Final Report

Falls Church Tower

Falls Church, VA

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Structural Option

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Executive Summary

The information found in this document is the culmination of a year-long thesis project involving the study and analysis of a luxury apartment building located in Virginia. At the request of the owner the actual name and location of the apartment building will remain anonymous. For the purposes of this report the building will be referred to as Falls Church Tower and as its name suggests I have located it in Falls Church, Virginia. To clarify, the name and more importantly the location provided have no influence throughout the following analyses as the true location was used to attain the appropriate data for studies such as seismic and wind loading.

The main focus of this report is redesign of the existing lateral force resisting system. The current ordinary concrete moment frame in combination with the post-tensioned flat plate floor system provides an extremely efficient means of resisting most kinds of loads. The engineers of SK&A have designed a framing system that minimizes the floor to floor height of the building while avoiding the overcrowding of columns.

But in spite of these positive aspects there is still room for improvement. The depth portion of the report addresses this by proposing the implication of a concrete shear wall system. This would allow for the elimination of oversized columns; their excessive girth only serving the purpose of resisting lateral loads.

This, in turn, prompts an architectural response that is aesthetic as well as practical in a structural sense. The architectural breadth portion focuses on redesigning the building's column layout to account for the change in column sizes. This presents the opportunity to align the columns to a more definable grid that allows for more flexibility within the building in addition to providing some redundancy to the frame which simplifies the analysis of applied loads.

The second breadth of this report involves a relative cost analysis of the moment frame and shear wall systems. This breadth is important in determining whether or not the alternative system is worth consideration given the fact that even the most superior systems won't make it past the shop drawing phase if the price tag is too high.

Introduction

Falls Church Tower is a luxury apartment building located in Falls Church, Virginia. The high rise apartment building stands eleven stories tall with a penthouse centered on the main roof. Three and a half levels of parking are offered beneath the building and private pool sits adjacent to the plaza. The building encloses 364,000 square feet of gross floor area which excludes mechanical rooms, underground rooms, and garage space. The first floor contains the lobby, a residential gym, and a lounge as well as some living space with the remaining floors serving as strictly residential space. Overall the building contains 213 residential units with a wide view of the surrounding area courtesy of the building's curved facade. The structural system of the building is primarily the concrete moment frame and post-tensioned slab supplemented by retaining walls, grade beams, and girders framing the stairwells and elevators. The moment frame acts as the main lateral force resisting system while the post tensioned slabs transfer a majority of the gravity loads.

General Information

Project Team

Falls Tower is co-owned by Equity Residential and Sunburst Hospitality Corporation. The area that is referred to as Falls Church Tower is actually comprised of the tower itself as well as a cluster of townhouses which is where the joint ownership arises. Sunburst Hospitality has ownership of the townhouses while Equity Residential has ownership of the high rise tower.

Donohoe Construction Company headed the project as general contractor instituting a Design-Bid-Build delivery method. The entire project (high rise tower and townhouses) began July 10th, 2006 and reached substantial completion March 20th, 2009 costing an estimated $92,000,000.

Architecture

Falls Church high rise apartment tower possesses a distinctly post-modern look. Located at the edge of the courthouse district, the luxury apartments serve as a transitional building between residential and commercial developments. The brick veneer and alternating recesses of the facade supplement the surrounding apartment complexes while the vertical glazed elements implement the sleek look of the inner district office buildings. The tower's shape is defined by the contour of the main road running just south of it and provides a contrast to the curved façade of the hotel to the north. In addition to its visual contribution, Falls Church is only a mere ten minutes from the National Mall in Washington D.C. as well as six major parks making it a prime location for commuter living.

The typical building facade consists of brick, mortar collector, building insulation, and 1/2" gypsum board with either an air barrier or self-adhering sheathing behind the brick. Any visible concrete is used to emphasize the vertical elements of the building, specifically the glazed facade leading up to the penthouse as well as the circular corner of the building that houses mechanical equipment inside the top ring.

Structure

The structural system of the Abingdon Heights high rise tower is comprised primarily of cast-in-place concrete. Exceptions include the cornices on the main roof which are constructed of W8x10, W8x15, and W8x21 steel beams. The foundation of the building is a three part system utilizing a 5 inch thick slab on grade with strap beams at key locations. The columns sit on spread footings of 5000 psi concrete and basements 1 through 3 have a retaining wall running the full length of the perimeter.

The gravity loads on the building are resisted by a flat plate system involving rectangular columns of 5000, 6000, and 8000 psi concrete. The columns are typically arranged in 24'x24' bays in a radial pattern with 6' spans for the corridor. The floor system is a one-way, post-tensioned slab with perpendicular tendons running between columns. The tendons themselves are primarily two and three strand tendons spaced four to five on center.

The lateral loads on the building are resisted by the moment frame composed of rectangular columns that alternate in direction. The columns range anywhere between 12 inches and 24 inches on the short face, and 12 inches to 48 inches on the long face. The typical reinforcing for a column will be #8 and #9 bars running both directions with some variation in bar size.

Mechanical

The mechanical system was designed on the premise that the east half of the west half of the building would be treated as two separate towers. All mechanical equipment is located in the penthouse roof mechanical room. The main units are two 6000 CFM AAON RN series packaged AHU’s. Two condenser water pumps (primary and secondary) and one hot water pump are provided for both towers courtesy of Bell and Gossett. Two 2334 MBH Burnham V1112 fossil fuel boilers provide hot water for the 8000 gallon tank. Cooling is achieved by induced draft provided by BAC’s two cell cooling tower.

Electrical

Electrical service for Falls Church Tower is provided by Dominion Virginia Power Co. via 277/480V-3 Phase-4 Wire systems. These are, in turn, run from two D.V.P underground vault transformers. The source feeds into two 2500A switchboards which lead into two 1000KVA-480V Delta-208Y/120V dry type transformers. These supply two 2500A-3 Phase-4 Wire-120/208V plug-in busway risers which provide 400A and 600A service. In addition there is a 450kW, 575KVA-3Ph-4W-277/480V indirect diesel emergency generator located on level B1 with the rest of the equipment.

Lighting

The curved design of the building allows for more direct sun lightning throughout the day minimizing the use of artificial lighting. Typical artificial lighting for apartments takes the form of compact fluorescent pendent fixtures located in the kitchen, dining room, and bathrooms. Corridor lighting consists of wall mounted pendants that are also compact fluorescent.

Navigation

Occupants enter the building from either the lobby entrance located on the north face or from the main elevator in the parking garage beneath the building. Upon entering the lobby occupants are presented with two elevators. The corridor that runs past these elevators leads to the east and west ends of the building. The east end of the building provides an additional elevator and both ends of the building have a stairwell that leads from the basement to the roof.

Structural Depth

Falls Church Tower is an interesting building for the fact that the cleverly designed façade gives the illusion that underneath the post-modern exterior there is a simple frame supporting everything. As it turns out, the structural system is a complex array of irregularly oriented columns arranged in a fashion that does not adhere to any one particular grid. Combine this with the asymmetric curve of the building and the reduction of the floor area at the west end as the height increases to reveal a considerable challenge. The following sections will provide a brief overview of the existing structural system and better explain the elements involved.

Foundation

The foundation system of Falls Church Tower was designed in accordance with the geotechnical report provided by Whitlock, Dairymple, Poston and Associates. The report indicated a soil bearing pressure of 4 ksf along the southern face of the tower and a bearing pressure of 10 ksf for the remainder of the structure.

The foundation system from levels B3 Ext. through B1 consist of retaining walls, spread footings, and a precast slab on grade. The retaining wall runs the full perimeter of the building with a thickness of 1'-4"on the B3 Ext. level and 1'-0" for B3 through B1. The footings under the retaining walls have a width ranging from 2' to 3'. The 2' width is used for sections of the buildings where the B1 retaining wall is offset towards the interior of the building by 3'-6". A section of a typical retaining wall can be seen in Figure 1-2 and Figure 1-3.

The column footings have a range of 6'x6' to 12'x12' throughout the structure. The larger footings (10'x10' to 12'x12') being located in the basement parking section beneath the plaza. A typical footing detail can be seen in Figure 1-1. The slab on grade is 5 ksi, normal weight concrete that is 5" thick with 6x6-W2.0xW2.0 welded wire fabric placed on a vapor barrier on top of 6" of #57 washed crushed stone.

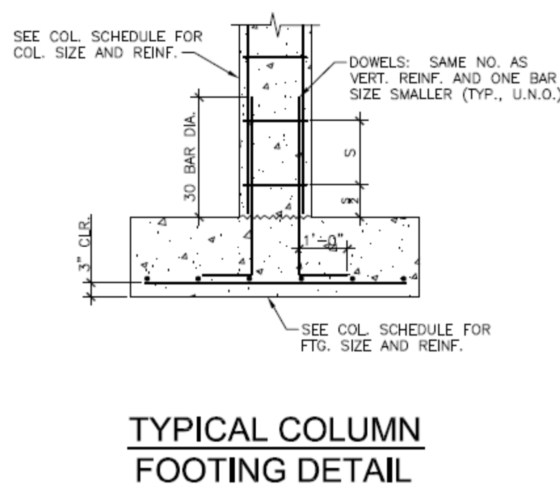


Figure 1-1

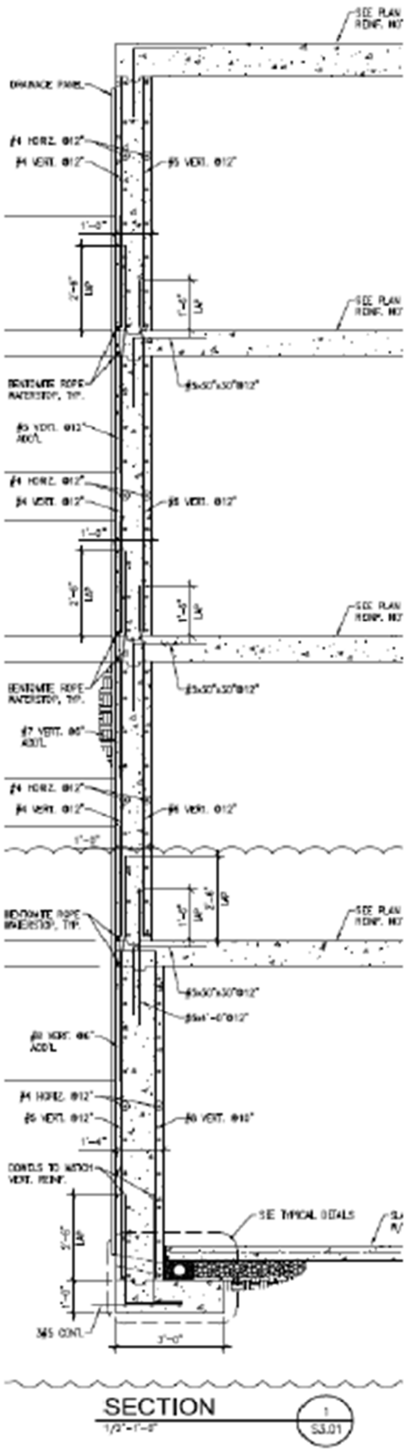


Figure 1-2

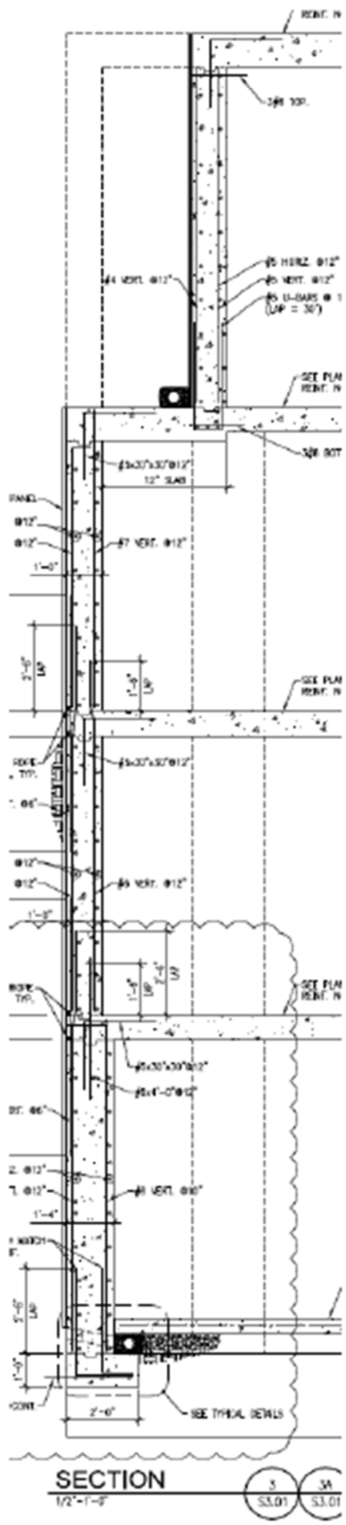


Figure 1-3

Gravity Load System

The main gravity load resisting system is composed of a flat plate supported by an intricate array of columns. Levels B3 Ext. through B1 plate systems are typically a 5 ksi, 9” thick, normal weight slab with a two way mat of #4 bottom bars at 12” on center except for slabs on grade which are 5 kis, 5” thick normal weight concrete. The penthouse roof and the elevator machine room roof use a 6” thick, one-way slab with the same properties and is support by a system of concrete beams. The plate systems from level 1 through the main roof utilize a 7” thick post tensioned slab. The typical tendons are two to three strands thick and spaced 5' on center. For a typical post tension layout plan refer to Figure 1-4.

The tower columns don't necessarily have a standard bay size due to the building's curved shape and the stair cases in both the east and west wings which interrupt any attempt at a rectilinear layout. The most typical bay size established throughout the building would be the 28'x24' bays located in the western half of the building's curved section. A standard column layout can be seen in Figure 1-5

In addition to the flat plate system the structural engineers also incorporated concrete beams into the design where necessary. As previously mentioned a system of beams is used to support the penthouse and mechanical room roofs. There are also strap (grade) beams used in the west section of B3 Ext. foundation and the east edge of B3 foundation which can be seen in Figure 1-6. Lastly, beams are used to frame all stairs and elevator shafts.

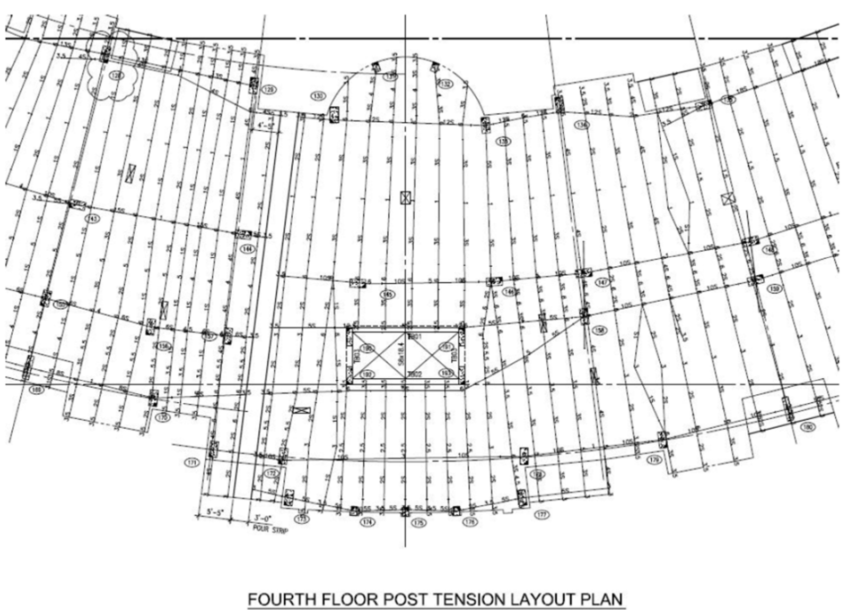


Figure 1-4
(for a larger view refer to Appendix A)

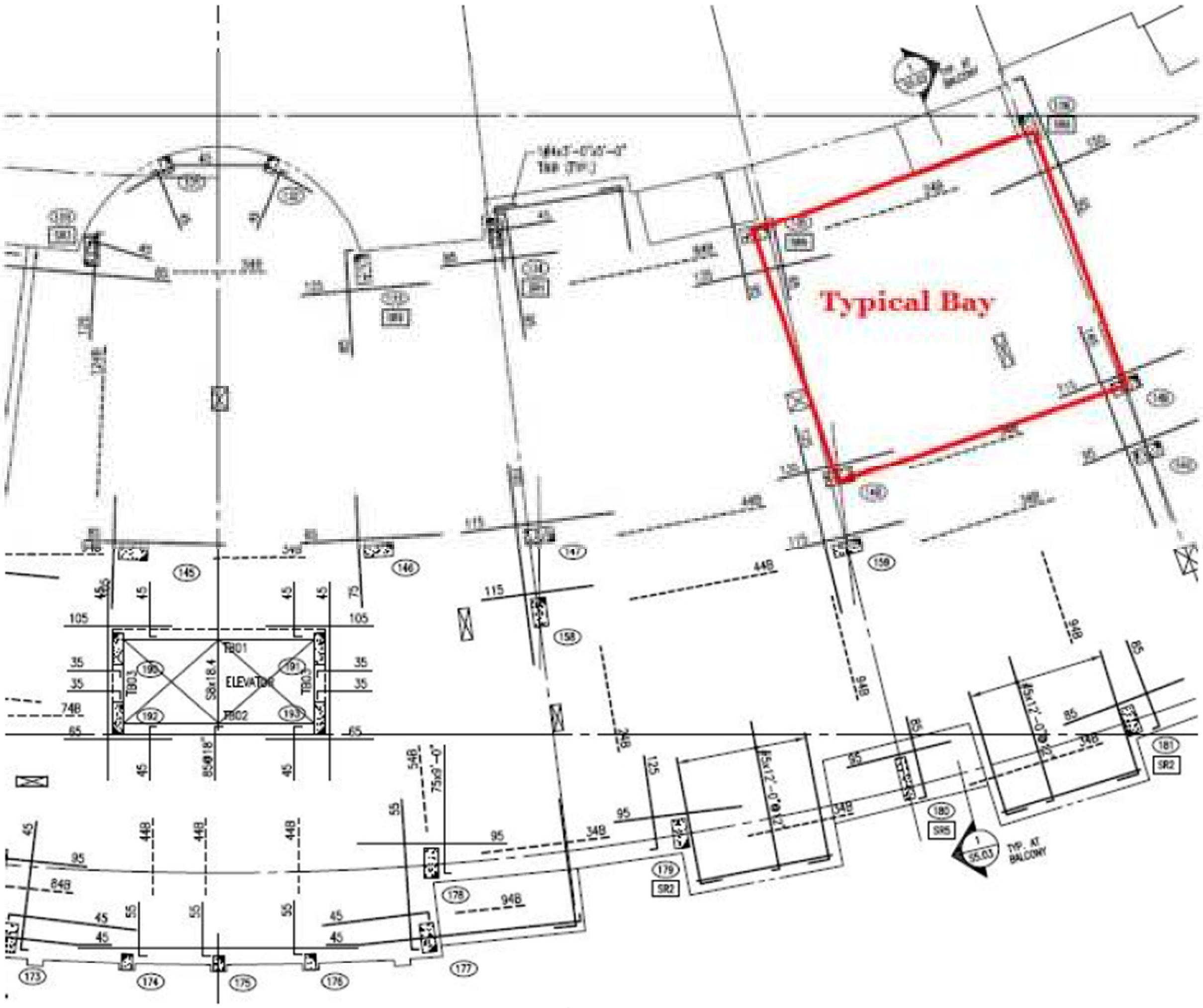


Figure 1-5

Lateral Load System

The lateral system of the building is an ordinary concrete moment frame. The tower columns' dimensions range from 12” to 24” on the short face and 12” to 48” on the long face. The two most typical columns that occur throughout the building are 16”x32” and 12”x36”. The 16”x32” dimension is common for most of the interior columns whereas the 12”x36” columns are used to frame the stairs and elevator shafts.

The irregular layout of the columns is shown in Figure 1-6. Note how the columns outside the central core of the building follow a radial pattern around a center point located just north of the building while the central core columns appear to be arranged in a somewhat orthogonal manner.

This layout allows the columns absorb forces acting on the building from a variety of directions in addition to being well arranged to handle torsional forces.

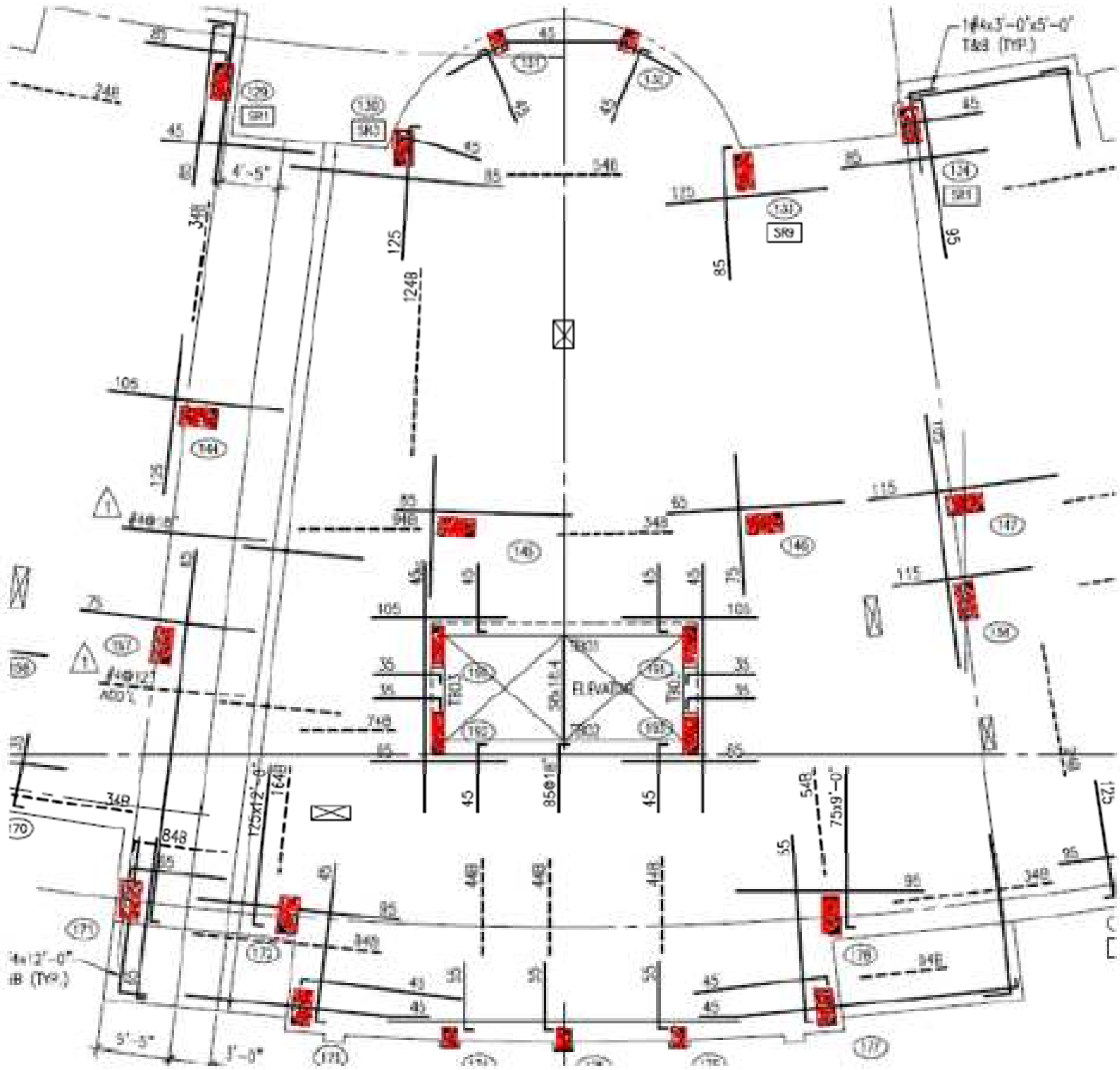


Figure 1-6 : Column Layout

Load Paths

Transfer of live loads through the structure is fairly straight forward for a concrete moment frame. For wind loads the pressure is transferred through the building envelop into the structure supporting the envelope. In the case of Falls Church Towers, the load is transferred into the post tensioned slabs where it then proceeds to down the columns.

Seismic loads behave differently in that the forces they produce are induced by and directly related to the relative displacement of the ground floor from the floors above. In the case of a seismic load the main force produced is shear force which must be taken into consideration when working with concrete moment frames given the inflexible nature of the material and the lack of lateral bracing.

Figure 4-1 illustrates a load path through a typical frame section of Falls Church Tower. The thin red arrows represent the transferred forces from the slab and higher floors. The thick red arrow represents the cumulative force consisting of the forces from the higher floors and a fraction of the forces from the slab. The thin dark red arrow represents the reduced force in the slab.

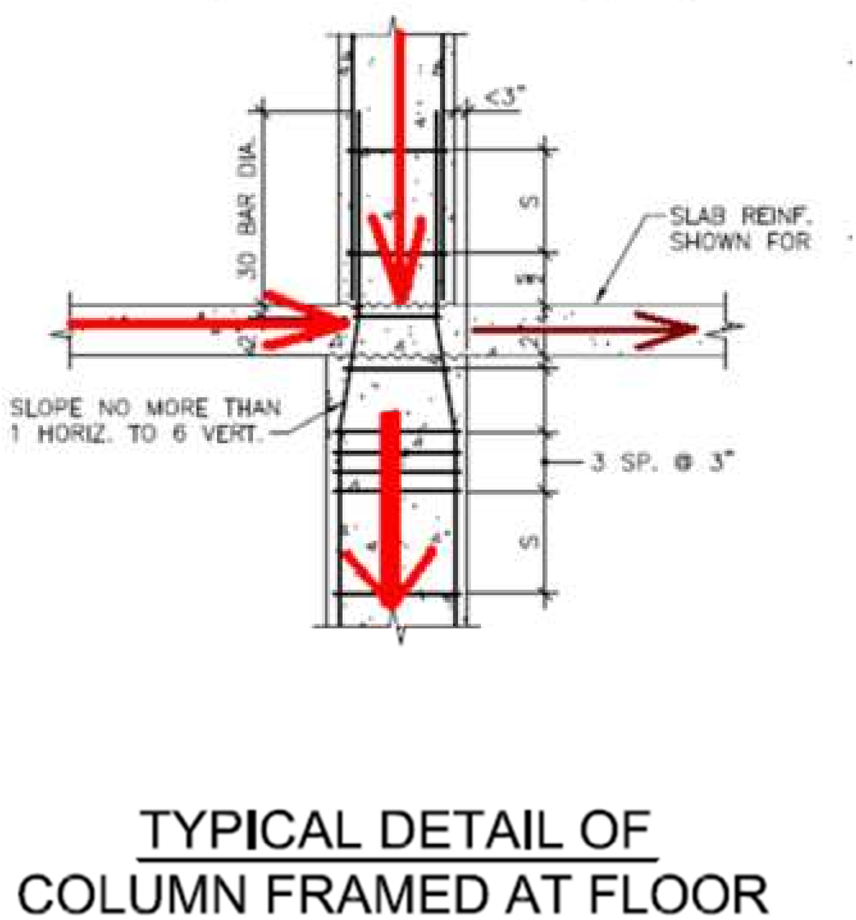


Figure 1-7

Design Confirmation

Previous technical reports have verified the existing system's ability to with stand both gravity and lateral forces. The post tensioned slab was checked as part of the criteria for technical report 2 and the tendon sizing and spacing proved adequate to support the 1.2D+1.6L load combination over the critical column span shown below in Figure 1-8.

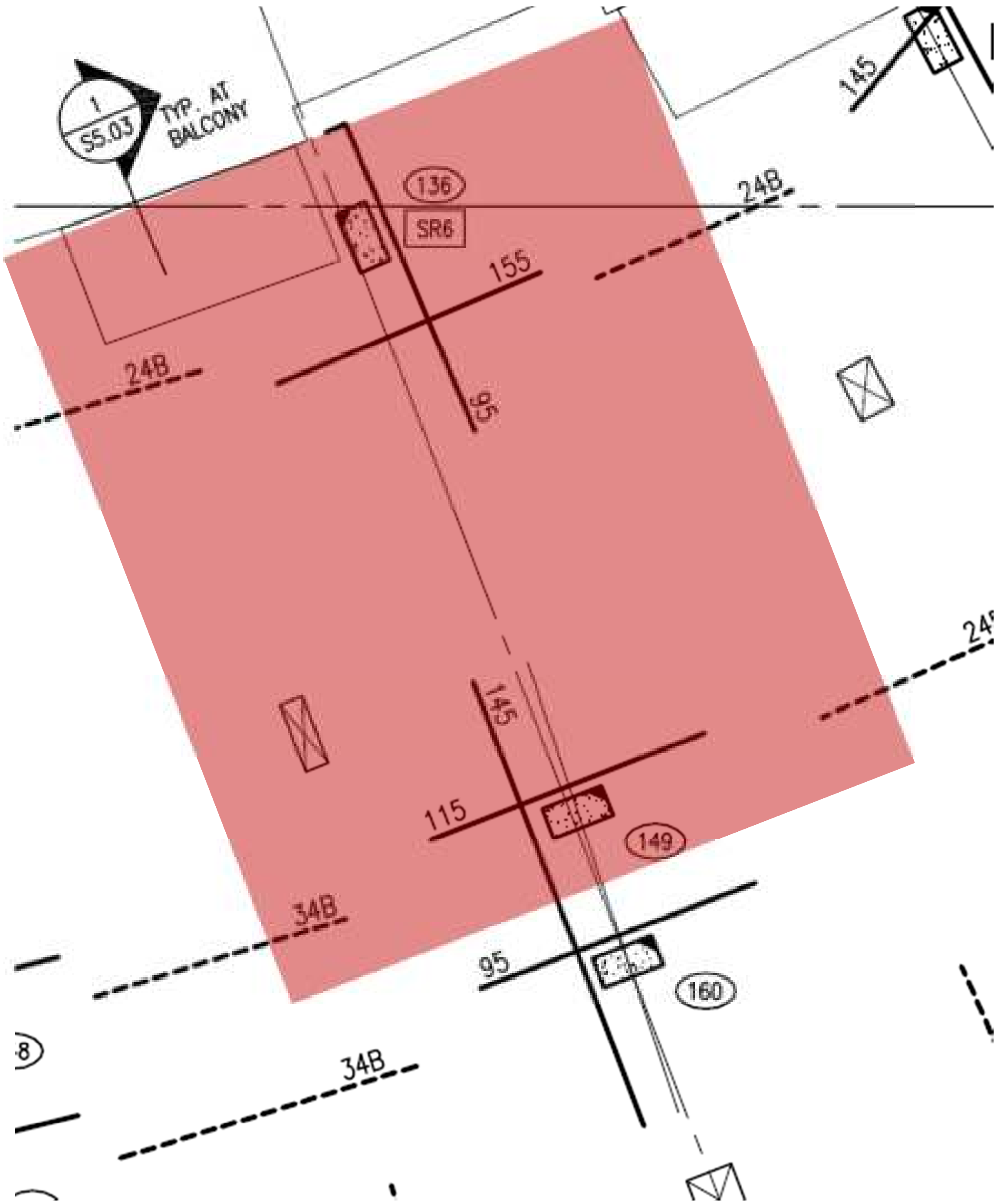


Figure 1-7

The existing moment frame was shown to withstand the controlling seismic load calculated in technical report 3 as per the drift values provide d in Chart 1-1. From the chart it can be seen that the existing structure is well within the allowable limits which might lead one to believe the structure to be over designed. Results from seismic and wind analyses can be found in Appendix C and Appendix D respectively.

Floor	Seismic Drift			Wind Drift		
	x	y	Allowable	x	y	Allowable
Pent. Roof	0.013593	0.065152	4.4400	0.0133	0.013064	0.3700
Mech Roof	0.0144	0.0928	2.0400	0.0122	0.011976	0.1700
Main Roof	0.0174	0.0984	2.4000	0.0108	0.011735	0.2000
11	0.0189	0.0994	2.1600	0.0095	0.010212	0.1800
10	0.0190	0.0984	2.1600	0.0082	0.007719	0.1800
9	0.0187	0.0961	2.1600	0.0070	0.005937	0.1800
8	0.0182	0.0922	2.1600	0.0059	0.00415	0.1800
7	0.0170	0.0861	2.1600	0.0047	0.003594	0.1800
6	0.0155	0.0776	2.1600	0.0037	0.003482	0.1800
5	0.0133	0.0659	2.1600	0.0027	0.003362	0.1800
4	0.0107	0.0522	2.1600	0.0017	0.003116	0.1800
3	0.0077	0.0355	2.1600	0.0009	0.002684	0.1800
2	0.0029	0.0133	2.6400	0.0003	0.001065	0.2200
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Total	0.1874	0.9731	30.9600	0.0809	0.0821	4.0800

Table 1-1: Existing Drift Values

Live Load Areas	ASCE 7-05 Required Loading		Loads Used By Engineer
Private Rooms	40 psf	ASCE 7-05 Table 4-1	40 psf + 20 psf (Partition Allowance)
Public Rooms/Corridors	100 psf	ASCE 7-05 Table 4-1	100 psf
Tenant Storage	125 psf	ASCE 7-05 Table 4-1	125 psf
Roof	20 psf	ASCE 7-05 Table 4-1	30 psf
Stairways	100 psf	ASCE 7-05 Table 4-1	100 psf
Balconies	100psf	ASCE 7-05 Table 4-1	-
Theater	60 psf	ASCE 7-05 Table 4-1	-
Garage	40 psf	ASCE 7-05 Table 4-1	50 psf
Plaza	100 psf	ASCE 7-05 Table 4-1	350 psf
Mechanical	-		150 psf
Elevator Machine Room	-		125 psf

Table 1-2: Gravity Live Loads

Dead Loads	Load Values
Floor Finish	16 psf
Slab: B3 - 1	109 psf
Slab: 2 - Main Roof	85 psf
MEP	15 psf
Steel	15 psf
Miscellaneous	10 psf
Roof Waterproofing	5.5 psf

Table 1-3: Gravity Dead Loads

Applicable Codes

Codes Used for Original Design

- International Building Code 2000
- Arlington County Building Code
- American Concrete Institute (ACI 318 and ACI 301)
- American Society for Testing and Materials
- American Institute of Steel Construction Manual

Codes Implemented for Thesis Analysis

- American Society of Civil Engineers (ASCE 7-05)
- International Building Code 2006

Resources

- AISC Steel Construction Manual (13th Edition)
- ACI 318 – 08
- RS Means Costworks

Materials and Properties

Concrete

- Footings 3000 psi
- Retaining Wall Footings 5000 psi
- Foundation Walls
    - B3 and B3 Ext. Level 5000 psi
    - B2 and B1 Level 4000 psi
    - Site Retaining Wall 5000 psi
- Formed Slabs and Beams 5000 psi
- Columns 5000, 6000, and 8000 psi
- Slabs on Grade 5000 psi
- Pea-Gravel Concrete 2500 psi
- All Other Concrete 4000 psi

Reinforcing Steel

- Reinforcing Bars ASTM A615
- Welded Wire Fabric ASTM A185
- Reinforcing Bar Mats ASTM A185
- Reinforcing Bars in Garage Slabs ASTM A775

Steel

- Wide Flange Members ASTM A992
- Stiffener Plates ASTM A572
- Other ASTM A36

Proposal

Through the use of large column sections and an irregular arrangement of columns the engineers of SK&A managed to design an unorthodox structural system capable of distributing lateral loads evenly throughout. However, the excessive size of their columns greatly increases the seismic load on the building which, as determined earlier, is the controlling load case for the system. This need for larger columns in turn may have led to the irregular column plan by way of minute changes in column orientation to avoid the impedance of private and public spaces.

The solution to this dilemma is the design of a shear wall system. Shear walls will improve the building by absorbing a large majority of the lateral loads imposed by wind and seismic forces. This reduction in lateral stress will allow for the resizing of columns which will reduce the overall cost of the structural system.

Design Goals

The ultimate goal of this thesis is to explore the feasibility of redesigning the lateral system in a way that would decrease the overall building weight while simultaneously reducing the story drift of the building. All this is to be achieved without sacrificing too much of the current floor plan or the owner's money.

Design Process

The first step in the process of designing a shear resisting system was material selection. Initially, the idea of a braced steel frame system was entertained given the lightweight aspect of steel as opposed to concrete. Furthermore, a steel braced frame would also be advantageous given the relatively short amount of time it takes to erect one whereas a concrete frame requires curing, shoring, formwork, and finishing.

But with regard to Falls Church Tower, steel does possess some disadvantages. To begin with, there is a certain level of difficulty presented when it comes to forging a connection between steel and concrete. Additionally, the shape of the building prevents it from wholly adhering to a definable grid system. Without a proper layout the steel becomes susceptible to lateral torsional buckling. Finally the incorporation of a steel structure would most certainly increase the height of the building thus increasing the overall cost by a significant amount in addition to increasing the effect of wind pressure loads.

With a concrete shear wall system the building's height remains the same thus keeping the cost down. Additionally a cast in place concrete shear wall system will improve the serviceability of the post-tensioned slab by giving it something to tie into. When it comes to concrete it is far easier to manipulate in a space where columns need to be hidden.

Design Considerations

Having chosen to design a concrete shear wall as the alternative lateral force resisting system the part of the next part of the design process required a re-evaluation of the column layout of the entire building. It was this phase that brought together the structural depth and the architectural breadth portion of the design.

The structural aspects involved in designing a column layout included making sure that the distance between adjacent columns was less than or equal to the span limit for 7" thick post tensioned slab. If not the slab critical section would have to be redesigned to span the larger distance most likely resulting an increase in the slab depth and overall height of the building which is counter intuitive to the goals of this thesis. The architectural aspects of the design of the column layout dealt primarily with locating the columns one of two different grids which will be covered in more detail throughout the architectural breadth portion of the study.

Strength Checks

After defining an acceptable column layout it was necessary to perform strength checks for the three most typical column locations in the building. These locations are defined as exterior, interior and "large area" interior placements which are depicted in the figures below. The "large area" interior columns differ (as their title suggests) from the other interior columns in that they are not doubled up in the center of the building. Each of these typical columns were sized using the excel spreadsheets on the following page.

Figure 2-1:
Typical Exterior
Column

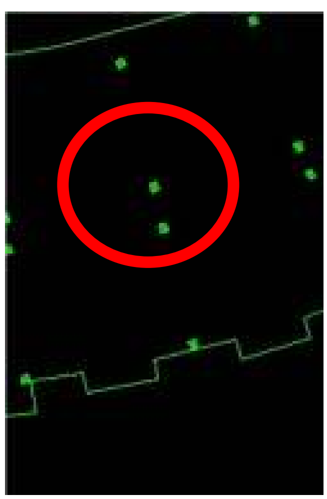


Figure 2-2:
Typical Interior
Column

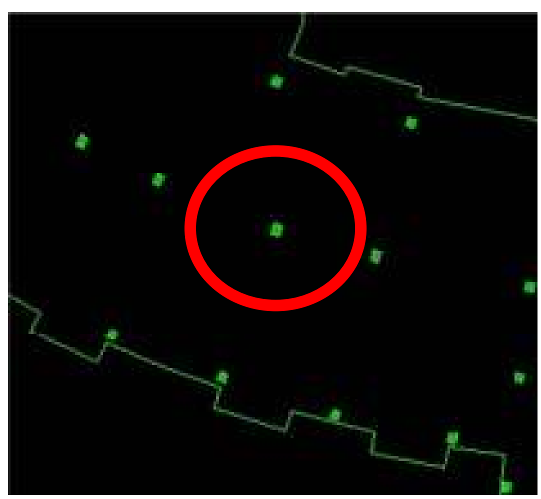


Figure 2-3:
Typical "Large Area"
Interior Column

Floor	Pu (Kips)	ΦPn (Kips, f'c=5)	Ag (sf)	Ag (in^2)	Width (in)	Length (in)
1	1372.20	1421.42	3.56	512.00	16	32
2	1247.45	1292.72	3.33	480.00	16	30
3	1122.70	1151.28	2.89	416.00	16	26
4	997.96	1022.58	2.67	384.00	16	24
5	873.21	881.14	2.22	320.00	16	20
6	748.46	752.44	2.00	288.00	16	18
7	623.72	681.72	1.78	256.00	16	16
8	498.97	540.28	1.33	192.00	12	16
9	374.22	434.20	1.00	144.00	12	12
10	249.48	434.20	1.00	144.00	12	12
11	124.73	434.20	1.00	144.00	12	12

Table 1-4:
Typical Exterior Column

Floor	Pu (Kips)	ΦPn (Kips, f'c=5)	Ag (sf)	Ag (in^2)	Width (in)	Length (in)
1	1288.30	1292.72	3.33	480.00	16	30
2	1171.14	1222.00	3.11	448.00	16	28
3	1053.98	1080.56	2.67	384.00	16	24
4	936.82	1022.58	2.67	384.00	16	24
5	819.66	881.14	2.22	320.00	16	20
6	702.51	752.44	2.00	288.00	16	18
7	585.35	611.00	1.56	224.00	14	16
8	468.19	540.28	1.33	192.00	12	16
9	351.03	434.20	1.00	144.00	12	12
10	233.88	434.20	1.00	144.00	12	12
11	116.72	434.20	1.00	144.00	12	12

Table 1-5:
Typical Interior Column

Floor	Pu (Kips)	ΦPn (Kips, f'c=5)	Ag (sf)	Ag (in^2)	Width (in)	Length (in)
1	1778.92	1845.74	4.89	704.00	16	44
2	1617.38	1633.58	4.22	608.00	16	38
3	1455.84	1504.88	4.00	576.00	16	36
4	1294.31	1363.44	3.56	512.00	16	32
5	1132.77	1222.00	3.11	448.00	16	28
6	971.24	1022.58	2.67	384.00	16	24
7	809.70	881.14	2.22	320.00	16	20
8	648.17	681.72	1.78	256.00	16	16
9	486.64	540.28	1.33	192.00	12	16
10	325.11	434.20	1.00	144.00	12	12
11	163.58	434.20	1.00	144.00	12	12

Table 1-6:
Typical "Large Area" Interior Column

When performing the strength checks for these columns it was important to start at the 11 floor and work down to be sure to get the smallest size possible for each column while following certainly dimensional criteria. This criteria made sure that the column could be properly stacked from floor to floor without producing and major eccentricities and therefore any unnecessary moments. An initial hand calculation is shown in Appendix E. In which the demand load Pu was set equal to the nominal strength equation as follows:

$$Pu \leq \varphi(0.8)(0.65)[0.85f'c(Ag - 0.0018Ag) + fy(0.0018)Ag]$$

The initial result of this calculation produced a column size of 40 in^2 which is only capable of resisting a shear force of around 0.382 K (assuming allowable reinforcing). To avoid the failure of any columns from shear a minimum column size was established based on the existing column sizes and the resultant minimum column size chosen was a 12"x 12" with 4#9 bars. Starting with this column size and proceeding to the subsequent floors using the spreadsheet allowed for minor alterations in column sizes until the necessary strength was achieved.

After the design of the typical columns was complete the main roof columns supporting the penthouse roof and the mechnanical roof were sized according to the same criteria used for the typical columns. Hand calculation for these columns can be referenced in Appendix E.

Once all all of the columns were sized AutoCAD was used to consruct a simple model of the building's new structural system. The model itself was not very extensive, just lines to represent columns and flat plane areas represent floor slabs. This model was then imported into ETABS as a .dxf and assigned material and section properties as appropriate.

Shear Wall Design

The design of the shear wall system was a trial and error process involving multiple layouts. The criteria for this part of the study included basic shear wall placement considerations and the design standards specified in Chapter 14 of ACI 318-08. Due to the facts that Falls Church Tower has such an unorthodox shape and the floor area decreases as the building's height increases, the most efficient way to design the shear walls was to model multiple layouts until the resulting story drift values were less that the drift values of the existing structure. Stength checks would then be done for the layout that produced acceptable drift values and modifications would be made as necessary.

In order to reduce the impact on the existing floor plan it was decided that the initial placement of shear walls would occur around the elevator shafts and stair wells with additional walls being placed at other locations if the need arose. When deteming the location of shear walls there are a few considerations that must be kept in mind.

The shear walls must resist not only loads in the X and Y directions, but also the loads produced from torsional effects. For example, if shear walls are placed too close to the center of rigidity they wont be able to resist loads procduced by eccentrically applied forces. This is because the the moment arm from the center of rigidity to the wall is not large enough as shown in Figure 3- 1.

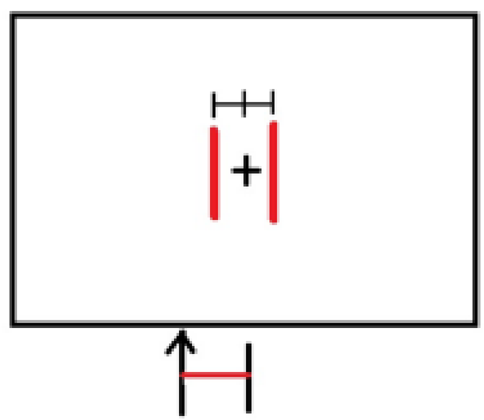


Figure 3-1

One the other hand, if the walls are placed too far from the center of rigity the magnitude of the diaphram deflection will exceed serviceability limitations as depicted in Figure 3-2.

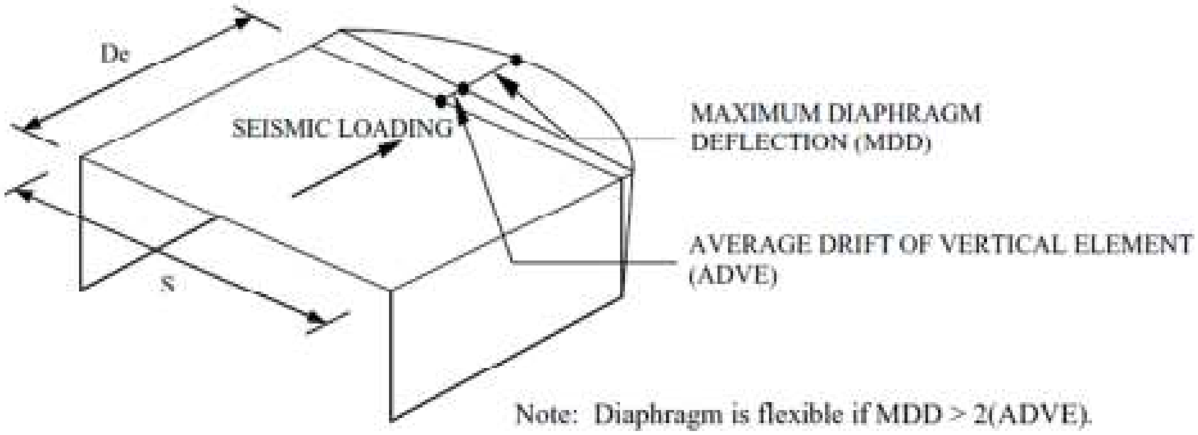


Figure 3-2 (from ASCE7-05)

It is because of these considerations that shear walls are commonly placed around stairwells and elevator shafts as was done with Falls Church Tower. Throughout the design process many layouts were produced for the shear walls keeping in mind the aforementioned criteria. The following figures show the different layouts tested.

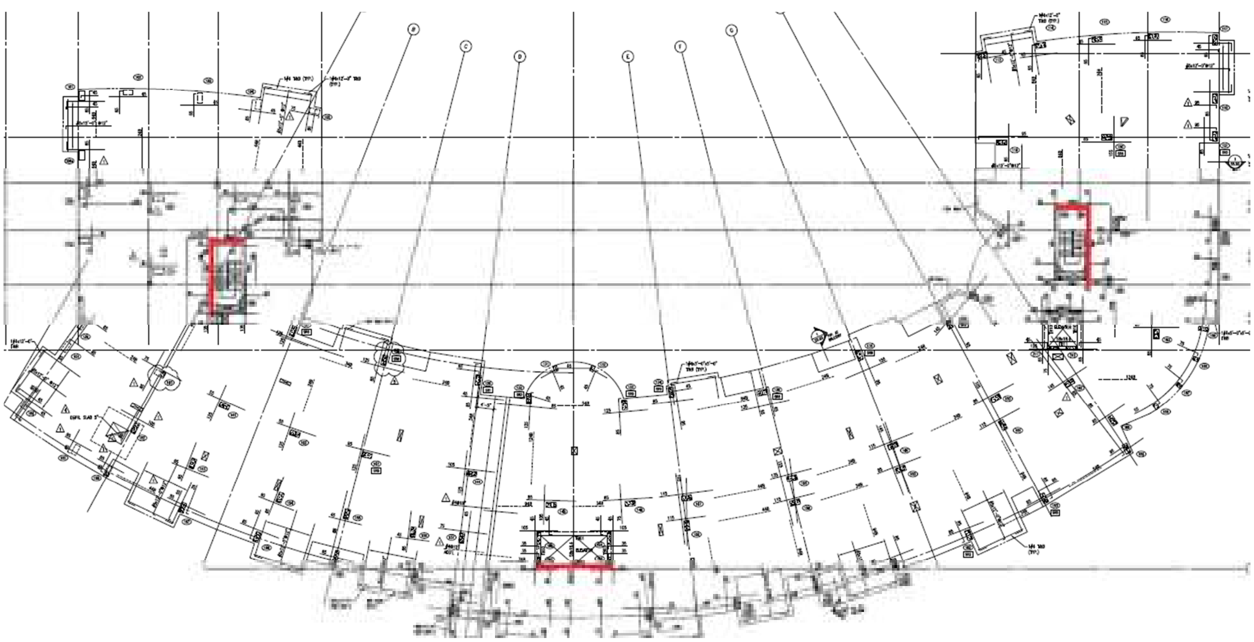


Figure 3-3: Layout 1

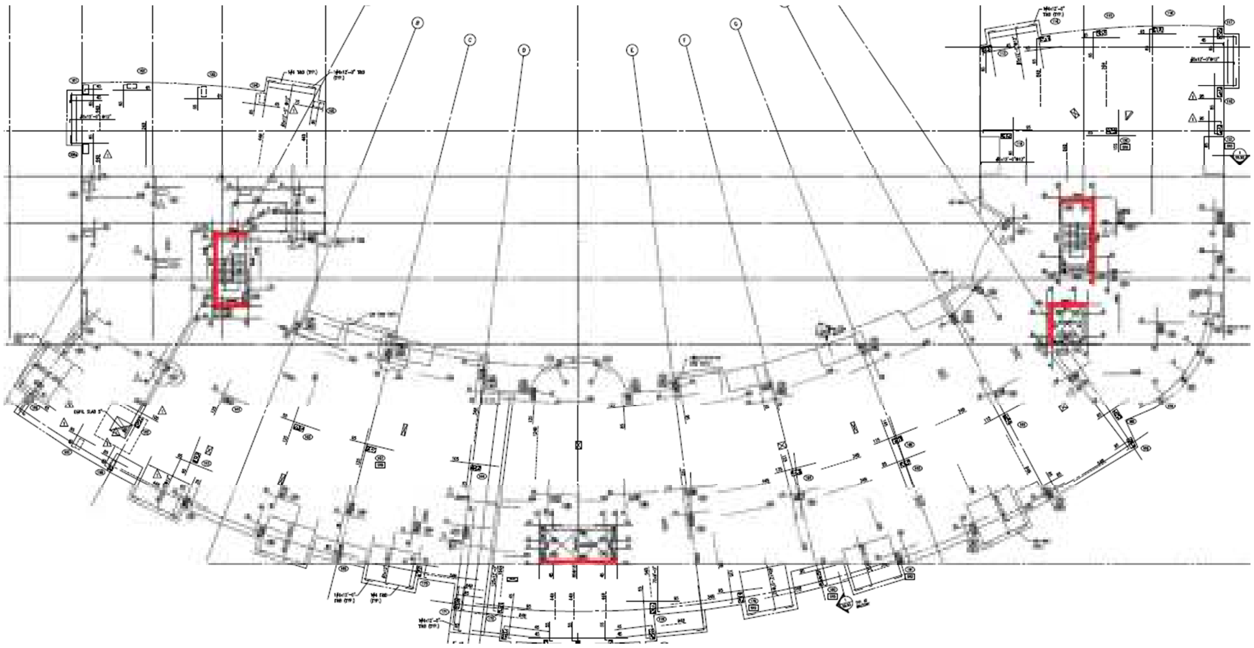


Figure 3-4: Layout 2

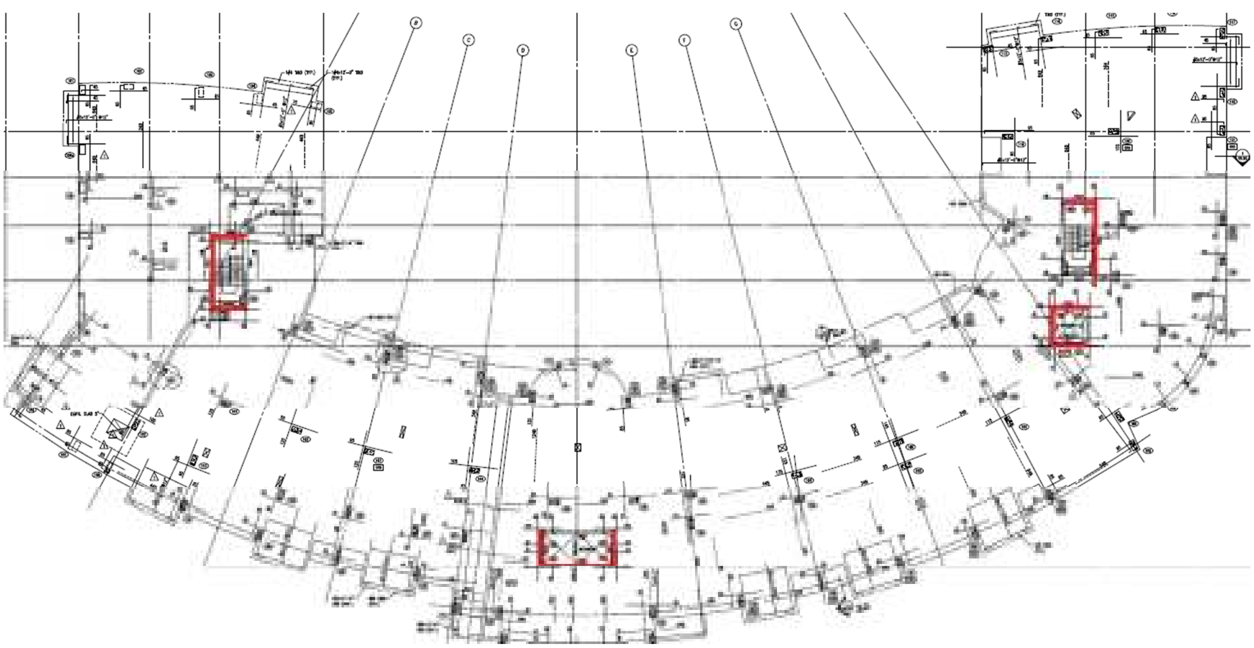


Figure 3-5: Layout 3

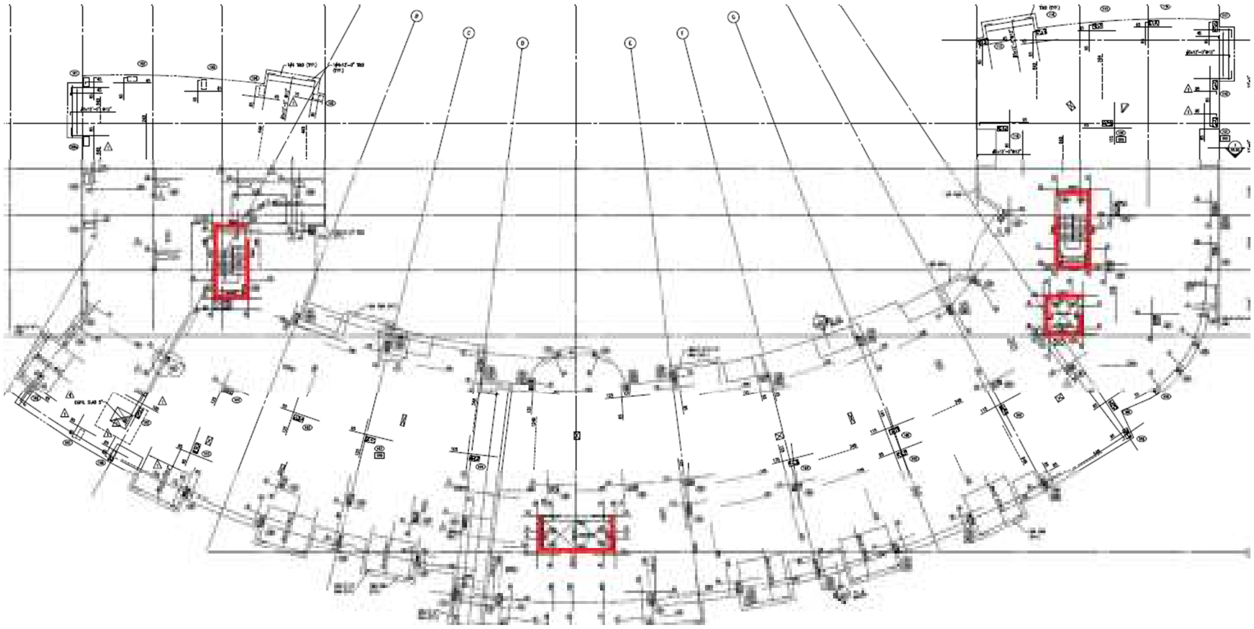


Figure 3-6: Layout 4

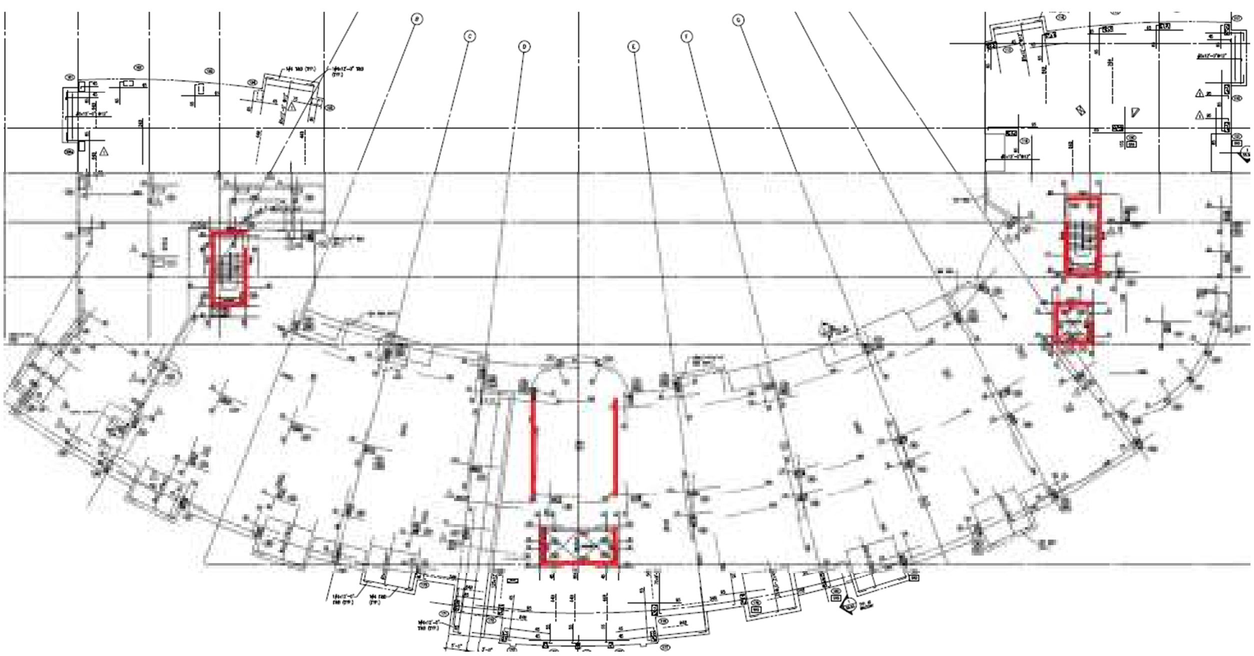


Figure 3-7: Layout 5

It was with Layout 5 that the story drifts were less than those of the existing structure. A comparison between the existing drift values and the revised drift values can be viewed in the tables below. The load cases applied to the structure were seismic in both directions and wind in both directions. Seismic design was done through ETABS with some amendments made to the design done in technical report 3. The change in the lateral force resisting system prompted a change in the response modification factor (R) used to determine the base shear of the building. The response modification factor change from 3 to 1½ increases the base shear of the buidling by a factor of two when compared to an ordinary concrete moment frame. As with technical report 3 the seismic loading in the Y direction controlled. Because the height of the building has remained the same the design wind pressures are those of technical report 3. Details for seismic and wind pressures can be found in Appendix D and Appendix C respectively.

After determining the layout of the shear walls hand calculations were performed according to Chapter 14 of ACI 318-08 to ensure the walls possessed the strength to support factored floor loads. These calculations can be found in Appendix F. The shear walls were verifiedto be 12 inches thick with #9 reinforcing spaced at 12 inches on center.

A stress analysis was then run through ETABS to determine the magnitude of shell stresses in the shear walls from lateral loads. Figure 3-8 shows the distribution of maximmum stresses throughout the shear wall system. Yellow shades indicate areas of little or no stress whereas shades of purple indicate areas of high stress. The stresses increase linearly across the spectrum from yellow to purple. The results from these checks showed that the walls of Layout 5 are indeed adequate to support the lateral loads.

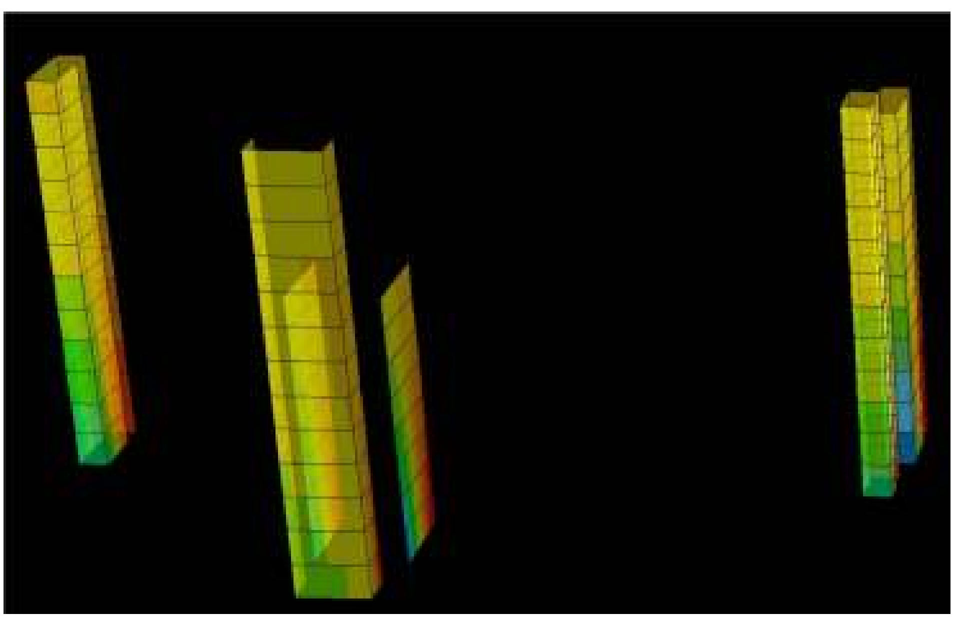


Figure 3-8: Stress Diagram

Seismic Story Drift of Existing				
Floor	Y (in)	X (in)	Total Y	Total X
Pent. Roof	0.065	0.014	0.065	0.014
Mech. Roof	0.093	0.014	0.158	0.028
Main Roof	0.098	0.017	0.256	0.045
11	0.099	0.019	0.356	0.064
10	0.098	0.019	0.454	0.083
9	0.096	0.019	0.550	0.102
8	0.092	0.018	0.642	0.120
7	0.086	0.017	0.729	0.137
6	0.078	0.015	0.806	0.153
5	0.066	0.013	0.872	0.166
4	0.052	0.011	0.924	0.177
3	0.036	0.008	0.960	0.184
2	0.013	0.003	**0.973**	**0.187**

Table 1-7:
Seismic Drift Values for Existing Structure

Seismic Story Drift of Revised				
Floor	Y (in)	X (in)	Total Y	Total X
Pent. Roof	0.052	0.088	0.052	0.013
Mech. Roof	0.057	0.052	0.108	0.025
Main Roof	0.059	0.094	0.167	0.037
11	0.060	0.099	0.228	0.047
10	0.060	0.099	0.288	0.055
9	0.059	0.097	0.346	0.061
8	0.057	0.095	0.403	0.065
7	0.053	0.089	0.457	0.068
6	0.048	0.081	0.505	0.072
5	0.041	0.070	0.546	0.075
4	0.033	0.055	0.579	0.078
3	0.022	0.037	0.601	0.081
2	0.009	0.015	**0.610**	**0.082**

Table 1-8:
Seismic Drift Values for Revised Structure

Breadth 1: Architecture

As previously mentioned the structural depth and the architectural breadth experienced some overlap in the design process. Part of the design process involved locating the columns in a way that adhered to a grid. In this case two grids were used to place the columns. One being a radial grid and the other being a rectangular grid. The radial grid encompassed a majority of the curved portion of the building as shown below while the rectangular grid was located at the core of the building as well as the east and west arms.

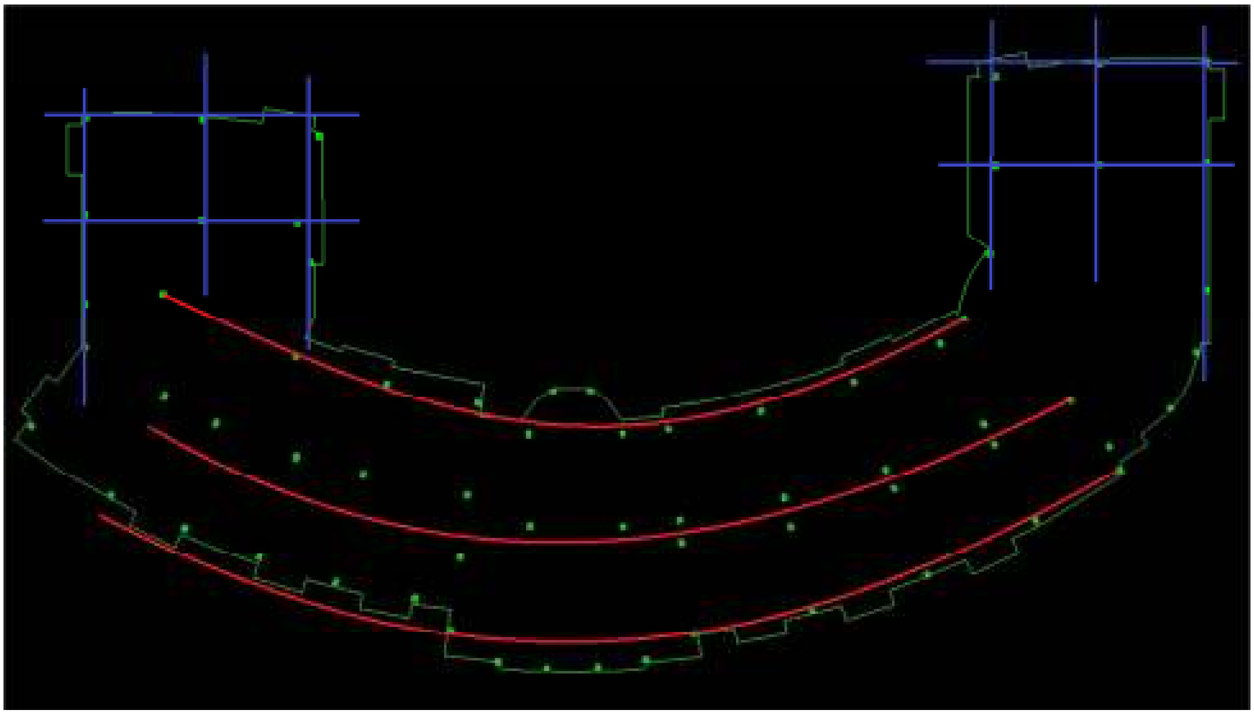


Figure 4-1: Grid Scheme

It was initially thought that the revision of the column layout would have a significant effect on the floor plan of the building. However, the revised layout had very little effect on the building as a whole. As shown below a few partition walls will have to be moved and the balcony on the north face of the west arm of the building will have to be moved but in either case there is no major redesign required.

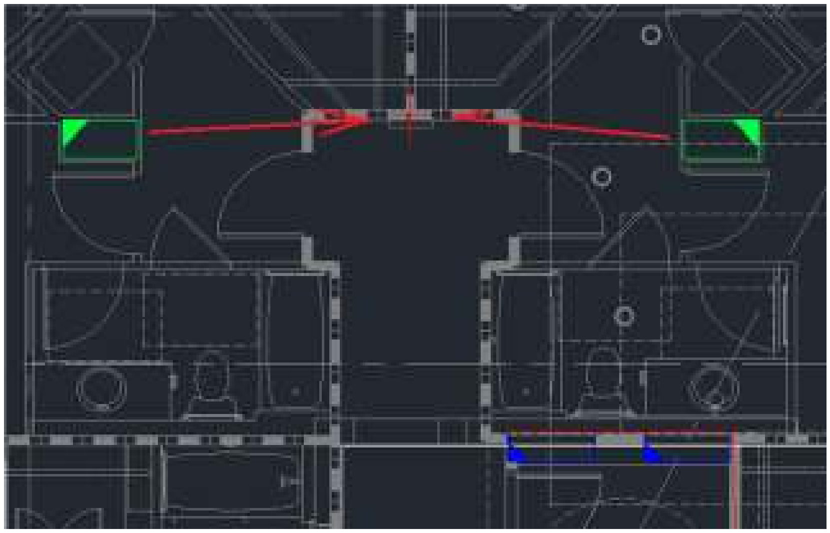


Figure 4-2

Figure 4-3

The only significant impact the revision of the column layout had was on the southern façade in the center of the building. The exisitng layout has five columns across the face of the façade whereas the revised layout only has four. The revised façade can viewed below in Figure 4-4. The reduction of the number of columns increases the area of glazing from an average of 270.5 square feet to 311.35 square feet per floor. This increase in glazing affects the overall cost of the building albeit a negligable amount of $586.19. Additionally the change in glazing will allow more light to enter the space but as with the cost this change in lighting is negligable.

Figure 4-3: Redesigned Facade

Breadth 2: Cost and Scheduling

The purpose of the cost and scheduling breadth study was to determine the impact the redesign of the lateral framing would have on both the overall cost of the building and the duration of system installation. The advantage of this redesign in terms of cost and scheduling is that the material and, for the most part, the system itself does not change. This means that the type and number of crews used during construction will be identical. For the purpose of clarification the exact cost and schedule of the existing structural system could not be obtained due to anonymity constraints by the owner. Therefore a construction cost and scheduling estimate was performed on the existing structure in addition to the redesigned structure.

Cost Estimate

The cost estimate for the exsting stucture and the redesign were performed using RS Means Costworks. The 2010 master format and a commercial labor type were chosen as the criteria for the estimate. Because estimate were done for both the existing and redesigned structure the year of the cost data is not as relevant as it normally would be but even so th 2007 year data was used to get as close as possible to the actual costs during the initial time of construction in 2006.

For both estimates the only elements of the building taken into consideration were the lateral framing systems seeing as those are the only parts of the building that changed. The cost of the slabs (material, placement, and reinforcing steel) was omitted as well since the shape and thickness of the slab didn't change. However, it was necessary to include the formwork for the slabs to account for the fact that the relocation and resizing of the columns had an effect on that aspect.

The cost estimate for the existing structure included material, placement, steel weight, and labor for the columns as well as formwork and labor for both the slabs and the columns. The following table shows the comparison between the final costs of both designs. The redesign of the column layout and the incorporation of a shear wall system reduced the cost of the structural system by nearly $150,000. This is mainly attributed to the reduced material and formwork cost of the alternative system which more than made up for the additional cost of the shear walls.

Element	Existing Structure	Redesigned Structure
Columns	$1,071,333.38	$535,663.77
Slabs	$2,334,217.66	$2,334,574.61
Shear Walls	-	$386,345.68
Total Time	**$3,405,551.04**	**$3,256,584.05**
Difference	**-$148,966.99**	

Table 2-1: Cost Estimates

Schedule

The schedule estimate for the existing structure and the redesigned structure were done using the same means and criteria as the cost estimate. Furthermore, the schedule estimate also took into account the same elements of the existing and redesigned structure as the cost estimate. But for all their similarities the schedule estimate was not simply a gross summation of values as was the cost estimate. Certain activities, such as placing reinforcing steel and erecting column formwork can be overlapped. Based on the processes involved in the schedule estimate it was determined that the only factors that needed to considered were the time required to erect column formwork, the time required to place the concrete for the columns, and the time required to erect slab formwork. The resason for this being the fact the existing structure schedule was estimated and that the whole purpose of this breath study is to compare the different schedules. The results of the schedule estimates are shown below with the redesigned lateral system reducing the construction time by 276 days. This can be attributed mainly to the reduction of the total column formwork.

	Time Required (Days)	
Element	**Existing Structure**	**Redesigned Structure**
Column Formwork	515	243
Column Concrete	15	11
Slab Formwork	617	617
Total Time	**1,147**	**871**
Difference	**-276**	

Table 2-2: Schedule Estimates

Conclusion

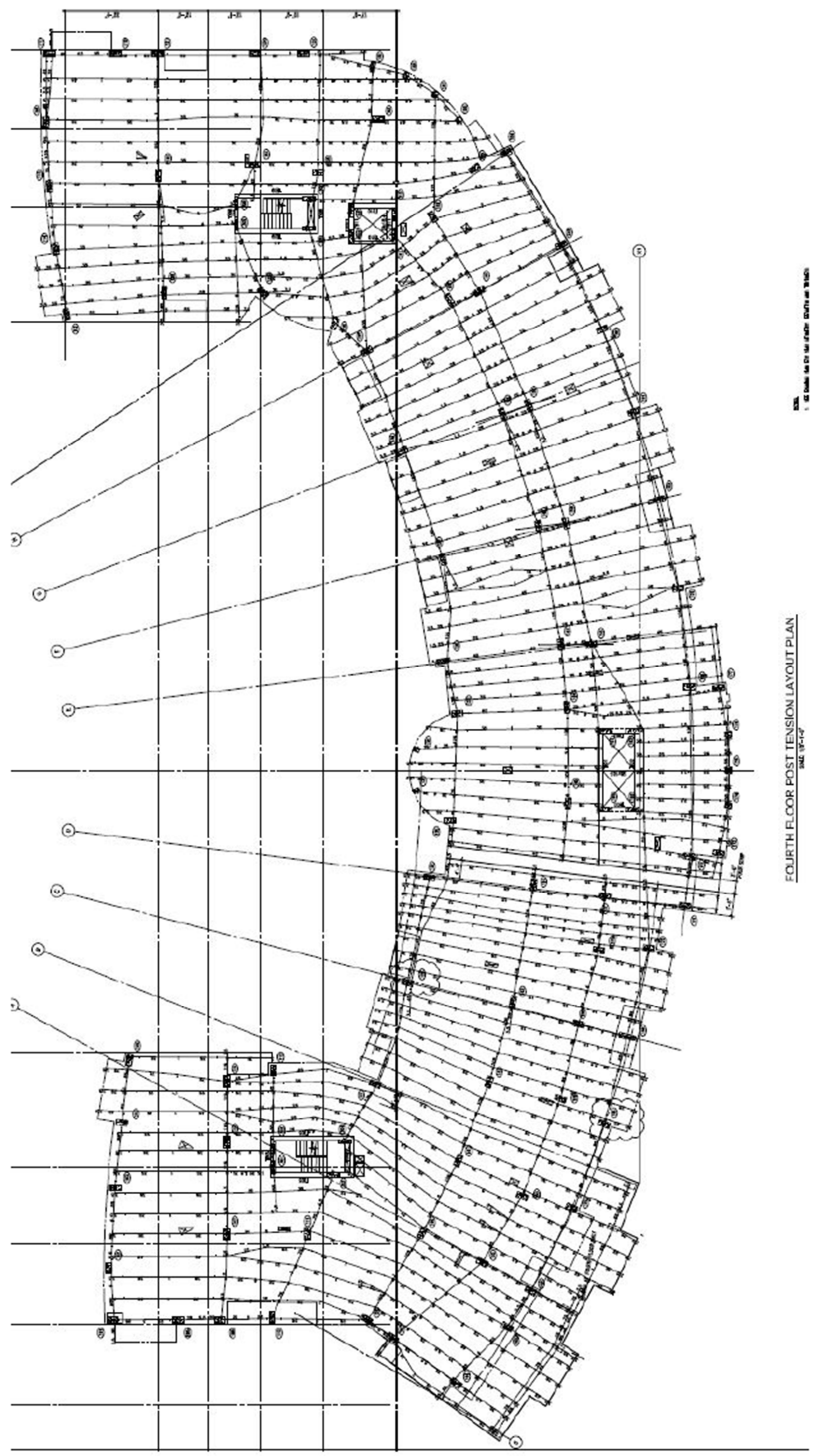
The depth study explored the option of altering the existing lateral force resisting system by designing a new column layout, reducing the size of the columns, and incorporating shear walls into the structure. This change reduced the overall weight of the structural system but also the reduction modification factor which increases the base shear by a factor of 2. Hand calculations wre done to determine the size of the columns in the new layout and an ETABS model was used to model and analyze several different shear wall layouts. The fifth layout proved to be the one capable of resisting the required lateral and gravity loads. As before the seimic loads controlled over the wind loads in terms of deflection and were used as the basis for the design. The shear walls of the fifth layout are located around the central and west elevator cores, the east and west stairwells, and running perpendicular to the face of the central elevators.

Two breadth studies were done in addition to the depth study. The architectural breadth study was done simultaneously with the depth due to the fact that a change in the column layout could have an effect on the floorplan of the building. As it turns out the impact on the floor plan was minimal in that at most a few partition walls would have to be shortened or lengthened a small amount. The only significant effect the revised column layout had on the architeture of the building was the southern glass façade. The five column originally located along this façade were reduced to four thus increasing the square footage of glazing.

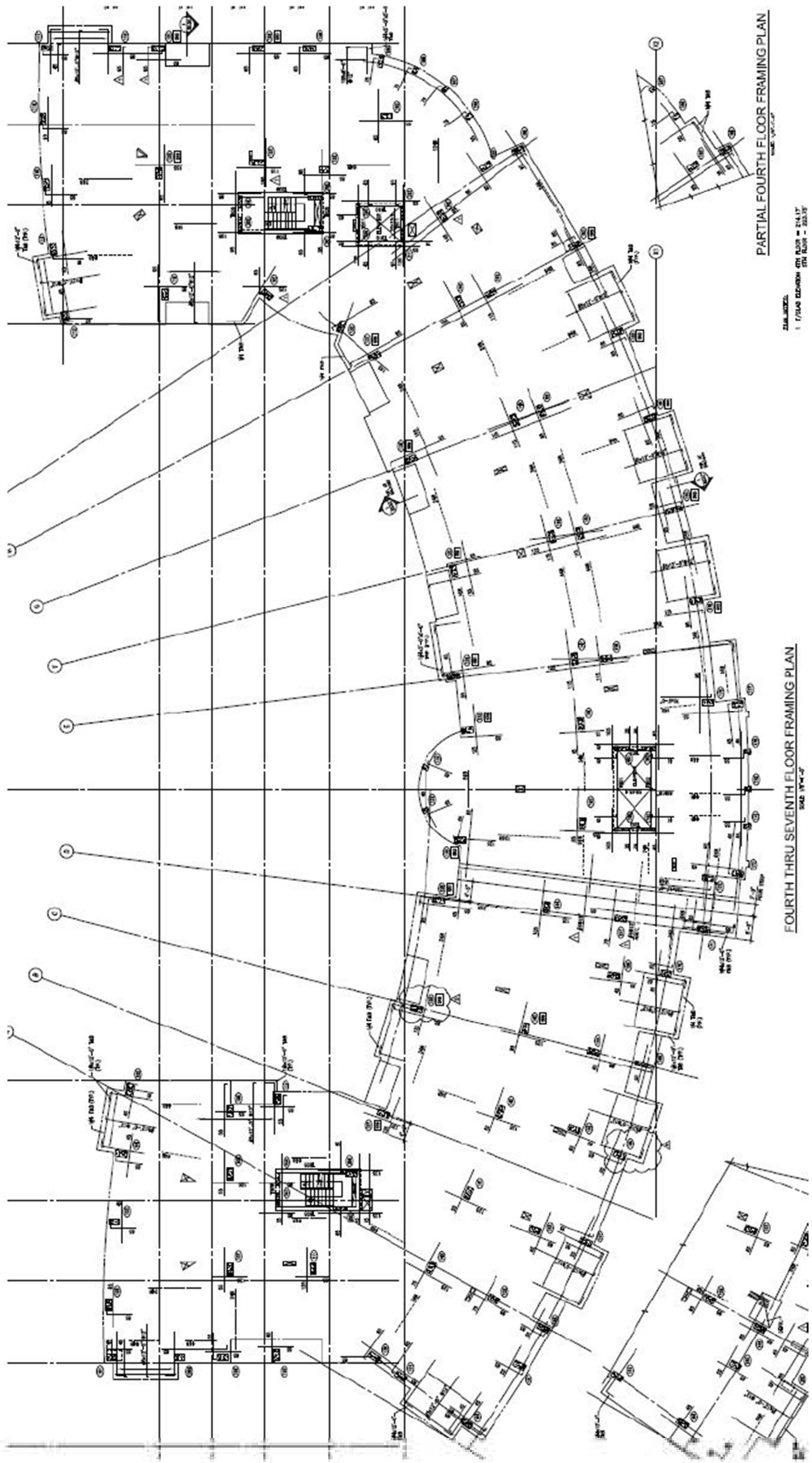
The cost and schedule analysis were performed using RS Means Costworks which is software offered on the RS Means website that aids in building cost estimation. Using the unit prices provided by this program a cost estimate and a schedule estimate were produced for both the redesigned structure and the old structure. From the cost estimates it was determined that the shear wall sysem would reduce the overall price of the building by nearly $150,000. The scheduling estimate also produced satisfactory results showing that the shear wall system would decrease overall construction time by 276 days. Given the results of the depth study and both breadth studies it is clear that the goals of this project were well met.

Appendix

Appendix A: Existing Structure



Typical Post Tension Layout



Typical Column Layout

Appendix B: Building Weight Table

Floor	Ext. Wall Weight (K)	Column Weight (K)	Slab Weight (K)	Beam Weight (K)	Area (sf)	Add. Dead Load (psf)	Total Weight (K)
Penthouse Roof	54.95		170.67	100.42	2354.00	15.50	362.52
Mech. Roof	9.82		20.95	100.42	289.00	15.50	135.67
Main Roof	171.83	18.68	1450.35	44.29	17147.00	15.50	1950.93
11	264.51	71.03	1703.00	44.29	20134.00	26.00	2606.32
10	268.95	71.31	1711.80	44.29	20238.00	26.00	2622.53
9	293.11	80.20	2288.15	44.29	27052.00	26.00	3409.09
8	317.27	107.38	2288.15	44.29	27052.00	26.00	3460.44
7	322.58	149.51	2433.97	44.29	28776.00	26.00	3698.53
6	327.88	195.49	2433.97	44.29	28776.00	26.00	3749.81
5	327.88	217.58	2433.97	44.29	28776.00	26.00	3771.89
4	327.88	256.85	2433.97	44.29	28776.00	26.00	3811.17
3	330.24	277.91	2384.66	44.29	28193.00	26.00	3770.12
2	360.99	316.93	2452.24	44.29	28992.00	26.00	3928.24
1	388.02	396.07	3339.50	44.29	30708.00	26.00	4966.29
B1	805.98	751.71	6072.17	182.00	55836.00	10.00	8370.22
B2	939.41	764.72	5827.59	167.73	53587.00	10.00	8235.32
B3	886.78	615.07	5038.61	206.70	46332.00	10.00	7210.47
B3 Ext.	222.29	286.48	809.46	33.53	13398.00	10.00	1485.75
Total Building Weight							**67545.30**

Appendix C: Wind Loads

Nathan Eck | Wind Loads | 9-27-10 | 1

Location: Arlington, VA
Residential Building:
Topography: Homogeneous

Basic Wind Speed	Exposure	Building Classification
$V = 90$ mph	B	Category II

Velocity Pressure (q_z)

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$I = 1.00$ from Table 6-1
$K_d = 0.85$ from Table 6-4
$V = 90$ mph from Figure 6-1

Height Above Ground Level, z	K_z*
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
180	1.17
200	1.20
250	1.28
300	1.35
350	1.41
400	1.47
450	1.52
500	1.56

*K_z values per level were determined through interpolation and can be referenced on the q_z value chart

$K_{zt} = 1.00$ based on Section 6.5.7

*refer to chart for q_z values

2

Gust Effect Factor

n_1 = natural frequency $= \frac{100}{H}$; $H = 136'4''$

$n_1 = \frac{100}{136.33} = \boxed{0.734\ Hz} < 1\,Hz$ ∴ the building is flexible

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

$g_v = \boxed{3.4}$

$g_Q = \boxed{3.4}$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$$g_R = \sqrt{2\ln(3600)(0.734)} + \frac{0.577}{\sqrt{2\ln(3600)(0.734)}}$$

$g_R = 3.970 + 0.145 = \boxed{4.12}$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$; $\bar{z} = \left.0.6h\right|_{max\ z_{min}}$; $z_{min} = 30\,ft$, $h = 117.83\,ft$

$\bar{z} = \left.70.7\right|_{max\ 30}$ ⇒ $\bar{z} = \boxed{70.7}$

$c = 0.30$

$I_{\bar{z}} = 0.3\left(\frac{33}{70.7}\right)^{1/6} = \boxed{0.264}$

$R = \sqrt{\frac{1}{\beta}R_n R_h R_B(0.53+0.47R_L)}$; $R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}}$; $N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$; $n_1 = 0.734$, $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

β assumed 1.5% or 0.015

$R_n = \frac{7.47(4.21)}{(1+10.3(4.21))^{5/3}} = \boxed{0.057}$

$N_1 = \frac{(0.734)(412.53)}{71.86}$; $N_1 = \boxed{4.21}$

$L_{\bar{z}} = 320\left(\frac{70.7}{33}\right)^{1/3} = \boxed{412.53}$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{70.7}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = \boxed{71.86}$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$; $\eta = 4.6 n_1 (h/\bar{V}_{\bar{z}}) = 4.6(0.734)(117.83/71.86) = \boxed{5.54}$

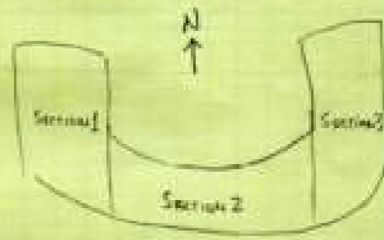
$= \frac{1}{5.54} - \frac{1}{2(5.54^2)}(1-e^{-2(5.54)})$

$= \boxed{0.164}$

3

Note: In order to obtain values for B and L in the calculation of R_B and R_L the building will be divided into three sections. The values for B and L will be taken for each section, assuming they are all rectangular for ease of calculation. This will be done for both N-S and E-W directions. The three sections will be specified on floors 2-7 seeing as these floors are the most typical layout for the building.

Section	Variable	N-S	E-W
1	B_1	65'	118'
	L_1	118'	65'
2	B_2	174.18'	66'
	L_2	66'	174.18'
3	B_3	65'	104'
	L_3	104'	65'



R_B Values

Section 1

N-S

$R_B = R_h$ for $\eta = 4.6 n_1 B / \bar{V}_z$

$= 4.6(0.734)(65/71.86)$

$= 3.05$

$$R_B = \frac{1}{3.05} - \frac{1}{2(3.05^2)}\left(1 - e^{-2(3.05)}\right)$$

$= 0.274$

E-W

$\eta = 4.6(0.734)(118/71.86) = 5.54$

$R_B = 0.164$

Section 2

N-S

$\eta = 4.6(0.734)(174.18)/71.86$

$= 8.18$

$R_B = 0.115$

E-W

$\eta = 4.6(0.734)(66/71.86)$

$= 3.10$

$R_B = 0.271$

Section 3

N-S

$\eta = 4.6(0.734)(65/71.86)$

$= 3.05$

$R_B = 0.274$

E-W

$\eta = 4.6(0.734)(104/71.86)$

$= 4.89$

$R_B = 0.184$

4

R_L Values

Section 1

N-S

$R_L = R_n$ for $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$

$= 15.4(0.734)(118/71.86)$

$= 18.56$

$R_L = \boxed{0.052}$

E-W

$\eta = 15.4(0.734)(65/71.86)$

$= 10.22$

$R_L = \boxed{0.093}$

Section 2

N-S

$\eta = 15.4(0.734)(66/71.86)$

$= 10.38$

$R_L = \boxed{0.092}$

E-W

$\eta = 15.4(0.734)(174.15/71.86)$

$= 27.4$

$R_L = \boxed{0.036}$

Section 3

N-S

$\eta = 15.4(0.734)(104/71.86)$

$= 16.36$

$R_L = \boxed{0.059}$

E-W

$\eta = 15.4(0.734)(65/71.86)$

$= 10.72$

$R_L = \boxed{0.093}$

*Refer to R value chart for R values for each section

Background Response

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$$

Section 1

N-S

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{65+117.83}{417.53}\right)^{0.63}}}$$

$Q = \boxed{0.852}$

E-W

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{118+117.83}{417.53}\right)^{0.63}}}$$

$Q = \boxed{0.832}$

Section 2

N-S

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{174.15+117.83}{418.53}\right)^{0.63}}}$$

$Q = \boxed{0.815}$

E-W

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{66+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.852}$

Section 3

N-S

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{65+117.83}{417.53}\right)^{0.63}}}$$

$Q = \boxed{0.852}$

E-W

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{104+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.837}$

5

Gust Effect Factor

$$G_f = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_{\bar{z}}}\right)$$

* Largest Gust Factor controls

Section 1

N-S: $$G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.852^2)+(4.12^2)(0.306^2)}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.886}\ *$$

E-W: $$G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.836)^2+(4.12)^2(0.242)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.859}$$

Section 2

N-S: $$G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4^2)(0.815^2)+(4.12)^2(0.203)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.842}$$

E-W: $$G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4^2)(0.856)^2+(4.12)^2(0.304)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.855}\ *$$

Section 3

N-S: $$G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.852)^2+(4.12)^2(0.309)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.886}\ *$$

E-W: $$G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.837)^2+(4.12)^2(0.256)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.855}$$

External Pressure Coefficients (Figure 6-6)

C_p: Windward = 0.8

Leeward:

	N-S	E-W
Section 1	-0.34	-0.5
Section 2	-0.5*	-0.27
Section 3	-0.38	-0.5*

* These values control for their respective directions

Internal Pressure Coefficient (Figure 6-5)

$GC_{pi} = \pm 0.18$

6

Design Wind Pressures

Windward: $p = q_z G C_p - q_h (GC_{pi})$; $q_z = 10.05$

Sample Calc: $p = (10.05\,psf)(0.886)(0.8) - 18.24(-0.18) = 10.41\,psf$

Leeward: $p = q_h G C_p - q_h (GC_{pi})$; $q_h = 18.24$

$p = (18.24)(0.886)(-0.5) + 18.24(-0.18) = -11.36\,psf$

Floor	Height Above Ground (ft)	K_z	K_{zt}	K_d	V	I	q_z (psf)
B1	0.00	0.570	1.00	0.85	90.00	1.00	10.05
1.000	10.00	0.570	1.00	0.85	90.00	1.00	10.05
2.000	21.00	0.628	1.00	0.85	90.00	1.00	11.07
3.000	30.58	0.704	1.00	0.85	90.00	1.00	12.41
4.000	40.17	0.761	1.00	0.85	90.00	1.00	13.41
5.000	49.75	0.809	1.00	0.85	90.00	1.00	14.26
6.000	59.33	0.847	1.00	0.85	90.00	1.00	14.93
7.000	68.92	0.886	1.00	0.85	90.00	1.00	15.62
8.000	78.50	0.924	1.00	0.85	90.00	1.00	16.29
9.000	88.08	0.954	1.00	0.85	90.00	1.00	16.81
10.000	97.67	0.983	1.00	0.85	90.00	1.00	17.33
11.000	107.25	1.008	1.00	0.85	90.00	1.00	17.77
Penthouse	118.83	1.035	1.00	0.85	90.00	1.00	18.24

Velocity Pressure Values

R Values						
Section	Direction	Rn	Rh	R_B	R_L	R
1	N-S	0.057	0.164	0.274	0.052	0.308
	E-W	0.057	0.164	0.164	0.093	0.242
2	N-S	0.057	0.164	0.115	0.092	0.203
	E-W	0.057	0.164	0.271	0.036	0.304
3	N-S	0.057	0.164	0.274	0.059	0.309
	E-W	0.057	0.164	0.184	0.093	0.256

R Values

Floor	Height Above Ground (ft)	K_z	q_z (psf)	q_h (psf)	Windward (psf)	Leeward (psf)	Total Pressure (psf)
B1	0.00	0.570	10.05	18.24	10.41	-11.36	21.77
1	10.00	0.570	10.05	18.24	10.41	-11.36	21.77
2	21.00	0.628	11.07	18.24	11.13	-11.36	22.49
3	30.58	0.704	12.41	18.24	12.08	-11.36	23.44
4	40.17	0.761	13.41	18.24	12.79	-11.36	24.15
5	49.75	0.809	14.26	18.24	13.39	-11.36	24.75
6	59.33	0.847	14.93	18.24	13.87	-11.36	25.23
7	68.92	0.886	15.62	18.24	14.35	-11.36	25.72
8	78.50	0.924	16.29	18.24	14.83	-11.36	26.19
9	88.08	0.954	16.81	18.24	15.20	-11.36	26.56
10	97.67	0.983	17.33	18.24	15.57	-11.36	26.93
11	107.25	1.008	17.77	18.24	15.88	-11.36	27.24
MainRoof	117.83	1.035	18.24	18.24	16.21	-11.36	27.58
Mech. Roof	126.33	1.056	18.61	18.24	-	-11.36	27.83
Pent. Roof	136.33	1.081	19.50	18.24	-	-11.36	28.46

Design Wind Pressure

Appendix D: Seismic Loads

Seismic Load

Spectral Response Acceleration Parameters

Soil Site Class: C

$S_s = 0.16$ (ASCE-7 Figure 22-1)

$S_1 = 0.051$ (ASCE-7 Figure 22-2)

$S_{MS} = S_s F_a$; $F_a = 1.2$ (ASCE-7 Table 11.4-1)

$S_{MS} = (0.16)(1.2) = \boxed{0.192}$

$S_{M1} = S_1 F_v$; $F_v = 1.7$ (ASCE-7 Table 11.4-2)

$S_{M1} = (0.051)(1.7) = \boxed{0.087}$

Design Spectral Acceleration Parameters

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.192) = 0.128$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.087) = 0.058$

Seismic Base Shear

$T = C_t h_n^x$; $C_t = 0.016$; $x = 0.9$; $h_n = 137.75 ft$

$T = (0.016)(137.75)^{0.9} = 1.34$ sec

$T_L = 8$ sec (Figure 22-15)

$T_L > T$

$C_s = \frac{S_{DS}}{R/I}$; $R = 1.5$; $I = 1$

$C_s = \frac{0.128}{(1.5/1)} = 0.0853$

$V = C_s W_t$; $W_t =$ [illegible]

$V = (0.0853)($[illegible]$) =$ [illegible]

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$; $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$; $k = 1.67$

Values for $\sum_{i=1}^{n} w_i h_i^k$, $w_x h_x^k$, C_{vx}, and F_x are given in seismic loading chart

C

Floor	Weight (K)	Height (ft)	$w_x h_x^k$	C_{vx}	F_x (K)	Story Shear (K)	Moment (ft-K)
Penthouse Roof	362.52	136.33	68999522.63	0.0073	26.43	-	3602.54
Mech. Roof	135.67	126.33	11769657.81	0.0012	4.51	-	569.43
Main Roof	1950.93	117.83	898875571.84	0.0954	344.25	30.93	40562.70
11	2606.32	107.25	1246034121.90	0.1322	477.20	375.18	51179.81
10	2622.53	97.67	1076873124.91	0.1142	412.42	852.38	40280.71
9	3409.09	88.08	1404275369.42	0.1490	537.80	1264.80	47369.74
8	3460.44	78.50	1187906832.24	0.1260	454.94	1802.60	35712.76
7	3698.53	68.92	1068180026.45	0.1133	409.09	2257.54	28194.29
6	3749.81	59.33	851065939.79	0.0903	325.94	2666.63	19337.88
5	3771.89	49.75	640470132.47	0.0679	245.28	2992.57	12202.91
4	3811.17	40.17	455914098.65	0.0484	174.60	3237.85	7013.85
3	3770.12	30.58	283918480.10	0.0301	108.73	3412.45	3325.08
2	3928.24	21.00	162336606.29	0.0172	62.17	3521.19	1305.59
1	4966.29	10.00	69562161.05	0.0074	26.64	3583.36	266.41
		$\Sigma w_i h_i^k$ =	9426181646	Base Shear =	3610	Overturning Moment =	156093.97
						Resisting Moment =	4868869

Seismic Loads

Appendix E: Column Design

Floor	Pu (Kips)	ΦPn (Kips, f'c=5)	Ag (sf)	Ag (in^2)	Width (in)	Length (in)
1	1288.30	1292.72	3.33	480.00	16	30
2	1171.14	1222.00	3.11	448.00	16	28
3	1053.98	1080.56	2.67	384.00	16	24
4	936.82	1022.58	2.67	384.00	16	24
5	819.66	881.14	2.22	320.00	16	20
6	702.51	752.44	2.00	288.00	16	18
7	585.35	611.00	1.56	224.00	14	16
8	468.19	540.28	1.33	192.00	12	16
9	351.03	434.20	1.00	144.00	12	12
10	233.88	434.20	1.00	144.00	12	12
11	116.72	434.20	1.00	144.00	12	12

Interior Column Design

Floor	Pu (Kips)	ΦPn (Kips, f'c=5)	Ag (sf)	Ag (in^2)	Width (in)	Length (in)
1	1778.92	1845.74	4.89	704.00	16.00	44.00
2	1617.38	1633.58	4.22	608.00	16.00	38.00
3	1455.84	1504.88	4.00	576.00	16.00	36.00
4	1294.31	1363.44	3.56	512.00	16.00	32.00
5	1132.77	1222.00	3.11	448.00	16.00	28.00
6	971.24	1022.58	2.67	384.00	16.00	24.00
7	809.70	881.14	2.22	320.00	16.00	20.00
8	648.17	681.72	1.78	256.00	16.00	16.00
9	486.64	540.28	1.33	192.00	12.00	16.00
10	325.11	434.20	1.00	144.00	12.00	12.00
11	163.58	434.20	1.00	144.00	12.00	12.00

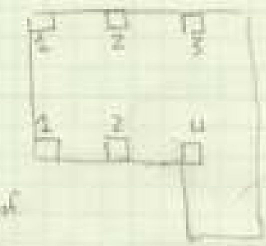
“Large Area” Interior Column Design

Floor	Pu (Kips)	ϕPn (Kips, f'c=5)	Ag (sf)	Ag (in^2)	Width	Length
1	1372.20	1421.42	3.56	512.00	16.00	32.00
2	1247.45	1292.72	3.33	480.00	16.00	30.00
3	1122.70	1151.28	2.89	416.00	16.00	26.00
4	997.96	1022.58	2.67	384.00	16.00	24.00
5	873.21	881.14	2.22	320.00	16.00	20.00
6	748.46	752.44	2.00	288.00	16.00	18.00
7	623.72	681.72	1.78	256.00	16.00	16.00
8	498.97	540.28	1.33	192.00	12.00	16.00
9	374.22	434.20	1.00	144.00	12.00	12.00
10	249.48	434.20	1.00	144.00	12.00	12.00
11	124.73	434.20	1.00	144.00	12.00	12.00

Exterior Column Design

Main Roof Column Design

Mechanical East Roof



Column 1:

$A_T = 29$ sf Reinforcement = 4 #9

$\phi P_n = 0.8(0.65)[0.85 f'_c[A_g - A_s] + f_y A_s]$

$P_u = \omega_R A_T + \omega_w$ (wall weight) ; $\omega_R = 242$ psf $\omega_w = 30$ psf

$P_u = (242)(29) + (30)(9.17)(6.33/2) = 7715.5$ lbs $= 7.72$ K

* assume a minimum column size of 12"x12" to be conservative

$\phi P_n = 0.8(0.65)[0.85(5)[144-4] + 60(4)] = 434.2$ K $>> 7.72$ K ∴ ok

Column 2:

$A_T = 42.5$ sf

$P_u = \omega_R A_T = (242)(42.5$ sf$) + (30)(6.33/2)(7.67) = 11070.8$ lbs $= 11.1$ K $<< 434.2$ K ∴ ok

Column 3:

$A_T = 51.7$ sf

$P_u = \omega_R A_T = (242)(51.7) + 30(6.33/2)(16.08) = 14056.3$ lbs $= 14.1$ K $<< 434.2$ K ∴ ok

Column 4:

$A_T = 117.3$ sf

$P_u = \omega_R A_T = (242)(117.3) + 30(29.5)(6.33/2) = 30440.9$ lbs $= 30.4$ K $<< 434.2$ K ∴ ok

Mechanical West Roof

* assume 12x12 columns

Column 1:

$A_T = 44.44$ sf

$P_u = \omega_R A_T = (242)(44.4) + 30(6.33/2)(14.25')$

$= 12214.4$ lbs $= 12.21$ K $<< 434.2$ K ∴ ok

Column 2:

$A_T = 49.1$

$P_u = (242)(44.44) + (30)(6.33/2)(10.91') = 11572.2$ lbs $= 11.9$ K $<< 434.2$ K ∴ ok

2

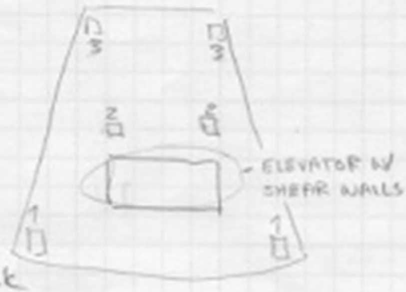
Main Roof Column Design

Column 1:

$A_T = 179.9$ sf

$P_u = (242)(179.9) + (30)(8.17)(40.25)$

$= 53401$ lbs $= 53.4$ K < 434.2 K $\therefore$ ok



Column 2:

$A_T = 840.9$

$P_u = (242)(840.9) + (30)(8.17)(17.33)$

$= 86745.4$ lbs $= 86.7$ K < 434 K $\therefore$ ok

Column 3:

$A_T = 214.2$ sf

$P_u = (242)(210.64) + (30)(8.17)(29.16) = 58121$ lbs $= 58$ K $\ll 434.2$ $\therefore$ ok

AMPAD

Appendix F: Shear Wall Design

Shear Wall Design 1

ACI Requirements

Vertical Reinforcement = $A_{sv_{min}} = 0.0012\ A_g$
Horizontal Reinforcement = $A_{sh_{min}} = 0.002\ A_g$
Reinforcement Spacing: $s \le 18$ in
$s \le 3(T_w)$; T_w = wall thickness

ACI 14.5.2

$\phi P_n = 0.5 f'_c A_g \left[1 - \left(\frac{k l_c}{32h}\right)^2\right]$; k = effective length factor = 1.0
$l_c = 9'7" = 9.58' = 115"$
h = thickness = $h_{min} \ge 1/25(l_c) = 1/25(115) = 4.6$ in
$\ge 1/25(L)$

Wall 1: East Arm of Building, Around Elevator Shaft → For 11th Floor Shear Walls

N/S Direction

A: $\phi P_n = 0.55 \phi f'_c A_g \left[1 - \left(\frac{k l_c}{32h}\right)^2\right]$; $l_c = 115"$
$h = 12"$ $\quad h_{min} \ge 1/25(136) = 5.44"$
$k = 1.0$

$A_t = 241.6$ sf = 34787 in²

(roof load) = $W_R = 242$ psf (Tech 1)
P_u = weight from mechanical roof columns 1 and 2 + wall weight + roof weight + floor weight
column weight = (1')(1')(6.83)(0.150 K/cf) = 1.02 K/per column
mechanical roof weight = (242)(28)(2) = 13552 lbs = 13.6 K
mechanical room wall weight = (30)(6.83)(9.17)(2) = 3757.9 lb = 3.76 K
main roof weight = (241.6)(242) = 58467.2 lbs = 58.5 K
floor weight = (49.52sf)(229.2psf) = 11349.9 lbs = 11.35 K

$P_u = 2(1.02) + 13.6 + 3.76 + 58.5 + 11.35 = 89.25$ K

$\phi P_n = (0.55)(0.65)(5)(139" \times 12")\left[1 - \left(\frac{115}{32(12)}\right)^2\right] = 2714.14$ K

Shear Wall Design

2

B: $\phi P_n = 0.55\phi f'_c A_g\left[1-\left(\frac{k l_c}{32h}\right)^2\right]$

$= 0.55(0.65)(5)(51'' \times 12'')\left[1-\left(\frac{115}{32(12)}\right)^2\right]$

$= 995.8$ K

P_u: Mechanical Roof Weight = (242)(42.5)(2) = 20570 lbs = 20.6 K
Mechanical Room Column Weight = 1.02 K per column
Mechanical Room Wall Weight = (30)(6.53)(767)(2) = 21467 lbs = 21.5 K
Main Roof Weight = (98.9)(242) = 23934 lb = 23.9 K
Floor Weight = (148.7)(229.2) = 34082 lbs = 34.1 K

$P_u = 2(1.02) + 20.6 + 21.5 + 23.9 + 34.1 = 102.14$ K < 995.8 ∴ ok

C: P_u: Mechanical Roof Weight = (242)(70.5) = 17061 lbs = 17.1 K
Mech Room Column Weight = 1.02 per column
Mech Room Wall Weight = (12.5)(6.83)(30) = 2561.3 lbs = 2.6 K
Main Roof Weight = (84)(242) = 20328 lbs = 20.3 K
Floor Weight = (24.81)(229.2) = 5686 lbs = 5.68 K

$P_u = 2(1.02) + 17.1 + 2.6 + 20.3 + 5.68 = 47.72$

$\phi P_n = 0.55(0.65)(5)(95'' \times 12'')\left[1-\left(\frac{115}{32(12)}\right)^2\right] = 1913$ K > 47.72 ∴ ok

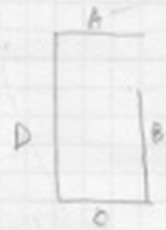
D: P_u: Mechanical Roof Weight = 20.3 K
Mech Room Column Weight = 1.02 per column
Mech Room Wall Weight = (19.3)(6.83)(30) = 3954 lb = 3.95 K
Main Roof Weight = (242)(56.9) = 13769 lb = 13.8 K
Floor Weight = 5.68

$P_u = 2(1.02) + 20.3 + 3.95 + 13.8 + 5.68 = 45.77$ K

$\phi P_n = 1913 > 45.77$ K ∴ ok

Shear Wall Design 3

Wall 2: East Arm of the Building; Around the Stairwell



A: P_u: Main Roof weight = $(88.1)(242) = 21320$ lb = 21.3 K

$\phi P_n = 0.55(0.65)(5)(12" \times 97")\left[1 - \left(\frac{115}{32(12)}\right)^2\right]$
$= 1894$ K > 21.3 K ∴ ok

B: $P_u = (276.4)(242) = 66889$ lbs = 66.9 K

$\phi P_n = 0.55(0.65)(5)(12" \times 170")\left[1 - \left(\frac{115}{32(12)}\right)^2\right] = 3320$ K > 66.9 K ∴ ok

C: $P_u = (100)(242) = 24200$ lbs = 24.2 K

$\phi P_n = 0.55(0.65)(5)(12 \times 97)\left[1 - \left(\frac{115}{32(12)}\right)^2\right] = 1894$ K > 24.2 K ∴ ok

D: $P_u = (257.7)(242) = 62363$ lb = 62.4 K

$\phi P_n = 0.55(0.65)(5)(12 \times 219)\left[1 - \left(\frac{115}{32(12)}\right)^2\right] = 4276.2$ K > 62.4 K ∴ ok

Wall 3: Center of Building; Around Elevator Shaft

A, C: $P_u = (95.27)(242) = 23055$ lb = 23.1 K

$\phi P_n = 0.55(0.65)(12" \times 116")(5)\left(1 - \left(\frac{116}{32(12)}\right)^2\right)$
$= 1840$ K > 23.1 K ∴ ok

B: $P_u = (199.3)(242) = 48230$ lbs = 48.2 K

$\phi P_n = 0.55(0.65)(12 \times 240)(5)\left(1 - \left(\frac{116}{32(12)}\right)^2\right)$
$= 3806$ K > 48.2 K ∴ ok

Shear Wall Design 4

Wall 4: West Arm of Building; Around the Stairwell

A: P_u: Stair Roof = (16.8)(242) = 4065 lbs = 4.1 K

Stairwell wall weight = (30)(?)(6.83) = 1638 lbs = 1.64 K

Column Weight = 1.02 K per column

Floor Weight = (16.8)(229.2) = 3851 lbs = 3.85 K

Wall Weight = (30)(9.58/2)(22.75) = 3269 lbs = 3.27 K

P_u = 4.1 + 1.64 + 2(1.02) + 3.85 + 3.27 = 14.9 K

ϕP_n = 0.55(0.65)(5)(96" × 12")$\left[1-\left(\frac{115}{32(12)}\right)^2\right]$

= 1874.5 K > 14.9 K ∴ ok

B: P_u: Main Roof Weight = (242)(144.2) = 34896 lbs = 34.9 K

Stair Roof Weight = (242)(60.9) = 14738 lbs = 14.7 K

Floor Weight = (229.2)(60.9) = 13958 lbs = 14 K

Column Weight = 1.02 K

Stair Wall Weight = (30)(6.83)(18.42) = 3774.3 lbs = 3.77 K

P_u = 2(1.02) + 14 + 14.7 + 34.9 + 3.77 = 69.41 K

ϕP_n = 0.55(0.65)(5)(176" × 12")$\left[1-\left(\frac{115}{32(12)}\right)^2\right]$ = 3436.6 K > 69.41 K ∴ ok

C: Stair Roof = 4.1 K

Stair wall weight = 1.64 K

Column Weight = 1.02 K per column

Main Roof Weight = (242)(143.6) = 34751 lbs = 34.8 K

Floor Weight = 3.85 K

P_u = 2(1.02) + 4.1 + 1.64 + 34.8 + 3.55 = 46.9 K

ϕP_n = 1879.5 K > 46.9 K ∴ ok

D: P_u: Stair Roof = 14.7 K

Stair Wall Weight = 3.77 K

Column Weight = 1.02 K per column

Main Roof Weight = (242)(301.2) = 72890 lbs = 72.9 K

Floor Weight = 14 K

P_u = 2(1.02) + 14.7 + 3.77 + 72.9 + 14 = 107.4 K

ϕP_n = 0.55(0.65)(5)(12" × 221")$\left[1-\left(\frac{115}{32(12)}\right)^2\right]$ = 4355.3 K > 107.4 ∴ ok

Shear Wall Design 5

Max Load Check for all Shear Walls → 9 Floor

Wall 1:

A: P_u: Initial Load Supported by 11th Floor Walls = 89.25 K
Cummulative Wall Weight from higher floors = 10(11.58)(9.58)(0.150) = 166 K
Cummulative Floor Weight from higher floors = 10(229.2)(150.8) = 345633 lbs = 345.6 K

$P_u = 89.25 + 166 + 345.6 = 600.9\ K$

$\phi P_n = 2714.14 K > 600.9 K \therefore ok$

B: $P_u = 102.14 K + 10(6.1K) + 10(0.229)(205) = 469.45 K$

$\phi P_n = 995.8 K > 469.45 K \therefore ok$

C: $P_u = 47.72 + 10(9.58)(8.16)(0.150) + (0.229)(89.3)(10) = 369.5 K$

$\phi P_n = 1913 K > 369.5 K \therefore ok$

D: $P_u = 46.77 + 10(9.58)(8.16)(0.150) + (0.229)(91.4)(10) = 349.43 K$

$\phi P_n = 1913 K > 349.43 K \therefore ok$

Wall 2:

A: $21.3K + (292)(186.8) + 10(8.02)(9.58)(0.150) + 9(186.8)(0.229) = 567.4 K$

$\phi P_n = 1894 K > 567.4 K \therefore ok$

B: $66.9K + (10)(9.58)(14.10)(0.15) + (0.229)(332.5)(10) = 1034 K$

$\phi P_n = 3320 K > 1034 K \therefore ok$

C: $24.2 + 10(9.58)(0.15)(8.08) + 10(89.1)(0.229) = 344 K$

$\phi P_n = 1894 K > 344 K \therefore ok$

D: $62.4 K + 10(9.58)(0.15)(15.16) + 10(0.229)(201.4) = 784.6 K$

$\phi P_n = 4276 K > 784.6 K$

Shear Wall Design 6

Wall 3:

A: $P_u = 23.1 + 12(9.58)(0.150)(9.67) + 10(0.229)(188) = 551.7\ K$

$\phi P_n = 2265\ K > 551.7\ K \therefore ok$

B: $P_u = 48.2 + 12(9.58)(0.15)(20) + 10(0.229)(284) = 1043\ K$

$\phi P_n = 4686.3\ K > 1043\ K \therefore ok$

C: $P = 23.1 + 12(9.58)(0.15)(9.67) + 10(0.229)(168.9) = 576.6\ K$

$\phi P_n = 2265\ K > 576.6\ K \therefore ok$

Wall 4:

A: P_u: Initial P_u + 3(Wall Weight) + 3(Floor Weight) + 7(Floor Weights) + Shear Wall Weight

$P_u = 14.9\ K$

Wall Weight $= 3(80)(9.58)(8) = 6897\ lbs = 6.9\ K$

Floor Weight 1 $= 3(0.229)(15.91) = 10.93\ K$

Floor Weight 2 $= 7(0.229)(107.9) = 173\ K$

Shear Wall Weight $= 10(0.15)(9.58)(8) = 114.9\ K$

$P_u = 6.9 + 10.93 + 173 + 114.9 = 305.7\ K$

$\phi P_n = 1874\ K > 305.7\ K \therefore ok$

B. $P_u = 69.41 + 10(0.229)(89.5) + 10(9.58)(14.66)(0.15) = 645.3\ K$

$\phi P_n = 3436.6\ K > 645.3\ K \therefore ok$

C: $P_u = 46.4 + 10(0.229)(161.8) + 10(0.15)(9.58)(8) = 531.9\ K$

$\phi P_n = 1874.5\ K > 531.9\ K \therefore ok$

D: $P_u = 107.4\ K + 10(0.229)(149.4) + 10(0.15)(9.58)(18.42) = 714.22\ K$

$\phi P_n = 4315\ K > 714.2\ K \therefore ok$