EXECUTIVE SUMMARY

This thesis project proposes a partial redesign of 554-556 Third Avenue. It explores one possibility for providing extra support for heavy cladding at cantilevered edges of slabs. In the proposed design, this extra support is achieved through a system of exposed roof trusses spanning two directions. Wide flange tension members are hung from the trusses at the exterior edge of the cantilevers. The trusses are supported by a combination of several columns and the shear wall core.

The structural design consists of the truss design and design of tension members to support the cladding. Additionally columns and shear walls supporting the new system are checked and redesigned as necessary to ensure that they are capable of supporting the trusses. Several column sizes and reinforcement requirements are changed in order to support the increased loads. Braced frames are also designed to replace the shear walls that extend above roof level and will be interrupted by the new trusses.

The design changes directly affect the building architecture and the management of the construction process. A heavy precast architectural panel system was chosen for the new truss system to support. The new cladding and exposed steel roof truss give the building a very different exterior appearance. However, care was taken to ensure that it still fits into the neighborhood. Adding trusses and changing the cladding system will create additional costs, scheduling issues, and site obstacles as well. These issues are also discussed within the report.

ACKNOWLEDGEMENTS

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BUILDING INFORMATION

The Aurora, commonly referred to by its address, 554-556 Third Avenue, is a thirty story residential high rise in Midtown Manhattan. While the footprint only occupies about 3500 square feet near the southwest corner of Third Avenue and Thirty-Seventh Street, the building is approximately 120,000 square feet. The main roof is 418 feet above curb level. On the roof of the building is a penthouse that extends an additional 50 feet into the air.

Figure 1: Building Location

The architecture of 554-556 Third Avenue is simple and functional. The ground floor offers retail space at street level as well as a lobby for the residents of the building. Floors two through twenty-four are ExecuStay suites, with a fitness room on the fifth floor. The lower floors are comprised of tightly packed residential units. They leave little room for adjustment. Residential units were designed for maximum space efficiency and column placement is crucial. Floors twenty-five through thirty each house three condominiums with double height ceilings. While the double height stories offer a feeling of luxury to the condominium owners, their initial purpose was to allow for floors to be added at mid-height if zoning restrictions limiting the number of floors in the building change. Floor plans demonstrating the layouts of different floors are shown in figure 2(a)-(d).

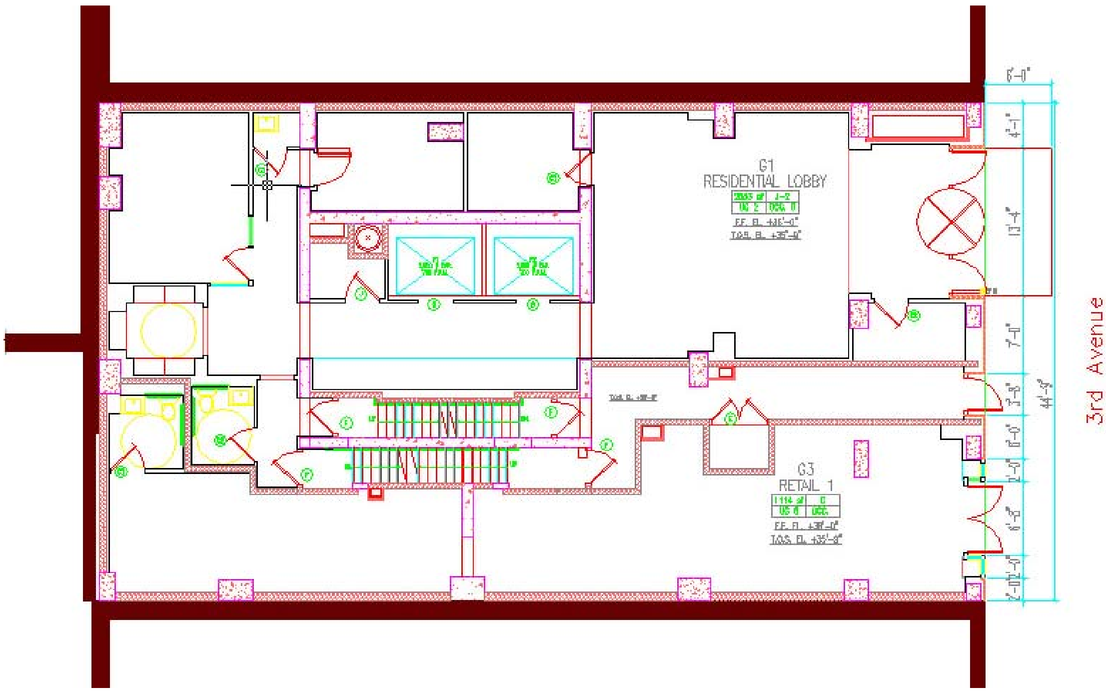


Figure 2(a): Ground Floor Plan

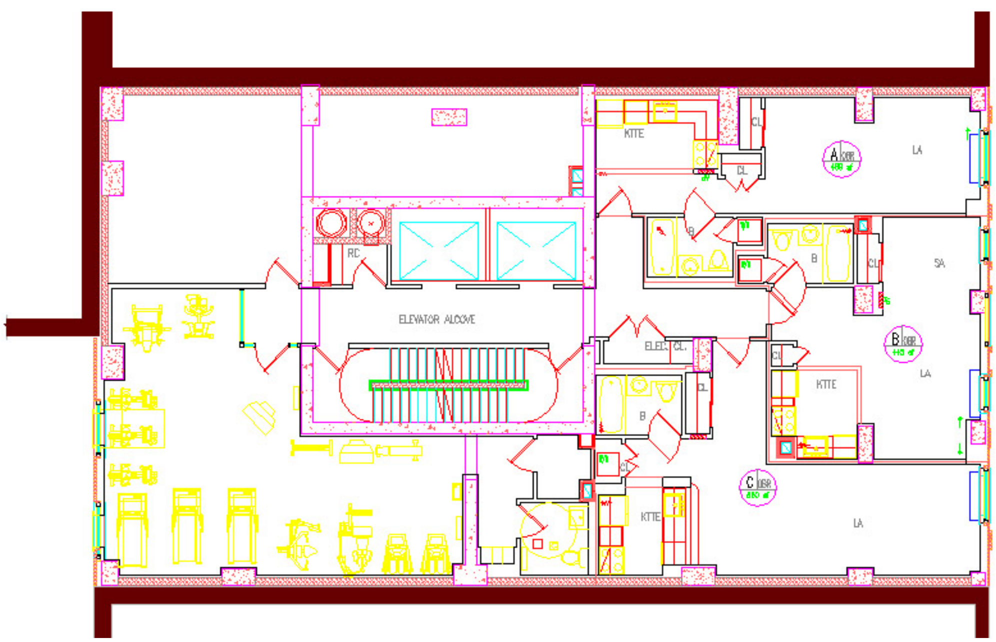


Figure 2(b): Fifth Floor Plan

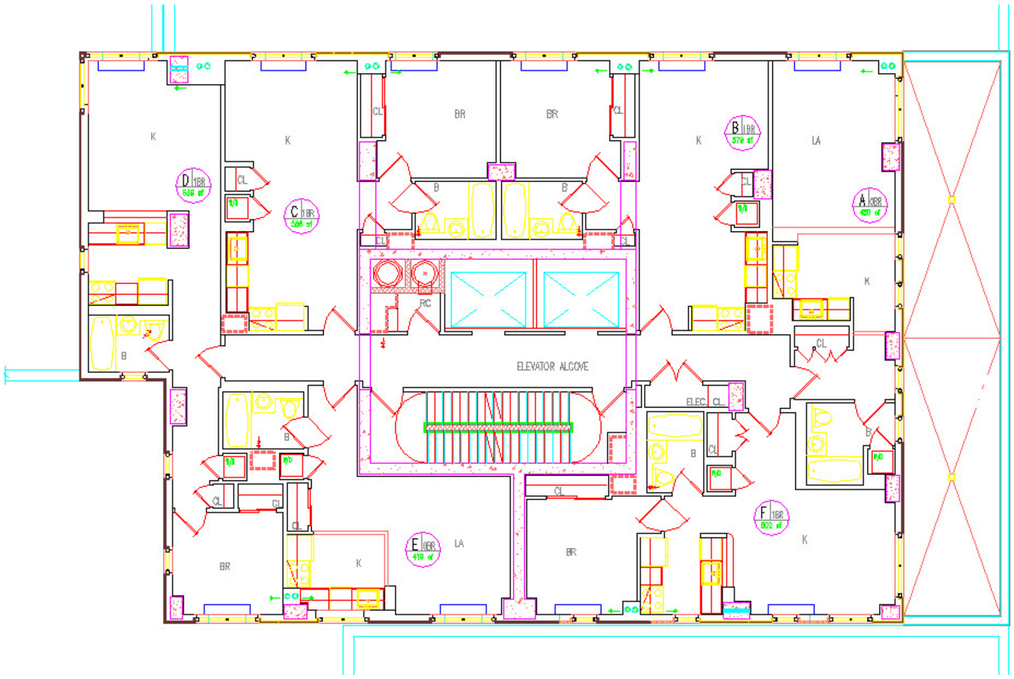


Figure 2(c): Floor Plan 8-24

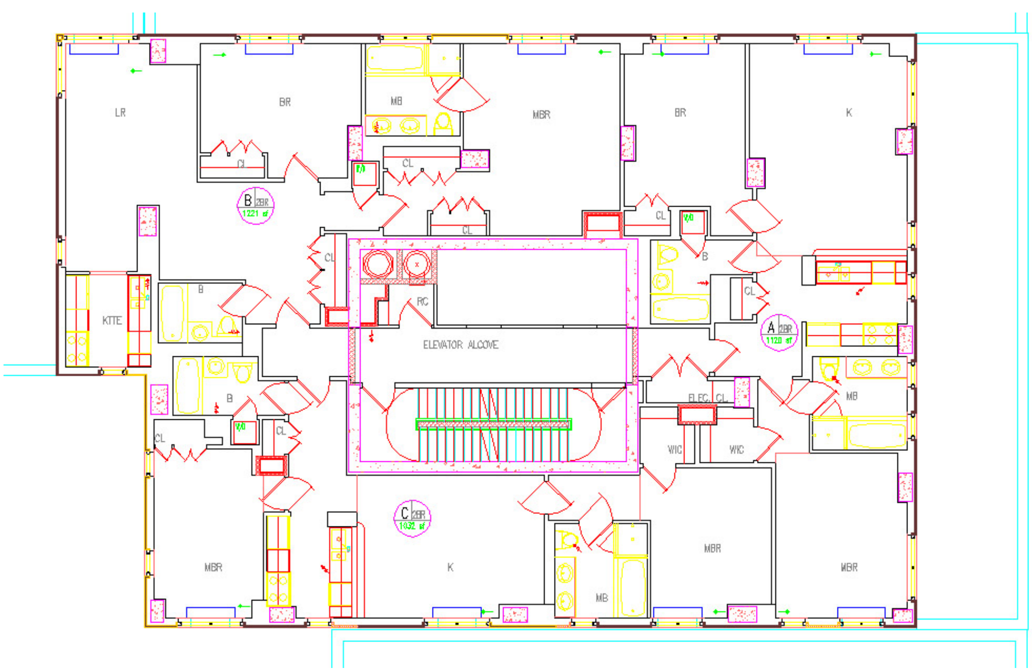


Figure 2(d): Floor Plan 29-30

The site is very slender and is restricted by four-story brick buildings on three sides. The adjacent liquor store on the corner was originally going to be purchased by the owner to create a wider corner site for the building. However, the owner chose not to sell. Instead, air rights were purchased above this and another adjacent building. The upper floors of 554-556 Third Avenue cantilever over these two adjacent buildings. Cantilevers begin at level six on the north side and eight on the west side. Each floor cantilevers independently of the other floors and supports its own section of the façade. In order to keep the façade light enough for the cantilevered slabs to support, the architect used light weight metal panels as cladding. The metal panel cladding resembles a mixture of concrete and brick finishes with windows interspersed. On the non-cantilevered street side of the building, a ten foot setback occurs at level eight. This setback is required by city code. The purpose of this zoning requirement is to prevent pedestrians from feeling closed in when walking along a street lined with high-rise buildings.

Figure 3 (a)-(c): Construction photographs of the current building from left to right:
(a) West side cantilever
(b) Full building from street showing cladding system
(c) Double height floor for condominiums

Building Information

The Building Code of the City of New York (NYC Building Code) and ACI 318-98 were the codes used for the structural design of this building. The structural system is comprised of flat plates that distribute gravity and lateral loads among the columns and shear walls. The shear walls, which surround the stair and elevator core, are the main lateral force resisting members.

A CM at risk with a guaranteed maximum price was used as the delivery method for 554-556 Third Avenue. Excavation began in August of 2001, and the structural work was completed in late June 2002. The superstructure work started in December of 2001 and lasted approximately eight months, topping out in July of 2002. During the portion of the work requiring a tower crane, a small section of Third Avenue was blocked. Concrete was pumped up to level six and placed with a crane and bucket above level six once the tower crane was in place. The Aurora at 554-556 Third Avenue is scheduled for a June 2003 opening.

STRUCTURAL EXISTING CONDITIONS

Gravity System

The Aurora makes use of a flat plate reinforced concrete system to support gravity loads and act as a diaphragm for shear loads. Gravity loads are supported by two-way reinforced slabs that distribute the load into eighteen reinforced concrete columns. Figure 4 shows a typical framing plan for floors nine through twenty-five. The slab spans up to twenty feet in each direction, but span lengths vary due to the asymmetrical column layout. Typical slab thickness is nine inches, with the exception of the area enclosed by the shear wall core and floors six and eight, which are all twelve inches deep. Concrete strengths are outlined below.

<u>Foundation:</u>

Mat Foundation: 5000 psi

<u>Slabs:</u>

Cellar: 5000 psi

Ground – Twenty: 5950 psi

Twenty-One – Roof: 5000 psi

<u>Columns and Shear Walls:</u>

Supporting Ground – Twenty: 8000 psi

Supporting Twenty-One – Roof: 5000 psi

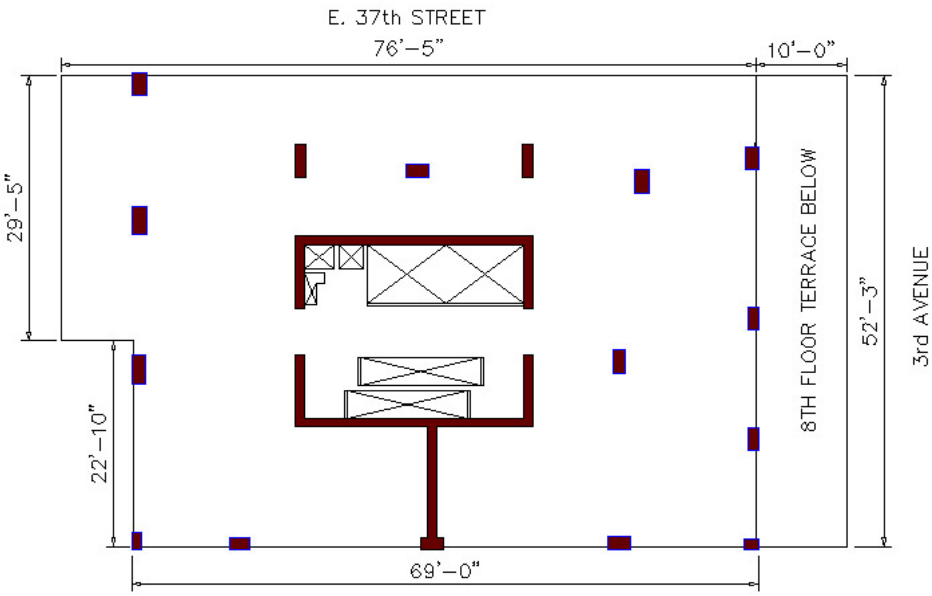


Figure 4: Typical Floor Plan Floors 9-20

The building cantilevers eight feet on two sides beginning at floors six and eight. The cantilevered slabs each support their own weight, as well as the additional dead and live loads that will be imposed. The design gravity loads are listed below. These include the self weight of the system as well as the required live loads imposed by the NYC Building Code.

<u>Dead Loads</u>

Slab Self-Weight:	112.5 psf for 9" slabs
	150 psf for 12" slabs
	300 psf for top roof
Superimposed Dead Loads:	Mechanical = 10 psf
	Partitions = 5 psf
	Flooring = 5 psf
	Total = 20 psf
Cladding Loads :	Metal Panel Siding = 30 psf

Live Loads (as specified by NYC Building)

Roof:

Snow Load = 30psf

Concentrated Live Load = 200 lbs on area of 2 ft by 2 ft

Interior Floors:

Dwellings-Apartments = 40 psf

Equipment Rooms = 75 psf

Storage Light = 100 psf

Exercise Room = 100 psf

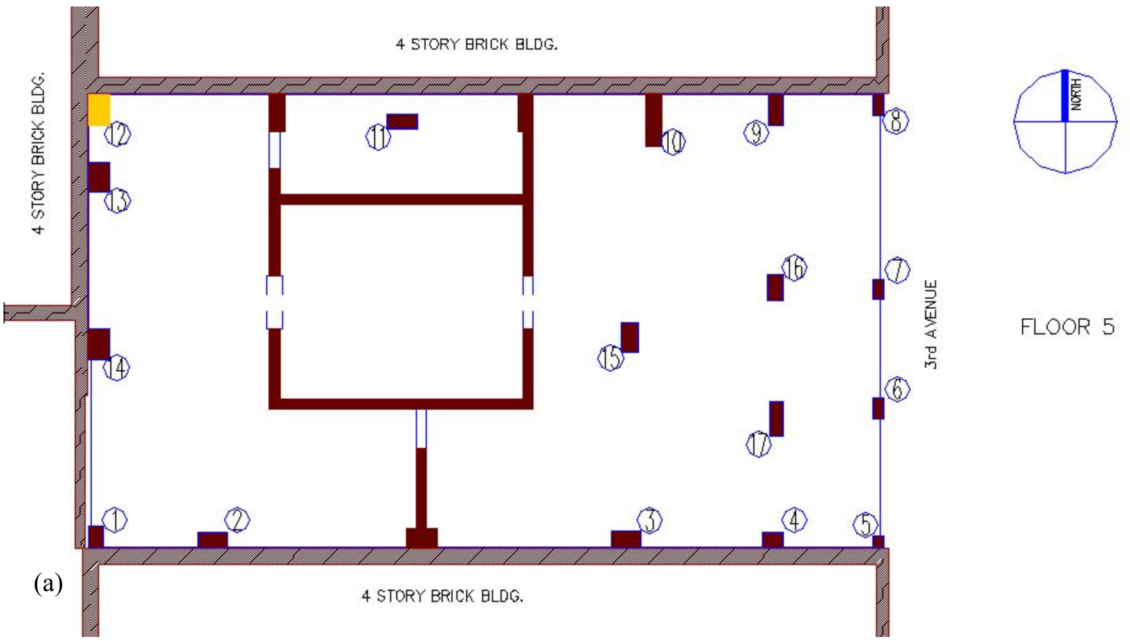
Telephone Equipment Room = 80 psf

Retail Sales – Basement & First Floor = 100 psf

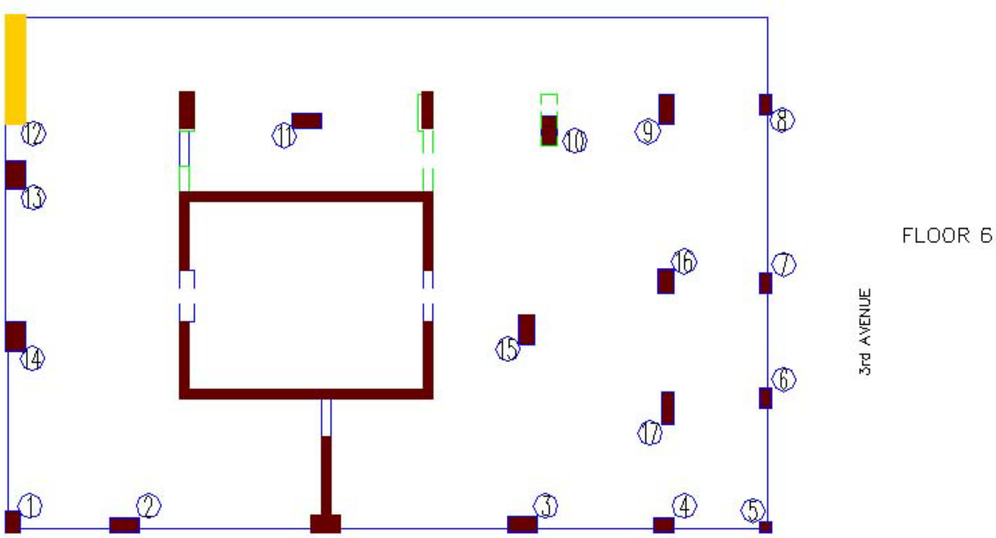
Due to tight site conditions, including encroaching foundation walls from some of the adjacent four story brick buildings, several of the columns shift partway up the building through the use of column walks. As the columns move up the building, they shift toward the exterior. Five of the columns stop below floor eight where the setback occurs. The remaining columns extend to the main roof. The only gravity resisting members for the elevator machine room and bulkhead, which are above the main roof, are the shear walls described below.

As previously mentioned, several columns shift slightly from floor to floor. The most severe case occurs when column 12 shifts to become column 18 between the fifth and eighth floors as shown in figure 5. This allows a column to be located at the outer edge of the north side cantilever. The location of this column is critical because it is where the north cantilever meets the west cantilever. Without this column the northwest corner could not support its own weight. However, the shifting column causes a large tension force in the slab at the top level of the sloped portion. Therefore, post-tensioning is used in this part of the slab at floor eight to relieve the

concrete of tensile forces it cannot handle. As the design was issued, the cantilevers prevented a heavier cladding system from being used due to the limitations in the strength of such a thin slab. A quick ADOSS analysis proved that the slab would fail if it were subjected to loading from even a 55 psf brick system at the edges of the cantilevers.



(a)



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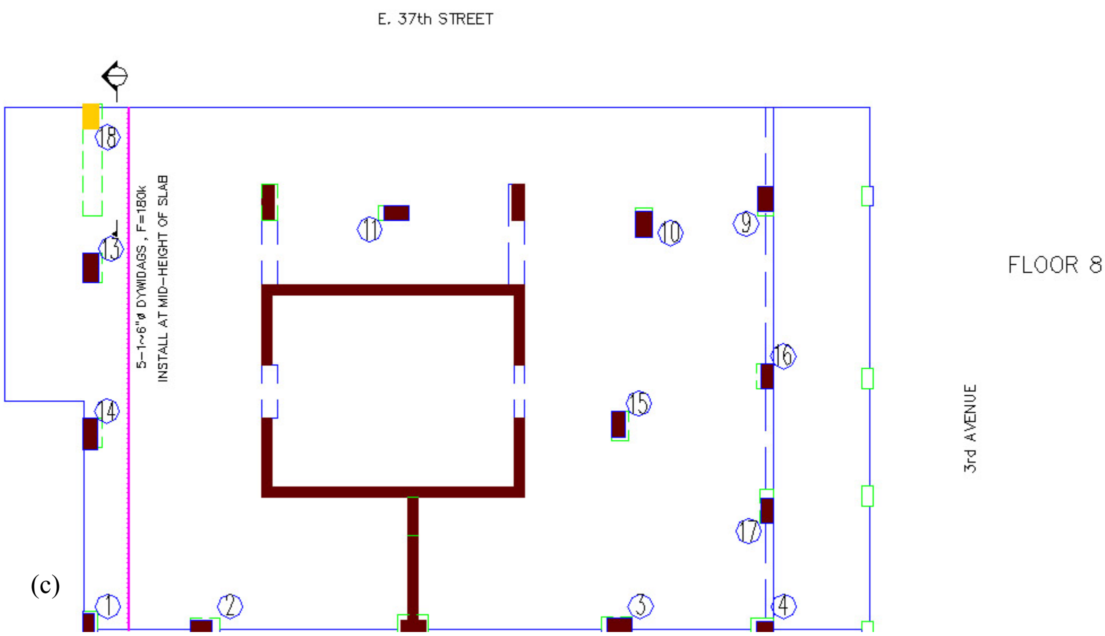

(d)

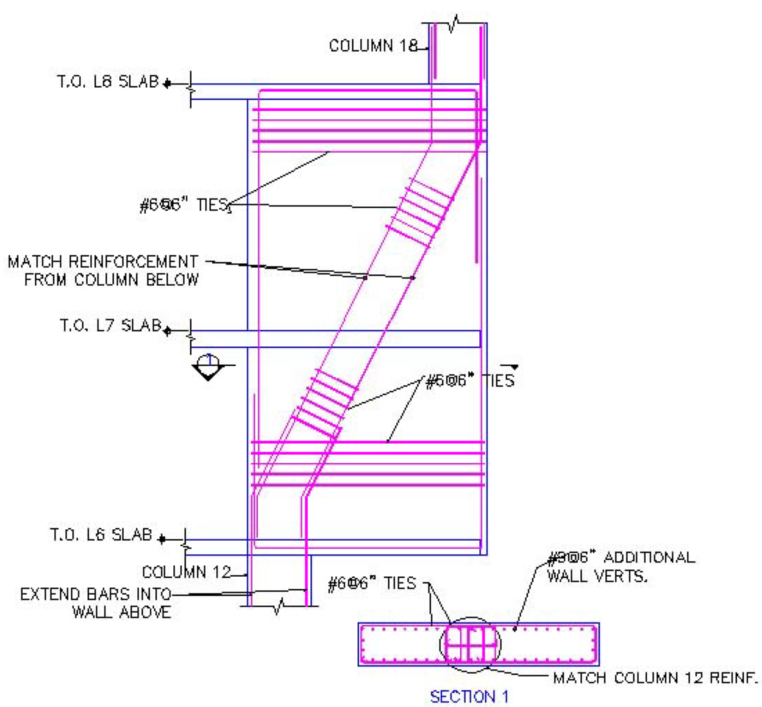

Figure 5(a)-(d): Column walk between floors six and eight

Lateral System

Shear walls in the core of the building provide the primary lateral system for 554-556 Third Avenue. This system of shear walls houses the elevators and stairs. Therefore, while the walls are relatively uniform throughout the building, many floors have openings in the shear walls to provide doorways into the elevator and stair lobby. The walls are connected above and below each of these door openings by link beams. This creates complications in distributing the lateral forces to the different walls in the frame. A typical shear wall layout is shown in figure 6. In addition to the shear walls, concrete columns can be expected to resist a smaller fraction of the lateral loads. The effect of these columns is ignored in the distribution of shear forces included in Appendix A.

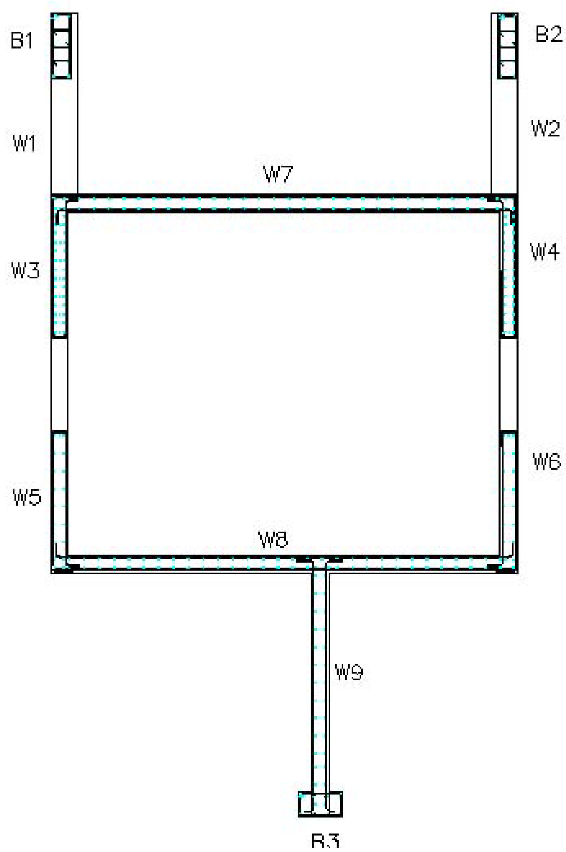

Figure 6: Typical Shear Wall Plan Floors 9-25

Lateral loads were determined based on the wind and seismic calculations shown in the appendix. Each floor was analyzed for the maximum lateral load at that level. This was the wind load for all levels other than the top, where seismic loads control. The design shear spreadsheet in Appendix A shows the maximum lateral force at each level and distinguishes whether wind or seismic loads govern at that floor. The loads on this spreadsheet are later divided for individual shear walls to resist. The slabs act as diaphragms for transferring lateral loads from the face of the building to the lateral resisting shear walls. The distribution of lateral loads by floor is shown in figure 7.

The spreadsheet entitled “Distribution of Forces” in Appendix A provides a wall by wall distribution of the shear forces. The walls are labeled as shown on the typical shear wall plan for floors nine through twenty-five in figure 6. Lateral forces are distributed based on the relative stiffness of the resisting shear walls in the direction of each load. The “Direct Stiffness by Floor” spreadsheet determines the relative stiffness of each shear wall in a particular direction. “Center of Rigidity” incorporates the effects of eccentricity into the total forces. This spreadsheet calculates the center of rigidity as well as the eccentricity of the force on each floor. The information is referenced in the “Distribution of Forces” spreadsheet, where the torsion effects are calculated and added to the forces distributed through the direct stiffness method. Torsion effects are based on the difference between the center of rigidity of the shear walls and the centroid of the lateral load. In the case were the center of rigidity occurs at the center of the wall, a minimum eccentricity of five percent of the building width is assumed. Additionally, the effects of torsion are only considered where they increase the shear force on the wall. In areas were torsion forces would reduce the overall shear, torsion is neglected.

Link beams are ignored in the distribution of forces and the calculation of stiffness. This simplifies the calculation of stiffness, allowing all walls to behave in a similar manner. Had the link beams been included in the calculations, they would

increase the stiffness of the walls that they connect, drawing a larger percentage of these forces to these walls. It is likely that due to this simplification, the shear walls that are connected by these link beams resist larger loads than those calculated in the appendix.

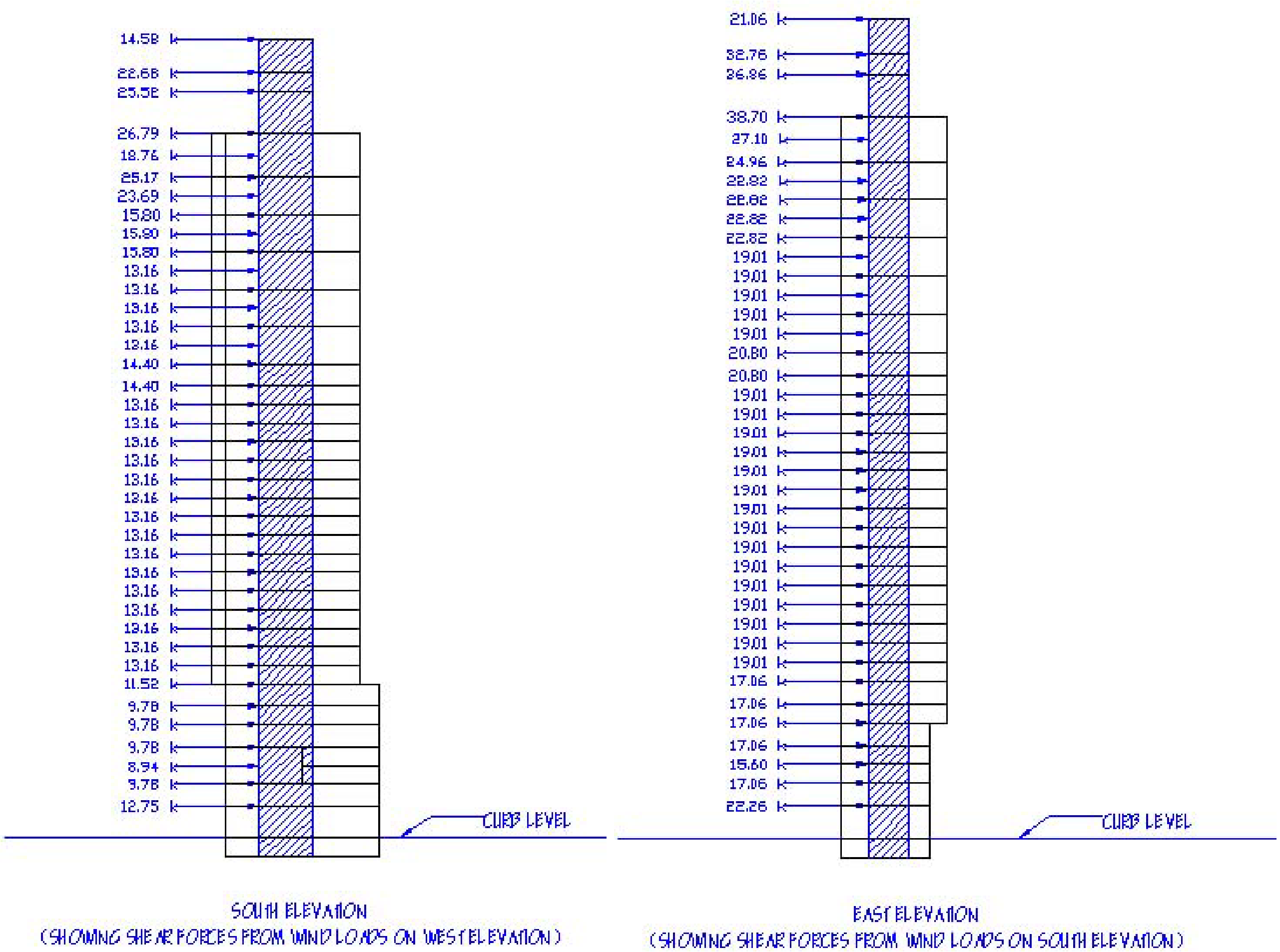

Figure 7: Shear forces from wind loads

Foundation

The Aurora rests on a mat foundation. The depth of the mat ranges from 60 inches on the west side to 42 inches on the east side. This foundation behaves as a fixed support to resist overturning moments transferred down through the shear wall lateral system. The maximum overturning moment to be resisted in the north-south direction is 172,141 ft-k. In the east-west direction, a moment of 125,257 ft-k needs to be resisted. These values, along with the overall building dead load were determined through the use of spread sheets that can be found in the appendix. The dead load is sufficient to resist the uplift forces from the overturning moments. Data from the geotechnical report indicates that the bearing capacity of the soil below the foundation is two tons per square foot. Based on this value and the area of the mat, the total bearing capacity of the soil under the mat is great enough to resist the sum of forces from dead load and the downward forces from overturning moments.

PROPOSAL

Problem Statement

The current structural system of 554-556 Third Avenue is incapable of supporting a heavy cladding system. The system was designed in accordance with the provisions of ACI 318-95, using the live loads specified by the Building Code of the City of New York (NYC Building Code). Structural changes must adhere to the provisions of the NYC Building Code, ACI 318-02, and the AISC Manual of Steel Construction for Load and Resistance Factor Design. The current structural system is efficient but limits the architect in the building's exterior appearance. Since the slabs must support the cladding in the cantilevered regions, the architect was forced to use a lightweight metal cladding rather than a heavier system such as brick and block or precast architectural concrete.

Design Criteria

The main goal of a new structural system for this building is to remove the tight weight restriction from the cladding selection. The new structural design should offer the cantilevers enough support to allow the use of a heavier cladding system. The criteria for achieving this are as follows:

- Height may be increased
- Overall floor area must not be increased according to code
- Reductions to usable floor area must be minimal
- System must not interrupt interior floor plan
- Design should adhere to provisions of ACI 318-02, NYC Building Code, AISC LRFD Manual

STRUCTURAL REDESIGN

Truss and Braced Frame System

The extra support for the cladding system in cantilevered areas can be achieved through the use of tension supports hanging from a roof truss. Rather than changing the entire framing system of the building, extra support for cantilevered edges carrying heavy cladding loads will be supplied from above. Wide flange members will be used in tension to pick up the excess cladding loads and transfer them to the series of roof trusses above.

Design of the truss system must not only be structurally sound, but it must also add architectural interest to the top of the building since the steel will remain exposed. The truss design shown in figure 8 can be supported by several of the interior columns and the shear walls. It spans two directions to pick up the cladding loads from both cantilevers. Since the truss will be supported directly on the shear walls in the core of the building, the penthouse in the central core that was an extension of the shear walls will change to braced frames that will be supported by the truss.

The entire truss and braced frame system was modeled in RISA for analysis. Top and bottom chords were modeled as continuous members, while all other connections were pinned. The supports were placed in locations where a truss node could be aligned with a column or shear wall. Due to the asymmetry of the column layout, the support conditions do not occur at the same place on all symmetric trusses. The columns and shear walls that support the trusses, as well as the lines of the trusses, are highlighted in yellow on the roof floor plan in figure 9.

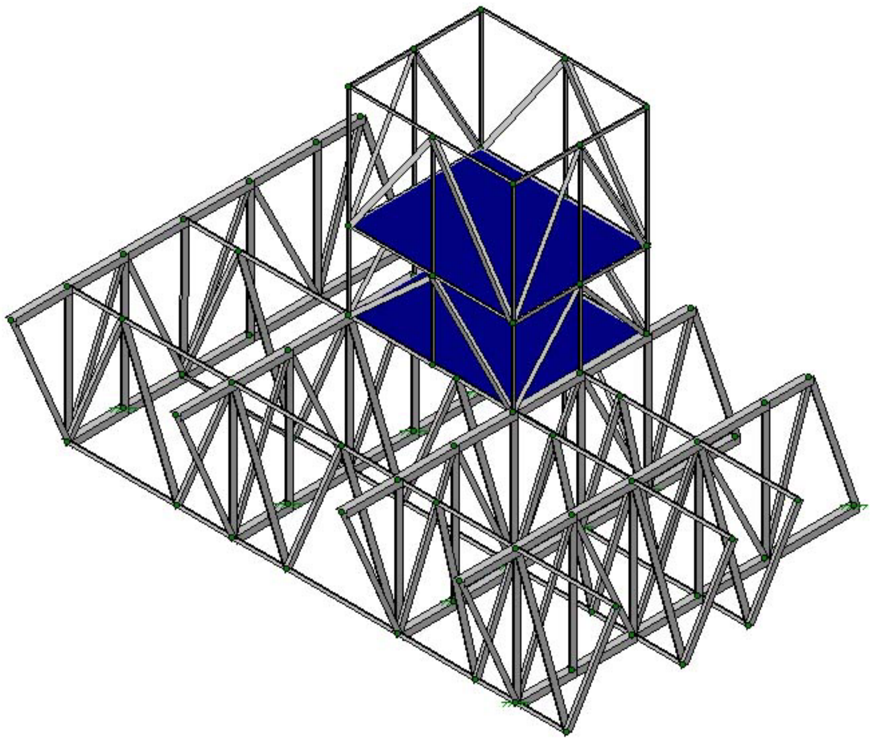

Figure 8: Truss and braced frames

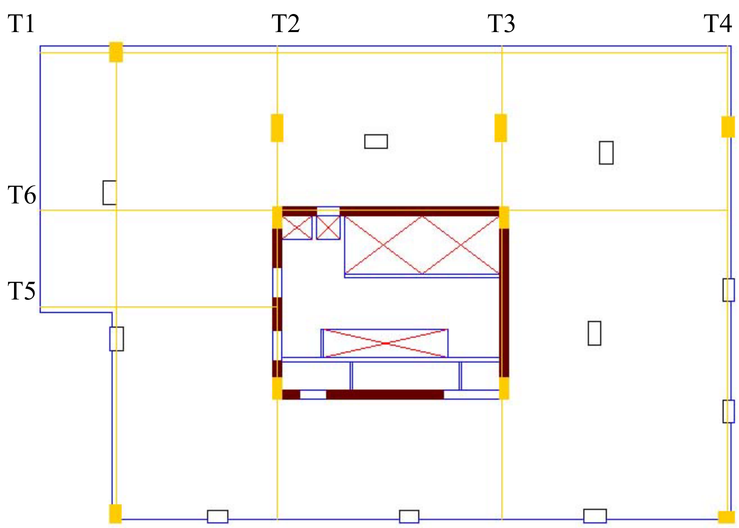

Figure 9: Roof floor plan with truss connection locations

The design calls for wide flange shapes for all members. The section numbers that correspond to different shapes refer to the color coded drawing in figure 10. Top chords for trusses in the longitudinal direction are W12x83 (section 9), and bottom chords in this direction are W12x72 (section 3). In the transverse direction the upper chords are W14x90s (section 8), and lower chords are W14x211s (section 2). All vertical truss members are W14x90s (section 4). Vertical members for the braced frames are W14x48s (section 5). Diagonal truss members are W12x79s (section 1), and diagonal bracing members are W8x40s (section 6). The beams in the central core are W4x13s (section 7). A summary of the chosen shapes is shown in figures 10, 11 and 12.

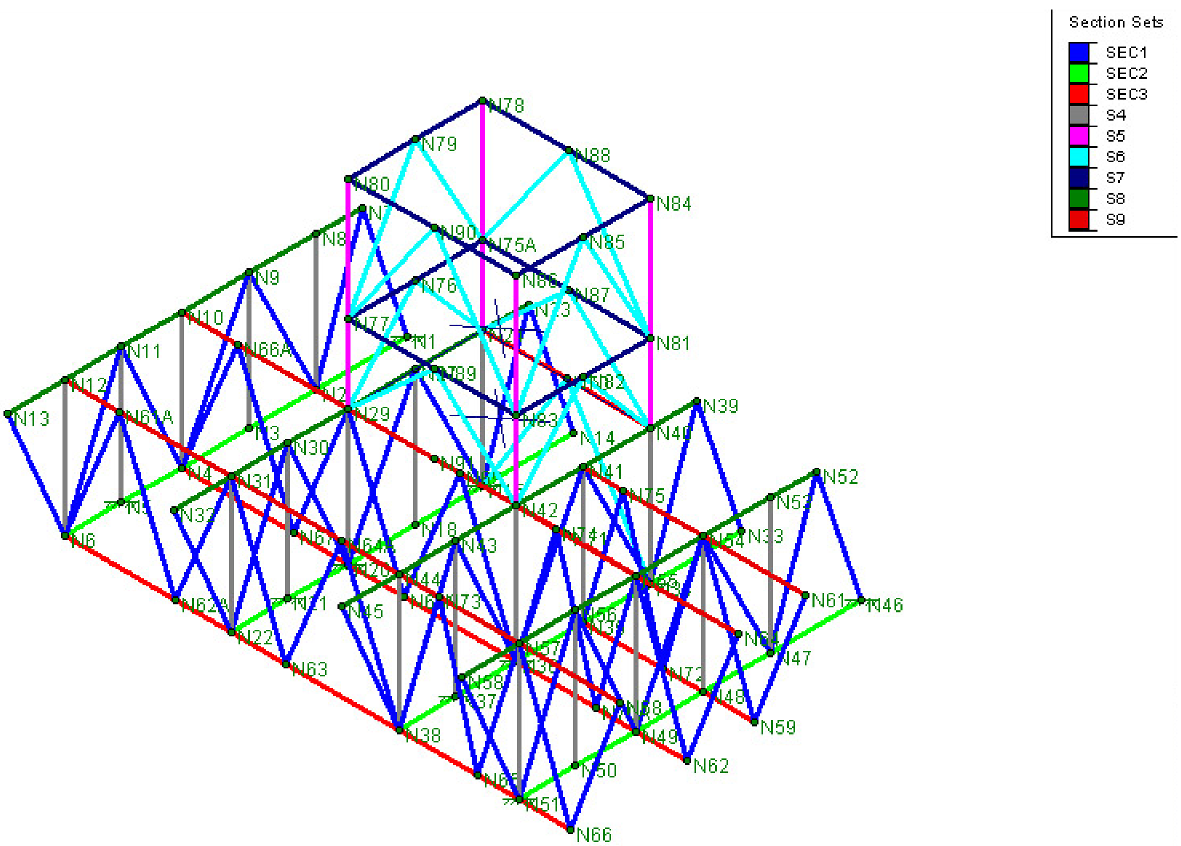

Figure 10: Truss with section groups color coded

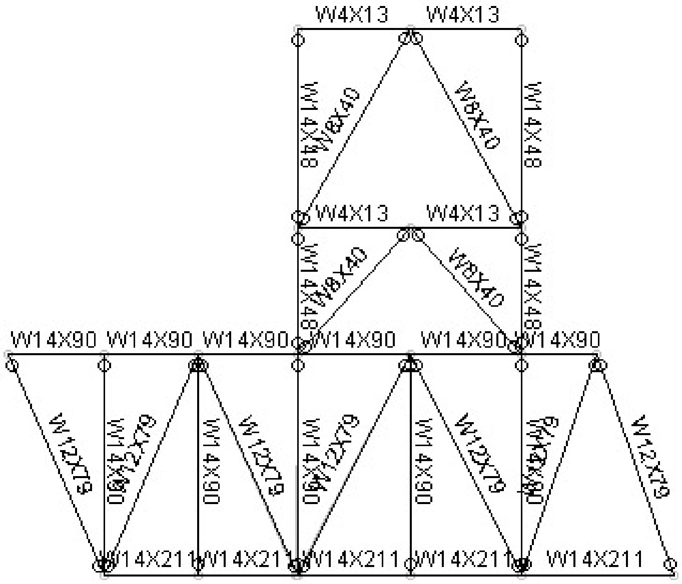

Figure 11: East Elevation of Truss with Section Labels

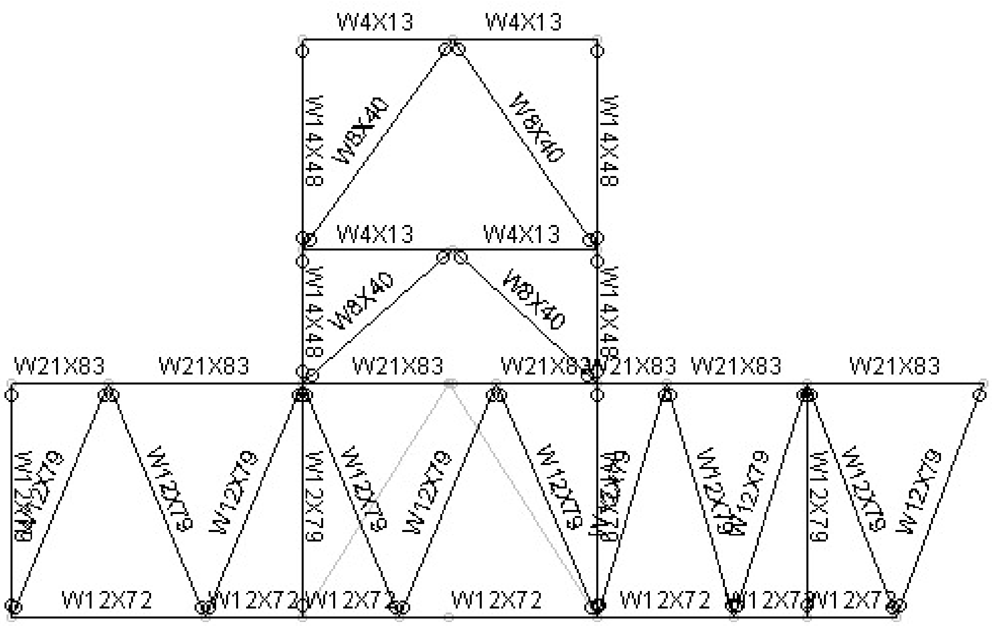

Figure 12: South Elevation of Truss with Section Labels

The analysis performed by RISA includes second order p-delta effects. RISA calculates these effects through an exact p-delta analysis using the stiffness matrix. Through this analysis, joint deflections were determined. A deflected shape diagram is provided in figure 13. The critical load combination for joint deflection at the top of the braced frames is the dead, live, and wind load combination. The maximum joint deflection at the top of the braced frames is 0.409 inches. This is equal to about h/600, well below the h/400 common rule of thumb for deflection. Since the braced frames are not supporting an area that will be inhabited by people, the drift limitation is not as critical for comfort. However, it is still important to limit drift in order to prevent dangerous interaction between flexible frames and a rigid cladding system. All drifts calculated by RISA are acceptable.

In sizing the steel members, RISA analyzes the system for the specified sections and then provides suggested alternate shapes based on minimum weight. The size of the shapes was economized according to RISA's suggestion with the exception of areas where consistency with the rest of the design was more important than using the absolute lightest weight member. For instance, the columns for the braced frame are larger than necessary in order to maintain consistency with the vertical truss members below. RISA also performs a steel code check in which all members are compared with either equation H1-1a or H1-1b from the LRFD handbook, depending on the ratio of the required axial strength to the factored nominal strength. If a member fails to meet the requirements of these equations it is automatically highlighted in the steel code checks output. All members were adjusted until every location passed the code check equation.

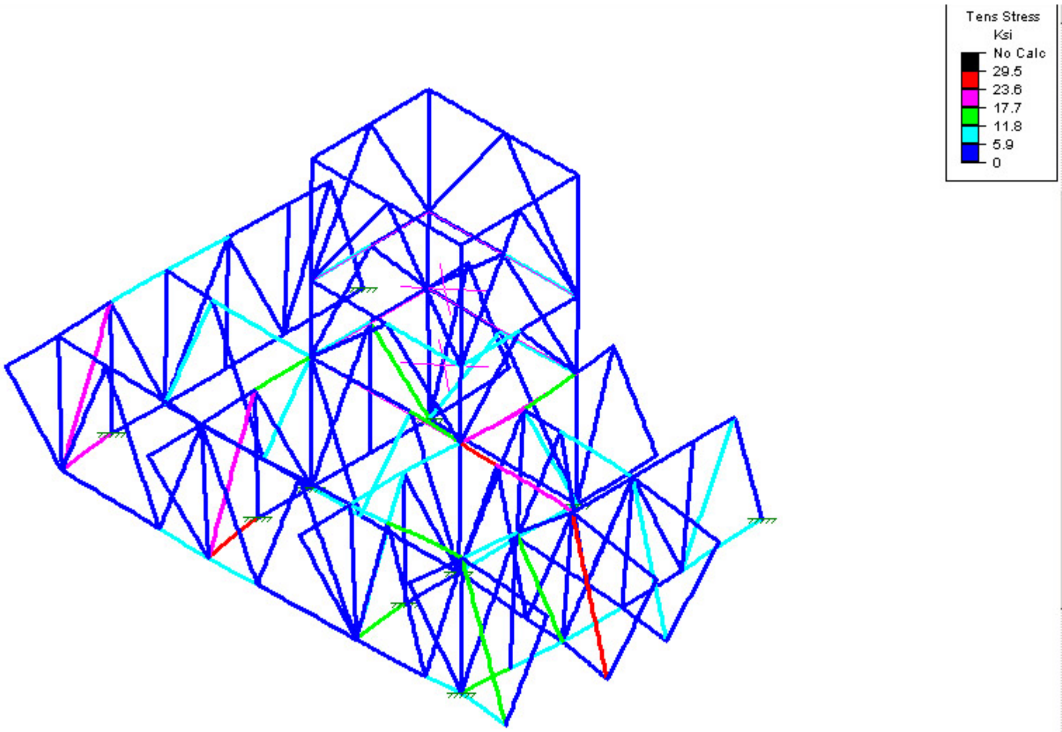

Figure 13: Deflected Shape Showing Tension in Members

Truss Connections

Due to the massive size of this truss system, it will need to be erected on site. Heavy bracing connections must be used since all truss members are large wide flange shapes. A sample connection is shown but not sized in figure 14. This connection occurs at the column eighteen support. Connections between the bottom truss chord and the concrete column or shear wall below will be accomplished through base plates and anchor bolts. It is important that all points of connection are leveled so that additional stresses are not introduced as a result of uneven supports.

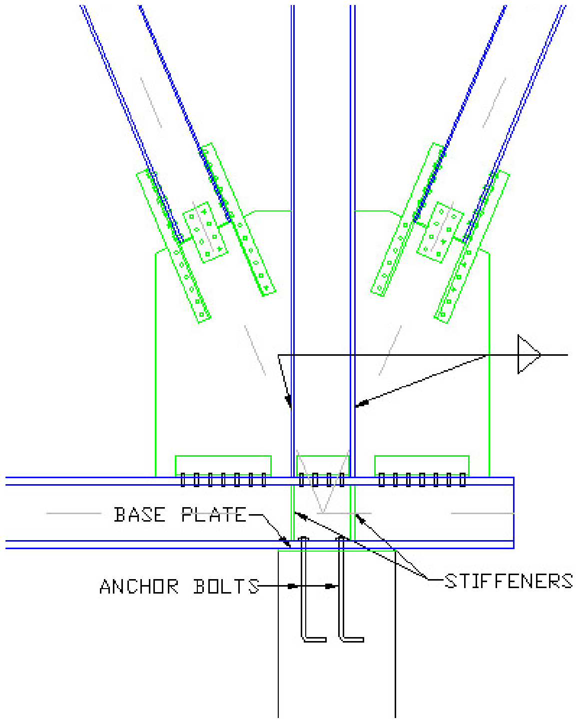

Figure 14: Typical truss connection detail

Similar heavy bracing type connections to the one shown in figure 14 will occur at several locations where truss web members frame into the top of the wide flange bottom truss chord. Stiffeners will most likely be necessary to prevent local buckling.

Tension Members

Wide flange shapes will be hung from the truss to support the cladding at each level. Since the support is from above, the sections will need to be largest at the top instead the bottom as in regular columns. All tension members are sized as W14x68, W14x34, or W14x22 shapes. Cladding load spreadsheets in Appendix B show the loads at each level of each member and which of the three steel shapes is necessary at that location. Where necessary, tension splices such as the one shown in figure 15 will

be used. A sample connection was designed for a W14x34 connection. In this case, all plates will be ¾ inch thick A36 steel. Eight A325-N 7/8 inch bolts will be needed at each web and flange. This connection develops the full tensile strength of a W14x34 member. This way the same connection can be used at every location where a W14x34 member is spliced to another shape that is the same size or larger. All edge distances are 1 ¼ inches, and spacing between bolts is three inches. The web plate has a six inch gauge and the two flange plates each have a gauge of four inches.

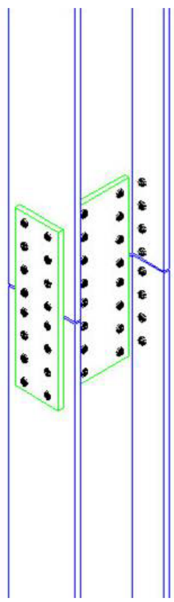

Figure 15: Bolted tension splice

Hanger Connections

A typical hanger connection was designed for the tension members that support the cladding system. The critical case for this connection is the tension member that will carry 825 kips at the roof. Therefore, figure 16 details the connection at the 825 kip load. This connection begins with a 1 1/4 inch plate welded to the lower chord of the truss. The plate will be welded directly below the beam web to prevent local flange failure at the points of connection. An additional plate will be welded to each side of the first plate and each side of the column web. These

connector plates can only be as wide as the depth of the column being connected allows. Therefore, in order to develop the necessary yield strength, they need to be 1 ½ inches thick. To compensate for the variance between the thickness of the column web and the plate attached to the truss chord, an additional ¾ inch plate will be welded to the column web. This plate will fill the gap between the web and the connector plate. This extra plate will be just enough larger than the area of contact of the connector plate to allow room for welds. Figure 16(d) shows the connection without the connector plate so that this extra web plate is visible. All welds are specified as full penetration in order to develop the necessary connection strength for the critical connection.

Connections similar to this one occur at the top of every tension column when it frames into the wide flange bottom chord of the truss above. The lightest load at one of these hanger connections is 212 kips. The connection can be considerably reduced in locations of much lighter loads. However, it is economical to maintain some consistency in all similar connections.

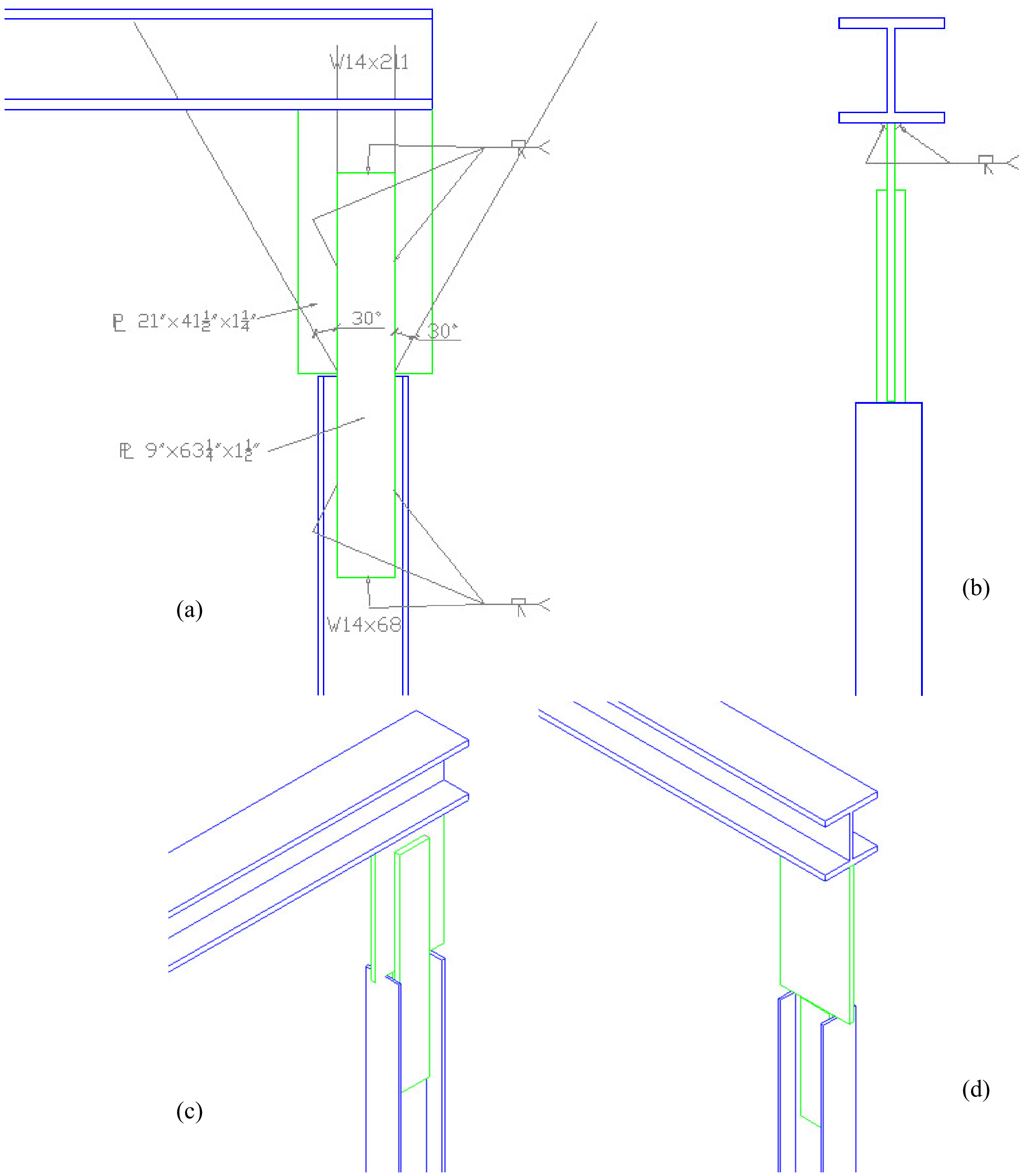

Figure 16 (a)-(d): Hanger Connection
(a)-(c) Views of the 825k hanger connection from different angles
(d) Hanger connection with connector plate omitted to show web plate

Slabs and Column Walk

Slabs will be poured before the tension members are connected. Therefore, the slabs will support their full weight, as was the case in the original design. Before cladding is added, the tension members will be hung from above and laterally braced to the slab. Architectural precast panels will span horizontally between the tension members and will not interact with the slabs. Therefore, no additional reinforcement will be needed in the slabs when the heavier cladding loads are applied. The cantilevered slabs will no longer need to provide support for the columns at the extreme edges. This could lead to a decrease in reinforcement in the cantilevers. However, this decrease was not calculated since the slab will still be sufficient as designed. A typical connection between the column and cladding at floor slab level is shown in figure 17. The wide flange tension members will be continuous through the slab. Lateral ties similar to those used to tie back masonry construction will be used to tie the cladding to the slab. No gravity loads will be transferred to the slab. These ties will only transfer lateral load.

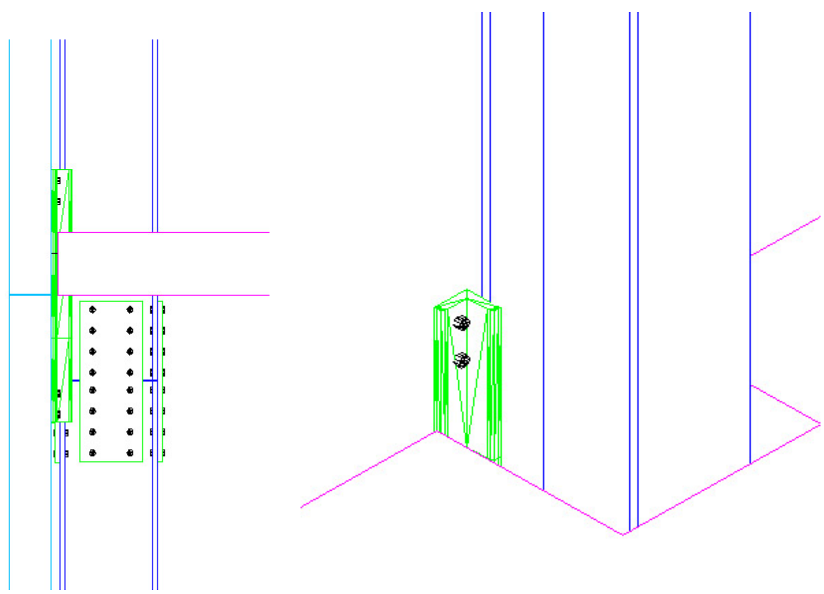

Figure 17: Panel to column connection at slab

Although the slab will not be required to directly carry any load from the precast panels, the slab on floor eight will still be affected by the increased load. The column walk described in the existing structural conditions section transfers a tensile force to the slab at the top floor of the column walk, floor eight. This tension is proportional to the axial compression in the column. When the axial load in the column is increased by the truss reaction and the cladding supported directly by that column, the tension in the slab increases at the same rate. The new tension in the slab at level eight is approximately 938 kips of unfactored load. The allowable stress in one of the 1 3/8 inch Dywdags used for post tensioning is 180 kips. The original design calls for five of these Dywdags, which can carry a total of 900 kips. Therefore, the new design calls for one additional strand at 180 kips for a total capacity of 1080 kips. The axial load in the sloped portion of the column was checked in compression and is sufficient.

Columns

All columns carrying more load than they supported in the initial building design were checked at critical locations under the new design loads. This includes all columns supporting the truss. Some columns that will not support the truss will still experience greater loads with the new design as well. These are exterior columns that will be directly subjected to the increased cladding loads.

Columns supporting the truss were modeled as fixed connections in RISA. The output provided reactions at each support point that were then incorporated into the column load calculations. Columns supporting the truss system were checked to ensure that they could withstand the extra loading as well as the biaxial moments imposed by the eccentric loading. From an architectural as well as a construction point of view it was preferred to maintain some symmetry in the trusses, even though the columns supporting the system are not aligned symmetrically. Therefore, the

truss supports are located wherever a truss node can be aligned with a column, even if it can not be centered on the column. This causes varying amounts of eccentricity in the columns, leading to biaxial moments.

Only moments due to the truss load eccentricities were incorporated into the column checks. For a more accurate and conservative analysis, unbalanced moments at the slabs must also be considered in these checks. Finding these moments in the asymmetric frames of this building would be a complicated process without the assistance of a program which performs a finite analysis. Therefore, these effects were ignored, but it is acknowledged that they would be an important consideration before construction of this system. Additionally, moments from the portion of the wind shears resisted by the column in question should also be considered. The majority of wind forces are resisted by shear walls, so the moments from wind on these columns should be minimal.

Since the initial lateral analysis of the building determined that the column frames are responsible for taking some of the lateral loads, the columns must be assumed to be in a sway frame. Therefore, the maximum slenderness ratio where slenderness can be neglected is 22, according to ACI 318-02. The maximum slenderness ratio in any of the columns checked is 30 in the top double height floor of column four. Column four must be checked for slenderness effects all the way down to the ground floor, where the column dimensions increase enough that the slenderness ratio drops below 22. Column five is the only other column in which slenderness is considered. It has a slenderness ratio of 48 at its critical location.

PCA Column was used to check column capacities at critical locations. The locations that were checked in each column are highlighted on the column spreadsheets in the appendix. In the locations where slenderness is a consideration, slab spans and depths were used in place of beam dimensions in order to find the rigidity in the top and bottom of the column. The width and clear span of these slab beams was estimated due to the inconsistency of column lines. All loads for non-

slender columns were multiplied by a factor of 1.08 to account for the change in phi factors between ACI 318-89 and ACI 318-02. The version of PCA column used references ACI 318-89, while the load factors used are those from the 2002 code. For slender columns, service loads were used in conjunction with the new set of factors which were input manually.

As a result of these checks, several columns must be redesigned. Some simply require a change in reinforcement, whereas alterations must be made to the geometry of others. Increasing column dimensions is avoided wherever possible, but in some places it is necessary. The original and new column schedules for only critical locations of those columns changing dimensions are shown in figure 18. All information that changed is shown in red on the new schedule.

Original Column Schedule

Level					
Roof	24x14 6#9 #3@12"	14x20 4#9 #3@12"	26x16 6#9 #3@12"	26x16 6#9 #3@12	14x36 12#9 #4@12
25	24x14 6#9 #3@12"	14x20 4#9 #3@12"	26x16 8#9 #3@12"	26x16 10#9 #3@12"	14x36 10#9 #4@12
21	24x14 4#9 #3@12"	14x20 4#9 #3@12"	30x18 6#9 #3@12"	30x20 8#9 #3@12"	14x44 12#9 #4@12"
9	24x14 6#9 #3@12"	14x20 6#9 #3@12"	30x18 6#11 #4@12"	30x20 8#11 #4@12"	14x44 10#9 #4@12"
Cellar	TYPICAL 6#11 #4@12	TYPICAL 6#11 #4@12"	TYPICAL 8#11 #4@12"		TYPICAL 16#11 #4@12"
	C1	C4	C9	C18	B2

Revised Column Schedule

Level					
Roof	24x14 6#9 #4@4" Added #4 Tie	16x20 6#9 #4@3" Added #4 Tie	30x18 6#9 #3@4"	32x24 6#9 #4@4"	14x42 10#9 #4@12
25	24x14 6#9 #3@12"	16X20 6#9 #3@12"	30x18 6#9 #3@12"	32x24 8#9 #3@12"	14x42 10#9 #4@12
21	26x18 8#11 #3@12"	16x20 6#9 #3@12"	30x18 10#9 #3@12"	32x24 12#11 #3@12"	14x44 12#9 #4@12"
9	26x18 6#9 #3@12"	16x20 6#9 #3@12"	30x18 6#11 #4@12"	32x24 8#11 #4@12"	14x44 10#9 #4@12"
Cellar	TYPICAL 6#11 #4@12"	TYPICAL 6#11 #4@12"	TYPICAL 8#11 #4@12"		TYPICAL 16#11 #4@12"

Figure 18: Original and new column schedules for critical columns and locations

Some of the truss supports will experience tension with the new design. A schematic drawing in figure 19 displays how the members can be stressed in tension. The particular truss shown in the figure is supported by bulb 2 and a shear wall. In this case, the bulb and the right side of the shear wall are in compression, but the left side of the shear wall is in tension. This creates an overturning moment that is resisted by the entire shear wall core. Flexural checks for the shear wall core are discussed in the overturning moment section of this report.

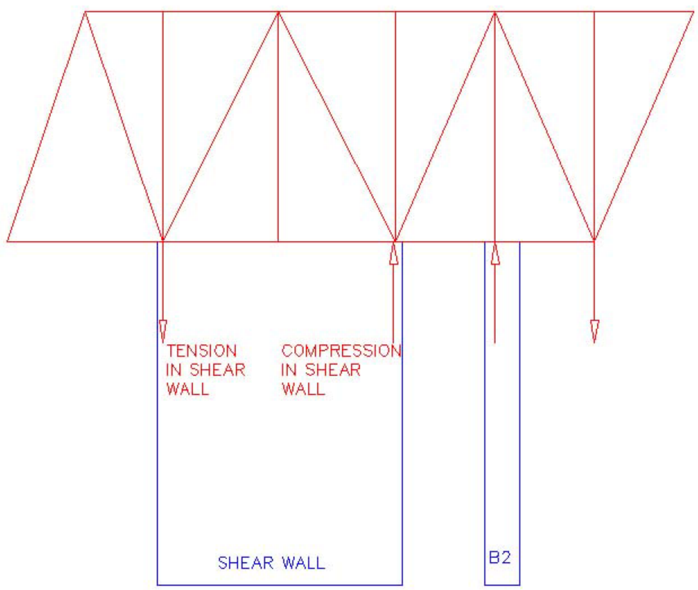

Figure 19: Schematic of vertical support reactions

Column four will also experience tension in the dead load only load case. However, it will only be in tension until enough dead load accumulates to counteract the uplift force from the truss. The maximum design tension in this column is 28 kips. The steel reinforcement in the column can withstand this tension, but extra care must be taken to ensure that the appropriate lap lengths for tension members are used.

Lateral Loads

Adding weight to the building through the heavy cladding and the roof truss affects seismic loads. Original and new seismic calculations can be found in the Appendix A and B, respectively. Since the design shears were originally governed by wind forces, and the same holds true for the new design, shear wall capacities are only checked to ensure that they can carry the extra loads imposed by the truss connections.

The lateral reactions at the truss connections result from the wind forces on the penthouse above roof level. The loads will be resisted at the support points of the truss. It is assumed that the connections will transfer the loads directly to the columns at roof level through the anchor bolt connections. However, once the lateral loads reach the floor slab at level thirty, they will be distributed to columns and shear walls based on stiffness. The assumption is that the thirtieth floor slab will act as a rigid diaphragm to redistribute the loads to the shear walls and frames. Therefore, below floor thirty, the story shears from lateral forces will be distributed exactly as they were in the original building design. For this reason, shear forces in the walls and columns were only checked at the top level.

The shear walls directly below the roof were checked for the design wind loads distributed by stiffness plus lateral reactions from the trusses at the top level of the wall. In areas were uplift forces create tension in the shear walls, the reinforcement is assumed to take the full shear load. The horizontal reinforcement in shear walls five and six must be increased from number four bars at twelve inches to number four bars at six inches for this reason. All other shear walls can resist the combination of shears from wind loads and truss reactions as originally designed.

Since column four is subjected to axial tension, the shear reinforcement must be able to take the entire shear force at the top of the column. Therefore, in the area of the column that is under tension, the ties must be increased from number three ties at twelve inches to number four ties at four inches on center with an extra tie in the

middle. The extra reinforcement should be continued until the column accumulates enough dead load to become in compression, which occurs at floor twenty-nine. In order to accommodate the additional shear reinforcement, the column must be increased by two inches in the smaller dimension and vertical reinforcement should be changed from four number nines to six number nine bars. The increase in vertical reinforcement is necessary to hold the extra middle tie in place.

Column one will also require additional shear reinforcement due to the horizontal reactions from the trusses. Rather than the original number three ties at twelve inches on center, it will require number four ties at nine inches on center. The changes in shear reinforcement are reflected in the column schedules in figure 18. Calculations for column shear reinforcement due to truss reactions were done by hand and are not included in this report.

The two truss connections at wall seven transfer lateral forces to the wall in opposing directions. The same is true of wall eight. This creates horizontal tension in the shear walls. Horizontal reinforcement was checked to ensure that it can carry the tension loads. A fracture plane of 45 degrees to the horizontal was assumed in order to determine the area of reinforcement that will contribute to the tensile resistance. Based on this assumption, the spacing of the reinforcement must be cut in half for the tension area of wall seven. This means that for the top thirteen feet of shear wall seven, number four bars at six inches on center with tension splice will be needed. Wall eight will need tension splices for the top thirteen feet, but the area of reinforcement is sufficient as designed. Below thirteen feet from the top, both walls can return to their original design horizontal reinforcement.

Overturning

Hanging the cantilevers on only one side of the building creates overturning moments that will add to the overturning moments previously calculated for wind forces alone. The critical direction for overturning moments is about the east-west

axis. This is the direction with the higher moments due to both wind and truss loading. It is also the direction with the smaller lever arm for the self weight to act along when resisting overturning moments. Overturning was checked in this direction for the two load cases involving wind loads as specified in ACI 318-02.

1. 1.2D+1.0L+1.6W
2. 0.9D+1.6W

The building dead load was conservatively assumed to act at the midpoint of the upper floors of the building. This point is closer to the point about which overturning would occur than the midpoint of the lower floors is. The second load case is critical with a factor of safety of 2.54. The first load case is critical for bearing failure with a 4.3 factor of safety. The bearing capacity of the soil is 40,000 psf.

The shear wall core of the building experiences a moment of its own. Shear wall flexural checks were used to determine whether or not the shear walls can resist the moment created by compression and tension forces from truss reactions. Overturning moments from wind were resolved into compressive and tensile forces on opposite sides of the shear wall core. These forces were then combined with the reactions at truss supports calculated by RISA and the compressive forces from the column load take down. Wind overturning moments were assumed to act in the same directions as the moments from the truss reactions in order to find the critical loading. Load cases one and two mentioned above were also compared to find the critical loading. These checks were performed with the aid of spreadsheets that can be found in Appendix B. Wall capacities were compared with imposed loads at every level and were determined to be in accordance with ACI 318-02.

ARCHITECTURE

New York Architecture

New York is known world wide for its crowded streets and abundance of sky scrapers. Much of Midtown and Lower Manhattan is lined with very tall buildings. However, mixed in with all of these notably tall and unique buildings are residential areas lined with low rise apartment buildings. An example of this in Greenwich Village is shown in figure 20(a). The combination of commercial areas filled with sky scrapers such as Times Square shown in figure 20(b) and smaller scale residential areas is part of what gives New York its unique personality.

Figure 20(a)-(b): Contrast between Greenwich Village (left) and Times Square (right)
(Photos taken from www.wirednewyork.com)

Current Building Architecture

The Aurora is a thirty story residential building in New York City. Most of the surrounding buildings are low-rise apartment buildings with retail space on the ground floor. The Aurora maintains the theme of residential space above ground floor retail space, but it dwarfs the adjacent buildings. Although the height of the building is somewhat intrusive, the exterior finishes attempt to blend into the neighborhood. The Aurora is a tall building compared to the buildings immediately surrounding it, but it is far smaller than any of the notably tall buildings in the city.

It also lacks any boldly unique qualities such as those in many commercial buildings. Therefore, it does very little to distinguish itself from the rest of the buildings in the area.

Figure 21: Rendering of the original design for The Aurora

Cladding

The structural changes that will be made to The Aurora will remove the restrictions to the architectural decision of a cladding system. The system will no longer need to be a lightweight metal system. Instead, a much heavier architectural precast panel system will work. The color combination chosen for the original metal panel system provides a transition between the brightly colored red and burgundy four story apartment buildings to the south and the beige high rise building across Thirty-Seventh Street to the north. The exterior architecture of the building represents an attempt to blend into the surroundings unobtrusively.

Architecture

Figure 22: Construction photo showing low rise brick buildings in foreground and beige high-rise in the background

It is important to maintain this idea of fitting into the neighborhood in order to maintain a balance. Therefore, the pattern chosen for the new cladding system takes the colors of the surrounding buildings and mixes them to give the building its own identity. A sample of the panels chosen from Architectural Precast Association is shown in figure 23. In addition to the red and tan shades taken from the surrounding buildings, this cladding incorporates a shade of yellow. The yellow helps distinguish the building from those surrounding it without overstating itself. It also incorporates the name of the building into the overall design. The name Aurora is associated with light. The yellow will tie in the idea that the tall slender building represents a beam of light reaching toward the sky. However this will remain a subtle idea while the building fits unobtrusively into the neighborhood. An additional benefit of having the darker colors mixed into the panels is that the panels are dark enough not show dirt as easily some colors. This will make the building easier to maintain.

Figure 23: Sample panel finish from Architectural Precast Association

Truss Design

The roof trusses designed as structural members will be exposed as architectural elements. The trusses span two directions and overhang the building on the north and west sides. Areas where the trusses overhang the building coincide with the cantilevered sections. In this way, the trusses emphasize the cantilevered design of the building. At the same time, they give more definition to the building and prevent it from disappearing into the crowded New York City skyline. Similar approaches have been used successfully in other high rise buildings. A prime example of this is the IBM building in Baltimore Maryland, designed by Skidmore Owings and Merrill. This building is shown in figure 24.

Figure 24: IBM Building, Baltimore Maryland
(Photo taken from www.som.com)

One concern with putting architectural trusses on such a tall building is whether or not they will actually be visible from street level. The drawings in figure 25 demonstrate how the trusses will appear from various locations along the street. Not only will the trusses be visible, but they will draw attention away from the somewhat awkward tall and slender penthouse on the roof.

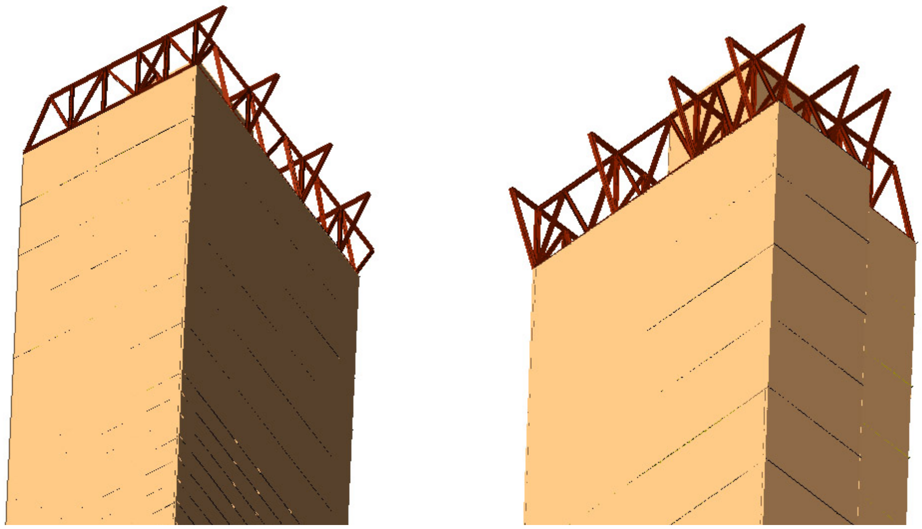

Figure 25: Trusses will be visible from street level

Interruptions to Floor Layout

As stated in the design criteria, floor layouts are very tight, making any changes to column sizes difficult. The structural design changes call for some increased column dimensions in certain areas due to the heavy truss and cladding loads. The areas of the floor plan where column dimensions increase must be analyzed to ensure that the columns do not disrupt the floor layouts. A floor plan is shown in figure 26 with the new column sizes. As a result of the changes, some of the windows on the west side must be shifted slightly. This is not expected to significantly impact the exterior appearance of the building. All floors above and below this floor allow room for the window to shift to the new location shown on the floor plan below. Therefore, the windows will still line up vertically, even with the proposed shift. The original column sizes were left on the plan for comparison with the new sizes outlined in green. Areas where changes occur were clouded for clarity.

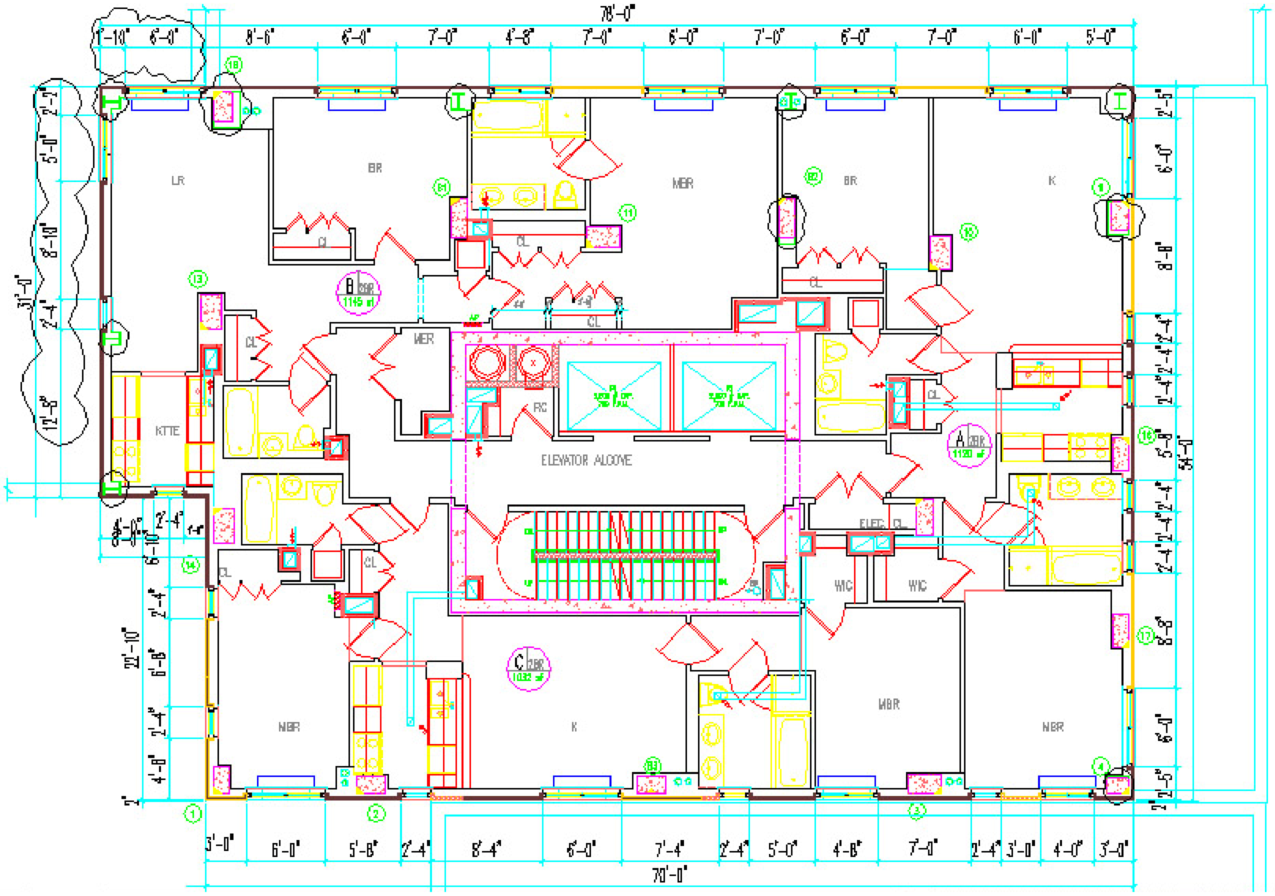

Figure 26: Floor twenty-five layout with column size changes

Exterior Architecture

The final product of the exterior architectural changes proposed is demonstrated in the before and after renderings in figure 27. The combination of the roof trusses and the cladding change give the building a very different overall appearance. Removing the strong vertical and horizontal lines apparent in the initial design makes a large difference in the apparent dimensions of the building. The actual dimensions did not change at all, but the new design looks much less slender. The lack of strong vertical lines and the horizontal roof truss act together to give the building a wider appearance on the same slender site.

The cladding style and the use of large roof trusses set The Aurora apart from its neighbors. However, at the same time the colors chosen for the both the siding and the truss blend well with the environment of the building.

Figure 27: Renderings of the building before (left) and after (right) proposed changes
(Rendering courtesy of H. Thomas O'Hara Architects)

CONSTRUCTION MANAGEMENT

Schedule Background

The excavation process revealed some unexpected encroaching foundation walls from adjacent existing buildings. This meant that the structural engineers needed to make revisions to the design documents after the site work phase had begun. This unexpected obstacle meant that the design phase needed to extend past the site work phase. A primavera schedule of 554-556 Third Avenue is shown in figure28(a). This schedule reflects the dates for design and major construction phases.

A crane was needed for both the cast in place concrete and the metal panel siding. Therefore, the enclosure could not begin until the concrete pouring was complete. If this building had a more typical brick façade, enclosures could have begun before the superstructure was complete. The start date for the enclosures would only have to lag the start date of the superstructure by a couple of weeks. This would allow enough time for a few stories to be poured before the bricklaying began.

Sequencing Issues

One of the initial intents of the new design and cladding system was to save some time on the schedule. The idea was to offer more overlap time by changing the cladding to a brick system. Since a brick façade would not require a crane to erect, the enclosure phase could begin before concrete pouring was complete. Theoretically, the masons could follow closely behind the concrete, just staying a few floors below where the concrete pouring was taking place. This would allow earlier enclosure of at least some of the building so that more tasks could overlap. However, since the new support system holds the siding from above, the truss and steel tension supports must be in place before the cladding is attached. Therefore, until the concrete and steel construction is complete, even a brick cladding system could only be attached up to floor six or on the non-cantilevered sides. If the masonry work cannot begin until the

entire structure is complete, changing to a brick cladding system would only lengthen the project duration.

Additionally, the brick system would have nothing to support it at the east edge of floors six and seven. The setback occurs at floor eight, so the east edge of floors six and seven is not directly below the roof trusses. Therefore, no tension member will be located at the northeast corner of these floors. In order for the new system to support the cladding at this corner, the cladding must be able to cantilever ten feet, the length of the setback. A shelf angle supporting brick cannot support this kind of cantilever. However, the architectural precast system can support itself for this distance. Therefore, the precast panels will be more practical as well as faster and easier to erect.

Schedule Impact

Activity durations for the additional construction proposed for 554-556 Third Avenue were estimated using the MC^2 estimating program. According to this estimate, the steel erection should take approximately 1.5 months. This time will have to be added to the overall schedule since enclosures cannot begin until the system supporting the cladding system is in place. The precast concrete cladding system should take 77 days to install, while the metal system currently in place was estimated at 81 days. The actual duration for this task was 150 days, much longer than the estimate indicated. The difference between the two estimates is minimal in comparison with the difference between the metal estimate and actual duration. Therefore, it is assumed that the concrete system will take approximately the same amount of time to install as it took to install the metal system.

Figure 28(a) and (b) shows some of the key milestone dates for the actual building construction and how these dates would change as a result of the proposed changes. The overall impact to the schedule is the duration of the structural steel erection, approximately a month and a half.

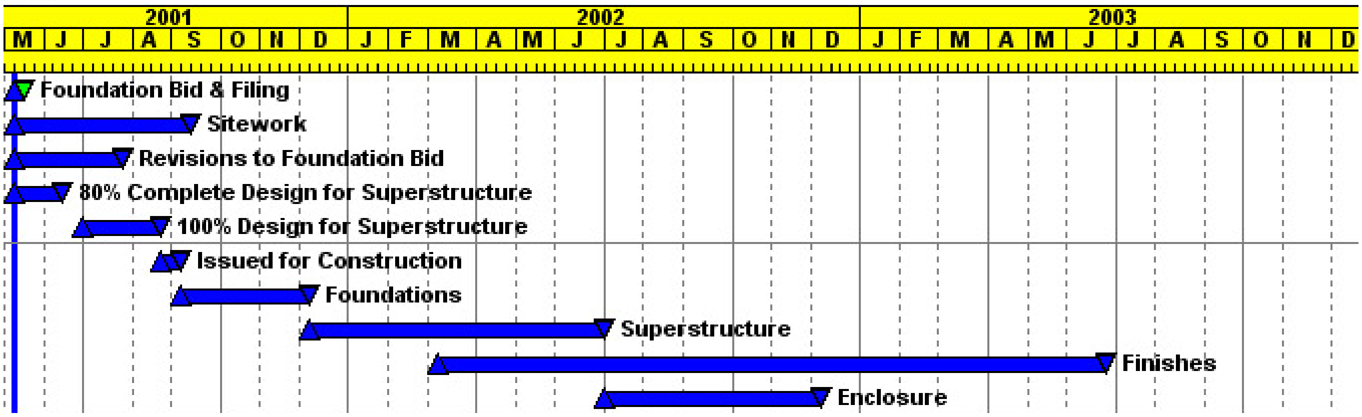

Figure 28(a): Current Schedule

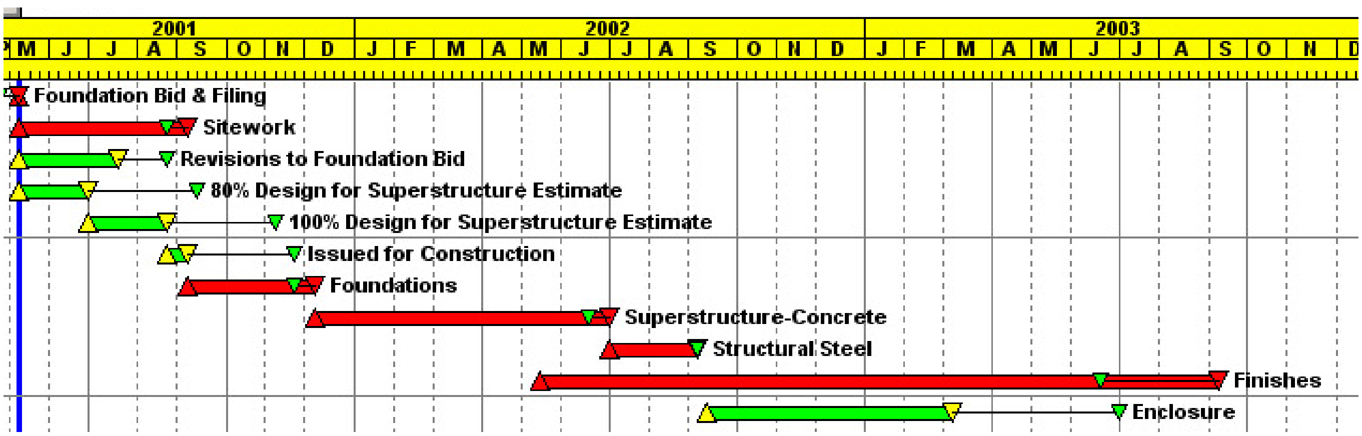

Figure28 (b): Schedule with Proposed Changes

Site Logistics

Construction of The Aurora is already complicated due to the tight site restrictions. Adding a large truss to the roof will add more obstacles to the site, but the work will still be feasible. The biggest issue to overcome on the site of 554-556 Third Avenue is the lack of staging area. There is no room on site for the steel shake-out. Therefore, steel will need to be sorted off site and brought to the site in the order of erection. This requires more coordination with the fabricator, but it is not uncommon for projects such as this in the middle of a large city. A lane of Third Avenue will need to be reserved for the delivery trucks supplying the steel. The

tower crane is already oriented in such a way that it blocks a lane of traffic, so the delivery trucks should be able to pull along the side of Third Avenue behind the crane. Additionally, concrete trucks were needed for the placement of the concrete, but they will all be gone by the time the steel erection begins. Trucks carrying the steel can occupy area along the curb where concrete trucks parked during the earlier stages of construction. Shop owners will need to be notified of the street blockage.

The actual capacity of the crane already on site is eight tons. This is the capacity at the appropriate reach and angle as provided by the construction manager on site. The largest piece of steel to be lifted is a 40 foot long W14x211. That gives the member a total weight of about 4.7 tons, well below the maximum crane capacity. Therefore, the crane already on site can be kept for an extra month and a half and used for the steel erection.

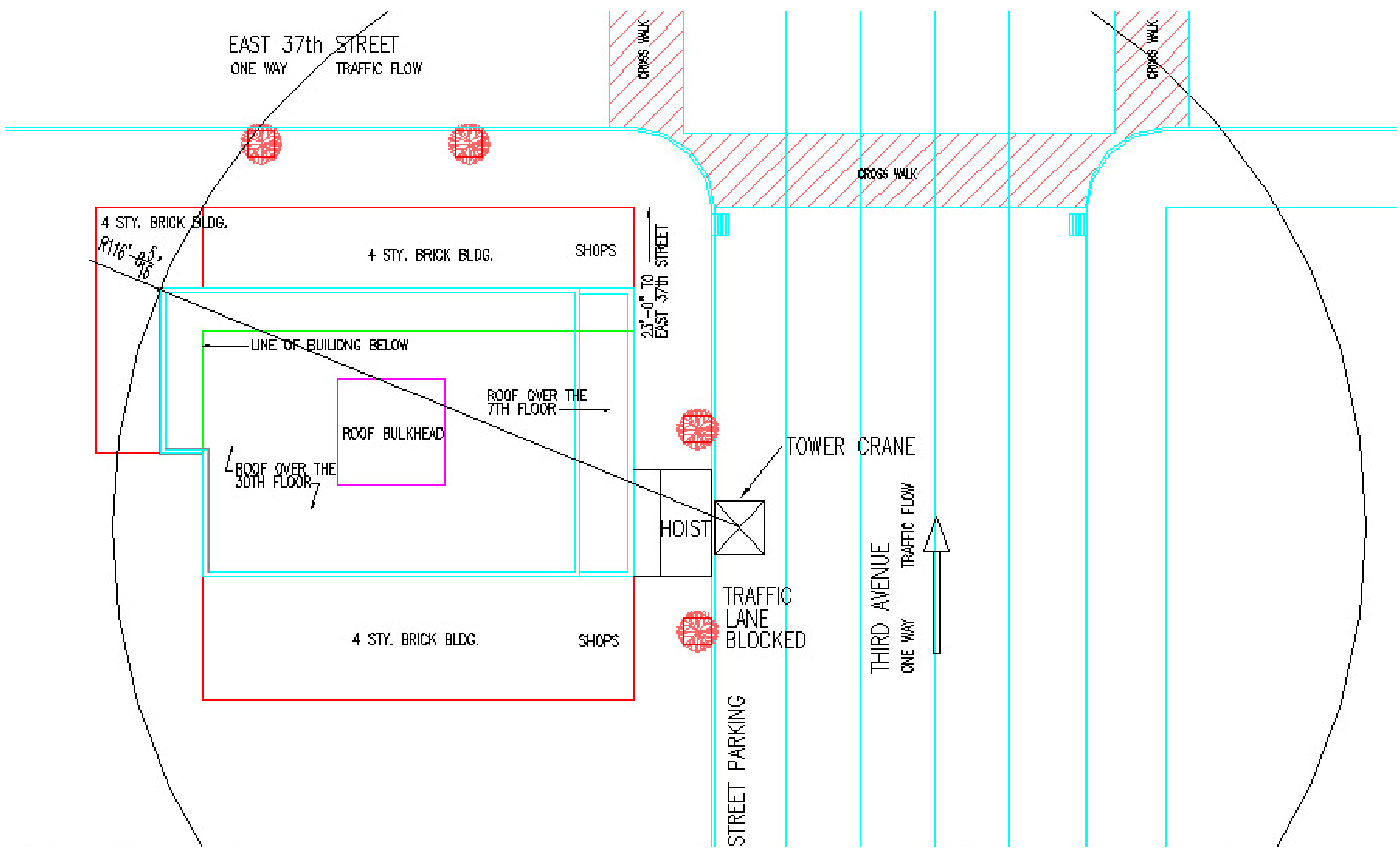

Figure 29: Site Layout

Cost Background

Actual costs for construction of The Aurora were not released. Therefore, an average of the cost from a D4 Cost 2002 analysis and an R.S. Means estimate will be assumed for comparison purposes. The average of these two estimates yields an approximate building cost of $17.5 million.

The D4 computer program was first used to determine an approximate cost for 554-556 Third Avenue based on the known costs for projects with similar sizes and occupancy types to this building. The costs were then adjusted to fit the actual square footage of the building and the New York City location. The final cost estimated by D4 was $16.7 million. R.S. Means was used for another square footage estimate. The base cost was $115.09 for a reinforced concrete frame. This cost was adjusted for a 239 ft perimeter and an average story height of 12.46 ft. It was then multiplied by the total square footage of 120,639 ft^2. An additional cost was added for the basement, and then the overall cost was multiplied by a location factor of 1.35 for New York City. The final cost produced by this estimate was $18.2 million.

Added Costs

The main additional costs will result from the added steel and precast architectural panels. Additional costs are also likely to result from site issues, but they will not be considered here. One such cost is maintaining the Third Avenue road closure for the extra month an a half that will be added to the schedule.

Overall costs for the added steel were estimated using R.S. Means Construction Cost Data 2003 and were then adjusted for 2002 since that is the year when the truss erection would have taken place had it been part of the original design. This estimate priced the steel addition at $270,000. The Means estimate took into account the location of the project and the year of the work. A similar estimate was made with the aid of the computer program MC^2. This computer program estimated the overall cost of the steel addition at $219,000. The estimated cost of structural steel

that will be added to the building is approximately 1.5 percent of the total estimated cost before the addition.

Costs for metal panel siding and precast architectural panels were also estimated and compared. Both costs were estimated using the MC^2 estimating program. For the 82,000 square foot perimeter of the building to be covered in the selected cladding system, the cost of the metal panel system is $962,000. The cost to clad the same surface area in six inch precast architectural panels was estimated at $2,235,000. This is an increase of $1,273,000, or 130 percent. The cost difference amounts to about 7.3 percent of the total initial estimated cost.

The overall cost impact of the structural steel and architectural precast panel siding is around $1.5 million. Breakdowns for these costs can be found in Appendix C. This translates to about 8.6 percent of the initial building cost.

SUMMARY AND CONCLUSIONS

As a solution to the limited options for a cladding system for 554-556 Third Avenue, a hanging support system for the cladding is suggested. The truss designed allows a heavy architectural precast panel system to be used to clad the building. The new system maintains the current floor area with minimal disruptions to the layout.

The truss system serves a dual purpose. In addition to supporting the exterior walls, it adds architectural interest to the exterior of the building. The combination of the new heavier cladding system and the series of exposed trusses on the roof give the building a much more substantial appearance. The truss will make the building more intriguing to pedestrians walking along Third Avenue. A more interesting building will entice more tenants, allowing the owner to charge more for the suites and condominiums within the building. If changing the architecture of the building increases the value of the real estate, it may be worthwhile for the owner to consider some costly alterations to the design.

Although the new system has architectural advantages, it is clear that such a system will complicate construction and add time to the schedule in addition to increasing the overall cost. Based on the location of the building, the owner can already expect to make a large profit due to the high price of apartment rentals in New York City. Delaying the opening of building may cause a greater initial loss in revenue than the owner is willing to accept.

Therefore, the recommendation is to use the original design with the lightweight metal panels. A Midtown Manhattan location will produce large revenue regardless of the exact finish on the building's exterior. Without changing the quality of interior spaces, rental rates are not likely to substantially increase. Unless the architect or owner has a strong desire for a finish that cannot be achieved with metal panels, the new design does not seem like a practical alternative.