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AE 481W Thesis Proposal
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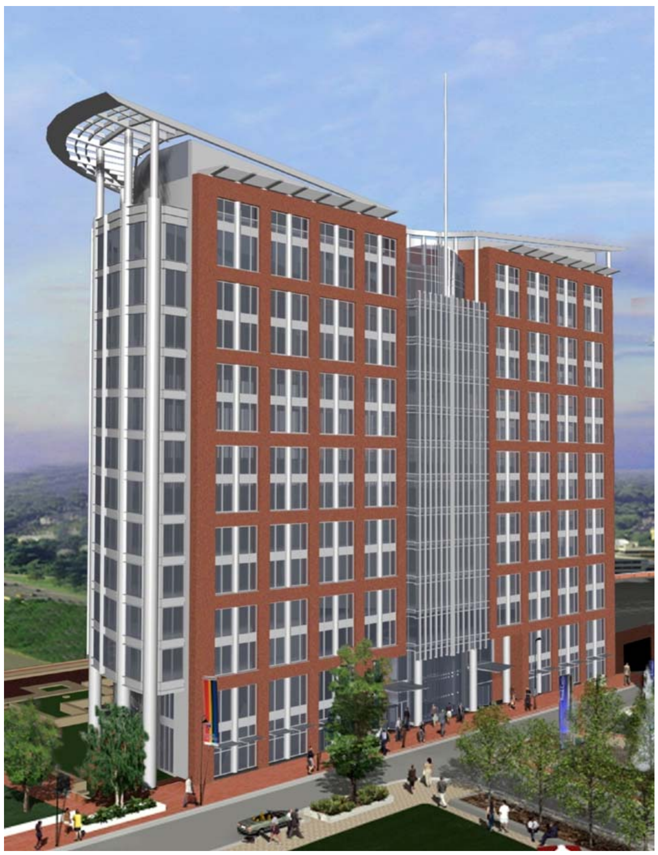
TOWERS CRESCENT BUILDING F



CHANTILLY, VA

FLOOR, LATERAL SYSTEM, AND FOUNDATION OPTIMIZATION

Executive Summary

Towers Crescent Building F is a 230' speculative office building containing 5 levels of parking and 15 floors of offices. The foundation is auger cast piles. The gravity system is a reinforced concrete flat slab with drop panels and drop bands framing into reinforced concrete columns. The lateral system is a combination of reinforced concrete shear walls, and the moment resisting frames created by the monolithically cast columns and slab. The slab acts as a rigid diaphragm and distributes lateral load to the lateral load resisting elements according to stiffness.

This building was never built according to its original design due to certain expensive structural features, such as curved drop bands and the pile foundation system. Materials costs would also have been high due to the large volume of concrete in the slab. Also, based on simple lateral analysis procedures the lateral system was determined to be insufficient to bear the calculated seismic loads.

Because of these problems, the floor system will be redesigned as a two way pre-stressed slab with supporting edge beams. This will simplify the required formwork, reduce materials costs, and contribute to lateral stiffness. Next, seismic loads will have to be reduced through more rigorous analysis procedures. The stiffness of the lateral system will also have to be determined more accurately. These analyses will involve construction of a 3-dimensional model. Once they are complete I will make changes to the lateral system to increase its strength and efficiency. Lastly, the pile foundation system will be replaced with caissons. Once the redesign is complete, I will estimate the cost of construction of one floor using the original design compared to my design.

Background

Towers Crescent Building F is a 230' speculative office building with 339,063 square feet of office space on 15 floors and 416,065 square feet of parking on 6 levels. The bottom 3 levels are exclusively parking, and the 4th through 6th levels are mixed use. These floors comprise the base of the structure; the office tower extends 13 floors above the base. Table 1 presents the square footage of the building at each floor, broken down by parking area, and area relevant to floor area ratio (FAR) calculations.

Description of Structural System

Foundation and Slab on Grade

The building utilizes a foundation system of 80T 16"Φ auger cast piles. Pile caps are laid out on a roughly regular 30' x 30' structural grid as well as one semicircular line which follows the rounded face of the building. Pile groups range from 3 in the parking areas to 42 for the interior tower columns. A common pile cap for the areas supporting only parking is 6'-6" x 6'-6", 44" deep, containing 10 #6 reinforcing bars in each direction, and caps 4 piles. A common pile cap in the area beneath the office tower is 15' x 20', 55" deep, containing 20 #11 reinforcing bars in each direction, and caps 20 piles. The slab-on-grade is 6" thick stone concrete at f'c = 4 ksi reinforced with 6 x 6 #8/#8 W.W.F. It is placed over a vapor barrier on top of 6" of washed gravel fill.

Table 1. Floor Area

Floor	Gross SF (FAR)	Gross SF (Parking)
15	22943	
14	23636	
13	23636	
12	23636	
11	23636	
10	23636	
9	23636	
8	23636	
7	23636	
6	23636	
5	23636	
4	23636	
3	19461	
2/P6	16881	24783
Mezz/P5	386	51910
1/P4	19396	51904
P3		98459
P2		98459
P1		90550

Columns

Columns are reinforced concrete, with material strengths as follows:

- Base to 2nd floor - 8 ksi
- 2nd floor to 8th floor - 7 ksi
- 8th floor to 13th floor - 6 ksi
- 13th floor to main roof - 5 ksi
- Main roof to penthouse roof - 4 ksi.

The parking areas are held up by a mostly regular grid of concrete columns (typically 24" x 24" with 6 #9 reinforcing bars) extending usually from the pile caps to the P-4 or P-6 level. The tower is held up by a rectangular grid of columns as well as a semicircular line which follows the curvature of the building. A typical internal tower column on the rectangular grid runs as follows:

- Base - P2: 24"x48", 16 #18
- P2 - P3: 24"x48", 16 #14
- P3 - 2/P6 level: 24"x48", 20 #11
- 2/P6 level - 4th floor: 24"x30", 16 #11

- 4th floor - 5th floor: 24”x24”, 16 #11
- 5th floor - 6th floor: 24”x24”, 12 #11
- 6th floor - 7th floor: 24”x24”, 10 #11
- 7th floor - 9th floor: 24”x24”, 8 #11
- 9th floor - 13th floor: 24”x24”, 6 #11
- 13th floor - main roof: 24”x24”, 4 #11

A typical column along the semicircular line runs as follows:

- Base - P3: 42” Φ, 8 #11
- P3 - 2/P6 level: 42” Φ, 7 #11
- 2/P6 level - 4th floor: 36” Φ, 7 #11
- 4th floor - main roof: 36” Φ, 6 #11
- Main roof - penthouse roof: 36” Φ, 6 #11, W14x82

The reinforcement is spliced by overlapping bars.

Floors
The floors are 9” minimum flat structural concrete slab (f’c = 4 ksi) reinforced by a bottom mat of #5 rebar at 12” O.C. in each direction. Where the slab is 10” thick, it is reinforced by #5 rebar at 9” O.C. in each direction, and where it is 12” thick, it is reinforced by #7 rebar at 12” O.C. in each direction. Additional reinforcement is provided as needed, almost always top reinforcement (#5 or #6) to resist the tensile stresses which result from the negative moments, especially around the columns. Around every column there is a drop panel 5-1/2” below the lowest adjacent slab soffit at 1/6 the column span in each direction, a drop band 5-1/2” below the lowest adjacent slab soffit at 1/4 the column span in each direction, or a similar system. In addition, there are typically 8-1/2” drop bands around the edge of the floor.

Lateral System
There is a structural core area in the center of the tower with 4 large concrete shear walls. Table 2 describes these walls at level P6:

Table 2. Description of Shear Walls at Level P6

Along Gridline	Length	Vertical Reinforcement	Horizontal Reinforcement	End Column Dimensions	End Column Reinforcement
FQ.3	22'10”	#6 @ 6” O.C.	#5 @ 9” O.C.	36” x 12”	(12) #11
FP	29'2”	#5 @ 6” O.C.	#4 @ 12” O.C.	30” x 24”	(16) #10
FN	29'2”	#5 @ 6” O.C.	#4 @ 12” O.C.	30” x 24”	(16) #10
FM	28'	#6 @ 6” O.C.	#5 @ 9” O.C.	30” x 24”	(16) #10

Walls are 16” thick from the foundation to level P6, and 12” thick above. Wall reinforcement and end column size and reinforcement vary throughout the height of the building. Length is constant. The shear walls are attached to concrete columns at either end to provide resistance against overturning moment as well as added shear capacity.

These four walls run from north to south through the building's narrow section and resist lateral loads in that direction. Six shorter shear walls, 9'-6" effective length, including end columns of the same width and containing similar reinforcement, run perpendicular to them. Three intersect wall FP, and three intersect wall FN, therefore these walls will act together as web and flange to one beam section. In addition, since the flat floor slab is cast monolithically with the building's concrete columns, the resulting frames will have an inherent moment resisting capacity and hence contribute somewhat to the lateral stiffness of the tower. Some will brace against the shear walls.

The load path for lateral loads is as follows. In the case of wind loading, the curtain wall will receive the load and distribute it to the minimum 9" thick floor slabs above and below. The slabs will then act as rigid diaphragms and thus distribute lateral load to the lateral load resisting elements according to stiffness. In the case of seismic loading, load will be distributed from all massive elements, through their structural connections with the slab, which again, will act as a rigid diaphragm.

At the base, the floor area of Towers Crescent Building F increases dramatically. At these levels the central tower area is surrounded by additional structure which will be used for parking. These areas contain additional moment resisting frames produced by the monolithic casting of the slab-beams and columns, some of which provide a small but significant resistance.

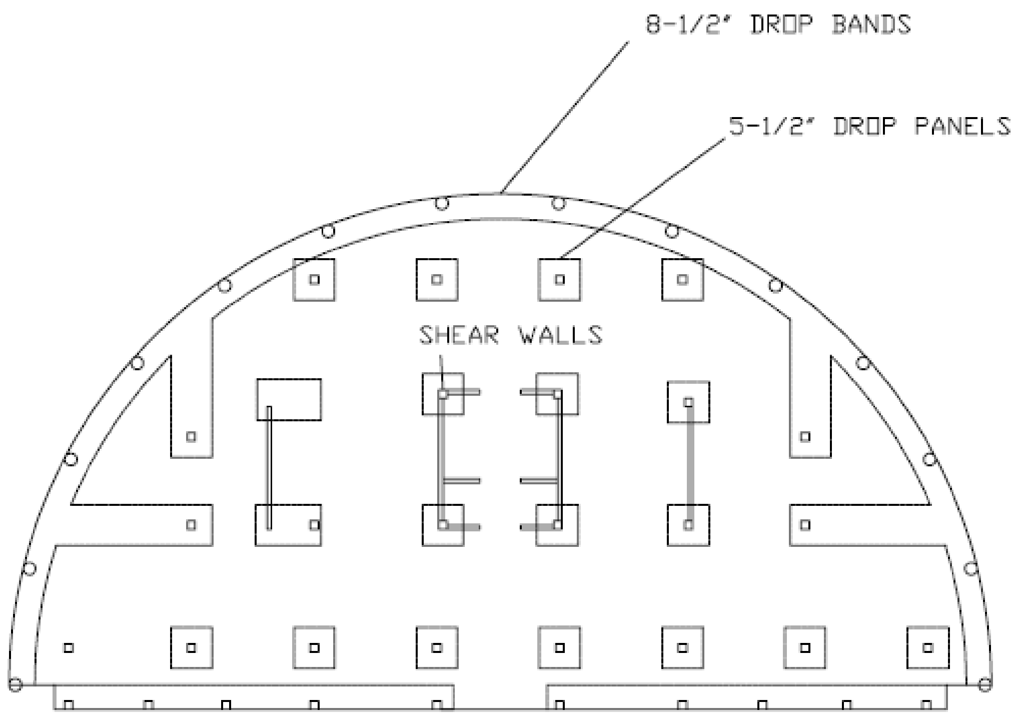
Two large sections of the parking structure are separated by expansion joints. These sections will rely entirely on moment resisting frames for their lateral resistance. Since they are far shorter than the tower, they are anticipated to deflect less than the central slab area under wind or seismic loads. Therefore I have assumed that, whatever structural contact would occur between them during a wind or seismic event, these sections would act as restraints against the motion of the central section. Assuming linear behavior, they could properly be modeled as a series of springs which resisted compressive but not tensile forces.

Figure 1 displays a typical floor plan for the tower, including lateral load resisting elements.

Building Codes and Design Standards
Towers Crescent Building F was designed by the 2000 USBC Virginia statewide building code, which is a variation of the IBC 2000 model code. This references the ASCE 7-98 design standard. Nevertheless, I have chosen to design by ASCE 7-02, due to my greater familiarity with it. ASCE 7 sections 6.0 and 9.0, on wind and seismic loads, respectively, are especially relevant to this assignment. Concrete structural elements would have been designed originally by the standards of ACI 318-99, but again, I will be designing based on the updated code ACI 318-05. Steel structures would have been designed either with

the ASD manual of steel construction, 9th edition, or the LRFD manual of steel construction, 1st edition. I will use the LRFD 3rd edition.

Figure 1. Typical Tower Floor Plan



Problem Statement

Prohibitive Cost
The current plan calls for certain structural features which have rendered this building prohibitively expensive (it was not built according to the current plan, and has been sent to another firm to be value engineered). Among these are the curved drop bands, which would require elaborate formwork assemblies, and the pile foundation system. The flat slab requires a large volume of concrete as well.

Possibly Inadequate Lateral System
Next, the lateral load resisting system must be designed to resist seismic and wind loads as determined by ASCE 7-02 sections 9.0 and 6.0, respectively. Furthermore, the building must not deflect more than 0.25% of its height under these loads. Using simple analysis procedures, it was determined that the current lateral system is inadequate for strength criteria in one direction and for deflection criteria in both.

Solution

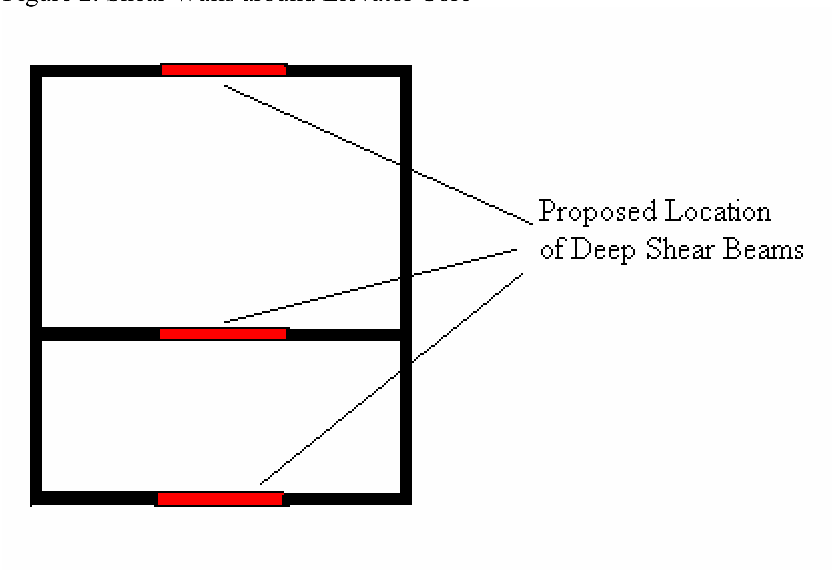
Value Engineering

Certain structural features of Towers Crescent Building F will have to be replaced. The pile foundation system will be replaced with an alternative, such as caissons. Next, pre-stressing will be investigated as a means to reducing the thickness of the slab, thus saving material costs and reducing the total weight of the structure. Drop panels and drop bands will be eliminated in favor of supporting edge beams. These will simplify the required formwork and reduce the required slab thickness, as well as increasing the stiffness of the lateral frames. I will investigate the plenum space required for mechanical equipment prior to sizing these beams.

Rigorous Seismic Analysis

The seismic loads will have to be reduced through a more rigorous analysis, and the lateral stiffness of the building will have to be determined more accurately. This having been done, the lateral system will be optimized to efficiently bear the controlling lateral loads. The most important anticipated change is the introduction of deep shear beams to connect the shear walls on either side of the elevator core. See figure 2 below. Shear walls may be reduced in thickness as well.

Figure 2. Shear Walls around Elevator Core



Solution Method

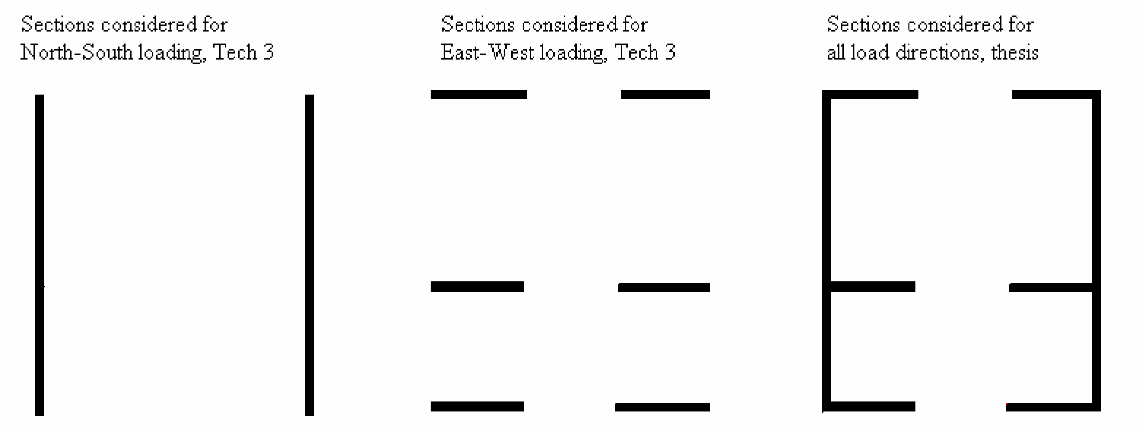
Equivalent Frames

The slab will be redesigned using equivalent frame analysis. Gravity design load will be computed according to the provisions of IBC 2003 1605-1606. Then, ADOSS, a computer implementation of the equivalent frame method prescribed in ACI 318-05 chapters 13 and 18, will be used to distribute it. The estimated moments due to wind loading will then be added to those determined above, and the new pre-stressed slab with edge beams will be designed using two way load balancing and "Method 3" from 1963 ACI.

Modal Analysis and 3-Dimensional Modeling

To the end of reducing lateral design loads on Towers Crescent Building F I will perform a modal analysis. This will require building a 3-dimensional model in ETABS which represents the spatial distribution of the mass and stiffness of the structure. A 2-dimensional model will not suffice since the lateral force resisting systems in orthogonal directions are not independent. In the analysis for previous technical reports they were considered as such, however, this is over-conservative, since when orthogonal shear walls intersect, they serve as web and flange for one beam section. Figure 3 below pictures the beam sections which were employed in the lateral stiffness analysis of technical report 3, compared to the beam sections which will be employed in the thesis report.

Figure 3. Cantilever Beam Sections for Core Shear Walls



Modal analysis of the 3-dimensional model will yield the fundamental period of the building. It is anticipated that, once this is known, it will be possible to reduce the base shear calculated through the equivalent lateral force procedure by a factor of nearly 1.7. Currently, the building period is approximated by $T_a = 0.02h^{0.75} = 1.17$ seconds. The actual period is likely significantly higher. With 19% of the building mass attached to wall FQ.3, one of the four shear walls which resist lateral load in the North-South direction, its period is over 5 seconds. The period of the entire building is expected to be similar. Thus, provision 9.5.5.3 of ASCE 7-02 will apply and the fundamental period of the building will be taken to be C_uT_a, where C_u is slightly below 1.7. This in turn will reduce the seismic response coefficient by the factor C_u.

Furthermore, it is anticipated that the modal base shear will be less than 85% of the base shear calculated by the equivalent lateral force procedure. Therefore, base shear will be taken to be 85% of the modal base shear, resulting in a total reduction of seismic loads by approximately half from those calculated in technical report 3. After these reductions, it is likely that wind loading will control the design of the lateral system in one or both directions.

Further measures will be taken to optimize the lateral design of Towers Crescent Building F. The 3-dimensional model will enable the design to make use of heretofore neglected lateral resisting elements, such as curved and irregular frames. Specifically, there is a large curved frame around the entire North face of the building which will contribute significantly more than it has been calculated to contribute using 2-dimensional procedures. In addition, the 3-dimensional model will distribute loads more accurately than 2-dimensional procedures, since it will satisfy all compatibility requirements simultaneously, instead of merely one floor at a time. Finally, all lateral frames will be stiffer due to the introduction of edge beams, which will increase the rigidity of the beams and torsional members of the equivalent frame (Edge beams will not be so deep as to reduce plenum space below what is necessary for mechanical systems).

Once these analyses have been completed, I will optimize the efficiency of the lateral system. The anticipated changes include adding shear beams to connect the shear walls around the elevator core, which will drastically increase torsional resistance and building stiffness against East-West lateral loads, and reducing the thickness of the shear walls.

Drilled Piers

Gravity loads at the foundation will be determined by a column load takedown. I will employ a spreadsheet developed by John Barry of Thornton Tomasetti in this task. Gravity loads having been determined, I will design caissons with sufficient load bearing capacity to resist them, as well as base shear and any overturning moment from the lateral loads. Equations for the vertical and lateral load bearing capacity of drilled piers are found in chapter 12 of *Principles of Foundation Engineering* by Braja M. Das.

Breadth

As stated above, I will determine the required plenum depth for mechanical systems. Next, I will estimate the cost of building a typical tower floor, including formwork, reinforcement, and concrete placement, using the original design and my design, and compare the two.

Tasks and Tools

I. Design Two Way Pre-Stressed Slab for Typical Floor

- a) Determine required plenum depth for mechanical systems.
- b) Pick slab edge beam sizes based on ceiling height requirements and ACI 318-05 Table 9.5(a).
- c) Estimate slab thickness, 1” to 3” thinner than that required by ACI 318-05 Table 9.5(a) for non-prestressed slabs
- d) Find self-weight and superimposed dead load; determine live load and the controlling load combination by IBC 2003 1605-1606.
- e) Distribute gravity load in ADOSS.

f) Apply estimated moments due to lateral loads.
g) Design edge beams, slab, and slab reinforcement with two way load balancing and "Method 3" from 1963 ACI.
h) Check punching shear.

II. Modal Analysis and Seismic Design
a) Determine stiffness of all equivalent frame members.
b) Assemble 3-dimensional model of all lateral resisting elements, with rigid diaphragms to connect them, in ETABS.
c) Determine new floor masses and centers of mass.
d) Apply floor mass to the center of mass at each floor, perform modal analysis and obtain fundamental frequency.
e) Find new seismic loads.
f) Perform wind load analysis per ASCE 7-02 section 6.0 for East and West faces.
g) Apply wind and seismic loads at required eccentricities, analyze, and determine maximum deflection and member stresses.
h) Make changes to lateral system, such as adding shear beams, reducing shear wall thicknesses, etc., and repeat analysis, until the lateral system is optimized.

III. Foundation Redesign
a) Determine gravity load on foundations through column load takedown.
b) Find shear at the base of each column due to lateral loading from ETABS model.
c) Calculate overturning moment for various lateral elements.
d) Design caissons sufficient to resist the above by the equations found in chapter 12 of *Principles of Foundation Engineering* by Braja M. Das.

IV. Determine cost of alternative floor systems using takeoffs and RS Means.

Timetable

January 2006

Su	Mo	Tu	We	Th	Fr	Sa
1	2	3	4	5	6	7
8	9	10	11	12	13	14
15	16	17	18	19	20	21
22	23	24	25	26	27	28
29	30	31				

February 2006

Su	Mo	Tu	We	Th	Fr	Sa
			1	2	3	4
5	6	7	8	9	10	11
12	13	14	15	16	17	18
19	20	21	22	23	24	25
26	27	28				

March 2006

Su	Mo	Tu	We	Th	Fr	Sa
			1	2	3	4
5	6	7	8	9	10	11
12	13	14	15	16	17	18
19	20	21	22	23	24	25
26	27	28	29	30	31	

April 2006

Su	Mo	Tu	We	Th	Fr	Sa
						1
2	3	4	5	6	7	8
9	10	11	12	13	14	15
16	17	18	19	20	21	22
23	24	25	26	27	28	29
30						

Blue – Find and distribute design moments for slab
Red – Design pre-stressed slab
Green – Determine stiffness of equivalent frame members
Purple – Assemble model
Indigo – Redesign lateral system
Pink - Determine loads at foundation
Black - Design Caissons
Rose - Estimate floor costs
Lime – Write report