Trump Palace Tower

Architectural Engineering Spring 2004 Senior Thesis

Daniel J. Tate Structural Emphasis

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Executive Summary

This report contains the results of senior thesis work performed on the Trump Palace Tower located in northern Miami. Thesis work began by first analyzing the existing structure to obtain an understanding of how it functioned. An analysis of alternative floor systems suggested that the current 8" thick post-tensioned concrete slab may not be the most efficient slab system. The current system is a good solution for long spans, but over shorter spans the slab appeared to become less efficient.

This report investigates the possibility of using an alternative floor system in the Trump Palace Tower. The original monolithic post-tensioned slab was redesigned by dividing the floor into three critical areas and redesigning each of these areas separately. Upon completion of the redesign, comparisons based on overall constructability were performed between the two systems to determine which was more efficient. Although the redesigned slab was capable of resisting the required loads, it was found to be less efficient than the original design. The redesigned floor had several advantages but it also complicated construction and increased overall costs. After comparing the advantages and disadvantages of each system, the original post-tensioned system was recommended over the redesigned floor system.

Also examined in this report was the durability of the building's balcony/façade in the Miami region. It was determined that the balconies would last about 20 years without any significant maintenance. Solar impact was also studied to determine its influence on the air-conditioning between the north and south sides of the building.

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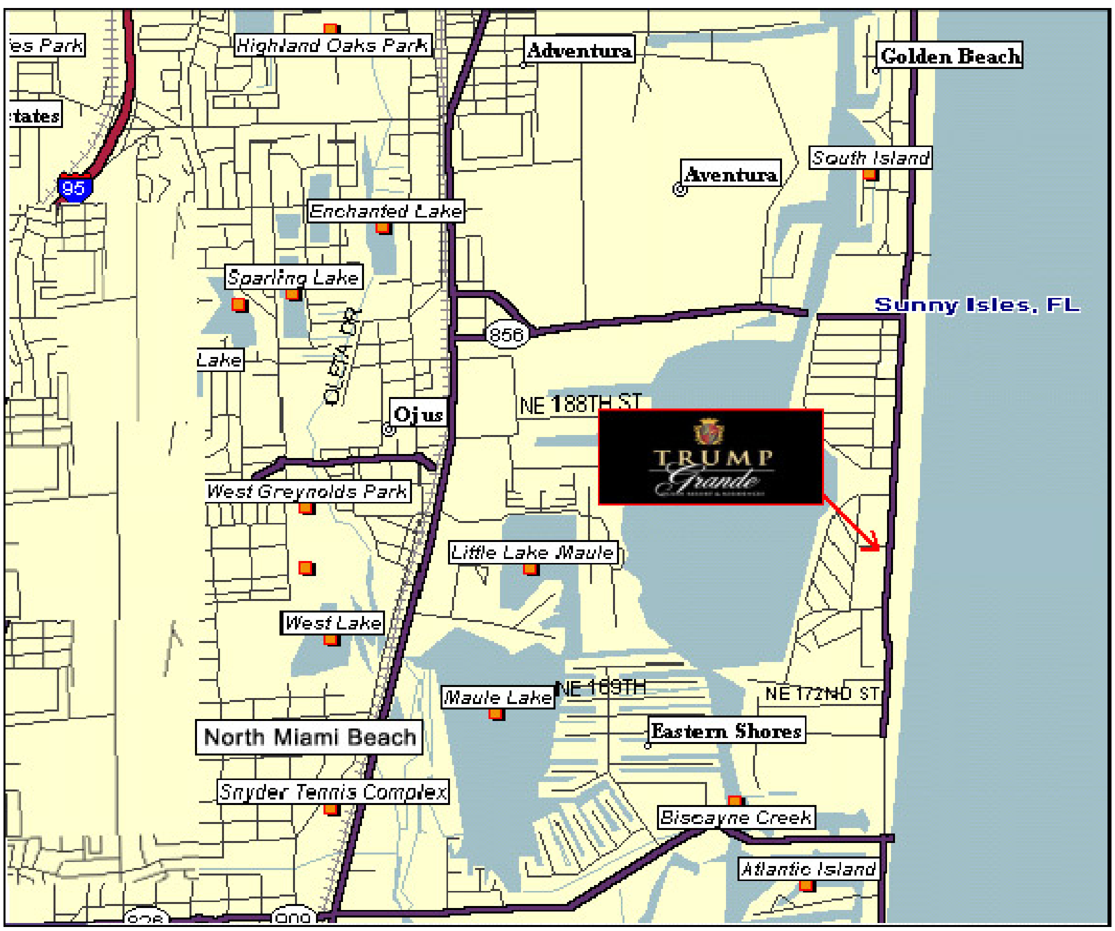
Background Information

The Trump Palace Tower is a 55 story, 278 unit high rise condominium being built off the northern Miami coast. The building encloses approximately 1,000,000 SF over its height, while its foot print is only about 18,000 SF. The main roof is located 488'-4" above grade but the height to the top of the highest architectural element is 550'-4", making it the second tallest building in Miami. The building is located a stone's throw from the Atlantic coast at 18101 Collins Ave. Sunny Isles Beach, FL.



Picture courtesy of Trumpgrande.com

Picture courtesy of Trumpgrande.com

The Trump Palace is one of two symmetrical buildings being constructed in a development project known as Trump Grande. Of the two symmetrical buildings seen in the picture above, the Trump Palace is the structure to the right (south).

Primary Project Team	
General Contractor	Coscan Waterways Inc.
Architect	Sieger Suarez Architectural Partnership
Structural Engineer	Gopman Consulting Engineers
Owner	Sunny Isles Luxury Ventures
Developer	Dezer Properties
Mechanical Engineer	Florida Engineering Services
Landscape Architect	EGS2 Corporation
Civil Engineer	EDC

The architecture for the Trump Palace was designed with both function and form in mind while considering its purpose as a high-rise luxury condominium and resort. The shear wall/column system is completely symmetric which facilitated the structural design. Much of this symmetry is due to the 3 elliptical cores that define much of the building. At the middle of each core is a void which contains room for elevator shafts, air-conditioning and mechanical equipment.

Adequate parking was provided for the building's occupants including a 2 story garage under the structure. To surround its occupants in a lavish environment, a tropically landscaped water activities center was added that includes a temperature-controlled lagoon pool with sloped, beach-grade entry, two temperature controlled lap pools, Jacuzzis, waterfalls and a snack bar & grill. A magnificent two story lobby entrance greats the Palace's occupants. Once inside the lobby, large aquariums and marble floors surround the occupants as they make their way towards the 6 private, high-speed elevators that will lead them to their

domain. Each residence features 10' ceilings and floor to ceiling windows, taking advantage of the natural lighting provided by the warm Miami sun. Individual HVAC systems with personal climate control were included in each unit to ensure comfort. Spacious outdoor terraces were included in every unit with panoramic ocean and intracoastal views. The residential units were designed for maximum space efficiencies and governed by where the large shear walls went through the building. These restrictions leave little room for room adjustment. There are two floors of multipurpose rooms located on levels 18 and 19 that include a 5000 SF duplex spa with his-and-her Jacuzzi, sauna, steam room, treadmills, stationary bikes and state of the art Nautilus equipment.

The exterior walls of the Palace are composed of both CMU blocks and concrete with a white stucco finish. Aluminum trim work is used for the exterior windows and balconies on the outdoor terraces. Hurricane rated, blue tinted-exterior glazing is standard for all exterior windows. The roof is constructed out of concrete and is also finished with white stucco.

Construction began on the Trump Palace in February 1, 2003 and is scheduled to be completed in July 1, 2005. The original contract sum for the building was $101,504,072.68. A design/bid/build project delivery method was used for this structure. The building is zoned as a mixed-use resort district (MU-R).

Existing Structural System

Description of Structure

The Trump Palace Tower works by using a system of post-tensioned slabs, shear walls and columns to resist gravity and lateral loads. The slabs are post-tensioned and poured thick to make them extra rigid, ensuring that the lateral loads are transmitted to the vertical resisting elements. Concrete beams also aid in the transfer of gravity and lateral loads into the shear walls/columns. The shear walls in this structure have three jobs which in effect makes them behave as a giant cantilevered beams fixed at the foundation. First, the shear walls have to transfer variable shear into the foundation which reaches a maximum at the base. The shear walls also need to transmit the bending moment. These moments will create vertical tension on the loaded edge of the shear wall and compression on the far edge. Lastly, the shear walls must transfer compression from ordinary gravity loads from the structure into the foundation.

The shear wall/column system is almost completely symmetric which facilitates the design. Much of this symmetry is due to the 3 elliptical cores present in this structure. At the middle of each core is a void which contains room for elevator shafts, air-conditioning and mechanical equipment. Shear walls will surround each of these voids to offer the additional support their empty space requires. The total area of shear walls/columns does not decrease significantly as you move towards the top of the building. However, the amount of steel reinforcement in these elements does decrease. For example, one part of shear wall 'A' is reinforced with 2 layers of 28 #11 bars @ 6" running from the foundation to level 4. From

level 4 to level 10 this reinforcement drops to 2 layers of 15 #11 bars @12". The reinforcement is again reduced to 2 layers of 15 #9bars @ 12" from level 10 to level 45. The strength of the concrete used in the shear walls and columns also decreases as you move higher up in the building. The shear walls are typically between 3'-6" to 4'-0" thick. The system of shear walls/columns concludes in the foundation, an impressive structural feature on this high rise. The foundation is a single mat that is 9' thick, supported by 350 30" piles and 250 18" stiffening piles. The foundation encompasses 26,000 square feet, requiring 9,000 yards of concrete and 1,500 tons of rebar. All of this to resist the overturning moment created by the lateral forces that are present during a severe hurricane.

Design Codes

The Trump Palace Tower has been designed in accordance with the Standard Building Code (SBCII 97). Calculations were prepared in accordance with all applicable codes and standards including ASTM, AISI, AISC and AWS. The reinforced concrete has been designed per ACI-318-95. Construction of this building complies with other various local codes and standards including:

- Standard Building Code (SBCII, 1997)
- American Concrete Institute (ACI 318-95)
- The South Florida Building Code – Dade County 1997 Edition
- The Life Safety Code – NFPA 101, 1994 Edition
- The Fair Housing Act 1968 – as amended
- The Florida Accessibility Code
- Americans with Disabilities Act (ADA) & ANSI 117.1
- FAA (limit to building height)

Required Loads

Dead Loads

- 8" slab = 100 psf
- 9 " slab = 112.5 psf
- 5" roof slab = 62.5 psf

Superimposed Dead Loads

- slabs = 25 psf (assumed)

Live Loads

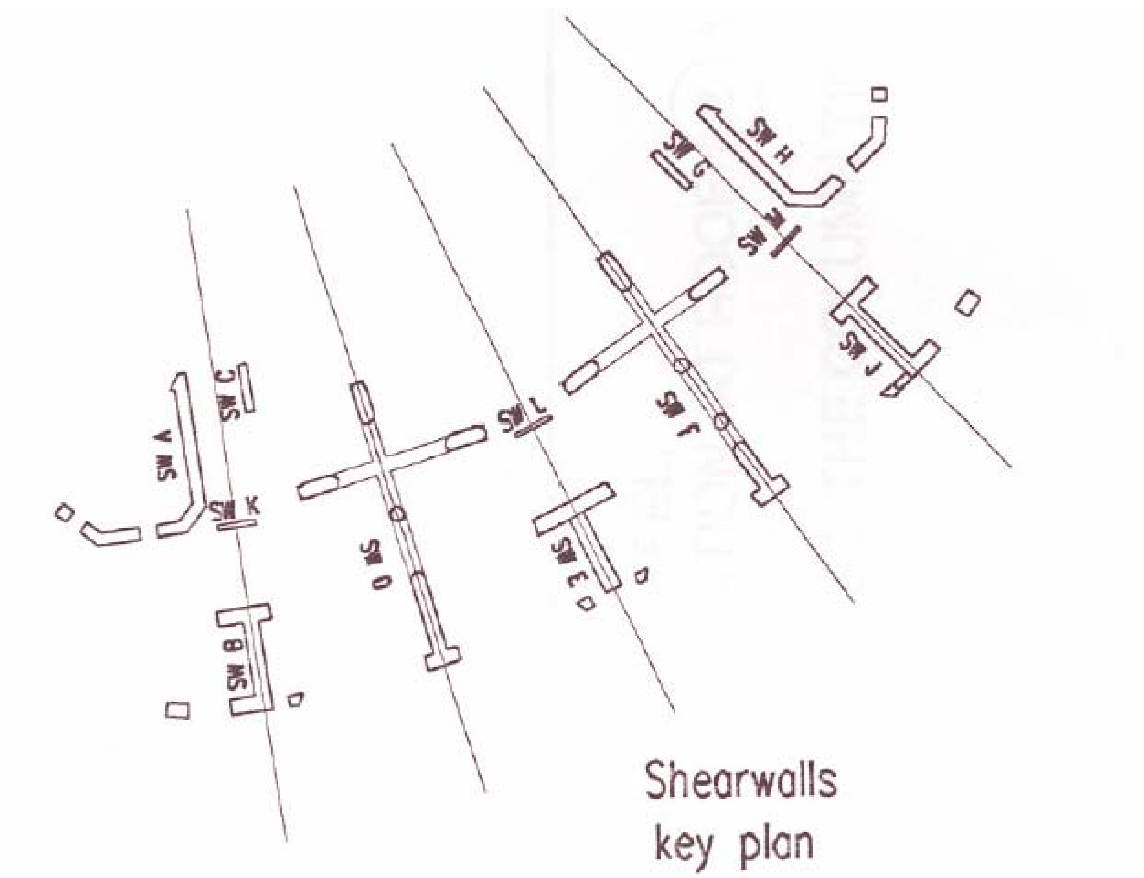
- apartments = 40 psf
- private balconies = 60 psf
- landscape decks = 100 psf
- lobby = 100 psf
- storage = 125 psf
- fan rooms = 50 psf
- pump and chiller room = 150 psf
- cooling towers = 100 psf
- path of egress – commercial = 100 psf
- path of egress – residential = 80 psf
- roofs – sloped = 20 psf
- roofs – flat = 30 psf

Gravity System

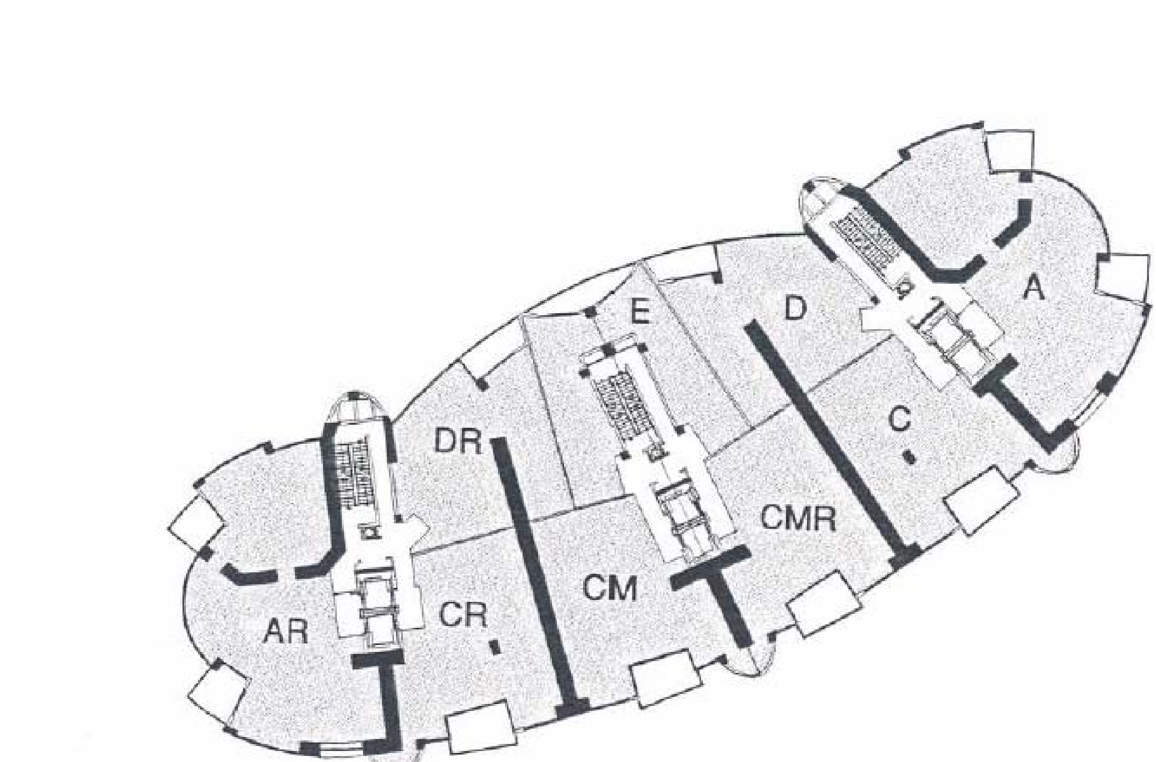
Gravity loads are supported by 8" thick post-tensioned concrete slabs which run from the lobby to the 45th floor. The concrete strength for these slabs is rated at 6000 psi from the lobby to the 28th floor. From the 29th floor to the roof the strength of the concrete is reduced to 5000 psi. The main roof features a 5" concrete slab on metal deck where the concrete has a strength of 4000 psi. Span lengths vary throughout the floor, but there is a maximum span length of 35'. Typical framing for the Trump Palace Tower begins in the post-tensioned concrete slabs which then direct the gravity loads into a system of shear walls/columns. The shear walls/columns are the backbone of the tower and are also designed to resist tremendous lateral loads due to the building's location in a hurricane region. The reinforced concrete shear walls/columns have the same strength requirements as they traverse vertically through the tower. The following is a summary of their strength requirements:

Shear Wall/Column Strength Requirements		
f'_c	E_c	Level
10ksi	4200ksi	Foundation to L18
8ksi	4200ksi	L18 to L28
6ksi	3700ksi	L28 to L40
5ksi	3500ksi	L40 and above

Below is a key plan for the shear walls that run throughout the structure. Here you can see how the shear walls are aligned 9° from each other towards an imaginary focal point, much like spokes on a wheel. It is this unorthodox configuration of shear walls that give the building its elliptical shape.



You can get a better idea of how the shear walls are laid out inside the building by looking at the floor plan below. From the floor plan, the elevator shafts become apparent which run up the center of each core inside the building. The shear walls surround each of these voids to offer the additional support they require.

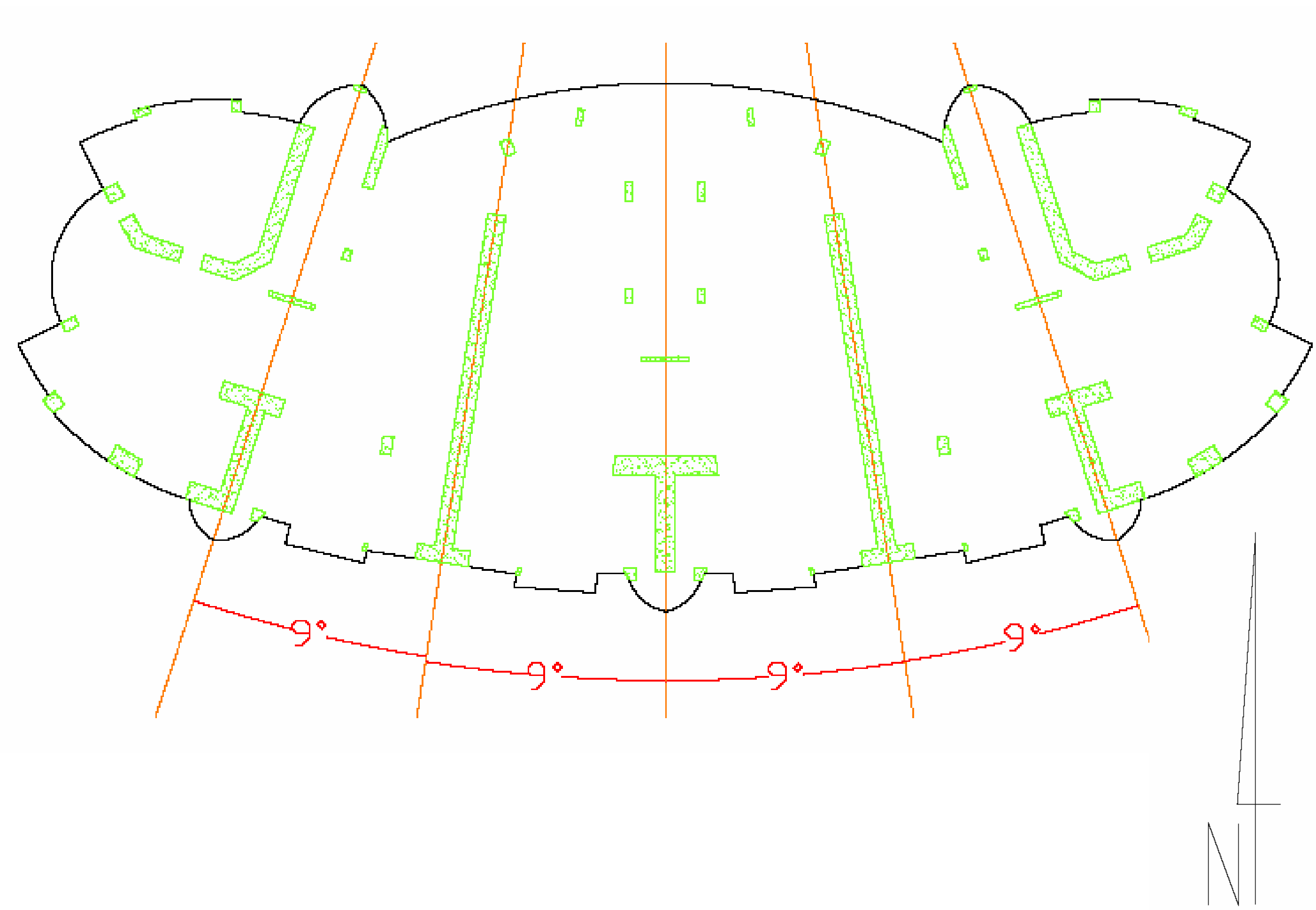


Key plan for levels 6-17

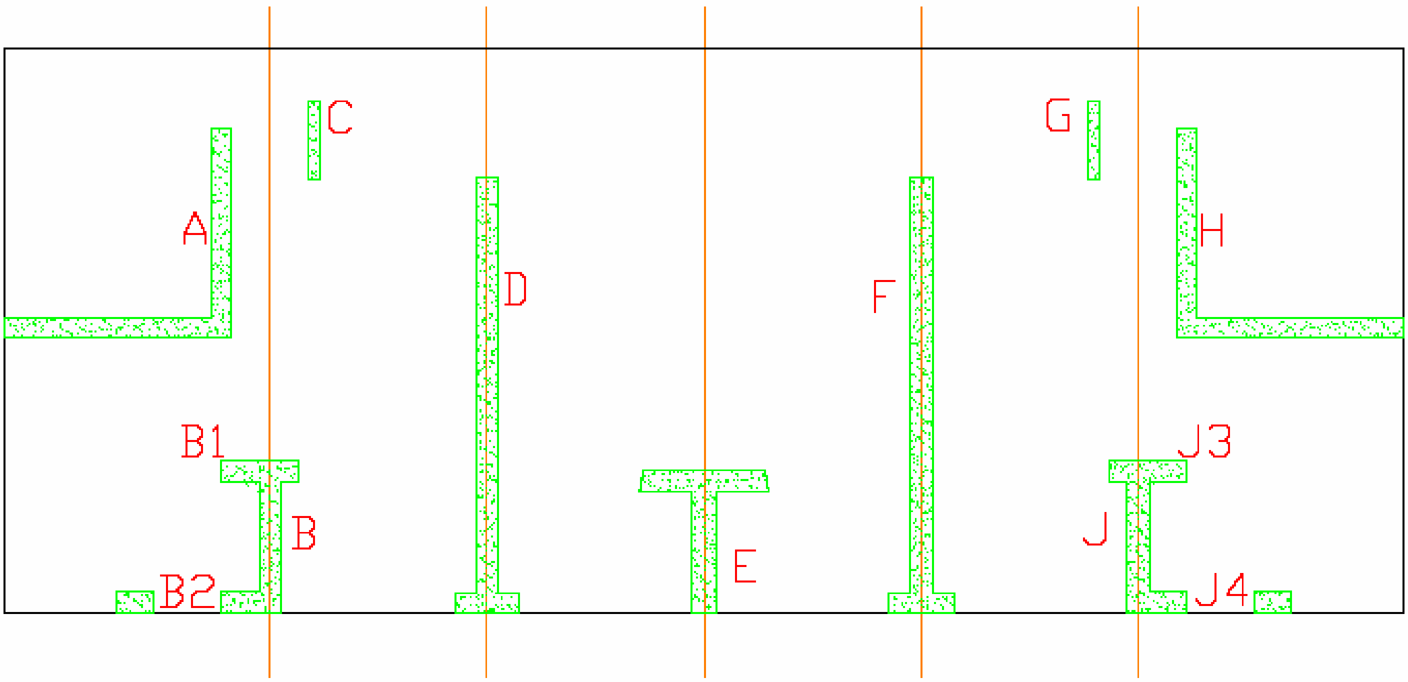
Lateral Loads

The lateral forces encountered by the Trump Palace Tower are resisted by its shear walls. Due to the unusual configuration of shear walls and also due to the elliptical shape of this structure I decided to simplify the structure for my own analysis. Below is a typical floor plan for floors 6 through 18 along with the simplified design I created to complete my analysis.

Existing Floor Plan for Levels 6-18

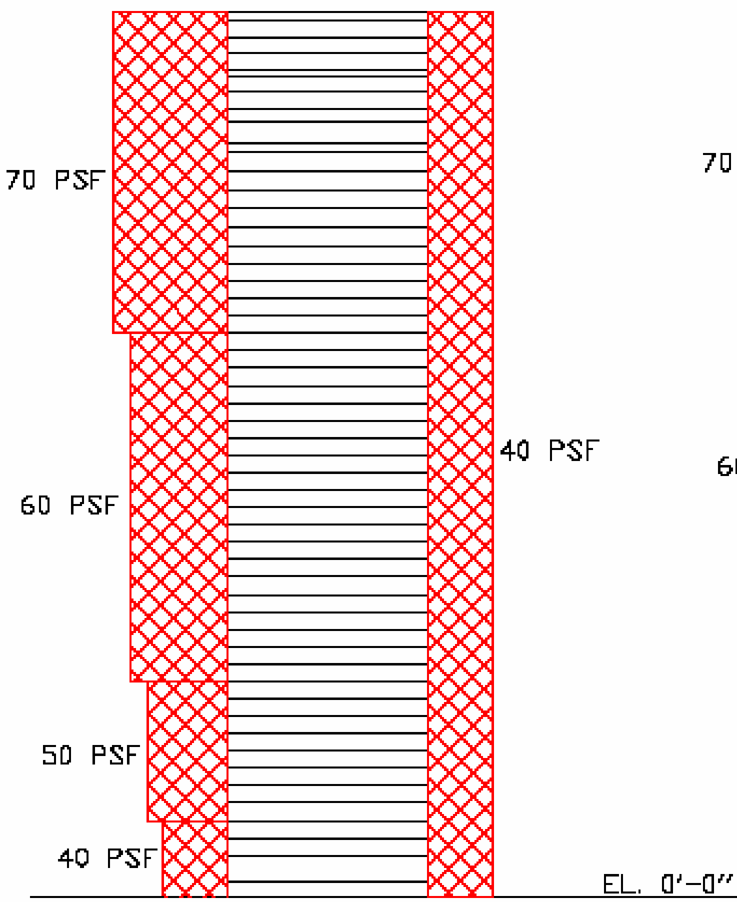


Simplified Floor Plan

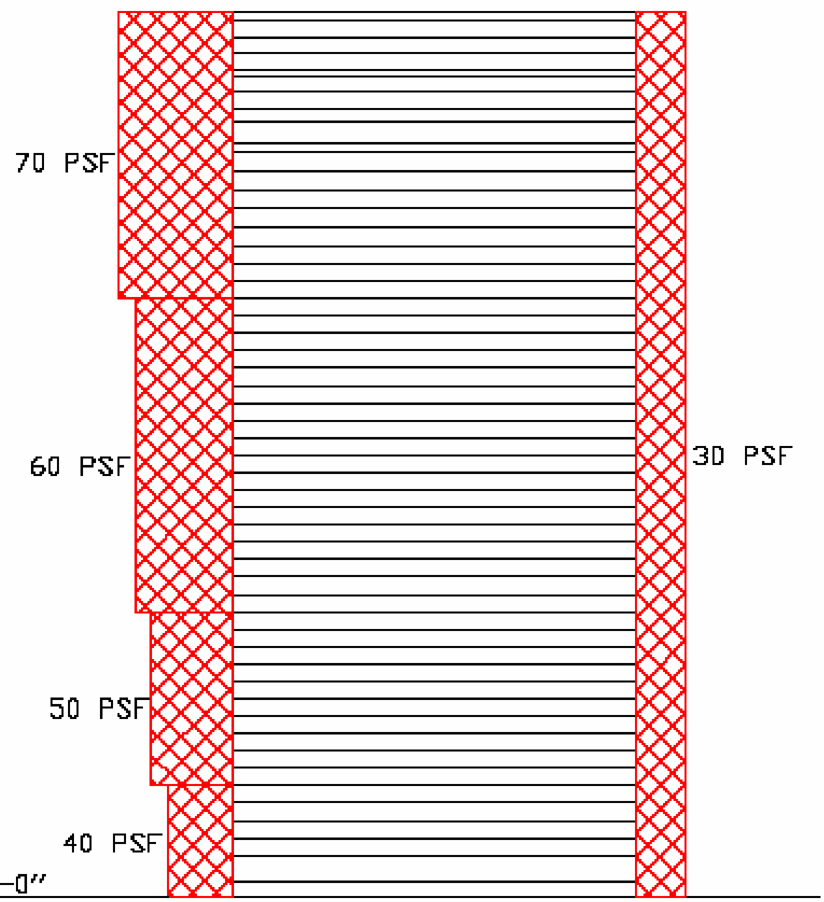


The main reason I chose to simplify the existing floor plan was to place the shear walls on a rectangular coordinate system. Another benefit of this simplified design was the rectangular exterior shape which was modified from its existing elliptical form. It must be noted that I am not suggesting an actual design change from the existing structure. This alternative floor plan was implemented only to understand how the existing lateral system functions. Columns were neglected from my simplified design but in reality they would offer a very small fraction of additional lateral support. This simplified design facilitated the analysis of the lateral system.

In hurricane prone regions such as Miami, the wind loading of buildings is an especially important design factor. Generally, building codes tend to be very conservative in their wind factor specifications. I calculated some rough wind loads using ASCE-7 1998. The following diagrams illustrate my results as a wind pressure distribution over the height of the building. Spreadsheets for calculation of my shear forces and overturning moments can be found in Appendix A.



N/S Wind Pressure Diagram



E/W Wind Pressure Diagram

Due to the tower's unique shape, the actual wind loads on this building were calculated via wind tunnel analysis. Wind tunnel testing not only increases the accuracy of wind load knowledge, but also tends to identify overall lateral load reduction when compared to more general load requirements. Significant cost savings can result. RWDI completed the wind tunnel analysis for the Trump Palace Tower using a 1:400 scale model of the structure and all surroundings within a full scale radius of 1600 ft. Their recommended design wind loads were based on a 50 year return period. The overturning moments and base shears calculated by RWDI were almost half the values that I calculated via analytical methods. Several confounding factors may account for such a variance in results. I made several assumptions in order to complete my wind analysis, the main one being that I assumed the tower to be perfectly rectangular. In reality the building has an elliptical shape to it. The tower also has openings which extend all the way through it in the north/south direction between floors 18 and 21. I also assumed a flat roof, where in actuality there exist three ovular cooling towers. The assumptions that I made were extremely conservative which in turn probably explain why my loads were much higher than RWDI's loads used for design. Also, since RWDI based their analysis on a 50 year return period, they were able to use a much lower wind velocity then prescribed by ASCE; V=115mph compared to V=150mph.

The lateral forces encountered by the Trump Palace Tower are resisted by both shear walls and columns. In the simplified design, the columns were removed meaning that the entire lateral load was now assumed to be resisted by just the shear walls. This simplified lateral support system was analyzed using the direct stiffness method. Using this method of

analysis, the lateral loads are distributed into each shear wall based on both its relative stiffness at that level and the direction of the applied load. Placing all of the shear walls on a rectangular coordinate system facilitated this analysis since the walls would be resisting the loads in only one direction (just an x component rather than an x and y component on a rectangular coordinate system). The spreadsheet entitled "Direct Stiffness by Floor for Each Shear Wall" was used to calculate the stiffness (k value) for each shear wall and can be found in Appendix A. This k value illustrates the proportion of the total load each wall resists at a given level and direction.

The shear walls that resisted forces in the north/south direction only had to resist the direct shear due to symmetry in the shear walls. The shear walls that resisted forces in the east/west direction had to resist the direct shear as well as torsion from the lateral loads. Torsional effects developed in the east/west direction because a difference existed in the location of the center of rigidity and the centroid of lateral loads. This difference is known as eccentricity and basically functions as a moment arm taken about the center of rigidity. In the east/west direction, the eccentricity was usually around 8' as you traverse the height of the building. The spreadsheets entitled "Center of Rigidity" and "Distribution of Forces" lists these results.

The orientation of the shear walls in this structure did not lend itself well to a computer analysis. Because of this the drift analysis was performed by hand and put into a spreadsheet. A limiting overall drift of L/400 was used on the building which resulted in an acceptable drift of 16.5". Next, this displacement was divided by the number of levels above

ground to get the acceptable story drift at each level. Drifts for each level were than calculated by using the equation $\Delta = F/k$. This method of analysis does not take into effect the compounding flexural effects over the height of the building, but it sufficed for an approximation of within 10% of the actual deflection. The results for the wind analysis can be found in the spreadsheet entitled "Story Drift" in Appendix A.

In the north/south direction the overall calculated displacement due to drift was 8.5". This is well under the 16.5" tolerance even with an additional 10% deflection to account for the flexure. However, a segment of the individual story drifts were slightly over the L/400 tolerance. This occurred between levels 7 through 17. In the east/west direction the overall calculated displacement due to drift was 14.5" which is also under the 16.5" tolerance. However, just as in the north/south direction a segment of story drifts exceeded the L/400 tolerance between levels 3 through 17.

After examining the results I found that the shorter east/west side of the building deflects almost twice as much as the longer north/south side of the building. At first glance this seemed to defy logic; the longer side deflecting more than the shorter side. However, after considering the orientation of the shear walls it became apparent that this is exactly what the designers had in mind. The total area of the shear walls oriented in the north/south direction is 5 times greater than that of the east/west direction. The area of the shear walls is directly proportional to the amount of stiffness in a given direction. Therefore, the north/south direction yielded a smaller drift since it was much stiffer than the east/west direction; regardless of the length of each side.

It is interesting that in both the north/south and east/west directions, there were story drifts that exceeded tolerance at the same levels. In the existing design, the north/south direction contains openings which extend through the building. These openings were neglected in my design to simplify the wind calculations. Interestingly enough, these openings are between levels 18 and 21; directly above where the story drifts failed in the north/south direction. After examining the results it seems that the openings may have been strategically placed at these levels to dampen the magnitude of the story drifts below them. If this is the case than the unusual openings and sky-lobby may be more than just architectural features. These openings may have been specifically created to reduce the effects of lateral loads. While this may account for the high drifts in the north south direction, it doesn't offer a solution for the high drifts in the east/west direction. As to the east/west direction, remember that the shear walls in actuality contribute a fraction of their force in both the x and y directions. I assumed in my simplified design that they only acted in one specific direction. The fraction that the shear walls actually add to resist the forces in the east/west direction may be enough to make the drifts more acceptable in that direction.

Foundation

Both the gravity and lateral loads eventually terminate in the building's foundation. The Trump Palace Tower rests on a 7000 psi mat foundation that is 108" thick. Furthermore, this foundation lies on top of a network of 350 – 30" diameter augercast piles, each having a minimum depth of 74'-0". There are also approximately 250 – 18" diameter structural stiffening piles under the slab that reach a minimum depth of 35'-0". This foundation behaves as a fixed support to resist overturning moments transferred down through the shear wall lateral system. From RWDI's Wind-Induced Structural Responses report, the maximum overturning moment to be resisted in the north-south direction is 1,760,000 ft-k. In the east-west direction a moment of 1,082,000 ft-k needs to be absorbed. The enormous dead load of the building is enough to resist any uplift forces encountered by the overturning moments.

Proposal

Problem Statement

The nature of this building does not lend itself to many other options when considering a structural redesign. Concrete is the material of choice in southern Florida due to the abundance of its constituent materials in this region. Large steel structures are also uncommon in this area because there are no local steel mills. Steel must be shipped great distances which makes its material cost unreasonable. As a result, laborers in this region are much more familiar working with concrete than with steel. A steel structure would drive construction costs up in this region more so than in a region where steel and concrete were competitive materials. Lastly, steel would be a poor material in a high rise such as the Trump Palace tower where hurricane force winds are prevalent. The story drifts would be unacceptable and it would be much more prone to overall failure. Due to these restrictions I decided to modify the existing structure. A previous analysis of alternative floor systems suggested that the current 8" thick post-tensioned concrete slab may not be the most efficient slab system. The current system is a good solution for long spans, but over shorter spans the slab becomes less efficient.

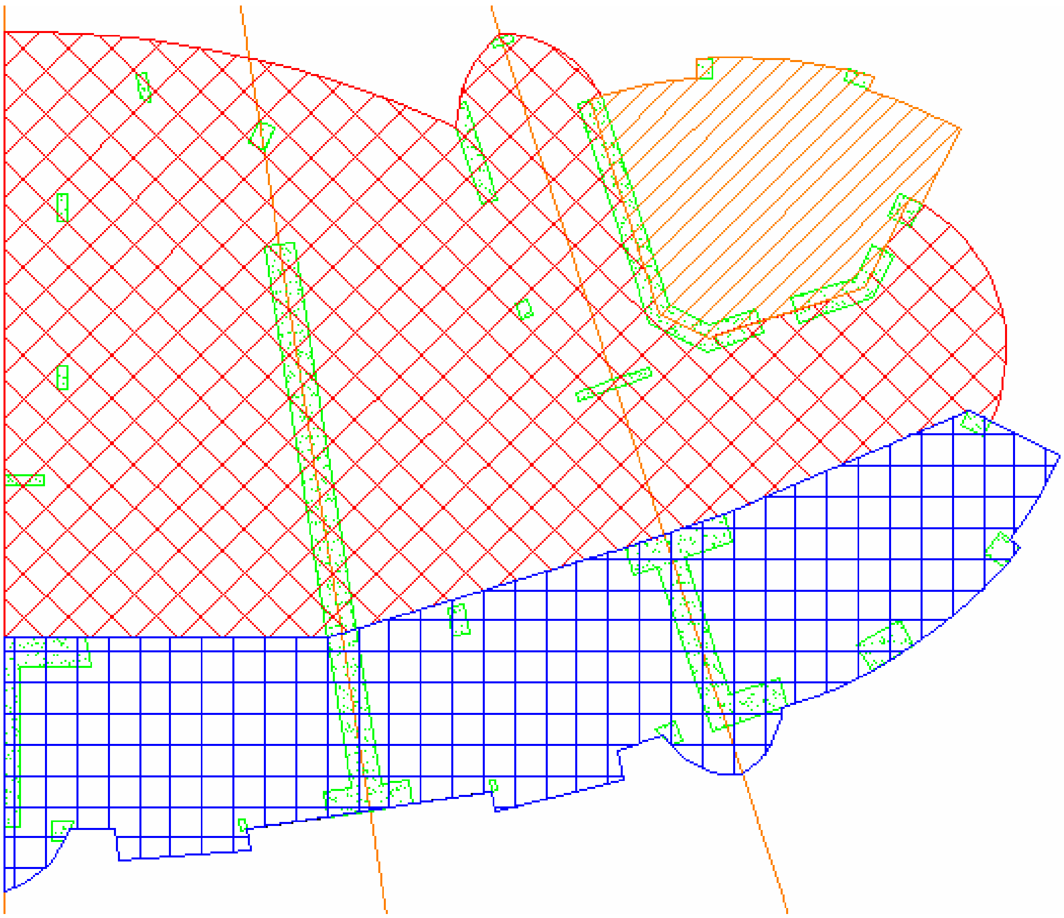
Design Criteria

The proposed redesign will focus on removing the post-tensioning cables where possible and introducing a combination of one and two way floor slabs. This alternative system will then be compared to the existing design to determine it is in fact a more efficient design. The efficiency of the two designs will be evaluated by considering overall constructability. The proposed redesign is controlled by the following design criteria:

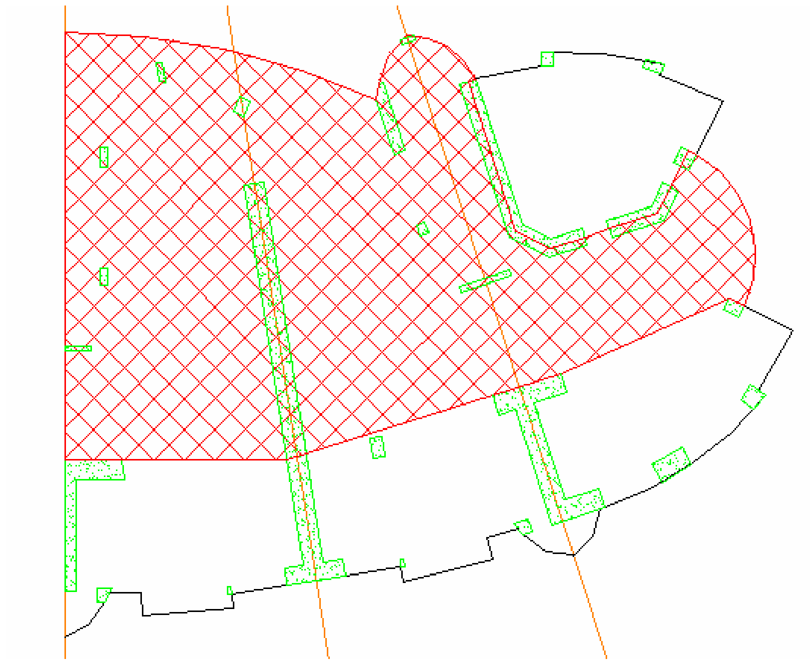
- building height may not be increased (due to a local FAA regulation, building heights may not exceed 550'-4")
- at least 10' floor to ceiling heights must be maintained
- the interior floor plan must not be interrupted by additional columns
- the design must conform to ACI 318-02

Structural Redesign

The existing post-tensioning cables in the slab created a one-way action that acted parallel to the length of the building. To begin my redesign, I examined the existing frames in order to determine how the slabs would behave in the system without these post-tensioning cables. Symmetry in the building facilitated this process since I only had to examine one half of the total floor area. After examining the frames I determined that there would be three critical areas that would need to be redesigned. The concrete strength used for my redesign is 6000 psi, which is the same strength used in the actual design.



Critical Area 1



The slab action in this area is a one way edge supported slab that must support a maximum span length of 35'. Spans of this length are usually controlled by either bending or deflection. In bending it was determined that without post-tensioning, the slab would need to be increased from 8" to 10" in thickness. The increase in slab thickness added a significant 25psf dead load. The redesigned slab also required #11 bars @12" o.c. to achieve its required moment capacity. The existing slab contained small #4 bars for temperature and shrinkage reinforcement. The extra 2" in slab thickness is a problem because the building height can not be increased due to an FAA regulation. This 2" per slab translates into 8' of

additional building height. I decided to continue with the analysis however, and worry about possibly taking some height off the cooling towers if necessary to meet code.

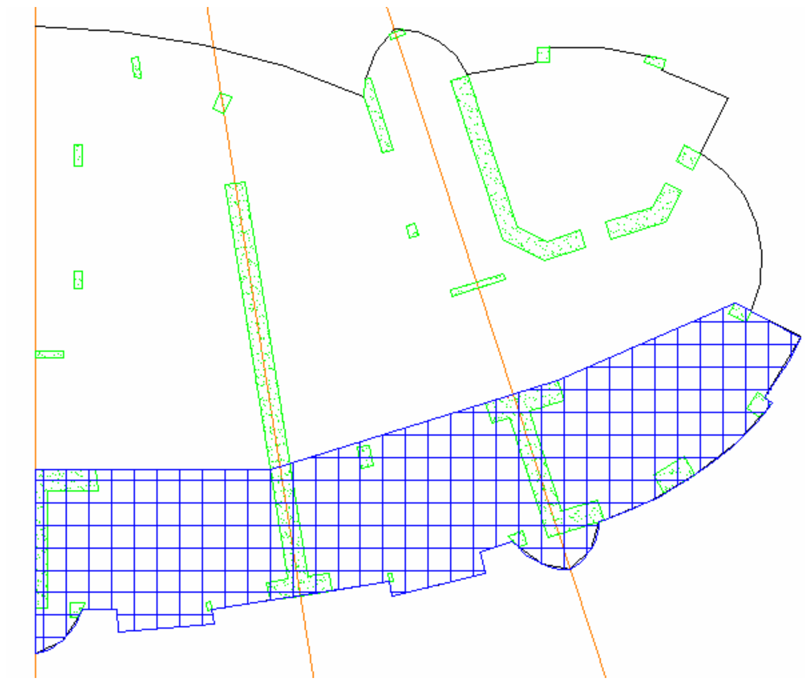
ACI Code 9.5.2 specifies a minimum thickness in Table 9.5(a) for non-prestressed slabs of normal weight concrete. According to this table, with a span length of 35’ the minimum slab thickness would need to be 15” (L/28, both ends continuous)! Lesser thickness may be used if calculation of deflections indicates no adverse effects. Since the ACI recommended thickness was unreasonably thick, I decided to calculate the immediate deflections by the same method as for beams. Using an unfactored load, I calculated a deflection of 2.24” for a maximum span of 35’. This was found to be under the L/180 deflection limit which was 2.33”.

Taking the post-tensioning out of critical area one did not yield any advantages. In order to meet the design criteria and the 35’ maximum span, the redesigned slab was:

- increased from 8” to 10”
- reinforced with #11 bars at 12” o.c.
- designed with a deflection approaching the L/180 limit

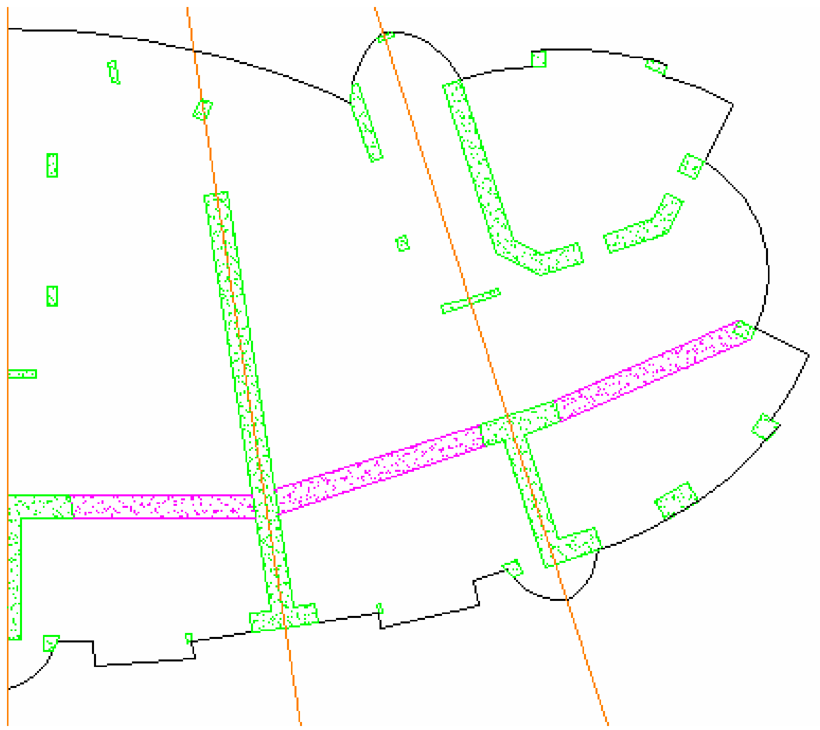
It is an engineer’s duty to design safe structures. While the slab I redesigned for critical area 1 will theoretically support its required loads, the slab is also approaching the limits of what a slab of this nature can do. A 10” slab thickness is the maximum thickness in common practice for a 1 way edge supported slab. In addition, the amount of reinforcement required to make the slab work is also approaching the limits of standard practice. Considering all of this and the slab still barely satisfied its deflection criteria. Due to the inefficiency of the redesigned slab I recommend that the existing post-tensioned slab remain in place for critical area 1.

Critical Area 2

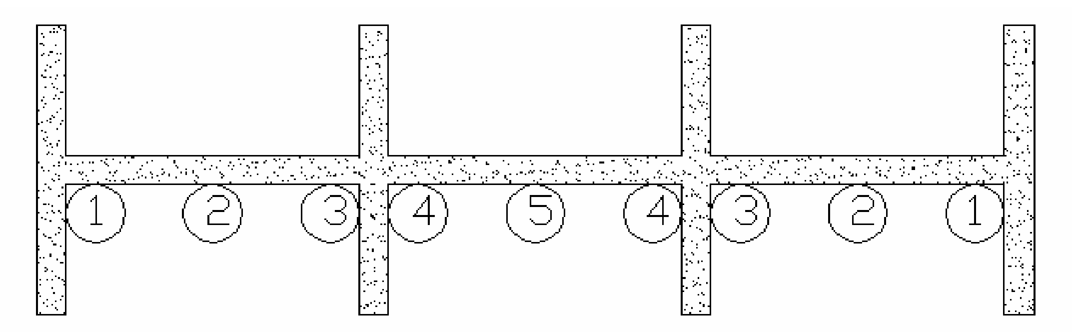


In the second critical area, the original floor slab was designed for a six span condition. The 40' spans in this area were only achieved through the use of post-tensioning cables. Since a non-prestressed conventional slab cannot achieve a 40' span I decided to add a girder and change the load flow, making it orthogonal to the original load flow. This new configuration now made it possible for a non-prestressed conventional slab by reducing the 40' span to a 20' span. As a result, the new slab became a one end continuous slab. The girder was designed for a 3 span condition under differential loading. In the picture below you

can see where the concrete girder (magenta) has been added. The addition of this girder eliminated the column that was below it.



The slab for critical area 2 was redesigned as a 6" one way edge supported slab. Reinforcement for this slab included #9 bars at 18" o.c. This redesigned slab is 2" less in thickness compared to the original slab which reduces the dead load by 25psf. Under these conditions, the girder was designed to be 36" wide and 18" deep. Reinforcement varied throughout the girder and was based on the worse moments calculated by a differential loading analysis. The required reinforcement for the girder is summarized in the table below:



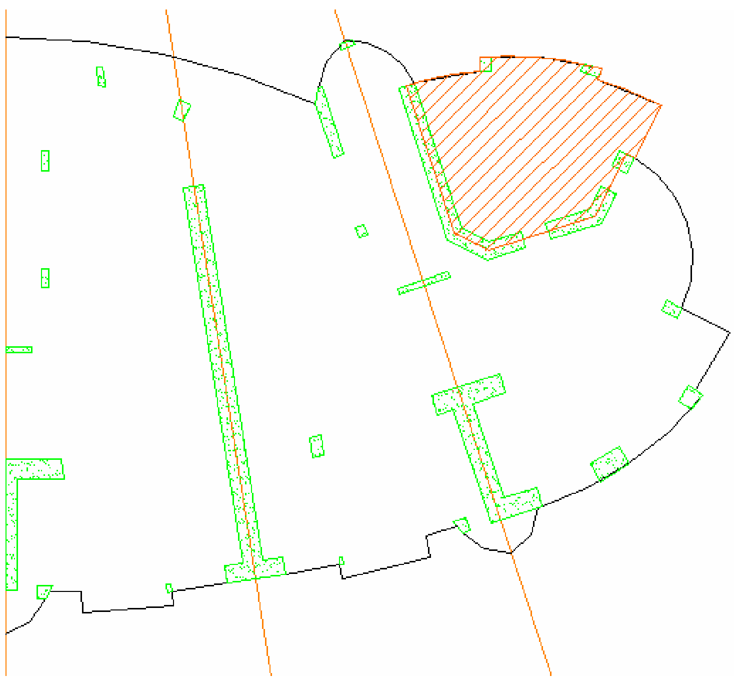
Girder Reinforcement Summary Base on Worst Moments		
Location	worst moments (ft-kips)	reinforcement
1	236.4	Use 4-#9 bars
2	253.0	Use 4-#9 bars
3	400.4	Use 6-#10 bars
4	442.0	Use 6-#10 bars
5	181.1	Use 4-#8 bars

The girder also required shear reinforcement which included #4 double stirrups @ 4" near supports. At 6" the reinforcement dropped to single stirrups and at 17' from the support the reinforcement dropped to no stirrups.

Once again, Table 9.5(a) from ACI code 9.5.2 was used to check the minimum thickness of the slab. According to this table, with a span length of 20' the minimum slab thickness would need to be 10" (L/24, one end continuous). However, lesser thickness may be used if calculation of deflections indicates no adverse effects. Since the ACI recommended thickness was thicker then the 6" slab I designed, I decided to calculate the

immediate deflections by the same method as for beams. Using an unfactored load, I calculated a deflection of 0.19" for a maximum span of 20'. This was found to be well under the L/180 deflection limit which was 1.33". The redesign of critical area 2 was successful.

Critical Area 3



The framing that surrounded critical area 3 seemed ideal for a two-way edge supported slab. The precise determination of moments in two-way slabs with various conditions of continuity at the supported edges is mathematically formidable and not suited to design practice. For this reason, various simplified methods have been adopted for determining moments, shears, and reactions for such slabs. The coefficient method is among these simplified methods of analysis and is the one I used to design this slab. The method

makes use of tables of moment coefficients for a variety of conditions. These coefficients are based on elastic analysis but also account for inelastic redistribution. In consequence, the design moment in either direction is smaller by an appropriate amount than the elastic maximum moment in that direction. The moments in the middle strips in the two directions are computed from $M(a)= C(a)wL^2(a)$ and $M(b)= C(b)wL^2(b)$.

After analysis via the coefficient method it was determined that an 8" two-way edge supported slab would be acceptable. Span 'A' was 28' in length while span 'B' was 34'. Consistent with the assumptions of the analysis of two-way edge supported slabs, the main flexural reinforcement is placed in an orthogonal pattern, with reinforcing bars parallel and perpendicular to the supported edges. Reinforcement was selected by first determining 'p' (rho) from a table and applying the equation 'p'(rho) = As/bd. From this equation the area of steel could be determined and the reinforcement was selected. A summary of the slab required reinforcement is detailed below.

Two-Way Edge Supported Slab Reinforcement	
Short Direction	Long Direction
Midspan: #6 bars at 12" o.c.	Midspan: #5 bars at 12" o.c.
Continuous Edge: If 2 of every 3 positive bars are bent up and likewise for the adjacent panel, the negative moment steel area furnished at the continuous edge will be 4/3 times the positive moment steel in the span. OK, inelastic redistribution.	Continuous Edge: As in the short direction, bending up 2 of every 3 bottom bars from the panels adjacent to the continuous edge will provide the necessary reinforcement.
Discontinuous Edge: The negative moment at the discontinuous edge is 1/3 the positive moment in the span. It is adequate to bend up every third bar from the bottom to provide negative moment steel at the discontinuous edge.	Discontinuous Edge: It is adequate to bend up every third bar from the bottom to provide negative moment steel at the discontinuous edge.

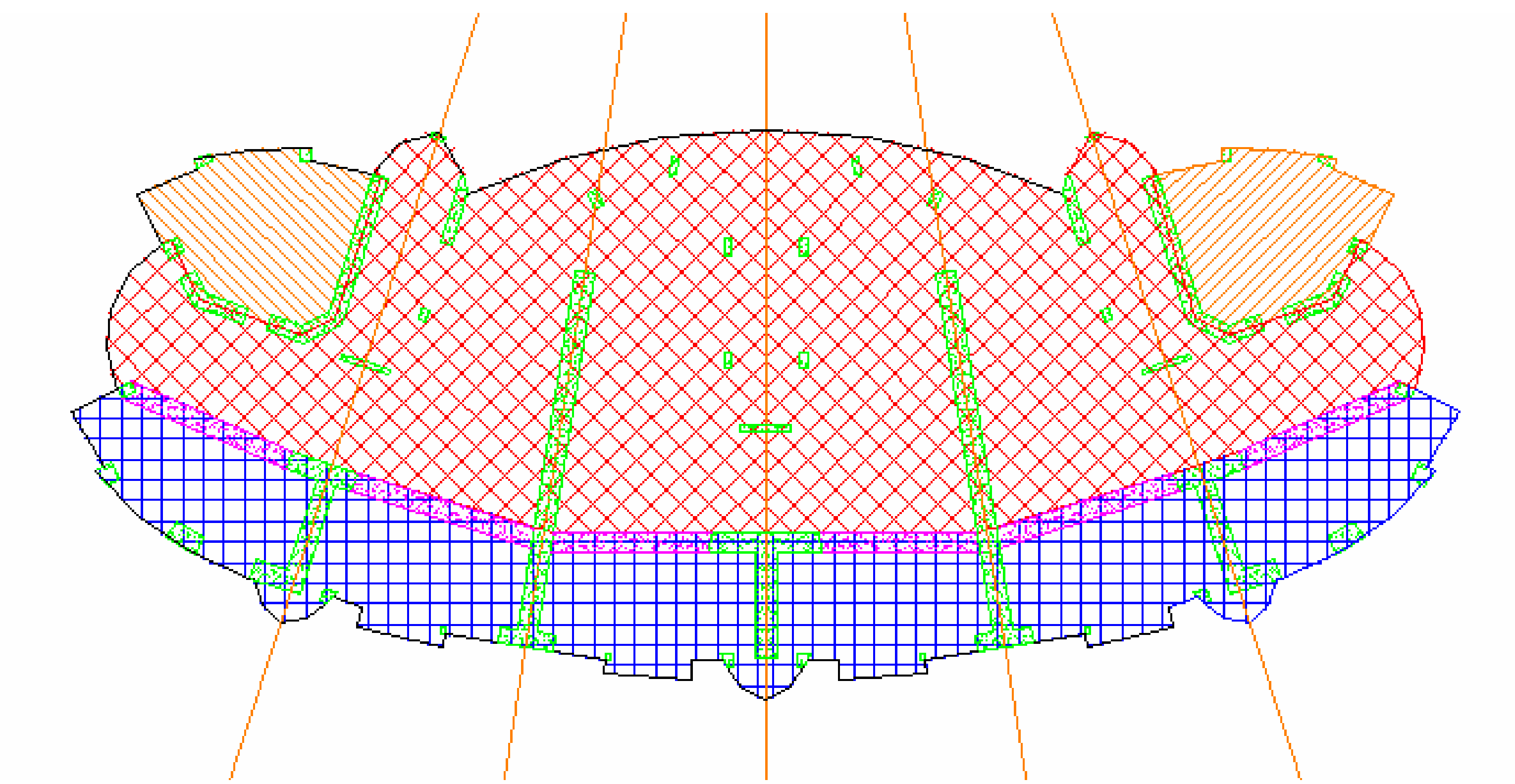
Edge supported slabs are typically thin relative to their span, and may show large deflections even though strength requirements are met, unless certain limitations are imposed in the design to prevent this. The deflection of an edge supported slab can be estimated with reasonable accuracy based on the moment coefficients used in the flexural analysis. The deflection components of concern here are the long-term deflections due to sustained loads and the immediate deflection due to live load. Through this analysis the deflection at the center of the slab panel was calculated and compared to ACI code limitations.

The immediate deflection at midspan due to dead load, including a time dependent increment, was 0.188". The live load deflection was calculated to be 0.12". The deflection

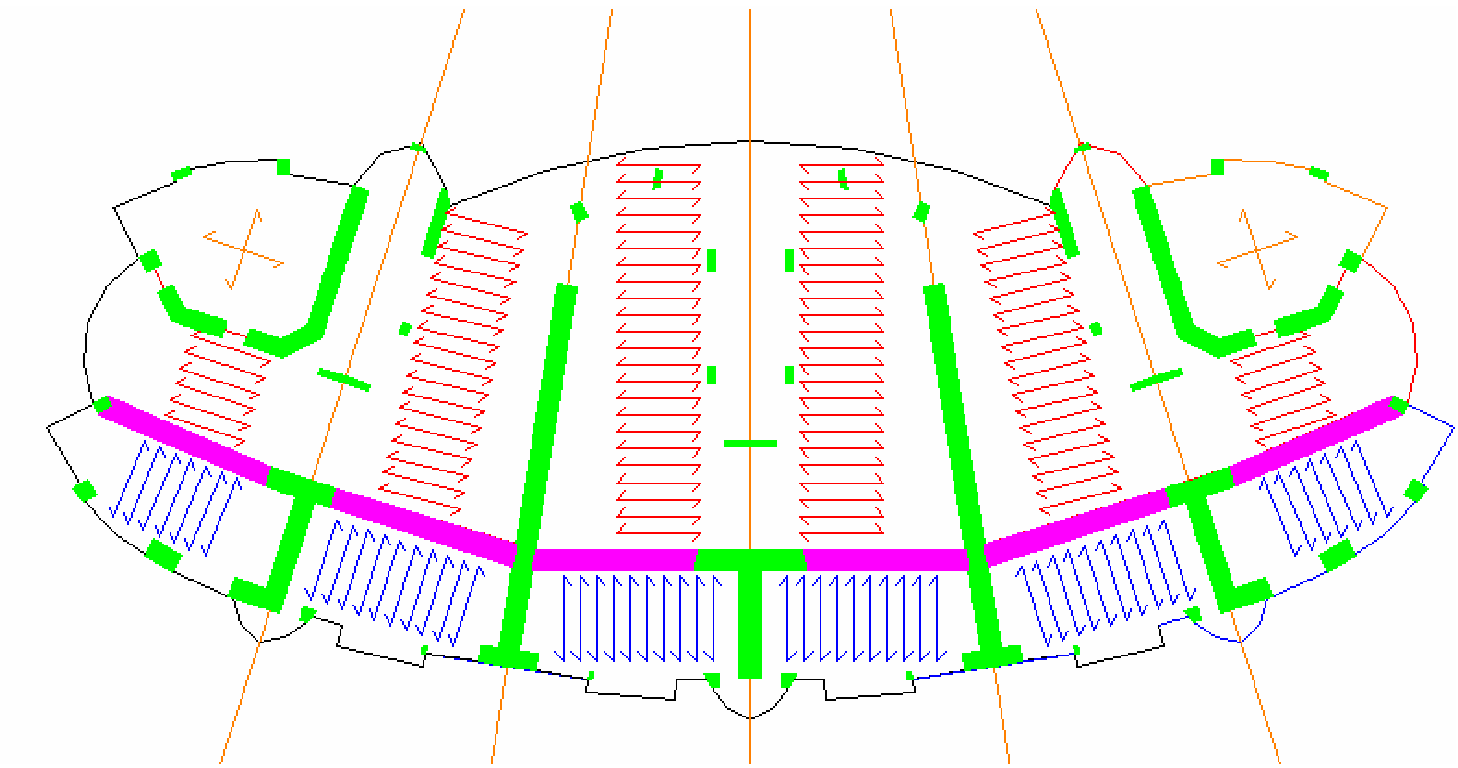
causing potential damage is the sum of the incremental time-dependent dead load deflection occurring after three months and the immediate deflection due to live load:

$\Delta = 0.188" + 0.12" = 0.31"$

Assuming an L/480 deflection limit the maximum allowable deflection according to ACI 318-02 was 0.7". The slab I designed for critical area three is therefore adequate to support its required loads. Below is a picture that illustrates the three critical areas that were considered over the entire floor.



Conclusions



The picture above describes the load paths after the floor was redesigned. Overall, the redesign was successful for critical areas 2 and 3. Critical area 1 had to remain post-tensioned however in order to accommodate its long 35' span length. The redesigned slab has some benefits over the existing design. By reducing the slab thickness from 8" to 6" in area 2, the dead load of the slab was reduced by 25 psf. This saves 27 CY per floor, or about 1300 CY over the height of the building. This comes out to a weight reduction of 120

kips per floor, or 5400 kips over the height of the building. This weight reduction would relieve some of the bearing forces experienced by the foundation. At the same time, this weight loss is not significant enough to raise the overturning moment to the point where the foundation would need to be redesigned. The introduction of the large girder in area 2 allowed me to remove columns 10 and 22. This may have given the architect more freedom when he was initially designing the rooms. However, as the rooms are currently designed with the columns located in the wall of a closet, this is not much of a benefit.

There are also several disadvantages with my redesign. In areas where post-tensioning was removed the slabs required heavy reinforcement to cover their spans. The material cost and the labor required to place this reinforcement is probably more expensive than the original post-tensioned system. It is also clear that my redesigned system would complicate construction and add time to the schedule. This is mainly due to the fact that my redesigned floor is segmented into different areas that feature different slab construction. Compared to the original monolithic slab, my redesigned slab would increase overall costs.

Although the redesigned slab is capable of resisting the required loads, it was found to be less efficient than the original design. Therefore my recommendation is to use the original design.

Façade/ Balcony Durability

The building envelope accounts for 15-30% of the initial cost of a building, is the principle factor governing the whole costs and energy use and is a major influence on the achievable service life of a building. The Trump Palace Tower is located in an environmentally harsh region; one where both high winds and a marine environment can wreak havoc on the building's façade. In this investigation I decided to verify that the Trump Palace's façade system met the regions governing code requirements. I focused on the building's balconies and areas around them and divided the investigation into 3 subcategories: concrete, glazing/ aluminum frames and balcony glass railings.

Concrete

To protect the concrete reinforcement from the elements, minimum concrete cover was established as per ACI 318-95 7.7. Any formed concrete that was exposed to earth or weather had the following minimum requirements:

#6 bar and larger	2" Cover
#5 bar and smaller	1-1/2" Cover

In addition to this, the concrete on the balconies was subject to a special provision for corrosion. The provision stated that the concrete on all exposed balconies had to be at least 5000 psi and contain master builders rheocrete 222 corrosion inhibiting admixture at the

rate of 1.0 gallon per cubic yard of concrete. This admixture was to be used in conjunction with other compatible admixtures and in accordance with the manufacturer's recommendations. The water in solution was counted as mixing water for the purpose of determining the water to cement ratio of the concrete. The maximum water to cement ratio was set at 0.40. The concrete used in the Trump Palace exceeds this provision by using 6000 psi concrete on the balconies (although this was for structural purposes). The water to cement ratio was specified at 0.40 to make the concrete a less permeable material. At this ratio, cracks are less likely to develop which would otherwise allow salt water to infiltrate the material and corrode the reinforcement. The concrete balconies were also sloped 2" over their length to prevent pooling of water.

Glazing/Aluminum Frames

The main threat to the exterior glazing is the substantial wind loads they must absorb. The windows of the Trump Palace Tower were designed accordingly to the "Recommended Wind Loads for Cladding Design" provided by the RWDI wind tunnel analysis. The RWDI wind report included peak positive/negative pressures over a 50 year return period. The wind loads yielded through the wind tunnel analysis were applied to the building's cladding system in the same manner as would wind loads calculated by building code analytical methods. It is also important to note that the wind loads provided in this report applied to cladding elements behind the balcony guard rails. Based on the data provided by RWDI, 3/8" tempered glass windows were selected for the building's units.

An additional requirement for windows was also required to prevent damage from flying objects in the event of a hurricane. According to the Standard Building Code (SBC) Dade County Edition, the contractor was required to provide Miami – Dade product control approved hurricane shutters for all windows, sliding glass doors and store fronts 30' or less above finished grade except where store fronts were engineered and installed to meet or exceed SBC impact requirements. Luckily, residential units don't begin until level 4 which is above the required 30' so this code doesn't apply to the balconies. This did, however apply to the levels below including the lobby where appropriate measures were applied. The glass in the lobby, for example, had specific structural performance criteria: maximum deflection of L/180 of the span without exceeding allowable stress and safety factor of 1.50 under wind loads prescribed by South Florida Building Code, Dade County 1994 edition.

To protect from corrosion, exposed aluminum framing elements were to be finished with a Fluoropolymer Three-Coat System. This included the manufacturer's standard three-coat, thermocured system consisting of specially formulated inhibitive primer, fluoropolymer color coat, and a clear fluoropolymer top coat, with both the color and clear coats containing not less than 70 percent polyvinylidene fluoride resin by weight, complying with AAMA 2605. A coating was also provided which has been field tested under normal range weather conditions for a minimum of 20 years without significant peel, blister, flake, chip, crack, or check in the finish, and without chalking in excess of 8 (ASTM D 659) and without fading in excess of 5 NBS units.

In addition to corrosion resistance, extruded aluminum sills were sloped for positive wash, one piece full width of opening. Water penetration resistance, required by the Southern Florida Building Code, was met by pitching pan type extruded aluminum with approximately 3-1/2" back leg or longer if necessary. Finally, weather stripping included double weather stripping, water resistant polypropylene pile and virgin PVC vinyl in strict accordance with Miami-Dade County Product Approval.

Balcony Glass Rails

By code, the top rail of railing systems was designed to be capable of supporting the following loads applied as indicated:

1. Uniform load of 50 lbs per linear ft. in any direction
2. 200 lbs per linear ft minimum each post
3. Concentrated load need not be assumed to act concurrently with uniform loads.

The infill area was designed to withstand a horizontal concentrated load of 200 lbs per linear ft. applied to 1 SF at any point in the system, including glass panels, intermediate rails, balusters, or other elements composing the infill area. Also, each section of top rails was supported by a minimum of 3 glass panels or by another means so that it remained in place should any one panel fail.

The railing was also designed to allow for thermal movements resulting from the following maximum change (range) in ambient and surface temperatures by preventing

buckling, opening of joints, overstressing of components, failure of connections, and other detrimental effects. Engineering calculations were based on surface temperature of materials due to both solar heat gain and nighttime-sky heat loss.

ΔTemperature (Range): ambient ---------------- 120°F (67° C)
: material surfaces ---- 180°F (100°C)

In addition to these guidelines, the balcony railing had to resist the applicable wind loads defined in the RWDI Wind-Induced Structural Responses Report. The report indicated that for the design of all balcony guardrails it is recommended that a net wind load of 100 psf be considered. The aluminum elements in the balcony guard rail are governed by the same finishing requirements stated above for the aluminum frames for corrosion protection.

Overall Durability

The Trump Palace Tower appears to be very durable according to the results of my investigation. Water penetration and the effects of high wind loads were of chief concern for the longevity of the balconies. The exterior aluminum pieces will be the first elements that will need major maintenance since its finish is only expected about 20 years. However, concrete on the balconies and weather proofing materials should be routinely inspected to prevent the elements from getting a foothold.

Solar Impact

I decided to investigate the impact that the sun would have on the Trump Palace Tower for three reasons. First of all, buildings in a warm sunny climate such as those in Southern Florida will inherently require more energy to cool them. The occupants of these buildings will find themselves constantly cranking up their air conditioning units to maintain a comfortable environment. This increased energy consumption translates into a more expensive energy bill. Secondly, the shear height of the Trump Palace eliminates most of the obstructions between the sun and the building. This creates a clear line of site between the building and the sun that makes the building even more susceptible to the sun's influence. In other words, the building will be receiving the entire daily impact of the sun's energy. Lastly, since the Trump Palace is located directly off the coast of the Atlantic Ocean, it will be receiving more of the sun's energy due to the reflection off the water.

Due to Miami's location in the northern hemisphere, the south side of the building will be receiving more direct sunlight. Therefore, occupants who reside in the south end of the building will be paying more for their electric due to an increase in their air conditioning load. You can imagine how these occupants might be upset to discover that their northern neighbors are enjoying a cheaper energy bill! In an attempt to eliminate the difference in electricity costs I decided to investigate possible solutions.

Before I could investigate any possible solutions, I needed somewhere to start from so I could compare my alternatives. To accomplish this I first had to determine the magnitude of the sun's impact as it currently existed between the north and south sides of the building.

This was accomplished using the Hourly Analysis Program (HAP). HAP seemed to work very well for this analysis since it took many influencing factors into account; location, cooling schedule, room size, exterior glazing, etc. Another factor that makes this building susceptible to solar impacts is its abundant exterior glazing. Each room features 10' floor to ceiling exterior windows and sliding glass doors. In fact out of 1470 SF of total exterior wall, 1070 SF of it is glazing. After all of the variables were entered, a monthly simulation was run. A unit on the north side of the building required 196762 KBTU per year compared to 227616 KBTU for the south unit. As the building is currently designed, an occupant in a south unit will be paying up to 16% more for air conditioning than an occupant in a north unit.

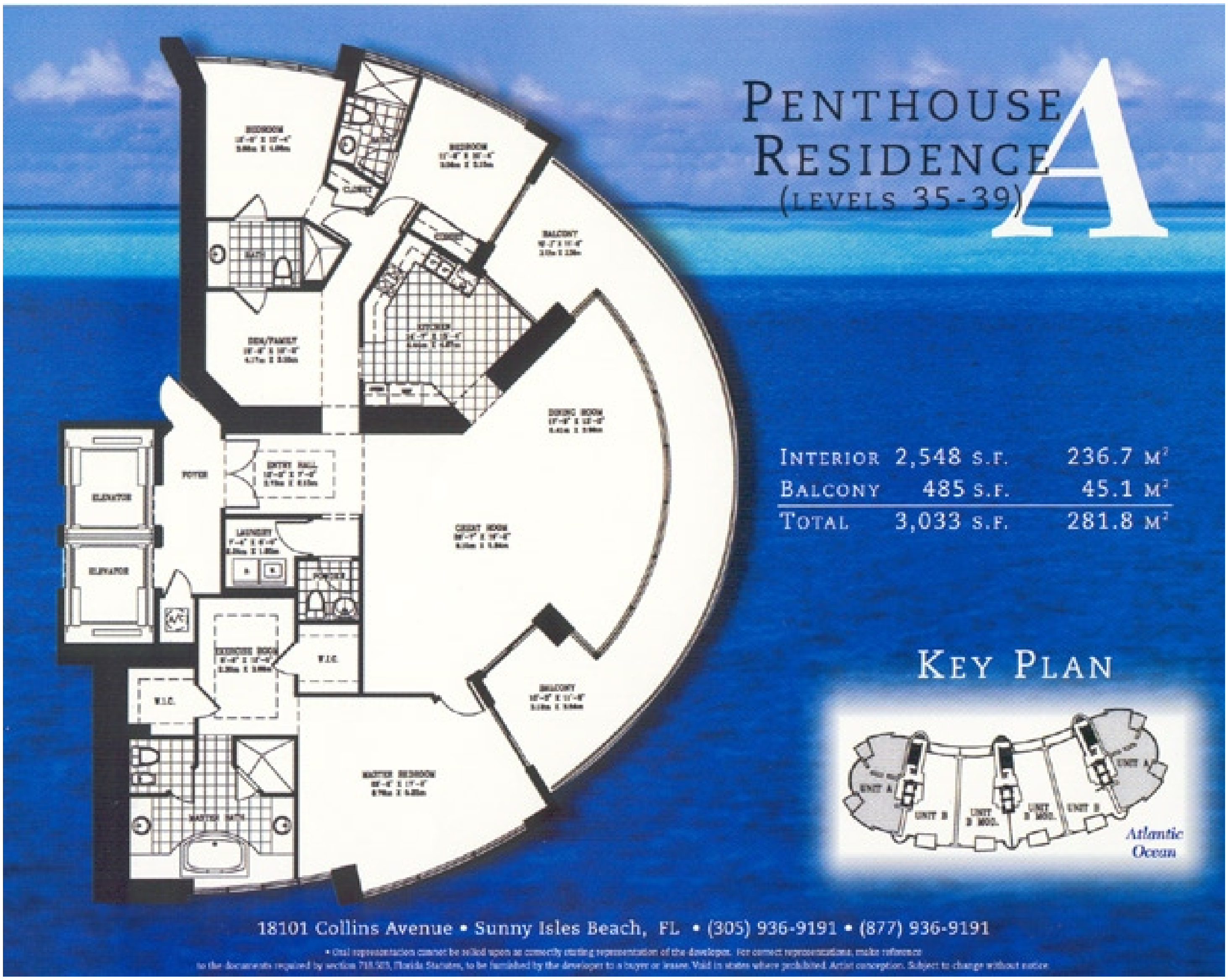
Now that I had some numbers that represented the existing conditions I could begin working towards a solution. My first idea was to simply reduce the area of glass on the south side of the building. This seemed to be the most direct approach so I began to change my numbers in HAP and I ran some new simulations. I discovered that I would need to reduce the amount of exterior glazing by almost 25% in the south units in order to achieve a balance between the north and south side. I determined that this was not a viable solution since it was too drastic a change in the building's architecture. The spacious 10' floor to ceiling windows would need to be reduced to a mere 7.5' all around to achieve this effect!

My next approach was to try a more thermal efficient exterior glazing. In the original simulation, the glass that was used had a U-value of 0.4. This is considered a pretty good thermally efficient number. However, after doing some research I discovered some glass that had an even lower U-value of 0.35. I decided to run another HAP analysis using the U-

value of 0.35 for the south windows. This yielded a terminal cooling coil load of 211102 KBTU, compared to 196762 KBTU for the north units. In this scenario the occupant in the south unit would be paying only 7% more for air conditioning compared to the occupant in the north unit.

I recommend that for the units on the south end of the building that a more thermal efficient glass be installed in order to more closely balance the energy consumption between the north and south sides of the building. Upgrading the glass on the south side, however, will not be enough to completely balance these costs but it will bring them within a more acceptable difference. Another possible solution would be to use a combination of the upgraded glass while also reducing the amount of glass on the south side of the building. Using the upgraded glass, only 10% of the total exterior glazing would need to be removed in order to achieve a complete balance between the north and south units. This could be accomplished by using 9' high windows instead of 10' high floor to ceiling windows. However, this may still be too drastic a change in the building's architecture and the occupants themselves may not want to sacrifice their wonderful views to save a little money on their electric bills. This is why I recommend just using the higher grade glass by itself.

(See Appendix for Monthly Simulation Results)



Picture courtesy of Trumpgrande.com

Above is the unit that was analyzed for solar impact. You can see from the key plan that these units are symmetrical and on opposite ends of the building (the shaded areas). The shaded area to the right on the key plan is the north unit, while the shaded unit to the left is the south.

Appendix A

N/S Wind Pressure Calcs.											
(ft)	kz	qz	qh	Gww	Cpww	Pww	Glw	Cplw	Plw	Total P	Design P
						(psf)			(psf)	(psf)	(psf)
0-15	1.030	57.989	107.900	0.869	0.800	40.314	0.869	0.500	46.883	87.197	80
20	1.080	60.804	107.900	0.869	0.800	42.271	0.869	0.500	46.883	89.153	80
25	1.120	63.056	107.900	0.869	0.800	43.837	0.869	0.500	46.883	90.719	80
30	1.160	65.308	107.900	0.869	0.800	45.402	0.869	0.500	46.883	92.285	80
40	1.220	68.686	107.900	0.869	0.800	47.751	0.869	0.500	46.883	94.633	80
50	1.270	71.501	107.900	0.869	0.800	49.707	0.869	0.500	46.883	96.590	90
60	1.310	73.753	107.900	0.869	0.800	51.273	0.869	0.500	46.883	98.156	90
70	1.340	75.442	107.900	0.869	0.800	52.447	0.869	0.500	46.883	99.330	90
80	1.380	77.694	107.900	0.869	0.800	54.013	0.869	0.500	46.883	100.895	90
90	1.400	78.820	107.900	0.869	0.800	54.796	0.869	0.500	46.883	101.678	90
100	1.430	80.509	107.900	0.869	0.800	55.970	0.869	0.500	46.883	102.852	90
120	1.480	83.324	107.900	0.869	0.800	57.927	0.869	0.500	46.883	104.809	90
140	1.520	85.576	107.900	0.869	0.800	59.492	0.869	0.500	46.883	106.375	100
160	1.550	87.265	107.900	0.869	0.800	60.667	0.869	0.500	46.883	107.549	100
180	1.580	88.954	107.900	0.869	0.800	61.841	0.869	0.500	46.883	108.723	100
200	1.610	90.643	107.900	0.869	0.800	63.015	0.869	0.500	46.883	109.898	100
250	1.680	94.584	107.900	0.869	0.800	65.755	0.869	0.500	46.883	112.637	100
300	1.730	97.399	107.900	0.869	0.800	67.712	0.869	0.500	46.883	114.594	100
350	1.780	100.214	107.900	0.869	0.800	69.669	0.869	0.500	46.883	116.551	110
400	1.820	102.466	107.900	0.869	0.800	71.234	0.869	0.500	46.883	118.117	110
450	1.860	104.718	107.900	0.869	0.800	72.800	0.869	0.500	46.883	119.683	110
500	1.890	106.407	107.900	0.869	0.800	73.974	0.869	0.500	46.883	120.857	110
550	1.920	108.096	107.900	0.869	0.800	75.148	0.869	0.500	46.883	122.031	110

E/W Wind Pressure Calcs.

(ft)	kz	qz	qh	Gww	Cpww	Pww (psf)	Glw	Cplw	Plw (psf)	Total P (psf)	Design P (psf)
0-15	1.030	57.989	107.900	0.921	0.800	42.726	0.921	0.300	29.813	72.539	70
20	1.080	60.804	107.900	0.921	0.800	44.800	0.921	0.300	29.813	74.613	70
25	1.120	63.056	107.900	0.921	0.800	46.460	0.921	0.300	29.813	76.272	70
30	1.160	65.308	107.900	0.921	0.800	48.119	0.921	0.300	29.813	77.932	70
40	1.220	68.686	107.900	0.921	0.800	50.608	0.921	0.300	29.813	80.421	70
50	1.270	71.501	107.900	0.921	0.800	52.682	0.921	0.300	29.813	82.495	70
60	1.310	73.753	107.900	0.921	0.800	54.341	0.921	0.300	29.813	84.154	70
70	1.340	75.442	107.900	0.921	0.800	55.586	0.921	0.300	29.813	85.398	80
80	1.380	77.694	107.900	0.921	0.800	57.245	0.921	0.300	29.813	87.058	80
90	1.400	78.820	107.900	0.921	0.800	58.075	0.921	0.300	29.813	87.887	80
100	1.430	80.509	107.900	0.921	0.800	59.319	0.921	0.300	29.813	89.132	80
120	1.480	83.324	107.900	0.921	0.800	61.393	0.921	0.300	29.813	91.206	80
140	1.520	85.576	107.900	0.921	0.800	63.052	0.921	0.300	29.813	92.865	80
160	1.550	87.265	107.900	0.921	0.800	64.297	0.921	0.300	29.813	94.110	80
180	1.580	88.954	107.900	0.921	0.800	65.541	0.921	0.300	29.813	95.354	90
200	1.610	90.643	107.900	0.921	0.800	66.786	0.921	0.300	29.813	96.599	90
250	1.680	94.584	107.900	0.921	0.800	69.689	0.921	0.300	29.813	99.502	90
300	1.730	97.399	107.900	0.921	0.800	71.764	0.921	0.300	29.813	101.576	90
350	1.780	100.214	107.900	0.921	0.800	73.838	0.921	0.300	29.813	103.650	90
400	1.820	102.466	107.900	0.921	0.800	75.497	0.921	0.300	29.813	105.310	100
450	1.860	104.718	107.900	0.921	0.800	77.156	0.921	0.300	29.813	106.969	100
500	1.890	106.407	107.900	0.921	0.800	78.401	0.921	0.300	29.813	108.213	100
550	1.920	108.096	107.900	0.921	0.800	79.645	0.921	0.300	29.813	109.458	100

North/South Windloading

Level	Story Height (ft)	Height Above Curb (ft)	Tributary Height (ft)	Tributary Width (ft)	Wind Pressure (psf)	Force (kips)	Shear Sum (kips)	Overturning Moment (kip-ft)
54	5	545.00	2.5	250	110	68.75	68.75	0
53	10	540.00	7.5	250	110	206.25	275	343.75
52	10	530.00	10	250	110	275	550	3093.75
51	10	520.00	10	250	110	275	825	8593.75
50	4	510.00	7	250	110	192.5	1017.5	16843.75
49	10	506.00	7	250	110	192.5	1210	20913.75
48	10	496.00	10	250	110	275	1485	33013.75
47	8	486.00	9	250	110	247.5	1732.5	47863.75
46	13.67	478.00	10.835	250	110	297.9625	2030.4625	61723.75
45	5.33	464.33	9.5	250	110	261.25	2291.7125	89480.17238
44	11.67	459.00	8.5	250	110	233.75	2525.4625	101695
43	11.66	447.33	11.665	250	110	320.7875	2846.25	131167.1474
42	11.67	435.67	11.665	250	110	320.7875	3167.0375	164354.4224
41	11.67	424.00	11.67	250	110	320.925	3487.9625	201313.75
40	11.66	412.33	11.665	250	110	320.7875	3808.75	242018.2724
39	10.67	400.67	11.165	250	110	307.0375	4115.7875	286428.2974
38	10.67	390.00	10.67	250	110	293.425	4409.2125	330343.75
37	10.66	379.33	10.665	250	110	293.2875	4702.5	377390.0474
36	10.67	368.67	10.665	250	110	293.2875	4995.7875	427518.6974
35	10.67	358.00	10.67	250	110	293.425	5289.2125	480823.75
34	10.66	347.33	10.665	250	100	266.625	5555.8375	537259.6474
33	10.67	336.67	10.665	250	100	266.625	5822.4625	596484.8751
32	10.67	326.00	10.67	250	100	266.75	6089.2125	658610.55
31	10.66	315.33	10.665	250	100	266.625	6355.8375	723582.4474
30	10.67	304.67	10.665	250	100	266.625	6622.4625	791335.6751
29	10.67	294.00	10.67	250	100	266.75	6889.2125	861997.35
28	10.66	283.33	10.665	250	100	266.625	7155.8375	935505.2474
27	10.67	272.67	10.665	250	100	266.625	7422.4625	1011786.475
26	10.67	262.00	10.67	250	100	266.75	7689.2125	1090984.15
25	10.66	251.33	10.665	250	100	266.625	7955.8375	1173028.047
24	10.67	240.67	10.665	250	100	266.625	8222.4625	1257837.275
23	10.67	230.00	10.67	250	100	266.75	8489.2125	1345570.95
22	10.66	219.33	10.665	250	100	266.625	8755.8375	1436150.847
21	11	208.67	10.83	250	100	270.75	9026.5875	1529488.075
20	11	197.67	11	250	100	275	9301.5875	1628780.538
19	11	186.67	11	250	100	275	9576.5875	1731098
18	10.67	175.67	10.835	250	100	270.875	9847.4625	1836440.463
17	10.67	165.00	10.67	250	100	266.75	10114.2125	1941512.888
16	10.66	154.33	10.665	250	100	266.625	10380.8375	2049431.535
15	10.67	143.67	10.665	250	100	266.625	10647.4625	2160091.263
14	10.67	133.00	10.67	250	90	240.075	10887.5375	2273699.688
13	10.66	122.33	10.665	250	90	239.9625	11127.5	2389869.713
12	10.67	111.67	10.665	250	90	239.9625	11367.4625	2508488.863
11	10.67	101.00	10.67	250	90	240.075	11607.5375	2629779.688
10	10.66	90.33	10.665	250	90	239.9625	11847.5	2753632.113
9	10.67	79.67	10.665	250	90	239.9625	12087.4625	2879926.463
8	10.67	69.00	10.67	250	90	240.075	12327.5375	3008899.688
7	10.66	58.33	10.665	250	90	239.9625	12567.5	3140434.513
6	10.67	47.67	10.665	250	80	213.3	12780.8	3274404.063
5	10.67	37.00	10.67	250	80	213.4	12994.2	3410775.199
4	16.04	26.33	13.355	250	80	267.1	13261.3	3549423.313
3	10.29	10.29	13.165	250	80	263.3	13524.6	3762134.565

East/West Windloading

Level	Story Height (ft)	Height Above Curb (ft)	Tributary Height (ft)	Tributary Width (ft)	Wind Pressure (psf)	Force (kips)	Shear Sum (kips)	Overturning Moment (kip-ft)
54	5	545.00	2.5	125	100	31.25	31.25	0
53	10	540.00	7.5	125	100	93.75	125	156.25
52	10	530.00	10	125	100	125	250	1406.25
51	10	520.00	10	125	100	125	375	3906.25
50	4	510.00	7	125	100	87.5	462.5	7656.25
49	10	506.00	7	125	100	87.5	550	9506.25
48	10	496.00	10	125	100	125	675	15006.25
47	8	486.00	9	125	100	112.5	787.5	21756.25
46	13.67	478.00	10.835	125	100	135.4375	922.9375	28056.25
45	5.33	464.33	9.5	125	100	118.75	1041.6875	40672.80563
44	11.67	459.00	8.5	125	100	106.25	1147.9375	46225
43	11.66	447.33	11.665	125	100	145.8125	1293.75	59621.43063
42	11.67	435.67	11.665	125	100	145.8125	1439.5625	74706.55563
41	11.67	424.00	11.67	125	100	145.875	1585.4375	91506.25
40	11.66	412.33	11.665	125	100	145.8125	1731.25	110008.3056
39	10.67	400.67	11.165	125	100	139.5625	1870.8125	130194.6806
38	10.67	390.00	10.67	125	90	120.0375	1990.85	150156.25
37	10.66	379.33	10.665	125	90	119.98125	2110.83125	171398.6195
36	10.67	368.67	10.665	125	90	119.98125	2230.8125	193900.0806
35	10.67	358.00	10.67	125	90	120.0375	2350.85	217702.85
34	10.66	347.33	10.665	125	90	119.98125	2470.83125	242786.4195
33	10.67	336.67	10.665	125	90	119.98125	2590.8125	269125.4806
32	10.67	326.00	10.67	125	90	120.0375	2710.85	296769.45
31	10.66	315.33	10.665	125	90	119.98125	2830.83125	325694.2195
30	10.67	304.67	10.665	125	90	119.98125	2950.8125	355870.8806
29	10.67	294.00	10.67	125	90	120.0375	3070.85	387356.05
28	10.66	283.33	10.665	125	90	119.98125	3190.83125	420122.0195
27	10.67	272.67	10.665	125	90	119.98125	3310.8125	454136.2806
26	10.67	262.00	10.67	125	90	120.0375	3430.85	489462.65
25	10.66	251.33	10.665	125	90	119.98125	3550.83125	526069.8195
24	10.67	240.67	10.665	125	90	119.98125	3670.8125	563921.6806
23	10.67	230.00	10.67	125	90	120.0375	3790.85	603089.25
22	10.66	219.33	10.665	125	90	119.98125	3910.83125	643537.6195
21	11	208.67	10.83	125	90	121.8375	4032.66875	685227.0806
20	11	197.67	11	125	90	123.75	4156.41875	729586.4369
19	11	186.67	11	125	90	123.75	4280.16875	775307.0431
18	10.67	175.67	10.835	125	80	108.35	4388.51875	822388.8994
17	10.67	165.00	10.67	125	80	106.7	4495.21875	869214.3944
16	10.66	154.33	10.665	125	80	106.65	4601.86875	917178.3785
15	10.67	143.67	10.665	125	80	106.65	4708.51875	966234.2994
14	10.67	133.00	10.67	125	80	106.7	4815.21875	1016474.194
13	10.66	122.33	10.665	125	80	106.65	4921.86875	1067852.579
12	10.67	111.67	10.665	125	80	106.65	5028.51875	1120319.699
11	10.67	101.00	10.67	125	80	106.7	5135.21875	1173973.994
10	10.66	90.33	10.665	125	80	106.65	5241.86875	1228766.779
9	10.67	79.67	10.665	125	80	106.65	5348.51875	1284645.099
8	10.67	69.00	10.67	125	70	93.3625	5441.88125	1341713.794
7	10.66	58.33	10.665	125	70	93.31875	5535.2	1399778.667
6	10.67	47.67	10.665	125	70	93.31875	5628.51875	1458783.899
5	10.67	37.00	10.67	125	70	93.3625	5721.88125	1518840.194
4	16.04	26.33	13.355	125	70	116.85625	5838.7375	1579892.667
3	10.29	10.29	13.165	125	70	115.19375	5953.93125	1673546.017

Direct Stiffness by Floor for Each Shear Wall

DIRECT STIFFNESS BY FLOOR FOR EACH SHEARWALL												
From Levels 1 to 18												
Wall	Direction	Width	Length	Height	H/L	f'c (PSI)	Ec (PSI)	I	EI	Δ/P	k	k/Sumki
A	N/S	3	33	185	5.606061	10000	4200000	8984.25	3.773E+10	5.72E-05	17491.93	0.042783
B	N/S	3.5	24.5	185	7.55102	10000	4200000	4289.286	1.802E+10	0.000119	8432.935	0.020626
C	N/S	2	12.5	185	14.8	10000	4200000	325.5208	1.367E+09	0.001549	645.741	0.001579
D	N/S	3.5	69	185	2.681159	10000	4200000	95815.13	4.024E+11	5.75E-06	173863.8	0.425247
E	N/S	4	23	185	8.043478	10000	4200000	4055.667	1.703E+10	0.000125	7985.041	0.01953
F	N/S	3.5	69	185	2.681159	10000	4200000	95815.13	4.024E+11	5.75E-06	173863.8	0.425247
G	N/S	2	12.5	185	14.8	10000	4200000	325.5208	1.367E+09	0.001549	645.741	0.001579
H	N/S	3	33	185	5.606061	10000	4200000	8984.25	3.773E+10	5.72E-05	17491.93	0.042783
J	N/S	3.5	24.5	185	7.55102	10000	4200000	4289.286	1.802E+10	0.000119	8432.935	0.020626
SUMS								222884			408853.9	1
A	E/W	3	36	185	5.138889	10000	4200000	11664	4.899E+10	4.42E-05	22616.28	0.394057
B1	E/W	3.5	12.5	185	14.8	10000	4200000	569.6615	2.393E+09	0.000885	1130.047	0.019689
B2	E/W	3.5	15.5	185	11.93548	10000	4200000	1086.13	4.562E+09	0.000465	2150.917	0.037477
D	E/W	3	10	185	18.5	10000	4200000	250	1.05E+09	0.002014	496.4944	0.008651
E	E/W	3.5	20	185	9.25	10000	4200000	2333.333	9.8E+09	0.000217	4605.945	0.080252
F	E/W	3	10	185	18.5	10000	4200000	250	1.05E+09	0.002014	496.4944	0.008651
H	E/W	3	36	185	5.138889	10000	4200000	11664	4.899E+10	4.42E-05	22616.28	0.394057
J3	E/W	3.5	12.5	185	14.8	10000	4200000	569.6615	2.393E+09	0.000885	1130.047	0.019689
J4	E/W	3.5	15.5	185	11.93548	10000	4200000	1086.13	4.562E+09	0.000465	2150.917	0.037477
SUMS								29472.92			57393.41	1
FROM LEVELS 18 TO 28												
Wall	Direction	Width	Length	Height	H/L	f'c (PSI)	Ec (PSI)	I	EI	Δ/P	k	k/Sumki
A	N/S	3	33	98	2.969697	8000	4200000	8984.25	3.773E+10	8.97E-06	111488.7	0.048432

Direct Stiffness by Floor for Each Shear Wall (Continued)

B	N/S	3.5	24.5	98	4	8000	4200000	4289.286	1.802E+10	1.82E-05	55031.45	0.023906
C	N/S	2	12.5	98	7.84	8000	4200000	325.5208	1.367E+09	0.000232	4309.115	0.001872
D	N/S	3.5	69	98	1.42029	8000	4200000	95815.13	4.024E+11	1.05E-06	954014.4	0.414432
E	N/S	4	23	98	4.26087	8000	4200000	4055.667	1.703E+10	1.91E-05	52292.51	0.022716
F	N/S	3.5	69	98	1.42029	8000	4200000	95815.13	4.024E+11	1.05E-06	954014.4	0.414432
G	N/S	2	12.5	98	7.84	8000	4200000	325.5208	1.367E+09	0.000232	4309.115	0.001872
H	N/S	3	33	98	2.969697	8000	4200000	8984.25	3.773E+10	8.97E-06	111488.7	0.048432
J	N/S	3.5	24.5	98	4	8000	4200000	4289.286	1.802E+10	1.82E-05	55031.45	0.023906
SUMS								222884			2301980	1
A	E/W	3	36	98	2.722222	8000	4200000	11664	4.899E+10	7E-06	142760.3	0.389853
B1	E/W	3.5	12.5	98	7.84	8000	4200000	569.6615	2.393E+09	0.000133	7540.95	0.020593
B2	E/W	3.5	15.5	98	6.322581	8000	4200000	1086.13	4.562E+09	7E-05	14291.85	0.039029
D	E/W	3	10	98	9.8	8000	4200000	250	1.05E+09	0.000301	3322.774	0.009074
E	E/W	3.5	20	98	4.9	8000	4200000	2333.333	9.8E+09	3.29E-05	30358.23	0.082903
F	E/W	3	10	98	9.8	8000	4200000	250	1.05E+09	0.000301	3322.774	0.009074
H	E/W	3	36	98	2.722222	8000	4200000	11664	4.899E+10	7E-06	142760.3	0.389853
J3	E/W	3.5	12.5	98	7.84	8000	4200000	569.6615	2.393E+09	0.000133	7540.95	0.020593
J4	E/W	3.5	15.5	98	6.322581	8000	4200000	1086.13	4.562E+09	7E-05	14291.85	0.039029
SUMS								29472.92			366190	1
FROM LEVELS 28 TO 40												
Wall	Direction	Width	Length	Height	H/L	f'c (PSI)	Ec (PSI)	I	EI	Δ/P	k	k/Sumki
A	N/S	3	33	128	3.878788	6000	3700000	8984.25	3.324E+10	2.2E-05	45452.98	0.045296
B	N/S	3.5	24.5	128	5.22449	6000	3700000	4289.286	1.587E+10	4.52E-05	22139.02	0.022063
C	N/S	2	12.5	128	10.24	6000	3700000	325.5208	1.204E+09	0.000584	1711.602	0.001706
D	N/S	3.5	69	128	1.855072	6000	3700000	95815.13	3.545E+11	2.37E-06	421927.2	0.420474
E	N/S	4	23	128	5.565217	6000	3700000	4055.667	1.501E+10	4.76E-05	20995.08	0.020923
F	N/S	3.5	69	128	1.855072	6000	3700000	95815.13	3.545E+11	2.37E-06	421927.2	0.420474
G	N/S	2	12.5	128	10.24	6000	3700000	325.5208	1.204E+09	0.000584	1711.602	0.001706
H	N/S	3	33	128	3.878788	6000	3700000	8984.25	3.324E+10	2.2E-05	45452.98	0.045296

Direct Stiffness by Floor for Each Shear Wall (Continued)

J	N/S	3.5	24.5	128	5.22449	6000	3700000	4289.286	1.587E+10	4.52E-05	22139.02	0.022063
SUMS								222884			1003457	1
A	E/W	3	36	128	3.555556	6000	3700000	11664	4.316E+10	1.71E-05	58519.17	0.392246
B1	E/W	3.5	12.5	128	10.24	6000	3700000	569.6615	2.108E+09	0.000334	2995.304	0.020077
B2	E/W	3.5	15.5	128	8.258065	6000	3700000	1086.13	4.019E+09	0.000176	5690.774	0.038145
D	E/W	3	10	128	12.8	6000	3700000	250	925000000	0.000759	1317.634	0.008832
E	E/W	3.5	20	128	6.4	6000	3700000	2333.333	8.633E+09	8.23E-05	12144.03	0.0814
F	E/W	3	10	128	12.8	6000	3700000	250	925000000	0.000759	1317.634	0.008832
H	E/W	3	36	128	3.555556	6000	3700000	11664	4.316E+10	1.71E-05	58519.17	0.392246
J3	E/W	3.5	12.5	128	10.24	6000	3700000	569.6615	2.108E+09	0.000334	2995.304	0.020077
J4	E/W	3.5	15.5	128	8.258065	6000	3700000	1086.13	4.019E+09	0.000176	5690.774	0.038145
SUMS								29472.92			149189.8	1
FROM LEVELS 40 TO 49												
Wall	Direction	Width	Length	Height	H/L	f'c (PSI)	Ec (PSI)	I	EI	Δ/P	k	k/Sumki
A	N/S	3	33	52	1.575758	5000	3500000	8984.25	3.144E+10	1.91E-06	524184.7	0.06857
B	N/S	3.5	24.5	94	3.836735	5000	3500000	4289.286	1.501E+10	1.93E-05	51779.25	0.006773
C	N/S	2	12.5	94	7.52	5000	3500000	325.5208	1.139E+09	0.000246	4065.178	0.000532
D	N/S	3.5	69	52	0.753623	5000	3500000	95815.13	3.354E+11	3.11E-07	3217632	0.420906
E	N/S	4	23	94	4.086957	5000	3500000	4055.667	1.419E+10	2.03E-05	49222.47	0.006439
F	N/S	3.5	69	52	0.753623	5000	3500000	95815.13	3.354E+11	3.11E-07	3217632	0.420906
G	N/S	2	12.5	94	7.52	5000	3500000	325.5208	1.139E+09	0.000246	4065.178	0.000532
H	N/S	3	33	52	1.575758	5000	3500000	8984.25	3.144E+10	1.91E-06	524184.7	0.06857
J	N/S	3.5	24.5	94	3.836735	5000	3500000	4289.286	1.501E+10	1.93E-05	51779.25	0.006773
SUMS								222884			7644545	1
A	E/W	3	36	52	1.444444	5000	3500000	11664	4.082E+10	1.53E-06	653374.1	0.462426
B1	E/W	3.5	12.5	94	7.52	5000	3500000	569.6615	1.994E+09	0.000141	7114.062	0.005035
B2	E/W	3.5	15.5	94	6.064516	5000	3500000	1086.13	3.801E+09	7.42E-05	13475.89	0.009538

Direct Stiffness by Floor for Each Shear Wall (Continued)

D	E/W	3	10	52	5.2	5000	3500000	250	875000000	5.49E-05	18201.11	0.012882
E	E/W	3.5	20	94	4.7	5000	3500000	2333.333	8.167E+09	3.5E-05	28597.57	0.02024
F	E/W	3	10	52	5.2	5000	3500000	250	875000000	5.49E-05	18201.11	0.012882
H	E/W	3	36	52	1.444444	5000	3500000	11664	4.082E+10	1.53E-06	653374.1	0.462426
J3	E/W	3.5	12.5	94	7.52	5000	3500000	569.6615	1.994E+09	0.000141	7114.062	0.005035
J4	E/W	3.5	15.5	94	6.064516	5000	3500000	1086.13	3.801E+09	7.42E-05	13475.89	0.009538
SUMS								29472.92			1412928	1

Center of Rigidity

CENTER OF RIGITY													
SUPPORTING LEVELS 2-18													
WALL	DIRECTION	WIDTH	LENGTH	AREA	X	ki	ki/Sumki	Kili/Sumki	di	kidi	kidi^2	e=0	
A	N/S	3	33	99	112.5	17491.93	0.042783	4.8130698	0	0	0		
B	N/S	3.5	24.5	85.75	112.5	8432.935	0.020626	2.3204012	0	0	0		
C	N/S	2	12.5	25	112.5	645.741	0.001579	0.1776817	0	0	0		
D	N/S	3.5	69	241.5	112.5	173863.8	0.425247	47.840268	0	0	0		
E	N/S	4	23	92	112.5	7985.041	0.01953	2.1971592	0	0	0		
F	N/S	3.5	69	241.5	112.5	173863.8	0.425247	47.840268	0	0	0		
G	N/S	2	12.5	25	112.5	645.741	0.001579	0.1776817	0	0	0		
H	N/S	3	33	99	112.5	17491.93	0.042783	4.8130698	0	0	0		
J	N/S	3.5	24.5	85.75	112.5	8432.935	0.020626	2.3204012	0	0	0		
							dx	112.5			0		
					Y								
A	E/W	3	36	108	45.5	22616.28	0.394057	17.929593	8.486785	191939.5	1628949.1	e=8	
B1	E/W	3.5	12.5	43.75	22.75	1130.047	0.019689	0.4479358	-14.26321	-16118.1	229895.9		
B2	E/W	3.5	15.5	54.25	1.75	2150.917	0.037477	0.0655843	-35.26321	-75848.24	2674652.8		
D	E/W	3	10	30	1.5	496.4944	0.008651	0.0129761	-35.51321	-17632.11	626172.98		
E	E/W	3.5	20	70	1.75	4605.945	0.080252	0.1404413	-35.26321	-162420.4	5727466.3		
F	E/W	3	10	30	1.5	496.4944	0.008651	0.0129761	-35.51321	-17632.11	626172.98		
H	E/W	3	36	108	45.4	22616.28	0.394057	17.890188	8.386785	189677.9	1590787.4		
J3	E/W	3.5	12.5	43.75	22.75	1130.047	0.019689	0.4479358	-14.26321	-16118.1	229895.9		
J4	E/W	3.5	15.5	54.25	1.75	2150.917	0.037477	0.0655843	-35.26321	-75848.24	2674652.8		
							dy	37.013215			16008646		
SUPPORTING LEVELS 19-28													
WALL	DIRECTION	WIDTH	LENGTH	AREA	X	ki	ki/Sumki	Kili/Sumki	di	kidi	kidi^2	e=0	
A	N/S	3	33	99	112.5	111488.7	0.048432	5.4485599	0	0	0		
B	N/S	3.5	24.5	85.75	112.5	55031.45	0.023906	2.6894406	0	0	0		
C	N/S	2	12.5	25	112.5	4309.115	0.001872	0.2105906	0	0	0		
D	N/S	3.5	69	241.5	112.5	954014.4	0.414432	46.623616	0	0	0		
E	N/S	4	23	92	112.5	52292.51	0.022716	2.5555861	0	0	0		

Center of Rigidity (Continued)

F	N/S	3.5	69	241.5	112.5	954014.4	0.414432	46.623616	0	0	0		
G	N/S	2	12.5	25	112.5	4309.115	0.001872	0.2105906	0	0	0		
H	N/S	3	33	99	112.5	111488.7	0.048432	5.4485599	0	0	0		
J	N/S	3.5	24.5	85.75	112.5	55031.45	0.023906	2.6894406	0	0	0		
							dx	112.5			0		
					Y								
A	E/W	3	36	108	45.5	142760.3	0.389853	17.738317	8.816469	1258642	11096777	e=8.3	
B1	E/W	3.5	12.5	43.75	22.75	7540.95	0.020593	0.4684907	-13.93353	-105072.1	1464024.9		
B2	E/W	3.5	15.5	54.25	1.75	14291.85	0.039029	0.0682999	-34.93353	-499264.9	17441084		
D	E/W	3	10	30	1.5	3322.774	0.009074	0.0136109	-35.18353	-116906.9	4113198.7		
E	E/W	3.5	20	70	1.75	30358.23	0.082903	0.1450801	-34.93353	-1060520	37047710		
F	E/W	3	10	30	1.5	3322.774	0.009074	0.0136109	-35.18353	-116906.9	4113198.7		
H	E/W	3	36	108	45.4	142760.3	0.389853	17.699331	8.716469	1244366	10846477		
J3	E/W	3.5	12.5	43.75	22.75	7540.95	0.020593	0.4684907	-13.93353	-105072.1	1464024.9		
J4	E/W	3.5	15.5	54.25	1.75	14291.85	0.039029	0.0682999	-34.93353	-499264.9	17441084		
							dy	36.683531			105027581		
SUPPORTING LEVELS 29-40													
WALL	DIRECTION	WIDTH	LENGTH	AREA	X	ki	ki/Sumki	Kili/Sumki	di	kidi	kidi^2	e=0	
A	N/S	3	33	99	112.5	45452.98	0.045296	5.0958455	0	0	0		
B	N/S	3.5	24.5	85.75	112.5	22139.02	0.022063	2.4820603	0	0	0		
C	N/S	2	12.5	25	112.5	1711.602	0.001706	0.1918919	0	0	0		
D	N/S	3.5	69	241.5	112.5	421927.2	0.420474	47.303297	0	0	0		
E	N/S	4	23	92	112.5	20995.08	0.020923	2.3538102	0	0	0		
F	N/S	3.5	69	241.5	112.5	421927.2	0.420474	47.303297	0	0	0		
G	N/S	2	12.5	25	112.5	1711.602	0.001706	0.1918919	0	0	0		
H	N/S	3	33	99	112.5	45452.98	0.045296	5.0958455	0	0	0		
J	N/S	3.5	24.5	85.75	112.5	22139.02	0.022063	2.4820603	0	0	0		
							dx	112.5			0		
					Y								
A	E/W	3	36	108	45.5	58519.17	0.392246	17.847215	8.628833	504952.2	4357148.3	e=8.1	

ae

Center of Rigidity (Continued)

B1	E/W	3.5	12.5	43.75	22.75	2995.304	0.020077	0.4567549	-14.12117	-42297.18	597285.59		
B2	E/W	3.5	15.5	54.25	1.75	5690.774	0.038145	0.0667529	-35.12117	-199866.6	7019549.1		
D	E/W	3	10	30	1.5	1317.634	0.008832	0.0132479	-35.37117	-46606.24	1648517.2		
E	E/W	3.5	20	70	1.75	12144.03	0.0814	0.1424497	-35.12117	-426512.3	14979611		
F	E/W	3	10	30	1.5	1317.634	0.008832	0.0132479	-35.37117	-46606.24	1648517.2		
H	E/W	3	36	108	45.4	58519.17	0.392246	17.80799	8.528833	499100.3	4256743		
J3	E/W	3.5	12.5	43.75	22.75	2995.304	0.020077	0.4567549	-14.12117	-42297.18	597285.59		
J4	E/W	3.5	15.5	54.25	1.75	5690.774	0.038145	0.0667529	-35.12117	-199866.6	7019549.1		
							dy	36.871167			42124206		
SUPPORTING LEVELS 41-45/49													
WALL	DIRECTION	WIDTH	LENGTH	AREA	X	ki	ki/Sumki	Kili/Sumki	di	kidi	kidi^2	e=0	
A	N/S	3	33	99	112.5	524184.7	0.06857	7.7140996	0	0	0		
B	N/S	3.5	24.5	85.75	112.5	51779.25	0.006773	0.7620029	0	0	0		
C	N/S	2	12.5	25	112.5	4065.178	0.000532	0.0598247	0	0	0		
D	N/S	3.5	69	241.5	112.5	3217632	0.420906	47.351885	0	0	0		
E	N/S	4	23	92	112.5	49222.47	0.006439	0.7243763	0	0	0		
F	N/S	3.5	69	241.5	112.5	3217632	0.420906	47.351885	0	0	0		
G	N/S	2	12.5	25	112.5	4065.178	0.000532	0.0598247	0	0	0		
H	N/S	3	33	99	112.5	524184.7	0.06857	7.7140996	0	0	0		
J	N/S	3.5	24.5	85.75	112.5	51779.25	0.006773	0.7620029	0	0	0		
							dx	112.5			0		
					Y								
A	E/W	3	36	108	45.5	653374.1	0.462426	21.040367	3.12897	2044388	6396827.4	e=2.6	
B1	E/W	3.5	12.5	43.75	22.75	7114.062	0.005035	0.1145458	-19.62103	-139585.2	2738806.1		
B2	E/W	3.5	15.5	54.25	1.75	13475.89	0.009538	0.0166907	-40.62103	-547404.4	22236131		
D	E/W	3	10	30	1.5	18201.11	0.012882	0.0193228	-40.87103	-743898	30403877		
E	E/W	3.5	20	70	1.75	28597.57	0.02024	0.0354199	-40.62103	-1161663	47187946		
F	E/W	3	10	30	1.5	18201.11	0.012882	0.0193228	-40.87103	-743898	30403877		
H	E/W	3	36	108	45.4	653374.1	0.462426	20.994125	3.02897	1979050	5994483.6		
J3	E/W	3.5	12.5	43.75	22.75	7114.062	0.005035	0.1145458	-19.62103	-139585.2	2738806.1		
J4	E/W	3.5	15.5	54.25	1.75	13475.89	0.009538	0.0166907	-40.62103	-547404.4	22236131		
							dy	42.37103			170336885		

Distribution of Forces (N/S)

Distribution of Loads(N/S)						
Floor	Wall	P	Ki/SumKi	FiDirect		F Total (Kips)**
2 to 6	Total	192060				134.442
	A		0.042783	8216.873		5.751810916
	B		0.020626	3961.389		2.77297225
	C		0.001579	303.3382		0.212336734
	D		0.425247	81672.91		57.17103354
	E		0.01953	3750.99		2.625693116
	F		0.425247	81672.91		57.17103354
	G		0.001579	303.3382		0.212336734
	H		0.042783	8216.873		5.751810916
	J		0.020626	3961.389		2.77297225
7 to 14	Total	216068				151.2476
	A		0.042783	9244.003		6.470802255
	B		0.020626	4456.573		3.119601001
	C		0.001579	341.2563		0.238879378
	D		0.425247	91882.23		64.31756157
	E		0.01953	4219.874		2.953911591
	F		0.425247	91882.23		64.31756157
	G		0.001579	341.2563		0.238879378
	H		0.042783	9244.003		6.470802255
	J		0.020626	4456.573		3.119601001
15 to 18	Total	240075				168.0525
	A		0.042783	10271.09		7.189763645
	B		0.020626	4951.736		3.466215313
	C		0.001579	379.1727		0.265420917
	D		0.425247	102091.1		71.46379193
	E		0.01953	4688.738		3.282116395
	F		0.425247	102091.1		71.46379193
	G		0.001579	379.1727		0.265420917

Distribution of Forces (N/S) (Continued)

	H		0.042783	10271.09		7.189763645			
	J		0.020626	4951.736		3.466215313			
19 to 28	Total	240075				168.0525			
	A		0.048432	11627.23		8.139058834			
	B		0.023906	5739.266		4.017486394			
	C		0.001872	449.4004		0.314580298			
	D		0.414432	99494.8		69.64635725			
	E		0.022716	5453.621		3.817534445			
	F		0.414432	99494.8		69.64635725			
	G		0.001872	449.4004		0.314580298			
	H		0.048432	11627.23		8.139058834			
	J		0.023906	5739.266		4.017486394			
29 to 34	Total	240075				168.0525			
	A		0.045296	10874.53		7.612174049			
	B		0.022063	5296.717		3.70770174			
	C		0.001706	409.4974		0.286648181			
	D		0.420474	100945.2		70.66166519			
	E		0.020923	5023.031		3.516121677			
	F		0.420474	100945.2		70.66166519			
	G		0.001706	409.4974		0.286648181			
	H		0.045296	10874.53		7.612174049			
	J		0.022063	5296.717		3.70770174			
35-40	Total	264083				184.8581			
	A		0.045296	11962.01		8.373407308			
	B		0.022063	5826.399		4.078479636			
	C		0.001706	450.448		0.315313596			
	D		0.420474	111040		77.72797888			
	E		0.020923	5525.345		3.867741168			
	F		0.420474	111040		77.72797888			
	G		0.001706	450.448		0.315313596			
	H		0.045296	11962.01		8.373407308			
	J		0.022063	5826.399		4.078479636			
41-49	Total	264083				184.8581			

Distribution of Forces (N/S) (Continued)

	A		0.06857	18108.11		12.6756782			
	B		0.006773	1788.729		1.252110349			
	C		0.000532	140.4327		0.098302925			
	D		0.420906	111154		77.80781708			
	E		0.006439	1700.404		1.190282897			
	F		0.420906	111154		77.80781708			
	G		0.000532	140.4327		0.098302925			
	H		0.06857	18108.11		12.6756782			
	J		0.006773	1788.729		1.252110349			
**(.7load factor adjustment)									

Distribution of Forces (E/W)

Distribuiton of Loads (E/W)									
Floor	Wall	P	Ki/SumKi	FiDirect	M	Kidi/Sumkidi^2	Ftorsional	Ftotal	F total (kips)**
2 to 8	Total	67221			537768				33.6105
	A		0.394057	26488.91		0.011989738	6447.69755	32936.6	16.46830154
	B1		0.019689	1323.547		-0.001006837	-541.444763	782.1021	0.391051033
	B2		0.037477	2519.223		-0.004737955	-2547.92041	-28.69789	-0.014348945
	D		0.008651	581.5101		-0.001101412	-592.304027	-10.79391	-0.005396957
	E		0.080252	5394.63		-0.010145794	-5456.08332	-61.45329	-0.030726643
	F		0.008651	581.5101		-0.001101412	-592.304027	-10.79391	-0.005396957
	H		0.394057	26488.91		0.011848463	6371.72417	32860.63	16.43031485
	J3		0.019689	1323.547		-0.001006837	-541.444763	782.1021	0.391051033
	J4		0.037477	2519.223		-0.004737955	-2547.92041	-28.69789	-0.014348945
9 to 18	Total	76824			614592				38.412
	A		0.394057	30273.03		0.011989738	7368.7972	37641.83	18.82091604
	B1		0.019689	1512.625		-0.001006837	-618.794015	893.8309	0.446915466
	B2		0.037477	2879.111		-0.004737955	-2911.90905	-32.79759	-0.016398795
	D		0.008651	664.583		-0.001101412	-676.918888	-12.3359	-0.006167951
	E		0.080252	6165.291		-0.010145794	-6235.52379	-70.23233	-0.035116164
	F		0.008651	664.583		-0.001101412	-676.918888	-12.3359	-0.006167951
	H		0.394057	30273.03		0.011848463	7281.97048	37555.01	18.77750268
	J3		0.019689	1512.625		-0.001006837	-618.794015	893.8309	0.446915466
	J4		0.037477	2879.111		-0.004737955	-2911.90905	-32.79759	-0.016398795
19 to 28	Total	86427			717344				43.2135
	A		0.389853	33693.83		0.011983918	8596.59163	42290.43	21.14521325
	B1		0.020593	1779.791		-0.001000424	-717.647834	1062.143	0.531071625
	B2		0.039029	3373.117		-0.004753655	-3410.00573	-36.88832	-0.018444158
	D		0.009074	784.2306		-0.001113107	-798.480608	-14.25001	-0.007125003
	E		0.082903	7165.052		-0.010097539	-7243.40884	-78.3568	-0.039178402
	F		0.009074	784.2306		-0.001113107	-798.480608	-14.25001	-0.007125003
	H		0.389853	33693.83		0.011847991	8499.08557	42192.92	21.09646022
	J3		0.020593	1779.791		-0.001000424	-717.647834	1062.143	0.531071625
	J4		0.039029	3373.117		-0.004753655	-3410.00573	-36.88832	-0.018444158

Distribution of Forces (E/W) (Continued)

29 to 36	Total	86427			700059				43.2135
	A		0.392246	33900.69		0.011987221	8391.76222	42292.45	21.14622461
	B1		0.020077	1735.207		-0.001004106	-702.93372	1032.273	0.516136484
	B2		0.038145	3296.717		-0.004744698	-3321.5684	-24.85131	-0.012425653
	D		0.008832	763.3172		-0.001106401	-774.545653	-11.22845	-0.005614225
	E		0.0814	7035.144		-0.010125113	-7088.17632	-53.03231	-0.026516154
	F		0.008832	763.3172		-0.001106401	-774.545653	-11.22845	-0.005614225
	H		0.392246	33900.69		0.011848301	8294.50965	42195.2	21.09759833
	J3		0.020077	1735.207		-0.001004106	-702.93372	1032.273	0.516136484
	J4		0.038145	3296.717		-0.004744698	-3321.5684	-24.85131	-0.012425653
37 to 40	Total	96030			777843				48.015
	A		0.392246	37667.43		0.011987221	9324.17625	46991.61	23.49580313
	B1		0.020077	1928.007		-0.001004106	-781.037132	1146.97	0.57348515
	B2		0.038145	3663.019		-0.004744698	-3690.62997	-27.61098	-0.01380549
	D		0.008832	848.1302		-0.001106401	-860.605913	-12.47569	-0.006237844
	E		0.0814	7816.827		-0.010125113	-7875.7481	-58.92141	-0.029460706
	F		0.008832	848.1302		-0.001106401	-860.605913	-12.47569	-0.006237844
	H		0.392246	37667.43		0.011848301	9216.11788	46883.55	23.44177395
	J3		0.020077	1928.007		-0.001004106	-781.037132	1146.97	0.57348515
	J4		0.038145	3663.019		-0.004744698	-3690.62997	-27.61098	-0.01380549
41 to 49	Total	96030			249678				48.015
	A		0.462426	44406.74		0.012002026	2996.64196	47403.38	23.70168875
	B1		0.005035	483.509		-0.000819466	-204.602552	278.9065	0.139453234
	B2		0.009538	915.892		-0.003213657	-802.379574	113.5124	0.05675622
	D		0.012882	1237.043		-0.004367216	-1090.39777	146.645	0.073322494
	E		0.02024	1943.641		-0.006819797	-1702.75324	240.8881	0.120444041
	F		0.012882	1237.043		-0.004367216	-1090.39777	146.645	0.073322494
	H		0.462426	44406.74		0.011618449	2900.87108	47307.61	23.65380331
	J3		0.005035	483.509		-0.000819466	-204.602552	278.9065	0.139453234
	J4		0.009538	915.892		-0.003213657	-802.379574	113.5124	0.05675622
**(.5 load adjustment factor)									

Story Drift (N/S)

Story Drift(N/S)					
Floor	k total	P direct	h/400	story drift	acceptable
3	408853.93	134442	0.351	0.328826	y
4	408853.93	134442	0.351	0.328826	y
5	408853.93	134442	0.351	0.328826	y
6	408853.93	134442	0.351	0.328826	y
7	408853.93	151247.6	0.351	0.369931	n
8	408853.93	151247.6	0.351	0.369931	n
9	408853.93	151247.6	0.351	0.369931	n
10	408853.93	151247.6	0.351	0.369931	n
11	408853.93	151247.6	0.351	0.369931	n
12	408853.93	151247.6	0.351	0.369931	n
13	408853.93	151247.6	0.351	0.369931	n
14	408853.93	151247.6	0.351	0.369931	n
15	408853.93	168052.5	0.351	0.411033	n
16	408853.93	168052.5	0.351	0.411033	n
17	408853.93	168052.5	0.351	0.411033	n
18	2301979.7	168052.5	0.351	0.073003	y
19	2301979.7	168052.5	0.351	0.073003	y
20	2301979.7	168052.5	0.351	0.073003	y
21	2301979.7	168052.5	0.351	0.073003	y
22	2301979.7	168052.5	0.351	0.073003	y
23	2301979.7	168052.5	0.351	0.073003	y
24	2301979.7	168052.5	0.351	0.073003	y
25	2301979.7	168052.5	0.351	0.073003	y
26	2301979.7	168052.5	0.351	0.073003	y
27	2301979.7	168052.5	0.351	0.073003	y
28	1003456.7	168052.5	0.351	0.167474	y
29	1003456.7	168052.5	0.351	0.167474	y
30	1003456.7	168052.5	0.351	0.167474	y
31	1003456.7	168052.5	0.351	0.167474	y
32	1003456.7	168052.5	0.351	0.167474	y
33	1003456.7	168052.5	0.351	0.167474	y
34	1003456.7	168052.5	0.351	0.167474	y
35	1003456.7	184858.1	0.351	0.184221	y
36	1003456.7	184858.1	0.351	0.184221	y
37	1003456.7	184858.1	0.351	0.184221	y
38	1003456.7	184858.1	0.351	0.184221	y
39	1003456.7	184858.1	0.351	0.184221	y
40	7644545.3	184858.1	0.351	0.024182	y
41	7644545.3	184858.1	0.351	0.024182	y
42	7644545.3	184858.1	0.351	0.024182	y
43	7644545.3	184858.1	0.351	0.024182	y
44	7644545.3	184858.1	0.351	0.024182	y
45	7644545.3	184858.1	0.351	0.024182	y
46	7644545.3	184858.1	0.351	0.024182	y
47	7644545.3	184858.1	0.351	0.024182	y
48	7644545.3	184858.1	0.351	0.024182	y
49	7644545.3	184858.1	0.351	0.024182	y
Total			16.5	8.573124	y

Story Drift (E/W)

Story Drift (E/W)					
Floor	k total	P direct	h/400	story drift	acceptable
3	57393.412	33610.5	0.351	0.585616	n
4	57393.412	33610.5	0.351	0.585616	n
5	57393.412	33610.5	0.351	0.585616	n
6	57393.412	33610.5	0.351	0.585616	n
7	57393.412	33610.5	0.351	0.585616	n
8	57393.412	33610.5	0.351	0.585616	n
9	57393.412	38412	0.351	0.669275	n
10	57393.412	38412	0.351	0.669275	n
11	57393.412	38412	0.351	0.669275	n
12	57393.412	38412	0.351	0.669275	n
13	57393.412	38412	0.351	0.669275	n
14	57393.412	38412	0.351	0.669275	n
15	57393.412	38412	0.351	0.669275	n
16	57393.412	38412	0.351	0.669275	n
17	57393.412	38412	0.351	0.669275	n
18	366190.01	38412	0.351	0.104896	y
19	366190.01	43213.5	0.351	0.118008	y
20	366190.01	43213.5	0.351	0.118008	y
21	366190.01	43213.5	0.351	0.118008	y
22	366190.01	43213.5	0.351	0.118008	y
23	366190.01	43213.5	0.351	0.118008	y
24	366190.01	43213.5	0.351	0.118008	y
25	366190.01	43213.5	0.351	0.118008	y
26	366190.01	43213.5	0.351	0.118008	y
27	366190.01	43213.5	0.351	0.118008	y
28	149189.79	43213.5	0.351	0.289655	y
29	149189.79	43213.5	0.351	0.289655	y
30	149189.79	43213.5	0.351	0.289655	y
31	149189.79	43213.5	0.351	0.289655	y
32	149189.79	43213.5	0.351	0.289655	y
33	149189.79	43213.5	0.351	0.289655	y
34	149189.79	43213.5	0.351	0.289655	y
35	149189.79	43213.5	0.351	0.289655	y
36	149189.79	43213.5	0.351	0.289655	y
37	149189.79	48015	0.351	0.321838	y
38	149189.79	48015	0.351	0.321838	y
39	149189.79	48015	0.351	0.321838	y
40	1412927.9	48015	0.351	0.033983	y
41	1412927.9	48015	0.351	0.033983	y
42	1412927.9	48015	0.351	0.033983	y
43	1412927.9	48015	0.351	0.033983	y
44	1412927.9	48015	0.351	0.033983	y
45	1412927.9	48015	0.351	0.033983	y
46	1412927.9	48015	0.351	0.033983	y
47	1412927.9	48015	0.351	0.033983	y
48	1412927.9	48015	0.351	0.033983	y
49	1412927.9	48015	0.351	0.033983	y
Total			16.5	14.61638	y

Appendix B

Appendix B

North Side (U value = 0.4)

Air System Simulation Results (Table 1) : 0.4

Month	Terminal Cooling Coil Load (kBTU)	Terminal Cooling Eqpt Load (kBTU)	Terminal Unit Clg Input (kWh)	Terminal Heating Coil Load (kBTU)	Terminal Heating Coil Input (kWh)	Terminal Fan (kWh)	Lighting (kWh)
January	10236	775	538	37	11	267	2126
February	10549	719	511	21	6	241	1920
March	15058	838	621	0	0	267	2126
April	17500	802	613	0	0	258	2058
May	20269	816	650	0	0	267	2126
June	21344	779	643	0	0	258	2058
July	23288	803	668	0	0	267	2126
August	22049	797	676	0	0	267	2126
September	19488	775	649	0	0	258	2058
October	15710	819	646	0	0	267	2126
November	11993	805	603	0	0	258	2058
December	9279	733	504	62	18	267	2126
Total	**196762**	**9461**	**7321**	**120**	**35**	**3139**	**25035**

Air System Simulation Results (Table 2) :

Month	Electric Equipment (kWh)
January	106
February	95
March	106
April	102
May	106
June	102
July	106
August	106
September	102
October	106
November	102
December	106
Total	**1244**

North Side (U value = 0.35)

Air System Simulation Results (Table 1) : 0.35

Month	Terminal Cooling Coil Load (kBTU)	Terminal Cooling Eqpt Load (kBTU)	Terminal Unit Clg Input (kWh)	Terminal Heating Coil Load (kBTU)	Terminal Heating Coil Input (kWh)	Terminal Fan (kWh)	Lighting (kWh)
January	9805	787	539	29	9	242	2126
February	9997	727	511	16	5	219	1920
March	14076	838	621	0	0	242	2126
April	16214	802	613	0	0	235	2058
May	18732	816	650	0	0	242	2126
June	19747	779	643	0	0	235	2058
July	21503	803	668	0	0	242	2126
August	20466	797	676	0	0	242	2126
September	18178	775	649	0	0	235	2058
October	14792	819	646	0	0	242	2126
November	11410	807	604	0	0	235	2058
December	8964	744	508	51	15	242	2126
Total	183886	9495	7328	96	28	2853	25035

Air System Simulation Results (Table 2) :

Month	Electric Equipment (kWh)
January	106
February	95
March	106
April	102
May	106
June	102
July	106
August	106
September	102
October	106
November	102
December	106
Total	1244

South Side (U value = 0.4)

Air System Simulation Results (Table 1) : 0.4

Month	Terminal Cooling Coil Load (kBTU)	Terminal Cooling Eqpt Load (kBTU)	Terminal Unit Clg Input (kWh)	Terminal Heating Coil Load (kBTU)	Terminal Heating Coil Input (kWh)	Terminal Fan (kWh)	Lighting (kWh)
January	15412	822	540	16	5	361	2126
February	15416	752	512	0	0	326	1920
March	18313	838	621	0	0	361	2126
April	18961	802	613	0	0	350	2058
May	20159	816	650	0	0	361	2126
June	19561	779	643	0	0	350	2058
July	21413	803	668	0	0	361	2126
August	21878	797	676	0	0	361	2126
September	21554	775	649	0	0	350	2058
October	20137	819	646	0	0	361	2126
November	18630	808	605	0	0	350	2058
December	16182	831	517	3	1	361	2126
Total	**227616**	**9643**	**7338**	**19**	**6**	**4256**	**25035**

Air System Simulation Results (Table 2) :

Month	Electric Equipment (kWh)
January	106
February	95
March	106
April	102
May	106
June	102
July	106
August	106
September	102
October	106
November	102
December	106
Total	**1244**

South Side (U value = 0.35)

Air System Simulation Results (Table 1) : 0.35

Month	Terminal Cooling Coil Load (kBTU)	Terminal Cooling Eqpt Load (kBTU)	Terminal Unit Clg Input (kWh)	Terminal Heating Coil Load (kBTU)	Terminal Heating Coil Input (kWh)	Terminal Fan (kWh)	Lighting (kWh)
January	14358	821	540	15	4	325	2126
February	14267	756	512	0	0	294	1920
March	16950	838	621	0	0	325	2126
April	17514	802	613	0	0	314	2058
May	18678	816	650	0	0	325	2126
June	18216	779	643	0	0	314	2058
July	19900	803	668	0	0	325	2126
August	20343	797	676	0	0	325	2126
September	20004	775	649	0	0	314	2058
October	18673	819	646	0	0	325	2126
November	17195	808	605	0	0	314	2058
December	15004	836	517	3	1	325	2126
Total	**211102**	**9651**	**7338**	**18**	**5**	**3826**	**25035**

Air System Simulation Results (Table 2) :

Month	Electric Equipment (kWh)
January	106
February	95
March	106
April	102
May	106
June	102
July	106
August	106
September	102
October	106
November	102
December	106
Total	**1244**

South Side (U value=0.4) (75%glass)

Air System Simulation Results (Table 1) : 75%

Month	Terminal Cooling Coil Load (kBTU)	Terminal Cooling Eqpt Load (kBTU)	Terminal Unit Clg Input (kWh)	Terminal Heating Coil Load (kBTU)	Terminal Heating Coil Input (kWh)	Terminal Fan (kWh)	Lighting (kWh)
January	12669	821	540	16	5	273	2126
February	12501	750	512	0	0	247	1920
March	14936	838	621	0	0	273	2126
April	15467	802	613	0	0	264	2058
May	16676	816	650	0	0	273	2126
June	16476	779	643	0	0	264	2058
July	17937	803	668	0	0	273	2126
August	18379	797	676	0	0	273	2126
September	17995	775	649	0	0	264	2058
October	16675	819	646	0	0	273	2126
November	15138	808	605	0	0	264	2058
December	13075	831	517	3	1	273	2126
Total	**187923**	**9640**	**7338**	**19**	**6**	**3213**	**25035**

Air System Simulation Results (Table 2) :

Month	Electric Equipment (kWh)
January	106
February	95
March	106
April	102
May	106
June	102
July	106
August	106
September	102
October	106
November	102
December	106
Total	**1244**

Appendix C