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Structural Option
Advisor: Walter Schneider / Ali Memari
110 Third Avenue
110 Third Avenue
New York, NY 10003
Spring 2006

Final Report

Design and Analysis of 110 Third Avenue

Tony Nicastro
Structural Option
110 Third Avenue
New York, NY 10003

110 Third Avenue

Project Team

OWNER: TOLL BROTHERS INC.

ARCHITECT: GREENBERG FARROW

STRUCTURAL ENGINEER: AXIS DESIGN GROUP

MEP ENGINEER: MGJ ASSOCIATES INC.

GEOTECHNICAL ENGINEER: LUNGAN

CM AGENT: TISHMAN CONSTRUCTION

CONSULTANT: LZA/THORTON-TOMASETTI

Structural System

•CIP concrete system
•8” two-way slab system
•Loads are carried from the two-way slab system to concrete columns ranging from 12x12 to 40x12
•Concrete columns recessed from perimeter approximately 10” to allow for non-bearing exterior panels
•The only beams present in the structure surround the elevator core and stairwell, and also grade beams in the basement level that extend to the face of the building.
•Roof is flat slab system with roof drains nested under pavers
•Footings range from 4’6” square up to 15’ x 9’6”
•Shear walls extend entire height of the building and are located around the elevator core.

Architecture

•Net Square Feet: 107,100 SF
•Usage:
  •Primary Occupancy- Residential
  •Secondary Occupancy- Retail, Floor 1
•Number of Stories: 21 above grade, 2 below
•The exterior walls of 110 Third Ave. consist of a “window wall” system. This system is fixed window units fabricated with flush aluminum panels finished to match the window wall that rests on the slab.
•On the North and East sides of the building are balconies from floors 8 through 16 and 16 through 21, respectively.

Electrical/Lighting System

•Electrical service is brought into 110 Third Avenue by Con-Edison service 120/208V 3 Phase 4 wire distributed to two switchboards located on the cellar level.

•Switchboard 1 services the residential portions of the building, retail space, and gym area Switchboard 2 powers utilities such as the sprinkler system, fire pumps and elevators.

•Circuit wire sizes are most commonly 2 #12-3/4”C, and branch circuit breakers are most commonly 1 pole, 20 Amp.

Mechanical System

•2400#/hr and 2,400,000 BTU/hr. steam supply

•Heat exchanger supplies individual units via individual hot water unit heaters.

•A second heat exchanger serves the primary condenser water loop and is tied to a 2-cell cooling tower serving the water-source heat pumps at 990 CPM per tower with 330 tons capacity per tower.

•CFM total is 48680

•Common spaces are conditioned by a dedicated VAV box rated at 1040 CFM.

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/ajn137/

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Executive Summary

The requirements set forth by the designers of 110 Third Avenue are basic in that they meet the needs of economy and the future occupants of the building. This report consists of a new design of the structural system and concludes, independently of existing conditions, the best system for the site and conditions impacting 110 Third Avenue.

The new structural system removes columns throughout the building thus opening the floor plan while resisting the same loadings. Increasing the bay sizes to adapt the floor plan impacted all other structural systems in the building, while having little impact on mechanical and lighting issues. Larger bays lent the design of the new building to a post-tensioned system, because PT does not become cost effective until bays reach spans of twenty or more feet. The post-tensioned cast-in-place floor slab can support the larger bay sizes without increasing the overall depth of the slab. The effectiveness of post-tensioning is judged based on the advantages it provides for the building against the costs of both the old and new system.

The lateral system was also evaluated for effectiveness, but it was found that using a combined lateral resisting system consisting of moment frames and shear walls was actually the best system all along. When designing columns initially, it was assumed they were leaning columns and would only take moments created by uneven floor loading patterns. The lateral force resisting system, under this assumption, consisted of only shear walls. After some further investigation, it became clear that shear walls in their original configuration in 110 Third Avenue would not be adequate to prevent large story drifts. Options for developing a proper lateral system were moving the shear walls toward the extremities of the building or reverting back to a combined system and redesigning the columns. From the way architects had intended the floor plan to operate and 110 Third Avenue to look, it was clear that putting shear walls near or on the exterior of the building would greatly disrupt the architecture. The window wall system had to be kept intact to preserve the original look of the building. As a consequence, the best lateral system was clearly moment frames combined with shear walls. Redesign of the columns simply increased their reinforcing, and the end result was a building that functioned quite effectively.

Cost analysis of the floor slabs proved, unfortunately, that the old system was about 8% cheaper than the new. However, removal of around 50% of the original columns freed up the living spaces a bit and will also cut down on overall formwork costs. The increased cost of the new floor system will be offset by savings in labor and formwork. In the end, both systems are comparable, but the original CIP flat plate system should be used in the New York City setting because of the low availability of PT contractors in the area.

Introduction

110 Third Avenue is a residential mid-rise tower that sits in the heart of Manhattan between Gramercy and East Village. Standing at 210’ to the bulkhead slab, it offers 21 stories of mid-sized apartments totaling approximately 107,000 square feet of inhabitable space. The structural system is entirely concrete, with columns scattered carefully throughout the floor plans. The slab is CIP flat plate and is 8” thick, and the lateral force resisting system utilizes the slab and columns as a moment frame in addition to using shear walls around the elevator core of the building. All concrete used in 110 Third Avenue is 5000 psi. The neighborhood surrounding the building is typical for Manhattan, with mid-rise buildings reaching the maximum height limitations all around. The street located below these towers is constantly bustling with locals and NYU students alike. The addition of 110 Third Avenue will prove to be a good addition and add even more character to an already well known, well defined city.

This report focuses on a new design of 110 Third Avenue that features a new column layout that will hopefully better serve tenants by reducing the overall number of columns and increasing the bay sizes to free up more living space. A new floor slab system designed as a two-way post-tensioned cast-in-place flat plate slab replaces the old flat plate regularly reinforced system. A new lateral system that utilizes the new column layout is evaluated for drift requirements and compared to the old system. In addition, the façade of the building has also been reexamined for possible flaws or omissions in the design and a section with suggestions for façade improvement follows. Finally, a cost comparison of the new and old floor systems give a basis for deciding which system is more suited to the stipulations incurred on the site that face the owner, designers, and contractors alike. The ramifications of constructing the new post-tensioned floor slab in New York City are also addressed and factored into the evaluation of which system is a better option for the given environment.

Included are appendices to assist the reader in following thought processes and supports conclusions reached in the body of the report. Please note this report is intended for educational purposes only and does not substitute for the original design.

Building Description

- **Building Name:** 110 Third Ave.
- **Location and Site:** 110 Third Ave., New York, New York 10003
- **Building Occupant Name:**
    - Primary Occupancy: Residential- Toll Brothers, Inc.
    - Secondary Occupancy: Retail, Floor 1- Not Yet Determined
- **Size:** 107,100 Sq. Ft.
- **Number of Stories:**
    - Above Grade: 21 Stories with total height above grade, 227'-6"
    - Below Grade: "Cellar" and "Sub-Cellar"
- **Primary Project Team:**

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Inspection Group Inc.							
2279 Arby Court							
Wantagh, NY 11793							

- **Dates of Construction:** Demolition began in December, New construction in late February of 2006.
- **Project Cost Information:** Not Available, but estimated at $110 million
- **Project Delivery Method:** Design-bid-build

- **Architecture:** 110 Third Avenue serves as a great addition to the New York skyline with twenty-one stories of residential condominiums. The exterior façade is reminiscent of the repeating patterns found quite often in 1960's post-modern architecture. The spiraling balconies and tapered neck of the building alter the Roheian approach to box skyscrapers slightly to adjust for more modern tastes. The prime downtown location in the heart of Manhattan allows tenants to experience the very best of the city that never sleeps in their own private haven. First floor apartments offer 2 bedrooms, 2.5 baths with living room, kitchen and access to a private recreation room downstairs complete with a private terrace. All tenants have access to an in-house gym located on the cellar level. Floors 2 through 15 have four or five units per floor, and units feature either one or two bedrooms plus bathroom(s), living room, and kitchen. Floors 16 through 21 have only three units with three bedrooms, 2.5 baths, living room, and kitchen.

- **Model Code**: Building Code of the City of New York, including latest amendments ("N.Y.C. Code")

- **Zoning:** 110 Third Avenue occupies a residentially zoned site that previously consisted of a parking lot, 1 story residential building, 3 story residential building, and a 4 story residential building

- **Historical Requirements:** None

- **Building Envelope:** The exterior walls of 110 Third Ave. consist of a "window wall"system. This system is fixed window units fabricated with flush aluminum panels finished to match the window wall that rests on the slab. Surrounding the windows are glazed aluminum window wall framing. The window units themselves consist of a 1/4" thick nominal aluminum composite panel affixed to the exterior face window-wall unit with conceded fasteners and/or adhesives finished to match the window-wall. Also present is an insulating spandrel panel. On the North and East sides of the building are balconies from floors 8 through 16 and 16 through 21, respectively. Each balcony is cantilevered 5' from the building face. The roof is concrete slab supporting mechanical equipment, but it also hosts several private terraces and a common terrace for occupants. The roof itself is composed of a layer of fluid applied roofing membrane, drainage panels, 4" polystyrene, adjustable paver pedestals, topped with a layer of precast concrete pavers. Surrounding the living spaces is a 4'-0" high perimeter parapet planter all around the roof.

- **Structural:** The structural system of 110 Third Avenue is predominantly cast-in-place concrete. Most floors have 8" CIP slab, but beginning with floor 15 the slab increases to as much as 24" to support cantilevered portions of the building and mechanical equipment on the roof. All slabs and columns have f'_c= 5000 psi. Loads are carried from the two-way slab system to concrete columns ranging from 12x12 to 40x12. The columns are continuous throughout the height of the building except for a few columns that terminate at floor 16 due to a setback in the building perimeter, and a few columns that originate on the drawings at floor 11 due to the reduction of the elevator core to column-sized portions. Footings range from 4'6" square up to 15' x 9'6". The only beams present in the structure are in the basement level and are grade beams extending from perimeter East-face and West-Face footings to the outside wall. There are also beams surrounding the elevator core and around the stairwell. Shear walls extend throughout the height of the building and are located mostly on the North and South sides of the building. The roof is a flat slab system that is drained by roof drains nested under pavers. Supporting columns are recessed from the façade on average 10", and therefore allow the designer to use non-bearing prefabricated panels.

- **Electrical:** Electrical service is brought into 110 Third Avenue by Con-Edison service 120/208V 3 Phase 4 wire distributed to two switchboards located on the cellar level. Switchboard 1 services the residential portions of the building, retail space, and gym area with meters every third floor. Switchboard 2 powers utilities such as the sprinkler system, fire pumps and elevators. Circuit wire sizes are most commonly 2 #12-3/4"C, and branch circuit breakers are most commonly 1 pole, 20 Amp. All circuits and feeders have a full size insulated green ground conductors and are connected to the ground bus in their respective panels. Minimum size conductor and conduit is #12 THHN CU, ¾"C (EMT). All mounted wall switches, dimmer, etc., are at 4'0" A.F.F. to center line of devices. Receptacles are mounted at 15" A.F.F. The electronic ballasts meet or exceed both the minimum ballast efficiency factor (B.E.F.) as specified by Con-Edison and total harmonic distortion (T.H.D.) requirement or 20% or less. All fluorescent light fixtures have energy saving lamps and are equipped with electronic energy saving ballasts.

- **Lighting:** typical suite lighting is achieved by ceiling mounted compact fluorescent bowls and linear fluorescent prismatic wrap-arounds. Circulation lighting is primarily wall mounted compact fluorescent sconces. Following is a brief outline of interior lighting throughout 110 Third Avenue.

1. Kitchen Fixtures:

   a. Recessed ceiling downlighting.
   b. Continuous undercabinet task lighting.
   c. Pendant task lighting above island countertops.

2. Bathrooms:

   a. Recessed ceiling downlighting.
   b. Mirror task lighting.

3. Walk-in Closets:

   a. Utility wall Sconce.

4. Apartment Halls:

   a. Recessed ceiling downlighting.

5. Common Residential Corridors And Elevator Lobbies:
   Recessed ceiling downlighting

- **Mechanical:** Con Edison provides the heat to 110 Third Avenue through a 2400#/hr and 2,400,000 BTU/hr. steam supply that feed into Heat Exchanger 1. This heat exchanger supplies individual units throughout the building via individual hot water unit heaters. Heat Exchanger 2, also located in the basement, serves the primary condenser water loop and is tied to a 2-cell cooling tower is located on the roof serving the water-source heat pumps at 990 CPM per tower with 330 tons capacity per tower. CFM total is 48680. Common spaces are conditioned by a dedicated VAV box rated at 1040 CFM. Stairwells are heated by individual electric heaters mounted to underside of landing fully recessed. Living rooms feature baseboard water heating as per hot water fin runtal RF—2 at 600 BTU/hr @180 F. Bedrooms feature individual heating recirculation units.

- **Fire Protection:** The system is a sprinkler alarm system per NYC building code and includes elevator recall and fan shutdown. Ducts feature smoke detector wired to the central alarm system. Other system features have not been designed yet. Pull station locations are yet to be determined.

- **Transportation:** The core of 110 Third Avenue features all transportation systems including two elevators servicing all floors. The central stair case abuts the two elevator shafts to the north, but additional stairs connecting individual apartments to private terraces and service staircases exist on the cellar level. Also featured is a refuse room located southeast of the elevator shafts that transports waste to exit the building.

- **Telecommunications:** 110 Third Avenue features a telephone room located in the basement. Each apartment has approximately nine combination voice/data CAT-E cable/ TV coaxial RG-6 cable outlets. There are typically two combination outlets found in each bedroom, and two or three found in each living room. Each kitchen has a voice CAT-5E cable for a wall phone. Also, located in all apartment hall closets is the apartment NID panel.

- **Plumbing:** All connections made to 110 Third Avenue come directly from Third Avenue as opposed to 13th or 14th streets which surround the site. The building has full sewer, gas, and water services. Sewage and water lines, on the first floor, run through the center of the building to retail spaces and the two luxury apartments located in the rear. Each subsequently higher floor matches the points of connection and ties to lower floors with vertical pipes located in the walls next to the bathrooms and kitchens. The cellar possesses a domestic water booster pump, DCVA, and meter for the entire building. Present on the first floor are three none-freeze wall hydrants.

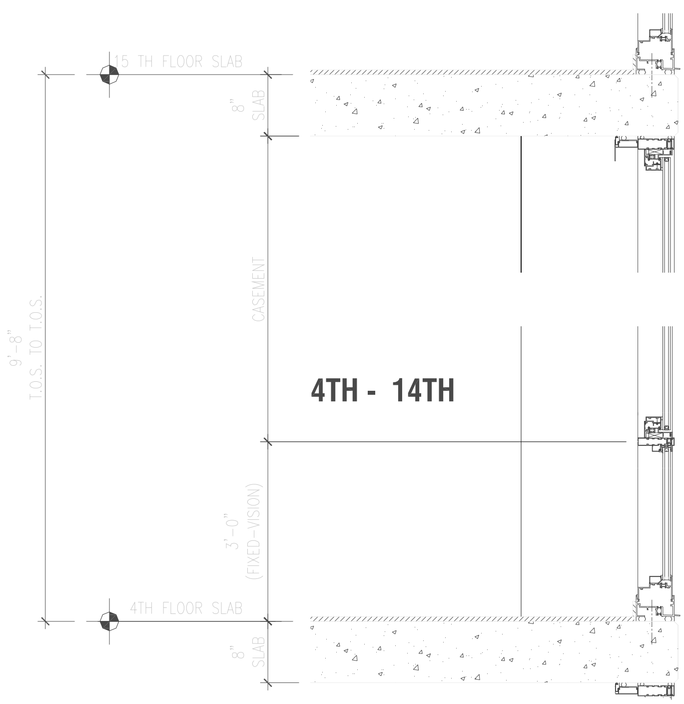
Roof drains are connected all the way to the basement level where there are three sump pumps. One sump pump services the basement level while the other two are elevator sump pumps. Gas is provided by Con-Edison through a 4” line connected on the basement level extending vertically to stoves on each floor.

Structural System Information

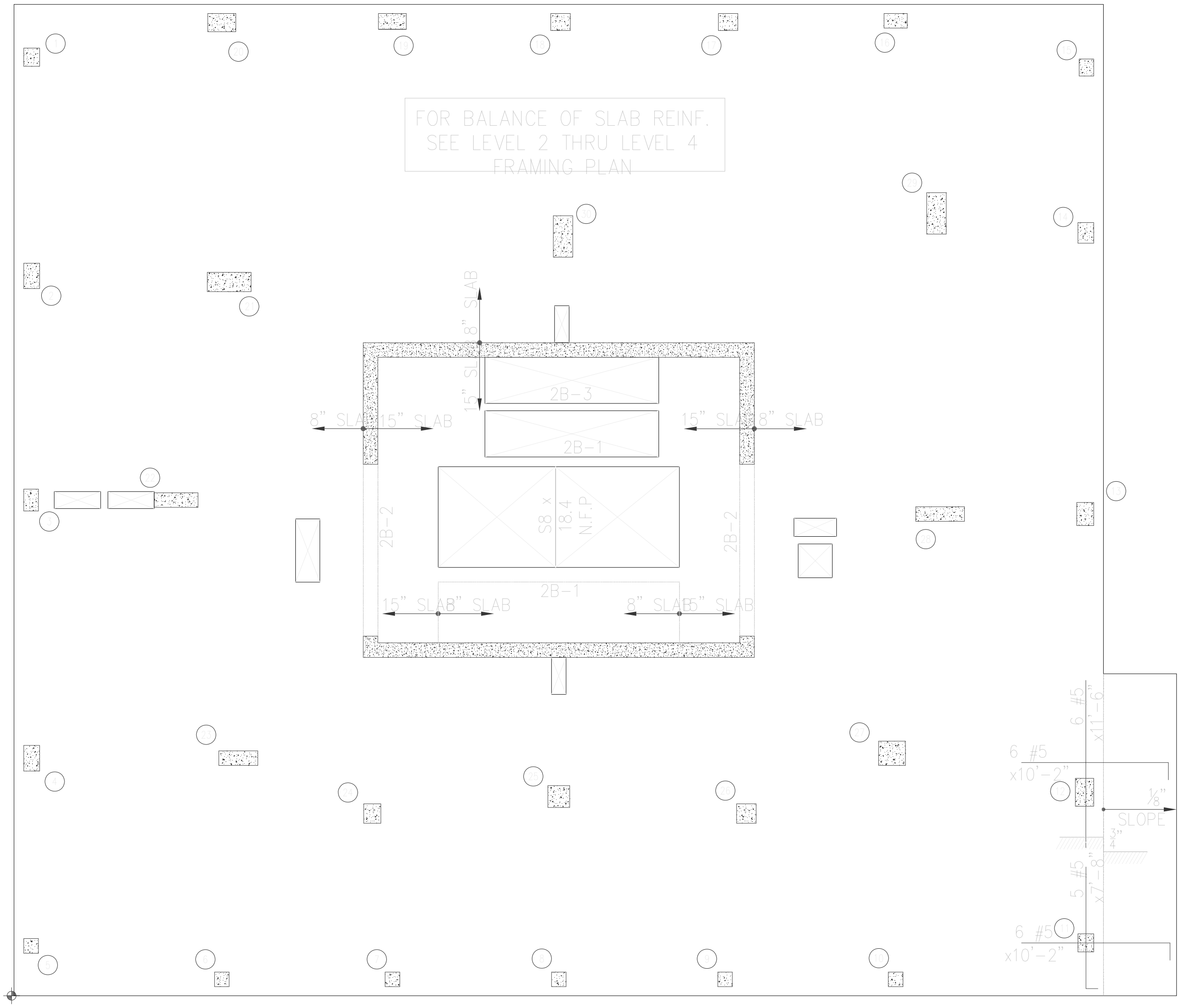
110 Third Avenue is a great example of economic residential design in an urban setting. The design of the structural system is nearly uniform throughout the height of the building, changing mildly at the 16th floor to accommodate a small setback in overall width of the building. The placement of the main lateral resisting elements around the elevator core saves precious exterior wall space for windows and a curtain wall that are aesthetically beneficial. The foundation is quite typical, but the placement of the columns in irregular-shaped bays shows the designers consideration for well placed structural elements throughout the building. Each apartment space revolves around the architects intent for the flow of the building and individual units, and the placement of columns caters to these needs.

Existing Structural Floor System

110 Third Avenue is completely a flat plate system with columns roughly sorted into a 7x5 element bay. The building extends 68' in the North-South direction (5 columns) and 75' in the East-West direction (7 columns). A flat plate system supports the loads placed on the building and directly transfers the loading to the columns. No drop panels assist in the distribution of weight or add to the building's resistance to punching shear. A central shear wall system centered around the elevator core provides lateral stability and resistance to wind and seismic loading.



Typical Floor Section



Typical Floor Plan for floors 5 through 10, other floors are very similar

Design weight of floor framing is 8" thick concrete flat plate slab at 100 PSF (S-001)
A typical flat plate slab system serves the entirety of 110 Third Avenue, with a typical slab thickness of 8". Slab size increases around the elevator core to 15", and increases to 24" near the elevator core on the roof level to support mechanical equipment. Slabs are continued, in portions of each floor, past the perimeter to form balconies. The balconies have a ¾" step down from the 8" slab that makes up the entire interior space, and are therefore 7 ¼ in. thick. The flat plate slab is a great approach to a mid-rise residential tower because it saves on formwork and labor costs. All slabs are 5000 psi concrete.

Additionally, please note there is a height restriction on 110 Third Avenue limiting the overall height from grade to bulkhead floor slab to 210'. 110 Third Avenue now stands at this 210' and has no additional room to increase height. The only ways to accommodate any additional height in the redesigned floor system would be to subtract from the habitable area's height or apply for a variance from zoning regulations that limit 110 Third Avenue.

Foundation

The foundation structure of 110 Third Avenue consists mainly of footings occurring at regular intervals underneath the columns. There is also a perimeter wall footing that ranges from 2'-0" to 9'-8" in width. The footings range from 4'-6" square to 9'-6" x 15'-0" to 11'-0" x 12'-6", and there also are also grade beams connecting East and West face foundations with the exterior. These grade beams are 18x24 with 3 #11 top and bottom continuous reinforcement. The bottom of footings bear on gravely sand (NYC classification 7-65 and 6-65) with a minimum allowable bearing capacity of 4 tons per square foot. Also note that overturning moment in the foundation will be examined in a later report to insure lateral system does work.

Framing

The framing of 110 Third Avenue is an economical approach to mid-rise residential facilities. It consists of an inner core of shear walls around the elevator and stairwell that resists lateral loads, and a column layout setback from the perimeter to allow for a lightweight, prefabricated aluminum and glass panel to serve as the exterior façade. In addition, a flat plate slab provides support against gravity loads and transfers weight directly to the columns. This may leave the building vulnerable to punching shear, and this aspect of the building will be evaluated in the future. The columns are irregularly sized, and a pattern really doesn't develop in their sizing except around the perimeter where a regular grid is present. Column sizes range from 12" x 12" to 40" x 12" and are spaced at intervals that suit the needs of the architecture of the apartment. All columns are 5000 psi concrete

Slabs

A typical flat plate slab system serves the entirety of 110 Third Avenue, with a typical slab thickness of 8". Slab size increases around the elevator core to 15", and increases to 24" near the elevator core on the roof level to support mechanical equipment. Slabs are continued, in portions of each floor, past the perimeter to form balconies. The balconies have a ¾" step down from the 8" slab that makes up the entire interior space, and are therefore 7 ¼ in. thick. The flat plate slab is a great approach to a mid-rise residential tower because it saves on formwork and labor costs. All slabs are 5000 psi concrete

Lateral System

The lateral system of 110 Third Avenue is a combined system that utilizes both shear walls and a moment frame consisting of columns and floor slabs to resist lateral loads. Shear walls are located around the elevator core as described in previous sections. They are continuous from floor 2 to the roof, and on the ground floor and first floor they are supported by additional length and reinforcement. Designers placed these shear walls only around the elevator core presumably because architectural concerns prevented them from being located elsewhere.

In conjunction with the shear walls, the moment frame provides the second essential part to the lateral force resisting system. The stability of the columns and floor acting as a rigid diaphragm provide additional support. Without including this frame when considering how lateral forces act on the building, overall story drift would be excessive and a nuisance.

Loads and Load Cases

Loading conditions on the vast majority of the building are relatively light due to their use as residential space. A table below provides a complete description of loads according to drawing S.001 provided by Axis Design Group. When factored according to ASCE-07, loading throughout the apartments is only 94 psf. Low loading consequently makes the existing system, the 8" flat plate system, a very good choice in order to maximize space. Most other systems aren't competitive simply because they cannot maintain a depth of only 8".

Floor	Partition	Ceiling & Mech.	Floor Finish	Live	Total Imposed
Lobby	-	5	40	100	145
Apartment	12	-	5	40	65
Roof	-	5	25	30	60
Retail	-	5	15	100	120
Storage	-	5	-	100	105
Stairs	-	-	-	100	100
Private Roof Terrace	-	-	65	60	200
Public Roof Terrace	-	-	65	100	200
Mechanical	-	25	40	150	215
Gym	-	5	15	100	215
Courtyard	-	-	65	60	215

D = dead load;
D_i = weight of ice;
E = earthquake load;
F = load due to fluids with well-defined pressures and maximum heights;
F_a = flood load;
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials;
L = live load;
L_r = roof live load;
R = rain load;
S = snow load;
T = self-straining force;
W = wind load;

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r$ or S or $R)$
3. $1.2D + 1.6(L_r$ or S or $R) + (L$ or $0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r$ or S or $R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Exceptions:

1. The load factor on L in combinations (3),

Max wind loading: 1.6W
Max Seismic loading: 1.0E

As detailed above, ASCE7-02 gives seven loading combinations that could be applied to 110 Third Avenue. Evaluation of considered lateral loadings (W and E) shows that W and E are never combined in any ratios. Therefore, the ETABS model presented later in this report considers the maximum factored wind load of 1.6W and the maximum seismic load of 1.0E separately. Taking these loads separately accurately reflects the provisions laid out by ASCE7-02. Note that several wind loading patterns must also be considered as per ASCE7-02 figure 6-9. In this report, case 1 and case 3 are the only cases considered since cases 2 and 4 almost never control.

	Fy (N-S)			Fx (E-W)		
Level	Seismic	Wind	Controlling	Fx (E-W)	Fx (E-W)	Controlling
21(roof)	13.1	22.4	WIND	13.1	13.8	WIND
20	26.4	41.7	WIND	26.4	25.8	SEISMIC
19	24.7	38.7	WIND	24.7	23.9	SEISMIC
18	23.0	38.3	WIND	23.0	23.7	WIND
17	21.4	38.0	WIND	21.4	23.4	WIND
16	19.8	37.6	WIND	19.8	23.2	WIND
15	18.2	37.2	WIND	18.2	22.9	WIND
14	16.6	36.8	WIND	16.6	22.7	WIND
13	15.1	36.3	WIND	15.1	22.4	WIND
12	13.6	35.9	WIND	13.6	22.1	WIND
11	12.1	35.4	WIND	12.1	21.8	WIND
10	10.7	34.8	WIND	10.7	21.5	WIND
9	9.3	34.3	WIND	9.3	21.1	WIND
8	8.0	33.7	WIND	8.0	20.7	WIND
7	6.7	33.0	WIND	6.7	20.3	WIND
6	5.5	32.3	WIND	5.5	19.9	WIND
5	4.3	31.4	WIND	4.3	19.3	WIND
4	3.3	30.5	WIND	3.3	18.7	WIND
3	2.2	29.9	WIND	2.2	18.4	WIND
2	1.3	28.9	WIND	1.3	17.7	WIND
1	0.5	30.3	WIND	0.5	18.6	WIND

The above table shows that wind is generally the controlling load for 110 Third Avenue with the rare exception of the 19th and 20th floors in the E-W direction. Each loading utilizes its respective load factor of 1.0E or 1.6W.

Overturning

The foundation system in 110 Third Avenue resists overturning. The overturning moment in the N-S direction is 81347 ft-kips, and in the E-W direction it is 50168 ft-kips.

Floor	FLOOR SHEAR (Kips) N-S	FLOOR SHEAR (Kips) E-W	Floor Height	M (N-S)	M (E-W)
21	22.4	13.8	12.000	269.0205	166.1308
20	64.1	39.6	9.667	619.9979	382.8349
19	102.8	63.5	9.667	993.966	613.7061
18	141.2	87.1	9.667	1364.509	842.4283
17	179.1	110.6	9.667	1731.491	1068.916
16	216.7	133.8	10.000	2166.995	1337.663
15	253.9	156.7	11.000	2792.659	1723.738
14	290.6	179.4	9.667	2809.485	1733.976
13	327.0	201.8	9.667	3160.544	1950.471
12	362.8	223.9	9.667	3507.095	2164.139
11	398.2	245.7	9.667	3848.871	2374.809
10	433.0	267.1	9.667	4185.557	2582.286
9	467.3	288.2	9.667	4516.789	2786.341
8	500.9	309.0	9.667	4842.131	2986.699
7	533.9	329.3	9.667	5161.051	3183.028
6	566.2	349.1	9.667	5472.889	3374.912
5	597.6	368.5	9.667	5776.789	3561.816
4	628.1	387.2	9.667	6071.6	3743.016
3	658.0	405.6	10.000	6579.887	4055.667
2	686.9	423.3	10.000	6868.965	4233.003
1	717.2	441.9	12.000	8606.287	5302.359

Overturning Moment	N-S	81346.5789	ft-kips
	E-W	50167.9383	ft-kips

As per the seismic analysis performed in Technical Report 1, the weight of the building is as follows:

Level	w_x
21(roof)	178.74
20	382.98
19	382.98
18	382.98
17	382.98
16	382.98
15	382.98
14	382.98
13	382.98
12	382.98
11	382.98
10	382.98
9	382.98
8	382.98
7	382.98
6	382.98
5	382.98
4	382.98
3	382.98
2	382.98
1	382.98
Total	7838.34

Assume a worst case scenario with a support at each end of the building. Weight of the building is 7,838.34 k as above. Therefore, each end of the building has support 7,838.34/2 = 3919.17 k to resist uplift.

N-S Direction: Axial load = M/ L = 81347 ft-kip/68 ft. = 1196 k
E-W Direction: Axial load = M/ L = 50168 ft-kip/75 ft. = 669 k

The allowable uplift force of 3919.17 is greater than both applied moments (1196 k and 669 k), so the weight of the building is great enough to resist the downward forces from the overturning moment.

Problem Statement

Designers of 110 Third Avenue faced a very simple design problem: create an efficient design suitable for residential construction with a height limit by putting the most floors in as possible while making sure to avoid interference with architectural design. Several interesting solutions were incorporated into the design of the structural system and can be examined further. First, the floor system is a two-way flat plate, but this simple system might not be the best solution with regard to ease of installation and economy. The maximum height limitation for 110 Third Avenue is 210'-0", and the reasonable maximum number of stories for such a restriction is twenty one. In order to maximize occupiable volume per floor, the floor system must remain slim and not exceed 8". There exists no room for a plenum space for mechanical equipment, and any slab system exceeding 8" would not have nearly as many advantages as a flat plate system. The criteria for an improved floor system design are as follows:

1) Equal to or less than 8" thick
2) Maintain strength of system without compromising span length
3) Must keep costs equal to or lower than a flat plate system
4) Ease of construction/installation

Second, as seen in Tech Report 3, a major difference in lateral force resisting system analysis was discovered and should be reevaluated. Designers assumed use of slabs and columns to resist lateral forces, not just shear walls. Until a few years ago, there were no computer programs that could easily analyze a structure in this manner, but tools are now available that will allow this analysis to be performed. Both shear walls and the use of slabs and columns as a moment frame acted together to drastically reduce the drift with minimal force in the slab. The columns have no additional size or reinforcement and the slab simply includes a few additional top bars at the columns for the wind moment. Due to time constraints during the completion of Technical Report 3, a completely new model could not be created in time for this report.

The drift should be further analyzed in the future using revised load cases (without factors) and the combined system previously specified. If these two adjustments are made to the computer model, it should produce perfectly reasonable drifts. Finally, the Excel file, although seemingly off in its forces, also uses reasonable values for base shear and weight of the building (242.8 k base shear and 7838.8 k weight). The wind forces applied to both the ETABS and Excel model are identical except for the 1.6 factor, indicating they should be off by a multiplier of 1.6, not 3. The report shows that the lateral system was competently designed, although using ETABS did not necessarily demonstrate exact loading and resisting conditions. The difference in results using computer models is clearly explained from the different approach a combination system takes. The use of the combined frame and shear wall reduces lateral movement for a given size and reinforcing of shear walls.

Proposed Solutions

Floor System Redesign

Although a flat plate system seems well suited to conditions present in 110 Third Avenue, such as height restriction and desire for high occupancy, other alternate floor systems may be equally as viable if not more advantageous. The most viable option is a post-tensioned two-way slab that will allow for greater spans, but subsequent redesign of columns and the lateral system must be performed.

Post Tensioned Two-Way Slab

The use of PT presents many benefits that are conducive to the requirements presented by 110 Third Avenue. PT slabs are typically thinner than an ordinary reinforced concrete slab. A thinner slab could quite possibly mean the incorporation of an extra story into the design (although this may be overly ambitious). According to http://www.concretecentre.com, "the amount of prestress can be adjusted to control deflection, thus enabling the minimum depth of slab to be used. Deflection calculation can also be simpler than for reinforced concrete because the section is uncracked."

The presence of irregular grids in 110 Third Avenue offers a severe challenge to any system that can't readily adapt to differing bay sizes and shapes. A PT slab is an especially exciting prospect since it has the same flexibility to accommodate irregular design that a normal slab does. Post Tensioned slabs are also easily erected and could possibly save on construction time and erection costs such as formwork.

A possible downside to the use of PT is most sources claim a PT slab won't become economical until spans reach around 20'. Spans in 110 Third Avenue are approximately around this 20' mark in the long direction, so the floor layout as it is may not be best suited for a PT slab. However, if necessary, the floors could be redesigned to have fewer columns.

A redesign of these columns will be performed in a manner that will avoid interference with the architecture already present in the designs of 110 Third Avenue. Of course, the redesign of the columns will influence the lateral system since it relies on a combined system of shear walls and a moment frame consisting of the floor slab and columns. The procedure for redesigning 110 Third Avenue using post-tensioning will consist of:

1) Design the floor slab assuming a larger bay size to make the system economical
2) Reduce number of columns to accommodate the larger bay size
3) Resize the columns
4) Analyze the lateral system using the new column layout and adjust the shear walls and columns as necessary

Combination Lateral System Analysis

A new lateral analysis will be performed using ETabs that will incorporate the use of a combined lateral force resisting system. The old model, which did not incorporate actual column and bay sizes, will have to be completely redone with accurate column sizes spaced irregularly to provide the proper degree of accuracy. In addition, the new ETabs model will place slab-beams running between columns to approximate the moment frame. Finally, the factored loads input into the model will be changed to unfactored loads and compared to hand analysis once again to verify the design. Adjusting the analysis in this way will allow the combination system to be evaluated and compared to the previously analyzed shear-wall-only system.

Solution Method

The post tensioned system will be checked at initial and service conditions for the given loading. Also, the strands much be checked to make sure they are within the acceptable range for placement of PT reinforcement. Capacity is evaluated at initial condition, after jacking, and after losses. Shear stresses will be checked as well. Needless to say, more research will need to be done to ensure proper design of a PT slab, and most of the knowledge of PT systems currently comes from CE 543. Also, RAM Concept could possibly aid in the design of a PT system.

Once design and analysis of the floor system is complete, the columns can be resized based on the new weights and loadings from the new bay sizes. After they have been resized for vertical loading, a lateral analysis will insure they will be sufficient for wind and seismic loadings in combination with the existing shear walls. It may be necessary, if the combination system fails with regard to story drift, to increase the size of the shear walls.

The current floor system will be analyzed for punching shear and then the addition of stud rails will determine whether the floor system can be reduced in thickness.

Lateral analysis will be performed using ETabs, as stated before, and will use a completely new and separate model from Tech 3. The new model will not modify column placement and size, but rather will maintain true-to-life column sizing and spacing to make sure an accurate end analysis is obtained. In this manner, shear walls and the moment frame created by the slabs and the columns will be analyzed as a combination system. Also, the new system will be compared to the shear wall system previously analyzed using the output found in Tech 3.

Tasks and Tools

1. Two-way Slab Post Tensioned Floor System Alternative

    Task 1: Determine loading

Task 2: Design slab
- a) determine minimum thickness
- b) find applied moments
- c) check capacity of slab
- d) check deflections

2. Redesign of columns

Task 1: Determine vertical loading based on the loading criteria listed on page 3
Task 2: Size the columns using the analysis learned in AE431

3. Perform a Lateral Analysis of the New System

Task 1: Input model into ETabs
Task 2: Analyze model for drifts and member forces
Task 3: Compare drifts to serviceability criterion
Task 4: Conclude whether the system is sufficient in all aspects of design

4. Combination Lateral Force Resisting System Analysis

Task 1: Input model into ETabs
Task 2: Analyze model for drifts and member forces
Task 3: Compare drifts to serviceability criterion
Task 4: Compare conclusions to actual design of lateral system and previous system.

Breadth Work

Construction Management

The redesign of the floor system must be evaluated in comparison to the existing system. To do this, issues such as cost, constructability, and labor must be addressed. The post tensioned system will be examined and compared to the current system. After a comparison of each system has been constructed, a final determination will be made of which system is best.

Also, the construction process will be examined first hand in New York City in February. Any issues that have arisen or could potentially arise will be examined in-depth. Installation of the floor system will also be examined in detail to provide a firm basis of comparison for other systems.

Building Technology

The exterior walls of 110 Third Ave. consist of a "window wall" system. This system is fixed window units fabricated with flush aluminum panels finished to match the window wall that rests on the slab. Surrounding the windows are glazed aluminum window wall framing. The window units themselves consist of a 1/4" thick nominal aluminum composite panel affixed to the exterior face window-wall unit with conceded fasteners and/or adhesives finished to match the window-wall. Also present is an insulating spandrel panel. The roof is concrete slab supporting mechanical equipment, but it also hosts several private terraces and a common terrace for occupants. The roof itself is composed of a layer of fluid applied roofing membrane, drainage panels, 4" polystyrene, adjustable paver pedestals, topped with a layer of precast concrete pavers. Surrounding the living spaces is a 4'-0" high perimeter parapet planter all around the roof.

These key features of the building envelope must perform as intended, otherwise water penetration could pose a significant threat to the health of the building and the satisfaction of tenants. Each part of the building envelope will be examined for adequacy, and potential issues that could arise during construction will be listed. The construction process is key to ensuring proper performance of the window wall system.

Timetable

Item	Week 1 1/9 to 1/16	Week 2 1/16 to 1/23	Week 3 1/23 to 1/30	Week 4 1/30 to 2/6	Week 5 2/6 to 2/13	Week 6 2/13 to 2/20	Week 7 2/20 to 2/27	Week 8 2/27 to 3/6
1. Post Tensioning Research	X	X						
2. Design of PT Floor System		X	X					
3. Design of new Columns			X	X				
4. Design of new lateral system					X	X		
5. Breadth 1- Compare Floor systems						X	X	
6. Develop ETabs model								X
7. Find drifts and evaluate								
8. Breadth 2- Build. Tech.								
9. Present								
10. Review								

Item	Week 9 3/6 to 3/13	Week 10 3/13 to 3/20	Week 11 3/20 to 3/27	Week 12 3/27 to 4/3	Week 13 4/3 to 4/10	Week 14 4/10 to 4/17	Week 15 4/17 to 4/24	Week 16 4/24 to 4/28
1. Post Tensioning Research	Break							
2. Design of PT Floor System	Break							
3. Design of new Columns	Break							
4. Design of new lateral system	Break							
5. Breadth 1- Compare Floor systems	Break							
6. Develop ETabs model	Break	X	X					
7. Find drifts and evaluate	Break		X					
8. Breadth 2- Build. Tech.	Break			X				
9. Submit	Break				X			
10. Present and Review	Break					X	X	X

Design Work

Column Design

The first step in designing the new column layout that would be implemented in 110 Third Avenue was to consider the architectural ramifications of removing columns and subsequently upsizing them elsewhere. It's plain to see that the goal of the layout is to maintain a free perimeter throughout the building, as evidenced by the column setback from the window-wall building envelope system. Also, designers could not avoid having to place columns in somewhat awkward spots such as the middle edge of a living room or bedroom. The designers, however, did take precaution whenever possible to tuck columns away from sight whenever possible. As a result, the original layout of 110 Third Avenue had a completely irregular layout, and this is another reason why designers chose a flat plate system. A flat plate system can handle irregularities in bay sizes and shape without much consequence in the design, especially with the use of current computer modeling programs.

The new column layout possesses the same principles as the old one: hide columns when possible and keep the floor plan open and flowing. Removal of about 50% of the columns opened up the spaces greatly, and careful consideration was taken when enlarging the other remaining columns. Each column, after being upsized, was to avoid unnecessary intrusion into the living spaces. In some cases, this required creating quite rectangular columns, e.g. 19" x 30", to maintain maximum window space and non-interference with flow between rooms. The new plan has only 14 columns in the building, and all but four are along the edges or corner of the building. The four which are interior columns are tucked in the corner of the stairwell and in public hallway space. Also note that the four columns along the south (building south) edge of the building step back in plan on floors seventeen and higher. They also are sized smaller due to smaller axial loading at these floors.

The second step in creating a column layout was to define how these columns were to be designed. Initially, each column was treated as a leaning column where the only moment experienced by the column would be due to uneven floor loading patterns. Once these uneven floor loads were found, each column was sized and reinforcing designed using the computer program PCACOL. Please also note the hand calculation provided in the appendix of this report that verifies accurate design on the part of PCACOL.

Once initial sizes of the columns were found, they did not change until the design of the lateral system proved that approaching the columns as leaning columns was an inefficient means of design. Each column and its reinforcing was governed to an extreme degree by gravity loads due to the "leaning column" assumption, but each column could easily handle being treated as a member in a moment frame. Further design in ETabs used these columns to reduce story drifts and assumed an increased moment loading on each.

After the lateral system design was completed, a second inspection of the columns showed the reinforcing of each column had to be reexamined. Using ETabs to assist in the design of the columns based upon the moments the program generated, PCACOL was once again utilized to design reinforcement. In the process of designing these columns using ETabs, the program noted on its plan that no columns would have to change in cross sectional area. After applying the moments again in PCACOL, this program said that two columns needed to be upsized by one or two inches. The two columns were the most rectangular ones on the plan: columns 7 and 8. To be cautious, these two columns were upsized despite ETabs claiming they were ok, because PCACOL said they could not be designed within the given parameters. The slight difference in designs is strange, but not wholly unexpected. Quite frequently in Etabs, a smaller moment could be found on the columns than would be created from uneven floor loading. Instead of using these smaller loads in PCACOL, the maximum loading in both directions for all cases was used. Therefore, an additional burden was being placed on columns designed using PCACOL than what Etabs was generating. This could explain why two columns needed to increase in size in PCACOL. Despite the slight discrepancies between programs, a final column schedule could be developed with little trouble. See the following details of the final column designs for information on size and reinforcing:

Column ID	Dimensions	Reinforcement	A_s (in.^{2})	Clear Cover (in.)	Spacing (in.)		
1	21"x21"	12-#10	15.24	1.88	4.06		
2	23"x19"	12-#10	15.24	1.88	3.39		
3	22"x19"	12-#10	15.24	1.88	3.39		
4	26"x15"	16-#8	12.64	1.88	1.56		
5	28"x28"	16-#10	20.32	1.88	4.47		
6	28"x27"	16-#10	20.32	1.88	4.22		
7	30"x18"	16-#10	20.32	1.88	1.98	<------	*used to be 29" x 18"
8	30"x19"	12-#14	27	1.88	2.22	<------	*used to be 30" x 19"
9	26"x26"	16-#10	20.32	2	2.74		
10	26"x26"	16-#10	20.32	1.88	3.97		
11	20"x20"	8-#10	10.16	1.88	6.22		
12	23"x20"	8-#9	8	1.88	6.43		
13	23"x20"	8-#9	8	1.88	6.43		
14	20"x20"	16-#10	20.32	1.88	2.47		
11(17+)	16"x16"	12-#11	18.72	2	2.12		
12(17+)	11"x11"	4-#10	5.08	1.88	4.71		
13(17+)	11"x11"	4-#10	5.08	1.88	4.71		
14(17+)	16"x16"	12-#11	18.72	2	2.12		

Below is a table detailing where all column loadings come from, and in the appendix further details of column loadings based on tributary area can be found.

Column Loadings

Per Floor Loading	(1-16)													
Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Total Load (k)	54.5	60.15	60.46	56.44	110.77	108.78	79.47	82.29	105.65	103.68	54.13	63.05	63	53.1

Per Floor Loading	(17-21)													
Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Total Load (k)	54.5	60.15	60.46	56.44	110.77	108.78	65.68	82.29	76.93	77.08	44.63	37.76	37.56	39.1

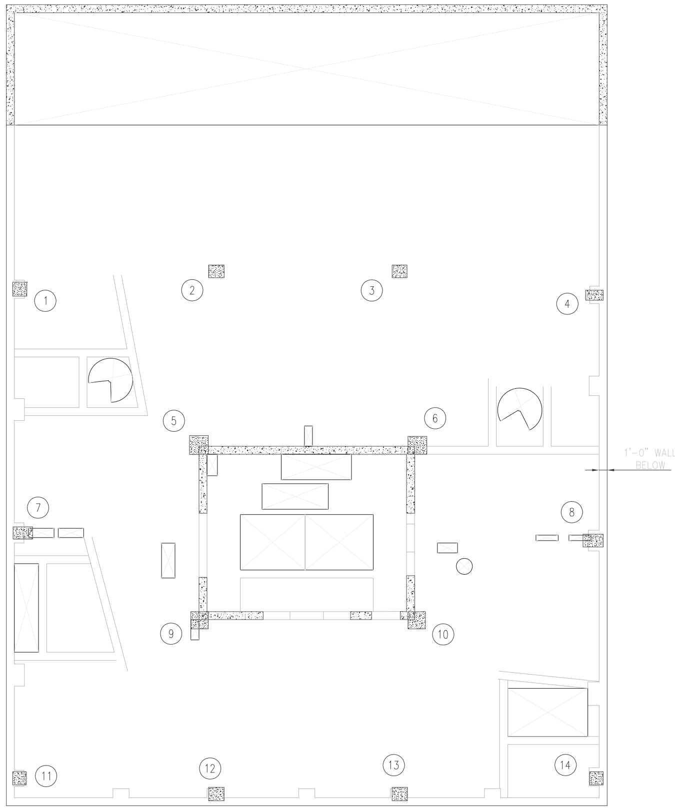
Per Floor Loading	Roof													
Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Total Load (k)	71.36	78.69	79.11	73.85	154.52	151.92	103.98	107.66	147.37	144.79	70.83	82.5	82.43	69.47

Column weight per floor	Floor Height (ft.)	
(Assume columns are 20x20")	9.66667	4.027779167
	10	4.166666667
	11	4.583333333

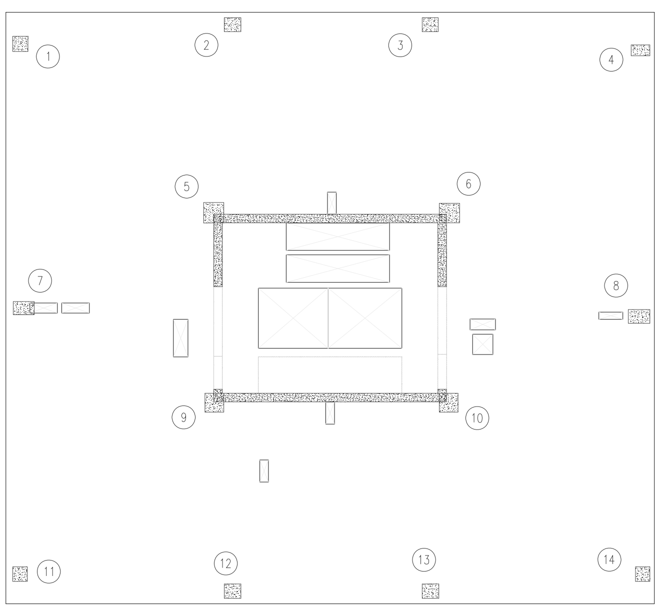
Balcony Loading			Column													
Dimen.	Area	Floor	1	2	3	4	5	6	7	8	9	10	11	12	13	14
5'x22.125'	110.625	5 to 15								10.325						20.65
5x27,5x22.125	110.625,136.34	16	19.0876	19.0876						10.325						20.65
5x27	136.34	17 to 21	19.0876	19.0876												

Table gives axial load in kips

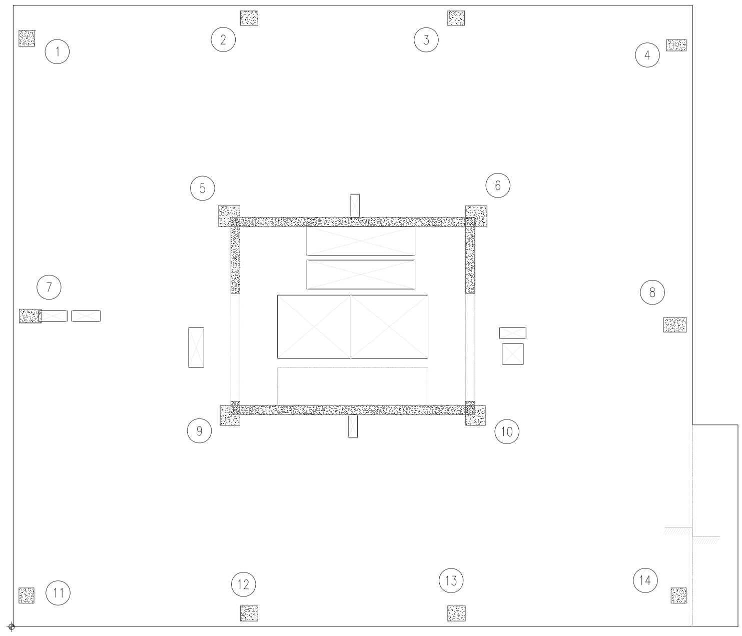
			Column													
Floor	Floor Height	Column Weight	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Roof 21	11.000	4.58	95.03	102.36	83.69	78.43	159.10	156.50	108.56	112.24	151.95	149.37	75.41	87.08	87.01	74.05
20	9.667	4.03	172.65	185.63	148.18	138.90	273.90	269.31	178.27	198.56	232.91	230.48	124.07	128.87	128.60	117.18
19	9.667	4.03	250.26	268.89	212.67	199.37	388.70	382.12	247.98	284.88	313.87	311.59	172.73	170.66	170.19	160.31
18	9.667	4.03	327.88	352.16	277.16	259.84	503.50	494.93	317.69	371.20	394.83	392.70	221.39	212.45	211.78	203.44
17	9.667	4.03	405.49	435.42	341.64	320.30	618.29	607.73	387.39	457.51	475.78	473.80	270.04	254.23	253.36	246.56
16	10.000	4.17	483.25	518.83	406.27	380.91	733.23	720.68	471.03	554.30	585.60	581.65	328.34	321.45	320.53	324.48
15	11.000	4.58	542.33	583.56	471.31	441.93	848.58	834.04	555.08	651.49	695.83	689.91	387.06	389.08	388.11	402.81
14	9.667	4.03	600.86	647.74	535.80	502.40	963.38	946.85	638.58	748.14	805.51	797.62	445.21	456.16	455.14	480.59
13	9.667	4.03	659.39	711.92	600.29	562.87	1078.18	1059.66	722.08	844.78	915.19	905.33	503.37	523.24	522.17	558.37
12	9.667	4.03	717.91	776.09	664.78	623.34	1192.98	1172.47	805.58	941.42	1024.87	1013.04	561.53	590.32	589.20	636.15
11	9.667	4.03	776.44	840.27	729.27	683.81	1307.78	1285.28	889.08	1038.07	1134.55	1120.75	619.69	657.40	656.23	713.93
10	9.667	4.03	834.97	904.45	793.75	744.27	1422.57	1398.08	972.57	1134.71	1244.22	1228.45	677.84	724.47	723.25	791.70
9	9.667	4.03	893.50	968.63	858.24	804.74	1537.37	1510.89	1056.07	1231.35	1353.90	1336.16	736.00	791.55	790.28	869.48
8	9.667	4.03	952.02	1032.80	922.73	865.21	1652.17	1623.70	1139.57	1327.99	1463.58	1443.87	794.16	858.63	857.31	947.26
7	9.667	4.03	1010.55	1096.98	987.22	925.68	1766.97	1736.51	1223.07	1424.64	1573.26	1551.58	852.32	925.71	924.34	1025.04
6	9.667	4.03	1069.08	1161.16	1051.70	986.14	1881.76	1849.31	1306.56	1521.28	1682.93	1659.28	910.47	992.78	991.36	1102.81
5	9.667	4.03	1127.61	1225.34	1116.19	1046.61	1996.56	1962.12	1390.06	1617.92	1792.61	1766.99	968.63	1059.86	1058.39	1180.59
4	9.667	4.03	1186.14	1289.52	1180.68	1107.08	2111.36	2074.93	1473.56	1704.24	1902.29	1874.70	1026.79	1126.94	1125.42	1237.72
3	10.000	4.17	1244.80	1353.83	1245.31	1167.69	2226.30	2187.88	1557.20	1790.70	2012.11	1982.55	1085.09	1194.16	1192.59	1294.99
2	10.000	4.17	1303.47	1418.15	1309.93	1228.29	2341.23	2300.82	1640.83	1877.15	2121.92	2090.39	1143.38	1261.37	1259.75	1352.25
1	12.000	5.00	1362.97	1483.30	1375.39	1289.73	2457.00	2414.60	1725.30	1964.44	2232.57	2199.07	1202.51	1329.42	1327.75	1410.35
	209.000															



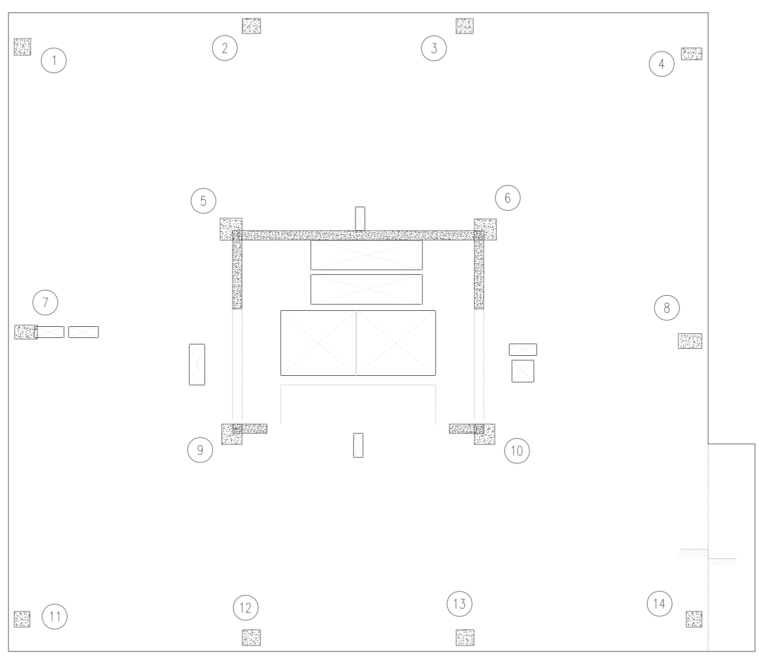
GROUND FLOOR FRAMING PLAN



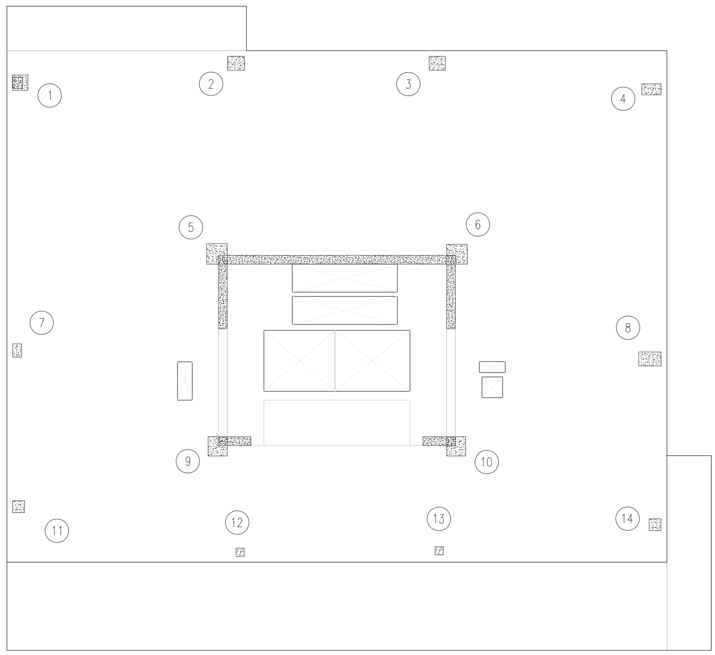
LEVEL 2 THRU LEVEL 4 FRAMING PLAN



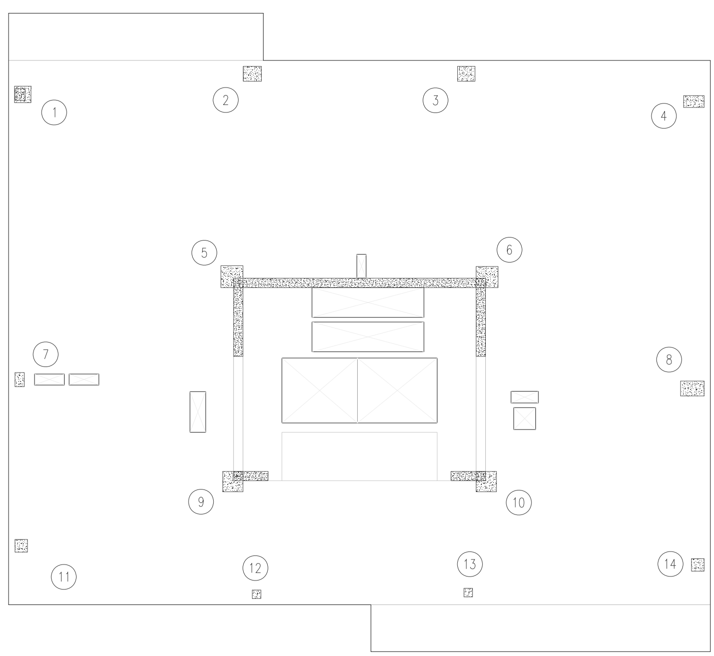
LEVEL 5 THRU 10 FRAMING PLAN



LEVEL 11 THRU 15 FRAMING PLAN



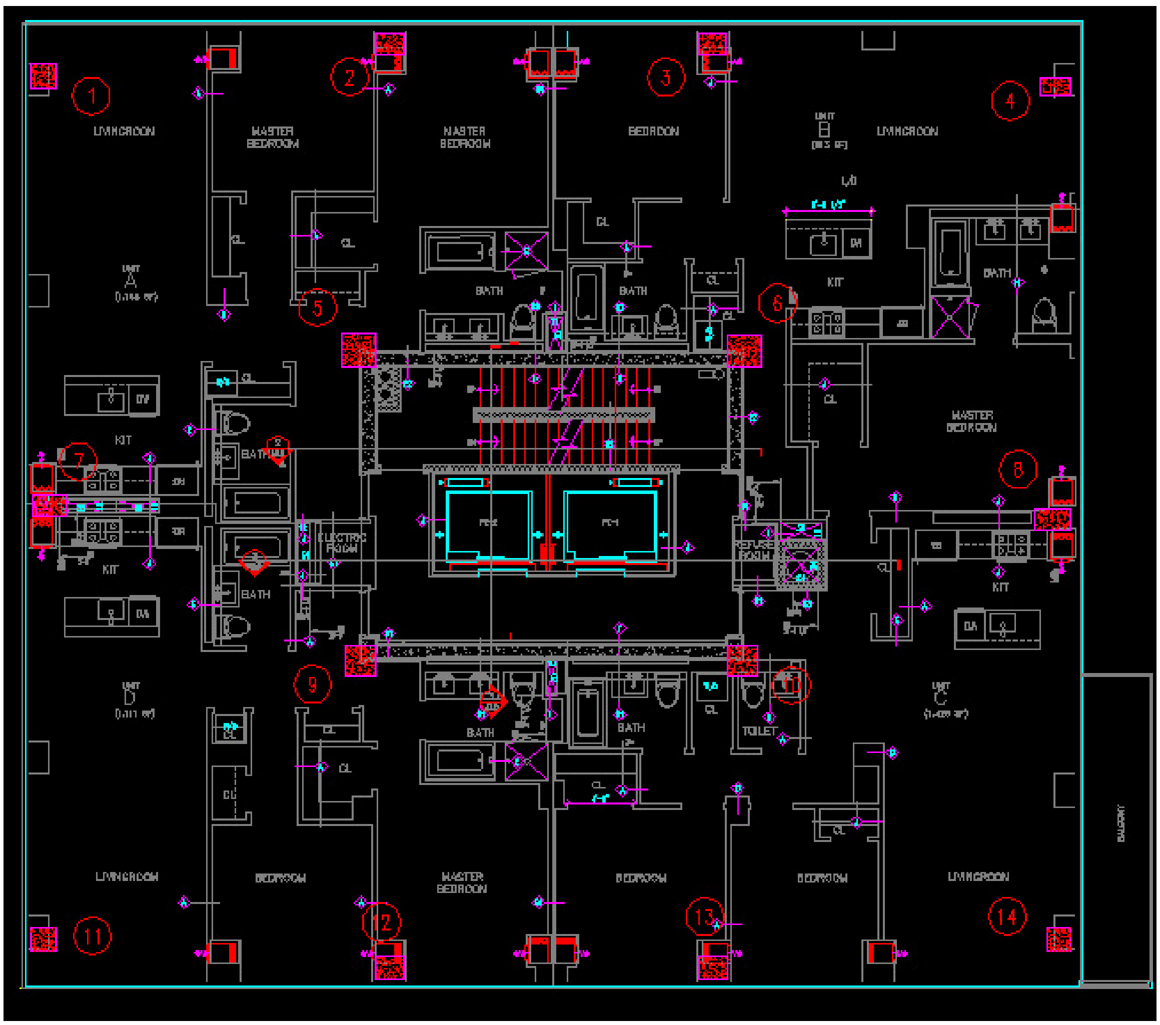
LEVEL 16 FRAMING PLAN



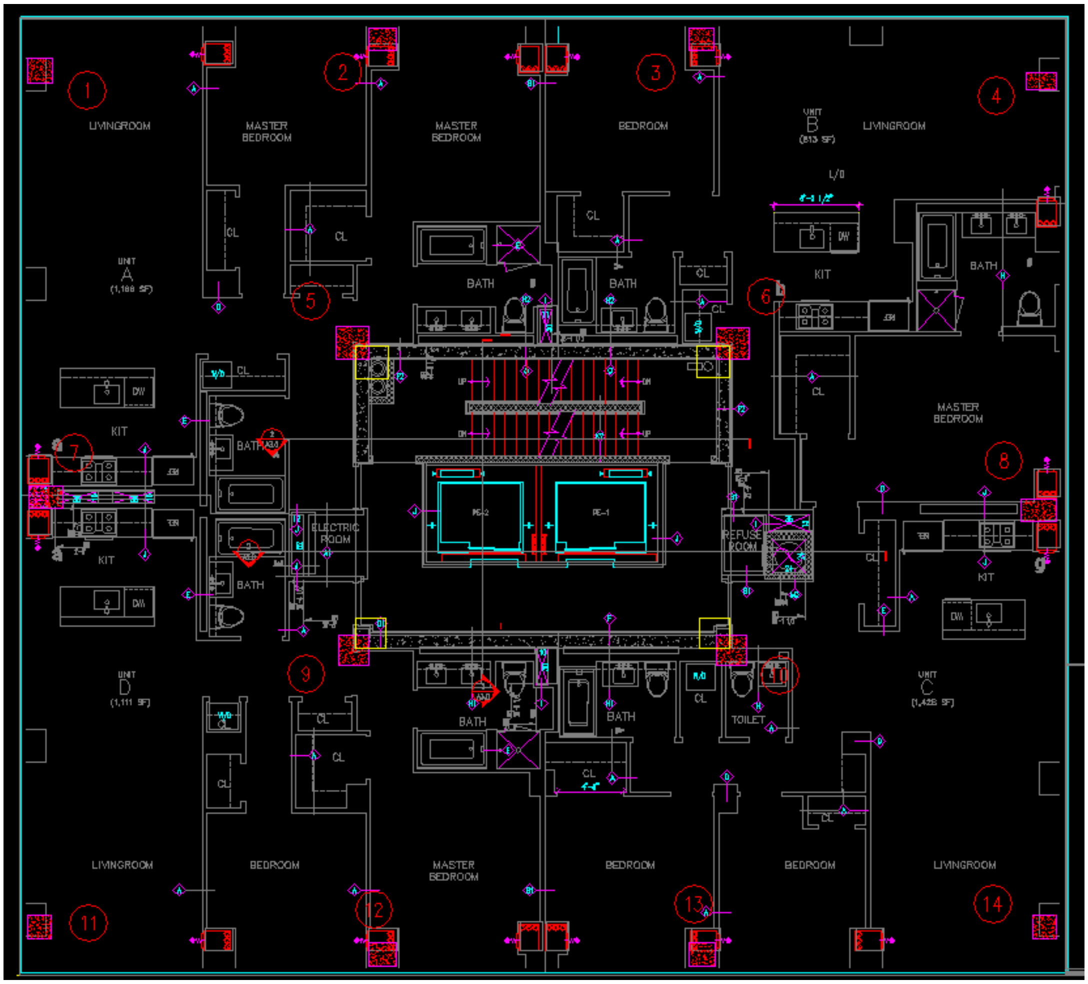
LEVEL 17 THRU 21 FRAMING PLAN

Architectural Impact of New Column Design

The new column layout necessitates a few changes be made in the current floor plan. Below is an architectural study of how the new layout impacts the old floor plan. The new columns are larger in size than the old ones, but many old columns have been removed freeing more inhabitable space. The only negative this new arrangement has on the floor plan is the encroachment of columns 5,6,9 and 10 into the common hallways. If architects do not wish to rearrange the hallways, this problem can easily be solved by offsetting the columns into the stairwell. Code allows a maximum of 8” overlap between door swing and radial distance from the stairs to allow for safe movement through the stairwell. Since the columns are a maximum of 28”, they would encroach upon the stairwell an acceptable distance without hindering flow.



Typical new floor plan for floors 5-10



Possible repositioning of columns 5, 6, 9, and 10 if they pose an architectural problem

Post-Tensioned Floor Slab Design

A post-tensioned two-way floor slab system poses several advantages over a regularly reinforced system. The larger bay sizes created by eliminating columns are more conducive to PT once a 20' x 20' bay is reached. After 110 Third Avenue was designed using these larger bay sizes, it became quite apparent that post-tensioning would be a viable alternate floor system. Unfortunately, the initial ambition of actually using a thinner floor system with post-tensioning proved to be unrealistic. Punching shear controlled the thickness of the floor slab almost all over each floor, so trying to use a thinner slab would be impossible. Also, using a drop panel system with drop panels equal in depth to the current floor thickness would create an excess amount of formwork that would offset any cost benefits from a thinner floor.

The majority of post-tensioned analysis was performed using RAM Conce*pt*. This program is new to the AE computer labs and presented many challenges as students learned to navigate their way through the finer points of their models. The results of modeling a new floor system for 110 Third Avenue, however, were better than expected. RAM Conce*pt* produced a design that agrees with the expected layout for a basically square floor plan with slightly irregular bay sizes. In the model, banding of tendons runs in the north-south direction along column lines, while a regular spacing of tendons runs in the east-west direction. Although not visible on the plans, each longitudinal band consists of banding simply because RAM Conce*pt* assumes that an excess of 7 strands per tendon will be carried into a second closely spaced tendon. Conce*pt* also provided a better analysis than possible by hand calculation. Since Conce*pt* uses a finite analysis method, it stands at an advantage over using design strips, and then analyzing it using conventional analysis techniques. Conce*pt* claims to predict the elastic behavior of a slab much more accurately than frame models.

Note that in the model Conce*pt* produced, almost all columns failed its punching shear check. As Conce*pt* programmers freely admit in their help section, the punching shear check is flawed. As a precaution, punching shears for all columns were checked using a hand analysis, and all columns passed.

Punching Shear Check

Loading	214	psf factored
F'c	5000	psi
d	6.5	inches

	Interior	Edge	Corner
α_s	40	30	20

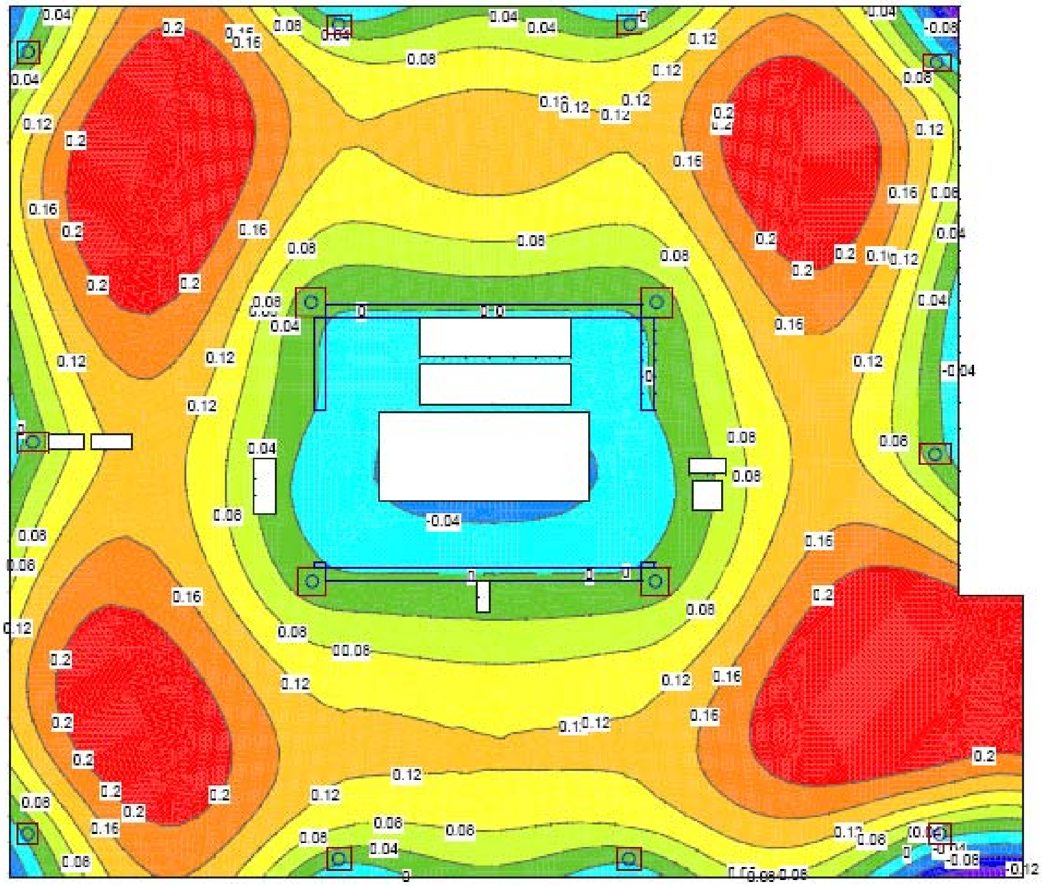
f_{pc} From Concept model

Column	Length	Width	b_o	Tributary Area (ft.^2)	V_u (kips)	α_s	β_p	f_{pc} (psi)	V_p	ΦV_c (kips)	OK?
1	21	21	110	254.87	53.41831	20	3.364	207	0	160.8455	YES
2	23	19	110	281.06	59.02891	30	3.500	125	0	152.8245	YES
3	22	19	108	282.54	59.38353	30	3.500	125	0	150.0459	YES
4	26	15	108	263.74	55.40194	20	2.315	186	0	115.5573	YES
5	28	28	138	551.85	116.3271	40	3.500	204	0	207.6694	YES
6	28	27	136	542.57	114.3924	40	3.500	201	0	204.063	YES
7	29	18	120	371.36	78.17849	30	3.500	268	0	191.8141	YES
8	30	19	124	384.51	80.90194	30	3.500	268	0	198.2079	YES
9	26	26	130	526.33	111.0649	40	3.500	225	0	199.6232	YES
10	26	26	130	517.12	109.094	40	3.500	217	0	198.1022	YES
11	20	20	106	252.95	53.08768	20	3.302	229	0	156.1508	YES
12	23	20	112	294.62	61.88691	30	3.500	158	0	161.0085	YES
13	23	20	112	294.41	61.84197	30	3.500	169	0	162.8103	YES
14	20	20	106	248.11	52.05192	20	3.302	172	0	147.3144	YES
11(17)	16	16	90	208.57	43.88164	20	3.000	229	0	123.2151	YES
12(17)	11	11	70	176.45	37.30518	30	3.500	158	0	100.6303	YES
13(17)	11	11	70	175.52	37.10616	30	3.500	169	0	101.7564	YES
14(17)	16	16	90	182.71	38.3476	20	3.000	172	0	115.7124	YES

*note some fpc<125, so use 125 as minimum per ACI
*note conservative estimate of Vp

Following are excerpts from a report prepared by RAM Conce*pt* that show the reinforcing plan, deflections, and status of the floor among other things.

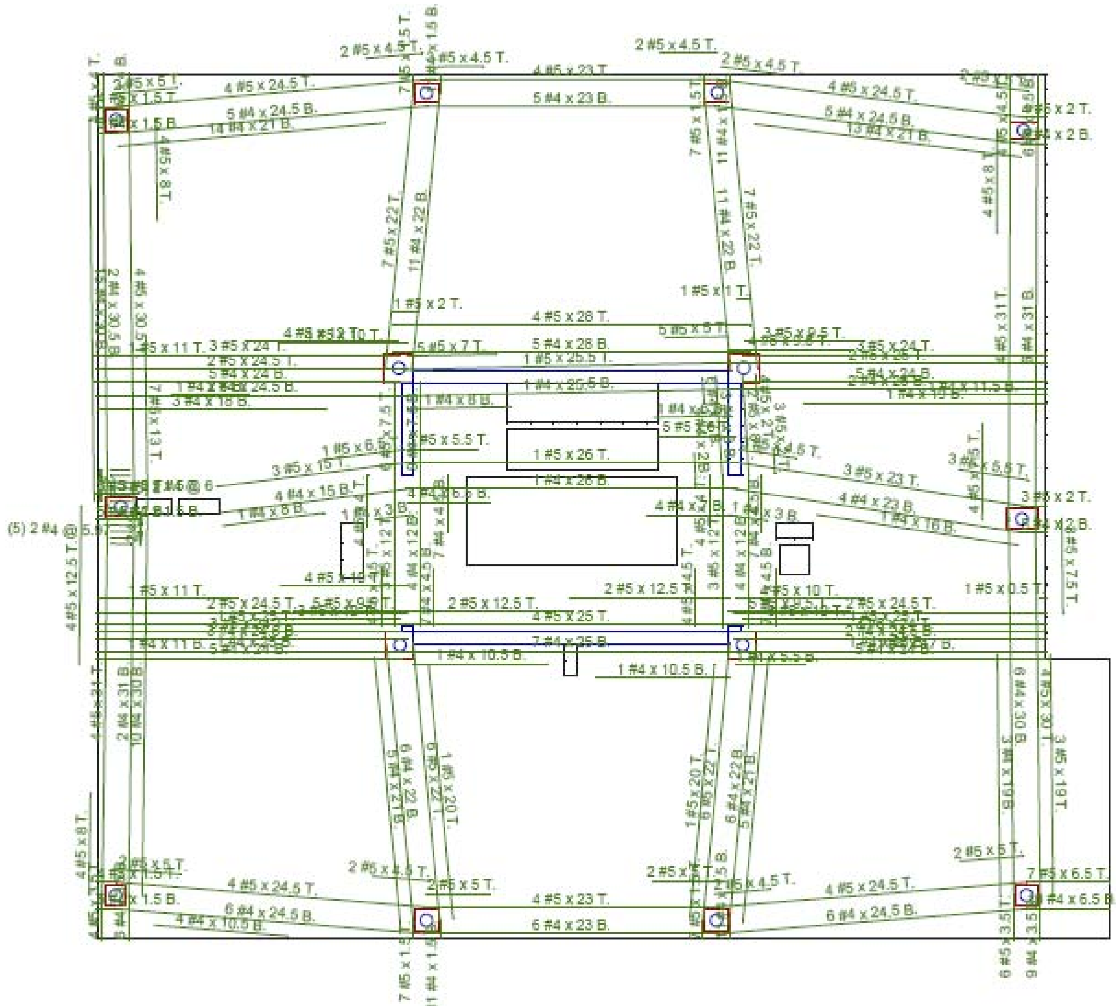
Deflection Plan



Deflection Plan in inches

Note maximum deflection is .2 inches which is about L/1200 assuming a 20’ span. This will meet all deflection criteria for the floor.

Reinforcing Plan



Post-tensioned reinforcing

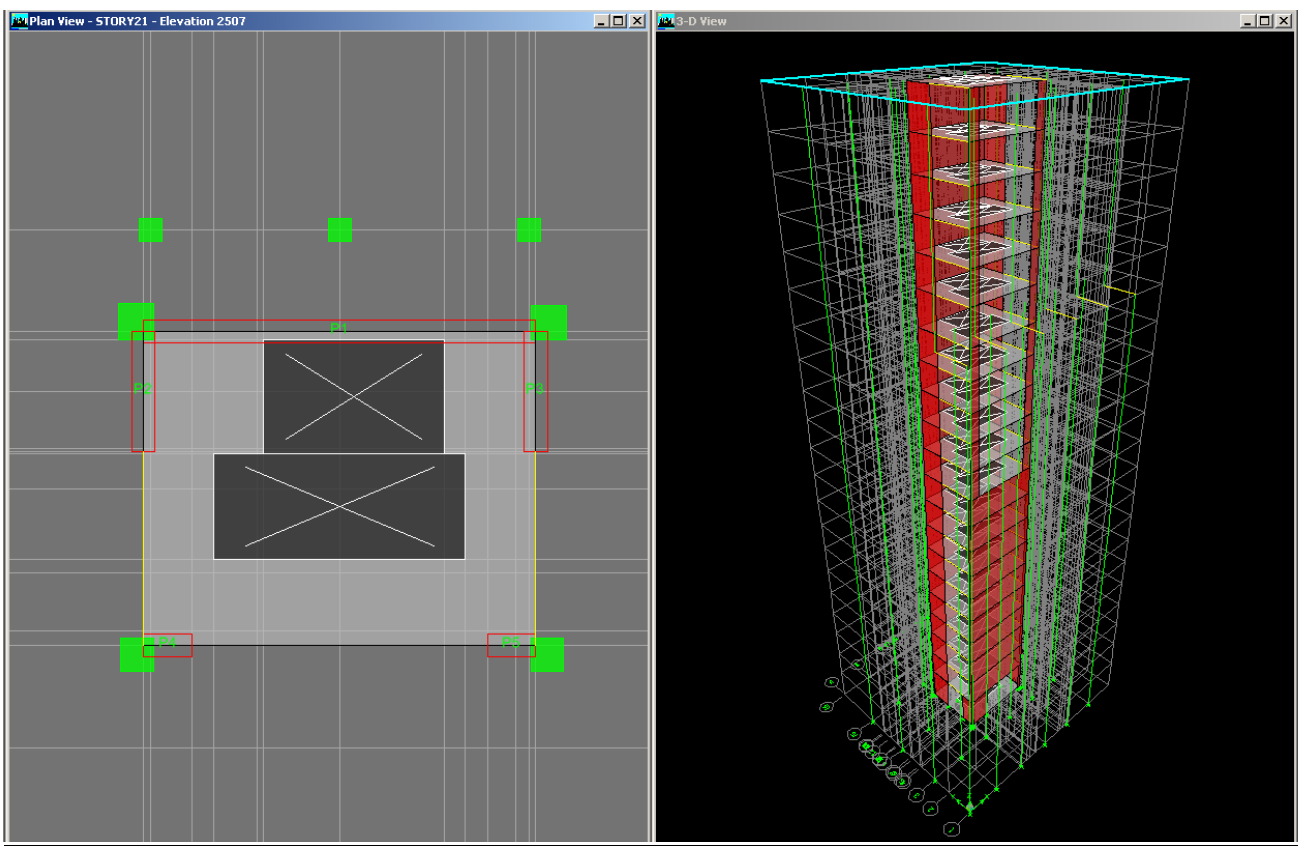
Lateral System Design

The process of designing a lateral system was a two-step process. First, a shear-wall-only system was attempted. Second, after realizing a shear wall system was impractical, another system was designed using a combined moment-frame and shear wall system to resist lateral forces.

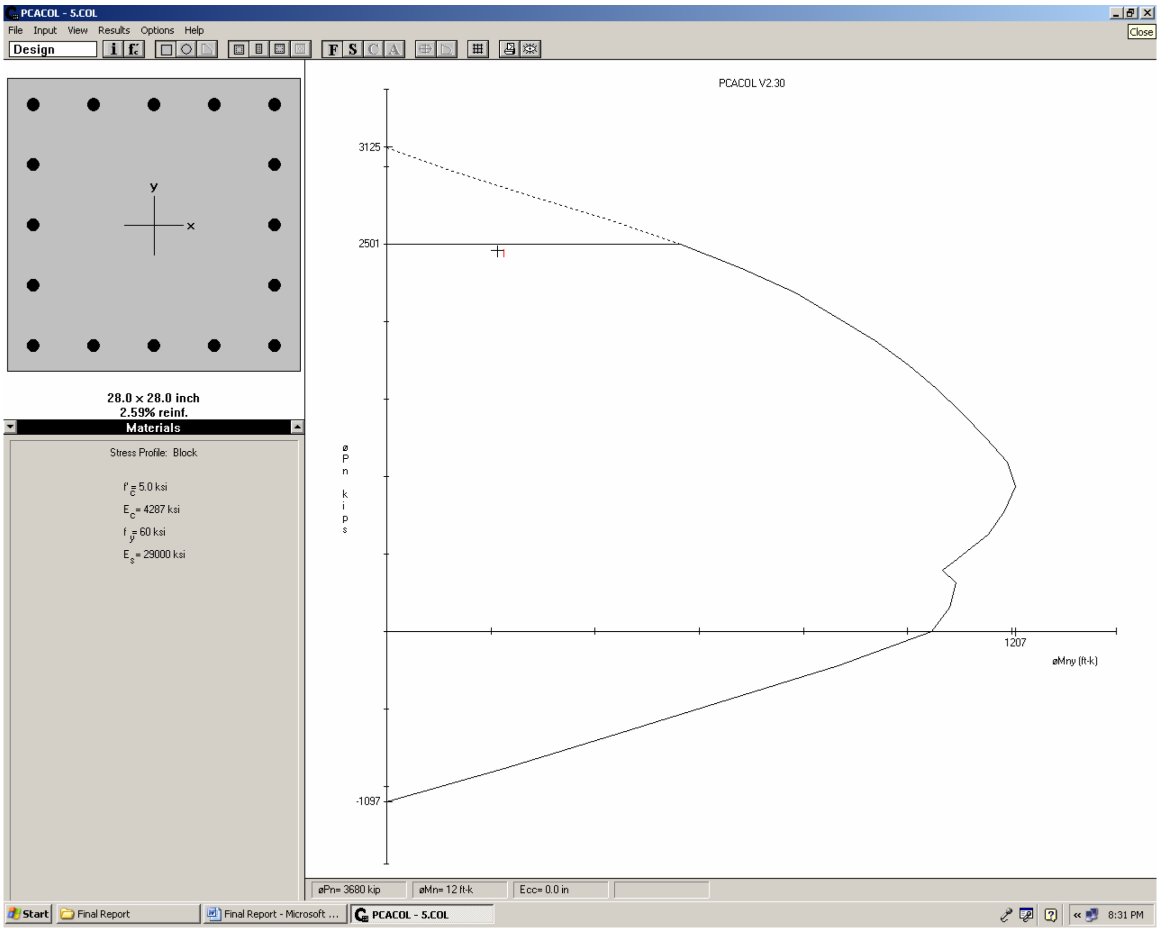
Initial Design

A computer model using ETABS was generated to assist in the lateral analysis of 110 Third Avenue. The shear walls act as vertical cantilever beams which transfer lateral forces from the superstructure to the foundation. In 110 Third Avenue, the shear walls are coupled together with link beams, as reflected in the ETABS model. In the included first ETabs analysis, each floor is assumed to act as a rigid diaphragm for loads in the plane of the floor. Thus, the shear walls alone are assumed to resist all lateral forces. Normalized bays with even column spacing are used in the model, even though the actual building has varying sizes of bays and columns. Both hand-calculated loads and those generated by ETABS were used in the analysis. Using this simplified model made its construction in ETABS more efficient, and should not have posed any problem to analyzing the structure. From a practical standpoint, the structure should not drift more than H/400 to prevent serviceability issues from arising.

Below are some graphics of the computer model generated using ETABS. They are provided simply as reference to demonstrate the setup of the model.



The original design of the lateral system was simple in that it relied on shear walls as the sole resisting elements in the building. All columns were treated as leaning columns, and had been designed with only uneven floor loading creating moments. The first Etabs model reflects this assumption in that it contains dummy columns intended to provide stability only. A lateral system that consisted only of shear walls was intended to be fast, simple, and offer an amount of redundancy that would make 110 Third Avenue even safer. If a shear wall were to fail, the columns would still be able to handle the increased moment placed on them. From the PCACOL program, most columns are well below their ΦM_n. A 25% increase, based on seismic code provisions, on each one would not cause significant harm to the building.



Column 5 Interaction Diagram displaying a low Mn value

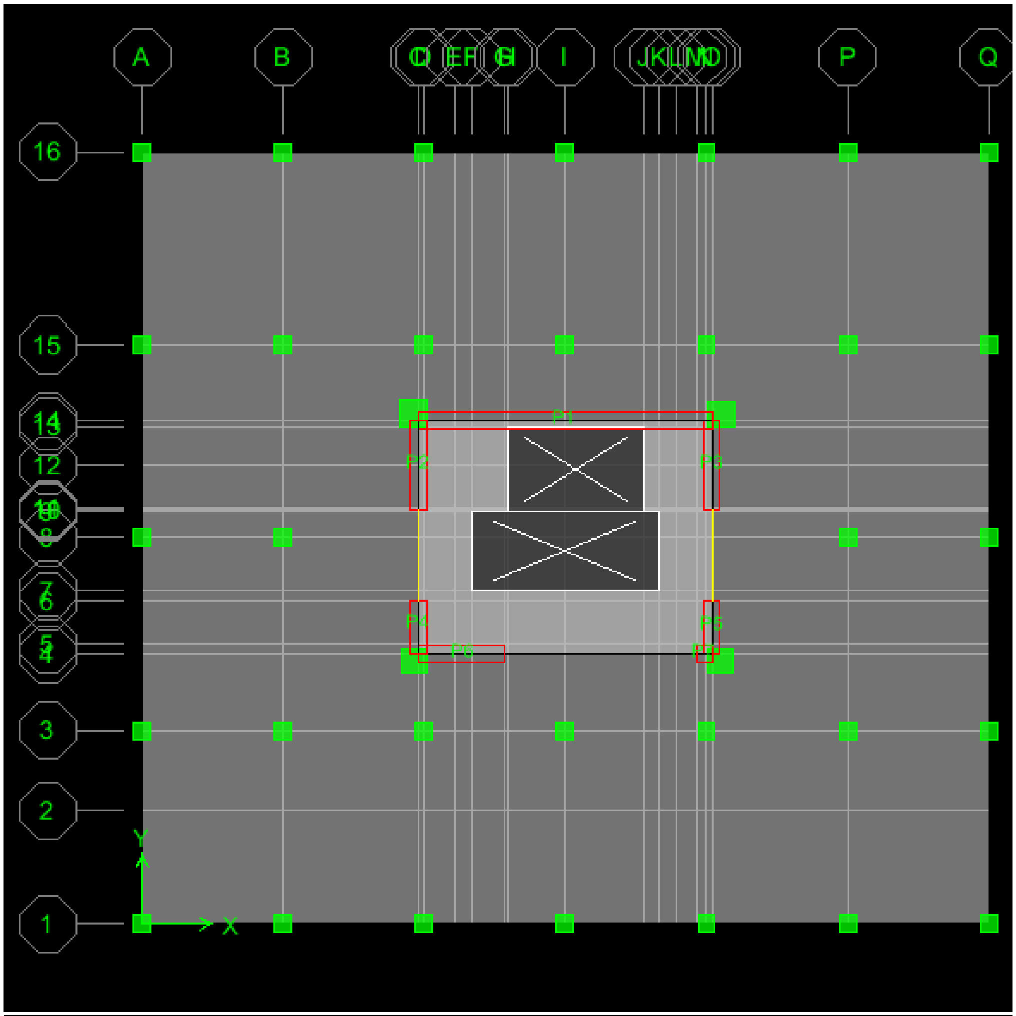
Based on the performance of a shear wall lateral-force-resisting-system in Tech Report 3, an improved system had to be devised for the final design. Increasing all shear walls to a total of 15 inches of thickness reduced drifts but not to a point within an acceptable limit of H/400. The implications of increasing the thickness include a slight encroachment of hallway space in the building. The architectural impact of taking an extra 1.5 inches on either side of the hallway (see plans) would be minimal and likely would not cause any significant problems.

Also, when designing a new column layout, four new columns were placed at the corners of the shear walls. The addition of these columns impacted the length of the shear walls and how they interacted as a complete system. Although cast monolithically, ETabs would not recognize shear walls framing into the columns while still acting as a rigid wall system. Therefore, for analysis purposes the shear walls were continued through these columns to their original intersection points from Tech Report 3. As a consequence, the columns then had to be analyzed for the additional load carrying capacity required by assuming the shear walls continue through a portion of each of them. This assumption relies on the columns to resist the pier moment at the edge of the shear walls. By finding the moments in the shear walls outputted by ETabs and dividing along the length of the wall, the moment could be resolved into a force couple. This force couple could be applied to the columns as an increased axial load. The assumption that all of the moment found in the shear walls is carried into the columns, however, is a greatly over-exaggerated requirement to place on the columns. The following tables detail the original assumption that the corner columns at the shear walls carry the entirety of the moments in the shear walls to which they are cast monolithically.

Pier	Force (in k)	Column(s) Acting On	Floor of Max Moment	Length of Pier at Floor of Occurrence	Axial Force on Column	Units
1	19571.749	5	2	94	208.21	k
2	154703.37	5,6	1	309	500.66	k
3	18414.116	6	2	94	195.89	k
4	32888.34	10	1	57	576.99	k
5	107429.88	9,10	2	309	347.67	k
6	19702.125	9	1	89.5	220.14	k
7	16641.206	9	1	57	291.95	k

Column	New Column Load	Units
5	3165.87	k
6	2818.70	k
9	3029.69	k
10	2711.16	k

As evident above, not all of the moment in the shear walls is going to transfer through a few inches of overlap into the columns. It would be unnecessary to increase the size of the corner columns simply to accommodate the few inches of overlap. See the “final design” section of this report for further information and a resolution regarding how to treat these pier moments and axial forces.



Overall view of the location of shear walls in the central core and the leaning columns from the first model analysis

Lateral System Results- Shear Wall Only

ASCE7-02 does not provide a detailed description of story drift limits due to wind (sec. B.1.2) but does give drift limits cause by seismic forces (sec. 9.5.2.8). The following table compares allowable drifts to actual drifts due to seismic forces.

Allowable Story Drifts based on ASCE7-02 sec. 9.5.2.8

Use Group	II	
Allowable Drift:	$.015h_{sx}$	L/67

Floor	Height (in.)	Allowable Drift (in)	Seismic X	Drift (in.)	OK?	Seismic Y	Drift (in.)	OK?
21	144.00	2.16	0.002595	0.37368	OK	0.005613	0.808272	OK
20	116.00	1.74	0.002663	0.308908	OK	0.005683	0.659228	OK
19	116.00	1.74	0.002759	0.320044	OK	0.005773	0.669668	OK
18	116.00	1.74	0.002859	0.331644	OK	0.005871	0.681036	OK
17	116.00	1.74	0.002941	0.341156	OK	0.005999	0.695884	OK
16	120.00	1.80	0.003496	0.41952	OK	0.006106	0.73272	OK
15	132.00	1.98	0.003459	0.456588	OK	0.006142	0.810744	OK
14	116.00	1.74	0.003314	0.384424	OK	0.00616	0.71456	OK
13	116.00	1.74	0.002997	0.347652	OK	0.006144	0.712704	OK
12	116.00	1.74	0.002507	0.290812	OK	0.006089	0.706324	OK
11	116.00	1.74	0.001671	0.193836	OK	0.00599	0.69484	OK
10	116.00	1.74	0.000851	0.098716	OK	0.005807	0.673612	OK
9	116.00	1.74	0.000834	0.096744	OK	0.005599	0.649484	OK
8	116.00	1.74	0.000751	0.087116	OK	0.005338	0.619208	OK
7	116.00	1.74	0.000678	0.078648	OK	0.005016	0.581856	OK
6	116.00	1.74	0.000629	0.072964	OK	0.004611	0.534876	OK
5	116.00	1.74	0.000565	0.06554	OK	0.004117	0.477572	OK
4	116.00	1.74	0.000485	0.05626	OK	0.003519	0.408204	OK
3	120.00	1.80	0.000388	0.04656	OK	0.002799	0.33588	OK
2	120.00	1.80	0.000265	0.0318	OK	0.001899	0.22788	OK
1	144.00	2.16	0.00014	0.02016	OK	0.000778	0.112032	OK

The criterion of drift must be less than or equal to H/400 was used to evaluate drifts caused by wind in the N-S and E-W directions. The following table evaluates ASCE7-02 loading and NYC building code loading in terms of drift.

Wind Drift Check

Drift based on good judgement, not code		
Allowable Drift:	$.0025h_{sx}$	H/400

ASCE7-02 Loadings

Floor	Height (in.)	Allowable Drift (in)	WINDX Wind X	Drift (in.)	OK?	WINDY Wind Y	Drift (in.)	OK?
21	144.00	0.36	0.002247	0.323568	OK	0.004574	0.658656	NOT OK
20	116.00	0.29	0.002316	0.268656	OK	0.004636	0.537776	NOT OK
19	116.00	0.29	0.002419	0.280604	OK	0.004723	0.547868	NOT OK
18	116.00	0.29	0.00254	0.29464	NOT OK	0.004825	0.5597	NOT OK
17	116.00	0.29	0.002664	0.309024	NOT OK	0.004966	0.576056	NOT OK
16	120.00	0.30	0.003241	0.38892	NOT OK	0.005097	0.61164	NOT OK
15	132.00	0.33	0.003306	0.436392	NOT OK	0.005172	0.682704	NOT OK
14	116.00	0.29	0.00327	0.37932	NOT OK	0.005242	0.608072	NOT OK
13	116.00	0.29	0.003066	0.355656	NOT OK	0.00529	0.61364	NOT OK
12	116.00	0.29	0.00268	0.31088	NOT OK	0.005312	0.616192	NOT OK
11	116.00	0.29	0.001923	0.223068	OK	0.005302	0.615032	NOT OK
10	116.00	0.29	0.001108	0.128528	OK	0.005219	0.605404	NOT OK
9	116.00	0.29	0.001021	0.118436	OK	0.005113	0.593108	NOT OK
8	116.00	0.29	0.001001	0.116116	OK	0.004955	0.57478	NOT OK
7	116.00	0.29	0.000959	0.111244	OK	0.004735	0.54926	NOT OK
6	116.00	0.29	0.000898	0.104168	OK	0.004427	0.513532	NOT OK
5	116.00	0.29	0.000814	0.094424	OK	0.004021	0.466436	NOT OK
4	116.00	0.29	0.000707	0.082012	OK	0.003495	0.40542	NOT OK
3	120.00	0.30	0.000571	0.06852	OK	0.002826	0.33912	NOT OK
2	120.00	0.30	0.000401	0.04812	OK	0.001948	0.23376	OK
1	144.00	0.36	0.000178	0.025632	OK	0.000807	0.116208	OK

L/ Value

Wind X	Wind Y
445.04	218.63
431.78	215.70
413.39	211.73
393.70	207.25
375.38	201.37
308.55	196.19
302.48	193.35
305.81	190.77
326.16	189.04
373.13	188.25
520.02	188.61
902.53	191.61
979.43	195.58
999.00	201.82
1042.75	211.19
1113.59	225.89
1228.50	248.69
1414.43	286.12
1751.31	353.86
2493.77	513.35
5617.98	1239.16

NYC Building Code Loadings

Floor	Height	Allowable Drift (in)	NYCX Wind X	Drift (in.)	OK?	NYCY Wind Y	Drift (in.)	OK?
21	144.00	0.36	0.002247	0.323568	OK	0.004574	0.658656	NOT OK
20	116.00	0.29	0.002316	0.268656	OK	0.004636	0.537776	NOT OK
19	116.00	0.29	0.002419	0.280604	OK	0.004723	0.547868	NOT OK
18	116.00	0.29	0.00254	0.29464	NOT OK	0.004825	0.5597	NOT OK
17	116.00	0.29	0.002664	0.309024	NOT OK	0.004966	0.576056	NOT OK
16	120.00	0.30	0.003241	0.38892	NOT OK	0.005097	0.61164	NOT OK
15	132.00	0.33	0.003306	0.436392	NOT OK	0.005172	0.682704	NOT OK
14	116.00	0.29	0.00327	0.37932	NOT OK	0.005242	0.608072	NOT OK
13	116.00	0.29	0.003066	0.355656	NOT OK	0.00529	0.61364	NOT OK
12	116.00	0.29	0.00268	0.31088	NOT OK	0.005312	0.616192	NOT OK
11	116.00	0.29	0.001923	0.223068	OK	0.005302	0.615032	NOT OK
10	116.00	0.29	0.001108	0.128528	OK	0.005219	0.605404	NOT OK
9	116.00	0.29	0.001021	0.118436	OK	0.005113	0.593108	NOT OK
8	116.00	0.29	0.001001	0.116116	OK	0.004955	0.57478	NOT OK
7	116.00	0.29	0.000959	0.111244	OK	0.004735	0.54926	NOT OK
6	116.00	0.29	0.000898	0.104168	OK	0.004427	0.513532	NOT OK
5	116.00	0.29	0.000814	0.094424	OK	0.004021	0.466436	NOT OK
4	116.00	0.29	0.000707	0.082012	OK	0.003495	0.40542	NOT OK
3	120.00	0.30	0.000571	0.06852	OK	0.002826	0.33912	NOT OK
2	120.00	0.30	0.000401	0.04812	OK	0.001948	0.23376	OK
1	144.00	0.36	0.000178	0.025632	OK	0.000807	0.116208	OK

L/ Value

Wind X	Wind Y
445.04	218.63
431.78	215.70
413.39	211.73
393.70	207.25
375.38	201.37
308.55	196.19
302.48	193.35
305.81	190.77
326.16	189.04
373.13	188.25
520.02	188.61
902.53	191.61
979.43	195.58
999.00	201.82
1042.75	211.19
1113.59	225.89
1228.50	248.69
1414.43	286.12
1751.31	353.86
2493.77	513.35
5617.98	1239.16

Final Design

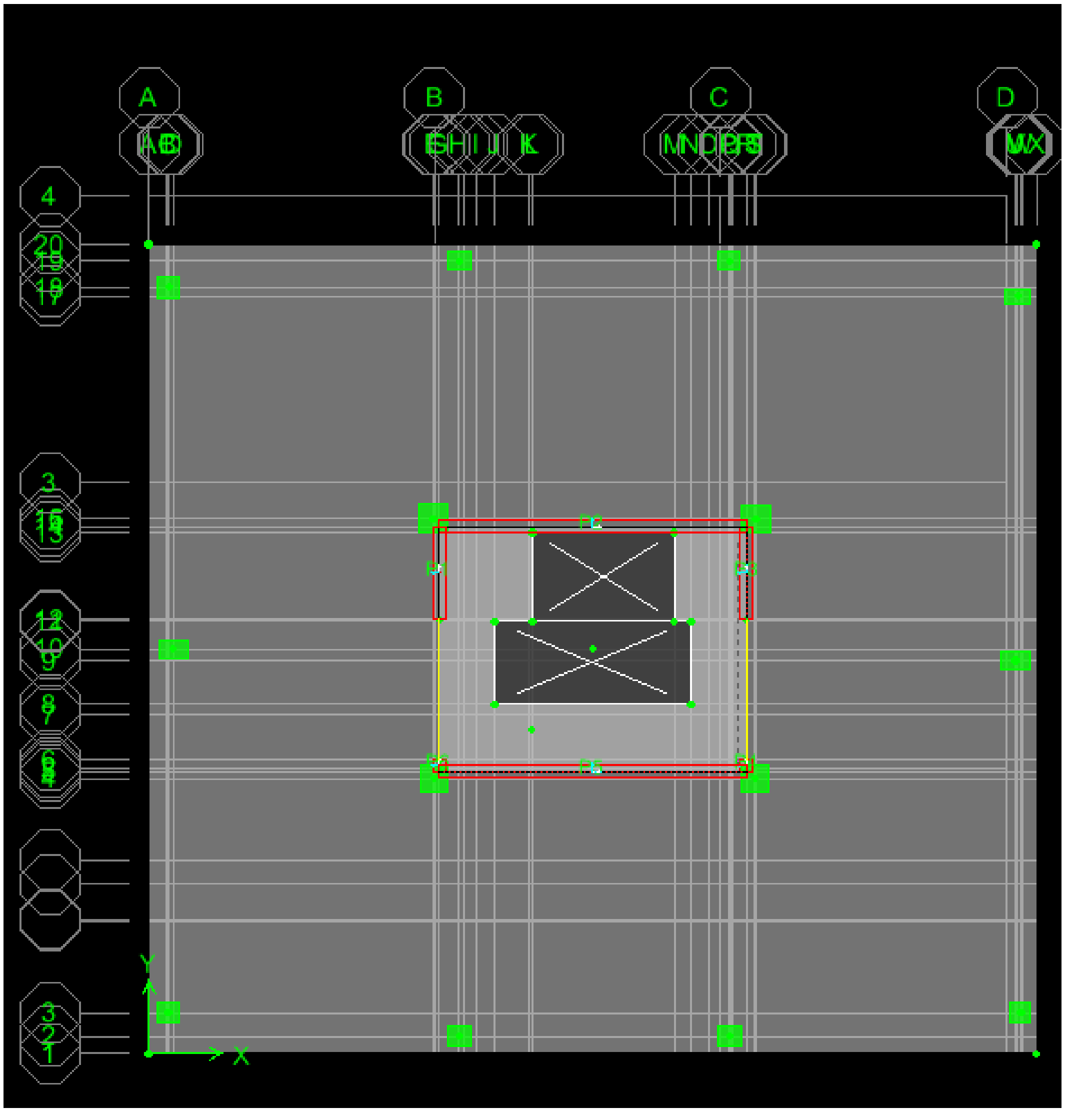
Initial design of the lateral system was conducted assuming only the shear walls would resist lateral forces. After a thorough analysis using ETabs, it was determined that such a system would be disadvantageous in a number of ways. First, as Technical Report 3 proved, a shear wall system where all walls were 12" thick was not adequate in terms of story drift. A thicker, 15" wall system was analyzed, but again story drifts were too high. It was plain to see that increasing the shear wall thickness would not adequately reduce story drifts to a reasonable amount unless the walls were disruptively thick. It would be impossible to add shear walls farther away from the center of mass in order to create a more effective shear wall system. The basic architecture of the building barred the placing of any structural elements along the outside perimeter of the building, and a shear wall within the residential units would also be disruptive. Each unit is relatively small to support the implementation of a shear wall large enough to have a positive impact on the lateral system.

Another system had to be devised to reduce overall story drifts to an acceptable level. The lateral system was designed after the columns, and it was clear that treating the columns as leaning columns was a waste of valuable space. They supported mainly axial loading and originally did not factor into the lateral system at all. It seemed the designers of 110 Third Avenue were right all along in using a combined lateral resisting system.

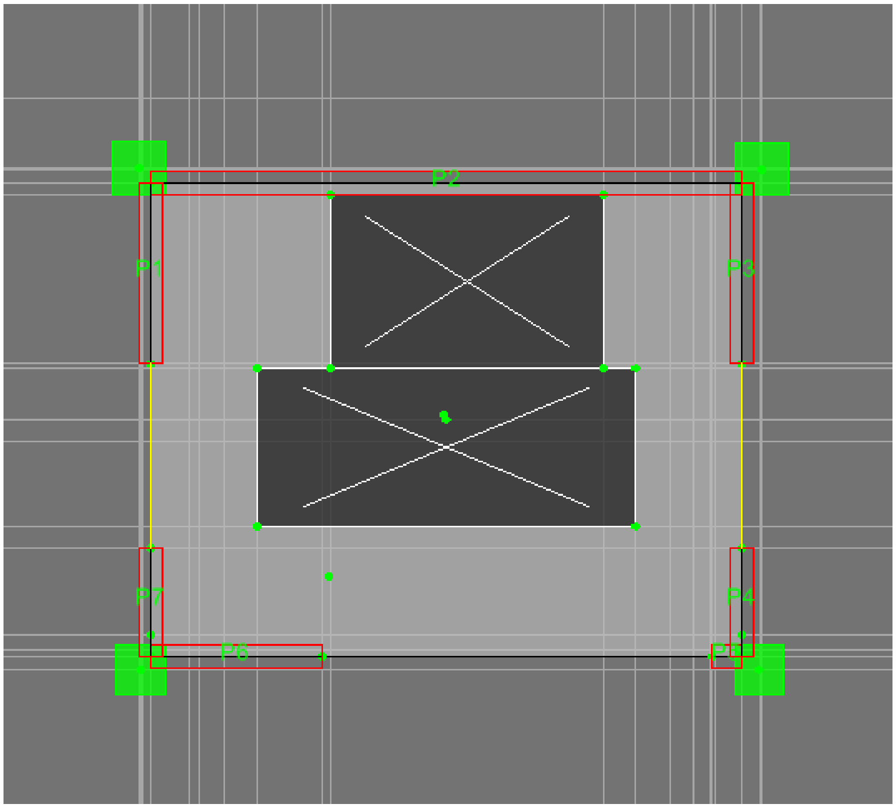
Another model was created in ETabs using the columns and floor slab as a moment frame in addition to the shear walls to resist lateral forces. The result was a highly effective model that produced relatively small drifts. Even better, the size of the shear walls could be reduced back down to 12" thick. The combined lateral system was definitely the better choice.

Reinforcement design for the shear walls surrounding the elevator core was based on ACI Chapter 11 and 21, an example from The Seismic Design Handbook and another example from Design of Concrete Structures. The interesting setup that places columns at the ends of the shear walls posed a dilemma for designing the boundary elements in the wall. The increased load at the ends of the wall due to resolving the moment on the wall into a force couple placed an added burden on whatever element was considered to be the end of the wall. Instead of placing an extra axial load on the column, a boundary element could be designed at the end of the shear wall leading up to the column. Interestingly enough, designs proved that a boundary element could be confined to a 12" x 12" area easily concealed within all shear walls since all of them are 12" thick. The example shear wall design provided in Appendix D can be applied to most piers with little variation since $P_{umax} = 501$ k < $\Phi P_n = 507.56$ k except for pier 4 which will require a little more reinforcing for $P_u = 577$ k. In all cases, the boundary element can be confined 12" x 12", the standard shear wall size. This route of design was advantageous over placing an added burden on the columns and having to subsequently upsize them. The result is a shear wall reinforcing system that has either 12 #6 bars or 12#7 bars, in the case of pier 4, acting as the boundary element adjacent to the column.

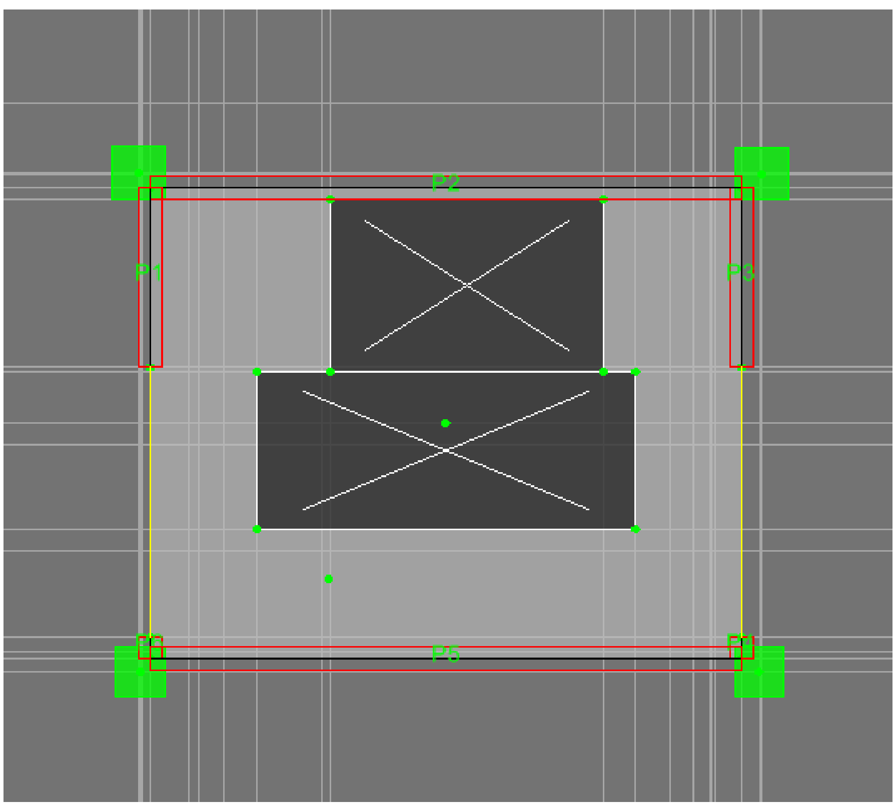
Floor Plans of Final Design



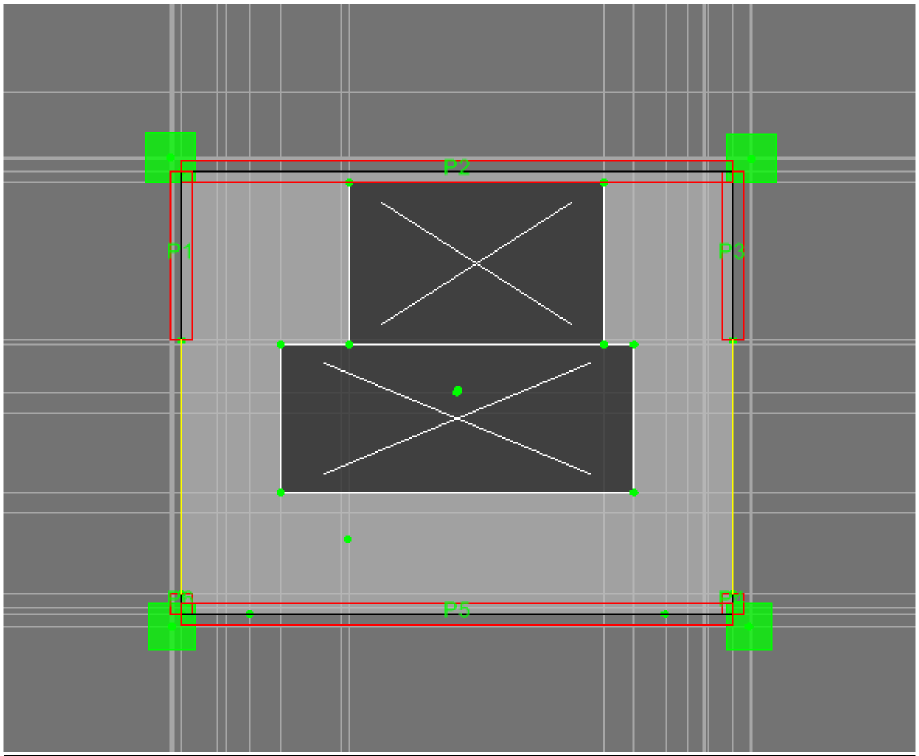
The redesigned model that removes the leaning columns and introduces the actual structure to the model



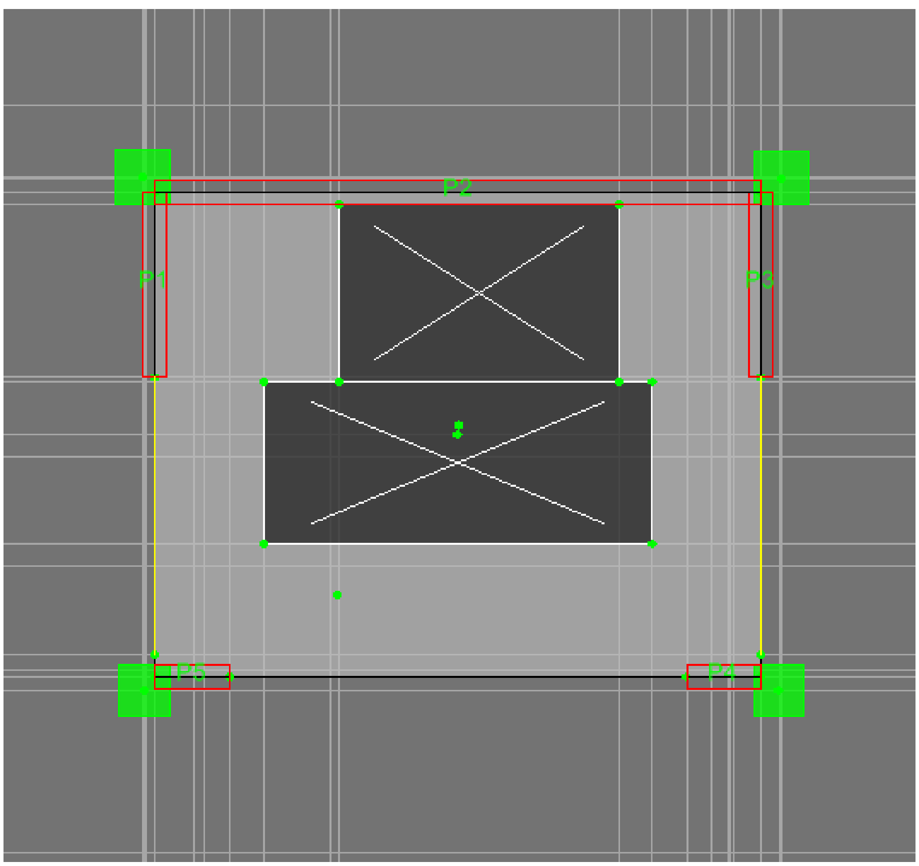
Pier Labels- Floor 1



Pier Labels- Floors 2 through 9



Pier Labels- Floor 10



Pier Labels- Floors 11 through 21

Lateral System Results- Combined System

Allowable Story Drifts based on ASCE7-02 sec. 9.5.2.8

Use Group	II	
Allowable Drift:	$.015h_{sx}$	L/67

Floor	Height (in.)	Allowable Drift (in)	Seismic X	Drift (in.)	OK?	Seismic Y	Drift (in.)	OK?
21	144.00	2.16	0.000678	0.097632	OK	0.00044	0.06336	OK
20	116.00	1.74	0.000715	0.08294	OK	0.000494	0.057304	OK
19	116.00	1.74	0.000751	0.087116	OK	0.000564	0.065424	OK
18	116.00	1.74	0.000783	0.090828	OK	0.000647	0.075052	OK
17	116.00	1.74	0.000779	0.090364	OK	0.000733	0.085028	OK
16	120.00	1.80	0.000885	0.1062	OK	0.000855	0.1026	OK
15	132.00	1.98	0.000996	0.131472	OK	0.000952	0.125664	OK
14	116.00	1.74	0.001044	0.121104	OK	0.001024	0.118784	OK
13	116.00	1.74	0.001043	0.120988	OK	0.001075	0.1247	OK
12	116.00	1.74	0.000976	0.113216	OK	0.001101	0.127716	OK
11	116.00	1.74	0.000754	0.087464	OK	0.001088	0.126208	OK
10	116.00	1.74	0.000527	0.061132	OK	0.00103	0.11948	OK
9	116.00	1.74	0.000514	0.059624	OK	0.001022	0.118552	OK
8	116.00	1.74	0.000473	0.054868	OK	0.000999	0.115884	OK
7	116.00	1.74	0.000429	0.049764	OK	0.000973	0.112868	OK
6	116.00	1.74	0.000381	0.044196	OK	0.000934	0.108344	OK
5	116.00	1.74	0.000329	0.038164	OK	0.000879	0.101964	OK
4	116.00	1.74	0.000287	0.033292	OK	0.000798	0.092568	OK
3	120.00	1.80	0.000244	0.02928	OK	0.000684	0.08208	OK
2	120.00	1.80	0.000178	0.02136	OK	0.000479	0.05748	OK
1	144.00	2.16	0.000118	0.016992	OK	0.000193	0.027792	OK

Table showing 110 Third Avenue meets seismic drift requirements

Wind Drift Check

Drift based on good judgement, not code		
Allowable Drift:	.0025h_{sx}	H/400

ASCE7-02 Loadings

			WINDX			WINDY			L/ Value	
Floor	Height (in.)	Allowable Drift (in)	Wind X	Drift (in.)	OK?	Wind Y	Drift (in.)	OK?	Wind X	Wind Y
21	144.00	0.36	0.000752	0.108288	OK	0.000417	0.060048	OK	1329.79	2398.08
20	116.00	0.29	0.000791	0.091756	OK	0.00047	0.05452	OK	1264.22	2127.66
19	116.00	0.29	0.000834	0.096744	OK	0.000542	0.062872	OK	1199.04	1845.02
18	116.00	0.29	0.000875	0.1015	OK	0.000631	0.073196	OK	1142.86	1584.79
17	116.00	0.29	0.000885	0.10266	OK	0.000725	0.0841	OK	1129.94	1379.31
16	120.00	0.30	0.001014	0.12168	OK	0.000858	0.10296	OK	986.19	1165.50
15	132.00	0.33	0.001168	0.154176	OK	0.000972	0.128304	OK	856.16	1028.81
14	116.00	0.29	0.001261	0.146276	OK	0.001065	0.12354	OK	793.02	938.97
13	116.00	0.29	0.001298	0.150568	OK	0.001141	0.132356	OK	770.42	876.42
12	116.00	0.29	0.001254	0.145464	OK	0.001196	0.138736	OK	797.45	836.12
11	116.00	0.29	0.001013	0.117508	OK	0.001211	0.140476	OK	987.17	825.76
10	116.00	0.29	0.000645	0.07482	OK	0.001179	0.136764	OK	1550.39	848.18
9	116.00	0.29	0.000625	0.0725	OK	0.001202	0.139432	OK	1600.00	831.95
8	116.00	0.29	0.000598	0.069368	OK	0.001211	0.140476	OK	1672.24	825.76
7	116.00	0.29	0.000579	0.067164	OK	0.001216	0.141056	OK	1727.12	822.37
6	116.00	0.29	0.000549	0.063684	OK	0.001204	0.139664	OK	1821.49	830.56
5	116.00	0.29	0.000507	0.058812	OK	0.00117	0.13572	OK	1972.39	854.70
4	116.00	0.29	0.000451	0.052316	OK	0.001096	0.127136	OK	2217.29	912.41
3	120.00	0.30	0.000384	0.04608	OK	0.000968	0.11616	OK	2604.17	1033.06
2	120.00	0.30	0.000291	0.03492	OK	0.000711	0.08532	OK	3436.43	1406.47
1	144.00	0.36	0.000147	0.021168	OK	0.0003	0.0432	OK	6802.72	3333.33

NYC Building Code Loadings

			NYCX			NYCY			L/ Value	
Floor	Height	Allowable Drift (in)	Wind X	Drift (in.)	OK?	Wind Y	Drift (in.)	OK?	Wind X	Wind Y
21	144.00	0.36	0.000412	0.059328	OK	0.000443	0.063792	OK	2427.18	2257.34
20	116.00	0.29	0.000435	0.05046	OK	0.000469	0.054404	OK	2298.85	2132.20
19	116.00	0.29	0.000461	0.053476	OK	0.000503	0.058348	OK	2169.20	1988.07
18	116.00	0.29	0.000487	0.056492	OK	0.000541	0.062756	OK	2053.39	1848.43
17	116.00	0.29	0.000495	0.05742	OK	0.000572	0.066352	OK	2020.20	1748.25
16	120.00	0.30	0.000568	0.06816	OK	0.000624	0.07488	OK	1760.56	1602.56
15	132.00	0.33	0.000652	0.086064	OK	0.000693	0.091476	OK	1533.74	1443.00
14	116.00	0.29	0.000701	0.081316	OK	0.000742	0.086072	OK	1426.53	1347.71
13	116.00	0.29	0.000718	0.083288	OK	0.000784	0.090944	OK	1392.76	1275.51
12	116.00	0.29	0.000689	0.079924	OK	0.000811	0.094076	OK	1451.38	1233.05
11	116.00	0.29	0.00055	0.0638	OK	0.00082	0.09512	OK	1818.18	1219.51
10	116.00	0.29	0.00036	0.04176	OK	0.000801	0.092916	OK	2777.78	1248.44
9	116.00	0.29	0.000354	0.041064	OK	0.000806	0.093496	OK	2824.86	1240.69
8	116.00	0.29	0.000329	0.038164	OK	0.000807	0.093612	OK	3039.51	1239.16
7	116.00	0.29	0.000302	0.035032	OK	0.000808	0.093728	OK	3311.26	1237.62
6	116.00	0.29	0.000281	0.032596	OK	0.0008	0.0928	OK	3558.72	1250.00
5	116.00	0.29	0.000256	0.029696	OK	0.000779	0.090364	OK	3906.25	1283.70
4	116.00	0.29	0.000223	0.025868	OK	0.000735	0.08526	OK	4484.30	1360.54
3	120.00	0.30	0.000181	0.02172	OK	0.000655	0.0786	OK	5524.86	1526.72
2	120.00	0.30	0.000136	0.01632	OK	0.000504	0.06048	OK	7352.94	1984.13
1	144.00	0.36	0.00012	0.01728	OK	0.000247	0.035568	OK	8333.33	4048.58

ASCE Case 3 Loadings

			ASCE Case 3			ASCE Case 3			L/ Value	
Floor	Height	Allowable Drift (in)	Wind X	Drift (in.)	OK?	Wind Y	Drift (in.)	OK?	Wind X	Wind Y
21	144.00	0.36	0.000942	0.135648	OK	0.000187	0.026928	OK	1061.57	5347.59
20	116.00	0.29	0.000989	0.114724	OK	0.000303	0.035148	OK	1011.12	3300.33
19	116.00	0.29	0.001039	0.120524	OK	0.00046	0.05336	OK	962.46	2173.91
18	116.00	0.29	0.001085	0.12586	OK	0.000646	0.074936	OK	921.66	1547.99
17	116.00	0.29	0.001088	0.126208	OK	0.00084	0.09744	OK	919.12	1190.48
16	120.00	0.30	0.001239	0.14868	OK	0.001136	0.13632	OK	807.10	880.28
15	132.00	0.33	0.001419	0.187308	OK	0.001423	0.187836	OK	704.72	702.74
14	116.00	0.29	0.001529	0.177364	OK	0.001646	0.190936	OK	654.02	607.53
13	116.00	0.29	0.001573	0.182468	OK	0.001795	0.20822	OK	635.73	557.10
12	116.00	0.29	0.001524	0.176784	OK	0.001843	0.213788	OK	656.17	542.59
11	116.00	0.29	0.001231	0.142796	OK	0.001657	0.192212	OK	812.35	603.50
10	116.00	0.29	0.000779	0.090364	OK	0.001309	0.151844	OK	1283.70	763.94
9	116.00	0.29	0.000752	0.087232	OK	0.001351	0.156716	OK	1329.79	740.19
8	116.00	0.29	0.000729	0.084564	OK	0.001371	0.159036	OK	1371.74	729.39
7	116.00	0.29	0.000708	0.082128	OK	0.001399	0.162284	OK	1412.43	714.80
6	116.00	0.29	0.000674	0.078184	OK	0.0014	0.1624	OK	1483.68	714.29
5	116.00	0.29	0.000626	0.072616	OK	0.001369	0.158804	OK	1597.44	730.46
4	116.00	0.29	0.000561	0.065076	OK	0.001283	0.148828	OK	1782.53	779.42
3	120.00	0.30	0.000487	0.05844	OK	0.001137	0.13644	OK	2053.39	879.51
2	120.00	0.30	0.00037	0.0444	OK	0.000799	0.09588	OK	2702.70	1251.56
1	144.00	0.36	0.000193	0.027792	OK	0.000237	0.034128	OK	5181.347	4219.409

Table showing 110 Third Avenue meets allowable drift requirements

	Load							
	Wind						Seismic	
	Wind X	Wind Y	NYC X	NYC Y	ASCE3X	ASCE3Y	Seismic X	Seismic Y
Floor	Drift (in.)	Drift (in.)	Drift (in.)	Drift (in.)	Drift (in.)	Drift (in.)	Drift (in.)	Drift (in.)
21	0.108288	0.060048	0.059328	0.063792	0.135648	0.026928	0.097632	0.06336
20	0.091756	0.05452	0.05046	0.054404	0.114724	0.035148	0.08294	0.057304
19	0.096744	0.062872	0.053476	0.058348	0.120524	0.05336	0.087116	0.065424
18	0.1015	0.073196	0.056492	0.062756	0.12586	0.074936	0.090828	0.075052
17	0.10266	0.0841	0.05742	0.066352	0.126208	0.09744	0.090364	0.085028
16	0.12168	0.10296	0.06816	0.07488	0.14868	0.13632	0.1062	0.1026
15	0.154176	0.128304	0.086064	0.091476	0.187308	0.187836	0.131472	0.125664
14	0.146276	0.12354	0.081316	0.086072	0.177364	0.190936	0.121104	0.118784
13	0.150568	0.132356	0.083288	0.090944	0.182468	0.20822	0.120988	0.1247
12	0.145464	0.138736	0.079924	0.094076	0.176784	0.213788	0.113216	0.127716
11	0.117508	0.140476	0.0638	0.09512	0.142796	0.192212	0.087464	0.126208
10	0.07482	0.136764	0.04176	0.092916	0.090364	0.151844	0.061132	0.11948
9	0.0725	0.139432	0.041064	0.093496	0.087232	0.156716	0.059624	0.118552
8	0.069368	0.140476	0.038164	0.093612	0.084564	0.159036	0.054868	0.115884
7	0.067164	0.141056	0.035032	0.093728	0.082128	0.162284	0.049764	0.112868
6	0.063684	0.139664	0.032596	0.0928	0.078184	0.1624	0.044196	0.108344
5	0.058812	0.13572	0.029696	0.090364	0.072616	0.158804	0.038164	0.101964
4	0.052316	0.127136	0.025868	0.08526	0.065076	0.148828	0.033292	0.092568
3	0.04608	0.11616	0.02172	0.0786	0.05844	0.13644	0.02928	0.08208
2	0.03492	0.08532	0.01632	0.06048	0.0444	0.09588	0.02136	0.05748
1	0.021168	0.0432	0.01728	0.035568	0.027792	0.034128	0.016992	0.027792
Total Drift	1.897452	2.306036	1.039228	1.655044	2.32916	2.783484	1.537996	2.0088519

*Assume story drifts can be added due to the rigid diaphragm

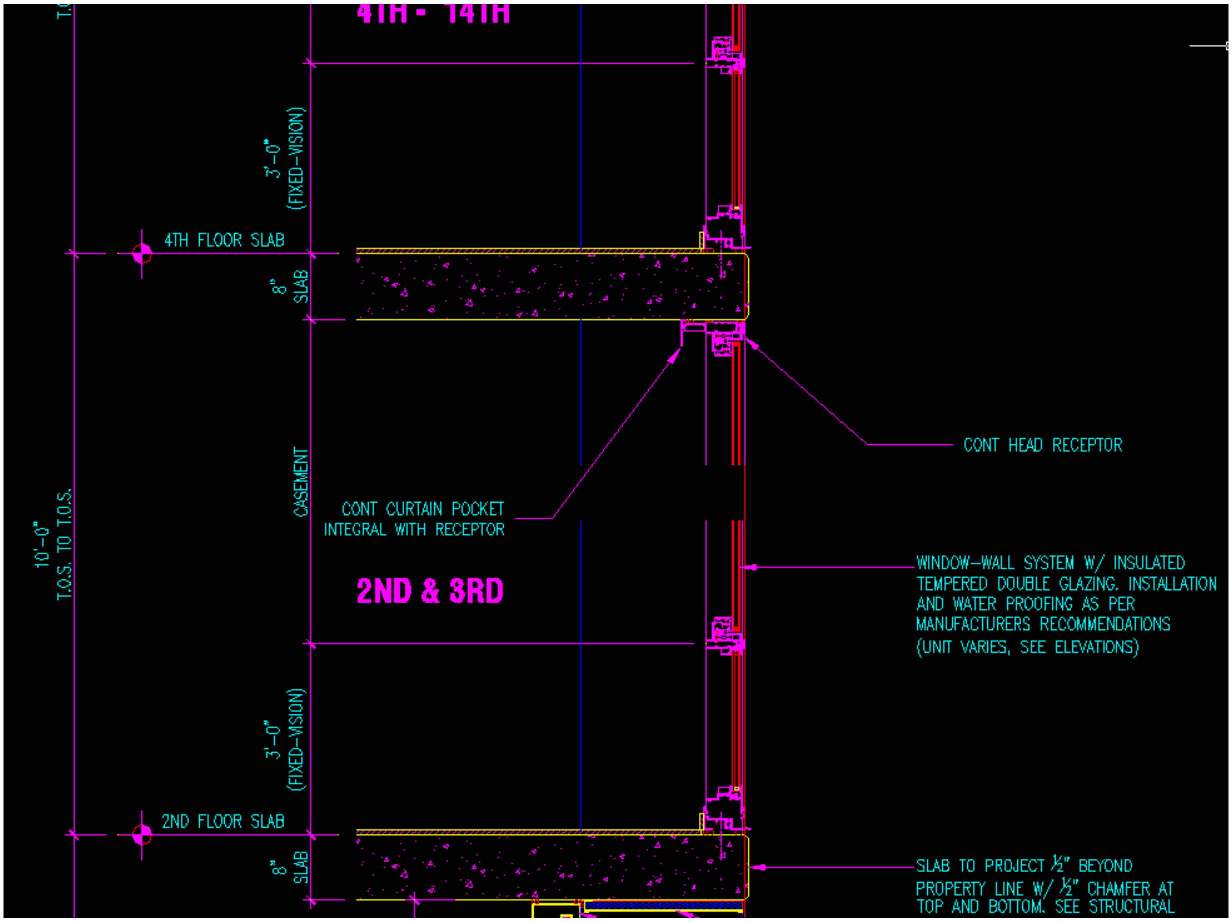
As evidenced from the preceding tables, the final lateral system design acts more effectively than the first and conforms to the drift limitations set forth in this report. Therefore it is safe to conclude that a combined lateral system consisting of shear walls and columns and slabs acting as a moment frame is the best choice for 110 Third Avenue.

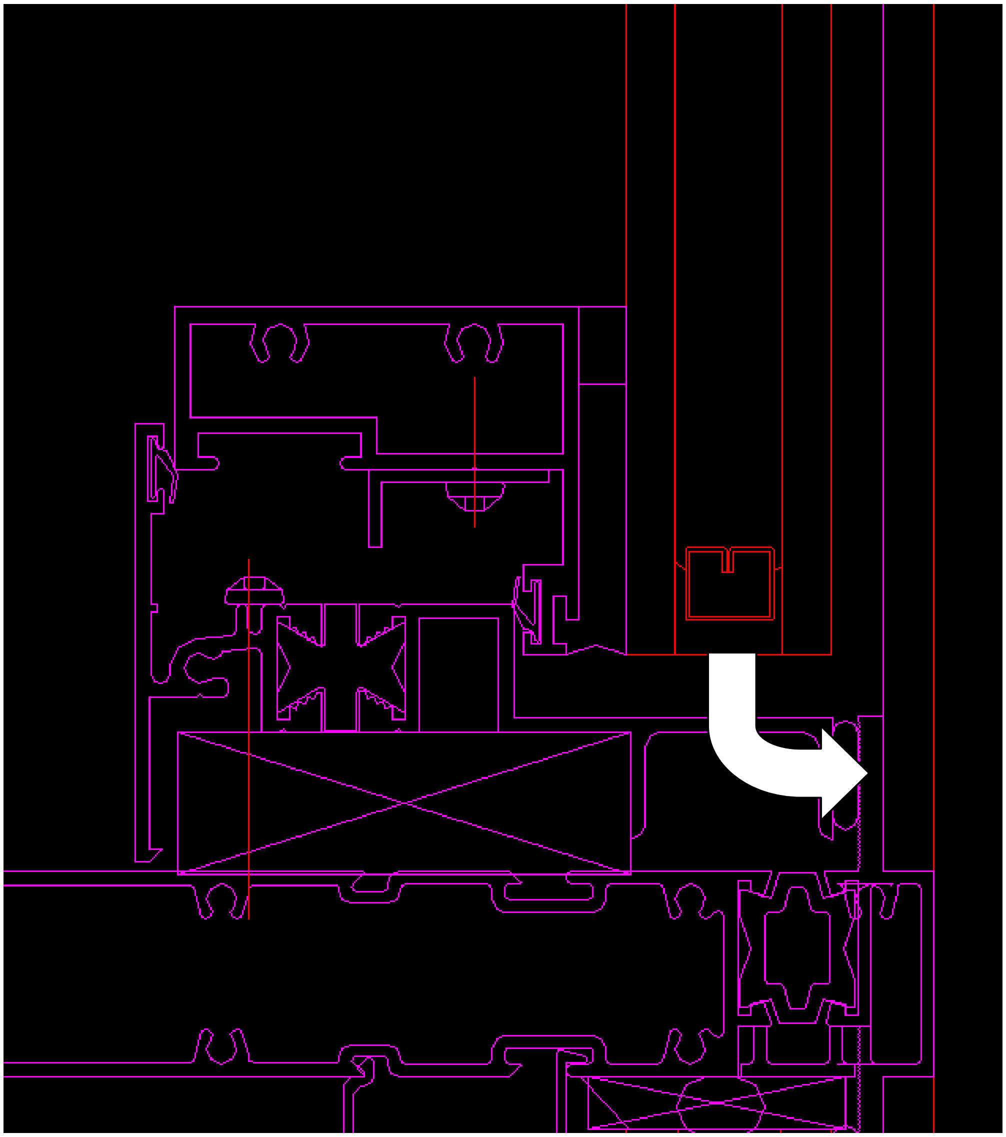
Breadth Topic 1: Building Envelope Design

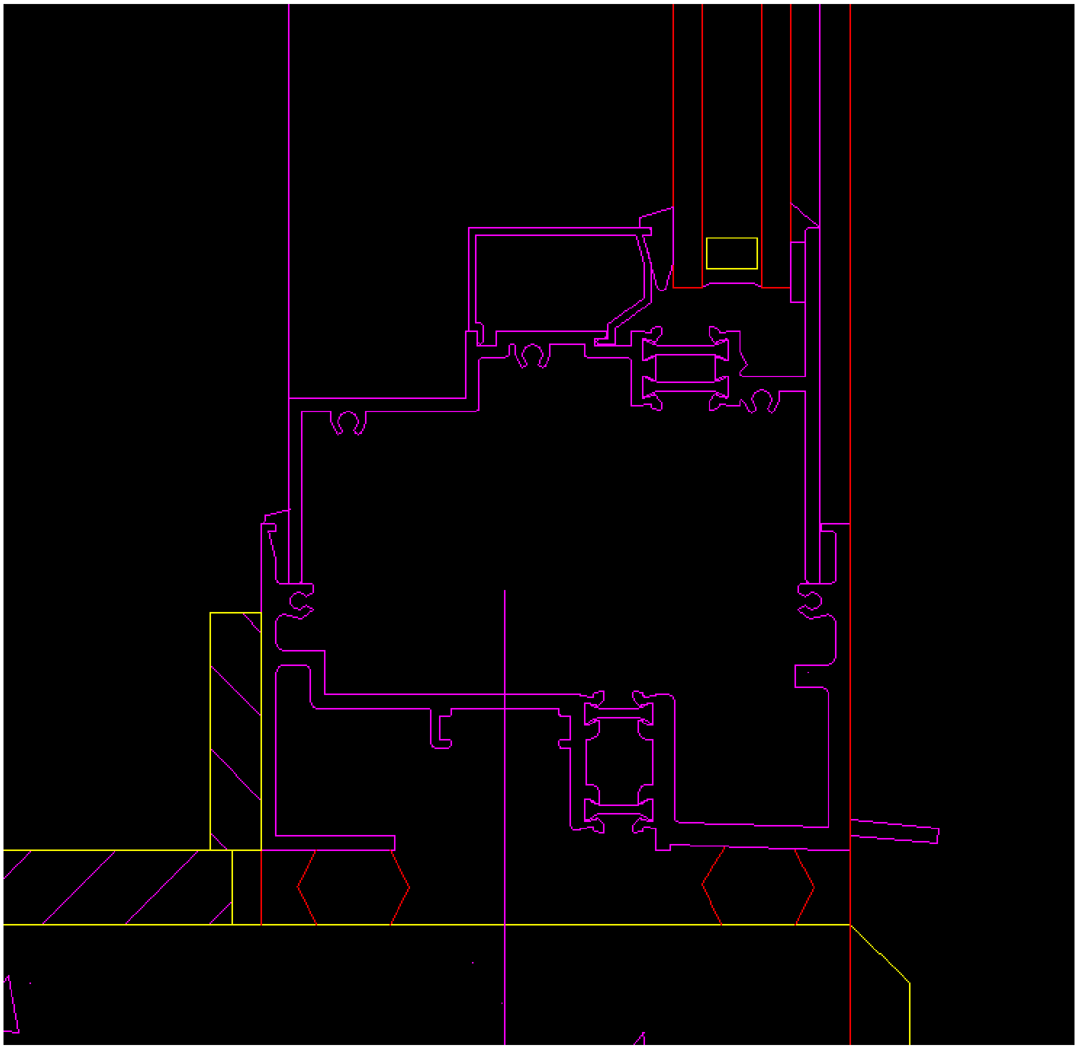
All systems of a building must function together to create an inhabitable whole. Most important to the building's operation is the interface between systems, including the building envelope. The building envelope is where the building comes together, where two or more systems meet at all times. As a consequence, the ability of the building to function efficiently and as intended often greatly influences the performance of the building. In conjunction with Simpson Gumpertz and Heger, this section evaluates the building envelope design of 110 Third Avenue. SGH provided valuable information regarding problems that may arise such as water intrusion and gave alternatives to prevent such problems from ever arising.

The window-wall system that serves as the majority of the façade for 110 Third Avenue could have several water penetration issues develop over the course of the building's life. The window wall system is a barrier system that should repel water and not allow any penetration whatsoever. If water were to penetrate the system, there exists no way for the water to exit except through the interior. The wall sections provided by the architect do not show some critical transition points between materials. Most of the time these details are omitted simply because the contractor who will eventually install the system already knows the typical type of system that should be put in place. Sometimes, however, the contractor cannot anticipate certain issues that may arise in the particular system, and therefore a proper detail should be prepared.

In the case of 110 Third Avenue, the wall system provides no way for the glazing pocket to drain. Operable windows need a way to drain, so there should be some way for them to drain along the sill track. The sill track should have weep holes that can give the water a way to escape the system without the tendency to penetrate through to the inside.


4TH - 14TH
3'-0" (FIXED-VISION)
4TH FLOOR SLAB
8" SLAB
CASEMENT
CONT CURTAIN POCKET INTEGRAL WITH RECEPTOR
CONT HEAD RECEPTOR
10'-0" T.O.S. TO T.O.S.
2ND & 3RD
WINDOW-WALL SYSTEM W/ INSULATED TEMPERED DOUBLE GLAZING. INSTALLATION AND WATER PROOFING AS PER MANUFACTURERS RECOMMENDATIONS (UNIT VARIES, SEE ELEVATIONS)
3'-0" (FIXED-VISION)
2ND FLOOR SLAB
8" SLAB
SLAB TO PROJECT ½" BEYOND PROPERTY LINE W/ ½" CHAMFER AT TOP AND BOTTOM. SEE STRUCTURAL


Operable Window



Base of Wall (Typ.)

In addition, the gaskets should have fully vulcanized corners, because the gaskets tend to shrink over time. If no provision is made for drainage, they will eventually leak.

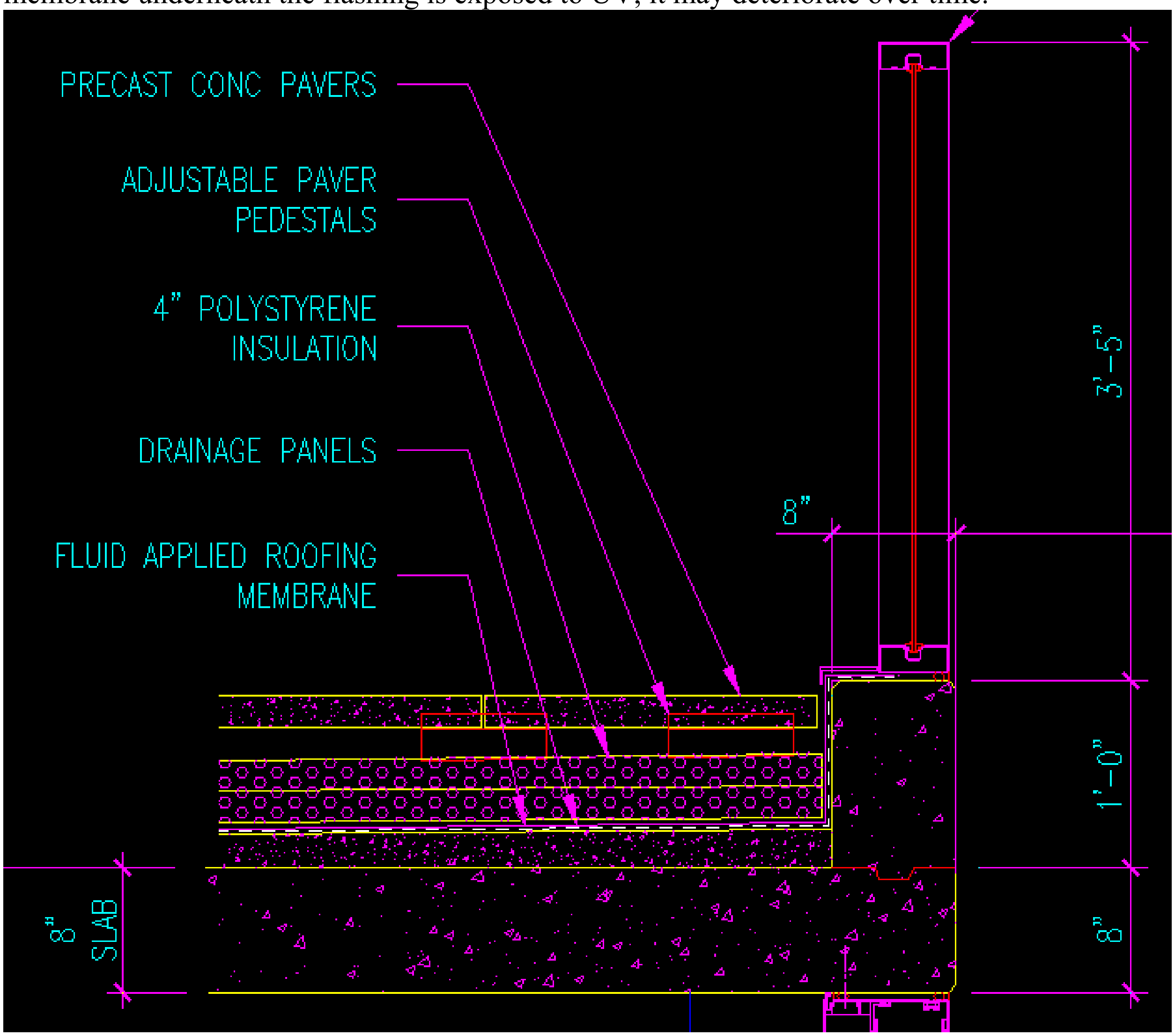
Wherever concrete is exposed, which is every floor in between the window wall system and along edges, clear sealer should be applied to prevent the concrete from absorbing water.

The contractor should install a double sealant joint where the window system ties into concrete at the head and the jamb. The existing system only has a single sealant joint, which, if it fails, would allow water to penetrate the system. The second joint that is visible on the drawings is simply an air tight joint to prevent air intrusion. Sealant joints and clear sealer are maintenance items. They must be checked on a regular basis, and in order to support this regular maintenance, a swing stage should be installed on the top of building to allow for regular check-ups. Clear sealers have a five to ten year lifespan, and must be replaced in order to prevent deterioration and subsequent absorption of water by the concrete.

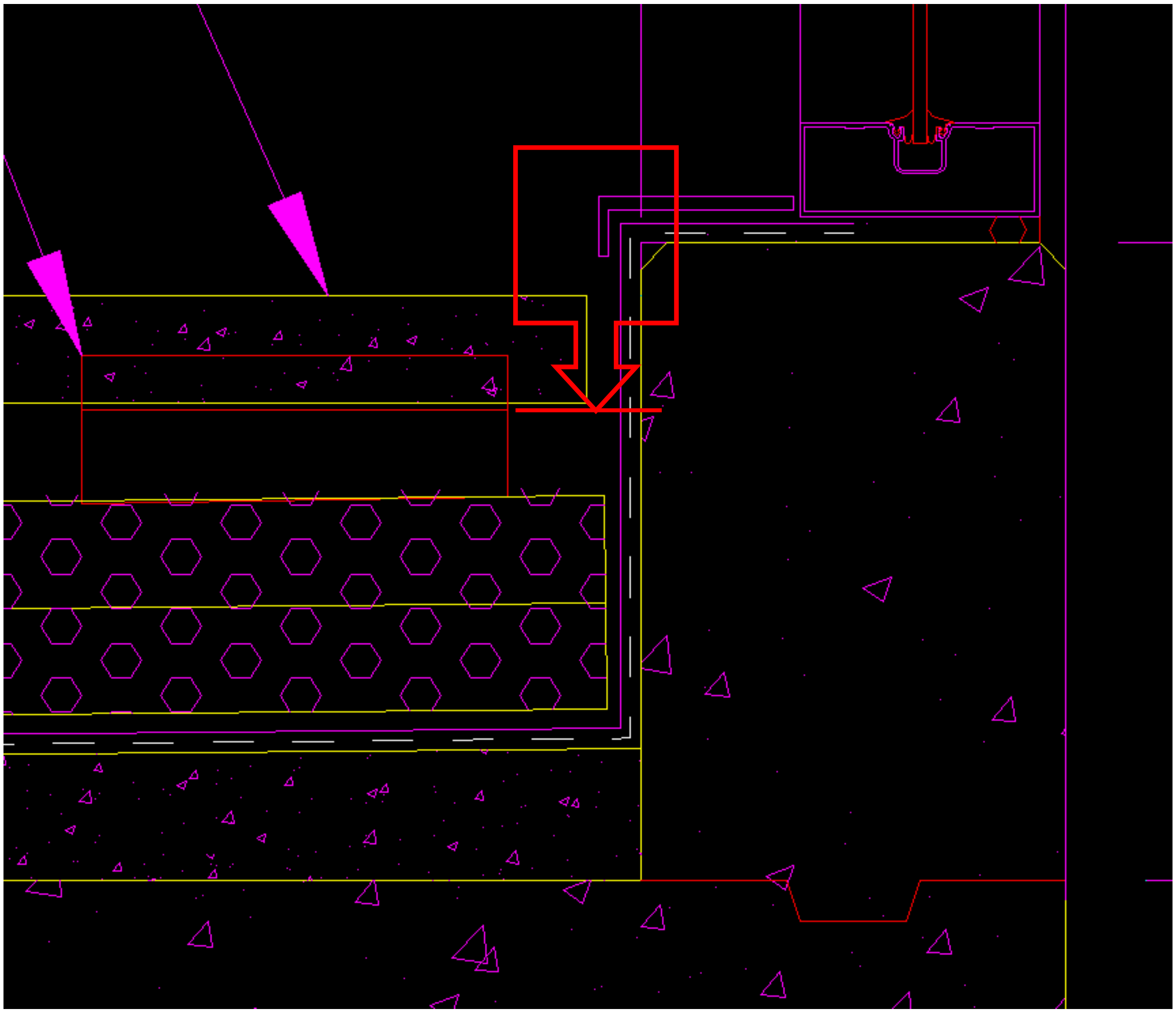
Where the concrete floor slab meets the one foot parapet at the roofline, the cold joint that exists between these two elements is not watertight. The addition of a waterstop and waterstop slurry would help prevent the intrusion of moisture through the concrete and, eventually, into the interior of the building. Another option to address this issue is to cast the entire piece monolithically. Also, if both elements are cast separately, vertical control joints should be put along the parapet panel to prevent cracking.

Roof

The roof drainage system in 110 Third Avenue seems to be pretty sufficient to drain water and prevent any issues from forming with regard to leakage. However, the addition of a protection board underneath the drainage panels and above the roofing materials would help make the system even better. Also, the sheet metal that serves as flashing at the ends of the parapet should continue all the way down past the pedestal. If the membrane underneath the flashing is exposed to UV, it may deteriorate over time.



Roof Section



Extend sheet metal flashing past adjustable paver pedestals

Breadth 2: Cost and Constructability of Post-Tensioned Slabs

Possibly the most important part of any structural design, the cost of systems must be compared to each other to determine which one is a better option for contractors and developers to construct in a given area. Below is the estimated cost per floor using a CIP post-tensioned floor slab.

Estimate

Concrete Costs

Materials:	100 per cu. yds	x	122.5 cu. yds	=	12250
Labor:	50 per cu. yds	x	122.5 cu. yds	=	6125
Total:	150 per cu. yds	x	122.5 cu. yds	=	18380

Post-Tensioning Costs

Materials:	1 per pounds	x	3734 pounds	=	3734
Labor:	0.5 per pounds	x	3734 pounds	=	1867
Total:	1.5 per pounds	x	3734 pounds	=	5601

Formwork Costs

Materials:	1 per sq. ft.	x	4961 sq. ft.	=	4961
Labor:	1 per sq. ft.	x	4961 sq. ft.	=	4961
Total:	2 per sq. ft.	x	4961 sq. ft.	=	9923

Mild Steel Reinforcing Costs

Materials:	1000 per tons	x	4.129 tons	=	4129
Labor:	500 per tons	x	4.129 tons	=	2065
Total:	1500 per tons	x	4.129 tons	=	6194

Total Costs

Materials:	5.054 per sq. ft.	x	4961 sq. ft.	=	25070
Labor:	3.027 per sq. ft.	x	4961 sq. ft.	=	15020
Total:	8.081 per sq. ft.	x	4961 sq. ft.	=	40090

Presented next is the cost data for a regularly reinforced slab system as detailed on the original plans for 110 Third Avenue. The estimate was performed using ICE 2000 software.

Estimate Detail - Standard Construction Project

Detail - Without Taxes and Insurance

Estimator :
Project Size : sqft

ItemCode	Description	Quantity	UM	Lab.Unit	Mat.Unit	Eqp.Unit	Sub.Unit	Eqp.Rent.Unit	Temp.Mat.Unit	Other Unit	Tot.UnitCost	TotalCost
03111.612	SLAB FORM W/2.6 BM/SF	5,100.00	SQFT	2.5380	1.263						3.801	19,385.61
03111.624	SLAB EDGE FORM	190.67	SQFT	2.1511	0.853						3.004	572.72
03150.650	SCREEDS FOR SLAB	612.00	LNFT	0.9219	0.320						1.242	760.04
03150.900	FORM RELEASING AGENT	5,290.67	SQFT	0.2095	0.023						0.233	1,230.08
03210.130	SUPPORTED SLAB REBAR	46.33	CWT	32.3636	26.750						59.114	2,738.94
03311.500	**CONC IN SUPPORTED SLAB**		****									
03311.532	5000 PSI W/CRANE	125.93	CUYD	13.9420	60.000						73.942	9,311.21
03315.986	* SUPPORTED SLAB AREA *	5,100.00	SQFT									
03350.130	MACHINE TROWEL FINISH	5,100.00	SQFT	0.3304							0.330	1,685.04
03350.131	POINT & PATCH	5,290.67	SQFT	0.1102	0.013						0.123	650.75
03390.010	PROTECT & CURE	5,100.00	SQFT	0.1102	0.019						0.129	659.94
	Total Estimate											**$36,994**

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The overall cost per floor for a regularly reinforced system is $36,994 while the cost per floor for a post-tensioned system is approximately 8% higher at $40,090. The cost differential between floors, when added up over the entire height of the building can be significant. Post-tensioning tends to be a more costly system, and is therefore not necessarily a better option.

Raw cost data is only a part of how one system compares to another. Availability of contractors to perform the work in a timely fashion also highly influences what type of construction is prevalent in a given area. For example, post tensioning is much more common in Washington D.C. than in New York City because there are many more contractors who can perform the work in the areas surrounding Washington. Several professionals have mentioned that the reason not much post-tensioning work is done in New York is simply because the nearest contractors are in New Jersey. Unionized iron workers prevent other trained laborers experienced in post-tensioning from gaining work in the city thus making it hard for contractors to find laborers nearby with the proper experience for PT.

The constructability of a CIP post-tensioned slab in New York City is similar to any other city. The same equipment applies to a regularly reinforced CIP slab that applies to a post-tensioned slab. The main difference is in the crews that construct the reinforcing. As mentioned previously, a special crew and special jacking equipment is needed for laying the post-tensioning in the slab. This special need puts PT at a disadvantage for the New York metropolitan area.

Upon traveling to New York City to examine the site firsthand to scout any potential issues that may arise during construction relating to the installation of post-tensioning, it was clear to see that the regular reinforcing and PT differed little in terms of needs from the surrounding area. 110 Third Avenue is an open site with easy street access. At the time of the visit, contractors were approximately three months behind schedule. Toll Brothers did not disclose the reasons for the delay, but provided a good outlook for scheduling in the next few months. At the time of presenting this report, the foundations should be reaching completion. Unfortunately, because of the outstanding delay in construction, the objectives of the trip to the site were not wholly met. None of the building systems had been completed so as to examine the construction process, but upon speaking with professionals it was clear the PT, if it were common in NYC, would not complicate construction at all. Included below are some sample pictures from the site visit showing the delay to the project and its status in early February.

The real advantages and disadvantages between a regularly reinforced floor slab and a post-tensioned one lie in how they influence other systems and construction. If the PT system were desired because it improves the architecture of the building and living conditions of tenants, then the added cost is only marginal. However, if construction costs, availability of labor, and erection time are the most significant factors to the owners (which is probably the case), then a regularly reinforced slab may be the best option.

Summary

Designing 110 Third Avenue started with the simple principle of finding a comparable structural system to the one already in place and possibly providing the advantage of reduced cost and architectural benefits. To do this, the column layout was rearranged. Over 50% of preexisting columns were removed and the remaining columns were upsized in order to allow for more inhabitable space throughout each floor. The result was a more open layout that can be adjusted by architects to suit the increased space available. There are, however, a few areas where columns impact the floor plan in a negative way. The four columns located around the shear walls inhibit traffic through the common hallways. This problem can easily be solved by offsetting the columns further into the core if architects were to determine the existing hallways couldn't be shifted slightly since new space has opened in the living quarters. The new columns range from 11" x 11" to 30" x 19" in cross sectional area and are cast monolithically with the slab. With the assistance of PCACOL, columns reinforcement was designed to meet the needs of axial loading as well as lateral loads.

After designing the columns, a floor slab model was developed using RAM Conce*pt*. The floor slab is 8" thick, maintaining its thickness from the original designs. Post-tensioning is used throughout the floor with banding around the columns. The floor meets all deflection criteria. The floor slab, as well as the columns and shear walls, are 5000 psi concrete.

The lateral system developed was originally a shear-wall-only system that assumed leaning columns throughout the building. When models produced excessive story drifts, a revised model was constructed assuming a combined system featuring shear walls and moment frames created by the slabs and columns. The combined system reduced drifts to well below the required amount, but as a result columns had to be redesigned to handle the increased moment placed on them. After examining the columns and the moments on them in Etabs, they were reevaluated in PCACOL for new reinforcement plans. All of the columns could stay the same size, according to Etabs, but PCACOL produced an analysis that said columns 7 and 8 had to be upsized by an inch. To err on the side of caution, these columns have been upsized in the final model.

Also evaluated was the building envelope system. Designers often leave out necessary details because the contractor who will eventually install the system already knows the typical type of system that should be put in place. Sometimes, however, the contractor cannot anticipate certain issues that may arise during construction, and if a barrier system malfunctions the results could be disastrous.

Finally, an analysis was performed evaluating the costs of each floor system. A post-tensioned floor system was about 8% more expensive than its regularly reinforced counterpart, but note that formwork and labor costs will be saved using the new column layout. Also, the low availability of PT contractors in New York City adversely affects the price and ability to erect a PT floor system.

Conclusions

The new design for 110 Third Avenue functions well within the bounds of code requirements and can be considered a valid design for the future. The column layout has a definite advantage over the previous layout because it frees living space for use. Granted, some slight rearrangements of rooms and hallways would be necessary to accommodate the upsized columns, but no significant barriers stand in the way of utilizing the new system. The new column system will save money in terms of labor, because there are fewer columns. There will also be less formwork because of reduced surface area.

The post-tensioned cast-in-place slab also serves as a good alternative to the existing flat plate system. However, an increased cost comes with the new system, and contractors in the New York metropolitan area do not usually perform PT construction. The cost differential between floor systems is minor and will be offset by savings in the columns. Unfortunately, unionized iron workers preventing the infusion of laborers skilled in post-tensioning will make it expensive to construct the system in New York. If the building were not in New York City, it might very well be a viable alternative if not an advantageous system to build.

After examining alternatives for the lateral system, a combined system incorporating columns and slabs into a moment frame in addition to shear walls is definitely the best system. Using only shear walls may be a simpler analysis, but in no way compares to the advantages received by utilizing the columns and floor slabs. Drifts would be excessive due to the centrally located shear walls, so it's no wonder that using perimeter columns greatly increases the stiffness of the building.

Also, upgrading the window-wall and roof systems will ensure no problems will occur with the barrier system previously planned to be installed in 110 Third Avenue. These minor additions are not costly and will prevent costly damages in the future.

As a whole, the design produced and detailed in this report will function well as a new 110 Third Avenue. It can be concluded that both the preexisting building and the new one with new columns, new slab, and new lateral system are comparable to each other. In the end, the old floor slab system may have an advantage over post-tensioning because of cost and availability issues.

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I would sincerely like to thank everyone who helped contribute to the development of my thesis throughout the months. Without their help, none of my research would have been possible. It's through the generosity of contributors, professional and personal, who give young students the ability to succeed and rise to the challenge that's been set before us.

I'd like to thank Nathan Shuman formerly of Axis Design Group based in New York City for providing the plans and specifications for 110 Third Avenue. Without your assistance, I wouldn't even have a thesis building. Thank you for answering my continued questions throughout the semester and never delaying in your response.

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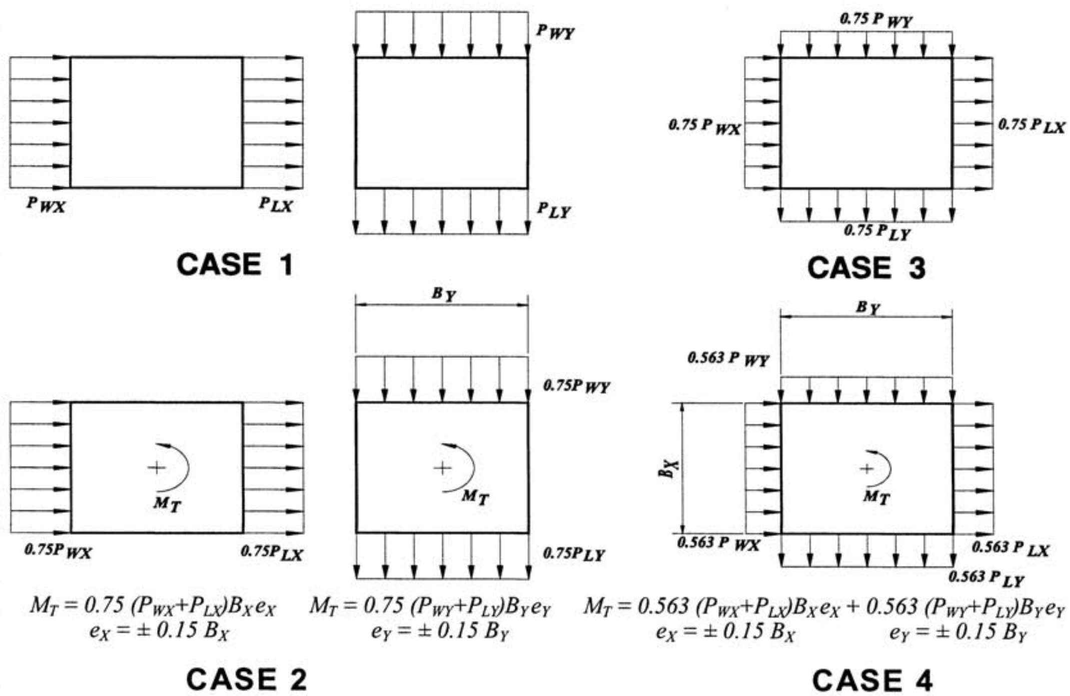
Appendix A: ASCE7-02 References

Main Wind Force Resisting System– Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y): Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

6.5.12.3 Design Wind Load Cases. The main wind force-resisting system of buildings of all heights, whose wind loads have been determined under the provisions of Sections 6.5.12.2.1 and 6.5.12.2.3, shall be designed for the wind load cases as defined in Figure 6-9. The eccentricity e for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (e_X, e_Y). The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (e_X, e_Y):

$e = e_Q + 1.7 I_Z (g_Q Q e_Q)^2 + (g_R R e_R)^2$

$1.7 I_Z (g_Q Q)^2 + (g_R R)^2$ **(Eq. 6-21)**

where

e_Q = eccentricity e as determined for rigid structures in Figure 6-9

e_R = distance between the elastic shear center and center of mass of each floor

I_Z, g_Q, Q, g_R, R shall be as defined in Section 6.5.8

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

Exception: One-story buildings with h less than or equal to 30 ft, buildings two stories or less framed with light-framed construction and buildings two stories or less designed with flexible diaphragms need only be designed for Load Case 1 and Load Case 3 in Figure 6-9.

TABLE 9.5.2.8
ALLOWABLE STORY DRIFT, Δ_a[a]

Structure	Seismic Use Group		
	I	II	III
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 9.5.2.8 is not waived.
[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

SECTION B.1
DEFLECTION, VIBRATION, AND DRIFT

B.1.1 Vertical Deflections. Deformations of floor and roof members and systems due to service loads shall not impair the serviceability of the structure.

B.1.2 Drift of Walls and Frames. Lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems due to wind effects shall not impair the serviceability of the structure.

B.1.3 Vibrations. Floor systems supporting large open areas free of partitions or other sources of damping, where vibration due to pedestrian traffic might be objectionable, shall be designed with due regard for such vibration.

Mechanical equipment that can produce objectionable vibrations in any portion of an inhabited structure shall be isolated to minimize the transmission of such vibrations to the structure.

Building structural systems shall be designed so that wind-induced vibrations do not cause occupant discomfort or damage to the building, its appurtenances, or its contents.

Appendix B: Column Design

Column Design   Floors 1-16

Column ID   Axial Loading

1. $254.87\ ft^2\ (1.2\ (125\ psf) + 1.6\ (40\ psf))$
$= 54.5\ k$

2. $281.06\ ft^2\ (1.2(125\ psf) + 1.6\ (40\ psf))$
$= 60.15\ k$

3. $282.54\ ft^2\ (1.2(125\ psf) + 1.6(40\ psf)) = 282.54\ (214)$
$= 60.46\ k$

4. $263.74\ (214\ psf)$
$= 56.44\ k$

5. $(551.85\ ft^2 - 135.7\ ft^2)(214\ psf) + (135.7\ ft^2)(1.6\ (100\ psf))$
$= 110.77\ k$

6. $(542.57\ ft^2 - 135.7\ ft^2)(214\ psf) + 135.7\ ft^2\ (160\ psf)$
$= 108.78\ k$

7. $371.36\ ft^2\ (214\ psf)$
$= 79.47\ k$

8. see spreadsheet for balcony loading
$(384.51\ ft^2)(214\ psf) + \cancel{1/3\ (5' - 22' - 1\tfrac{1}{2}'')(100(1.2) + 100\ (1.6))}$
$= 82.29\ k + \cancel{10.325 = 92.62\ k}$

9. $(526.33\ ft^2 - 129.3\ ft^2)(214\ psf) + (129.3\ ft^2)(160\ psf)$
$= 105.65\ k$

10. $(517.12\ ft^2 - 129.3\ ft^2)(214\ psf) + (129.3\ ft^2)(160\ psf)$
$= 103.68\ k$

11. $(252.95\ ft^2)(214\ psf)$
$= 54.13\ k$

12. $(294.62\ ft^2)\ (214\ psf)$
$= 63.05\ k$

13. $(294.41\ ft^2)(214\ psf)$
$= 63.00\ k$

14. Balcony - see spreadsheet
$(248.11\ ft^2)(214\ psf) + \cancel{2/3\ (110.625\ ft^2)\ (280\ psf)}$
$= 53.10\ k$

Colum Design Floors 17-21

Column ID	Axial Loading
1	54.5 k
2	60.15 k
3	60.46 k
4	56.44 k
5	110.77 k
6	108.78 k
7	$306.92\ ft^2\ (1.2(125\ psf) + 1.6(40\ psf))$ = 65.68 k
8	$384.51\ ft^2\ (214\ psf)$ = 82.29 k
9	$(392.10\ ft^2 - 129.29\ ft^2)(214\ psf) + (129.3\ ft^2)(160\ psf)$ = 76.93 k
10	$(392.80\ ft^2 - 129.89\ ft^2)(214\ psf) + (129.3)(160)$ = 77.08 k
11	$208.57\ ft^2(214\ psf)$ = 44.63 k
12	$(176.45\ ft^2)(214\ psf)$ = 37.76 k
13	$175.52\ ft^2\ (214\ psf)$ = 37.56 k
14	$(182.71\ ft^2)(214\ psf)$ = 39.1 k

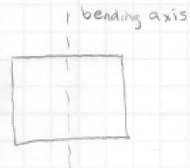
Column Design   Roof

Column ID	Axial Loading
1	254.87 ft² (1.2 (100 psf) + 1.6 (100 psf)) = 71.36 k
2	281.06 ft² (280) = 78.69 k
3	282.54 ft² (280) = 79.11 k
4	263.74 (280 psf) = 73.85 k
5	551.85 (280 psf) = 154.52 k
6	542.57 (280 psf) = 151.92 k
7	371.36 ft² (280 psf) = 103.98 k
8	(384.51 ft²)(280 psf) = 107.66 k
9	(526.33)(280 psf) = 147.37 k
10	(517.12 ft²)(280 psf) = 144.79 k
11	(252.95 ft²)(280 psf) = 70.83 k
12	(294.62 ft²)(280 psf) 82.50 k
13	(294.41 ft²)(280 psf) 82.43 k
14	(248.11 ft²)(280 psf) 69.47 k

1

Column Design Check

Objective: Design column (5) for floors 1-15



Axial loading = 2457.00 k
Moment = 213.07 'k

$f'_c = 5000$ psi , $f_y = 60$ ksi

$e = \frac{M_u}{P_u} = \frac{213.07(12)}{2417.25} = 1.06$

if $h = 24$, $\gamma = \frac{24-5}{24} = .792$, $\frac{e}{h} = \frac{1.06}{24} = .044$

if $h = 26$, $\gamma = \frac{26-5}{26} = .808$, $\frac{e}{h} = \frac{1.06}{26} = .041$

if $h = 28$, $\gamma = \frac{28-5}{28} = .821$, $\frac{e}{h} = \frac{1.06}{28} = .038$

- size column : assume $\gamma \approx .8$, $\rho = .03$, $\frac{e}{h} \approx .04$

From chart A.7 of Design of concrete Structures , pg. 758 ,

$K_n = 1.02$, *assume chart is applicable to $f'_c = 5$ ksi

$A_g = \frac{P_u}{\Phi f'_c K_n} = \frac{2457}{.65(5)(1.02)} = 741.18 \text{ in}^2$

$h \approx \sqrt{741 \text{ in}^2} = 27.22 \text{ in} \rightarrow 27 \text{ in}^2$

Try 27 x 27 in column

- Determine steel

$K_n = \frac{2457}{.65(5)(27^2)} = 1.037 = 1.04$

$R_n = \frac{P_u e}{\Phi f'_c A_g h} = \frac{213.07(12)}{.65(5)(27^3)} = .040$

$\gamma = .8$, $\xrightarrow{\text{Chart A.7}}$ $\rho = .032$

Try 27 x 27 column

$\gamma = \frac{27-5}{27} = .815$

ρ will still be approximately .032

$A_{s_{min}} = \rho A_g = .032(27^2) = 23.33 \text{ in}^2$

$\frac{23.33 \text{ in}^2}{16 \text{ bars}} = 1.458 \text{ in}^2/\text{bar} \rightarrow$ #11 bar $A = 1.56$

2 Column design check (5)

Check % of steel : $\frac{1.56(16)}{27 \times 27} = .034 < .04$ OK

$\gamma = \frac{h - 5.5}{h} = \frac{27 - 5.5}{27} = .796$ (for #11 bar)

$\rho = .032$ ∴ OK to use #11 bars

$h_{min} = b_{min} = 2(1.5) + 2(3/8) + 5(11/8) + 1.5(4 \text{ spaces})(11/8) = 18.9'' < 27''$ OK
(cover, stirrup, diam.)

* Try smaller bar size :

$A_{s_{min}} = 23.33\ in^2$

$\frac{23.33\ in^2}{20\ bars} = 1.17\ in^2 \Rightarrow$ #10 $(A_s = 1.27)$

Check % of steel : $\frac{1.27(20)}{27^2} = .035 < .04$ OK

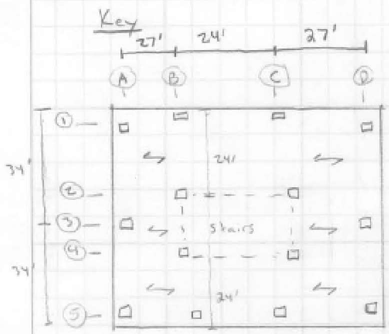
$h_{min} = b_{min} = 2(1.5) + 2(3/8) + 6(10/8) + 1.5(5 \text{ spaces})(10/8) = 20.63'' < 27''$ OK

$\boxed{\text{use } 27 \times 27 \text{ column w/ } 6 - \#10 \text{ in each face}}$

Design differs slightly from PCACOL design (outputs 28x28) probably due to the use of a design aid with $f'_c = 4$ ksi for this hand check. More axial strength in the column would require less reinforcement, which is consistent with these results.

1 Column Moments

Key



Span lengths are conservative (longer than actual)

DL = 100 psf self weight (assume 8" slab)

DL = 25 psf Apt.

LL = 40 psf Apt.

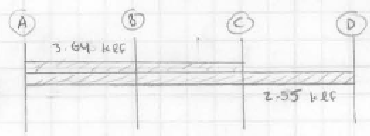
LL = 100 psf stairs

Note: Assume 34' wide bays to be conservative despite offset gridlines (2) and (4)

Along (1):

$w_D = 1.2(17')(125\text{ psf}) = 2550\text{ plf} = 2.55\text{ klf}$

$w_{L+D} = 2.55\text{ klf} + 1.6(40\text{ psf})(17') = 3.64\text{ klf}$



For worst negative moment at columns

$FEM_{AB} = \frac{3.64(27')^2}{12} = 221.13\ 'k$

$FEM_{BC} = \frac{3.64(24)^2}{12} = 174.72\ 'k$

$FEM_{CD} = \frac{2.55(27)^2}{12} = 154.91\ 'k$

$FEM = \frac{wl^2}{12}$

cast monolithically

- Estimate column moments

Assume 50% of ΔFEM goes into column above and below joint

(A) $\Delta FEM = 221.13\ 'k$

$\cdot 50\% = 110.57\ 'k$ into col. above and below

221.13 'k total column moment (for Floor 2-16)

(B) $\Delta FEM = 221.13 - 174.72 = 46.41\ 'k$ total into column moment

(C) $\Delta FEM = 174.72 - 154.91 = 19.81\ 'k$ total

(D) $\Delta FEM = 154.91\ 'k$ total

2

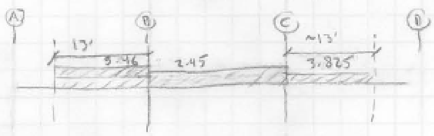
Along (2):

$w_D = 1.2\left(17' + \frac{34'}{4}\right)(125\ psf) = 3.825\ klf$

$w_{L+D}\ (Apt.) = 1.6\left(17' + \frac{34'}{4}\right)(40\ psf) + 3.825 = 5.46\ klf$

$w_L\ (Stair) = (1.6)\left(\frac{34}{4}\right)(100\ psf) + 1.6(17')(40\ psf) = 2.45\ klf$

w_D (Stair region) $= 1.2\,(17)(125) = 2.55\ klf$



Assume 13' cantilever to be conservative

$FEM_{BA} = 50\% \cdot \frac{w\ell^2}{12} = .5\left(\frac{5.46\ klf\,(13')^2}{12}\right) = 38.45\ 'k$

$FEM_{BC} = \frac{w\ell^2}{12} = \frac{(2.45 + 2.55)(24)^2}{12} = 240.00\ 'k$

$FEM_{CD} = 50\% \cdot \frac{w\ell^2}{12} = .5\left(\frac{3.825\,(13^2)}{12}\right) = 26.93\ 'k$

- Estimate Column moments

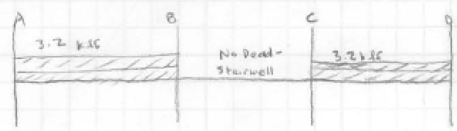
(B) $\Delta FEM = 240 - 38.45 = 201.55\ 'k$

(C) $\Delta FEM = 240 - 26.93 = \boxed{213.07\ 'k}$ max. moment @ (B) and (C)

Along (3):

$w_D = 1.2\,(34')(125\ psf) = 5.1\ klf$

$w_{L+D} = 5.1 + (1.6)(34')(40\ psf) = 7.28\ klf$

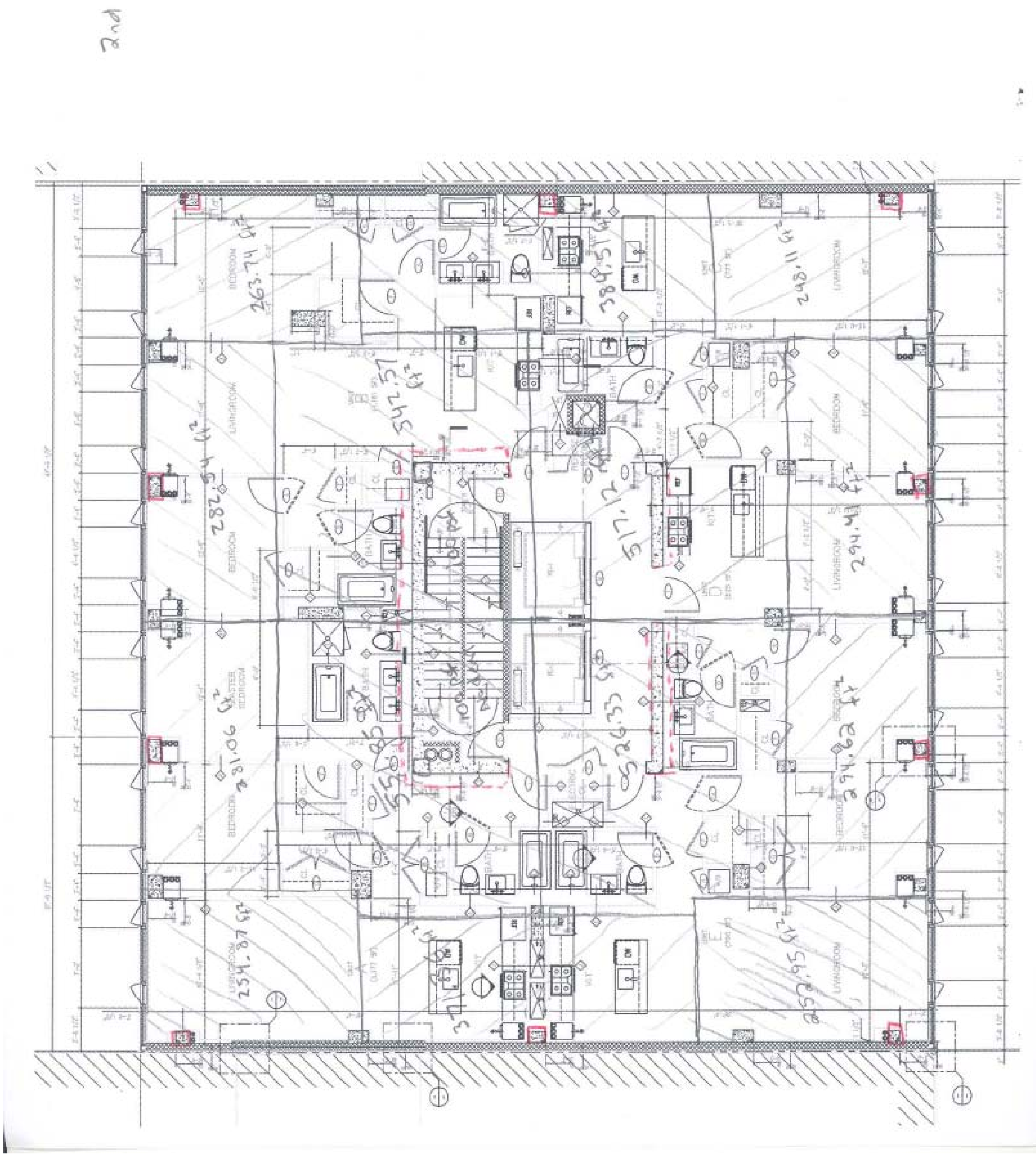


$FEM_{AB} = \frac{w\ell^2}{12} = \frac{7.28\,(27)^2}{12} = 442.26\ 'k \approx FEM_{CD}$

(A), (B), (C), (D) $\Delta FEM = 442.26\ 'k$

Along (4): Same as along (2)

Along (5): same as along (1)



A sample quick sketch determining where to keep or remove columns from the existing layout

Appendix C: Floor Slab Design

Punching Shear Check

Loading

- $w_u = 1.2(100 + 25) + 1.6(40) = 214$ psf for all columns except 5, 6, 9, 10

$w_u = 1.6(100) = 160$ psf for columns 5, 6, 9, 10 by stairwell

* note: use larger w_u for columns 5, 6, 9, 10 just to be safe

- $d = 8" - \underset{\text{cover}}{.75"} - \underset{\text{stirrup}}{.5"} - \frac{.5"}{2} = 6.5"$

Column 12 — 11" × 11" ← (smallest column)

Floors 17+

$b_o = (11" + 6.5")4 = 70"$

- V_u = Load (Tributary area − b_o area)

$= .214\text{ k}\left(176.45\text{ ft}^2 - (17.5/12)^2\right) = 37.3\text{ k}$

- $\phi V_c = .75\left(\beta_p\sqrt{f'_c} + .3 f_{pc}\right)b_o d + V_p$ (ACI 318-05 11.12.2.2)

where $\beta_p = \min\begin{cases} 3.5" \\ \alpha_s d/b_o + 1.5 = 30(11" - 1.5" - .5" - .5")/70" + 1.5" = 5.14" \end{cases}$

$\beta_p = 3.5"$

code req't: 125 psi < f_{pc} = 158 psi (from RAM Concept model)

V_p = vertical component = 0 conservatively

$\phi V_c = .75\left(3.5\sqrt{5000} + .3(158\text{ psi})\right)70"(6.5") + 0$

$\phi V_c = 100.63$ k OK

- $\phi V_c = .75 \times 4 \times \sqrt{5000} \times 70" \times 6.5" = 96.5$ k OK

Column 12 will not fail in punching shear, this is the most critical case because it is the smallest column

RAM Concept Model Details

Materials

Concrete Mix

Mix Name	Density (pcf)	f'ci (psi)	f'c (psi)	fcui (psi)	fcu (psi)	Poissons Ratio	Ec Calc	User Eci (psi)	User Ec (psi)
3000 psi	150	3000	3000	3725	3725	0.2	ACI 8.5.1 (no Wc)	2500000	3000000
4000 psi	150	3000	4000	3725	4975	0.2	ACI 8.5.1 (no Wc)	2500000	3000000
5000 psi	150	3000	5000	3725	6399	0.2	ACI 8.5.1 (no Wc)	2500000	3000000
6000 psi	150	3000	6000	3725	7450	0.2	ACI 8.5.1 (no Wc)	2500000	3000000

PT Systems

System Name	Type	Aps (sq. in.)	Eps (ksi)	fse (ksi)	fpy (ksi)	fpu (ksi)	Duct Width (inches)	Strands Per Duct	Min Radius (feet)
½" Unbonded	unbonded	0.153	28000	175	243	270	0.5	1	6
½" Bonded	bonded	0.153	28000	160	243	270	3	4	6
0.6" Unbonded	unbonded	0.217	28000	175	243	270	0.6	1	8
0.6" Bonded	bonded	0.217	28000	160	243	270	4	4	8

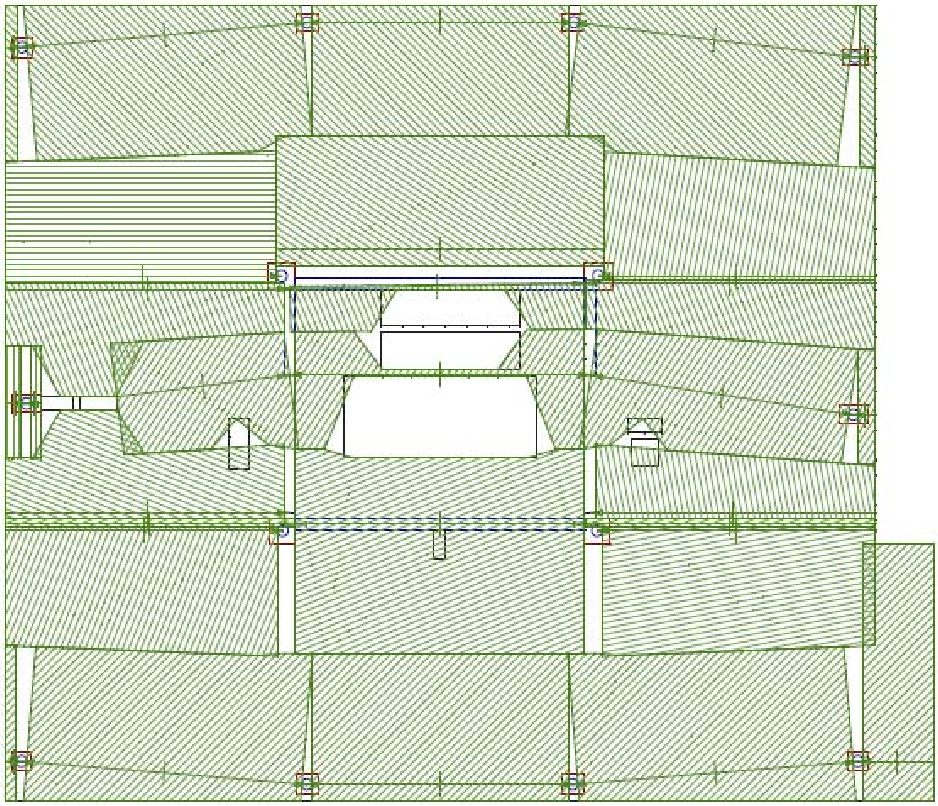
PT Stressing Parameters

System Name	Jacking Stress (ksi)	Seating Loss (inches)	Anchor Friction	Wobble Friction (1/feet)	Angular Friction (1/radians)	Long-Term Losses (ksi)
½" Unbonded	216	0.25	0	0.0014	0.07	22
½" Bonded	216	0.25	0.02	0.001	0.2	22
0.6" Unbonded	216	0.25	0	0.0014	0.07	22
0.6" Bonded	216	0.25	0.02	0.001	0.2	22

Reinforcing Bars

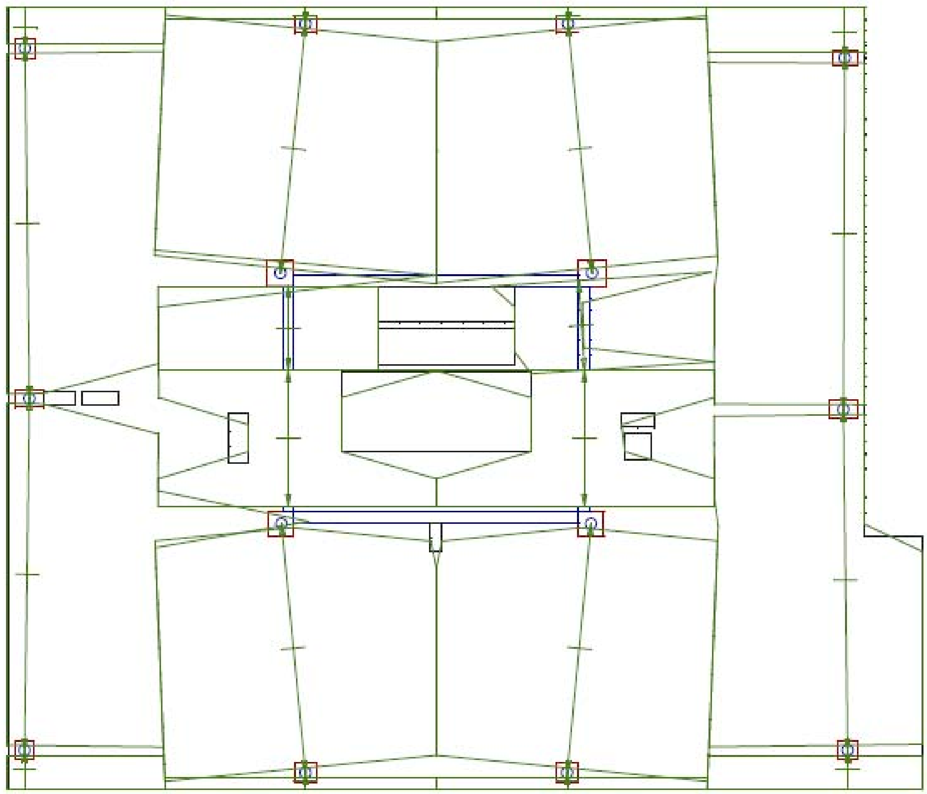
Bar Name	As (ksi)	ES (ksi)	Fy (ksi)
#3	0.11	29000	60
#4	0.2	29000	60
#5	0.31	29000	60
#6	0.44	29000	60
#7	0.6	29000	60
#8	0.79	29000	60
#9	1	29000	60
#10	1.27	29000	60
#11	1.56	29000	60

Lateral Design Spans Plan



Generated by RAM Concept

Longitude Design Spans Plan



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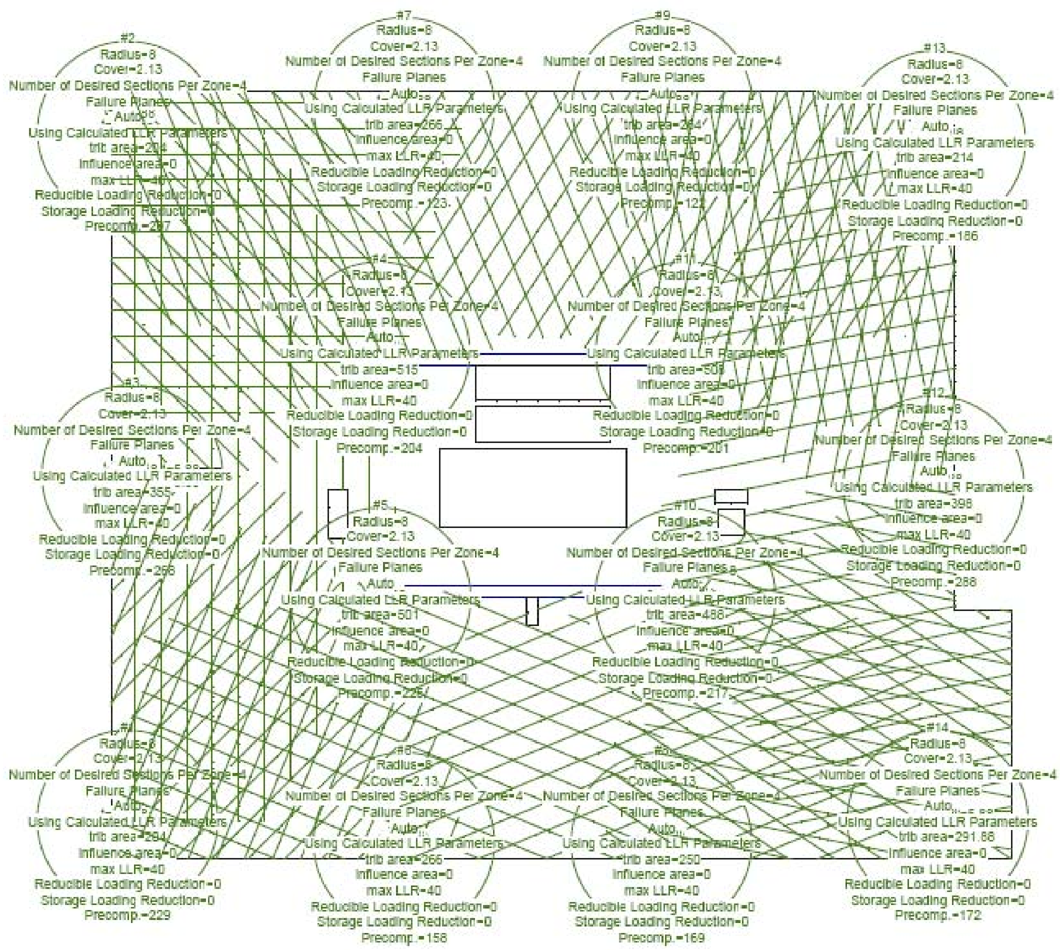
Punching Checks Plan



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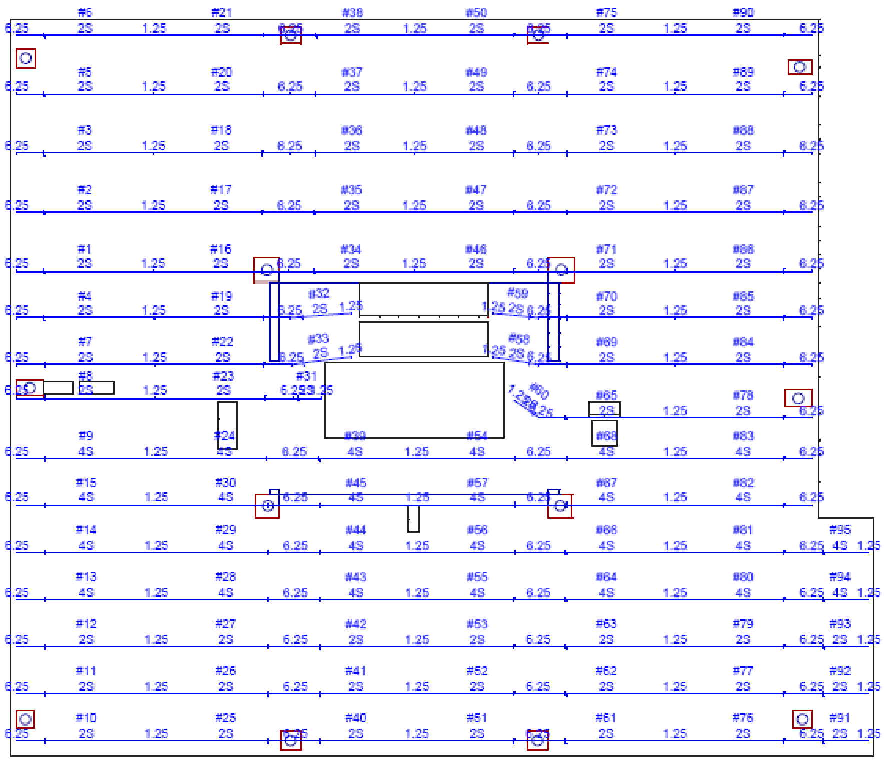
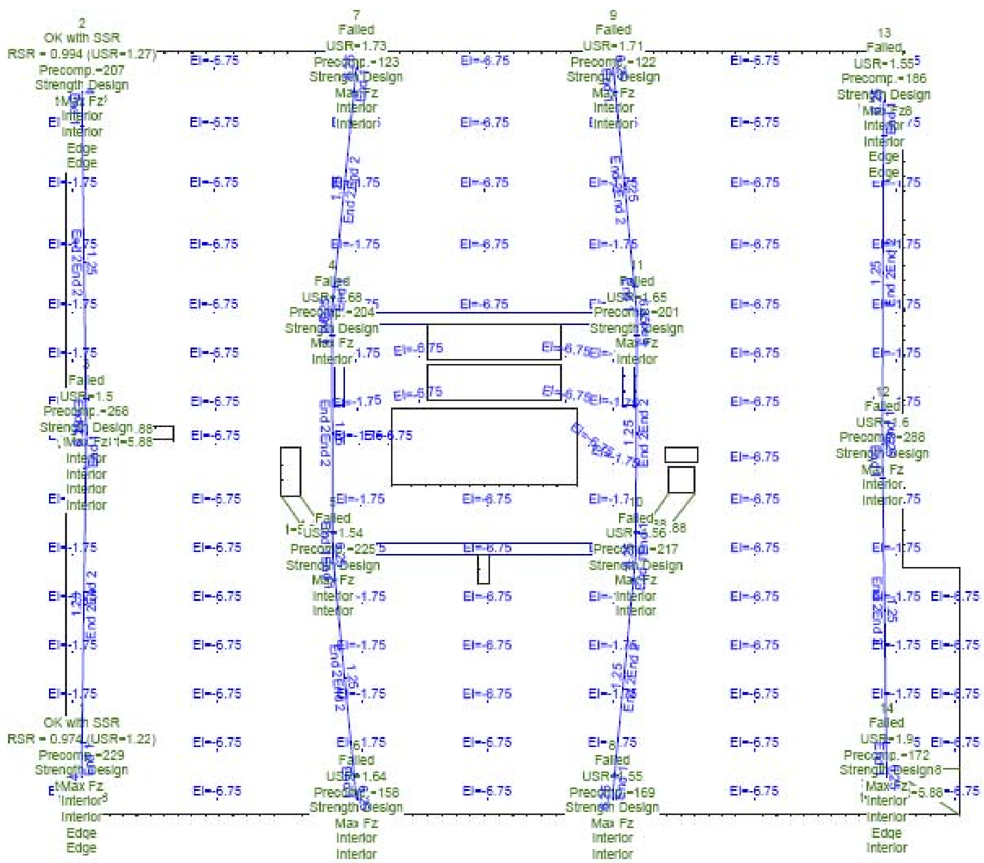
Latitude Tendon: Standard Plan



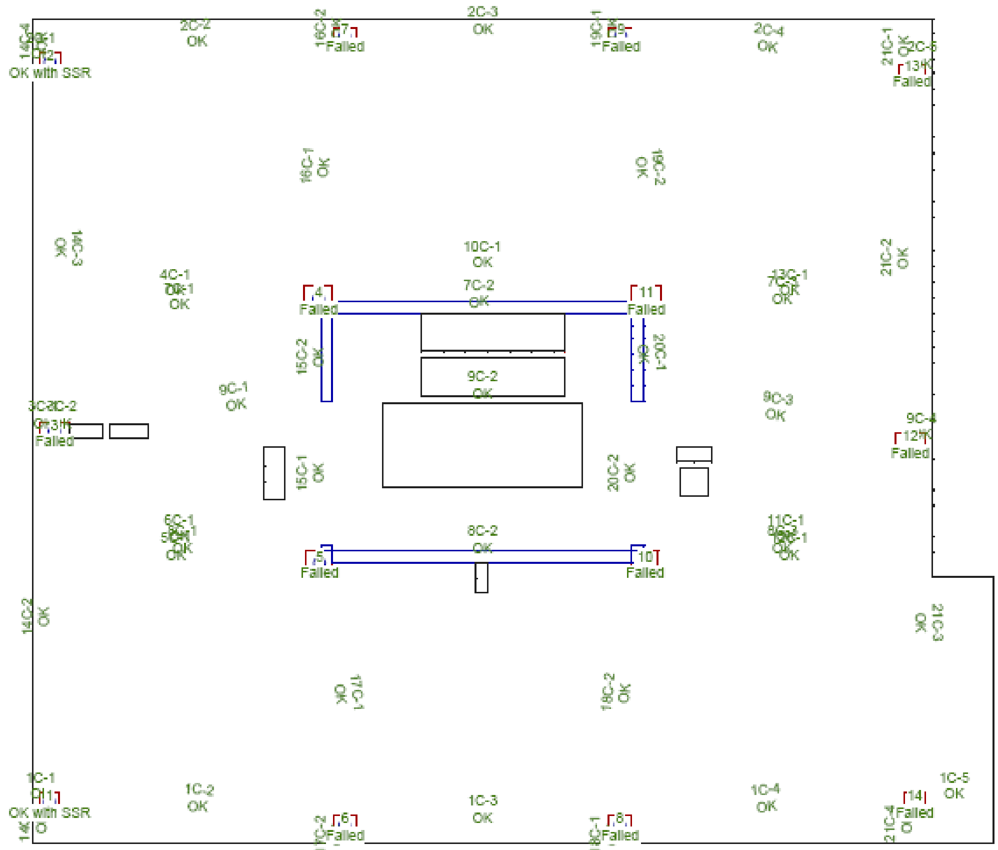
Generated by RAM Concept

Longitude Tendon: Standard Plan



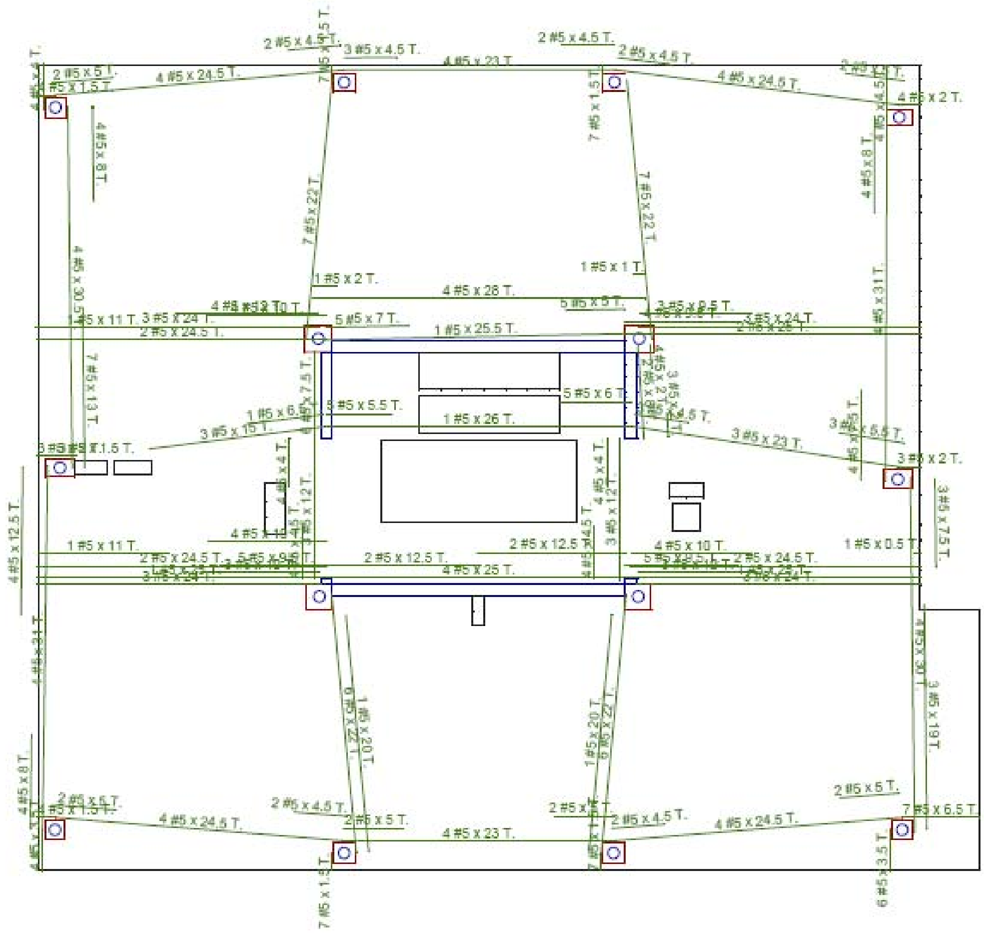
Generated by RAM Concept

Design Summary: Status Plan



Generated by RAM Concept

Design Summary: Top Reinforcement Plan



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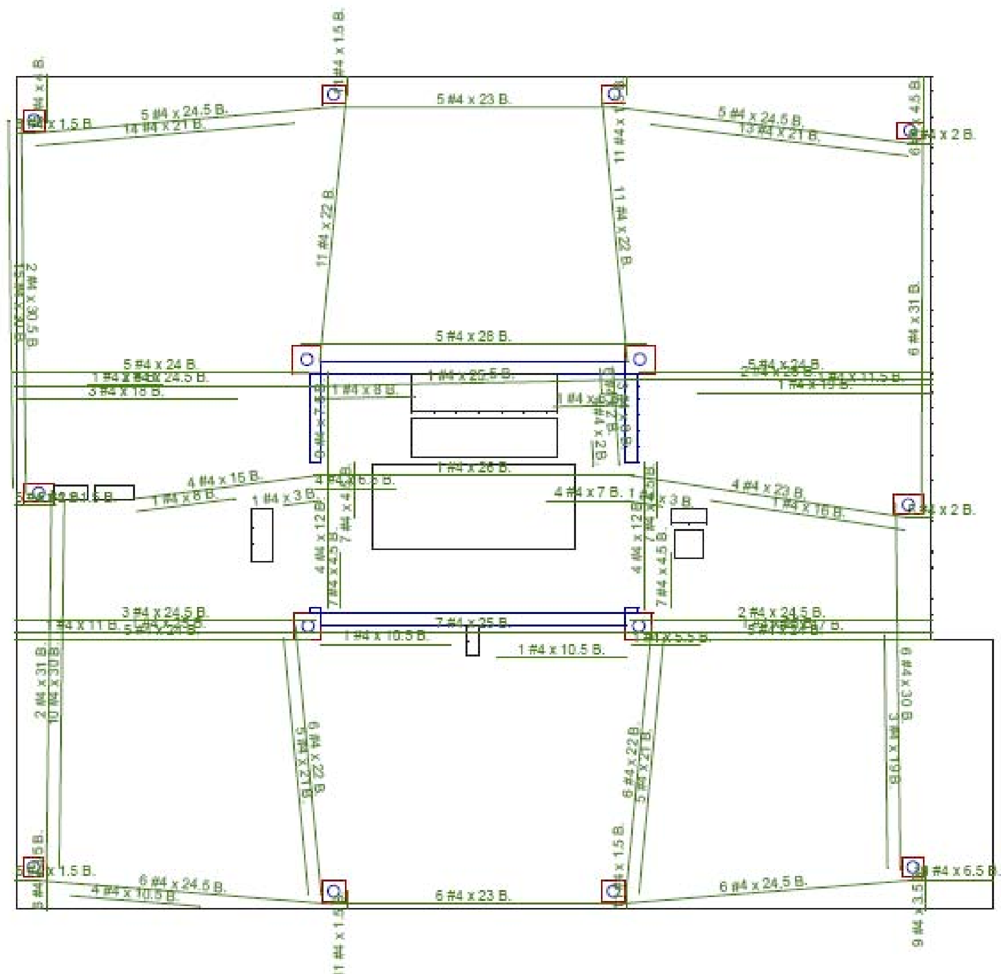
Design Summary: Bottom Reinforcement Plan



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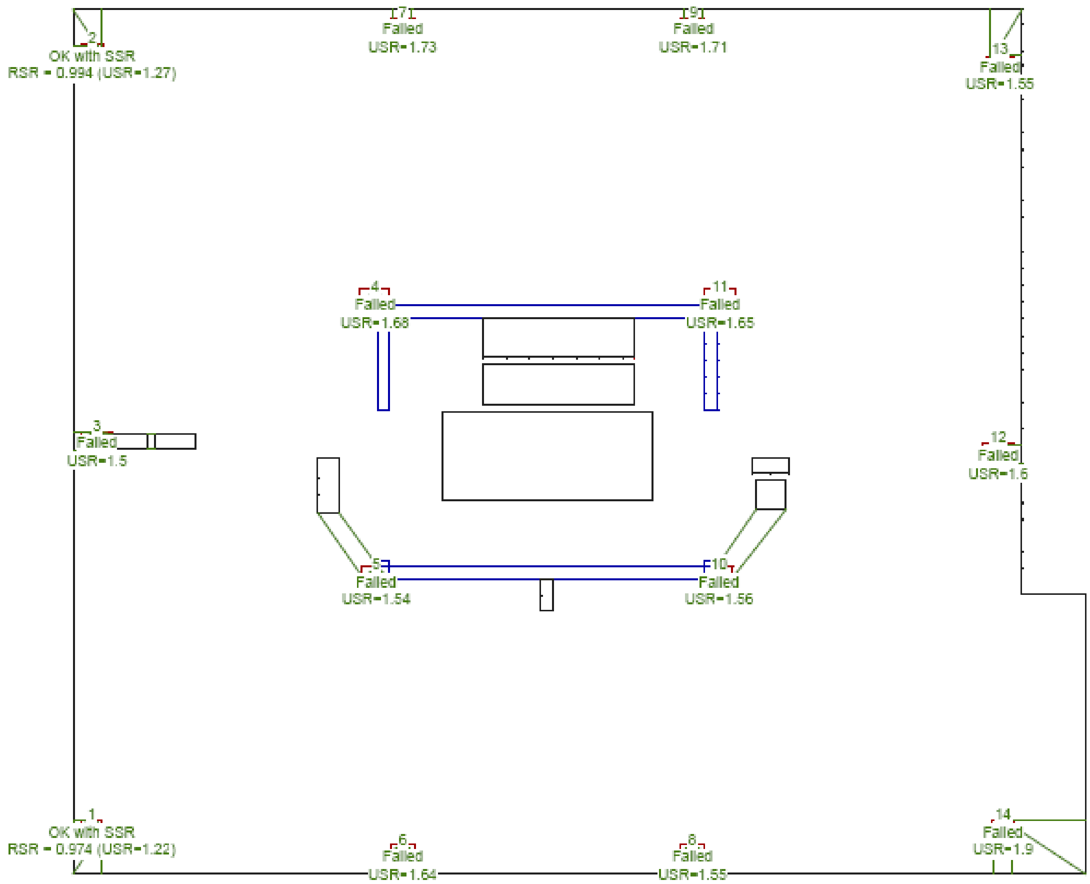
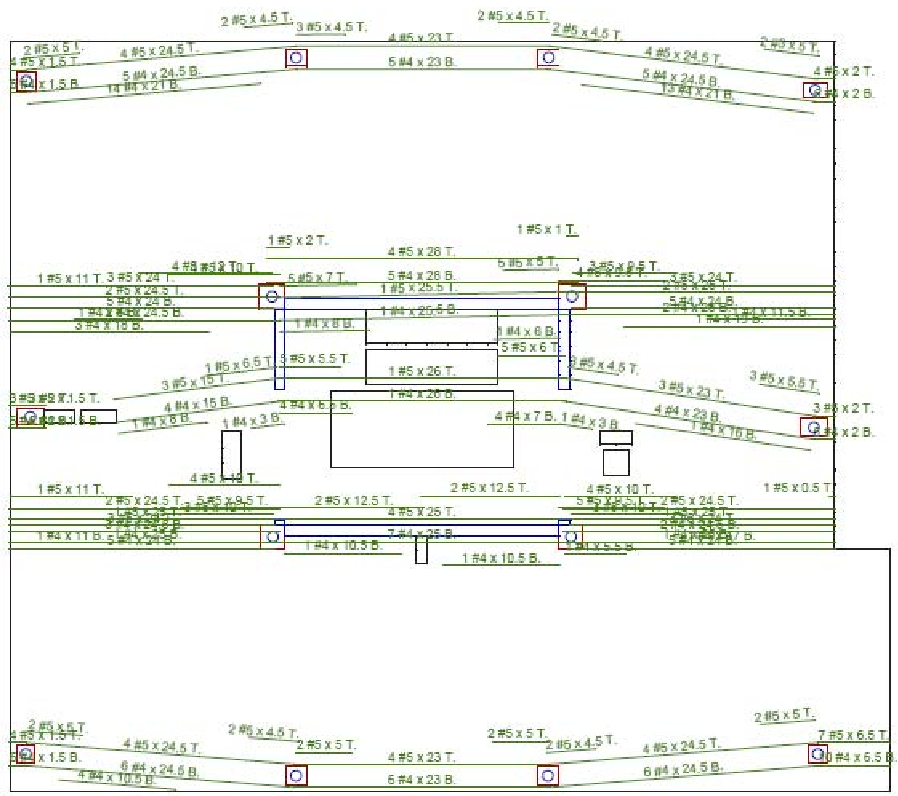
Design Summary: Punching Shear Status Plan



Generated by RAM Concept

Design Summary: Latitude Reinforcement Plan



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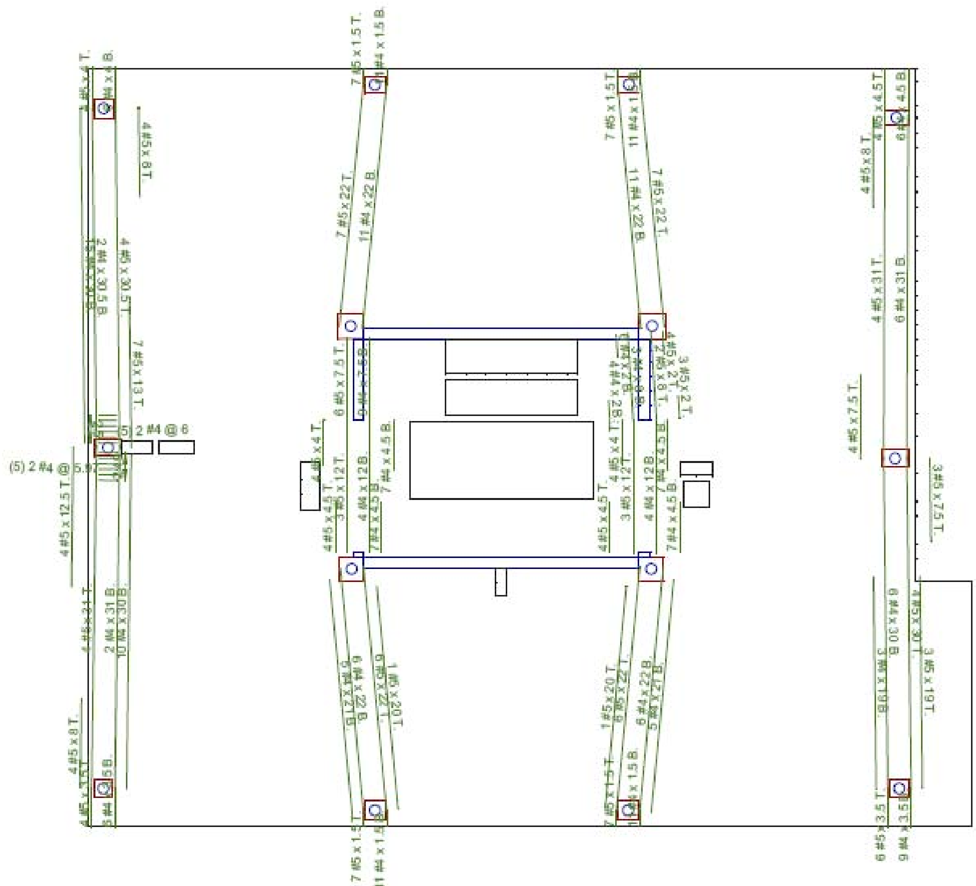
Design Summary: Longitude Reinforcement Plan



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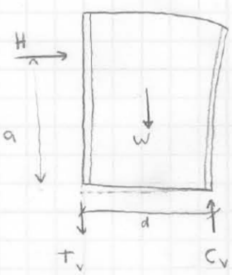
Appendix D: Lateral System Design

1 Shearwall Design pg. 553

Pier 1 - Combined System



$M_u = 19{,}571.75$ in·k $= 1630.98$ 'k (from ETab

$P_u = 208.1$ k (axial compression on end due to resolved force couple from lateral loading)

$l_w = 94"$

$h_w = 12' = 144"$

$t_w = 12"$

(a) Check whether boundary elements are needed:

- $I_{n.a.} = \frac{(12")(94")^3}{12} = 830{,}584 \text{ in}^4$
- $A_g = (12")(94") = 1128 \text{ in}^2$
- $f_c = \frac{P_u}{A_g} + \frac{M_u\, h_w/2}{I_{n.a.}} = \frac{208.1 \text{ k}}{1128 \text{ in}^2} + \frac{19{,}571.75 \text{ in·k}\,(144")/2}{830{,}584 \text{ in}^4}$

$f_c = 1.88$ ksi $> .2f'_c = .2(5) = 1$ (ACI 26.6.2.3)

∴ Boundary elements are required

(b) Determine minimum longitudinal and transverse reinforcement reinforcement

- $2A_{cv}\sqrt{f'_c} = \frac{2(12)(144")\sqrt{5000}}{1000} = 244.4$ kips

V_u = horizontal max shear on wall 1 = 212.88 kips

$2A_{cv}\sqrt{f'_c} = 244.4 > V_u = 212.88$

∴ 1 curtain of reinforcement is required

- Required longitudinal + transverse reinforcement in wall:

$\rho_v = \frac{A_{sv}}{A_{cv}} = \rho_n \geq .0025$ (max. spacing = 18")

where $A_{cv} = 12(12) = 144 \text{ in}^2$ (per ft. of wall)

$A_{sv} = \rho_n(A_{cv}) = .0025(144 \text{ in}^2) = .36 \text{ in}^2/\text{ft.}$

A_s of #6 $= .44 \text{ in}^2 > .36 \text{ in}^2$

2 Shearwall Design

$S_{(required)} = \frac{.44}{.36}(12) = 14.67'' \rightarrow 14''$ in.

(c) Determine reinforcement requirements for shear

Assume one curtain of No. 6 bars @ 14 in o.c. both ways.

$\frac{h_w}{l_w} = \frac{144''}{94''} = 1.53 < 2$

∴ design in accordance with ACI App. A and 11.10.9.2 through 11.10.9.5

11.10.9.3

$s \le \min \begin{cases} l_w/5 = 94/5 = 18.8'' \\ 3h = 3(144'') \\ 18'' \end{cases}$

11.10.9.4

$\rho_l = .0025 + .5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - .0025)$

where $\rho_t = \frac{.44 \text{ in}^2/\text{ft}}{A_{cv}} = \frac{.44\left(\frac{12}{14}\right)}{144} = .0026$

$\rho_l = .0025 + .5\left(2.5 - \frac{144}{94}\right)(.0026 - .0025)$

$\rho_l = .00256$

∴ use #6 @ 14 in. o.c. in both horizontal and vertical directions

(d) check adequacy of boundary element acting as a short column under factored vertical forces due to gravity & lateral loads

$P_u = 208.1$ k

- Try a boundary element 15 in. x 15 in. w/12 #6 bars

$A_g = (15 \text{ in})^2 = 225 \text{ in}^2$

$A_{st} = (12)(.44 \text{ in}^2) = 5.28 \text{ in}^2$

$\rho_{st} = \frac{5.28}{225} = .0234$

3 Shearwall Design

$\rho_{min} = .01 < \rho_{st} < \rho_{max} = .06$ OK

Axial load capacity of a short column:

$\phi P_n = .80\,\phi\left[.85 f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$

$= .80(.70)\left[.85(5\text{ ksi})\left(225 - 5.28\text{ in}^2\right) + (60\text{ ksi})(5.28\text{ in}^2)\right]$

$= 700.34\text{ k} > 208.1\text{ k}$ OK

- Try a smaller boundary element: 12 in. x 12 in. w/ 12 #6 bars

$A_g = 12^2 = 144\text{ in}^2$

$A_{st} = 5.28\text{ in.}^2$

$\rho_{st} = \frac{5.28}{144} = .0367$

$\rho_{min} = .01 < \rho_{st} < \rho_{max} = .06$ OK

Axial load capacity:

$\phi P_n = .80(.70)\left[.85(5)(144 - 5.28) + 60(5.28)\right]$

$= 507.56\text{ k} > 208.1\text{ k}$

$\boxed{\therefore \text{ use a } 12'' \times 12'' \text{ boundary element w/ 12 \#6 bars}}$

$b_{min} = 1.5''(2) + 4(\underset{d_{\#6}}{.75}) + 3(\underset{\text{spaces}}{.75}) = 8.25\text{ in.} < 12\text{ in}$ OK

(e) Determine lateral (confinement) reinforcement for boundary element

$s_{max} = \begin{cases} 1/4\,(12'') = 3'' \\ 4\text{ in} \end{cases}$

- required cross-sectional area of confinement reinforcement in short dir.

$A_{sh} \geq \begin{cases} .09 s h_c \dfrac{f'_c}{f_{yh}} \\ .3 s h_c \left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f'_c}{f_{yh}} \end{cases}$

Assume #5 hoops @ 3 in. o.c. and 3 in. from center line of the #6 vertical bars to the face of the column.

4 shearwall design

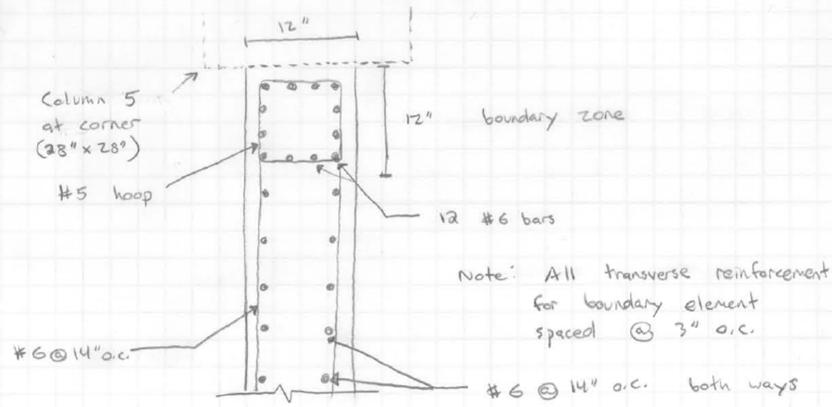
$h_c = (12-6) + 1.41 + .625 = 8.035$ in.

$A_{ch} = (8.035 + .625)(6'' + 1.41 + 1.25) = 75$ in.^{2}

$$A_{sh} > \begin{cases} .09(3'')(8.035)\left(\frac{5}{60}\right) = .18 \text{ in}^2 \\ .3(3'')(8.035)\left(\frac{144}{75} - 1\right)\frac{5}{60} = .554 \text{ in}^2 \quad \text{governs} \end{cases}$$

A_{sh} (provided) $= (2 \text{ legs})(.31) = .62 \text{ in}^2 > .554 \text{ in}^2$ OK

* same for long direction (12 x 12 boundary element)



Half section of structural wall at base

* Note boundary element design and shearwall reinforcement design can be applied to all piers with little variation since $P_{u_{max}} = 501$ k $< \phi P_n = 507.56$ k except pier 4 which will require a little more reinforcing for $P_u = 577$ k. In all cases, the boundary element can be confined to 12" x 12", the standard shearwall size.

Appendix E: Breadth Topics

Breadth 2: CM

#4 = .668 lb/ft

- Assume #4 @ 16" o.c. top + bottom, continuous each way in slab. (from dwg. S-102, note #2)

- $\left(\frac{75' \text{ long}}{16''} + \frac{68' \text{ wide}}{16''} \right) \left(\frac{12 \text{ in.}}{\text{ft.}} \right) (2 \text{ top + bottom}) = \#$ of bars

  # of bars = 214.5 → 215 bars total

- Assuming bars over gaps are continuous will account for additional reinforcement included in the plans.

- $\frac{75'}{16''} \left(\frac{12 \text{ in.}}{\text{ft}} \right) (2) = 112.5$ bars lengthwise

  (113 bars)(75') = 8,475 ' of bar

  = (8475)(.668) = 5661.3 lb. rebar

- $\frac{68'}{16''} \left(\frac{12 \text{ in}}{1 \text{ ft}} \right) (2) = 102$ bars width

  (102 bars)(68') = 6936 ' of bar

  = (6936)(.668) = 4633.248 lb. rebar

Total sq. ft / floor = 68' x 75' = 5100 sq. ft.

$\frac{4633.248 \text{ lb. rebar}}{5100 \text{ sq. ft.}} = .9085 \text{ lb/ft}^2$

A calculation to determine the concentration of reinforcing in the floor slab for input into ICE 2000