The Towers at
The City College of New York
New York City, NY

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Structural Option
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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

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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 multiply 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

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.

Figure 2: Existing foundation
(photo courtesy GPI)

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.
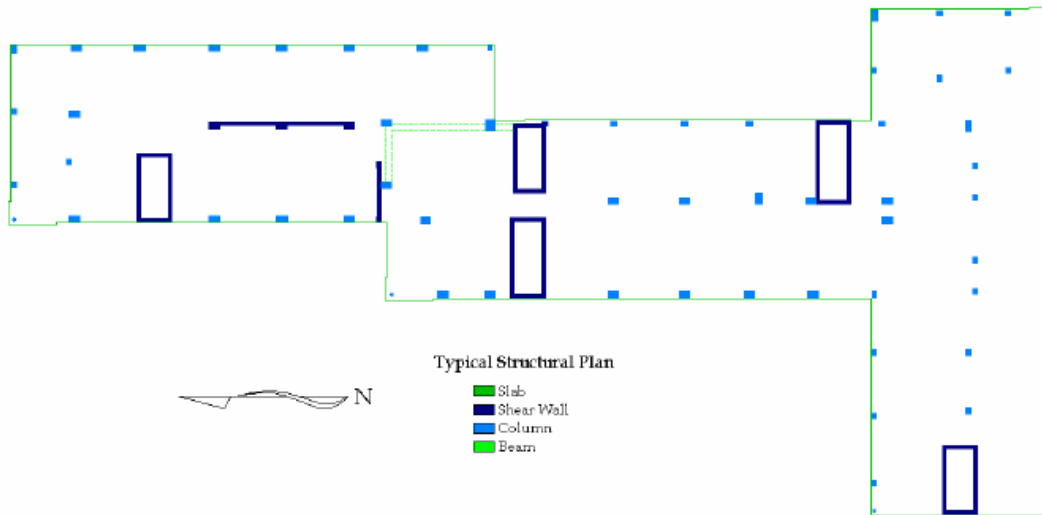
Figure 4: Typical structural plan of existing building

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: $\dfrac{L}{360}$

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

$\dfrac{L}{400}$ (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
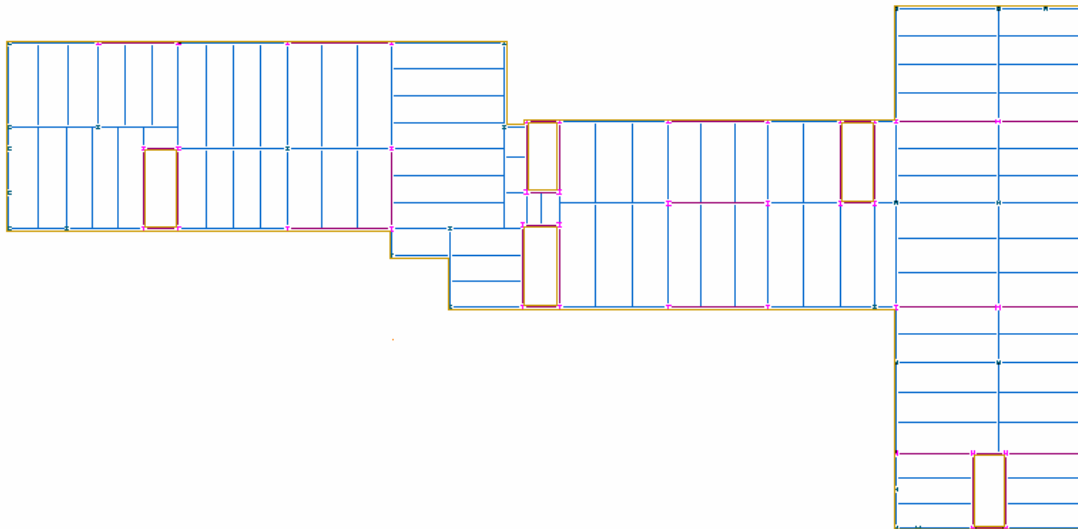

Figure 6: Typical plan showing structural layout, Levels 1 and 2

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
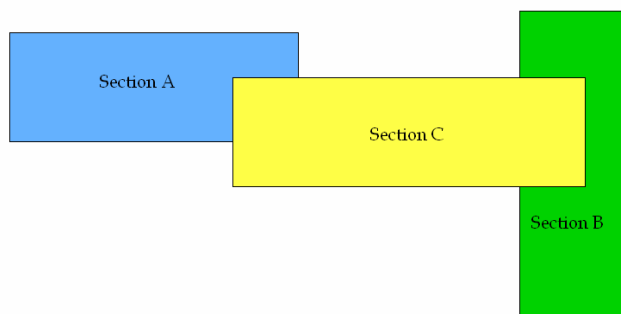

Figure 7: Building components used for lateral load calculations

are calculated based on the UBC Section 2312-1990 with modifications provided in the New York City building code.

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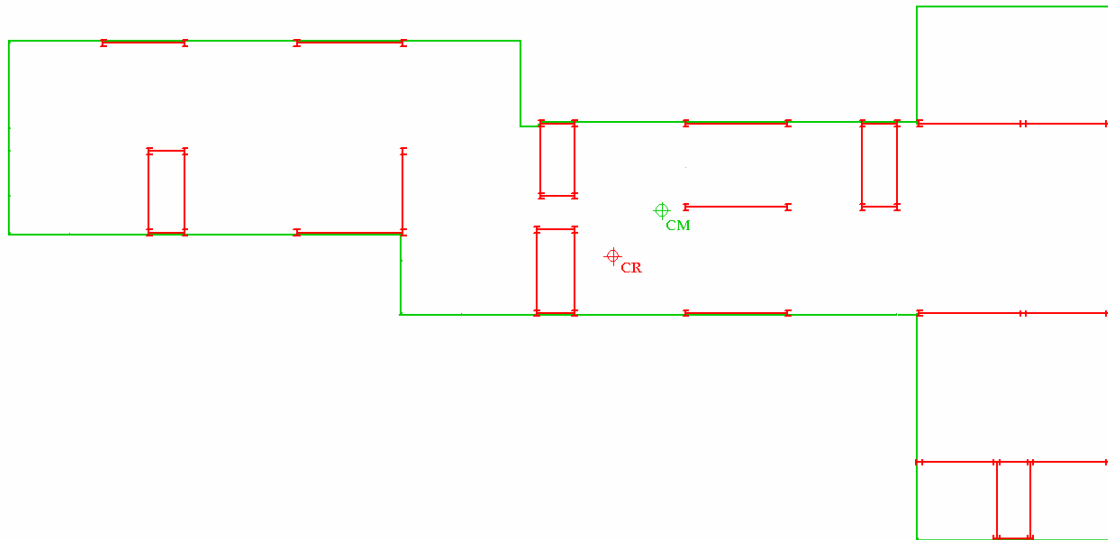
Figure 8: Center of rigidity and center of mass for the 3rd floor

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)

East – West Wind
Loading Diagrams

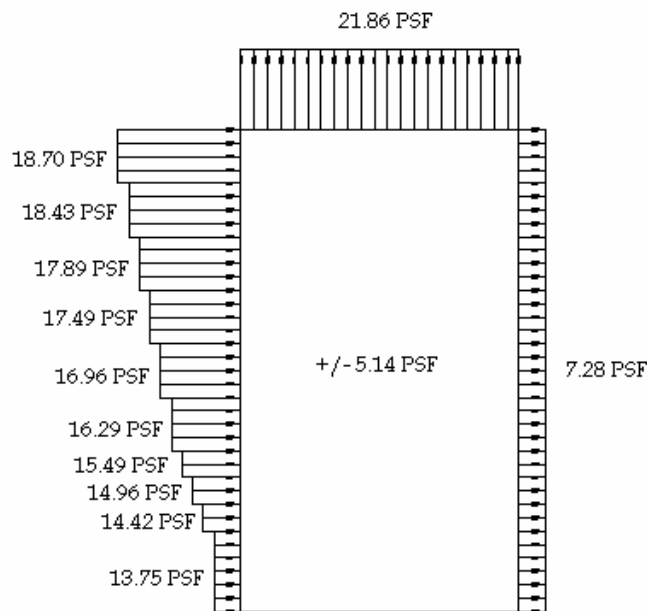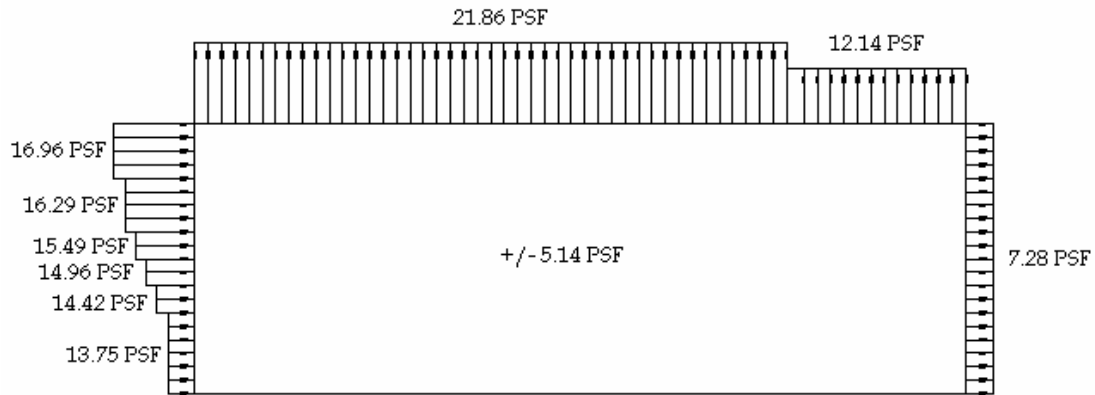

Figure 9: Wind pressure on Section A

Figure 10: Wind pressure on Section B
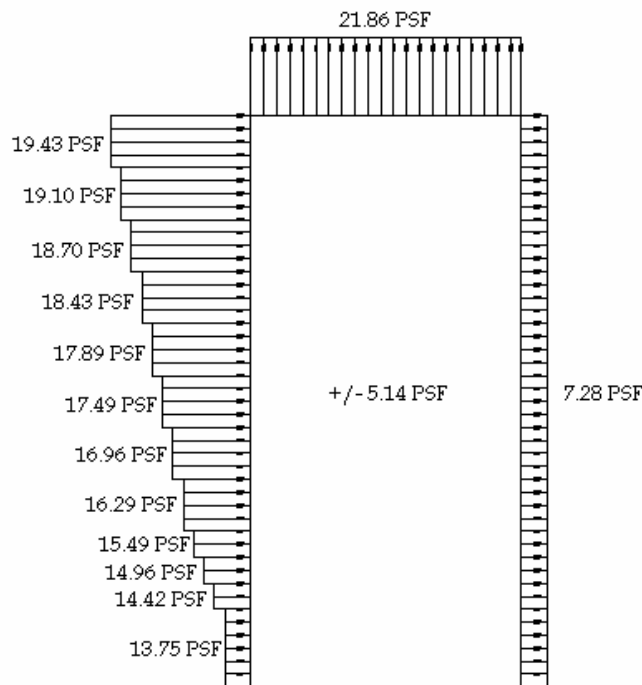
Figure 11: Wind pressure on Section C

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	$R = 6$
(dual system: steel eccentric braced frames with ordinary moment resisting frames)	$C_T = 0.035$

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.

Story Force	Story Shear
34.58 k	34.58 k
28.50 k	60.38 k
45.22 k	105.60 k
39.02 k	144.62 k
34.14 k	178.76 k
29.26 k	208.02 k
34.67 k	242.69 k
27.38 k	270.07 k
20.54 k	290.61 k
13.46 k	304.07 k
6.72 k	310.79 k

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

6.0 - STRUCTURAL DEPTH

6.1 - COMPOSITE BEAMS

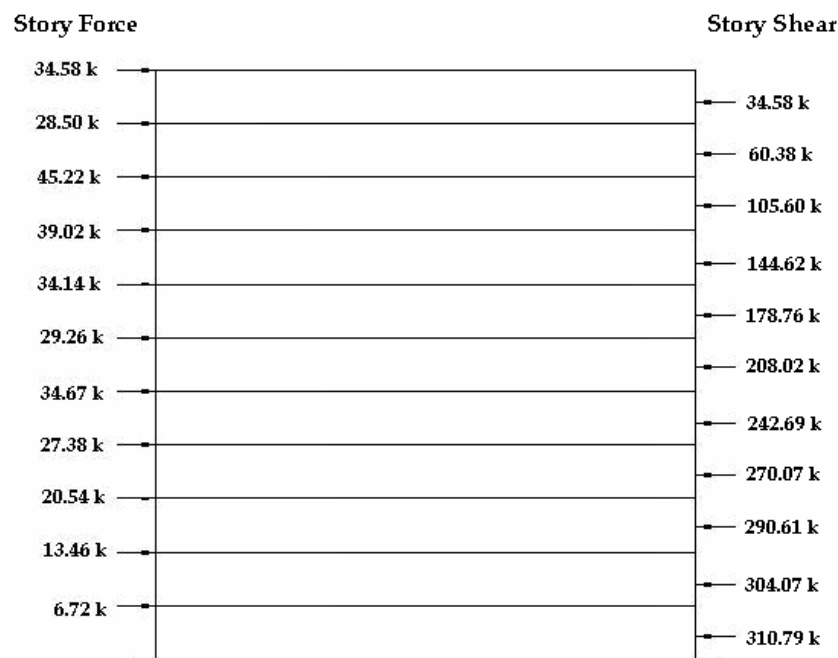A composite steel gravity system was chosen because of the high strength to weight ratio. Steel was selected for this system because long spans could be achieved. This results in larger column spacing and a regular grid. Composite steel uses shear studs to create a bond and transfer forces from the concrete 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′.
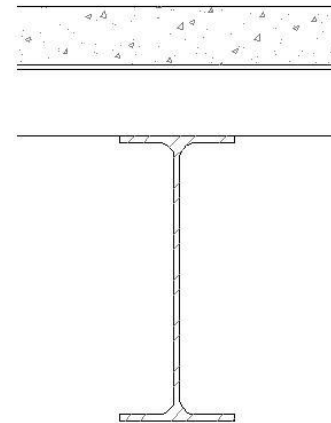
Figure 13: Composite steel section

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.
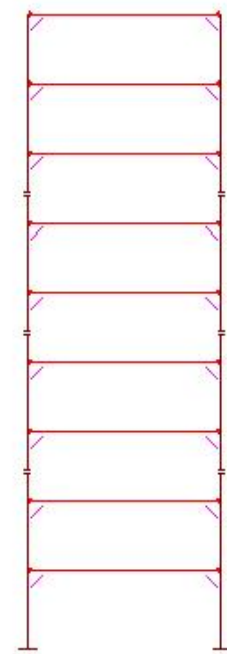
Figure 14: Typical moment frame braced with kicker angles
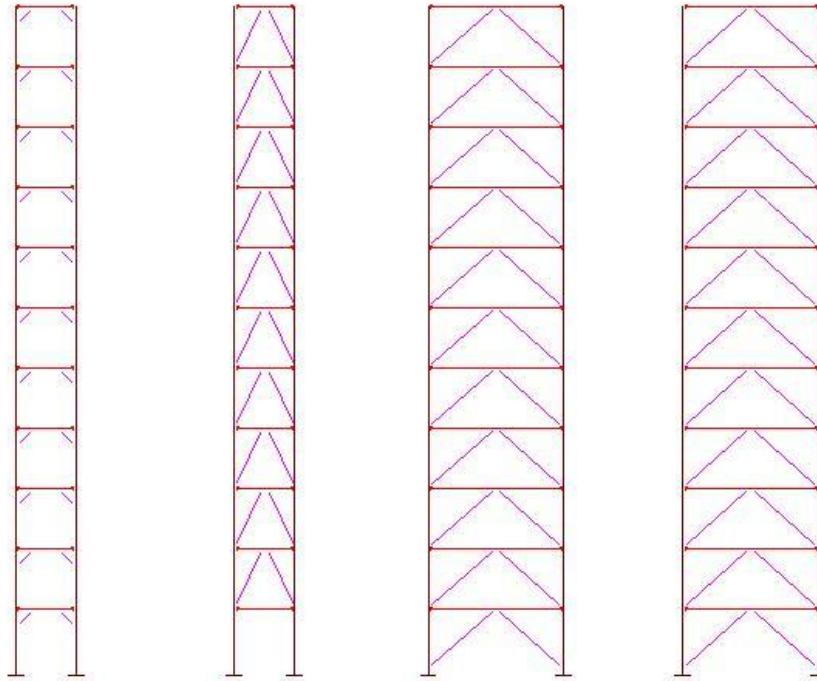
Figure 15: Typical braced frame layout around stair core

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
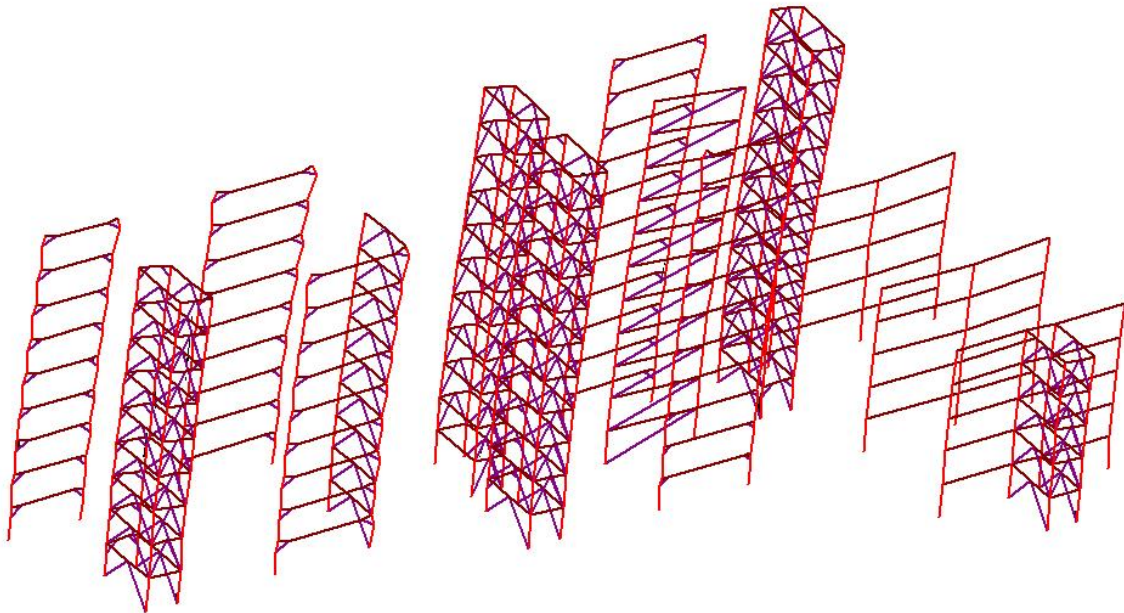
Figure 16: Frame deflections under controlling load factors in the x-direction
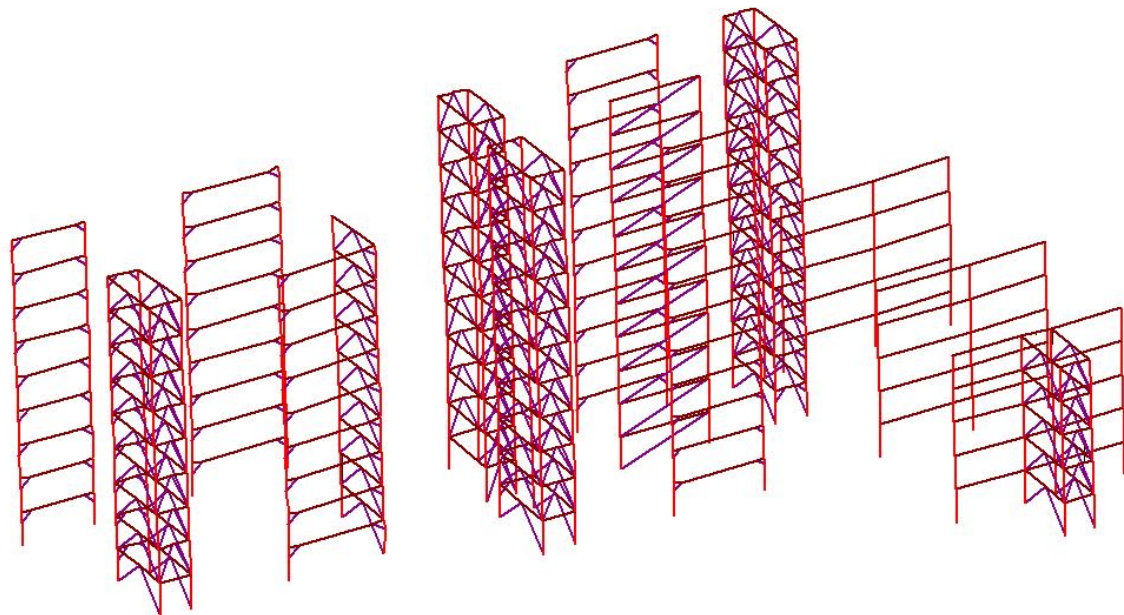
Figure 17: Frame deflections under controlling load factors in the y-direction

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.
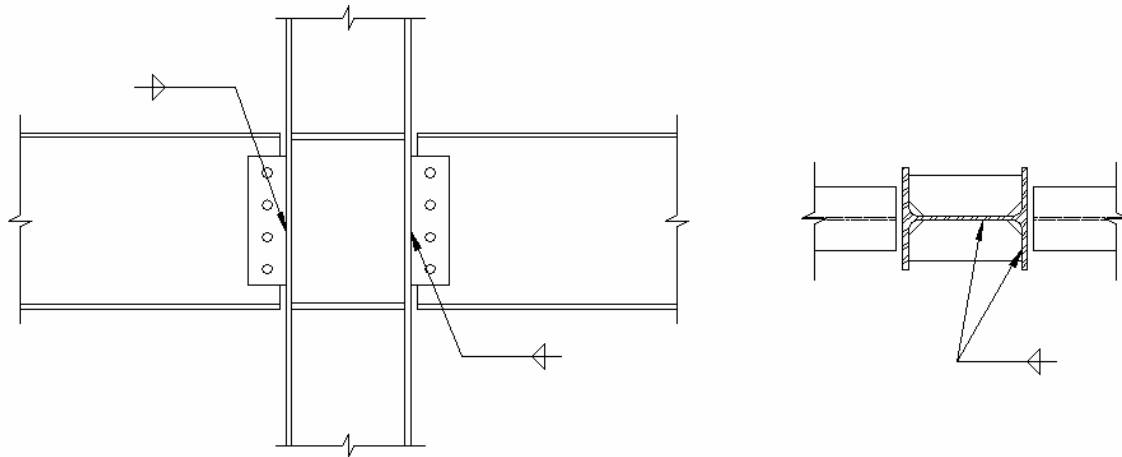
Figure 18: Typical girder to column shear tab connection shown with stiffener plates
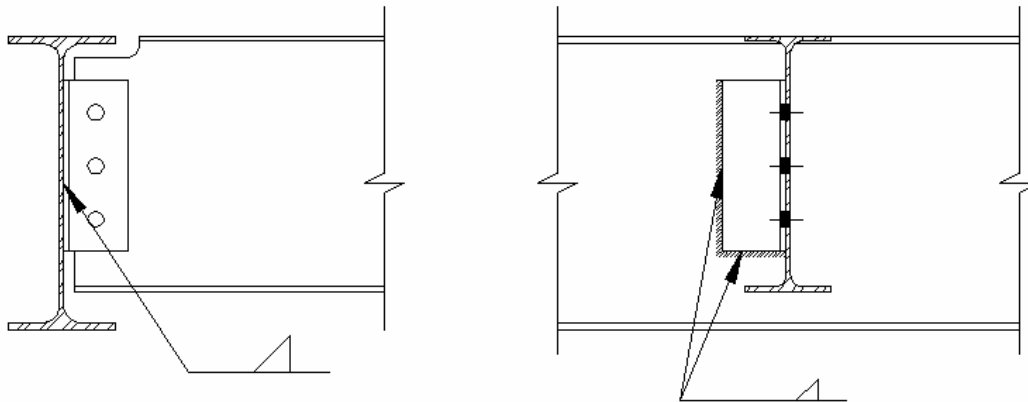
Figure 19: Typical beam to girder single angle connection

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.
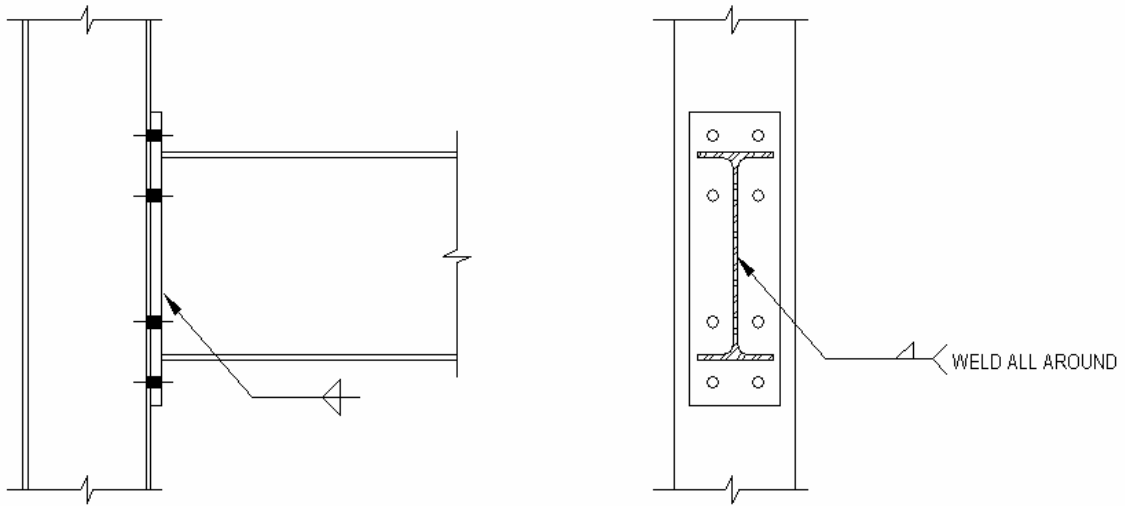
Figure 20: Typical frame beam to frame column fully restrained moment connection
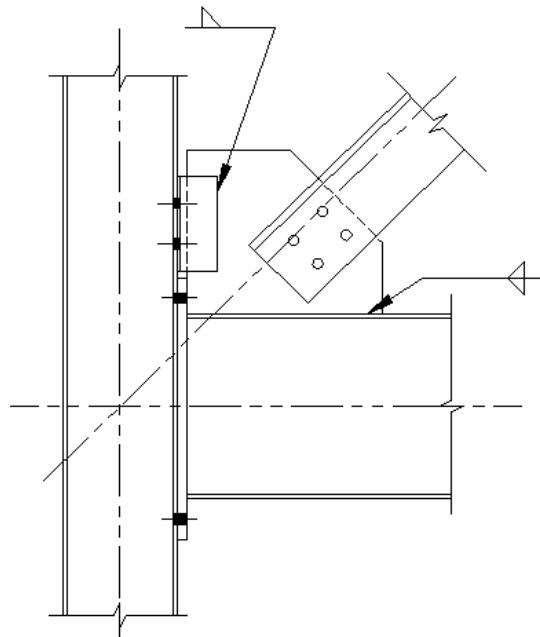
Figure 21: Typical light bracing connection in a moment frame

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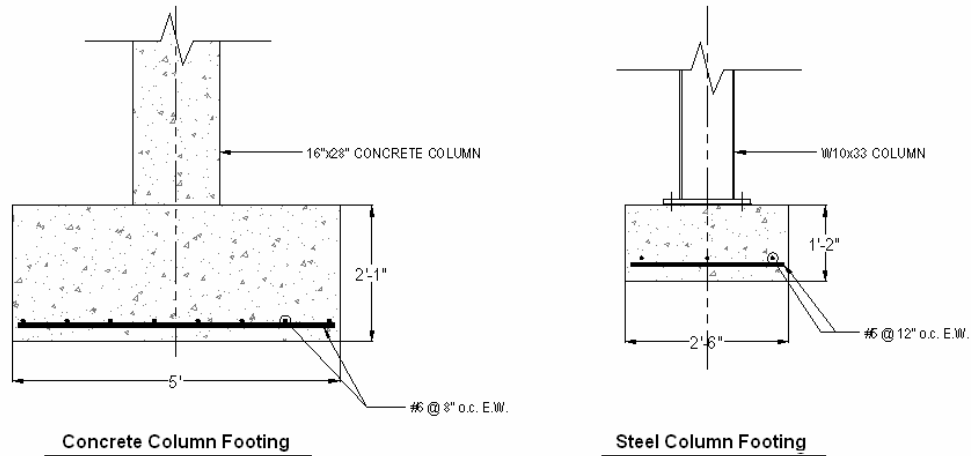
Figure 22: Comparison of gravity column footings for existing concrete structure and proposed steel structure

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.

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.
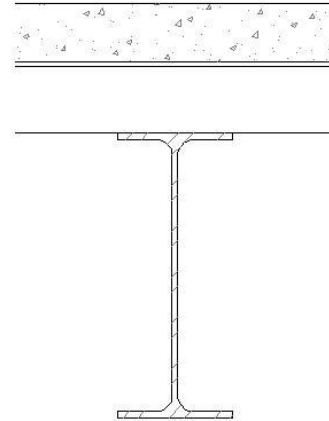
*Figure 23: Section of composite
floor slab and beam*

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.

*Figure 24: Typical moment
frame*

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
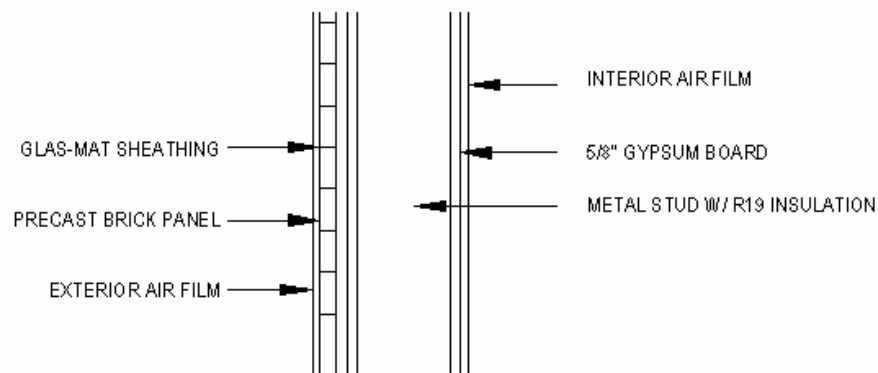
Figure 25: Existing building envelope
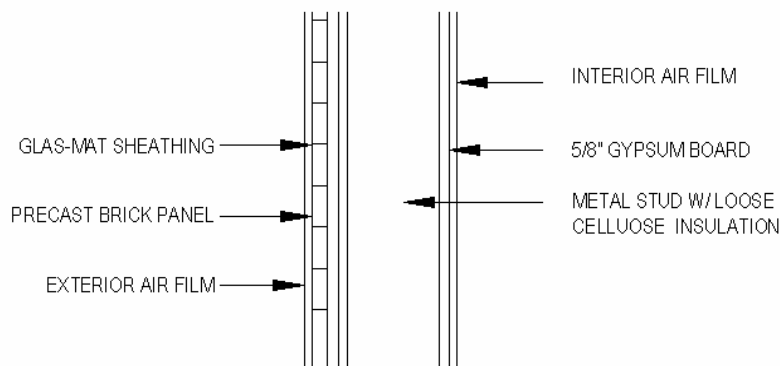
Figure 26: Proposed building envelope

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 MC² 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.

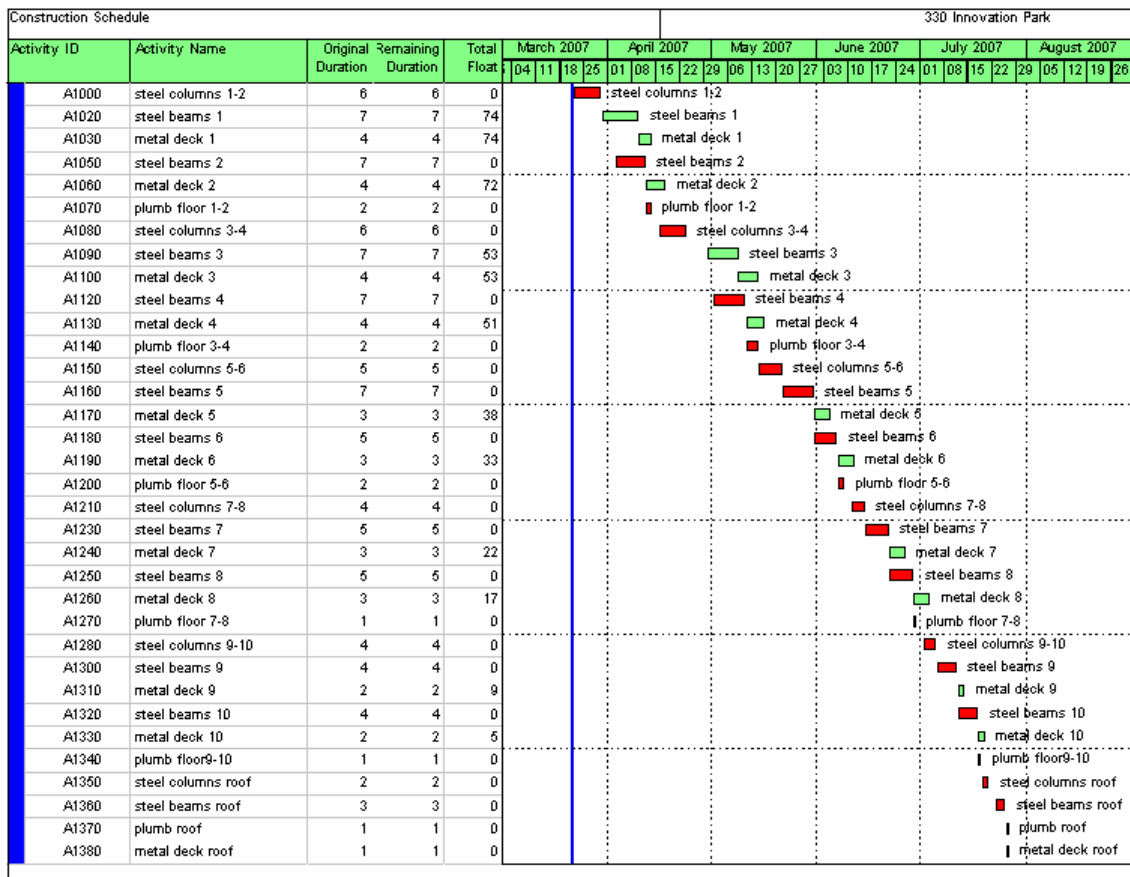| Activity ID	Activity Name	Original Duration	Remaining Duration	Total Float
A1000	steel columns 1-2	6	6	0
A1020	steel beams 1	7	7	74
A1030	metal deck 1	4	4	74
A1050	steel beams 2	7	7	0
A1060	metal deck 2	4	4	72
A1070	plumb floor 1-2	2	2	0
A1080	steel columns 3-4	6	6	0
A1090	steel beams 3	7	7	53
A1100	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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I especially want to thank my family for putting up with me for the past 4 months, giving me the encouragement I needed to finish this project and reassuring me that graduation is around the corner!

APPENDIX

APPENDIX A

A.1 – WIND LOADS

Velocity Wind Pressure

Windward pressure

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

$$q_z = (0.00256)(1.03)(0.085)(1.0)(95 \text{mph})^2 (1.0)$$

$$q_z = 20.23 \text{psf}$$

Height	kz	qz	
0 - 15	1.03	20.23	psf
15 - 20	1.08	21.21	psf
20 - 25	1.12	22.00	psf
25 - 30	1.16	22.78	psf
30 - 40	1.22	23.96	psf
40 - 50	1.27	24.94	psf
50 - 60	1.31	25.73	psf
60 - 70	1.34	26.32	psf
70 - 80	1.38	27.10	psf
80 - 90	1.4	27.49	psf
90 - 100	1.43	28.08	psf
100 - 110	1.455	28.57	psf

Leeward pressure

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I$$

$$q_h = (0.00256)(1.455)(0.085)(1.0)(95 \text{mph})^2 (1.0)$$

$$q_h = 28.57 \text{psf}$$

Design Wind Pressure

Windward wall

$$p = q_z GCp - (GCpi) q_h$$

$$p = (20.23 \text{psf})(0.85)(0.8) - (\pm 0.18)(28.57)$$

$$p = 13.75 \pm 5.14 \text{psf}$$

Height	qz		p			
0 - 15	20.23	psf	13.75	+/-	5.14	psf
15 - 20	21.21	psf	14.42	+/-	5.14	psf
20 - 25	22.00	psf	14.96	+/-	5.14	psf
25 - 30	22.78	psf	15.49	+/-	5.14	psf
30 - 40	23.96	psf	16.29	+/-	5.14	psf
40 - 50	24.94	psf	16.96	+/-	5.14	psf
50 - 60	25.73	psf	17.49	+/-	5.14	psf
60 - 70	26.32	psf	17.89	+/-	5.14	psf
70 - 80	27.10	psf	18.43	+/-	5.14	psf
80 - 90	27.49	psf	18.70	+/-	5.14	psf
90 - 100	28.08	psf	19.10	+/-	5.14	psf
100 - 110	28.57	psf	19.43	+/-	5.14	psf

Leeward wall with north-south wind

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.3) - (\pm 0.18)(28.57)$$
$$p = -7.28 \pm 5.14\,psf$$

Roof with north south wind

$$\frac{h}{L} = \frac{110'}{294'-8''} = 0.373$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.9) - (\pm 0.18)(28.57) \qquad 0 - 110'$$
$$p = -21.86 \pm 5.14\,psf$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.5) - (\pm 0.18)(28.57) \qquad 110' - 220'$$
$$p = -12.14 \pm 5.14\,psf$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.3) - (\pm 0.18)(28.57) \qquad 220' - 294'-8''$$
$$p = -7.28 \pm 5.14\,psf$$

Leeward wall east-west wind

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.5) - (\pm 0.18)(28.57)$$
$$p = 12.14 \pm 5.14\,psf$$

Roof with east-west wind

$$\frac{h}{L} = \frac{110'}{144'-4''} = 0.762$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.9) - (\pm 0.18)(28.57) \qquad 0 - 110'$$
$$p = -21.86 \pm 5.14\,psf$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.5) - (\pm 0.18)(28.57) \qquad 110' - 220'$$
$$p = -12.14 \pm 5.14\,psf$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\,psf)(0.85)(-0.3) - (\pm 0.18)(28.57) \qquad 220' - 294'-8''$$
$$p = -7.28 \pm 5.14\,psf$$

A.2 – SEISMIC LOADS

Building Period

$$T = Ct(h_n)^{\frac{3}{4}}$$

$$T = (0.035)(110')^{\frac{3}{4}}$$

$$T = 1.189s$$

Design Base Shear

$$V = \frac{CvI}{RT}w = \frac{(0.12)(1.0)}{(6)(1.189s)}(20330k)$$

$$V = 342k$$

$$Ft = 0.07VT = (0.07)(342k)(1.189s)$$

$$Ft = 28.5k$$

$$Fi = \frac{(V - Ft)w_i h_i}{\Sigma w_i h_i}$$

Level	h_i	w_i	$w_i h_i$	F_i
1	10	2290	22900	6.72
2	20	2290	45800	13.46
3	30	2330	69900	20.54
4	40	2330	93200	27.38
5	50	2360	118000	34.67
6	60	1660	99600	29.26
7	70	1660	116200	34.14
8	80	1660	132800	39.02
9	90	1710	153900	45.22
10	100	970	97000	28.50
roof	110	1070	117700	34.58

$\Sigma w_i h_i =$	1067000

APPENDIX B

B.1 – COMPOSITE BEAM CHECK

Loads:

    Superimposed Dead = 25 psf
    Superimposed Live = 40 psf
    Construction Dead = 45 psf + self weight

Beam Length = 23'-0"
Beam Spacing = 8'-0"

$f'c$ = 4 ksi
Fy = 50 ksi

$$w_D = (25\text{psf} + 45\text{psf})(8') + 12\text{plf}$$
$$w_D = 572\text{plf}$$
$$w_L = (40\text{psf})(8')$$
$$w_L = 320\text{plf}$$

$$w_U = 1.2w_D + 1.6w_L$$
$$w_U = 1.2(572\text{plf}) + 1.6(320\text{plf})$$
$$w_U = 1.2\text{klf}$$

$$Vu = \frac{w_U L}{2} = \frac{(1.2\text{klf})(23')}{2}$$
$$Vu = 13.8^k$$

$$Mu = \frac{w_U L^2}{8} = \frac{(1.2\text{klf})(23')^2}{8}$$
$$Mu = 79.4^{'k}$$

$$b_{eff} = \min \begin{cases} \dfrac{23'\left(12''/1'\right)}{4} = 69'' \\[2mm] (8')\left(12''/1'\right) = 96'' \end{cases}$$

Assume 1" of concrete in compression:

$$y_2 = 4.5'' - 1''/2$$
$$y_2 = 4''$$
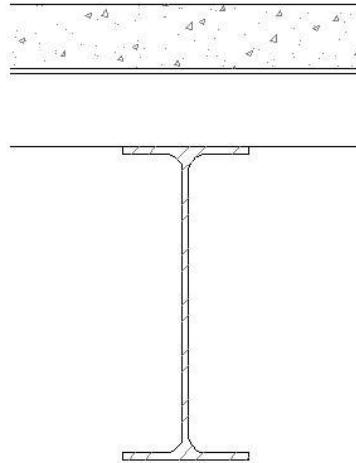
From LRFD 13th Edition Table 3-19:

Try W 10x12
  PNA at location 6
  $\phi Mn = 84.2^{'k} > Mu = 79.4^{'k}$
  $\sum Qn = 68.9^{k}$

$$a = \frac{\sum Qn}{0.85f'cb_{eff}} = \frac{68.9^{k}}{(0.85)(4ksi)(69")}$$
$$a = 0.29" < 1" \therefore ok$$

Use W 10x12 Beam

B.2 – GRAVITY COLUMN CHECK

Column K-3 at Level 3

Loads:

$Pu = 124.7^k$

$Mu_X = 3.4^{'k}$

$Mu_Y = 2.5^{'k}$

$kLx = kLy = 10'$

W 10x33:

$\phi Pn = 330^k$ LRFD 13th Edition Table 4-1

$\phi Mn_X = \phi Mn_Y = 134^{'k}$ LRFD 13th Edition Table 3-10

$$\frac{Pu}{\phi Pn} = \frac{124.7^k}{330^k} = 0.378 > 0.2 \therefore \text{Equation H1-1a}$$

$$\frac{Pu}{\phi Pn} + \frac{8}{9}\left(\frac{Mu_X}{\phi Mn_X} + \frac{Mu_Y}{\phi Mn_Y}\right) \le 1.0$$

$$\frac{124.7^k}{330^k} + \frac{8}{9}\left(\frac{3.4^{'k}}{134^{'k}} + \frac{2.5^{'k}}{134^{'k}}\right) = 0.42 < 1.0 \therefore \text{ok}$$

Check local buckling:

Use W 10x33 Column

B.3 – FOUNDATION CHECK

$P = 143.6^k$

$Pu = 184.4^k$

Bearing pressure = 40,000 psf

Footing Size:

$40ksf = \dfrac{143.6^k}{B^2}$

$B = 2'-6''$

$q = \dfrac{184.4^k}{(2.5')^2}$

$q = 29.5ksf$

Two Way Shear:

$Vc = \phi 4\sqrt{f'c} = (0.75)(4)\left(\sqrt{4000psi}\right)$

$Vc = 189.7psi$

$d^2\left(Vc + \dfrac{q}{4}\right) + d\left(Vc + \dfrac{q}{2}\right)w = \left(\dfrac{q}{4}\right)\left(BL - w^2\right)$

$d^2\left(189.7psi + \dfrac{204.9psi}{4}\right) + d\left(189.7psi + \dfrac{204.9psi}{2}\right)(16'') = \left(\dfrac{204.9psi}{4}\right)\left((30'')^2 - (10'')^2\right)$

$d = 5.5''$

Punching Shear:

$4d^2 + 2d(b+c) = \dfrac{Pu}{Vc}$

$4d^2 + 2d(16''+14'') = \dfrac{184400lb}{189.7psi}$

$d = 10''$

Punching Shear Controls

Use h = 14″

$$d = 14" - 3" - \frac{5}{8}"$$

$$d = 10.375"$$

Flexure:
Critical Section

$$l = 1.25' - \frac{8"}{12}$$

$$l = 0.583'$$

$$Mu = \frac{(29.5ksf)(0.583')^2}{2}$$

$$Mu = 5.02'^k$$

$$Mu = \phi Asfy \left(d - \frac{a}{2}\right)$$

$$a = \frac{Asfy}{0.85f'cb} = \frac{As(60ksi)}{(0.85)(4ksi)(12")}$$

$$a = 1.47As$$

$$\left(5.02'\right)(12"/1') \le 0.9As(60ksi)\left(10.375" - \frac{1.47As}{2}\right)$$

$$As \ge 0.11in^2 / ft$$

Use #5 @ 12″ o.c.

Check Minimum Steel:

$$\rho = \frac{0.31in^2 / ft}{(14")(12")} = 0.00185 > \rho_{min} = 0.0018 \therefore ok$$

Check Tension Controlled Section:

$$\varepsilon = \frac{0.003}{c}(d - c)$$

$$c = \frac{a}{\beta_1} = \frac{(1.47)(0.31in^2)}{0.85}$$

$$c = 0.536"$$

$$\varepsilon = \frac{0.003}{0.536"}(10.375" - 0.536)$$

$$\varepsilon = 0.0551 > 0.005 \therefore \phi = 0.9$$

Check Bearing:

$$\phi Bn = \phi 0.85 f'c A_1 \sqrt{\frac{A_2}{A_1}} = (0.65)(0.85)(4ksi)(224in^2)\sqrt{\frac{900in^2}{224in^2}}$$

$$\phi Bn = 992.3^k > Pu = 184.4^k \therefore ok$$

**Use 2'-6" x 2'-6" x 14" thick spread footing
reinforced with #5 bars at 12" o.c.**

B.4 – BRACING MEMBER CHECK

Brace at 3rd floor in the y-direction
2 L 6x6x½

Load Combination: 1.2D + 1.6W
$Tu = 46.5^k$

Tension Yield:
$$\varphi Tn = \varphi FyAg = (0.9)(36ksi)(11.5in^2)$$
$$\varphi Tn = 372.6^k > Tu = 46.5^k \Rightarrow ok$$

Tension Fracture:
$$\varphi Tn = \varphi FuAe$$
$$Ae = UAn$$

$$U = 1 - \frac{\overline{x}}{L} = 1 - \frac{1.67"}{3"}$$
$$U = 0.443$$

$$An = 11.5in^2 - 2\left(\frac{3}{4}" + \frac{1}{8}"\right)\left(\frac{1}{2}"\right)$$
$$An = 10.625in^2$$

$$\varphi Tn = (0.75)(58ksi)(0.443)(10.635in^2)$$
$$\varphi Tn = 204.7^k > Tu = 46.5^k \Rightarrow ok$$

B.5 – TYPICAL SINGLE ANGLE CONNECTION

Vu = 13.8k

W12x14
$t_w = 0.255"$
$d = 11.9"$
$A = 4.16\text{in}^2$
$S_{net} = 4.71\text{in}^3$

Minimum weld size:
$t_w = 0.23"$ (W14x22 girder)
$t_{weld} = \frac{1}{8}"$
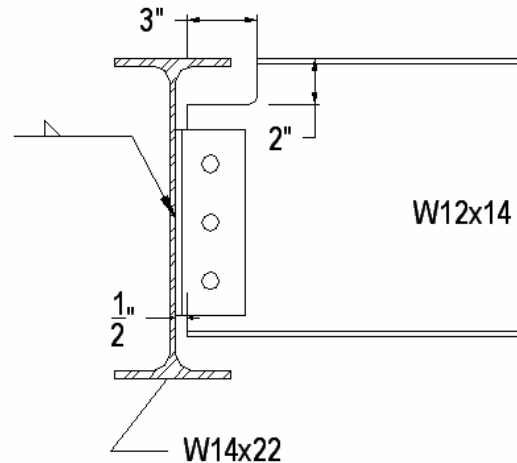
Eccentric shear strength of the weld:
$Rn = CC_1Dl$
$kl = 3"$
$\therefore k = \dfrac{3"}{6"} = 0.5$
$x = 0.083$

$xl = (0.083)(6") = 0.489"$
$e_x = 3.1"-0.498" = 2.602"$

$a = {e_x}\!\!\diagup\!\!{l} = {2.602"}\!\!\diagup\!\!{6"}$
$a = 0.434$

a	k=0.5
0.4	2.24
0.434	C
0.5	1.95

C=2.12

$Rn = (2.12)(1.0)(2)(6")$

$Rn = 25.44^{k}$

$\varphi Rn = (0.75)(25.44^{k})$

$\varphi Rn = 19.1^{k} > Vu = 13.8^{k} \Rightarrow$ use 1/8″ weld

Check weld base metal:

$\varphi Rn = \varphi(0.6Fu)t_{w}l$

$\varphi Rn = (0.75)(0.6)(65ksi)(0.23")(6")$

$\varphi Rn = 40.4^{k} > Vu = 13.8^{k} \Rightarrow$ ok

Minimum angle thickness:

$d_{b} = \frac{3}{4}"$

$\therefore t_{min} = \frac{3}{8}" \Rightarrow$ use L 3x3x3/8 x 6″ angle

Bolt shear:

$\varphi Vn = \varphi FvAb$

$\varphi Vn = (0.75)(48ksi)(2)\left(\dfrac{\pi \left(\frac{3}{4}" \right)^{2}}{4} \right)$

$\varphi Vn = 31.8^{k} > Vu = 13.8^{k} \Rightarrow$ ok

Angle Bearing and Tearout:

$Rn = 1.2L_{c}tFu \leq 2.4d_{b}tFu$

       Angle diameter $= \frac{3}{4}" + \frac{1}{16}" + \frac{1}{16}" = 0.875"$

       $1.2(3" - 0.875")\left(\frac{3}{8}" \right)(58ksi) = 55.5^{k}$

       $1.2\left(3" - \frac{0.875"}{2} \right)\left(\frac{3}{8}" \right)(58ksi) = 27.7^{k}$ Edge bolts control

       $2.4\left(\frac{3}{4}" \right)\left(\frac{3}{8}" \right)(58ksi) = 39.2^{k}$ Tearout controls

$$\varphi Rn = (0.75)(27.7^k + 39.2^k)$$
$$\varphi Rn = 50.2^k > Vu = 13.8^k \Rightarrow ok$$

Angle block shear:
$$Rn = 0.6FuAnv + UbsFuAnt \leq 0.6FyAgv + UbsFuAnt$$

$$Agt = (1.5")(\tfrac{3}{8}") = 0.5625in^2$$
$$Agv = (4.5")(\tfrac{3}{8}") = 1.6875in^2$$

$$Ant = 0.5625in^2 - 0.5(0.875")(\tfrac{3}{8}") = 0.398in^2$$
$$Anv = 1.6875in^2 - 1.5(0.875")(\tfrac{3}{8}") = 1.195in^2$$

$$0.6(58ksi)(1.195in^2) + (1.0)(58ksi)(0.398in^2) = 64.7^k$$
$$0.6(36ksi)(1.6875in^2) + (1.0)(58ksi)(0.398in^2) = 59.5^k \Rightarrow controls$$

$$\varphi Rn = (0.75)(59.5^k)$$
$$\varphi Rn = 44.6^k > Vu = 13.8^k \Rightarrow ok$$

APPENDIX C

C.1 – LEED CERTIFICATION – BUILDING ENVELOPE

Existing Wall System:
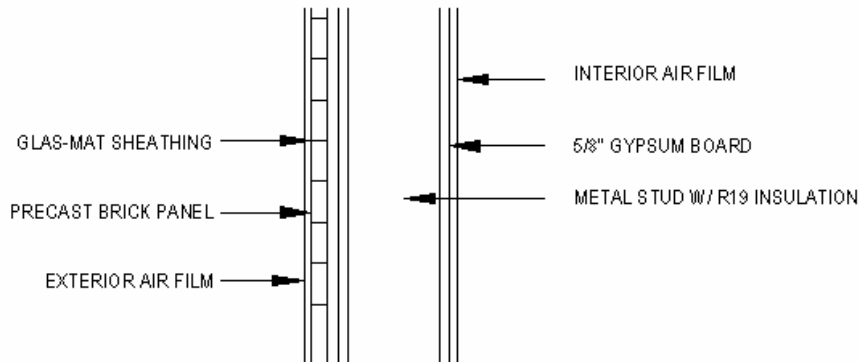
Material	R-Value
Outside Air Film	0.17
1 ¾" Precast Thin Brick	0.14
5/8" Glas-Mat Sheathing	3.0
6" Metal Stud w/ Insulation	19
5/8" Gypsum Board	0.56
Interior Air Film	0.68

Exterior air film: $\quad h_o = 6\,\dfrac{BTU}{hr - ft^2 - °F}$

Interior air film: $\quad h_i = 1.46\,\dfrac{BTU}{hr - ft^2 - °F}$

Wall system R value: $\Sigma R = 23.55$

$$Q = \frac{\Delta T}{\frac{1}{h_i} + \Sigma R + \frac{1}{h_o}} = \frac{72°F - 13°F}{\frac{1}{6} + 23.55 + \frac{1}{1.46}}$$

$$Q = 2.42\,\frac{BTU}{hr}$$

(Assuming 1 s.f. of wall area)

$$T_0 - T_1 = \frac{Q}{h_o}$$

$$13°F - T_1 = \frac{2.42}{6}$$

$$T_1 = 13.4°F$$

$$T_1 - T_2 = QR$$

$$13.4°F - T_2 = (2.42)(0.14)$$

$$T_2 = 13.7°F$$

$$T_2 - T_3 = QR$$

$$13.7°F - T_3 = (2.42)(3.0)$$

$$T_3 = 21.0°F$$

$$T_3 - T_4 = QR$$

$$21.0°F - T_3 = (2.42)(19)$$

$$T_4 = 67°F$$

$$T_4 - T_5 = QR$$

$$67.0°F - T_5 = (2.42)(0.56)$$

$$T_5 = 68.4°F$$
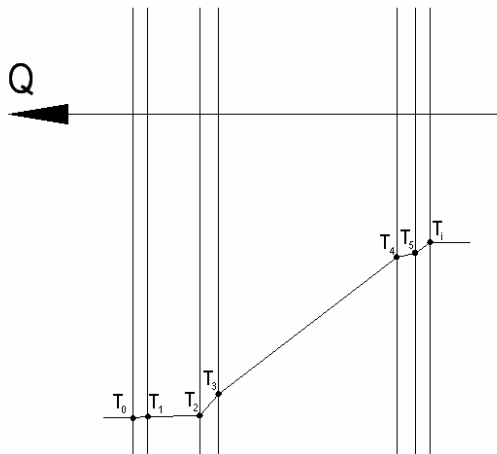
Heat Transfer, Q	2.42 BTU/hr
T_1	13.4
T_2	13.7
T_3	21.0
T_4	67.0
T_5	68.4

Alternative Wall System 1: Loose cellulose Insulation
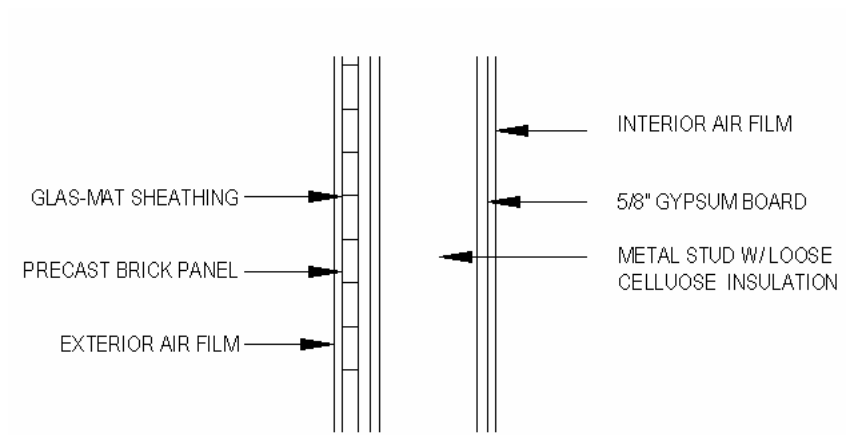
Material	R-Value
Outside Air Film	0.17
1 ¾" Precast Thin Brick	0.14
5/8" Glas-Mat Sheathing	3.0
6" Metal Stud w/ Insulation	22.8
5/8" Gypsum Board	0.56
Interior Air Film	0.68

Exterior air film: $\quad h_o = 6 \frac{BTU}{hr - ft^2 - °F}$

Interior air film: $\quad h_i = 1.46 \frac{BTU}{hr - ft^2 - °F}$

Wall system R value: $\Sigma R = 27.35$

$$Q = \frac{\Delta T}{\frac{1}{h_i} + \Sigma R + \frac{1}{h_o}} = \frac{72°F - 13°F}{\frac{1}{6} + 27.35 + \frac{1}{1.46}}$$

$$Q = 2.09 \frac{BTU}{hr}$$

(Assuming 1 s.f. of wall area)

Alternative Wall System 2: Increase Insulation Thickness
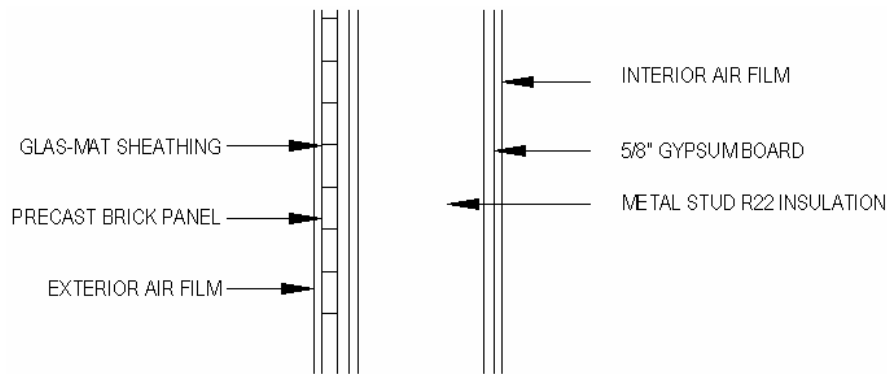
Material	R-Value
Outside Air Film	0.17
1 ¾″ Precast Thin Brick	0.14
5/8″ Glas-Mat Sheathing	3.0
8″ Metal Stud w/ Insulation	22
5/8″ Gypsum Board	0.56
Interior Air Film	0.68

Exterior air film: $\quad h_o = 6 \dfrac{BTU}{hr - ft^2 - °F}$

Interior air film: $\quad h_i = 1.46 \dfrac{BTU}{hr - ft^2 - °F}$

Wall system R value: $\Sigma R = 26.55$

$$Q = \dfrac{\Delta T}{\frac{1}{h_i} + \Sigma R + \frac{1}{h_o}} = \dfrac{72°F - 13°F}{\frac{1}{6} + 26.55 + \frac{1}{1.46}}$$

$$Q = 2.15 \dfrac{BTU}{hr}$$

(Assuming 1 s.f. of wall area)

C.2 – LEED CREDITS

Yes	?	No				
10				**Sustainable Sites**		**14** Points
Y			Prereq 1	**Construction Activity Pollution Prevention**		Required
1			Credit 1	**Site Selection**		1
1			Credit 2	**Development Density & Community Connectivity**		1
1			Credit 3	**Brownfield Redevelopment**		1
1			Credit 4.1	**Alternative Transportation**, Public Transportation Access		1
1			Credit 4.2	**Alternative Transportation**, Bicycle Storage & Changing Rooms		1
			Credit 4.3	**Alternative Transportation**, Low-Emitting & Fuel-Efficient Vehicles		1
1			Credit 4.4	**Alternative Transportation**, Parking Capacity		1
1			Credit 5.1	**Site Development,** Protect of Restore Habitat		1
1			Credit 5.2	**Site Development,** Maximize Open Space		1
1			Credit 6.1	**Stormwater Design,** Quantity Control		1
			Credit 6.2	**Stormwater Design,** Quality Control		1
			Credit 7.1	**Heat Island Effect,** Non-Roof		1
			Credit 7.2	**Heat Island Effect,** Roof		1
1			Credit 8	**Light Pollution Reduction**		1
Yes	?	No				
3				**Water Efficiency**		**5** Points
1			Credit 1.1	**Water Efficient Landscaping**, Reduce by 50%		1
			Credit 1.2	**Water Efficient Landscaping**, No Potable Use or No Irrigation		1
1			Credit 2	**Innovative Wastewater Technologies**		1
1			Credit 3.1	**Water Use Reduction**, 20% Reduction		1
			Credit 3.2	**Water Use Reduction**, 30% Reduction		1
2				**Energy & Atmosphere**		**17** Points
Y			Prereq 1	**Fundamental Commissioning of the Building Energy Systems**		Required
Y			Prereq 2	**Minimum Energy Performance**		Required
Y			Prereq 3	**Fundamental Refrigerant Management**		Required
2			Credit 1	**Optimize Energy Performance**		1 to 10
					10.5% New Buildings or 3.5% Existing Building Renovations	1
				2	14% New Buildings or 7% Existing Building Renovations	2
					17.5% New Buildings or 10.5% Existing Building Renovations	3

	21% New Buildings or 14% Existing Building Renovations	4
	24.5% New Buildings or 17.5% Existing Building Renovations	5
	28% New Buildings or 21% Existing Building Renovations	6
	31.5% New Buildings or 24.5% Existing Building Renovations	7
	35% New Buildings or 28% Existing Building Renovations	8
	38.5% New Buildings or 31.5% Existing Building Renovations	9
	42% New Buildings or 35% Existing Building Renovations	10

	Credit 2	**On-Site Renewable Energy**	1 to 3
		2.5% Renewable Energy	1
		7.5% Renewable Energy	2
		12.5% Renewable Energy	3
	Credit 3	**Enhanced Commissioning**	1
	Credit 4	**Enhanced Refrigerant Management**	1
	Credit 5	**Measurement & Verification**	1
	Credit 6	**Green Power**	1

continued…

Yes	?	No			
4			**Materials & Resources**		**13** Points

Yes	?	No			
Y			Prereq 1	**Storage & Collection of Recyclables**	Required
			Credit 1.1	**Building Reuse**, Maintain 75% of Existing Walls, Floors & Roof	1
			Credit 1.2	**Building Reuse**, Maintain 100% of Existing Walls, Floors & Roof	1
			Credit 1.3	**Building Reuse**, Maintain 50% of Interior Non-Structural Elements	1
1			Credit 2.1	**Construction Waste Management**, Divert 50% from Disposal	1
			Credit 2.2	**Construction Waste Management**, Divert 75% from Disposal	1
1			Credit 3.1	**Materials Reuse**, 5%	1
1			Credit 3.2	**Materials Reuse**, 10%	1
1			Credit 4.1	**Recycled Content**, 10% (post-consumer + ½ pre-consumer)	1
			Credit 4.2	**Recycled Content**, 20% (post-consumer + ½ pre-consumer)	1
			Credit 5.1	**Regional Materials**, 10% Extracted, Processed & Manufactured Regionally	1
			Credit 5.2	**Regional Materials**, 20% Extracted, Processed & Manufactured Regionally	1
			Credit 6	**Rapidly Renewable Materials**	1
			Credit 7	**Certified Wood**	1

Yes   ?   No

7			Indoor Environmental Quality	15 Points
Y			Prereq 1 Minimum IAQ Performance	Required
Y			Prereq 2 Environmental Tobacco Smoke (ETS) Control	Required
			Credit 1 Outdoor Air Delivery Monitoring	1
			Credit 2 Increased Ventilation	1
			Credit 3.1 Construction IAQ Management Plan, During Construction	1
			Credit 3.2 Construction IAQ Management Plan, Before Occupancy	1
			Credit 4.1 Low-Emitting Materials, Adhesives & Sealants	1
1			Credit 4.2 Low-Emitting Materials, Paints & Coatings	1
1			Credit 4.3 Low-Emitting Materials, Carpet Systems	1
			Credit 4.4 Low-Emitting Materials, Composite Wood & Agrifiber Products	1
1			Credit 5 Indoor Chemical & Pollutant Source Control	1
1			Credit 6.1 Controllability of Systems, Lighting	1
1			Credit 6.2 Controllability of Systems, Thermal Comfort	1
1			Credit 7.1 Thermal Comfort, Design	1
			Credit 7.2 Thermal Comfort, Verification	1
1			Credit 8.1 Daylight & Views, Daylight 75% of Spaces	1
			Credit 8.2 Daylight & Views, Views for 90% of Spaces	1
Yes	?	No		

			Innovation & Design Process	5 Points
			Credit 1.1 Innovation in Design: Provide Specific Title	1
			Credit 1.2 Innovation in Design: Provide Specific Title	1
			Credit 1.3 Innovation in Design: Provide Specific Title	1
			Credit 1.4 Innovation in Design: Provide Specific Title	1
			Credit 2 LEED® Accredited Professional	1
Yes	?	No		
26			Project Totals (pre-certification estimates)	69 Points

C.3 – MATERIAL QUANTITY TAKEOFFS

Concrete Quantities

Formwork		
	Quantity	
Columns	44000	sf
Beams	3500	sf
Slab	185000	sf
Walls	46330	sf
Total	278830	sf

Concrete		
	Quantity	
Columns	900	cy
Beams	60	cy
Slab	4500	cy
Walls	1400	cy
Total	6860	cy

Reinforcing		
	Quantity	
Columns	2150	cwt
Beams	200	cwt
Slab	18100	cwt
Walls	2000	cwt
Total	22450	cwt

Miscellaneous		
	Quantity	
Formwork Hardware	35400	ea
Formwork Release	311500	sf
Chamfer Strips	27000	lf
Slab Finishing	181000	sf
Slab Screeds	21720	lf
Shoring	362000	sf

Steel:

Steel		
	Quantity	
Beams	6000	cwt
Columns	1900	cwt
Angles	200	cwt
Frames	36200	cwt
Total	44300	cwt

Composite Slab		
	Quantity	
Metal Deck	181000	sf
Shear Studs	17660	ea
Concrete	1960	cy
WWF Reinforcing	1990	lf
Slab Finishing	181000	sf
Slab Screeds	21720	lf

Miscellaneous		
	Quantity	
Anchor Bolts	228	ea
Baseplates	57	ea
Grout	89	sf
Red Oxide	16000	sf
Gypsum Board	20000	bdf

C.4 – CONSTRUCTION COST

Material Costs:

Concrete		
	Unit Cost	
5000 psi Concrete	$ 60.00	/cy
3000 psi Concrete	$ 55.00	/cy
Formwork	$ 1.00	/sf
Reinforcing	$ 27.00	/cwt
Shoring	$ 3.00	/sf
Formwork Hardware	$ 0.50	ea
Formwork Release	$ 0.50	/sf
Chamfer Strips	$ 0.50	/lf
Slab Screeds	$ 1.00	/lf

Steel		
	Unit Cost	
Structural Steel	$ 35.00	/cwt
Anchor Bolts	$ 2.00	ea
Baseplates	$ 25.00	ea
Grout	$ 6.00	/sf
Shear Studs	$ 10.00	ea
Shop Paint	$ 1.00	/sf
Gypsum Board	$ 2.00	/bdf
Metal Deck	$ 5.00	/sf
WWF Reinforcing	$ 8.20	/cwt

Labor Costs:

	Concrete Crews	Rate / day
C235	Concrete Pump 6 Common Laborers 1 Common Laborer Foreman 1 Vibrator Operator	$ 1,512.00
C276	Concrete Finishing 1 Common Laborer 3 Concrete Finishers	$ 826.00
C311	Formwork 3 Carpenters 1 Carpenter Foreman 2 Common Laborers	$ 1,152.00
C321	Reinforcing Steel 6 Reinforcing Rodmen 1 Reinforcing Rodman Foreman	$ 1,780.00

	Steel Crews	Rate / day
C235	Concrete Pump 6 Common Laborers 1 Common Laborer Foreman 1 Vibrator Operator	$ 1,512.00
C276	Concrete Finishing 1 Common Laborer 3 Concrete Finishers	$ 826.00
C311	Formwork 3 Carpenters 1 Carpenter Foreman 2 Common Laborers	$ 1,152.00
C321	Wiremesh 1 Reinforcing Rodmen 6 Common Laborers	$ 1,321.00
C360	Baseplate Grout Crew 1 Concrete Finisher	$ 216.00
C509	Miscellaneous Metals 3 Steelworkers	$ 760.00

C510	Structural Steel 8 Steelworkers 1 Steelworker Foreman	$ 3,000.00
C917	Fireproofing 1 Misc. Fireproofing Laborer 1 Fireproofing Laborer	$ 330.00
C990	Paint 1 Common Laborer 5 Painters	$ 1,188.00

Equipment Costs:

Equipment Costs		
Concrete Pump	$ 6.00	/cy
Crane	$ 300.00	/day

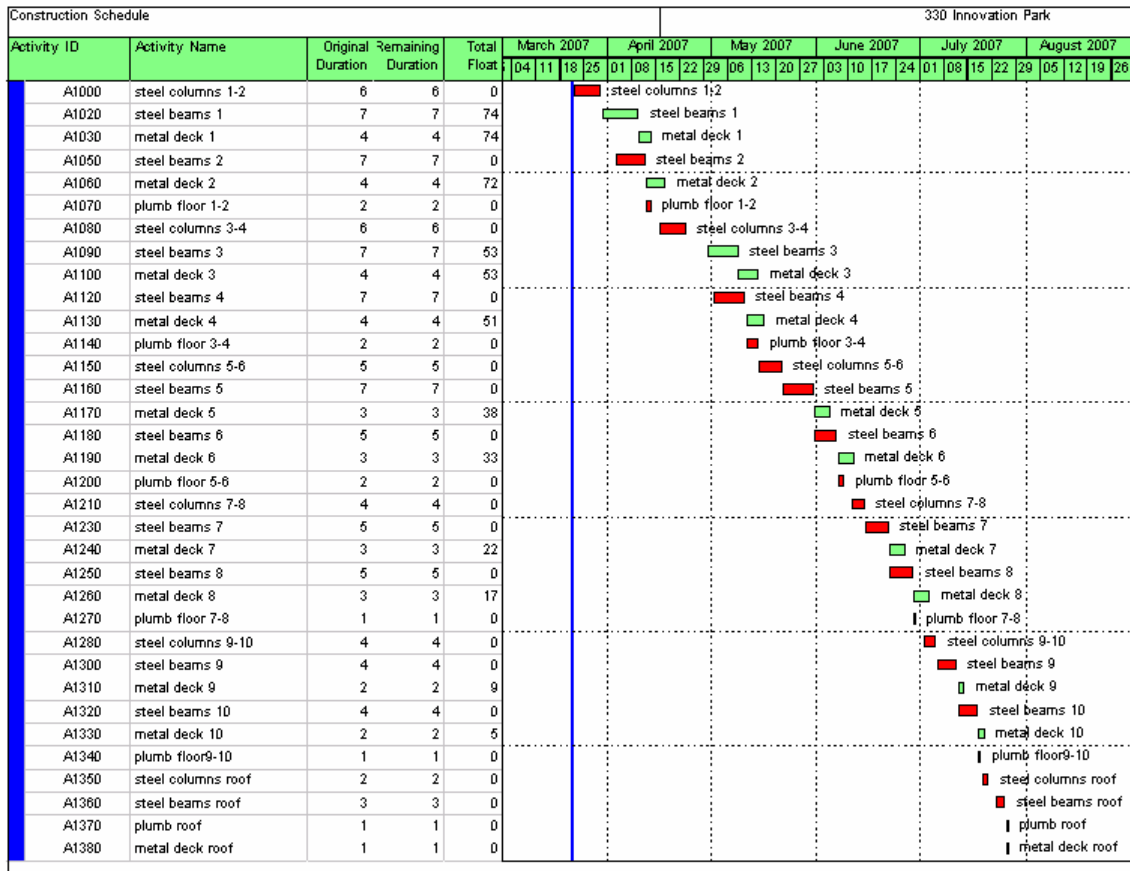|	Crews	# Crews	Crew Production	Total Production	# Days	Material Quantity	Material Unit Cost	Labor Unit Cost	Equipment Unit Cost	Total Material Cost	Total Labor Cost	Total Equipment Cost	Total Cost
Column Formwork	C311	4	600 sf/day	2400 sf/day	18	44000 sf	$1.00 /sf	$1,152.00 /day		$44,000.00	$84,480.00		$128,480.00
Slab Formwork	C311	7	600 sf/day	4200 sf/day	44	185000 sf	$1.00 /sf	$1,152.00 /day		$185,000.00	$355,200.00		$540,200.00
Beam Formwork	C311	6	600 sf/day	3600 sf/day	1	3500 sf	$1.00 /sf	$1,152.00 /day		$3,500.00	$6,720.00		$10,220.00
Wall Formwork	C311	7	600 sf/day	4200 sf/day	11	46330 sf	$1.00 /sf	$1,152.00 /day		$46,330.00	$88,953.60		$135,283.60
Formwork Hardware						35400 sf	$0.50			$17,700.00			
Formwork Release	C311	7	500 sf/day	3500 sf/day	89	311500 sf	$0.50 /sf	$1,152.00 /day		$155,750.00	$717,696.00		$873,446.00
Chamfer Strips	C311	7	700 lf/day	4900 lf/day	6	27000 lf	$0.50 /lf	$1,152.00 /day		$13,500.00	$44,434.29		$57,934.29
Column Reinforcing	C321	7	50 cwt/day	350 cwt/day	6	2150 cwt	$27.00 /cwt	$1,780.00 /day		$58,050.00	$76,540.00		$134,590.00
Slab Reinforcing	C321	7	50 cwt/day	350 cwt/day	52	18100 cwt	$27.00 /cwt	$1,780.00 /day		$488,700.00	$644,360.00		$1,133,060.00
Beam Reinforcing	C321	3	50 cwt/day	150 cwt/day	1	200 cwt	$27.00 /cwt	$1,780.00 /day		$5,400.00	$7,120.00		$12,520.00
Wall Reinforcing	C321	7	50 cwt/day	350 cwt/day	6	2000 cwt	$27.00 /cwt	$1,780.00 /day		$54,000.00	$71,200.00		$125,200.00
Column Pour	C235	1	100 cy/day	100 cy/day	9	900 cy	$60.00 /cy	$1,512.00 /day	$6.00 /cy	$54,000.00	$13,608.00	$5,400.00	$73,008.00
Slab Pour	C235	3	100 cy/day	300 cy/day	15	4500 cy	$60.00 /cy	$1,512.00 /day	$6.00 /cy	$270,000.00	$68,040.00	$27,000.00	$365,040.00
Beam Pour	C235	1	100 cy/day	100 cy/day	1	60 cy	$60.00 /cy	$1,512.00 /day	$6.00 /cy	$3,600.00	$907.20	$360.00	$4,867.20
Wall Pour	C235	4	100 cy/day	400 cy/day	4	1400 cy	$60.00 /cy	$1,512.00 /day	$6.00 /cy	$84,000.00	$21,168.00	$8,400.00	$113,568.00
Slab Finishing	C276	7	500 sf/day	3500 sf/day	52	181000 sf		$826.00 /day			$299,012.00		$299,012.00
Slab Screeds	C311	3	250 lf/day	750 lf/day	29	21720 lf	$1.00 /lf	$1,152.00 /day		$21,720.00	$100,085.76		$121,805.76
Shoring	C235	3	600 sf/day	1800 sf/day	70	362000 sf	$3.00 /sf	$1,512.00 /day		$1,086,000.00	$105,840.00		$1,191,840.00
Crane					140		2		500 /day		$0.00	$140,000.00	$140,000.00
										$2,591,250.00	$2,705,364.85	$181,160.00	$5,460,074.85

Concrete Structure Estimate

	Crews	# Crews	Crew Production	Total Production	# Days	Material Quantity	Material Unit Cost	Labor Unit Cost	Equipment Unit Cost	Total Material Cost	Total Labor Cost	Total Equipment Cost	Total Cost
Steel Beams	C510	2	100 cwt/day	200 cwt/day	30	6000 cwt	$35.00 /cwt	$3,000.00 /day		$210,000.00	$180,000.00		$390,000.00
Steel Columns	C510	2	100 cwt/day	200 cwt/day	10	1900 cwt	$35.00 /cwt	$3,000.00 /day		$66,500.00	$57,000.00		$123,500.00
Steel Angles	C510	2	100 cwt/day	200 cwt/day	1	200 cwt	$35.00 /cwt	$3,000.00 /day		$7,000.00	$6,000.00		$13,000.00
Steel Frame	C510	3	100 cwt/day	300 cwt/day	121	36200 cwt	$35.00 /cwt	$3,000.00 /day		$1,267,000.00	$1,086,000.00		$2,353,000.00
Anchor Bolts	C509	1	100 /day	100	2	228	$2.00	$760.00		$456.00	$1,732.80		$2,188.80
Base Plates	C510	1	65 /day	65 /day	1	57	$25.00	$3,000.00 /day		$1,425.00	$2,630.77		$4,055.77
Grout	C360	2	35 sf/day	70 sf/day	1	89 sf	$6.00 /sf	$216.00		$534.00	$549.26		$1,083.26
Shear Studs	C509	3	400 /day	1200 /day	15	17659	$10.00	$760.00 /day		$176,590.00	$33,552.10		$210,142.10
Red Oxide	C990	2	400 sf/day	800 sf/day	20	16000 sf	$1.00 /sf	$1,188.00 /day		$16,000.00	$47,520.00		$63,520.00
Gypsum Board	C917	1	300 bdf/day	300 bdf/day	67	20000 bdf	$2.00 /bdf	$330.00 /day		$40,000.00	$22,000.00		$62,000.00
Metal Deck	C510	10	600 sf/day	6000 sf/day	30	181000 sf	$5.00 /cwt	$3,000.00 /day		$905,000.00	$905,000.00		$1,810,000.00
WWF Reinforcing	C320	2	100 sqs/day	200 sqs/day	10	1990 sqs	$8.20 /cwt	$1,321.00 /day		$16,318.00	$26,287.90		$42,605.90
Concrete Slab	C235	3	100 cy/day	300 cy/day	7	1960 cy	$55.00 /cy	$1,512.00 /day	$6.00 /cy	$107,800.00	$29,635.20	$11,760.00	$149,195.20
Slab Finishing	C276	6	500 sf/day	3000 sf/day	60	181000 sf		$826.00 /day			$299,012.00		$299,012.00
Slab Screeds	C311	6	250 lf/day	1500 lf/day	14	21720 lf	$1.00 /lf	$1,152.00 /day		$21,720.00	$100,085.76		$121,805.76
Crane					93	2			$300 /day		$0.00	$55,800.00	$55,800.00
										$2,836,343.00	$2,797,005.79	$67,560.00	$5,700,908.79

Steel Structure Estimate

C.4 – CONSTRUCTION SCHEDULE

Crew Production:

Crew Production		
C235	100	ct/day
C276	155	sf/day
C311	250	lf/day
C320	100	sqs/day
C360	35	sf/day
C509	100	/day
C510	100	cwt/day
	600	sf/day
C917	300	bdf/day
C990	400	sf/day

| 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

APPENDIX D

1st AND 2nd FLOOR PLAN

The Towers at The City
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STRUCTURAL PLAN - FLOOR 1 - 2

SCALE: $\frac{1}{32}$" = 1'-0"

S1.1

The Towers at The City College of New York

STRUCTURAL PLAN - FLOOR 3 - 6

SCALE: $\frac{1}{32}$" = 1'-0"

S1.2
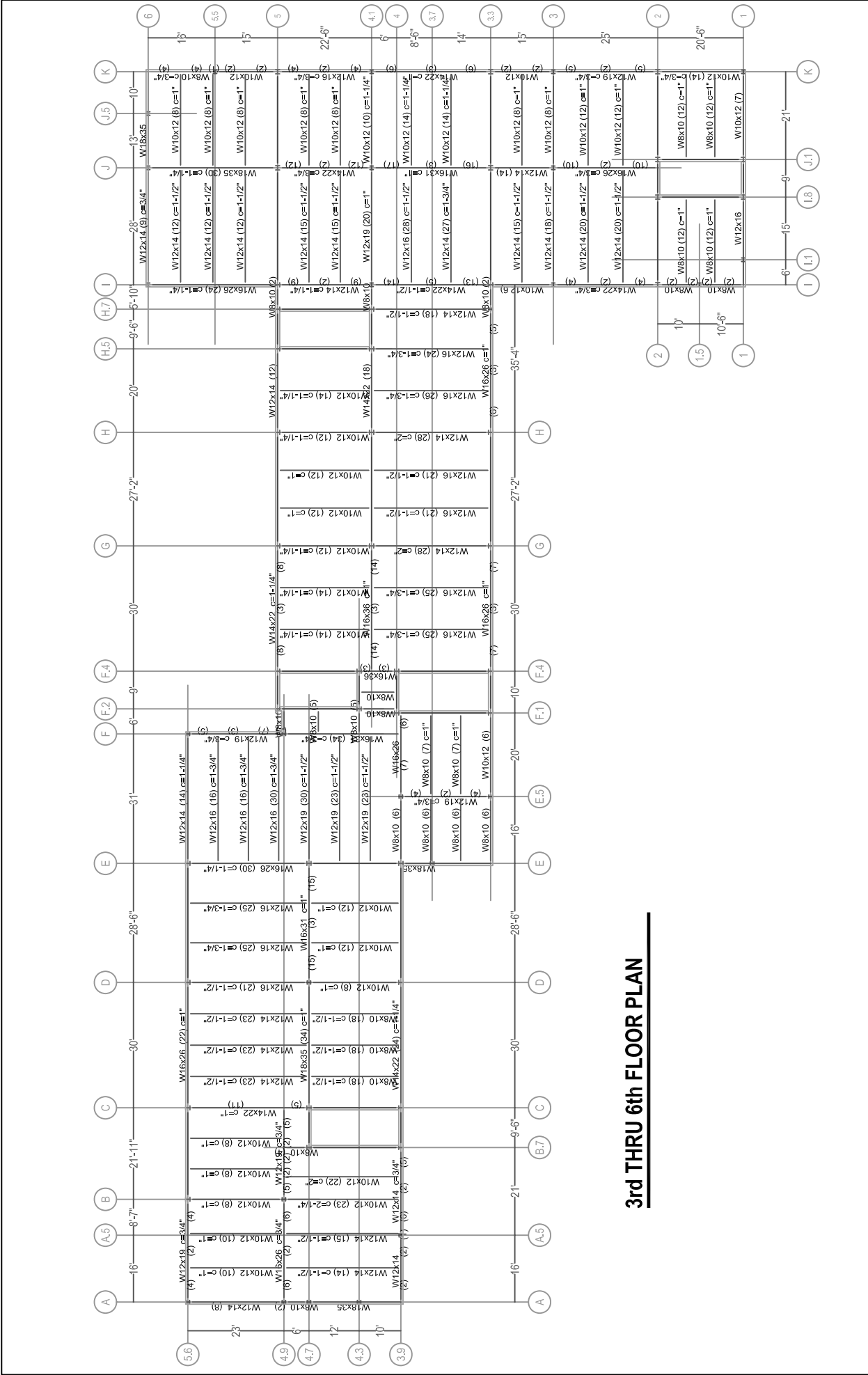
3rd THRU 6th FLOOR PLAN
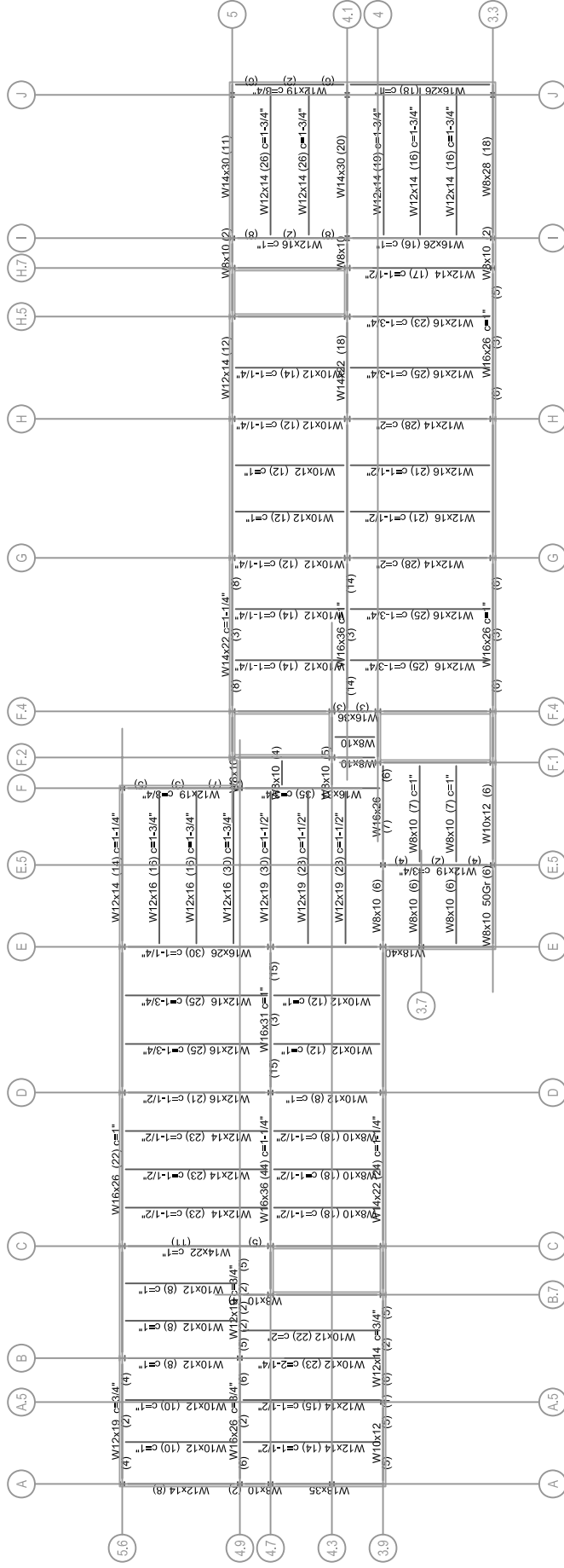
The Towers at The City College of New York

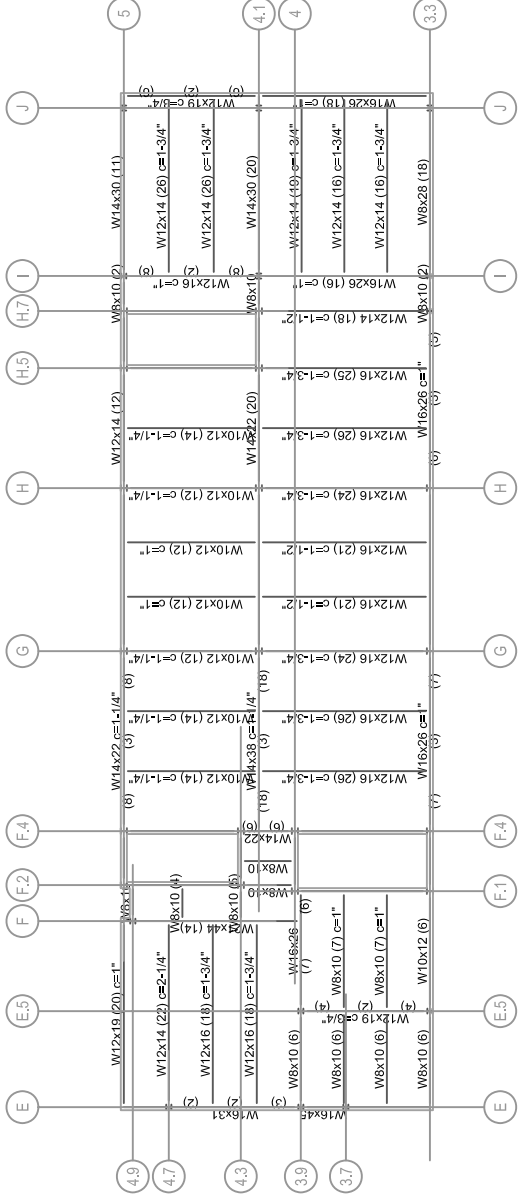**STRUCTURAL PLAN - FLOOR 7 - 9**

SCALE: $\frac{1}{32}" = 1'-0"$

S1.3

7th THRU 9th FLOOR PLAN

S1.4
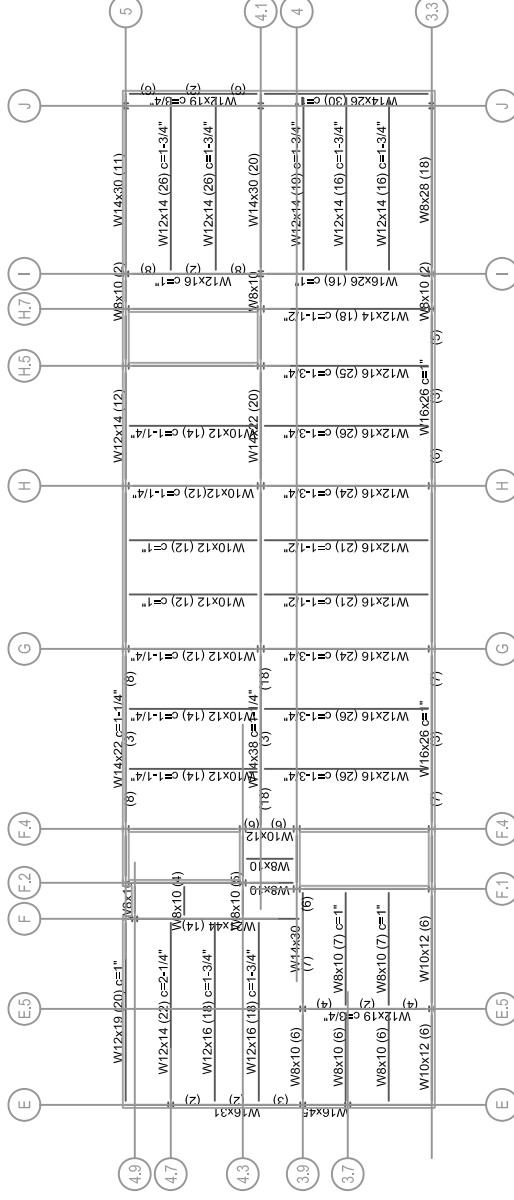
STRUCTURAL PLAN - FLOOR 10 - ROOF

SCALE: $\frac{1}{32}$" = 1'-0"

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ROOF PLAN

10th FLOOR PLAN
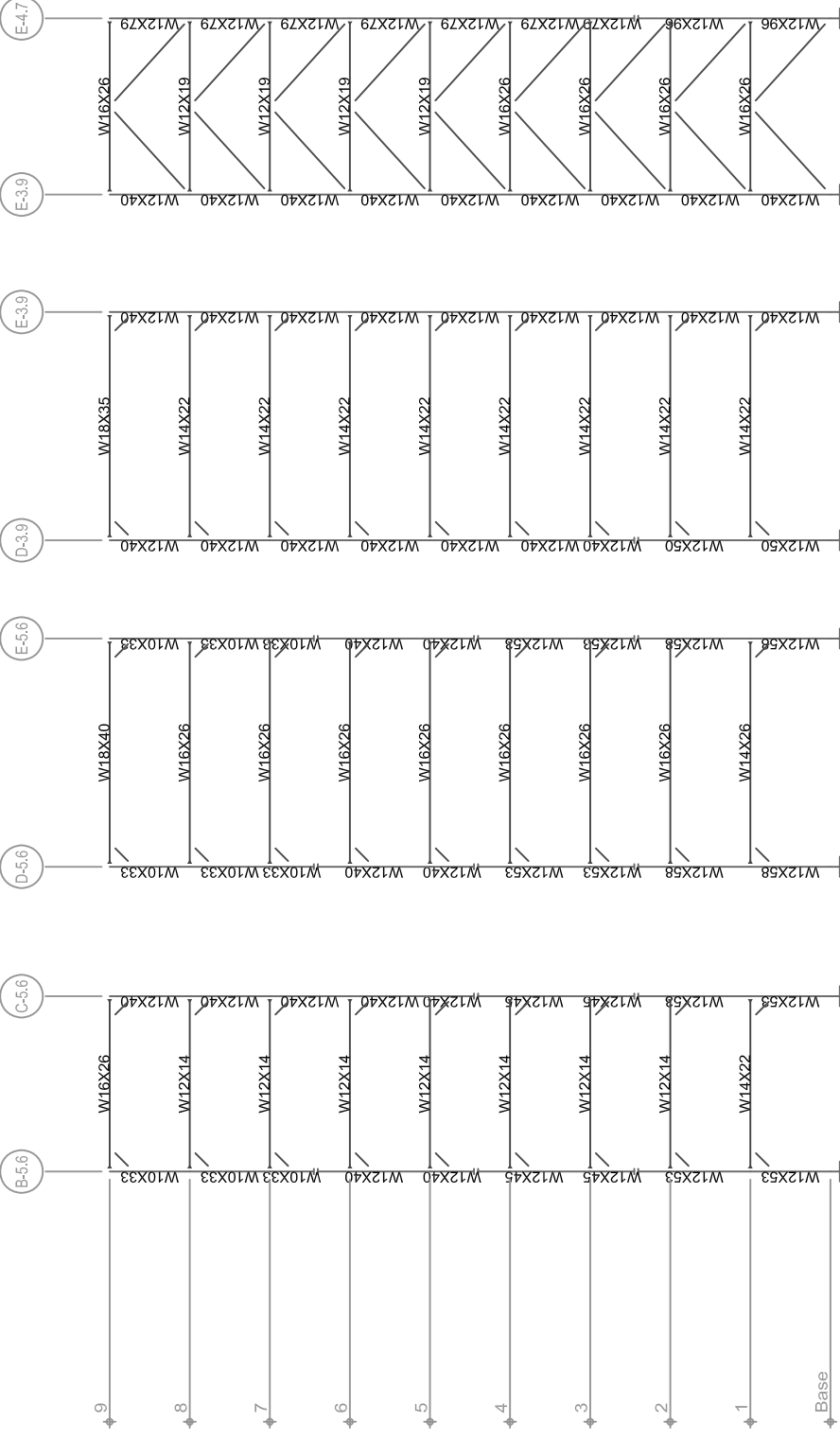
The Towers at The City College of New York

FRAME ELEVATIONS

SCALE: $\frac{1}{32}$" = 1'-0"

S2.1

FRAME ELEVATIONS

SCALE: $\frac{1"}{32}$ = 1'-0"

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College of New York

S2.2

S2.3

FRAME ELEVATIONS

SCALE: $\frac{1}{32}" = 1'\text{-}0"$

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Frame H-4.1 / G-4.1 (top members): W12X40, W12X40, W12X40, W12X50, W12X50, W12X58, W12X58, W12X65, W12X65, W12X79, W12X79

Diagonals: W16X31 (×11)

Bottom members: W12X40, W12X40, W12X53, W12X53, W12X65, W12X65, W12X79, W12X79, W12X96, W12X96

Frame H-5 / G-5: top and bottom chords W12X65; verticals W14X22 (×10), W14X26

Frame F-4.3.3 / F-1-3.3: W10X12, W12X58

Frame F-4.4 / F-1.4: W10X12, W12X58

Frame F-4.4.3 / F-2.4.3: W10X12, W12X58

Frame F-4-5 / F-2-5: W10X12, W12X58
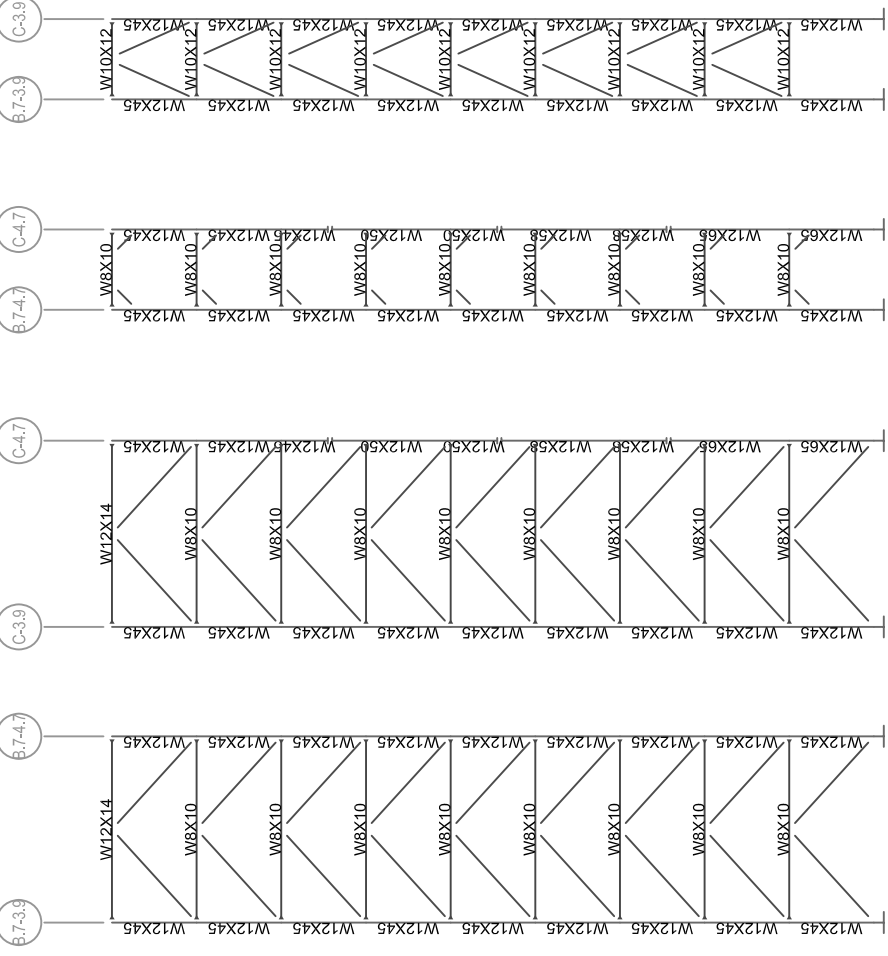
FRAME ELEVATIONS

SCALE: $\frac{1}{32}$" = 1'-0"

The Towers at The City
College of New York
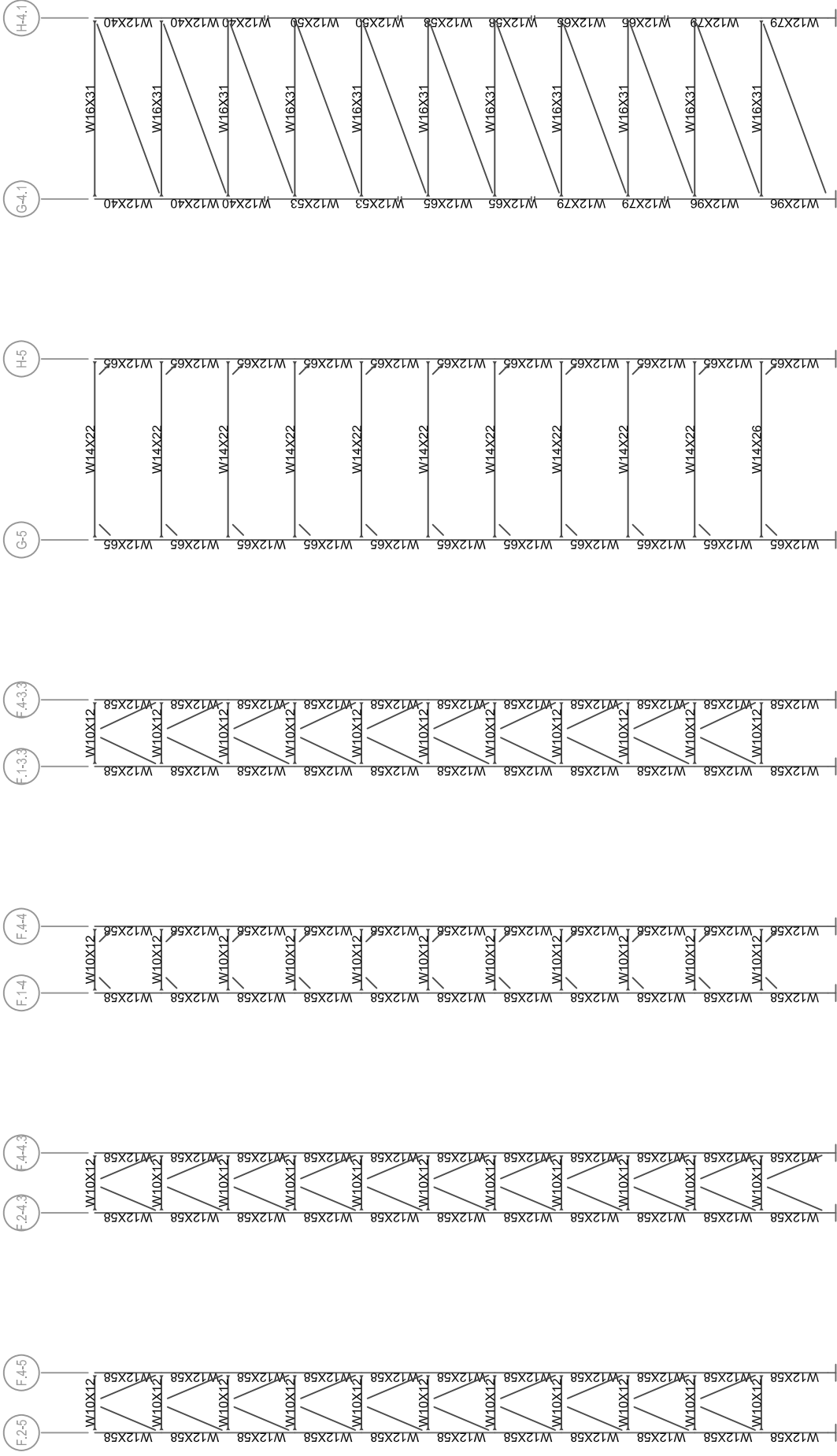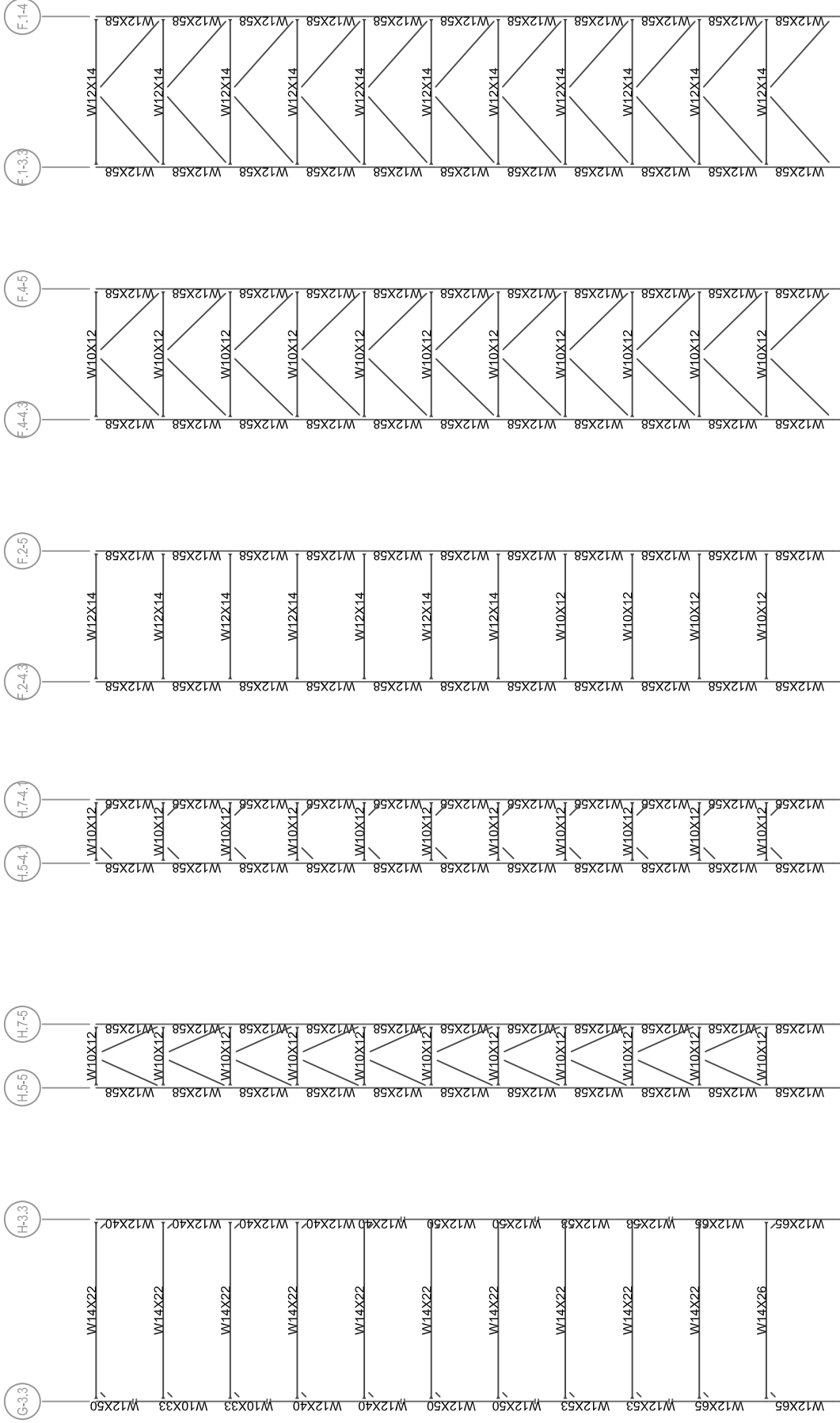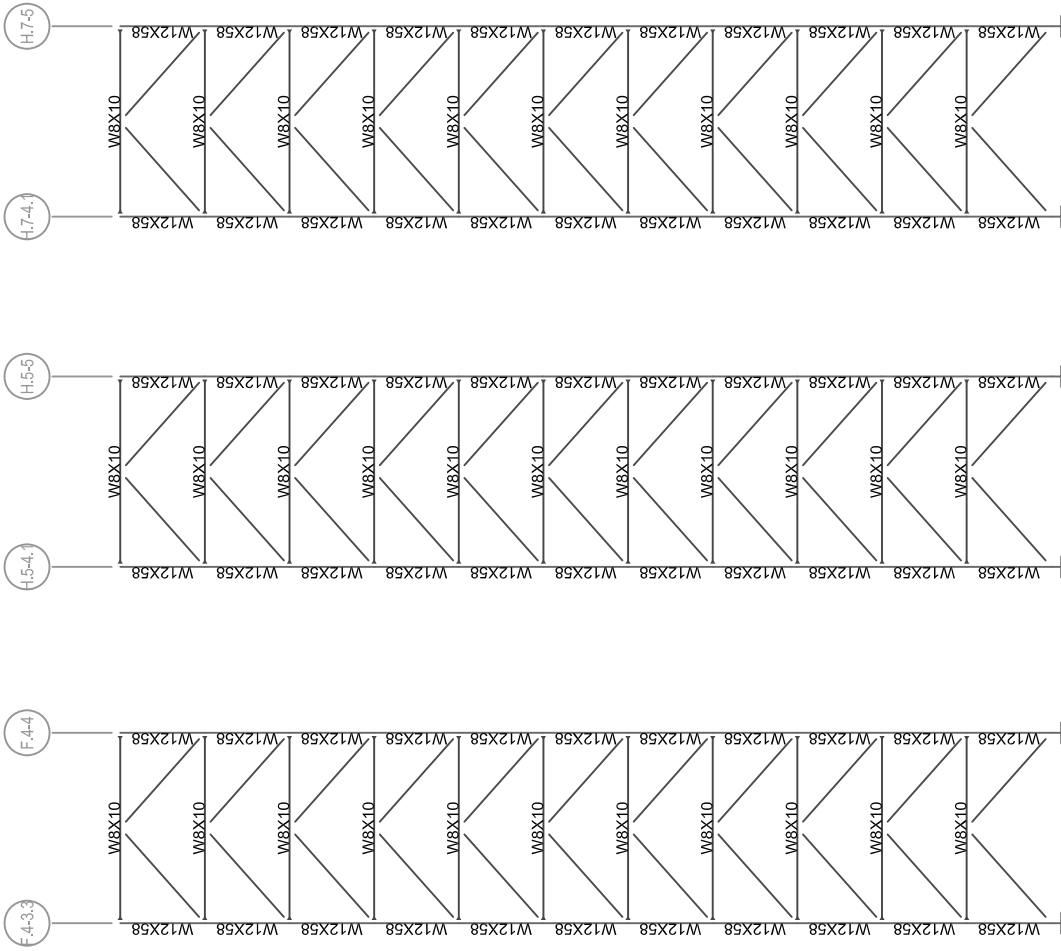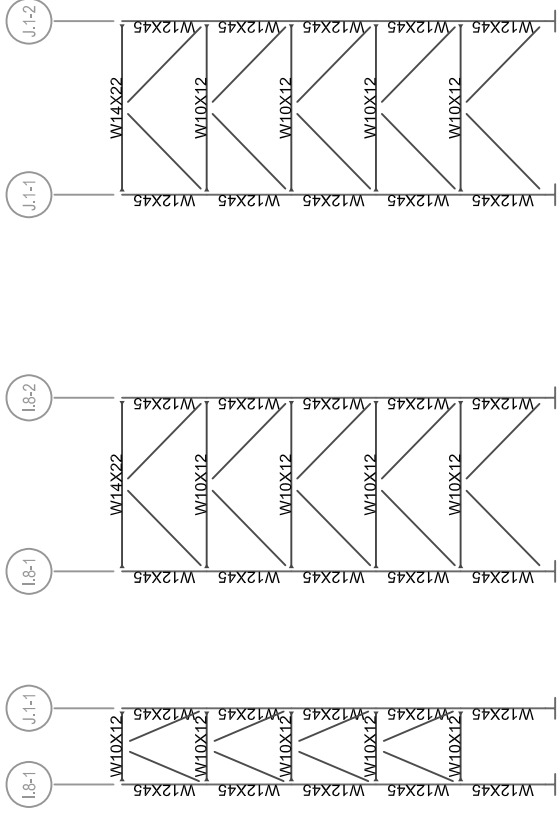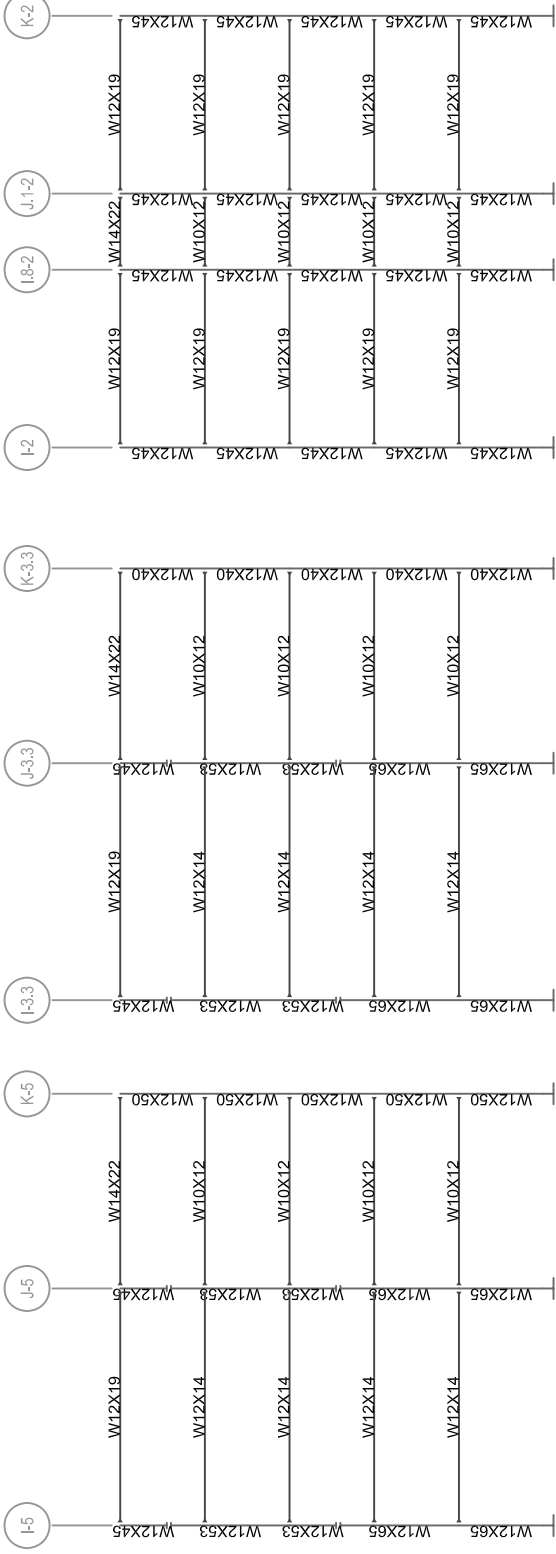
S2.4

FRAME ELEVATIONS

SCALE: $\frac{1}{32}$" = 1'-0"

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College of New York

S2.5

FRAME ELEVATIONS

Frame — Grid K-2 to I-2

| Grid | K-2 | J.1-2 | I.8-2 | I-2 |

- Beams (top, left to right): W12X45, W12X45, W12X45, W12X45
- Columns: W12X19, W12X19, W12X19, W12X19, W12X19
- Second beam line: W12X45, W12X45, W12X45, W12X45, W12X45 (W14X22 at left, W10X12 interior)
- Columns: W12X19, W12X19, W12X19, W12X19, W12X19
- Bottom beam: W12X45, W12X45, W12X45, W12X45, W12X45

Frame — Grid K-3.3 to I-3.3

- Top beam: W12X40, W12X40, W12X40, W12X40
- Columns: W14X22, W10X12, W10X12, W10X12, W10X12
- Middle beam: W12X45, W12X53, W12X53, W12X65, W12X65
- Columns: W12X19, W12X14, W12X14, W12X14, W12X14
- Bottom beam: W12X45, W12X53, W12X53, W12X65, W12X65

Frame — Grid K-5 to I-5

- Top beam: W12X50, W12X50, W12X50, W12X50
- Columns: W14X22, W10X12, W10X12, W10X12, W10X12
- Middle beam: W12X45, W12X53, W12X53, W12X65, W12X65
- Columns: W12X19, W12X14, W12X14, W12X14, W12X14
- Bottom beam: W12X45, W12X53, W12X53, W12X65, W12X65

Braced Frame — Grid J.1-2 to J.1-1

- Top beam: W12X45, W12X45, W12X45, W12X45, W12X45
- Columns: W14X22, W10X12, W10X12, W10X12, W10X12
- Bottom beam: W12X45, W12X45, W12X45, W12X45, W12X45

Braced Frame — Grid I.8-2 to I.8-1

- Top beam: W12X45, W12X45, W12X45, W12X45, W12X45
- Columns: W14X22, W10X12, W10X12, W10X12, W10X12
- Bottom beam: W12X45, W12X45, W12X45, W12X45, W12X45

Braced Frame — Grid J.1-1 to I.8-1

- Top beam: W12X45, W12X45, W12X45, W12X45, W12X45
- Columns: W10X12, W10X12, W10X12, W10X12, W10X12
- Bottom beam: W12X45, W12X45, W12X45, W12X45, W12X45