support Stress (top)	-109	psi	-270	psi
Support Stress (bottom)	-66	psi	-239	psi

After Losses Full Service (X-Direction): | **After Losses Full Service (Y-Direction):**

	X-Direction		Y-Direction	
Midspan Moment	6	kip-in/ft width	4	kip-in/ft width
Support Moment	-11	kip-in/ft width	-7	kip-in/ft width
Service Compression	77	psi	224	psi
Midspan Stress (top)	-140	psi	-269	psi
Midspan Stress (bottom)	-14	psi	-178	psi
Support Stress (top)	30	psi	-148	psi
Support Stress (bottom)	-184	psi	-300	psi

Stress Limits (Class U):

			X-Direction	Y-Direction	
f'ci	3000				
Initial Compression	-1800	OK	-109	-270	
Initial Tension	164	OK	-66	-239	
f'c	6000				
Service Compression	-2700	OK	-184	-300	(can be 1/3 larger if transient loads)
Service Tension	581	OK	30	-148	

Concrete Section Properties:

A	84	in^2
S	98	in^2

Number of Strands:

fpe	160	ksi				
X-Ap	0.040	in^2/ft width	0.28	tendons/ft	42.8	in. spacing (1 tendon=0.144in^2)
Y-Ap	0.134	in^2/ft width	0.93	tendons/ft	12.9	in. spacing (1 tendon=0.144in^2)

rho p 0.000585

Ultimate Loads (X-Direction): | **Ultimate Loads (Y-Direction):**

	X-Direction		Y-Direction	
Balancing Moment	1.82	kip-ft/ft width	5.29	kip-ft/ft width
Primary Moment	1.21	kip-ft/ft width	3.53	kip-ft/ft width
Secondary Moment	0.61	kip-ft/ft width	1.76	kip-ft/ft width
Mu	4.53	kip-ft/ft width	4.53	plf

Strength of Section (X-Direction): | **Strength of Section (Y-Direction):**

	X-Direction		Y-Direction	
fpu	270	ksi	270	ksi
fpy	243	ksi	243	ksi
gamma	0.28	(pfy/pfu greater than 0.9)	0.28	(pfy/pfu greater than 0.9)
beta 1	0.75		0.75	
fps	267	ksi	267	ksi
a	0.176	in	0.583	in
phi Mn	4.59	kip-ft/ft width OK	14.61	kip-ft/ft wid OK

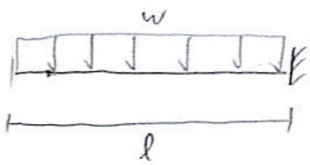
Shear Wall Spot Check:

E-W1 Shear Wall (15th Floor) Spot Check

Height (ft)	8	
Length (ft)	8.5	
Thickness (in)	24	
Diameter bars (in)	0.75	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	6	
Factored Moment (ft-k)	12592.8	
Factored Shear (k)	425.6	
Factored Axial (k)	1100	
Moment of Inertia (ft^4)	102.3542	
Stress (ksi)	4.080488	
0.2*f'c (ksi)	1.2	**BE Required**
2Acv*SQRT(f'c)	379.2425	**2 Curtains Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.003068	**OK**
hw/lw	0.941176	
alpha	3	
Vn (kip)	1019.486	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	917.5374	**WALL OK**

Deflection of Cantilever with Constant Stiffness vs. Varying Stiffness:

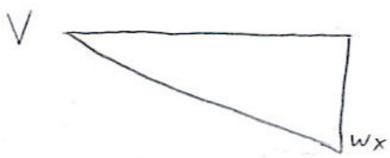
Moment Calculation:



$$\boxed{P = -w}$$

$$V = \int w \, dx$$

$$V = -wx + C_1 \qquad V(0) = 0 \quad \therefore C_1 = 0$$



$$\boxed{V = -wx}$$

$$M = \int V \, dx = \int -wx \, dx$$

$$M = -\frac{wx^2}{2} + C_1 \qquad M(0) = 0 \quad \therefore C_1 = 0$$

$$\boxed{M = \frac{-wx^2}{2}}$$

Comparison of Constant Stiffness vs. Varying Stiffness (x^2):

$(EI)_1 =$ constant

$(EI)_2 = x^2$

$$\int EI = Cx \qquad \int EI(l) = Cl$$

$$\int EI = \frac{x^3}{3} \qquad \int EI(l) = \frac{l^3}{3}$$

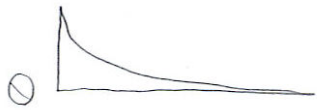
For the two stiffness distributions to have the same total,

$$\int (EI)_1 = \int (EI)_2$$

$$Cl = \frac{l^3}{3}$$

$$C = \frac{l^2}{3}$$

Deflection of Constant Stiffness:



$$\theta = \int \frac{M}{(EI)_1} dx = \frac{1}{(EI)_1} \int \frac{-wx^2}{2} dx$$

$$\theta = \frac{1}{(EI)_1}\left(\frac{-wx^3}{6}\right) + C_1 \qquad \theta(l) = 0 \quad \therefore C_1 = \frac{wl^3}{6(EI)_1}$$

$$\boxed{\theta = \frac{1}{(EI)_1}\left(\frac{wl^3}{6} - \frac{wx^3}{6}\right)}$$

$$\Delta = \int \theta \, dx = \int \frac{1}{EI}\left(\frac{wl^3}{6} - \frac{wx^3}{6}\right) dx$$

$$\Delta = \frac{1}{(EI)_1}\left(\frac{wl^3x}{6} - \frac{wx^4}{24}\right) + C_1 \qquad \Delta(l) = 0 \quad \therefore C_1 = \frac{-wl^4}{8(EI)_1}$$

$$\Delta = \frac{1}{(EI)_1}\left(\frac{-wl^4}{8} + \frac{wl^3x}{6} - \frac{wx^4}{24}\right)$$

$$\boxed{\Delta = \frac{w}{24(EI)_1}\left(-x^4 + 4l^3x - 3l^4\right)}$$

Deflection of Varying Stiffness:

$$\theta = \int \frac{M}{(EI)_2} dx = \int \frac{-wx^2}{2x^2} dx$$

$$\theta = \frac{-wx}{2} + C_1 \qquad \theta(l) = 0 \quad \therefore C_1 = \frac{wl}{2}$$

$$\boxed{\theta = \frac{-wx}{2} + \frac{wl}{2}}$$

$$\Delta = \int \theta \, dx = \int \left(\frac{-wx}{2} + \frac{wl}{2}\right) dx$$

$$\Delta = \frac{-wx^2}{4} + \frac{wlx}{2} + C_1 \qquad \Delta(l) = 0 \quad \therefore C_1 = \frac{-wl^2}{4}$$

$$\boxed{\Delta = \frac{-w}{4}\left(x^2 - 2lx + l^2\right)}$$

Maximum Deflection Comparison With Same Total Stiffness:

For EI = constant

$$\Delta(0) = \frac{-3wl^4}{24(EI)_1} = \frac{-wl^4}{8(EI)_1} \qquad (EI)_1 = \frac{l^2}{3}$$

$$\Delta(0) = \frac{-3wl^2}{8} = -0.375\,wl^2$$

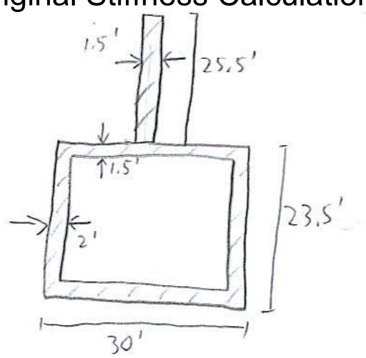
For $EI = x^2$

$$\Delta(0) = \frac{-wl^2}{4} = -0.25\,wl^2$$

For the same total stiffness, a stiffness that varies by x^2 will have $\frac{0.25}{0.375}$ = 67% of the deflection of a constant stiffness

Shear Wall Stiffness Calculations:

Original Stiffness Calculations:



$$I_1 = \tfrac{1}{12}(1.5 \cdot 25.5^3) = 2,073 \text{ ft}^4$$

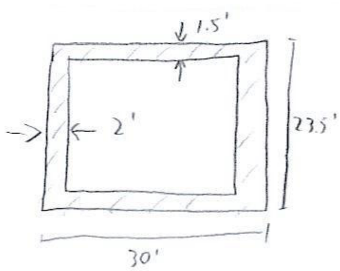
$$I_2 = \tfrac{1}{12}(30 \cdot 23.5^3) - \tfrac{1}{12}(26 \cdot 20.5^3) = 13,779 \text{ ft}^4$$

$$\bar{y} = \frac{1.5 \cdot 25.5 \cdot \left(\frac{23.5}{2} + \frac{25.5}{2}\right)}{1.5 \cdot 25.5 + (30 \cdot 23.5 - 26 \cdot 20.5)} = 4.46'$$

$$I = I_1 + A_1 y_1^2 + I_2 A_2 y_2^2 = 34,635 \text{ ft}^4$$

$$(EI)_1 = 34,635 \cdot 5154 \cdot 144 = \boxed{2.57 \times 10^{10} \text{ K-ft}^2}$$

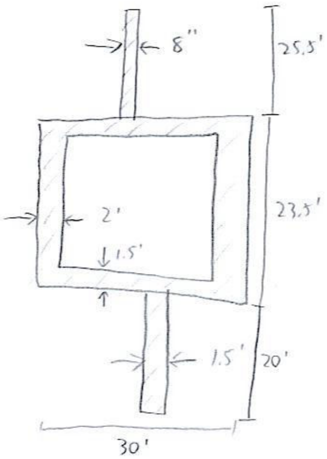
$$(EI)_2 = 34,635 \cdot 4463 \cdot 144 = \boxed{2.23 \times 10^{10} \text{ K-ft}^2}$$



$$I = 13,779 \text{ ft}^3$$

$$(EI)_3 = 13,779 \cdot 4463 \cdot 144 = \boxed{0.89 \times 10^{10} \text{ K-ft}^2}$$

New Stiffness Calculations:



$$I_1 = \frac{1}{12}\left(\frac{8}{12} \cdot 25.5^3\right) = 921\ ft^3$$

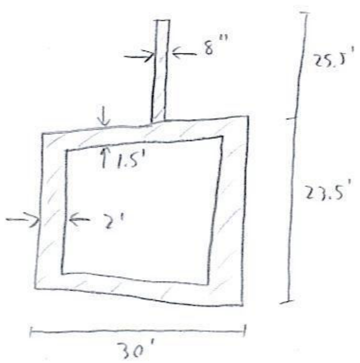
$$I_2 = 13{,}779\ ft^3$$

$$I_3 = \frac{1}{12}\left(1.5 \cdot 20^3\right) = 1000\ ft^3$$

$$\bar{y} = \frac{\frac{8}{12} \cdot 25.5\left(\frac{23.5}{2} + \frac{25.5}{2}\right) - 1.5 \cdot 20\left(\frac{23.5}{2} + 10\right)}{\frac{8}{12} \cdot 25.5 + 1.5 \cdot 20 + (30 \cdot 23.5 - 26 \cdot 20.5)} = -1.08'$$

$$I = 46{,}400\ ft^3$$

$$(EI)_1 = 46{,}400 \cdot 5154 \cdot 144 = \boxed{3.44 \times 10^{10}\ k\text{-}ft^2}$$



$$I_1 = 921\ ft^3$$

$$I_2 = 13{,}779\ ft^3$$

$$\bar{y} = \frac{\frac{8}{12} \cdot 25.5 \cdot \left(\frac{23.5}{2} + \frac{25.5}{2}\right)}{\frac{8}{12} \cdot 25.5 + (30 \cdot 23.5 - 26 \cdot 20.5)} = 2.20'$$

$$I = 34{,}554\ ft^3$$

$$(EI)_2 = 34{,}554 \cdot 5154 \cdot 144 = \boxed{2.71 \times 10^{10}\ k\text{-}ft^2}$$

$$(EI)_3 = 34{,}554 \cdot 4463 \cdot 144 = \boxed{2.22 \times 10^{10}\ k\text{-}ft^2}$$

$$(EI)_4 = \boxed{0.89 \times 10^{10}\ k\text{-}ft^2}$$

Ductwork Estimate:

		Original			33% smaller			50% Larger		
Floor	**Length**	**Width**	**Depth**	**Weight (lb)**	**Width**	**Depth**	**Weight (lb)**	**Width**	**Depth**	**Weight (lb)**
Roof	18	14	26	146.4	12	22	124.44	18	30	175.68
29	11	14	26	89.46666667	12	22	76.04666667	18	30	107.36
28	10.5	14	26	85.4	12	22	72.59	18	30	102.48
27	10.5	14	26	85.4	12	22	72.59	18	30	102.48
26	10.5	14	26	85.4	12	22	72.59	18	30	102.48
25	10.5	14	26	85.4	12	22	72.59	18	30	102.48
24	9.75	14	26	79.3	12	22	67.405	18	30	95.16
23	9.75	14	22	71.37	12	18	59.475	18	26	87.23
22	9.75	14	22	71.37	12	18	59.475	18	26	87.23
21	9.75	14	22	71.37	12	18	59.475	18	26	87.23
20	9.75	14	22	71.37	12	18	59.475	18	26	87.23
19	9.75	14	22	71.37	12	18	59.475	18	26	87.23
18	9.75	14	22	71.37	12	18	59.475	18	26	87.23
17	9.75	14	22	71.37	12	18	59.475	18	26	87.23
16	9.75	14	16	59.475	12	12	47.58	18	18	71.37
15	9.75	14	16	59.475	12	12	47.58	18	18	71.37
14	10.5	14	16	64.05	12	12	51.24	18	18	76.86
13	9.75	14	16	59.475	12	12	47.58	18	18	71.37
12	9.75	14	16	59.475	12	12	47.58	18	18	71.37
11	9.75	14	16	59.475	12	12	47.58	18	18	71.37
10	9.75	14	10	47.58	12	8	39.65	18	12	59.475
9	9.75	14	10	47.58	12	8	39.65	18	12	59.475
8	9.75	14	10	47.58	12	8	39.65	18	12	59.475
7	9.75	14	10	47.58	12	8	39.65	18	12	59.475
6	9.75	14	10	47.58	12	8	39.65	18	12	59.475
5	9.75	10	8	35.685	8	8	31.72	12	10	43.615
4	9.75	10	6	31.72	8	6	27.755	10	8	35.685
3	11	10	6	35.78666667	8	6	31.31333333	10	8	40.26
			Total	1858.873333			1552.755			2249.375
			Cost	$11,005			$9,192			$13,316

Shear Wall Estimate:

Original Shear Walls

	Item	Amount	Unit	Unit Cost	Daily Output	Total	Time
Concrete, Ready Mix	6000psi	1223	CY	109		$133,307	
	8000psi	960	CY	179		$171,840	
Placing Concrete	Walls, 8" thick, with crane and bucket	0	CY	15.87	90	$0	0.00
	Walls, 15" thick, with crane and bucket	2183	CY	13.61	105	$29,711	20.79
Formwork	Modular prefabricated plywood, to 8' high, 4use	62155	SFCA	1.95	1260	$121,202	49.33
Mild Rebar	#3-#7	102	Ton	1230	3	$125,460	34.00
	#8-#18	24	Ton	1125	4	$27,000	6.00
Splices	#14-#11		Ea.	67		$0	
	#10-#9	252	Ea.	42		$10,584	
	#9-#8	52	Ea.	39		$2,028	
	#8-#7	452	Ea.	38		$17,176	
					Total	$638,308	110.12
					Cost/VF	$2,007	

New Shear Walls

	Item	Amount	Unit	Unit Cost	Daily Output	Total	Time
Concrete, Ready Mix	6000psi	1131	CY	109		$123,279	
	8000psi	982	CY	179		$175,778	
Placing Concrete	Walls, 8" thick, with crane and bucket	157	CY	15.87	90	$2,492	1.74
	Walls, 15" thick, with crane and bucket	1956	CY	13.61	105	$26,621	18.63
Formwork	Modular prefabricated plywood, to 8' high, 4use	66675	SFCA	1.95	1260	$130,016	52.92
Mild Rebar	#3-#7	81	Ton	1230	3	$99,630	27.00
	#8-#18	25	Ton	1125	4	$28,125	6.25
Splices	#14-#11	40	Ea.	67		$2,680	
	#10-#9	154	Ea.	42		$6,468	
	#9-#8	26	Ea.	39		$1,014	
	#8-#7	354	Ea.	38		$13,452	
					Total	$609,555	106.54
					Cost/VF	$1,917	