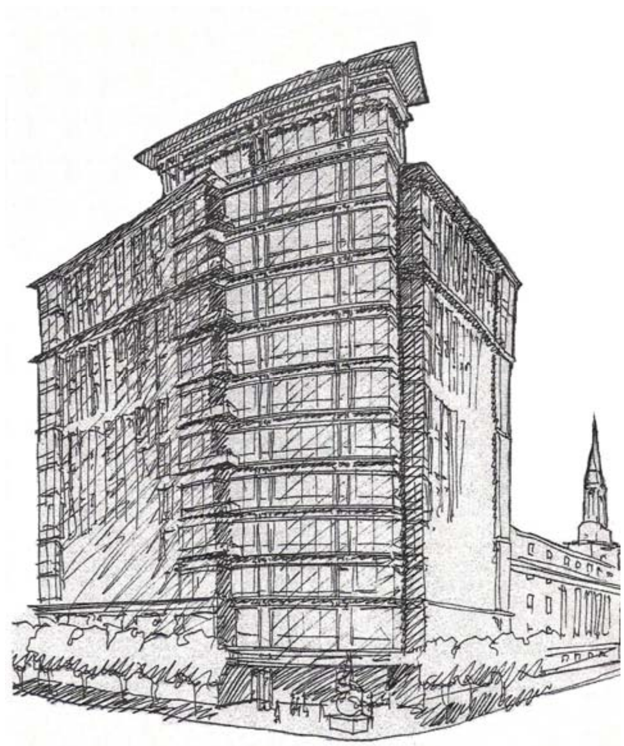
The Executive Tower
NW Washington, DC



Sean Howard
April 3, 2006
Structural
Dr. Linda Hanagan

Executive Tower

Washington, DC

Sean Howard

Structural

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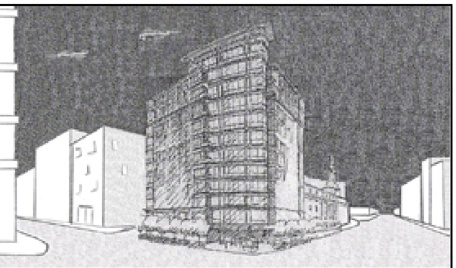


Project Team

- Owner: Kaempfer Company
- Architect: Hellmuth, Obata + Kassabaum, Inc. (HOK)
- Structural Engineer: Tadger, Cohen, Edelson Assoc.
- MEP: GHT ltd
- Geotechnical engineers: Schnabel Engineering
- General Contractor: Tompkins Builders
- Size: 132,268 sqft
- 12 Stories
- Class A office building

Architecture

- Trademark curved façade at south east corner
- Precast concrete and glass envelope
- Granite exterior for first and second floors
- Penthouse viewing of Washington DC including the Capitol Dome, the Washington Monument, and the White House

Structural

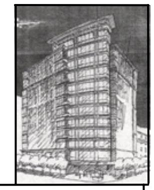
- MAT Foundation
  - 42" thick
  - 13'x13'x1' additional footings
- Cast-in-Place Concrete Construction
- Concrete Flat Plate system with typical:
  - Columns: 20"x20"
  - Plates: 8'x10'x8"
  - Slab: 8"

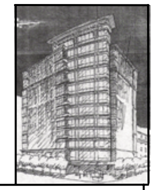
Mechanical

- Cooling towers in penthouse
- 13 VAV water cooled AC units for each floor plus one for the lobby and fitness room
- Heating and Air conditioning is all monitored by computer from building engineers office

Lighting/Electrical

- typical incandescent 3-phase lighting at 277V
- 480/277V 3-phase Electrical feed





EXECUTIVE SUMMARY

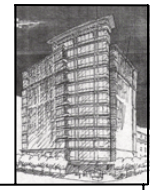
The Executive Tower is one of the highest rental rates in the Washington DC area at $47 per sqft-month. At this rate, constructing buildings with a maximum floor space is ideal. However, in the case of the Executive Tower, and most buildings the DC area, it has a height restriction of 130' measured from the north edge of the building to the ceiling of the 11th floor with an 18' penthouse space above not included in the height. Concrete systems are typically used in DC in order to achieve thinner ceiling spaces and get a maximum number of floors over a plot of land. The same concept was used in this report where an architectural study, mechanical study, and post tension design were used with similar goals of ultimately lowering the building height enough to construct a 12th floor typical to floors three through nine.

The architectural breadth developed a new design for the entrance into Retail 2. The building height is measure at the north corner. If the north corner were even with the south end, the Executive Tower has the potential of being constructed 5' – 6" lower. This entrance was designed to be recessed into the ground by 3' – 0" after drawing a few sketches and comparing their advantages and disadvantages.

The mechanical breadth study rerouted a new duct system to optimizing the air flow through each duct. By doing this air was more evenly distributed through the system so the duct sizes were able to be sized to thinner sections. The controlling duct size in the existing system was 12 inches. After the rerouting and excel calculations, this number was able to be reduced to nine inches, saving three inches per floor.

The depth study of this report was converting the Executive Tower's floor system from a reinforced flat slab to a post tension slab to reduce the thickness up to three inches, from eight inches to five inches. A model was constructed using RAM Concept to calculate the various arrangements of the columns in the Executive Tower through a finite analysis. The results were conclusive that a post tensioned slab was necessary to decrease the slab, however, through the analysis it was only able to be reduced by two inches. The five inch slab was failing in both flexure and deflection in most of the long spans of the floor system.

As a result of the new systems, the Executive Tower building height was able to be reduced by 9' – 3". The necessary reduction needed to be at least 11' – 0". The Executive Tower is only 1' – 10" over the 130' with the addition of the 12th floor, however this is still capable of being reduced under this limit by lowering the ceiling height per floor by only two inches, from 9' ceilings to 8' – 10" ceilings.



BUILDING DESCRIPTION

The Executive Tower 132,000 sqft Class A office building in the heart of Washington, DC located two blocks northeast of the White House and can see in plane sight the Washington Monument and Capitol Dome from the penthouse courtyard. This eleven story office building offers both sectioned and open floors plans to numerous companies such as Bloomberg Financial, Merrill Lynch, and AIG.

ARCHITECTURE

Executive Tower uses a curtain wall system consisting of glass with aluminum framing and precast concrete stretching horizontally around the whole building at each floor level with few precast concrete lines in the vertical direction. The first and second floors on the fronts facing New York Ave, H St and most of 14th St are showcased with granite paneling at floor level and over exterior columns. The east side borders and existing church also has all precast concrete panels at level 6 and is cmu block wall at level 5 and below. The west face on the south end features the building's trademark curved façade which links the skewed street of New York Ave and 14 St. This is further pronounced by keeping this shape separate from the rest of the building by not having a granite paneling at the 1st and 2nd level, cantilevering the corner by 19 feet and extending the façade above the 11th floor to make the outdoor viewing area. The roof of the building stands out by having precast capitals at level 10 and the roof of level 11 and is topped with a larger precast capital along the curved roof of the viewing area.

The lobby rests on the southwest corner at New York and 14th St. and is inviting to the eyes with its high ceilings, and wood, marble and granite veneers line the walls and floor. To reach the elevator lobby, one must walk through the rotunda, a cylindrical wood veneer room in the center of the buildings footprint. The first floor also houses the fitness center and retail with loading bay accessed on H St. The 2nd through 11th floors are all tenant space. The penthouse and main roof contain the main mechanical room, cooling towers, emergency generators, building engineer's office and a covered outdoor view area that over looks the White House, Washington Monument and the Capitol dome.

PROJECT TEAM

Owner……………………………………………….1399 New York Ave Associates
Managing Group……………………………………………...Kaempfer Company
Architects……………………………Hellmuth, Obata + Kassabaum, Inc. (HOK)
Structural Engineers…………………………….Tadger, Cohen, Edelson Assoc
MEP Engineers…………………………………………………………...GHT ltd
Geotechnical Engineers……………………………..….Schnabel Engineering
Construction Management…………………………………….Tompkins Builders

BUILDING SYSTEMS

STRUCTURAL SYSTEMS

FLOOR SYSTEM

The floor system of the Executive Towers is a two-way flat plate concrete slab, a typical systems used in and around the DC area to allow a maximum number of floors to be constructed in a region with specific height restrictions. The typical thickness for this slab is 8" reinforced with #4 at 12" O.C. The slab around the exterior of the building has an additional 3½" thickness acting as wide exterior beams. Drop Panels at interior and exterior column locations of 10'x8'x8" allow of for the thinner slabs across the longer span.

COLUMN

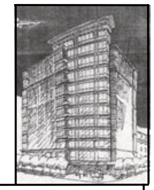
The columns of the Executive Tower consist of all cast in-place-concrete, mostly rectangular spread out variably throughout the floor system as seen in figure 2.1. The flat plate concrete slab allows the column location to be irregular and having a typical bay is virtually non-existent in the Executive Tower. However, the typical column consists of 20"x20" with roughly 6 #10 bars of reinforcement.

FOUNDATION

A mat foundation is utilized to maximize ground contact and distribution of the buildings loads. An additional 13'x13'x1' spread footings at column locations. The MAT is a 42" thick slab fully reinforced with #10@12" O.C. each way bottom steel and #7@12" O.C. each way top steel. Sheeting and shoring is placed on the north, south and west side of building and underpinning is required only on the east side.

LATERAL RESISTANCE

The lateral resisting system consists of six shear walls forming the enclosure of the elevator shafts in the center of the building. The shear walls are all 12" thick extending the height of the building and is reinforced with #6@8" horizontal steel through the height of the building.



MECHANICAL SYSTEM

The mechanical rooms are located in the penthouse of the executive tower, which contains cooling towers that feed the 13 VAV water cooled A/C units located at on each floor including one for the fitness center, lobby and penthouse.

The building's entire central air system is monitored by the building's engineer in the penthouse. Through this system he can change cooling and heating temperatures, flow rates and change exchange ratios.

FIRE PROTECTION

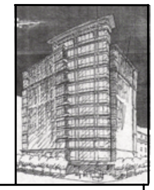
Executive Towers uses 2 hour rating in most area such load bearing walls and columns. For non load bearing separations a one hour rating is used. Throughout the tenant spaces, lobby, and fitness room a wet sprinkler system is used with a standpipe in the main stairwell located in the center of the building.

PLUMBING

A Duplex booster pump with hydrocumulator tank located in the P1 parking level pumps the domestic water throughout the entire building and to two electric water heaters located in the penthouse mechanical room.

TRANSPORTATION

Executive Tower consists of a four elevator core in the center of the building which can be used to access the three below grade parking levels and to the 11th floor. The elevator 1 located in the top left corner of the core is used to access the penthouse and main roof. There is a single stairwell adjacent to the elevator core.



PROPOSAL

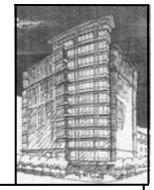
PROBLEM STATEMENT

The Executive Tower rests in the downtown area of Washington DC. As with most buildings in this district, the Executive Tower is restricted to a maximum height set by the DC zoning regulations based on the width of the adjacent street. The limiting height requirement is equivalent of 30 feet over the width of that street. These standards are put in place to insure the District of Columbia skyline does not bleed out the view of the national landmarks such as the Washington Monument and the Capitol Dome. As a result of these ordinances, building owners in the DC area requested buildings with as many rentable floors within the limits as possible. To accommodate this, most buildings in Washington are concrete structure utilizing various floor framing systems to minimize the space need in between floors. The engineers of the Executive Tower used a concrete flat slab system with drop panels to accommodate DC's ordinances.

The Executive Tower is surrounded on three sides with H, 14th and New York Ave. Adjacent, to its east, is the New York Ave Presbyterian Church. Limited to the defined area of 13,278.58 sqft, the Executive Tower built up to 128' – 4" just under its maximum height restriction of 130 ft. It is due to the high land value in Washington DC that building owners go to great lengths in order to get the maximum number of floors within their limits. In the case of the Executive Tower, the building tops out at 11 stories, 1' – 8" short of the maximum building height.

PROBLEM SOLUTION

In a city where maximum rentable floor area is ideal, designing and coordinating various systems to achieve this goal is a necessity. In Technical Report 2, alternative framing systems that could be used for the Executive Tower were studied. It was found that the two steel systems would be inefficient at meeting floor depth required to create even eleven stories under the 130' height limit, much less a 12th floor. Two concrete systems, flat plate and flat slab post tensioning, were



purposed and found to be adequate to meet height limits. However, the post tensioning system proved to provide the most advantages by decreasing the depth of the floor slab.

Complying to the DC regulations regarding height of the building, a new design of the building's framing system and other methods will be performed to trim the ceiling space in between floors in effort to construct a 12^{th} typical floor under the 130' height restriction. The typical height per floor is currently 11' – 6". In order is reach this goal by just thinning the ceiling thickness would require each floor, including the 12^{th}, to be a height of 10' – 8". This is equivalent to a reduction of 10 inches per floor. Three components will be analyzed and designed to achieve this goal.

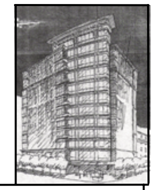
First, a conversion will take place of the framing systems from flat slab to post tension. The findings from Technical Report 2 concluded that post tensioning provided the most advantages such as a lighter structure and by reducing the ceiling space. The result from a post tensioning analysis found the slab could be trimmed by ½". Upon further review, if a post tensioning system with drop panel were used, it would result in thinner slab than the previous study. The two-way post tension slab will comply with ACI 318-05 and DC regulations. Through this analysis, it is predicted the typical slab thickness can be reduce up to 3" per floor resulting in a savings of 2' – 9" of total slab thickness throughout the total building's height.

Two additional breadth studies will be performed; both methods will contribute to thinning the ceiling space thicknesses and lowering the overall building height under the 130' height to add an additional floor.

A study of alternate MEP duct systems will reduce ceiling depths further. The first breadth study is of the mechanical system ducts used in the Executive Tower. The typical ceiling depth is 2' – 6" constructed from the 8" floor slab, MEP ducts, MEP units, recessed lighting fixtures and sprinkler systems. The MEP duct work is the controlling thickness in this space at 12 inches. In this study different MEP systems or alternative routes will be explored in efforts is reduce the heights of the MEP duct to contribute to shrinking the ceilings depths. Similar to the post tensioning, it is a goal for the total floor thickness to be reduced by 3" totaling 2' – 9" to be used to construct the 12th story.

The second breadth study will involve a new design the Executive Tower's entrance into Retail 2 on the first floor at the northwest corner as seen in Figure 9-1 (following page). The architectural design of the landscape and structure on the south end of the building will focus on Retail 2 to lower the building but not inhibit this entrance. The landscaping grade slopes of the north side to the south side creating a difference of 5' – 6" (Figure 9-2, following page). The Executive Tower's height restriction is determined by using the top of slab elevation above the 11th story and the ground elevation at the 1st floor on the north end. By designing the building at this area to be recess, the Executive Tower can subtract up to 5' – 6" from its total height to be used in creating a 12th story.

The goals set forth by this proposal are just estimation of what is ideal. Assuming these three studies are successful, six inches of the ceiling depth per floor combine from both the slab and MEP duct thickness plus a reduction of five and half feet from the total building height. These number summed is equivalent to 138 inches or 11' – 6". The total building height should then be 129' – 6" which is six inches lower than the DC height restrictions.



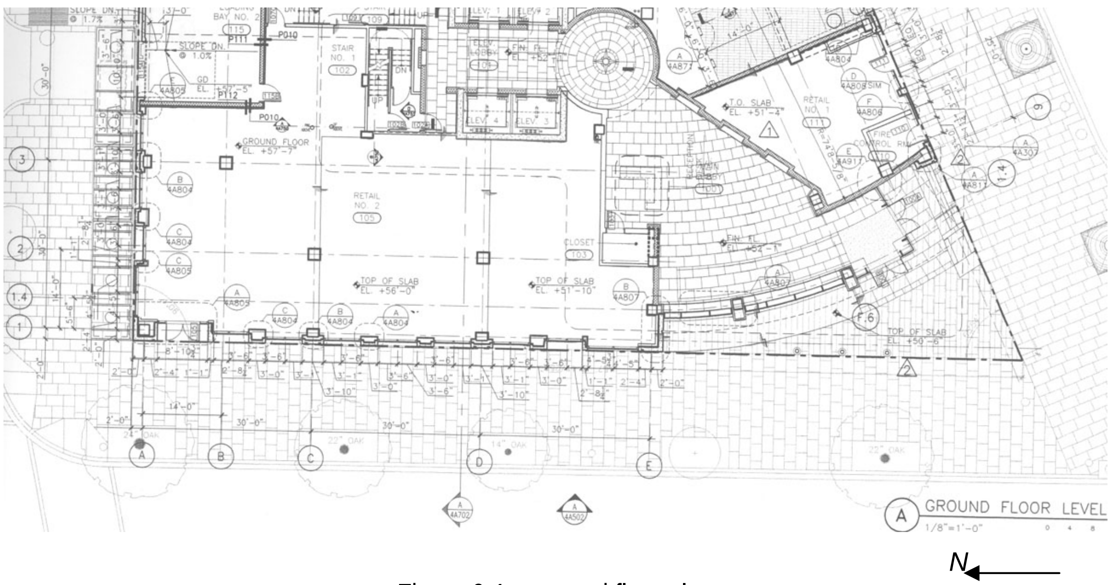


Figure 9-1 – ground floor plan

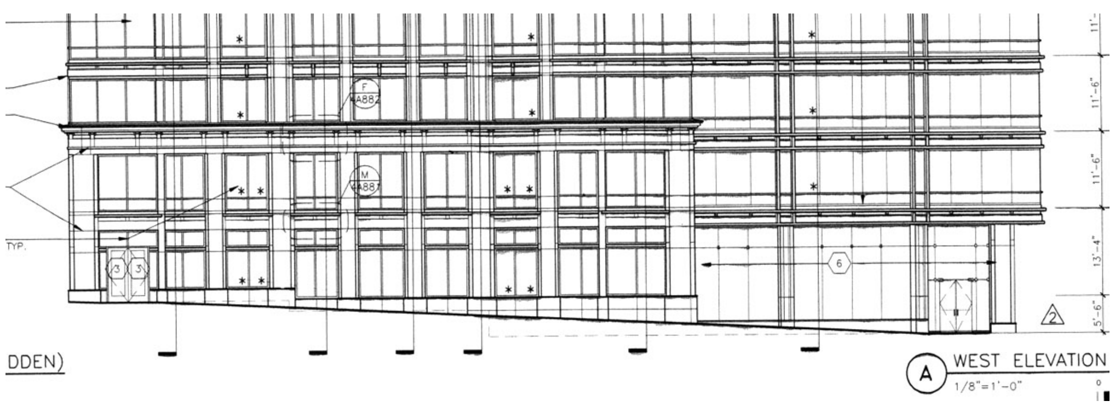


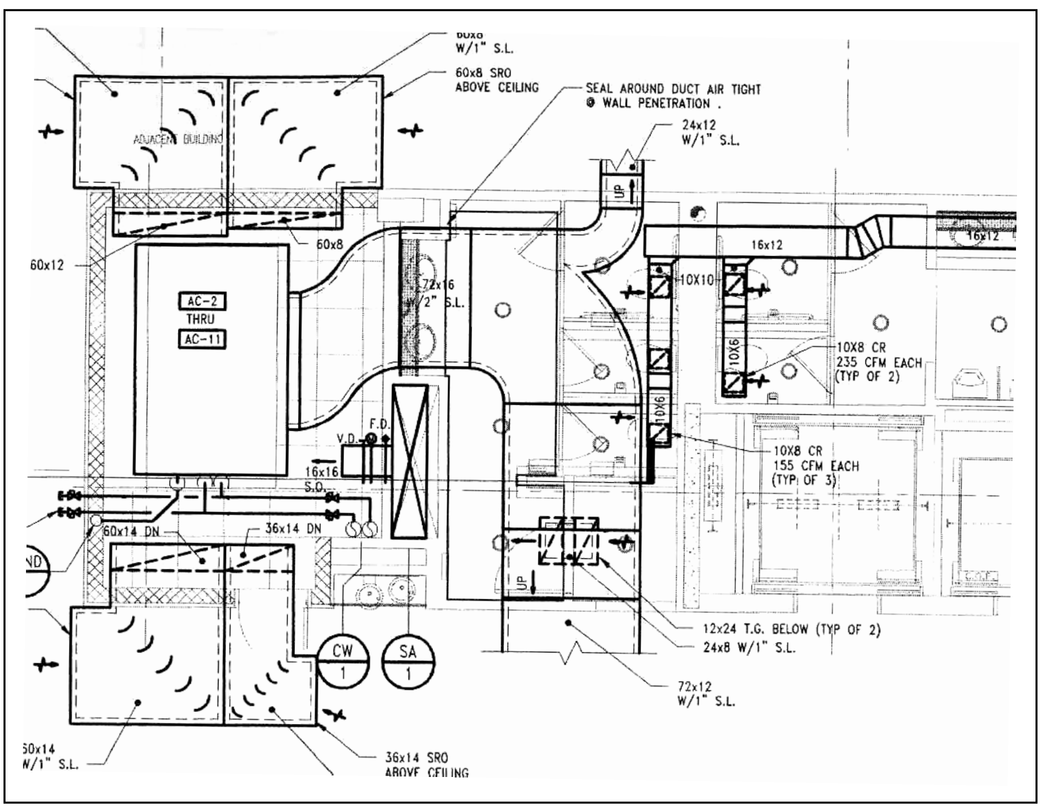
Figure 9-2 – west elevation of floor 1-3

MECHANICAL BREADTH

INTRODUCTION

The Executive Tower's mechanical system is compiled of cooling towers on the penthouse floor that feeds the entire building below. The supply is located in the mechanical room on each floor in the main corridor adjacent to the restrooms. The main supply follows a path over the restroom and splits to feeds to the corridor and the rear of the building. The ducts at this point are nominally 14 inches for the main feed and 12 and 10 after the split (see Figure 10-1).

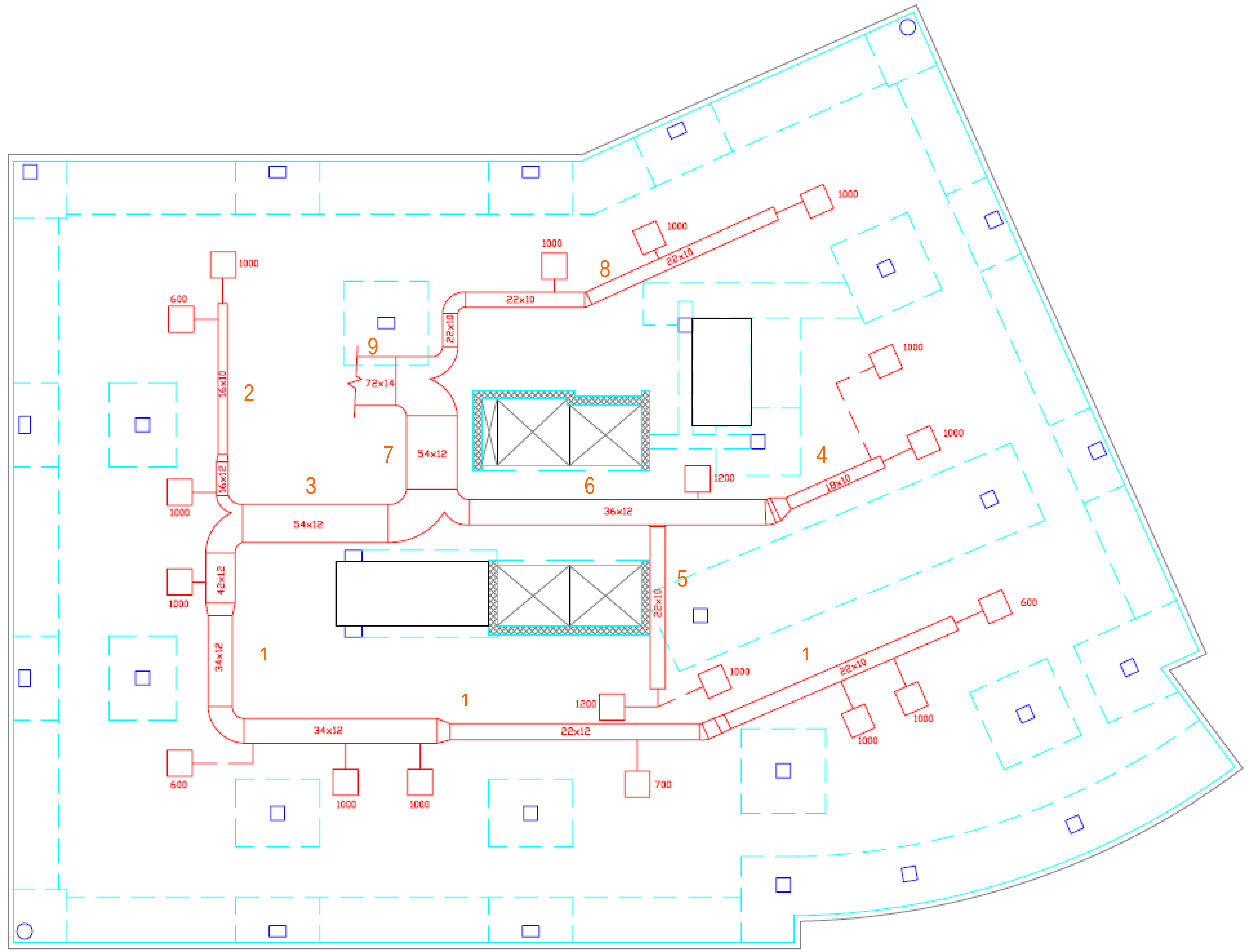
The goal for this study is to cut the depths of these ducts to reduce the ceiling depths per floor up to three inches. At first, it was assumed that by doing this would require a completely alternative system such as a DOAS system which would allow the total air flow per floor to be reduced up to 15%. However, upon further investigation it was realized that by rerouting ducts to evenly distribute air could produce a more efficient system.



DESIGN

Some of the parameters set while following this procedure are designing ducts with similar air velocities, a friction loss of less the 0.65, and the assumption that the ceiling entering and within the restroom can be considered to be lower than the rest of the floor. The current duct system is laid out on the following page. The ceiling over the restrooms is a non-critical area and is going to be allowed to be lowered for this study if needed. In the table below, the air flow through each leg of the duct is used to calculate the air velocity, friction loss and equivalent diameter ducts. Designing the new duct system to have similar air velocities and friction losses will insure the new system is still equivalent to the old system.

Duct section	Duct Size	Equiv. Dia.	Air Flow	Velocity	Friction Loss
	(in x in)	(in)	(cfm)	(fpm)	(water/100')
1	22x10	16	2600	1900	0.31
	22x12	18	3300	2200	0.33
	34x12	21	5900	2300	0.32
	42x12	23	6900	2300	0.30
2	16x10	14	1600	1600	0.26
	16x12	15	2600	2100	0.40
3	54x12	26	9500	2500	0.30
4	18x10	15	2000	2300	0.29
	36x12	22	3200	1200	0.09
5	22x10	16	2200	1600	0.24
6	36x12	22	5400	2000	0.24
7	72x12	30	14900	3100	0.39
8	22x10	16	3000	2300	0.40
9	72x16	35	17900	2800	0.28


1000
600
16x10
2
16x12
1000
3
54x12
42x12
1000
34x12
1
1
34x12
22x12
600
1000
1000
1200
700
22x10
9
72x14
7
54x12
1000
22x10
8
22x10
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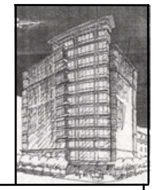
1
22x10
600
1000
1000

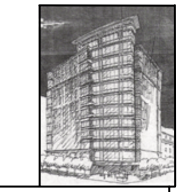


Rearranging the air velocities in descending order, it was found that the ducts with the faster air flow were the ones in the restroom or in the corridor. The ducts around the offices were all approximately 2300 fpm to reduce the noise in these areas. In the new plans, the ducts are sized to be less than 2300 fpm around offices, 2700 fpm in the corridor and less than 3100 fpm over the restroom and into the mechanical room.

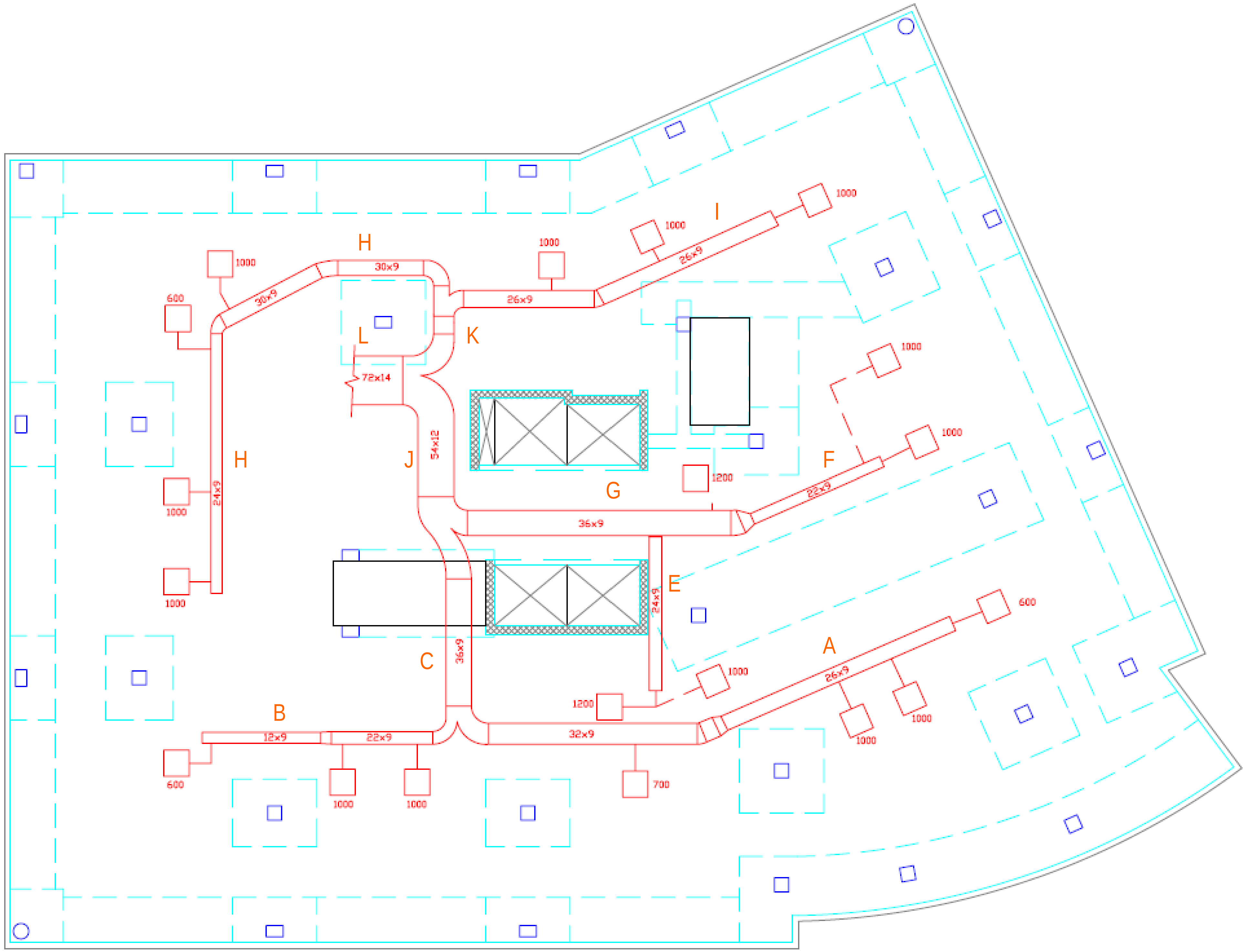
Duct section	Velocity
7	3100
9	2800
3	2500
8	2300
4	2300
1	2300
2	2100
6	2000
5	1600

CONCLUSION

The new plan is designed on the following page and follows the parameters initially set. The deepest section ducts are 14 inches and 12 inches. This is five inches deeper than the goal of sizing the new ducts; however these deep sections only occur over the restroom and part of the corridor. This section of the building does not detract from the overall design to lower from nine foot ceilings to eight and half feet. Using this assumption, the remaining ducts are all controlling with nine inch section depths still allowing the building ceiling depth to be lowered three inches per floor. The rerouted duct system can be seen on the following page along with the design calculations.



SEAN HOWARD
STRUCTURAL



Duct section	Duct Size	Equiv. Dia.	Air Flow	Velocity	Friction Loss
	(in x in)	(in)	(cfm)	(fpm)	(water/100')
A	26x9	16	600	450	0.02
	26x9	16	1600	1200	0.12
	32x9	17.5	2600	1550	0.20
	32x9	17.5	3300	2200	0.46
B	12x9	11.5	600	900	0.10
	22x9	15	1600	1300	0.17
	22x9	15	2600	2100	0.43
C	36x9	19	5900	3000	0.62
E	24x9	16	2200	1600	0.24
F	22x9	15.7	2000	1900	0.30
G	36x9	19	5400	2800	0.58
H	24x9	15.5	1000	750	0.06
	24x9	15.5	2000	1500	0.14
	30x9	17.5	2600	1600	0.08
	30x9	17.5	3600	2200	0.34
I	26x9	16	1000	700	0.05
	26x9	16	2000	1400	0.28
	26x9	16	3000	2100	0.39
J	54x12	26	11300	3000	0.43
K	40x9	19.5	6600	3000	0.60
L	72x14	32	17900	3000	0.30

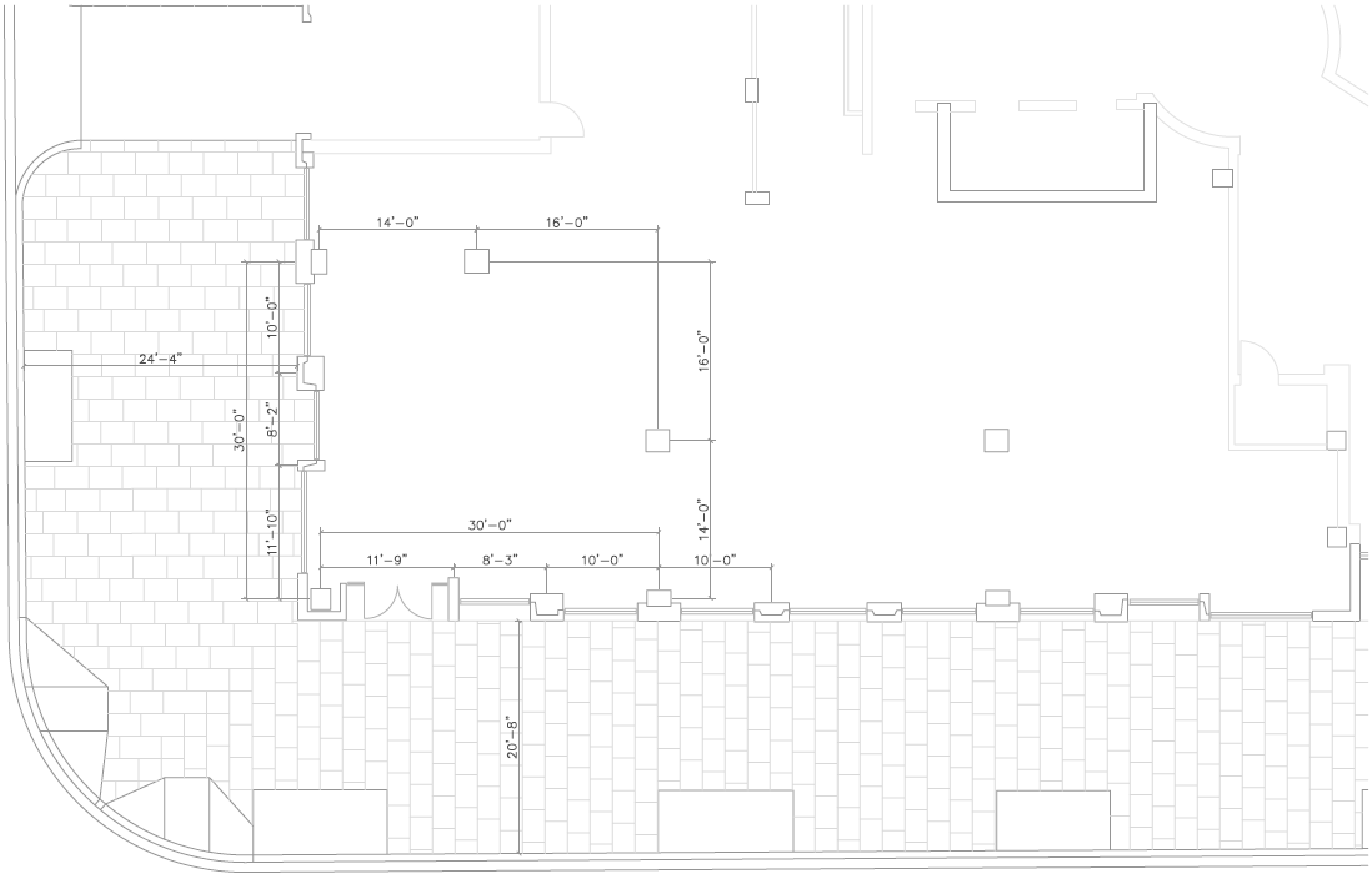
ARCHITECTURAL BREADTH

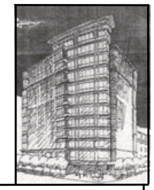
The architectural breadth study on the Executive Tower looks closely at the building's North grounds. Currently the building rests on a sloping terrain that creates an elevation difference of 5' – 6" between the North and South sides. As stated in the proposal, a 12th typical floor is to be added to the Executive Tower in between the floors three and nine. The floor heights of these typical floors are 11' – 6".

It is ideal that the building be designed to gain all of the five and half feet to be saved for developing the 12th floor. However, a few rules were enforced to keep the overall architectural look of the Executive Tower the least affected by the new design. In designing the Executive Tower's first floor the 2003 International Building Code was reference for the building openings, doorways and ramps. The District of Columbia Zoning Regulation was referenced for specific streetscape designing issues.

Three trial sketches were drawn before designing to determine which version would fit best for the buildings layout and overall design. On the next is a drawing of the current first floor plan.

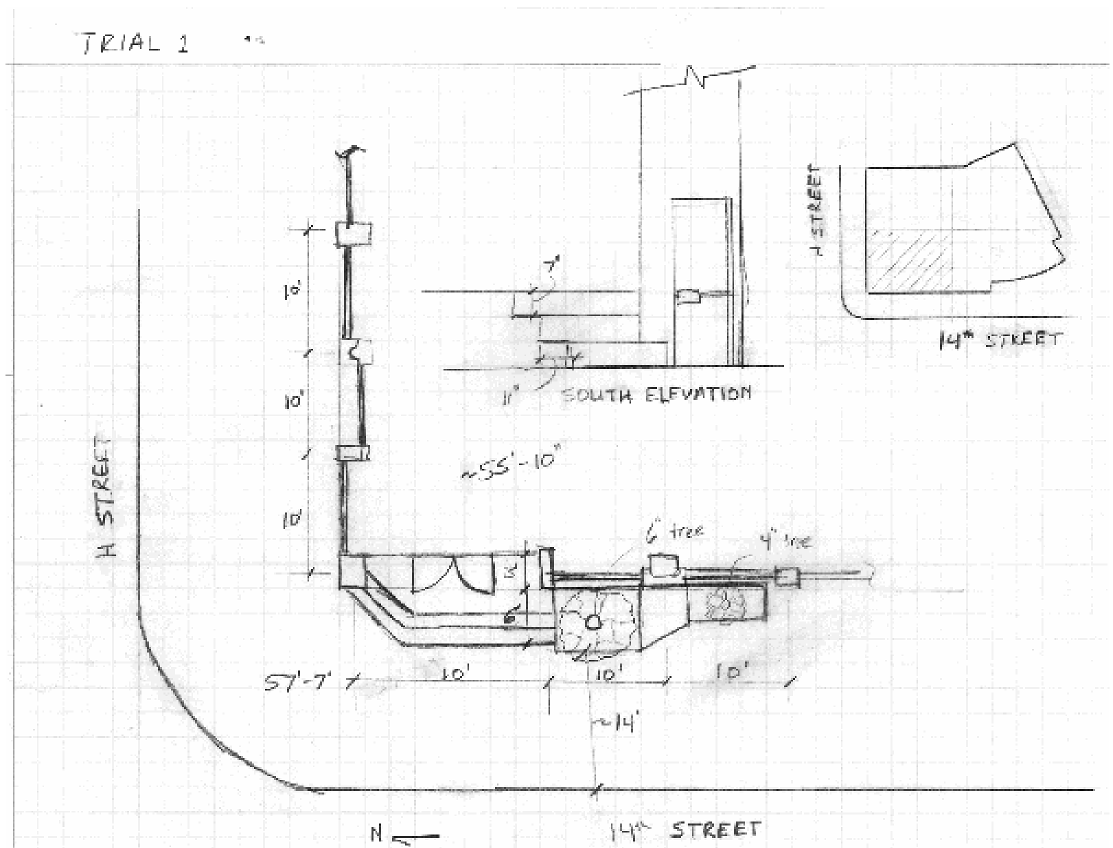
Picture of the North End of the Executive Tower (right). The Picture is blown up to see the retail entrance more easily.


14'–0"
16'–0"
10'–0"
24'–4"
16'–0"
30'–0"
8'–2"
11'–10"
14'–0"
30'–0"
11'–9"
8'–3"
10'–0"
10'–0"
20'–8"



TRIAL 1

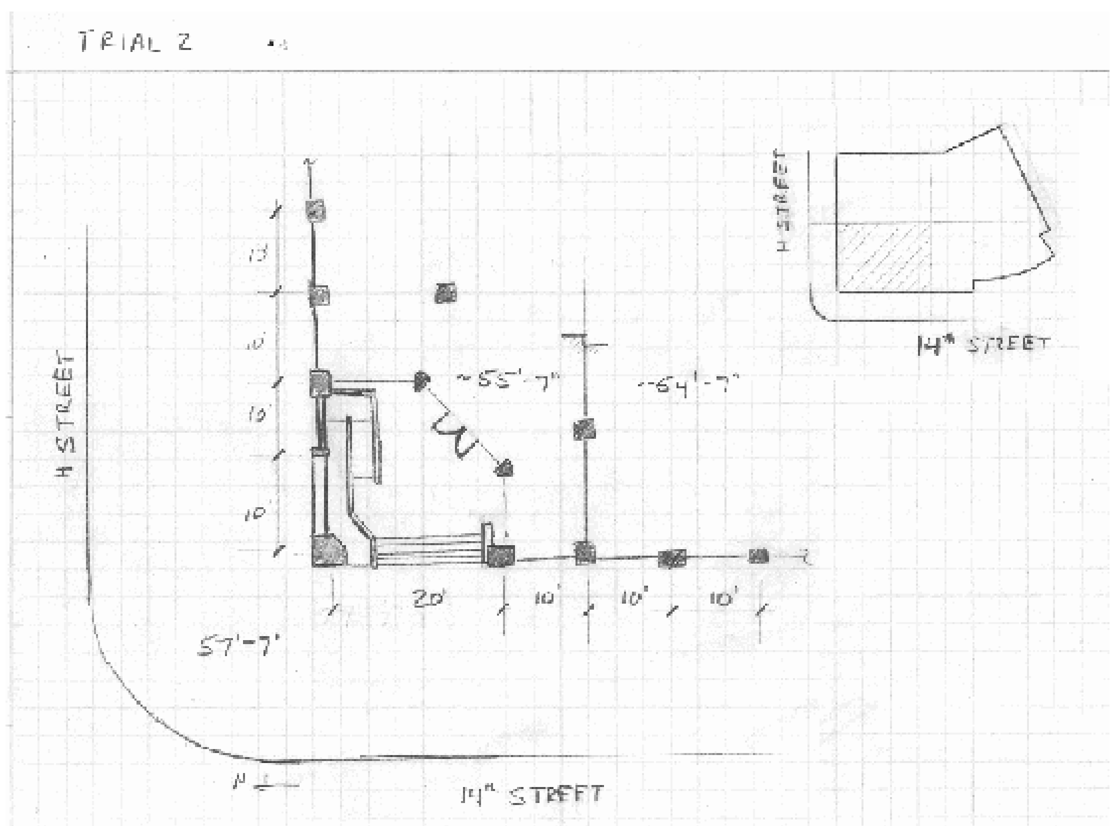
Trial one shows the simplest form where the building will be dropped approximately 2' – 9" while keeping the doorway to Retail 2 in the same place. The stairs were placed 6 ft from the building front leaving roughly 14 ft of space on the sidewalk. A planter of a maximum 5' width according to DC Streetscape code 1106.10 is placed to divert the flow of pedestrians from the steps. This setup would be an acceptable solution; however, this does not leave room for a disabilities ramp and according to 1106.10 of the DC code the depth of the sidewalk is to be taken from the edge of the property line to the curb. Since the steps leading to the entrance way cross the property line, this solution is against DC regulation and must find a different approach to lowering the building.



Trial 1

TRIAL 2

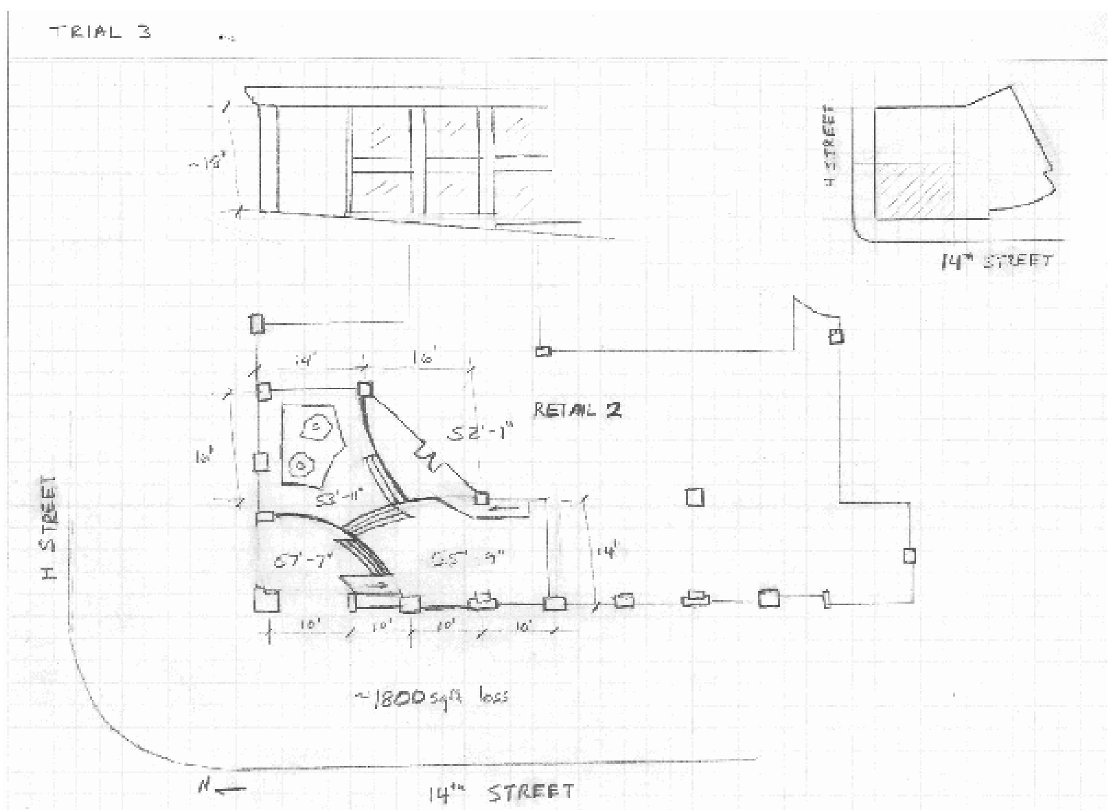
Trial two takes the approach of creating a small inlet to drop the building approximately two feet and allowing the space for a wheelchair ramp. It is a provision of this study to attempt at leaving the overall structure mostly unchanged. In this trial the first nonbearing column is removed to allow more space to create the inlet. The façade on the north wall remains the same and a ramp is constructed to IBC 2003 regulations adjacent to the north wall. In this trial, the majority of the façade remains unchanged and a minimum amount of floor space from Retail 2 is lost. The drawback from using this trial is the possibility of the entranceway feeling too low as people walk down the stairs.



Trial 2

TRIAL 3

Trial three takes into consideration a wide open atrium space to enter Retail 2. The space removes no columns from the original design. People enter through at the corner where previously window panels were. The plan takes advantage of using all five and half feet the elevation has to lower the overall building height by creating a three tier gradual step down system. By doing this, less material can be taken away in the other studies making the proposal more feasible. Handicap ramps can be constructed between the first and second tier and the second and ground level to allow access to Retail 2 to all people. A small green space can be built in the atrium on the third level to create a friendlier environment. In using trial three, the entrance height would approximately be 6' – 10" and this would be in violation of IBC provision 1003.2-ceiling height. Thus as seen in the section sketch, a space from the second floor would need to be remove to allow headroom at the entrance.



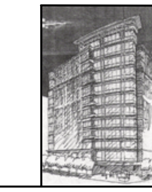
Trial 3

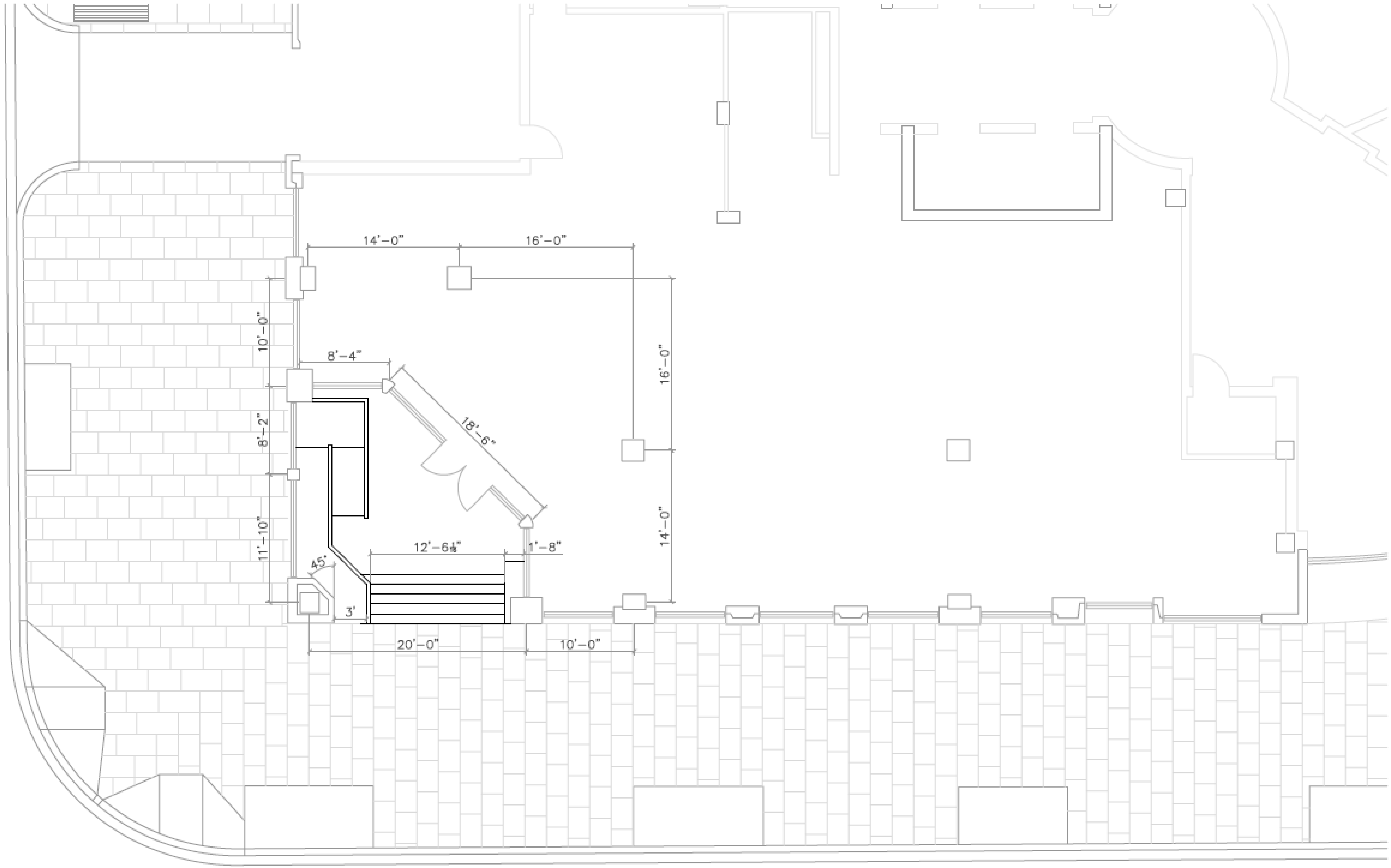
DESIGN SUMMARY AND CONCLUSION

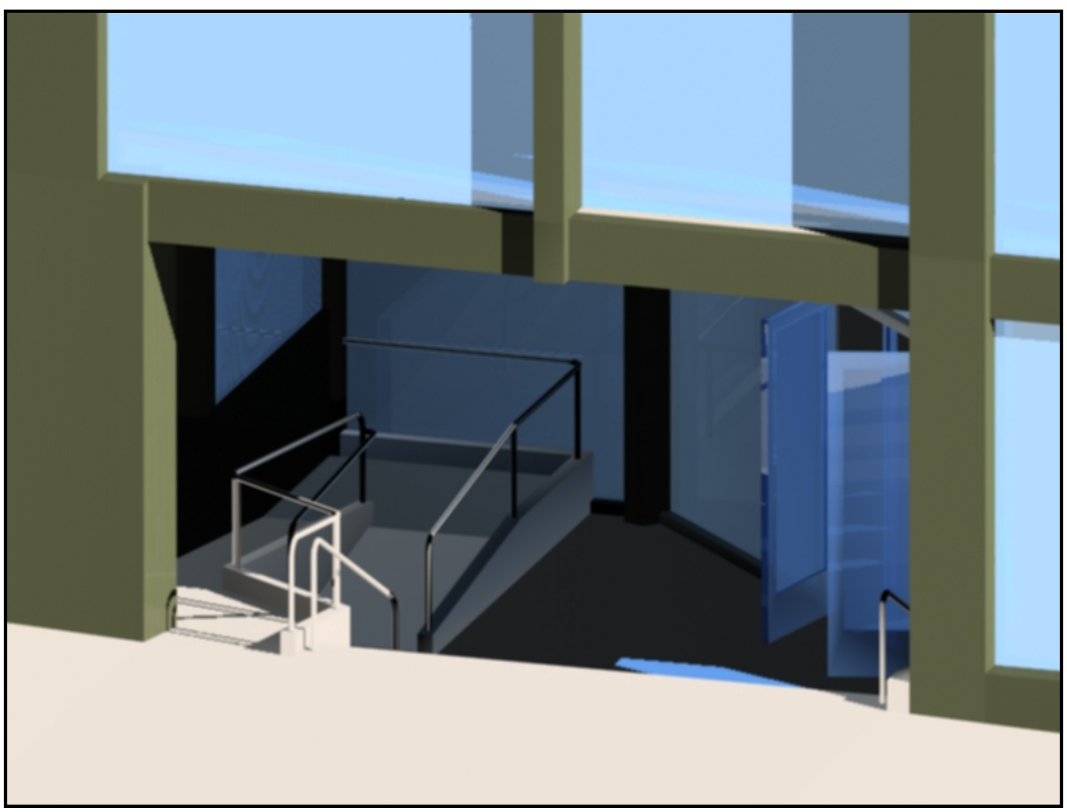
The loss of rentable space from Retail 2 is approximately 400 sqft whereas using trial three would result in a loss of over four times that at 1,800 sqft; a total of 900 sqft from Retail 2 at $38 per sqft and the equivalent space from the second floor office space at $47 per sqft creating a loss of monthly revenue of over $76,500. The rent lost from the area in trial two resulted in approximately $16,700 per month. Aside from the lost funds, construction of trial three would probably be too large scale and distracting from the main entrance on the south side of the building.

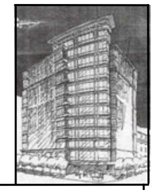
After review the three choices in the design of the first floor entrance it was decided to use trial two for the starting design. Trial two fits the purpose of lowering the building at least two feet without retracting too much from the overall design. The details for the full design are as follows.

The building is lowered three feet below its original level. The steps are to DC code at a 12 inch run by 6 inch drop. The wheelchair ramp switches back (as originally expected) to allow for a 12 to 1 grade. The left side the wall remains unchanged from the original design. Only the non-loading bearing column 10' from the corner was removed to make enough room for this design. The floor plan for this design can be seen on the following page including a 3D rendering on the next page.




14'–0"
16'–0"
10'–0"
8'–4"
18'–6"
16'–0"
8'–2"
11'–10"
14'–0"
45°
1'–8"
3'
20'–0"
10'–0"





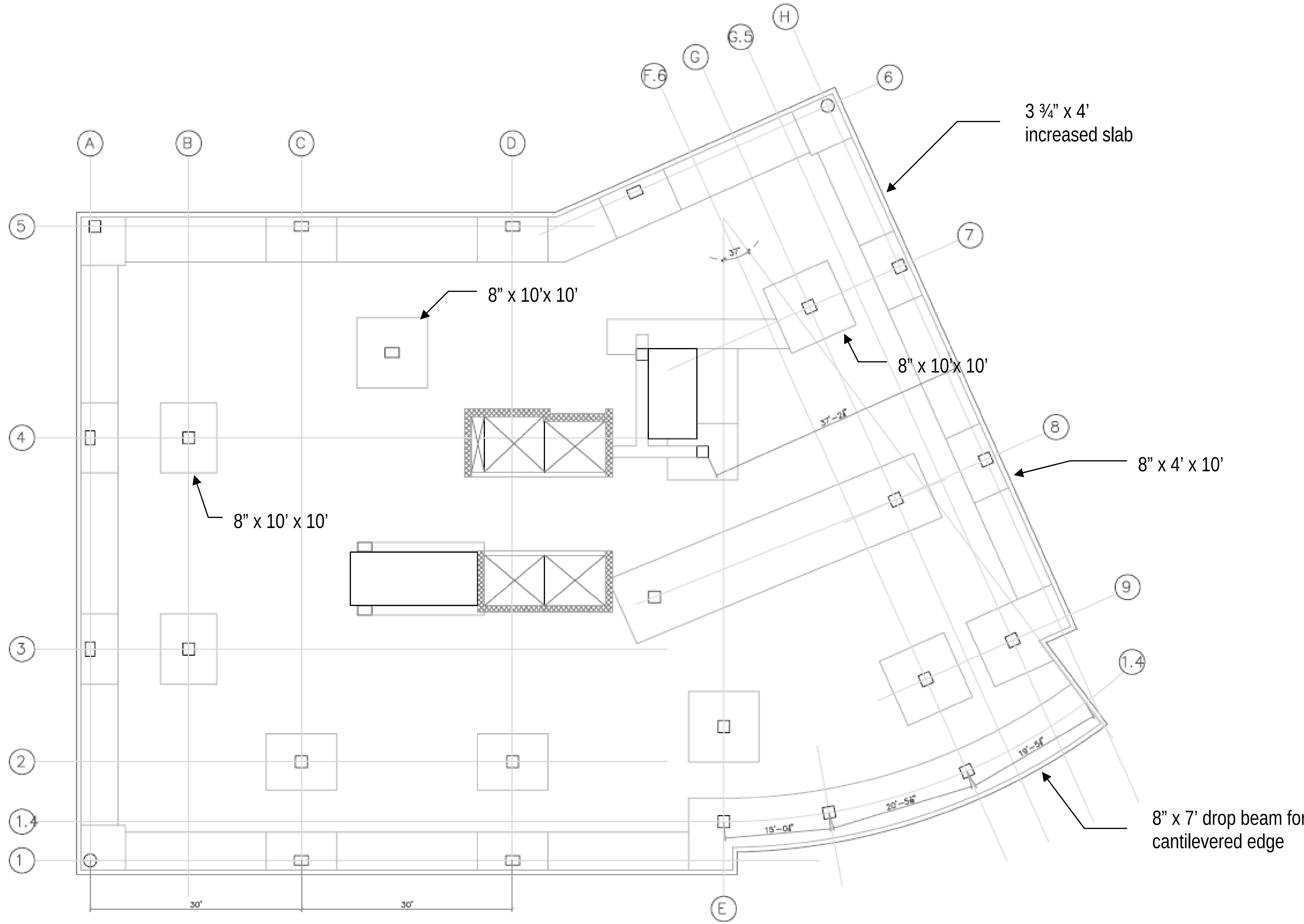
POST TENSIONING

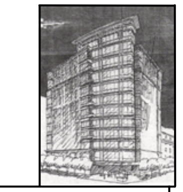
The third and final step in completing the proposal is the application of a two-way post tensioned slab in order to reduce the depth of the framing system for the Executive Tower by three inches. The existing system is an eight inch two-way flat slab with eight inch drop panels at all column locations. An increased slab thickness of three and three quarter inches acts at a perimeter beam around the entire building except for in one place. The curve perimeter section is supported by three columns with a 19 foot cantilever on the south end. This section of the slab has an eight inch by seven foot drop beam added to the thickness of the slab. A detailed drawing of the structural floor plan can be found on the following page (24).

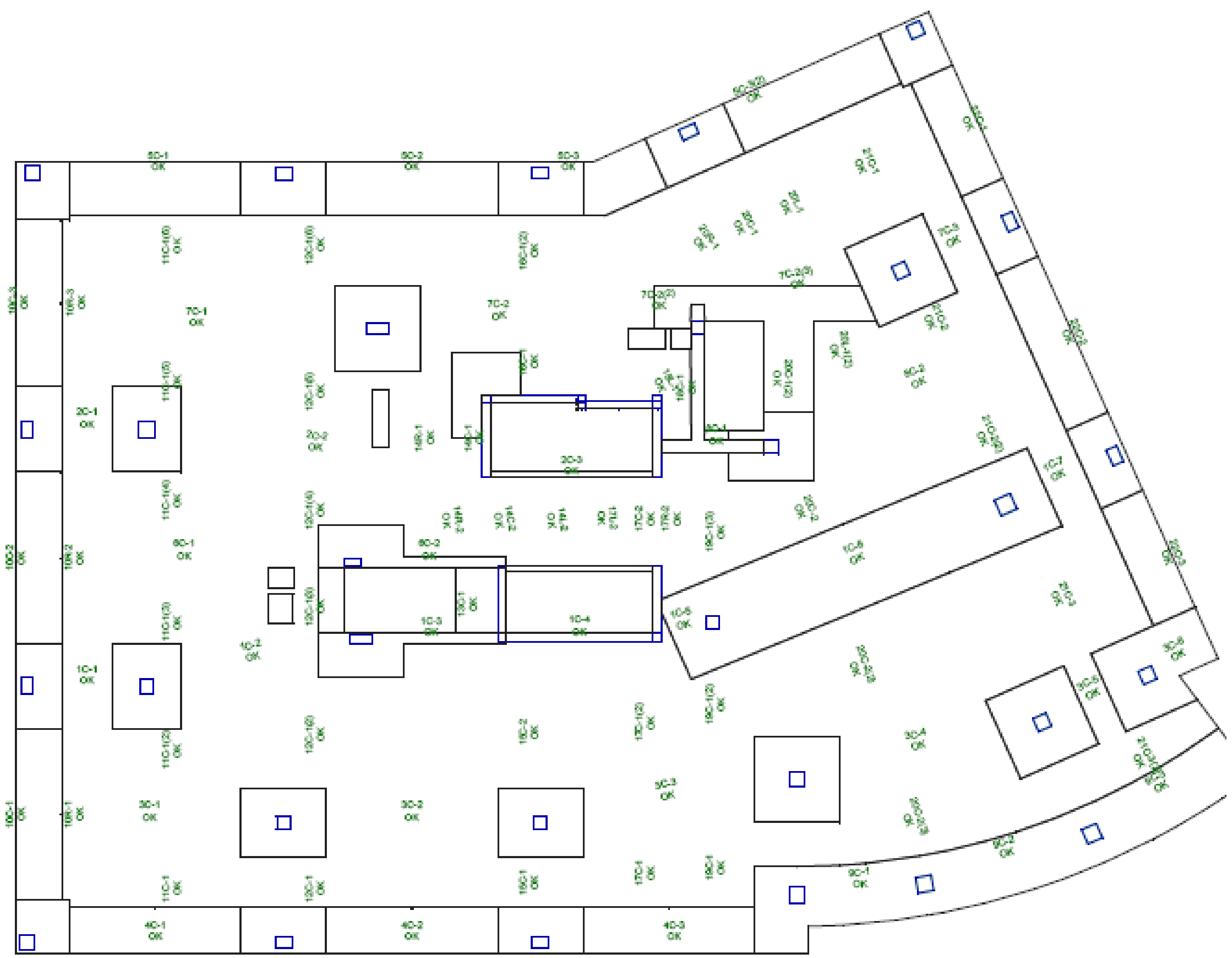
In order to achieve the goal of a three inch reduction, it was decided as of Technical Report 2 to convert the current system to a two-way post tensioned slab. In order to analyze the post tensioning due the Executive Tower's disorganized column layout, a structure program that undertook a finite analysis was used.

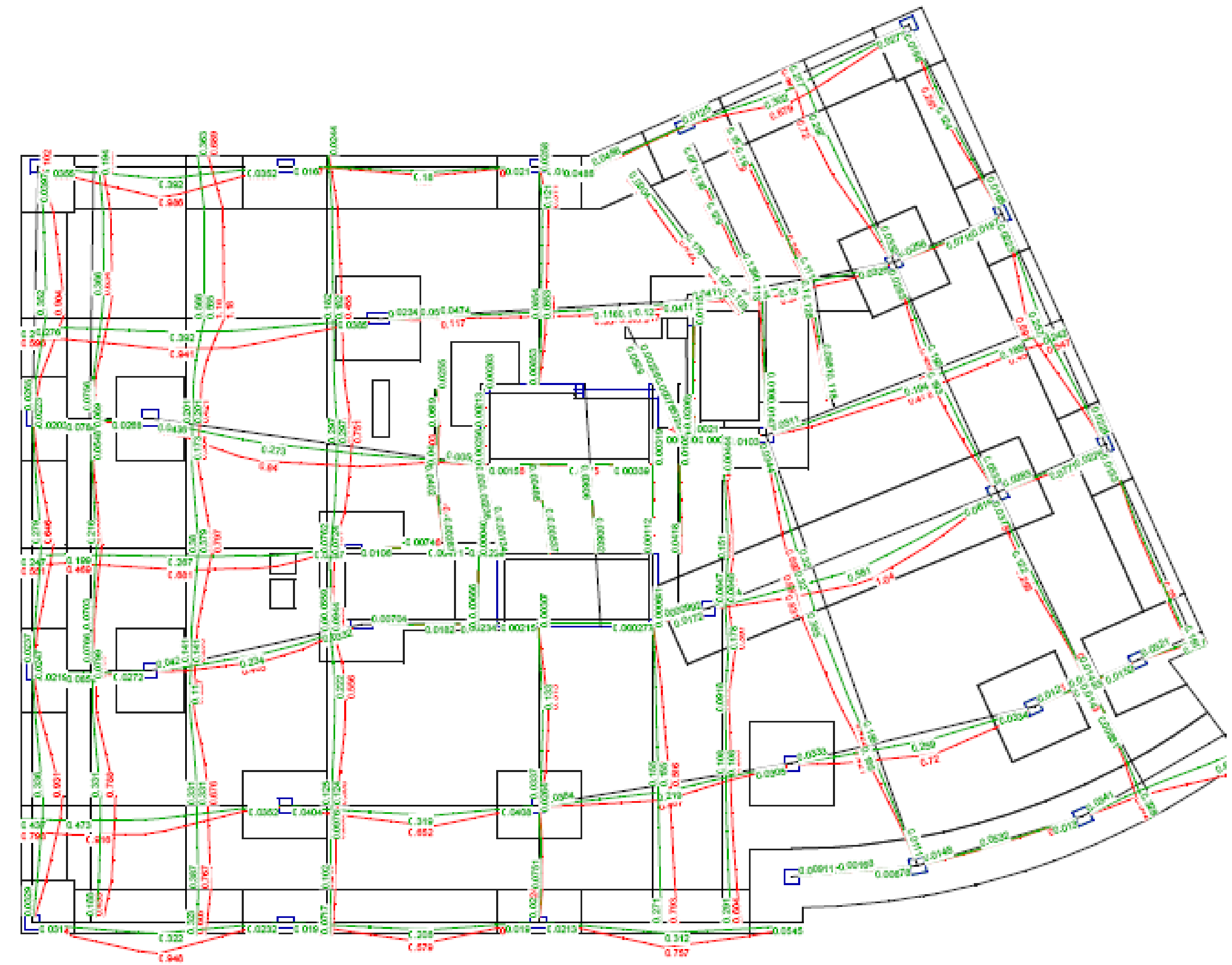
The Executive Tower was constructed in RAM Concept by developing the original system without any post tensioning tendons and then allowing it to run its analysis. The results were conclusive, the original system worked for the most part in RAM Concept. The areas of failure are due to sections of the slab that were reinforced more because the #4 @ 12" web was insufficient. The results of this analysis can be seen including the deflections on page 25. This is in agreement with the findings from Technical Report 1.

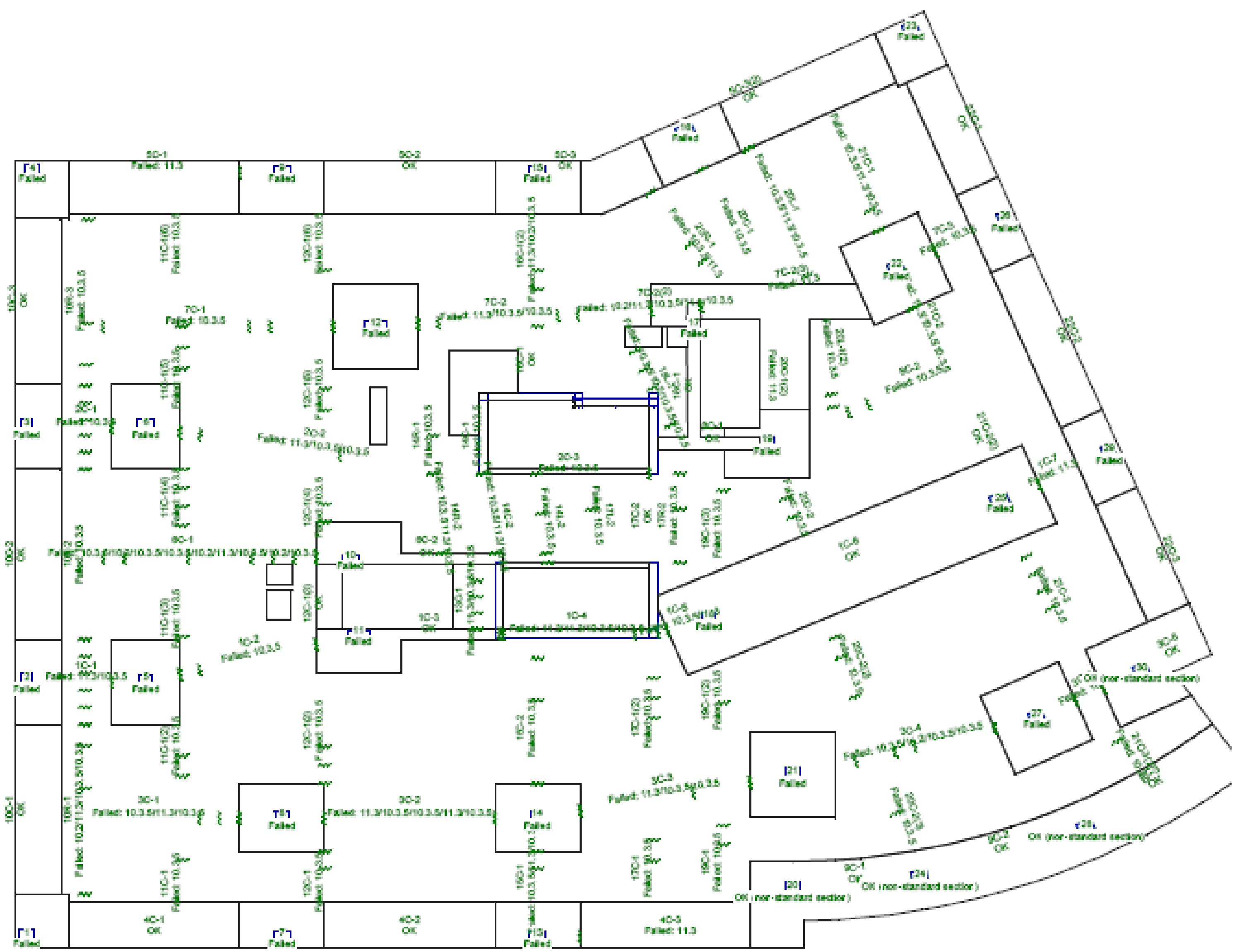
On page 26, RAM was then run with a flat slab system with the slab reduced by three inches proving the application of a post tensioning system is necessary to achieve the goal of a thinner slab. Note the slab fails in multiple places and where it does not fail the deflections in the five inch slab are considerably greater, some as high as five inches.

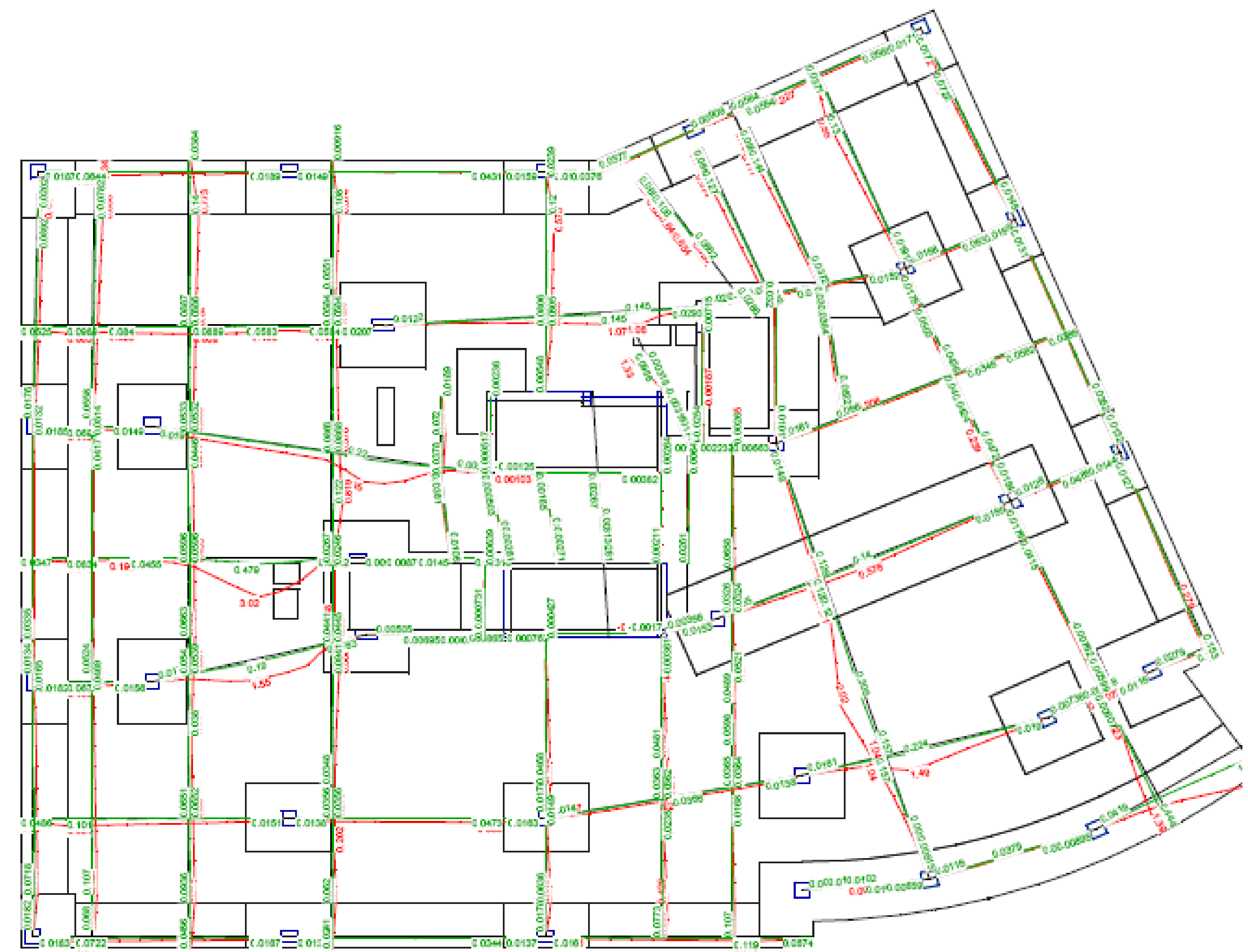

3 ¾" x 4'
increased slab
8" x 10'x 10'
8" x 10'x 10'
8" x 10' x 10'
8" x 4' x 10'
8" x 7' drop beam for
cantilevered edge
30'
30'
A
B
C
D
E
F.6
G
G.5
H
1
1.4
2
3
4
5
6
7
8
9
1.4









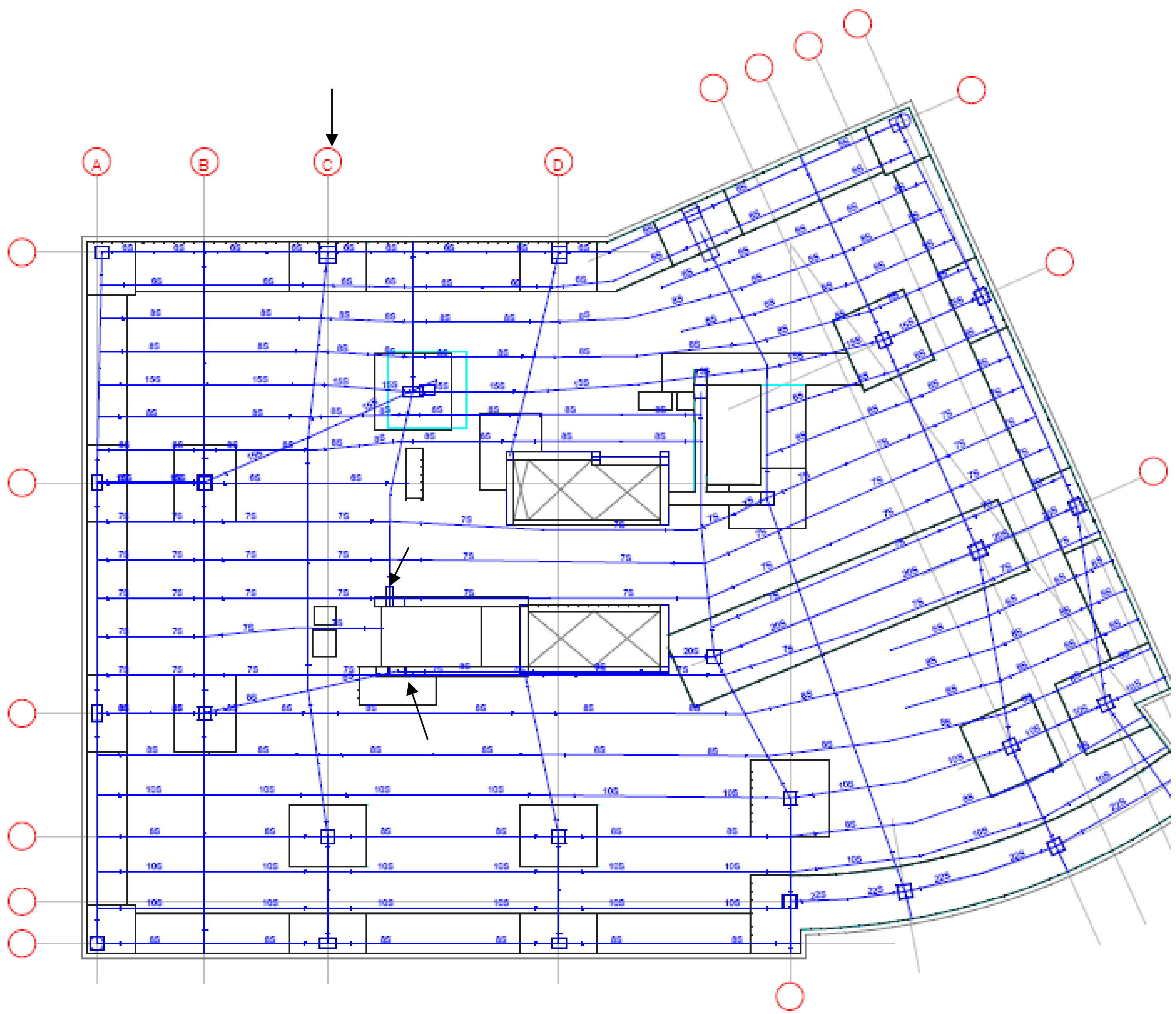


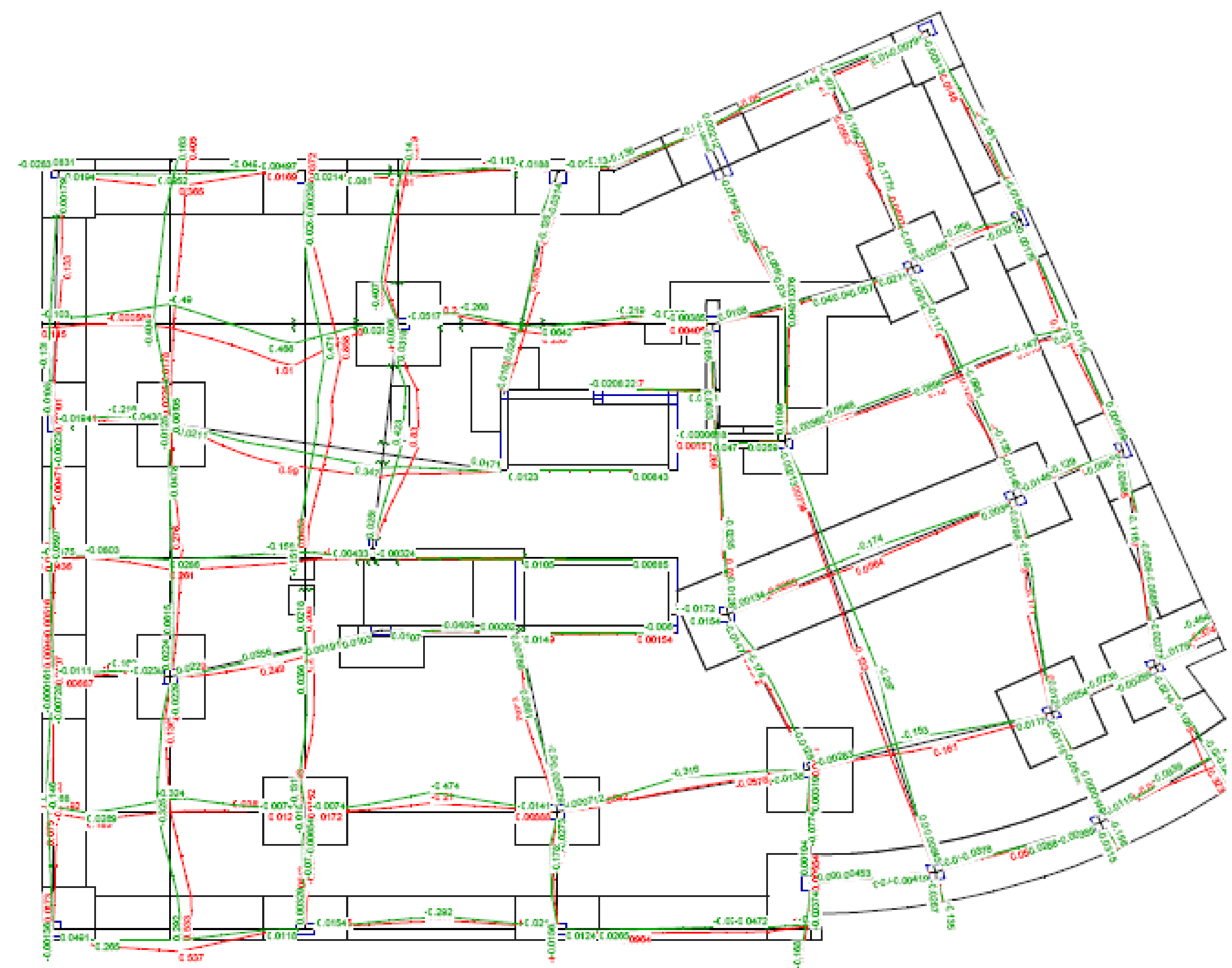
TRIAL 1

To develop a workable post tensioning system, the column strips need to be laid out meeting as many columns as possible. In the case of the Executive Tower, the columns do not line up along one column line grid. The column strips needed to be skewed in several places. The end result was a tendon layout just as irregular. The longitudinal tendons were bundle in groups of 15 making the longitude direction the strong direction and the distributed tendons in the latitude. Running the strong tendons in this direction proved to be next to impossible. First, the tendon along column line C was too long of a distance to make the section work (see next page). It was impossible to trend the tendon to the right of the opening to the two columns indicated by the arrows due to the stairwell in between them, so two tendons (out of plane of the latitude direction) were laid out span from one column to the other with the low point of the tendon underneath the low point of column line C in an attempt to help support this section of the slab. After extending 15 strands at both of these locations, the slab continued to fail. Any more strands at these points and the slab would have been compressively stressed to the maximum resulting in failure again. Second, many of the longitudinal tendons take too steep of directional changes making it less effective and constructible. It is ideal the tendon stay perfectly straight to properly jack the tendons to their necessary stresses. Third, the distributed tendons in the latitude direction are spread out evenly but some of the spans were too long to work under service loads; also, the latitude tendons were unable to be design to effective following the curve of the building.

The advantage of constructing this layout was the discovery that a post tension is ideal for the Executive Tower's unique column layout and necessary in cutting the slab thickness. Also shown below is the deflection plan with this post tensioning layout on page 28. Even though some spans failed and were unable to be constructed to pass, most of the floor plan was acceptable and the largest deflection was 1.01 inches on a 37 foot span calculating a deflection ratio of L/439.

Due to the orientation of the slab openings and the column layout it was decided to try running the tendons in the opposite directions. By doing this, the longitude tendons (now the distributed tendons) can be stopped at the elevator cores leaving the slab in the corridor without post tensioning.


A
B
C
D



TRIAL 2

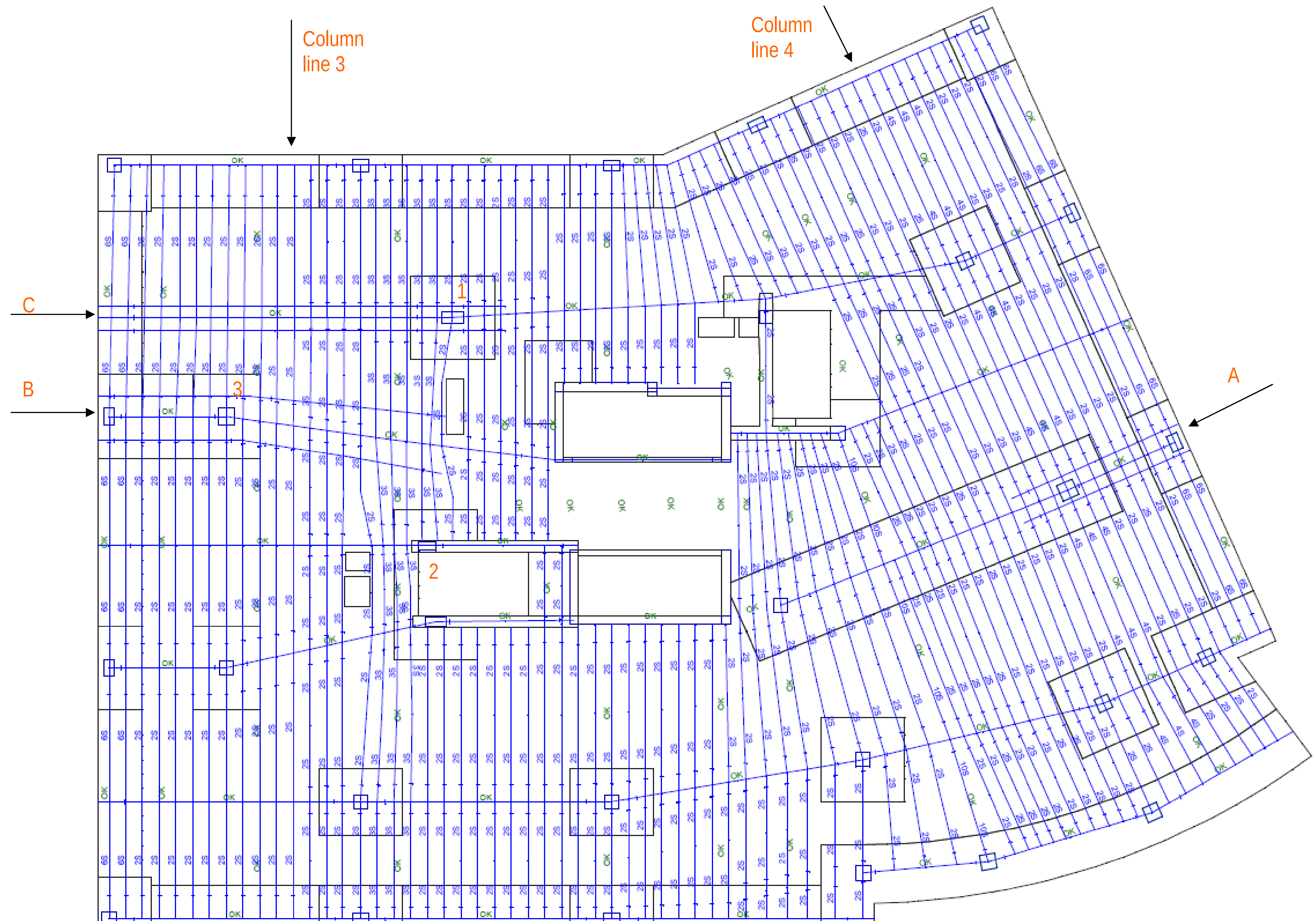
In trial two, the tendons were rotated 90 degrees to attempt to create shorter and straighter column strip spans, a tendon free corridor and enforce a deflection criterion of L/360 or better. With the exception of a few spans that needed a creative design solution, the trial two created a significantly better layout than that of trial one. The Trial two plan is on page 30.

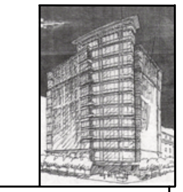
Trial two is a more realistic construction plan compared to trial one. The strong tendons run in the latitudinal direction which has few turns and produces natural breaks in the building structure to anchor tendons. Only four latitude tendons stretch the entire length of the building. The remaining five are anchored along the right side of the two elevator cores. This creates a smoother transition in designing for the 24 degree skew the building plan takes in the middle of the floor plan and allows the use of fewer tendons in slabs that do not required large stress to be sufficiently supported. In trial two by spanning the strong tendons in the latitudinal direction, the strong tendons are now in line with several beams in the Executive Tower floor plan making it ideal for these beams to support the distributed tendons in the other directions. The beams at the stairwells are great places to stop distributed tendons. Most of the MEP openings in the slab are oriented parallel to the distributed tendons. Having these openings in the same direction makes it easier to spread tendons to still support the slab without disrupting the MEP duct work.

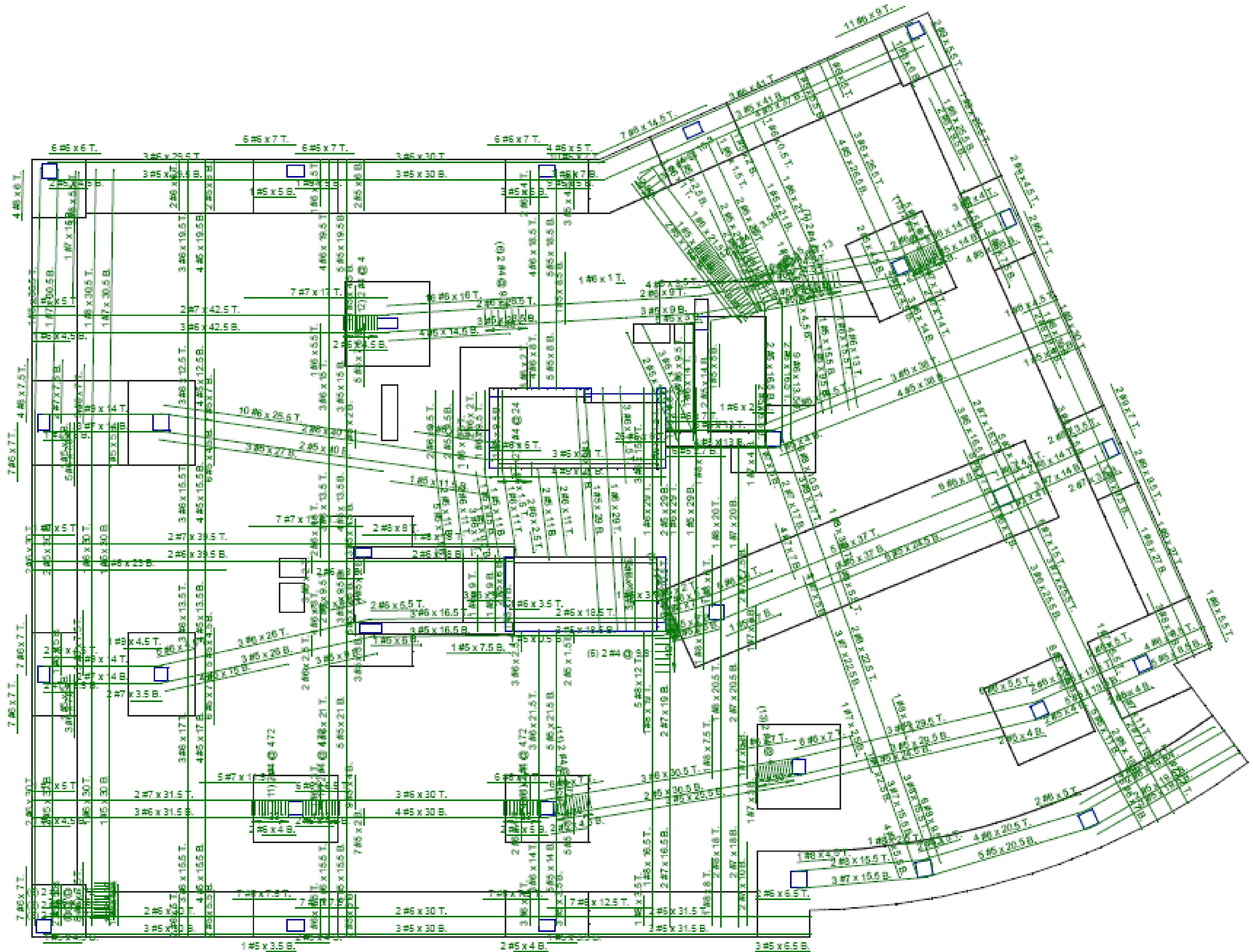
In the process of laying out the column strips, it was assumed the edge beams around the perimeter would act compositely with the slab creating a tee beam. Also due to the Executive Tower's column arrangement, when designing the column strips for the distributed direction (longitude) it was assumed the columns strips along column lines three and four would act as equivalent frames. The column strips were drawn perfectly straight stopping at each strong tendon that runs the in latitude direction to insure the slab is checked at each span of the distributed tendons.

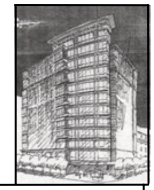
A few disadvantages are places in the slab where even with substantial post tensioning and reinforcement would still fail. These areas are discussed further in the design section.

SEAN HOWARD
STRUCTURAL









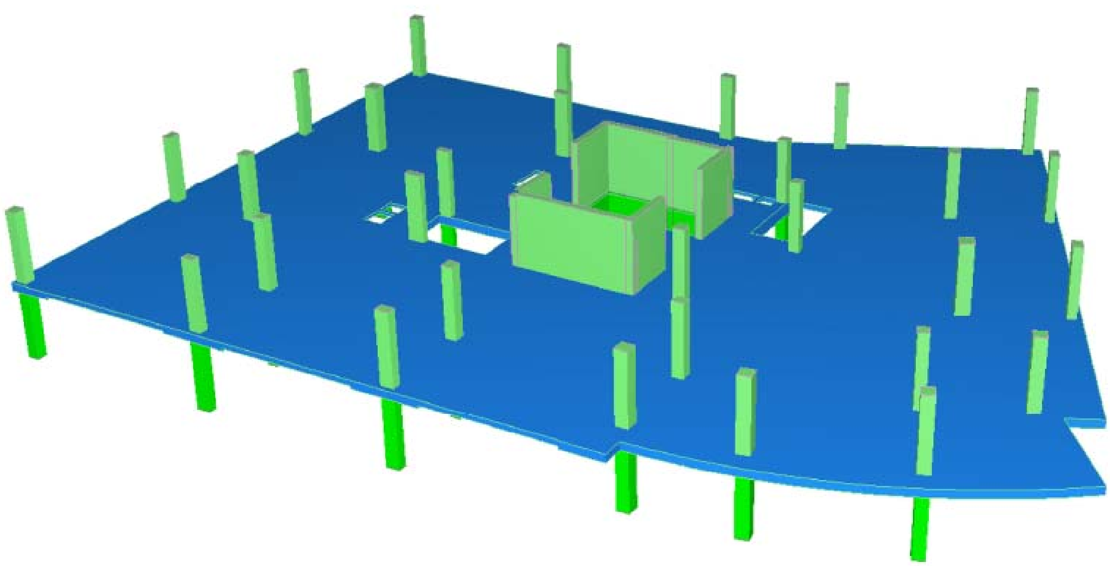
DESIGN

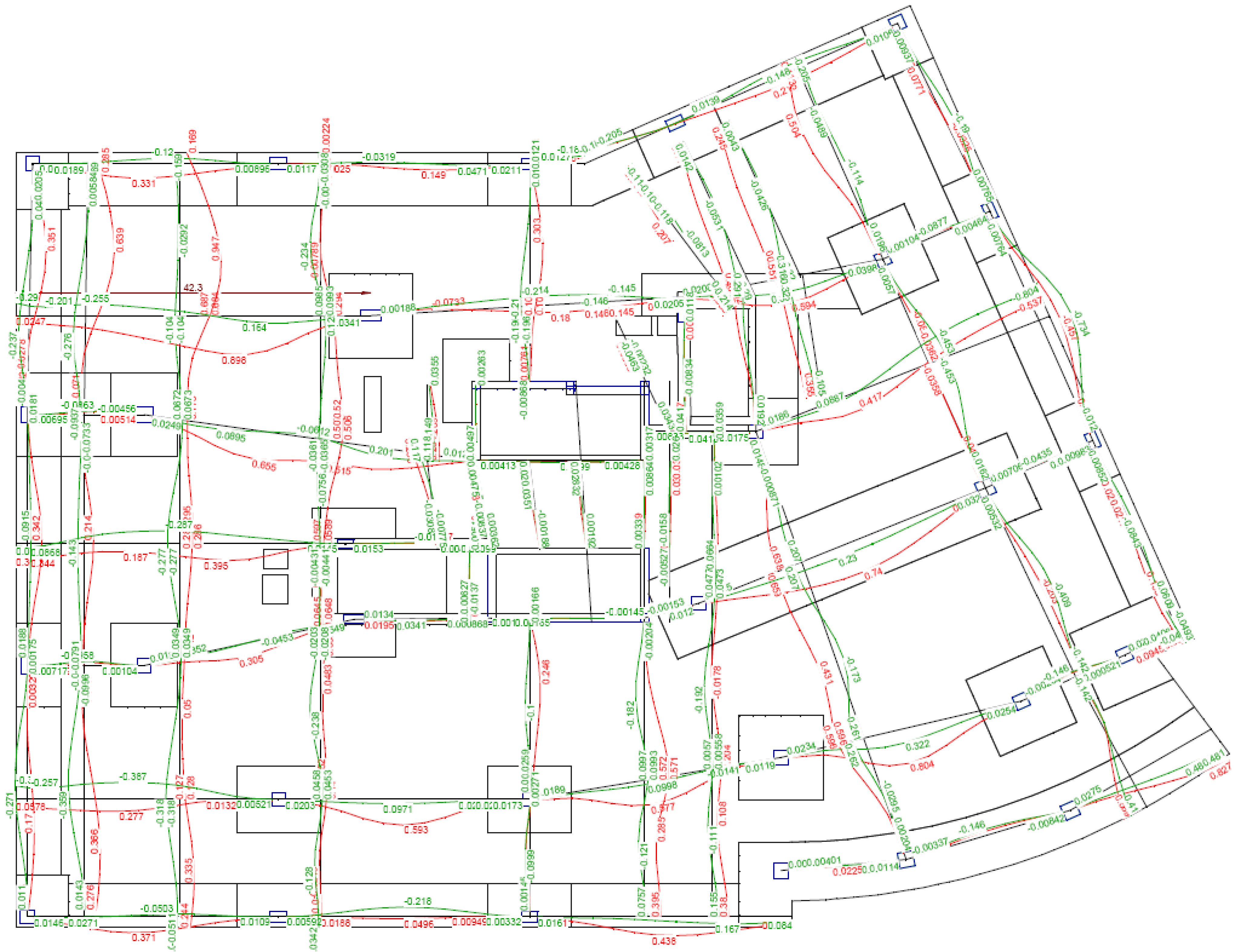
After designing the second post tension plan in RAM, it was found that the initial goal of reducing the slab to five inches was too aggressive. With the thickness reduced this much, the slab still continuously failed in similar locations as trial one. It was decided to only reduce the slab thickness by two inches. This however, does not sway opinion of using trial two over trial one. Trial two still proves to be the more suitable design solution for the Executive Tower.

Three areas initially caused problems in the design phase in the RAM Concept model. These areas are marked by the arrows on the previous page (30). Section A is a 10 foot span at the end of a 37 foot span. Along the 37 foot span is an eight inch drop panel to help control the deflection in this area. Without tendons in this section, the 37 foot span would deflect up to 0.98” causing the 14 foot span to have an upwards deflection of 0.3”. Due to the large deflection over a short distance, the slab was cracking in both tension and compression at the edge of the drop beam. The first design solution was to add more tendons at this area to help carry the loads. However, after extending 27 tendons, the slab would begin to reach its pre-compressive limit and would fail. As a result of this, the main tendon was cut down to nine strands and set at its maximum uplift balancing load for the 37 foot span and inverted over the 14 foot span developing a downward balancing load. This caused a combination of uplift for the 37 foot span and a downward loading for the 14 foot span resulting in an improved deflection over the 14 foot span however still failing. Six strands were then run over the 14 foot span and anchored just after the column to increase the downward load in this area. The results were verified by the deflection plan now show only -0.74 and +0.044 which has a control deflection of L/600 between the two of them.

A similar area of failure occurred at section B indicated by the arrow on the previous page. This area was deflecting too much from the long span of 40’ compared to the short span of 14’. Similarly, the main tendon was reduced to 10 strands and two four strand tendons were placed on either side creating uplift in the long span and downward load in the short span. The result improved the short span but still failed, plus the reduction of tendons in the long span was now causing flexural failure. To fix the short span, the slab was increased in thickness equivalent to the edge of 9 ¾". The new section passed and stiffened the connection of the column and long causing it the long span to deflect less, but still fail in flexural. A creative solution to this involved revising the distributed tendons in the longitudinal direction. Fifteen strands spread evenly at one foot spacing were altered to span from column 1 to column 2 instead of resting on the main tendon in the 40 foot span. The result of this is an uplifting point load at these crossing tendons equivalent to their balancing load times the width of the 40 foot span column strip which is 13.5'.

Section C was failing in deflection as a result of a 44 foot span. The conclusion was to apply the same solution of section B and have the distributed tendons span from the edge beam to column 3. The result for both sections was a deflection limit of L/732 and L/587, respectively.





PUNCHING SHEAR

Punching shear in the Executive Tower was found to be the controlling factor in determining the size of columns. The punching shear equation for a prestressed concrete was used from ACI 318-05 11.12.2.2, without being in excessive of 11.12.3.1 (both shown below).

$$V_c = (\beta_p(f'c)^{(1/2)}+0.3f_{pc})*b_o*d+V_p \qquad 11.12.2.2$$

$$2*(f'c)^{(1/2)}*b_o*d \qquad 11.12.3.1$$

The results from this spreadsheet can be found in Appendix E, but three columns are shown below and discussed. In the existing structure, shear reinforcement was not necessary since at every column location had 16" of concrete due to drop panels. Punch shear was checked however to determine if this holds true for 14" of concrete. In all but three columns, punch shear passed without the use of steel reinforcement. Columns 1, 8 and 24 were test without steel reinforcement and failed mostly by only a few kips. The formula was then calculated again this time factoring in #4 bars at six inch spacings, which was found to be acceptable.

	Size (in x in)	d (in)	b_o (in)	f_{pc} (psi)	f'c (psi)	α_s	β_p	Vc (lb)	ØVc (lb)	Vu (lb)	check?	Vs w/ #4@6'	new ØVc (lb)	check?
1	20x20	14	58	260	4000	20	3.5	102711	77033	86300	no good	24000	95033.1	OK
8	20x20	14	136	225	4000	40	3.5	240839	180629	181000	no good	24000	198629.3	OK
24	24x24	14	152	200	4000	40	3.5	269173	201880	207000	no good	24000	219879.8	OK

LATERAL DESIGN

The shear walls were developed using the same method from Technical Report 3. Six shear walls are located enclosing the elevator core and five frames lining the perimeter of the building due to the thickened slab acting as a perimeter beams. The frames were modeled in STAAD with 100 kips point loads at each floor to find the relative stiffnesses. One hundred kips virtual loads were used instead of one to get a deflection off of STAAD with two more significant figures. The shear wall stiffnesses were found through the following equation:

$$R= Et/(4*(h/L)^3+3*(h/L))$$

Through an excel spreadsheet the shear walls and frames were all simultaneously calculated for direct shear and torsion. These loads were calculated for each floor. The loads per floor per element were then divided by the relative stiffness for those points to find the story drift and building drift. By designing this way, it is assumed the frames and shear walls will be taking all of the lateral loads, and as a result, the concrete strength for the shear walls needed to be increased to have a building deflection of less than the L/400 limit. In reality, the slab and all the columns would contribute to resisting the lateral loads which is why the shear walls on the original plan were sized smaller.

POST TENSION CONCLUSIONS

It has been found that converting to a post tensioned floor system was the correct process in order to meet the proposal. However, to much disappointment, reducing to a five inch slab proved inadequate to support the floor in flexure or deflections. Punch shear was not checked for a five inch slab, just a six inch slab, but by observation many more of the column in Appendix E were within a few kips of failure. Had the slab been kept at five slabs, punch shear would be become a reoccurring problem in several columns. As for the slab itself, accept in the areas discussed above the slab was sufficiently supported with one strand per foot distributed tendons in the longitudinal direction and strong tendon in the latitudinal direction mark on the tendon layout on page 30.

CONCLUSION

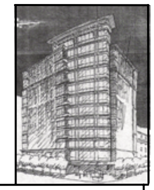
COST ESTIMATION

A cost estimation was calculated to compare reasons for progressing with the construction of a more complex framing system. The values for labor and material were found from MS Means 2005. RAM Concept automatically calculates building materials quantities for concrete, post tension and steel reinforcement. Using these numbers an estimate of $170,000 was found for the flat slab system per floor and roughly $160,000 per floor to convert the system to post tension minus the two inches of concrete. However, post tensioning is a slower process and was estimated to cost about $100,000 from general conditions in addition to the cost per floor. Therefore, the cost for post tension is roughly $90,000 more per floor than the flat slab system.

The Executive Tower rents per month at $47 per sqft of office space and $38 per sqft of retail space. With the addition of the 12th floor, the Executive Tower collects $552,250 per month minus the $16,700 lost from the architectural breadth study. The total structural difference can be found by multiplying the $90,000 per floor by 12 floors to yield $1,080,000. The number of months to pay off the cost is equivalent to $1,080,000 total cost divided by the $535,550 per month equaling 2.02 months.

Flat Slab	Units	Materials	Labor	Equip	Total w/ O&P	Amount	Schedule	Cost
Concrete cost with forms	CY	190	90.5	16.5	380	354.8		134824
Post tension	LB	0.46	0.7	0.03	1.85	0		0
Steel reinforcement	tons	850	305	0	1475	23.17		34175.75
General condition	days						±0	0
								$168,999.75

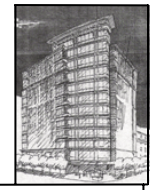
Flat Slab w/ Post Tension	Units	Materials	Labor	Equip	Total w/ O&P	Amount	Schedule	Cost
Concrete cost with forms	CY	190	90.5	16.5	380	308.4		117192
Post tension	LB	0.46	0.7	0.03	1.85	12510		23143.5
Steel reinforcement	tons	850	305	0	1475	12.56		18526
General condition	days						+30	100,000
								$258,861.50
							Difference of	$89,861.75



BUILDING HEIGHT SUMMARY

The original building height for the Executive Tower was 128' – 4", just 1' – 8" short of the height restriction set by the Washington DC Zoning Regulations. Since this 130 foot height is measured from the north side of the building (the shorter side), the Executive Tower had the capability of be lowered up to five and half feet by making it even grade with the south side. After evaluated a few sketches and fully designing one, it was determined that lowing the building only three feet was most suitable for the Executive Tower's overall look, square footage lost and head room regulations. Through the study of the mechanical duct work on each floor, ceiling space depth was able to be reduced by three inches per floor by rerouting and optimizing the duct layout on each floor. In the structural depth study, the task of design the Executive Tower as a fully post tension building was adopted with goals of reducing the slab from eight inches to five inches. However, after the constant failure of the first trial and the troubles met in the second trial, it was decide to abandon this goal and design the slab to be six inches thick. These numbers were plugged into an Excel spreadsheet seen below and found the new building height to be 131' – 10"; 1' – 10" higher than the DC Zoning Regulations will allow.

	Orginal Height	Arch. Breadth	Mech. Breadth	Post Tension	New Floor Heights	New Building Heights	Under 130'?
12	-	-	- 3"	- 2"	11' - 1"	131' - 10"	No Good
11	11' - 6"	-	- 3"	- 2"	11' - 1"	120' - 9"	
10	11' - 6"	-	- 3"	- 2"	11' - 1"	109' - 8"	
9	11' - 6"	-	- 3"	- 2"	11' - 1"	98' - 7"	
8	11' - 6"	-	- 3"	- 2"	11' - 1"	87' - 6"	
7	11' - 6"	-	- 3"	- 2"	11' - 1"	76' - 5"	
6	11' - 6"	-	- 3"	- 2"	11' - 1"	65' - 4"	
5	11' - 6"	-	- 3"	- 2"	11' - 1"	54' -3"	
4	11' - 6"	-	- 3"	- 2"	11' - 1"	43' - 2"	
3	11' - 6"	-	- 3"	- 2"	11' - 1"	32' - 1"	
2	11' - 6"	-	- 3"	- 2"	11' - 1"	21' - 0"	
1	13' - 4"	- 3' - 0"	- 3"	- 2"	9' - 11"	9' - 11"	
	128' - 4"						



FINAL REMARKS

Though the Executive Tower's proposal to add a 12th floor typical to floors three through nine seemed to fail, the building is still very capable of meeting its 130' height limitation. The building height of the new system is only off by 1' – 10". The story height per floor is now 11' – 1". To reduce the building height less than 130' at this point only requires the floor to ceiling height to be two inches lower per floor. As a result, instead of the tenants have 9' – 0" ceilings, they will have 8' – 10". This would have to be a decision made by the architects and owners of the building to determine if lowering the ceiling heights is what their tenants will want, but if by doing this the owner gains over $500,000 per month, in my opinion it would be well worth it.

ACKNOWLEDGEMENTS

I would like to thank Renee Gibbs and Jason Lee, the people that first helped me and eventually allowed the access to use the Executive Tower and the plans for my thesis project. The staff of Mesen Associates for giving me the work related background through a summer internship. Thanks to Dr. Hanagan, my instructor and thesis advisor, for her advice when deciding whether to design steel or a post tensioned system. As well as the additional comments of not procrastinating if designing the post tensioned system; this was not taken lightly. To Capt. Jess Janks of the 502nd Out-of-State Pen Pal Brigade for her master editing skills. To my family for their support. To Becky for putting up with me and supporting me for the past few months. And finally to all the people that felt it necessary to pull me out of thesis when I needed but didn't want to admit it.

Thank you

REFERENCES

CostWorks 2005 Building Cost Data. Kingston, MA. R.S. Means, incorporated 2005.

ACI 318-05. Farmington Hills, MI. American Concrete Institute.

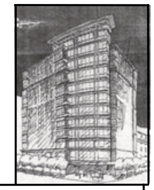
Manual of Steel Construction, Load and Resistance Factor Design. Chicago, IL. American Institute of Steel Construction Inc.

Downtown Streetscape Regulations, 2001. Washington, DC. Department of Transportation.

District of Columbia Municipal Zoning. Washington, DC. District of Columbia Office of Zoning.

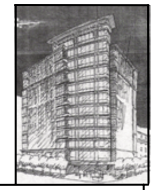
Naaman, Antoine E. Ann Arbor, MI. Prestressed Concrete Analysis and Design, Second Edition, 2004.

International Building Code 2003. Falls Church, VA. The International Code Council.



APPENDIX A

Post Tension
Hand Checks



<u>Material</u>

Normal wt 150 pcf

$f'_c = 4000$ psi

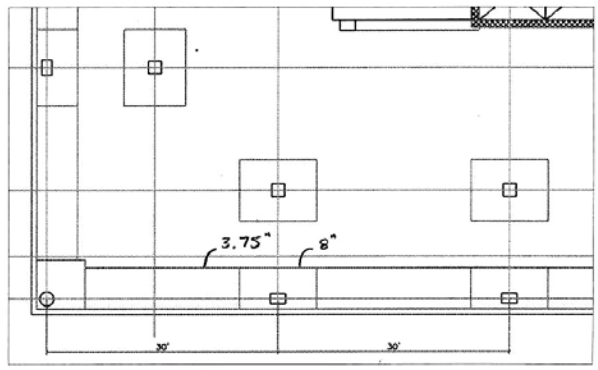
$f'_{ci} = 3000$ psi

$f_y = 60$ ksi

$f_{pu} = 270$ ksi

$f_{se} = 174$ ksi

$P_{eff} = 26.6$ k/ten



<u>Loading</u>

$LL_o = 100$ psf $\quad A_T = 255\ ft^2$ ∴ no LL reduction

Super DL = 20 psf

$$\text{Self wt} = (6/12)(150) + \frac{(4.5'\times 4.5')(\frac{8}{12})(150)}{255\ ft^2} + \frac{(20.5'\times 4')(\frac{3.75}{12})(150)}{255\ ft^2} + \frac{(4\times 10')(\frac{8}{12})(150)}{255}$$

$$= 113.7 \text{ psf}$$

<u>Slab Section Properties</u>

$A = 792\ in^2$

$$S = \frac{(8.5\times 12)(6^2)}{6} + \frac{(4\times 12)(3.75^2)}{6} = 724.5\ in^3$$

<u>Parameters</u>

At Jacking

$f'_{ci} = 3000$ psi

comp $= .6 f'_{ci} = 1800$ psi

Tension $= 3\sqrt{f'_{ci}} = 164$ psi

At service Loads

$f'_c = 4000$ psi

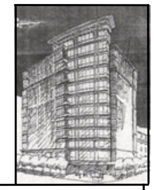
comp $= .45 f'_c = 1800$ psi

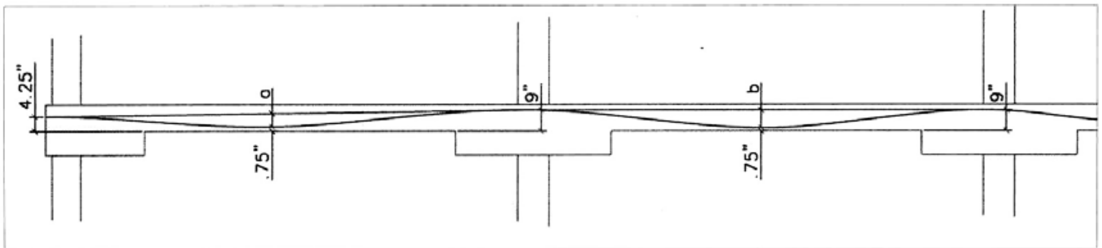
tension $= 6\sqrt{f'_c} = 380$ psi

Ave Precomp

$P/A = 125$ psi min

1/4





$a = \left(\frac{4.25+9}{2}\right) - .75 = 5.875''$

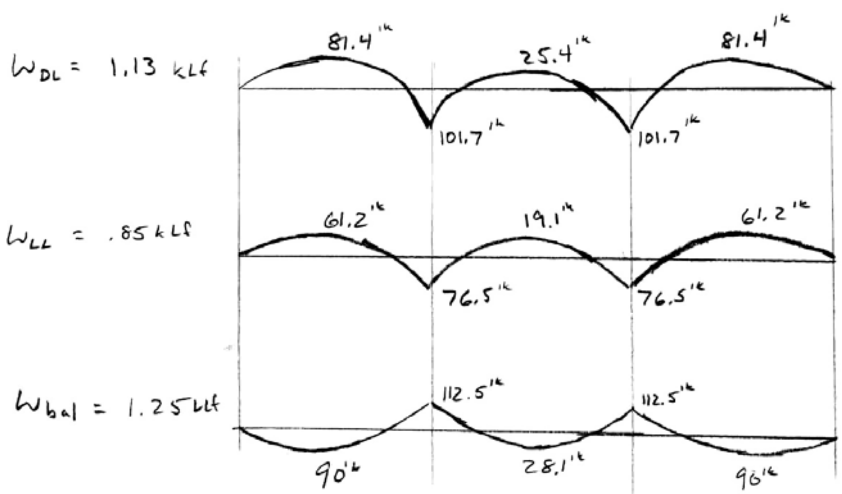
$b = 9 - .75 = 8.25''$

$P = (9 \text{ tendon})(26.6^k) = 239^k$

$P/A = \frac{239(1000)}{792 \text{ in}^2} = 302 \text{ psi} > 125 \text{ ok}$

$w_b = \frac{(239^k)(8)(5.87/12'')}{30^2} = 1.04 \text{ klf}$

$= \frac{239(8)(8.25/12'')}{30^2} = 1.46 \text{ klf}$



2/4

After jacking

Interior span

midspan stress

$$f_{Top} = \frac{(+25.4 + 28.1)(12)(1000)}{724\ in^3} - 302\ psi = -346\ psi < -1800\ psi\ \underline{ok}$$

$$f_{bot} = \frac{(-25.4 + 28.1)(12)(1000)}{724\ in^3} - 302\ psi = -257\ psi < -1800\ psi\ \underline{ok}$$

end stress

$$f_{Top} = \frac{(112.5 - 101.7)(12)(1000)}{724} - 302 = -123\ psi < -1800\ psi\ \underline{ok}$$

$$f_{bot} = \frac{(-112.5 + 101.7)(12)(1000)}{724} - 302 = -481\ psi < 1800\ psi\ \underline{ok}$$

Exterior spans

midspan stress

$$f_{Top} = \frac{(81.4 - 90)(12)(1000)}{724} - 302 = -444\ psi < 1800\ psi\ \underline{ok}$$

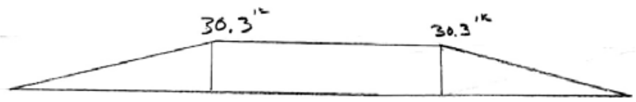
$$f_{bot} = \frac{(-81.4 + 90)(12)(1000)}{724} - 302 = -159.4\ psi < 1800\ psi\ \underline{ok}$$

Ultimate Strength

$$M_1 = P \cdot e$$

$$= 239 \cdot (9 - 4.875)/12'' = 82.1'^k$$

$$M_{sec} = M_{bal} - M_1 = 112.5 - 82.1 = 30.3'^k$$



at midspan

$$M_u = 1.2(81.4) + 1.6(61.2) + 1.0\left(\frac{30.3}{2}\right) = 210.7'^k$$

at support

$$M_u = 1.2(-101.7) + 1.6(-76.5) + 1.0(30.3) = -214.1'^k$$

3/4

Reinforcement

Bottom & Top bar

				ft-k	ft	ft-k/ft	#4	check?
mid	210.7	CS	.60	126.4	4'	31.6	95.6	good
		ms	.40	84.3	4.5'	18.7	55.1	good
support	214.1	CS	.75	160.6	4'	40.15	95.6	good
		ms	.25	53.5	4.5'	11.8	55.1	good

#4 @ 12"

C.S

$A_s = .2\ in^2/ft$

$d = 9"$

$a = \frac{.20(60)}{.85(4)(12)} = .29\ in$

$\phi M_n = \phi(.2)(60)(9.0 - \frac{.29}{2})$

$\phi M = 95.6^{'k}$

M.S

$A_s = .2\ in^2/ft$

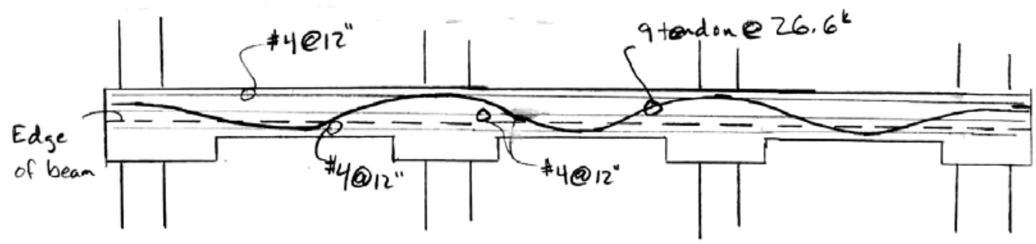
$d = 5.25$

$a = \frac{.20(60)}{.85(4)(12)} = .29\ in$

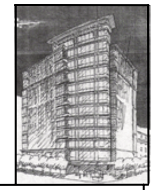
$\phi M_n = \phi(.2)(60)(5.25 - \frac{.29}{2})$

$\phi M_n = 55.1^{'k}$

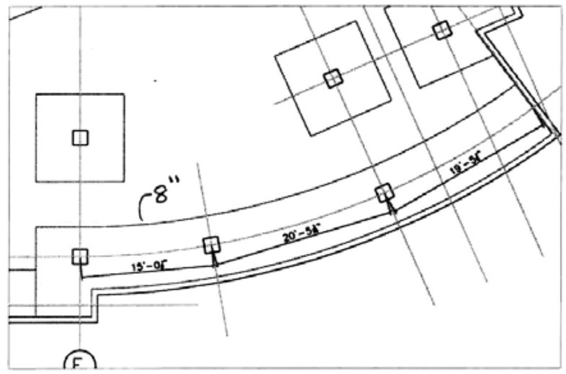
USE #4@12" o.c
+ 9 tendors PT



4/4



Assuming curve to act as a straight system



Materials

Normal wt 150 pcf

$f'_c = 4000\,psi$

$f'_{ci} = 3000\,psi$

$f_y = 60\,ksi$

$f_{pu} = 270\,ksi$

$P_{eff} = 26.6\ k/ten$

Loading

$LL_o = 100\,psf$ $\quad A_T = 235.7\ ft$ (for largest bay ∴ no LL Reduction)

Sup DL = 20 psf $\quad$ Ave width = 11.5'

$$\text{Self wt} = (.6/12)(150) + \frac{(7 \times 55)(\frac{8}{12})(150)}{632.5\,ft^2} = 135.8\,psf$$

Slab Properties

$A = 1500\,in^2$

$$S = \frac{(11.5 \times 12)(6)^2}{6} + \frac{(7 \times 12)(8)^2}{6} = 1724\,in^3$$

Parameters

At jacking

$f'_{ci} = 3000\,psi$

comp = $.6 f'_{ci}$ = 1800 psi

Tension = $3\sqrt{f'_{ci}}$ = 164 psi

At service loads

$f'_c = 4000\,psi$

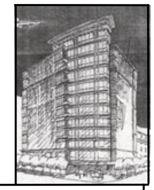
comp = $.45 f'_c$ = 1800 psi

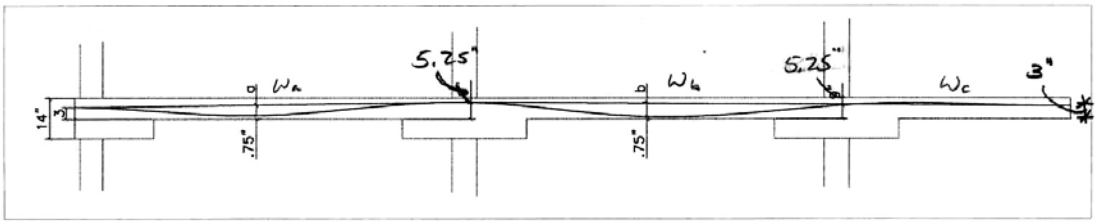
tension = $6\sqrt{f'_c}$ = 380 psi

Ave Precomp

P/A = 125 psi min

1/4





$a = \dfrac{(5.25 + 3)}{2} - .75 = 3.37''$

$b = 4.5''$

$P = (9 \text{ tendons})(26.6) = 239^k$

$P/A = \dfrac{239(1000)}{1500 \text{ in}^2} = 159.3 \text{ psi} > 125$ ok

$w_a = \dfrac{(239)(8)(3.37''/12)}{15^2} = 2.39 \text{ klf}$

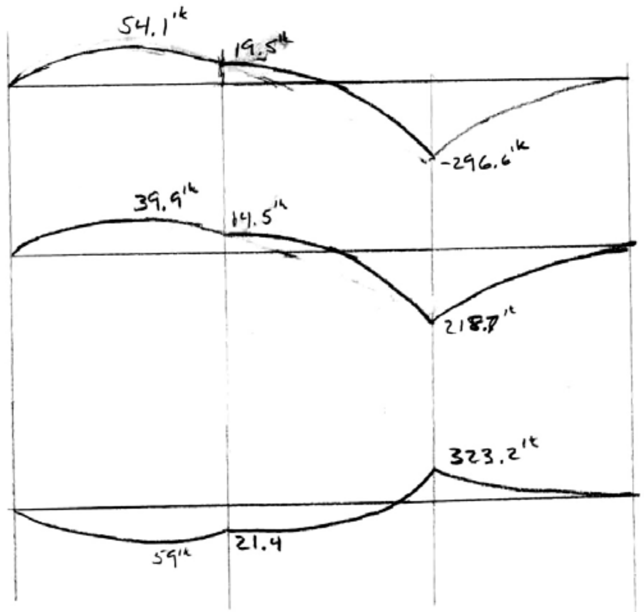
$w_b = 1.7 \text{ klf}$

$w_c = .94 \text{ klf}$

$W_{DL} = 1.56$ kft

$W_{LL} = 1.15$

$W_b = 1.7$ kLf



2/4

After jacking

"15' foot" span

midspan stress

$$f_{top} = \frac{(54.1-59)(12)(1000)}{1724 in^3} - 159.3 psi = -193.1 psi < 1800 psi \ \underline{ok}$$

$$f_{bot} = \frac{(-54.1+59)(12)(1000)}{1724} - 159.3 = -125.2 psi < 1800 psi \ \underline{ok}$$

support

$$f_{top} = \frac{(19.5-21.4)(12)(1000)}{1724} - 159.3 = -172.5 psi < 1800 psi \ \underline{ok}$$

$$f_{bot} = +13.2 - 159.3 = -146.1 psi < 1800 psi \ \underline{ok}$$

"20.5'/19.5'" span

support

$$f_{top} = \frac{(-296.6 + 323.2^{'k})(12)(1000)}{1724} - 159.3 = +25.8 psi < 164 psi \text{ tension } \underline{ok}$$

$$f_{bot} = -25.8 - 159.3 = -185.1 psi < 1800 psi \ \underline{ok}$$

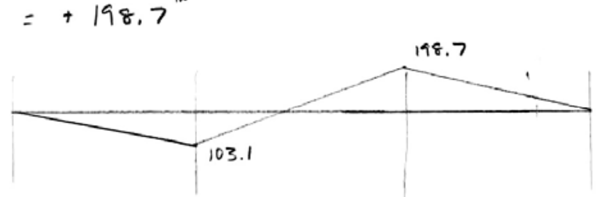
Ultimate Strength

$$M_1 = P \cdot e$$

$$= 239(6.25)/12 = 124.5^{'k}$$

$$M_{sec} = M_{bal} - M_1 = -103.1^{'k}$$

$$= +198.7^{'k}$$



"15' foot"

Mid $M_u = 1.2(54.1^{'k}) + 1.6(39.9^{'k}) + 1.0\left(\frac{103.1}{2}\right) = 77.2^{'k}$

sup $M_u = 1.2(19.5) + 1.6(14.5) + 1.0(-103.1) = -56.5^{'k}$

"20.5 / 19.5"

sup $M_u = 1.2(-296.6) + 1.6(-218.7) + 1.0(198.7) = -507.14$

3/4

Reinforcement

		ft-k	ft	ft-k/ft	#4	check	
77.2	cs .60	46.3	7'	6.6	11.8	ok	
	ms .40	30.8	4.5'	6.8	4.6	2.2	ft-k/ft ①
-56.5	cs .75	-42.4	7'	-6.0	-11.8	ok	
	ms .25	-14.1	4.5'	-3.1	-4.6	ok	
-507.1	cs .75	-380.3	7'	-54.3	-11.8	42.5	ft-k/ft ②
	ms .25	-126.8	4.5'	-28.17	-4.6	24.1	ft-k/ft ③

#4 @ 12" o.c.

C.S

$A_s = .20 \text{ in}^2/\text{ft}$ $\quad a = \dfrac{.20(60)}{.85(4)(12)} = .29 \text{ in}$ $\quad \phi M_n = \phi(.2)(60)(13.25 - \frac{.29}{2})/12"$

$d = 13.25$ $\quad \phi M_n = 11.8^{\text{1k}}/\text{ft}$

M.S.

$A_s = .2 \text{ in}^2/\text{ft}$ $\quad a = .29 \text{ in}$ $\quad \phi M_n = \phi(.2)(60)(5.25 - \frac{.29}{2})/12"$

$d = 5.25$ $\quad \phi M_n = 4.6^{\text{1k}/\text{ft}}$

① try 1#5

$M_n = (2.2^{\text{1k}/\text{ft}})(4.5') = 9.9^{\text{1k}}$ $\quad \phi M_n = \phi(.31)(60)(5.25 - \frac{.10}{2})/12$

$a = \dfrac{.31(60)}{.85(4)(4.5 \times 12)} = .10"$ $\quad \phi M_n = 7.25 < 9.9^{\text{1k}}$ USE 2#5

② try 8#8

$M_n = (42.5)(7') = 297.5^{\text{1k}}$ $\quad \phi M_n = \phi(6.28)(60)(13.0 - \frac{1.32}{2})/12$

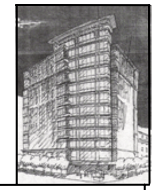
$a = \dfrac{6.28(60)}{.85(4)(7 \times 12)} = 1.32$ $\quad \phi M_n = 348.7^{\text{1k}} > 297.5^{\text{1k}}$ ok

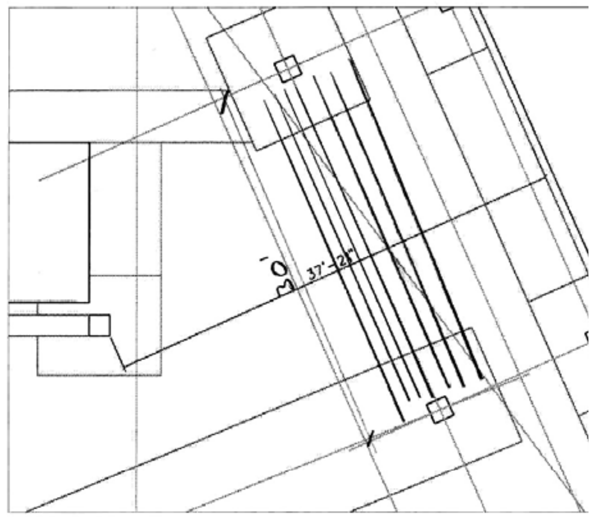
③ try 8#8

$M_n = (24.1)(4.5) = 108.4^{\text{1k}}$ $\quad \phi M_n = \phi(6.28)(60)(5 - \frac{2.05}{2})/12$

$a = \dfrac{6.28(60)}{.85(4)(4.5 \times 12)} = 2.05$ $\quad \phi M_n = 113.0^{\text{1k}} > 108.4^{\text{1k}}$ ok

4/4





Materials

Normal wt conc 150pcf

$f'_c = 4000 psi$

$f'_{ci} = 3000 psi$

$f_y = 60ksi$

$f_{pu} = 270 ksi$

$P_{eff} = 26.6$ k/ten

Loading

$LL_o = 100psf$

$A_T = 2341 ft^2$

$$LL = LL_o\left(.25 + \frac{15}{\sqrt{A_T}}\right)$$

$$= 100\left(.25 + \frac{15}{\sqrt{2341}}\right) = 56.0 psf$$

Sup Dead = 20psf

Slab Properties

$A = 1260 ft^2$

$S = 1260 ft^2$

Parameters

At jacking

$f'_{ci} = 3000psi$

comp $= .6f'_{ci} = 1800psi$

tensin $= 3\sqrt{f'_{ci}} = 164 psi$

At service loads

$f'_c = 4000psi$

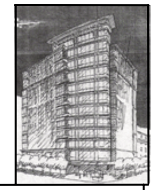
comp $= .45 f'_c = 1800psi$

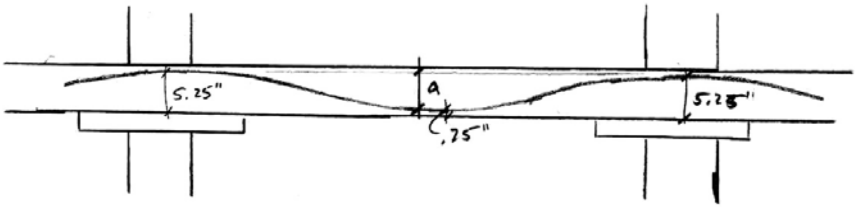
tensin $= 6\sqrt{f'_c} = 380 psi$

Ave Precomp

$P/A = 125$ psi min

1/4





$a = 5.25" - .75 = 4.5"$

$P = (16 \text{ tendons})(26.6) = 425.6 \text{ k}$

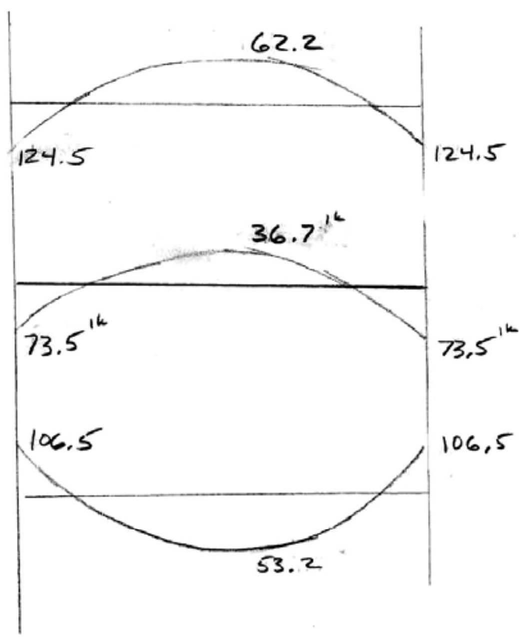
$P/A = \dfrac{425.6^k(1000)}{1260 \text{ in}^2} = 337.8 \text{ psi} > 125 \text{ psi}$ ok

$w_a = \dfrac{(425.6)(8)(4.5"/12)}{30^2} = 1.42 \text{ klf}$

$w_{DL} = 1.66$ klf

$w_{LL} = .98$ klf

$w_b = 1.42$ klf



2/4

After jacking

mid span stress

$f_{top} = \dfrac{(62.2 - 53.2)(12)(1000)}{1260} - 337.8 \text{ psi} = -252.1 < 164 \text{ psi ok}$

$f_{bot} = \dfrac{(-62.2 + 53.2)(12)(1000)}{1260} - 337.8 = -423.5 > -1800 \text{ psi ok}$

Support

$f_{top} = \dfrac{(-124.5 + 106.5)(12)(1000)}{1260} - 337.8 = -509.2 > -1800 \text{ psi ok}$

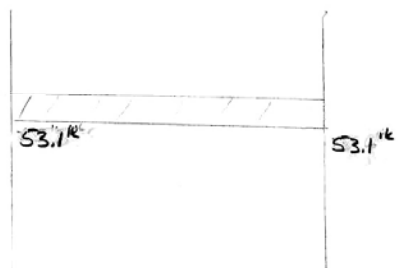
$f_{bot} = \dfrac{(124.5 - 106.5)(12)(1000)}{1260} - 337.8 = -166.4 < 164 \text{ psi ok}$

Ultimate

$M_1 = P \cdot e$

$= 425.6^k (4.5') / 12 = 159.6^{'k}$

$M_{sec} = M_{bal} - M_1 = 106.5 - 159.6 = 53.1^{'k}$



Mid

$M_u = 1.2(62.2) + 1.6(36.7) - 1.0(53.1)$

$= 86.26^{'k}$

Support

$M_u = 1.2(-124.5) + 1.6(-73.5) + 1.0(106.5)$

$= 160.5^{'k}$

3/4

Reinforcement

		ft-k	ft	ft-k/ft	#4	check
80.26	CS .60	48.2	8.7	5.54	11.8	ok
	MS .40	32.1	8.7	3.69	4.6	ok
-160.5	CS .75	120.4	8.7	13.84	11.8	2.04 ①
	MS .25	40.1	8.7	4.61	4.6	ok

#4 @ 12" o.c.

C.S.

$A_s = .20 \text{ in}^2/\text{ft}$ $\quad a = \frac{.20(60)}{.85(4)(12)} = .29"$ $\quad \phi M_n = \phi(.2)(60)(13.25 - \frac{.29}{2})/12$

$d = 13.25"$ $\quad = 11.8^{1k}/\text{ft}$

M.S

$A_s = .20 \text{ in}^2/\text{ft}$ $\quad a = .29"$ $\quad \phi M_n = \phi(.2)(60)(5.25 - \frac{.29}{2})/12$

$d = 5.25"$ $\quad = 4.6^{1k}/\text{ft}$

① $M_u = 2.04(8.7) = 17.7^{1k}$

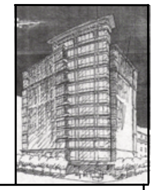
Try 1#6

$a = \frac{.44(60)}{.85(4)(8.7 \times 12)} = .07"$ $\quad \phi M_n = \phi(.44)(60)(5.25 - \frac{.07}{2})/12$

$= 10.3^{1k}$ no good

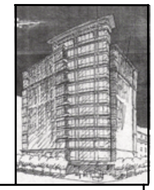
use 2#6

4/4



APPENDIX B

Seismic Loads



SEISMIC

Location DC

Category II site class D

seismic group I
importance factor = 1.0

$S_s = 18.0\%g$
$S_1 = 6.2\%g$

$S_{m_s} = F_a S_s$
$= (1.6)(.18)$
$= .288$

$F_a = 1.6$
$F_v = 2.4$

$S_{m_1} = F_v S_1$
$= (2.4)(.062)$
$= .149$

SEISMIC DESIGN CAT.
I
R = 5

$S_{DS} = \frac{2}{3} S_{ms} = .192$
$S_{D1} = \frac{2}{3} S_{m1} = .099$
$C_{s\,min} = .044\, I\, S_{DS}$
$= .044(1.0)(.192) = .0084$

$T = C_T h_n^x$
$C_T = .016$
$h = 147.5'$
$x = .9$
$T = .016(147.5)^{.9}$
$= 1.43$

$C_s = \frac{S_{DS}}{(R/I)}$
$= \frac{.192}{(5/1.0)} = .038$

$C_{s\,max} = \frac{S_{D1}}{T(R/I)}$
$= \frac{.099}{1.43(5/1.0)} = .014$ ← controls

1/2

Dead load estimation

- concrete, 6" slab

Area of 8" drops 2341.2 ft^2
Area of 3 3/4 edge beam 1365.9 ft^2

Equivalent Floor Thickness

$$T = 6" + \frac{(8)(2341.2)}{12{,}200 ft^2} + \frac{(3\,3/4)(1365.9)}{12200}$$

$$T = 7.95"$$

Curtain wall = 300 pLf (450') ⇒ 135^k

equivalent floor load = $\frac{135^k}{12200}$ = 11.06 psf → 12 psf

Total Dead load

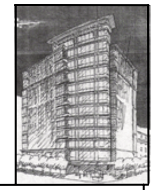
Concrete	99.4 psf
Sprinkler	5 psf
MEP	5 psf
Finishes	10 psf
Curtain wall	12 psf
	131.4 psf

Total Floor load = (131.4 psf)(11860) ⇒ 1550^k

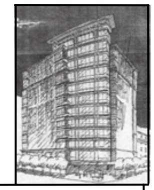
Total roof load = (131.4 psf)(3600) ⇒ 473^k

Total Building load = (1603)(12) + 473
= 19709^k

2/2



SEISMIC	ht (ft)	load (k)	$W*ht^k$	C_{vx}		story force (k) =VCs
roof	147.5	473	243137	0.0581		15.52
pent.	129.5	1550	677125	0.1618		43.21
12	118.7	1550	607289	0.1451		38.76
11	107.8	1550	538400	0.1287		34.36
10	97.0	1550	471842	0.1128		30.11
9	86.2	1550	407114	0.0973		25.98
8	75.3	1550	343815	0.0822		21.94
7	64.5	1550	283323	0.0677		18.08
6	53.7	1550	225320	0.0539		14.38
5	42.8	1550	169682	0.0406		10.83
4	32.0	1550	117969	0.0282		7.53
3	21.2	1550	70510	0.0169		4.50
2	10.3	1550	28601	0.0068		1.83
		19073	4184127	1.0000		267.02



APPENDIX C

Wind Loads

Wind

Exposure B case 2

$k_{zt} = 1.0$ (no hills)

$k_d = .85$

$V = 90_{mph}$

$I = 1.0$

$q_z = 0.00256 k_z k_{zt} k_d V^2 I$

Period of Building

$C_t = 0.016$

$h = 147.3'$

$x = .9$

$T = 0.016(147.3')^{.9}$

$= 1.48$

$f = \frac{1}{T}$

$= .67 < 1.0 \therefore$ flexible building

Windward pressure: $P_w = q_z G C_p$

$C_p = .8$

leeward

$C_p = -0.3$ N-S

$= -0.5$ E-W

Leeward Pressure: $P_L = q_h G C_p$

$Z_g = 1200\,ft$

$\alpha = 7.0$

$k_h = 2.01\left(\frac{147.5}{1200}\right)^{2/7.0} = 1.104$

$q_z = 0.00256(1.104)(1.0)(.85)(90_{mph})^2(1.0) = 19.46$

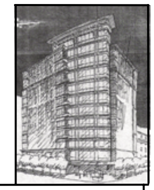
$G = .839$ N-S

$= .843$ E-W

$P_L = (19.46)(.839)(-.3) = -4.89$ N-S

$= (19.46)(.843)(-.5) = -8.20$ E-W

z (ft)	k_z	q_z	N-S			E-W		
			P_L (psf)	P_w (psf)	P_L+P_w (psf)	P_L (psf)	P_w (psf)	P_L+P_w (psf)
147.5	1.104	19.464	-4.97	12.96	17.93	-8.39	13.13	21.52
138.5	1.085	19.117	-4.97	12.72	17.69	-8.39	12.89	21.28
124.1	1.051	18.527	-4.97	12.33	17.30	-8.39	12.49	20.88
113.25	1.024	18.049	-4.97	12.01	16.98	-8.39	12.17	20.56
102.4	0.995	17.537	-4.97	11.67	16.64	-8.39	11.83	20.22
91.6	0.964	16.987	-4.97	11.31	16.28	-8.39	11.46	19.85
80.75	0.930	16.386	-4.97	10.91	15.88	-8.39	11.05	19.44
69.9	0.892	15.724	-4.97	10.47	15.44	-8.39	10.60	18.99
59.1	0.850	14.988	-4.97	9.98	14.95	-8.39	10.11	18.50
48.25	0.802	14.144	-4.97	9.41	14.38	-8.39	9.54	17.93
37.4	0.746	13.151	-4.97	8.75	13.72	-8.39	8.87	17.26
26.6	0.677	11.931	-4.97	7.94	12.91	-8.39	8.05	16.44
15.75	0.583	10.272	-4.97	6.84	11.81	-8.39	6.93	15.32
5.15	0.423	7.464	-4.97	4.97	9.94	-8.39	5.03	13.42



APPENDIX D

Laterals Load Distribution
Building Drifts

Height	Seismic		Wind - Governing		Story Forces	
	E-W	N-S	E-W	N-S	E-W	N-S
(ft)	(pLf)	(pLf)	(pLf)	(pLf)	(kips)	(kips)
roof	107.0	168.7	96.8	80.7	14.0	7.4
pent.	298.0	469.7	249.0	207.0	36.1	19.0
12	267.3	421.3	263.7	218.4	38.2	20.1
11	237.0	373.5	223.1	184.3	32.3	17.0
10	207.7	327.3	218.8	180.2	31.7	16.6
9	179.2	282.4	214.8	176.2	31.2	16.2
8	151.3	238.5	210.9	172.3	30.6	15.8
7	124.7	196.5	205.6	167.1	29.8	15.4
6	99.2	156.3	200.2	161.8	29.0	14.9
5	74.7	117.7	194.5	156.1	28.2	14.4
4	51.9	81.8	186.8	148.6	27.1	13.7
3	31.0	48.9	177.9	139.8	25.8	12.9
2	12.6	19.9	164.3	126.6	23.8	11.6

Lateral Rigidity	h (ft)	Wall 1 (E-W)	Wall 2 (E-W)	Wall 3 (E-W)	Wall 4 (E-W)	Wall 5 (N-S)	Wall 6 (N-S)	F1 (E-W)	F2	F2 (E-W)	F2 (N-S)	F3 (N-S)	F4	F4 (E-W)	F4 (N-S)	F5	F5 (E-W)	F5 (N-S)	ΣR (E-W)	ΣR (N-S)
roof	147.5	4.1	4.1	3.4	3.4	33.2	43.5	8.9	9.2	8.4	3.7	5.9	3.3	1.3	3.0	9.8	2.0	9.6	35.5	98.9
pent.	129.5	6.0	6.0	4.9	4.9	48.8	64.0	8.9	9.2	8.4	3.7	5.9	3.3	1.3	3.0	10.8	2.2	10.6	42.8	136.0
12	118.7	7.8	7.8	6.4	6.4	63.3	82.8	10.0	10.2	9.3	4.2	6.6	3.7	1.5	3.4	11.1	2.3	10.9	51.6	171.1
11	107.8	10.4	10.4	8.6	8.6	84.0	109.9	11.1	11.5	10.5	4.7	7.4	4.2	1.7	3.8	12.7	2.6	12.4	63.8	222.2
10	97.0	14.3	14.3	11.7	11.7	114.7	150.0	12.6	13.0	11.8	5.3	8.4	4.8	2.0	4.4	14.6	3.0	14.3	81.5	297.1
9	86.2	20.3	20.3	16.7	16.7	162.4	212.0	14.5	14.9	13.6	6.1	9.7	5.6	2.3	5.1	17.1	3.6	16.7	108.0	412.1
8	75.3	30.3	30.3	25.0	25.0	240.4	313.2	17.1	17.5	15.9	7.1	11.4	6.6	2.7	6.0	20.4	4.2	20.0	150.5	598.1
7	64.5	48.1	48.1	39.6	39.6	376.7	489.2	20.7	21.0	19.2	8.6	13.8	8.1	3.3	7.4	25.3	5.3	24.7	223.9	920.4
6	53.7	82.9	82.9	68.4	68.4	636.5	822.3	26.2	26.4	24.1	10.7	17.4	10.5	4.3	9.6	32.8	6.8	32.1	364.0	1528.6
5	42.8	161.0	161.0	132.9	132.9	1192.4	1527.6	36.2	35.6	32.5	14.5	23.9	14.8	6.0	13.5	46.7	9.7	45.7	672.3	2817.6
4	32.0	375.5	375.5	311.1	311.1	2592.0	3268.7	56.8	53.7	49.0	21.9	37.0	24.0	9.8	21.9	73.2	15.2	71.6	1504.1	6013.0
3	21.2	1201.8	1201.8	1004.3	1004.3	7037.4	8596.3	113.6	101.2	92.4	41.2	73.5	50.5	20.5	46.1	146.0	30.3	142.8	4669.1	15937.4
2	10.3	7281.6	7281.6	6285.6	6285.6	27291.3	31355.4	450.5	367.7	335.8	149.7	291.5	217.9	88.6	199.1	574.0	119.3	561.5	28128.5	59848.4

E-W	story forces (kips)	story shear (kips)	Wall 1 prop. (%)	Wall 1 Shear (kips)	Wall 2 prop. (%)	Wall 2 Shear (kips)	Wall 3 prop. (%)	Wall 3 Shear (kips)	Wall 4 prop. (%)	Wall 4 Shear (kips)	F1 prop. (%)	F1 Force (kips)	F2 prop. (%)	F2 Force (kips)	F4 prop. (%)	F4 Force (kips)	F5 prop. (%)	F5 Force (kips)
roof	14.0	14.0	0.115	1.6	0.115	1.6	0.094	1.3	0.094	1.3	0.251	3.52	0.166	2.33	0.038	0.53	0.057	0.81
pent.	36.1	50.1	0.141	7.0	0.141	7.0	0.116	5.8	0.116	5.8	0.208	7.51	0.138	4.98	0.031	1.13	0.052	1.89
12	38.2	88.4	0.151	13.4	0.151	13.4	0.124	11.0	0.124	11.0	0.194	7.40	0.128	4.89	0.029	1.11	0.045	1.71
11	32.3	120.7	0.163	19.7	0.163	19.7	0.134	16.2	0.134	16.2	0.174	5.63	0.116	3.75	0.027	0.87	0.041	1.34
10	31.7	152.5	0.175	26.7	0.175	26.7	0.144	22.0	0.144	22.0	0.155	4.91	0.103	3.27	0.024	0.76	0.037	1.18
9	31.2	183.6	0.188	34.5	0.188	34.5	0.155	28.4	0.155	28.4	0.134	4.18	0.090	2.80	0.021	0.66	0.033	1.02
8	30.6	214.2	0.201	43.1	0.201	43.1	0.166	35.5	0.166	35.5	0.114	3.47	0.076	2.32	0.018	0.55	0.028	0.86
7	29.8	244.0	0.215	52.4	0.215	52.4	0.177	43.2	0.177	43.2	0.092	2.76	0.062	1.84	0.015	0.44	0.023	0.70
6	29.0	273.0	0.228	62.2	0.228	62.2	0.188	51.3	0.188	51.3	0.072	2.09	0.048	1.39	0.012	0.34	0.019	0.54
5	28.2	301.2	0.239	72.1	0.239	72.1	0.198	59.6	0.198	59.6	0.054	1.52	0.036	1.00	0.009	0.25	0.014	0.41
4	27.1	328.3	0.250	82.0	0.250	82.0	0.207	67.9	0.207	67.9	0.038	1.02	0.025	0.67	0.006	0.18	0.010	0.27
3	25.8	354.1	0.257	91.2	0.257	91.2	0.215	76.2	0.215	76.2	0.024	0.63	0.016	0.41	0.004	0.11	0.006	0.17
2	23.8	378.0	0.259	97.8	0.259	97.8	0.223	84.5	0.223	84.5	0.016	0.38	0.010	0.25	0.003	0.08	0.004	0.10
base	378.0			97.8		97.8		84.5		84.5		45.02		29.88		7.00		11.00

N-S	story forces (kips)	story shear (kips)	Wall 5 prop. (%)	Wall 5 Shear (kips)	Wall 6 prop. (%)	Wall 6 Shear (kips)	F1 prop. (%)	F1 Force (kips)	F2 prop. (%)	F2 Force (kips)	F3 prop. (%)	F3 Force (kips)	F4 prop. (%)	F4 Force (kips)	F5 prop. (%)	F5 Force (kips)
roof	7.4	7.4	0.335	2.5	0.440	3.3	0.090	0.67	0.038	0.28	0.060	0.44	0.030	0.23	0.097	0.72
pent.	19.0	26.5	0.359	9.5	0.470	12.4	0.065	1.25	0.027	0.52	0.043	0.83	0.022	0.42	0.078	1.48
12	20.1	46.6	0.370	17.2	0.484	22.5	0.058	1.17	0.024	0.49	0.039	0.78	0.020	0.40	0.063	1.28
11	17.0	63.5	0.378	24.0	0.495	31.4	0.050	0.85	0.021	0.36	0.033	0.56	0.017	0.29	0.056	0.95
10	16.6	80.1	0.386	30.9	0.505	40.4	0.042	0.70	0.018	0.29	0.028	0.47	0.015	0.24	0.048	0.80
9	16.2	96.3	0.394	38.0	0.515	49.5	0.035	0.57	0.015	0.24	0.024	0.38	0.012	0.20	0.041	0.66
8	15.8	112.1	0.402	45.1	0.524	58.7	0.029	0.45	0.012	0.19	0.019	0.30	0.010	0.16	0.033	0.53
7	15.4	127.5	0.409	52.2	0.531	67.8	0.022	0.35	0.009	0.14	0.015	0.23	0.008	0.12	0.027	0.41
6	14.9	142.4	0.416	59.3	0.538	76.6	0.017	0.26	0.007	0.10	0.011	0.17	0.006	0.09	0.021	0.31
5	14.4	156.8	0.423	66.3	0.542	85.0	0.013	0.18	0.005	0.07	0.008	0.12	0.005	0.07	0.016	0.23
4	13.7	170.4	0.431	73.5	0.544	92.6	0.009	0.13	0.004	0.05	0.006	0.08	0.004	0.05	0.012	0.16
3	12.9	183.3	0.442	80.9	0.539	98.9	0.007	0.09	0.003	0.03	0.005	0.06	0.003	0.04	0.009	0.12
2	11.6	194.9	0.456	88.9	0.524	102.1	0.008	0.09	0.003	0.03	0.005	0.06	0.003	0.04	0.009	0.11
base	194.9			88.9		102.1		6.76		2.80		4.48		2.36		7.75

Torsion		Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1 (E-W)	F1 (N-S)	F2	F3	F4	F5	ΣRx^2
Dist. from CR		10.5	9.5	8.5	9.5	14.5	13.0	63.5	50.1	60.3	40.0	35.5	49.0	
roof	Rx	43	43	32	28	315	628	565	446	554	236	117	480	142888
pent.		63	57	42	47	706	832	565	446	554	236	117	529	154188
12		82	74	55	61	914	1077	635	513	564	148	131	544	167062
11		109	99	73	81	1213	1428	705	574	631	168	149	622	193620
10		150	136	100	112	1658	1950	800	650	715	192	170	715	229335
9		213	193	142	159	2347	2756	921	746	821	224	199	838	279016
8		318	288	212	237	3474	4071	1086	875	962	264	234	1000	351943
7		505	457	337	376	5444	6359	1314	1054	1159	324	288	1240	467545
6		871	788	581	650	9197	10690	1664	1322	1454	420	373	1607	667997
5		1690	1529	1130	1263	17231	19859	2299	1781	1959	592	525	2288	1068865
4		3943	3568	2645	2956	37454	42493	3607	2688	2956	960	852	3587	2005907
3		12619	11417	8537	9541	101690	111752	7214	5071	5577	2020	1793	7154	4869893
2		76456	69175	53428	59713	394359	407620	28607	18419	20257	8716	7735	28126	19441801

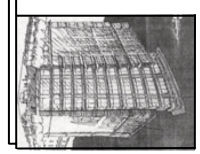
Rx/Rx^2	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1 (E-W)	F1 (N-S)	F2	F3	F4	F5
roof	0.000	0.000	0.000	0.000	0.002	0.004	0.004	0.003	0.004	0.002	0.001	0.003
pent.	0.000	0.000	0.000	0.000	0.005	0.005	0.004	0.003	0.004	0.002	0.001	0.003
12	0.000	0.000	0.000	0.000	0.005	0.006	0.004	0.003	0.003	0.001	0.001	0.003
11	0.001	0.001	0.000	0.000	0.006	0.007	0.004	0.003	0.003	0.001	0.001	0.003
10	0.001	0.001	0.000	0.000	0.007	0.009	0.003	0.003	0.003	0.001	0.001	0.003
9	0.001	0.001	0.001	0.001	0.008	0.010	0.003	0.003	0.003	0.001	0.001	0.003
8	0.001	0.001	0.001	0.001	0.010	0.012	0.003	0.002	0.003	0.001	0.001	0.003
7	0.001	0.001	0.001	0.001	0.012	0.014	0.003	0.002	0.002	0.001	0.001	0.003
6	0.001	0.001	0.001	0.001	0.014	0.016	0.002	0.002	0.002	0.001	0.001	0.002
5	0.002	0.001	0.001	0.001	0.016	0.019	0.002	0.002	0.002	0.001	0.000	0.002
4	0.002	0.002	0.001	0.001	0.019	0.021	0.002	0.001	0.001	0.000	0.000	0.002
3	0.003	0.002	0.002	0.002	0.021	0.023	0.001	0.001	0.001	0.000	0.000	0.001
2	0.004	0.004	0.003	0.003	0.020	0.021	0.001	0.001	0.001	0.000	0.000	0.001

Torsional Perportions			Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1 (E-W)	F1 (N-S)	F2	F3	F4	F5
E-W	Torsional Moment (kip-ft)	Story Moment (kip-ft)	Resultant Forces from Torsion (kips)											
roof	99	99	0.030	0.030	0.022	0.020	0.219	0.436	0.392	0.309	0.384	0.164	0.081	0.333
pent.	354	255	0.145	0.131	0.097	0.108	1.620	1.910	0.934	0.737	0.916	0.390	0.194	0.875
12	624	270	0.306	0.277	0.204	0.228	3.414	4.022	1.026	0.828	0.911	0.239	0.212	0.879
11	852	228	0.481	0.435	0.320	0.358	5.341	6.288	0.831	0.677	0.744	0.198	0.176	0.733
10	1076	224	0.703	0.636	0.469	0.524	7.782	9.151	0.782	0.635	0.698	0.188	0.166	0.699
9	1296	220	0.991	0.897	0.660	0.738	10.906	12.806	0.726	0.588	0.647	0.177	0.157	0.660
8	1512	216	1.368	1.238	0.912	1.019	14.928	17.493	0.666	0.537	0.590	0.162	0.144	0.613
7	1723	210	1.861	1.684	1.241	1.387	20.057	23.430	0.592	0.475	0.522	0.146	0.129	0.558
6	1928	205	2.513	2.273	1.677	1.874	26.540	30.849	0.511	0.406	0.446	0.129	0.114	0.493
5	2127	199	3.363	3.043	2.249	2.513	34.285	39.516	0.428	0.332	0.365	0.110	0.098	0.426
4	2318	191	4.557	4.123	3.056	3.416	43.283	49.106	0.344	0.256	0.282	0.092	0.081	0.342
3	2500	182	6.479	5.862	4.383	4.898	52.208	57.374	0.270	0.190	0.209	0.076	0.067	0.268
2	2668	168	10.494	9.494	7.333	8.196	54.126	55.946	0.247	0.159	0.175	0.075	0.067	0.243
base	3771	2668	10.5	9.5	7.3	8.2	54.1	55.9	8	6	7	2	2	7

N-S	Torsional Moment (kips-ft)	Story Moment (kip-ft)	Resultant Forces from Torsion (kips)											
roof	40	40	0.012	0.012	0.009	0.008	0.088	0.176	0.158	0.125	0.155	0.066	0.033	0.135
pent.	143	103	0.059	0.053	0.039	0.044	0.654	0.771	0.377	0.297	0.369	0.157	0.078	0.353
12	251	109	0.123	0.112	0.082	0.092	1.376	1.621	0.412	0.333	0.366	0.096	0.085	0.353
11	343	92	0.193	0.175	0.129	0.144	2.149	2.530	0.333	0.271	0.298	0.079	0.070	0.294
10	432	90	0.283	0.256	0.188	0.210	3.127	3.677	0.312	0.254	0.279	0.075	0.067	0.279
9	520	88	0.398	0.360	0.265	0.296	4.375	5.137	0.289	0.234	0.258	0.070	0.062	0.263
8	606	86	0.548	0.496	0.365	0.408	5.978	7.006	0.264	0.213	0.234	0.064	0.057	0.243
7	689	83	0.744	0.673	0.496	0.554	8.017	9.366	0.233	0.187	0.206	0.058	0.051	0.220
6	769	80	1.002	0.907	0.669	0.748	10.587	12.306	0.200	0.159	0.175	0.051	0.045	0.193
5	847	78	1.339	1.211	0.895	1.000	13.646	15.728	0.167	0.129	0.142	0.043	0.038	0.166
4	920	74	1.809	1.637	1.213	1.356	17.184	19.496	0.133	0.099	0.109	0.035	0.031	0.132
3	990	69	2.565	2.320	1.735	1.939	20.667	22.712	0.103	0.072	0.080	0.029	0.026	0.102
2	1053	63	4.140	3.745	2.893	3.233	21.352	22.070	0.093	0.060	0.066	0.028	0.025	0.091
base	2615	1053	4	4	3	3	21	22	3	2	3	1	1	3

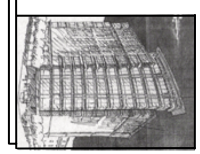
story forces (kips)	E-W	N-S
roof	14.0	7.4
pent.	36.1	19.0
12	38.2	20.1
11	32.3	17.0
10	31.7	16.6
9	31.2	16.2
8	30.6	15.8
7	29.8	15.4
6	29.0	14.9
5	28.2	14.4
4	27.1	13.7
3	25.8	12.9
2	23.8	11.6
base	378.0	194.9

story shear (kips)	E-W	N-S
roof	14.0	7.4
pent.	50.1	26.5
12	88.4	46.6
11	120.7	63.5
10	152.5	80.1
9	183.6	96.3
8	214.2	112.1
7	244.0	127.5
6	273.0	142.4
5	301.2	156.8
4	328.3	170.4
3	354.1	183.3
2	378.0	194.9
base	389.1	295.4



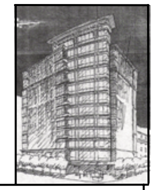
Total Forces (E-W)	Wall 1 Story Shear (kips)			Wall 2 Story Shear (kips)			Wall 3 Story Shear (kips)			Wall 4 Story Shear (kips)			Wall 5 Story Shear (kips)			Wall 6 Story Shear (kips)			F1 (E-W) StoryForces (kips)		
roof	1.61	-0.03	1.61	1.61	0.03	1.64	1.33	-0.02	1.33	1.33	0.02	1.34	0.00	0.22	0.22	0.00	0.44	0.44	3.52	-0.39	3.52
pent.	7.05	-0.15	7.05	7.05	0.13	7.18	5.80	-0.10	5.80	5.80	0.11	5.91	0.00	1.62	1.62	0.00	1.91	1.91	7.51	-0.93	7.51
12	13.37	-0.31	13.37	13.37	0.28	13.65	11.00	-0.20	11.00	11.00	0.23	11.23	0.00	3.41	3.41	0.00	4.02	4.02	7.40	-1.03	7.40
11	19.68	-0.48	19.68	19.68	0.43	20.11	16.19	-0.32	16.19	16.19	0.36	16.55	0.00	5.34	5.34	0.00	6.29	6.29	5.63	-0.83	5.63
10	26.71	-0.70	26.71	26.71	0.64	27.34	21.98	-0.47	21.98	21.98	0.52	22.50	0.00	7.78	7.78	0.00	9.15	9.15	4.91	-0.78	4.91
9	34.54	-0.99	34.54	34.54	0.90	35.43	28.43	-0.66	28.43	28.43	0.74	29.17	0.00	10.91	10.91	0.00	12.81	12.81	4.18	-0.73	4.18
8	43.14	-1.37	43.14	43.14	1.24	44.38	35.52	-0.91	35.52	35.52	1.02	36.54	0.00	14.93	14.93	0.00	17.49	17.49	3.47	-0.67	3.47
7	52.42	-1.86	52.42	52.42	1.68	54.10	43.18	-1.24	43.18	43.18	1.39	44.57	0.00	20.06	20.06	0.00	23.43	23.43	2.76	-0.59	2.76
6	62.20	-2.51	62.20	62.20	2.27	64.48	51.29	-1.68	51.29	51.29	1.87	53.17	0.00	26.54	26.54	0.00	30.85	30.85	2.09	-0.51	2.09
5	72.14	-3.36	72.14	72.14	3.04	75.18	59.58	-2.25	59.58	59.58	2.51	62.09	0.00	34.29	34.29	0.00	39.52	39.52	1.52	-0.43	1.52
4	81.98	-4.56	81.98	81.98	4.12	86.10	67.92	-3.06	67.92	67.92	3.42	71.34	0.00	43.28	43.28	0.00	49.11	49.11	1.02	-0.34	1.02
3	91.15	-6.48	91.15	91.15	5.86	97.02	76.17	-4.38	76.17	76.17	4.90	81.07	0.00	52.21	52.21	0.00	57.37	57.37	0.63	-0.27	0.63
2	97.84	-10.49	97.84	97.84	9.49	107.34	84.46	-7.33	84.46	84.46	8.20	92.65	0.00	54.13	54.13	0.00	55.95	55.95	0.38	-0.25	0.38
base	97.84	-10.49	97.84	97.84	9.49	107.34	84.46	-7.33	84.46	84.46	8.20	92.65	0.00	54.13	54.13	0.00	55.95	55.95	45.02	-7.75	45.02

Total Forces (E-W)	F1 (N-S) Story Force (kips)			F2 StoryForces (kips)			F3 StoryForces (kips)			F4 StoryForces (kips)			F5 StoryForces (kips)		
roof	0.00	0.31	0.31	2.33	0.38	2.72	0.00	0.16	0.16	0.53	-0.08	0.53	0.81	0.33	1.14
pent.	0.00	0.74	0.74	4.98	0.92	5.89	0.00	0.39	0.39	1.13	-0.19	1.13	1.89	0.87	2.77
12	0.00	0.83	0.83	4.89	0.91	5.80	0.00	0.24	0.24	1.11	-0.21	1.11	1.71	0.88	2.59
11	0.00	0.68	0.68	3.75	0.74	4.50	0.00	0.20	0.20	0.87	-0.18	0.87	1.34	0.73	2.07
10	0.00	0.63	0.63	3.27	0.70	3.97	0.00	0.19	0.19	0.76	-0.17	0.76	1.18	0.70	1.88
9	0.00	0.59	0.59	2.80	0.65	3.44	0.00	0.18	0.18	0.66	-0.16	0.66	1.02	0.66	1.69
8	0.00	0.54	0.54	2.32	0.59	2.91	0.00	0.16	0.16	0.55	-0.14	0.55	0.86	0.61	1.47
7	0.00	0.47	0.47	1.84	0.52	2.36	0.00	0.15	0.15	0.44	-0.13	0.44	0.70	0.56	1.26
6	0.00	0.41	0.41	1.39	0.45	1.83	0.00	0.13	0.13	0.34	-0.11	0.34	0.54	0.49	1.04
5	0.00	0.33	0.33	1.00	0.36	1.37	0.00	0.11	0.11	0.25	-0.10	0.25	0.41	0.43	0.83
4	0.00	0.26	0.26	0.67	0.28	0.95	0.00	0.09	0.09	0.18	-0.08	0.18	0.27	0.34	0.62
3	0.00	0.19	0.19	0.41	0.21	0.61	0.00	0.08	0.08	0.11	-0.07	0.11	0.17	0.27	0.44
2	0.00	0.16	0.16	0.25	0.18	0.42	0.00	0.08	0.08	0.08	-0.07	0.08	0.10	0.24	0.34
base	0.00	6.13	6.13	29.88	6.89	36.77	0.00	2.14	2.14	7.00	-1.69	7.00	11.00	7.12	18.13



Total Forces (N-S)	Wall 1 Story Shear (kips)			Wall 2 Story Shear (kips)			Wall 3 Story Shear (kips)			Wall 4 Story Shear (kips)			Wall 5 Story Shear (kips)			Wall 6 Story Shear (kips)			F1 (E-W) StoryForces (kips)		
roof	0.00	0.01	**0.01**	0.00	0.01	**0.01**	0.00	0.01	**0.01**	0.00	0.01	**0.01**	2.49	0.09	**2.58**	3.26	-0.18	**3.26**	0.00	0.16	**0.16**
pent.	0.00	0.06	**0.06**	0.00	0.05	**0.05**	0.00	0.04	**0.04**	0.00	0.04	**0.04**	9.50	0.65	**10.15**	12.45	-0.77	**12.45**	0.00	0.38	**0.38**
12	0.00	0.12	**0.12**	0.00	0.11	**0.11**	0.00	0.08	**0.08**	0.00	0.09	**0.09**	17.21	1.38	**18.59**	22.54	-1.62	**22.54**	1.00	0.41	**1.41**
11	0.00	0.19	**0.19**	0.00	0.18	**0.18**	0.00	0.13	**0.13**	0.00	0.14	**0.14**	24.01	2.15	**26.16**	31.41	-2.53	**31.41**	0.00	0.33	**0.33**
10	0.00	0.28	**0.28**	0.00	0.26	**0.26**	0.00	0.19	**0.19**	0.00	0.21	**0.21**	30.93	3.13	**34.06**	40.43	-3.68	**40.43**	0.00	0.31	**0.31**
9	0.00	0.40	**0.40**	0.00	0.36	**0.36**	0.00	0.26	**0.26**	0.00	0.30	**0.30**	37.96	4.37	**42.34**	49.55	-5.14	**49.55**	0.00	0.29	**0.29**
8	0.00	0.55	**0.55**	0.00	0.50	**0.50**	0.00	0.37	**0.37**	0.00	0.41	**0.41**	45.08	5.98	**51.06**	58.72	-7.01	**58.72**	0.00	0.26	**0.26**
7	0.00	0.74	**0.74**	0.00	0.67	**0.67**	0.00	0.50	**0.50**	0.00	0.55	**0.55**	52.19	8.02	**60.21**	67.77	-9.37	**67.77**	0.00	0.23	**0.23**
6	0.00	1.00	**1.00**	0.00	0.91	**0.91**	0.00	0.67	**0.67**	0.00	0.75	**0.75**	59.29	10.59	**69.88**	76.61	-12.31	**76.61**	0.00	0.20	**0.20**
5	0.00	1.34	**1.34**	0.00	1.21	**1.21**	0.00	0.89	**0.89**	0.00	1.00	**1.00**	66.34	13.65	**79.99**	84.99	-15.73	**84.99**	0.00	0.17	**0.17**
4	0.00	1.81	**1.81**	0.00	1.64	**1.64**	0.00	1.21	**1.21**	0.00	1.36	**1.36**	73.46	17.18	**90.65**	92.65	-19.50	**92.65**	0.00	0.13	**0.13**
3	0.00	2.56	**2.56**	0.00	2.32	**2.32**	0.00	1.73	**1.73**	0.00	1.94	**1.94**	80.93	20.67	**101.60**	98.86	-22.71	**98.86**	0.00	0.10	**0.10**
2	0.00	4.14	**4.14**	0.00	3.75	**3.75**	0.00	2.89	**2.89**	0.00	3.23	**3.23**	88.89	21.35	**110.25**	102.13	-22.07	**102.13**	0.00	0.09	**0.09**
base	0.00	4.14	**4.14**	0.00	3.75	**3.75**	0.00	2.89	**2.89**	0.00	3.23	**3.23**	88.89	21.35	**110.25**	102.13	-22.07	**102.13**	0.00	3.07	**3.07**

Total Forces (N-S)	F1 (N-S) Story Force (kips)			F2 StoryForces (kips)			F3 StoryForces (kips)			F4 StoryForces (kips)			F5 StoryForces (kips)		
roof	0.67	0.13	**0.79**	0.28	-0.16	**0.28**	0.44	-0.07	**0.44**	0.23	-0.03	**0.23**	0.72	0.13	**0.85**
pent.	1.25	0.30	**1.54**	0.52	-0.37	**0.52**	0.83	-0.16	**0.83**	0.42	-0.08	**0.42**	1.48	0.35	**1.83**
12	1.17	0.33	**1.51**	0.49	-0.37	**0.49**	0.78	-0.10	**0.78**	0.40	-0.09	**0.40**	1.28	0.35	**1.63**
11	0.85	0.27	**1.12**	0.36	-0.30	**0.36**	0.56	-0.08	**0.56**	0.29	-0.07	**0.29**	0.95	0.29	**1.24**
10	0.70	0.25	**0.96**	0.29	-0.28	**0.29**	0.47	-0.07	**0.47**	0.24	-0.07	**0.24**	0.80	0.28	**1.08**
9	0.57	0.23	**0.80**	0.24	-0.26	**0.24**	0.38	-0.07	**0.38**	0.20	-0.06	**0.20**	0.66	0.26	**0.92**
8	0.45	0.21	**0.67**	0.19	-0.23	**0.19**	0.30	-0.06	**0.30**	0.16	-0.06	**0.16**	0.53	0.24	**0.77**
7	0.35	0.19	**0.53**	0.14	-0.21	**0.14**	0.23	-0.06	**0.23**	0.12	-0.05	**0.12**	0.41	0.22	**0.63**
6	0.26	0.16	**0.41**	0.10	-0.17	**0.10**	0.17	-0.05	**0.17**	0.09	-0.04	**0.09**	0.31	0.19	**0.51**
5	0.18	0.13	**0.31**	0.07	-0.14	**0.07**	0.12	-0.04	**0.12**	0.07	-0.04	**0.07**	0.23	0.17	**0.40**
4	0.13	0.10	**0.23**	0.05	-0.11	**0.05**	0.08	-0.04	**0.08**	0.05	-0.03	**0.05**	0.16	0.13	**0.29**
3	0.09	0.07	**0.16**	0.03	-0.08	**0.03**	0.06	-0.03	**0.06**	0.04	-0.03	**0.04**	0.12	0.10	**0.22**
2	0.09	0.06	**0.15**	0.03	-0.07	**0.03**	0.06	-0.03	**0.06**	0.04	-0.03	**0.04**	0.11	0.09	**0.20**
base	6.76	2.43	**9.19**	2.80	-2.74	**2.80**	4.48	-0.85	**4.48**	2.36	-0.67	**2.36**	7.75	2.82	**10.57**

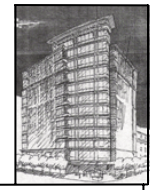


	wall 1				wall 2				wall 3			
	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}
roof	1.6	4.1	0.128	4.284	1.6	4.1	0.130	4.406	1.3	3.4	0.128	4.288
pent.	7.0	6.0	0.269	4.156	7.2	6.0	0.274	4.276	5.8	4.9	0.269	4.161
12	13.4	7.8	0.426	3.887	13.6	7.8	0.435	4.002	11.0	6.4	0.426	3.891
11	19.7	10.4	0.513	3.461	20.1	10.4	0.524	3.567	16.2	8.6	0.513	3.465
10	26.7	14.3	0.557	2.948	27.3	14.3	0.570	3.043	22.0	11.7	0.557	2.952
9	34.5	20.3	0.561	2.391	35.4	20.3	0.575	2.472	28.4	16.7	0.561	2.394
8	43.1	30.3	0.526	1.830	44.4	30.3	0.541	1.897	35.5	25.0	0.526	1.833
7	52.4	48.1	0.458	1.304	54.1	48.1	0.472	1.356	43.2	39.6	0.458	1.307
6	62.2	82.9	0.364	0.847	64.5	82.9	0.377	0.884	51.3	68.4	0.364	0.849
5	72.1	161.0	0.256	0.483	75.2	161.0	0.267	0.507	59.6	132.9	0.257	0.484
4	82.0	375.5	0.150	0.227	86.1	375.5	0.158	0.240	67.9	311.1	0.151	0.228
3	91.2	1201.8	0.063	0.077	97.0	1201.8	0.067	0.082	76.2	1004.3	0.064	0.077
2	97.8	7281.6	0.013	0.013	107.3	7281.6	0.015	0.015	84.5	6285.6	0.013	0.013

	wall 4				wall 5				wall 6			
	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}
roof	1.3	3.4	0.129	4.410	2.6	33.2	0.025	0.666	3.3	43.5	0.024	0.591
pent.	5.9	4.9	0.275	4.281	10.2	48.8	0.047	0.641	12.4	64.0	0.044	0.567
12	11.2	6.4	0.435	4.006	18.6	63.3	0.072	0.594	22.5	82.8	0.067	0.523
11	16.5	8.6	0.524	3.571	26.2	84.0	0.084	0.522	31.4	109.9	0.076	0.456
10	22.5	11.7	0.571	3.047	34.1	114.7	0.087	0.438	40.4	150.0	0.079	0.380
9	29.2	16.7	0.576	2.476	42.3	162.4	0.085	0.351	49.5	212.0	0.075	0.301
8	36.5	25.0	0.541	1.900	51.1	240.4	0.077	0.266	58.7	313.2	0.067	0.225
7	44.6	39.6	0.473	1.359	60.2	376.7	0.065	0.189	67.8	489.2	0.056	0.158
6	53.2	68.4	0.378	0.886	69.9	636.5	0.051	0.124	76.6	822.3	0.043	0.102
5	62.1	132.9	0.267	0.508	80.0	1192.4	0.036	0.073	85.0	1527.6	0.030	0.059
4	71.3	311.1	0.158	0.241	90.6	2592.0	0.022	0.037	92.6	3268.7	0.018	0.029
3	81.1	1004.3	0.068	0.083	101.6	7037.4	0.011	0.015	98.9	8596.3	0.008	0.012
2	92.7	6285.6	0.015	0.015	110.2	27291.3	0.004	0.004	102.1	31355.4	0.003	0.003

APPENDIX E

Punching Shear Check



αs = 20 corner column
30 edge column
40 interior column

βp = $(\alpha sd/b_o+1.5)$ ⟶ (Lesser of two)
3.5

Vc = $(\beta p(f'c)^{(1/2)}+0.3f_{pc})*b_o*d+Vp$
$2*(f'c)^{(1/2)}*b_o*d$

Vs = A_v*f_y*d/s

	Size (in x in)	d (in)	b_o (in)	f_{pc} (psi)	f'c (psi)	αs	βp	Vc (lb)	ØVc (lb)	Vu (lb)	check?	Vs w/ #4@6'	new ØVc (lb)	check?
1	22"Ø	14	74	260	4000	20	3.5	131045	98284	86300	OK			
	20x20	14	58	260	4000	20	3.5	102711	77033	86300	no good	24000	95033.1	OK
2	24x18	14	88	190	4000	30	3.5	155837	116878	114000	OK			
3	24x16	14	84	150	4000	30	3.5	148754	111565	86900	OK			
4	24x16	14	84	150	4000	30	3.5	148754	111565	101000	OK			
5	22"Ø	14	74	300	4000	20	3.5	131045	98284	69900	OK			
	20x20	14	64	300	4000	20	3.5	113336	85002	69900	OK			
6	30x16	14	148	220	4000	40	3.5	262090	196567	183000	OK			
7	18x18	14	128	180	4000	40	3.5	226672	170004	110000	OK			
8	20x20	14	136	225	4000	40	3.5	240839	180629	181000	no good	24000	198629.3	OK
9	24x16	14	84	360	4000	30	3.5	148754	111565	73200	OK			
10														
11														
12	24x16	14	84	270	4000	30	3.5	148754	111565	79900	OK			
13	24x24	14	152	270	4000	40	3.5	269173	201880	191000	OK			
14														
15														
16	20x20	14	136	200	4000	40	3.5	240839	180629	142000	OK			
17	24x12	14	128	125	4000	40	3.5	226672	170004	111000	OK			
18														
19														
20	24x12	14	128	250	4000	40	3.5	226672	170004	81100	OK			
21														
22														
23	20x20	14	136	150	4000	40	3.5	240839	180629	174000	OK			
24	24x24	14	152	200	4000	40	3.5	269173	201880	207000	no good	24000	219879.8	OK
25	24x16	14	84	350	4000	30	3.5	148754	111565	64500	OK			
26	24x16	14	84	300	4000	30	3.5	148754	111565	90400	OK			
27	20x20	14	136	200	4000	40	3.5	240839	180629	151000	OK			
28	20x20	14	136	200	4000	40	3.5	240839	180629	155000	OK			
29	20x20	14	136	200	4000	40	3.5	240839	180629	174000	OK			
30	20x20	14	136	225	4000	40	3.5	240839	180629	165000	OK			
31	20x20	14	136	175	4000	40	3.5	240839	180629	137000	OK			
32	20x20	14	136	200	4000	40	3.5	240839	180629	124000	OK			
33	22"Ø	14	74	225	4000	20	3.5	131045	98284	78100	OK			
	20x20	14	62	225	4000	20	3.5	109794	82346	78100	OK			
34	24x16	14	84	190	4000	30	3.5	148754	111565	82100	OK			
35	24x16	14	84	160	4000	30	3.5	148754	111565	79600	OK			
36	22x22	14	144	150	4000	30	3.5	255006	191255	89200	OK			
37	22x22	14	144	170	4000	30	3.5	255006	191255	72100	OK			
38	22x22	14	144	180	4000	30	3.5	255006	191255	81000	OK			