Erie on the Park

Chicago, IL

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Final Report - Senior Thesis 2006

Erie on the Park

Chicago, IL

Primary Project Team:

Owner: Smithfield Properties LLC
Architect: Lucien Lagrange Architects
Structural Engineer: Thornton-Tomasetti Engineers
General Contractors: Wooton Construction, Ltd.

General Project Info:

- 25 Story Condominium Complex
- First 5 stories are a tenant parking garage, 6th floor has a fitness center, and floors 7 - 25 are residential units
- 165,000 sq ft Residential Space
- $51M Overall Project Cost

Architecture:

- Chevron shapes accentuate the use of steel and draw your eyes skyward
- The use of steel allows for greater flexibility of floor layouts
- 30' Ceiling in Ground Floor Lobby provides grand entrance for tenants

Structural:

- Concrete shear walls and columns resist lateral and gravity loads in first 3 stories
- Steel columns and brace frame system supports upper 22 stories
- Mega braces moved to exterior to resist torsional forces

Mechanical:

- Vertical fan coil system supplies fresh air to each floor
- AHU's located in mechanical penthouse on 25th floor
- 300 CFM line diffusers distribute air to the rooms of the units

Electrical:

- 1600 Amp, 480/277 Volt, 3 Phase, 4 wire primary switchboard
- 1600 Amp, 208/120 Volt, 3 Phase, 4 wire secondary switchboard
- 125 Amp, 208/120 Volt, 1 Phase, 4 wire tenant switchboard
- 450 kW back-up generator

smithfield

Thornton-Tomasetti Group

PENNSTATE

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Structural
http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/trm906

Executive Summary

Erie on the Park is a 25 story condominium complex on W. Erie St. in Chicago, IL. By using steel for the main structural system the architect on this project goes against the normal practice of using concrete as the major structural system for a residential high-rise building. In doing this he allows himself greater flexibility when designing the layout of each of the tenant spaces, and provides a strong architectural statement with the steel chevrons punctuating the building's façade. The entrance to the building is through a grand lobby with a 30' high ceiling. The next three stories are part of a parking garage with many spaces for tenants to park their cars out of the elements. The sixth floor has a fitness center. Floors five through 24 are condominiums that provide a dynamic living space and spectacular views of the Chicago skyline through floor-to-ceiling windows.

This report describes the in depth study of redesigning the structural system of this 25 story condominium complex as three different concrete structures. All three designs incorporated a flat-plat floor system in the attempt to reduce the costs of construction while minimizing the actual depth of the floor system. To further reduce the depth of the floor slab the inclusion of post-tensioning tendons was investigated. The three designs considered for this study were a shear wall lateral force resisting system, a shear wall and moment frame lateral force resisting system that utilizes a 10" reinforced concrete flat-slab as the beams of the frame, and a shear wall and moment frame lateral force resisting system that utilizes an 8" post-tensioned concrete flat-slab as the slab frame. The reason for these various designs was to first see the effect of integrating a moment frame with the shear walls and second to determine the effect of stiffening the floor slab through post-tensioning.

In conjunction with this in depth analysis of the floor and lateral force resisting system, two breadth studies were conducted. The first breadth study delved into the construction management issues of altering the systems from steel to concrete and then the issues between the various concrete systems. In this study the costs and construction schedule of each of the systems were determined and any constructability issues were investigated. The other breadth topic investigated was the implementation of a modification to the plumbing and storm water drainage systems to achieve LEED points. This modification is a rain water collection and storage system that then uses the rain water to flush the toilets in the building in an attempt to reduce the overall usage of potable. The costs and savings of this system were determined as well as any design implications.

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Introduction

Building Information

'Erie on the Park' is a 25 story, 290' condominium complex in the River North neighborhood of Chicago, IL. These condos, owned and maintained by Smithfield Properties, LLC, were designed by Lucien Lagrange, an Architecture firm native to Chicago. Lucien Lagrange then enlisted Thornton-Tomasetti to design the structural system and Wooton Construction to be the general contractors.

Project Team:

Owner/Developer: Smithfield Properties LLC
Property Manager: Draper and Kramer, Incorporated
Architect: Lucien Lagrange Architects
General Contractor: Wooton Construction, Ltd.
Shell Construction: Area Erectors, Inc.
Structural Engineer: Thornton-Tomasetti Engineers
Electrical Engineer: Innovative Building Concepts, Inc.
Elevator Engineer: Jenkins & Huntington, Inc.
Geotechnical Engineer: STS Consultants, Ltd.
Lighting Consultant: Schuler & Shook, Inc.
Mechanical Systems Installation: Advance Mechanical Systems, Inc.
Fire Protection: Global Fire Protection Company
Steel Supplier: Zalk Josephs Fabricators LLC
Window Supplier: Trainor Glass Company

These condominiums are considered mid to high end real estate. They are located less than a mile northwest of downtown Chicago providing an easy commute for those tenants who work near the Loop. The building has such amenities as an on-site covered parking garage, fitness center, and a 24 hour doorman. The individual units come equipped with granite counter tops and handmade European cabinets. Most of the units have their own terraces and the floor to ceiling windows provide exceptional views of the city.

Architecture

This 25 story condominium complex was constructed using a steel frame structure which is contrary to the typical concrete structures used in residential high rises. The parallelogram shape that this building has assumed was dictated by the intersection of the two streets adjacent to the property, W. Erie St. and N. Kingsbury St. The unique shape and steel structure have allowed for very innovative and flexible floor plans. This structure also provides unobstructed floor to ceiling views of the great

Chicago cityscape and the stepping back of the upper floors allow for expansive terraces. The architecture is focused on images and views. The steel and glass façade with the large chevrons provide an image of modernity and make the structure seem light and graceful. The 30' lobby gives the image of the grand entrance. Even the interior décor of stainless steel appliances, granite counter tops, and European cabinets give a modern feeling to the individual units while the floor to ceiling windows and the steel braces frame spectacular views of the surrounding areas.

Foundation

The foundation is made-up of hardpan caissons and grade-beams. The caissons are drilled up to a depth of 85'. This depth is required to find soil with a net bearing pressure of 30 KSF. The caisson shaft diameters range from 30" to 54" and the bell diameters range from 4' to 11'. The grade beams average about 36"x60" with the largest width being 72" and the greatest depth being 100". The grade beams frame into the caisson caps which have a minimum width of 6" larger than their respective caisson and a depth of 3'. These sizes are increased to the width and/or depth of the largest grade beam framing in to them. These three structural elements have concrete with a 28 day compressive strength of 6000 PSI, and use epoxy coated, deformed rebar in accordance with ASTM A615.

Electrical

The electricity for 'Erie on the Park' enters the building through four conduits from the city's power grid. One of the conduits goes through a metered switchboard that distributes power to the buildings common areas. This power is distributed through four separate panels to (1) the mechanical penthouse, (2) the two elevators, (3) the emergency lighting, and (4) the receptacles and lighting of the corridors and common areas. The second conduit enters an unmetered switchboard that distributes power to the different floors and through a 1600Amp busduct. At each of the floors this power is split and metered before it ultimately reaches each of the tenant units. The third of the four supply conduits is dedicated to the emergency lighting for the entire building, and the fourth supply is dedicated to the fire pump and its controller. There is also a 450kW gas powered generator that in the event of a power outage would service the elevators, the emergency lighting, and the fire pump and controller.

Lighting

The typical lighting scheme in 'Erie on the Park' is 6" recessed downlights in the public and private corridors on the tenant levels. The living rooms, kitchens and dining rooms of the tenant units have track lighting that range in length from 4' to 20', in increments of 4', depending on the size and layout of the condominium. The service areas and parking levels typically use 8' fluorescent strip lights. The lighting in the lobby and around the entrance incorporates more artistic lighting in the form of directional, accent and floor recessed lighting. The entrance to the building has recessed pathway lights that light your way into the lobby. There are also a number of accent lights that illuminate the shear wall that lines the entrance to the building. In the lobby there are directional and accent lights that add to the grandeur of the 30' ceiling.

Mechanical

The mechanical system supply air is brought in at the 25th floor through two 4'x6' louvers to a pair of Carrier air handling units. Each AHU is capable of supplying 9,000 CFM each to the lower floors. Vertical fan coils to move conditioned air vertically between the 25 floors of this building. The air is distributed to each of the tenant rooms through 12 inch circular ducts. These ducts are suspended in the ceiling cavity by running through the openings in the open-web steel joists that support the floor slab. The ducts terminate at three foot linear distributors that are capable of 300 CFM.

Existing Structure

Columns

There are concrete columns from the ground level to the third floor, an overall elevation of 40'. These columns are either circular with a 30" diameter or rectangular with dimensions varying from 26" to 36" on each side. The circular columns are toward the eastern end of the building where they are only framing into concrete slabs. The rectangular columns are towards the western end of the building and frame into a steel mezzanine half way between the ground and second floor. The 28 day compressive strength of the concrete is 8000 PSI.

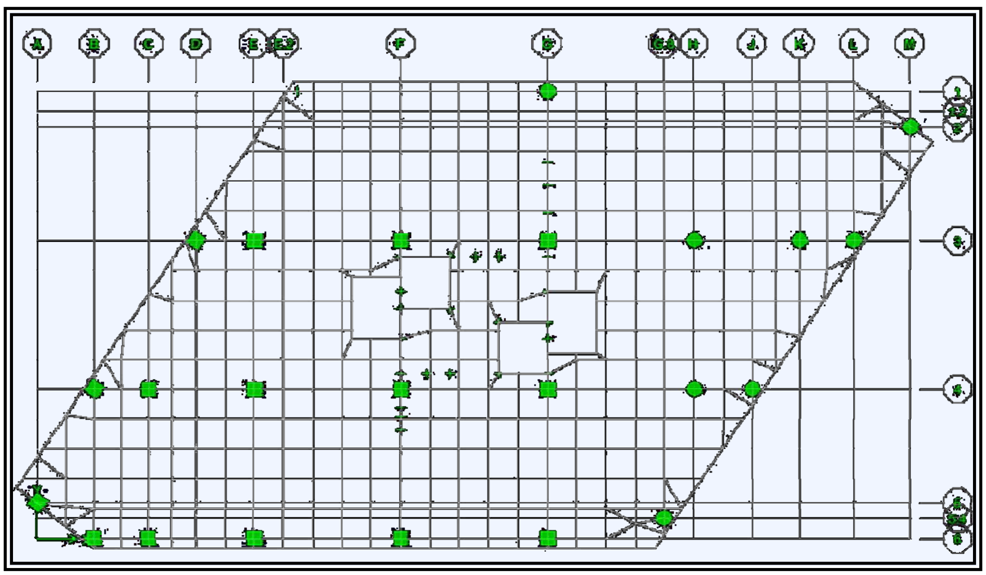


Figure 1: General column layout of first 3 floors.

At the third floor the concrete columns transition to steel W-shapes that continue the remaining 250' to the roof. The columns are ASTM A992 Grade 50 rolled W14 steel shapes. The largest columns are W14x257 and are part of the lateral system. The columns that are primarily part of the gravity system are W14x132's at the third floor down to W14x61's supporting the roof. These columns were generally erected in two story lifts, which are about 21'.

Floor System

The first through third floors incorporate a two-way, concrete, flat-slab system. The first floor is slab-on-grade and is 10" thick

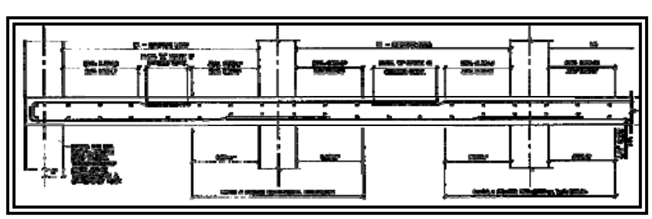


Figure 2: Detail of typical two-way flat-slab.

south of column line 4 and 12" thick north of column line 4. The second and third levels both have 12" thick slabs with 12"x24" beams running in the E-W direction along column lines 3 and 4 from column line E to H as added support around the openings for the elevators. The rebar in these slabs and beams are grade 60 deformed steel bars and those in the slab-on-grade have been epoxy coated for added protection against rusting. The concrete used in these slabs has a 28 day strength of 6000 PSI.

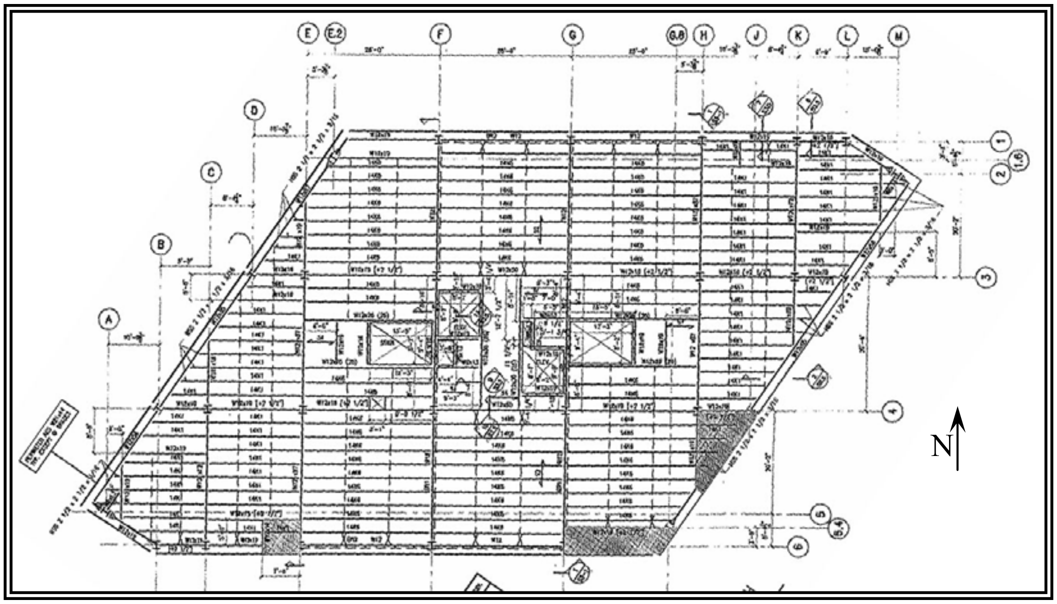


Figure 3: General floor plan of upper floors.

The mezzanine level and floors 4-6 have steel girders and beams with a partially composite slab on steel deck as their floor system. The beams are typically W18x35 and span 26'-4" in the N-S direction and the girders are W18x55 and span 26'-0" in the E-W direction. The sizes and lengths of the beams and girders begin to vary greatly east of column line H and west of column line E due to the acute angles of these corners of the building. The deck is 3" 18 gage composite steel decking. Normal weight concrete, with a 28 day compressive strength of 4000 PSI, is poured to a depth of 4 ½".

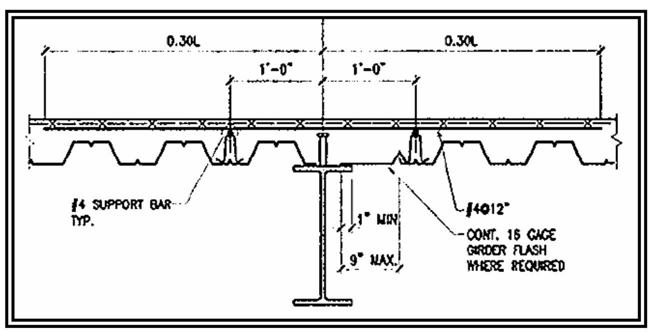


Figure 4: Detail of composite slab-on-deck system.

The seventh through 25th floors are steel joist construction where 14K6 joists, 2' O.C., span 26' between W12x87 beams that span 26'-4". These joists support a 2" concrete slab on 0.6C26 non-composite steel deck with 6x8xW1.4xW1.4 WWF. As with the lower floors this regular pattern is only applicable between column lines E and H. Outside of these column lines the trusses are reduced in size to either 14K1 or 12K1 depending on the span, and likewise the W-shape girders are reduced in size depending on the span. The roof is comprised primarily of W21x26 beams 8'-8" O.C. spanning 26' between W12x96 girders. These girders in turn span 26'-4". On top of the beams is a 3" 22 gage, hot dipped galvanized, steel deck.

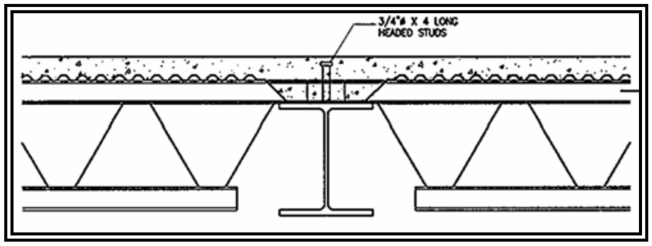


Figure 5: Detail of partially composite joist system.

Lateral System

The lateral system between the ground level and the third level is comprised of cast-in-place concrete shear walls, most with a 28 day compressive strength of 8000 PSI. There are two 27', 18" thick shear walls running in the N-S direction, and there are three running in the E-W direction with lengths of 26', 29'-4", and 52' which are also 18" thick. The 52' shear wall has pilasters that act as columns at its beginning, midpoint, and end thus these take the majority of the compressive forces and because of this concrete with an f'_c = 6000 PSI was used to save on concrete costs without compromising any noticeable shear strength. These walls resist the lateral

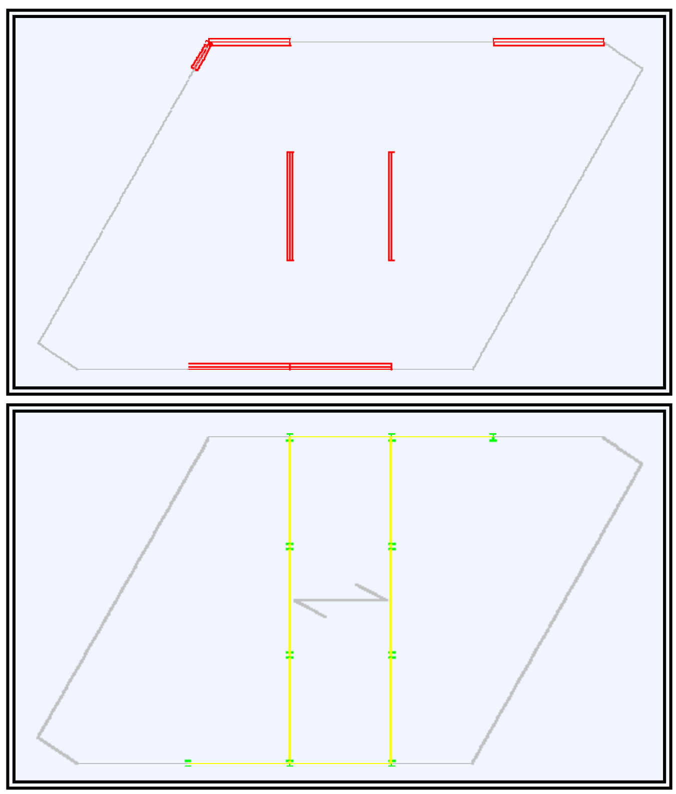


Figure 6: (Top) Lower level shear walls.
(Bottom) Upper level steel braces.

loads transferred down from steel brace frames on the upper floors. The braced frames, made up of W8 and W10 shapes, distribute the shear load through large three story steel chevrons as seen in the figure to the right. The three bay brace is in the N-S direction along column lines F and G and the two bay brace is in the E-W direction between F-1 and H-1 and between E-6 and G-6. The three bay brace resists most of the lateral forces in the N-S direction and the two bay braces resist the lateral forces in the E-W direction as well as helping out with any torsion since they are on the exterior of the building and have a much larger eccentricity than the three bay braces.

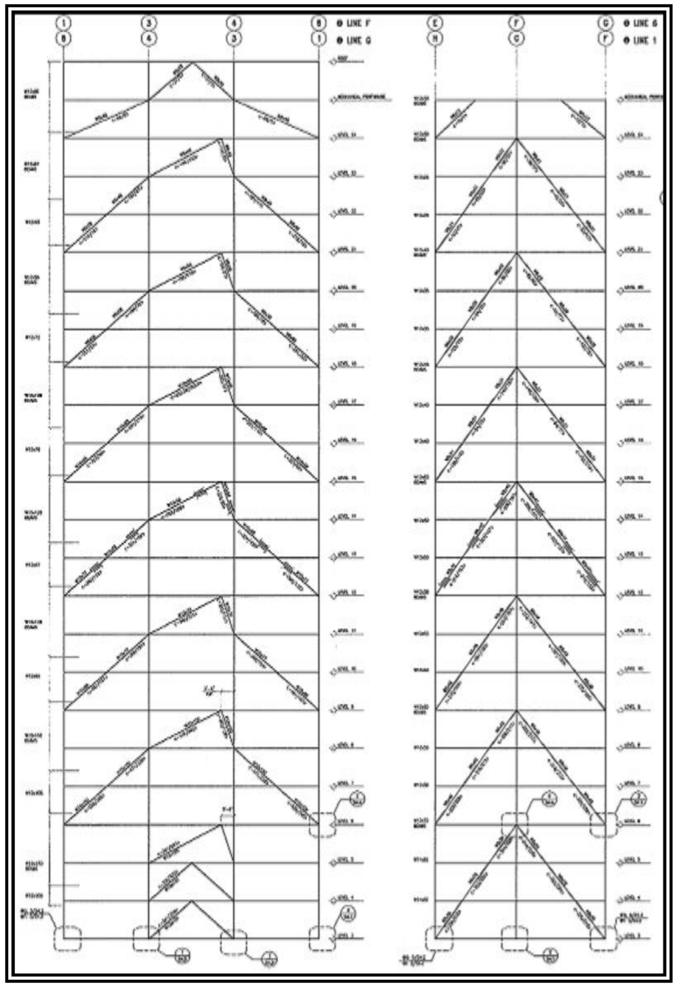


Figure 7: Braced frame layouts. (Left) N-S Braces (Right) E-W Braces

Proposal

Problem Statement

Due to the needs of the client, the companies involved and their drive to provide quality products, the design of this building is the most efficient steel system for the requirements of this project. This was found to be true when investigating the lateral system against the wind and seismic loads prescribed in ASCE7-02 for Technical Report 3. However when investigating alternate floor systems it was found that even though the open-web steel joist floor system is extremely light and inexpensive there may be problems with walking induced vibrations. For these reasons I intend to propose a material alteration of the structural system of both the gravity and the lateral systems from a steel system to a concrete system. In lieu of this alteration I will redesign the floor and lateral systems using concrete instead of steel in a way that will reduce floor vibration, provide space for MEP components, and will not disrupt the architecture or the current floor plans. I will determine whether the new systems are more efficient based on the criteria of overall cost, construction time, ease of construction, susceptibility to vibration, overall building weight, and coordination with the other trades.

Structural Redesign

Floor System

Alternate 1: The first floor system that was investigated is a two-way flat plate reinforced concrete system. This system will consist of a 10" reinforced concrete slab that frames into 16" square columns. The floor to floor height will remain the same as the original design at 10'-8" which means that there is a ceiling cavity of 4" for any MEP equipment.

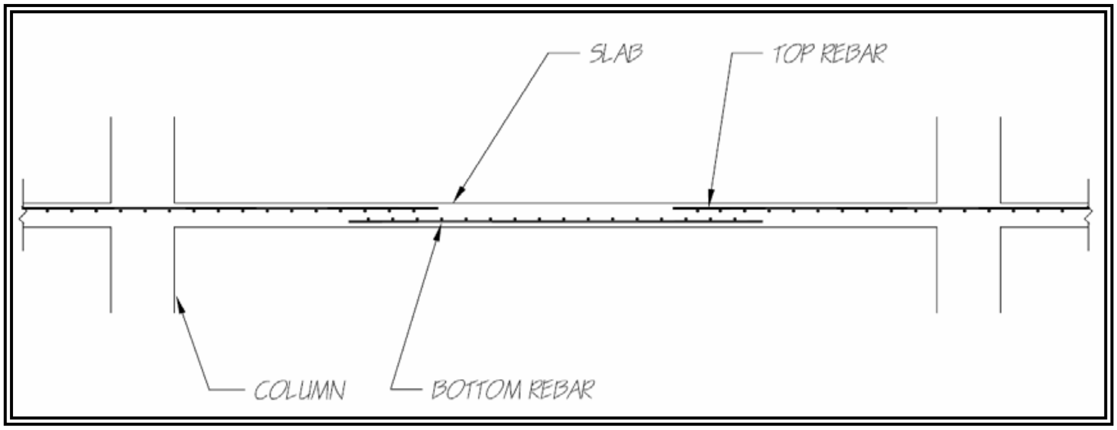


Figure 8: Cross Section of Two-way Flat Plate System

Alternate 2: The second floor system investigated was a two-way post tensioned flat plate concrete system with similar sized columns. It is the intent of this system to reduce the thickness of the floor slab to provide a larger ceiling cavity for MEP equipment. Reducing the thickness of the floor slab will reduce the dead loads into the columns and the punching shear in the slab and thus allow for smaller column sections and less overall weight that the foundation needs to support.

Design Gravity Loads

The gravity loads used for the design of both the reinforced concrete flat-slab and the post-tensioned flat-slab were gathered from ASCE7-02.

Dead Loads:

Concrete (incl. rebar)	150 PSF
Terrace Pavers	25 PSF
Ceiling - Units	10 PSF
Ceiling - Parking	3 PSF
Flooring	4 PSF
Roofing	7 PSF
Mechanical Room	250 PSF

Live Loads:

Floor – Private	40 PSF
Floor – Public	100 PSF
Balconies – if < 60 ft2	60 PSF
Balconies – if > 60 ft2	100 PSF
Fire Escapes	100 PSF
Corridors	100 PSF
Garage	40 PSF
Roof	20 PSF
Partitions	15 PSF

Snow Loads:

Roof Load	25 PSF
Drifting	25 PSF

Shear Design

Prior to beginning the reinforced concrete design, the CRSI Design Guide and the ACI code were consulted to determine preliminary sizes for the columns and the floor slabs. These preliminary estimates were based on loading, bay sizes and deflection limitations. The deflection limitations from Table 9.5(c) in the ACI dictated a 10" minimum slab depth corresponding with the 26' bays. Thinner slab sizes would be possible for the smaller bays but it was decided to stay with 10" slab for the entire floor as it would be easier during construction. Confronting the CRSI with the knowledge of a 10" slab, 26' bay size, and a superimposed load of about 50 PSF an initial column size of 14" square was determined for an interior column. It was decided to use a 15" column and concrete with an f'_c = 5000 PSI to account for the variety of loading schemes from balconies and corridors. The following equations from ACI chapter 11 were used to check and verify this decision.

Shear Loading:

$$V_u = w_u*l_1*l_2/2 \qquad \text{(Eq. 1a)}$$

$$V_u = w_u*l_1*l_2 \qquad \text{(Eq. 1b)}$$

Wide Beam Shear:

$$V_c = \Phi*2*\sqrt{(f'_c)}*b_w*d \qquad \text{(Eq. 2)}$$

Punching Shear:

$$V_c = \Phi*(2+4/\beta_c)*\sqrt{(f'_c)}*b_o*d \qquad \text{(Eq. 3)}$$

$$V_c = \Phi*(\alpha_s*d/b_o+2)*\sqrt{(f'_c)}*b_o*d \qquad \text{(Eq. 4)}$$

$$V_c = \Phi*4*\sqrt{(f'_c)}*b_o*d \qquad \text{(Eq. 5)}$$

When designing the post-tensioned concrete floor system the same column sizes were used as in the reinforced concrete design but another approach was used to determine a preliminary slab thickness. Since wide beam shear rarely controls and the columns were the same dimensions in both directions equations 2 and 3 will not control, and equation 4 requires a great deal of information so it was used later as a design check. This leaves equations 5 and equation 6 (below) which were rewritten as a function of 'd' in relation to f'c of the concrete and the factored loads then solved by graphing against equation 1b for the minimum value of 'd'. This procedure is shown below using equation 5 as an example:

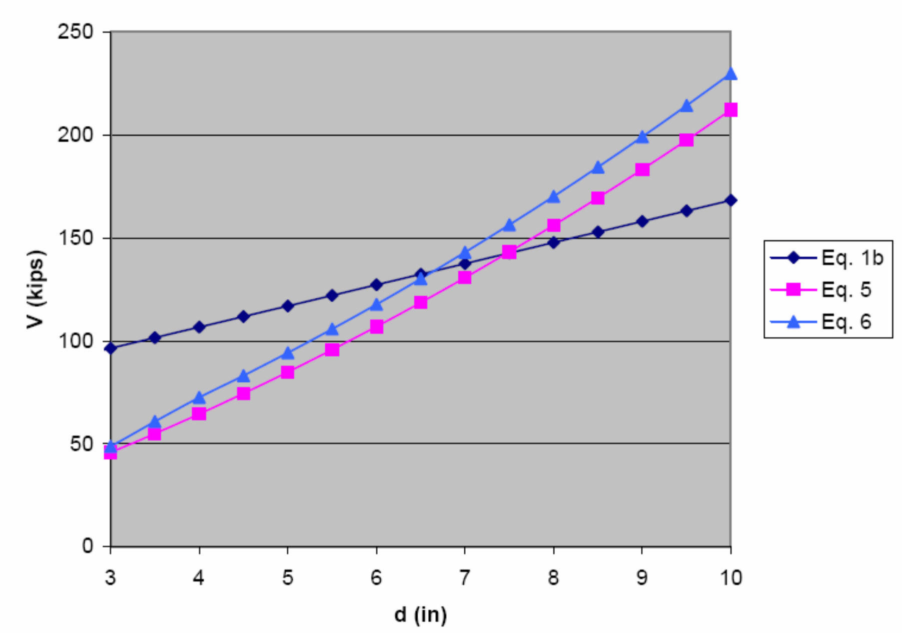
$$V_c = \Phi^* (\beta_p^*\sqrt{(f'_c)} + 0.3^* f_{pc})^*b_o^*d + V_p \qquad \text{(Eq. 6)}$$

(1) $w_u = 1.2^*(12.5^*(d+1.0)+SDL) + 1.6^*(LL)$

(2) $V_u < V_c$

(3) $w_u^*l_1^*l_2 < \Phi^*4^*\sqrt{(f'_c)}^*b_o^*d$

(4) $(15^*\mathbf{d}+22.5+1.2^*SDL+1.6^*(LL))\ ^*l_1^*l_2 < \Phi^*4^*\sqrt{(f'_c)}^*(60+4^*\mathbf{d})^*\mathbf{d}$



This approach is applicable because the post-tensioning creates forces in the slab that counter the dead loads thus greatly reducing the deflections under service loads, as well as creating a vertical force in the area around the column that opposes the shear forces which is demonstrated in the following diagram. The minimum depth of 'd' was found to be 6.5" by equation 6 so the minimum slab depth, assuming 1.0" of cover, is then 7.75” but for ease of construction and to provide greater sag in the tendons a thickness of 8.0” was used. This value was then checked against equations 2, 3, and 4 and found to be sufficient for resisting punching shear and wide beam shear.

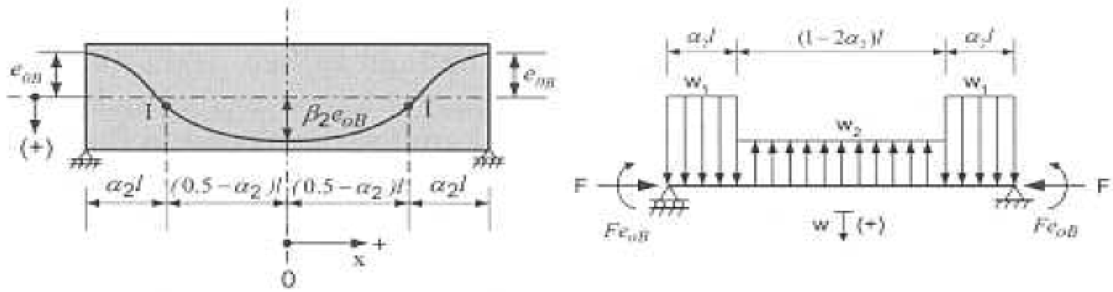


Figure 9: Parabolic tendon profile (left) and the induced forces from that profile (right).

Flexural Design

The floor system was designed a number of ways. The first two ways considered the floor slab as just a gravity force resisting element thus designing it to resist flexure from the design live and dead loads. The floor slab was first designed as a reinforced concrete flat-plate using the Direct Design method (ACI ch. 13.6) and then designed again using the Equivalent Frame analysis (ACI ch. 13.7). These two methods were then compared based on the design moments and the time to calculate each. The Direct Design method is much faster than the Equivalent Frame analysis, but it calculates lower negative moments at interior columns and therefore is un-conservative at these critical sections. The Equivalent Frame analysis, on the other hand, is un-conservative at exterior columns and mid-span of interior beams.

		Span 1		Span 2		Span 3	
		Total	Per Foot	Total	Per Foot	Total	Per Foot
Direct Design							
Interior	CS	274	21.1	254	19.5	274	21.1
	MS	92	7.1	85	6.5	92	7.1
Middle	CS	163	12.5	110	8.5	163	12.5
	MS	108	8.3	73	5.6	108	8.3
Exterior	CS	136	10.5	---	---	136	10.5
	MS	0	0.0	---	---	0	0.0
Equivalent Frame							
Interior	CS	314	24.2	293	22.5	314	24.2
	MS	105	8.1	98	7.5	105	8.1
Middle	CS	165	12.7	80	6.2	165	12.7
	MS	110	8.5	53	4.1	110	8.5
Exterior	CS	103	7.9	---	---	103	7.9
	MS	0	0	---	---	0	0

Table 1: Comparison of design moments calculated using Direct Design and Equivalent Frame Analysis

The third way the floor slab was calculated was as a post-tensioned flat-plate system. Considering the slab to be post-tensioned allowed for a thinner slab with greater section properties than a regular reinforced slab. By draping the tendons in a parabolic shape they produce a uniformly distributed load in the slab that acts in opposition to the gravity loads which reduces the deflection of the slab under service loading cases. The precompression of the slab also eliminates tension cracks, which means that the entire cross section is utilized to resist moments caused by live, dead, and even lateral loads.

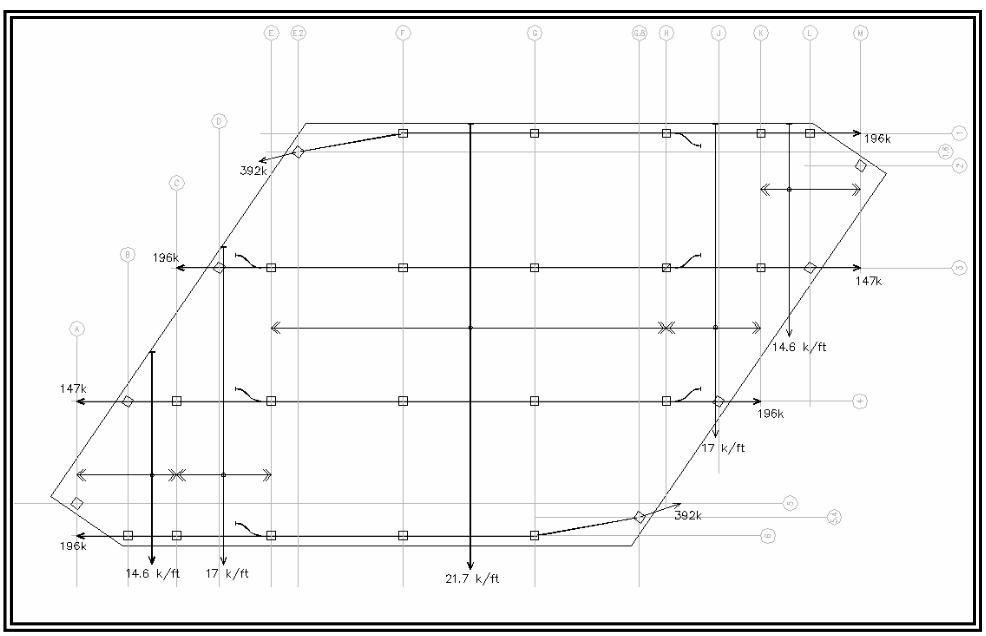


Figure 10: Typical post-tensioning tendon layout.

The fourth, and final, floor slab design considered the slab to be an integral part of the lateral force resisting frame. For this design, ADOSS was used to perform a frame analysis to determine the design moments on the slab due to the gravity loads and the controlling wind cases.

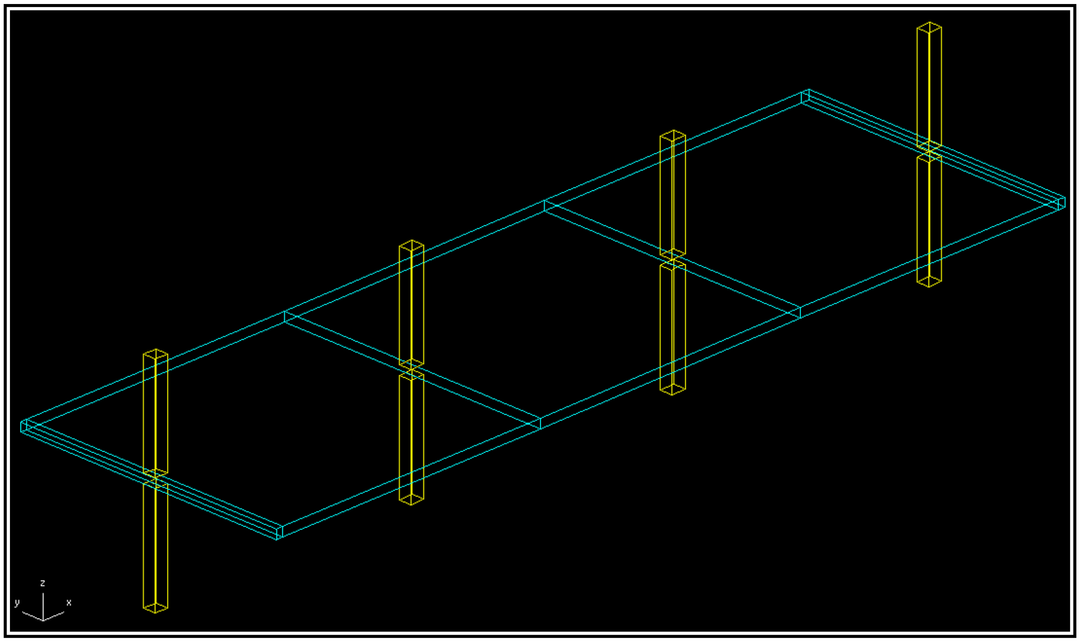


Figure 11: Isometric view of a typical N-S frame.

Columns

The initial column sizes were found during the shear design in the CRSI Handbook to be 14" square. It was decided that a 15" square column would be a better choice because of shear due to the various loading schemes from interior floor loads, terrace loads, and the loads due to the frame acting as a moment frame. The columns were designed initially for the factored axial loads that they would experience and it was found that the columns became very large and were an intrusion to the open nature of the floor plan. A second design iteration was performed using higher strength concrete and this was enough to decrease the columns to more reasonable sizes. A comparison of these sizes can be seen in Table 2 below. Due to the large axial forces on the columns towards the lower floors, they did not have to be altered to accommodate the additional moment caused by

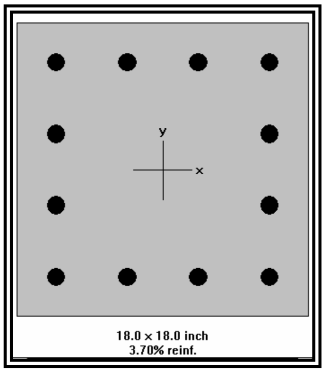


Figure 12: Column rebar design.

the frame acting to resist lateral forces. The columns, in the upper stories, where the lateral forces were greater and the axial forces lower, required additional rebar to account for the magnified moments caused by sway frames.

Column G-3, G-4, F-3, F-4 Below:	P_u (kips)	Initial Sizes		Final Sizes	
		Size	f'_c	Size	f'_c
Roof	116.3	15.0	5000	15.0	5000
Mechanical	405.3	15.0	5000	15.0	5000
24	516.6	15.0	5000	15.0	5000
23	627.8	15.0	5000	15.0	5000
22	741.8	15.0	5000	15.0	5000
21	855.9	15.0	5000	15.0	5000
20	969.9	16.0	5000	15.0	6000
19	1084.0	18.0	5000	16.0	6000
18	1198.0	18.0	5000	16.0	6000
17	1312.1	20.0	5000	16.0	6000
16	1426.2	20.0	5000	18.0	8000
15	1540.2	20.0	5000	18.0	8000
14	1654.3	22.0	5000	18.0	8000
13	1768.3	22.0	5000	18.0	8000
12	1882.4	22.0	5000	20.0	8000
11	1996.4	24.0	5000	20.0	8000
10	2110.5	24.0	5000	20.0	8000
9	2224.6	24.0	5000	20.0	8000
8	2338.6	26.0	5000	22.0	8000
7	2452.7	26.0	5000	22.0	8000
6	2566.7	26.0	5000	22.0	8000
5	2680.8	26.7	5000	22.0	8000
4	2791.5	27.3	5000	22.0	8000
3	2902.3	27.8	5000	30.0	8000
2	3013.1	28.3	5000	30.0	8000
Mezz.	3094.6	28.7	5000	30.0	8000

Table 2: Column sizes.

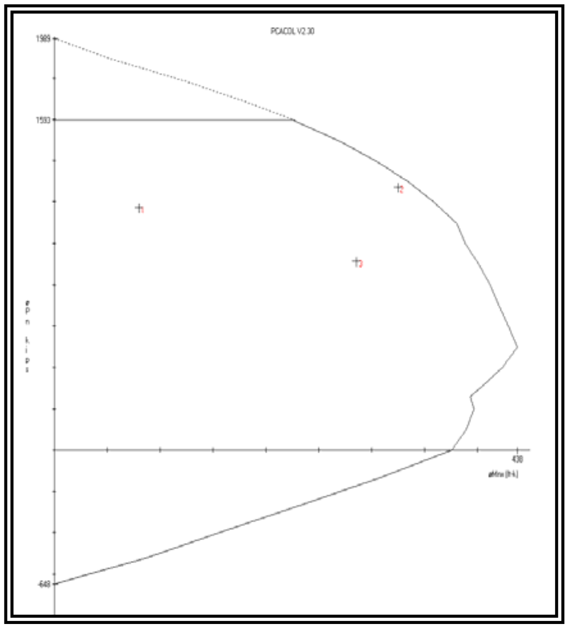


Figure 13: Column interaction diagram.

Lateral System

The first alternative lateral system that was investigated is a concrete shear wall system positioned around the elevator core that continues from the foundation through all levels of the building to the roof. The other alternative that was investigated is a system where the shear walls and slab-frame are working together to resist the lateral forces. This is possible for very little extra cost due to the fact that the columns, slabs, and shear walls are all poured monolithically which provides the moment connections needed for the frame to act integrally with the shear walls.

Lateral Design Loads

The lateral systems were analyzed for the wind and seismic loading schemes set forth by the Analytical Method from ASCE7-02 chapter 6 and the Equivalent Lateral Force Method from ASCE7-02 chapter 9, respectively. The lateral loads determined from these industry accepted procedures were then be put into an ETABS model of the building where the forces in each component will be calculated based on relative stiffness. The walls were then designed based on the worst load combination of gravity and lateral forces. Below are the lateral forces for each of the wind cases and the seismic forces for the shear wall and combined system. The criteria for calculating these loads can be found in appendices A2 through A4.

	Wind								Seismic			
	Case 1		Case 2		Case 3		Case 4		10" Slab		8" PT Slab	
Story	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	Shear Walls	Frame	Shear Walls	Frame
Roof	22.05	11.55	16.54	8.66	16.54	8.66	12.41	6.50	14.80	16.44	12.08	13.43
Mechanical	36.39	34.62	27.29	25.96	27.29	25.96	20.49	19.49	41.60	46.23	37.49	41.66
24	39.60	26.87	29.70	20.15	29.70	20.15	22.29	15.13	29.82	33.13	24.76	27.51
23	38.58	26.22	28.93	19.66	28.93	19.66	21.72	14.76	27.45	30.50	22.79	25.33
22	37.75	25.75	28.32	19.31	28.32	19.31	21.26	14.50	25.19	27.98	20.91	23.24
21	43.85	25.75	32.89	19.31	32.89	19.31	24.69	14.50	25.47	28.30	21.15	23.50
20	43.85	25.75	32.89	19.31	32.89	19.31	24.69	14.50	23.18	25.76	19.25	21.39
19	43.85	25.75	32.89	19.31	32.89	19.31	24.69	14.50	21.00	23.33	17.43	19.37
18	42.86	25.26	32.15	18.95	32.15	18.95	24.13	14.22	20.74	23.05	17.22	19.14
17	41.88	24.77	31.41	18.58	31.41	18.58	23.58	13.95	18.58	20.65	15.43	17.15
16	41.39	24.53	31.04	18.40	31.04	18.40	23.30	13.81	16.55	18.38	13.74	15.26
15	41.14	24.41	30.85	18.31	30.85	18.31	23.16	13.74	14.62	16.25	12.14	13.49
14	40.15	23.92	30.12	17.94	30.12	17.94	22.61	13.47	12.82	14.25	10.65	11.83
13	40.15	23.92	30.12	17.94	30.12	17.94	22.61	13.47	10.99	12.21	9.12	10.14
12	39.17	23.44	29.38	17.58	29.38	17.58	22.05	13.19	9.57	10.64	7.95	8.83
11	39.17	23.44	29.38	17.58	29.38	17.58	22.05	13.19	8.13	9.03	6.75	7.50
10	37.94	22.83	28.45	17.12	28.45	17.12	21.36	12.85	6.80	7.55	5.64	6.27
9	37.53	22.62	28.14	16.97	28.14	16.97	21.13	12.74	5.59	6.21	4.64	5.15
8	36.34	22.04	27.25	16.53	27.25	16.53	20.46	12.41	4.50	5.00	3.73	4.15
7	35.60	21.67	26.70	16.25	26.70	16.25	20.04	12.20	3.52	3.91	2.93	3.25
6	36.23	22.16	27.17	16.62	27.17	16.62	20.40	12.48	2.60	2.89	2.15	2.39
5	34.14	21.00	25.61	15.75	25.61	15.75	19.22	11.82	1.83	2.03	1.51	1.68
4	30.71	19.01	23.04	14.25	23.04	14.25	17.29	10.70	1.14	1.27	0.92	1.02
3	30.05	18.72	22.53	14.04	22.53	14.04	16.92	10.54	0.73	0.81	0.59	0.66
2	35.51	22.33	26.63	16.75	26.63	16.75	19.99	12.57	0.40	0.45	0.33	0.36
Mezzanine	39.85	25.39	29.89	19.04	29.89	19.04	22.43	14.29	0.03	0.04	0.03	0.03
Ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 3: Wind and seismic forces.

Shear Walls

The building was first designed as if shear walls were going to resist the lateral forces caused by the wind and seismic forces. In doing so a drift limit of ℓ/400 was imposed to prevent cracking of the façade; this limit was found in *ASCE7-02 CB.1.2 Drift of Walls and Frames*. The original intent was to have the shear walls around just the elevator core, but not only was this not enough to limit the deflections to the drift limit but the setbacks of the upper stories created a large amount of torsion that needed to be controlled. Therefore, more shear walls were needed. The architectural plans were consulted to determine likely places to add shear walls without disrupting the current floor plan. It was not initially evident where these additional walls would go because of the variation of the floor plans. It was finally decided to put them along same column lines that some of the original braces were located with openings in them at intermittent floors for doorways between the different rooms of the condominiums. This design was enough to bring the drift limit down to ℓ/560 which is considerably less than the required ℓ/400.

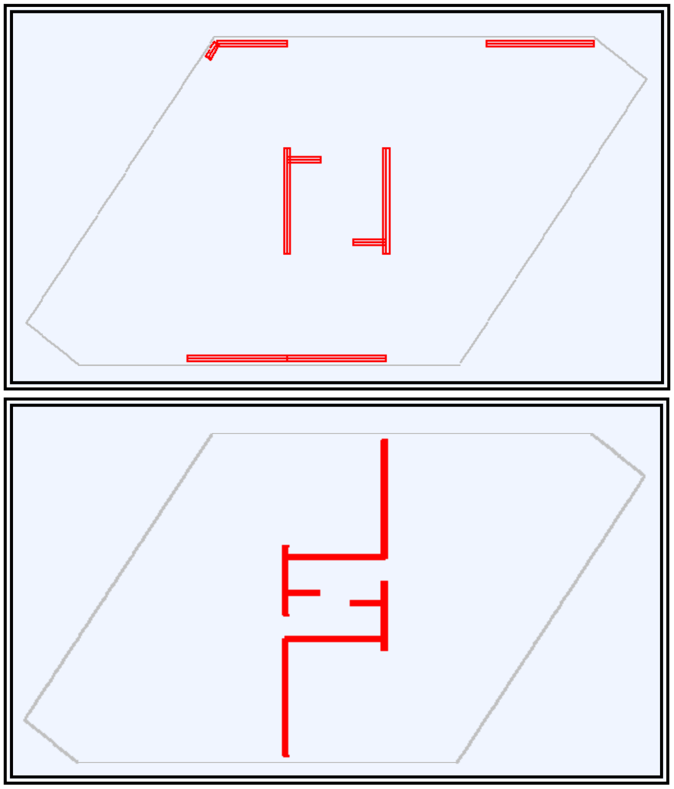


Figure 14: Final shear wall locations (top) stories 1-5 and (bottom) stories 6 to the roof.

Frame System

A frame system was initially investigated to attempt to reduce the materials needed for construction by combining two different types of lateral force resisting systems, the frame and the wall. By introducing the frame system, less shear walls may be required to satisfy the drift limit. This would be true due to the inherent nature of each of the two systems. The frame system deflects in shear and the shear walls defect in flexure. When these two systems are combined they produce a double curvature in the deflected shape of the building which is much stiffer than either of the two systems alone. This interaction is demonstrated below:

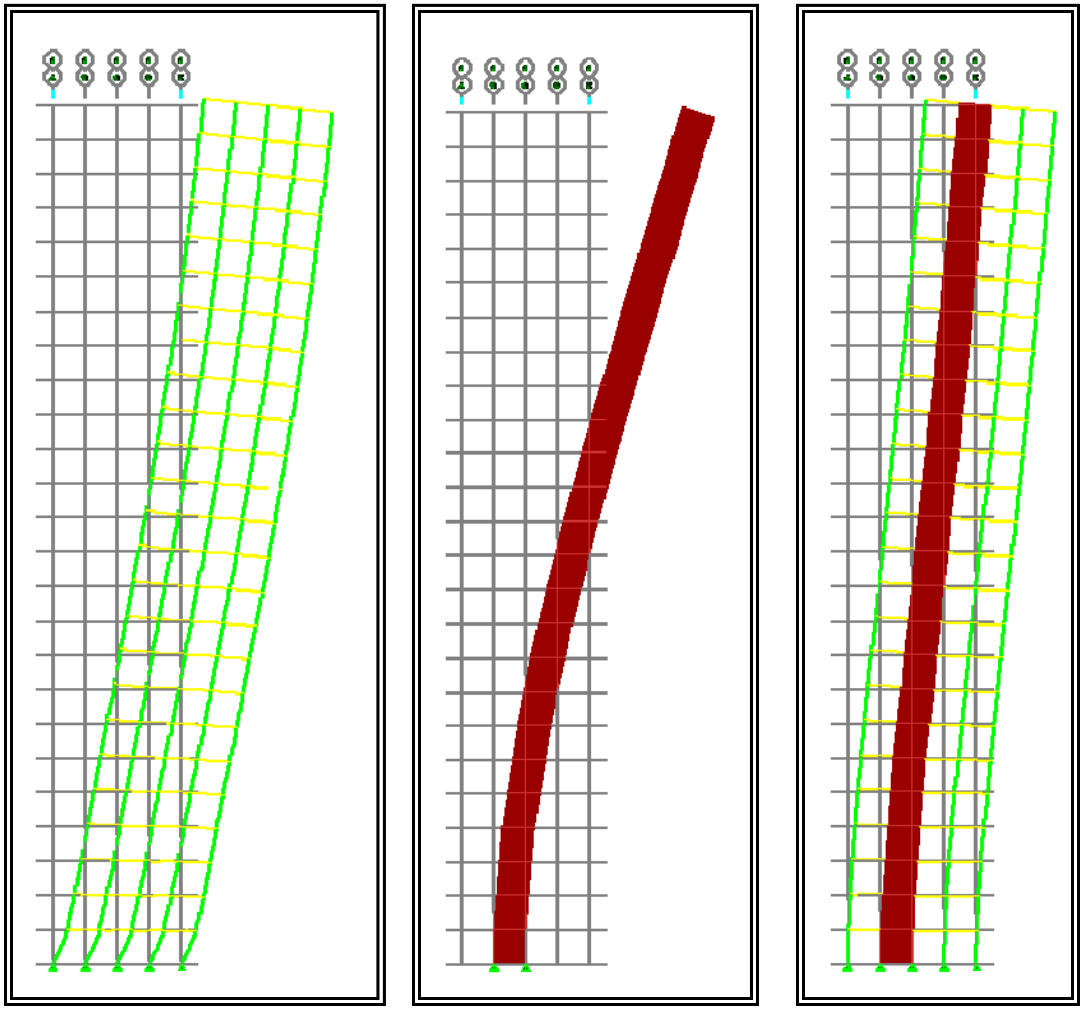
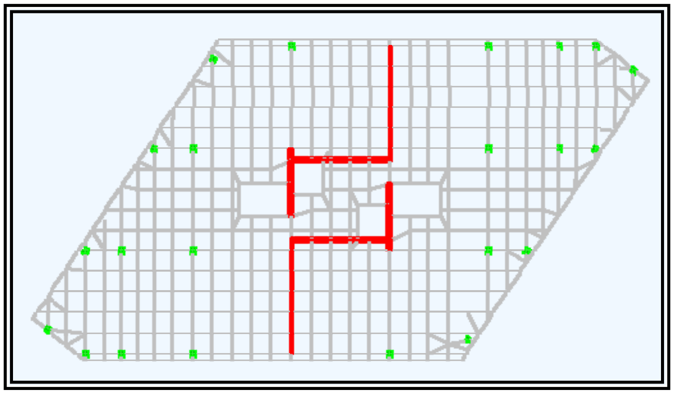


Figure 15: (Left) Frame reaction to lateral forces.
(Middle) Shear wall reaction to lateral forces.
(Right) Deflection of integrated shear wall and frame system.

This frame system was first investigated using the 10" flat-plate floor system and 15" square columns from the roof to the 5th floor and 30" columns from there to the ground floor. By incorporating the frame into the model used to determine the shear walls, the drift drastically reduced from the ℓ/560 with just the walls to ℓ/1015. From this point shear walls were removed until the total drift was ℓ/840. Removing shear walls beyond this point created either excessive deflection or a severe torsional state.



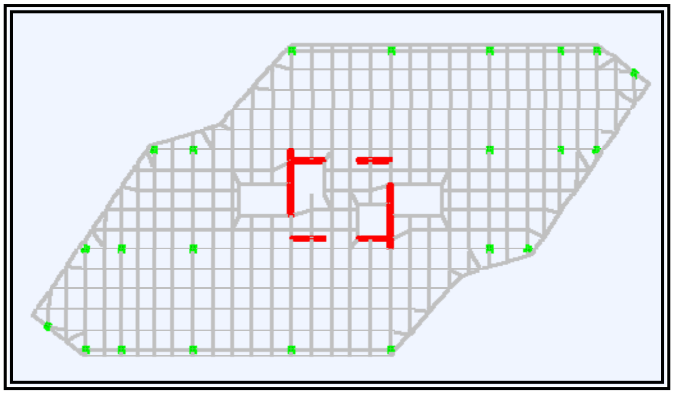
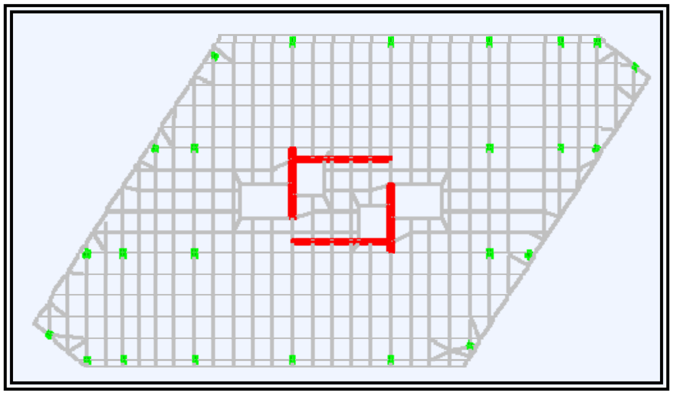


Figure 16: Final shear wall locations (top) stories 5-15 and (bottom) stories 16 to the roof.

The second frame system investigated used the 8" post-tensioned flat-plate system with the same column arrangement as the prior frame. The greatest difference between this system and the previous frame system is that the floor is much stiffer because the post-tensioning allows for the full moment of inertia of the slab to act in bending. From the shear wall model the addition of the post-tensioned floor slabs reduced the drift from the original ℓ/560 to ℓ/1180. This was greater than the reduction of the reinforced concrete floor system



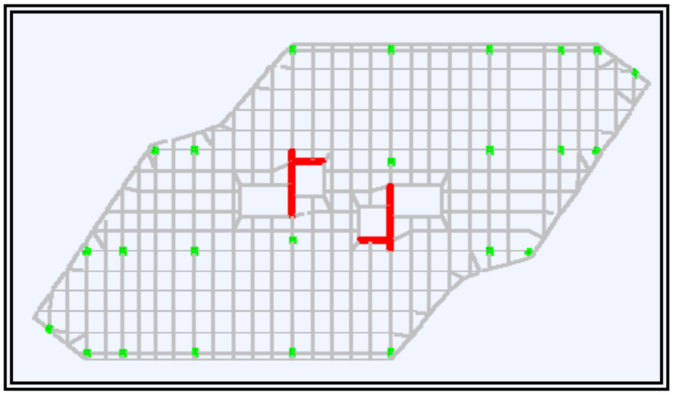


Figure 17: Final shear wall locations (top) stories 5-15 and (bottom) stories 16 to the roof.

of the previous model, as expected. In this model two entire shear walls were removed and still the overall drift was ℓ/510 which is well within the limit of ℓ/400.

The shear wall layout of the three separate designs is further demonstrated below. The image on the left shows the location of the shear walls of the model in which the shear walls alone are resisting the lateral forces. The image in the middle is of the flat-plate design where the frame and shear walls are acting together. Finally, the image on the right is of the post-tensioned, flat-plate floor system with the frame and shear walls acting integrally. Notice in the PT model how many fewer shear walls are required to meet the drift limits because of the floor slab being much stiffer than the conventionally reinforced flat-plate.

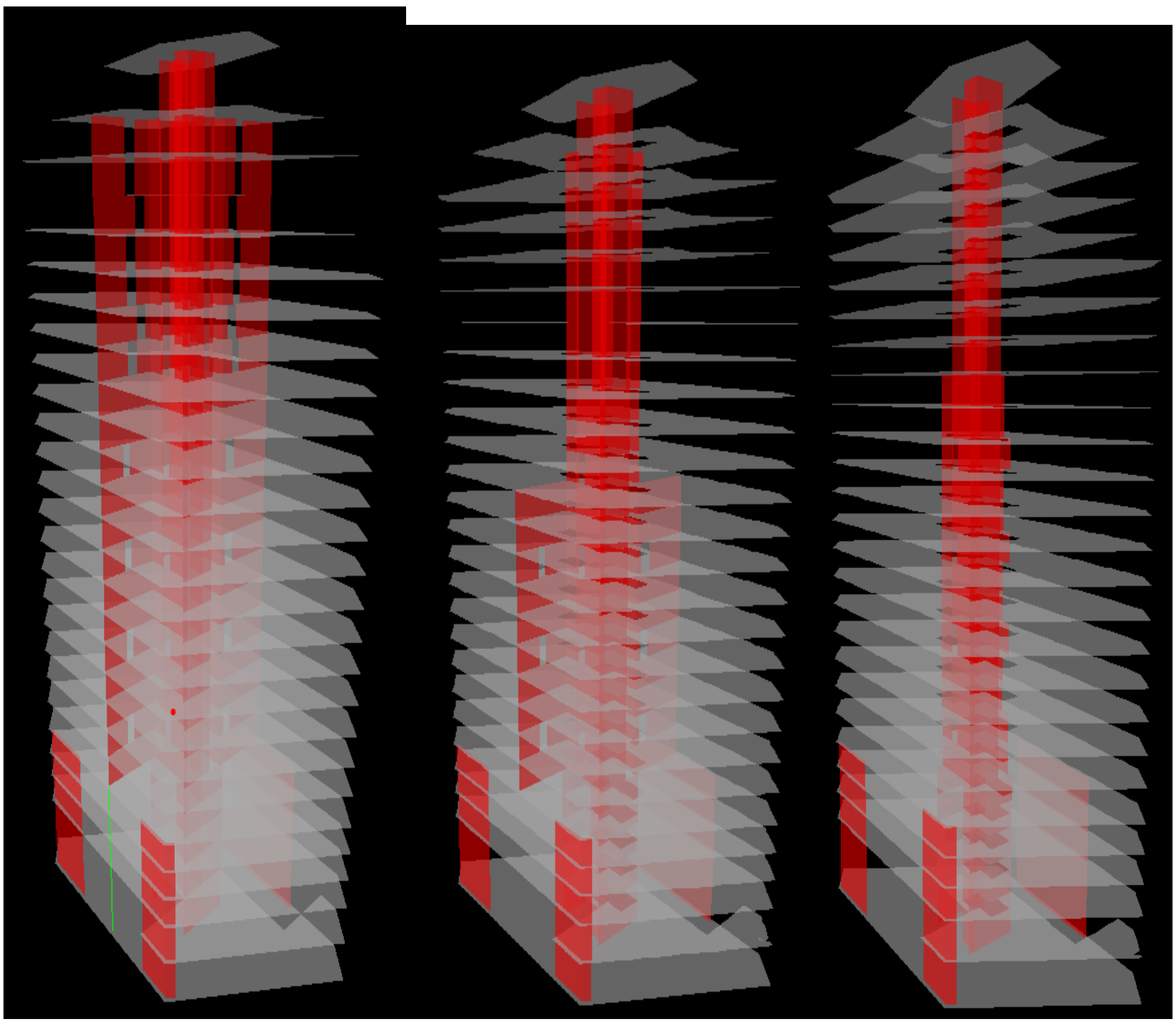


Figure 18: Shear wall layout of shear wall system (left), RC frame (middle), and PT frame system (right)

Breadth Studies

Construction Management

Changing the structural system of the building led to inherent changes in the constructability, construction schedule, and cost of 'Erie on the Park'. Probably due to the fact that steel is a long lead item and would inevitably push back the start date of construction of the superstructure, the first three stories have been designed and constructed in concrete. This allows construction to proceed earlier than it would have if the entire structure were constructed in steel, thus moving up the date of top-out and similarly the completion date. The drawback with this strategy is the interface between the steel and concrete systems. The tolerances for anchor rod placement in concrete are on the order of 1" where as the tolerances for steel are in the range of 1/8". This discrepancy can cause time delays due to initial rod placement or in fixing incorrect rod placement. Building the structure with one material, steel or concrete, would alleviate this system interface issue.

A constructability drawback to post-tensioned concrete systems is that all the other trades (i.e. mechanical, electrical, plumbing, telecommunication) have to know where their systems are going to require openings in the floor slab. This is so the tensioning tendons can be designed and laid out so they do not interfere with these openings and are still able to support the floor slab. This requires the other trades to be brought onto the project earlier so that they can design their system and where the through slab chases will be before the structural engineer finishes the design of the slab. This is necessary because drilling through a post-tensioned slab is not typically allowed due to the possibility of cutting a tensioning strand in the process.

After designing the alternative structural systems a cost estimate and construction schedule were determined for the initial steel system, a flat-plate with frame and shear wall interaction, and a post-tensioned flat-plate also with frame and shear wall interaction.

Design	Materials	Labor	Equipment	Cost	$ per SQFT	Incl O&P	$ per SQFT
Original	$3,224,006.22	$1,403,984.64	$254,026.53	$4,882,017.40	$22.91	$5,370,219.14	$25.21
RC Frame	$1,817,819.38	$2,013,858.70	$71,573.95	$3,903,252.03	$18.32	$4,683,902.44	$21.98
PT Frame	$1,612,342.94	$1,975,439.01	$69,008.05	$3,656,790.00	$17.16	$4,388,147.99	$20.60

Table 4: Cost breakdown for each structural system

	Steel		Decking		Floor Slabs		Concrete Columns	
Design	Crews	Duration	Crews	Duration	Crews	Duration	Crews	Duration
Original	1 - E2	105	2 - E4	24	2 - C8	42	1 - C14A	10
RC Frame	0	0	0	0	3 - C2	162	1 - C14A	45
PT Frame	0	0	0	0	2 - C2	194	1 - C14A	45

	Shear Walls		Post-Tensioning		Total Duration		Total
Design	Crews	Duration	Crews	Duration	Days	Months	Crews
Original	2 - C2	20	0	0	140	7	8
RC Frame	2 - C2	107	0	0	167	8.4	6
PT Frame	2 - C2	80	2 - C4	50	199	10	7

Table 5: Duration and crews per task for each structural system

LEED/Mechanical

Designing a building so that it is environmentally friendly has become a concern of owners, contractors, designers, and tenants over the past decade. Each has their own reasons for this movement to make 'green' buildings. Building owners are interested in this because they are able to save money on utilities by making their buildings more water and energy efficient. Contractors and designers are interested in green buildings because few of them are certified and experienced in building and designing these buildings so there is less competition for these contracts and they are able to charge a premium for their services. Tenants want green buildings because of the better materials which lead to fewer costs of replacing defective materials. Companies are profoundly interested in leasing space in green buildings because the higher quality of the indoor environment produces workers who are more content and more efficient.

When designing a green building there are five areas where designers focus their efforts to reduce the use of materials and provide a better indoor atmosphere. These focuses are planning for a sustainable site, water efficiency, energy and atmosphere, materials and resources, and the quality of the indoor environment. The largest green building certifying agency in the United States is the U. S. Green Building Council (USGBC) and if a building is designed to meet their criteria it would receive a LEED (Leadership in Energy and Environmental Design) rating. There are four levels of ratings that a building could obtain depending on how many of the criteria are met: Certified, Silver, Gold, and Platinum. The USGBC claims that a Silver rating is within the margin of error of the original cost estimate.

The intent of this case study was to collect the rain water that falls on the roof and investigate the design and cost implications if it were to be used for the toilets in each of the condominiums and common areas. The average rainfall per year in Chicago is 36.5”. The monthly average is about 3”, with August receiving the most rainfall of 4.62” and February receiving the least amount with only 1.63”. The rainfall would be collected in two large cisterns located below grade next to the elevator pits and since that area needs to be excavated already additional excavation costs are minimal. The cisterns have been designed for the 5 year, 60 minute design storm and have a capacity of 10,000 gallons each. Should the storm produce rain in excess of this amount, there will be an overflow drain which allows the extra water to flow into the sewer using the same system that is already in place for the grey water that the building produces. The water that is collected will be pumped to a 400 gallon tank in the mechanical space on the 25th floor by two 5 HP pumps that work in series to overcome the head pressure. From there it will drain down into the toilets. Since there is not enough rain during the year to fully supply the toilets with water, and since rainfall is unpredictable, this system will have to run in parallel with a utility water supplied system.

Assumptions

80	gal/person/day
330	residents
256	toilets
4	flushes/toilet/day
1.6	gal/flush
7.48	gal/ft3
36.5	in rain/year
18300	ft^2 roof area

	Total Usage	Toilet Usage	Rain
Day	26400	1638.4	---
Month	792000	49152	34221
Year	9636000	598016	416355.5
% of Total	100	6.206	4.321

Table 6: Water usage (in gallons)

Equipment	Size	Quantity	Cost/Item	Total
Cistern	10000 gal	2	$10,000.00	$20,000.00
Compression Tank	400 gal	1	$5,250.00	$5,250.00
Pump/Controls	5HP 3500RPM	2	$5,500.00	$11,000.00
Piping	3/4"	2500	$7.70	$19,250.00

Table 7: Cost for the rainwater collection system components.

	Base Cost	Incl. O&P
Cost of System	$55,500.00	$63,825.00
% of Building Cost	0.1088	0.1251

Table 8: Overall cost for the rainwater collection system.

Savings per Year	Quantity	Cost/gal	Total	%
Water	9636000	$0.002434	$23,453.06	---
Rain	-416355.5	$0.002434	-$1,013.37	4.321

Table 9: Yearly water and cost savings

Implementing this system costs $64,000 but it reduces the water usage by 4.3%, saving about $1,000 each year on utility costs. This system also satisfies LEED points which would get the building that much closer to being certified. The first LEED point this satisfies is Sustainable Site 6.1 which is storm water quantity control. This requires a 25% reduction of storm water runoff during a two year, 24 hour design storm. The system, as designed, would reduce the runoff of this design storm by 47.4%. The other LEED point that this system begins to satisfy is Water Efficiency 3.1. This point requires a 20% reduction in water usage and since no-one can adjust how much it rains this collection system only reduces the water usage by 4.3% or 1/5th the required amount. This is a giant step in the direction of achieving this LEED point, though. The rest of the water use reduction would have to be achieved by using high-efficiency fixtures possibly with motion sensors to reduce the potable water demand.

Conclusion

Recommendation

After investigating the four different systems I believe I can make a reasonable determination of which would be the best suited for the 'Erie on the Park' condominium complex. The steel system has a high potential for walking vibration issues and it is more expensive than any of the other systems, but it has the fastest erection schedule. The post-tensioned, frame-wall dual system has a floor slab that is heavy enough that there will not be any vibration from tenants walking about. It is also the most cost effective system, but the construction process takes the longest of the different systems due to tensioning of the PT strands. Of the other two systems neither of them will have walking vibration problems and their overall cost and construction schedule are between those of the steel and the PT frame systems.

There are other things to consider than just the cost and schedule. For instance, changing the structural system from a steel system to a concrete system increased the overall weight of the building by 50%. This additional weight is a huge concern when designing the foundation in an area like Chicago where the soil has little bearing capacity because of the river. In some cases this increase of weight may be the difference between a shallow foundation and a deep foundation which is an enormous cost increase. Since this building already has a deep foundation, the cost increase of strengthening the caissons would be minimal in comparison to switching from a shallow foundation. There are also the other trades to consider, such as the mechanical, lighting, electrical, and plumbing. In the original design they were run in the ceiling cavity, through the open-web joists. In the redesigned systems it is still possible to have these utilities run above the ceiling but either the ceiling itself will have to be lowered to increase the ceiling cavity depth or the overall building height will have to be increased to maintain the same floor to ceiling height. The second of those options is an option, but not one that I plan on entertaining when, for the PT frame system, the ceiling only needs to be lowered 2".

System	Steel	Shear Walls	RC Frame	PT Frame
Vibration	Yes	No	No	No
Cost	25.21	---	21.98	20.6
Schedule	7	---	8.4	10
Weight	19680	33250	32880	27560
Ceiling Cavity	12	4	4	6
Ease of Construction	Easy	Easy	Easy	Medium

Table 10: System comparison summary

I believe that the post-tensioned flat-slab working integrally as a frame with the shear walls is the best system for this building. I believe this because it is the least expensive system, they are already using a deep foundation, it is not as heavy as the other two concrete systems, and the additional construction time could be made up elsewhere in the overall building schedule. Most of all, though, is that this system truly offers the architects at Lucien Lagrange and the owner flexibility in the floor plan and totally unobstructed views of the city. Although the steel braces provide some flexibility they still have to be considered when designing the façade and laying out the interior walls. With the PT slab and integrated lateral system the shear walls are all confined to the center of the building near the elevator core and only columns dot the exterior of the floor plan.

Acknowledgements

I would like to thank all the design professionals who allotted their time to help me out with acquiring information necessary to the completion of this thesis project. I would like to extend my gratitude to all of the professors of the Architectural Engineering department who have been exceptional teachers as well as mentors over the few past years. Lastly, I would like to thank my family and friends for all your support and for keeping me sane during all of this. Thank you!