2010

HUNTER COLLEGE SCHOOL OF SOCIAL WORK

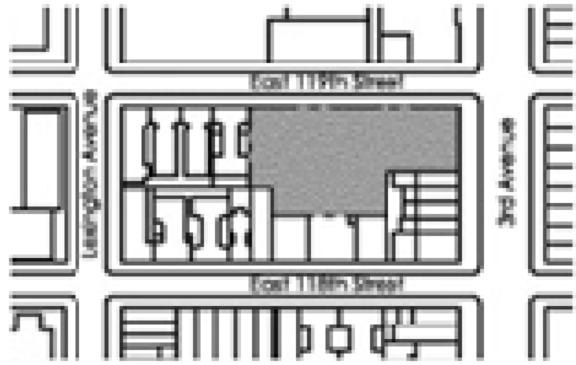
Vanessa Rodriguez | Structural
Advisor: Prof. Ali Memari
4/7/2010

Hunter College School of Social Work

New York, NY

Vanessa Rodriguez | Structural Emphasis

2009 - 2010



Project Team

Owner	The City University of New York
Developer	The Brodsky Organization
Architect	Cooper Robertson & Assoc.
Structural	Ysrael A. Seinuk, P.C.
LEED	Viridean Energy, LLC
MEP	WSP Flack + Kurtz
CM	Turner Construction Company

Structural System

Mat Foundation of thickness between 30" and 40" on a subgrade of undisturbed soil or compacted back with a bearing capacity of 1.5 tons

Gravity system of steel columns and fully composite metal deck

Column sizes vary from W14x68 to W14x233

Lateral system consists of cross bracing of hollow structural steel diagonal members and moment connections

Project Information

Floors	8 stories
Project Size	148,000 sq. ft.
Construction Time	July 2009 to August 2011

Mechanical System

hot water panel radiators

Two Hot water boiler are in the basement level with a gross output of 3300 mbh

Cooling towers located on the 5th floor

Electrical System

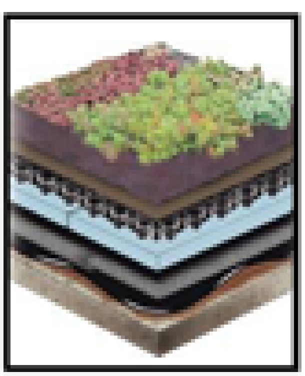
208Y/120V 3- phase, 4 wire electrical system

Two main 2000 amp switchboards power the panel boards on each floor

Six different buses feed these panel boards; maximum 2000 amps

400KW emergency generator

Sustainability



- Green Roofs
- Energy Star label appliances
- Low mercury lamps for all fluorescent, compact fluorescent, and HID lamps
- GREENGUARD Certification for systems furniture and seating.
- Low Emitting Carpet.

Architectural Design

The entrance lobby, conceived as an interior street, is glazed from floor to ceiling along 119th Street to provide a transparent and welcoming appearance from the exterior and to link the interior of the building to its neighborhood surroundings. The School of Social Work building will be LEED certified.

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Executive Summary

This report is the culmination of a yearlong study performed on the Hunter College School of Social Work project located on Third Avenue between 118th and 119th street. It is designed to be both a college and university space. The structure is comprised of a composite steel floor system that utilizes steel braced and moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The cellar floor is a reinforced slab on a mat foundation. The total height is 133ft above ground level.

The focus of this report is energy efficiency and how it can be implemented using facade and green roof redesign. It ties structural engineering concepts with existing enclosure installation methods to provide a secure barrier against water and the temperature of the outside world.

Enclosure design is important to ensure the life of a structure in addition to continual building maintenance. Simple and inexpensive measures can be taken to significantly improve the buildings energy efficiency. This project goal was inspired by the School of Social Work building's current goal of achieving LEED certification.

Along with the installation of a new LEED certified façade and the expansion of the green roofs, the structures supporting these systems were also analyzed. This includes the gravity framing system as well as the storm water management tank dunnage platform.

In addition to these changes, the lateral system was converted into a completely braced frame system instead of a combined system, the savings due to these changes would pay for the green roof additions four times over.

The lateral system used a combination of diagonal and chevron bracing, depending on the bay span. The chevron connection was detailed using the Uniform Force Method, and The diagonal member was analyzed as special case 2: Uniform Force Method.

Introduction

The structure of Hunter College School of Social Work is comprised of a composite steel floor system that utilizes steel braced and moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The cellar floor is a reinforced slab on a mat foundation. The total height is 133ft above ground level.

The building's design responds to the School of Social Work's mission by providing an open and engaging face to the neighborhood and opportunities for community use of parts of the facility. The entrance lobby, conceived as an interior street, is glazed from floor to ceiling along 119th Street to provide a transparent and welcoming appearance from the exterior and to link the interior of the building to its neighborhood surroundings. Classrooms and lecture halls occupy the lower levels with academic departments and offices on upper floors. An auditorium on the second floor is expressed on the facade, with a glazed wall allowing views of activity in and outside the building. A rear landscaped terrace will link the School to a planned CUNY Residential building adjacent to the site on 118th Street. The School of Social Work building will be LEED certified. -Cooper Robertson & Associates

Building Statistics

Name:	Hunter College School of Social Work
Location:	2180 Third Ave. New York, New York
Site:	East Harlem
Building Occupant Name:	The City University of New York
Occupancy or Function Types:	School and Faculty Offices
Size:	Approximately 148,000 Square Feet
Total Number of Stories:	5+3+ Penthouse
Dates of Construction:	Demolition started July 2009. Finish date is August 2011
Actual Cost Information:	This is not public information
Project Delivery Method:	Design-Bid-Build

Primary Project Team		
Owner	City University of New York	www.cuny.edu
Developer	East 118 Developer, LLC c/o The Brodsky Organization	www.brodskyorg.com
Construction Manager	Turner Construction Company	www.turnerconstruction.com
Design Architect	Cooper, Robertson & Partners	www.cooperrobertson.com
Architect of Record	SLCE Architects	www.slcearch.com
Structural Engineers	Ysrael A. Seinuk, P.C.	www.yaseinuk.com
MEP/FP/IT Engineer	WSP Flack + Kurtz	www.wspgroup.com
LEED Consultant	Viridian Energy and Environment, LLC	www.viridianee.com
Lighting Design	SBLD Studio	sbldstudio.com
Landscape Architect	Mathews Nielsen	www.mnlandscape.com
Audio/Visual & Acoustical	Cerami Associates	www.ceramiassociates.com
Security Consultants	Ducibella Venter & Santore	dvssecurity.com
Elevator Consultant	VDA	www.vdassoc.com
Signage Consultant	TWO TWELVE	www.twotwelve.com

Architecture

The building's design responds to the School of Social Work's mission by providing an open and engaging face to the neighborhood and opportunities for community use of parts of the facility. The entrance lobby, conceived as an interior street, is glazed from floor to ceiling along 119th Street to provide a transparent and welcoming appearance from the exterior and to link the interior of the building to its neighborhood surroundings. Classrooms and lecture halls occupy the lower levels with academic departments and offices on upper floors. An auditorium on the second floor is expressed on the faÃ§ade, with a glazed wall allowing views of activity in and outside the building. A rear landscaped terrace will link the School to a planned CUNY Residential building adjacent to the site on 118th Street. The School of Social Work building will be LEED certified.

The future building is meant to replace Hunter College School of Social Work's present building (below) while providing a modern environment for its graduate students. The existing building on 79th street is in stark contrast, to the proposed building, with its heavy gray stone façade. The ziggurat (set-backs) can still be seen as an important feature in the new building.

Figure 1: 79th and Third Ave. Location

Figure 2: Proposed Bldg., 119th and Thrid Ave. (North Elev.)

Setback laws in New York City were set to ensure daylight reached the streets and dwellings of New Yorkers. The use of the glass curtain wall removes the need for the setbacks on this building, yet they are kept as reminiscent of the past.

The 148,000-square-foot building "will have five large floors at its base and three smaller floors set back, and will exceed the current school by more than 38000 square feet" (NYTimes). In the elevation shown above, three distinct horizontal levels represent the building's various uses. These levels are architecturally visible, and along with its transparency, the new structure will provide a feeling of openness and welcome to the community of East Harlem. Along the large glass exposure facing Third Ave. there will be a public café along with community spaces.

Verticality is also a dominating architectural feature, showing the building's transition from community and commercial use to university use above.

The proposed facade of Hunter College School of Social Work resembles that of its neighbor; a luxury condominiums high rise. The triumph of engineering over physics is showcased with a seemingly heavy masonry middle section being upheld by a thin sheet of glass. However, the "masonry" referred to is really precast panels which have half bricks set into to make it look like a brick façade, this panel is then attached and "hung" off of the structural steel. The same goes for the curtain wall glass. It is attached by anchors to the building structural steel.

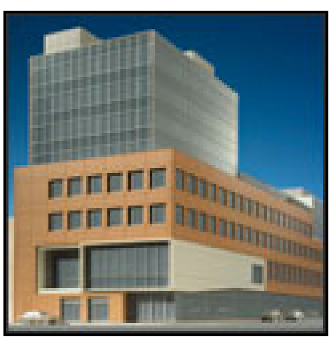


Figure 3: Rendering of the New School

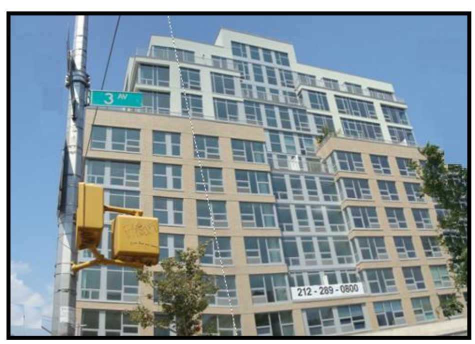


Figure 4: Neighboring Luxury Condominiums

Building Enclosure

Building Façade

In the North elevation (see drawing on page 3) the bottom band is UNITIZED C.W. 8"x 2 ½" two-aided curtain wall with custom cap with both transparent panels and spandrel shadow boxes. The left side of the middle band is architectural precast concrete while the right side is brick-faced precast panel in stack bond pattern with false jointing. The top band is UNITIZED C.W. 6 ¾" x 3" four-sided structurally glazed curtain wall with both transparent panels and spandrel shadow boxes. Above the main top band there is a vertical protrusion whose façade is 1"stucco on cmu substrate.

Similarly the South elevation has this same pattern of horizontal bands of varying material. There is however a change in the color of the stucco as you go up in elevation.

Unlike the North and South elevations, the East and West elevations don't present the horizontal banding clearly, instead it transitions into more vertical bands of varying material. From left to right these materials are 6" nominal cmu, 1" stucco, 6"nominal cmu again, brick-faced precast panel, and 1" stucco again. This vertical pattern applies up to the fifth floor, above that, the horizontal bands of stucco and glass curtain wall persist.

Windows and Glazing

Recycled aluminum windows shall have vision panels with factory glazed laminated "Low E" vision glass, tempered insulated glass, and insulated glass at shadow boxes and lecture hall. There is also tempered insulated glass widely used on the building façade. The clear "Low E" coating (U-value=0.32) was chosen to comply with the Energy Conservation Construction Code of New York State.

Typical Roofing

The typical roof is an IRMA roof, inverted roof membrane. The membrane is unreinforced with a nominal thickness of 90 mis and an exposed face color of white. Insulating Materials can be either Perlite Board Roof Insulation or Perlite/Polyisocyanurate Composite Board Roof Insulation. Flashing must be an elastomeric flashing sheet.

In the roofing construction, adhesives, sealants, and paints must be low-emitting and comply with the LEED specifications. The fasteners should be of at least sixty percent recycled steel as well as do other miscellaneous steel materials used on the roofing. Roof paver are specified as heavyweight concrete units.

Green Roofs

Green roofs are located on the first and second floors. These roofs vary from intensive to extensive green roofs. They are known to help with the heat island effect, keeping the building cool during hot summers and insulated during the winter months. Located on the library deck, this provides an environment conductive to learning.

Drainage materials for the green roof are three-dimensional molded panels of recycled material with drainage channels top and bottom sides and water retention reservoirs on the top side. This water is filtered with a non-woven, polymeric, geotextile fabric. After it is filtered a moisture mat composed of recycled, non-rotting, polypropylene fibers stitched through a polyethylene carrier sheet retains the water.

The growing medium is LiteTop lightweight engineered soil which provides a stable structure for the anchorage of the plants root system while remaining as light as possible to prevent excess loading of the roof structure. It also supplies essential nutrients, water and oxygen to the plant life.

Figure 5: Extensive Green Roof, American Hydrotech

Figure 6: Intensive Green Roof, American Hydrotech

Construction

Project delivery was design-bid-build. Demolition and abatement began July 2009 and expected completion date is August 2011. Turner Construction was the general contractor for the project. The site for Hunter College School of Social Work contained three buildings scheduled for demolition. Some of these buildings contained asbestos and the asbestos had to be contained before demolition could begin.

The new construction will be built against existing buildings and will therefore have to be careful not to damage its foundation. Because the water table is only a few feet below ground level, during excavation, dewatering will be a necessity especially during the winter months when melted snow brings up the water level. With the site located in an urban area, transportation of material to the site will be a major challenge.

Structural

The structural system for Hunter College School of Social Work is a steel frame system with composite slab on metal deck and composite and non-composite beams. Mat Foundation of varying thicknesses between 30" and 40" on a subgrade of undisturbed soil or compacted backfill with a bearing capacity of 1.5 tons. For the gravity system column sizes vary from W14x68 to W14x233. The lateral load resisting system consists of cross bracing of hollow structural steel diagonal members and moment connections.

Foundation System

There is one below-grade level in the Hunter College School of Social Work. This level known as the cellar contains a parking garage for the residential building adjacent, a library, computer labs, large kitchen areas, and mechanical rooms.

Slab thickness varies throughout the cellar level. It can be 30", 33", or 40". Steel reinforcement varies according to the slab thickness. For a 30" slab #7@11 are required top and bottom (T&B) each way, for a 33" slab #8@13 top and bottom, and for a 40" slab #9@13 top and bottom each way. The mat foundation will have a 2" mud slab above 12" of ¾ crushed stone to facilitate installation of waterproofing membrane. The subgrade is composed of undisturbed soil or compacted back fill with a required bearing capacity of 1.5 tons.

The soil is not considered susceptible to liquefaction for a Magnitude 6 earthquake and a peak ground acceleration of 0.16g. It is expected to encounter ground water during erection of the cellar level. Excavation depths are anticipated to vary from about 12ft to 20ft below existing ground surface grades. Footings shall bear on sound rock with a bearing capacity of 20 ton per square foot or on decomposed rock with a bearing capacity of 8 ton per square foot or on sand with a bearing capacity of 3 ton per square foot.

Foundation walls are designed to resist lateral pressures resulting from static earth, groundwater, adjacent foundations, and sidewalk surcharge loads. These walls will extend 14ft below existing ground surface grades. Concrete for foundations and site work shall be air-entrained normal weight stone concrete with a minimum compressive strength of 4000psi at 28 days and a maximum water to cement ratio of 0.45 by weight.

In the western portion of the six story faculty housing building footprint, it is recommended to excavate rock 12" below bottom of foundation in order to limit differential settlement between sections of the mat foundation bearing on rock and that bearing on soil.

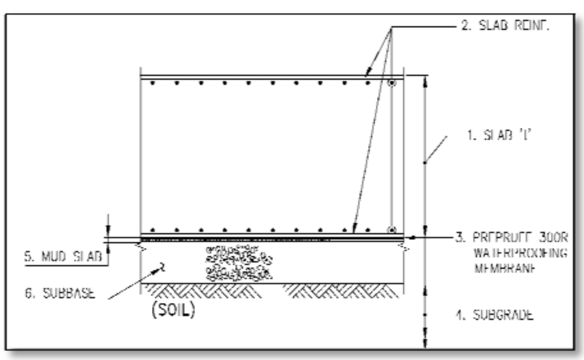


Figure 7: Mat Foundation Detail

Gravity System

Columns in the basement are 4000psi air-entrained concrete and vary in size from 32x48 to 36x60. The bay sizes vary from 30'x28', 30'x 28'2", 30'x31'5" and 30'x36' from north to south respectively.

All columns in the superstructure are W14s. Due to setbacks and varying story footprint, service loads carried by the columns at the ground level vary ranging from 137 to 1154kips. Because the service loads vary greatly throughout the floor, the column sizes vary as well; for example, on the ground floor column sizes range from w14x68 to w14x730. In the levels above the cellar, the bay sizes do not change.

There are non-composite beams as well as composite beams (with studs). Non-composite beams are found where beam to beam, and beam to column connections are designed to transfer the reaction for a simply supported, uniformly loaded beam. For composite beams, connections are designed to have 160% capacity of the reaction for a simply supported, uniformly loaded beam of the same size, span, fy, and allowable unit stress. For framed beam connections, including single plate connections, the minimum number of horizontal bolt rows should be provided based on 3" center-to-center.

Roof System

The roof is typically composed of 3 1/2 "light weight concrete over 3"-18 gage metal deck reinforced with 6x6-2.9x2.9 WWF. In a 200 square foot section the slab is 8" lightweight concrete slab reinforced with #4@12 top and bottom E.W. Columns are placed where needed and don't necessarily follow a typical framing layout. To provide additional vibration control, 4" concrete pads are located below mechanical equipment. Curbs on the roof are of CMU and concrete.

Floor System- Composite steel beam and deck floor system

The slab thickness for all floors is 3 ¼" thick 3500psi lightweight concrete placed over 3" deep 18 gage composite galvanized metal deck reinforced with 6x6- W2.9xW2.9 welded-wire-fabric. Exceptions on the ground floor are on the outdoor court, entry vestibules, and loading area; here 3" lightweight concrete is placed over 16 gage metal deck is used and instead of WWF, reinforcement is #4@12" o.c. top bars each way and 1-#5 bottom bars each rib. The exception for the second floor is the roof terrace where there is 5" of lightweight concrete over 3"-16 gage metal deck. On the roof level, the floor slab for the electrical control room is 8" lightweight concrete formed slab reinforced with to#4@12"o.c. top and bottom each way.

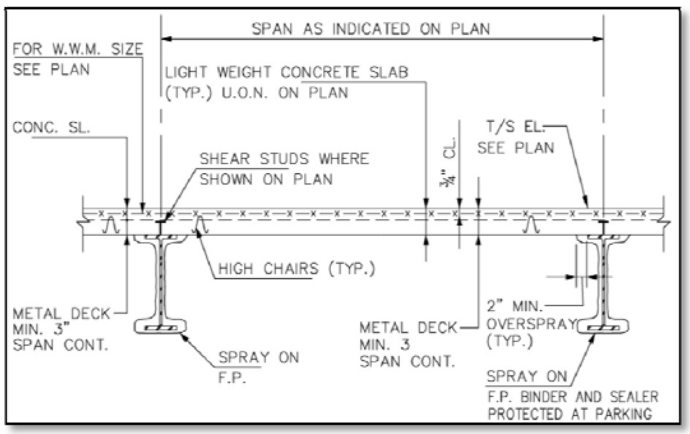


Figure 8: Typical Floor Construction. Metal Deck Perpendicular to Beams or Girders

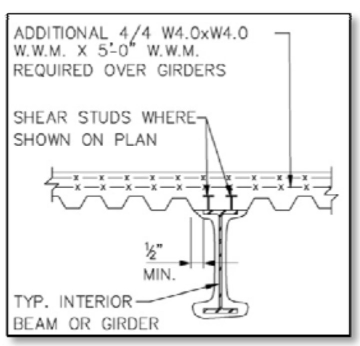


Figure 9: Typical Floor Construction. Metal Deck Parallel to Beams or Girders

Lateral System

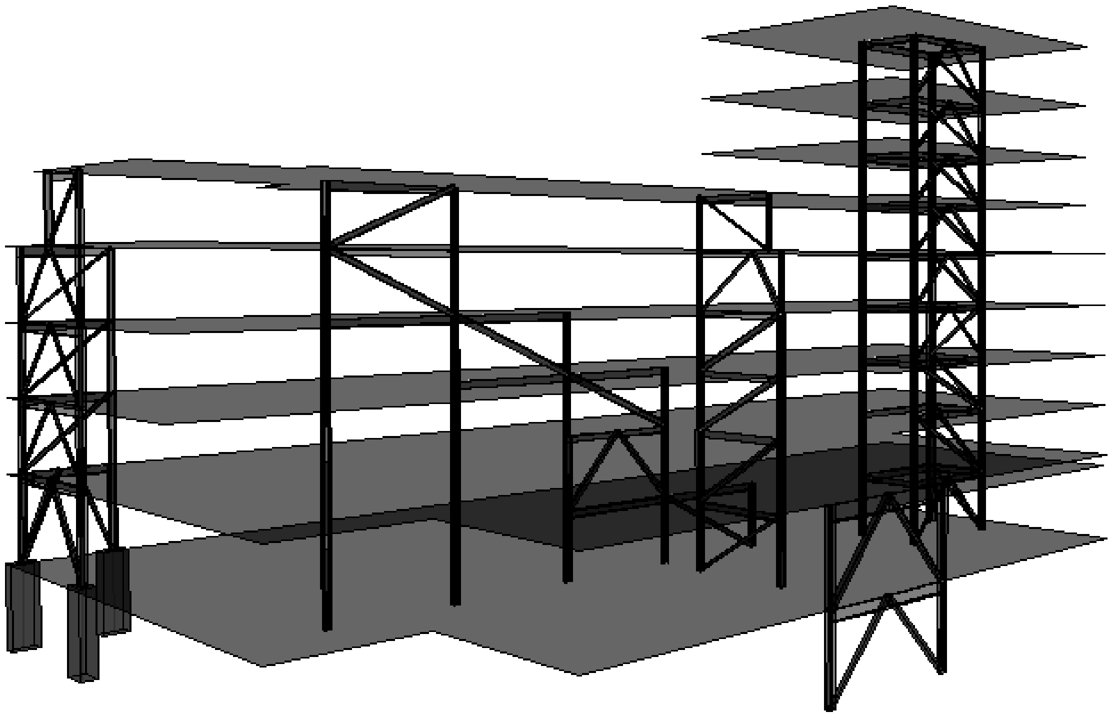


Figure 10: ETABS Model of the Lateral Force Resisting System

The lateral system is made up of braced frames and moment frames. Braced frames with column splices at four feet above floor level with vertical members attached using moment connections make up the lateral system. Locations of these frames are represented on figure 2 in red; they run all the way up to the top of the building. The only exception to this is the braced frame represented on figure 2 as blue since it changes as you go up in elevation. An elevation view of this truss is shown as figure 3. Braced frames were chosen to resist lateral forces because they are more efficient than moment frames in both cost and erection time. The exceptions are the two moment frames used to surround the storm water detention tank. Moment frames provide unobstructed access to the tank that would not be possible if it was a braced frame. The other two frames surrounding the tank are in fact braced frames.

The remainder of this report further analyses the existing lateral force system. ETABS was used for the lateral analysis of Hunter College School of Social Work, and hand calculations were performed to verify results from the program output. Members of the braced frame and moment frame were checked for strength and drift requirements. Throughout this report, frames will be referred to in reference to their location as shown in figure 2.

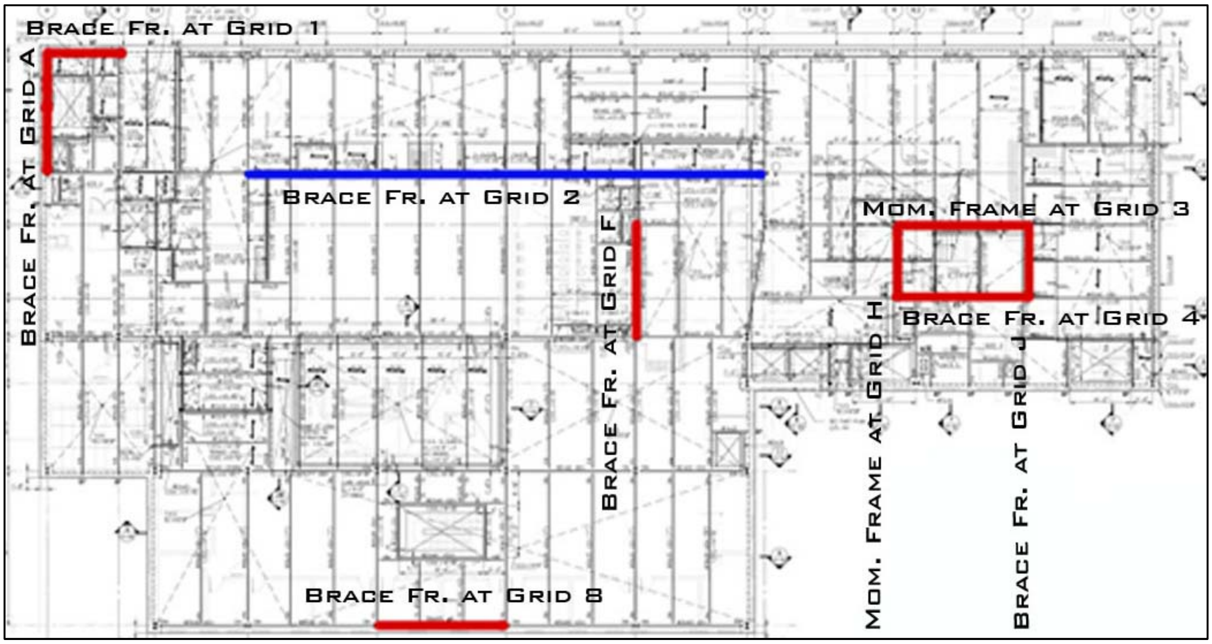
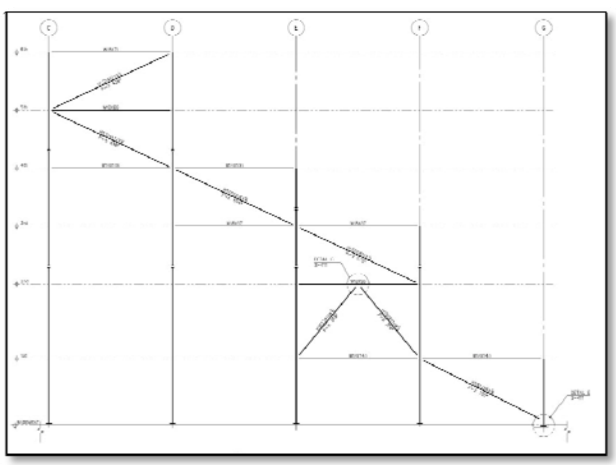


Figure 11: Location of Lateral Force Resisting System



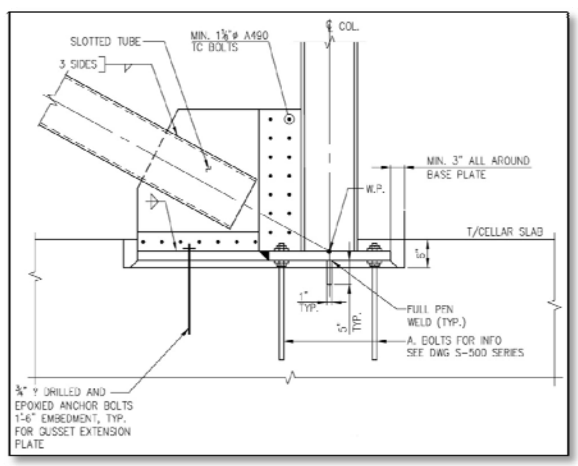


Figure 12: Truss Elevation at Grid 2

Figure 13: Lateral Load Connection

Problem Statement

Problem 1: the vertical core is made up of a combination of braced and moment frames.
Moment frames are more costly than braced frames. This is because they are many times field welded, making it riskier and more time consuming than braced connections.

Problem 2: building façade is susceptible to water and air infiltration
The façade is composed of various building materials which increases the potential for water and air infiltration. Water is the number one damaging agent to building materials. It rusts metals and fosters mold growth, making it an unhealthy breathing environment for its occupants.

As seen on the North elevation (below) the bottom band is 8"x 2 ½" two-aided curtain wall with custom cap with both transparent panels and spandrel shadow boxes. The left side of the middle band is architectural precast concrete while the right side is brick-faced precast panel in stack bond pattern with false jointing. The top band is 6 ¾" x 3" four-sided structurally glazed curtain wall with both transparent panels and spandrel shadow boxes. Above the main top band there is a vertical protrusion whose façade is 1"stucco on cmu substrate. Similarly the South elevation has this same pattern of horizontal bands of varying material.

Unlike the North and South elevations, the East and West elevations don't present the horizontal banding clearly, instead it transitions into more vertical bands of varying material. From left to right these materials are 6" nominal cmu, 1" stucco, 6"nominal cmu again, brick-faced precast panel, and 1" stucco again. This vertical pattern applies up to the fifth floor, above that, the horizontal bands of stucco and glass curtain wall persist.

Figure 14: North Elevation of Hunter College School of Social Work

Proposed Solutions and Methods

Problem 1: the vertical core is made up of a combination of braced and moment frames.
Solution 1: revise all moment frames to braced frames

The new vertical core which is a large part of the lateral load resisting system, should with stand gravity, seismic, and wind loads. The vertical core will be revised so that it is made up of braced frames only instead of a combination of braced frames and moment frames.

An etabs model of the existing lateral load resisting system will be created. A new model incorporating the changes of the vertical core will be compared to it. Changes in story drift, story shears, and relative stiffness of lateral elements will be analyzed along with lateral member spot checks.

Problem 2: building façade is susceptible to water and air infiltration
Solution 2: redesign of façade for improved waterproofing and incorporating thermal dampers

To ensure that the building is sealed tight against water penetration and that the outside temperature doesn't greatly affect the interior environment, there will be thermal dampers on exterior structural members. A redesign of the façade will be conducted for improved waterproofing and incorporation of the thermal dampers. Along with the redesign of the façade, the perimeter structural framing will be changed to better incorporate the new façade.

An analysis of the enclosure will be done to determine possible areas of improvement. Areas of weakness are expected to be wherever there is a transition of building material. Since this occurs often on the building façade, it is expected that there will be many areas in need of improvement.

Alternative materials through manufacturers' catalogs; which have been preapproved to be used in accordance with the LEED rating system, will be chosen if they better improve the building's performance with respect to energy efficiency. The effect of the alternative materials will be analyzed. These include the impact on the structural system, cost, and time.

Graduate Course Integration

Steel Connections will also be addressed in the redesign of façade connections to the structural steel. The connections will be analyzed for applicable failure modes. These include shear, bearing, tear-out, etc. The building enclosures class is expected to be heavily integrated with this thesis. Building façade connectivity to structural members will also be analyzed for ease of installation.

Following the main structural depth study, a minimum of two breath studies will also be performed for this proposal. These include a cost analysis including savings due to shorter erection time. The second breath will be a redesign of the green roof and building façade to increase energy efficiency.

Breadth I. Construction Impact and Cost Analysis

Changing the moment frames to braced frames is expected to have an impact on erection time, the savings associated with this will be analyzed. In addition, the new façade with thermal dampers will also have an effect on the erection time, it may either increase or decrease the construction schedule, however it is expected that the energy savings will supplant the added initial cost.

Breadth II. Redesign of green roof and façade for energy efficiency.

The building is currently going for LEED certification. Green roof filtration systems will be looked at in more detail and façade connectivity to structural members will be analyzed as well. A green roof redesign will be performed as well since they currently cover two roof levels. The water retention tank capacity may increase or decrease accordingly.

The viability of the new green roof and water retention tank will be analyzed against cost, time of placement, and complexity of labor.

Structural Depth Study

Code and Design Requirements

Applied to original Design

The Building Coded of the City of New York (most current) - Amended seismic design

AISC-LRFD, LRFD Specification for Structural Steel Buildings (applied except on the lateral force resisting frame)

AISC- ASD 1989, Specifications for Structural Steel Buildings- ASD and Plastic Design (for the design and construction of steel framing in lateral force resisting system)

ACI 318-89, Building Code Requirements for Structural Concrete

Substituted for thesis analysis

2006 International Building Code

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Material strength requirement summary

Structural Steel:
- All W Beams and Columns: ASTM A992, Fy=50ksi
- HSS Steel, Fy=46ksi
- Connection Material:Fy=36 ksi
- Base plates: ASTM 572 GR50, Fy=50ksi

Metal Decking:
- Units shall be 3” galvanized composite deck of 18 gage formed with integral locking lugs to provide a
  mechanical bond between concrete and deck

-Strength: Fy=40ksi

-Deflection of form due to dead load of concrete and deck does not exceed L/180 , but not more than ¾”

-Deflection of composite deck cannot exceed L/360 of deck span under superimposed live load.

Concrete:

-Caissons and Piers: 4000psi normal weight concrete

-Slabs on ground and footings: 4000psi normal weight concrete

-Retaining Walls: 4000 psi normal weight concrete

-Slab on deck: 3500psi lightweight concrete

- Foundations: 4000psi, air entrained, normal weight

-Walls, curbs, and parapets: 4000 psi

Reinforcement:

-Strength: 60ksi

Building Load Summary

Gravity Loads

Total building weight was found to be approximately 15,388kips. Detailed charts in Appendix A tabulate the columns and beams used in finding the total weight. Curtain wall weight was approximated to be 15 psf although curtain wall type varies as you go up in elevation. Glass curtain wall is used on the upper and lower sections of the building façade and precast masonry and stucco panels are used on the middle section of the building façade.

Calculation of the building weight was tedious due to the varying bay sizes, column and beam sizes, and varying lengths of these members. In erection of the structure, careful coordination must be taken in order to correctly identify and place these frame elements.

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
Penthouse	134	80750	0	38245	0	118995
Roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
					Total Building Weight:	15388153.12

Figure 15: Building Load Summary

ID	location	Live Loads (psf)			Dead Loads (psf)
		Design Live Loads	ASCE 705-05	NYC BLDG CODE 08	Design Dead Loads
1	loading dock	600	-	-	150
2	1st floor	100	100	100	130
3	podium	100	100	-	200
4	archive	350	-	-	75
5	offices	50	50	50	71
6	roof with garden	100	100	100	365
7	library stacks	100	100	100	71
8	classrooms	40	40	60	71
9	corridor	100	100	100	71
10	auditorium	60	60	100	85
11	roof with pavers on 2	100	-	-	150
12	roof	45	20	30	90
13	roof with drift	60	45	-	85
14	mechanical	100	125	100	120

Figure 16: Loading Schedule

Wind Load Summary

Since the Hunter College School of Social Work is located in New York City, the NYC Building Code governed the structural design. For this analysis, however, ASCE-7-05 was used along with Fanella Wind Analysis flowcharts. For detailed calculations please refer to Appendix A. In the north/south direction the base shear due to lateral wind loads was found to be 559 kips, much larger than in the East/West direction; 162 kips. This difference in base shear is due to building's rectangular shape as opposed to a square footprint. Wind forces were found to be much higher than seismic forces (figure 14). Seismic base shear was found to be 154 kips, less than wind-caused shear in either direction; north/south or east/west.

Due to the building's setbacks, it has differing roof levels, creating a potential for snow drifts. The allowable snow drift calculations were found to be 46psf (refer to Appendix A for details). The allowable snow drift values, along with the wind or seismic analysis, were not checked against the values originally found by the structural designers. The information needed was not provided on the construction documents for verification.

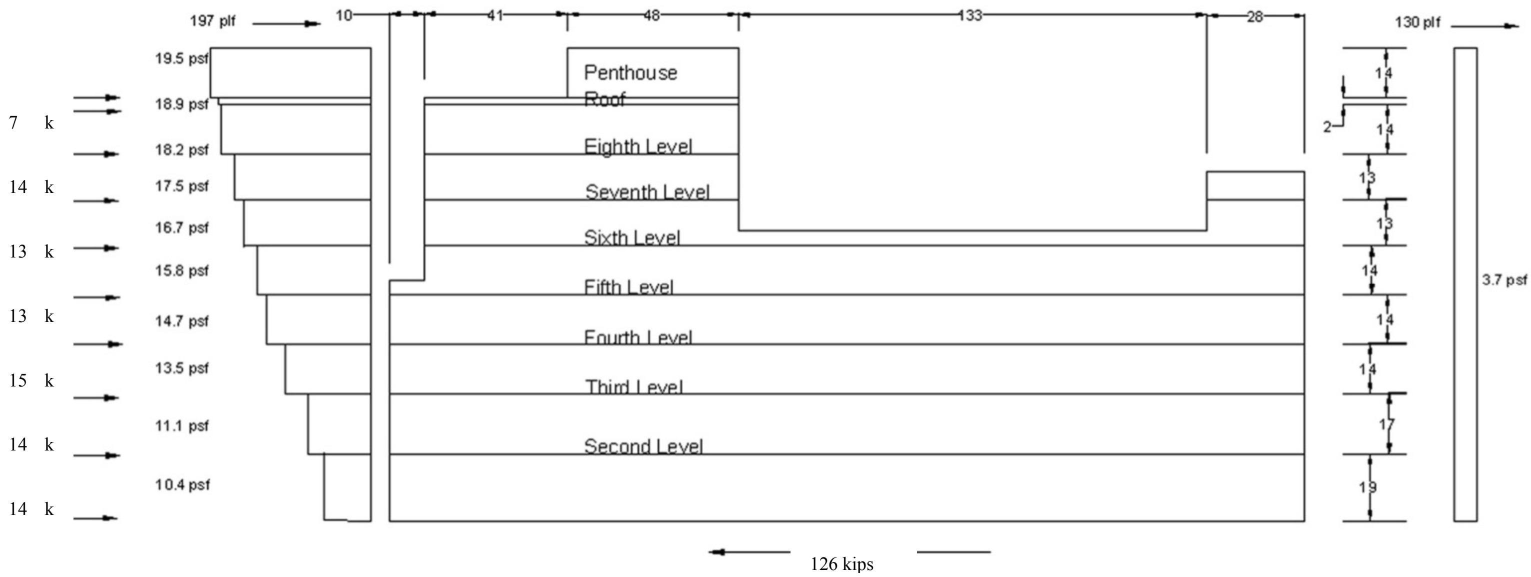


Figure 17: Wind Load Diagram using ASCE 7-05 in East/West winds direction

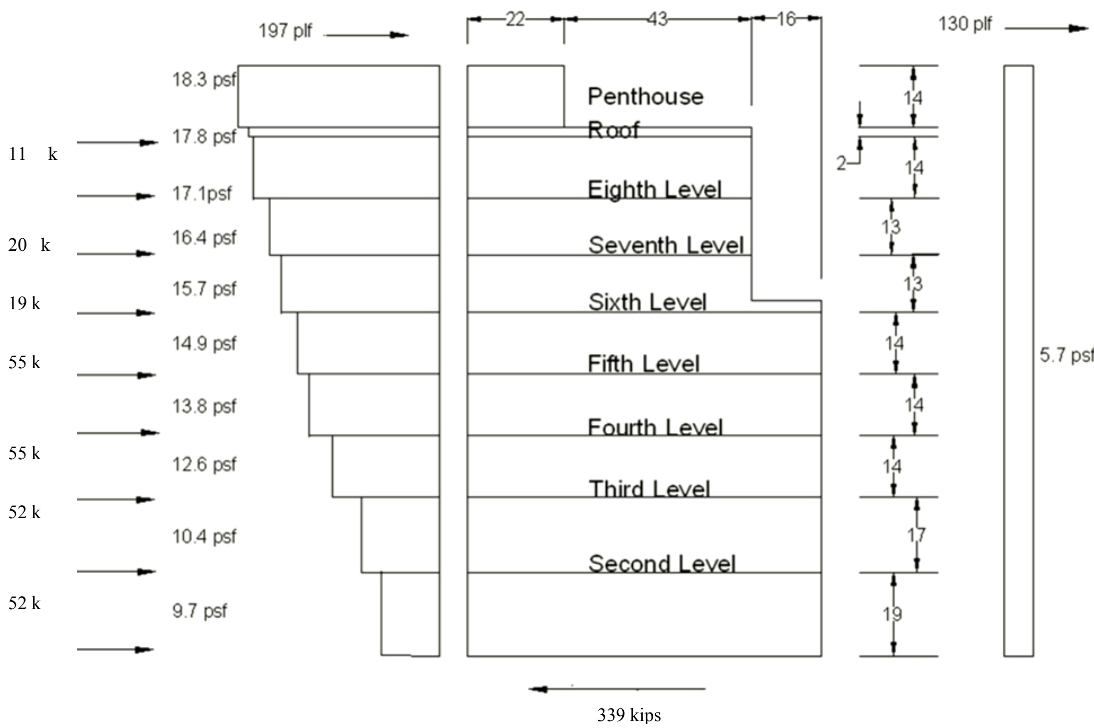


Figure 18: Wind Load Diagram using ASCE 7-05 in North/South winds direction

Refer to figures 11 through 13 for design forces, shears, moments, and assumptions for wind using ASCE7-05. For detailed calculations, refer to the appendix.

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
					N-S	E-W	N-S	E-W	N-S	E-W
Pent house	134	14	14	0.125	71	21	71	21	9580	2783
T.O. Parapet	120	0.25	0.125	0.9	5	1	77	22	605	176
Roof	118	1.7	0.9	7.0	39	11	115	33	4557	1324
8	104	14	7	6.5	64	19	179	52	6641	1930
7	91	13	6.5	6.5	59	17	238	69	5372	1561
6	78	13	6.5	7	59	17	297	86	4583	1331
5	64	14	7	7	58	17	354	103	3687	1071
4	50	14	7	7	54	16	408	119	2682	779
3	36	14	7	8.5	54	16	462	134	1953	568
2	19	17	8.5	9.5	52	15	514	149	987	287
Ground	0	19	9.5	7	44	13	559	162	0	0

Figure 19: Wind Design Forces and Shears

Seismic Summary

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Please refer to Appendix A for detailed calculations used to obtain building weight as well as base shear and overturning moment distribution for each floor as seen in figure 14 below. According to the construction documents, seismic analysis was not found to control this design. The site was declared not an issue for soil liquefaction.

Due to low approximations on the building weight the base shear may in actuality be higher than what is reported in figure 14. However it would not control because the shear cause by lateral wind loads is more than 3 times in magnitude.

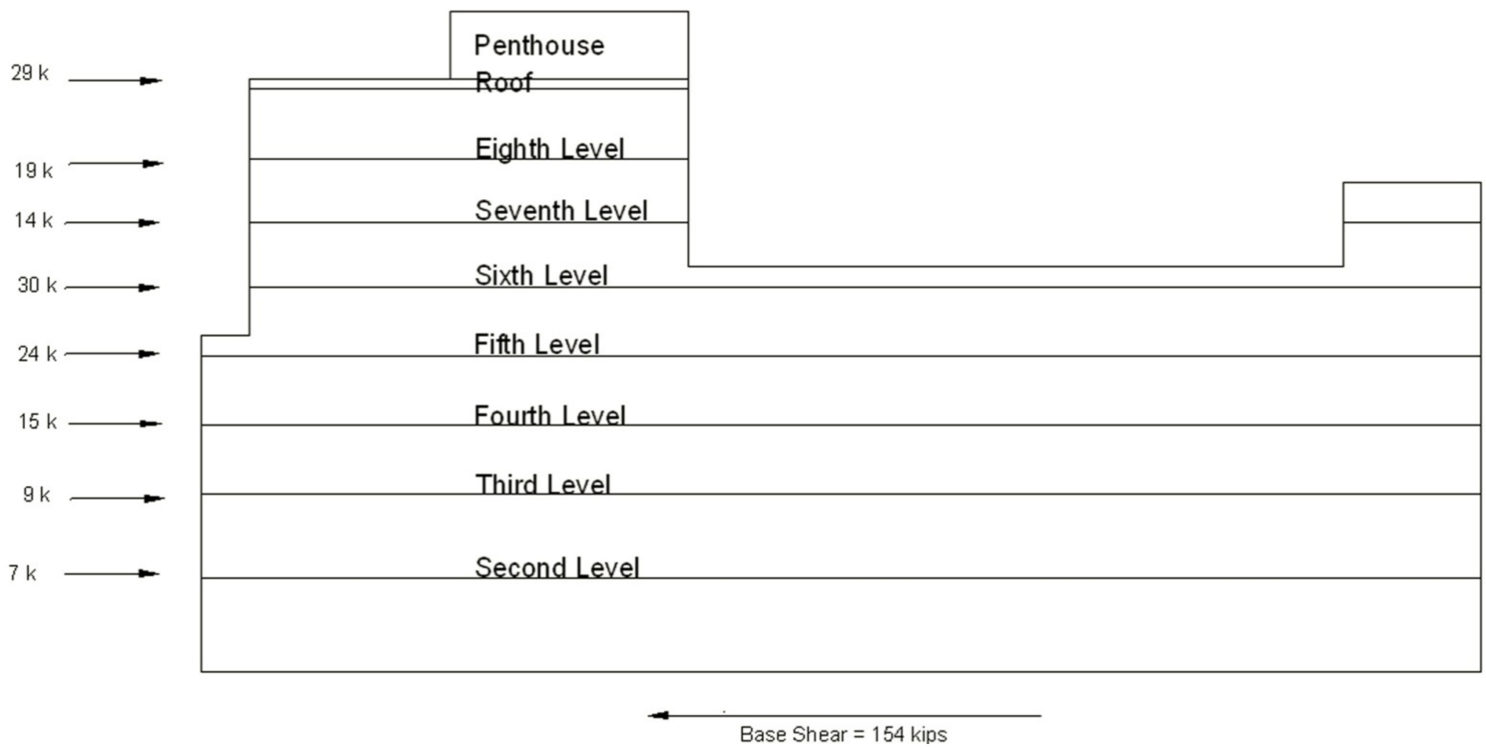


Figure 20: Seismic Force Diagram

Braced Frame Core Design

Introduction

The proposed lateralforce resisting core redesign consists of replacing two of the four moment frames to braced frames, to create a complete braced frame core. Braced frames are preferred over moment frames because they do not require field welds making them more cost effective.

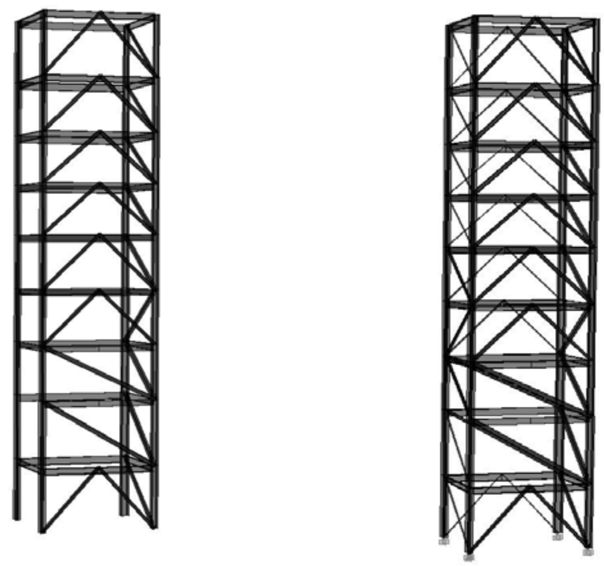


Figure 21: Original design (left) and Redesign (right) of lateral force resisting core

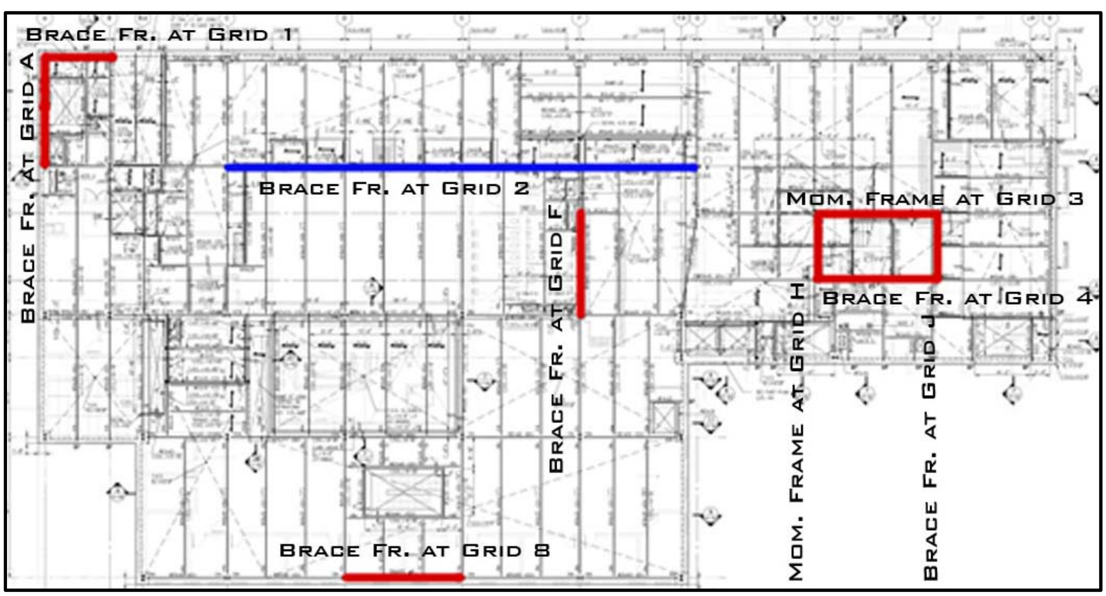


Figure 22: Location of Lateral Force Resisting System, In particular the location of the moment frames

Design Goals and Assumptions

The overall goal of this redesign is to effectively replace moment frames with braced frames as part of a braced frame lateral load resisting core. Other goals are as follows:

Design Goals

- Obtain initial sizes using relative stiffness method
- Use existing column sizes
- Use chevron braces for frame at grid 3 and diagonal member for frame at grid H to maintain symmetry.
- Develop ETABS model and confirm that strength and drift criteria has been satisfied.
- Design and detail the typical braced frame connections.
- Design the most critical braced frame column base plate

Design Assumptions

- P-delta effects not considered
- Columns and girders were kept the same
- Layout of braces are the samebraces of the frame opposite.
- Rigid diaphragm action as a result of the metal deck with concrete topping
- Diaphragms modeled with added mass value in accordance with loading diagrams found in the appendix
- Wind and seismic loads were determined according to ASCE 7-05

Methodology

1. Apply a looo kip load to an ETABS model to get relative stiffness since the the redesigned frame is expected to resist the same amount of force as it did previously.

Initial member sizes of braced frames were determined by first applying a 1000 kip load to an ETABS model of the original system and determining the relative stiffness of each frame. The frame redesigns are expected to resist the same amount of force as did the original frames. This is to ensure that the system is not overdesigned and that the other frames in the system are not over stressed. The connections at the base were modeled as fixed connections because on average the mat foundation is three feet deep with an area of approximately 28, 130 square feet.

Moments were released on the bracing members in the m_{33} direction. For the moment frames a reduced beam section was used in accordance with the program default because the moment frame design assumes 75% moment capacity. Rigid diaphragm mass definitions were assigned to every level in reference to the loading diagrams. The diaphragm definitions are presented in figure 5; for loading diagrams please see appendix. Section cuts were then taken at every story for every frame designed to resist the specified load, either X1000 or Y1000. Relative stiffness was determined based on how much of the 1000 kip load a frame member took with respect to the overall 1000 kip force. Gravity members were neglected for this analysis but were later accounted for in the building's weight for seismic analysis.

Story	Average weight per unit area	
	(psf)	(Kip-in)
Cellar	164	2.9474E-06
1	100	1.7972E-06
2	164	2.9474E-06
3	71	1.2760E-06
4	71	1.2760E-06
5	71	1.2760E-06
6	105	1.8871E-06
7	71	1.2760E-06
8	71	1.2760E-06
Roof	90	1.6175E-06

Figure 23: Diaphragm Additional Mass Assignments on ETABS model

Eighth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	0	0
2	0	0	H	-676	68
3	-175	17	F	0	0
4	-824	82	J	-322	32
8	0	0			0
total=	-999			-998	

Fourth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-178	18	A	45	-4
2	-572	57	H	-34	3
3	-45	5	F	-463	46
4	-203	20	J	-549	55
8	0	0			
total=	-999			-1000	

Seventh story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	0	0
2	0	0	H	-660	66
3	-210	21	F	0	0
4	-790	79	J	-338	34
8	0	0			
total=	-1000			-998	

Third story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-87	9	A	45	-5
2	-832	83	H	-24	2
3	6	-1	F	-456	46
4	-88	9	J	-563	56
8	0	0			
total=	-1000			-1000	

Sixth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	0	0
2	0	0	H	-660	66
3	-226	23	F	0	0
4	-774	77	J	-337	34
8	0	0			
total=	-1000			-997	

Second story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-143	14	A	32	-3
2	-653	65	H	-2	0
3	-32	3	F	-397	40
4	-171	17	J	-636	64
8	0	0			
total=	-1000			-1000	

Fifth story					
Grid	X Force	% X	Grid	Y Force	% Y
1	0	0	A	-6	1
2	-770	77	H	150	-15
3	80	-8	F	-354	35
4	-311	31	J	-788	79
8	0	0			
total=	-1001			-999	

First story					
Grid	X Force	% X	Grid	Y Force	% Y
1	-95	9	A	-103	10
2	-479	48	H	-50	5
3	-22	2	F	-347	35
4	-105	10	J	-488	49
8	-297	30			
total=	-999			-998	

Figure 24: Relative Stiffness for Frames resisting X1000 and Y1000 Lateral Force

2. The percentage of the force experienced by each level is then applied to a non-defined member structure on SAP

Relative stiffnesses are then translated into the percentage of the lateral force experienced by each floor level. These forces are applied to a generic frame in SAP which has the cocentric chevron braces but does not have the braces or any of the member defined with sizes.

3. The axial forces are then found on the bracing members and are sized accordingly

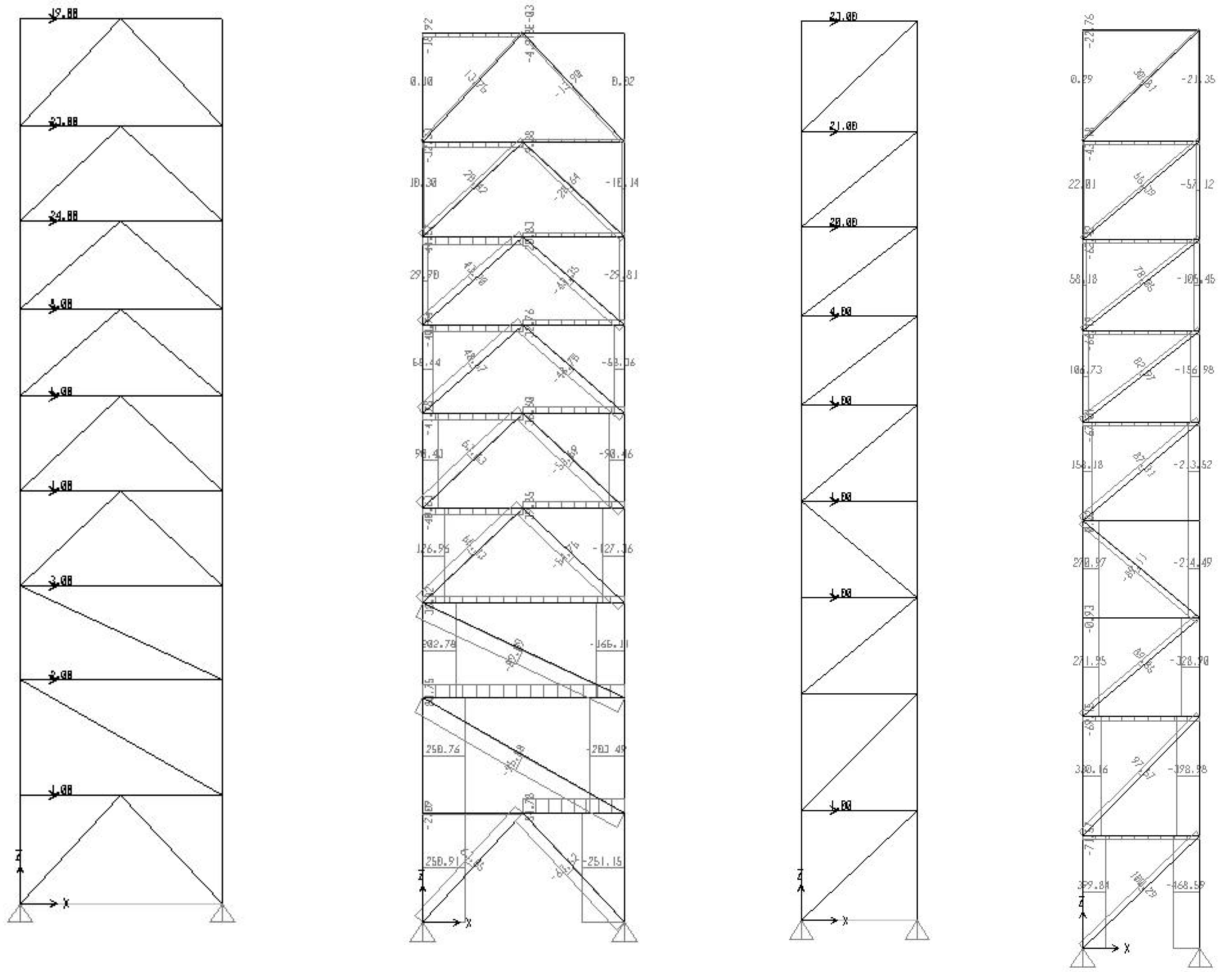


Figure 25: (from left to right) applied forces of frame at grid 3, resulting axial stresses on frame at grid 3, applied forces of frame at grid h, resulting axial stresses on frame at grid h

4. The new lateral system is modeled in ETABS. Drift limits are checked for the previous controlling wind case; which was 100 percent of the wind in the North/South or East/West direction. Seismic limits are also checked.

Once the redesign model is created in ETABS, incorporating the adequate member sizes, the lateral force resisting system is checked against wind drift for serviceability and seismic drift limit for strength requirements based on ASCE 7-05. The controlling wind case used was 100 percent of the wind in the North/South or East/West direction; the same as controlled in the original design. Wind drift was limited to H/400 which is typical for this type of structure. Seismic limits are checked using table 12.12-1 provided in the code.

Drift in the North/South direction was much larger than in the East/West direction due to the buildings rectangular shape. In both the original and the redesign, it can be seen that drift values were well below the allowable according to H/400. The redesign seems to have roughly the same serviceablitiy values as did the original design as can be seen from figure x below.

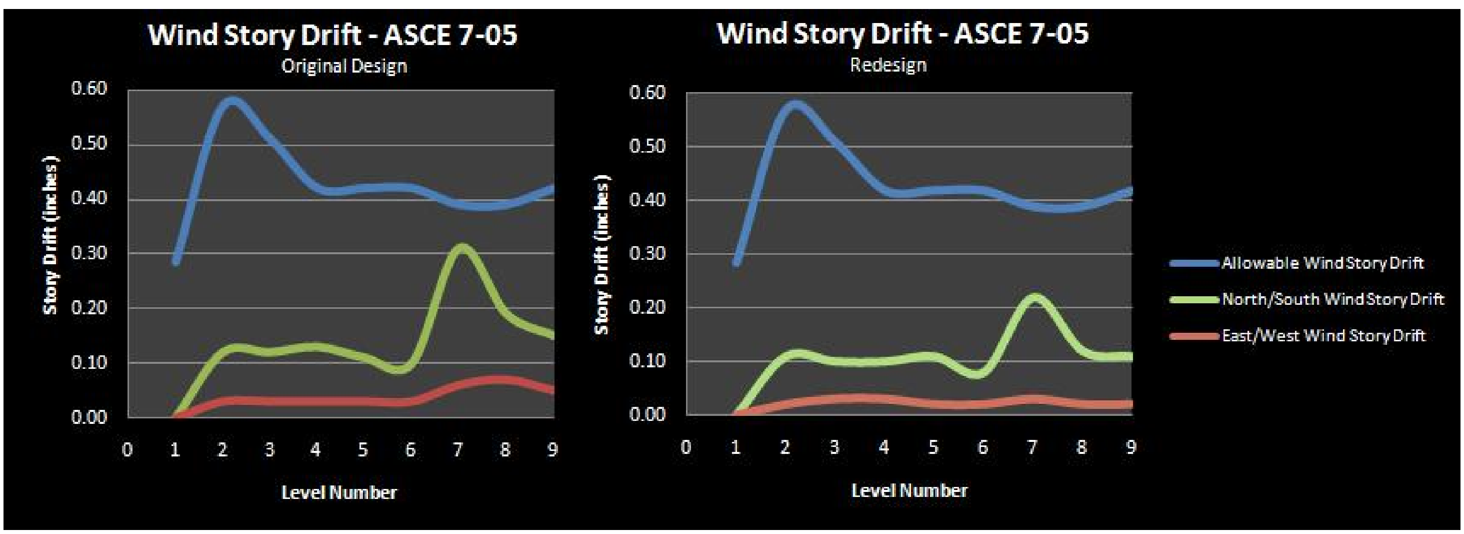


Figure 26: Wind Story Drifts vs. Allowable for the Original Design and New Design

The total wind drift allowed for the building is 3.54 inches. The maximum drift experienced due to the controlling wind case was 0.95 inches, well below the maximum allowed. Figure x on the following page tabulates the drift data of the original design and the new design.

Original Design - Wind Drift : East-West Direction										
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift =H/400 (in.)			Total Drift (in.)	Allowable Total Drift = H/400 (in.)		
9	118	-	0.05	<	0.42	TRUE	0.33	<	3.54	TRUE
8	104	14	0.07	<	0.39	TRUE	0.28	<	3.54	TRUE
7	91	13	0.06	<	0.39	TRUE	0.21	<	3.54	TRUE
6	78	13	0.03	<	0.42	TRUE	0.15	<	3.54	TRUE
5	64	14	0.03	<	0.42	TRUE	0.12	<	3.54	TRUE
4	50	14	0.03	<	0.42	TRUE	0.09	<	3.54	TRUE
3	36	14	0.03	<	0.51	TRUE	0.06	<	3.54	TRUE
2	19	17	0.03	<	0.57	TRUE	0.03	<	3.54	TRUE
1	0	19	0.00	<	0.29	TRUE	0.00	<	3.54	TRUE

Original Design - Wind Drift : North-South Direction										
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift =H/400 (in.)			Total Drift (in.)	Allowable Total Drift = H/400 (in.)		
9	118	-	0.15	<	0.42	TRUE	1.23	<	3.54	TRUE
8	104	14	0.19	<	0.39	TRUE	1.08	<	3.54	TRUE
7	91	13	0.31	<	0.39	TRUE	0.89	<	3.54	TRUE
6	78	13	0.10	<	0.42	TRUE	0.58	<	3.54	TRUE
5	64	14	0.11	<	0.42	TRUE	0.48	<	3.54	TRUE
4	50	14	0.13	<	0.42	TRUE	0.37	<	3.54	TRUE
3	36	14	0.12	<	0.51	TRUE	0.24	<	3.54	TRUE
2	19	17	0.12	<	0.57	TRUE	0.12	<	3.54	TRUE
1	0	19	0.00	<	0.29	TRUE	0.00	<	4.54	TRUE

Figure 27: Wind Drift Values for the Original Design of the Steel Frame Core

Redesign - Wind Drift : East-West Direction										
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift =H/400 (in.)			Total Drift (in.)	Allowable Total Drift = H/400 (in.)		
9	118	-	0.02	<	0.42	TRUE	0.19	<	3.54	TRUE
8	104	14	0.02	<	0.39	TRUE	0.17	<	3.54	TRUE
7	91	13	0.03	<	0.39	TRUE	0.15	<	3.54	TRUE
6	78	13	0.02	<	0.42	TRUE	0.12	<	3.54	TRUE
5	64	14	0.02	<	0.42	TRUE	0.10	<	3.54	TRUE
4	50	14	0.03	<	0.42	TRUE	0.08	<	3.54	TRUE
3	36	14	0.03	<	0.51	TRUE	0.05	<	3.54	TRUE
2	19	17	0.02	<	0.57	TRUE	0.02	<	3.54	TRUE
1	0	19	0.00	<	0.29	TRUE	0.00	<	3.54	TRUE

Redesign - Wind Drift : North-South Direction										
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift =H/400 (in.)			Total Drift (in.)	Allowable Total Drift = H/400 (in.)		
9	118	-	0.11	<	0.42	TRUE	0.95	<	3.54	TRUE
8	104	14	0.12	<	0.39	TRUE	0.84	<	3.54	TRUE
7	91	13	0.22	<	0.39	TRUE	0.72	<	3.54	TRUE
6	78	13	0.08	<	0.42	TRUE	0.50	<	3.54	TRUE
5	64	14	0.11	<	0.42	TRUE	0.42	<	3.54	TRUE
4	50	14	0.10	<	0.42	TRUE	0.31	<	3.54	TRUE
3	36	14	0.10	<	0.51	TRUE	0.21	<	3.54	TRUE
2	19	17	0.11	<	0.57	TRUE	0.11	<	3.54	TRUE
1	0	19	0.00	<	0.29	TRUE	0.00	<	4.54	TRUE

Figure 28: Wind Drift Values for the New Design of the Steel Frame Core

Seismic drift values were determined by applying the seismic forces determined in technical report 1. Unlike the wind drift requirements, seismic drift is not a serviceability requirement, it is a requirement that protects against building collapse. The limitation was taken to be $\Delta_{seismic}$=0.015h_{sx} (in.) based on ASCE 7-05. As is shown in the following tables, seismic drift was acceptable at all story levels in both East-West and North-South directions.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$\boxed{0.015h_{sx}}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

Figure 29: Allowable Story Drift due to Seismic Loading per ASCE 7-05 Table 12.12-1

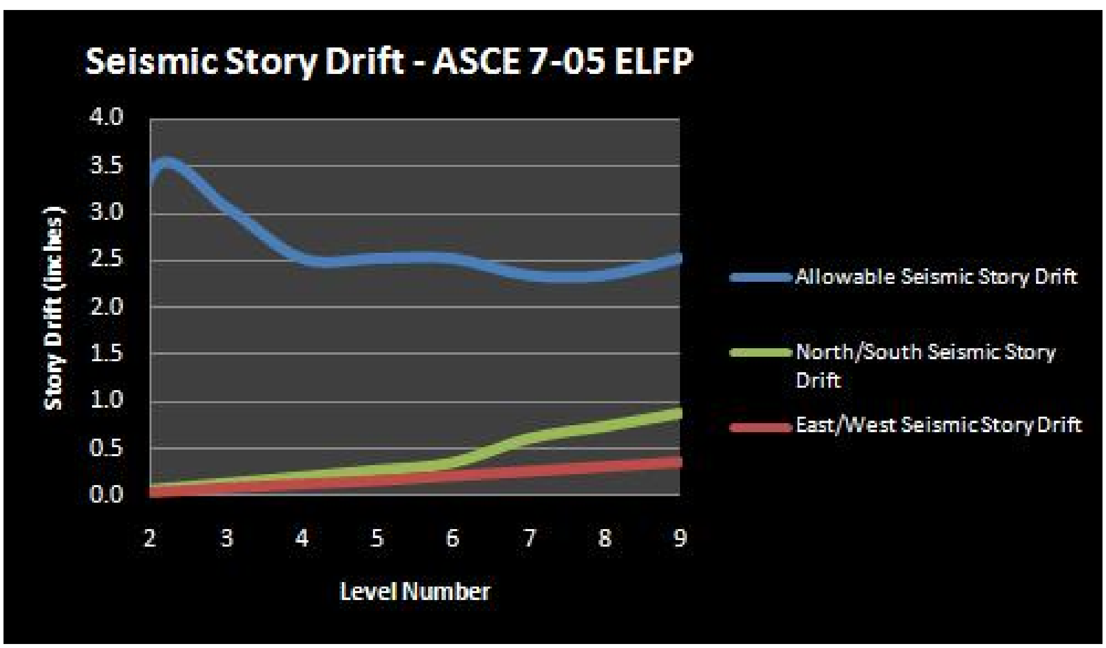


Figure 30: Seismic Drift vs. Allowable Drift

Seismic Drift : East-West Direction							
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift = $0.015h_{sx}$ (in.)			Total Drift (in.)
9	118	-	0.36	<	2.52	TRUE	0.19
8	104	14	0.31	<	2.34	TRUE	0.17
7	91	13	0.26	<	2.34	TRUE	0.15
6	78	13	0.21	<	2.52	TRUE	0.12
5	64	14	0.16	<	2.52	TRUE	0.10
4	50	14	0.12	<	2.52	TRUE	0.08
3	36	14	0.08	<	3.06	TRUE	0.05
2	19	17	0.03	<	3.42	TRUE	0.02
1	0	19	0.00	<	0.00	TRUE	0.00

Seismic Drift : North-South Direction							
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift = $0.015h_{sx}$ (in.)			Total Drift (in.)
9	118	-	0.88	<	2.52	TRUE	3.24
8	104	14	0.74	<	2.34	TRUE	2.36
7	91	13	0.61	<	2.34	TRUE	1.62
6	78	13	0.35	<	2.52	TRUE	1.01
5	64	14	0.27	<	2.52	TRUE	0.66
4	50	14	0.20	<	2.52	TRUE	0.39
3	36	14	0.13	<	3.06	TRUE	0.19
2	19	17	0.06	<	3.42	TRUE	0.06
1	0	19	0.00	<	0.00	TRUE	0.00

Figure 31: Seismic Drift

Consideration of Seismic P-Delta Effects

P-delta effects; otherwise known as secondary effects, looks at how Secondary moments caused by the eccentricity of the gravity loads above. These moments are determined using the design level seismic forces and elastic displacements. The secondary moment in a story is defined as the product of the total dead load, floor live load, and snow load above the story multiplied by the elastic drift of that story. The primary moment is defined as the seismic shear multiplied by the story height.

P-delta effects are usually negligable for shorter buildings, they are more important in high-rises. The IBC code allows p-delta effects to be ignored when Θ is less than 0.10. It also imposes a resistriction on secondary effects of Θ < 0.25 deeming the structure unstable. When Θ is between 0.10 and 0.25 then P-delta effects must be considered.

Drift values were found to be most significant in the East/West loading direction of the building, also referred to as the x-direction. Interstory drift values were obtained form ETABS and were used to determine the Θ-value of each story level. It was found that none of the Θ-values exceeded 0.10, therefore; according to the International Bulding Code, P-delta effects are small enough to be negligable.

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{[EQ. 1]}$$

Level	Px (kips)	Vx (kips)	Δ (inches)	hsx (ft.)	hsx (in.)	Θ	Θ ≤ 0.10 ?
Roof	736	36	0.88	-	-	-	-
8	1254	54	0.74	14	168	0.031	YES
7	1752	69	0.61	13	156	0.031	YES
6	3129	99	0.35	13	156	0.022	YES
5	4662	123	0.27	14	168	0.019	YES
4	6185	138	0.2	14	168	0.016	YES
3	7749	147	0.13	14	168	0.013	YES
2	11449	154	0.06	17	204	0.007	YES
1	15388	-	0	19	228	-	-

Figure 32: Consideration of P-Delta Effects

The eccentricity of the gravity loads due to the already existing deformation of the structure causes an additional moment on the structure whose value is the axial load multiplied by the eccentricity.

$$M = P \times e \qquad \text{[EQ. 2]}$$

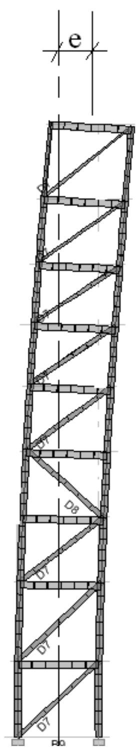


Figure 33: Secondary Effects caused by Gravity Load Eccentricity

5. The axial forces are the redesigned members are checked for strength capacity. As can be seen on figure 31, the stress loading diagrams of the redesigned frames. The values of the axial stress experienced by the braces are tabulated on the following page. These were compared to the axial capacity of the braces which were taken from the AISC Manual 13th edition. Thses axial capacity values take into account the effective length with k=1.0.

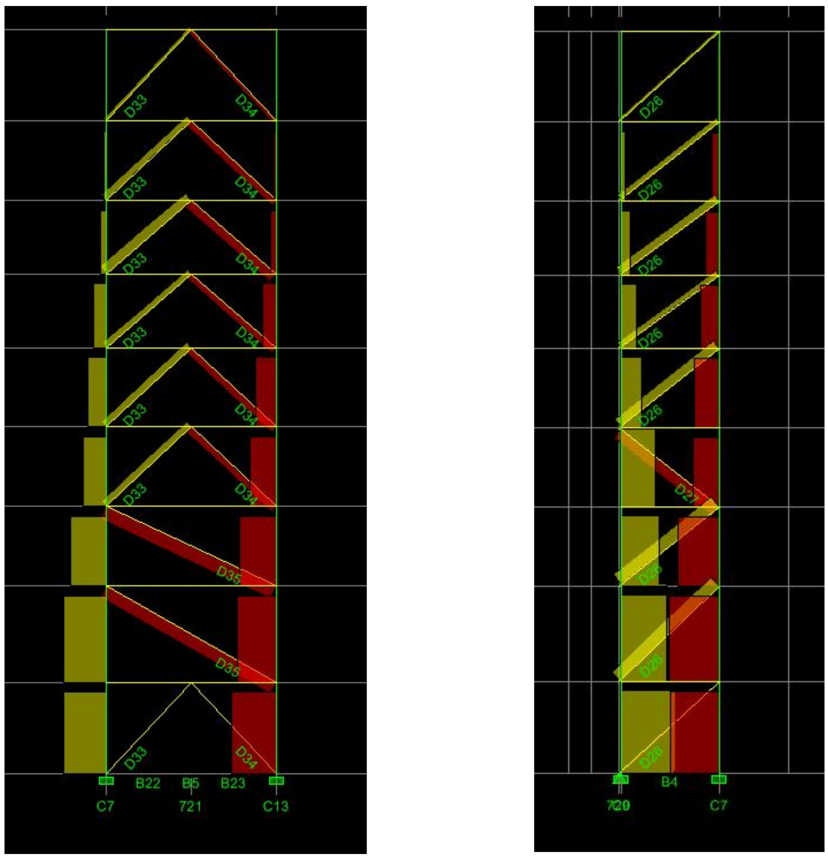
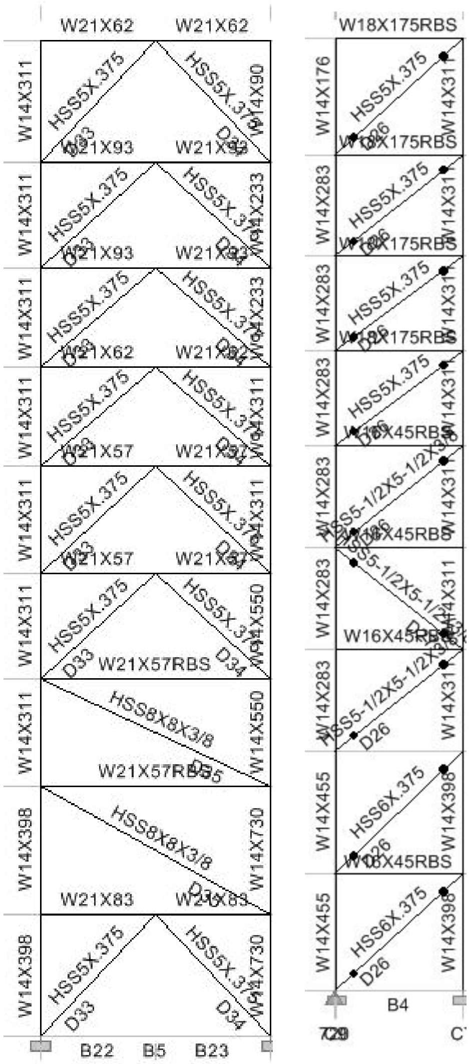


Figure 34: Axial Stresses Fill Diagram from Frames at Grids 3 and H.

The axial stress tabulated in the figure on the following page, where taken from ETABS member section cuts. The axial stress values are already factored using the 1.6 W load combination. The axial loads on the diagonal members due to the controlling wind case were far below the axial capacity of the HSS memebers. This is may be due to the higher stiffness of the other frames in the lateral resisting sytem. The other frames may be resisting most of the load compared to the redesigned frames at grid 3 and at grid h. Also, the I was able to decrease column sizes when going from moment frame to braced frame.



Braced Frame Schedule					
Concentrically Braced Frames on Grid H					
Story	HSS	Axial Capacity		Axial Stress	
8	5x5x 3/8	70.3	>	18	TRUE
7	5x5x 3/8	70.3	>	26	TRUE
6	5x5x 3/8	96.3	>	28	TRUE
5	5x5x 3/8	96.3	>	21	TRUE
4	5.5x5.5x 3/8	96.3	>	33	TRUE
3	5.5x5.5x 3/8	96.3	>	40	TRUE
2	5.5x5.5x 3/8	96.3	>	45	TRUE
1	6x6x 3/8	128	>	43	TRUE
cellar	6x6x 3/8	128	>	2	TRUE

Braced Frame Schedule					
Concentrically Braced Frames on Grid 3					
Story	HSS	Axial Capacity		Axial Stress	
8	5x5x3/8	77.2	>	14	TRUE
7	5x5x3/8	77.2	>	19	TRUE
6	5x5x3/8	77.2	>	22	TRUE
5	5x5x3/8	77.2	>	14	TRUE
4	5x5x3/8	77.2	>	15	TRUE
3	5x5x3/8	77.2	>	13	TRUE
2	8x8x3/8	135	>	23	TRUE
1	8x8x3/8	135	>	18	TRUE
cellar	5x5x3/8	77.2	>	1	TRUE

Figure 35: (from left to right) Frame at Grid 3, Frame at Grid H, Braced Frames Schedules

In the following section the bracing connection of a chevron bracing configuration is designed using the AISC Manual 13th edition. Force transfer in diagonal bracing connections is determined using the Uniform Force Method as is specified by the construction document.

Also a simple diagonal bracing member; such as the ones in the redesigned frame located at grid h, is analyzed to show how to determine the available strength of an existing diagonal bracing connections.

Graduate Course Integration: Design and detail of the Typical Braced Frame Connection

The Uniform Force Method looks to eliminate moments by selecting a connection geometry such that moments do not occur on the three connection interfaces. These are the gusset-to-column, gusset-to-beam, and beam-to-column connection. By elimination the introduction of moments, the connection can then be designed for shear and tension only.

The controlling geometries for the uniform force method include the beam depth, column depth, the distance from the face of the column flange or web to the centroid of the gusset-to-beam connection, the distance from the face of the beam flange to the centroid of the gusset-to-column connection, also the loading angle is an important factor. Once the connection geometry is chosen, the gusset-to-beam connection is designed for the required shear force and axial force.

There are three cases involved in the uniform force method for bracing connection design. Special case one, is used when the working point location is chosen at the corner of the gusset. For special case two the connection is designed to minimize the shear in the beam-to-column connection. This method is best used when the beam-to-column connection is already highly loaded because this type of connection is very uneconomical. Special case three is used when there is no gusset-to-column connection.

For the chevron connection on the following page was designed for an axial load of 205 kips. The brace-to-gusset and the gusset-to-beam weld size were designed to be 3/8" fillet welds although the required gusset-to-beam weld size was only required to ¼ in. This was done to keep things simpler and avoid an error when detailing the connections. The gusset plate was a ¾ in. gusset plate and it was designed against strength, buckling as a compression brace, and yielding as a tension brace. Among the limit states checked were the shear strength at the brace-to-gusset welds, the shear lag fracture in HSS brace, gusset-to-beam bolt connection, and local web yielding of the beam.

When checking the buckling of the gusset plate the whitmore section was assumed to be entirely in the gusset. Therefore, the whitmore section can spread across the joint into adjacent connected material of lesser thickness or adjacent connected material provided that a rational analysis is performed.

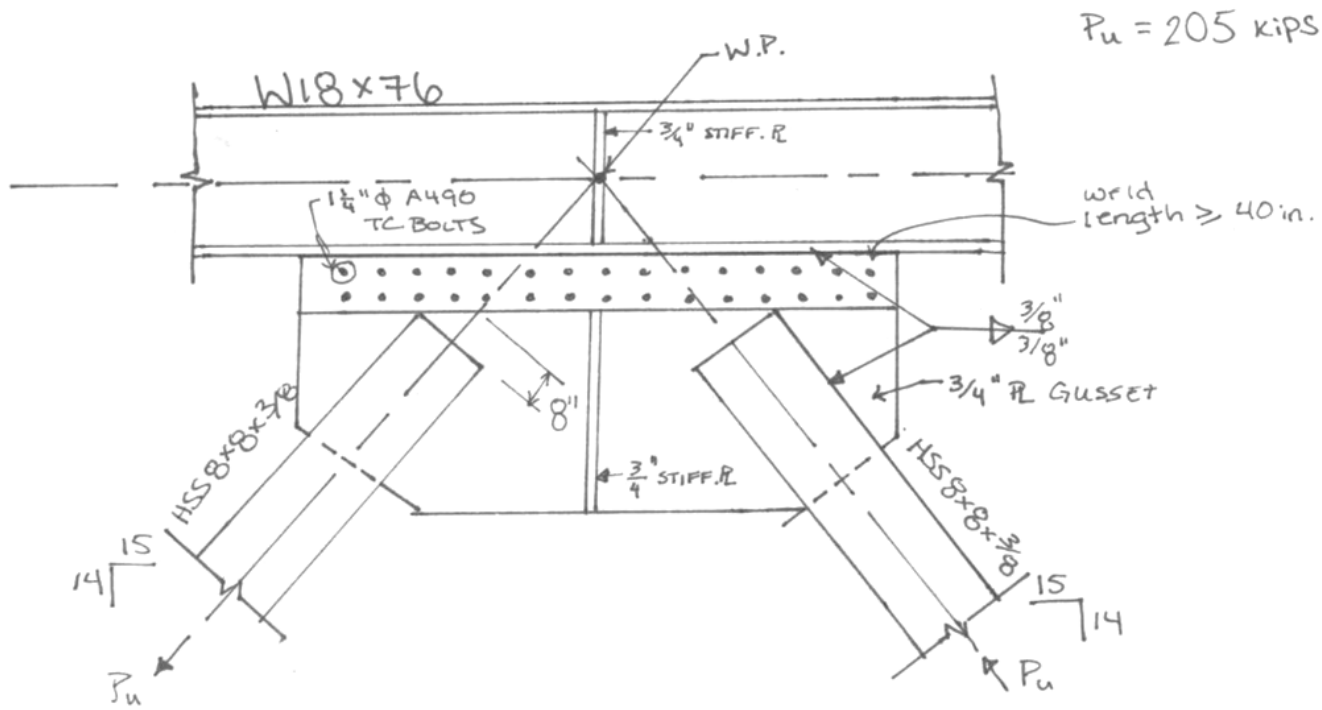


Figure 36: Chevron Connection Design

In order to calculate the interface forces of the chevron connection, the gusset-to-beam connection was designed as if each brace were the only brace and each brace's connection centroid was located at the ideal centroid locations to avoid inducing a moment on the gusset-beam interface, similarly to uniform form method special case 3.

Note that the beam to column connection was not designed as it was not of interest. Focus was given to the area where the diagonal member met to form the "inverted V" or chevron connection. On the following page the limit states pertaining to bracing connections are tabulated including that of the beam-to-column connection even though it was not applied to this thesis. For detailed hand-calculations of the chevron connection design please refer to the Brace Frame Connection Design subsection of the appendix. The following information can be found on the Penn State engineering website (www.engr.psu.edu/ae/steelstuff/economy.htm)

Limit-states considered for each interface of bracing connections		
Connection interface)	**Connection element**	**Limit states**
Brace-to-gusset (A)	Bolts to gusset	1
	Gusset	3, 4, 5, 6
	Bolts to brace	1
	Brace	5, 6, 7, 8
	Splice plates for WT's	5, 6, 7, 8
Gusset-to-beam (B)	Gusset	7
	Fillet weld	9
	Beam web	10
Gusset-to-column (C)	Bolts to gusset	1
	Fillet weld to gusset	9
	Gusset	6, 7, 8
	Bolts to column	2
	Clip angles	6, 7, 8, 11, 12
	Column	6, 11, 12
Beam-to-column (D)	Bolts to beam web	1
	Fillet weld to beam web	9
	Beam web	6, 7, 8
	Bolts to column	2
	Clip angles	6, 7, 8, 11, 12
	Column	6, 11, 12

Figure 38:Limit-states considered for each interface of bracing connections

Limit-states identification for bracing connections	
Limit state	**Number**
Bolt shear fracture	1
Bolt shear/tension fracture	2
Whitmore yielding	3
Whitmore buckling	4
Tear-out fracture	5
Bearing	6
Gross section yielding	7
Net section fracture	8
Fillet weld fracture	9
Beam web yielding (beyond k-distance)	10
Bending yielding (including prying action)	11
Bending fracture (including prying action)	12

Figure 37:Limit-states identification for bracing connections

Also a simple diagonal bracing member was analyzed to show how to determine the available strength of an existing diagonal bracing connections. The detailed calculations can be found in the appendix.

In this analysis special case two of the uniform force method was applied; shear in beam-to-column connection minimized. The purpose of this analysis was to avoid transfer of moment to horizontal members. This was achieved by using the following equation which can be found in the AISC Manual section 13-3.

$$\alpha - \beta tan\theta = e_b tan\theta - e_c \quad \text{[EQ. 3]}$$

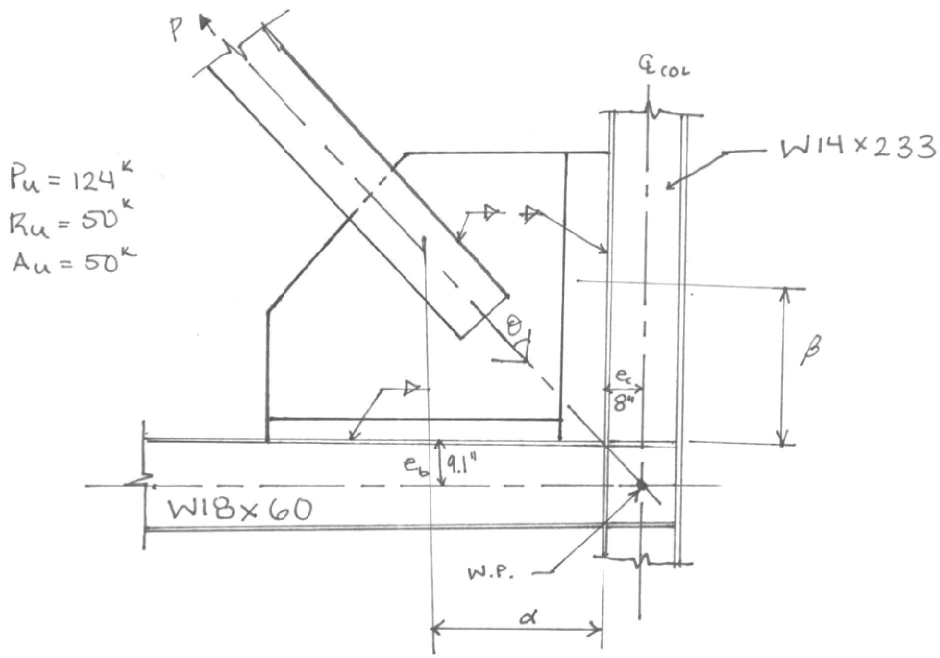


Figure 39: Diagonal Brace Connection

Interface Foces prior to special case two		
Connection ID	Shear (kips)	Axial (kips)
Gusset-to-column	40.4	30.8
Gusset-to-beam	67.8	35
Beam-to-column	85	80.8

Figure 40: Interface Forces Prior to Special Case 2 Application

Interface Forces applying special case two			
Connection ID	Shear (kips)	Axial (kips)	Moment (ft-k)
Gusset-to-column	75.4	30.8	-
Gusset-to-beam	0	67.8	51.3
Beam-to-column	50	80.8	-

Figure 41: Interface Forces Applying Special Case 2

Notice that after applying special case two, the shear forces in the gusset-to-beam connection went to zero while causing a moment on the gusset-to-beam connection. Because on this induced moment the connection will have to be larger and will requirea thicker gusset plate. As can be imagined, this special case interrupts the natural flow of forces assumed in the uniform force method.

Overturning and Foundation Impact Discussion

Overturning moment due to seismic loads is counteracted by the dead load of the building's weight. However, when this is not enough, additional measures need to be taken to resist this moment. Designing the foundation to assist in counteracting the overturn is a popular way to do this.

Values for overturning moment were calculated by multiplying the base shear by the frame height relative to ground level. Overturning was found to be resisted by all frames except the five-story braced frame at grid 1. This indicates an impact on the foundation. However, since seismic forces used were those determined using ASCE 7-05, they do not accurately represent the values used by the structural engineer. It is very possible that a "no impact on foundation" conclusion was found by the structural engineer.

Story	East-West Frames : Forces (kips)					North South Frames : Forces (kips)				Total Story Shear (kips)
	At Grid 1	At Grid 2	At Grid 3	At Grid 4	At Grid 8	At Grid A	At Grid F	At Grid H	At Grid J	
8	0.00	0.00	25.86	87.59	0.00	0.00	0.00	28.84	-28.87	113.41
7	0.00	0.00	47.75	174.56	0.00	0.00	0.00	58.18	-58.28	222.21
6	0.00	0.00	69.83	254.88	0.00	0.00	0.00	85.01	-85.22	324.50
5	0.00	271.65	-21.32	169.10	0.00	6.24	-15.93	-67.57	77.06	419.23
4	64.75	266.07	16.87	165.57	0.00	-4.60	-38.86	4.34	39.40	513.54
3	49.26	417.91	-2.07	141.78	0.00	-10.41	-36.83	-9.85	56.99	606.78
2	99.35	382.43	18.73	191.38	0.00	-21.27	-17.84	-4.31	36.00	684.47
1	64.33	335.91	19.49	141.56	216.62	58.83	-11.40	-7.71	-41.31	776.32

Figure 43: Story Forces due to Controlling load combination

	East-West Frames : Forces (kips)					North South Frames : Forces (kips)			
	At Grid 1	At Grid 2	At Grid 3	At Grid 4	At Grid 8	At Grid A	At Grid F	At Grid H	At Grid J
Overturning Moment (ft-k)	9856	12012	18480	18480	2926	12012	12012	18480	18480
Base Dimension (ft)	16.5	120	30	30	30	28	26	17	17
Force at edge column (k)	597.3	100.1	616	616	97.5	429	462	1087.1	1087.1
Edge Column DL (k)	430	1010	1390	1240	265	530	750	1300	1390
Overturning	NG	OK	OK	OK	OK	OK	OK	OK	OK

Figure 42: Story forces and Overturning Analysis

Center of Rigidity Discussion

Two methods were used to check against the center of rigidity coordinates determined by ETABS. The first method used SAP2000 for stiffness values while the second used ETABS for stiffness values. With the use of SAP2000, stiffness values were determined for each lateral system element by applying a one kip lateral load at the fourth story and taking the inverse of the resulting displacement at that level. The corresponding x and y coordinates of the center of rigidity were calculated using the following equations.

$$\overline{x} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \quad ; \quad \overline{y} = \frac{\sum k_{ix} y_i}{\sum k_{ix}} \qquad \text{[EQ. 4]}$$

For this first method, the center of rigidity was found to be at coordinates (79.2, 98.0) feet. Comparing this set of coordinates with the ETABS output, it is evident that there is a large gap of error. This error may be due to the neglecting of the center of rigidity effects of floors above and below story four.

Story four- Approximate COR Check using SAP2000 relative stiffness values				
Frame (dir)	Load Applied in Diaphragm (kips)	Displacement (in.)	Stiffness	Distance to Origin (ft)
1 (E-W)	1	0.01	105.26	132.5
2 (E-W)	1	0.00	227.27	104.5
3 (E-W)	1	0.00	238.10	92.5
4 (E-W)	1	0.00	625.00	75.4
8 (E-W)	1	0.00	0.00	0
A (N-S)	1	0.00	277.78	0
F (N-S)	1	0.01	142.86	136.5
H (N-S)	1	0.10	10.03	196.5
J (N-S)	1	0.01	161.29	226.5
Center of Rigidity in the x-direction:			79.2 ft compare to	113 ft
Center of Rigidity in the y-direction:			89 ft compare to	88 ft

Figure 44: Center of Rigidity values calculated using SAP2000

In ETABS; used for second method, wind forces calculated in accordance with ASCE 7-05 were applied in both directions at the center of pressure for each story. Section cuts were then taken at the fourth story on every lateral frame. Relative stiffness was determined based on the percentage of the total lateral load taken by the individual frames. The above equations for the center of rigidity was applied once again to obtain the values of (169.5, 83.5) feet. Although it was expected that this method would provide more accurate results, it did not, due to an unknown error. This same procedure was repeated was levels two and five, resulting in discrepancies between the calculated center of rigidity and the expected value.

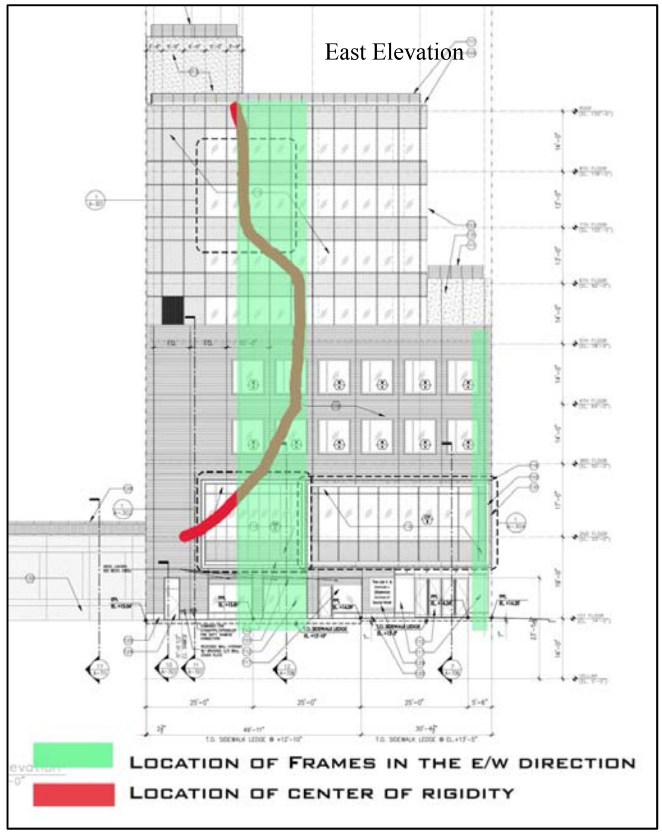
Story four- Approximate COR Check using ETABS relative stiffness values				
Frame (dir)	Load Applied in Diaphragm (kips)	Distribution (kips)	Percentage	Distance to Origin (ft)
1 (E-W)	321	41.00	0.13	132.5
2 (E-W)	321	165.31	0.51	104.5
3 (E-W)	321	10.54	0.03	92.5
4 (E-W)	321	103.01	0.32	75.4
8 (E-W)	321	0.00	0.00	0
A (N-S)	94	9.95	0.11	0
F (N-S)	94	33.63	0.36	136.5
H (N-S)	94	2.75	0.03	196.5
J (N-S)	94	47.84	0.51	226.5
Center of Rigidity in the x-direction:			169.54 ft compare to	113 ft
Center of Rigidity in the y-direction:			83.45 ft compare to	88 ft

Figure 45: Center of Rigidity values calculated using ETABS

ETABS output for center of rigidity; shown in Figure 46 takes into account the center of rigidities of levels above and below. As is shown in the table, there is a lot of changes in the y direction due to the various setbacks in the north south direction of the building. The x coordinates do not change as often as you go up in elevation because the only setback in the east-west direction occurs at the sixth story to seventh story transition where the building only a 5,290 square foot section (out of a total 28,130 square feet) of the building continues up the next three stories. A schematic diagram of the location of the center of rigidity for various buildings levels is shown as Figure 47. The locations of the center of rigidities for the diagram were taken from the table presented in Figure 46.

Center of Rigidity Calculated by ETABS		
Story	XCR	YCR
ROOF	216.733	74.103
STORY8	215.114	74.69
STORY7	210.446	75.703
STORY6	123.542	87.87
STORY5	112.238	89.533
STORY4	112.872	88.042
STORY3	114.427	81.942
STORY2	115.889	67.32
STORY1	n/a	n/a

Figure 46: Center of Rigidity output from ETABS



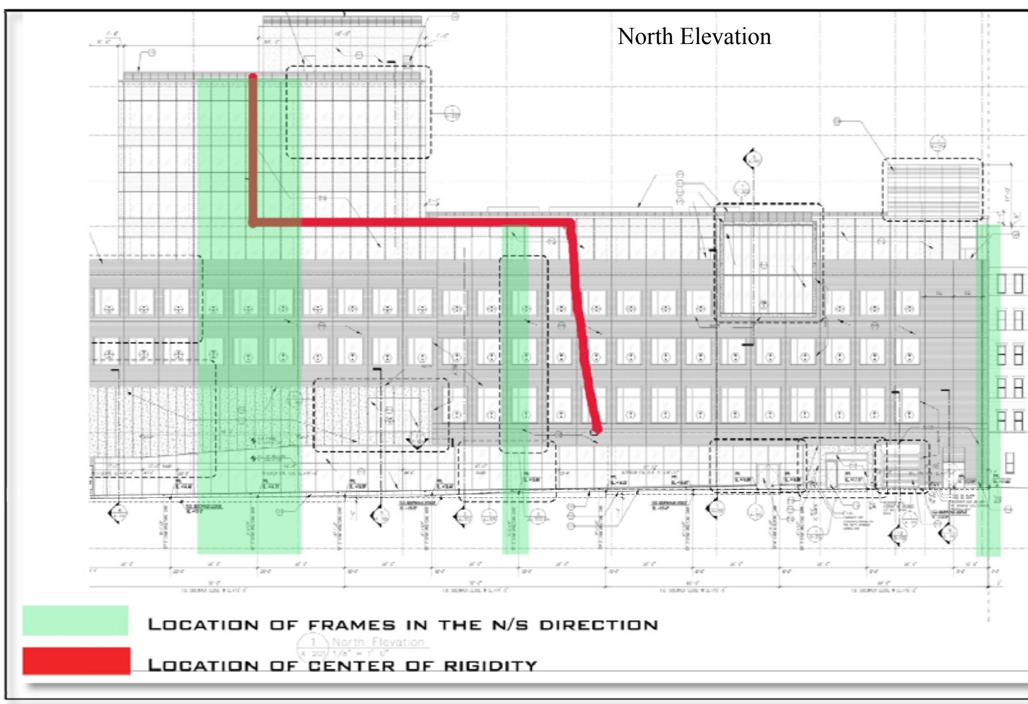


Figure 47: Schematic diagram of the location of the center of rigidity due to the lateral system

Structural Depth Summary

Comparison between Existing and New Braced Frames

Steel moment frames are expected to achieve ductility through the yielding of beams or columns. This means that the connections have to remain strong enough to withstand cyclical loading as is true of seismic loading.

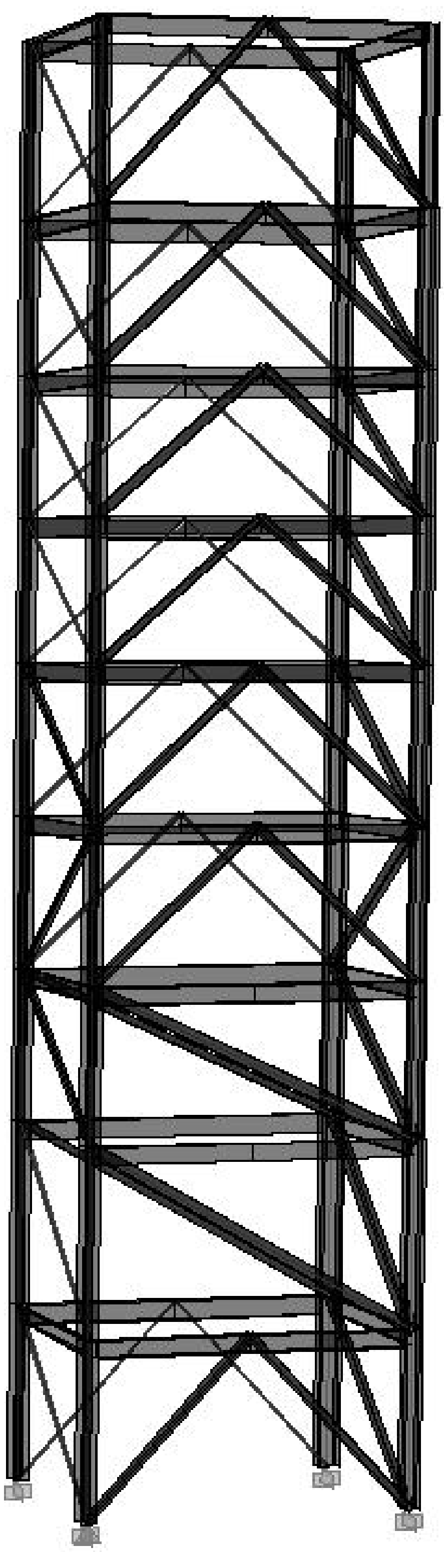
When going from moment frames to braced frame, the entire braced frame core now distributed to lateral load more evenly, this caused the initial column sizes to be overdesigned. I was able to bring down the column sizes, to the point where the combination frame core (moment frames and braced frames) was 35% higher in cost than a core of entirely braced frames. Achieving a savings of $77,100. The savings don't take into account the change in scheduling, therefore the overall savings are much higher.

Things that contributed to higher cost for the moment frames were the larger beam and column sizes which are significantly heavier per linear foot than in braced frames. Their massiveness is necessary to transfer loads, however these large sections leed to higher material costs and the need for larger erection equipment. [Richard]

While the actual design and detailing of a moment frame may only take a few hours to a day's work for an experienced engineer, that is only a small part of the process. In addition to designing the foundation anchorage, the engineer will need to produce steel and welding specifications, also review steel shop drawings and welding procedure specifications. A steel contractor will need to A steel sub-contractor will need to install the frame, and the general contractor will need to coordinate between the iron workers and the framers to make sure everything fits together. Field welds also increase the erection cost. In my estimates a cost of $620 per moment connection was assumed. [McEntee]

Some things to consider in design is that although the columns were optimized for the gravity load in this thesis, this may turn out to be more expensive in the long run, then instead sizing the columns at 75% capacity as opposed to near 100%. By designing at 75% capacity the need for doubler plates is eliminated.

Final Design



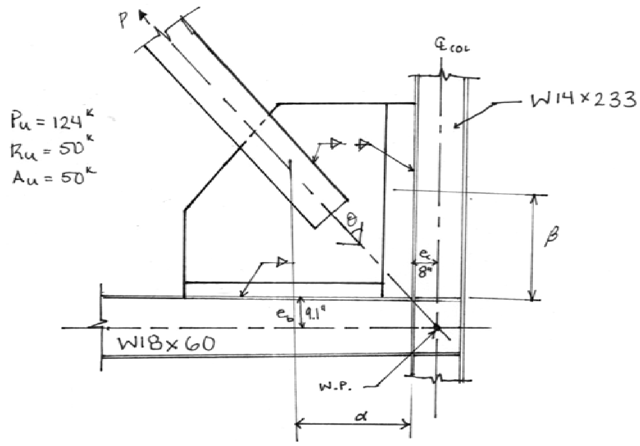


Figure 48: Diagonal Bracing Connection

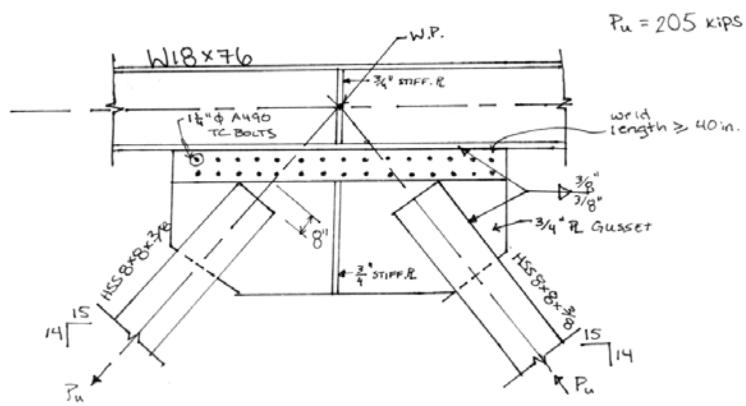


Figure 49: Chevron Bracing Connection

Original Design			New Design		
w14x	quantity	total length	w14x	quantity	total length
68	1	14	53	1	14
90	1	14	68	1	26
176	1	14	74	1	14
233	4	111	90	1	26
283	3	85	99	1	26
311	4	99	120	1	14
331	1	28	145	2	62
342	1	33	193	5	148
398	1	33	233	1	33
455	1	33	398	1	33
550	1	31			
730	1	33			
mom connections					
HSS			HSS		
5x5x3/8	11	573.1	5x5x3/8	11	1146.2
5.5x5.5x3/8	3	201.4	5.5x5.5x3/8	3	402.8
6x6x3/8	2	137	6x6x3/8	2	274
8x8x3/8	2	94.8	8x8x3/8	2	189.6

Figure 50: Member Sizes for Columns and Braces

Redesign of Façade

The focus of my thesis is energy efficiency and how it can be implemented using facade and green roof redesign. It ties structural engineering concepts with existing enclosure installation methods to provide a secure barrier against water and the temperature of the outside world. It will also provide sound isolation from street noise to foster a more comfortable learning environment for students.

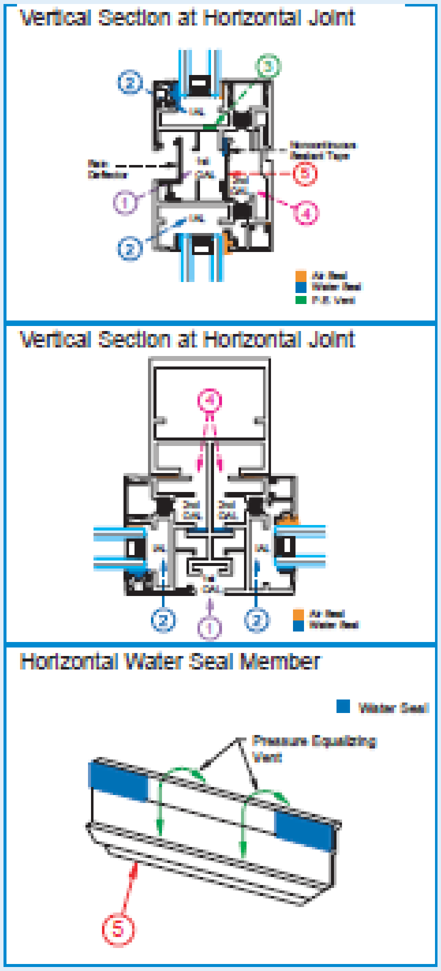
All of this has to be achieved while maintaining an inviting and transparent appearance to the community so that they can feel welcome. This may cause limitations in the window glazing chosen and its corresponding R-value. This in-depth analysis could not be achieved without the redesign of the structural system and its impact on cost.

Enclosure design is important to ensure the life of a structure in addition to continual building maintenance. Simple and inexpensive measures can be taken to significantly improve the buildings energy efficiency. This thesis topic was inspired by the building's current goal of achieving LEED certification. The Ting Wall system has been recognized by the LEED rating system; due to its long-lasting design, as a sustainable system.

Thermal Damper and Waterproofing

The glass curtain wall will be redesigned as a Ting Wall system. This system uses the functional isolation concept as opposed to the functional combination concept; the functions of sealing water and air are completely separated through the system. Durable water-tightness performance is achieved due to large tolerances to various structural movements.

The frame is designed to limit thermal conductivity by utilizing an I-Strut system for the thermal break to maximize the distance between the exterior and interior extrusion component. It also limits air infiltration through the Airloop system. In the summer there is a cooling effect due to natural air venting of the inter-connected airloops. Added insulation is provided in the winter by the "near still" air in the airloops.



1. **1st Outer Airloop™ (1st OAL)**
   The 1st Outer Airloop™ is a wet loop designed with instantaneous drainage capability. A continuous perimeter airspace, open to the exterior air, is formed in the panel extrusion frame around each individual panel and between adjacent panels on all sides.
2. **Inner Airloop™ (IAL)**
   The inner Airloop™ is a dry loop. This airspace is formed between the perimeter extrusion and the facing material of each panel. Horizontal cavities are connected to vertical cavities through miter-matched corners, allowing pressure-equalized air around all sides within each individual panel.
3. **Pressure Equalization Vent**
   The Inner Airloop™ is pressure equalized with the exterior air via vent holes connecting the Inner Airloop™ with the 1st Outer Airloop™, beyond the water path.
4. **2nd Outer Airloop (2nd OAL)**
   The 2nd Outer Airloop™ is also a dry loop. This airspace is formed around each panel -- between adjacent panels and between panels and mullions. This airspace is pressure equalized via a noncontinuous sealant tape attached to the horizontal water seal member (#5), which connects the 1st Outer Airloop and the 2nd Outer Airloop™, beyond the water path.

Figure 51: Airloop System

Perimeter Structural Framing Adjustments

The tingwall system chosen was system 75 which has a weight of 8 psf. This is much lower than the original system which has a weight of 12 to 15 psf. The cost of the tingwall system is about the same as a conventional unitized system; relatively 1:1. RAM modeler was used to determine the member sizes for the gravity columns and beams. The load applied to the diaphragms can be found in the *loading diagrams* section of the appendix. The line load applied from the Ting-Wall system was 10 psf along the perimeter, which is for a thermally broken system. The Foundation was modeled as a three feet mat foundation.

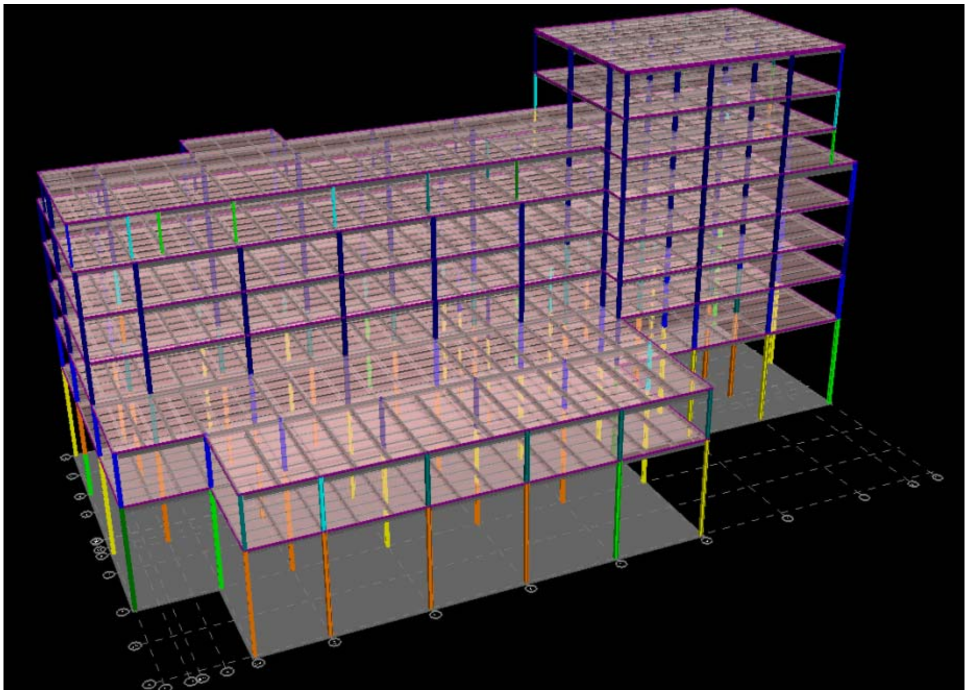


Figure 52: RAM model for Gravity Beams and Columns

Since the ting wall system is lighter than the existing façade, the structural steel weight was expected to decrease along with the cost. Take –offs were done for the structural steel material cost, labor cost, and equipment cost. An allocation factor of 1.06 was applied for New York, New York. It was found that the new gravity system would cost $2,771,500; that is about a 14% decrease in cost.

Structural Advantages for Ting Wall

Wind load forces are transferred into the mullion by mechanical inter-lock, thereby eliminating the need for screws which are subject to stress fatigue. Ting Wall claims that it is "Practically non-destructible if the building is standing after earthquake." And when considering floor live load, the tolerance for inter-floor spandrel beam deflection is up to ¾" deflection. This is possible because each Ting Wall panel is structurally isolated allowing it to use panel drifts to absorb the story drift with insignificant stress. Slotted casement allows vertical and horizontal movement independent of each other.

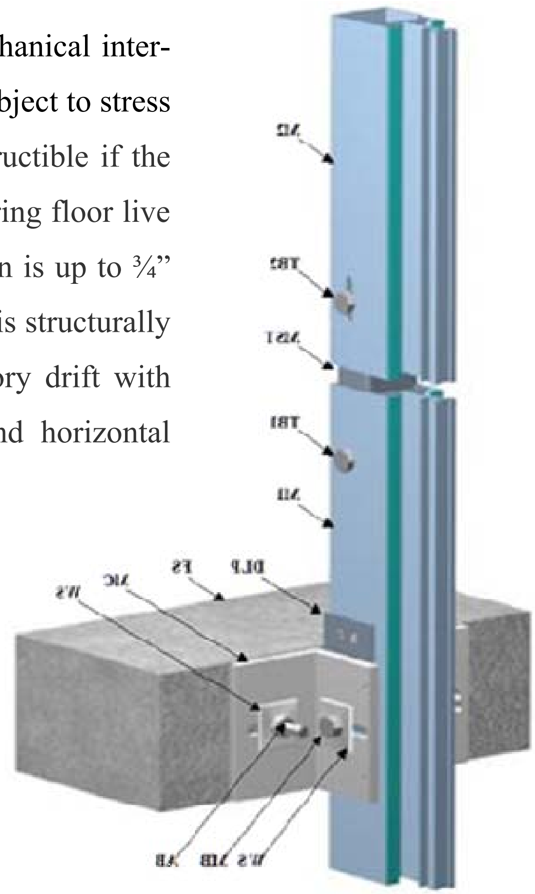


Figure 53: Ting Wall Structural System

Ting Wall Sustainability points toward LEED

- Sustainable site : 14pts
- Water efficiency: 5pts
- Energy and atmosphere: 17 pts
- Materials and resources: 13 pts
- Indoor environmental quality: 15 pts
- Innovation and Design Process : 5pt

Redesign of Green Roof

Hunter College School of Social Work is currently going for LEED Silver certification. Green roof filtration systems will be looked at closely to determine if any changes should be made. A green roof redesign will be performed since they currently cover two roof levels. The water retention tank capacity is expected to change. The viability of the new green roof and water retention tank will be analyzed.

The only allowed manufacturer listed in the building specifications for green roofs was American Hydrotech Inc. After much review of the drainage system found in the consruction documents, and of the web media presented by American Hydrotech Inc., I found that it appears to be well-designed and I am confident that if built as designed, that it will perform well. Below is a green roof detail that shows the design of the drainage system.

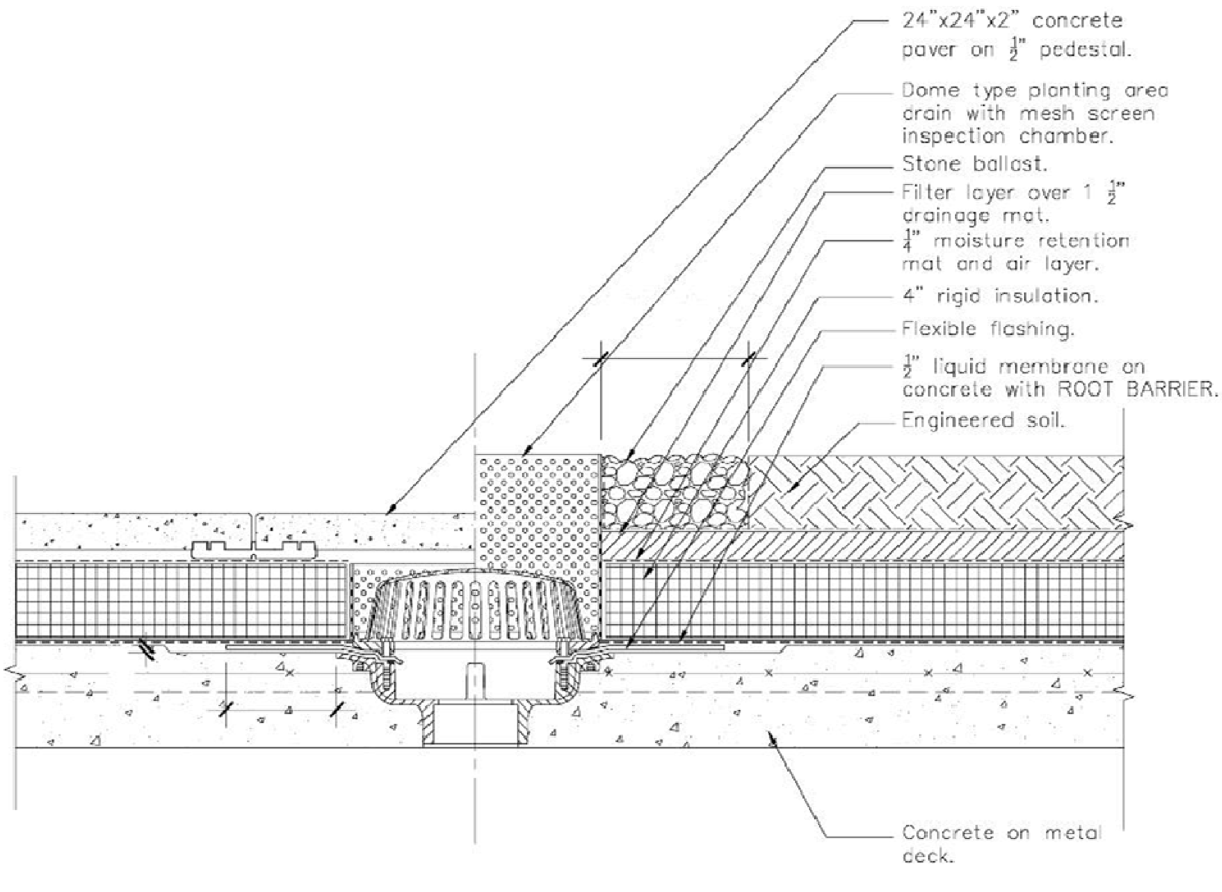


Figure 54: Detail at Green Roof Drain

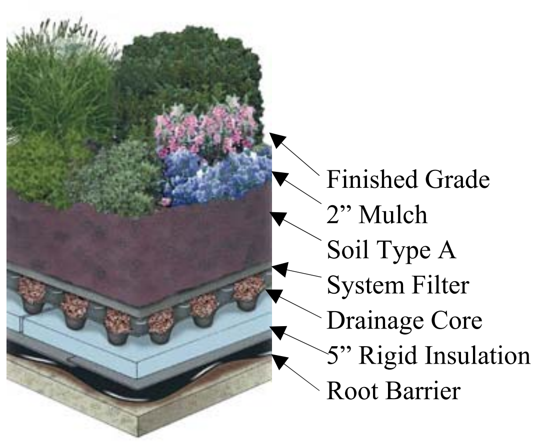
For my green roof redesign I have chosen to increase the available green roof area and to determine the impact on the storm water tank as well as the impact on energy savings and cost. As shown in Figure 55, The green roof on the ground level acts like a courtyard and the green roof on the second level allows for viewing into the courtyard. The second level green roof has seating areas, however I feel that the space is not intimate enough and I have proposed a new landscaping layout. The new layout will increase green roof coverage as well as provide students and faculty with more intimate spaces to sit and talk.

In addition to the second level green roof redesign I am also proposing an additional green roof on the fifth level, facing E119th Street. This will replace the existing IRMA roof, and will provide the long string of offices on the level with a green view which is uncommon in the city. Unlike the green roof on the second level, the roof on the fifth level will be an extensive green roof. This means that the growth media will be shallow and won't support much more than sedums. Also, pedestrian traffic will be prohibited, only access will be allowed to maintenance for accessing the mechanical system on the roof above the fifth floor. The added green roof space will help to improve the air quality, reduce combined sewer overflows, reduce noise, and extend waterproofing longevity.

Figure 55: Bird-View of Hunter College School of Social Work's proposed Green Roofs

Components of the Green Roof

The green roof uses a lightweight engineered soil to reduce the roof load. Shown below is an intensive green roof with an average planting media of eighteen inches. The original design calls for a green roof area of 4747 square feet on the second level. The new design increases the second level green roof area to 5100 square feet. The green roof on the first level is left unchanged with an area of 1222 square feet. Finally the additional green roof on the fifth level has an area of 3833 square feet.



Manufacturer	American Hydrotech Inc.
Growth Media	LiteTop Type A Engineered Soil
Avg. Planting Medium Depth	18 inches
Drainage Core	Gardendrain GR50
Moisture Retention Fabric	Hydrotech Moisture Retention Mat
Filter Fabric	Systemfilter SF

Figure 56: Green Roof Components Specifications

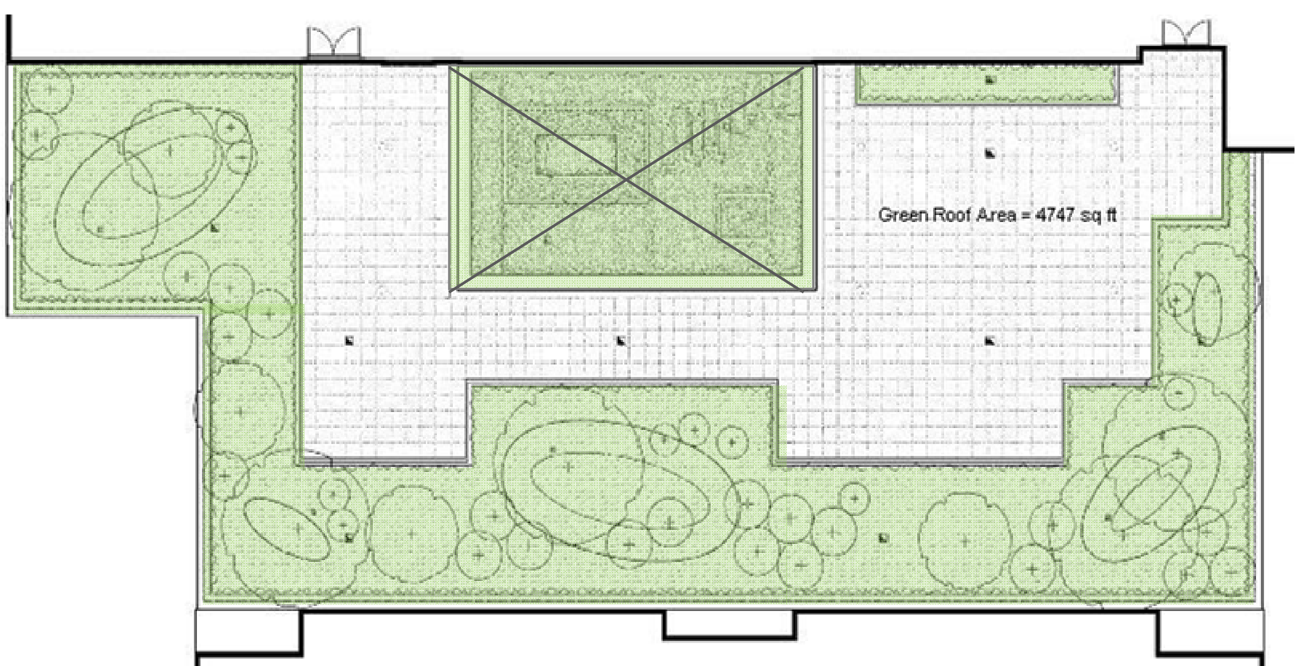


Figure 57: Original Green Roof Design

Final Green Roof Designs

On the First and Second Story Levels

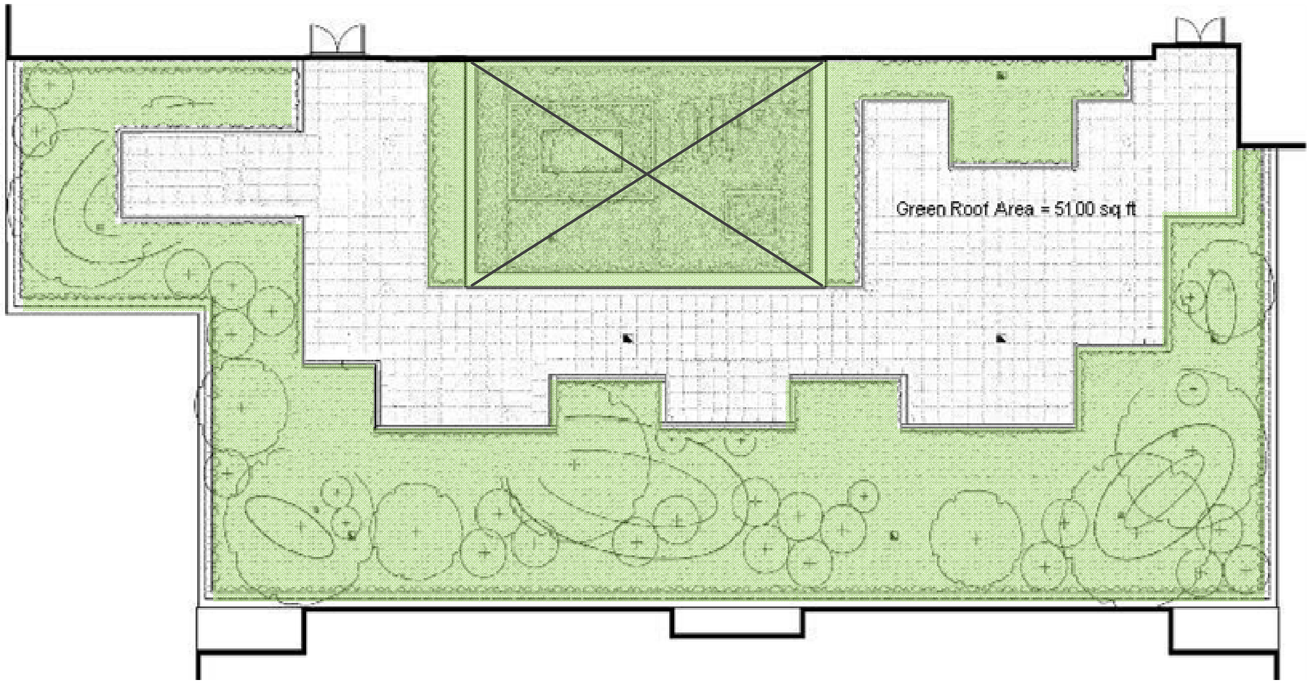


Figure 58: Redesigned Intensive Green Roof at Second Story Level

On the fifth story level

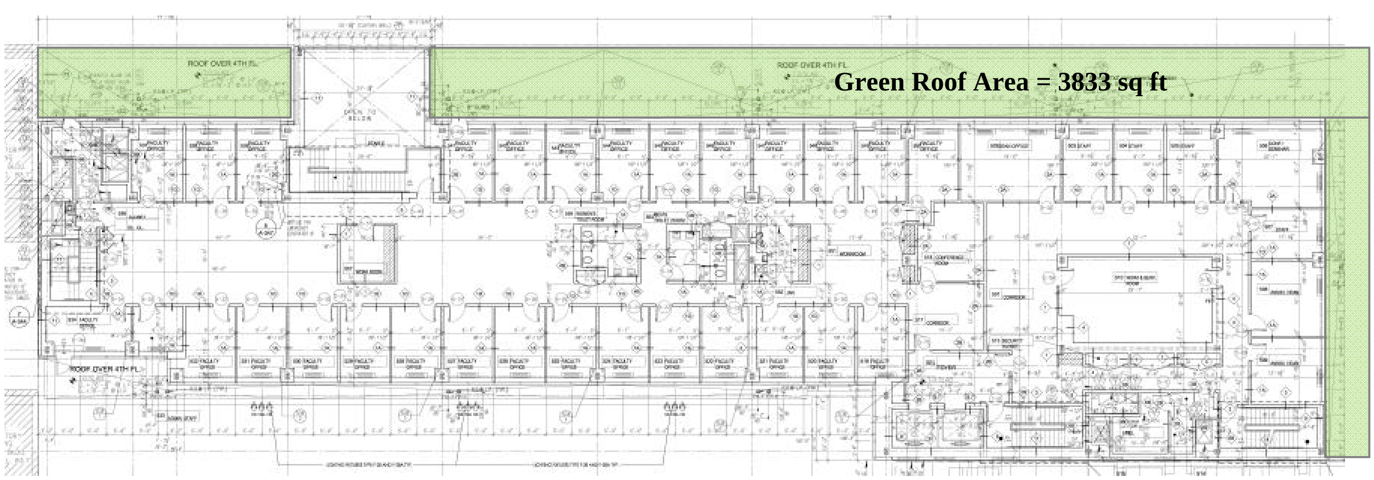


Figure 53: Redesigned Extensive Green Roof at Fifth Story Level

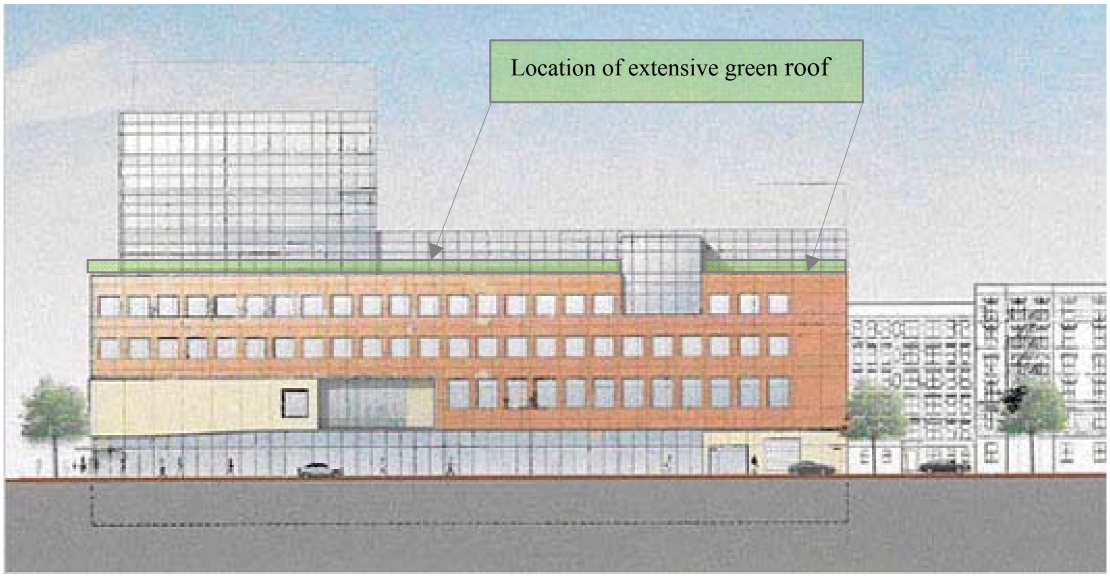


Figure 59: Location of Redesigned Extensive Green Roof

Figure 60: Extensive Green Roof Installed in Allentown, PA

The benefits of the fifth level green roof as the scenic views as well as avoiding the use of gravel near so much glass. The offices on the fifth level facing 199th Street as well as the ones on the back side of the building now have views of green roofs with the proposed redesign. The vegetation chosen for the fifth level green roof is Mexican sedum and coral carpet due to their ability to withstand harsh conditions. These sedum were also chosen because they require less than 4 inches of growing media which is ideal for extensive roofs.

Bright green sedum with yellow flowers in the spring. A fast grower compare to other sedums. Slightly frost-tender but always rebounds.

Dark Green in non-drought conditions and bright red in drought conditions. An extremely hardy and versatile plant.

Figure 55: Sedum types to be planted on the extensive roof

Stormwater Detention Tank Capacity

"Each 10,000-sq-ft green roof can capture between 6,000 and 12,000 gal of water in each storm event. This is rainfall that will never enter the combined sewer. At the same time, the evaporation of this rainfall will produce the equivalent of between 1,000 and 2,000 tons of air conditioning--enough heat removal to noticeably cool 10 acres of the city. This is a management practice that increases biodiversity and can literally add enjoyable landscape to all the boroughs of New York".

Currently there is a storm water management tank designed to hold 12, 000 gallons of rainwater runoff. The dimensions of the tank are 33’x19.5’x3.5’. the volume of the tank is equal to 16, 000 gallons. Determining the size of the tank needed for a particular roof depends on the regional 10-year, 24-hour rainfall, for New York City, this value is 5 inches (Based from New York State Stormwater Management Design manual, Fig 4.5, 10-yr Design Storm).

Tabulated below is the required stormwater capacity for each of the green roofs, both before and after my redesign. The required stormwater capacity before the redesign was 11823 gallons which is just under the designed for capacity of 12000 gallons. The new design calls for a 15000 gallon stormwater tank . Assuming that the current tank can handle the remaining 3000 gallons; since it has a volume of 16000 gallons, the structural integrity of the dunnage platform will be checked to insure that it had handle the extra stormwater load.

Original Design of Second Level Green Roof	
Roof	
Green Roof Surface Area (sq ft)	4747
Rain Fall	
Regional 10 yr storm (inches of rainfall)	5
Growth Media	
Growth media depth (inches)	18
Dry Weight (pounds per cubic ft)	38
Saturated Weight (pounds per cubic ft)	62
Moisture Retention Fabric	
Moisture retention fabric dry weight/sq ft	0.11
Moisture retention fabric saturated weight/ sq ft	1.2
Drainage Core	
top diameter of cups (inches)	1.5
botttom diameter of cups (inches)	0.25
cup height	2
number of cups per sq ft	36
Water retained (gallons per sq ft)	4.67
Weight of retained water (lbs per square foot)	39.92
Total gallons retained	22151.44
Run off coefficient	-0.50
Storwater Tank Capacity required (gallons)	11075.72

Original Design of First Level Green Roof	
Roof	
Green Roof Surface Area (sq ft)	1222
Rain Fall	
Regional 10 yr storm (inches of rainfall)	5
Growth Media	
Growth media depth (inches)	8
Dry Weight (pounds per cubic ft)	38
Saturated Weight (pounds per cubic ft)	62
Moisture Retention Fabric	
Moisture retention fabric dry weight/sq ft	0.11
Moisture retention fabric saturated weight/ sq ft	1.2
Drainage Core	
top diameter of cups (inches)	1.5
botttom diameter of cups (inches)	0.25
cup height	2
number of cups per sq ft	36
Water retained (gallons per sq ft)	2.27
Weight of retained water (lbs per square foot)	18.92
Total gallons retained	2771.90
Run off coefficient	0.27
Storwater Tank Capacity required (gallons)	748.41

Redesign of Second Level Green Roof	
Roof	
Green Roof Surface Area (sq ft)	5100
Rain Fall	
Regional 10 yr storm (inches of rainfall)	5
Growth Media	
Growth media depth (inches)	18
Dry Weight (pounds per cubic ft)	38
Saturated Weight (pounds per cubic ft)	62
Moisture Retention Fabric	
Moisture retention fabric dry weight/sq ft	0.11
Moisture retention fabric saturated weight/ sq ft	1.2
Drainage Core	
top diameter of cups (inches)	1.5
botttom diameter of cups (inches)	0.25
cup height	2
number of cups per sq ft	36
Water retained (gallons per sq ft)	4.67
Weight of retained water (lbs per square foot)	39.92
Total gallons retained	23798.68
Run off coefficient	-0.50
Storwater Tank Capacity required (gallons)	11899.34

Redesign of Fifth Level Roof - Extensive green roof	
Roof	
Green Roof Surface Area (sq ft)	3833
Rain Fall	
Regional 10 yr storm (inches of rainfall)	5
Growth Media	
Growth media depth (inches)	3.5
Dry Weight (pounds per cubic ft)	18
Saturated Weight (pounds per cubic ft)	34
Moisture Retention Fabric	
Moisture retention fabric dry weight/sq ft	0.11
Moisture retention fabric saturated weight/ sq ft	1.2
Drainage Core	
top diameter of cups (inches)	0.5
botttom diameter of cups (inches)	0.25
cup height	59/100
number of cups per sq ft	100
Water retained (gallons per sq ft)	0.72
Weight of retained water (lbs per square foot)	6.00
Total gallons retained	2757.78
Run off coefficient	0.77
Storwater Tank Capacity required (gallons)	2123.49

Figure 61: Stormwater Management Capacity for Green Roofs

Structural Integrity of Dunnage Base

The Dunnage platform was able to support the added load of 3000 gallons of water. Detailed hand calculations can be found in the appendix. Below is a summary of the structural steel member stresses and capacity both beore and after the green roof redesign.

		12000 Gallon Tank	15000 Gallon Tank
Member Size	ΦMn (ft-k)	Mu (ft-k)	Mu (ft-k)
W8x28	69	34.4	41.2
W12x40	160.5	75	88.2
W10x33	101	75	88
W8x35	130	75	88.2
Member Size	ΦPn (k)	Pu (k)	Pu (k)
W8x35	429.5	46	53.6

Figure 62: Dunnage Platform Stresses and Strength

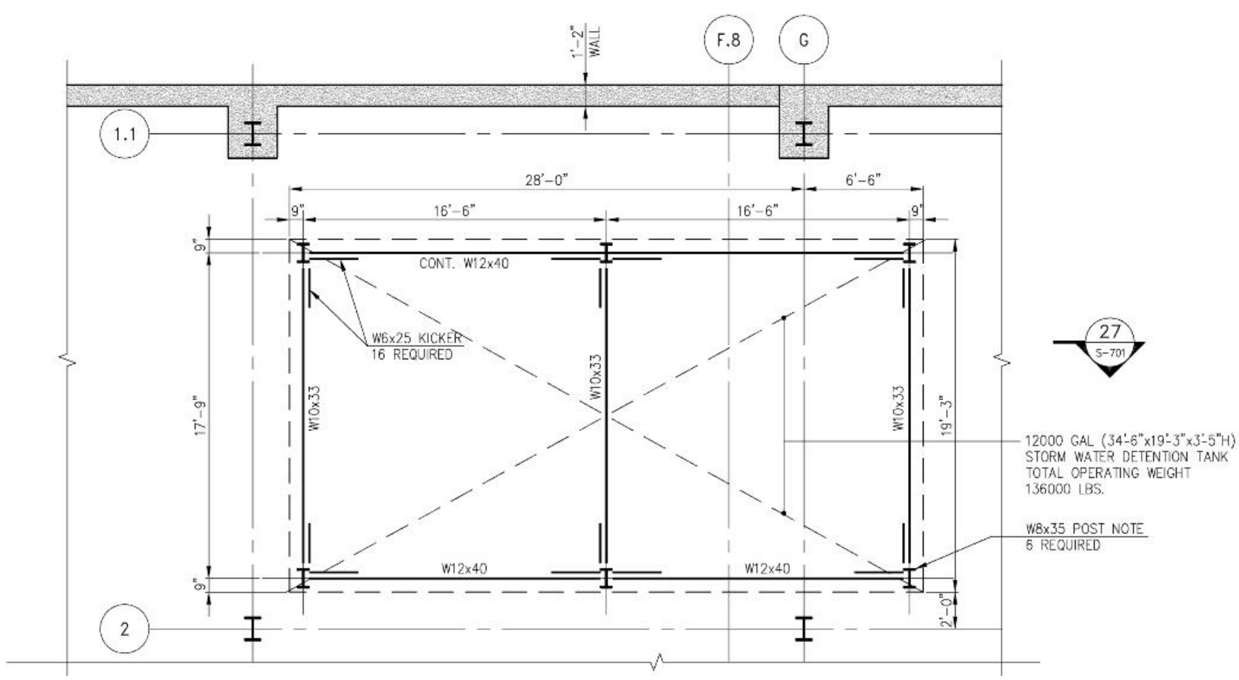


Figure 63: Watertank Dunnage Platform

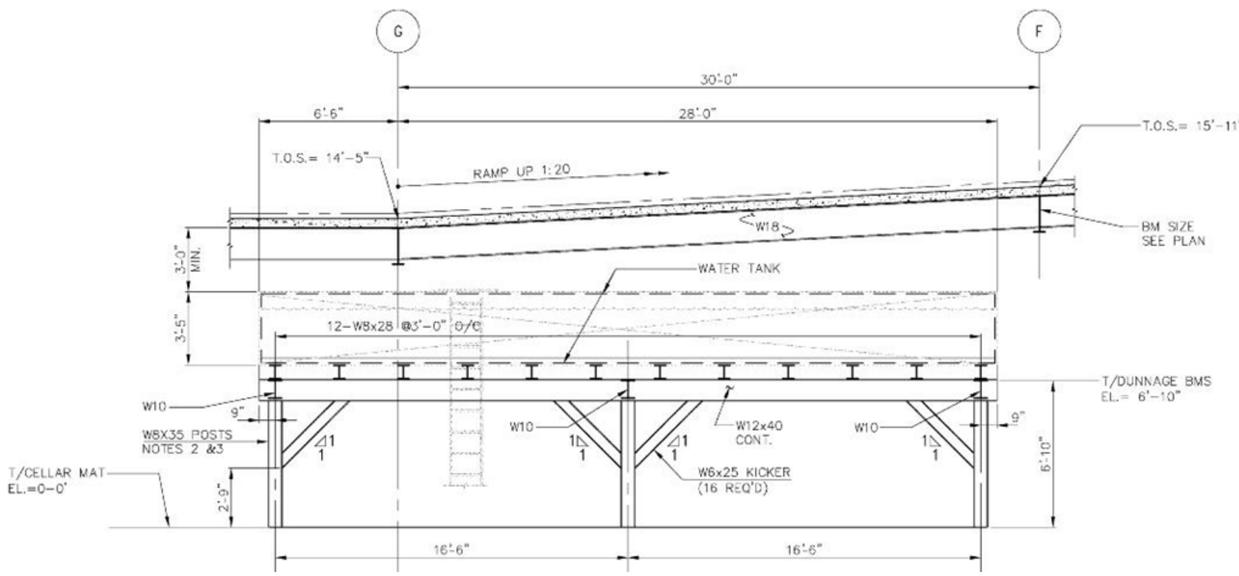


Figure 64: Section through Dunnage Platform Supporting Water Tank

Energy Savings Comparison between Existing and New Roof Plans

Energy savings with the green roof redesign are an additional $173 per year. This may not seem like much relative to the initial cost of green roofs, but every year the savings would amount to 8 square feet of extensive roof initial cost. With the tax incentive, the payback period is 11 square feet of extensive roof per year. This means that the extensive green roof would pay for itself in 384 years.

		Energy Savings Compared to a Conventional Roof			
		Electrical Savings	Gas Savings	Total Energy Cost Savings/roof	Total Energy Cost Savings/bldg
Original Design	First Floor	167.02 kWh /yr	31.21 Therms/yr	$79.99/yr	256.96/yr
	Second Floor	375.79 kWh/yr	70.22 Therms/yr	$179.97/yr	
	Fifth Floor	0	0	0	
New Design	First Floor	167.02 kWh /yr	31.21 Therms/yr	$79.99/yr	429.94/yr
	Second Floor	417.54kWh/yr	78.02 Therms /yr	$199.97/yr	
	Fifth Floor	313.16 kWh/yr	58.52 Therms/yr	$149.98/yr	

Figure 65: Energy Savings due to Green Roofs

With such an unreasonable pay-back period, one may wonder why not just install a reflective roof? The reason is that there are many benefits to green roofs that aren't easily quantified. These include environmental, social, and economic benefits.

Green roofs help to reduce the urban heat island effect by staying 40-50 degress (F) cooler than conventional roofs on a hot day. They can also reduce stormwater runoff by retaining a large portion of stormwater, therefore reducing the volume and velocity and reducing erosion and sedimentation of natural water sources. Air quality also improves with the implementation of green roofs because they filter airborne particles such as smog, sulpher dioxide and carbon dioxide.

Social benefits include esthetic appeal, education opportunities, usable green space, and the green roof industry creates jobs. Green roofs provide green space throughout urban areas where space is limited and provides a natural beauty of green roofs far different from the concrete hard-scape of urban areas.

Some economic benefits include the following:

- Reduce the life cycle cost of the roof
- Save on energy costs
- Provide sound insulation (1"soil=10 decibel reduction)
- Decrease need for storm water infrastructure expansion
- Credits for storm water impact fees

Under a law (A. 11226), New York building owners in New York who install green roofs on at least 50 percent of available roof top space can apply for a one-year property tax credit of up to $100, 000. The credit would be equal to $4.50 per square foot of roof area that is planted with vegetation, or approximately 25 percent of the typical costs associated with the materials, labor, installation and design of the green roof. This law would not have been applicable to the original roof since only 28% of the roof was green. With the new design 51% of the roof is green making the addition to the fifth floor roof well-worth the expense. The tax break money from the original green roofs alone would be $26, 861, this amount along with the tax break from the fifth level roof could potentially pay entirely for the new extensive roof provided that the cost is only $10 per square foot.

Cost and Schedule Analysis

TingWall

	New Gravity Frame Design	Original Gravity Frame Design
Adjusted for Location	$ 2, 309, 608	$ 2, 689, 200
Design Contingency (1.5%)	$ 34, 600	$ 40, 300
Escalation Contingency (3.5%)	$ 80, 800	$, 94, 100
Insurance (3%)	$ 69, 300	$ 80, 700
Bonds (10%)	$ 46, 200	$ 53, 800
Overhead and Profit (10%)	$ 221, 000	$ 268, 921
Total Structural Steel Cost	$ 2, 771, 500	$ 3, 227, 100

The cost of erecting a Ting Wall curtain wall is the same as a typical unitized curtain wall. The erection of Ting Wall may actually be easier because each panel unit involves one piece of facing material only. There is a true guarantee on completion date due to the ability of simultaneous multiple point erection, in other words, there is no left-to-right directional restriction in erection.

Roofing

An extensive roofing system costs about $5 to $10 per square foot (above the cost of a conventional roof), this includes drainage, filtering, paving, and growing medium. And has an additional roof load of 15-30 psf. The lifecycle costs include maintenance which is $1.50 per square foot (only for the first two years). For cost estimation, the extensive roof is taken to cost $10 per square foot.

For semi-intensive roofing the additional roof load is about 25-50 psf and the additional cost is about $10-$20 per square foot.

An intensive roof weighs 40-150+ psf. For intensive roofing systems, the life cycle cost includes irrigation for $3.00 per square foot. Intensive roofing costs $15 to $30 per square foot; for cost estimation, it was taken to be $20 per square foot.

	Green Roof (New Design)	Green Roof + IRMA Roof (Original)
Material Cost	$164,770	$119,380
Tax Deduction	$4.50/sq ft = $ 45,698.	n/a (50% or more of roof needs to be green)
Total Cost	$119, 072	$119, 380

For intensive roofs the installation and labor is $5.50 / sq ft. Other costs include design and specifications fee which can be between 5% and 10% of the total roofing cost. Project Administration and Site Review which can be 2.5% to 5% of the total roofing cost.

		Energy Savings Compared to a Conventional Roof			
		Electrical Savings	Gas Savings	Total Energy Cost Savings/roof	Total Energy Cost Savings/bldg
Original Design	First Floor	167.02 kWh /yr	31.21 Therms/yr	$79.99/yr	**256.96/yr**
	Second Floor	375.79 kWh/yr	70.22 Therms/yr	$179.97/yr	
	Fifth Floor	0	0	0	
New Design	First Floor	167.02 kWh /yr	31.21 Therms/yr	$79.99/yr	**429.94/yr**
	Second Floor	417.54kWh/yr	78.02 Therms /yr	$199.97/yr	
	Fifth Floor	313.16 kWh/yr	58.52 Therms/yr	$149.98/yr	

Lateral System

New Design				
w14x	quantity	total length	$/ft	total cost
53	1	14	$61.48	$860.72
68	1	26	$78.88	$2,050.88
74	1	14	$87.37	$1,223.18
90	1	26	$104.40	$2,714.40
99	1	26	$114.84	$2,985.84
120	1	14	$138.96	$1,945.44
145	2	62	$168.20	$10,428.40
193	5	148	$223.88	$33,134.24
233	1	33	$274.94	$9,073.02
398	1	33	$469.64	$15,498.12
HSS				
5x5x3/8	11	1146.2	$65.10	$74,617.62
5.5x5.5x3/8	3	402.8	$72.55	$29,223.14
6x6x3/8	2	274	$79.97	$21,911.78
8x8x3/8	2	189.6	$90.60	$17,177.76
			total:	$222,844.54

Original Design				
w14x	quantity	total length	$/ft	total cost
68	1	14	$78.88	$1,104.32
90	1	14	$104.40	$1,461.60
176	1	14	$202.18	$2,830.52
233	4	111	$274.94	$30,518.34
283	3	85	$328.28	$27,903.80
311	4	99	$360.76	$35,715.24
331	1	28	$410.00	$11,480.00
342	1	33	$429.20	$14,163.60
398	1	33	$469.64	$15,498.12
455	1	33	$536.90	$17,717.70
550	1	31	$638.00	$19,778.00
730	1	33	$846.80	$27,944.40
mom connections			$620/conn	$22,320.00
HSS				
5x5x3/8	11	573.1	$65.10	$37,308.81
5.5x5.5x3/8	3	201.4	$72.55	$14,611.57
6x6x3/8	2	137	$79.97	$10,955.89
8x8x3/8	2	94.8	$90.60	$8,588.88
			total:	$299,900.79

While the actual design and detailing of a moment frame may only take a few hours to a day's work for an experienced engineer, that is only a small part of the process. In addition to designing the foundation anchorage, the engineer will need to produce steel and welding specifications, also review steel shop drawings and welding procedure specifications. A steel contractor will need to A steel sub-contractor will need to install the frame, and the general contractor will need to coordinate between the iron workers and the framers to make sure everything fits together. Field welds also increase the erection cost. In my estimates a cost of $620 per moment connection was assumed. [McEntee]

Cost and Schedule Summary

Green roof savings = $300

Lateral System Savings = $77, 100

Ting Wall Savings = $455, 600

Total Building Savings = $533,000

Summary + Conclusions

The focus of this report is energy efficiency and how it can be implemented using facade and green roof redesign. It ties structural engineering concepts with existing enclosure installation methods to provide a secure barrier against water and the temperature of the outside world.

A personal goal of mine was to show how structural engineering enters all aspects of buildings design, whether it be mechanical systems, façade, roofing, architecture, acoustics, etc… And to prove that it *is* possible to take an idea far from the structural engineering realm as LEED Sustainable Design and approach it from a structural engineering standpoint.

Changes done to the gravity and lateral system, the green roofs, and the façade seem to have paid off with a savings of $533,000. I would have liked to have optimized the beams that were a part of the lateral system and seen how much more I could have saved.

The green roof system payback period is in the order of a few hundred years. It is my recommendation that it is in the best interest to choose a reflective roof instead in the case that social and environmental benefits of green roofs are not large motivators on a project; in other words if money is an issue then green roofs are not the answer.

Through this long journey I have learned the theory behind the Uniform Force Method, tips on reducing building weight, leading to lower building costs, and to avoid moment frames whenever possible, using them only if necessary by the architect's design. Also if you decide to use them, it is better to go with heavier members to reduce to detailing of connections.

Some things to consider in future designs is that although the columns were optimized for the gravity load in this thesis, this may turn out to be more expensive in the long run, then instead sizing the columns at 75% capacity as opposed to near 100%. By designing at 75% capacity the need for doubler plates is eliminated.

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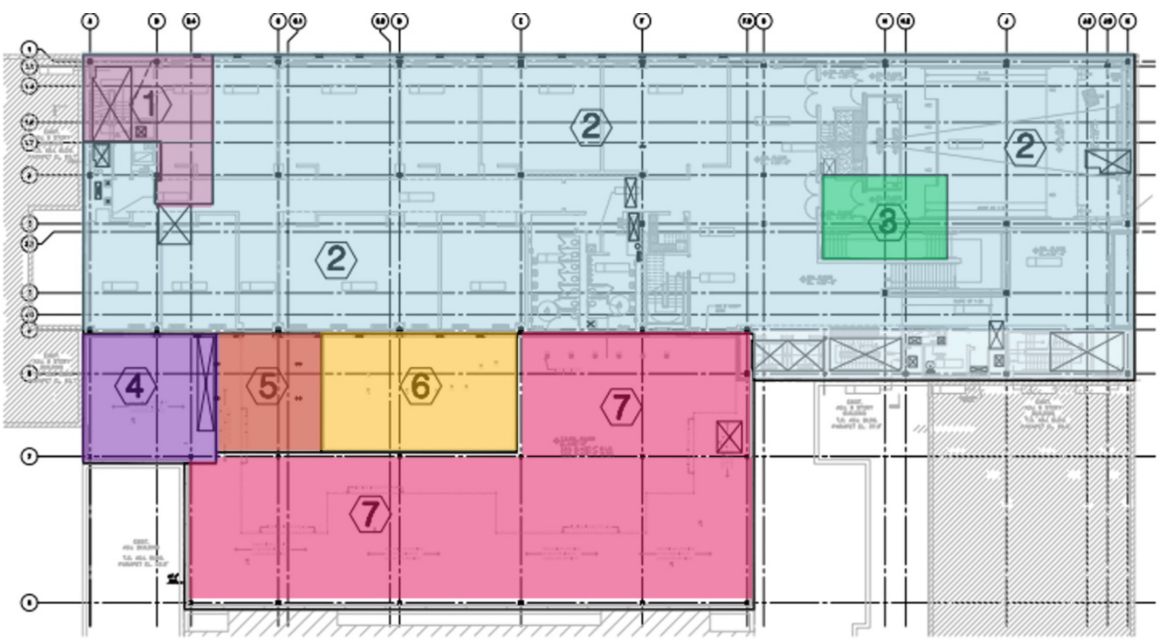
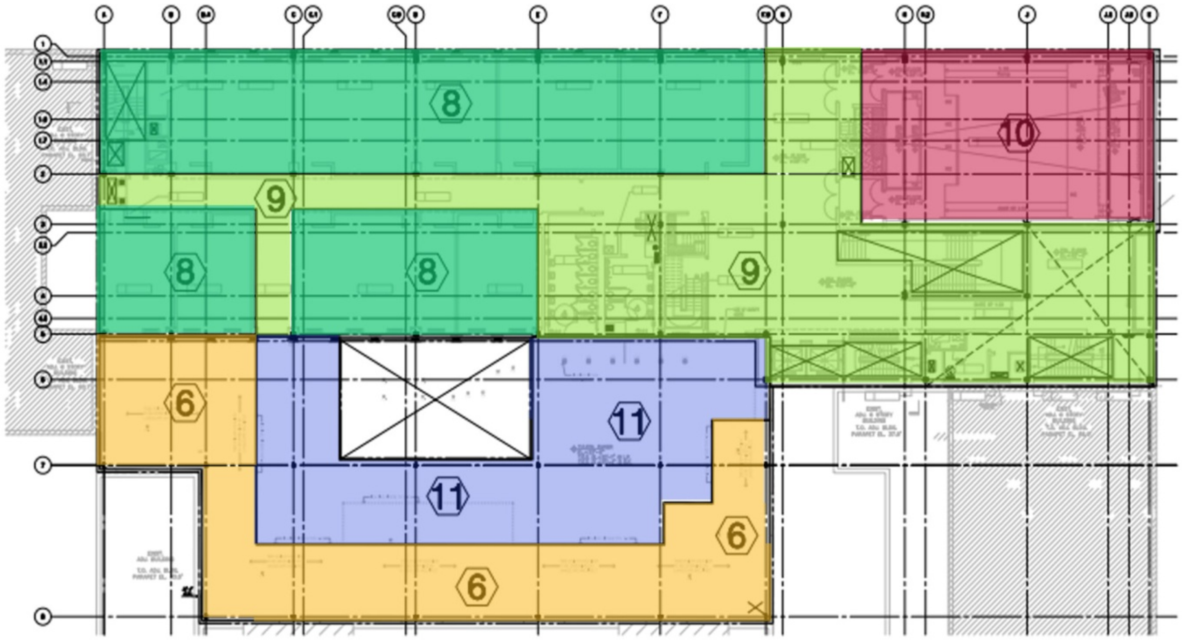
Appendix

Loading Diagrams

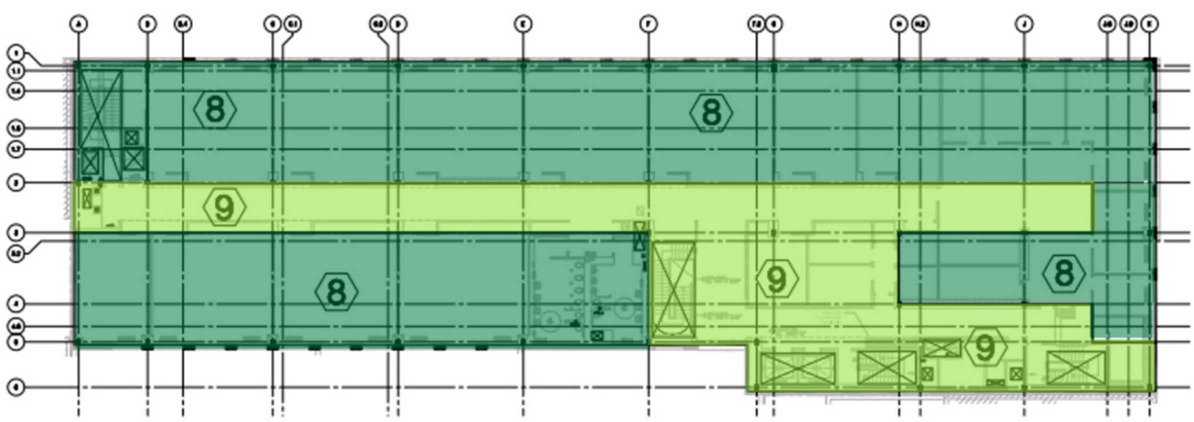
LOADING SCHEDULE		
ID	DL psf	LL psf
1. LOADING DOCK	150.0	600.0
2. 1ST FLOOR	130.0	100.0
3. PODIUM	200.0	100.0
4. ARCHIVE	75.0	350.0
5. OFFICES	71.0	50.0
6. ROOF WITH GARDEN	365.0	100.0
7. LIBRARY STACKS	71.0	100.0
8. CLASSROOMS	71.0	40.0
9. CORRIDOR	71.0	100.0
10. AUDITORIUM	85.0	60.0
11. ROOF WITH PAVERS ON 2	150.0	100.0
12. ROOF	90.0	45.0
13. ROOF WITH DRIFT	85.0	60.0
14. MECHANICAL	120.0	100.0



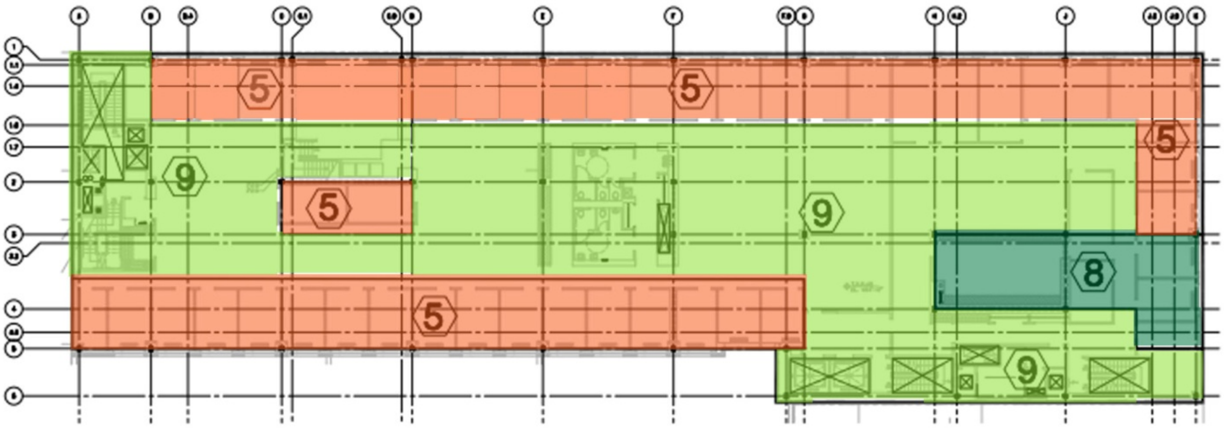
1 S-008 1ST FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



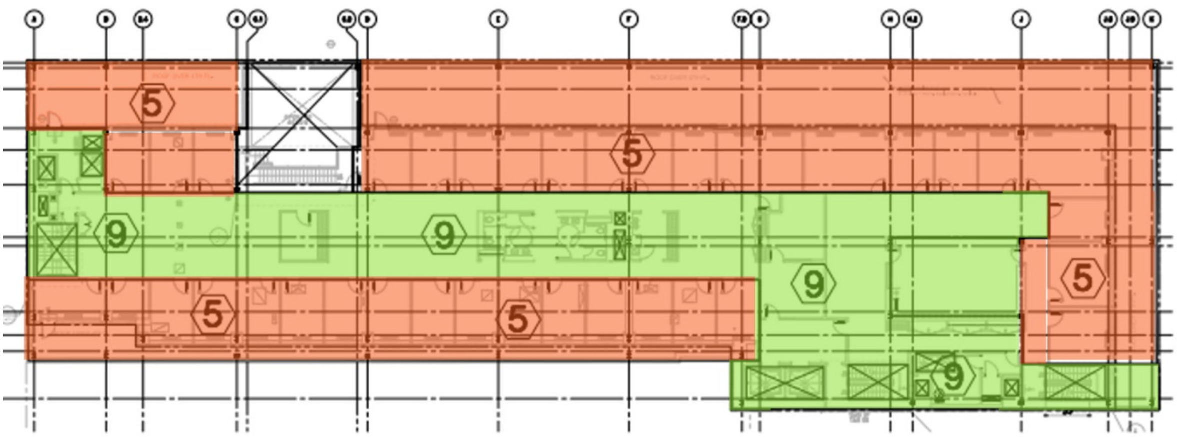
2 2ND FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



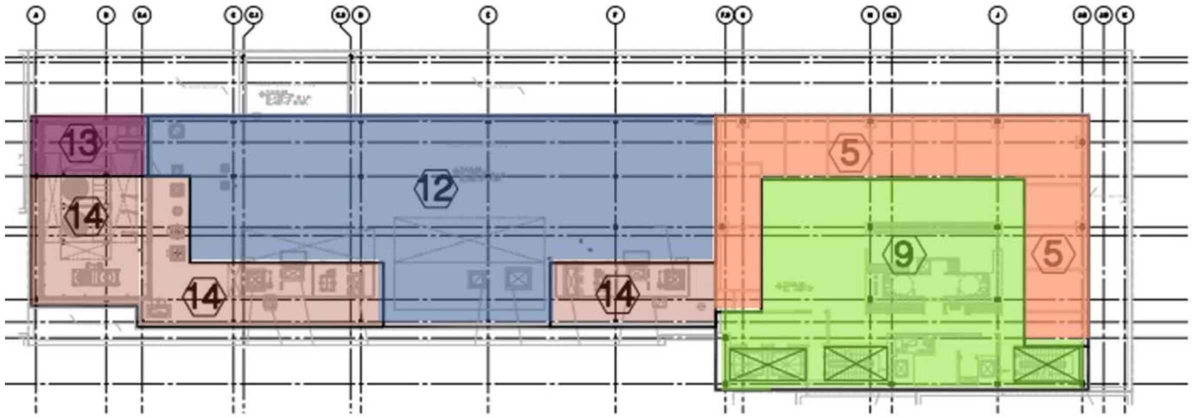
3 3RD FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



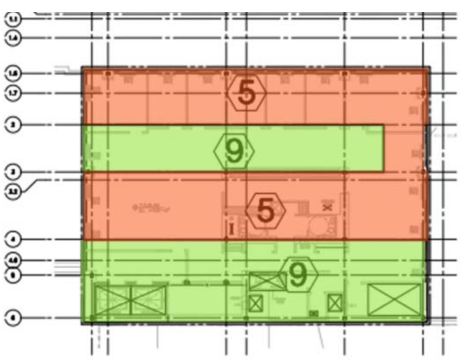
4TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



5TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"

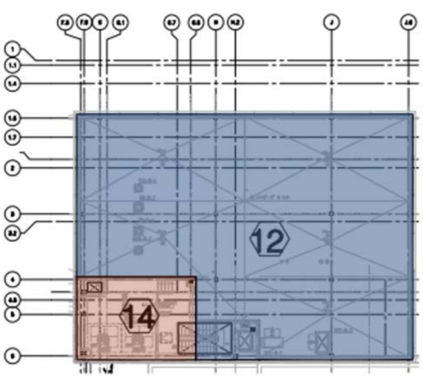


6 6TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"

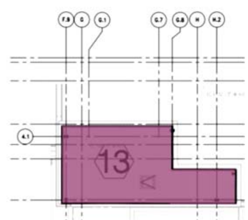


7 7TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"

8 8TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



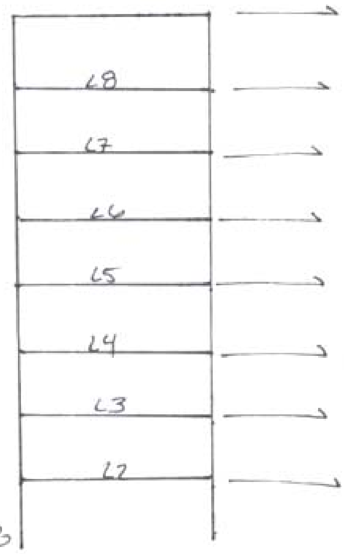
9 ROOF LOADING DIAGRAM
SCALE: ⅛"=1'-0"



10 PENTHOUSE LOADING DIAGRAM
SCALE: ⅛"=1'-0"

P-Delta Effects

ARE P-DELTA EFFECTS SIGNIFICANT ?



P_x (kips)	V_x (kips)	Δ (in.)	h_{sx} (ft)
736	36	0.88	—
1254	54	0.74	14 = 168"
1752	69	0.61	13 = 156"
3129	99	0.35	13 = 156"
4662	123	0.27	14 = 168"
6185	138	0.20	14 = 168"
7749	147	0.13	14 = 168"
11449	154	0.06	17 = 204"
			19 = 228"

TOTAL BLDG. WT. ∴ 15388

$C_d = 3\frac{1}{4}$ ORDINARY STEEL CONCENTRICALLY BRACED FRAME [TABLE 12.2-1 ASCE 7-05]

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

$$\theta_8 = \frac{(1254)(0.74)}{(54 \times 168)3.25} = 0.031$$

$$\theta_7 = \frac{(1752)(0.61)}{(69 \times 156)3.25} = 0.031$$

$$\theta_6 = \frac{(3129)(0.35)}{(99 \times 156)3.25} = 0.022$$

$$\theta_5 = \frac{(4662)(0.27)}{(123 \times 168)3.25} = 0.019$$

$$\theta_4 = \frac{(6185)(0.20)}{(138 \times 168)3.25} = 0.016$$

$$\theta_3 = \frac{(7749)(0.13)}{(147 \times 168)3.25} = 0.013$$

$$\theta_2 = \frac{(11449 \times 0.06)}{(154 \times 204)3.25} = 0.007$$

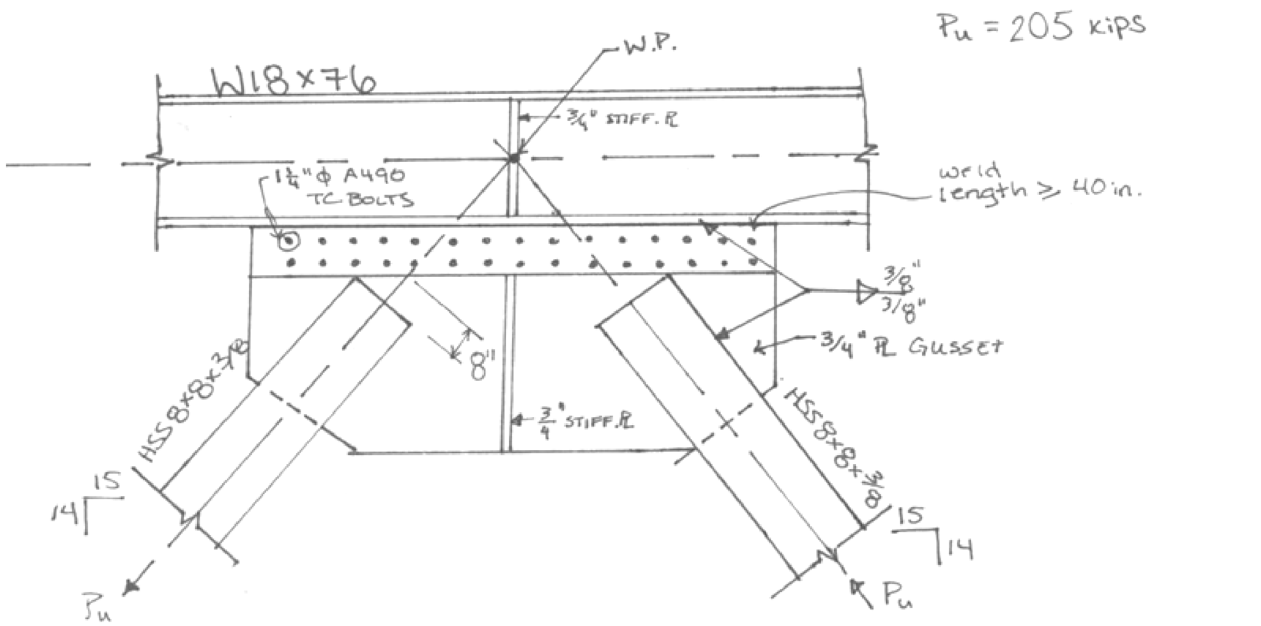
θ : STABILITY COEFFICIENT

$\theta \le 0.10$

THEREFORE WE CAN NEGLECT P-Δ EFFECT

[ASCE 7-05 § 12.8.7]

Brace Frame Connection Design



HSS CHEVRON BRACE CONNECTION DESIGN

- DETERMINE REQUIRED BRACE-TO-GUSSET WELD SIZE

SINCE THE BRACE LOADS ARE AXIAL, THE ANGLE BETWEEN THE LONGITUDINAL BRACE AXIS AND LINE OF FORCE IS $\theta_w = 0°$.

$$F_w = 0.60 F_{EXX}(1 + 0.5 \sin^{1.5}\theta_w) = 0.60(70^{ksi})(1 + 0.5 \sin^{1.5} 0°) = 42^{ksi}$$

$$W_{req'd} = \frac{P_u}{\phi\, 4\, F_w (0.707) L_w} + 1/16 \text{ in.}$$

$$= \frac{205}{(0.75)4(42\text{ ksi})(0.707)(8\text{in})} + 1/16 \text{ in}$$

$$= 0.288 + 1/16 = 0.350 \text{ in.}$$

USE 3/8" WELD.

NOTE: THE 1/16 in. ADDED TO THE WELD SIZE IS TO ACCOUNT FOR THE SLOT IN HSS.

THE MINIMUM WELD SIZE FOR THIS CONNECTION IS 3/16 in. THE REQUIRED IS LARGER THEREFORE, USE 3/8 in. FILLET WELDS. [TABLE J2.4]

- DETERMINE THE REQUIRED GUSSET PLATE THICKNESS

$$W_e = W_w - 3/8 - 1/16 = 5/16 \text{ in.}$$

$$t_{req'd} = \frac{\phi\,(0.60 F_{exx} W_e)(0.707)(2)}{\phi\,(0.60 F_{yi})}$$

$$= \frac{(0.75)(0.60)(70\text{ ksi})(1/4\text{ in.})(0.707)(2)}{1.00\,(0.60)(36\text{ ksi})} = 0.644 \text{ in.}$$

USE A 3/4 in. GUSSET PLATE

- CHECK GUSSET PLATE BUCKLING (COMPRESSION BRACE)

$$r = \frac{t_1}{\sqrt{12}} = \frac{3/4\text{ in.}}{\sqrt{12}} = 0.217 \text{ in.}$$

SINCE THE GUSSET IS ATTACHED BY ONE EDGE ONLY, THE BUCKLING MODE COULD BE A SIDESWAY TYPE AS SHOWN IN [T. C-C2.2]. IN THIS CASE $K = 1.2$

$$\frac{Kl}{r} = \frac{1.2(8'')}{0.217} = 44.2$$

Limiting slenderness ratio $4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{36}} = 134 > 44$

$$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2(29000\text{ ksi})}{(44.2)^2} = 146.5\text{ ksi} \qquad \text{[EQ E3-4]}$$

$$F_{cr} = \left[0.658^{F_y/F_e}\right]F_y = \left[0.658^{36/146.5}\right]\cdot 36\text{ ksi} = 32.5\text{ ksi} \qquad \text{[EQ E3-2]}$$

$$l = B + 2\left[(\text{CONNECTION LENGTH})\tan 30^\circ\right] = 8\text{ in.} + 2(8\text{ in.})\tan 30^\circ = 17.2\text{ in.}$$

NOTE: HERE, THE WHITMORE SECTION IS ASSUMED TO BE ENTIRELY IN THE GUSSET. THE WHITMORE SECTION CAN SPREAD ACROSS THE JOINT INTO ADJACENT CONNECTED MATERIAL OF EQUAL OR GREATER THICKNESS OR ADJACENT CONNECTED MAT. OF LESSER THICKNESS PROVIDED THAT A RATIONAL ANALYSIS IS PERFORMED.

$$A_w = l_w t_1 = (17.2\text{ in.})(3/4\text{ in.}) = 12.9\text{ in}^2 \qquad \text{[EQ E3-1]}$$

$$P_n = F_{cr} A_w = (32.5\text{ ksi})(12.9\text{ in}^2) = 420\text{ kips}$$

$$\phi P_n = 0.90(420\text{ kips}) = 378\text{ kips} > 205\text{ kips} \quad \therefore \text{OK}$$

- CHECK TENSION YIELDING OF GUSSET PLATE (TENSION BRACE)

$R_n = F_{yi} A_w = (36 \text{ ksi})(12.9 \text{ in}^2) = 464 \text{ kips}$ [EQ J4-1]

$\phi R_n = 0.90(464 \text{ kips}) = 418 \text{ kips} > 205 \text{ kips}$ ∴ OK

- CHECK SHEAR STRENGTH AT BRACE-TO-GUSSET WELDS

TRY MINIMUM WELD LENGTH, $L_w = 8$ in. [TABLE D3.1, CASE 6]

EFFECTIVE AREA, $A_e = 4L_w t = 4(8\text{in})(0.349\text{in.}) = 11.2 \text{ in}^2$

NOMINAL SHEAR STRENGTH, $V_n = 0.6F_y A_e = 0.6(46 \text{ ksi})(11.2 \text{ in}^2) = 309 \text{ kips}$

$\phi R_n = 1.00(309 \text{ kips}) = 309 \text{ kips} > 205 \text{ kips}$ ∴ OK

- CHECK SHEAR LAG FRACTURE IN HSS BRACE

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{8^2 + 2(8 \times 8)}{4(8+8)} = 3.0 \text{ in.}$$

$$U = 1 - \frac{\bar{x}}{L_w} = 1 - \frac{3.0}{8\text{in.}} = 0.625 \text{ in.}$$

$A_n = A_g - 2tt_1 = (10.4 \text{ in}^2) - 2(0.9)(0.349\text{in.})(3/4\text{in} + 1/8 \text{ in gap}) = 9.85 \text{ in}^2$

$A_e = UA_n = (0.625)(9.85) = 6.16 \text{ in.}^2$

$R_n = F_u A_e = 58 \text{ ksi } (6.16 \text{in}^2) = 357 \text{ kips}$

$\phi R_n = 0.75(357 \text{ kips}) = 268 \text{ kips} > 205 \text{ kips}$ ∴ OK

- CALCULATE INTERFACE FORCES

DESIGN THE GUSSET-TO BEAM CONNECTION AS IF EACH BRACE WERE THE ONLY BRACE AND LOCATE EACH BRACE'S CONNECTION CENTROID AT THE IDEAL CENTROID LOCATIONS TO AVOID INDUCING A MOMENT ON THE GUSSET-BEAM INTERFACE, SIMILARLY TO UNIFORM FORCE METHOD SPECIAL CASE 3.

$e_b = \frac{d}{2} = \frac{18.2}{2} = 9.1 \text{in.}$, $\theta = \tan^{-1}\left(\frac{15}{14}\right) = 47°$

Let $\alpha = e_b \tan\theta = 9.1 \tan(47°) = 9.76$ in → use 10 in.

$\beta = e_c = 0$

$r = \sqrt{(10+0)^2 + (0+9.1)^2} = 13.5$ in.

$$H_{ub} = \alpha P_u / r = \frac{(10)(205 \text{ kips})}{13.5 \text{ in.}} = 152 \text{ kips}$$

$$V_{ub} = e_b P_u / r = \frac{(9.1)(205 \text{ kips})}{13.5 \text{ in}} = 138 \text{ kips}$$

- DETERMINE REQUIRED GUSSET – TO – BEAM WELD SIZE

THE WELD LENGTH IS TWICE THE HORIZONTAL DISTANCE FROM THE WORK POINT TO THE CENTROID OF THE GUSSET – TO – BEAM CONNECTION, α, FOR EACH BRACE. THEREFORE $l = 2\alpha = 2(10\text{ in.}) = 20\text{ in.}$

$$D_{req'd} = \frac{1.25\,P_u}{1.392\,l} = \frac{1.25(205\text{ kips})}{1.392\,(20\text{in.})(2)} = 4.60$$

CAN USE A 1/4 in FILLET WELD, 40-in. LONG TOTAL. [TABLE J2.4]
BUT WILL USE A 3/8 in. FILLET WELD INSTEAD.

- CHECK GUSSET THICKNESS (AGAINST WELD SIZE FOR REQ'D STRENGTH)

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(4.60)}{58\text{ ksi}} = 0.49\text{ in.} < \frac{5}{8}\text{ in.} \therefore OK$$

- CHECK LOCAL WEB YIELDING OF THE BEAM

$$R_n = (N + 5k)F_y\,t_w = [20\text{ in.} + 5(1.08\text{ in.})](50\text{ ksi})(0.425\text{ in.}) = 540^{\text{kips}}$$

$$\phi R_n = 1.00\,(540) = 540\text{ kips}$$

$$205\text{ kips}(\cos 47^\circ) = 140\text{ kips}$$

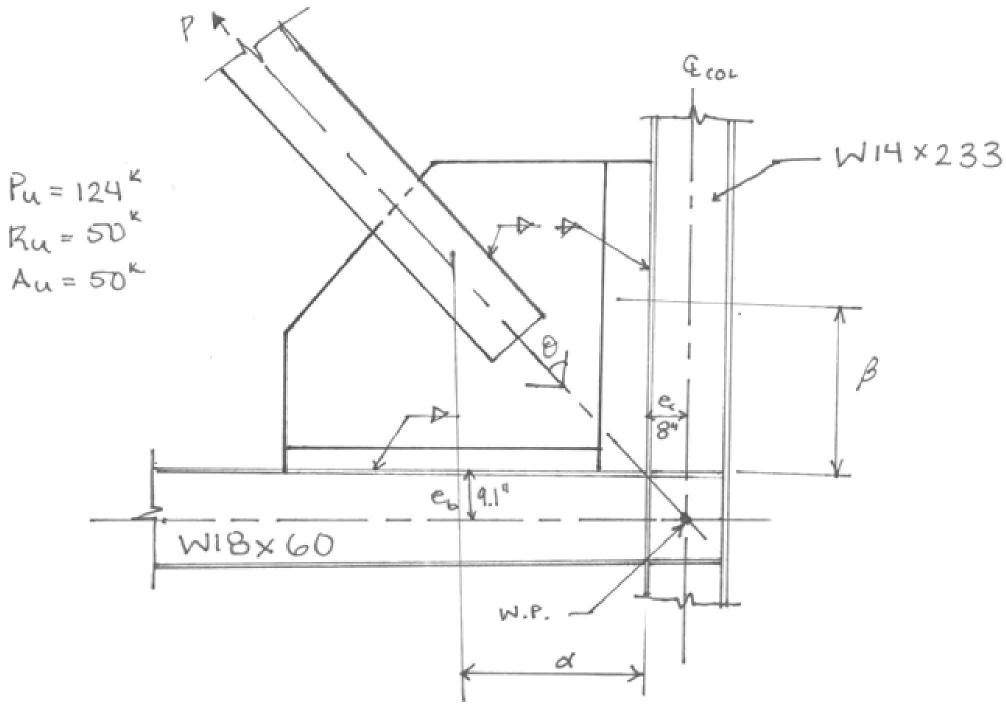
$$540 > 140 \quad \therefore OK \qquad \text{[EQ. J10-2]}$$

- DESIGN GUSSET – TO BEAM BOLT CONNECTION

$H_{ub} = 152$ kips
$V_{ub} = 138$ kips

$1\frac{1}{4}''\,\phi$ A490 BOLT < TENSION : $\phi r_n = 104$ kips; SHEAR : $\phi r_n = 110$ kips

(2 bolts)(104 kips) = 208 kips > 152 kips ∴ OK
(2 bolts)(110 kips) = 220 kips > 138 kips ∴ OK



CONNECTION DESIGN USING UNIFORM FORCE METHOD

↳ SPECIAL CASE 2 : SHEAR IN BEAM-TO-COLUMN CONNECTION MINIMIZED

GOAL: DO NOT TRANSFER MOMENT TO HORIZONTAL MEMBERS. USE THE FOLLOWING EQUATION TO ACHIEVE THIS :

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c \quad \text{[AISC MANUAL § 13-3]}$$

GIVEN: $e_b = 9.1"$ ASSUME: $\beta = 10\tfrac{1}{2}"$

$e_c = 8"$

$\tan\theta = 17/13$

$$\alpha = e_b \tan\theta - e_c + \beta \tan\theta$$
$$= 9.1(17/13) - 8 + 10.5(17/13)$$
$$= 17.6 \text{ in.}$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

$$= \sqrt{(17.6 + 8)^2 + (10.5 + 9.1)^2} = 32.2 \text{ in.}$$

CALCULATE THE INTERFACE FORCES

- ON THE GUSSET-TO-COLUMN CONNECTION

$$V_{uc} = \frac{\beta}{r} \cdot P_u = \frac{10\frac{1}{2}}{32.2}(124^k) = 40.4 \text{ kips}$$

$$H_{uc} = \frac{e_c}{r} \cdot P_u = \frac{8}{32.2}(124^k) = 30.8 \text{ kips}$$

- ON THE GUSSET-TO-BEAM CONNECTION

$$H_{ub} = \frac{\alpha}{r} \cdot P_u = \frac{17.6}{32.2}(124^k) = 67.8 \text{ kips}$$

$$V_{ub} = \frac{e_b}{r} \cdot P_u = \frac{9.1}{32.2} \cdot (124^k) = 35.0 \text{ kips}$$

- ON THE BEAM-TO-COLUMN CONNECTION, THE SHEAR IS,

$$R_{ub} + V_{ub} = 50 + 35 = 85 \text{ kips}$$

- AND THE AXIAL FORCE IS,

$$A_{ub} + H_{uc} = 50^k + 30.8^k = 80.8 \text{ kips}$$

- NEXT APPLYING SPECIAL CASE 2 WITH $\Delta V_{ub} = V_{ub} = 35^k$; CALCULATE THE NEW INTERFACE FORCES

- ON THE GUSSET-TO COLUMN CONNECTION

$$V_{uc} = 40.4 + 35 = 75.4 \text{ kips}$$

$$H_{uc} = 30.8 \text{ kips (unchanged)}$$

- ON THE GUSSET-TO-BEAM CONNECTION

$$H_{ub} = 67.8 \text{ kips (unchanged)}$$

$$V_{ub} = 35 - 35 = 0 \text{ kips}$$

$$M_{ub} = (\Delta V_{ub})\alpha = \frac{(35 \text{ kips})(17.6 \text{ in.})}{12 \text{ in/ft}} = 51.3 \text{ kip-ft}$$

- ON THE BEAM-TO-COLUMN CONNECTION THE SHEAR IS,

$$R_{ub} + \Delta V_{ub} - \Delta V_{ub}$$
$$= 50 + 35 - 35 = 50 \text{ kips}$$

- AND THE AXIAL FORCE IS,

$$A_{ub} + H_{uc} = 80.8 \text{ kips (unchanged)}$$

Storm Water Dunnage Platform Structural Capacity

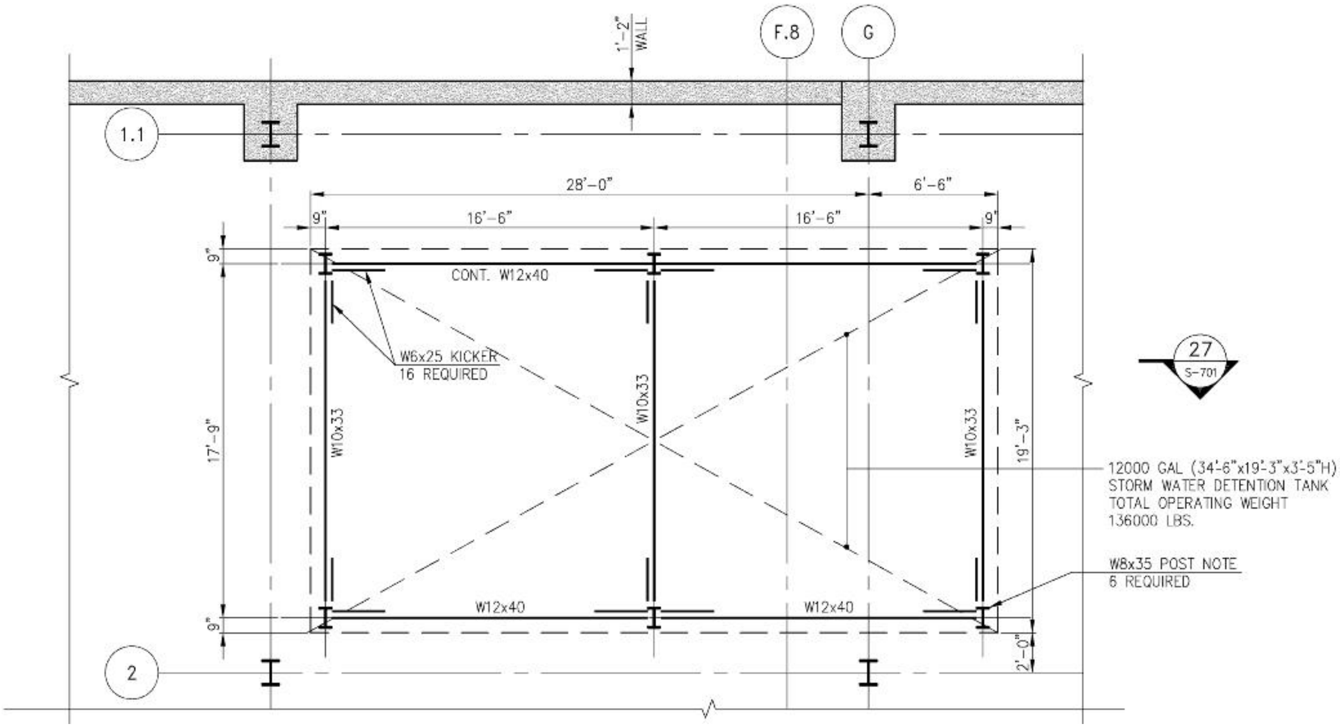


Figure 66: Watertank Dunnage Platform

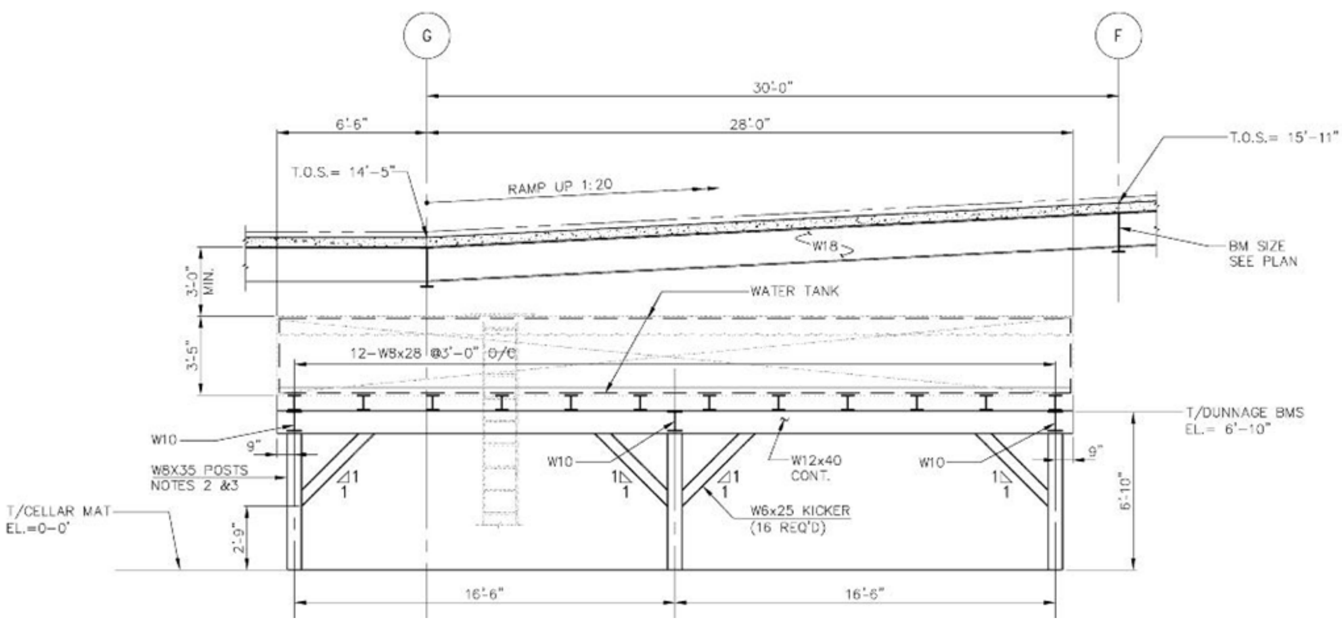


Figure 67: Section through Dunnage Platform Supporting Water Tank

12000 gallon Tank

ORIGINAL ROOF ON 1ST FLOOR : 748 gal
ORIGINAL ROOF ON 2ND FLOOR : 11075.5 gal
+
11,824 gal

11824 < 12000 ∴ Stormwater Tank is sufficient for the original green roof design

NOTE : FOR CALCULATIONS BELOW REFER TO AISC MANUAL 13TH ED.

OPERATING WT. OF WATER TANK :

136,000 lbs

$136000 / (34.5' \times 19.25') = \frac{136000}{664} = 204 \text{ psf}$

$\boxed{12-W8\times28 @ 3' o/c}$

$A_T = 19.25' \times 3' = 58.8 \text{ ft}^2$

$P_{TOT} = \cancel{242.5}\ 204 \text{ psf} \times 58.8 = 11934 \text{ lbs}$

$= 620 \text{ plf}$

$W_u = 1.2(620) = 744 \text{ plf} = 0.744 \text{ klf}$

$M_u = \frac{W_u l^2}{8} = \frac{(0.744)(19.25)^2}{8} = 34.4 \text{ ft}\cdot\text{k}$

$\phi M_n = 69'^k > M_u = 34.4'^k$ ∴ OK [TABLE 3-10]

15 000 gallon Tank

NEW DESIGN ON 2ND FLR : 11899 gal
NEW DESIGN ON 5TH FLR : 2123.5 gal
ORIGINAL ON 1ST FLR : 748 gal
+
14770.5 gal

14770.5 > 12000 ∴ STORMWATER TANK NEEDS TO BE DESIGN TO HOLD 15000 gallons of WATER.

TANK DIMENSIONS :

$34.5' \times 19.25' \times 3.5' = 2324 \text{ cf}$

$2324 \text{ ft}^3 = 17,385$ gallons

∴ TANK CAN PHYSICALLY HOLD THE EXTRA WATER BUT ADJUSTMENTS HAVE TO BE MADE ON THE SUPPORT PLATFORM.

ADDITIONAL WATER WT : 25,050 lbs

TOTAL WT : 161,050 lbs

$\frac{161050}{664 \text{ ft}^2} = 242.5 \text{ psf}$

$\boxed{12-W8\times28 @ 3' o/c}$

$A_T = 58.8 \text{ ft}^2$

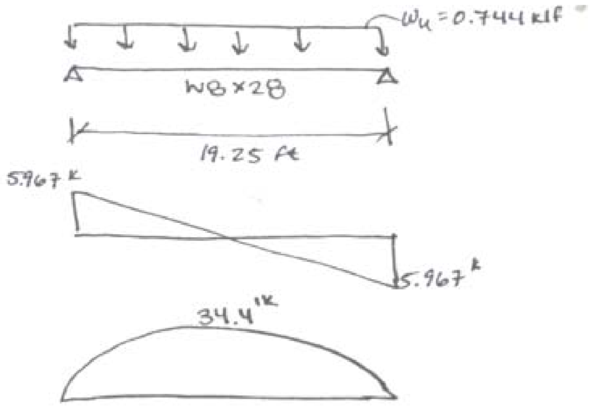
$P_{TOT} = 242.5 \text{ psf} \times 58.8 \text{ ft}^2 = 14262 \text{ lbs}$

$= 741 \text{ plf}$

$W_u = 1.2(741) = 889 \text{ plf} = 0.889 \text{ klf}$

$M_u = \frac{W_u l^2}{8} = \frac{(0.889)(19.25)^2}{8} = 41.2 \text{ ft}\cdot\text{k}$

$\phi M_n = 69'^k > M_u = 41.2'^k$ ∴ OK



$\boxed{\text{W12×40}}$

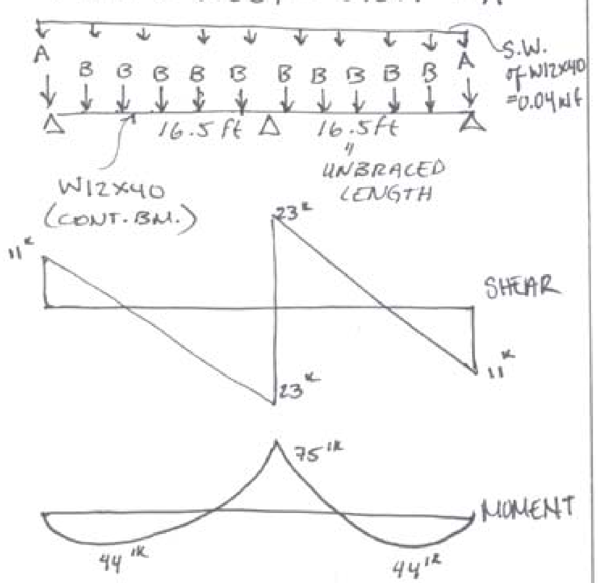
S.W. OF W8×28 : 28 lb/ft
↳ 539 lb per beam

$(5.967 \text{ kips}) \times 10 \text{ beams} = 59.67 \text{ kips}$

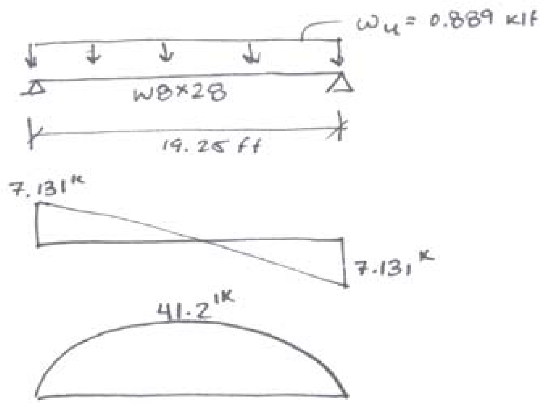
$$\frac{\left(1.5' + \frac{9''}{12''/'}\right) \times (204 \text{ psf}) \times (19.25')}{1000} = 8.9505 \text{ kips}$$

$8.9505 \div 2 = 4.475$ kips

$5.967 + 0.539 = 6.506^k = $ "B"
$4.475 + 0.539 = 5.014^k = $ "A"



$\phi M_n = 160.5'^k > M_u = 75'^k \therefore OK$



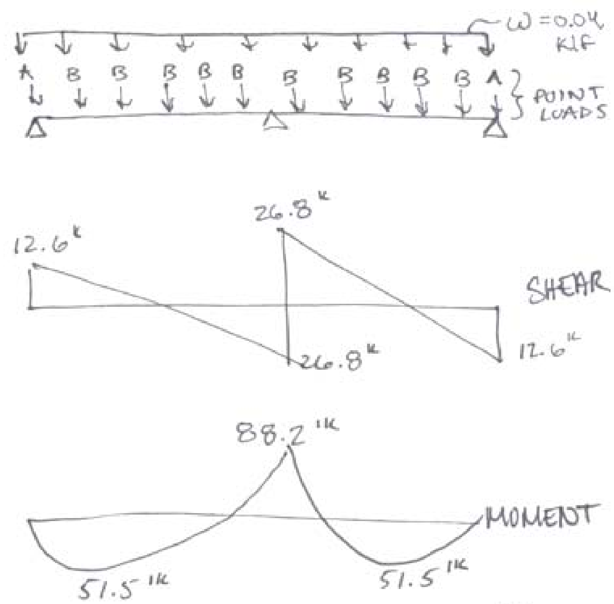
$\boxed{\text{W12×40}}$

0.539 kips per W8×28 beam
Note: there are 12 beams resting on the W12×40 girder.

$(7.131^k) + (0.539^k) = 7.670 \text{ kips} = $ "B"

$$\frac{(2.25 \text{ ft}) \times (242.5 \text{ psf}) \times (19.25 \text{ ft})}{1000} \div 2 = 5.252^k$$

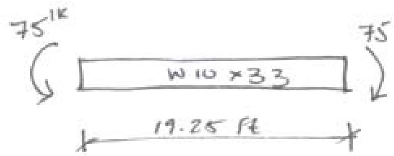
$(5.252^k) + (0.539^k) = 5.791 \text{ kips} = $ "A"



$\phi M_n = 160.5'^k > M_u = 88.2'^k \therefore OK$

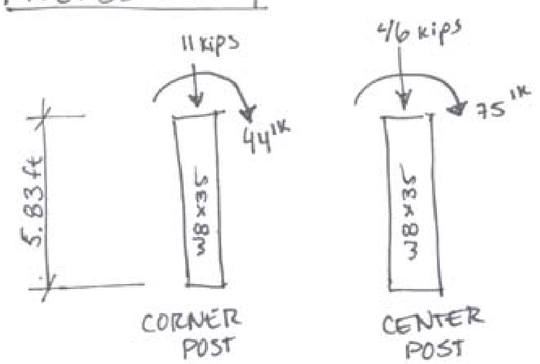
$\boxed{W10\times33}$

MOST CRITICAL:



$\phi M_n = 101'^K > M_u = 75'^K \therefore OK$

$\boxed{W8\times35 \text{ POST}}$



W8x35

$A_g = 10.3$
$I_x = 127$
$I_y = 42.6$
$r_x = 3.51$
$r_y = 2.03$

Assumptions

$KL = 6$ ft
$p \times 10^3 = 2.37$
$b \times 10^3 = 6.83$

$\frac{KL}{r_x} = 19.9$ $\quad \frac{KL}{r_y} = 34.5$ ← CONTROLS

$\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113 > 34.5$

∴ INELASTIC BEHAVIOR

$\phi F_{cr} = 41.7$ ksi

$\phi P_n = \phi F_{cr} A_g = 41.7(10.3) = 429.5^K$

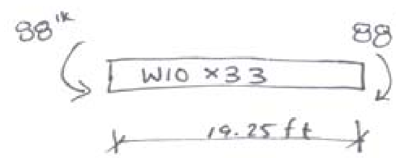
COMPARE TO TABLE 4-1: $\phi P_n = 482^K$

$P_u = 46$ kips $< \phi P_n \therefore OK$

$\phi M_n = 130'^K > 75'^K = M_u \therefore OK$

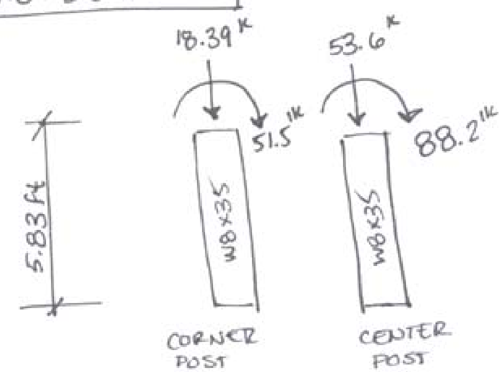
$\boxed{W10\times33}$

MOST CRITICAL:



$\phi M_n = 101'^K > M_u = 88'^K \therefore OK$

$\boxed{W8\times35 \text{ POST}}$



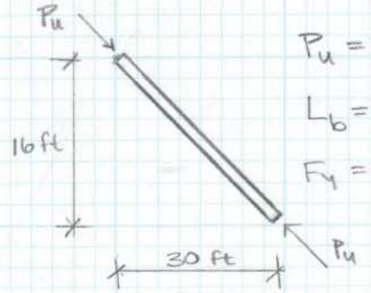
$\phi P_n = 429.5^K > P_u \therefore OK$

$\phi M_n = 130'^K > 88.2'^K \therefore OK$

Note: THE ALLOWABLE AXIAL LOAD IS MUCH HIGHER THE ACTUAL LOAD B/C THE POST MAY HAVE WITHSTAND THE INDUCED MOMENTS CAUSED BY THE LOAD ON THE GIRDERS (W12×40).

Braced Frame Diagonal Members Spot Checks

BRACE MEMBER AT LEVEL 1 — HSS 12 x 10 x 1/2



$P_u = 133K$ $(1.2D + 1.6W + 1.0L + 0.5L_r)$

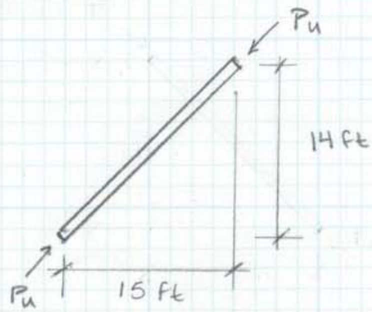
$L_b = \sqrt{16^2 + 30^2} = 34$ ft

$F_y = 42$ ksi

FROM TABLE 4-3 : $\phi P_n = 386$ kips $> P_u$ ∴ OK

$\frac{P_u}{\phi P_n} = \frac{133}{386} = 0.34 < 1.0$ ∴ OK

BRACE MEMBER AT LEVEL 5 — HSS 8 x 8 x 3/8



$P_u = 83.8K$ $(1.2D + 1.6W + 1.0L + 0.5L_r)$

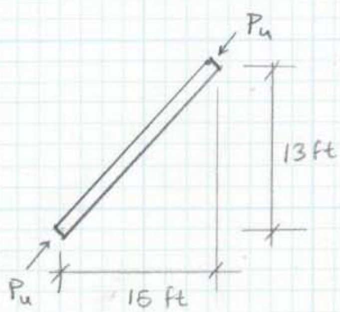
$L_b = 21$ ft

$F_y = 42$ ksi

FROM TABLE 4-4 : $\phi P_n = 275$ kips $> P_u$ ∴ OK

$\frac{P_u}{\phi P_n} = \frac{83.8}{275} = 0.30 < 1.0$ ∴ OK

BRACE MEMBER AT LEVEL 7 — HSS 10 x 8 x 3/8



$P_u = 85.7K$ $(1.2D + 1.6W + 1.0L + 0.5L_r)$

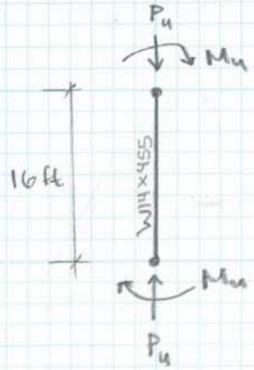
$L_b = 20$ ft

$F_y = 42$ ksi

FROM TABLE 4-3 : $\phi P_n = 333K > P_u$ ∴ OK

$\frac{P_u}{\phi P_n} = \frac{85.7}{333} = 0.26 < 1.0$ ∴ OK

COLUMN MEMBER AT LEVEL 1 W14X455



$P_u = 1193$ Kips

$M_{uy} = 22$ ft-kips $(1.2D + 1.6W + 1.0L + 0.5L_r)$

FROM TABLE 6-1

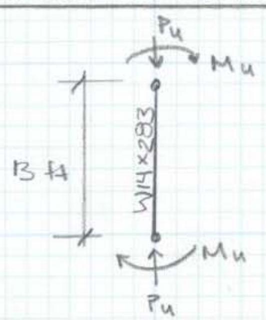
$KL = (16)\frac{r_x}{r_y} = (16)(1.67) = 26.72 \text{ ft}$

$p \times 10^{-3} = 0.247$

$b_x \times 10^{-3} = 0.261$

$$\frac{1193(0.247)}{1000} + \frac{22(0.261)}{1000} = 0.30 < 1.0 \therefore OK$$

COLUMN MEMBER AT LEVEL 7 W14X283



$P_u = 546$ kips

$M_{uy} = 70$ ft-kips

FROM TABLE 6-1

$KL = (13)(1.63) = 21.19 \text{ ft}$

$p \times 10^{-3} = 0.349$

$b_x \times 10^{-3} = 0.449$

$$\frac{(546)(0.349)}{1000} + \frac{(70)(0.449)}{1000} = 0.22 < 1.0 \therefore OK$$

Building Loads

Wind Loading

Distribution of Windward and Leeward Pressures

Level	Height Above Ground (ft)	q (psf)	Wind Pressures (psf)					
			N-S windward	N-S leeward	N-S side wall	E-W windward	E-W leeward	E-W side wall
Penthouse	134	20.75	23.10	-7.29	-20.18	23.36	-9.36	-20.41
T.O. Parapet	120	20.16	22.55	-7.29	-20.18	22.81	-9.36	-20.41
Roof	118	20.16	22.55	-7.29	-20.18	22.81	-9.36	-20.41
8	104	19.39	21.82	-7.29	-20.18	22.07	-9.36	-20.41
7	91	18.61	21.09	-7.29	-20.18	21.33	-9.36	-20.41
6	78	17.84	20.37	-7.29	-20.18	20.60	-9.36	-20.41
5	64	16.87	19.46	-7.29	-20.18	19.67	-9.36	-20.41
4	50	15.70	18.37	-7.29	-20.18	18.57	-9.36	-20.41
3	36	14.35	17.09	-7.29	-20.18	17.28	-9.36	-20.41
2	19	11.83	14.73	-7.29	-20.18	14.88	-9.36	-20.41
Ground	0	11.05	14.00	-7.29	-20.18	14.14	-9.36	-20.41

Appendix Figure 1: Calculated Wind Pressures in North/South Direction

Design Category	III
V (mph)	90
K_d	0.85
Importance Factor (I)	1.1
Exposure Category	B (urban areas)
K_{zt}=	1
n_1=	0.75
Gf	1.173 (N-S)
	1.189 (E-W)
q_p	20.16
GC_{pn}	+1.5 windward
	-1.0 leeward
P_p	21.56 windward
	19.16 leeward
GC_{pi}	n/a
z_g=	1200 ft
α=	7

Cp Value	N-S	E-W
Windward wall	0.8	0.8
Leeward Wall	-0.155	-0.239
Side Wall	-0.7	-0.7

Appendix Figure 2: Coefficients used to calculate Wind Loading and Gust Effect Factor Respectively

Gust Effect Factors		
	N-S	E-W
B (ft)	260	80.5
L (ft)	80.5	260
h (ft)	134	134
n_1	0.75	0.75
Structure:	Flexible	Flexible
g_q	3.4	3.4
g_v	3.4	3.4
g_r	4.12	4.12
z bar	80.4	80.4
ε bar	0.33	0.33
L bar	320	320
b bar	0.45	0.45
α bar	0.25	0.25
Iz bar	0.259	0.259
Lz bar	430.6	430.6
Q	0.792	0.843
Vz bar	74.21	74.21
N_1	4.352	4.352
n_h	6.23	6.23
n_b	12.087	3.742
n_l	12.529	40.466
R_h	0.148	0.148
R_b	0.079	0.232
R_L	0.077	0.024
R_n	0.055	0.055
R	0.06	0.101
G_f	1.173	1.189

	Level	Height Above Ground (ft)	Floor Height (ft)	K_z	q_z
windward	Penthouse	134	14	1.07	20.75
	T.O. Parapet	120	0.25	1.04	20.16
	Roof	118	1.7	1.04	20.16
	8	104	14	1	19.39
	7	91	13	0.96	18.61
	6	78	13	0.92	17.84
	5	64	14	0.87	16.87
	4	50	14	0.81	15.70
	3	36	14	0.74	14.35
	2	19	17	0.61	11.83
	Ground	0	19	0.57	11.05
Leeward	All	All	All	1.04	20.16

Appendix Figure 3:Kz and qz Factors

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
					N-S	E-W	N-S	E-W	N-S	E-W
Pent house	134	14	14	0.125	71	21	71	21	9580	2783
T.O. Parapet	120	0.25	0.125	0.9	5	1	77	22	605	176
Roof	118	1.7	0.9	7.0	39	11	115	33	4557	1324
8	104	14	7	6.5	64	19	179	52	6641	1930
7	91	13	6.5	6.5	59	17	238	69	5372	1561
6	78	13	6.5	7	59	17	297	86	4583	1331
5	64	14	7	7	58	17	354	103	3687	1071
4	50	14	7	7	54	16	408	119	2682	779
3	36	14	7	8.5	54	16	462	134	1953	568
2	19	17	8.5	9.5	52	15	514	149	987	287
Ground	0	19	9.5	7	44	13	559	162	0	0

Appendix Figure 4: Wind Story Forces, Shears, and Moments

FIND VELOCITY PRESSURES, q_z AND q_h:

DETERMINE BASIC WIND SPEED V FROM FIG. 6-1

$V = 90$ mph

DETERMINE WIND DIRECTIONALITY FACTOR K_d FROM TABLE 6-4 (ASCE 7-05)

$K_d = 0.85$

DETERMINE IMPORTANCE FACTOR I FROM TABLE 6-1 (ASCE 7-05)

CATEGORY III, $I = 1.1$

DETERMINE EXPOSURE CATEGORY FROM § 6.5.6 (ASCE 7-05)

CATEGORY B, URBAN AREA

ARE ALL 5 CONDITIONS OF §6.5.7.1 MET? NO

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

DETERMINE VELOCITY PRESSURE EXPOSURE COEFFICIENTS K_z AND K_h FROM TABLE 6-3 (ASCE 7-05)

$z_g = 1200$ ft
$\alpha = 7.0$
$z = 148$ ft ← NOTE: THIS IS THE MOST CRITICAL BUILDING HT.
EXPOSURE B, CASE 2

* REFER TO WIND ANALYSIS SPREADSHEET
$K_z = 1.07$ @ 134' (TOP OF PENTHOUSE)

DETERMINE VELOCITY PRESSURE AT HEIGHT z AND h

SAMPLE CALCULATION AT HT. = 134 ft (TOP OF PENTHOUSE)

$$K_z = 2.01\left(134/1200\right)^{(2/7)} = 1.07$$

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

$$= 0.00256(1.07)(1.0)(0.85)(90^2)(1.1)$$

$$= 20.75$$

GUST EFFECT FACTORS, G & G_f:

DETERMINE B, L, and H

$B\ (N-S) = 260\text{ft}, \quad L(N-S) = 80.5\text{ ft}$

$B\ (E-W) = 80.5\text{ft}, \quad L\ (E-W) = 260\text{ ft}$

$H = 134\text{ ft}$

DETERMINE n_1 & β

$n_1 = 100/H\ (\text{ft})$ AVERAGE VALUE

$= 100/134 = 0.75\text{ Hz}$

$\beta = 1.0$ PER ISO

IS $n_1 > 1\text{ Hz}$? NO

STRUCTURE IS FLEXIBLE

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad [\text{EQ. } 6\text{-}9]$$

$$g_R = \sqrt{2\ln(3600 \times 0.75)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.75)}} = 4.120$$

$\bar{z} = 0.6h \geq z_{min}$

$z_{min} = 30\text{ ft}$ [TABLE 6-2 ASCE 7-05]

$\bar{z} = 0.6(134) = 80.4\text{ft} > 30\text{ft} \therefore$ OK

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \quad [\text{EQ. } 6\text{-}5]$$

$c = 0.30$ [TABLE 6-2 ASCE 7-05]

$$I_{\bar{z}} = 0.30\left(\frac{33}{80.4}\right)^{1/6} = 0.259$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$\ell = 320\text{ ft}, \quad \bar{\epsilon} = 1/3.0$ [TABLE 6-2 ASCE 7-05]

$$L_{\bar{z}} = 320\left(\frac{80.4}{33}\right)^{(1/3)} = 430.6\text{ ft}$$

GUST EFFECT FACTORS, G & G_f CONTINUED:

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad \text{[EQ. 6-6]}$$

$$Q_{(N-S)} = \sqrt{\frac{1}{1+0.63\left(\frac{260+134}{430.6}\right)^{0.63}}} = 0.792$$

$$Q_{(E-W)} = \sqrt{\frac{1}{1+0.63\left(\frac{80.5+134}{430.6}\right)^{0.63}}} = 0.843$$

DETERMINE BASIC WIND SPEED: $V = 90$ mph [FIG. 6-1 ASCE 7-05]

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad \text{[EQ. 6-14]}$$

$$\bar{b} = 0.45, \quad \bar{\alpha} = 1/4.0$$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{80.4}{33}\right)^{(1/4)} 90\left(\frac{88}{60}\right) = 74.21$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.75)(430.6)}{74.21} = 4.352 \quad \text{[EQ. 6-12]}$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(4.352)}{(1+10.3(4.352))^{5/3}} = 0.055 \quad \text{[EQ. 6-11]}$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{for } \eta > 0$$

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.75)(134)}{74.21} = 6.230$$

$$R_h = \frac{1}{6.230} - \frac{1}{2(6.230)^2}\left(1-e^{-2(6.230)}\right) = 0.148$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{for } \eta > 0$$

$$\eta = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.6(0.75)(260)/74.21 = 12.087 \quad (N-S)$$

$$= 4.6(0.75)(80.5)/74.21 = 3.742 \quad (E-W)$$

$$R_B(N-S) = \frac{1}{12.087} - \frac{1}{2(12.087)^2}\left(1-e^{-(2\times 12.087)}\right) = 0.079 \quad (N-S)$$

$$R_B(E-W) = \frac{1}{3.742} - \frac{1}{2(3.742)^2}\left(1-e^{-(2\times 3.742)}\right) = 0.232 \quad (E-W)$$

GUST EFFECT FACTORS, G & GF CONTINUED:

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \text{ for } \eta > 0$$

$$\eta = 15.4\, n_1 L / \bar{V}_{\bar{z}} = 15.4(0.75)(80.5)/74.21 = 12.529 \quad (N-S)$$

$$= 15.4(0.75)(260)/74.21 = 40.466 \quad (E-W)$$

$$R_L\,(N-S) = \frac{1}{12.529} - \frac{1}{2(12.529)^2}\left(1 - e^{-(2\times 12.529)}\right) = 0.077 \quad (N-S)$$

$$R_L\,(E-W) = \frac{1}{40.466} - \frac{1}{2(40.466)^2}\left(1 - e^{-(2\times 40.466)}\right) = 0.024 \quad (E-W)$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad \text{[EQ. 6-10]}$$

$$= \sqrt{\frac{1}{1.0}(0.55)(0.148)(0.079)(0.53 + 0.47(0.077))} = 0.060 \quad (N-S)$$

$$= \sqrt{\frac{1}{1.0}(0.55)(0.148)(0.232)(0.53 + 0.47(0.024))} = 0.101 \quad (E-W)$$

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] \quad \text{[EQ. 6-8]}$$

$$= 0.925\left[\frac{1 + 1.7(0.259)\sqrt{(3.4)^2(0.792)^2 + (4.120)^2(0.060)^2}}{1 + 1.7(3.4)(0.259)}\right] = 1.173 \quad (N-S)$$

$$= 0.925\left[\frac{1 + 1.7(0.259)\sqrt{(3.4)^2(0.843)^2 + (4.120)^2(0.101)^2}}{1 + 1.7(3.4)(0.259)}\right] = 1.189 \quad (E-W)$$

BUILDINGS, MAIN WIND-FORCE RESISTING SYSTEMS

IS THE BUILDING ENCLOSED OR PARTIALLY ENCLOSED? YES

DOES THE BUILDING HAVE A PARAPET? YES

VELOCITY PRESSURE $q_P = 20.16$ mph

DETERMINE COMBINED NET PRESSURE COEFFICIENT GC_{pn}

$GC_{pn} = +1.5$ WINDWARD

$GC_{pn} = -1.0$ LEEWARD

DETERMINE COMBINED NET DESIGN PRESSURE ON THE PARAPET

$P_P = q_P GC_{pn}$ [EQ. 6-20]

$= (20.16) + 1.5 = 21.56$ (WINDWARD)

$= (20.16) - 1.0 = 19.16$ (LEEWARD)

IS THE BUILDING A LOW-RISE BUILDING AS DEFINED IN 6.2? NO

IS THE BUILDING RIGID? NO

DETERMINE VELOCITY PRESSURE q_z FOR WINDWARD WALLS ALONG THE HT. OF THE BUILDING AND q_h FOR LEEWARD WALLS, SIDE WALLS, & ROOF
(SEE SPREADSHEETS)

DETERMINE PRESSURE COEFFICIENTS C_p FOR THE ROOF FROM FIG 6-6

$\frac{L}{B} = \frac{80.5}{260} = 0.310$ (N-S)

$\frac{L}{B} = \frac{260}{80.5} = 3.230$ (E-W)

	C_p VALUE	
	N-S	E-W
WINDWARD WALL	0.8	0.8
LEEWARD WALL	-0.155	-0.239
SIDE WALL	-0.7	-0.7

WINDWARD WALLS: $P_z = q_z G_f C_p$ ← SAMPLE CALCULATION (SEE SPREADSHEET)

(N-S) $P_z = (20.16)(1.173)(0.8)$

C_P VALUES		
	N-S	E-W
WINDWARD	0.8	0.8
LEEWARD	-0.155	-0.239
SIDE WALL	-0.700	-0.700

G_f (N-S) = 1.173

G_f (E-W) = 1.189

NOT INCLUDING UPLIFT ON ROOF SINCE ROOF FRAMING MADE UP OF W-SHAPES

$q_i = q_h = q_z$ FOR TOP OF BUILDING = 20.16 psf

INTERNAL PRESSURE COEFFICIENT : $GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES — $P_z + P_h$ (EQ 6-17)

WINDWARD WALLS: (psf)

$$P_z = q_z GC_p - q_h (GC_{pi})$$
$$= (1.173)(0.8) q_z \pm 20.16 (0.18)$$
$$= 0.9384 q_z \pm 3.6288 \quad [N-S]$$

$$P_z = (1.189)(0.8) q_z \pm 20.16 (0.18)$$
$$= 0.9512 q_z \pm 3.6288 \quad [E-W]$$

LEEWARD WALLS & SIDE WALLS: (psf)

$$P_z = q_h GC_p - q_h (GC_{pi})$$
$$= (20.16)(1.173) C_p \pm 20.16 (0.18)$$
$$= 23.6477 C_p \pm 3.6288 \quad [N-S]$$

$$P_z = (20.16)(1.189) C_p \pm 20.16 (0.18)$$
$$= 23.9702 C_p \pm 3.6288 \quad [E-W]$$

Seismic Loads

Seismic Analysis Coefficients	
Ss=	0.37
S1=	0.07
Occupancy Category=	III
Site Class=	C (very dense soil and soft rock)
Fa=	1.2
Fv=	1.7
Sms=	0.45
Sm1=	0.119
Sds=	0.3
Sd1=	0.079
Ta=	1.182
0.8Ts=	0.211
SDC=	B
Ts=	0.226
R=	7
I=	1.1
Ta=	1.182
Cu=	0.211
TL=	6 sec
Cs=	0.006
Cs=	0.01
k=	1.755
W=	15388 kips
V=	153.88 kips

Appendix Figure 5: Coefficients used for Seismic Analysis per ASCE 7-05

Lateral Seismic Force, Fx							
Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)	Fx (kips)
penthouse	134	80750	0	38245	0	118995	6.76
roof	120	492300	3440	50726	70560	617026	28.87
8	104	403570	15938	37130	61740	518378	18.87
7	91	374170	24463	42135	57330	498098	14.34
6	78	1108370	24463	116396	127335	1376564	30.24
5	64	1201959	16940	169389	144690	1532978	23.80
4	50	1201959	86174	90008.7	144690	1522831.7	15.33
3	36	1201959	76816.5	140824.5	144690	1564290	8.85
2	19	3223770.5	76816.5	220889.5	178755	3700231.5	6.82
1	0	3356119.75	236557.1637	177844	168240	3938760.916	0.00

Appendix Figure 6: Equivalent Lateral Force Procedure

Base Shear and Overturning Moment Distribution							
Level	hx (ft)	Story Weight (k)	hxk Wx	Cvx	Fx=CvxV	Vx (k)	Mx (ft-k)
penthouse	134	119.0	643573	0.044	7	7	906
roof	120	617.0	2749581	0.188	29	36	4276
8	104	518.4	1796967	0.123	19	54	5668
7	91	498.1	1365943	0.093	14	69	6265
6	78	1376.6	2880199	0.197	30	99	7729
5	64	1533.0	2266636	0.155	24	123	7865
4	50	1522.8	1459971	0.100	15	138	6911
3	36	1564.3	842613	0.057	9	147	5294
2	19	3700.2	649294	0.044	7	154	2924
1	0	3938.8	0	0.000	0	154	0
Total	134	15388.2	14654776	1	154		47835
Base Shear=	154 kips						

Appendix Figure 7: Distribution of Shear and Moment on Building

SEISMIC GROUND MOTION VALUES & EQUIV. LAT. FORCE PROCEDURE

DETERMINE S_S AND S_1 FROM FIG. 22-1 THROUGH 22-14

$S_1 = 0.07$, $S_S = 0.350$

IS $S_S \leq 0.15$ & $S_1 \leq 0.04$? NO

IS THE STRUCTURE SEISMICALLY ISOLATED OR DOES IT HAVE DAMPING SYSTEMS ON SITE W/ $S_1 \geq 0.6$? NO

DETERMINE THE SITE CLASS IN ACCORDANCE W § 11.4.2 & Ch. 20

↳ SITE CLASS "C"

DETERMINE S_{MS} & S_{M1} by EQN 11.4-1 & 11.4-2

$F_a = 1.2$, $F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.370) = 0.45$

$S_{M1} = F_v S_1 = 1.7(0.07) = 0.119$

DETERMINE S_{DS} & S_{D1} BY EQN 11.4-3 & 11.4-4 RESPECTIVELY:

$S_{DS} = 2 S_{MS}/3 = 2(0.45)/3 = 0.30$

$S_{D1} = 2 S_{M1}/3 = 2(0.119)/3 = 0.079$

DETERMINE OCCUPANCY CATEGORY: III

IS $S_1 > 0.75$? NO

IS THE SIMPLIFIED DESIGN PROCEDURE OF 12.14 PERMITTED? NO

ARE ALL 4 CONDITIONS OF 11.6 SATISFIED? NO

$T_a = C_t h_n^x = 0.03(134)^{0.75} = 1.182$ (ECCEN. BRACED STEEL FRAMES)

$0.8 T_S = 0.8 \dfrac{S_{D1}}{S_{DS}} = 0.8\left(\dfrac{0.079}{0.30}\right) = 0.211$

$T_a \nless 0.8 T_S$ $\quad T = C_u T_a = 1.7(1.182) = 2.009$

DETERMINE SDC AS THE MORE SEVERE OF T. 11.6-1 & T. 11.6-2

SDC = B

DETERMINE R, RESPONSE COEFF. : 7 for truss frames

IMPORTANCE FACTOR : 1.1

DETERMINE T_L FROM FIG 22-15 THROUGH 22-20 : 6 sec.

DETERMINE C_s

$C_S = \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} \leq \dfrac{S_{DS}}{\left(\frac{R}{I}\right)}$ $\quad C_S = \dfrac{0.079}{(2.009)\left(\frac{7}{1.1}\right)} \leq \dfrac{0.30}{\left(\frac{7}{1.1}\right)} = 0.006$

IS $S_1 \geq 0.6$? NO

IS $C_s < 0.01$? YES

$C_s = 0.01$

DETERMINE EFFECTIVE SEISMIC WEIGHT W : 15,388 KIPS

DETERMINE BASE SHEAR

$$V = C_s W = 0.01(15388 \text{ kips}) = 153.88 \text{ kips}$$

IS $T \leq 0.5$ sec? NO

IS $T \geq 2.5$ sec? NO

$$k = 0.75 + 0.5T = 0.75 + 0.5(2.009) = 1.755$$

Gravity Loads

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
penthouse	134	80750	0	38245	0	118995
roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
				Total Building Weight:		15388153.12

Appendix Figure 8: Building Weight Calculations

Floor	Floor Area (sf)	Floor Dead Load (psf)	Floor Weight	Curtainwall length (ft)	Curtainwall height (ft)	Curtainwall weight (ft) (height*weight* 15 psf)
cellar level						
Ground						
loading dock	930	150	139500	701	16	168240
first floor level	14838	130	1928940			
podium	600	200	120000			
archive	900	75	67500			
Offices	1948	71	138308			
roof with garden	1330.84	365	485756.6			
library stacks	6705.847	71	476115.153			
second level						
roof with garden	4560	365	1664400	701	17	178755
classrooms	6784	71	481664			
corridors	7601.5	71	539706.5			
auditorium	2800	85	238000			
roof with pavers on 2	2000	150	300000			

Appendix Figure 9: Detailed Building Weight Calculations

Floor	Floor Area (sf)	Floor Dead Load (psf)	Floor Weight	Curtain wall length (ft)	Curtain wall height (ft)	Curtainwall weight (ft) (height*weight* 15 psf)
third level						
classrooms	11424	71	811104	689	14	144690
corridor	5505	71	390855			
fourth level						
offices	5712	71	405552	689	14	144690
classrooms	1200	71	85200			
corridors	10017	71	711207			
fifth level						
offices	7570.5	71	537505.5	689	14	144690
corridors	9358.5	71	664453.5			
sixth level						
offices	3050	71	216550	653	13	127335
corridors	2220	71	157620			
roof	4757.5	90	428175			
roof with drift	325	85	27625			
mechanical	2320	120	278400			
seventh level						
offices	2635	71	187085	294	13	57330
corridors	2635	71	187085			
eighth level						
offices	2335	71	165785	294	14	61740
corridors	2335	71	165785			
mechanical	600	120	72000			
roof level						
roof	4670	90	420300	294	16	70560
mechanical	600	120	72000			
penthouse level						
roof with drift	950	85	80750	248	0	0
		total:	12644927.3			1098030

Appendix Figure 10: Detailed Building Weight Calculations

LOCATION J3 : Accumulated Loads on Columns											
Level	tributary area	dead load (psf)	live load (psf)	influence area	LL red. Factor	live load (k)	dead load (k)	load comb.	load at floor (k)	accum. Load (k)	accum. load (k) by Turner
roof	525	90	45	2100	1.00	23.6	47.3	1.2D+0.5Lr	68.5	68.5	80
Eighth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	161.7	161
seventh	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	255.0	242
sixth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	348.2	337
fifth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	460.4	715
fourth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	572.6	852
third	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	684.8	997
second	675	85	100	3420	0.51	34.2	57.4	1.2D+1.6L	123.6	808.4	1123
Ground	675	130	100	3420	0.51	34.2	87.8	1.2D+1.6L	160.0	968.4	1349

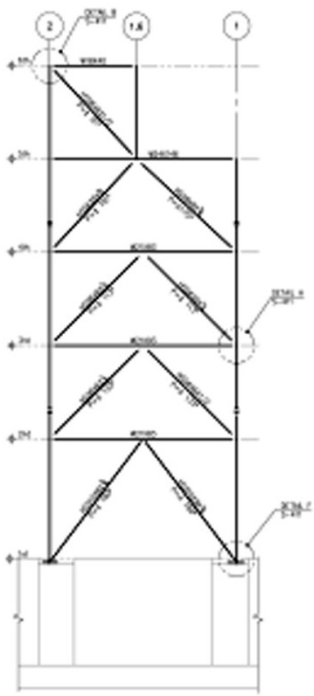
Appendix Figure 11: Accumulated Loads on Columns

At level 5 there is a large difference between the accumulated loads calculated by that which was provided by Turner Construction Company. This is due to the step- back of the floor levels above. Since the columns located at J1.6 at above levels don't continue to the fifth level, the fifth level is forced to carry the load from the J1.6 column at level 6. Below is a table depicting the adjusted accumulated loads and how they compare to values provided by Turner Construction Company.

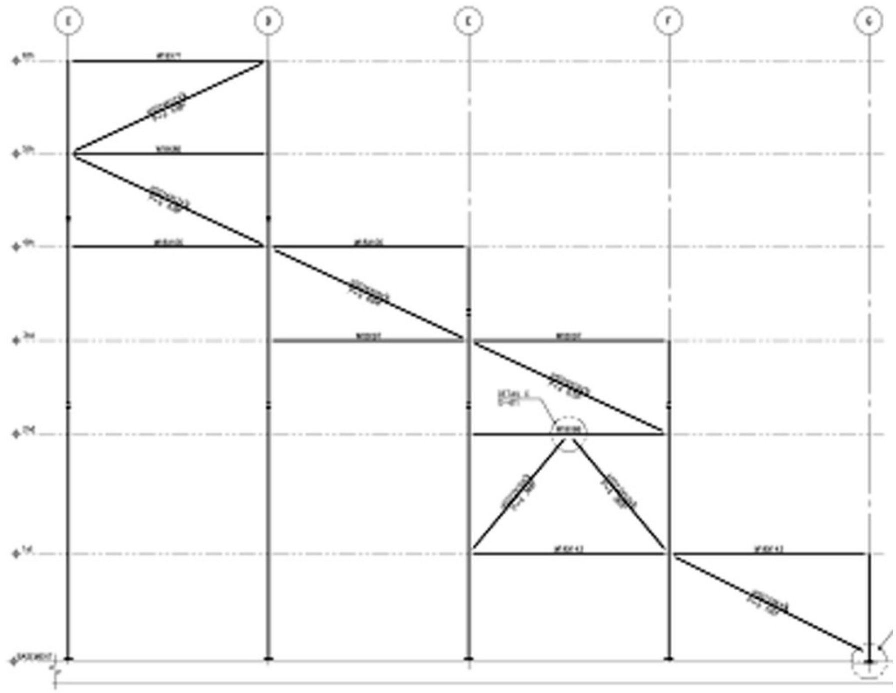
	accumulated load (k) by Turner for Loc. J1.6	**LOCATION J3 : Accumulated Loads on Columns**		
Level		Adjusted accumulated load (k)	accumulated load (k) provided by Turner	percent Error = |adj-prov| /adj*100
roof	n/a	68.5	80	17
eighth	n/a	161.7	161	0
seventh	n/a	255.0	242	5
sixth	266	348.2	337	3
fifth	n/a	726.4	715	2
fourth	n/a	838.6	852	2
third	n/a	950.8	997	5
second	n/a	1074.4	1123	5
Ground	n/a	1234.4	1349	9

Appendix Figure 12: Adjustment of Accumulated Loads on Columns

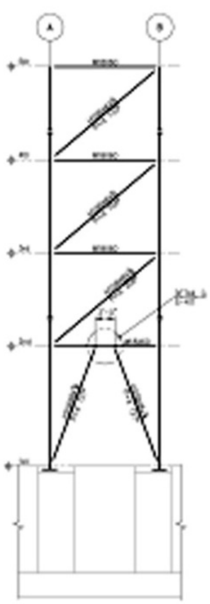
Braced Frames Elevations



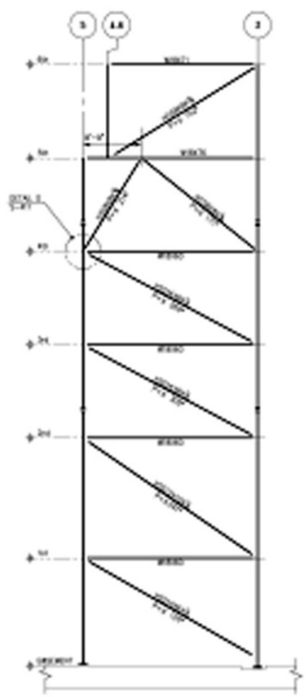
1 TRUSS @ GRID A



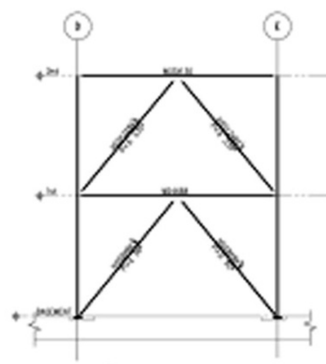
2 TRUSS @ GRID 2



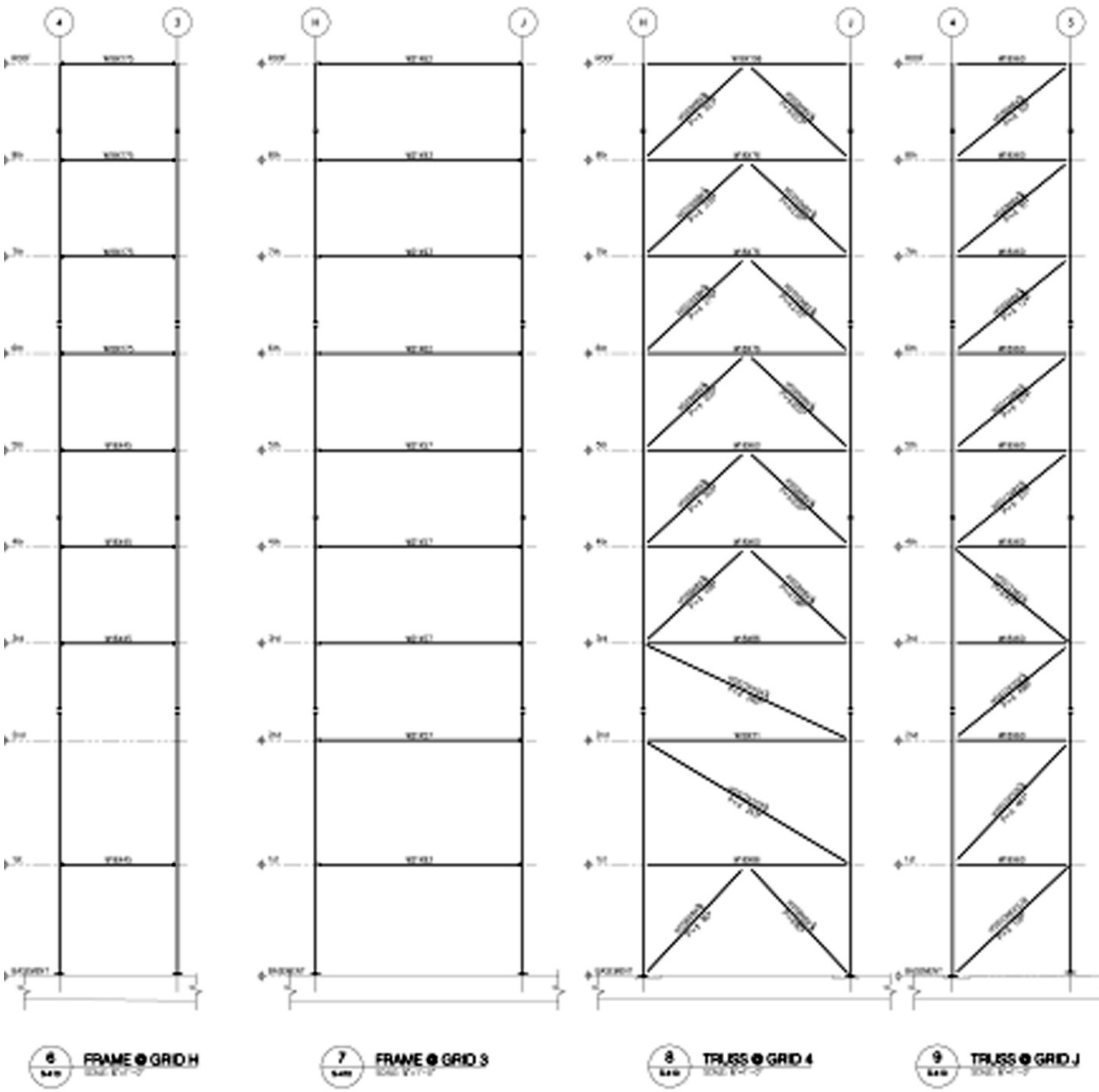
3 TRUSS @ GRID 1



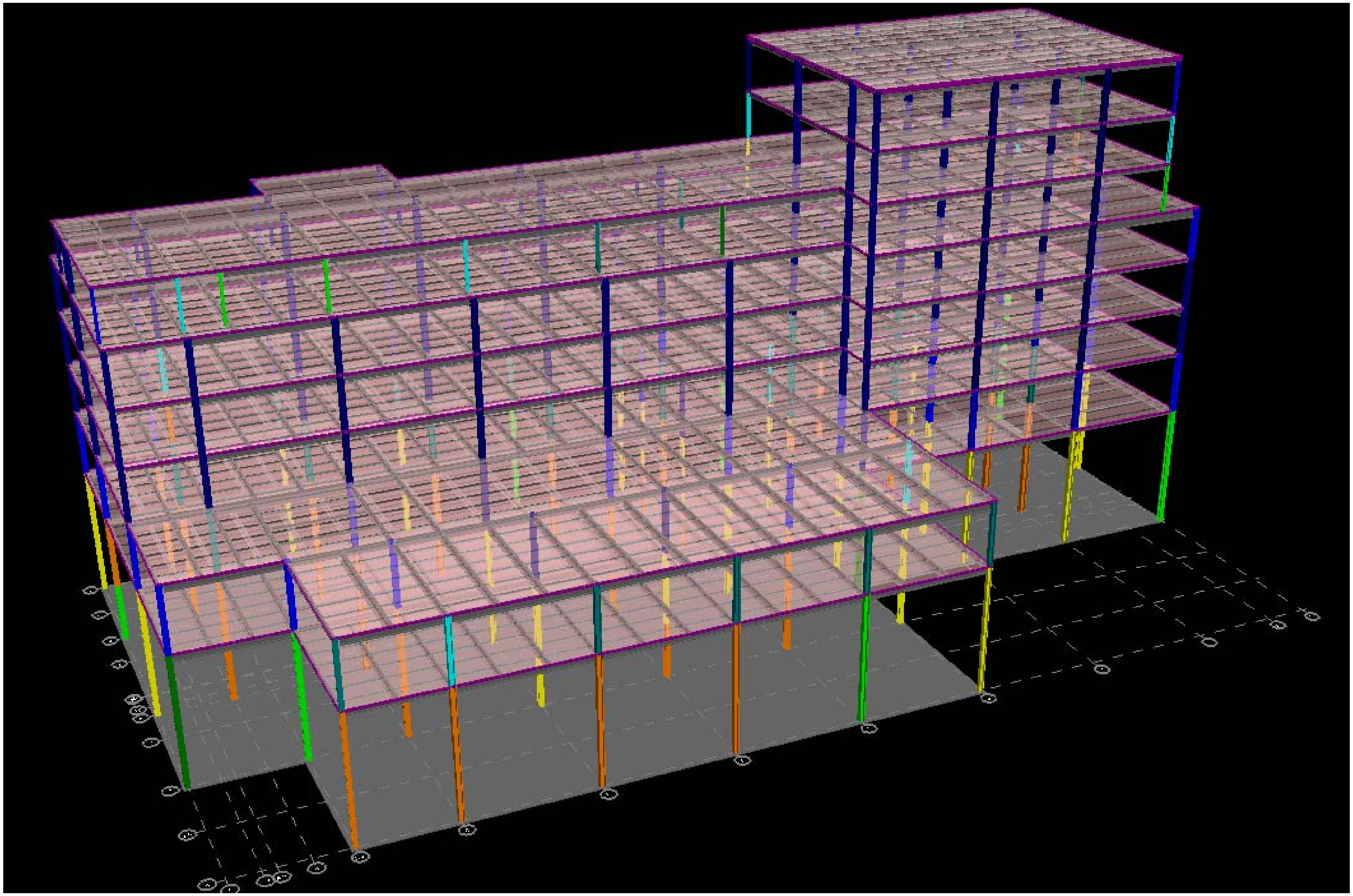
4 TRUSS @ GRID F

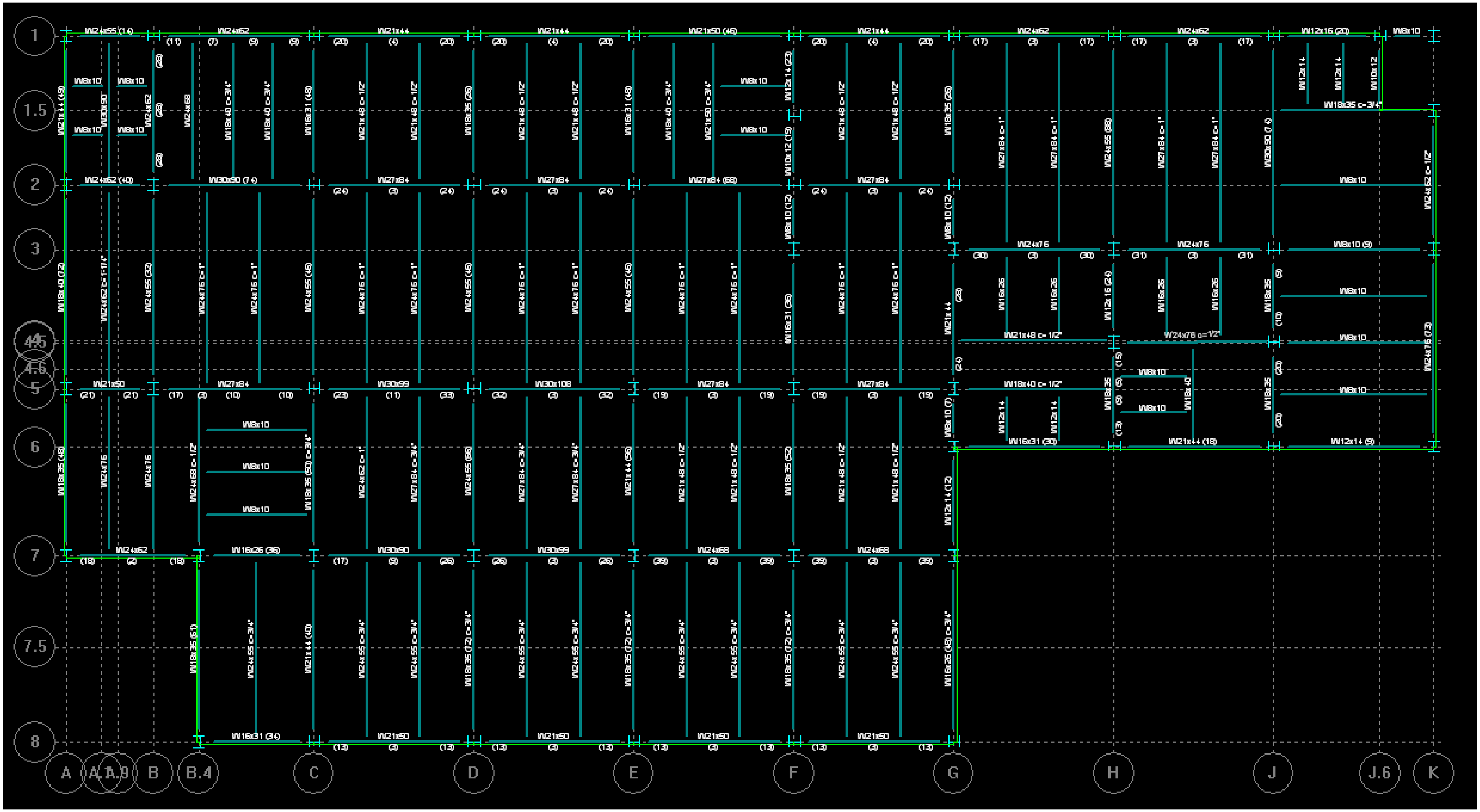


5 TRUSS @ GRID B

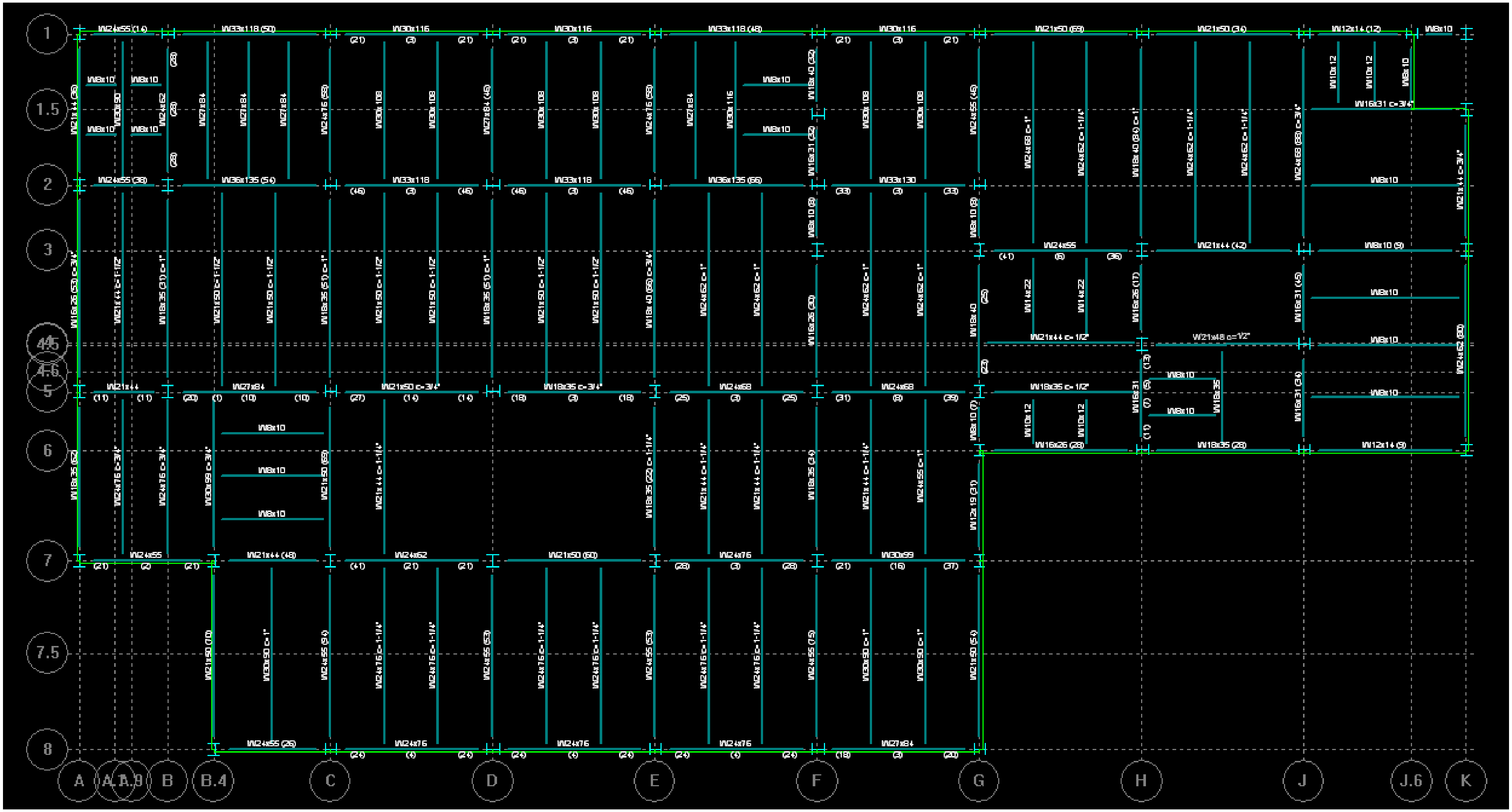

6 FRAME @ GRID H
7 FRAME @ GRID 3
8 TRUSS @ GRID 4
9 TRUSS @ GRID J

Redesigned Gravity Frame

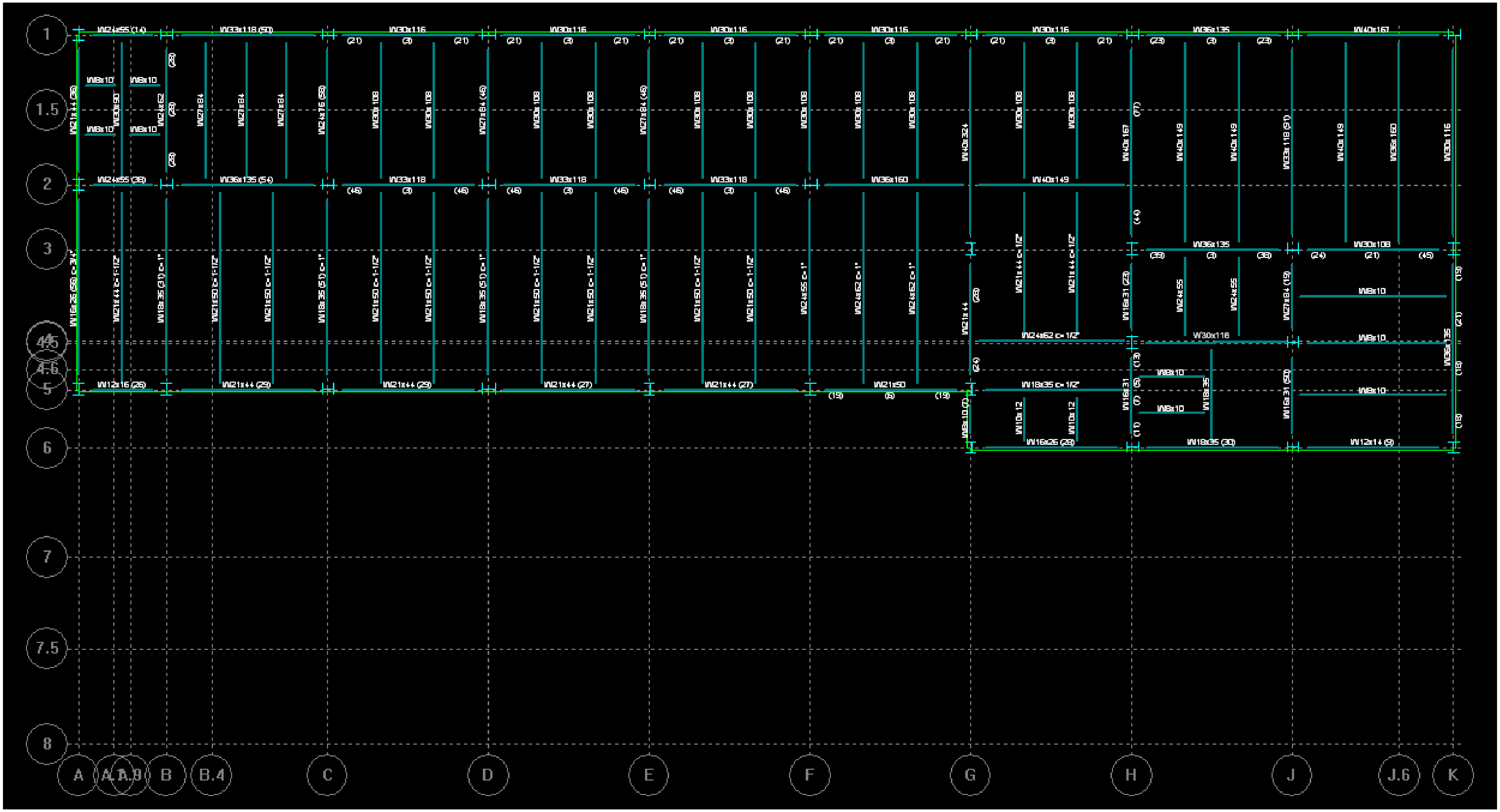




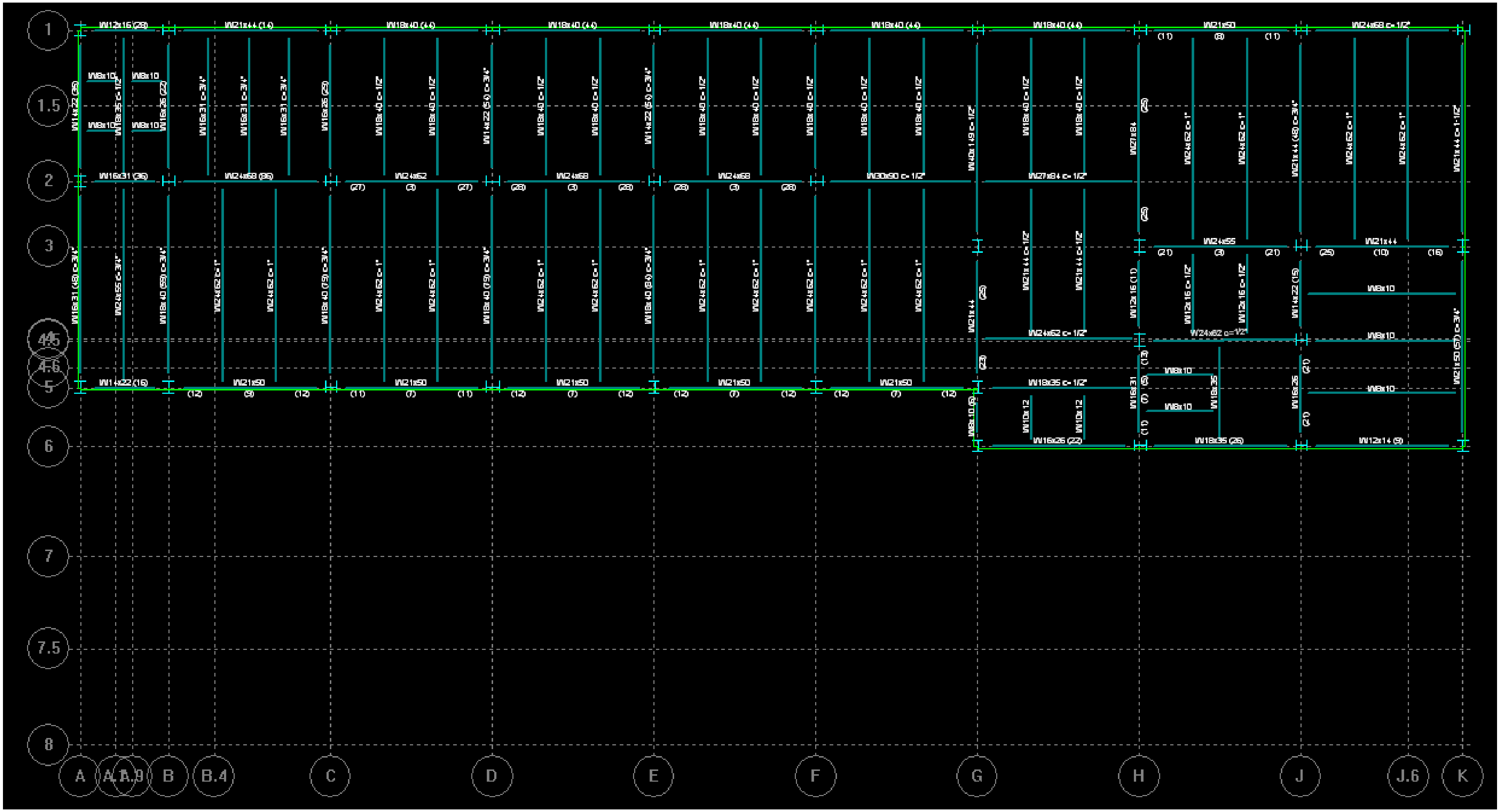
Appendix Figure 13: Ground Floor Beams



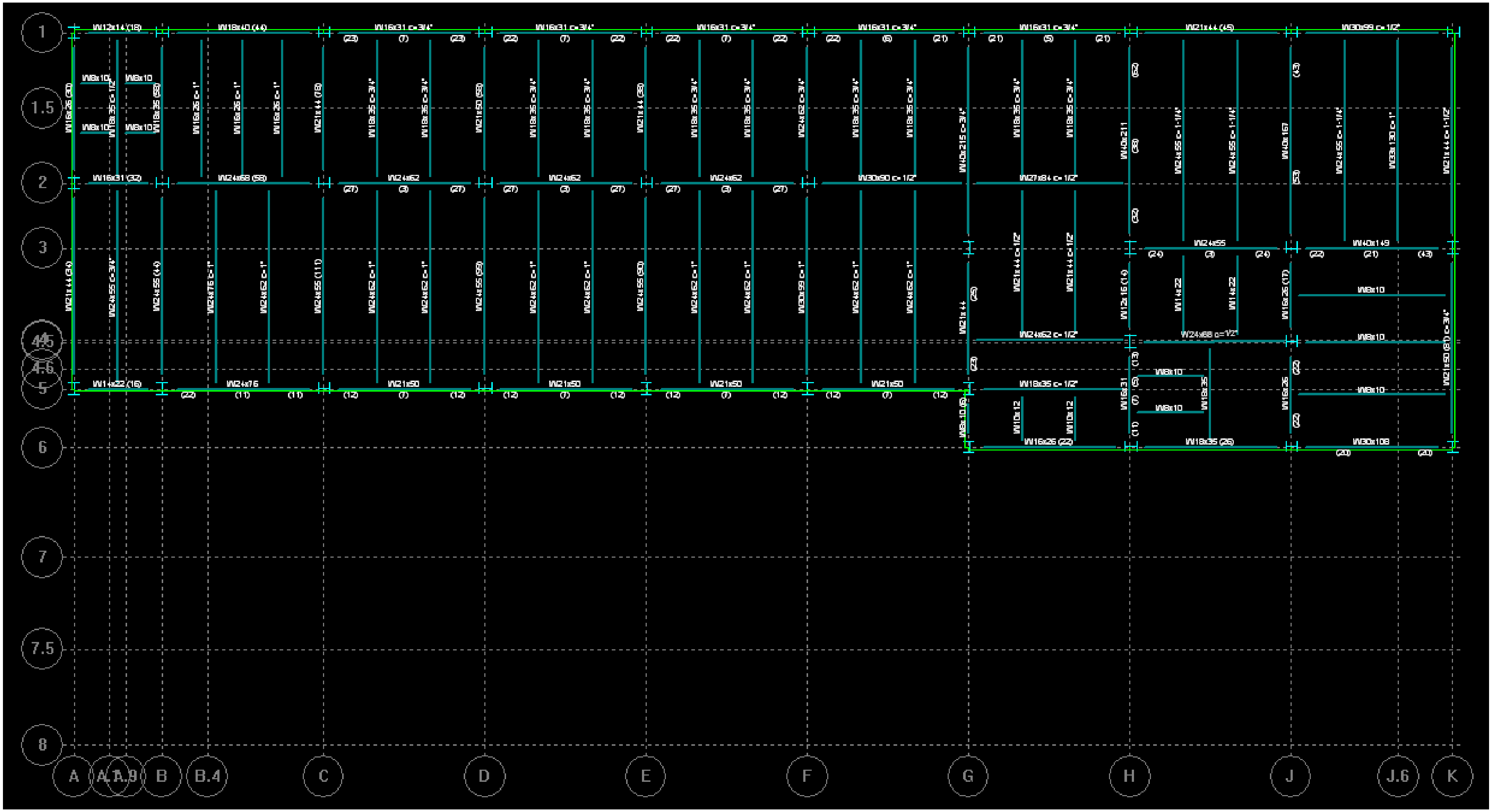
Appendix Figure 14: Second Floor Beams



Appendix Figure 15: Third Floor Beams

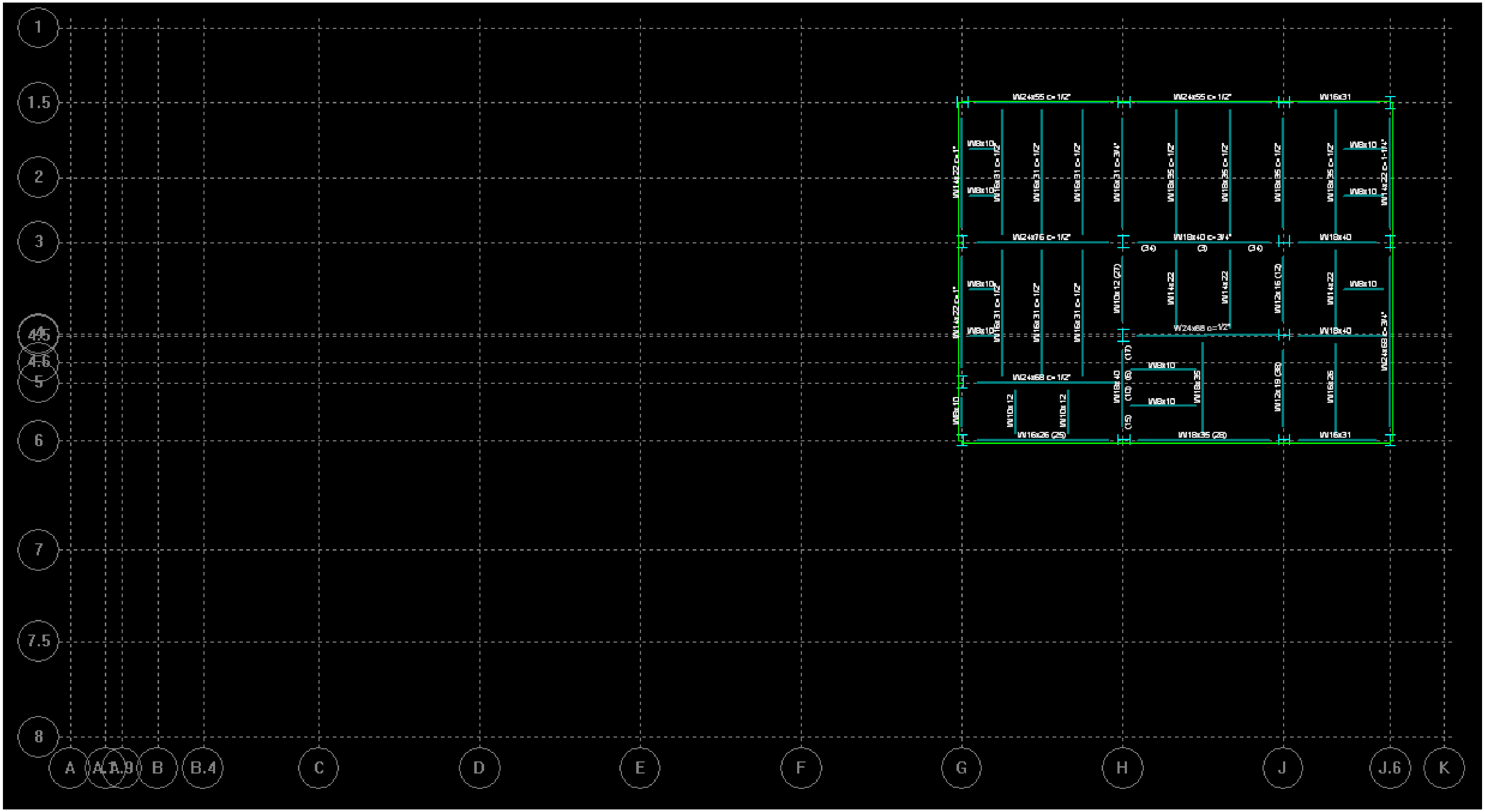


Appendix Figure 16: Fourth Floor Beams

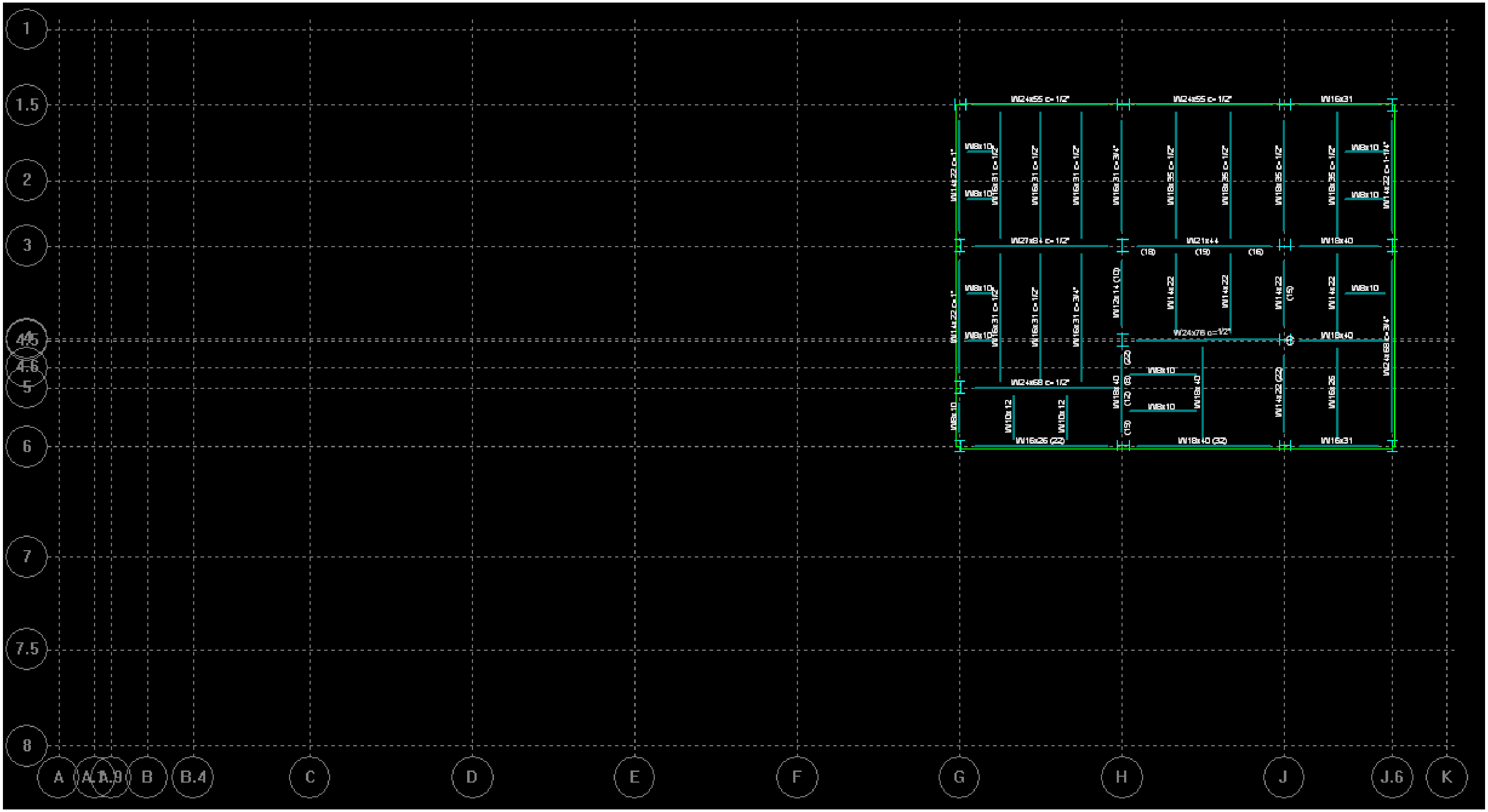


Appendix Figure 17: Fifth Floor Beams

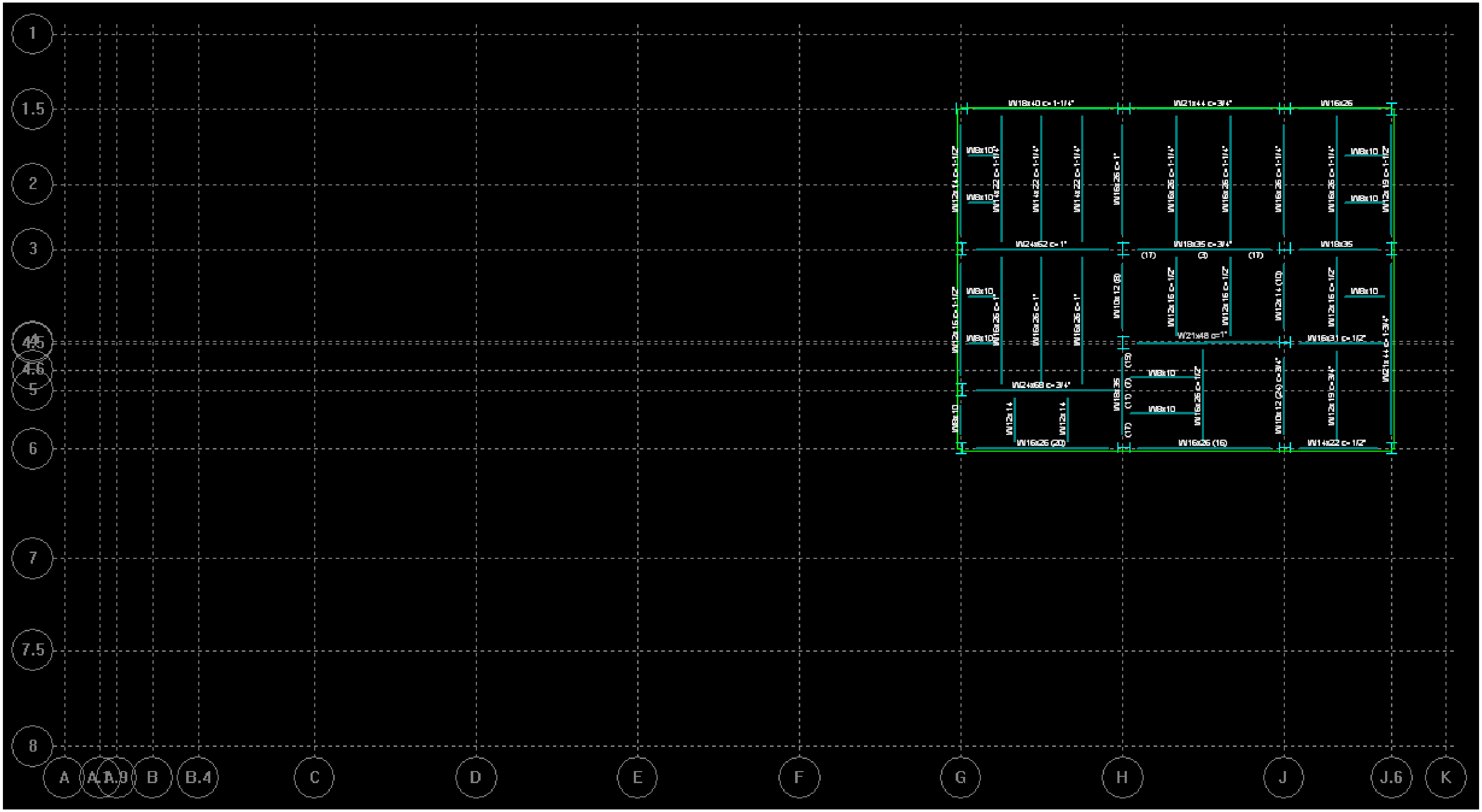
Appendix Figure 18: Sixth Floor Beams



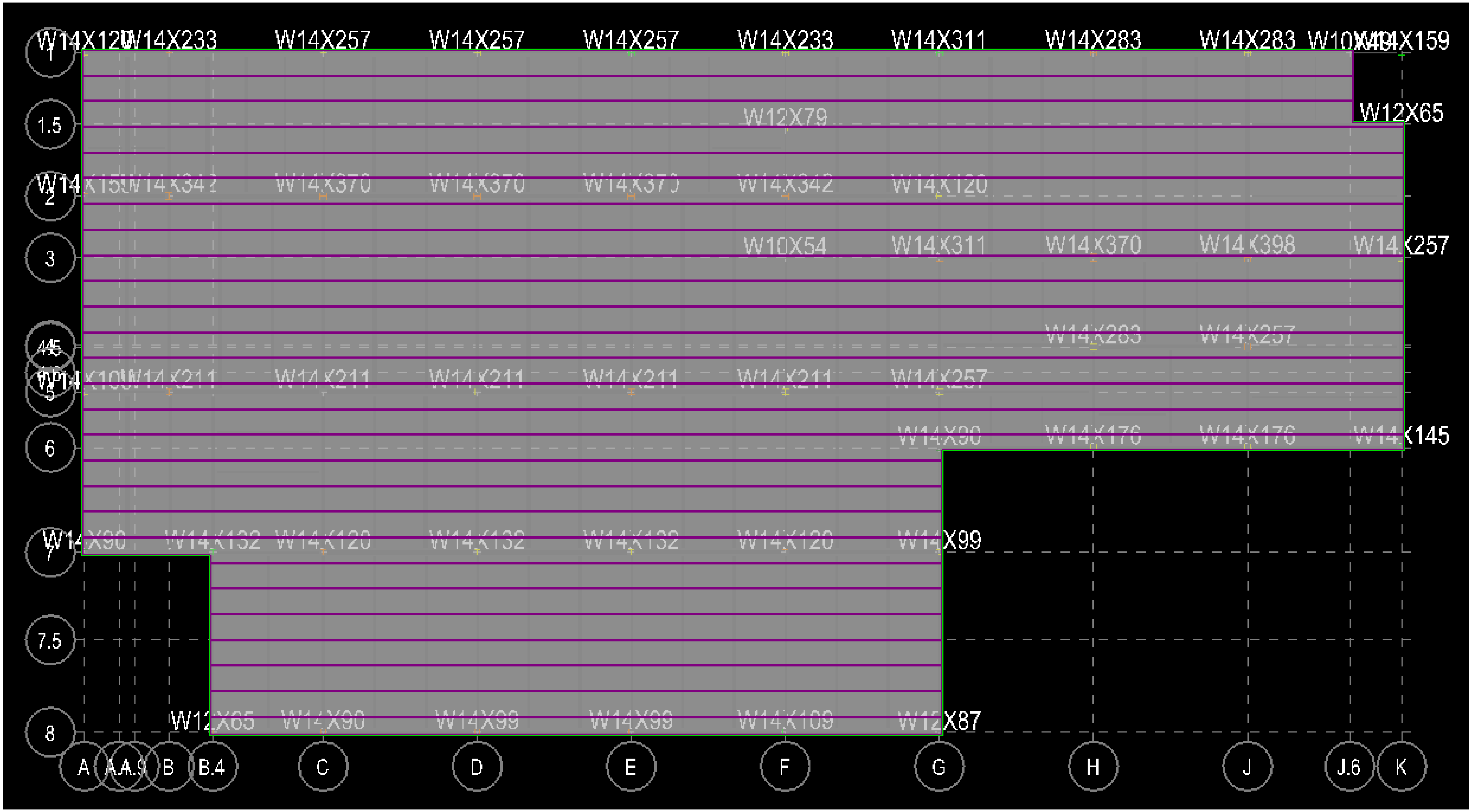
Appendix Figure 19: Seventh Floor Beams



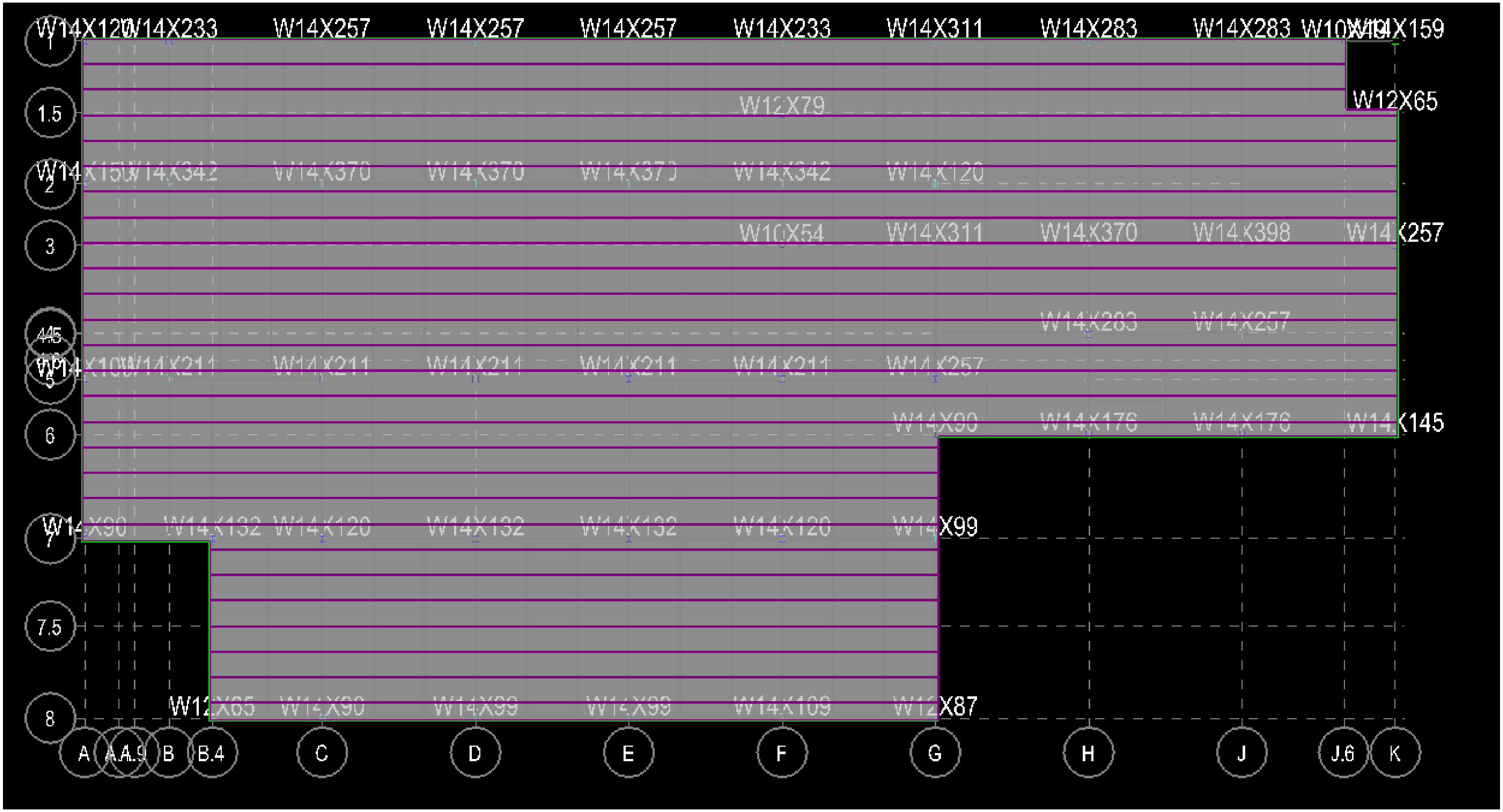
Appendix Figure 20: Eighth Floor Beams



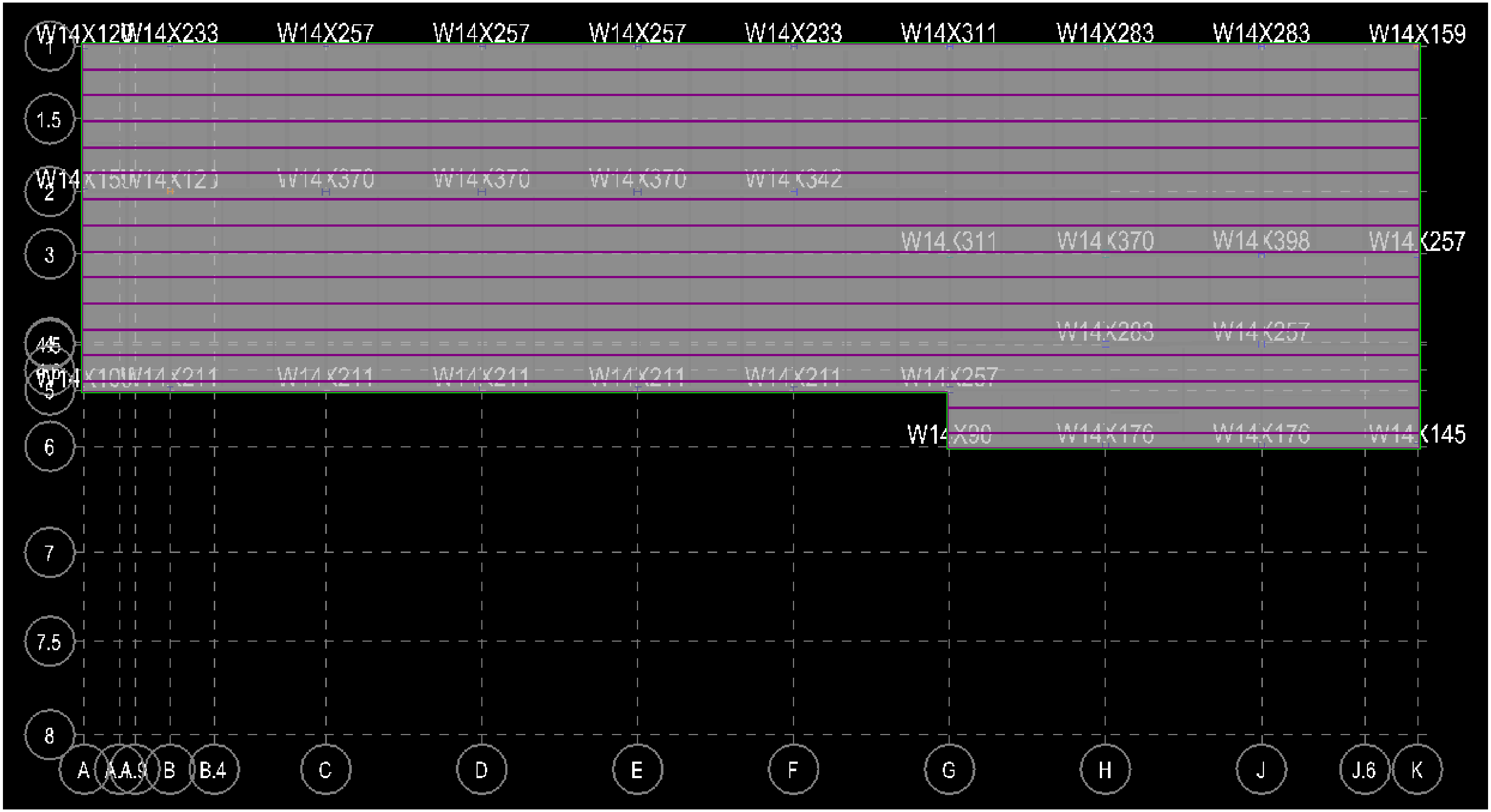
Appendix Figure 21: Roof Beams



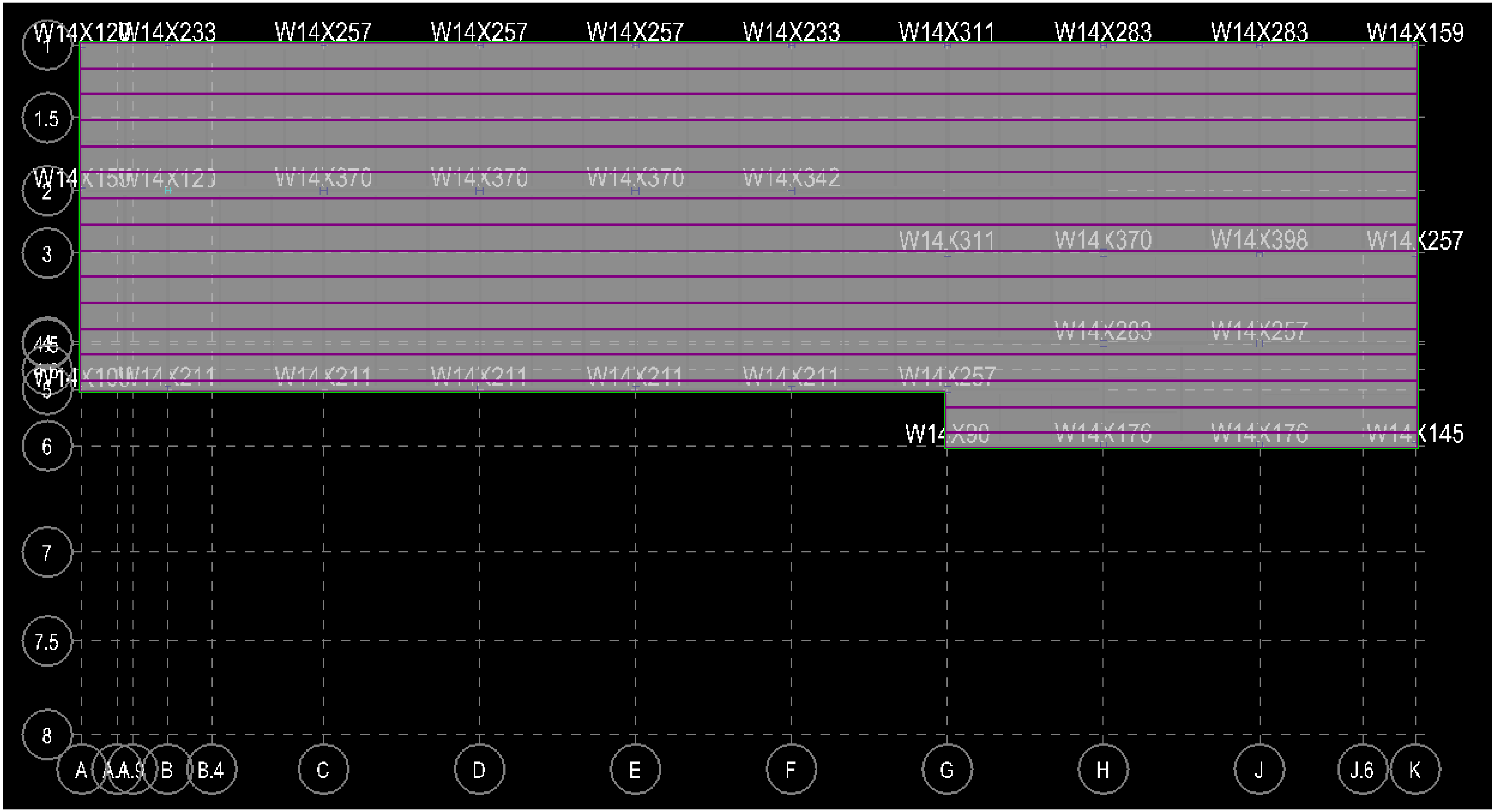
Appendix Figure 22: Ground Floor Columns



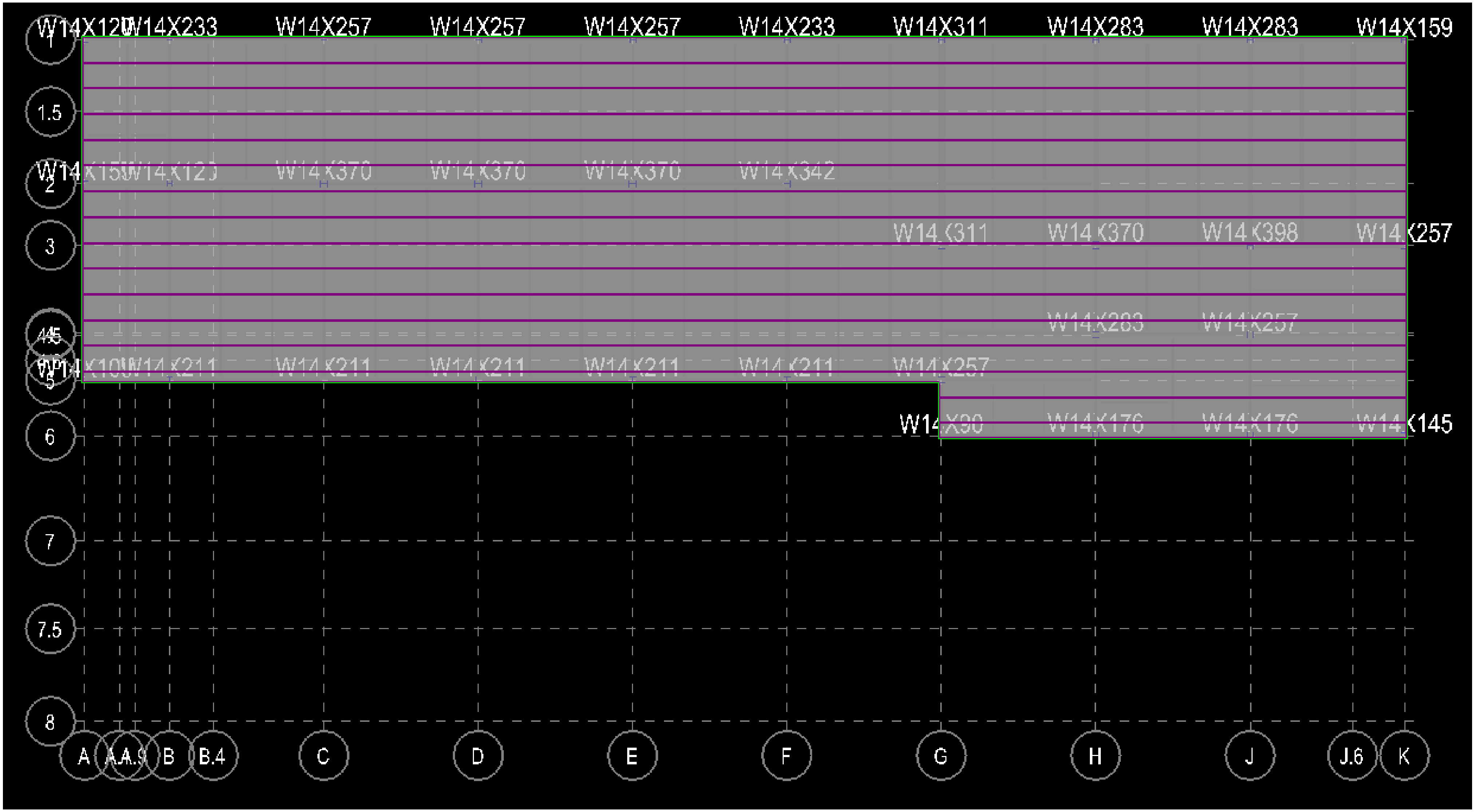
Appendix Figure 23: Second Floor Columns



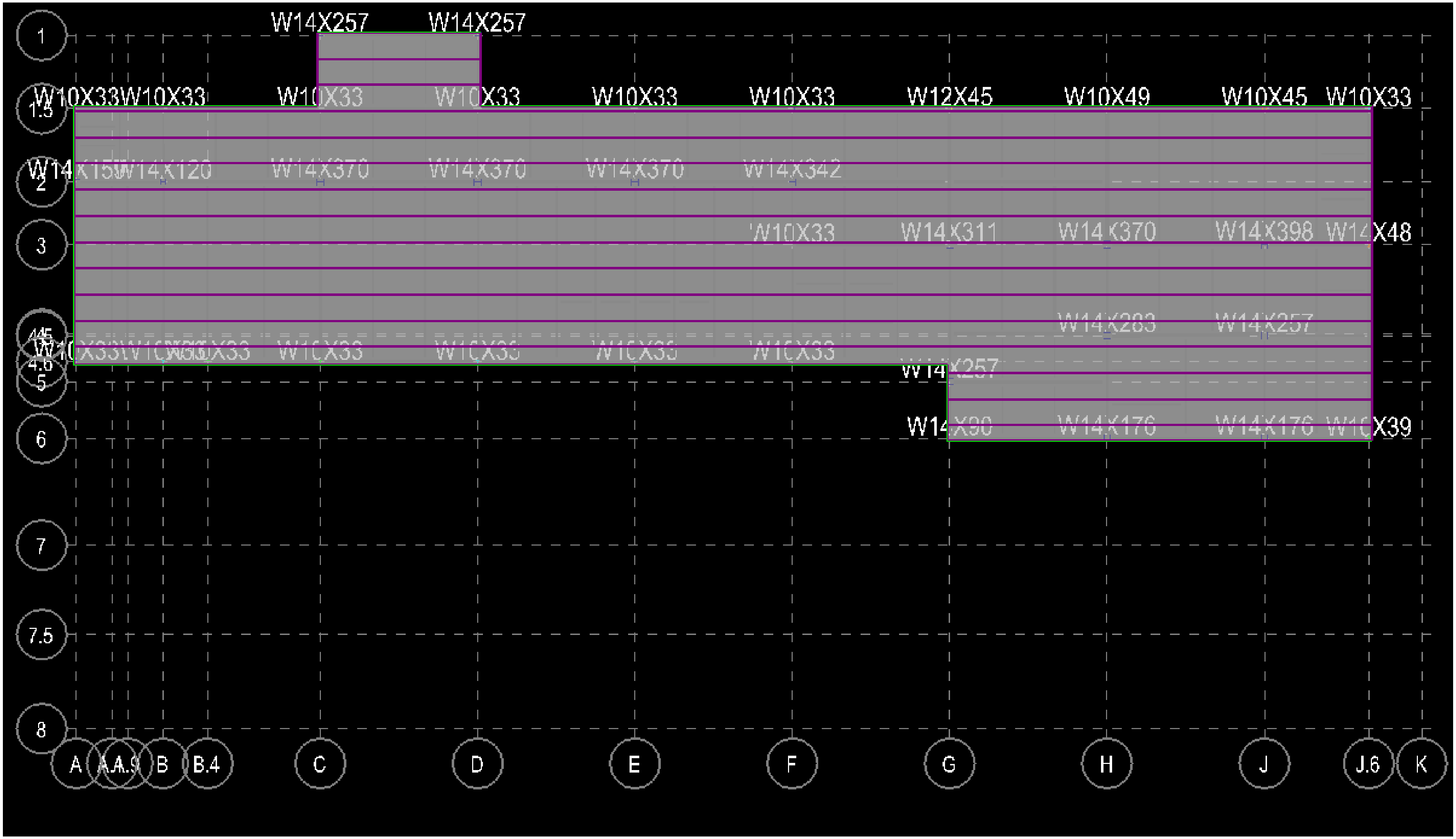
Appendix Figure 24: Third Floor Columns



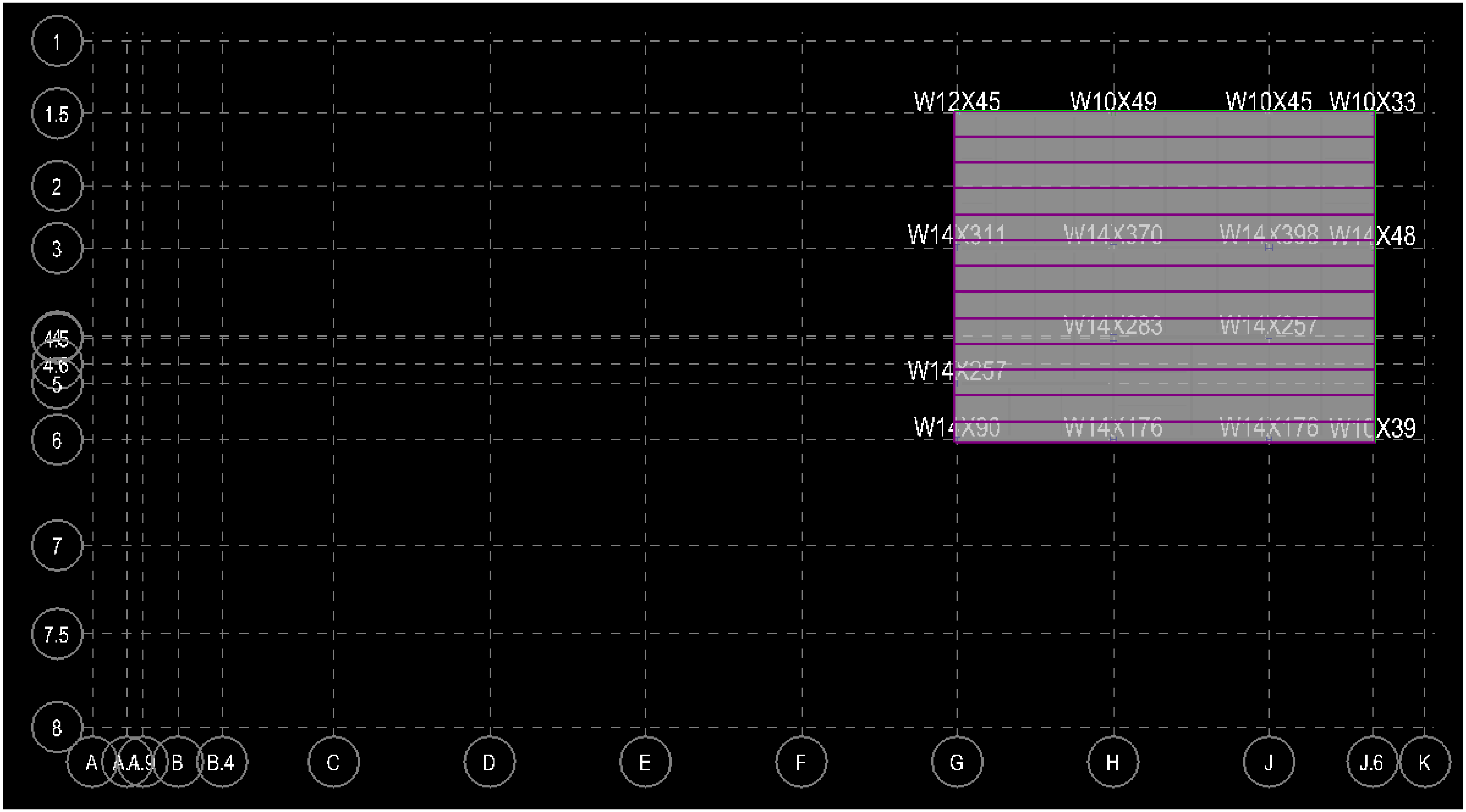
Appendix Figure 25: Fourth Floor Columns



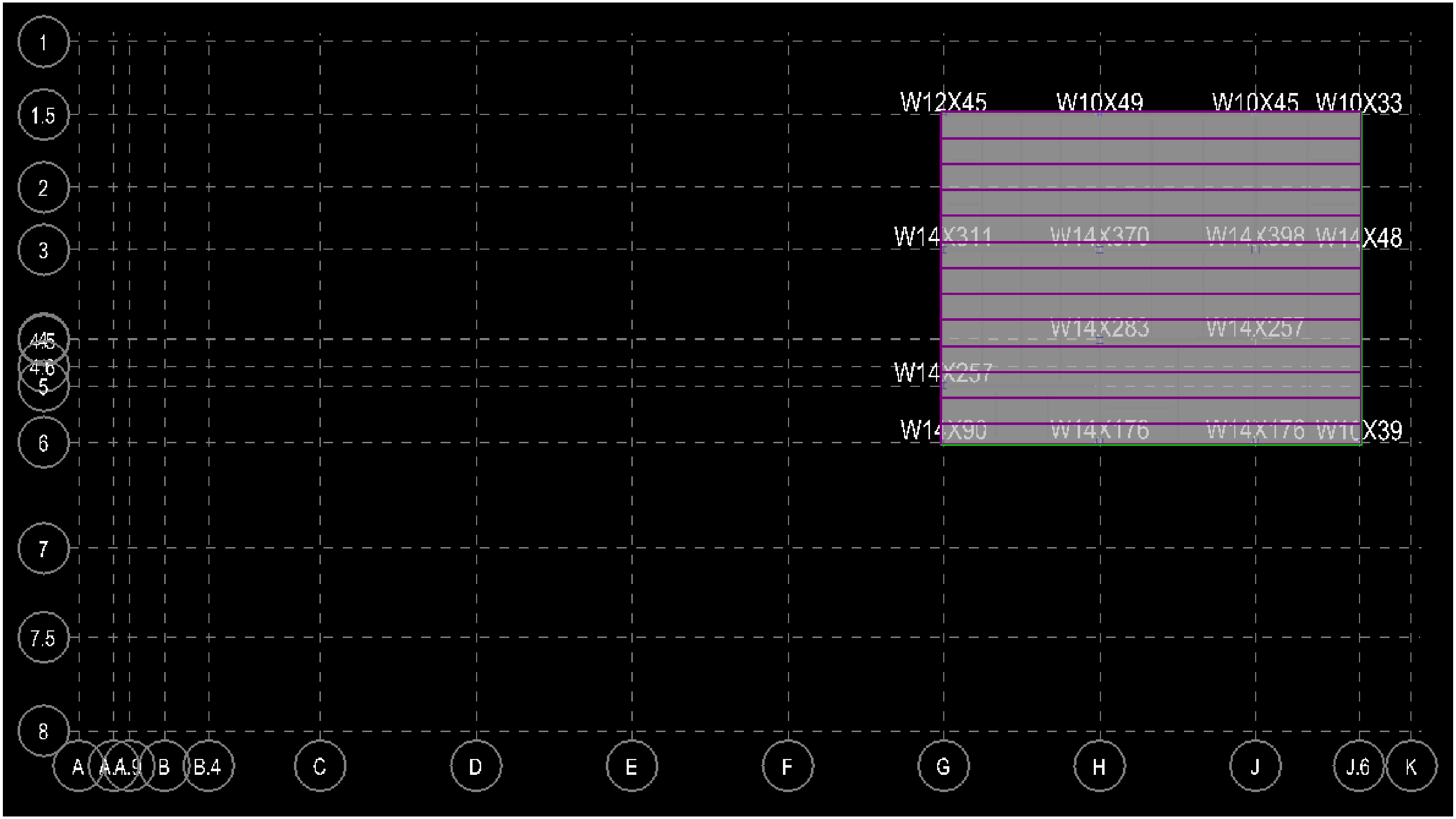
Appendix Figure 26: Fifth Floor Columns



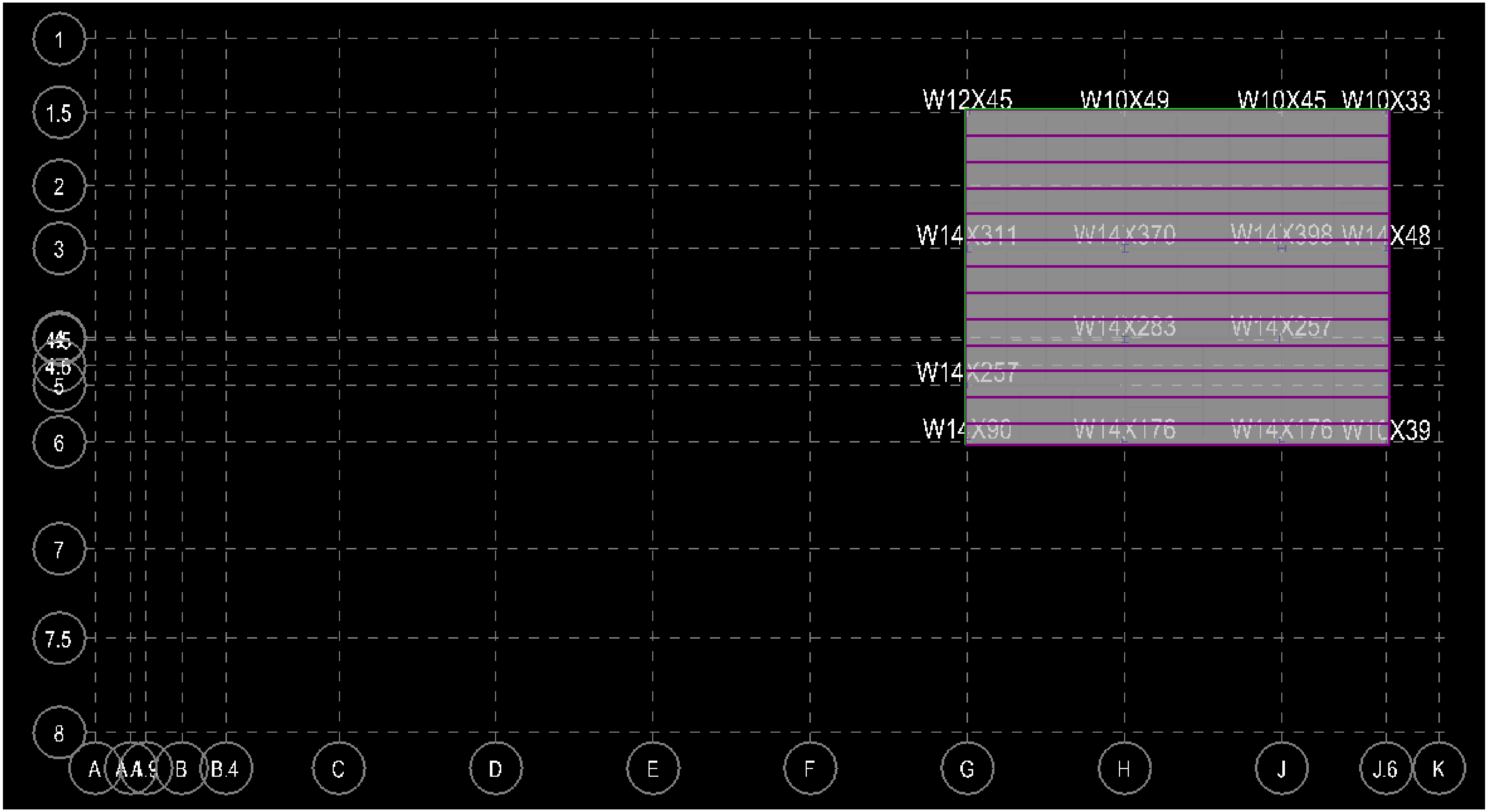
Appendix Figure 27: Sixth Floor Columns



Appendix Figure 28: Seventh Floor Columns



Appendix Figure 29: Eight Floor Columns



Appendix Figure 30: Roof Columns

Original Gravity Systemn Take-Offs

penthouse																				
beam type	w18x35	w12x40	w12x40	w12x40	w24x68	w33x263	w16x31	w16x31	w18x35	w18x40	w12x14	w12x16	w12x16	w12x16	w12x16	w16x31	w12x40	w30x90	w40x211	w27x84
length (ft)	24	15	20	22	78	39	6	21	17	17	17	17	17	17	17	17	32	35	39	35
beam weight lbs/ft	35	40	40	40	68	263	31	31	35	40	14	16	16	16	16	31	40	90	211	84
beam weight (lbs)	840	600	800	880	5304	10257	186	651	595	680	238	272	272	272	272	527	1280	3150	8229	2940
total weight:	38245 lbs																			
Unit Mat'l Cost	64	82.5	82.5	82.5	112	330	51	51	64	66	26.5	26.5	26.5	26.5	26.5	51	82.5	163	500	139
Mat'l Cost	1536	1237.5	1650	1815	8736	12870	306	1071	1088	1122	450.5	450.5	450.5	450.5	450.5	867	2640	5705	19500	4865
Unit Labor Cost	4.1	3.25	3.25	3.25	3.18	3.21	2.71	2.71	4.1	3.67	2.77	2.77	2.77	2.77	2.77	2.71	3.25	2.94	3.41	2.96
Labor Cost	98.4	48.75	65	71.5	248.04	125.19	16.26	56.91	69.7	62.39	47.09	47.09	47.09	47.09	47.09	46.07	104	102.9	132.99	103.6
Unit Equipment Cost	2.15	2.32	2.32	2.32	1.69	1.7	1.93	1.93	2.15	1.95	1.98	1.98	1.98	1.98	1.98	1.93	2.32	1.56	1.81	1.58
Equipment Cost	51.6	34.8	46.4	51.04	131.82	66.3	11.58	40.53	36.55	33.15	33.66	33.66	33.66	33.66	33.66	32.81	74.24	54.6	70.59	55.3
Total Item Cost	1686	1321.05	1761.4	1937.54	9115.86	13061.49	333.84	1168.44	1194.25	1217.54	531.25	531.25	531.25	531.25	531.25	945.88	2818.24	5862.5	19703.58	5023.9
Total Level Cost :	69807.76																			

roof																										
beam type	w16x26	w18x40	w18x35	w18x40	w18x26	w18x22	w14x22	w14x22	w14x22	w18x35	w12x19	w12x19	w12x40	w12x16	w12x16	w12x16	w12x16	w12x16	w12x16	w35x150	w27x84	w16x26	w12x22	w24x68	w14x22	w10x15
length (ft)	25	25	25	25	25	25	25	25	25	25	14	14	17	17	17	17	17	17	17	38	20	20	11	20	20	11
beam weight lbs/ft	26	40	35	40	26	22	22	22	22	35	19	19	40	16	16	16	16	16	16	150	84	26	22	68	22	15
beam weight (lbs)	650	1000	875	1000	650	550	550	550	550	875	266	266	680	272	272	272	272	272	272	5700	1680	520	242	1360	440	165
total:	52086 lbs																									
Unit Mat'l Cost	43	66	64	66	58	58	43	43	43	64	31.5	31.5	82.5	26.5	26.5	26.5	26.5	26.5	26.5	223	139	43	36.5	112	43	25
Mat'l Cost	1075	1650	1600	1650	1450	1450	1075	1075	1075	1600	441	441	1402.5	450.5	450.5	450.5	450.5	450.5	450.5	8474	2780	860	401.5	2240	860	275
Unit Labor Cost	2.44	3.67	4.1	3.67	3.67	3.67	2.46	2.46	2.46	4.1	2.77	2.77	3.25	2.77	2.77	2.77	2.77	2.77	2.77	3.02	2.96	2.44	2.77	3.18	2.46	4.06
Labor Cost	61	91.75	102.5	91.75	91.75	91.75	61.5	61.5	61.5	102.5	38.78	38.78	55.25	47.09	47.09	47.09	47.09	47.09	47.09	114.76	59.2	48.8	30.47	63.6	49.2	44.66
Unit Equipment Cost	1.74	1.95	2.15	1.95	1.95	1.95	1.76	1.76	1.76	2.15	1.98	1.98	2.32	1.98	1.98	1.98	1.98	1.98	1.98	1.6	1.58	1.74	1.98	1.69	1.76	2.9
Equipment Cost	43.5	48.75	53.75	48.75	48.75	48.75	44	44	44	53.75	27.72	27.72	39.44	33.66	33.66	33.66	33.66	33.66	33.66	60.8	31.6	34.8	21.78	33.8	35.2	31.9
Total Item Cost	1179.5	1790.5	1756.25	1790.5	1590.5	1590.5	1180.5	1180.5	1180.5	1756.25	507.5	507.5	1497.19	531.25	531.25	531.25	531.25	531.25	531.25	8649.56	2870.8	943.6	453.75	2337.4	944.4	351.56
Total Level Cost :	93326.94																									

8th level																										
beam type	w14x22	w16x31	w18x35	w16x26	w18x35	w12x16	w12x19	w33x118	w12x16	w18x35	w14x22	w12x83	w24x68	w24x76	w16x31	w21x62	w14x22	w12x16	w12x19	w21x147	w12x16	w12x19	w12x19	w16x26	w18x35	w33x130
length (ft)	200	25	25	78	26	34	17	37	9	20	20	20	30	30	20	30	40	55	10	30	15	15	15	20	45	30
beam weight lbs/ft	22	31	35	26	35	16	19	118	16	35	22	83	68	76	31	62	22	16	19	147	16	19	19	26	35	130
beam weight (lbs)	4400	775	875	2028	910	544	323	4366	144	700	440	1660	2040	2280	620	1860	880	880	190	4410	240	285	285	520	1575	3900
total:	37130 lbs																									
Unit Mat'l Cost	43	51	64	43	64	26.5	31.5	195	26.5	64	43	144	112	130	51	102	43	26.5	31.5	201	26.5	31.5	31.5	43	64	215
Mat'l Cost	8600	1275	1600	3354	1664	901	535.5	7215	238.5	1280	860	2880	3360	3900	1020	3060	1720	1457.5	315	6030	397.5	472.5	472.5	860	2880	6450
Unit Labor Cost	2.46	2.71	4.1	2.44	4.1	2.77	2.77	3	2.77	4.1	2.46	3.81	3.18	3.76	2.71	3.41	2.46	2.77	2.77	3.53	2.77	2.77	2.77	2.44	4.1	3.11
Labor Cost	492	67.75	102.5	190.32	106.6	94.18	47.09	111	24.93	82	49.2	76.2	95.4	112.8	54.2	102.3	98.4	152.35	27.7	105.9	41.55	41.55	41.55	48.8	184.5	93.3
Unit Equipment Cost	1.76	1.93	2.15	1.74	2.15	1.98	1.98	1.59	1.98	2.15	1.76	2.72	1.69	1.78	1.93	1.81	1.76	1.98	1.98	1.88	1.98	1.98	1.98	1.74	2.15	1.65
Equipment Cost	352	48.25	53.75	135.72	55.9	67.32	33.66	58.83	17.82	43	35.2	54.4	50.7	53.4	38.6	54.3	70.4	108.9	19.8	56.4	29.7	29.7	29.7	34.8	96.75	49.5
Total Item Cost	9444	1391	1756.25	3680.04	1826.5	1062.5	616.25	7384.83	281.25	1405	944.4	3010.6	3506.1	4066.2	1112.8	3216.6	1888.8	1718.75	362.5	6192.3	468.75	543.75	543.75	943.6	3161.25	6592.8
Total Level Cost :	67120.57																									

7th level																										
beam type	w14x22	w18x35	w16x31	w18x35	w16x26	w12x16	w12x19	w33x141	w12x19	w18x35	w14x22	w18x106	w12x16	w12x22	w12x22	w12x16	w24x68	w24x76	w16x26	w12x19	w12x16	w21x62	w14x22	w12x16	w21x147	w16x26
length (ft)	200	25	25	26	78	32	16	37	10	20	20	20	5	11	11	25	30	30	20	10	10	35	60	20	40	20
beam weight lbs/ft	22	35	31	35	26	16	19	141	19	35	22	106	16	22	22	16	68	76	26	19	16	62	22	16	147	26
beam weight (lbs)	4400	875	775	910	2028	512	304	5217	190	700	440	2120	80	242	242	400	2040	2280	520	190	160	2170	1320	320	5880	520
total:	42135 lbs																									
Unit Mat'l Cost	43	64	51	64	43	26.5	31.5	233	31.5	64	43	175	26.5	36.5	36.5	26.5	112	130	43	31.5	26.5	102	43	26.5	201	43
Mat'l Cost	8600	1600	1275	1664	3354	848	504	8621	315	1280	860	3500	132.5	401.5	401.5	662.5	3360	3900	860	315	265	3570	2580	530	8040	860
Unit Labor Cost	2.46	4.1	2.71	4.1	2.44	2.77	2.77	3.11	2.77	4.1	2.46	3.92	2.77	2.77	2.77	2.77	3.18	3.76	2.44	2.77	2.77	3.41	2.46	2.77	3.53	2.44
Labor Cost	492	102.5	67.75	106.6	190.32	88.64	44.32	115.07	27.7	82	49.2	78.4	13.85	30.47	30.47	69.25	95.4	112.8	48.8	27.7	27.7	119.35	147.6	55.4	141.2	48.8
Unit Equipment Cost	1.76	2.15	1.93	2.15	1.74	1.98	1.98	1.65	1.98	2.15	1.76	2.08	1.98	1.98	1.98	1.98	1.69	1.78	1.74	1.98	1.98	1.81	1.76	1.98	1.88	1.74
Equipment Cost	352	53.75	48.25	55.9	135.72	63.36	31.68	61.05	19.8	43	35.2	41.6	9.9	21.78	21.78	49.5	50.7	53.4	34.8	19.8	19.8	63.35	105.6	39.6	75.2	34.8
Total Item Cost	9444	1756.25	1391	1826.5	3680.04	1000	580	8797.12	362.5	1405	944.4	3620	156.25	453.75	453.75	781.25	3506.1	4066.2	943.6	362.5	312.5	3752.7	2833.2	625	8256.4	943.6
Total Level Cost :	75217.76																									

roof								
beam type	w16x31	w16x31	w30x90	w24x68	w16x26	w14x22	w24x117	w21x44
length (ft)	20	20	35	20	20	20	70	20
beam weight lbs/ft	31	31	90	68	26	22	117	44
beam weight (lbs)	620	620	3150	1360	520	440	8190	880
total:								
Unit Mat'l Cost	51	51	163	112	43	43	193	72.5
Mat'l Cost	1020	1020	5705	2240	860	860	13510	1450
Unit Labor Cost	2.71	2.71	2.94	3.18	2.44	2.46	3.36	3.32
Labor Cost	54.2	54.2	102.9	63.6	48.8	49.2	235.2	66.4
Unit Equipment Cost	1.93	1.93	1.56	1.69	1.74	1.76	1.79	1.76
Equipment Cost	38.6	38.6	54.6	33.8	34.8	35.2	125.3	35.2
Total Item Cost	1112.8	1112.8	5862.5	2337.4	943.6	944.4	13870.5	1551.6
Total Level Cost :								

roof (continued)								
beam type	w12x16	w21x50	w12x16	w16x26	w12x26	w44x290	w24x55	w21x50
length (ft)	40	10	15	20	20	39	25	20
beam weight lbs/ft	16	50	16	26	26	290	55	50
beam weight (lbs)	640	500	240	520	520	11310	1375	1000
total:								
Unit Mat'l Cost	26.5	82.5	26.5	43	43	500	91	82.5
Mat'l Cost	1060	825	397.5	860	860	19500	2275	1650
Unit Labor Cost	2.77	3.32	2.77	2.44	2.77	3.41	3.18	3.32
Labor Cost	110.8	33.2	41.55	48.8	55.4	132.99	79.5	66.4
Unit Equipment Cost	1.98	1.76	1.98	1.74	1.98	1.81	1.69	1.76
Equipment Cost	79.2	17.6	29.7	34.8	39.6	70.59	42.25	35.2
Total Item Cost	1250	875.8	468.75	943.6	955	19703.58	2396.75	1751.6
Total Level Cost :								

8th level
beam type
length (ft)
beam weight lbs/ft
beam weight (lbs)
total:
Unit Mat'l Cost
Mat'l Cost
Unit Labor Cost
Labor Cost
Unit Equipment Cost
Equipment Cost
Total Item Cost
Total Level Cost :

7th level				
beam type	w12x16	w12x19	w33x130	w18x35
length (ft)	15	15	40	45
beam weight lbs/ft	16	19	130	35
beam weight (lbs)	240	285	5200	1575
total:				
Unit Mat'l Cost	26.5	31.5	215	64
Mat'l Cost	397.5	472.5	8600	2880
Unit Labor Cost	2.77	2.77	3.11	4.1
Labor Cost	41.55	41.55	124.4	184.5
Unit Equipment Cost	1.98	1.98	1.65	2.15
Equipment Cost	29.7	29.7	66	96.75
Total Item Cost	468.75	543.75	8790.4	3161.25
Total Level Cost :				

6th level																										
beam type	w16x26	w12x16	w12x26	w12x16	w16x26	w16x26	w21x19	w18x40	w18x50	w36x135	w12x62	w18x46	w18x65	w18x65	w12x16	w18x65	w18x46	w18x35	w18x60	w21x57	w21x111	w18x50	w12x18	w18x35	w27x178	w24x62
length (ft)	30	30	52	195	100	58	51	20	25	37	25	34.5	34.5	34.5	34.5	34.5	34.5	103.5	34.5	34.5	34.5	34.5	36	12	30	26
beam weight lbs/ft	26	16	26	16	26	26	19	40	50	135	62	46	65	65	16	65	46	35	60	60	111	50	18	35	178	62
beam weight (lbs)	780	480	1352	3120	2600	1508	969	800	1250	4995	1550	1587	2242.5	2242.5	552	2242.5	1587	3622.5	2070	2070	3829.5	1725	648	420	5340	1612
total:	116396 lbs																									
Unit Mat'l Cost	43	26.5	43	26.5	43	43	72.5	66	82.5	223	199	76	107	107	26.5	107	76	64	107	102	201	82.5	36.5	64	266	102
Mat'l Cost	1290	795	2236	5167.5	4300	2494	3697.5	1320	2062.5	8251	4975	2622	3691.5	3691.5	914.25	3691.5	2622	6624	3691.5	3519	6934.5	2846.25	1314	768	7980	2652
Unit Labor Cost	2.44	2.77	2.77	2.77	2.44	2.44	3.32	3.67	3.87	3.02	3.81	3.67	3.92	3.92	2.77	3.92	3.67	4.1	3.92	3.41	3.53	3.87	2.77	4.1	3.07	3.18
Labor Cost	73.2	83.1	144.04	540.15	244	141.52	169.32	73.4	96.75	111.74	95.25	126.615	135.24	135.24	95.565	135.24	126.615	424.35	135.24	117.645	121.785	133.515	99.72	49.2	92.1	82.68
Unit Equipment Cost	1.74	1.98	1.98	1.98	1.74	1.74	1.76	1.95	2.06	1.6	2.72	1.95	2.08	2.08	1.98	2.08	1.95	2.15	2.08	1.81	1.88	2.06	1.98	2.15	1.63	1.69
Equipment Cost	52.2	59.4	102.96	386.1	174	100.92	89.76	39	51.5	59.2	68	67.275	71.76	71.76	68.31	71.76	67.275	222.525	71.76	62.445	64.86	71.07	71.28	25.8	48.9	43.94
Total Item Cost	1415.4	937.5	2483	6093.75	4718	2736.44	3956.58	1432.4	2210.75	8421.94	5138.25	2815.89	3898.5	3898.5	1078.125	3898.5	2815.89	7270.875	3898.5	3699.09	7121.145	3050.835	1485	843	8121	2778.62
Total Level Cost :	213569.7																									

5th level																										
beam type	w18x35	w21x50	w24x68	w21x83	w21x46	w14x22	w44x290	w16x26	w27x84	w24x76	w16x26	w21x44	w40x277	w24x76	w18x35	w18x40	w40x199	w36x150	w30x99	w14x22	w18x35	w36x150	w30x90	w10x15	w21x62	w21x55
length (ft)	304	69	34.5	38	29	29	29	56	28	28	224	56	68	28	28	120	40	40	40	16.5	16.5	30	90	110	60	60
beam weight lbs/ft	35	50	68	83	46	22	290	26	84	76	26	44	277	76	35	40	199	150	99	22	35	150	90	15	62	55
beam weight (lbs)	10640	3450	2346	3154	1334	638	8410	1456	2352	2128	5824	2464	18836	2128	980	4800	7960	6000	3960	363	577.5	4500	8100	1650	3720	3300
total:	169389 lbs																									
Unit Mat'l Cost	64	82.5	112	137	82.5	43	500	43	139	130	43	72.5	500	130	64	66	500	248	163	43	64	248	163	25	102	102
Mat'l Cost	19456	5692.5	3864	5206	2392.5	1247	14500	2408	3892	3640	9632	4060	34000	3640	1792	7920	20000	9920	6520	709.5	1056	7440	14670	2750	6120	6120
Unit Labor Cost	4.1	3.32	3.18	3.53	3.32	2.46	3.41	2.44	2.96	3.76	2.44	3.32	3.41	3.76	4.1	3.67	3.41	3.02	2.94	2.46	4.1	3.02	2.94	4.06	3.41	3.41
Labor Cost	1246.4	229.08	109.71	134.14	96.28	71.34	98.89	136.64	82.88	105.28	546.56	185.92	231.88	105.28	114.8	440.4	136.4	120.8	117.6	40.59	67.65	90.6	264.6	446.6	204.6	204.6
Unit Equipment Cost	2.15	1.76	1.69	1.88	1.76	1.76	1.81	1.74	1.58	1.78	1.74	1.76	1.81	1.78	2.15	1.95	1.81	1.6	1.56	1.76	2.15	1.6	1.56	2.9	1.81	1.81
Equipment Cost	653.6	121.44	58.305	71.44	51.04	51.04	52.49	97.44	44.24	49.84	389.76	98.56	123.08	49.84	60.2	234	72.4	64	62.4	29.04	35.475	48	140.4	319	108.6	108.6
Total Item Cost	21356	6043.02	4032.015	5411.58	2539.82	1369.38	14651.38	2642.08	4019.12	3795.12	10568.32	4344.48	34354.96	3795.12	1967	8594.4	20208.8	10104.8	6700	779.13	1159.125	7578.6	15075	3515.6	6433.2	6433.2
Total Level Cost :	310440																									

4th level																										
beam type	w18x35	w18x40	w12x62	w18x35	w16x26	w18x35	w24x68	w12x19	w12x19	w30x99	w33x130	w18x35	w33x144	w24x68	w27x84	w12x16	w18x40	w14x22	w21x50	w14x22	w24x68	w36x170	w21x44	w30x99	w16x31	w18x35
length (ft)	304	38	38	38	396	245	35	90	12	35	37	30	38	30	60	50	20	20	20	10	26	39	25	30	30	62
beam weight lbs/ft	35	40	62	35	26	35	68	19	19	99	130	35	144	68	84	16	40	22	50	22	68	170	44	99	31	35
beam weight (lbs)	10640	1520	2356	1330	10296	8575	2380	1710	228	3465	4810	1050	5472	2040	5040	800	800	440	1000	220	1768	6630	1100	2970	930	2170
total:	90008.7 lbs																									
Unit Mat'l Cost	64	66	199	58	43	58	112	31.5	31.5	163	215	58	279	112	139	26.5	66	43	82.5	43	112	281	72.5	163	51	58
Mat'l Cost	19456	2508	7562	2204	17028	14210	3920	2835	378	5705	7955	1740	10602	3360	8340	1325	1320	860	1650	430	2912	10959	1812.5	4890	1530	3596
Unit Labor Cost	4.1	3.67	3.81	3.67	2.44	3.67	3.18	2.77	2.77	2.94	3.11	3.67	3.21	3.18	2.96	2.77	3.67	2.46	3.32	2.46	3.18	3.07	3.32	2.94	2.71	3.67
Labor Cost	1246.4	139.46	144.78	139.46	966.24	899.15	111.3	249.3	33.24	102.9	115.07	110.1	121.98	95.4	177.6	138.5	73.4	49.2	66.4	24.6	82.68	119.73	83	88.2	81.3	227.54
Unit Equipment Cost	2.15	1.95	2.72	1.95	1.74	1.95	1.69	1.98	1.98	1.56	1.65	1.95	1.7	1.69	1.58	1.98	1.95	1.76	1.76	1.76	1.69	1.63	1.76	1.56	1.93	1.95
Equipment Cost	653.6	74.1	103.36	74.1	689.04	477.75	59.15	178.2	23.76	54.6	61.05	58.5	64.6	50.7	94.8	99	39	35.2	35.2	17.6	43.94	63.57	44	46.8	57.9	120.9
Total Item Cost	21356	2721.56	7810.14	2417.56	18683.28	15586.9	4090.45	3262.5	435	5862.5	8131.12	1908.6	10788.58	3506.1	8612.4	1562.5	1432.4	944.4	1751.6	472.2	3038.62	11142.3	1939.5	5025	1669.2	3944.44
Total Level Cost :	167066																									

3rd level																										
beam type	w18x35	w16x26	w30x108	w18x50	w12x16	w12x22	w12x19	w12x16	w12x22	w18x40	w18x35	w21x57	w27x84	w24x62	w24x84	w21x62	w21x50	w30x90	w24x306	w21x73	w12x19	w12x16	w24x94	w21x62	w21x44	w30x90
length (ft)	532	476	38	28	8	22	10	10	16.5	16.5	16.5	60	150	90	30	30	30	30	40	38	24	40	26	40	200	60
beam weight lbs/ft	35	26	108	50	16	22	19	16	22	40	35	57	84	62	84	62	50	90	306	73	19	16	94	62	44	90
beam weight (lbs)	18620	12376	4104	1400	128	484	190	160	363	660	577.5	3420	12600	5580	2520	1860	1500	2700	12240	2774	456	640	2444	2480	8800	5400
total:	140824.5 lbs																									
Unit Mat'l Cost	64	43	178	82.5	26.5	36.5	31.5	26.5	36.5	66	64	102	139	102	139	102	82.5	163	241	137	31.5	26.5	155	102	72.5	163
Mat'l Cost	34048	20468	6764	2310	212	803	315	265	602.25	1089	1056	6120	20850	9180	4170	3060	2475	4890	9640	5206	756	1060	4030	4080	14500	9780
Unit Labor Cost	4.1	2.44	2.94	3.87	2.77	2.77	2.77	2.77	2.77	3.67	4.1	3.41	2.96	3.18	3.27	3.41	3.32	2.94	3.36	3.53	2.77	2.77	3.27	3.41	3.32	2.94
Labor Cost	2181.2	1161.44	111.72	108.36	22.16	60.94	27.7	27.7	45.705	60.555	67.65	204.6	444	286.2	98.1	102.3	99.6	88.2	134.4	134.14	66.48	110.8	85.02	136.4	664	176.4
Unit Equipment Cost	2.15	1.74	1.56	2.06	1.98	1.98	1.98	1.98	1.98	1.95	2.15	1.81	1.58	1.69	1.74	1.81	1.76	1.56	1.79	1.88	1.98	1.98	1.74	1.81	1.76	1.56
Equipment Cost	1143.8	828.24	59.28	57.68	15.84	43.56	19.8	19.8	32.67	32.175	35.475	108.6	237	152.1	52.2	54.3	52.8	46.8	71.6	71.44	47.52	79.2	45.24	72.4	352	93.6

6th level

beam type	w14x26	w21x50	w21x44	w21x50	w24x55	w24x76	w18x35	w18x40	w14x22	w16x26	w12x16	w18x60	w18x26
length (ft)	44	90	60	30	30	30	20	32	40	20	60	19	19
beam weight lbs/ft	26	50	44	50	55	76	35	40	22	26	16	60	26
beam weight (lbs)	1144	4500	2640	1500	1650	2280	700	1280	880	520	960	1140	494
total:													
Unit Mat'l Cost	43	82.5	72.5	82.5	91	130	64	66	43	43	26.5	107	58
Mat'l Cost	1892	7425	4350	2475	2730	3900	1280	2112	1720	860	1590	2033	1102
Unit Labor Cost	2.46	3.32	3.32	3.32	3.18	3.76	4.1	3.67	2.46	2.44	2.77	3.92	3.67
Labor Cost	108.24	298.8	199.2	99.6	95.4	112.8	82	117.44	98.4	48.8	166.2	74.48	69.73
Unit Equipment Cost	1.76	1.76	1.76	1.76	1.69	1.78	2.15	1.95	1.76	1.74	1.98	2.08	1.95
Equipment Cost	77.44	158.4	105.6	52.8	50.7	53.4	43	62.4	70.4	34.8	118.8	39.52	37.05
Total Item Cost	2077.68	7882.2	4654.8	2627.4	2876.1	4066.2	1405	2291.84	1888.8	943.6	1875	2147	1208.78
Total Level Cost :													

beam type	w18x35	w21x62	w36x170	w21x44	w12x19	w12x16	w12x22	w12x19	w18x35	w30x90	w27x84	w27x84	w21x50
length (ft)	19	39	39	25	25	10	20	25	20	30	30	30	21.5
beam weight lbs/ft	35	62	170	44	19	16	22	19	35	90	84	84	50
beam weight (lbs)	665	2418	6630	1100	475	160	440	475	700	2700	2520	2520	1075
total:													
Unit Mat'l Cost	64	102	281	72.5	31.5	26.5	36.5	31.5	64	163	139	139	82.5
Mat'l Cost	1216	3978	10959	1812.5	787.5	265	730	787.5	1280	4890	4170	4170	1773.75
Unit Labor Cost	4.1	3.41	3.07	3.32	2.77	2.77	2.77	2.77	4.1	2.94	2.96	2.96	3.32
Labor Cost	77.9	132.99	119.73	83	69.25	27.7	55.4	69.25	82	88.2	88.8	88.8	71.38
Unit Equipment Cost	2.15	1.81	1.63	1.76	1.98	1.98	1.98	1.98	2.15	1.56	1.58	1.58	1.76
Equipment Cost	40.85	70.59	63.57	44	49.5	19.8	39.6	49.5	43	46.8	47.4	47.4	37.84
Total Item Cost	1334.75	4181.58	11142.3	1939.5	906.25	312.5	825	906.25	1405	5025	4306.2	4306.2	1882.97
Total Level Cost :													

5th level

beam type	w21x62	w18x40	w19x19	w18x46	w21x44	w30x99	w24x84	w27x94	w33x141	w10x15	w12x16	w21x62	w16x26
length (ft)	30	16.5	16.5	16.5	8.5	30	90	30	30	160	30	36	58
beam weight lbs/ft	62	40	19	46	44	99	84	94	141	15	16	62	26
beam weight (lbs)	1860	660	313.5	759	374	2970	7560	2820	4230	2400	480	2232	1508
total:													
Unit Mat'l Cost	102	66	58	76	72.5	163	139	155	233	25	26.5	102	43
Mat'l Cost	3060	1089	957	1254	616.25	4890	12510	4650	6990	4000	795	3672	2494
Unit Labor Cost	3.41	3.67	3.67	3.67	3.32	2.94	3.27	2.96	3.11	4.06	2.77	3.41	2.44
Labor Cost	102.3	60.555	60.555	60.555	28.22	88.2	294.3	88.8	93.3	649.6	83.1	122.76	141.52
Unit Equipment Cost	1.81	1.95	1.95	1.95	1.76	1.56	1.74	1.58	1.65	2.9	1.98	1.81	1.74
Equipment Cost	54.3	32.175	32.175	32.175	14.96	46.8	156.6	47.4	49.5	464	59.4	65.16	100.92
Total Item Cost	3216.6	1181.73	1049.73	1346.73	659.43	5025	12960.9	4786.2	7132.8	5113.6	937.5	3859.92	2736.44
Total Level Cost :													

beam type	w12x19	w18x50	w16x31	w18x35	w30x99	w21x44	w36x170	w21x62	w12x19	w12x16	w12x19	w36x135	w18x46
length (ft)	68	32	30	30	30	25	40	40	30	30	20	37	20
beam weight lbs/ft	19	50	31	35	99	44	170	62	19	16	19	135	46
beam weight (lbs)	1292	1600	930	1050	2970	1100	6800	2480	570	480	380	4995	920
total:													
Unit Mat'l Cost	31.5	82.5	51	64	163	72.5	281	102	31.5	26.5	31.5	223	76
Mat'l Cost	2142	2640	1530	1920	4890	1812.5	11240	4080	945	795	630	8251	1520
Unit Labor Cost	2.77	3.87	2.71	4.1	2.94	3.32	3.07	3.41	2.77	2.77	2.77	3.02	3.67
Labor Cost	188.36	123.84	81.3	123	88.2	83	122.8	136.4	83.1	83.1	55.4	111.74	73.4
Unit Equipment Cost	1.98	2.06	1.93	2.15	1.56	1.76	1.63	1.81	1.98	1.98	1.98	1.6	1.95
Equipment Cost	134.64	65.92	57.9	64.5	46.8	44	65.2	72.4	59.4	59.4	39.6	59.2	39
Total Item Cost	2465	2829.76	1669.2	2107.5	5025	1939.5	11428	4288.8	1087.5	937.5	725	8421.94	1632.4
Total Level Cost :													

4th level

beam type	w18x60	w21x50	w24x55	w12x19	w14x22	w12x22	w12x19	w12x22	w18x35	w21x44	w12x22	w12x16
length (ft)	30	30	40	36	30	15	20	16.6	16.5	28	10	20
beam weight lbs/ft	60	50	55	19	22	22	19	22	35	44	22	16
beam weight (lbs)	1800	1500	2200	684	660	330	380	365.2	577.5	1232	220	320
total:												
Unit Mat'l Cost	107	82.5	91	31.5	43	36.5	31.5	36.5	64	72.5	36.5	26.5
Mat'l Cost	3210	2475	3640	1134	1290	547.5	630	605.9	1056	2030	365	530
Unit Labor Cost	3.92	3.32	3.18	2.77	2.46	2.77	2.77	2.77	4.1	3.32	2.77	2.77
Labor Cost	117.6	99.6	127.2	99.72	73.8	41.55	55.4	45.982	67.65	92.96	27.7	55.4
Unit Equipment Cost	2.08	1.76	1.69	1.98	1.76	1.98	1.98	1.98	2.15	1.76	1.98	1.98
Equipment Cost	62.4	52.8	67.6	71.28	52.8	29.7	39.6	32.868	35.475	49.28	19.8	39.6
Total Item Cost	3390	2627.4	3834.8	1305	1416.6	618.75	725	684.75	1159.125	2172.24	412.5	625
Total Level Cost :												

3rd level

beam type	w30x99	w16x26	w12x19	w12x19	w21x50	w16x31	w21x44	w12x16	w21x62	w14x22
length (ft)	40	58	75	60	30	30	32	45	40	20
beam weight lbs/ft	99	26	19	19	50	31	44	16	62	22
beam weight (lbs)	3960	1508	1425	1140	1500	930	1408	720	2480	440
total:										
Unit Mat'l Cost	163	43	31.5	31.5	82.5	51	72.5	26.5	102	43
Mat'l Cost	6520	2494	2362.5	1890	2475	1530	2320	1192.5	4080	860
Unit Labor Cost	2.94	2.44	2.77	2.77	3.32	2.71	3.32	2.77	3.41	2.46
Labor Cost	117.6	141.52	207.75	166.2	99.6	81.3	106.24	124.65	136.4	49.2
Unit Equipment Cost	1.56	1.74	1.98	1.98	1.76	1.93	1.76	1.98	1.81	1.76
Equipment Cost	62.4	100.92	148.5	118.8	52.8	57.9	56.32	89.1	72.4	35.2

beam type	w18x60	w18x35	w33x130	w30x90	w21x44	w36x182	w12x22	w18x55	w12x19	w21x44
length (ft)	20	30	37	30	25	40	11	20	25	20
beam weight lbs/ft	60	35	130	90	44	182	22	55	19	44
beam weight (lbs)	1200	1050	4810	2700	1100	7280	242	1100	475	880
total:										
Unit Mat'l Cost	107	64	215	163	72.5	320	36.5	91	31.5	72.5
Mat'l Cost	2140	1920	7955	4890	1812.5	12800	401.5	1820	787.5	1450
Unit Labor Cost	3.92	4.1	3.11	2.94	3.32	3.14	2.77	3.87	2.77	3.32
Labor Cost	78.4	123	115.07	88.2	83	125.6	30.47	77.4	69.25	66.4
Unit Equipment Cost	2.08	2.15	1.65	1.56	1.76	1.67	1.98	2.06	1.98	1.76
Equipment Cost	41.6	64.5	61.05	46.8	44	66.8	21.78	41.2	49.5	35.2

6th level																
beam type	w16x26	w18x40	w12x26	w12x19	w18x35	w24x62	w24x76	w18x35	w18x50	w30x99	w12x19	w12x26	w12x16	w12x40	w24x117	w18x60
length (ft)	8.5	16.5	91	32	25	30	30	16.5	29	29	13	80	140	20	30	16.5
beam weight lbs/ft	26	40	26	19	35	62	76	35	50	99	19	26	16	40	117	60
beam weight (lbs)	221	660	2366	608	875	1860	2280	577.5	1450	2871	247	2080	2240	800	3510	990
total:																
Unit Mat'l Cost	43	66	43	31.5	64	102	130	64	82.5	163	31.5	43	26.5	82.5	193	107
Mat'l Cost	365.5	1089	3913	1008	1600	3060	3900	1056	2392.5	4727	409.5	3440	3710	1650	5790	1765.5
Unit Labor Cost	2.44	3.67	2.77	2.77	4.1	3.18	3.76	4.1	3.87	2.94	2.77	2.77	2.77	3.25	3.36	3.92
Labor Cost	20.74	60.555	252.07	88.64	102.5	95.4	112.8	67.65	112.23	85.26	36.01	221.6	387.8	65	100.8	64.68
Unit Equipment Cost	1.74	1.95	1.98	1.98	2.15	1.69	1.78	2.15	2.06	1.56	1.98	1.98	1.98	2.32	1.79	2.08
Equipment Cost	14.79	32.175	180.18	63.36	53.75	50.7	53.4	35.475	59.74	45.24	25.74	158.4	277.2	46.4	53.7	34.32
Total Item Cost	401.03	1181.73	4345.25	1160	1756.25	3206.1	4066.2	1159.125	2564.47	4857.5	471.25	3820	4375	1761.4	5944.5	1864.5
Total Level Cost :																

5th level						
beam type	w14x22	w12x50	w10x15	w12x16	w12x19	w24x84
length (ft)	20	20	25	30	10	25
beam weight lbs/ft	22	50	15	16	19	84
beam weight (lbs)	440	1000	375	480	190	2100
total:						
Unit Mat'l Cost	43	82.5	25	26.5	31.5	139
Mat'l Cost	860	1650	625	795	315	3475
Unit Labor Cost	2.46	3.25	4.06	2.77	2.77	3.27
Labor Cost	49.2	65	101.5	83.1	27.7	81.75
Unit Equipment Cost	1.76	2.32	2.9	1.98	1.98	1.74
Equipment Cost	35.2	46.4	72.5	59.4	19.8	43.5
Total Item Cost	944.4	1761.4	799	937.5	362.5	3600.25
Total Level Cost :						

Total Item Cost	37373	22457.68	693.5	2476.04	250	907.5	362.5	312.5	680.625	1181.73	1159.125	6433.2	21531	9618.3	4320.3	3216.6	2627.4	5025	9846	5411.58	870	1250	4160.26	4288.8	15516	10050
Total Level Cost :	243314.7																									

2nd level																										
beam type	w18x35	w16x26	w24x55	w24x62	w24x68	w30x99	w33x169	w18x40	w18x35	w16x31	w24x76	w12x19	w30x148	w30x173	w30x99	w27x84	w24x94	w24x76	w30x90	w33x118	w33x130	w24x55	w12x22	w21x44	w27x84	w21x50
length (ft)	494	308	324	108	36	36	36	93	62	20.5	31	52	30	30	30	30	86	26	26	30	30	22	22	93	31	31
beam weight lbs/ft	35	26	55	62	68	99	169	40	35	31	76	19	148	173	99	84	94	76	90	118	130	55	22	44	84	50
beam weight (lbs)	17290	8008	17820	6696	2448	3564	6084	3720	2170	635.5	2356	988	4440	5190	2970	2520	8084	1976	2340	3540	3900	1210	484	4092	2604	1550
total:	220889.5 lbs																									
Unit Mat'l Cost	64	43	91	102	112	163	279	66	64	51	130	31.5	244	285	163	139	155	130	163	195	215	91	36.5	72.5	139	82.5
Mat'l Cost	31616	13244	29484	11016	4032	5868	10044	6138	3968	1045.5	4030	1638	7320	8550	4890	4170	13330	3380	4238	5850	6450	2002	803	6742.5	4309	2557.5
Unit Labor Cost	4.1	2.44	3.18	3.18	3.18	2.94	3.21	3.67	4.1	2.71	3.76	2.77	3.04	3.15	2.94	2.96	3.27	3.76	2.94	3	3.11	3.18	2.77	3.32	2.96	3.32
Labor Cost	2025.4	751.52	1030.32	343.44	114.48	105.84	115.56	341.31	254.2	55.555	116.56	144.04	91.2	94.5	88.2	88.8	281.22	97.76	76.44	90	93.3	69.96	60.94	308.76	91.76	102.92
Unit Equipment Cost	2.15	1.74	1.69	1.69	1.69	1.56	1.7	1.95	2.15	1.93	1.78	1.98	1.62	1.67	1.56	1.58	1.74	1.78	1.56	1.59	1.65	1.69	1.98	1.76	1.58	1.76
Equipment Cost	1062.1	535.92	547.56	182.52	60.84	56.16	61.2	181.35	133.3	39.565	55.18	102.96	48.6	50.1	46.8	47.4	149.64	46.28	40.56	47.7	49.5	37.18	43.56	163.68	48.98	54.56
Total Item Cost	34703.5	14531.44	31061.88	11541.96	4207.32	6030	10220.76	6660.66	4355.5	1140.62	4201.74	1885	7459.8	8694.6	5025	4306.2	13760.86	3524.04	4355	5987.7	6592.8	2109.14	907.5	7214.94	4449.74	2714.98
Total Level Cost :	383564.4																									

1st level																										
beam type	w18x40	w18x35	w24x94	w24x76	w12x19	w24x62	w30x90	w30x90	w18x40	w21x50	w21x62	w24x117	w24x84	w21x44	w16x26	w18x65	W18X40	W18X46	W18X35	W21X44	W21X150	W18X46	W12X19	W21X44	W12X22	W24X55
length (ft)	72	252	36	36	30	26	30	30	20	26	30	30	30	20	155	186	418	38	38	38	38	38	34	60	28	26
beam weight lbs/ft	40	35	94	76	19	62	90	90	40	50	62	117	84	44	26	65	40	46	35	44	150	46	19	44	22	55
beam weight (lbs)	2880	8820	3384	2736	570	1612	2700	2700	800	1300	1860	3510	2520	880	4030	12090	16720	1748	1330	1672	5700	1748	646	2640	616	1430
total:	177844 lbs																									
Unit Mat'l Cost	66	64	155	130	31.5	102	163	163	66	82.5	102	193	139	72.5	43	107	66	76	64	72.5	201	76	31.5	72.5	36.5	91
Mat'l Cost	4752	16128	5580	4680	945	2652	4890	4890	1320	2145	3060	5790	4170	1450	6665	19902	27588	2888	2432	2755	7638	2888	1071	4350	1022	2366
Unit Labor Cost	3.67	4.1	3.27	3.76	2.77	3.18	2.94	2.94	3.67	3.32	3.41	3.36	3.27	3.32	2.44	3.92	3.67	3.67	4.1	3.32	3.53	3.67	2.77	3.32	2.77	3.18
Labor Cost	264.24	1033.2	117.72	135.36	83.1	82.68	88.2	88.2	73.4	86.32	102.3	100.8	98.1	66.4	378.2	729.12	1534.06	139.46	155.8	126.16	134.14	139.46	94.18	199.2	77.56	82.68
Unit Equipment Cost	1.95	2.15	1.74	1.78	1.98	1.69	1.56	1.56	1.95	1.76	1.81	1.79	1.74	1.76	1.74	2.08	1.95	1.95	2.15	1.76	1.88	1.95	1.98	1.76	1.98	1.69
Equipment Cost	140.4	541.8	62.64	64.08	59.4	43.94	46.8	46.8	39	45.76	54.3	53.7	52.2	35.2	269.7	386.88	815.1	74.1	81.7	66.88	71.44	74.1	67.32	105.6	55.44	43.94
Total Item Cost	5156.64	17703	5760.36	4879.44	1087.5	2778.62	5025	5025	1432.4	2277.08	3216.6	5944.5	4320.3	1551.6	7312.9	21018	29937.16	3101.56	2669.5	2948.04	7843.58	3101.56	1232.5	4654.8	1155	2492.62
Total Level Cost :	316826.2																									

Total Item Cost	6700	2736.44	2718.75	2175	2627.4	1669.2	2482.56	1406.25	4288.8	944.4	2260	2107.5	8131.12	5025	1939.5	12992.4	453.75	1938.6	906.25	1551.6
Total Level Cost :																				

2nd level

beam type	w18x40	w24x76	w27x94	w12x22	w21x44	w24x103	w33x118	w18x35	w16x26	w16x31	w18x35	w24x76	w21x147	w18x35	w21x62	w18x60	w14x370	w36x182	w21x50	w30x108	w18x50	w12x19	w12x22	w21x44	w18x35	w14x22
length (ft)	31	22	26	57.75	16.5	30	38	29	145	29	29	31	28	60	30	150	30	40	25	30	15	35	20	25	20	20
beam weight lbs/ft	40	76	94	22	44	103	118	35	26	31	35	76	147	35	62	60	70	182	50	108	50	19	22	44	35	22
beam weight (lbs)	1240	1672	2444	1270.5	726	3090	4484	1015	3770	899	1015	2356	4116	2100	1860	9000	2100	7280	1250	3240	750	665	440	1100	700	440
total:																										
Unit Mat'l Cost	66	130	155	36.5	72.5	172	195	64	43	51	64	130	201	64	102	107	198	320	82.5	178	82.5	31.5	36.5	72.5	64	43
Mat'l Cost	2046	2860	4030	2107.875	1196.25	5160	7410	1856	6235	1479	1856	4030	5628	3840	3060	16050	5940	12800	2062.5	5340	1237.5	1102.5	730	1812.5	1280	860
Unit Labor Cost	3.67	3.76	2.96	2.77	3.32	3.36	3	4.1	2.44	2.71	4.1	3.76	3.53	4.1	3.41	3.92	3.39	3.14	3.32	2.94	3.87	2.77	2.77	3.32	4.1	2.46
Labor Cost	113.77	82.72	76.96	159.9675	54.78	100.8	114	118.9	353.8	78.59	118.9	116.56	98.84	246	102.3	588	101.7	125.6	83	88.2	58.05	96.95	55.4	83	82	49.2
Unit Equipment Cost	1.95	1.78	1.58	1.98	1.76	1.79	1.59	2.15	1.74	1.93	2.15	1.78	1.88	2.15	1.81	2.08	3.42	1.67	1.76	1.56	2.06	1.98	1.98	1.76	2.15	1.76
Equipment Cost	60.45	39.16	41.08	114.345	29.04	53.7	60.42	62.35	252.3	55.97	62.35	55.18	52.64	129	54.3	312	102.6	66.8	44	46.8	30.9	69.3	39.6	44	43	35.2
Total Item Cost	2220.22	2981.88	4148.04	2382.188	1280.07	5314.5	7584.42	2037.25	6841.1	1613.56	2037.25	4201.74	5779.48	4215	3216.6	16950	6144.3	12992.4	2189.5	5475	1326.45	1268.75	825	1939.5	1405	944.4
Total Level Cost :																										

1st level

beam type	W21X44	W27X84	W27X114	W24X76	W24X84	W18X35	W21X44	W24X94	W21X55	W21X62	W18X40	W16X31	W21X62	W18X35	W21X50	W18X40	W21X68	W12X19	W18X40	W21X44	W21X62	W21X50	W24X84	W21X55	W24X146	W18X35
length (ft)	116	30	60	30	30	90	195	39	27	19	135	135	60	150	60	30	30	25	25	27	27	27	27	16.5	30	16.5
beam weight lbs/ft	44	84	114	76	84	35	44	94	55	62	40	31	62	35	50	40	68	19	40	44	62	50	84	55	146	35
beam weight (lbs)	5104	2520	6840	2280	2520	3150	8580	3666	1485	1178	5400	4185	3720	5250	3000	1200	2040	475	1000	1188	1674	1350	2268	907.5	4380	577.5
total:																										
Unit Mat'l Cost	72.5	139	188	130	139	64	72.5	155	102	102	66	51	102	64	82.5	66	112	31.5	66	72.5	102	82.5	139	102	241	64
Mat'l Cost	8410	4170	11280	3900	4170	5760	14137.5	6045	2754	1938	8910	6885	6120	9600	4950	1980	3360	787.5	1650	1957.5	2754	2227.5	3753	1683	7230	1056
Unit Labor Cost	3.32	2.96	3.07	3.76	3.27	4.1	3.32	3.27	3.41	3.41	3.67	2.71	3.41	4.1	3.32	3.67	3.41	2.77	3.67	3.32	3.41	3.32	3.27	3.41	3.36	4.1
Labor Cost	385.12	88.8	184.2	112.8	98.1	369	647.4	127.53	92.07	64.79	495.45	365.85	204.6	615	199.2	110.1	102.3	69.25	91.75	89.64	92.07	89.64	88.29	56.265	100.8	67.65
Unit Equipment Cost	1.76	1.58	1.63	1.78	1.74	2.15	1.76	1.74	1.81	1.81	1.95	1.93	1.81	2.15	1.76	1.95	1.81	1.98	1.95	1.76	1.81	1.76	1.74	1.81	1.78	2.15
Equipment Cost	204.16	47.4	97.8	53.4	52.2	193.5	343.2	67.86	48.87	34.39	263.25	260.55	108.6	322.5	105.6	58.5	54.3	49.5	48.75	47.52	48.87	47.52	46.98	29.865	53.4	35.475
Total Item Cost	8999.28	4306.2	11562	4066.2	4320.3	6322.5	15128.1	6240.39	2894.94	2037.18	9668.7	7511.4	6433.2	10537.5	5254.8	2148.6	3516.6	906.25	1790.5	2094.66	2894.94	2364.66	3888.27	1769.13	7384.2	1159.125
Total Level Cost :																										

Total Item Cost

Total Level Cost :

2nd level

beam type	w18x35	w18x65	w18x60	w18x76	w24x55	w21x101	w18x86	w24x68	w24x76	w27x102	w27x84	w27x336	w18x35	w18x60	w18x46
length (ft)	16.5	30	30	30	30	30	30	120	30	30	30	30	28	40	40
beam weight lbs/ft	35	65	60	76	55	101	86	68	76	102	84	336	35	60	46
beam weight (lbs)	577.5	1950	1800	2280	1650	3030	2580	8160	2280	3060	2520	10080	980	2400	1840
total:															
Unit Mat'l Cost	64	107	107	125	91	167	142	112	130	188	139	266	64	107	76
Mat'l Cost	1056	3210	3210	3750	2730	5010	4260	13440	3900	5640	4170	7980	1792	4280	3040
Unit Labor Cost	4.1	3.92	3.92	3.92	3.18	3.53	3.92	3.18	3.76	3.07	2.96	3.07	4.1	3.92	3.67
Labor Cost	67.65	117.6	117.6	117.6	95.4	105.9	117.6	381.6	112.8	92.1	88.8	92.1	114.8	156.8	146.8
Unit Equipment Cost	2.15	2.08	2.08	2.08	1.69	1.88	2.08	1.69	1.78	1.63	1.58	1.63	2.15	2.08	1.95
Equipment Cost	35.475	62.4	62.4	62.4	50.7	56.4	62.4	202.8	53.4	48.9	47.4	48.9	60.2	83.2	78
Total Item Cost	1159.125	3390	3390	3930	2876.1	5172.3	4440	14024.4	4066.2	5781	4306.2	8121	1967	4520	3264.8
Total Level Cost :															

1st level

beam type	W27X114	W16X26	W12X19	W14X22	W18X40	W18X50	W14X22	W18X60	W14X22	W21X62	W12X19	W12X16	W12X19	W24X76
length (ft)	30	15	30	60	20	30	12	20	20	20	40	20	40	30
beam weight lbs/ft	114	26	19	22	40	50	22	60	22	62	19	16	19	76
beam weight (lbs)	3420	390	570	1320	800	1500	264	1200	440	1240	760	320	760	2280
total:														
Unit Mat'l Cost	188	43	31.5	43	66	82.5	43	107	43	102	31.5	26.5	31.5	130
Mat'l Cost	5640	645	945	2580	1320	2475	516	2140	860	2040	1260	530	1260	3900
Unit Labor Cost	3.07	2.44	2.77	2.46	3.67	3.87	2.46	3.92	2.46	3.41	2.77	2.77	2.77	3.76
Labor Cost	92.1	36.6	83.1	147.6	73.4	116.1	29.52	78.4	49.2	68.2	110.8	55.4	110.8	112.8
Unit Equipment Cost	1.63	1.74	1.98	1.76	1.95	2.06	1.76	2.08	1.76	1.81	1.98	1.98	1.98	1.78
Equipment Cost	48.9	26.1	59.4	105.6	39	61.8	21.12	41.6	35.2	36.2	79.2	39.6	79.2	53.4
Total Item Cost	5781	707.7	1087.5	2833.2	1432.4	2652.9	566.64	2260	944.4	2144.4	1450	625	1450	4066.2
Total Level Cost :														

level1

column type	w14x99	w14x145	w14x90	w14x82	w14x109	w14x193	w14x159	w14x90	w14x68	w14x90	w14x233	w14x159	w14x90
height (ft)	20.833	23.417	23.417	21.167	23.417	36.333	36.333	21.5833	21.5833	23.4167	36.33333	37.33	33.33
column weight lbs/ft	99	145	90	82	109	193	159	90	68	90	233	159	90
column weight (lbs)	2062.467	3395.465	2107.53	1735.694	2552.453	7012.269	5776.947	1942.497	1467.664	2107.503	8465.667	5935.47	2999.7
total:	236557.2	lbs											
Unit Mat'l Cost	198	198	149	149	198	198	198	149	122	149	198	198	149
Mat'l Cost	4124.934	4636.566	3489.133	3153.883	4636.566	7193.934	7193.934	3215.912	2633.163	3489.088	7194	7391.34	4966.17
Unit Labor Cost	3.39	3.39	3.3	3.3	3.39	3.39	3.39	3.3	3.21	3.3	3.39	3.39	3.3
Labor Cost	70.62387	79.38363	77.2761	69.8511	79.38363	123.1689	123.1689	71.22489	69.28239	77.27511	123.17	126.5487	109.989
Unit Equipment Cost	2.42	2.42	2.35	2.35	2.42	2.42	2.42	2.35	2.29	2.35	2.42	2.42	2.35
Equipment Cost	50.41586	56.66914	55.02995	49.74245	56.66914	87.92586	87.92586	50.72076	49.42576	55.02925	87.92667	90.3386	78.3255
Total Item Cost	4245.974	4772.619	3621.439	3273.477	4772.619	7405.029	7405.029	3337.857	2751.871	3621.393	7405.097	7608.227	5154.485
Total Level Cost :	251485.8												

column type	w14x90	w14x90	w14x193	w14x132	w14x99	w14x120	w14x82	w14x193	w14x145	w14x99	w14x120	w14x90	w14x211
height (ft)	33.33	19.4167	37.33	37.33	33.33	33.33	19.4167	37.33	37.33	33.33	33.33	20.4167	37.33
column weight lbs/ft	90	90	193	132	99	120	82	193	145	99	120	90	211
column weight (lbs)	2999.7	1747.503	7204.69	4927.56	3299.67	3999.6	1592.169	7204.69	5412.85	3299.67	3999.6	1837.503	7876.63
Unit Mat'l Cost	149	149	198	198	149	198	149	198	198	149	198	149	198
Mat'l Cost	4966.17	2893.088	7391.34	7391.34	4966.17	6599.34	2893.088	7391.34	7391.34	4966.17	6599.34	3042.088	7391.34
Unit Labor Cost	3.3	3.3	3.39	3.39	3.3	3.39	3.3	3.39	3.39	3.3	3.39	3.3	3.39
Labor Cost	109.989	64.07511	126.5487	126.5487	109.989	112.9887	64.07511	126.5487	126.5487	109.989	112.9887	67.37511	126.5487
Unit Equipment Cost	2.35	2.35	2.42	2.42	2.35	2.42	2.35	2.42	2.42	2.35	2.42	2.35	2.42
Equipment Cost	78.3255	45.62925	90.3386	90.3386	78.3255	80.6586	45.62925	90.3386	90.3386	78.3255	80.6586	47.97925	90.3386
Total Item Cost	5154.485	3002.793	7608.227	7608.227	5154.485	6792.987	3002.793	7608.227	7608.227	5154.485	6792.987	3157.443	7608.227

level2 & 3

column type	w14x74	w14x99	w14x68	w14x82	w14x109	w14x120	w14x90	w14x176	w14x99	w14x90	w14x120	w14x90	w14x61
height (ft)	31	31	31	31	31	31	27	31	31	31	31	31	35
column weight lbs/ft	74	99	68	82	109	120	90	176	99	90	120	90	61
column weight (lbs)	2294	3069	2108	2542	3379	3720	2430	5456	3069	2790	3720	2790	2135
total :	153633	lbs											
Unit Mat'l Cost	122	198	122	149	198	198	149	198	198	149	198	149	122
Mat'l Cost	3782	6138	3782	4619	6138	6138	4023	6138	6138	4619	6138	4619	4270
Unit Labor Cost	3.21	3.39	3.21	3.3	3.39	3.39	3.3	3.39	3.39	3.3	3.39	3.3	3.21
Labor Cost	99.51	105.09	99.51	102.3	105.09	105.09	89.1	105.09	105.09	102.3	105.09	102.3	112.35
Unit Equipment Cost	2.29	2.42	2.29	2.25	2.42	2.42	2.25	2.42	2.42	2.25	2.42	2.25	2.29
Equipment Cost	70.99	75.02	70.99	69.75	75.02	75.02	60.75	75.02	75.02	69.75	75.02	69.75	80.15
Total Item Cost	3952.5	6318.11	3952.5	4791.05	6318.11	6318.11	4172.85	6318.11	6318.11	4791.05	6318.11	4791.05	4462.5
Total Level Cost :	196564.7												

column type	w14x109	w14x90	w14x61	w14x109	w14x90	w14x99	w14x132	w14x159	w14x99	w14x176	w14x90	w14x233	w14x159
height (ft)	31	31	35	17	14	31	31	31	31	18	13	31	35
column weight lbs/ft	109	90	61	109	90	99	132	159	99	176	90	233	159
column weight (lbs)	3379	2790	2135	1853	1260	3069	4092	4929	3069	3168	1170	7223	5565
Unit Mat'l Cost	198	149	122	198	149	198	198	198	198	198	149	198	198
Mat'l Cost	6138	4619	4270	3366	2086	6138	6138	6138	6138	3564	1937	6138	6930
Unit Labor Cost	3.39	3.3	3.21	3.39	3.3	3.39	3.39	3.39	3.39	3.39	3.3	3.39	3.39
Labor Cost	105.09	102.3	112.35	57.63	46.2	105.09	105.09	105.09	105.09	61.02	42.9	105.09	118.65
Unit Equipment Cost	2.42	2.25	2.29	2.42	2.25	2.42	2.42	2.42	2.42	2.42	2.25	2.42	2.42
Equipment Cost	75.02	69.75	80.15	41.14	31.5	75.02	75.02	75.02	75.02	43.56	29.25	75.02	84.7
Total Item Cost	6318.11	4791.05	4462.5	3464.77	2163.7	6318.11	6318.11	6318.11	6318.11	3668.58	2009.15	6318.11	7133.35

level4

column type	w14x53	w14x68	w14x68	w14x68	w14x90	w14x120	w14x120	w14x99	w14x48	w14x48	w14x90	w14x90	w14x53
height (ft)	10	24	10	10	24	10	24	10	28	28	24	10	10
column weight lbs/ft	53	68	68	68	90	120	120	99	48	48	90	90	53
column weight (lbs)	530	1632	680	680	2160	1200	2880	990	1344	1344	2160	900	530
total :	86174	lbs											
Unit Mat'l Cost	87.5	122	122	122	149	198	198	198	87.5	87.5	149	149	87.5
Mat'l Cost	875	2928	1220	1220	3576	1980	4752	1980	2450	2450	3576	1490	875
Unit Labor Cost	3.05	3.21	3.21	3.21	3.3	3.39	3.39	3.39	3.05	3.05	3.3	3.3	3.05
Labor Cost	30.5	77.04	32.1	32.1	79.2	33.9	81.36	33.9	85.4	85.4	79.2	33	30.5
Unit Equipment Cost	2.18	2.29	2.29	2.29	2.25	2.42	2.42	2.42	2.18	2.18	2.25	2.25	2.18
Equipment Cost	21.8	54.96	22.9	22.9	54	24.2	58.08	24.2	61.04	61.04	54	22.5	21.8
Total Item Cost	927.3	3060	1275	1275	3709.2	2038.1	4891.44	2038.1	2596.44	2596.44	3709.2	1545.5	927.3
Total Level Cost :	105604.4												

column type	w14x68	w14x68	w14x61	w14x61	w14x74	w14x68	w14x120	w14x99	w14x176	w14x233	w14x159	w14x311	w14x283
height (ft)	24	10	10	24	24	10	28	28	10	24	10	28	28
column weight lbs/ft	68	68	61	61	74	68	120	99	176	233	159	311	311
column weight (lbs)	1632	680	610	1464	1776	680	3360	2772	1760	5592	1590	8708	8708
Unit Mat'l Cost	122	122	122	122	122	122	198	198	198	198	198	198	198
Mat'l Cost	2928	1220	1220	2928	2928	1220	5544	5544	1980	4752	1980	5544	5544
Unit Labor Cost	3.21	3.21	3.21	3.21	3.21	3.21	3.39	3.39	3.39	3.39	3.39	3.39	3.39
Labor Cost	77.04	32.1	32.1	77.04	77.04	32.1	94.92	94.92	33.9	81.36	33.9	94.92	94.92
Unit Equipment Cost	2.29	2.29	2.29	2.29	2.29	2.29	2.42	2.42	2.42	2.42	2.42	2.42	2.42
Equipment Cost	54.96	22.9	22.9	54.96	54.96	22.9	67.76	67.76	24.2	58.08	24.2	67.76	67.76
Total Item Cost	3060	1275	1275	3060	3060	1275	5706.68	5706.68	2038.1	4891.44	2038.1	5706.68	5706.68

level5

column type	w14x61	w14x61	w14x43	w14x61	w14x61	w14x43	w14x48	w14x43	w14x43	w14x43
height (ft)	14	14	14	14	14	14	14	14	14	14
column weight lbs/ft	61	61	43	61	61	43	48	43	43	43
column weight (lbs)	854	854	602	854	854	602	672	602	602	602
total :	16940	lbs								
Unit Mat'l Cost	122	122	71	122	122	71	122	71	71	71
Mat'l Cost	1708	1708	994	1708	1708	994	1708	994	994	994
Unit Labor Cost	3.21	3.21	3.01	3.21	3.21	3.01	3.21	3.01	3.01	3.01
Labor Cost	44.94	44.94	42.14	44.94	44.94	42.14	44.94	42.14	42.14	42.14
Unit Equipment Cost	2.29	2.29	2.15	2.29	2.29	2.15	2.29	2.15	2.15	2.15
Equipment Cost	32.06	32.06	30.1	32.06	32.06	30.1	32.06	30.1	30.1	30.1
Total Item Cost	1785	1785	1066.24	1785	1785	1066.24	1785	1066.24	1066.24	1066.24
Total Level Cost :	33149.34									

column type	w14x61	w14x43	w14x68	w14x82	w14x74	w14x61	w14x61	w14x61	w14x74
height (ft)	14	14	14	18	18	18	18	18	18
column weight lbs/ft	61	43	68	82	74	61	61	61	74
column weight (lbs)	854	602	952	1476	1332	1098	1098	1098	1332
Unit Mat'l Cost	122	71	122	149	122	122	122	122	122
Mat'l Cost	1708	994	1708	2682	2196	2196	2196	2196	2196
Unit Labor Cost	3.21	3.01	3.21	3.3	3.21	3.21	3.21	3.21	3.21
Labor Cost	44.94	42.14	44.94	59.4	57.78	57.78	57.78	57.78	57.78
Unit Equipment Cost	2.29	2.15	2.29	2.25	2.29	2.29	2.29	2.29	2.29
Equipment Cost	32.06	30.1	32.06	40.5	41.22	41.22	41.22	41.22	41.22
Total Item Cost	1785	1066.24	1785	2781.9	2295	2295	2295	2295	2295

level6 & 7

column type	w14x109	w14x90	w14x99	w14x68	w14x61	w14x311	w14x283	w14x99	w14x48	w14x233	w14x233	w14x61	w14x48	w14x61	w14x61

level1										
column type	w14x159	w14x211	w14x90	w14x90	w14x211	w14x132	w14x90	w14x74	w14x132	w14x109
height (ft)	37.33	37.33	33.33	33.33	37.33	25.5833	21.5833	21.5833	25.9167	17.25
column weight lbs/ft	159	211	90	90	211	132	90	74	132	109
column weight (lbs)	5935.47	7876.63	2999.7	2999.7	7876.63	3376.996	1942.497	1597.164	3421.004	1880.25
total:										
Unit Mat'l Cost	198	198	149	149	198	198	149	122	198	198
Mat'l Cost	7391.34	7391.34	4966.17	4966.17	7391.34	5065.493	3215.912	2633.163	5131.507	3415.5
Unit Labor Cost	3.39	3.39	3.3	3.3	3.39	3.39	3.3	3.21	3.39	3.39
Labor Cost	126.5487	126.5487	109.989	109.989	126.5487	86.72739	71.22489	69.28239	87.85761	58.4775
Unit Equipment Cost	2.42	2.42	2.35	2.35	2.42	2.42	2.35	2.29	2.42	2.42
Equipment Cost	90.3386	90.3386	78.3255	78.3255	90.3386	61.91159	50.72076	49.42576	62.71841	41.745
Total Item Cost	7608.227	7608.227	5154.485	5154.485	7608.227	5214.132	3337.857	2751.871	5282.083	3515.723
Total Level Cost :										

level1 (cont.)									
column type	w14x233	w14x132	w14x398	w14x455	w14x211	w14x132	w14x730	w14x342	w14x159
height (ft)	37.33	21.9167	37.33	37.33	25.5833	22.0833	35.33	35.33	25.5833
column weight lbs/ft	233	132	398	455	211	132	730	342	159
column weight (lbs)	8697.89	2893.004	14857.34	16985.15	5398.076	2914.996	25790.9	12082.86	4067.745
total:									
Unit Mat'l Cost	198	198	198	198	198	198	198	198	198
Mat'l Cost	7391.34	4339.507	7391.34	7391.34	5065.493	4372.493	6995.34	6995.34	5065.493
Unit Labor Cost	3.39	3.39	3.39	3.39	3.39	3.39	3.39	3.39	3.39
Labor Cost	126.5487	74.29761	126.5487	126.5487	86.72739	74.86239	119.7687	119.7687	86.72739
Unit Equipment Cost	2.42	2.42	2.42	2.42	2.42	2.42	2.42	2.42	2.42
Equipment Cost	90.3386	53.03841	90.3386	90.3386	61.91159	53.44159	85.4986	85.4986	61.91159
Total Item Cost	7608.227	4466.843	7608.227	7608.227	5214.132	4500.797	7200.607	7200.607	5214.132
Total Level Cost :									

level2 & 3										
column type	w14x311	w14x283	w14x176	w14x132	w14x550	w14x233	w14x132	w14x61	w14x193	w14x176
height (ft)	31	31	31	35	31	31	31	35	31	31
column weight lbs/ft	311	283	176	132	550	233	132	61	193	176
column weight (lbs)	9641	8773	5456	4620	17050	7223	4092	2135	5983	5456
total :										
Unit Mat'l Cost	198	198	198	198	198	198	198	122	198	198
Mat'l Cost	6138	6138	6138	6930	6138	6138	6138	4270	6138	6138
Unit Labor Cost	3.39	3.39	3.39	3.39	3.39	3.39	3.39	3.21	3.39	3.39
Labor Cost	105.09	105.09	105.09	118.65	105.09	105.09	105.09	112.35	105.09	105.09
Unit Equipment Cost	2.42	2.42	2.42	2.42	2.42	2.42	2.42	2.29	2.42	2.42
Equipment Cost	75.02	75.02	75.02	84.7	75.02	75.02	75.02	80.15	75.02	75.02
Total Item Cost	6318.11	6318.11	6318.11	7133.35	6318.11	6318.11	6318.11	4462.5	6318.11	6318.11
Total Level Cost :										

level4								
column type	w14x132	w14x132	w14x311	w14x233	w14x99	w14x61	w14x193	w14x176
height (ft)	28	10	28	28	28	10	10	10
column weight lbs/ft	311	132	311	233	9	61	193	176
column weight (lbs)	8708	1320	8708	6524	252	610	1930	1760
total :								
Unit Mat'l Cost	198	198	198	198	198	122	198	198
Mat'l Cost	5544	1980	5544	5544	5544	1220	1980	1980
Unit Labor Cost	3.39	3.39	3.39	3.39	3.39	3.21	3.39	3.39
Labor Cost	94.92	33.9	94.92	94.92	94.92	32.1	33.9	33.9
Unit Equipment Cost	2.42	2.42	2.42	2.42	2.42	2.29	2.42	2.42
Equipment Cost	67.76	24.2	67.76	67.76	67.76	22.9	24.2	24.2
Total Item Cost	5706.68	2038.1	5706.68	5706.68	5706.68	1275	2038.1	2038.1
Total Level Cost :								

height (ft)	30	26	26	26	26	26	26	26	26	26	26	26	26	26	26
column weight lbs/ft	109	90	99	68	61	311	283	99	48	233	233	61	48	61	61
column weight (lbs)	3270	2340	2574	1768	1586	8086	7358	2574	1248	6058	6058	1586	1248	1586	1586
total :	48926	lbs													
Unit Mat'l Cost	198	149	198	122	122	198	198	198	87.5	198	198	122	122	122	122
Mat'l Cost	5940	3874	5148	3172	3172	5148	5148	5148	2275	5148	5148	3172	3172	3172	3172
Unit Labor Cost	3.39	3.3	3.39	3.21	3.21	3.39	3.39	3.39	3.05	3.39	3.39	3.21	3.21	3.21	3.21
Labor Cost	101.7	85.8	88.14	83.46	83.46	88.14	88.14	88.14	79.3	88.14	88.14	83.46	83.46	83.46	83.46
Unit Equipment Cost	2.42	2.25	2.42	2.29	2.29	2.42	2.42	2.42	2.18	2.42	2.42	2.29	2.29	2.29	2.29
Equipment Cost	72.6	58.5	62.92	59.54	59.54	62.92	62.92	62.92	56.68	62.92	62.92	59.54	59.54	59.54	59.54
Total Item Cost	6114.3	4018.3	5299.06	3315	3315	5299.06	5299.06	5299.06	2410.98	5299.06	5299.06	3315	3315	3315	3315
Total Level Cost :	64227.94														

level 8															
column type	w14x109	w14x61	w14x99	w14x43	w14x43	w14x311	w14x176	w14x74	w14x43	w14x90	w14x68	w14x53	w14x43	w14x43	w14x61
height (ft)	10	10	26	10	10	10	10	26	10	10	10	10	10	10	10
column weight lbs/ft	109	61	99	43	43	311	176	74	43	90	68	53	43	43	61
column weight (lbs)	1090	610	2574	430	430	3110	1760	1924	430	900	680	530	430	430	610
total :	15938	lbs													
Unit Mat'l Cost	198	122	198	71	71	198	198	122	71	149	122	87.5	71	71	122
Mat'l Cost	1980	1220	5148	710	710	1980	1980	3172	710	1490	1220	875	710	710	1220
Unit Labor Cost	3.39	3.21	3.39	3.01	3.01	3.39	3.39	3.21	3.01	3.3	3.21	3.05	3.01	3.01	3.21
Labor Cost	33.9	32.1	88.14	30.1	30.1	33.9	33.9	83.46	30.1	33	32.1	30.5	30.1	30.1	32.1
Unit Equipment Cost	2.42	2.29	2.42	2.15	2.15	2.42	2.42	2.29	2.15	2.25	2.29	2.18	2.15	2.15	2.29
Equipment Cost	24.2	22.9	62.92	21.5	21.5	24.2	24.2	59.54	21.5	22.5	22.9	21.8	21.5	21.5	22.9
Total Item Cost	2038.1	1275	5299.06	761.6	761.6	2038.1	2038.1	3315	761.6	1545.5	1275	927.3	761.6	761.6	1275
Total Level Cost :	24834.16														

roof						
column type	w8x35	w8x35	w8x35	w8x40	w8x35	w8x35
height (ft)	16	16	16	16	16	16
column weight lbs/ft	35	35	35	40	35	35
column weight (lbs)	560	560	560	640	560	560
total :	3440	lbs				
Unit Mat'l Cost	58	58	58	79	58	58
Mat'l Cost	928	928	928	1264	928	928
Unit Labor Cost	4.43	4.43	4.43	4.43	4.43	4.43
Labor Cost	70.88	70.88	70.88	70.88	70.88	70.88
Unit Equipment Cost	3.17	3.17	3.17	3.17	3.17	3.17
Equipment Cost	50.72	50.72	50.72	50.72	50.72	50.72
Total Item Cost	1049.6	1049.6	1049.6	1385.6	1049.6	1049.6
Total Level Cost :	6633.6					

Redesign of Gravity System Take-Offs

Column Cost								
size	length	Unit Mat'l Cost	Mat'l Cost	Unit Labor Cost	Labor Cost	Unit Equipment Cost	Equipment Cost	Total Item Cost
w10x33	222.6	54.5	12131.7	4.43	986.118	3.17	705.642	13823.46
w10x39	40.6	81	3288.6	4.43	179.858	3.17	128.702	3597.16
w12x45	40.6	82.5	3349.5	3.25	131.95	2.32	94.192	3575.642
w10x45	40.6	81	3288.6	4.43	179.858	3.17	128.702	3597.16
w14x48	40.6	87.5	3552.5	3.05	123.83	2.18	88.508	3764.838
w10x49	91.6	81	7419.6	4.43	405.788	3.17	290.372	8115.76
w10x54	51	81	4131	4.43	225.93	3.17	161.67	4518.6
w12x65	102	119	12138	3.81	388.62	2.72	277.44	12804.06
w12x79	51	144	7344	3.81	194.31	2.72	138.72	7677.03
w12x87	51	144	7344	3.81	194.31	2.72	138.72	7677.03
w14x90	234.6	149	34955.4	3.3	774.18	2.35	551.31	36280.89
w14x99	153	198	30294	3.39	518.67	2.42	370.26	31182.93
w14x109	143	198	28314	3.39	484.77	2.42	346.06	29144.83
w14x120	299	198	59202	3.39	1013.61	2.42	723.58	60939.19
w14x132	153	198	30294	3.39	518.67	2.42	370.26	31182.93
w14x145	92	198	18216	3.39	311.88	2.42	222.64	18750.52
w14x159	182	198	36036	3.39	616.98	2.42	440.44	37093.42
w14x176	280.2	198	55479.6	3.39	949.878	2.42	678.084	57107.562
w14x211	460	198	91080	3.39	1559.4	2.42	1113.2	93752.6
w14x233	184	198	36432	3.39	623.76	2.42	445.28	37501.04
w14x257	659.2	198	130521.6	3.39	2234.688	2.42	1595.264	134351.552
w14x283	316.6	198	62686.8	3.39	1073.274	2.42	766.172	64526.246
w14x311	224.6	198	44470.8	3.39	761.394	2.42	543.532	45775.726
w14x342	156	198	30888	3.39	528.84	2.42	377.52	31794.36
w14x370	447.6	198	88624.8	3.39	1517.364	2.42	1083.192	91225.356
w14x398	132.6	198	26254.8	3.39	449.514	2.42	320.892	27025.206

Beams Cost								
size	length	Unit Mat'l Cost	Mat'l Cost	Unit Labor Cost	Labor Cost	Unit Equipment Cost	Equipment Cost	Total Item Cost
w8x10	1768.58	16.5	29181.57	4.06	7180.4348	2.9	5128.882	41490.8868
w10x12	577.08	19.8	11426.184	4.06	2342.9448	2.9	1673.532	15442.6608
w10x15	16.5	25	412.5	4.06	66.99	2.9	47.85	527.34
w12x14	352	26.5	9328	2.77	975.04	1.98	696.96	11000
w12x16	235.17	26.5	6232.005	2.77	651.4209	1.98	465.6366	7349.0625
w12x19	139.08	36.5	5076.42	2.77	385.2516	1.98	275.3784	5737.05
w14x22	762.42	43	32784.06	2.46	1875.5532	1.76	1341.8592	36001.4724
w16x26	1131.33	43	48647.19	2.44	2760.4452	1.74	1968.5142	53376.1494
w16x31	1273.58	51	64952.58	2.71	3451.4018	1.93	2458.0094	70861.9912
w16x36	30	66	1980	3.05	91.5	2.18	65.4	2136.9
w18x35	2274.92	58	131945.36	3.67	8348.9564	1.95	4436.094	144730.4104
w18x40	1332.67	66	87956.22	3.67	4890.8989	1.95	2598.7065	95445.8254
w21x44	1605.42	72.5	116392.95	3.32	5329.9944	1.76	2825.5392	124548.4836
w21x48	473	82.5	39022.5	3.32	1570.36	1.76	832.48	41425.34
w21x50	1408.42	82.5	116194.65	3.32	4675.9544	1.76	2478.8192	123349.4236
w21x62	60	102	6120	3.41	204.6	1.81	108.6	6433.2
w24x55	1624.33	91	147814.03	3.18	5165.3694	1.69	2745.1177	155724.5171
w24x62	2045.25	102	208615.5	3.18	6503.895	1.69	3456.4725	218575.8675
w24x68	749.26	112	83917.12	3.18	2382.6468	1.69	1266.2494	87566.0162
w24x76	1232.34	125	154042.5	3.18	3918.8412	1.69	2082.6546	160043.9958
w27x84	951.17	139	132212.63	2.96	2815.4632	1.58	1502.8486	136530.9418
w30x90	349.25	163	56927.75	2.94	1026.795	1.56	544.83	58499.375
w30x99	189.42	163	30875.46	2.94	556.8948	1.56	295.4952	31727.85
w30x108	566	178	100748	2.94	1664.04	1.56	882.96	103295
w30x116	338	191	64558	3.04	1027.52	1.62	547.56	66133.08
w33x118	280	195	54600	3	840	1.59	445.2	55885.2
w33x130	70	215	15050	3.11	217.7	1.65	115.5	15383.2
w36x135	187	223	41701	3.02	564.74	1.6	299.2	42564.94
w36x160	70	281	19670	3.07	214.9	1.63	114.1	19999
w40x149	220	500	110000	3.41	750.2	1.81	398.2	111148.4
w40x167	110	500	55000	3.41	375.1	1.81	199.1	55574.2
w40x215	40	500	20000	3.41	136.4	1.81	72.4	20208.8
w40x211	40	500	20000	3.41	136.4	1.81	72.4	20208.8
w40x324	40	500	20000	3.41	136.4	1.81	72.4	20208.8

Lateral System Core Take-Offs

Original Design				
w14x	quantity	total length	$/ft	total cost
68	1	14	$78.88	$1,104.32
90	1	14	$104.40	$1,461.60
176	1	14	$202.18	$2,830.52
233	4	111	$274.94	$30,518.34
283	3	85	$328.28	$27,903.80
311	4	99	$360.76	$35,715.24
331	1	28	$410.00	$11,480.00
342	1	33	$429.20	$14,163.60
398	1	33	$469.64	$15,498.12
455	1	33	$536.90	$17,717.70
550	1	31	$638.00	$19,778.00
730	1	33	$846.80	$27,944.40
mom connections			$620/conn	$22,320.00
HSS				
5x5x3/8	11	573.1	$65.10	$37,308.81
5.5x5.5x3/8	3	201.4	$72.55	$14,611.57
6x6x3/8	2	137	$79.97	$10,955.89
8x8x3/8	2	94.8	$90.60	$8,588.88
			total:	$299,900.79

New Design				
w14x	quantity	total length	$/ft	total cost
53	1	14	$61.48	$860.72
68	1	26	$78.88	$2,050.88
74	1	14	$87.37	$1,223.18
90	1	26	$104.40	$2,714.40
99	1	26	$114.84	$2,985.84
120	1	14	$138.96	$1,945.44
145	2	62	$168.20	$10,428.40
193	5	148	$223.88	$33,134.24
233	1	33	$274.94	$9,073.02
398	1	33	$469.64	$15,498.12
HSS				
5x5x3/8	11	1146.2	$65.10	$74,617.62
5.5x5.5x3/8	3	402.8	$72.55	$29,223.14
6x6x3/8	2	274	$79.97	$21,911.78
8x8x3/8	2	189.6	$90.60	$17,177.76
			total:	$222,844.54