Benjamin M. Douglass
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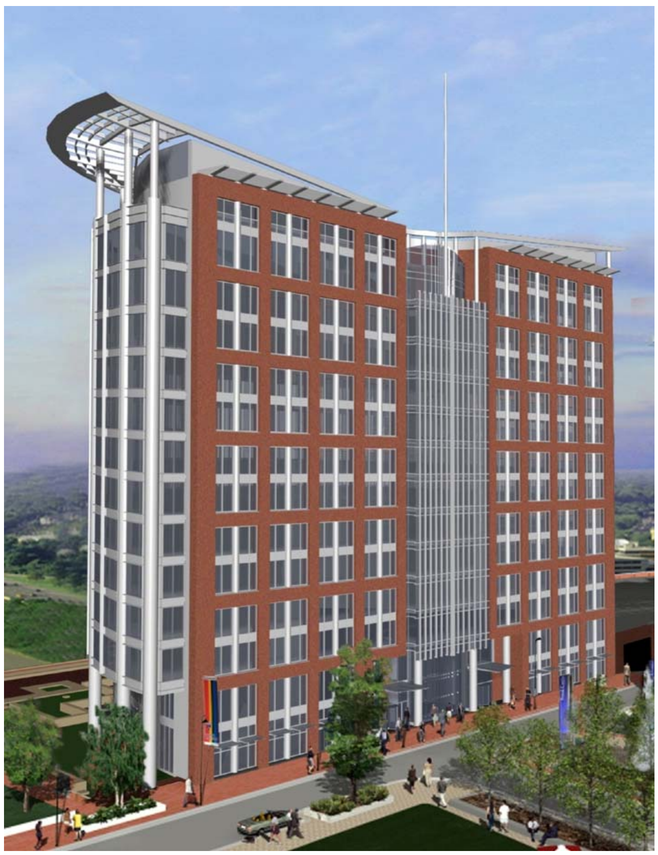
TOWERS CRESCENT BUILDING F



CHANTILLY, VA

FLOOR, LATERAL SYSTEM, AND FOUNDATION OPTIMIZATION

Executive Summary

Towers Crescent Building F is a 199' speculative office building containing 6 levels of parking and 13 floors of offices. The foundation is composed of caissons. The gravity system is a reinforced concrete flat slab with drop panels 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.

In this thesis, the floor system will be redesigned as a two way post-tensioned slab with supporting edge beams. This will reduce material costs and contribute to lateral stiffness. Next, since the calculations in technical report 3 showed the lateral system to be inadequate to carry the design loads, seismic loads will 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, I will attempt to downsize some of the foundation elements due to the decreased weight of the building. Once the redesign is complete, I will compare the cost and time of construction for erecting a floor of this building with a traditional formwork system versus a flying table system.

Background

Towers Crescent Building F is a 199' high speculative office building with 304,880 square feet of office space on 13 floors and 368,770 square feet of parking on 6 floors. The bottom 3 levels are exclusively parking; these floors comprise the base of the structure. Three additional levels of parking rise from the base in one location and the semicircular office tower sits on top of it in another. 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 50' deep, 30"-54"φ caissons. These caissons sit below each column (in one case, two caissons support one column) and below the shear walls. The slab-on-grade is 5" thick concrete at $f'_c = 3$ ksi reinforced with 6 x 6 W2.0xW2.0 W.W.F. placed 2" below the top of the slab. It is placed over a vapor barrier on top of 4" of gravel fill.

Columns

Columns are reinforced concrete, with material strengths as follows:

- Base to 4^{th} floor - 8 ksi
- 4^{th} floor to 8^{th} floor - 6 ksi
- 8^{th} floor to penthouse roof - 5 ksi

Table 1. Floor Area

Floor	Gross SF (FAR)	Gross SF (Parking)
13	23486	
12	24232	
11	24232	
10	24232	
9	24232	
8	24232	
7	24232	
6	24232	
5	24232	
4	24232	
3	24232	
2/P6	19614	23759
Mezz/P5		23759
1/P4	19460	23759
P3		99235
P2		99235
P1		99023

The parking areas are held up by a mostly regular grid of concrete columns (typically 24" x 24" with 8 #8 reinforcing bars) extending usually from the pile caps to the P-4 or P-6 level. The tower is held up by a partially radial and partially orthogonal grid of columns. A typical internal tower column on the rectangular grid runs as follows:

- Base – P4: 24"x36", 16 #10
- P4 - 4th floor: 24"x30", 10 #10
- 4th floor - 5th floor: 24"x24", 16 #10
- 5th floor - 6th floor: 24"x24", 12 #11
- 6th floor - roof: 24"x24", 12 #9

A typical column along the semicircular edge of the building runs as follows:

- Base – 3rd floor: 30"x30", 12 #10
- 4th floor - main roof: 24"x30", 10 #8

The reinforcement is spliced by overlapping bars.

Floors

The floors are 8" minimum flat structural concrete slab (f'_c = 5 ksi) reinforced by a bottom mat of #5 rebar at 12" O.C. in each direction. Where the slab is 12" or 14" thick, it is reinforced by #6 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 resulting from the negative moments, especially around the columns. Around most every column there is a 10'x10' or similarly sized drop panel extruding 8" below the lowest adjacent slab soffit.

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 P5:

Table 2. Description of Shear Walls at Level P5

Along Gridline	Length	Width	Vertical Reinforcement	Horizontal Reinforcement	Addl. End Reinforcement
FG.5	29’5-1/2”	16”	#6 @ 8” O.C.	#4 @ 12” O.C.	(4) #6
FJ.2	33’8”	12”	#5 @ 8” O.C.	#4 @ 12” O.C.	(8) #8
FK.7	33’8”	12”	#5 @ 8” O.C.	#4 @ 12” O.C.	(8) #8
FL.8	30’	16”	#6 @ 8” O.C.	#4 @ 12” O.C.	(3) #6

Wall reinforcement varies throughout the height of the building. The length and width of the shear walls, by contrast, are constant over the full height. 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’-11-5/8” in length, run perpendicular to them. Three intersect wall FJ.2, and three intersect wall FK.7, 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.

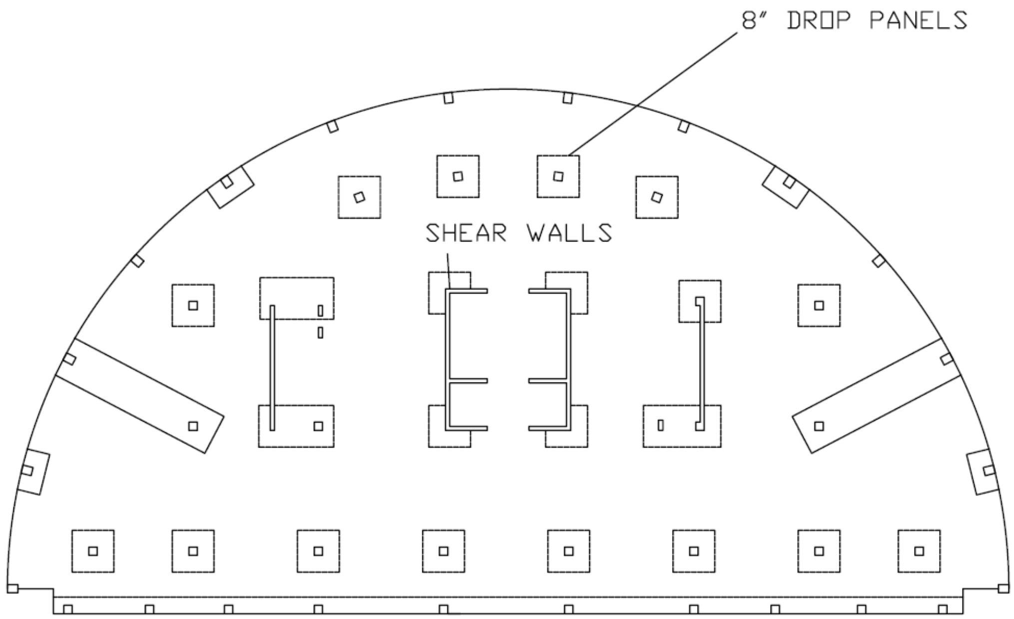
The load path for lateral loads is as follows. In the case of wind loading, the curtain wall will receive the load and distribute it to the minimum 8” thick floor slabs above and below. The slabs will then act as rigid diaphragms and thus distribute lateral load to the lateral load resisting elements 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 an additional parking structure. 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.

One large section of the parking structure is separated by an expansion joint. This section will rely entirely on moment resisting frames for its lateral resistance. Since it is far shorter than the tower, it is 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, this section would act as a restraint against the motion of the central section. Assuming linear behavior, it 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.

Fig. 1. Typical Tower Floor Plan



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.

Problem Statement

Continual Pressure to Drive down Cost

While a number of important changes have been made to this structure since the original design (e.g., eliminating the curved drop bands, reducing the floor to floor height, replacing the pile foundation system with caissons), the owner desires to further reduce the cost of construction.

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

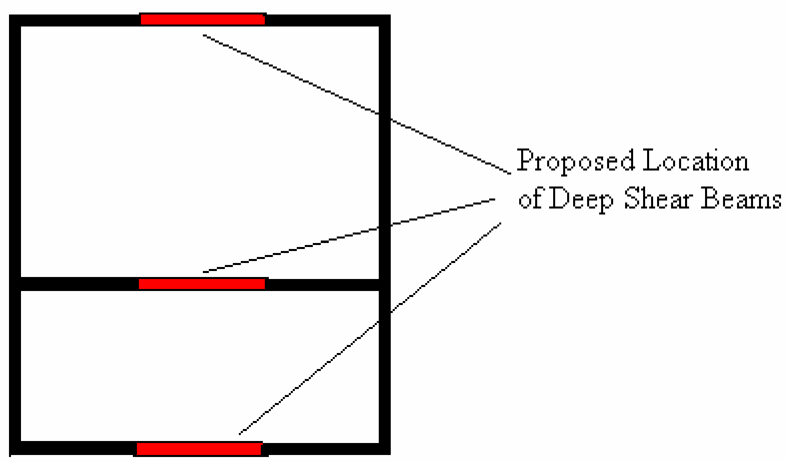
Value Engineering

Post-tensioning 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. Columns will be rearranged into a more regular grid in order to facilitate this process. Next, drop panels will be eliminated in favor of supporting edge beams. These will 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. Foundation members will be downsized once the weight of the structure has been thus reduced.

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

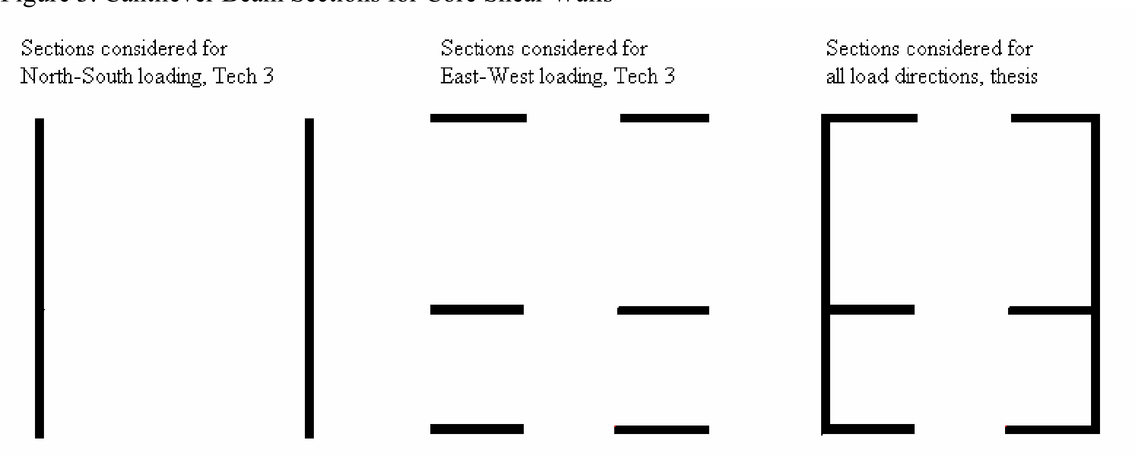
The slab will be redesigned using equivalent frame analysis. Gravity design load will be computed according to the provisions of IBC 2003 1605-1606. Then, ADOSS, a computer implementation of the equivalent frame method prescribed in ACI 318-05 chapters 13 and 18, will be used to distribute it. The estimated 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.

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

Figure 3. Cantilever Beam Sections for Core Shear Walls



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. I will also analyze the effect of any changes I make to the shear wall system on the mechanical ductwork. Next, I will compare the cost and time of construction for erecting a floor of this building with a traditional formwork system versus a flying table system.

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. Compare cost and time of construction for different formwork systems.