2010

HUNTER COLLEGE SCHOOL OF SOCIAL WORK

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

Hunter College School of Social Work

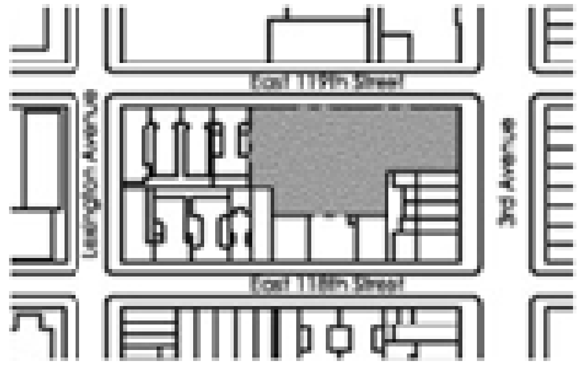
New York, NY

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

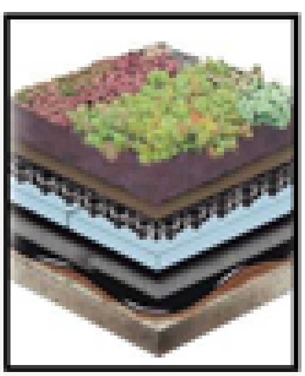
208Y/120V 3- phase, 4 wire electrical system

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.

Table of Contents

Executive Summary 4

Introduction 5

Building Statistics 6

Architecture 7

Building Enclosure 9

Construction 11

Structural 11

Problem Statement 17

Proposed Solutions and Methods 18

Structural Depth Study 20

Code and Design Requirements 20

Building Load Summary 22

Braced Frame Core Design 27

Introduction 27

Design Goals and Assumptions 28

Methodology 29

Graduate Course Integration: Design and detail of the Typical Braced Frame Connection 39

Overturning and Foundation Impact Discussion 43

Center of Rigidity Discussion 44

Structural Depth Summary 47

Comparison between Existing and New Braced Frames 47

Redesign of Façade 49

Thermal Damper and Waterproofing 49

Perimeter Structural Framing Adjustments 51

Redesign of Green Roof 53

Components of the Green Roof 55

Final Green Roof Designs 56

Stormwater Detention Tank Capacity 58

Structural Integrity of Dunnage Base 60

Energy Savings Comparison between Existing and New Roof Plans 61

Cost and Schedule Analysis 63

Cost and Schedule Summary 65

Summary + Conclusions 66

References 67

Credits/ Acknowledgements 68

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.

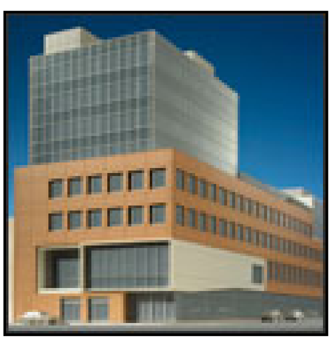


Figure 3: Rendering of the New School

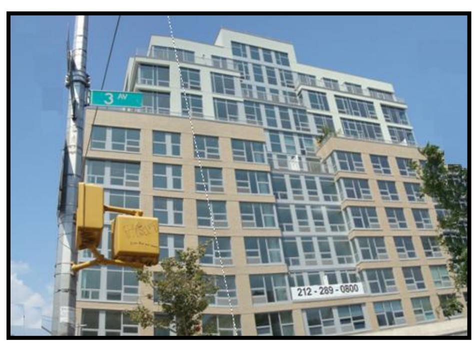


Figure 4: Neighboring Luxury Condominiums

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.

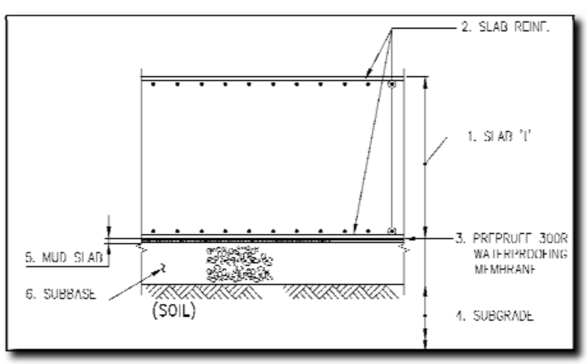


Figure 7: Mat Foundation Detail

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.

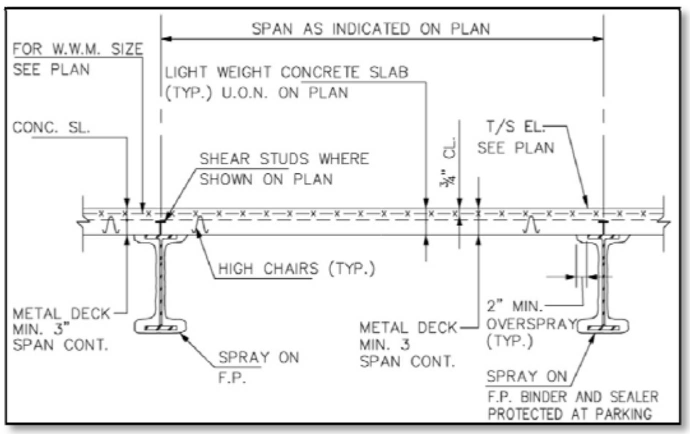


Figure 8: Typical Floor Construction. Metal Deck Perpendicular to Beams or Girders

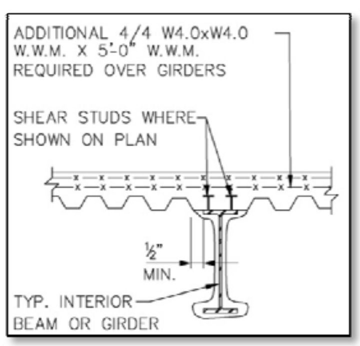


Figure 9: Typical Floor Construction. Metal Deck Parallel to Beams or Girders

Lateral System

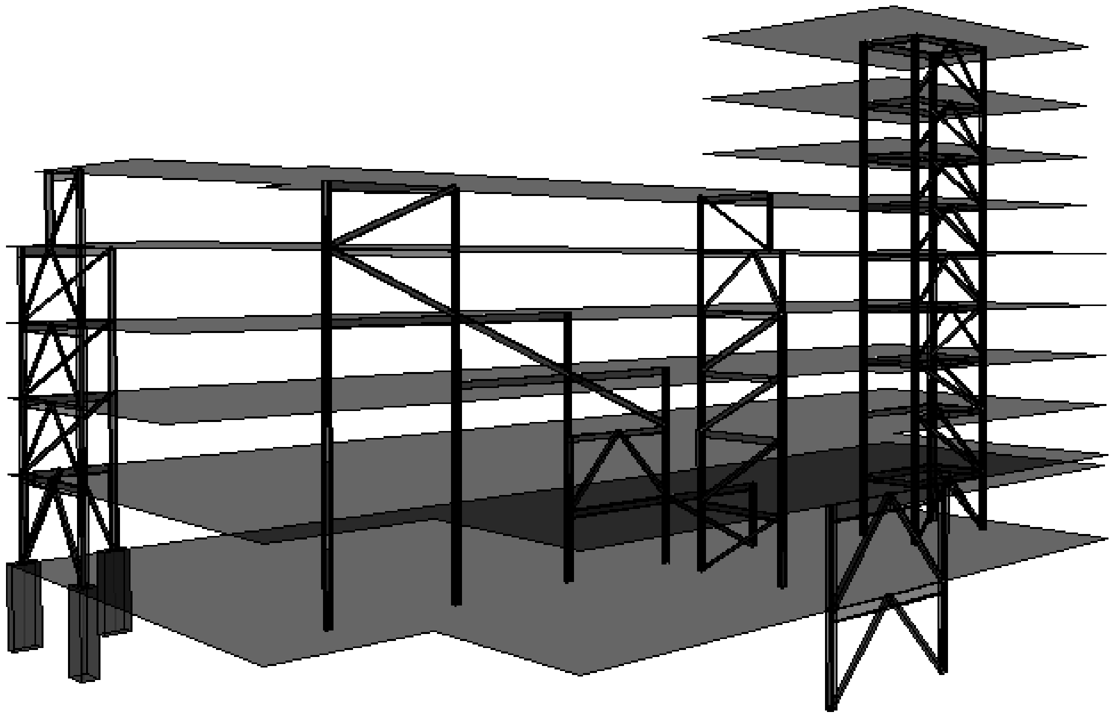


Figure 10: ETABS Model of the Lateral Force Resisting System

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.

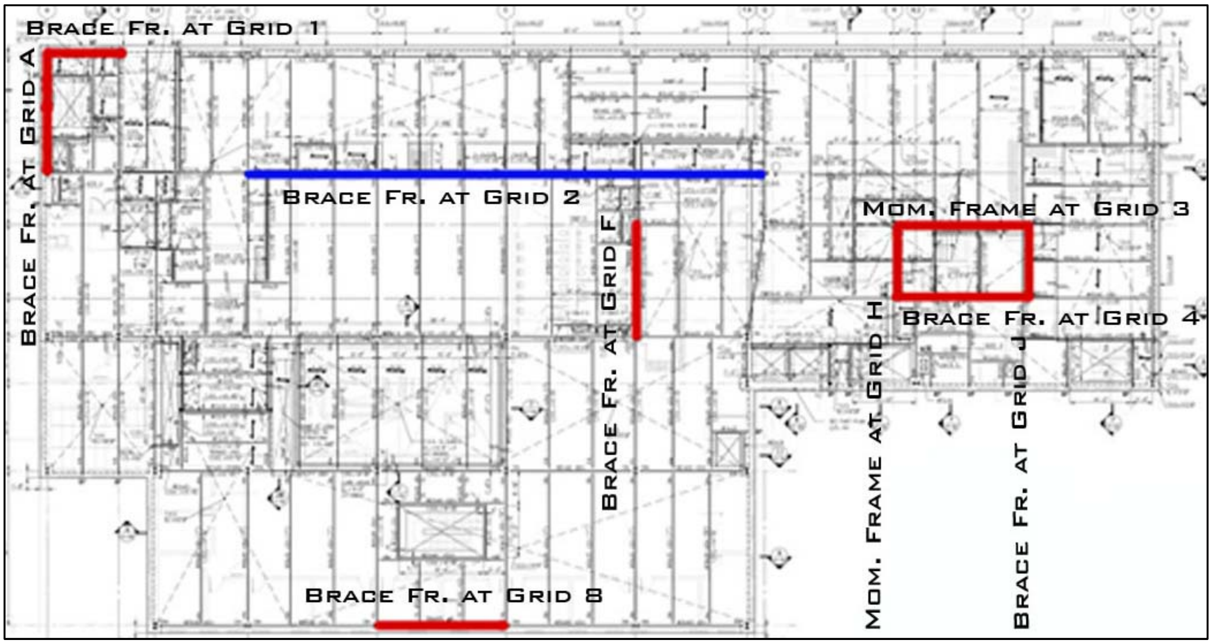
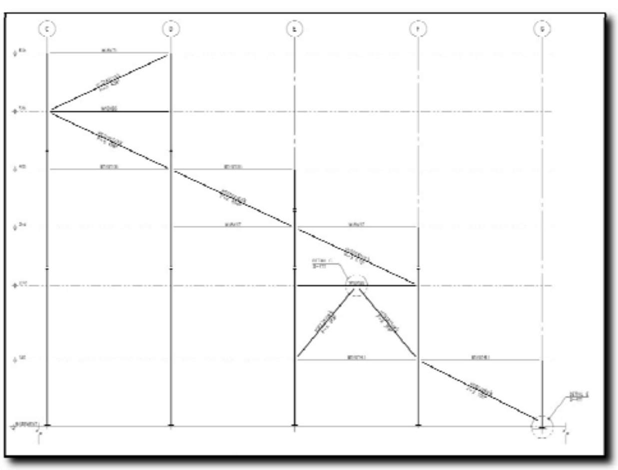


Figure 11: Location of Lateral Force Resisting System



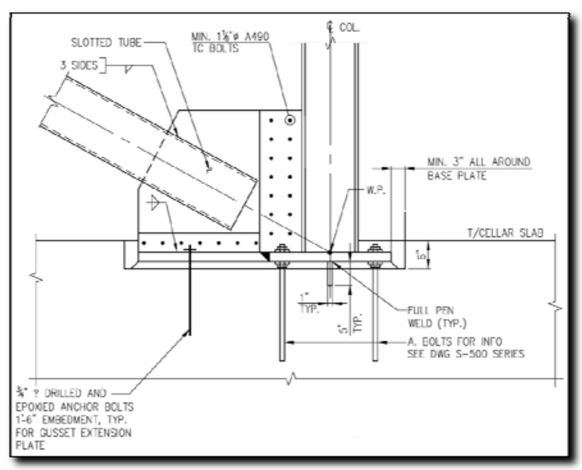


Figure 12: Truss Elevation at Grid 2

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.

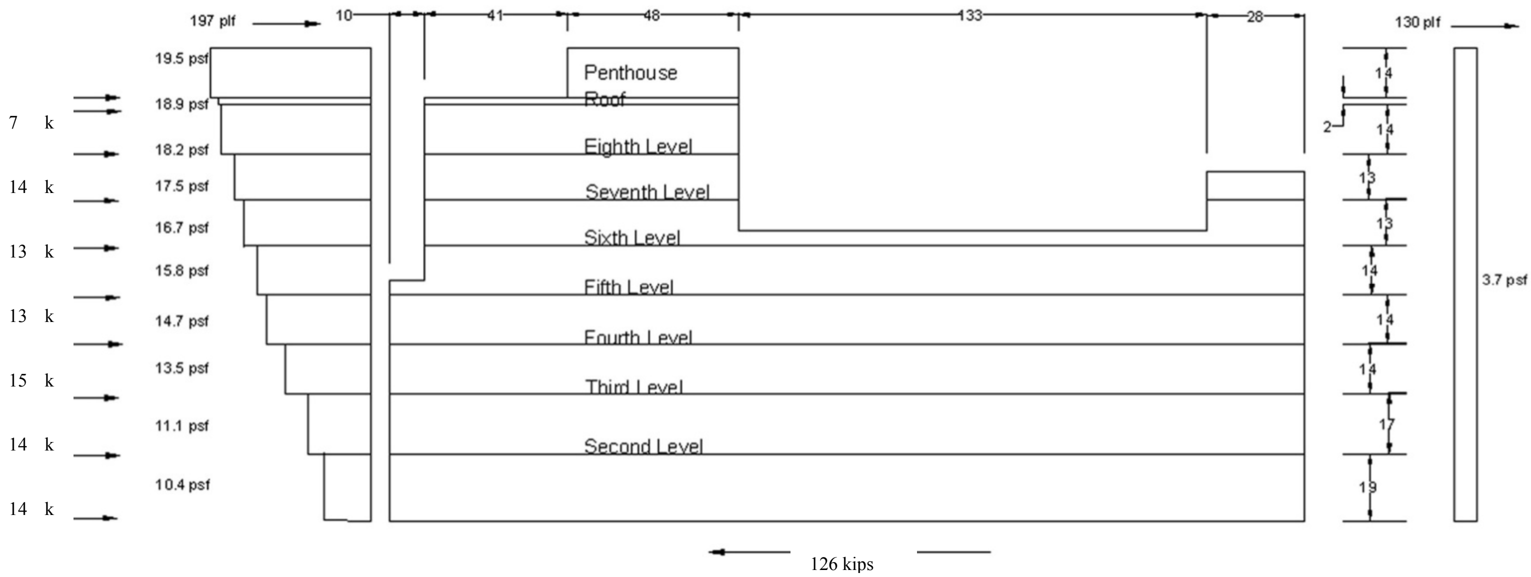


Figure 17: Wind Load Diagram using ASCE 7-05 in East/West winds direction

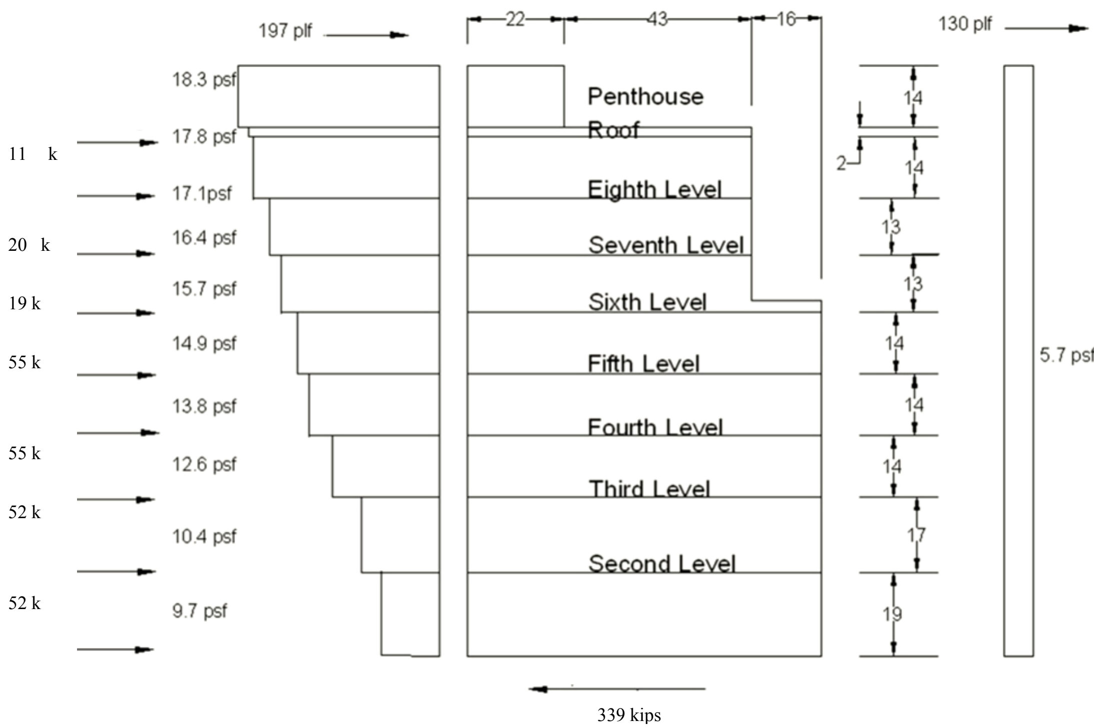


Figure 18: Wind Load Diagram using ASCE 7-05 in North/South winds direction

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.

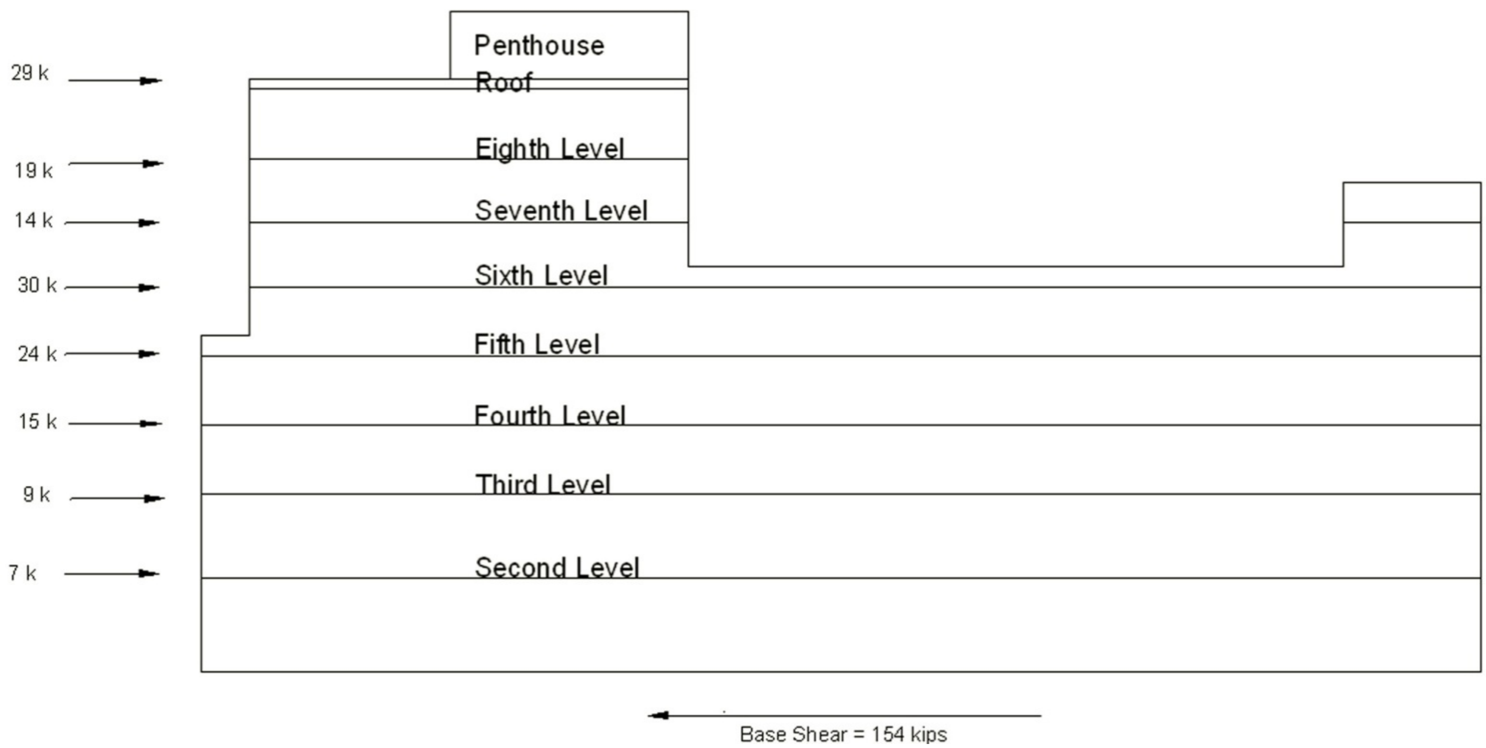


Figure 20: Seismic Force Diagram

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.

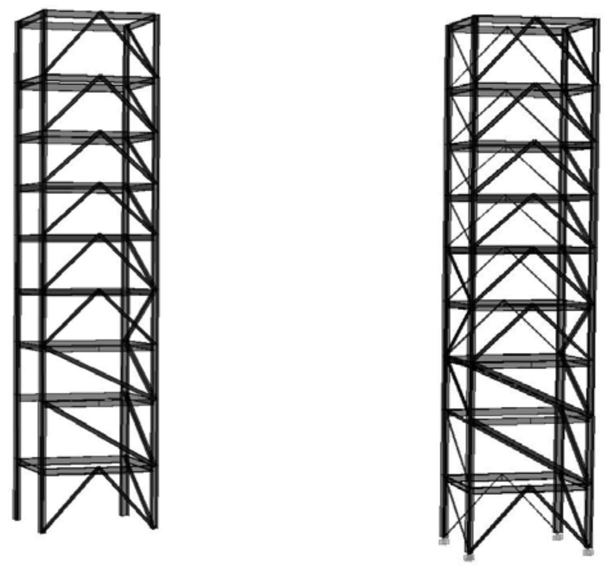


Figure 21: Original design (left) and Redesign (right) of lateral force resisting core

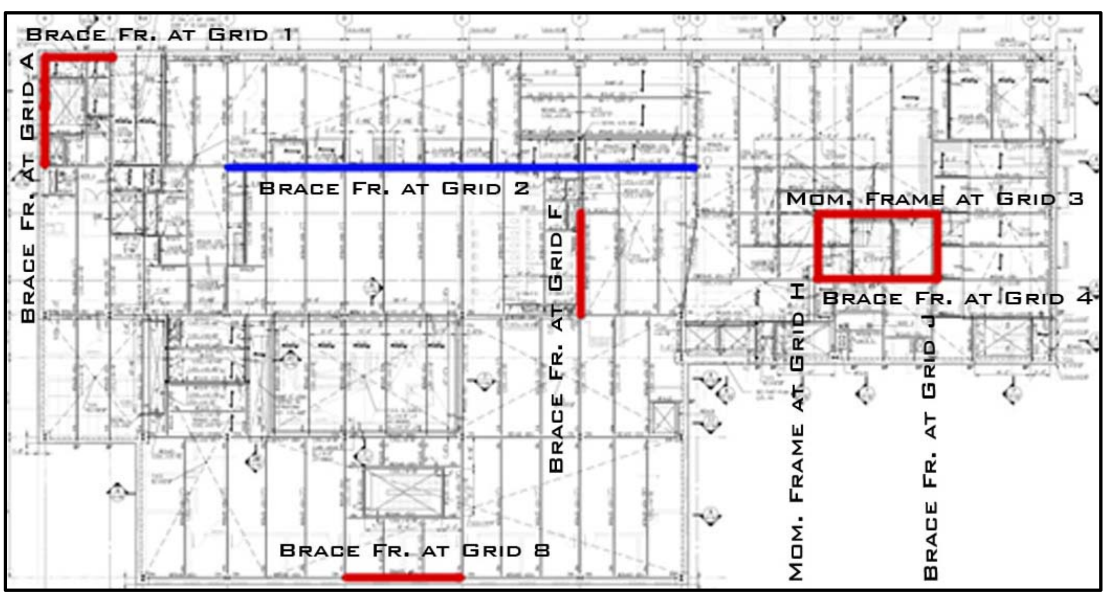


Figure 22: Location of Lateral Force Resisting System, In particular the location of the moment frames

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	

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	

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	

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	

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	

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	

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	

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

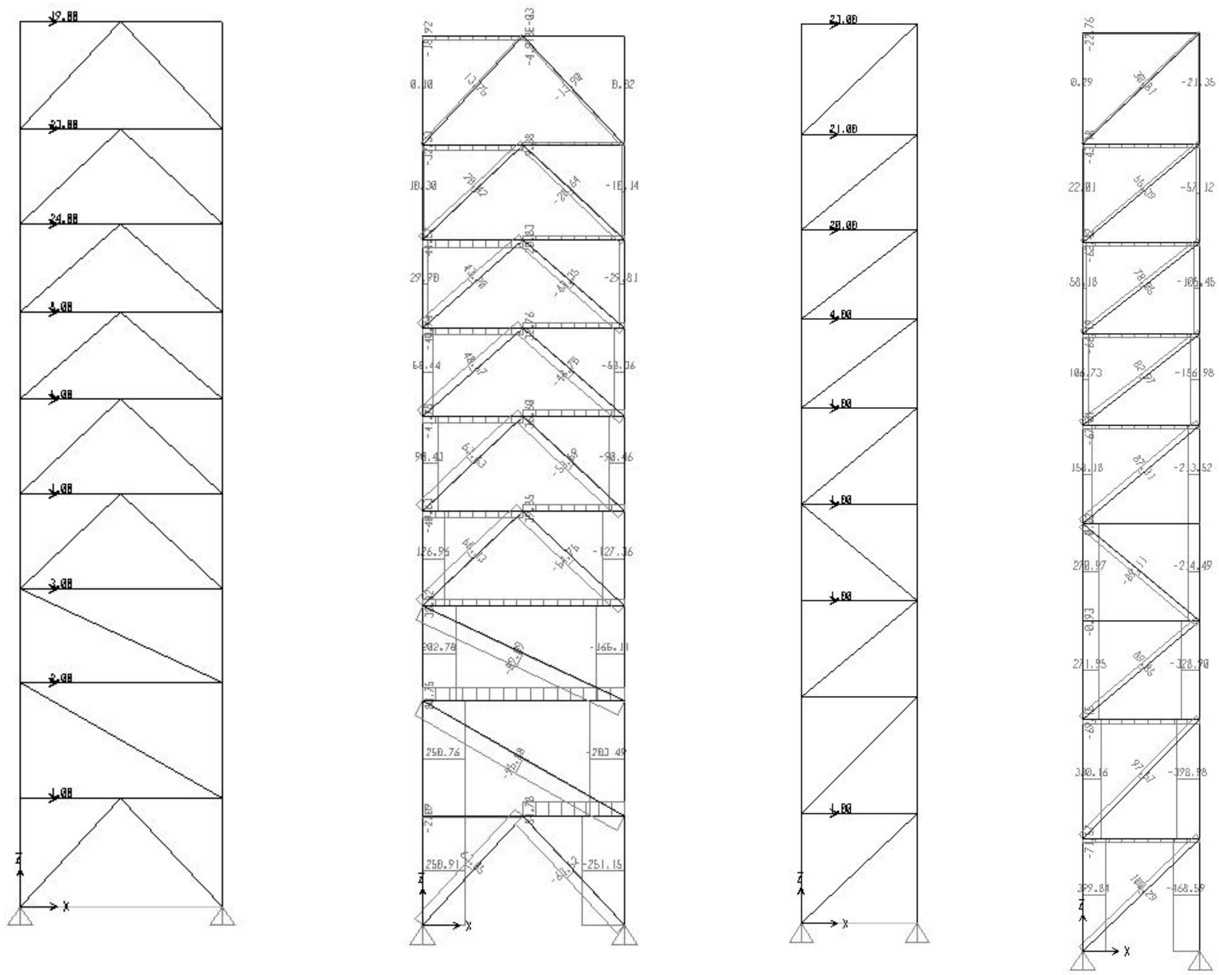


Figure 25: (from left to right) applied forces of frame at grid 3, resulting axial stresses on frame at grid 3, applied forces of frame at grid h, resulting axial stresses on frame at grid h

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.

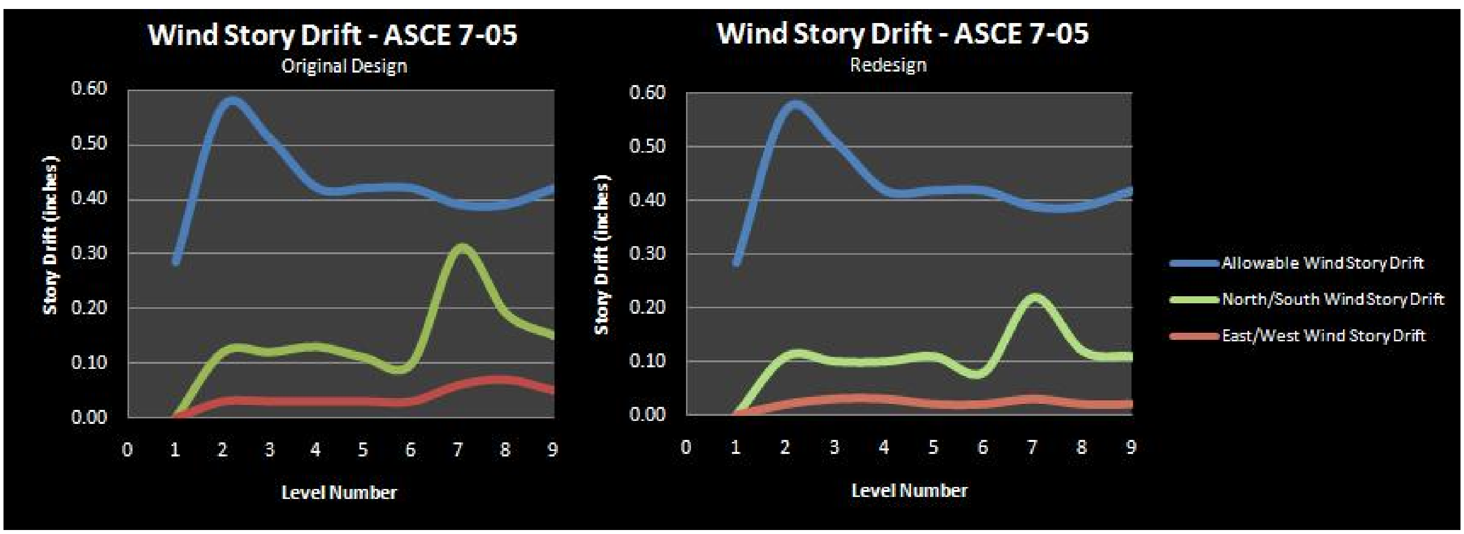


Figure 26: Wind Story Drifts vs. Allowable for the Original Design and New Design

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	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[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

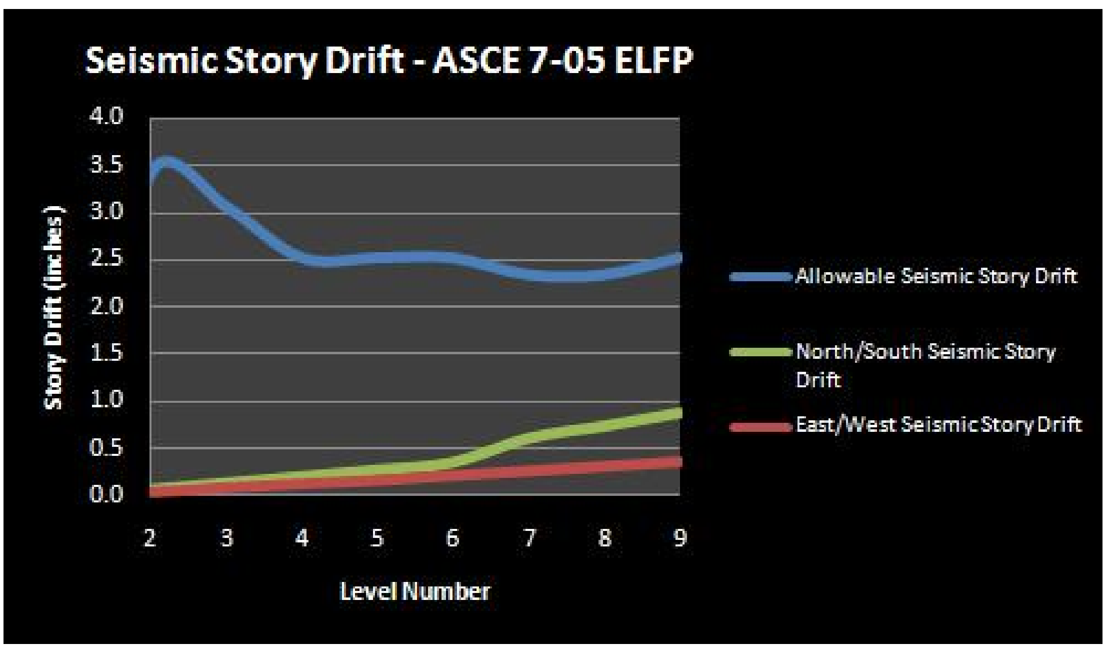


Figure 30: Seismic Drift vs. Allowable Drift

Seismic Drift : East-West Direction							
Floor	Story Height (ft)	inter-story ht	Story Drift (in.)	Allowable Story Drift = 0.015 h_{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 = 0.015 h_{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.

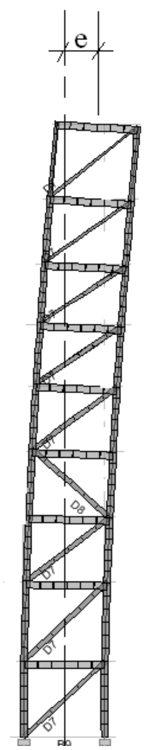
P-delta effects are usually negligable for shorter buildings, they are more important in high-rises. The IBC code allows p-delta effects to be ignored when Θ is less than 0.10. It also imposes a resistriction on secondary effects of $\Theta < 0.25$ deeming the structure unstable. When Θ is between 0.10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

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

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.

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

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.

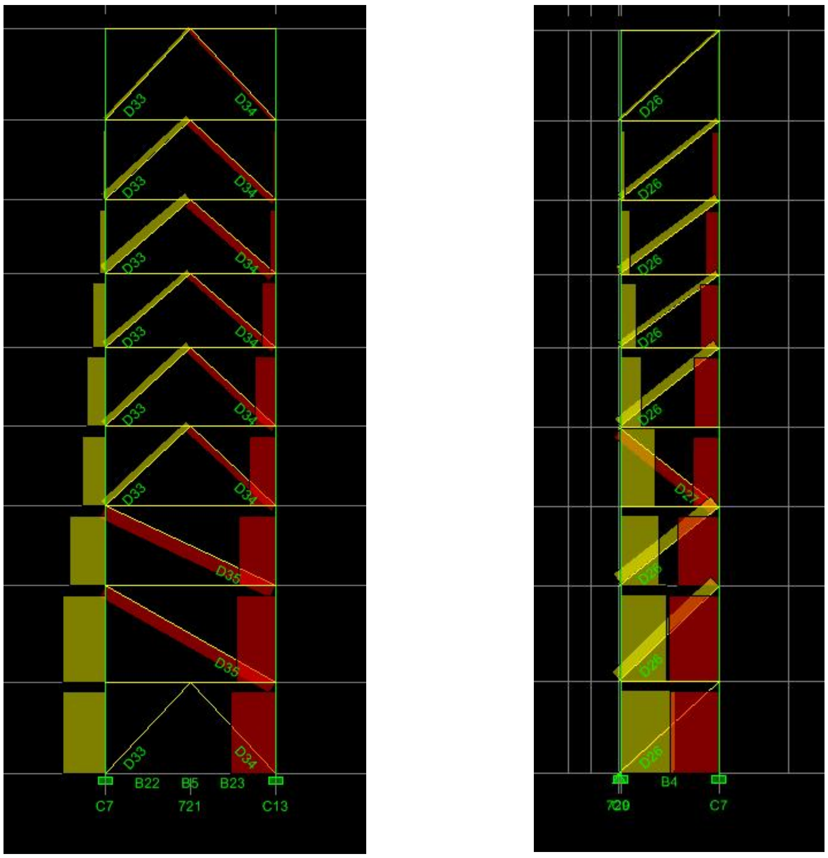
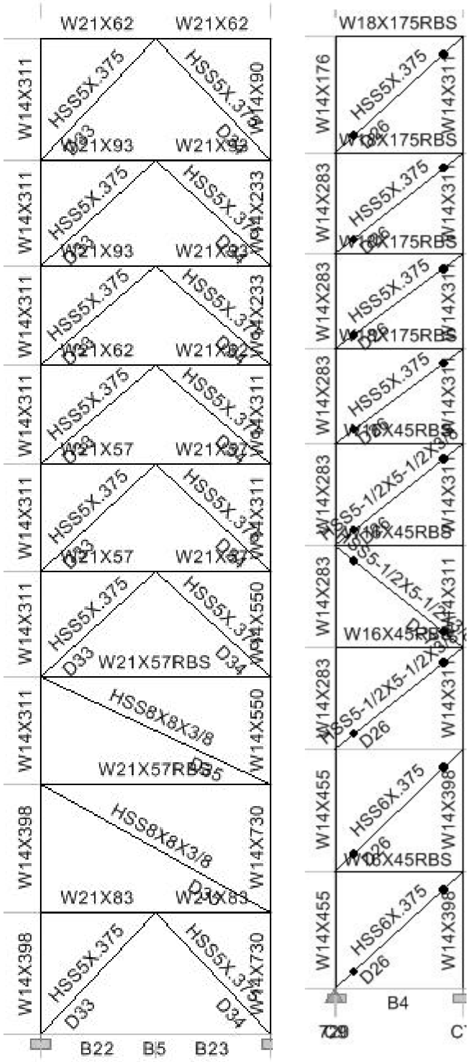


Figure 34: Axial Stresses Fill Diagram from Frames at Grids 3 and H.

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.

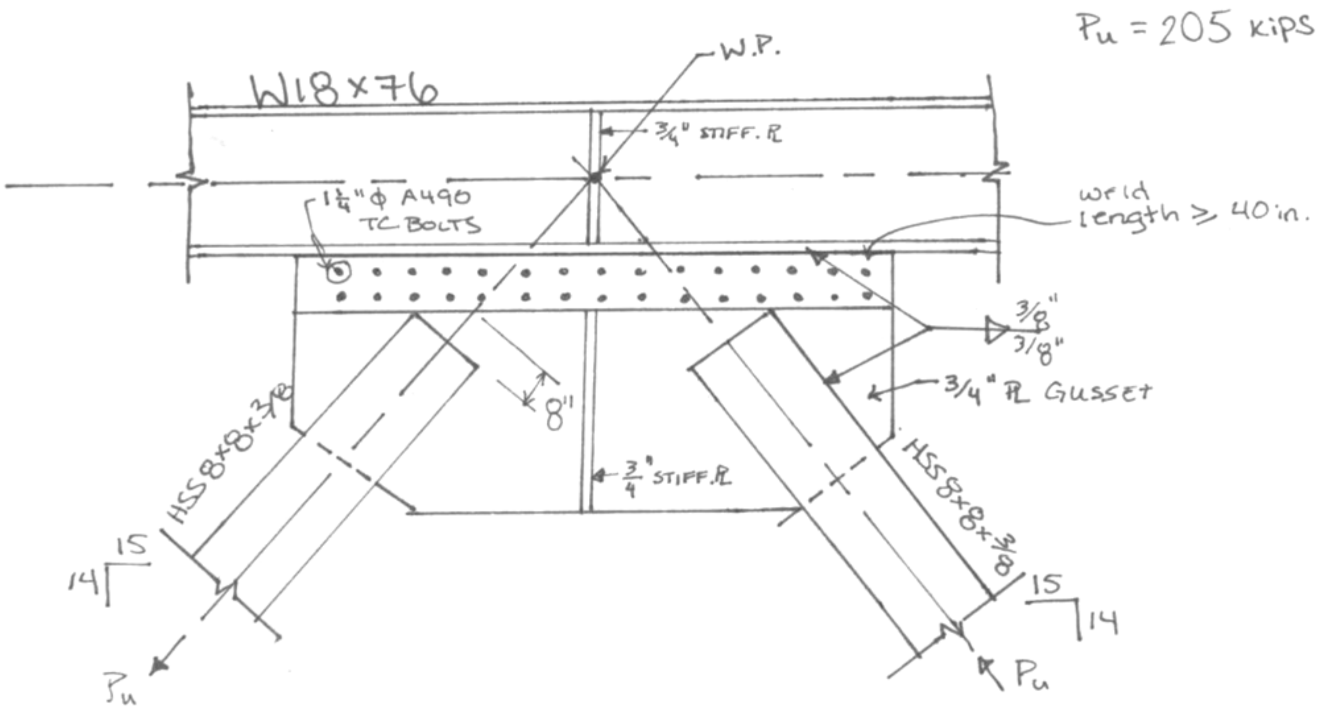


Figure 36: Chevron Connection Design

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.

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

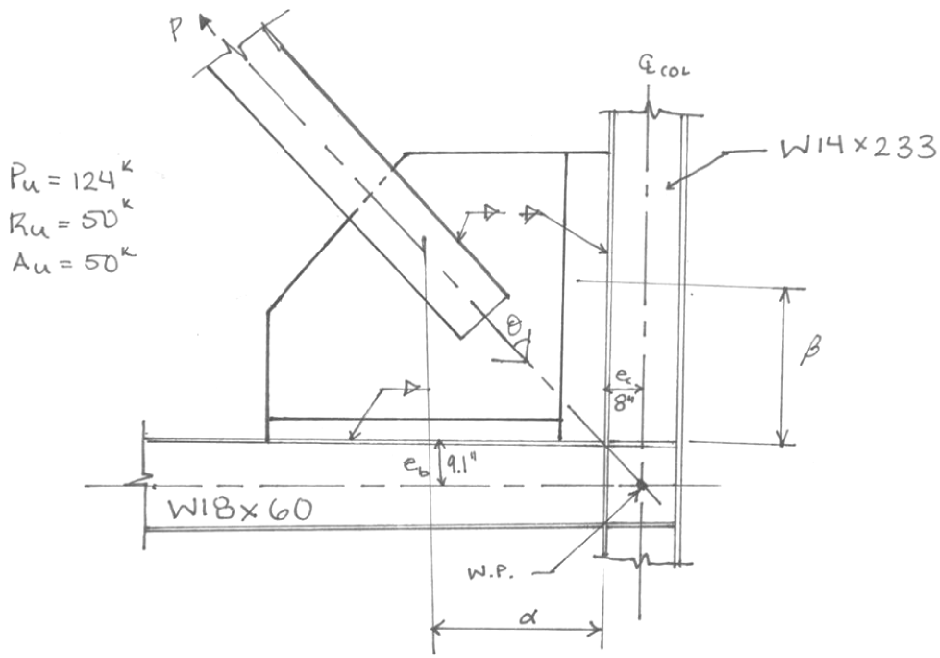


Figure 39: Diagonal Brace Connection

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.

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

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.

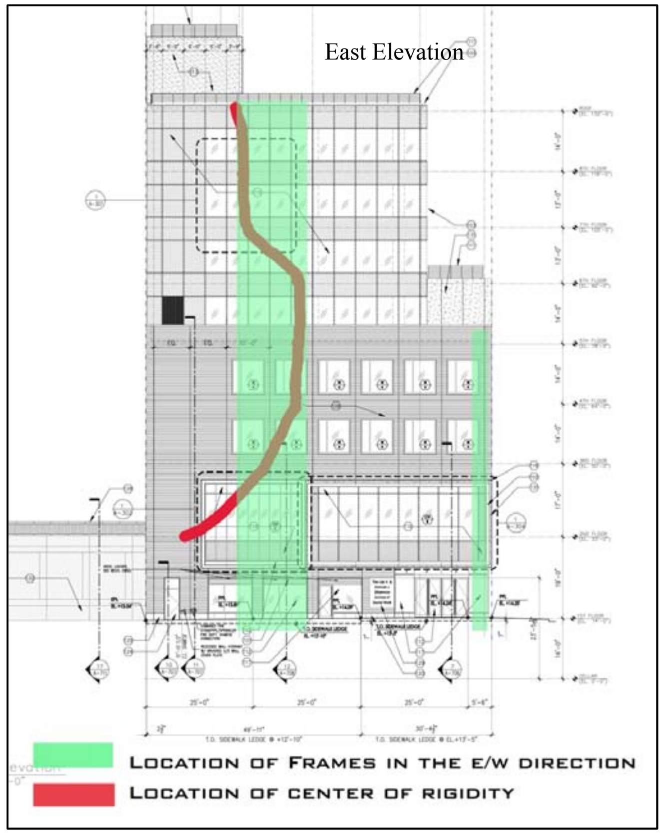
Story four- Approximate COR Check using ETABS relative stiffness values				
Frame (dir)	Load Applied in Diaphragm (kips)	Distribution (kips)	Percentage	Distance to Origin (ft)
1 (E-W)	321	41.00	0.13	132.5
2 (E-W)	321	165.31	0.51	104.5
3 (E-W)	321	10.54	0.03	92.5
4 (E-W)	321	103.01	0.32	75.4
8 (E-W)	321	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



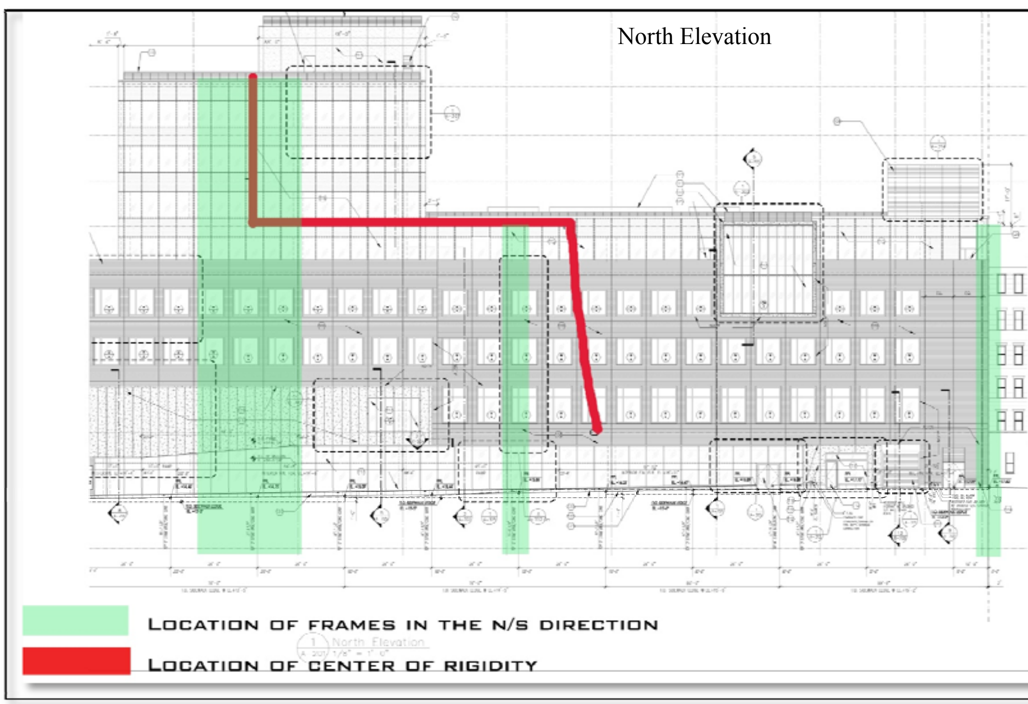


Figure 47: Schematic diagram of the location of the center of rigidity due to the lateral system

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.

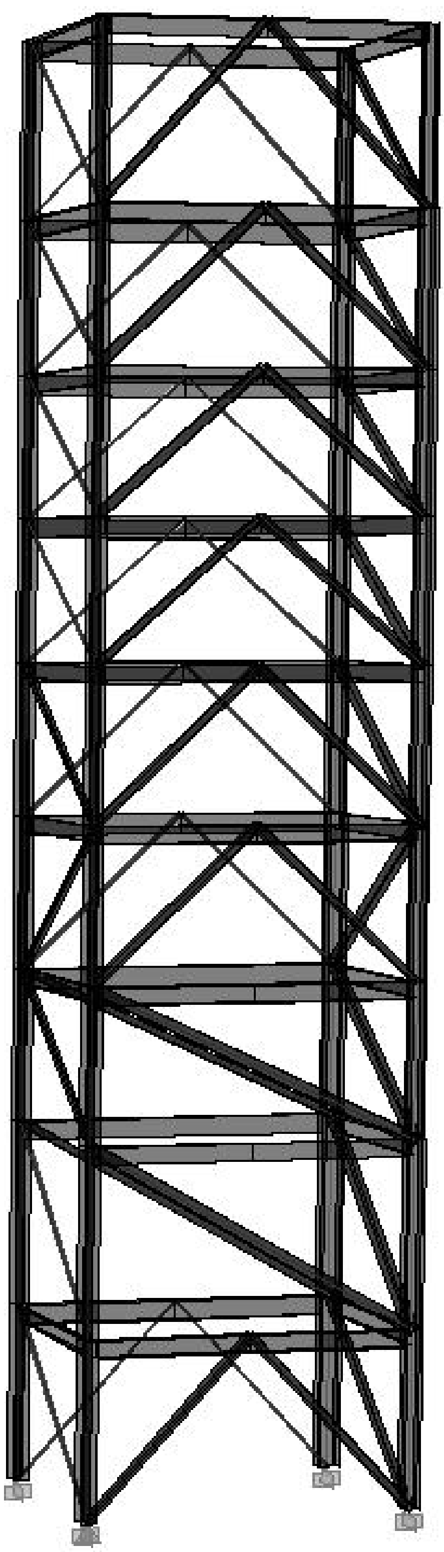
When going from moment frames to braced frame, the entire braced frame core now distributed to lateral load more evenly, this caused the initial column sizes to be overdesigned. I was able to bring down the column sizes, to the point where the combination frame core (moment frames and braced frames) 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



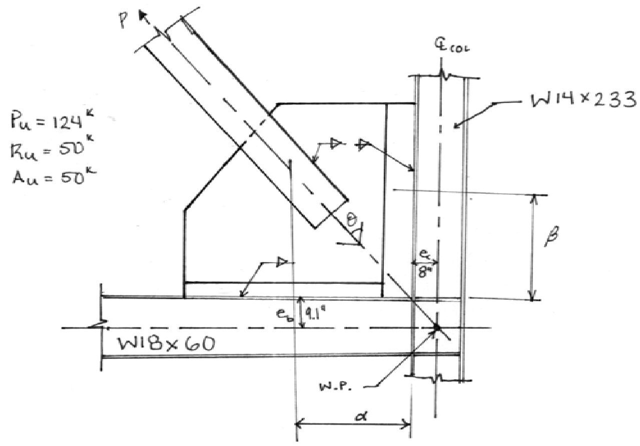


Figure 48: Diagonal Bracing Connection

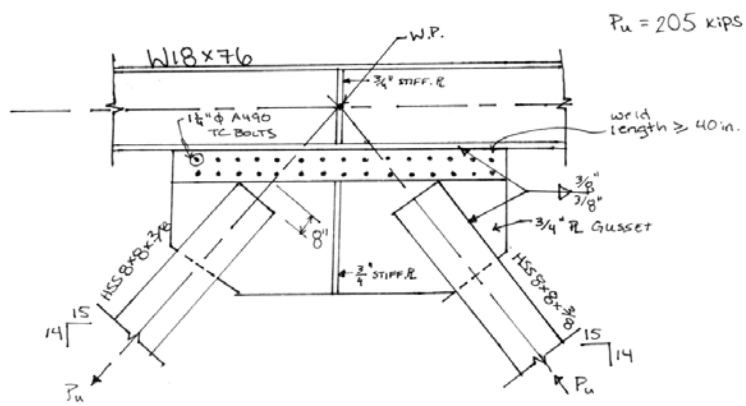


Figure 49: Chevron Bracing Connection

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.

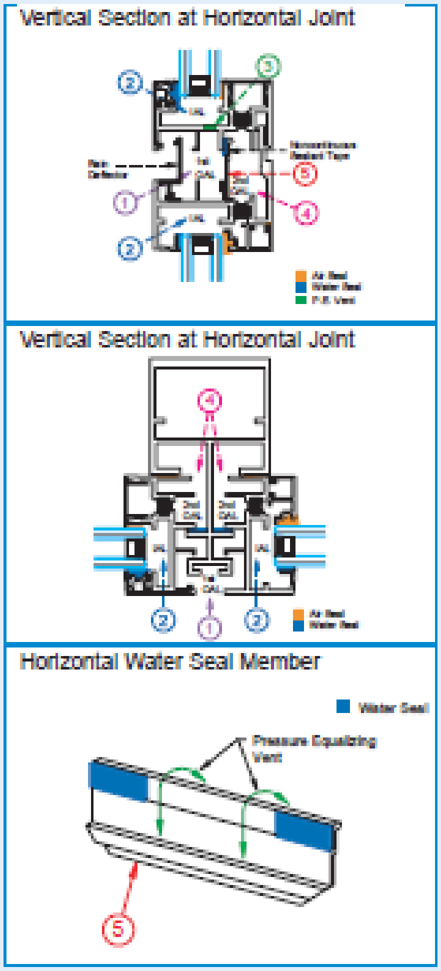
All of this has to be achieved while maintaining an inviting and transparent appearance to the community so that they can feel welcome. This may cause limitations in the window glazing chosen and its corresponding R-value. This in-depth analysis could not be achieved without the redesign of the structural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

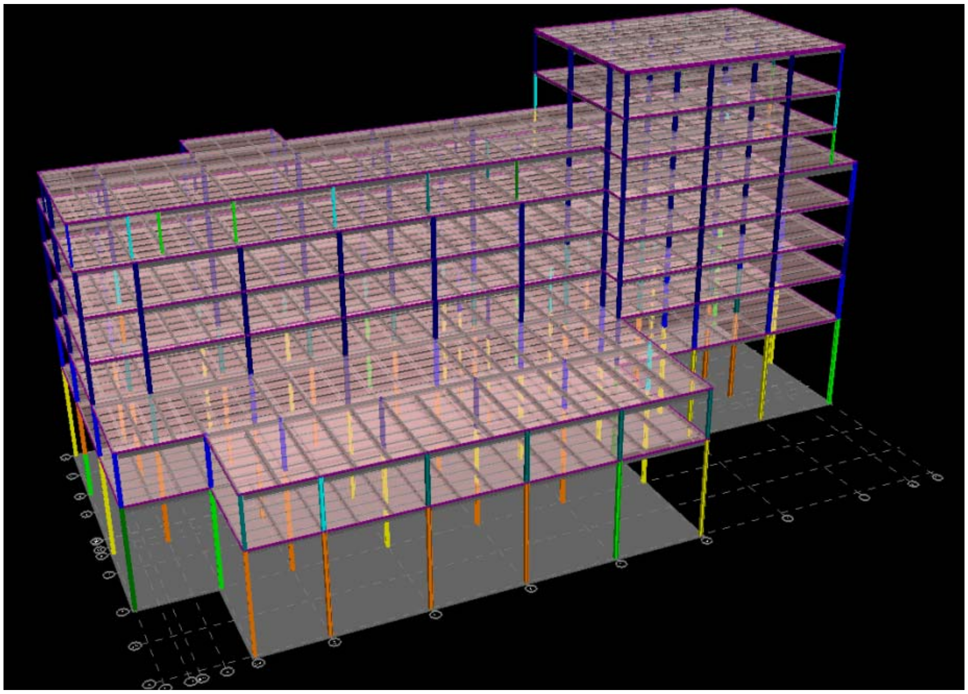


Figure 52: RAM model for Gravity Beams and Columns

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.

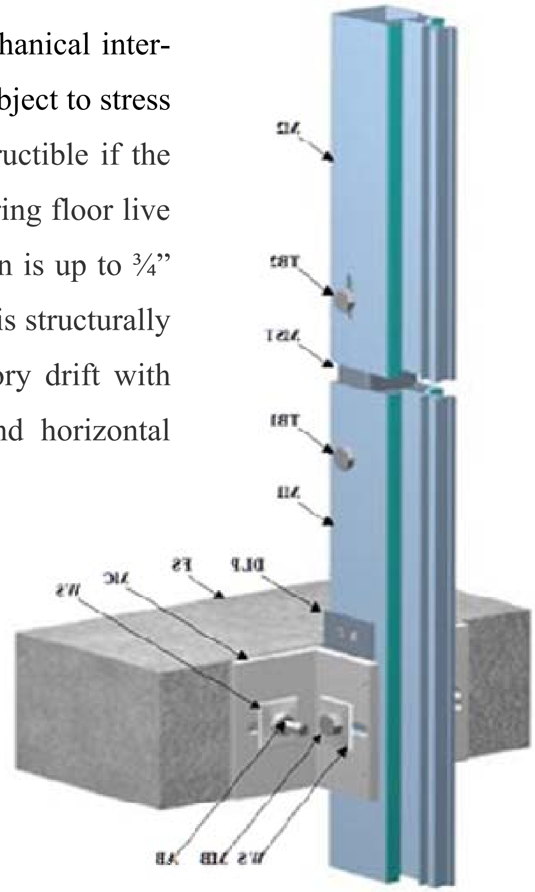


Figure 53: Ting Wall Structural System

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.

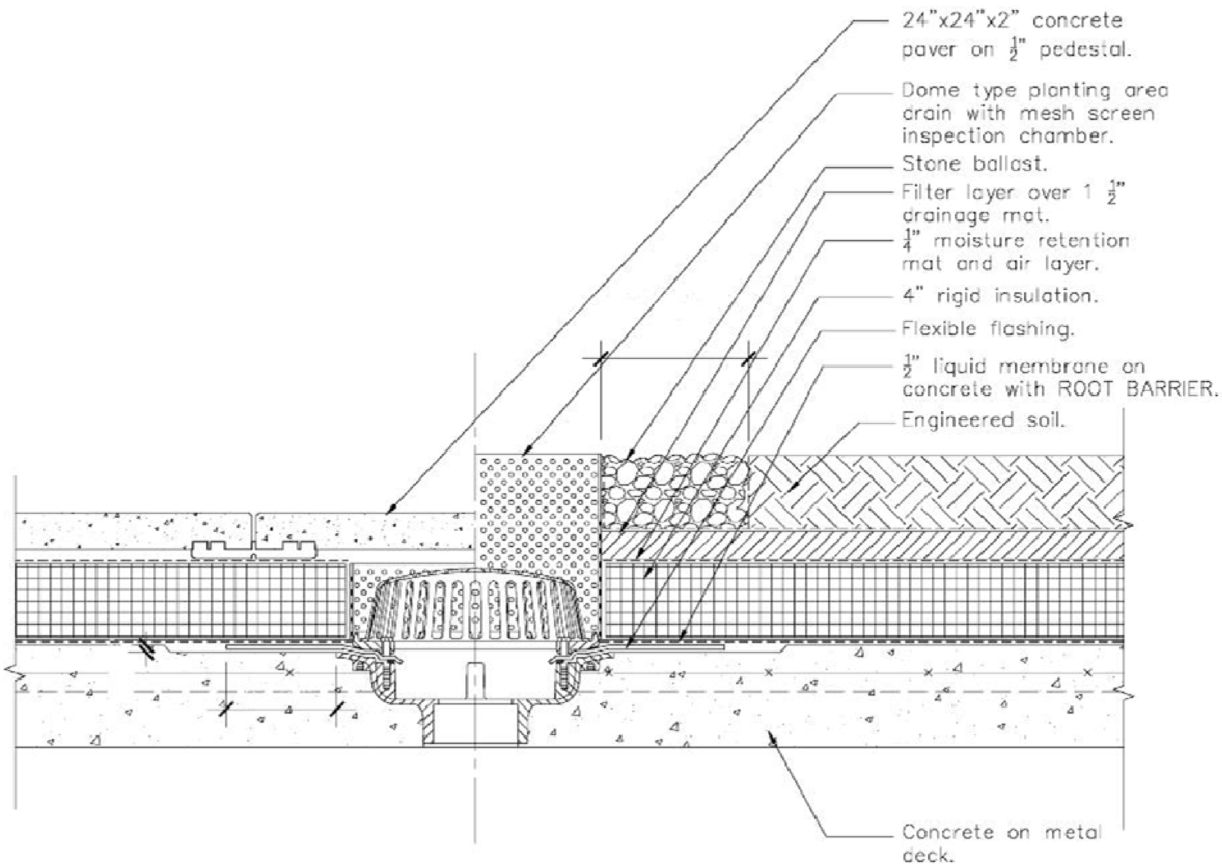


Figure 54: Detail at Green Roof Drain

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

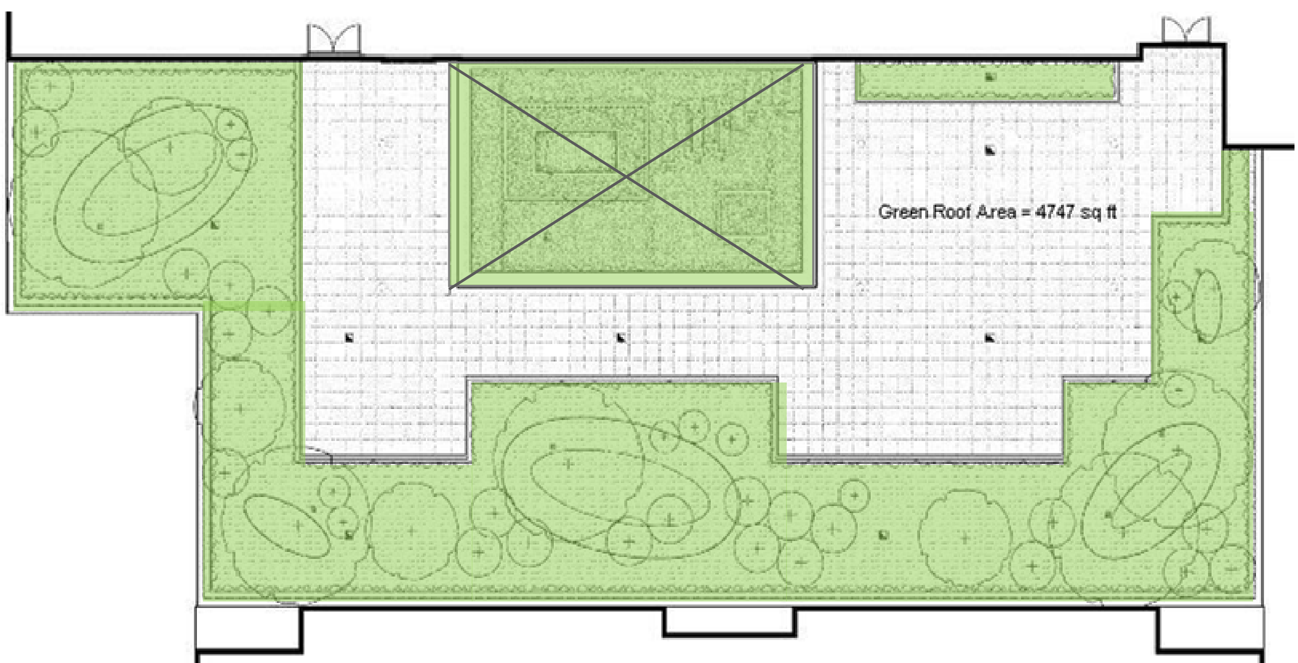


Figure 57: Original Green Roof Design

Final Green Roof Designs

On the First and Second Story Levels

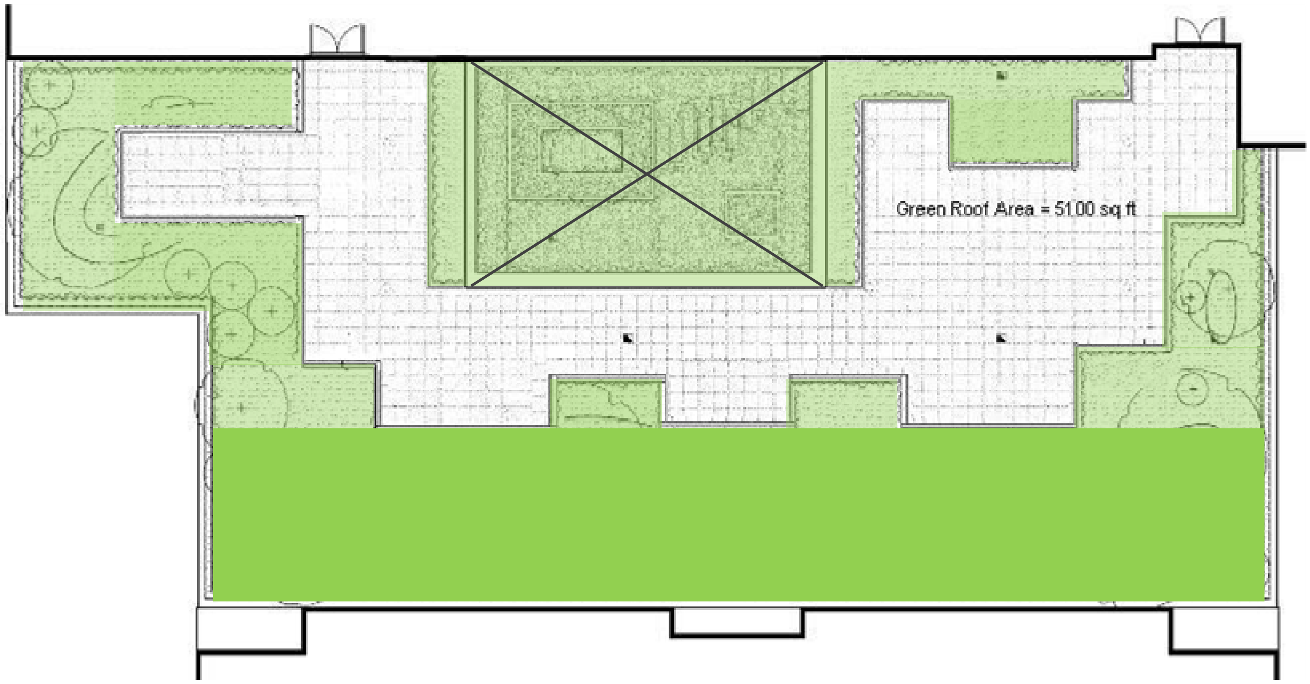


Figure 58: Redesigned Intensive Green Roof at Second Story Level

On the fifth story level

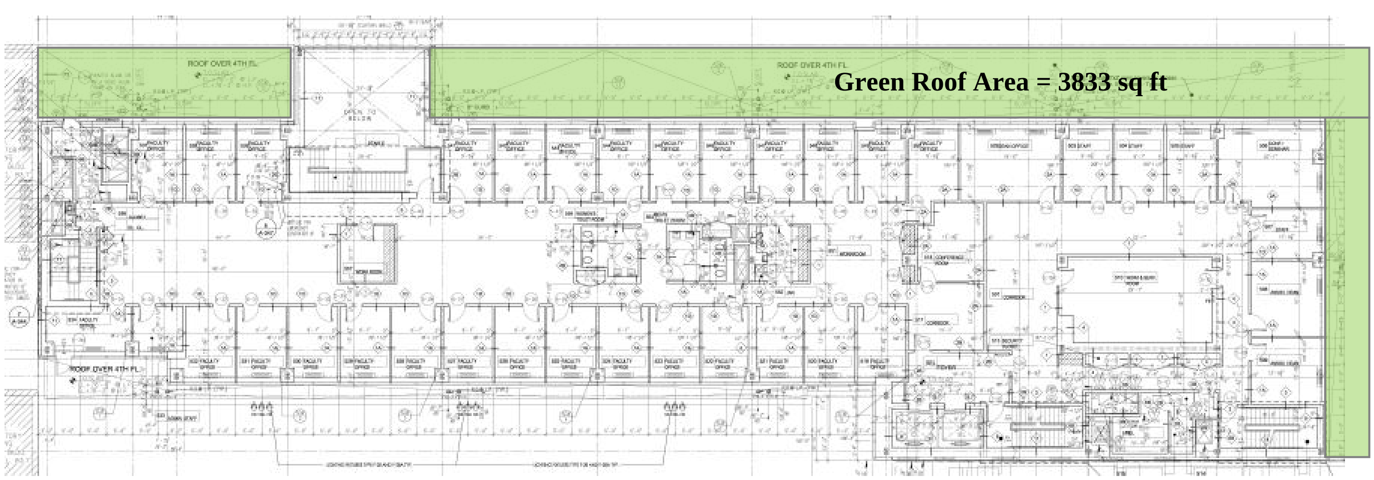


Figure 53: Redesigned Extensive Green Roof at Fifth Story Level

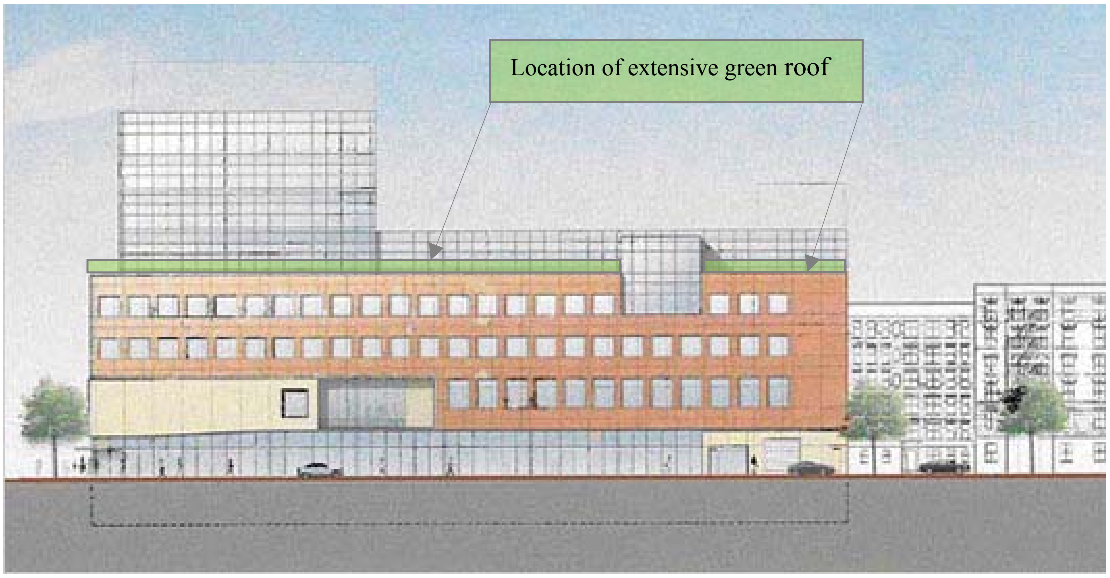


Figure 59: Location of Redesigned Extensive Green Roof

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

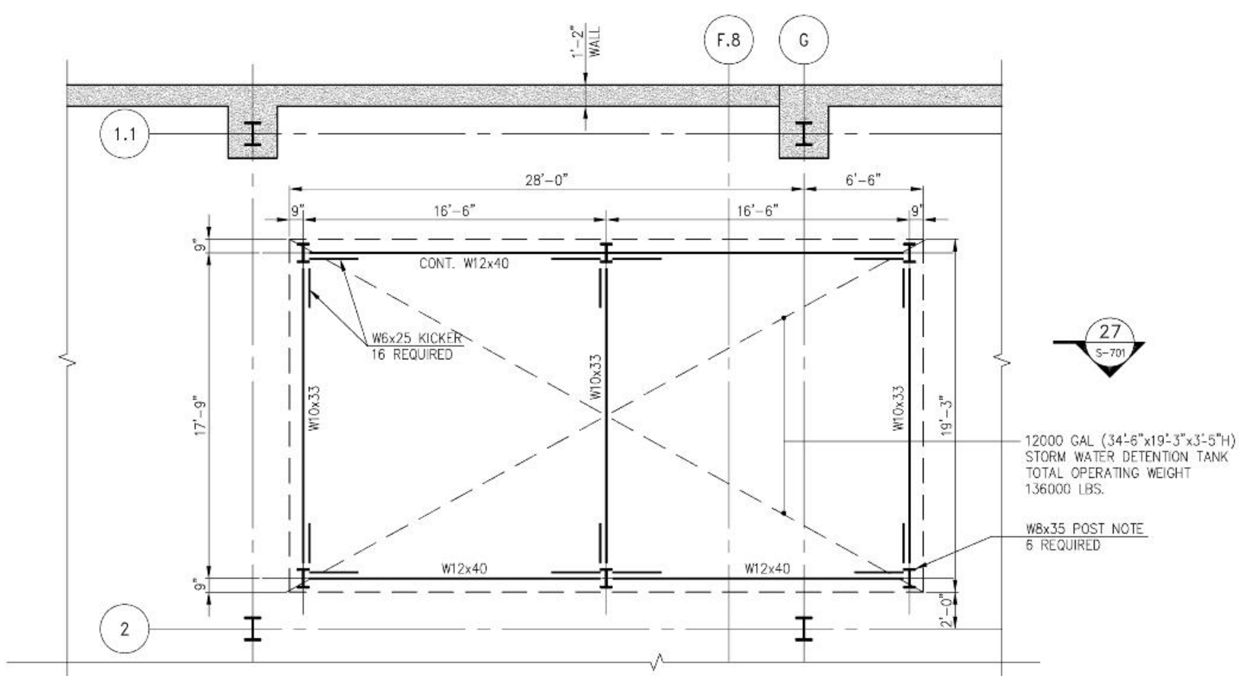


Figure 63: Watertank Dunnage Platform

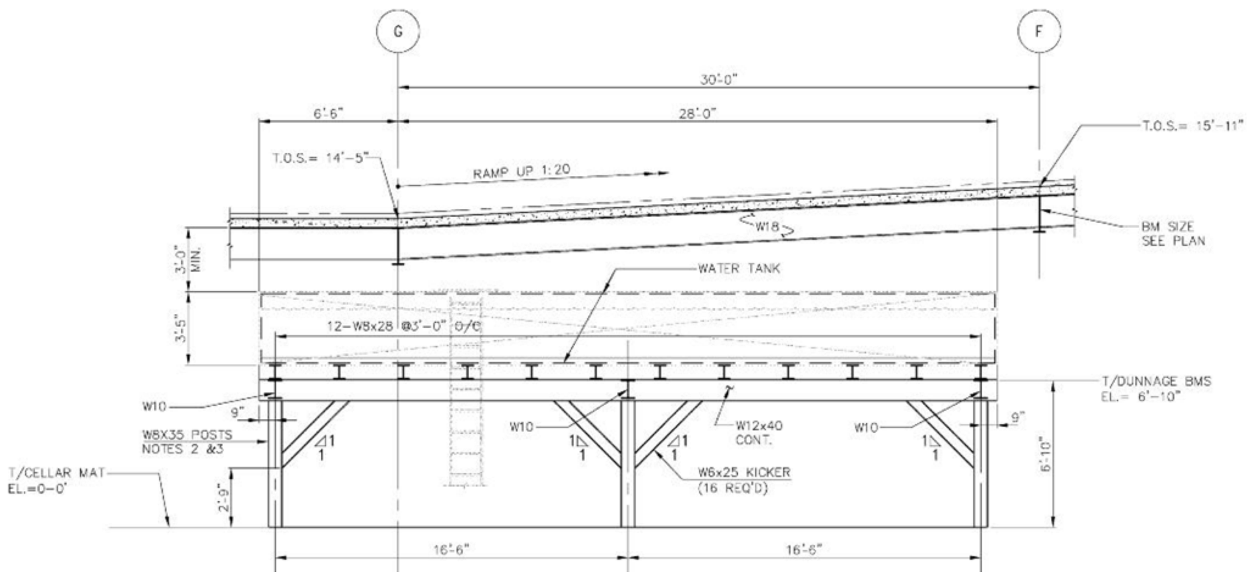


Figure 64: Section through Dunnage Platform Supporting Water Tank

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.

References

Model Codes and Standards

ASCE Standard 7-05 Minimum Design Loads for Buildings and Other Structures. Reston, VA: American Society of Civil Engineers/Structural Engineering Institute, 2005.

Building Code of the City of New York. New York: Department of Citywide Administrative Services, 2004.

International Code Council, 2006 International Building Code. Illinois: International Code Council Publications, 2006.

Steel Construction Manual, 13th Edition. Chicago: American Institute of Steel Construction, 2005.

Design References

Fisher, James M., and Griffis Lawrence G., "Design Guide 3: Serviceability Design Considerations for Steel Buildings (Second Edition)". Steel Design Guide Series Number 3, AISC 2003.

Textbooks

Geschwindner, Louis F., Unified Design of Steel Structures. John Wiley and Sons, Inc., 2008.

Articles

McEntee, Paul. "Steel Moment Frames – History and Evolution." *Structural Engineer magazine* Feb. 2009: 1-6. Web.

Richard, Dylan. "Steel moment frames 101: What to consider when creating wide-open spacesnd Evolution." *Structural Engineer magazine* Jun. 2008: 1-6. Web

peck, steven, and . "design guidelines for green roofs." 15-21. Web. <http://egov.cityofchicago.org/>

"New York City Green Roof Buildings to Earn Tax Creditroofs." *Storm Water Solutions* (2008): n. pag. Web. <Sustainable South Bronx>.

"Green Roofs and the Urban Heat Island Effect uildings to Earn Tax Creditroofs." *Buildings* (2009): n. pag. Web. <Buildings.com>.

Credits/ Acknowledgements

The writer is thankful to the following professionals, AE faculty, and students for their guidance throughout the year, as well as permission to use the Hunter College School of Social Work.

Turner Construction Company

Matthew Niskanen

AE Consultant and Advisor

Professor Ali Memari

AE Faculty

Professor M. Kevin Parfitt

Professor Robert Holland

AE Students

Cassandra Watson for her help with Electrical Drawings

Sabrina Duk for her help with Mechanical Drawings

Kyle Wagner, Ronza Abusaid, and Zack Yarnall for the continuous support. She is also thankful for the love and support provided by her mother, Annies. The past five years would not have been as great without my family away from home, you know who you are. Thank you for everything.