The Towers at The City College of New York

New York City, NY

Robin Scaramastro

Structural Option

Spring 2007

Advisor: Dr. Ali Memari

The Towers at The City College of New York

130th and St. Nicholas Terrace, New York City NY

Project Team

Owner/Developer: Capstone Development
Architect of Record: Goshow Architects
Structural/MEP Engineer: Greenman-Pedersen, Inc.
Civil Engineer: Langan Engineering

Building Statistics

Gross Size: 185,000 SF
Total Cost: $59 million
Delivery Method: Guaranteed Maximum Price
Schedule: April 2005 - August 2006

Structural

8" reinforced flat plate concrete slab floor system
Reinforced concrete columns on a regular grid
Reinforced concrete shear walls around stair towers and core elevators
42" thick mat slab foundations

Electrical

120/208V 3 phase 4 wire service
Seven sets of 4#750 MCM in 5" conduits to 4000A service entrance switchboard
Emergency power provided by battery back-up for emergency lighting and fire alarms

Mechanical

Two gas-fired boilers, 3million BTU/hr each
Each apartment's HVAC is provided by packaged terminal air conditioning units and hot water heating coils
HVAC for corridors and lobbies is provided by roof top units with a direct expansion cooling system and gas fired heaters

Architecture

Light and dark brown brick reflects the terra cotta and local dark stone in existing buildings
Floor-to-ceiling glass bays accentuate the building's corners
Setbacks coincide with adjacent building heights

TABLE OF CONTENTS

1.0 - EXECUTIVE SUMMARY

The Towers at the City University of New York is a new residence hall for CUNY students and faculty. It is the first dormitory for the Manhattan college in its 185 year history. The building is located at 130th Street and Saint Nicholas Terrace in the upper west side of New York City. The 11 story building is capable of housing 600 CUNY students and faculty in 165 apartments. The total cost of development and construction of the Towers was $54 million. Some of the features of the 181,000 square foot building are fully furnished apartments with private bedrooms, a laundry room, a fitness room, classroom spaces, administrative offices, a reception desk that is operational 24 hours a day, and numerous lounge and study spaces. Ground was broken in May 2005 and was completed in August 2006.

This report consists of a detailed study of an alternate steel structural system. This was done to eliminate columns in the floor to ceiling corner windows at the corners of the building and to create a regular column grid. RAM Structural System and RAM Frame were utilized in the design of the structure. The final design consists of a composite steel beams and girders with 4 ½″ composite metal deck slab. The beams are W10 or W12 shaped and the girders were kept at a maximum depth of 21″ to keep a 2′ plenum depth. All gravity columns are W10 shapes and all lateral columns are W12 shapes for added stiffness. The lateral system consists of braced frames around all stair and elevator cores and moment frames elsewhere throughout the building.

Two breadth studies were performed to understand the impact of the proposed structure on the other building systems. The first was an analysis to determine if The Towers could be LEED Certified. The second study was to study the impact of cost and scheduling of the existing concrete structure and the proposed steel structure. It was determined that a steel structure would cost approximately $6.0 million and take 19 weeks to construct.

2.0 - BUILDING INFORMATION

The Towers at the City College of New York is a new residence hall for CUNY students and faculty. It is the first dormitory for the Manhattan college in its 185 year history. The 11 story building is capable of housing 600 CUNY students and faculty in 165 apartments. The total cost of development and construction of the Towers was $54 million. Some features of the 181,000 square foot building include fully furnished apartments with private bedrooms, a laundry room, a fitness room, classroom spaces, administrative offices, a reception desk that is operational 24 hours a day, and numerous lounge and study spaces. Ground was broken in May 2005 and the building was completed in August 2006.

Figure 1 – Location of The Towers
(photo courtesy of Mapquest.)

2.1 - PROJECT TEAM

Architect:
Goshow Architects
Design Consultant:
Design Collective
Structural / MEP Engineer:
Greenman-Pedersen, Inc.
Owner / Developer:
Capstone Development
Civil Engineer:
Langan Engineering
Construction Manager:
Turner Construction Company

2.2 - LOCATION AND ZONING

The building is located at 130th Street and Saint Nicholas Terrace in the upper west side of New York City as seen in Figure 1. According to the guidelines set in the Building Code of the City of New York, the zoning district for The Towers is Residential 7-A. This means that the height limitations for an apartment building are dictated by the sky plane with a maximum front wall height of 60'-0". Also, the footprint of the building can only cover a maximum of 70% of the lot.

2.3 - ARCHITECTURE

Goshow Architects decided to use natural brown brick colors to reflect the terra cotta and local dark stone of the existing buildings on campus and in the upper west side of Manhattan. The architects also made the setbacks of the Towers to somewhat reflect the heights of adjacent buildings in the neighborhood. The floor to ceiling glass bays are used to accentuate the corners of the building and give it a unique look. The L-shape of the building provides privacy and protection for the quadrangle facing the center of campus. The existing granite walls of the buildings that previously occupied the site will be incorporated into the landscaping.

The façade of the Towers is a thin brick panel system. The brick panels consist of the brick, thin set adhesive cement bed over metal lath, 5/8″ glas-mat sheathing and vapor barrier. This panel is connected to 6″ cold formed metal studs. The studs are insulated with R19 batt insulation. The roof system consists of a multi-ply bitumen roof membrane over tapered R19 rigid insulation. The slope of the insulation is equal to ¼″ per roof and ½″ per foot within 24″ of the roof drains. The structure of the roof is a 9 ½″ thick reinforced flat plate concrete slab.

2.4 - EXISTING STRUCTURAL SYSTEM

The structural system that was originally chosen The Towers is cast in place reinforced concrete columns and floor slabs. The slabs are a two-way flat plate system that directly transfer the floor loads to the columns. The penthouse consists of structural steel tube columns, wide flange beams and steel angle bracing.

2.3.1 - FOUNDATION

Figure 2: Existing foundation (photo courtesy GPI)

Based on the soil borings and the geotechnical report, a shallow foundation was permissible for The Towers. The soil report indicated that solid bedrock was beneath 6′ – 12′ of firm soils at the site. The slabs and spread footings sit directly on top of the bedrock with a bearing pressure of 40,000 psf. Matt slab foundations that range in thickness from 36″ to 42″ are used to support the loads from the concrete shear walls around the stair and elevator cores. The foundation walls are cast in place reinforced concrete atop spread footings. Rectangular spread footings up to 30″ in depth support the gravity load from the concrete columns.

2.3.2 - FRAMING

A cast-in-place concrete system was chosen for The Towers. Rectangular columns are laid out on an irregular grid and large concrete beams are used in the central lobby area of the building that connects the two wings. The beams also support the cantilevered portion of the building at the third floor over the main entrance. The floor slab is tied in to the columns by studrails at each face, and reinforcing bars over the column transfer the floor loads into the columns. The thin brick prefabricated panels that make up the façade of the building are also connected to the top of the slab with steel angles. Expansion joints are used at the edges of the slab where they meet with the exterior wall panels. 2″ seismic expansion joints are also used at the corners of the building.

Figure 3: Concrete framing (photo courtesy GPI)

The penthouse of the building is structural steel. Steel tube columns are used as the columns and W-shapes are used as beams. Bracing is provided by steel angles for the beam to column connections. The penthouse consists of two levels. The floor of the first level is the cast in place roof slab. The second floor framing consists of W24x55 beams. The roof of the penthouse is framed with W12x14 beams. The exterior girders that carry the floor and roof framing are connected to the columns with moment connections, and are additionally braced with steel angle knee bracing.

2.3.3 - FLOOR SLAB

The typical structural slab for all 11 stories of the Towers is a two way 8″ elevated flat plate concrete slab. The slab is reinforced with #4 bars at 12″ on center. Extra bars are provided at column locations for added resistance against shear forces. For the basement, a 4″ slab on grade was used. The slab on grade is reinforced with welded wire fabric and is cast over a vapor barrier and 4″ of a porous fill base. The floor system for the first level is the flat plate concrete slab. The floor system of the structural steel penthouse consists of a 4 ½″ concrete slab with metal deck.

2.3.4 - LATERAL FORCE RESISTING SYSTEM

Lateral loads imposed on the building are resisted by concrete shear walls located throughout the building. One wall is located in the north wing of the building, and the other walls are around the stair towers and elevator core. The typical structural layout in Figure 1 below illustrates the locations of columns and shear walls. The floor slab acts as a rigid diaphragm to transfer the toads to the lateral force resisting system. The shear walls are 10" thick and are reinforced with two curtains of rebar.

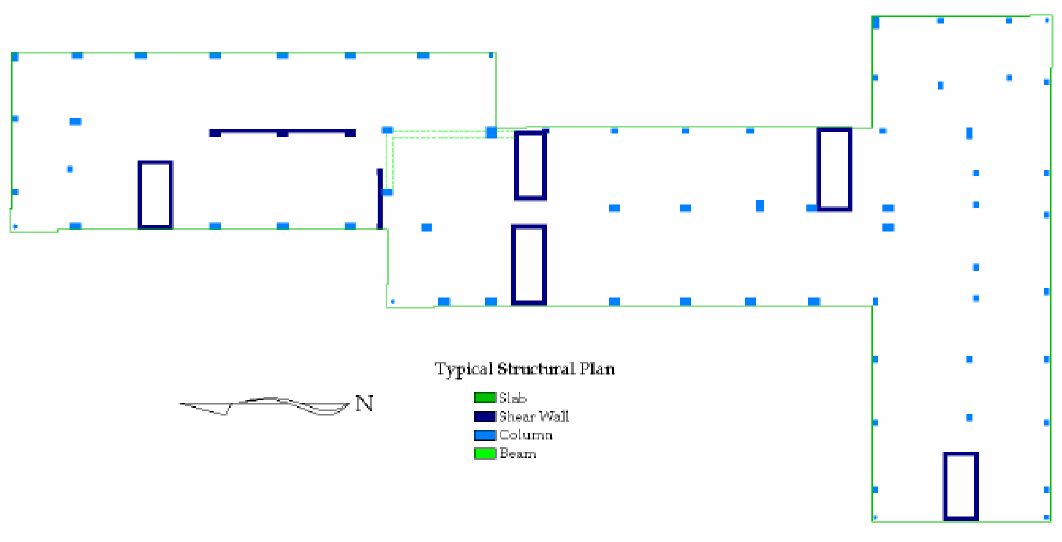


Figure 4: Typical structural plan of existing building

2.4 - OTHER BUILDING SYSTEMS

The electric power for The Towers is 120/208V, 3 phase 4 wire. Seven sets of 4 #750MCM cables are run from the service at 130th street to a 4000A service entrance located at the lower level of the building. Each apartment unit has its own electrical panel sized according to the New York City Electrical Code. There is no generator for emergency power for the building. Emergency lighting and fire alarms are powered by integral batteries inside the fixtures.

HVAC for the apartments is provided by packaged terminal air conditioning (or PTAC) units. The PTAC units use a 1/2 ton direct expansion cooling system and are located below each window. Heating is provided by a hot water heating coil which is served from the central hot water boiling plant.

3.0 - PROBLEM STATEMENT

Upon a site visit to The Towers, it was discovered that large, round concrete columns were placed in the corner windows of the apartments, obstructing any views of Saint Nicholas Park or downtown Manhattan. There are also many large, concrete columns placed throughout the building on an irregular grid, which may have impacted the ease of construction of The Towers. Some of these columns are obstructions in corridors and open spaces, which detract from the architecture of the building.

Figure 5: Photograph illustrating the concrete column obstructing the window (Photo courtesy of Robert Chin)

4.0 - PROPOSAL

A steel structure on a regular grid is proposed to eliminate the corner columns in the windows and make construction of the building more efficient. A composite steel deck will replace the flat plate concrete slab and steel braced frames and moment frames will replace the concrete shear walls. Using a steel frame will also cut down on the foundation loads which can decrease the required sizes for the spread footings. Although using steel will increase floor to floor height of the building, the zoning requirements per the Building Code of the City of New York allows for an increase of 3′ per floor.

The corner columns will be eliminated by cantilevering the beams supporting the portion of the building where the corner windows are located. By keeping a ceiling plenum depth of 2′, the floor to floor height will only be increased by 1′-4″ which only increases the total building height by approximately 13′.

5.0 - DESIGN CRITERIA

5.1 - DESIGN PROCEDURE

To determine the most appropriate layout for the proposed steel structure, trace paper will be placed over the existing architectural plans and several schematic layouts will be sketched. This is done to ensure that column lines will be in line with partition walls and do not interfere with any door or window openings and open spaces. The grid will be input into RAM Structural System and then the columns and beams will be laid out. Deflections for members will be limited to:

Dead: $\frac{L}{360}$

Total: $\frac{L}{240}$

$\frac{L}{400}$ (Cantilevers)

After the gravity framing system is in place, the lateral force resisting system will be laid out. Braced frames will used primarily around the stair and elevator cores where the frame was aligned with the interior wall. Moment frames with bracing kickers were used where door and window openings were located so that the architecture will not be impacted. Due to the plan irregularity of the building, the frames will be laid out to try to reduce the eccentricity between the center of rigidity and the center of mass of the building, thus reducing the torsional

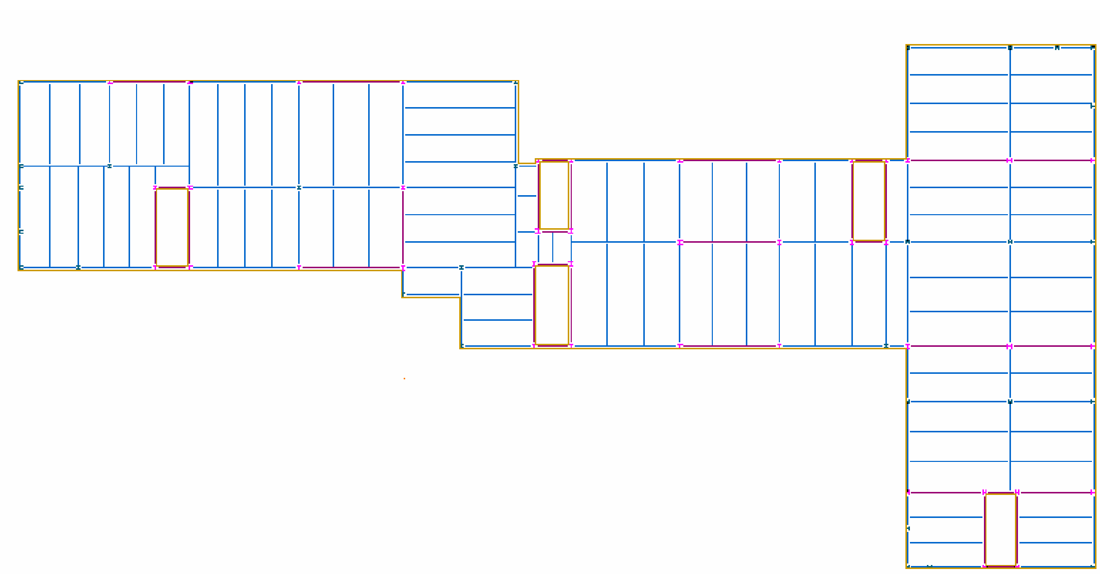


Figure 6: Typical plan showing structural layout, Levels 1 and 2

moment caused by the lateral loads.

An analysis of the gravity beams and columns will then be performed in RAM Structural System for the various dead and live loading conditions. RAM Frame will be used to evaluate the effects of wind and seismic loads imposed on the building. From RAM Frame, story displacements and story forces can be obtained to determine feasibility of the system.

All designs will be compliant with provisions set forth in the Building Code of the City of New York, which further references UBC 97 for seismic loads and ASCE 7-05 for wind loads.

5.2 - LOADING CONDITIONS

5.2.1 - GRAVITY LOADS

The following is the list of gravity dead and live loads for each of the building occupancies used in the proposed design of The Towers. These loads are in accordance with the Building Code of the City of New York. Dead loads include self weight of the members, finishes, MEP piping and sprinklers, and partitions. The loads listed below do not include the weight of the structural members; however they will be included in the RAM analysis. The live loads are reducible per section 27-566 of the building code.

DORMITORY	PSF
Construction Dead Load	
- 2” deck w/ 2 ½” N.W. concrete	45
Superimposed Dead Load	
- ceiling	4
- floor finish	2
- mechanical/electrical	5
- partitions (100-200 plf)	12
- misc.	2
Total Dead Load	70
Live Load	
- for partitioned dormitories	40

LOBBY/CORRIDOR	PSF
Construction Dead Load	
- 2” deck w/ 2 ½” N.W. concrete	45

Superimposed Dead Load	
- ceiling	2
- floor finish	2
- mechanical/electrical	5
Total Dead Load	60
Live Load	100

ROOF (MECHANICAL)	PSF
Construction Dead Load	
- 2" deck w/ 2 ½" N.W. concrete	45
Superimposed Dead Load	
- ceiling	2
- mechanical/electrical	6
- roofing and insulation	5
- misc	2
Total Dead Load	70
Live Load	
- weight of equipment and ponding water	150

EXTERIOR WALL LOADS	PSF
Dead Load	
- prefabricated thin brick panels with metal stud back-up wall	24
- curtain wall system	15

5.2.2 - LATERAL LOADS

Lateral loads imposed on The Towers are the result of wind and seismic forces. Per the City Building Code of the City of New York, the wind loads are calculated based on the methods provided in ASCE 7-05 and the seismic loads are calculated based on the UBC Section 2312-1990 with modifications provided in the New York City building code.

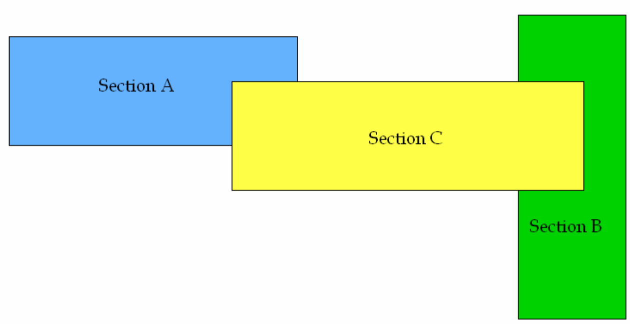


Figure 7: Building components used for lateral load calculations

To simplify the loading for wind, the building will be broken up into three components as shown in Figure 2. Section A, B and C consist of 8, 6, and 11 stories consecutively. It was determined that wind loading will control in the east-west directions because of the much greater loading area. The seismic loading will control in the north-south direction because large braces could not be implemented in this direction.

The following is a summary of the wind and seismic loads, as well as diagrams to illustrate the loading patterns on the building. See Appendix A for complete lateral loading calculations.

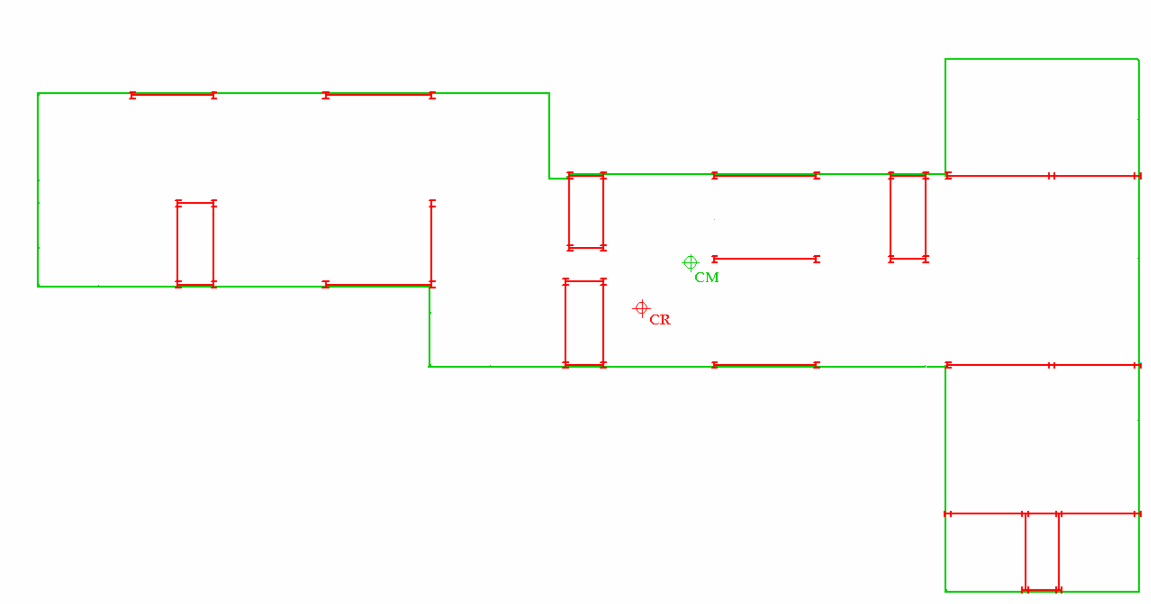


Figure 8: Center of rigidity and center of mass for the 3rd floor

WIND LOADS

- Basic Wind Speed	V = 95 mph	
- Importance Factor	I_W = 1.0	
	Category 4	
- Building Exposure	D	
- Mean Roof Height	110′-0″	
- Gust Factor (Rigid Structure)	G = 0.85	
- Topographic Factor	Kzt = 1.0	
- Wind Directionality Factor	Kd = 0.85	
- Velocity Pressure Coefficients	Kh = 1.455	
	Kz = 1.03	0 – 15′
	Kz = 1.08	15 – 20′
	Kz = 1.12	20 – 25′
	Kz = 1.16	25 – 30′
	Kz = 1.22	30 – 40′
	Kz = 1.27	40 – 50′

	Kz = 1.31	50 – 60′
	Kz = 1.34	60 – 70′
	Kz = 1.38	70 – 80′
	Kz = 1.40	80 – 90′
	Kz = 1.43	90 – 100′
	Kz = 1.455	100 – 110′
- Internal Pressure Coefficient	GCpi = +/- 0.18	
- Wall Pressure Coefficients	Cp = 0.8 (windward)	
	Cp = -0.5 (leeward ⊥ 294'-8")	
	Cp = -0.3 (leeward ⊥ 144'-4")	
	Cp = -0.7 (sidewall)	
- Roof Pressure Coefficients	Cp = -0.9 (0 – h)	
	Cp = -0.5 (h – 2h)	
	Cp = -0.3 (>2h)	

<u>East – West Wind</u>
Loading Diagrams

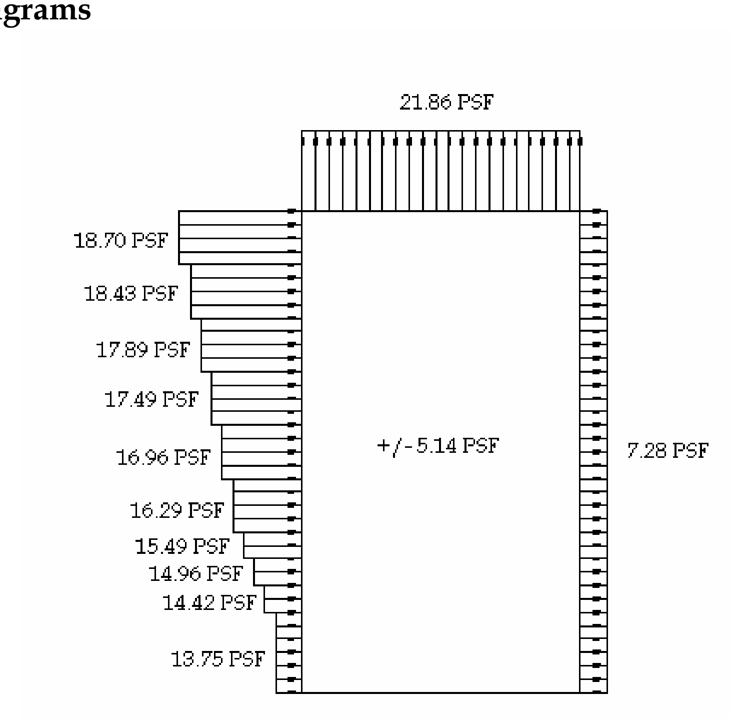


Figure 9: Wind pressure on Section A

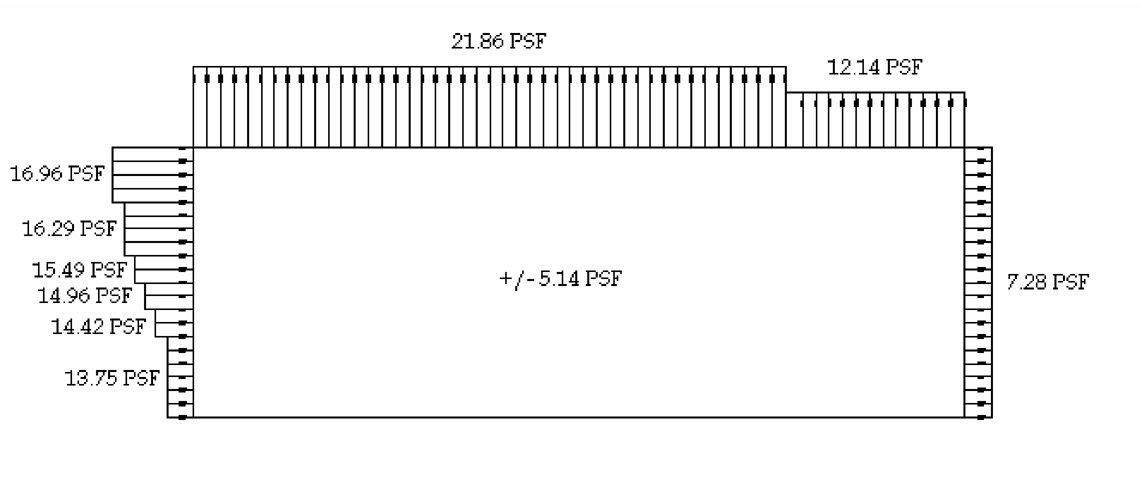


Figure 10: Wind pressure on Section B

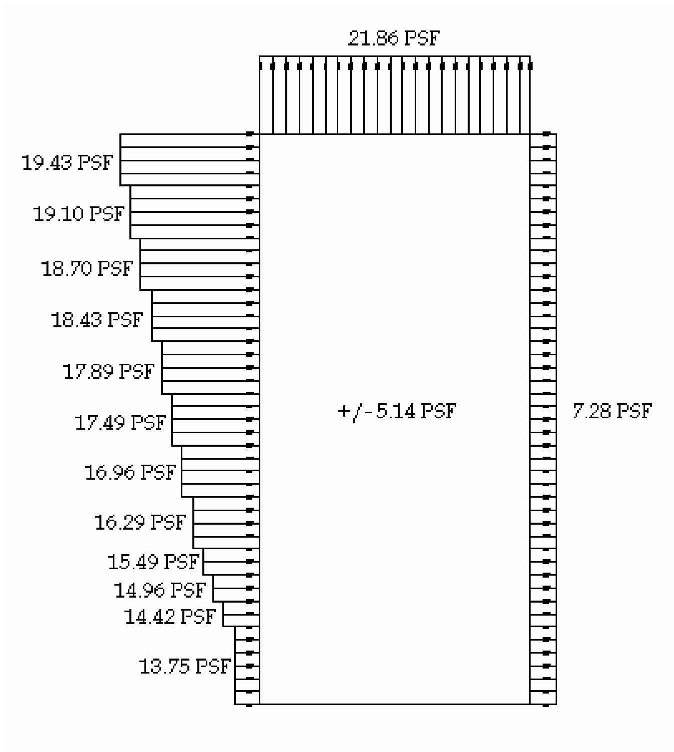


Figure 11: Wind pressure on Section C

SEISMIC LOADS

- Seismic Zone Factor	Z = 0.15
- Site Class: Hard Rock	A
- Mapped Spectral Acceleration (0.2s)	S_S = 35% g
- Mapped Spectral Acceleration (1.0s)	S_1 = 6.5% g
- Importance Factor	I = 1.0 I_p = 1.0
- Analysis Procedure	Static Force Procedure
- Plan Structural Irregularities	No
- Vertical Structural Irregularities	No
- Building Height	h_n = 110′
- Type of Lateral System (dual system: steel eccentric braced frames with ordinary moment resisting frames)	R = 6 C_T = 0.035

In calculating the weight of the Towers for seismic forces, 100% of the dead load and 25% of the live load are considered. The resultant story force acts at the center of mass of the floor. Figure X below shows the calculated story forces and the story shear applied at each floor under the seismic design criteria.

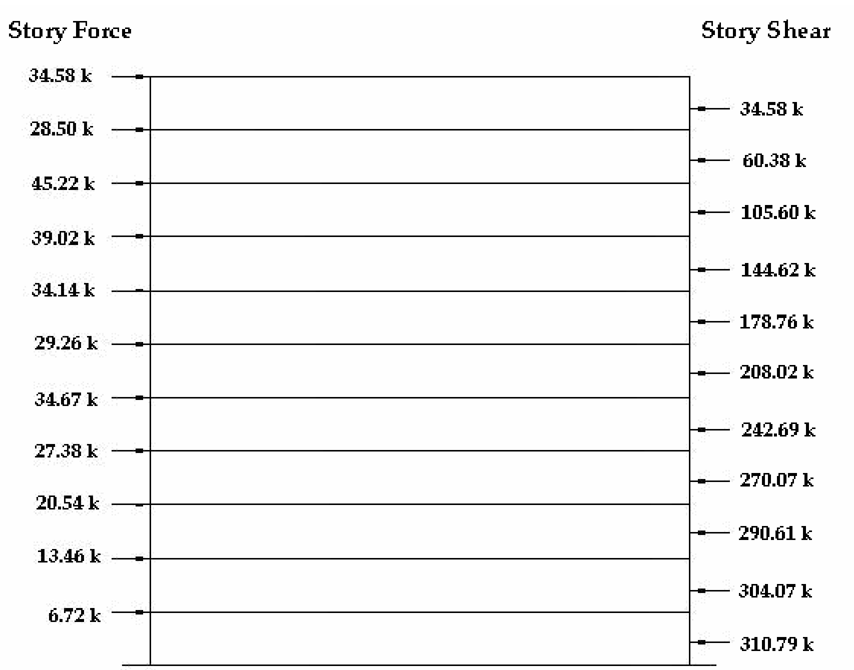


Figure 12: Calculated seismic story forces and story shear acting on the building

6.0 - STRUCTURAL DEPTH

6.1 - COMPOSITE BEAMS

A composite steel gravity system was chosen because of the high strength to weight ratio. Steel was selected for this system because long spans could be achieved. This results in larger column spacing and a regular grid. Composite steel uses shear studs to create a bond and transfer forces from the concrete slab to the steel beam. The beams were modeled and analyzed in RAM Structural System and design checks were performed to check the RAM results. These design checks can be found in Appendix B. The infill beams are W10 or W12 shapes equally spaced between girders. The girders are kept to a maximum W21 to keep the plenum depth at 2′.

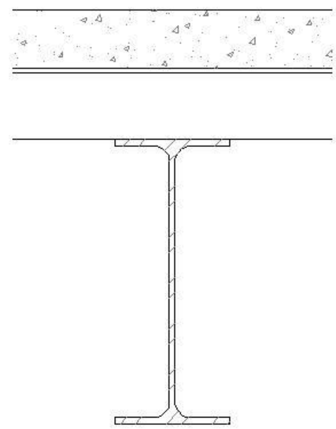


Figure 13: Composite steel section

The steel girders are able to be cantilevered at the building corners where the floor-to-ceiling windows are located. In the RAM analysis, the deflections were kept to a maximum of L/400 to create a stiffer member and reduce deflections and vibrations. The beams and girders were analyzed using LRFD method and were checked using the procedure outlined in the 13th Edition AISC Steel Manual. Hung gypsum board acts as fireproofing for the steel structural members and also provides an acoustical barrier between floors.

6.2 - COLUMNS

Loads are transferred to the gravity columns by girders. W10 shapes were chosen for the columns to keep the member within the partition and exterior walls. This creates less of an impact on the architectural floor plan. The columns were analyzed in RAM and checked using the method outlined in the 13th Edition of the LRFD. The column's capacity is checked for combined axial and flexural strength, and local buckling. Gypsum board will be used as fireproofing. The steel system creates less of an architectural impact on the floor plan as opposed to the existing concrete system. All columns are located within walls and out of the way, creating unobstructed rooms and corridors.

6.3 - COMPOSITE DECK

Using the maximum spacing of 11′-0″ between beams and the 2 hour fire rating requirement according to the New York City building code, a 2VLI18 composite deck with gypsum board fireproofing will be selected for the floor system. This deck does not need to be shored after the concrete is placed. The depth of the deck is 2″ with 2 ½″ normal weight concrete equals a total slab thickness of 4 ½″. The deck spans perpendicular to the steel infill beams.

6.4 - LATERAL FORCE RESISTING SYSTEM

Lateral forces are to be resisted by a dual system of eccentric braced frames with ordinary moment resisting frames. Eccentric braced frames are located around stair and elevator cores where the frame would be within a wall as shown in Figure 15. Moment frames braced with double angle kickers are located elsewhere throughout the building where eccentric braced frames could not be applied. See Figure 14 for typical moment frame. The frames are laid out to keep the eccentricity between center of mass and center of rigidity at a minimum to reduce the amount of torsion caused by lateral loads.

W12 shapes were applied as the lateral columns to provide additional stiffness for the frames. All braces used were 2L6x6x½ double angles.

See Appendix D for complete frame elevations with sizes.

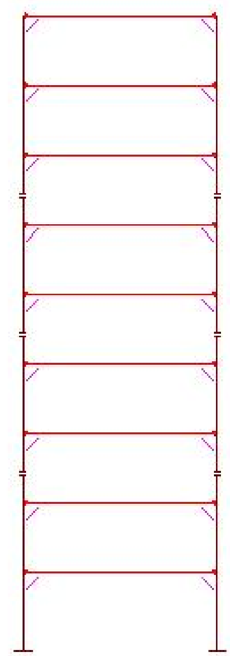


Figure 14: Typical moment frame braced with kicker angles

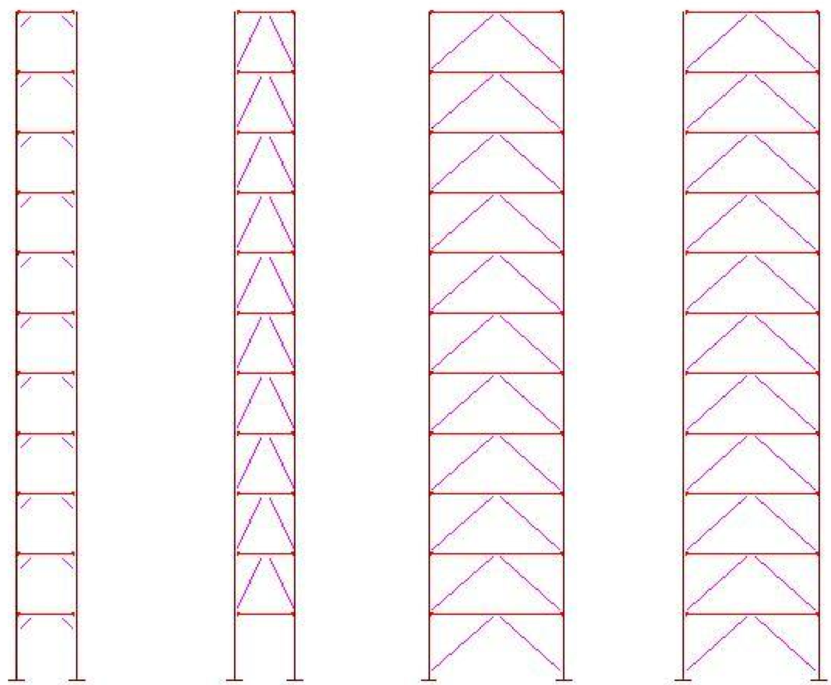


Figure 15: Typical braced frame layout around stair core

With the new steel structure, the earthquake loads in the x-direction and the wind loads in the y-direction will control. The following are diagrams depicting the deflected shape of the frame under the controlling earthquake loads in the x-direction and wind loads in the y-direction. The load combinations used were generated from RAM Frame and comply with ASCE 7-05. These load combinations are:

- 1.4 D
- 1.2 D + 1.6 L
- 1.2 D + 0.5 L + 1.6 W
- **1.2 D + 1.6 W** Controls y-direction deflection
- 0.9 D + 1.6 W
- **1.2 D + 0.5 L + 1.0 E** Controls x-direction deflection
- 1.2 D + 1.0 E
- 0.9 D + 1.0 E

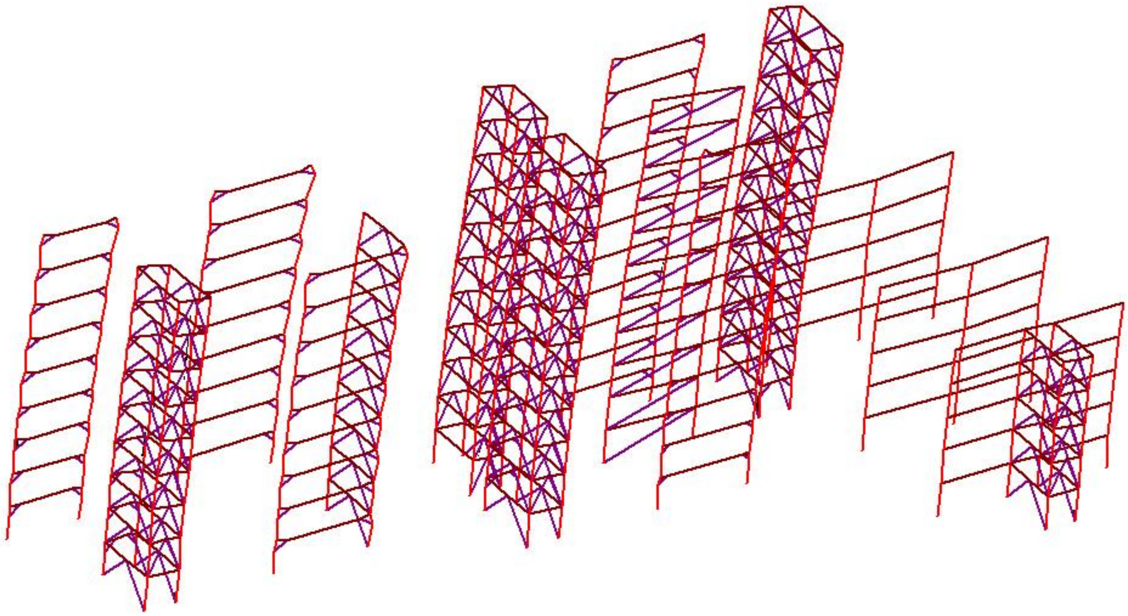


Figure 16: Frame deflections under controlling load factors in the x-direction

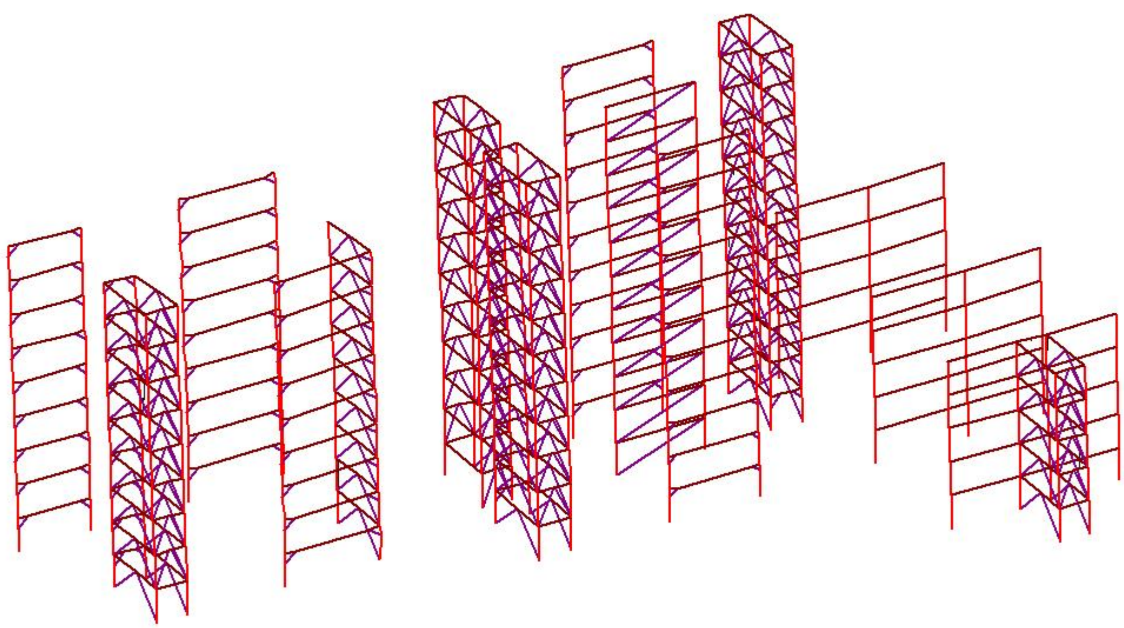


Figure 17: Frame deflections under controlling load factors in the y-direction

STORY	STORY DISPLACEMENT		STORY DRIFT	
	X	Y	X	Y
Roof	3.53″	1.87″	0.36″	0.17″
10	3.17″	1.70″	0.32″	0.08″
9	2.85″	1.62″	0.38″	0.21″
8	2.47″	1.41″	0.40″	0.21″
7	2.07″	1.20″	0.40″	0.21″
6	1.67″	0.99″	0.39″	0.27″
5	1.28″	0.72″	0.33″	0.18″
4	0.95″	0.54″	0.30″	0.17″
3	0.65″	0.37″	0.26″	0.16″
2	0.39″	0.21″	0.19″	0.12″
1	0.20″	0.09″	0.20″	0.09″

The displacement at the roof level in the x direction is slightly higher than the H/400 industry standard, which equals 3.50″.

6.5 - TYPICAL CONNECTIONS

For most of the connections, in the building, typical shear and moment connections can be applied. The connections are designed using the controlling ultimate factored loads obtained from the analysis. The procedure for connection design is outlined in the 13th edition of the LRFD Manual of Steel Construction.

For the beam to girder connections, single angles can be applied as the shear connection. The angles are shop welded to the girder and bolted in the field to the beam. Appendix B shows the complete design calculation for this type of connection

For typical gravity beam to gravity column connections, shear tabs can be used. The steel plates are shop welded to the columns and bolted to the girder in the field. In some instances, web stiffener plates are welded in the column web to protect the column web from crippling under panel zone shear. Shown below are schematics for the typical shear connections.

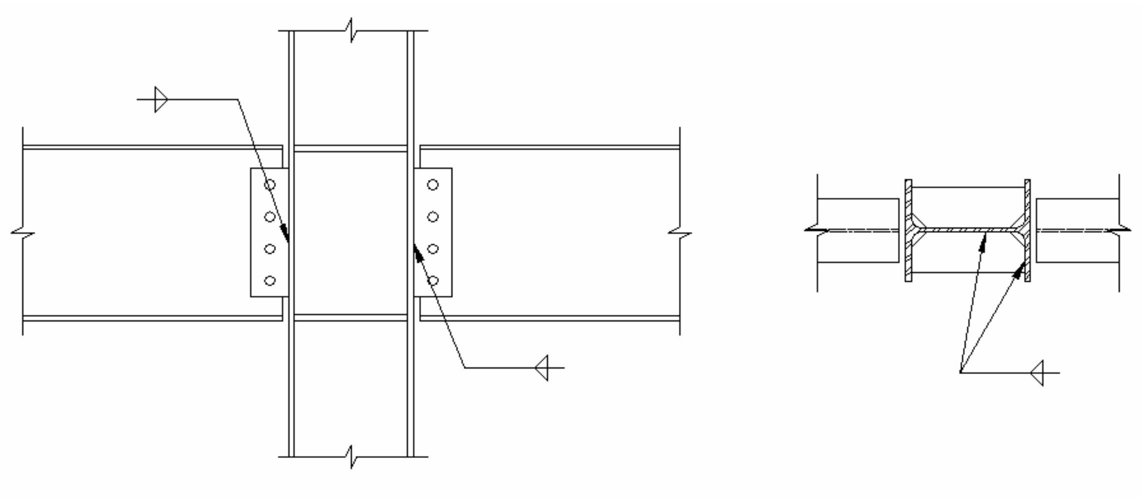


Figure 18: Typical girder to column shear tab connection shown with stiffener plates

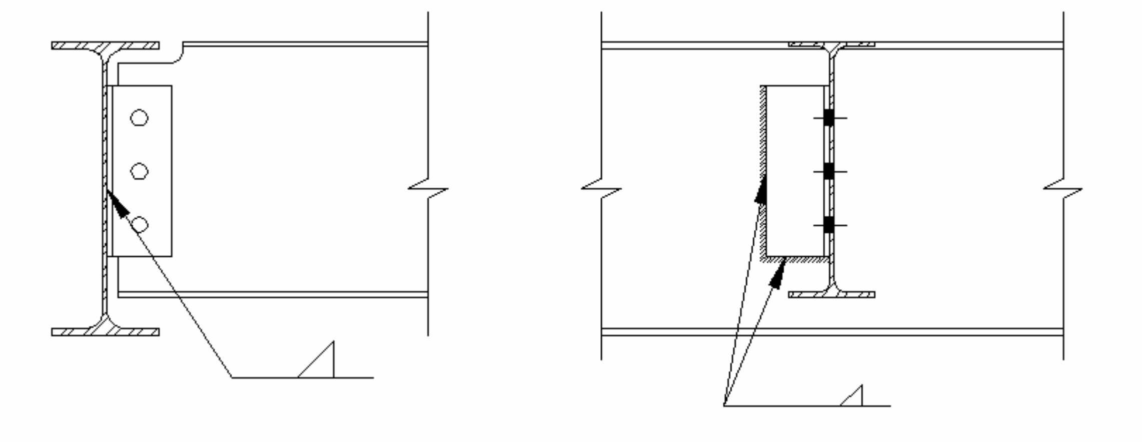


Figure 19: Typical beam to girder single angle connection

For the moment frames, end plate fully restrained moment connections can be used for the frame beam to frame column connections. These are designed using the procedure outlined in the AISC Design Guide 16 (Flush and Extended Multiple-Row Moment End-Plate Connections) Stiffener plates will have to be welded in the column web to reduce the effects of panel zone shear. For the braced frames, light bracing connections can be used.

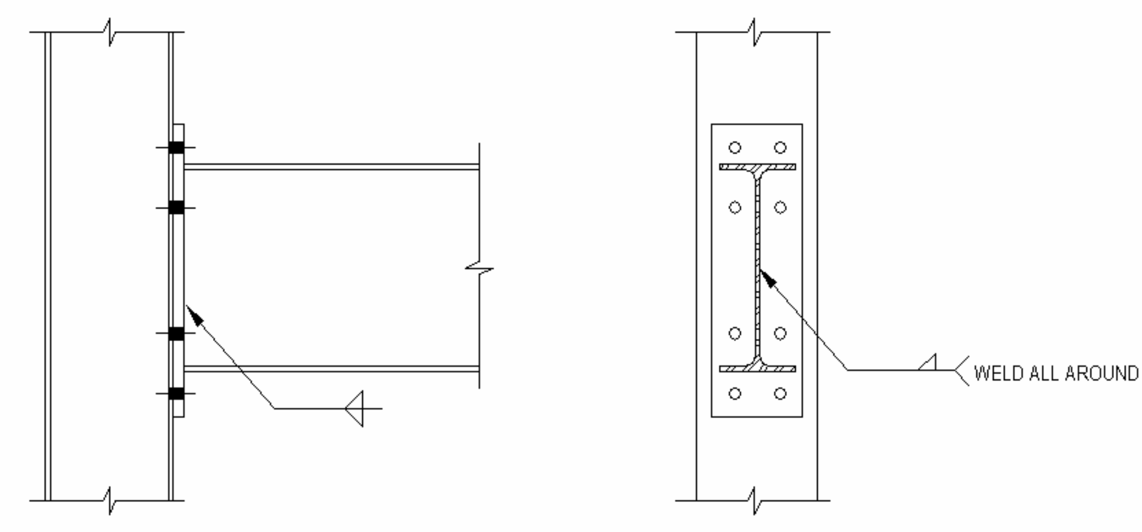


Figure 20: Typical frame beam to frame column fully restrained moment connection

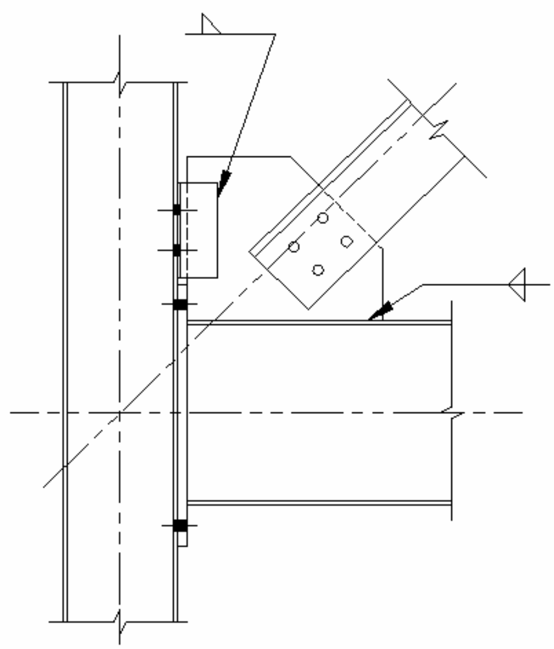


Figure 21: Typical light bracing connection in a moment frame

6.6 - IMPACT ON FOUNDATIONS

The steel structure produces a lighter frame which in turn produces less dead load on the foundations. A typical foundation for the same column was checked for the existing concrete structure and the proposed steel structure. The total unfactored dead and live load for the steel column was reduced by 47%. This resulted in a much smaller foundation size and less reinforcing. A decrease in foundation sizes will result in a cost savings in concrete.

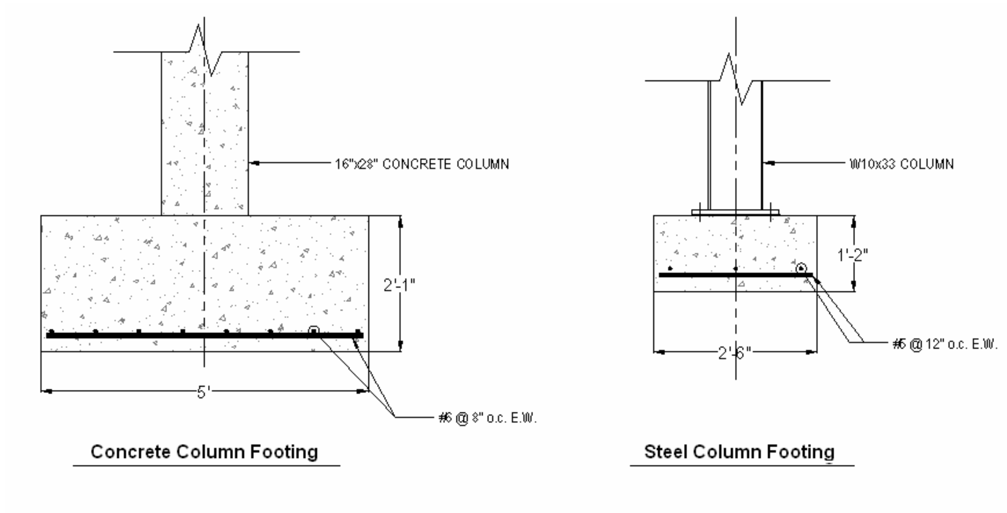


Figure 22: Comparison of gravity column footings for existing concrete structure and proposed steel structure

7.0 - STRUCTURAL DEPTH SUMMARY

GRAVITY SYSTEM

A composite steel frame was proposed for the gravity system of The Towers. The beams were W10 or W12 shapes and the girders were kept to a maximum depth of 21″ to keep a 2′ plenum depth. A 2″ composite metal deck with 2 ½″ of normal weight concrete was used as the floor system. This is capable of spanning the maximum beam spacing of 11′-0″ without needing shoring. The columns for the gravity loads are W10 shapes.

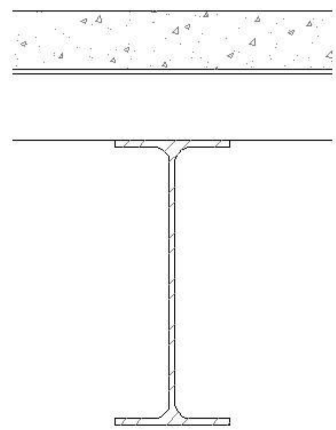


Figure 23: Section of composite floor slab and beam

The use of steel imposes less dead load on the foundations, which results in a decrease in foundation size and less reinforcing required for the gravity system. The beam to girder connections are single angles shop welded to the girder and bolted to the beam. The beam to column connections are shear tabs shop welded to the column flange and bolted to the beam.

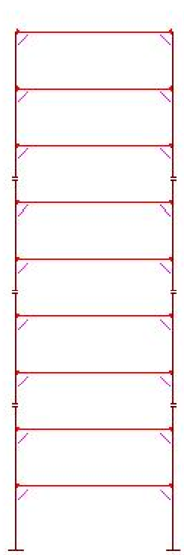


Figure 24: Typical moment frame

LATERAL SYSTEM

The lateral forces imposed on the building are resisted by a dual system of eccentric braced frames with ordinary moment resisting frames. Braced frames are used where they line up within a wall. Moment frames with kickers are used where the wall has window or door openings.

For the lateral system, light bracing connections consisting of L6x6x½″ angles are used to connect the double angle braces to the beams and columns. All lateral columns are W12 shapes for added stiffness. The lateral beams range in size from W10′s to W14′s. Fully restrained moment connections are required at the beam to column connections to resist wind and seismic loads.

8.0 - BREADTH TOPICS

8.1 – LEED CERTIFICATION

Designing a building to be LEED certified has become a growing trend in today's construction industry. With the trend in the rising earth's temperature and the fear of depleting fossil fuels, there are many people looking for ways to reduce the consumption of energy and become more efficient in building designs. The Unites States Green Building Council (USGBC) is a group dedicated to making buildings more environmentally efficient and healthier places to be in. The USGBC developed the LEED rating system as a set of guidelines to follow to create a sustainable building. Today, there are many incentives for buildings to go green. The benefits of LEED certification include tax rebates, operating cost reduction, conservation of energy and an overall healthier living environment.

For a building to become LEED certified, five aspects of design and construction of the building are analyzed to determine if it meets the requirements for energy and water efficiency. These aspects are:

- Sustainable Sites
- Water Efficiency
- Energy and Atmosphere
- Materials and Resources
- Indoor Environmental Quality

It was determined if there was a possibility for The Towers to become a LEED certified building. After analyzing the criteria provided by the USGBC for new building construction, it is possible for the building to gain the 26 points which will qualify it for LEED certification. Points can be earned from site selection because previously the site consisted of old buildings. The location of the building is close to four subway stops and many other bus stops. Also, the use of recycled steel and incorporating low VOC emitting paints and finishes can add points to the LEED rating. Appendix C contains a LEED checklist of possible rating points that The Towers can attain.

One specific point that was investigated was the thermal efficiency of the building envelope. The existing exterior wall consists of a 1 ¾" precast thin brick panel, 5/8" glas-mat sheathing, 6" cold formed metal stud with R19 insulation, and 5/8" gypsum board. Two options that were chosen to increase thermal efficiency of the wall system was to increase the thickness of the insulation and to use loose fill cellulose insulation.

Wall Insulation	Wall R-Value	Heat Transfer Through Wall (BTU/hr)
6″ R19 Batt Insulation	23.55	2.42
6″ Loose Fill Cellulose Insulation	27.35	2.09
8″ Batt Insulation	26.55	2.15

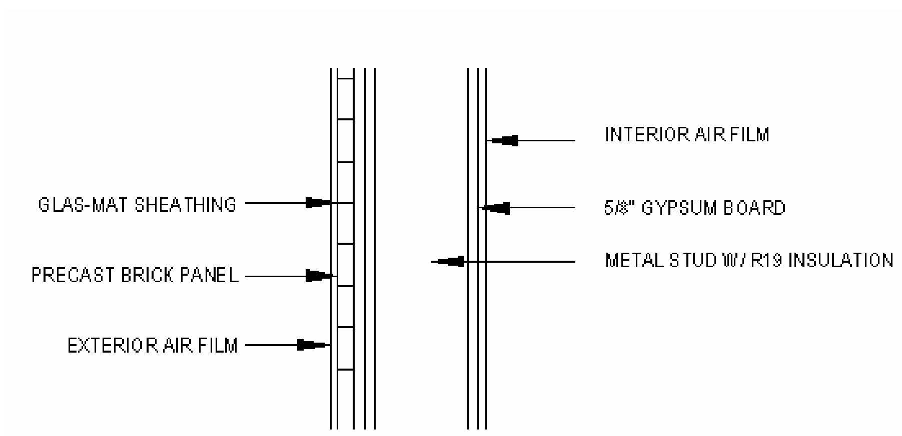


Figure 25: Existing building envelope

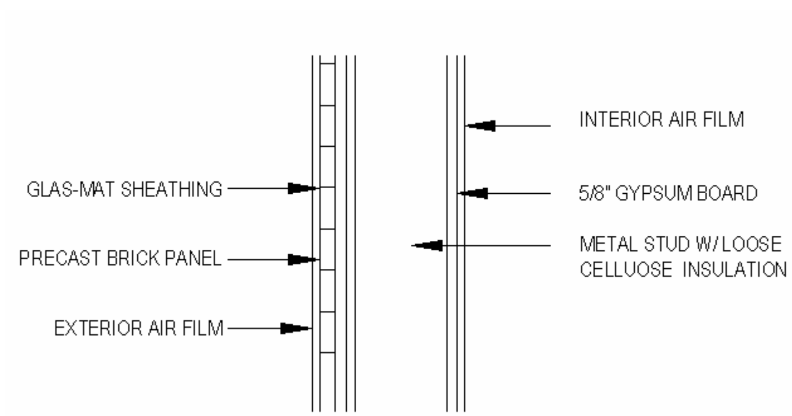


Figure 26: Proposed building envelope

Cellulose loose fill insulation will be proposed to insulate the wall. This will provide a higher R-value and less heat transfer without increasing the thickness of the wall. Cellulose loose fill insulation is made from recycled newspaper, which provides a higher R-Value than conventional fiberglass batting and earns points for the LEED rating.

8.2 – COST ANALYSIS

Changing a building's structural frame will impact the construction cost and schedule of the project. For the proposed steel structure, production costs from MC^2 software, RSMeans and Primavera scheduling software were used to perform a cost and schedule analysis. The cost and schedule for the steel structure was then compared to the existing concrete structure.

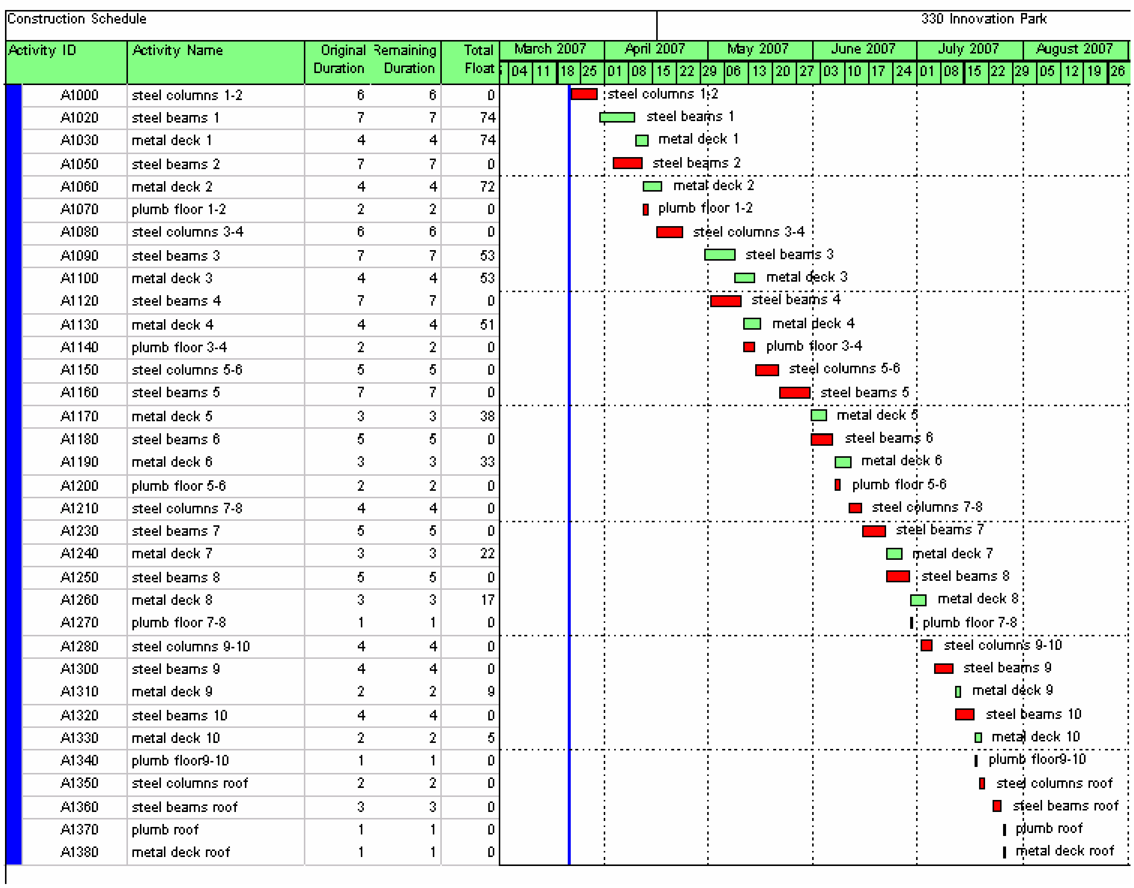
For The Towers, the exact cost of the concrete frame was unable to be obtained. A concrete estimate was done taking into consideration the material quantity and labor costs of formwork, reinforcing, 5000 psi concrete, shoring and required equipment. It was determined that the concrete structure cost approximately $5.5 million. All material takeoffs and labor rates are located in Appendix C. The following is a breakdown of the cost for the elements used in the steel structure. It was determined that the total cost for the steel structure will be approximately $6.0 million.

COMPOSITE DECK	
- Concrete	$149,200
- Wire Mesh Reinforcing	$42,600
- 2” Deck	$1,810,000
- Screeds for Slab	$121,800
- Slab Finish	$229,000
- Protect and Cure	$23,400
	$2,376,000
STEEL FRAMING	
- W Shapes	$2,870,000
- Angles	$150,000
- Shear Studs	$42,400
- Red Oxide	$63,500
- Base plates	$4,000
- Grout	$1,000
- Anchor Bolts	$2,800
- Gypsum Board Fireproofing	$62,000
	$3,621,000
TOTAL COST	$5,997,000

8.3 – SCHEDULE IMPACT

Using the crew production rates provided by RSMeans and MC² software, as well as the material quantities obtained for the cost estimate, durations for each of the stages of construction can be determined to produce a construction schedule. It was determined that for the existing concrete structure, the frame took approximately 140 days to complete.

Assuming three steel crews were to be used for the construction, the duration for the erection of the proposed structure was determined to be 95 days. One advantage of using steel as opposed to concrete is the reduction in construction time and less crews on the jobsite.



Activity ID	Activity Name	Original Duration	Remaining Duration	Total Float
A1000	steel columns 1-2	6	6	0
A1020	steel beams 1	7	7	74
A1030	metal deck 1	4	4	74
A1050	steel beams 2	7	7	0
A1060	metal deck 2	4	4	72
A1070	plumb floor 1-2	2	2	0
A1080	steel columns 3-4	6	6	0
A1090	steel beams 3	7	7	53
A1100	metal deck 3	4	4	53
A1120	steel beams 4	7	7	0
A1130	metal deck 4	4	4	51
A1140	plumb floor 3-4	2	2	0
A1150	steel columns 5-6	5	5	0
A1160	steel beams 5	7	7	0
A1170	metal deck 5	3	3	38
A1180	steel beams 6	5	5	0
A1190	metal deck 6	3	3	33
A1200	plumb floor 5-6	2	2	0
A1210	steel columns 7-8	4	4	0
A1230	steel beams 7	5	5	0
A1240	metal deck 7	3	3	22
A1250	steel beams 8	5	5	0
A1260	metal deck 8	3	3	17
A1270	plumb floor 7-8	1	1	0
A1280	steel columns 9-10	4	4	0
A1300	steel beams 9	4	4	0
A1310	metal deck 9	2	2	9
A1320	steel beams 10	4	4	0
A1330	metal deck 10	2	2	5
A1340	plumb floor9-10	1	1	0
A1350	steel columns roof	2	2	0
A1360	steel beams roof	3	3	0
A1370	plumb roof	1	1	0
A1380	metal deck roof	1	1	0

Figure 27: Construction schedule for steel structure erection obtained from Primavera

9.0 - SUMMARY

Using steel, the entire structure becomes lighter. This reduces the amount of dead load on the foundations under gravity columns. This reduces the amount of concrete and reinforcing steel needed for these foundations, decreasing the construction cost.

There are some drawbacks to the proposed steel structure. It was determined from the RAM analysis that the deflections under a code designed earthquake are about ¼" greater than the limit of H/400. The schedules are impacted because a building designed in steel has a much larger lead time that a concrete building. The individual steel members must be fabricated weeks before they will be put in place. This leaves little room for error in the final design stages of the building. The proposed steel structure will have added height of 1'-4" per floor. This will increase the area of the thin brick panel veneer needed for the façade.

There will also be an added cost associated with the steel connections. Fully restrained moment connections are required to resist the wind and seismic forces. The extensive amount of welding for these connections will drive up the cost.

The Towers could have the potential for a LEED certification with a proposed steel structure. The location of the site and the use of steel could possibly earn points. A more efficient building envelope was studied and the use of cellulose loose fill insulation can decrease the amount of heat transfer through the wall from 2.42 BTU/hr to 2.09 BTU/hr as well as help earn LEED points for material reuse.

The construction cost and schedule were impacted by changing the structure from concrete to steel. Although the exact concrete structure cost was unable to be obtained, it was determined that the approximate cost of the steel was equal to $5.4 million. Using MC2 and RSMeans, it was determined that the steel structure would cost approximately $5.6 million. The construction of the concrete frame took 140 days. Using production rates provided by RSMeans, the duration of the construction of the steel structure took 95 days.

10.0 - CONCLUSION

A thorough investigation of a proposed steel structural frame for The Towers was performed and was compared to the existing concrete structure. A steel structure was chosen to investigate to eliminate the need for columns in the corners of the building where the floor to ceiling windows are located. The two structures were compared based on construction cost, schedule and impact on foundations.

A steel structure is a viable option for a structural system for The Towers. Using steel, the corners of the building with the corner windows can be cantilevered, leaving the windows free of any structural members. The columns can be lined up on a regular grid, which will make the structure easy to construct. This grid also reduced the number of columns that the original structure had. From a construction standpoint, steel is a practical structural system because it is a common practice in New York City. The foundations for the proposed steel building are significantly smaller than the foundations for the concrete structure.

Based on schedule and foundation impact, a steel structure is a practical option for The Towers and could be a recommended solution. Although the cost for the steel is slightly higher than the concrete, nine weeks were saved in the erection time. The foundations for the steel structure are significantly smaller than the foundations for the existing concrete structure. Steel is also a common construction practice in New York City, which further makes constructing a steel structure possible.

There are some drawbacks for making the structure steel. A steel structure impacts the architecture considerably. The floor to floor heights for the steel and concrete designs is 10′-0″ and 8′-8″ respectively. Therefore, the steel produces an increase in height of 13′-3″. This will cause an increase in area of the precast thin brick veneer, which in turn will increase the cost. Fully restrained moment connections will be needed to resist the wind and seismic loads imposed on the building. Fully restrained connections require extensive welding which will add to the construction cost of the building. Also, the steel designs require a larger lead time, meaning the design of the structure must be complete in time for the steel to be fabricated.

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