# Final Report

## Piez Hall Extension

Oswego, NY

Min Gao Li
Structural Option
Faculty Advisor: Dr. Thomas E. Boothby
March 26, 2013

# TABLE OF CONTENTS

# Executive Summary

To begin the semester long project, the gravity and lateral loads were determined according to ASCE 7-10 guidelines. Assumptions had been made to predict the overall weight and height of the building. The accurate weight of the building was found to be 11518 kips while the overall height was 73' of 18.2' for each level. A schematic design was then created in ETABS. Upon the completion of the gravity system, the locations of lateral elements were experimented in ETABS to best resist seismic and wind loads. It was worth noting that most braced frames were determined to locate near elevator shaft and stair core to resist lateral loads. The braces were designed in a way that would account for any opening along the frame's elevation. All the frames in the model had their beams and braces end released because there were no moment connections. After all the members had been modeled properly, ETABS performed the analysis to find the optimal size of the members according to the inputted gravity and lateral loads. Hand calculations of the center of mass and center of rigidity were performed to check the adequacy and accuracy of the model. Column C-2 was chosen to check the member sizes selected by ETABS. It was determined that the model was accurate and the selected member sizes met both serviceability and strength requirement.

The outputs of the final ETABS model were used to determine building torsion, lateral load distribution, allowable story drift, and overturning moments. These values were then compared to the values of the existing concrete design. Since the seismic loads were decreased by 2 fold in the proposed design, its building torsion were also reduced by 60% in the north-south direction and 75% in the east-west direction. Both allowable story drift and overturning moment requirements were determined to be adequate for the new redesign.

A construction breadth study was conducted to determine the construction cost and time of the proposed design. Detailed cost estimation was performed to find the new structure system's cost, which turned out to be 41,171,435 US dollars. Although the steel system was more expensive than the existing concrete system, the expertise of the labors in steel construction at the building's location is still quite vast. A construction schedule for the new redesign was developed using Microsoft project. According to the schedule given by Cannon Design, the new system would decrease construction time by as much as 3 months compared to the existing concrete system. The author also considered constructability of the proposed design. Hence, a construction site logistics were established to map out the existing condition, excavation/mobilization, structure, and finishes phase of the project.

In the sustainability breadth, an energy analysis was conducted for the proposed extensive green roof system. It was found that this proposed roof system would reduce annually cooling load by 10% in summer and heating load by 25% in winter. The extensive green roof also featured lightweight, fast installation, cost effective, and low maintenance. The benefits of green roof included but not limited to improve acoustic performance and to reduce storm water run-off time. LEED and installation process of the green roof were also discussed.

# Building Introduction

The new Piez hall extension at Oswego University located in New York will provide high quality classrooms, teaching and research laboratories, as well as interaction spaces for all of the university's engineering departments. Inside the new facility, there will be a planetarium, meteorology observatory and a greenhouse.

FIGURE 2: AERIAL MAP FROM BING.COM SHOWING THE LOCATION OF THE SITE

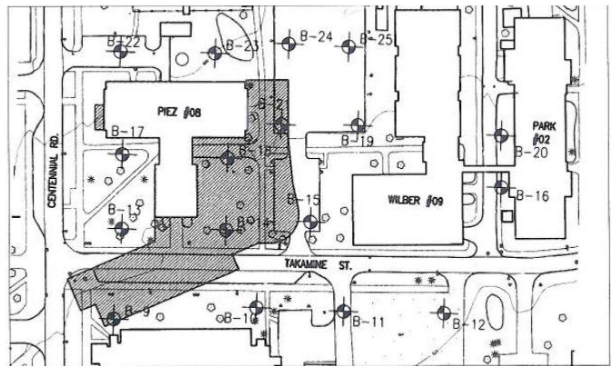



FIGURE 1: SITE MAP SHOWING EXISTING PIEZ HALL AND THE NEW EXTENSION (SHADED AREA)

The Piez hall addition will add approximately 155,000 square feet to the existing Piez hall. Snygg hall, which is next to the Piez hall, will be demolished to make way for the new addition. In the back of the U shaped Piez hall, there will be a walkway connecting Wilbur hall and the new addition. The construction of Piez hall extension began as early as April 2011. It is anticipated to be complete by April 2013 with an estimated cost of $110 million dollars. The building has 6 stories and it stands 64 feet high. The new 210,000 square feet concrete framed extension was designed by Cannon Design. The building is designed so that its exterior enclosure looks somewhat similar to the existing Piez hall (see Figure 3). The building is decorated with a skin of curtain wall. Brick is used in the south side facade. The second and third levels have spaces cantilevered slightly out to the west.

The Piez hall extension has numerous sustainability features to attain LEED Gold Certification. The building energy efficient curtain wall with a high R value will reduce heat loss. The mechanical system includes a large geothermal heat pump with a design capacity of 800 tons will be implanted to cool and heat the building. Occupied spaces have access to daylight. The roof has photovoltaic array, skylight and wind turbines. These features together will reduce the total energy use of the building to 47% and save 21% of the energy cost each year.

FIGURE 3: EXTERIOR RENDERING SHOWING THE BUILDING ENCLOSURE

# Structural Overview

## Foundation

According to the soil report for Oswego County, the proposed site will be suitable for supporting the renovation and addition with a shallow spread foundation system.  The maximum net allowable pressure on soil is 6,000psf for very dense till layers and 4,000 psf for medium dense clay and sand layers. All grade beams, foundation walls and piers will have a concrete strength of 4000psi while all other footings and slabs-on-grade will have a concrete strength of 3000psi.It is estimated that all foundations will undergo a total settlement less than 1 inch. Differential settlement is estimated to be less than 0.5 inch.  Details of typical footings are given in Figure 4.

Slab on grade have a thickness of 7 inches. Basement non-yielding walls have granular backfill with drains at locations where surcharge effect from any adjacent live loads may cause problems. These non-yielding walls are designed to resist lateral soil pressure of 65pcf where foundation drains are placed above groundwater level. Any cantilever earth retaining walls are designed based on 45pcf active earth pressure. All retaining wall are designed for a factor of safety equal to or greater than 1.5 against sliding and overturning. The frictional resistance can be estimated by multiplying the normal force acting at the base of the footing by a coefficient of friction of 0.32.

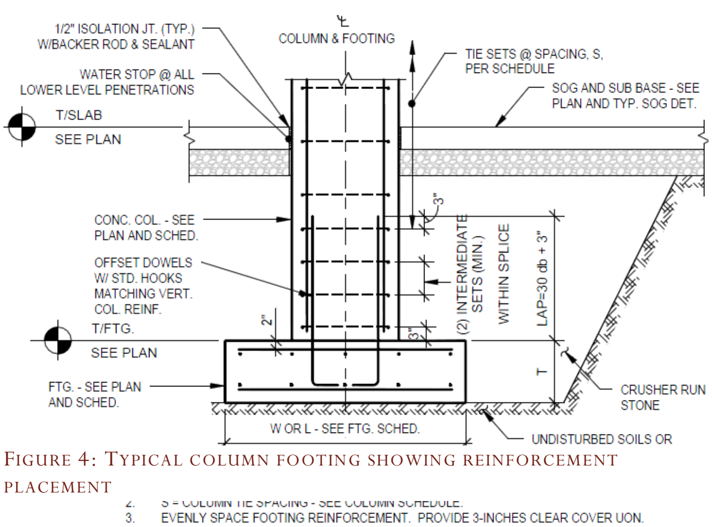



FIGURE 4: TYPICAL COLUMN FOOTING SHOWING REINFORCEMENT PLACEMENT

## Floor System

The typical floor structure of Piez Hall addition is a two-way cast-in-place flat slab with drop panels. The slab thickness of the floors is 12” throughout the entire building with primarily #6 @ 9” o.c top and #6 @ 12” o.c bottom bars in 5000 psi strength concrete. 42”x24”concrete beams spans a length of 46.2’ with 4 #8 @ top and 6 # 10 @ bottom reinforcement bars are placed in the edge of the floor slab primary located to support the cantilevered portion of the building in the second and third floor. Also, 24”x24” interior concrete beams are placed along the corridor of building to support areas where the slab is discontinuous such as stair and elevator shaft locations. A continuous 50”x10” edge beam each spans a length of 31.5’ is placed on the north side of the south wing where the conservatory is connected to the building. The total depth of the floor system is 20”. A typical framing plan of the south wing can be found in figure 10 and 11.

Typical slab thickness is 12”. A drop panel is placed in almost every column location to increase the slab thickness in order to magnify the moment carrying capacity near the column support and to resist punching shear. Typical drop panels are 10.5’x10.5’x8” (see Figure 6)

In the conservatory located in the middle of Piez Hall, the structural engineer employed composite steel floor system primary because lateral forces is not a concern due to the fact that the conservatory is embraced by the Piez hall building. Thus expensive moment connections are not necessary.

In addition, reinforcements for temperature change are #6 bars at 18” spacing, which is the maximum permitted spacing for temperature reinforcement. Typical steel reinforcement placement for the slab is given in figure 5.

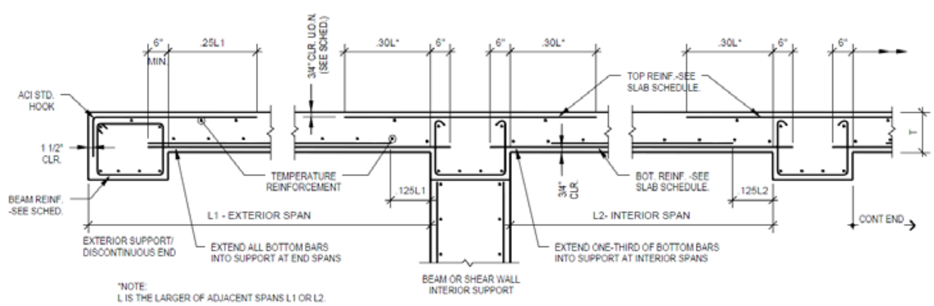



FIGURE 5: TYPICAL ONE WAY SLAB SHOWING REINFORCEMENT PLACEMENTS

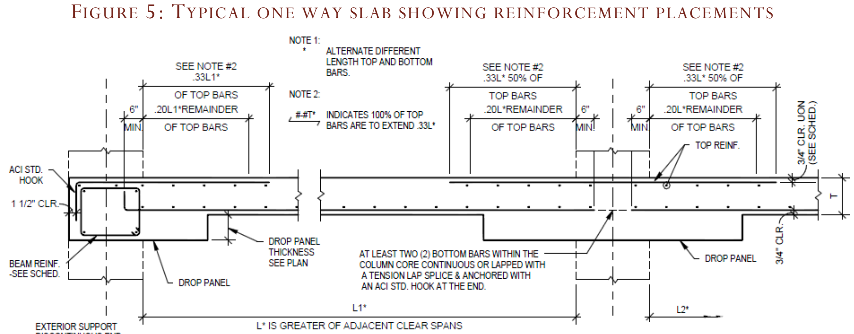



FIGURE 6: TYPICAL COLUMN STRIP DETAIL WITH DROP PANEL AND EDGE BEAM

## Framing System

Typical bay in the new south wing of the building are 31.5'x31.5'. Corridor areas have a bay size of 10.3'x31.5'. The 10.3' span is less than two third of its adjacent span of 31.5'. Thus, this limitation suspends the use of direct design method. The equivalent frame method will be used to analyze the slab.

Typical columns are 24"x24" square concrete columns with eight #8 vertical reinforcing bars and #3 ties at 15" spacing. The upper east part of the new addition is supported by circular concrete columns with 30" diameter extending from the foundation to the top of second floor. Typical beams are 24"x24" doubly reinforced concrete beams with #6 top reinforcing bars and #8 bottom reinforcing bars. Because beams are framed into slabs, beams are treated as T-section beams. Typical reinforcement placements for beams are shown in Figure 7.

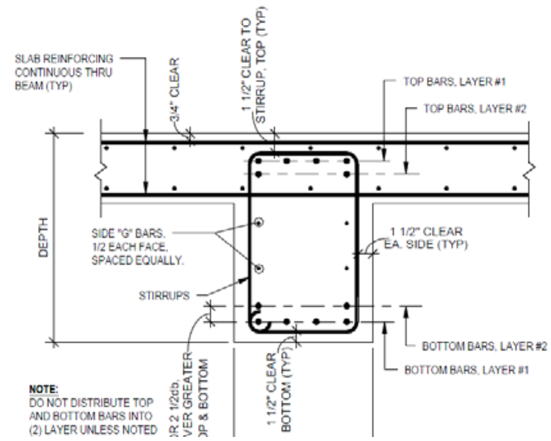



Figure 7: Typical beam section

The planetarium and conservatory in the middle of the "U" of building is built with structural steel framing. The floor system is a composite steel deck supported by W-shape beams. The sizes of the beams are typically W 14x22, W16x26, and W16x 31. Columns consist of various kinds of hollow structural steel and W10x33. Again, a typical framing plan of the south wing can be found in figure 10 and 11.

## Lateral System

Shear walls and diagonal bracing are the main lateral force resisting system in the Piez hall new addition. They are evenly distributed and orientated throughout the building to best resist the maximum lateral loads coming from all direction. Typical shear walls are 12" thick and consist of 5000psi concrete. Shear walls extend from the first level to the top of the roof. Loads travel through the walls and are distributed down to the foundation directly. Diagonal bracing are concrete struts that framed into concrete beams. They are located on the second to fourth level and placed on the sides of the cantilevered portion of the building. Since the building is a concrete building, concrete intersection points also serve as moment frames. Together, these elements create a strong lateral force resisting system.

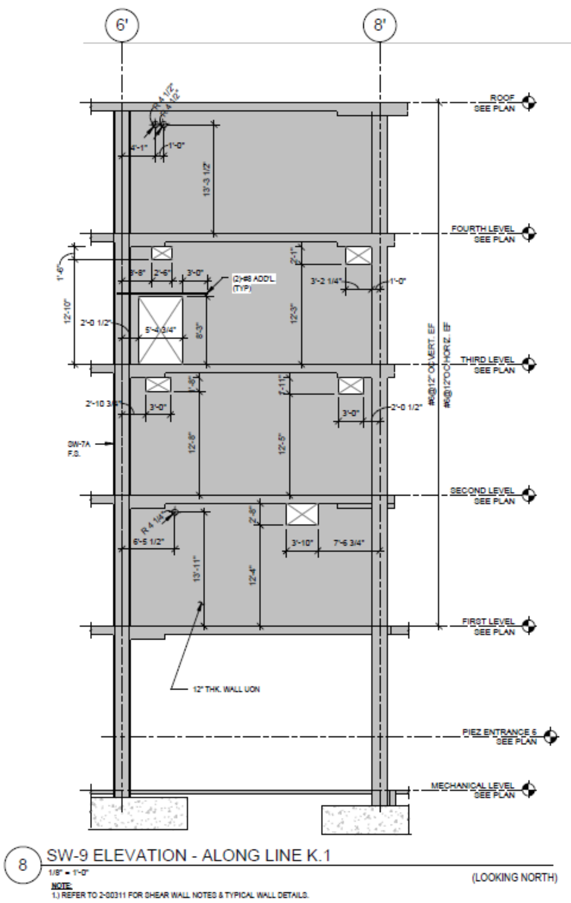



Figure 8: Typical concrete shear wall

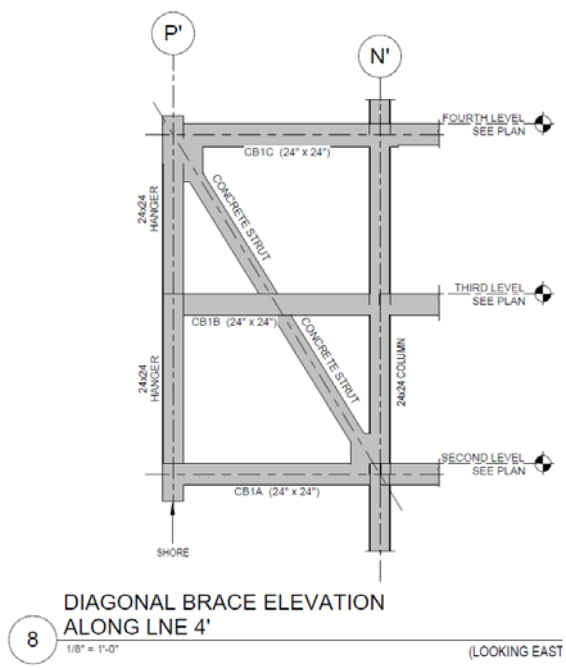



Figure 9: Typical concrete diagonal braces

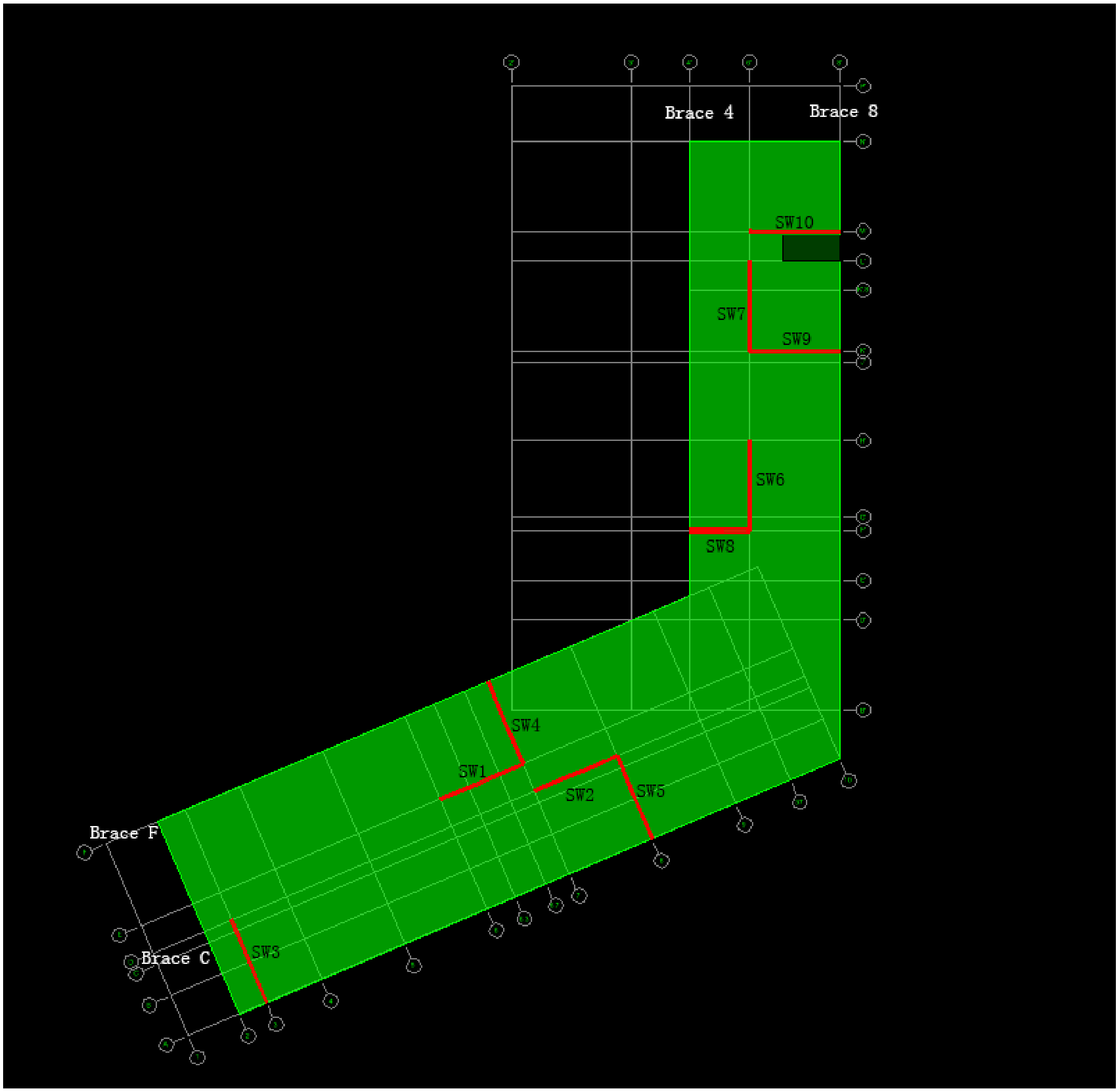



Figure 10: shear wall locations of a typical floor

## Roof System

There are three different kinds of roof system for the Piez hall extension. Steel decks and steel beams are used to support the roof for the planetarium. The roof for the cantilever part of the third level is designed to let people walk on top of them. Therefore, a fairly thick roof of 10" concrete is required. All other roof for the fourth level uses 6.5" thick concrete because they are not intended for excessive live load. On top of the roof, there are photovoltaic array, skylights, wind turbine and mechanical equipment that contribute to LEED.

## Design Codes

- Building Code Requirements for Structural Concrete (ACI 318-05)
- Specifications for Masonry Structures (ACI 530.1)
- Building Code Requirements for Masonry Structures (ACI 530)
- Masonry Structure Building Code Commentary (ACI)
- AISC Specifications and Code (AISC)
- Structural Welding Code – Steel (AWS D1.1 2002)
- Structural Welding Code – Sheet Steel
- Building Code of New York State 2007
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-02)

## Design Codes used for Thesis

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)
- International Building Code (2009 Edition)
- Building Code Requirement for Reinforced Concrete (ACI 318-11)
- Steel Construction Manual (AISC 14$^{th}$ Edition)

## Materials Used

| Concrete | | |
|---|---|---|
| **Usage** | **Strength (psi)** | **Weight (pcf)** |
| Footings | 3000 | Normal |
| Grade Beams | 4000 | Normal |
| Foundation Walls and Piers | 4000 | Normal |
| Columns and Shear Walls | 5000 | Normal |
| Framed Slabs and Beams | 5000 | Normal |
| Slabs-on-Grade | 3000 | Normal |
| Slabs-on-Steel-Deck | 3000 | Normal |
| All Other Concrete | 4000 | Normal |

TABLE 1: SUMMARY OF MATERIAL USED WITH STRENGTH AND DESIGN STANDARD

| Steel | | |
|---|---|---|
| **Type** | **Standard** | **Grade** |
| Typical Bars | ASTM A-615 | 60 |
| Welded Bars | ASTM A-706 | 60 |
| Steel Fibers | ASTM A-820 Type 1 | N/A |
| Wide Flange Shapes, WT's | ASTM A992 | 50 |
| Channels and Angles | ASTM A36 | N/A |
| Pipe | ASTM A53 | B |
| Hollow Structural Sections (Rectangular & Round) | ASTM A500 | B |
| High Strength Bolts, Nuts and Washers | ASTM A325 or ASTM A-490 | N/A |
| Anchor Rods | ASTM F1554 | 36 |
| Welding Electrode | AWS A5.1 or A5.5 | E70XX |
| All Other Steel Members | ASTM A36 UON | N/A |

TABLE 2: SUMMARY OF MATERIAL USED WITH STRENGTH AND DESIGN STANDARD

# Gravity Loads

Dead, live and snow loads are computed and compared to the loads listed on the structural drawings. After determining the loads using ASCE 7-10, spot checks on members of the structural system were checked to verify their adequacy to carry gravity loads.

## Dead and Live Loads

Although the Structural engineer has given a superimposed dead load of 15psf for all levels, but a more conservative and general superimposed dead load of 20psf were used in the calculation. Façade, column, shear wall and slab were all taken into account to obtain the overall dead load in each level. The exterior wall consists of curtain wall, CMU, precast concrete panels in different location. Thus to simplify the calculation, a uniform 30psf were taken as the load of the façade in all sides of the building. The overall weight of the building is found to be 29577 kips. This total weight is needed to compute the base shear for seismic calculation later on.

| Weight Per Level | | |
|---|---|---|
| **Level** | **Weight (kips)** | **Weight (psf)** |
| 1 | 5293.10 | 197.67 |
| 2 | 6449.73 | 221.54 |
| 3 | 6246.66 | 222.84 |
| 4 | 6246.66 | 222.84 |
| Roof | 3265.58 | 121.95 |
| Total Weight | 29577.02 | |

Table 3: Distribution of weight per level and total weight

Live Loads shown in the middle column of Table 4 are given by the structural engineer. The structural engineer is rather conservative to use all design live load to be 100psf when an 80psf can typically be used for educational occupancy. Since this is a University building, typical floor is likely to be classrooms which have live load of 50psf as defined by ASCE 7-10. Similarly, public spaces can be interpreted as corridor above the first floor which has a live load of 80psf.

| Live Load | | |
|---|---|---|
| **Space** | **Design Live Load (psf)** | **ASCE 7-10 Live Load (psf)** |
| Typical Floors | 100 | 50 |
| Public Spaces | 100 | 80 |
| Exit Corridors | 100 | 100 |
| Stairs | 100 | 100 |
| Lobbies | 100 | 100 |

TABLE 4: COMPARISON OF LIVE LOADS

## Snow Loads

Following the procedure outlined in ASCE 7-10, the result of snow loads were obtained. The resulting snow loads were found to be 46psf. This is close to what the structural engineer had calculated.

# Lateral Loads

## Wind Loads

Wind loads were calculated with the MWFR Analytical Procedure. A simplified building shape was used to approximate the size of the U-shaped building. After making such simplification, a building footprint of 237.92'x217.92'x64' was developed to calculate the wind pressure. This simplification overestimates the size of the original building, and therefore it was a conservative approach. This was done mainly to ease the use of the MWFR Analytical Procedure. The wind loads are collected by the components and cladding of the exterior enclosure. The façade then transfer these loads to the floor system, which further directs the load to the lateral force resisting system within the building and down all the way to the foundation. A base shear of 244 kips were found in the North-South direction and a 224kips base shear was found in the East-West direction.

The building was assumed to be a rigid building, hence a gust factor equals to 0.85 was used in the calculation as defined by section 6.5.8 of ASCE 7-10. Most calculations were performed using Microsoft Excel to avoid repetitive procedures. Wind pressures, including windward, leeward, sideward, uplift at roof and internal pressure were found in Table 5. Windward pressure was then distributed into each level of the building. Internal pressures have been calculated, but they were not included in both windward and leeward pressures because they eventually cancelled out. Figures 11 and 12 contain a diagram representing the wind forces in the N-S and E-W direction of the building. Since the simplified building was a fairly square box, the North-South direction wind pressure was the same as the East-West direct pressure except the building's base was 217' instead of 237'. For more details, refer to Appendix A for wind load calculation.

| Wind Pressures for all directions | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Wall** | **Floor** | **Distances (ft)** | **Wind Pressure (psf)** | **Internal Pressure (psf)** | | **Net Pressure (psf)** | |
| | | | | **0.18** | **-0.18** | **0.18** | **-0.18** |
| Windward Wall | 1 | 0.00 | 14.20 | 4.82 | -4.82 | 9.37 | 19.02 |
| | 2 | 16.00 | 14.33 | 4.82 | -4.82 | 9.51 | 19.16 |
| | 3 | 32.00 | 16.15 | 4.82 | -4.82 | 11.33 | 20.98 |
| | 4 | 48.00 | 17.37 | 4.82 | -4.82 | 12.54 | 22.19 |
| | Roof | 64.00 | 18.22 | 4.82 | -4.82 | 13.40 | 23.04 |
| Leeward Walls | All | All | -11.39 | 4.82 | -4.82 | -16.21 | -6.57 |
| Side Walls | All | All | -15.94 | 4.82 | -4.82 | -20.77 | -11.12 |
| Roof | Roof | 0 to h | -20.50 | 4.82 | -4.82 | -25.32 | -15.68 |
| | Roof | h to 2h | -11.39 | 4.82 | -4.82 | -16.21 | -6.57 |
| | Roof | > 2h | -6.83 | 4.82 | -4.82 | -11.66 | -2.01 |

TABLE 5: WIND PRESSURE IN EITHER DIRECTION

| Wind Forces N-S direction | | | | | | |
|---|---|---|---|---|---|---|
| **Floor** | **Elevation** | **Length (ft)** | **Tributary Height** | **Area (ft^2)** | **Story Forces (k)** | **Overturning Moment (k-ft)** |
| 1 | 0.00 | 237.92 | 8.00 | 1903.36 | 27.02 | 0.00 |
| 2 | 16.00 | 237.92 | 16.00 | 3806.72 | 54.57 | 873.08 |
| 3 | 32.00 | 237.92 | 16.00 | 3806.72 | 61.49 | 1967.79 |
| 4 | 48.00 | 237.92 | 16.00 | 3806.72 | 66.11 | 3173.32 |
| Roof | 64.00 | 237.92 | 8.00 | 1903.36 | 34.68 | 2219.64 |
| | | **Total Base Shear =** | | | **243.88** | |
| | | **Total Overturning Moment =** | | | | **8233.83** |

TABLE 6: WIND FORCES IN NORTH-SOUTH DIRECTION

| Wind Forces E-W direction | | | | | | |
|---|---|---|---|---|---|---|
| **Floor** | **Elevation** | **Length (ft)** | **Tributary Height** | **Area (ft^2)** | **Story Forces (k)** | **Overturning Moment (k-ft)** |
| 1 | 0.00 | 217.92 | 8.00 | 1743.36 | 24.75 | 0.00 |
| 2 | 16.00 | 217.92 | 16.00 | 3486.72 | 49.98 | 799.69 |
| 3 | 32.00 | 217.92 | 16.00 | 3486.72 | 56.32 | 1802.38 |
| 4 | 48.00 | 217.92 | 16.00 | 3486.72 | 60.55 | 2906.56 |
| Roof | 64.00 | 217.92 | 8.00 | 1743.36 | 31.77 | 2033.06 |
| | | **Total Base Shear =** | | | **223.37** | |
| | | **Total Overturning Moment =** | | | | **7541.68** |

TABLE 7: WIND FORCES IN EAST-WEST DIRECTION


FIGURE 11: WIND FORCES DIAGRAM IN NORTH-SOUTH DIRECTION

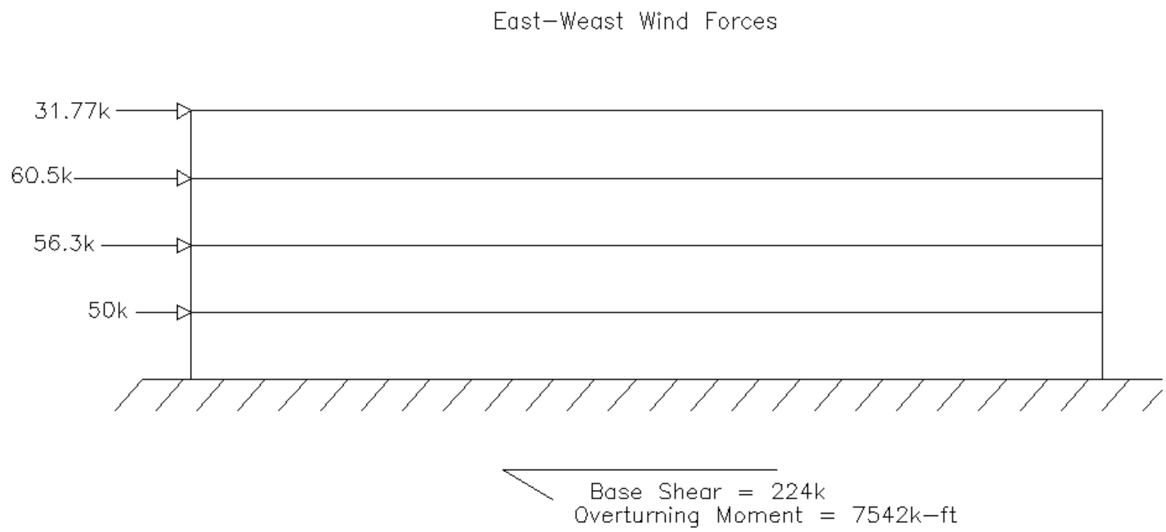



FIGURE 12: WIND FORCES DIAGRAM IN EAST-WEST DIRECTION

## Seismic Loads

The seismic loads were obtained using the equivalent lateral force procedure given in Chapters 12 of ASCE 7-10. Test boring results of the specification shows that the site is classified as class “C” for very dense soil and soft rocks. The corresponding spectral response accelerations were 0.194 for Ss and 0.078 for S1. The site coefficients were found to be Fa equals to 1.2 and Fv equals to 1.7. The approximate fundamental period of the building was estimated based on section 12.8.2.1 and was determined to be 0.676 second. This tells us that the structure was very stiff and it did not behave well during earthquakes. Similar to wind load, seismic load transfers from the floor slabs of the building to the lateral system of the building and down to the foundation.

In Figure 13, a seismic base shear of 1067 kips was determined, which has only 2.6% difference from the 1040 kips base shear that was given in the structural drawings. This slight difference was most likely due to the errors in calculating the total weight of the building. Also, seismic loads were determined to be the controlling force in this analysis in either direction. This was expected since the building has a very large base and a relatively low overall height. Moreover, it is indicated in the structural drawing that the building is designed to resist a seismic base shear of 1040 kips. Thus, it was determined that wind loads were not a controlling design factor for Piez Hall addition. However, the effect of wind load on component and cladding of the façade must be thoroughly investigated. Due to the amount of time permitted, this was not included in this report.

| Seismic Forces | | | | | | | |
|---|---|---|---|---|---|---|---|
| Level | Story Weight, Wx (kip) | Story Height, hx (ft) | $W*hx^k$ | Cvx | Story Forces (kip) | Story Shear (kip) | Overturning Moment (k-ft) |
| 1 | 5293.10 | 0.00 | 0.00 | 0.00 | 0.00 | 1067.07 | 0.00 |
| 2 | 6449.73 | 16.00 | 131711.66 | 0.12 | 124.84 | 1067.07 | 1997.47 |
| 3 | 6246.66 | 32.00 | 271175.87 | 0.24 | 257.03 | 942.23 | 8225.02 |
| 4 | 6246.66 | 48.00 | 421539.56 | 0.37 | 399.55 | 685.19 | 19178.54 |
| Roof | 3265.58 | 64.00 | 301359.17 | 0.27 | 285.64 | 285.64 | 18281.01 |
| Sum | 27501.74 | | 1125786.25 | | **1067.07** | | **47682.04** |

TABLE 8: SEISMIC FORCES DISTRIBUTION

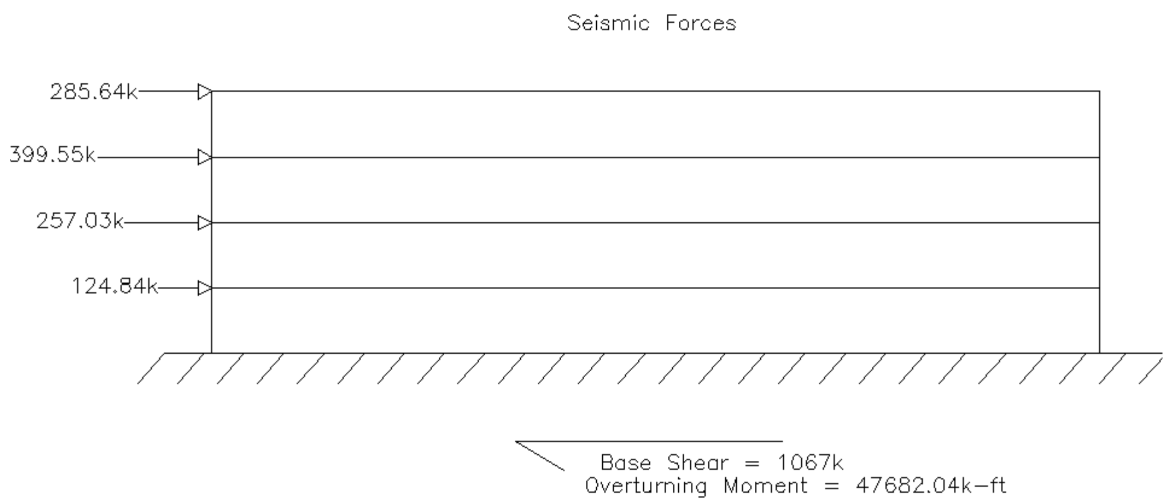



FIGURE 13: SEISMIC FORCES DIAGRAM IN EITHER DIRECTION

## Comparison of Wind and Seismic Forces

By comparing the lateral loads produced by both wind and seismic forces, it was clear that seismic loads produce the highest base shear and the largest overturning moment in either direction. The results were summarized in the table below.

| Comparison of Design Forces | | | |
|---|---|---|---|
| | N-S Wind | E-W Wind | Seismic |
| Base Shear (kip) | 244 | 244 | 1067 |
| Overturning Moment (k-ft) | 8234 | 7542 | 47682 |

TABLE 9: COMPARISON OF WIND AND SEISMIC BASE SHEAR

# Problem Statement

The current design of Piez Hall extension was determined to meet both strength and serviceability requirements as proved in Technical Reports 1 to 3. However, all shear walls resided in the new extension while the conservatory, planetarium, and old Piez hall had steel framing system with no lateral force resistance. (Typical plans of the conservatory and planetarium can be found in appendix G of page 97). The author was not sure why the structural designer used two different systems in one unified building. This approach might cause confusion during the construction phases of the building. Different groups of iron workers for steel and concrete rebar, concrete crews, and other contractors were needed to construct the building. Coordination and communication might become difficult between this many different groups of people and the budget to hire all of them may be very expensive as well. In this report, a unified design was explored to create a more flexible building while lowering construction cost, time, confusion, as well as seismic loads and building torsion.

# Proposed Solution

Based on technical report 2, either a composite system or a two way flat slab with drop panel will be selected to unify the design of Piez Hall. Since the old Piez hall, conservatory and planetarium is already a steel frame structure, the new extension will be redesigned as a composite system with long span trusses spanning a length of about 60 feet. K-series joists will be used in between the spans of the long span trusses. Moreover, since New York had many experienced steel crews and contractors, construction time and cost of the building should be reduced. The weight of the building will be greatly reduced as well, which benefits the foundation. After the redesign of gravity and lateral system of Piez Hall, the author will not have time to perform a foundation and vibration analysis. Thorough investigation will needed to determine the impact of the proposed design on the existing foundation. It was determined in technical report 3 that seismic load was the controlling lateral force in all directions, thus a flexible building with ductile members was desirable to dissipate energy in an earthquake. The redesign incorporated eccentric braced frames as the main lateral force resisting system. The shear walls in the current design will be eliminated and the column layout will be rearranged to achieve more usable interior spaces and longer deck span while it still meets all strength and serviceability requirements. A model of the proposed design will be generated using ETABS to compare with the current design. The model will be a unified composite steel system with long span trusses and eccentric braced frames. The criteria of comparison include constructability, strength, feasibility, construction cost, construction time, building torsion, and drift limits.

# Breadth Topics

## Construction Breadth

The redesign of Piez Hall addition might alter the construction process and the time and cost associated with it. The goal here was to lower the cost and time to construct the building. A construction schedule using Microsoft project was created for the proposed system. Detailed cost estimate was performed using RS means cost work. The cost and construction time of the proposed and current system was analyzed and compared. Another issue that needed to be addressed was the temporary supports and bracings that resist construction load. Since a structure has not developed its full strength during early construction phase, there exist many possibilities that the structure will collapse if temporary supports were not properly designed. Finally construction site logistics was established for the new proposed system.

## Sustainability Breadth

The current Piez Hall was rated LEED Gold. However, there were still rooms to improve. The goal for the redesign of Piez Hall was to improve sustainability by further reducing annual energy load of the building. An extensive green roof was incorporated into the proposed design. It benefited Piez Hall addition by increasing the thermal resistance of the roof assembly throughout the year, especially in summer by helping to reduce cooling costs. A green roof also acted as a sound barrier to improve the building's overall acoustic performance. Lastly, it reduced storm water run-off by 50 to 90%, which minimized the impact on the existing sewer system.  A thicker roof was accounted for the additional load brought by the green roof. An energy model will be created using Trace 700 to conduct an energy study for the green roof system.

# MAE Coursework

Concepts learn in course AE 530 (Computer Modeling), AE 538 (Earthquake Design), AE 537 (Building Performance Failure), and AE 542 (Building Enclosure Design) were incorporated into the proposed design of Piez Hall over the spring. ETABS knowledge learnt in AE 530 was used to create the model of Piez Hall extension. Seismic design concepts learnt in AE 538 was incorporated into the redesign in order to allow the structure to better resist seismic loads. Principles learnt in AE 537 were used to avoid human mistakes made in the construction phase of the building and to ensure better building performances after the building is constructed. Energy analysis and concepts learnt in AE 542 were applied to evaluate the amount of energy saved annually.

# Structural Depth

## Dead Loads

To begin the structural system re-design, the loads of the steel structure were determined. Live and snow loads were assumed to be the same as the loads used for the existing design. An additional 30 psf was added to the roof to account for the green roof system. The dead load of the structural was evaluated based on a schematic design created in ETABS. Table 10 showed a distribution of weight for each level.

| Weight Per Level | | |
|---|---|---|
| **Level** | **Weight (kips)** | **Weight (psf)** |
| 1 | 2190.28 | 64.49 |
| 2 | 2459.70 | 72.42 |
| 3 | 2459.70 | 72.42 |
| 4 | 1504.46 | 80.75 |
| Roof | 2903.54 | 85.49 |
| **Total Weight** | **11517.67** | |

TABLE 10: OVERALL WEIGHT OF THE PROPOSED DESIGN

## Wind Loads

Since the overall building height was increased in order to maintain the same floor to ceiling height ratio of the existing design, the wind load was increased as well. Due to conservative reasons, the author assumed the largest girder was a W40 and the deck was a VLI3 with a 3.5 inch concrete topping in order to obtain maximum wind load. With a floor system depth of 46.5 inches, the overall height was increased by 106 inches or 9 feet. The resulting overall height of Piez Hall addition was now 73 feet. This new building height did not violate the local zoning code since the Oswego campus allowed a maximum building height of 90 feet in the area.

The wind loads were determined using the Main Wind Force Resisting System (MWFRS) procedure (method 2) as defined in ASCE 7-10. Due to the building's complex geometry and its non-orthogonal nature, a rectangular building shape was assumed to simplify the wind load analysis. Most of the calculations for determining the wind pressures and story forces were performed in Microsoft Excel in order to minimize the amount of repetitive calculations. In the analysis, windward, leeward, sidewall, and roof suction pressures were determined. Internal pressures were neglected in calculating the design wind pressure because they will eventually cancel out in the equation.

Since the lateral resisting elements are evenly distributed throughout the building, a rigid diaphragm was assumed in ETABS. The guest effect factor was then equals to 0.85. Wind pressures in all directions can be seen in table 11. The story forces were then determined based on the wind pressures. The resulting base shear was 283kips in the N-S direction with an overturning moment of 10905kip-ft and 259kips in the E-W direction with an overturning moment of 9988kip-ft

| Wind Pressures for all directions | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Wall** | **Floor** | **Distances (ft)** | **Wind Pressure (psf)** | **Internal Pressure (psf)** | | **Net Pressure (psf)** | |
| | | | | 0.18 | -0.18 | 0.18 | -0.18 |
| Windward Wall | 1 | 0.00 | 14.20 | 4.93 | -4.93 | 9.27 | 19.12 |
| | 2 | 18.21 | 14.61 | 4.93 | -4.93 | 9.68 | 19.54 |
| | 3 | 36.42 | 16.54 | 4.93 | -4.93 | 11.61 | 21.47 |
| | 4 | 54.63 | 17.78 | 4.93 | -4.93 | 12.85 | 22.71 |
| | Roof | 72.83 | 18.61 | 4.93 | -4.93 | 13.68 | 23.53 |
| Leeward Walls | All | All | -11.63 | 4.93 | -4.93 | -16.56 | -6.70 |
| Side Walls | All | All | -16.28 | 4.93 | -4.93 | -21.21 | -11.36 |
| Roof | Roof | 0 to h | -20.93 | 4.93 | -4.93 | -25.86 | -16.01 |
| | Roof | h to 2h | -11.63 | 4.93 | -4.93 | -16.56 | -6.70 |
| | Roof | > 2h | -6.98 | 4.93 | -4.93 | -11.90 | -2.05 |

Table 11: wind pressure for proposed design

| Wind Forces N-S direction | | | | | | |
|---|---|---|---|---|---|---|
| **Floor** | **Elevation** | **Length (ft)** | **Tributary Height** | **Area (ft^2)** | **Story Forces (k)** | **Overturning Moment (k-ft)** |
| 1 | 0.00 | 237.92 | 9.10 | 2166.06 | 30.75 | 0.00 |
| 2 | 18.21 | 237.92 | 18.21 | 4332.13 | 63.29 | 1152.47 |
| 3 | 36.42 | 237.92 | 18.21 | 4332.13 | 71.65 | 2609.36 |
| 4 | 54.63 | 237.92 | 18.21 | 4332.13 | 77.03 | 4207.59 |
| Roof | 72.83 | 237.92 | 9.10 | 2166.06 | 40.30 | 2935.52 |
| | | **Total Base Shear =** | | | **283.03** | |
| | | **Total Overturning Moment =** | | | | **10904.93** |

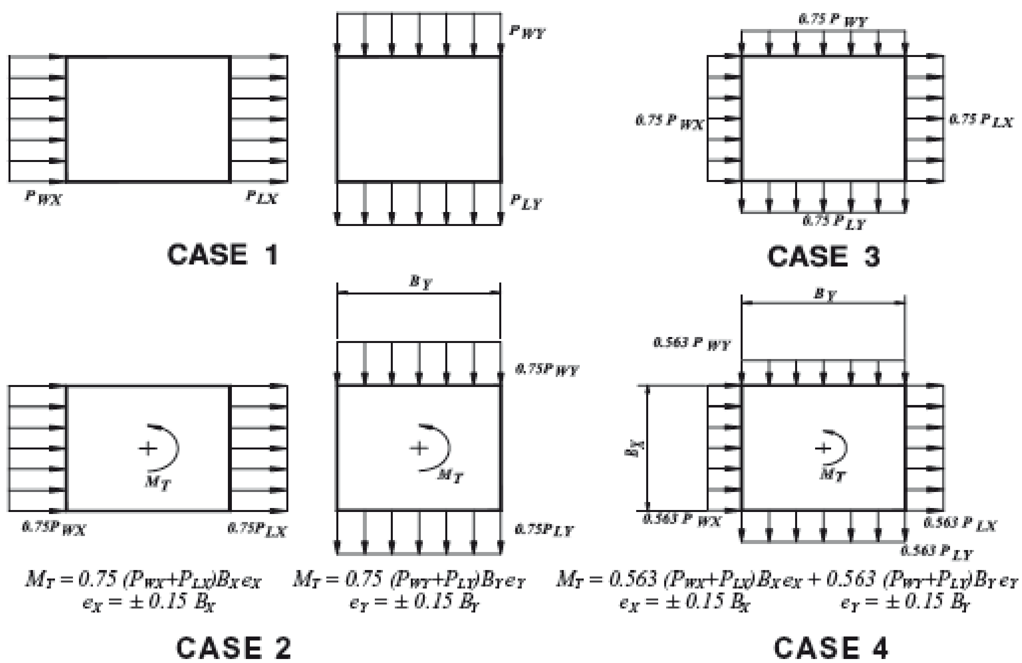
TABLE 12: NORTH SOUTH DIRECTION WIND FORCES FOR PROPOSED DESIGN

| Wind Forces E-W direction | | | | | | |
|---|---|---|---|---|---|---|
| **Floor** | **Elevation** | **Length (ft)** | **Tributary Height** | **Area (ft^2)** | **Story Forces (k)** | **Overturning Moment (k-ft)** |
| 1 | 0.00 | 217.92 | 9.10 | 1983.98 | 28.17 | 0.00 |
| 2 | 18.21 | 217.92 | 18.21 | 3967.96 | 57.97 | 1055.59 |
| 3 | 36.42 | 217.92 | 18.21 | 3967.96 | 65.63 | 2390.01 |
| 4 | 54.63 | 217.92 | 18.21 | 3967.96 | 70.55 | 3853.89 |
| Roof | 72.83 | 217.92 | 9.10 | 1983.98 | 36.92 | 2688.76 |
| | | **Total Base Shear =** | | | **259.24** | |
| | | **Total Overturning Moment =** | | | | **9988.24** |

TABLE 13: EAST WEST DIRECTION WIND FORCES FOR PROPOSED DESIGN

The four possible wind load cases from ASCE 7-10, as seen in figure 14, were considered to determine which wind case would control the design. After checking each wind load combinations in excel, it was found that wind load case 1 controlled in all directions.

| Main Wind Force Resisting System – Part 1 | | All Heights |
|---|---|---|
| Figure 27.4-8 | Design Wind Load Cases | |




**Case 1.** Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

**Case 2.** Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

**Case 3.** Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

**Case 4.** Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

**Notes:**

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   $P_{WX}$, $P_{WY}$: Windward face design pressure acting in the x, y principal axis, respectively.
   $P_{LX}$, $P_{LY}$: Leeward face design pressure acting in the x, y principal axis, respectively.
   $e$ ($e_X$, $e_Y$) : Eccentricity for the x, y principal axis of the structure, respectively.
   $M_T$: Torsional moment per unit height acting about a vertical axis of the building.

FIGURE 14: FOUR WIND LOAD COMBINATIONS DEFINED IN ASCE 7-10 CHAPTER 27

## Seismic Loads

Seismic loads were determined using the Equivalent Lateral Force Procedure outlined in chapter 11 and 12 of ASCE 7-10. In the analysis, the weight from the green roof, façade, column, slab, beam, girder, braced frames, and a superimposed dead load of 20psf were included. The overall weight of the entire addition was determined to be 11518 kips, which was almost 3 times lighter than the original design. The lighter Piez Hall had a much lower seismic load value. Also, the existing foundation of the building should be sufficient to resist the loads from the proposed design due to the lighter weight. Although the author should investigate the foundation system and potentially save some money by finding a lighter foundation system for the proposed design, he did not perform the analysis due to the amount of time permitted. Therefore, the foundation system of the re-design was remained unchanged from the original design.

A seismic base shears of 447kips was determined and the resulting overturning moment was 25349kip-ft. Table 14 showed the distribution of story forces.

| Seismic Forces | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Level** | **Story Weight, Wx (kip)** | **Story Height, hx (ft)** | **$W^*hx^k$** | **Cvx** | **Story Forces (kip)** | **Story Shear (kip)** | **Overturning Moment (k-ft)** |
| 1.00 | 2190.28 | 0.00 | 0.00 | 0.00 | 0.00 | 446.89 | 0.00 |
| 2.00 | 2459.70 | 18.21 | 57822.74 | 0.10 | 42.64 | 446.89 | 776.46 |
| 3.00 | 2459.70 | 36.42 | 122919.08 | 0.20 | 90.64 | 219.46 | 3301.19 |
| 4.00 | 1504.46 | 54.63 | 116870.74 | 0.19 | 86.18 | 133.28 | 4708.13 |
| Roof | 2903.54 | 72.83 | 308405.10 | 0.51 | 227.42 | 42.64 | 16563.15 |
| **sum** | **11517.67** | | **606017.65** | | **446.89** | | **25348.93** |

TABLE 14: SEISMIC FORCES FOR PROPOSED DESIGN

## Comparison of Seismic and Wind Loads

Due to the load combinations in ASCE 7-10 section 2.3.2, a 1.0W and 1.0E was used to compare with each other. Taking the maximum wind loads in the N-S direction, the resulting base shear was 283kips. The overturning moments was 10905kip-ft. Compared to a seismic base shear of 447kips and an overturning moment of 25349kip-ft, seismic loads was still the controlling lateral loads for Piez Hall addition. This makes sense because the building still had a very large base and a relatively low overall height.

## Gravity System Design

A schematic design was first determined and modeled in ETABS. The goal of the proposed design was to reduce the overall weight, construction cost and time of the building. During the modeling process, most of the interior columns were removed to achieve a wider bay. As a result, more interior space was available for the architect to use. 48LH17 trusses were chosen to span a maximum length of 60 feet with 31.5 feet o.c. Joist sizes were optimized depending on the span length. The longest joists were 24K5 and span a maximum of 40 feet with 12 feet o.c. After designing the frame layout, Vulcraft 3VLI18 lightweight composite deck was chosen for the floor system and a N18 roof deck was chosen for the roof.

Upon completing the framing layout, the composite beam/girder/columns of the gravity system were designed using ETABS and the member sizes were checked manually using AISC $14^{th}$ edition. Since the layout consisted of varying bay sizes, the members were designed to its optimal size. Typical framing layout with member sizes can be seen in figure 15.

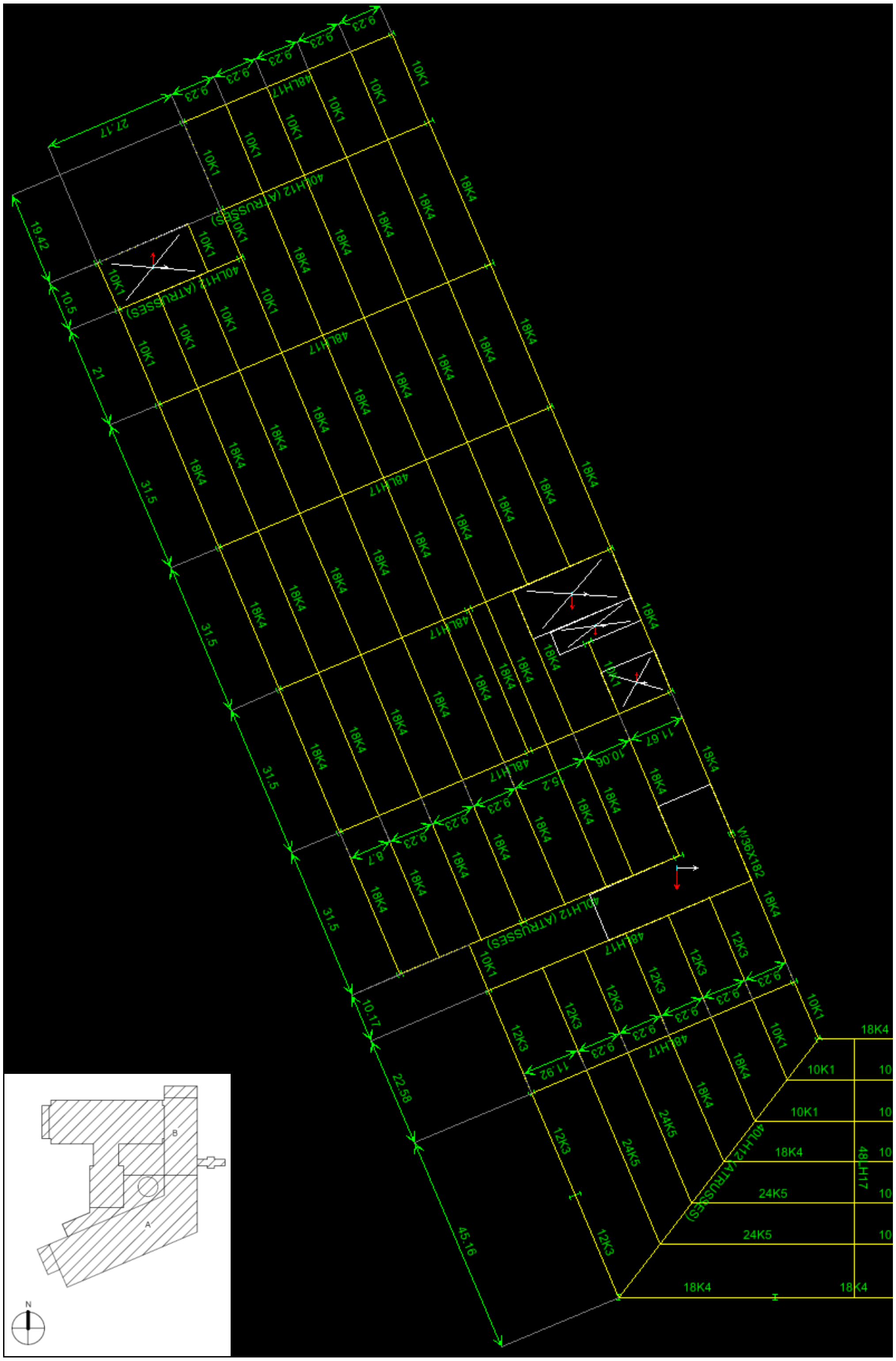



FIGURE 15: TYPICAL FLOOR PLAN WITH MEMBER SIZES

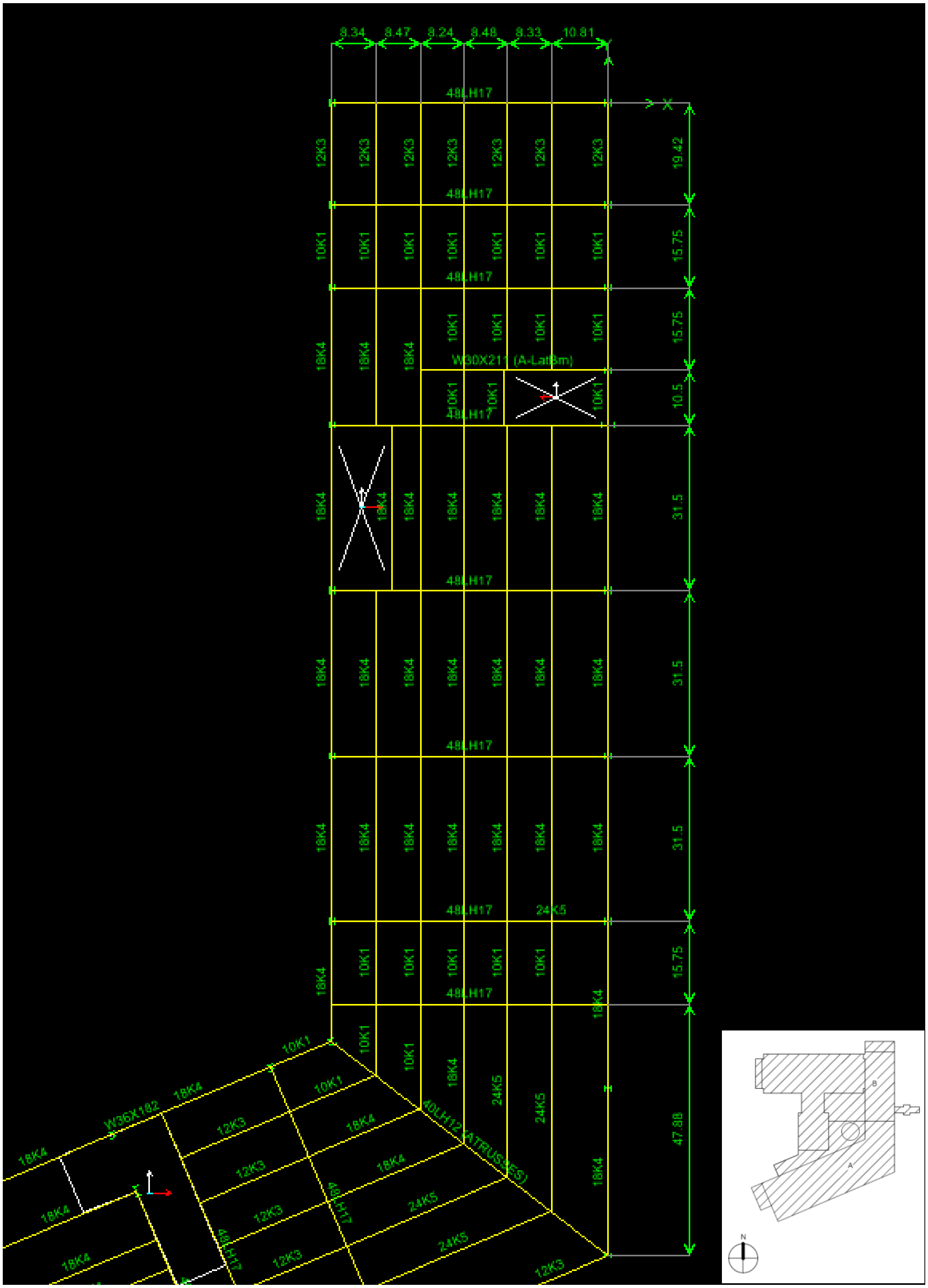



FIGURE 16: TYPICAL FLOOR PLAN WITH MEMBER SIZES

## Lateral System Design

After establishing the framing layout, the braced frames locations were determined through trial and error. 10 braced frames were placed throughout the building to best resist the lateral loads and to minimize building torsion. Most of the braced frame locations were either the same as the existing shear wall locations with a slight modification or placed around elevator shafts and stairwell cores. The locations and elevation of the braced frames were shown in the following figures.

To begin the analysis of the lateral system, the wind and seismic load for the re-design were calculated. Next, the resulting loads were input into ETABS in order for the program to determine the member sizes. After the member sizes are determined, hand calculation was performed to check the adequacy of these lateral-force resisting members.

At the south east corner, braced frames 6 was located along the perimeter of the building as shown in figure 17. This area was an auditorium in the first level and laboratories in the levels above. Although this frame may block the windows along the south wall, the author designed the location of the braces such that the negative impact will be minimized. He also found that the location of braced frame 6 was the most cost efficient and necessary to minimize overall building torsion. Moreover, the amount of windows in the southeastern corner was minimal compared to other parts of the building. Regardless, the architect of Piez Hall Addition might complain about the fact that the window areas were reduced. Moment frames were also considered, but the stiffness and construction cost/time for the moment connections were not worthwhile compared to brace frames.

The floor-to-floor height was increased from the existing 16' to 18.2' because of deeper floor. As a result, the overall building height will be increased from 64' to 73'. The purpose of this height increase was to maintain the same floor-to-ceiling heights ratio as the existing design.

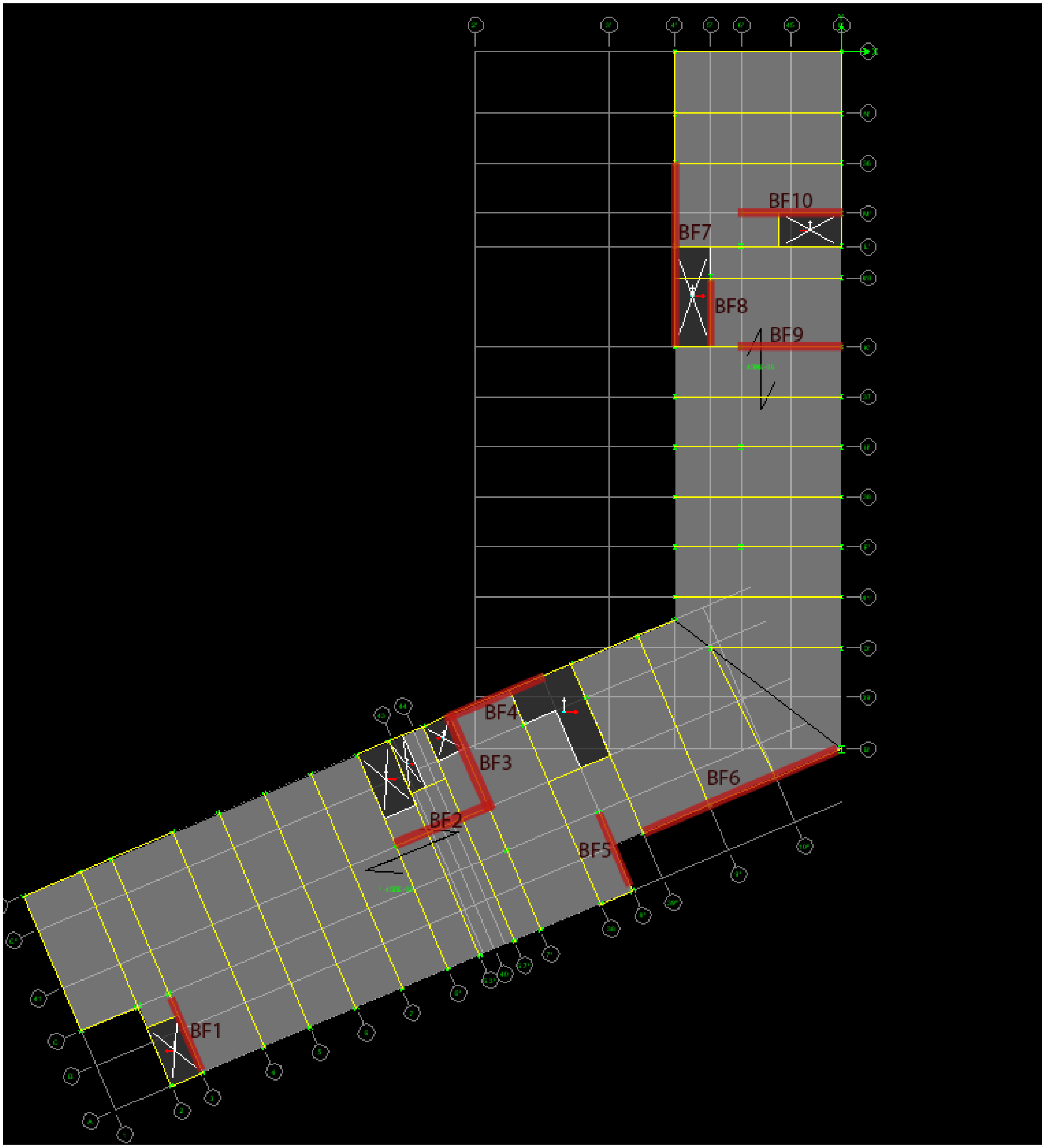



Figure 17: braced frames location for the proposed design

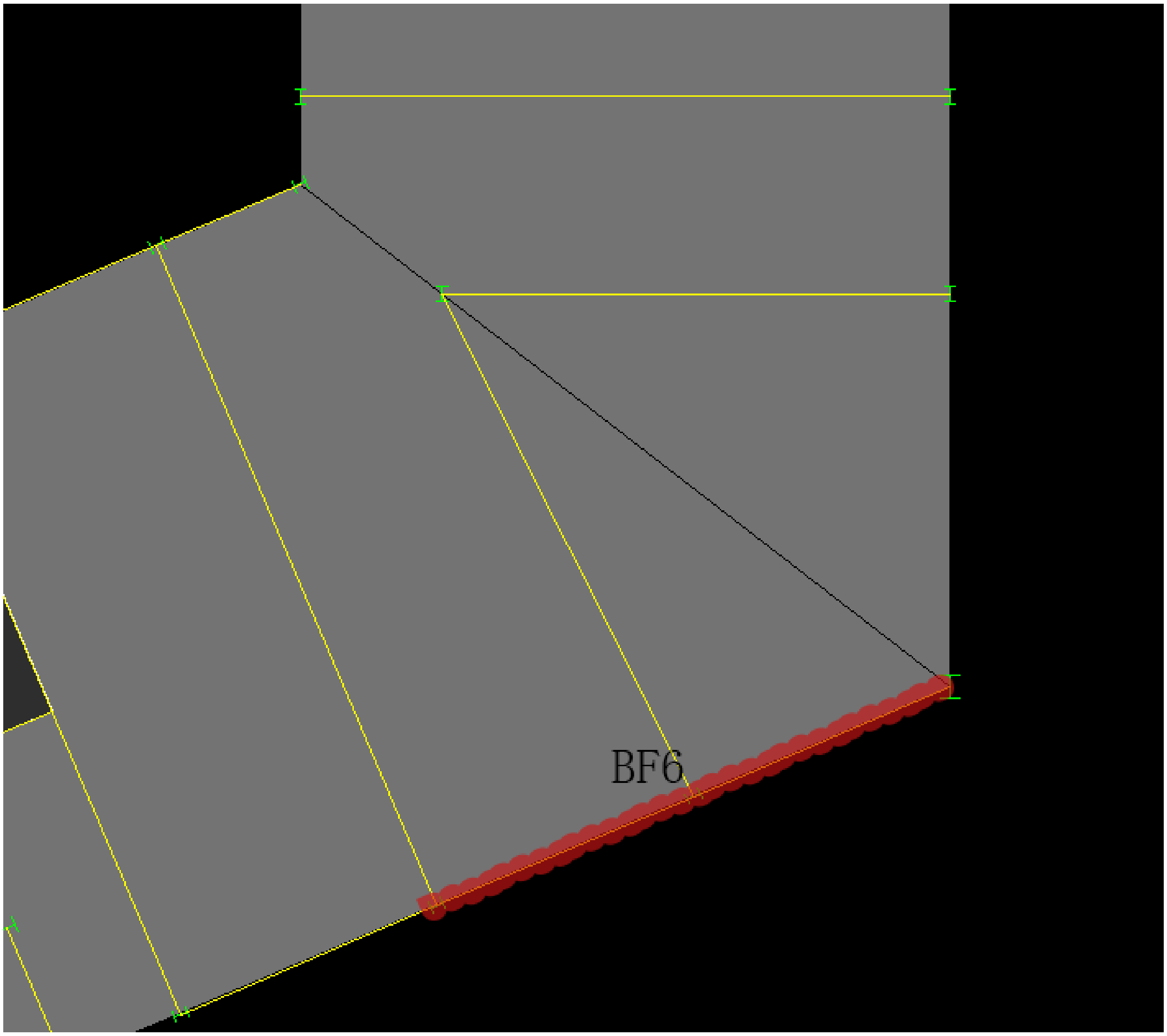
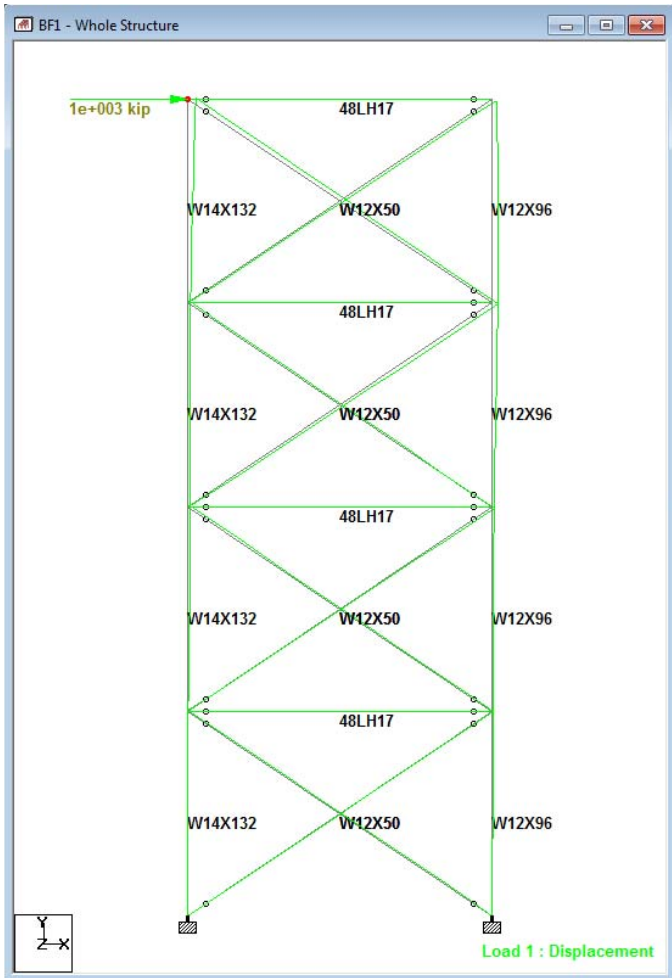



Figure 18: braced frame 6 of the proposed design




BF1 - Node Displacements:

All / Summary

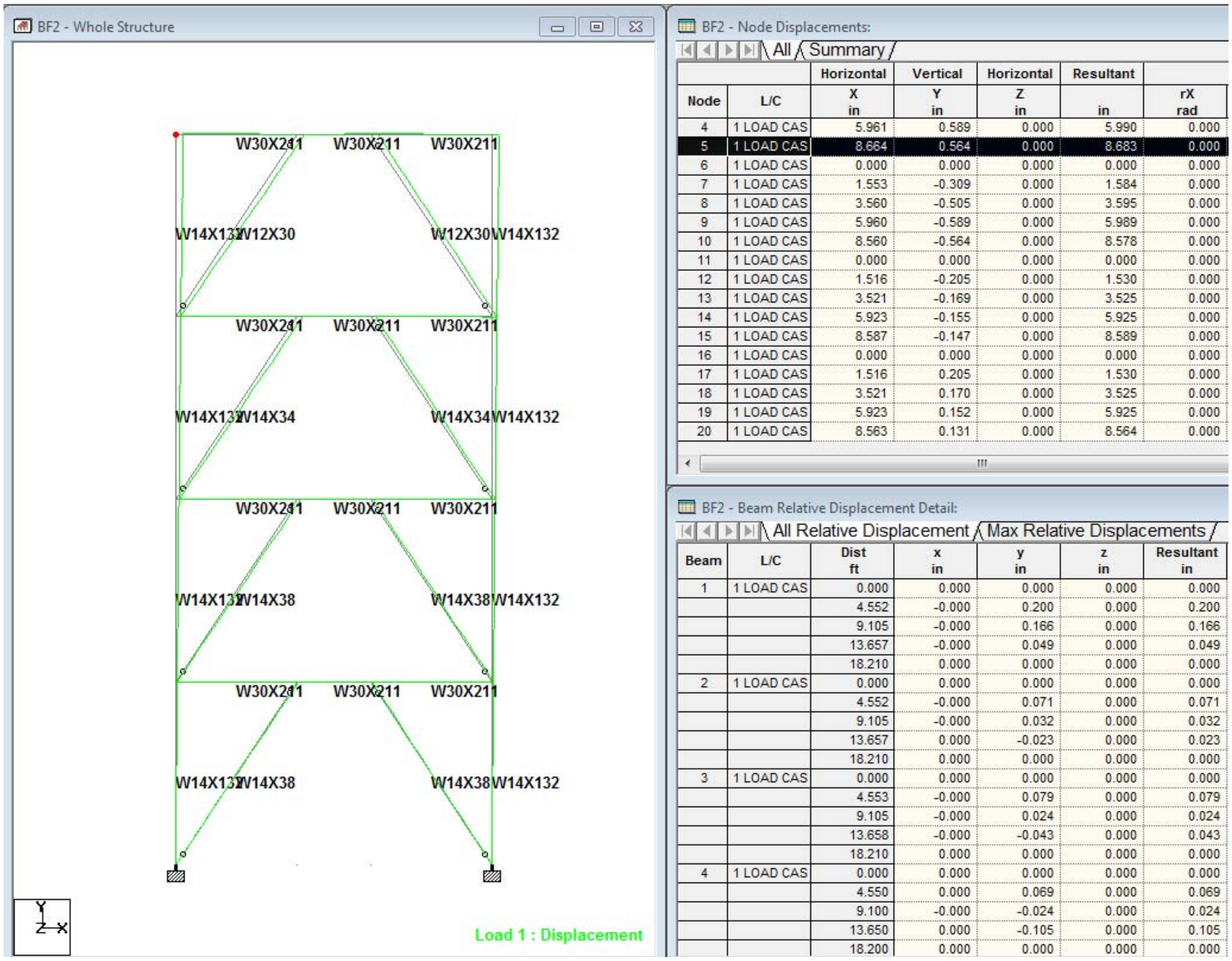
| Node | L/C | Horizontal X in | Vertical Y in | Horizontal Z in | Resultant in | rX rad |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.661 | 0.426 | 0.000 | 0.786 | 0.000 |
| 3 | 1 LOAD CAS | 3.399 | 0.788 | 0.000 | 3.489 | 0.000 |
| 4 | 1 LOAD CAS | 3.453 | 0.933 | 0.000 | 3.577 | 0.000 |
| 5 | 1 LOAD CAS | 8.979 | 1.058 | 0.000 | 9.041 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 1.397 | -0.663 | 0.000 | 1.547 | 0.000 |
| 8 | 1 LOAD CAS | 1.936 | -1.061 | 0.000 | 2.208 | 0.000 |
| 9 | 1 LOAD CAS | 6.056 | -1.397 | 0.000 | 6.215 | 0.000 |
| 10 | 1 LOAD CAS | 5.046 | -1.401 | 0.000 | 5.237 | 0.000 |

BF1 - Beam Relative Displacement Detail:

All Relative Displacement / Max Relative Displacements

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | -0.000 | 0.172 | 0.000 | 0.172 |
| | | 9.105 | -0.000 | 0.281 | 0.000 | 0.281 |
| | | 13.657 | -0.000 | 0.250 | 0.000 | 0.250 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | 0.000 | -0.022 | 0.000 | 0.022 |
| | | 9.105 | -0.000 | -0.155 | 0.000 | 0.155 |
| | | 13.657 | -0.000 | -0.210 | 0.000 | 0.210 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 3 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.553 | 0.000 | -0.019 | 0.000 | 0.019 |
| | | 9.105 | -0.000 | 0.151 | 0.000 | 0.151 |
| | | 13.658 | -0.000 | 0.246 | 0.000 | 0.246 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 4 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.550 | -0.000 | 0.529 | 0.000 | 0.529 |
| | | 9.100 | -0.000 | 0.604 | 0.000 | 0.604 |
| | | 13.650 | -0.000 | 0.378 | 0.000 | 0.378 |
| | | 18.200 | 0.000 | 0.000 | 0.000 | 0.000 |

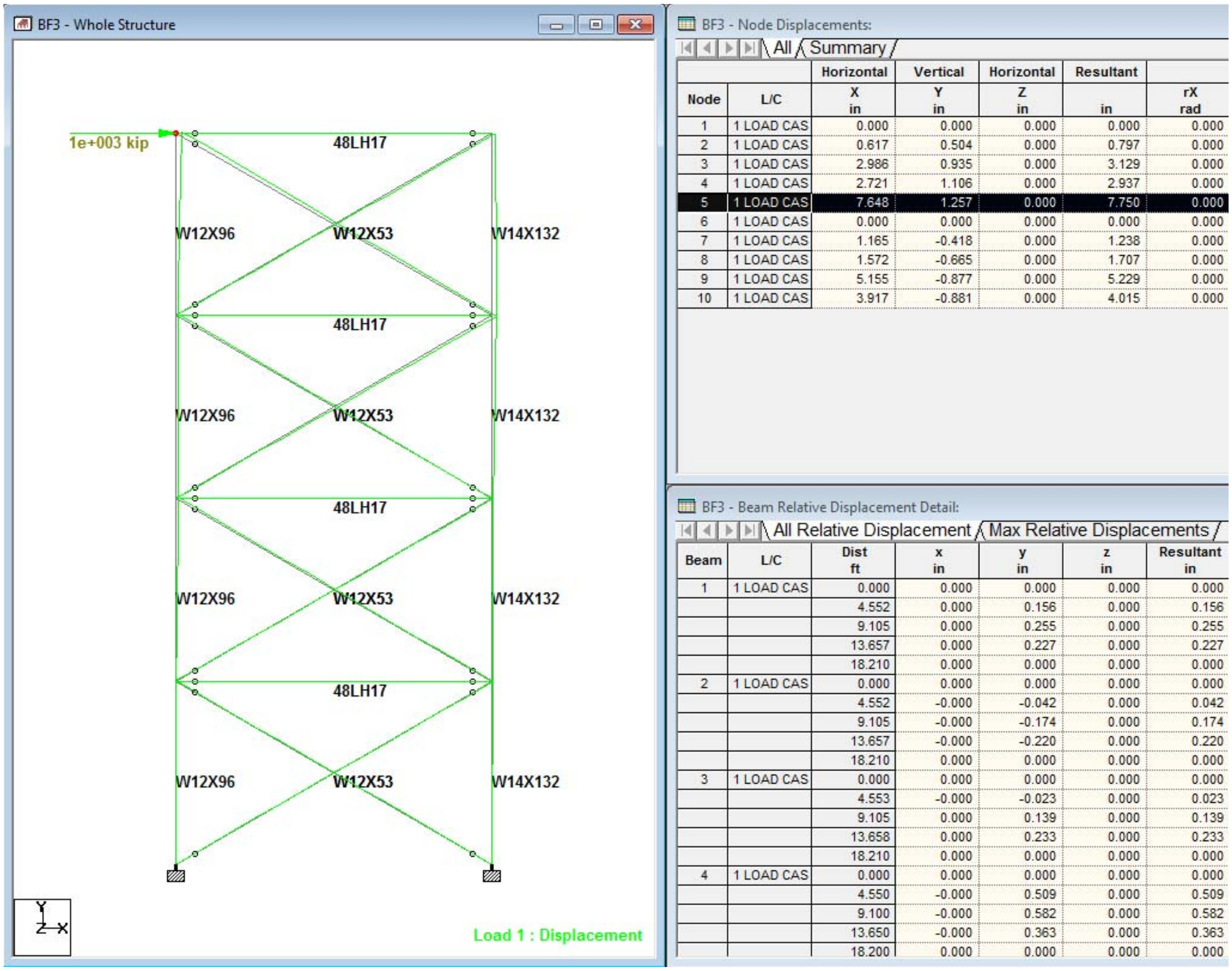
FIGURE 19: BRACED FRAME 1 OF PROPOSED DESIGN




| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 4 | 1 LOAD CAS | 5.961 | 0.589 | 0.000 | 5.990 | 0.000 |
| 5 | 1 LOAD CAS | 8.664 | 0.564 | 0.000 | 8.683 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 1.553 | -0.309 | 0.000 | 1.584 | 0.000 |
| 8 | 1 LOAD CAS | 3.560 | -0.505 | 0.000 | 3.595 | 0.000 |
| 9 | 1 LOAD CAS | 5.960 | -0.589 | 0.000 | 5.989 | 0.000 |
| 10 | 1 LOAD CAS | 8.560 | -0.564 | 0.000 | 8.578 | 0.000 |
| 11 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 12 | 1 LOAD CAS | 1.516 | -0.205 | 0.000 | 1.530 | 0.000 |
| 13 | 1 LOAD CAS | 3.521 | -0.169 | 0.000 | 3.525 | 0.000 |
| 14 | 1 LOAD CAS | 5.923 | -0.155 | 0.000 | 5.925 | 0.000 |
| 15 | 1 LOAD CAS | 8.587 | -0.147 | 0.000 | 8.589 | 0.000 |
| 16 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 17 | 1 LOAD CAS | 1.516 | 0.205 | 0.000 | 1.530 | 0.000 |
| 18 | 1 LOAD CAS | 3.521 | 0.170 | 0.000 | 3.525 | 0.000 |
| 19 | 1 LOAD CAS | 5.923 | 0.152 | 0.000 | 5.925 | 0.000 |
| 20 | 1 LOAD CAS | 8.563 | 0.131 | 0.000 | 8.564 | 0.000 |

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | -0.000 | 0.200 | 0.000 | 0.200 |
| | | 9.105 | -0.000 | 0.166 | 0.000 | 0.166 |
| | | 13.657 | -0.000 | 0.049 | 0.000 | 0.049 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | -0.000 | 0.071 | 0.000 | 0.071 |
| | | 9.105 | -0.000 | 0.032 | 0.000 | 0.032 |
| | | 13.657 | 0.000 | -0.023 | 0.000 | 0.023 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 3 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.553 | -0.000 | 0.079 | 0.000 | 0.079 |
| | | 9.105 | -0.000 | 0.024 | 0.000 | 0.024 |
| | | 13.658 | -0.000 | -0.043 | 0.000 | 0.043 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 4 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.550 | 0.000 | 0.069 | 0.000 | 0.069 |
| | | 9.100 | -0.000 | -0.024 | 0.000 | 0.024 |
| | | 13.650 | 0.000 | -0.105 | 0.000 | 0.105 |
| | | 18.200 | 0.000 | 0.000 | 0.000 | 0.000 |

Figure 20: braced frame 2 of proposed design




BF3 - Node Displacements:

All / Summary

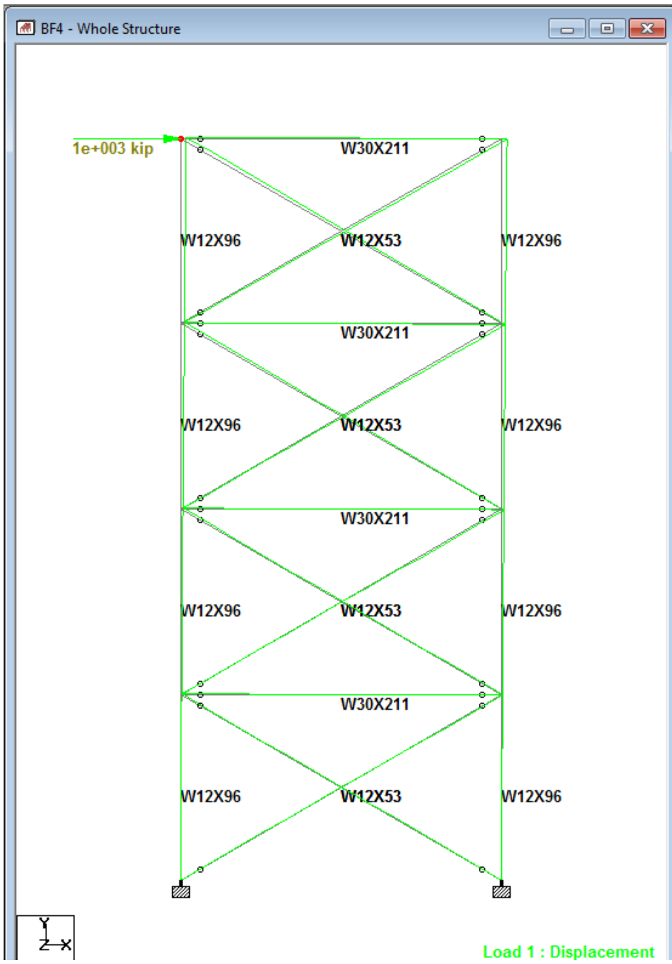
| Node | L/C | Horizontal X in | Vertical Y in | Horizontal Z in | Resultant in | rX rad |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.617 | 0.504 | 0.000 | 0.797 | 0.000 |
| 3 | 1 LOAD CAS | 2.986 | 0.935 | 0.000 | 3.129 | 0.000 |
| 4 | 1 LOAD CAS | 2.721 | 1.106 | 0.000 | 2.937 | 0.000 |
| 5 | 1 LOAD CAS | 7.648 | 1.257 | 0.000 | 7.750 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 1.165 | -0.418 | 0.000 | 1.238 | 0.000 |
| 8 | 1 LOAD CAS | 1.572 | -0.665 | 0.000 | 1.707 | 0.000 |
| 9 | 1 LOAD CAS | 5.155 | -0.877 | 0.000 | 5.229 | 0.000 |
| 10 | 1 LOAD CAS | 3.917 | -0.881 | 0.000 | 4.015 | 0.000 |

BF3 - Beam Relative Displacement Detail:

All Relative Displacement / Max Relative Displacements

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | 0.000 | 0.156 | 0.000 | 0.156 |
| | | 9.105 | 0.000 | 0.255 | 0.000 | 0.255 |
| | | 13.657 | 0.000 | 0.227 | 0.000 | 0.227 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | -0.000 | -0.042 | 0.000 | 0.042 |
| | | 9.105 | -0.000 | -0.174 | 0.000 | 0.174 |
| | | 13.657 | -0.000 | -0.220 | 0.000 | 0.220 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 3 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.553 | -0.000 | -0.023 | 0.000 | 0.023 |
| | | 9.105 | 0.000 | 0.139 | 0.000 | 0.139 |
| | | 13.658 | 0.000 | 0.233 | 0.000 | 0.233 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 4 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.550 | -0.000 | 0.509 | 0.000 | 0.509 |
| | | 9.100 | -0.000 | 0.582 | 0.000 | 0.582 |
| | | 13.650 | -0.000 | 0.363 | 0.000 | 0.363 |
| | | 18.200 | 0.000 | 0.000 | 0.000 | 0.000 |

FIGURE 21: BRACED FRAME 3 OF PROPOSED DESIGN




BF4 - Node Displacements:

All / Summary

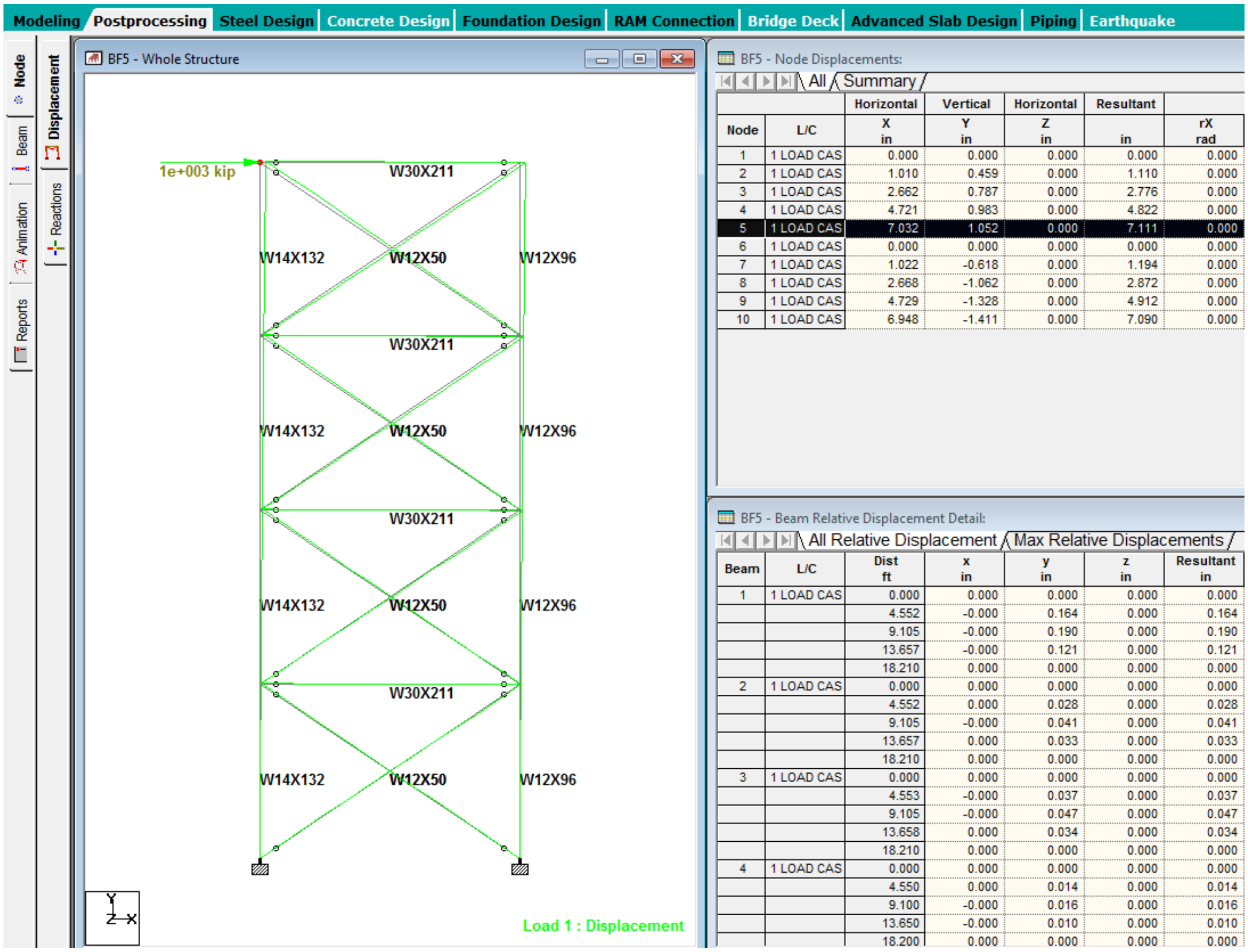
| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.948 | 0.540 | 0.000 | 1.090 | 0.000 |
| 3 | 1 LOAD CAS | 2.440 | 0.925 | 0.000 | 2.609 | 0.000 |
| 4 | 1 LOAD CAS | 4.283 | 1.157 | 0.000 | 4.436 | 0.000 |
| 5 | 1 LOAD CAS | 6.360 | 1.239 | 0.000 | 6.479 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 0.948 | -0.540 | 0.000 | 1.090 | 0.000 |
| 8 | 1 LOAD CAS | 2.439 | -0.925 | 0.000 | 2.609 | 0.000 |
| 9 | 1 LOAD CAS | 4.289 | -1.157 | 0.000 | 4.443 | 0.000 |
| 10 | 1 LOAD CAS | 6.262 | -1.229 | 0.000 | 6.381 | 0.000 |

BF4 - Beam Relative Displacement Detail:

All Relative Displacement / Max Relative Displacements

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| | | 9.096 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 18.192 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 27.289 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 36.385 | 0.000 | 0.000 | 0.000 | 0.000 |
| 18 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 9.096 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 18.192 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 27.289 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 36.385 | 0.000 | 0.000 | 0.000 | 0.000 |
| 19 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 9.095 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 18.190 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 27.285 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 36.380 | 0.000 | 0.000 | 0.000 | 0.000 |
| 20 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 9.095 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 18.190 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 27.285 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 36.380 | 0.000 | 0.000 | 0.000 | 0.000 |

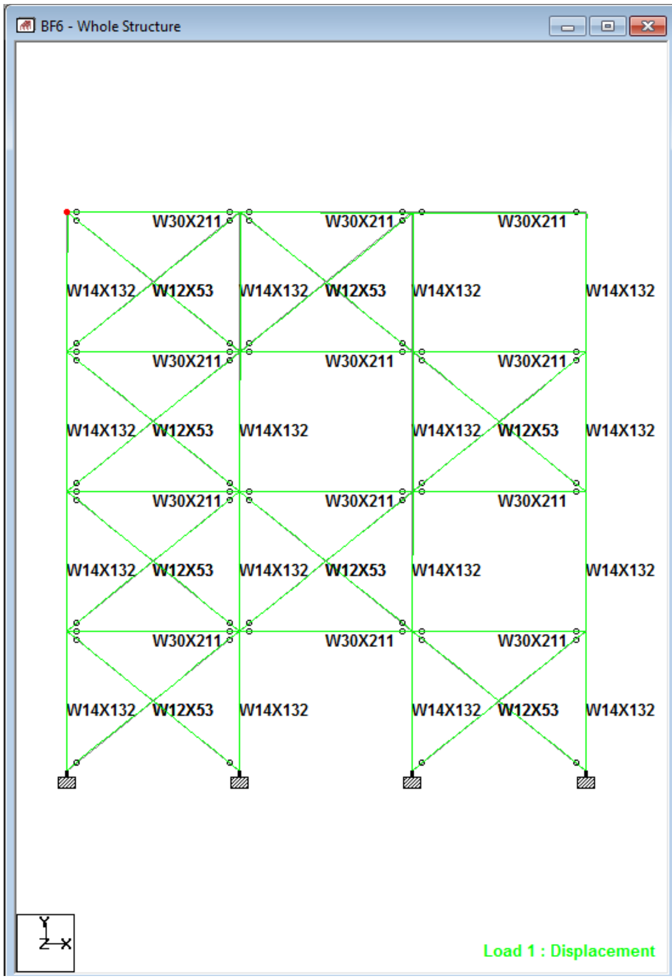
Figure 22: braced frame 4 of proposed design




| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 1.010 | 0.459 | 0.000 | 1.110 | 0.000 |
| 3 | 1 LOAD CAS | 2.662 | 0.787 | 0.000 | 2.776 | 0.000 |
| 4 | 1 LOAD CAS | 4.721 | 0.983 | 0.000 | 4.822 | 0.000 |
| 5 | 1 LOAD CAS | 7.032 | 1.052 | 0.000 | 7.111 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 1.022 | -0.618 | 0.000 | 1.194 | 0.000 |
| 8 | 1 LOAD CAS | 2.668 | -1.062 | 0.000 | 2.872 | 0.000 |
| 9 | 1 LOAD CAS | 4.729 | -1.328 | 0.000 | 4.912 | 0.000 |
| 10 | 1 LOAD CAS | 6.948 | -1.411 | 0.000 | 7.090 | 0.000 |

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | -0.000 | 0.164 | 0.000 | 0.164 |
| | | 9.105 | -0.000 | 0.190 | 0.000 | 0.190 |
| | | 13.657 | -0.000 | 0.121 | 0.000 | 0.121 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | 0.000 | 0.028 | 0.000 | 0.028 |
| | | 9.105 | -0.000 | 0.041 | 0.000 | 0.041 |
| | | 13.657 | 0.000 | 0.033 | 0.000 | 0.033 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 3 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.553 | -0.000 | 0.037 | 0.000 | 0.037 |
| | | 9.105 | -0.000 | 0.047 | 0.000 | 0.047 |
| | | 13.658 | 0.000 | 0.034 | 0.000 | 0.034 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 4 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.550 | 0.000 | 0.014 | 0.000 | 0.014 |
| | | 9.100 | -0.000 | 0.016 | 0.000 | 0.016 |
| | | 13.650 | -0.000 | 0.010 | 0.000 | 0.010 |
| | | 18.200 | 0.000 | 0.000 | 0.000 | 0.000 |

Figure 23: Braced frame 5 of proposed design




BF6 - Node Displacements:

All / Summary

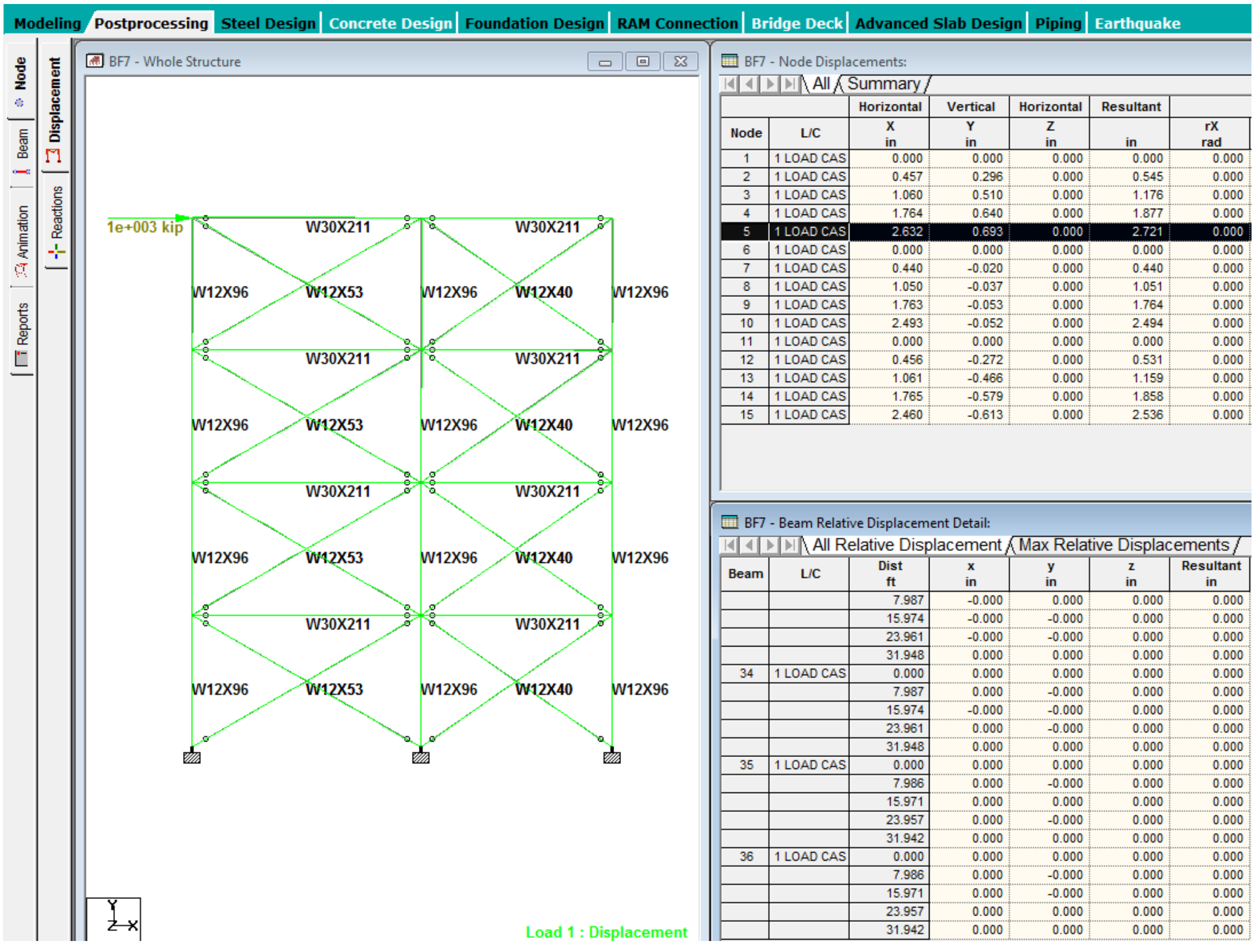
| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 3 | 1 LOAD CAS | 0.845 | 0.344 | 0.000 | 0.912 | 0.000 |
| 4 | 1 LOAD CAS | 1.417 | 0.430 | 0.000 | 1.481 | 0.000 |
| 5 | 1 LOAD CAS | 2.078 | 0.464 | 0.000 | 2.129 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 0.388 | -0.018 | 0.000 | 0.388 | 0.000 |
| 8 | 1 LOAD CAS | 0.830 | -0.020 | 0.000 | 0.830 | 0.000 |
| 9 | 1 LOAD CAS | 1.419 | -0.017 | 0.000 | 1.419 | 0.000 |
| 10 | 1 LOAD CAS | 1.959 | 0.012 | 0.000 | 1.959 | 0.000 |
| 11 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 12 | 1 LOAD CAS | 0.362 | -0.057 | 0.000 | 0.367 | 0.000 |
| 13 | 1 LOAD CAS | 0.871 | -0.111 | 0.000 | 0.878 | 0.000 |
| 14 | 1 LOAD CAS | 1.382 | -0.158 | 0.000 | 1.391 | 0.000 |
| 15 | 1 LOAD CAS | 1.918 | -0.201 | 0.000 | 1.928 | 0.000 |
| 16 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 17 | 1 LOAD CAS | 0.334 | -0.123 | 0.000 | 0.356 | 0.000 |
| 18 | 1 LOAD CAS | 0.920 | -0.217 | 0.000 | 0.945 | 0.000 |
| 19 | 1 LOAD CAS | 1.341 | -0.260 | 0.000 | 1.366 | 0.000 |
| 20 | 1 LOAD CAS | 1.917 | -0.260 | 0.000 | 1.935 | 0.000 |

BF6 - Beam Relative Displacement Detail:

All Relative Displacement / Max Relative Displacements

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | -0.000 | 0.059 | 0.000 | 0.059 |
| | | 9.105 | -0.000 | 0.063 | 0.000 | 0.063 |
| | | 13.657 | 0.000 | 0.036 | 0.000 | 0.036 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.552 | 0.000 | -0.005 | 0.000 | 0.005 |
| | | 9.105 | -0.000 | -0.002 | 0.000 | 0.002 |
| | | 13.657 | 0.000 | 0.002 | 0.000 | 0.002 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 3 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.553 | 0.000 | 0.015 | 0.000 | 0.015 |
| | | 9.105 | -0.000 | 0.018 | 0.000 | 0.018 |
| | | 13.658 | -0.000 | 0.013 | 0.000 | 0.013 |
| | | 18.210 | 0.000 | 0.000 | 0.000 | 0.000 |
| 4 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 4.550 | 0.000 | 0.004 | 0.000 | 0.004 |
| | | 9.100 | -0.000 | 0.005 | 0.000 | 0.005 |
| | | 13.650 | 0.000 | 0.003 | 0.000 | 0.003 |
| | | 18.200 | 0.000 | 0.000 | 0.000 | 0.000 |

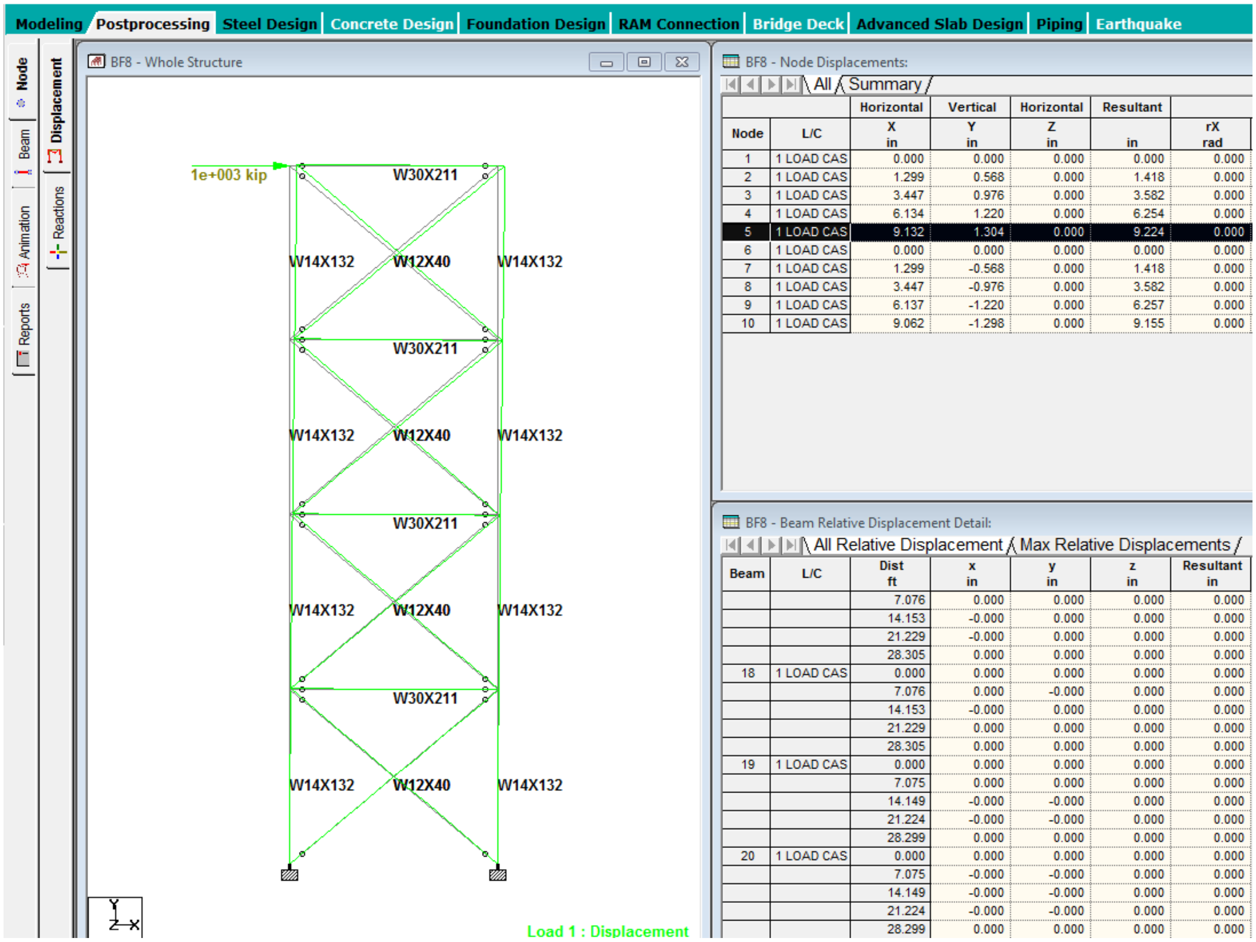
Figure 24: braced frame 6 of proposed design




| Node | L/C | Horizontal X in | Vertical Y in | Horizontal Z in | Resultant in | rX rad |
|---|---|---|---|---|---|---|
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.457 | 0.296 | 0.000 | 0.545 | 0.000 |
| 3 | 1 LOAD CAS | 1.060 | 0.510 | 0.000 | 1.176 | 0.000 |
| 4 | 1 LOAD CAS | 1.764 | 0.640 | 0.000 | 1.877 | 0.000 |
| 5 | 1 LOAD CAS | 2.632 | 0.693 | 0.000 | 2.721 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 0.440 | -0.020 | 0.000 | 0.440 | 0.000 |
| 8 | 1 LOAD CAS | 1.050 | -0.037 | 0.000 | 1.051 | 0.000 |
| 9 | 1 LOAD CAS | 1.763 | -0.053 | 0.000 | 1.764 | 0.000 |
| 10 | 1 LOAD CAS | 2.493 | -0.052 | 0.000 | 2.494 | 0.000 |
| 11 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 12 | 1 LOAD CAS | 0.456 | -0.272 | 0.000 | 0.531 | 0.000 |
| 13 | 1 LOAD CAS | 1.061 | -0.466 | 0.000 | 1.159 | 0.000 |
| 14 | 1 LOAD CAS | 1.765 | -0.579 | 0.000 | 1.858 | 0.000 |
| 15 | 1 LOAD CAS | 2.460 | -0.613 | 0.000 | 2.536 | 0.000 |

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| | | 7.987 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 15.974 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 23.961 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 31.948 | 0.000 | 0.000 | 0.000 | 0.000 |
| 34 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.987 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 15.974 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 23.961 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 31.948 | 0.000 | 0.000 | 0.000 | 0.000 |
| 35 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.986 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 15.971 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 23.957 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 31.942 | 0.000 | 0.000 | 0.000 | 0.000 |
| 36 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.986 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 15.971 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 23.957 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 31.942 | 0.000 | 0.000 | 0.000 | 0.000 |

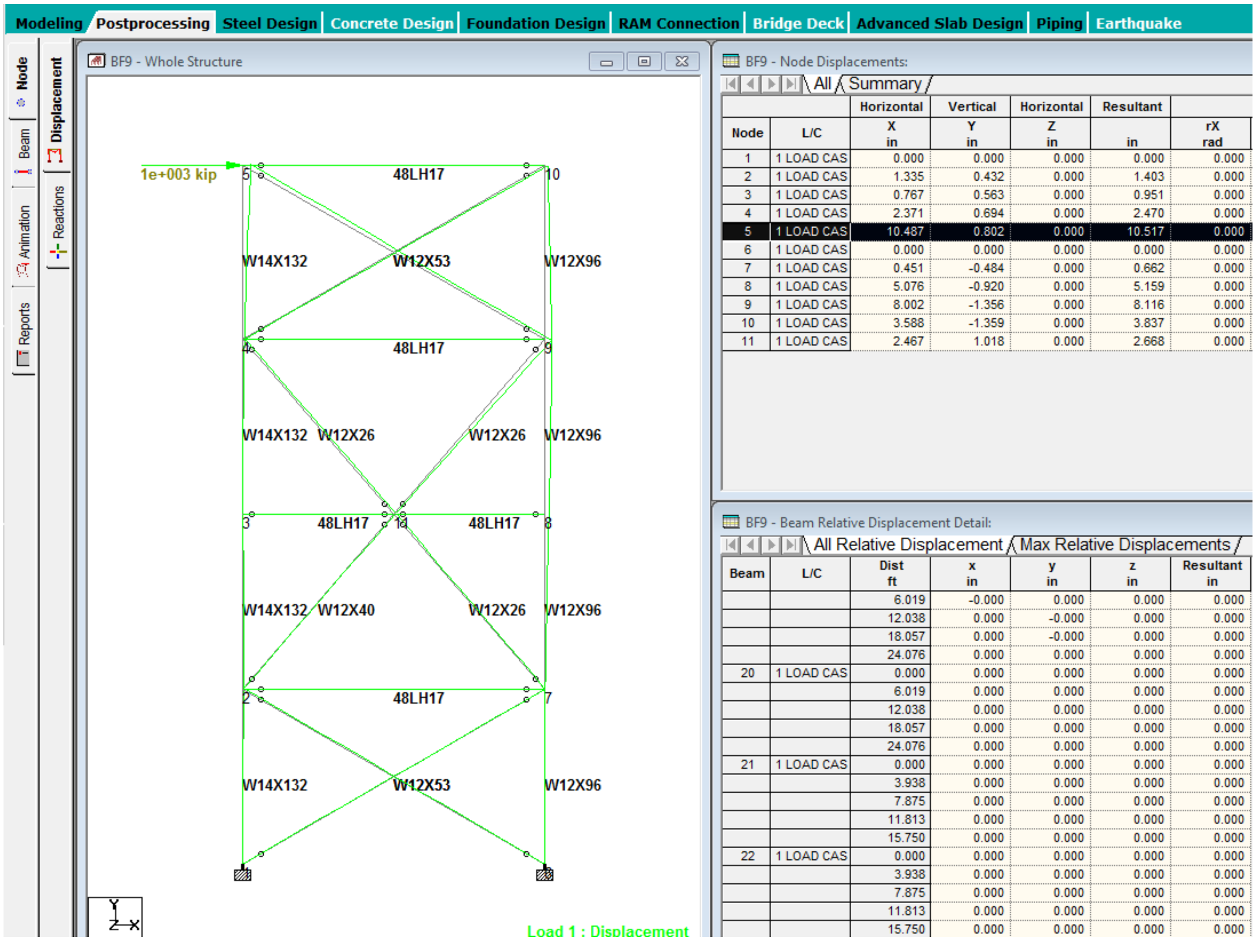
FIGURE 25: BRACED FRAME 7 OF PROPOSED DESIGN




| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 1.299 | 0.568 | 0.000 | 1.418 | 0.000 |
| 3 | 1 LOAD CAS | 3.447 | 0.976 | 0.000 | 3.582 | 0.000 |
| 4 | 1 LOAD CAS | 6.134 | 1.220 | 0.000 | 6.254 | 0.000 |
| 5 | 1 LOAD CAS | 9.132 | 1.304 | 0.000 | 9.224 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 1.299 | -0.568 | 0.000 | 1.418 | 0.000 |
| 8 | 1 LOAD CAS | 3.447 | -0.976 | 0.000 | 3.582 | 0.000 |
| 9 | 1 LOAD CAS | 6.137 | -1.220 | 0.000 | 6.257 | 0.000 |
| 10 | 1 LOAD CAS | 9.062 | -1.298 | 0.000 | 9.155 | 0.000 |

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| | | 7.076 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 14.153 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 21.229 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 28.305 | 0.000 | 0.000 | 0.000 | 0.000 |
| 18 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.076 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 14.153 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 21.229 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 28.305 | 0.000 | 0.000 | 0.000 | 0.000 |
| 19 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.075 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 14.149 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 21.224 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 28.299 | 0.000 | 0.000 | 0.000 | 0.000 |
| 20 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.075 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 14.149 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 21.224 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 28.299 | 0.000 | 0.000 | 0.000 | 0.000 |

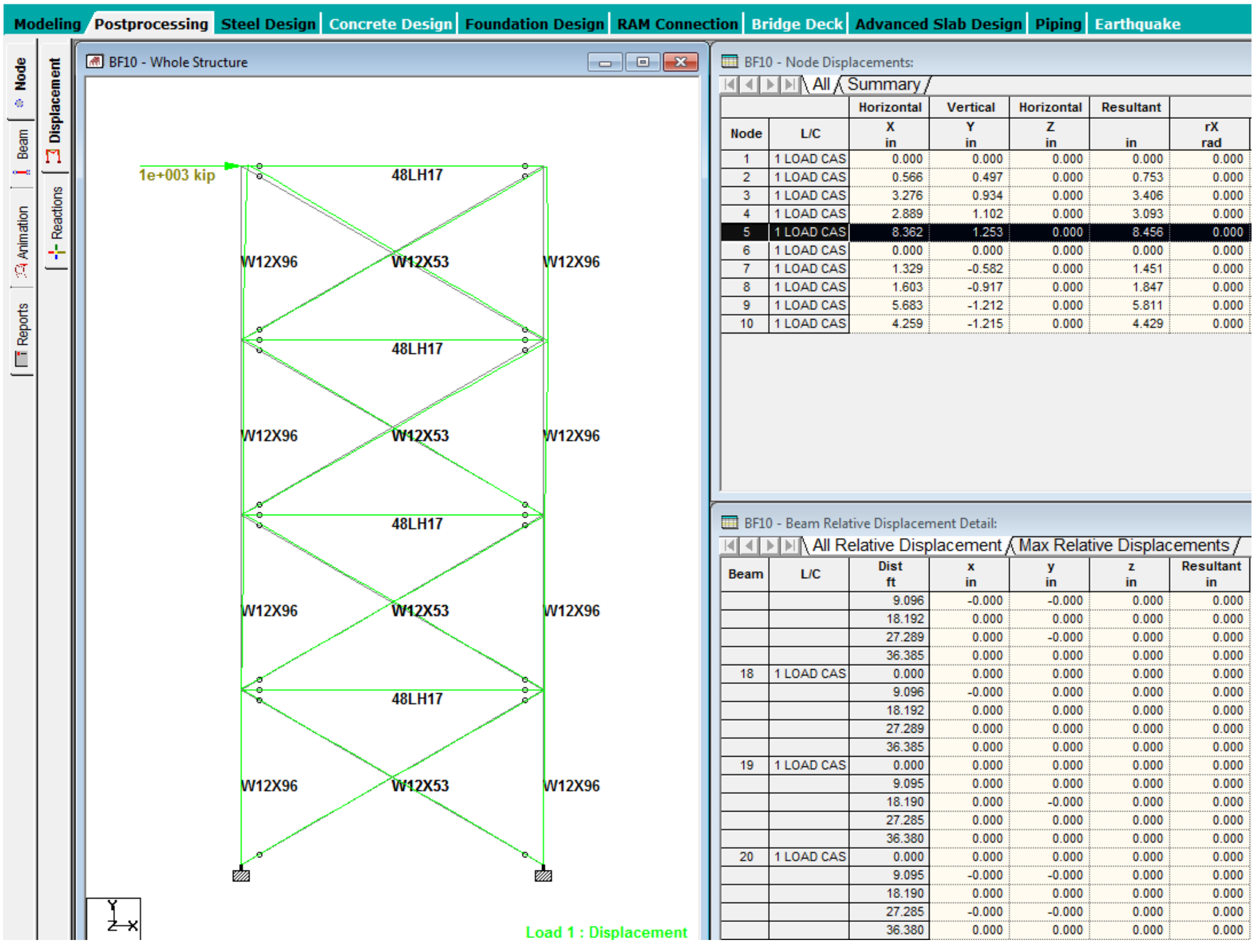
FIGURE 26: BRACED FRAME 8 OF PROPOSED DESIGN




| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 1.335 | 0.432 | 0.000 | 1.403 | 0.000 |
| 3 | 1 LOAD CAS | 0.767 | 0.563 | 0.000 | 0.951 | 0.000 |
| 4 | 1 LOAD CAS | 2.371 | 0.694 | 0.000 | 2.470 | 0.000 |
| 5 | 1 LOAD CAS | 10.487 | 0.802 | 0.000 | 10.517 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 0.451 | -0.484 | 0.000 | 0.662 | 0.000 |
| 8 | 1 LOAD CAS | 5.076 | -0.920 | 0.000 | 5.159 | 0.000 |
| 9 | 1 LOAD CAS | 8.002 | -1.356 | 0.000 | 8.116 | 0.000 |
| 10 | 1 LOAD CAS | 3.588 | -1.359 | 0.000 | 3.837 | 0.000 |
| 11 | 1 LOAD CAS | 2.467 | 1.018 | 0.000 | 2.668 | 0.000 |

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| | | 6.019 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 12.038 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 18.057 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 24.076 | 0.000 | 0.000 | 0.000 | 0.000 |
| 20 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 6.019 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 12.038 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 18.057 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 24.076 | 0.000 | 0.000 | 0.000 | 0.000 |
| 21 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 3.938 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.875 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 11.813 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 15.750 | 0.000 | 0.000 | 0.000 | 0.000 |
| 22 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 3.938 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 7.875 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 11.813 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 15.750 | 0.000 | 0.000 | 0.000 | 0.000 |

FIGURE 27: BRACED FRAME 9 OF PROPOSED DESIGN




| | | Horizontal | Vertical | Horizontal | Resultant | |
|---|---|---|---|---|---|---|
| Node | L/C | X in | Y in | Z in | in | rX rad |
| 1 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2 | 1 LOAD CAS | 0.566 | 0.497 | 0.000 | 0.753 | 0.000 |
| 3 | 1 LOAD CAS | 3.276 | 0.934 | 0.000 | 3.406 | 0.000 |
| 4 | 1 LOAD CAS | 2.889 | 1.102 | 0.000 | 3.093 | 0.000 |
| 5 | 1 LOAD CAS | 8.362 | 1.253 | 0.000 | 8.456 | 0.000 |
| 6 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 7 | 1 LOAD CAS | 1.329 | -0.582 | 0.000 | 1.451 | 0.000 |
| 8 | 1 LOAD CAS | 1.603 | -0.917 | 0.000 | 1.847 | 0.000 |
| 9 | 1 LOAD CAS | 5.683 | -1.212 | 0.000 | 5.811 | 0.000 |
| 10 | 1 LOAD CAS | 4.259 | -1.215 | 0.000 | 4.429 | 0.000 |

| Beam | L/C | Dist ft | x in | y in | z in | Resultant in |
|---|---|---|---|---|---|---|
| | | 9.096 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 18.192 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 27.289 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 36.385 | 0.000 | 0.000 | 0.000 | 0.000 |
| 18 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 9.096 | -0.000 | 0.000 | 0.000 | 0.000 |
| | | 18.192 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 27.289 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 36.385 | 0.000 | 0.000 | 0.000 | 0.000 |
| 19 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 9.095 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 18.190 | 0.000 | -0.000 | 0.000 | 0.000 |
| | | 27.285 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 36.380 | 0.000 | 0.000 | 0.000 | 0.000 |
| 20 | 1 LOAD CAS | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 9.095 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 18.190 | 0.000 | 0.000 | 0.000 | 0.000 |
| | | 27.285 | -0.000 | -0.000 | 0.000 | 0.000 |
| | | 36.380 | 0.000 | 0.000 | 0.000 | 0.000 |

FIGURE 28: BRACED FRAME 10 OF PROPOSED DESIGN

## ETABS Model

After all the required loads were determined, the proposed structural system was modeled in ETABS. Several assumptions were made when creating the model. The members were modeled as line elements and a list of member sizes were added to the auto selection list for the program to select the optimal member sizes based on the inputted loads. The base supports were modeled as fixed since the existing foundations were oversized for the proposed design because of its lighter weight. The floor slab was modeled as a rigid diaphragm and assigned a 3VLI20 composite deck to its area element. The self-mass of the floor slab were ignored in the material definitions and an additional self-weight was applied to the floor-slabs. The moment of the braces and beams ends were released since there were no moment connections. The columns were assumed to be continuous throughout full building height.

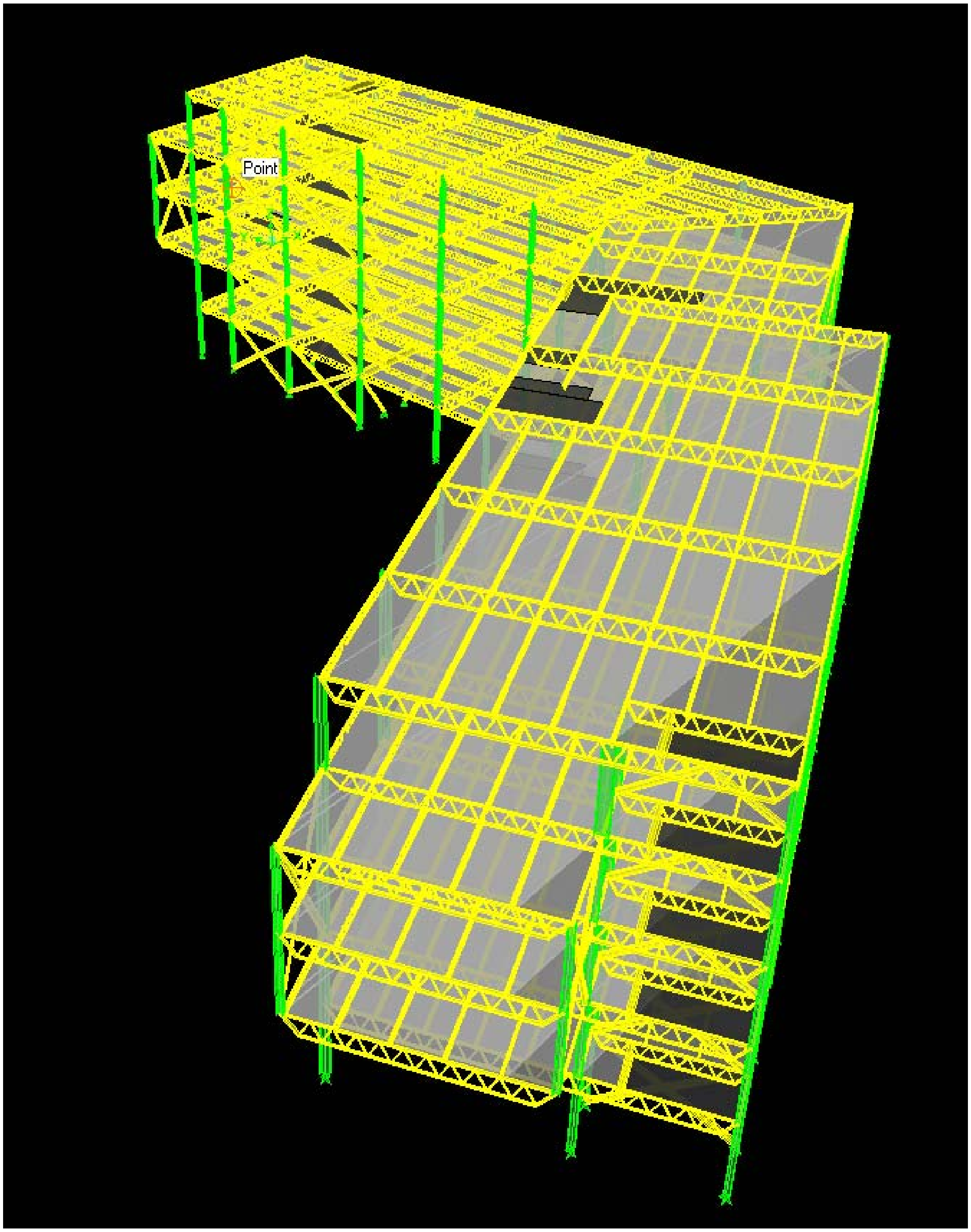



FIGURE 29: ETABS MODEL OF PROPOSED DESIGN

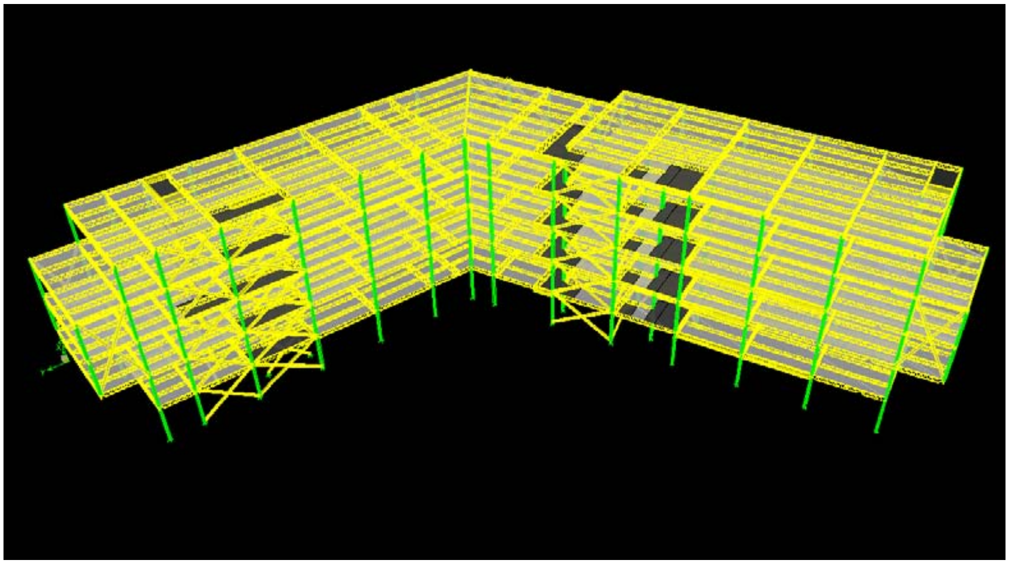


FIGURE 30: ETABS MODEL OF PROPOSED DESIGN

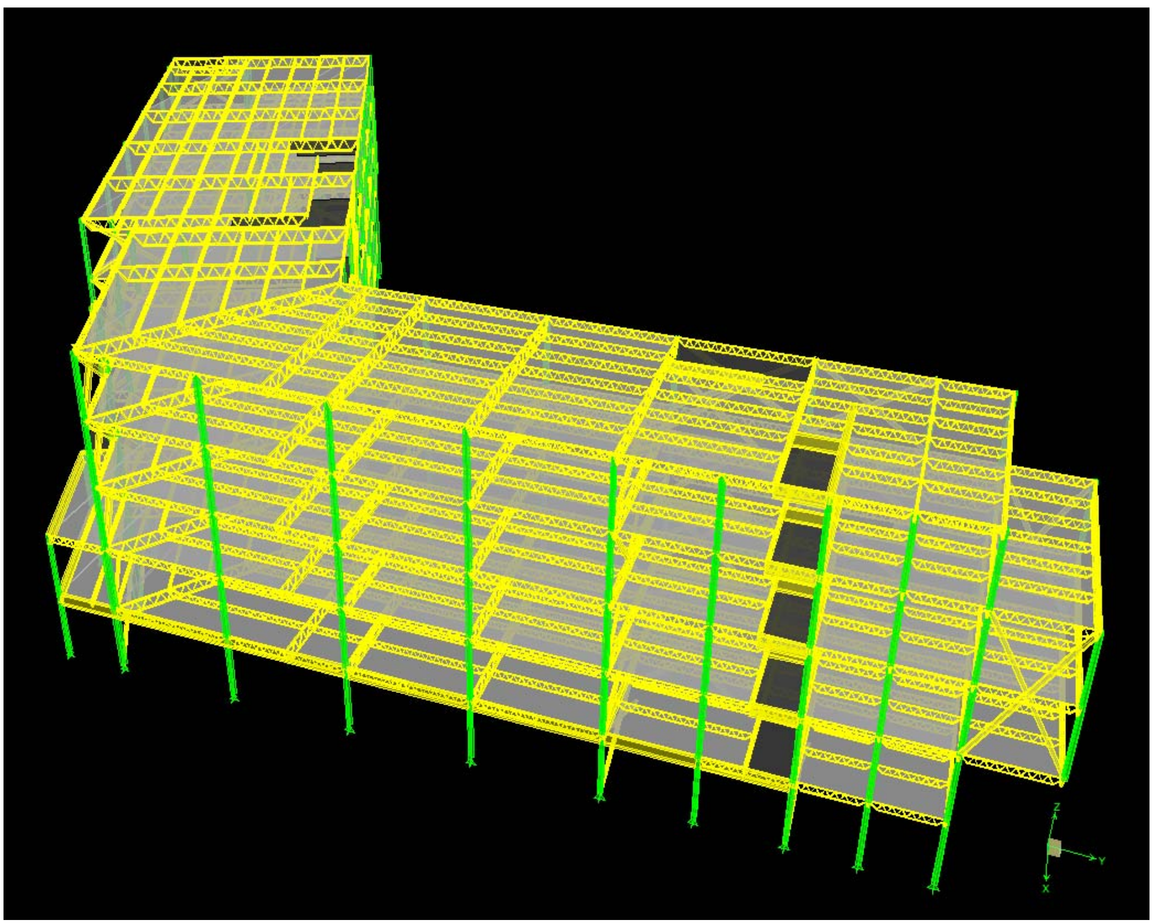



FIGURE 31: ETABS MODEL OF PROPOSED DESIGN

## Center of Mass and Rigidity

In order to check the accuracy of the ETABS model, the center of rigidity and the center of mass for level four of the building were calculated in excel, then compared to the outputs from the ETABS model. There outputs were given in table 16.

The center of rigidity was defined as the location at which an applied load would not cause any torsion. In order to calculate rigidity, the stiffness of each lateral resisting element was first determined in STAAD with an applied 1000k load at the top of the frame. The maximum horizontal displacement was then obtained for each frame and stiffness can be determined with the equation $k=F/\delta$. Since the layout of the braced frames were not orthogonal to each other (see figure 16), the stiffness of each wall was further separated into X and Y components. By separating the k value of the braced frames, it was treated as two orthogonal frames that resisted lateral loads in both X and Y direction. The X component of stiffness was obtained by multiplying the k value with sine of 115 degree and the Y component was obtained by multiplying the k value with negative cosine of 115 degree.

Similar to the analysis in technical report 3, a center position of the braced frame was needed for the calculation of the center of rigidity. The coordinates of the frames was determined by linking the structural drawings into AutoCAD and defining the origin at the top left corner of the building. Then a relative accurate position of the frames was found using the measure tool in the program. Compared to the output from ETABS, it was noted that a relative error of about 2% was found for the center of rigidity in both X and Y direction. This difference was probably due to the inaccuracy in determining the center position of the braced frames. Again, using AutoCAD to obtain the frame's position might be slightly different than the position coordinates ETABS used into its calculation.

The center of mass was found by taking the sum of the weight of the lateral resisting elements and the floor slab multiply by its relative position obtained in AutoCAD. Then divide that number by the weight of all those elements. Compared to ETABS output, the hand calculation for the center of mass produced a relative error of 4% in the X direction and less than 2% in the Y direction. In conclusion, the hand check of the center of mass and the center of rigidity for Piez Hall proved that the ETABS model was an accurate model.

| Stiffness and Coordinate Position of Braced Frames | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Label** | **Applied Force (kip)** | **Displacement (in)** | **Stiffness, K (k/in)** | **X direction, Kx (k/in)** | **Y direction, Ky (k/in)** | **X position of wall (in)** | **Y position of wall (in)** |
| BF 1 | 1000 | 8.98 | 111.37 | 43.43 | 102.46 | 2759.90 | 4031.86 |
| BF 2 | 1000 | 8.66 | 115.42 | 106.19 | 45.01 | 1655.94 | 3263.88 |
| BF 3 | 1000 | 7.65 | 130.75 | 50.99 | 120.29 | 749.05 | 1247.96 |
| BF 4 | 1000 | 6.36 | 157.23 | 144.65 | 61.32 | 1221.24 | 2885.88 |
| BF 5 | 1000 | 7.03 | 142.21 | 55.46 | 130.83 | 883.17 | 3108.26 |
| BF 6 | 1000 | 2.08 | 481.23 | 442.73 | 187.68 | 693.16 | 2793.37 |
| BF 7 | 1000 | 2.63 | 379.94 | 0.00 | 379.94 | 726.80 | 737.00 |
| BF 8 | 1000 | 7.03 | 142.25 | 0.00 | 142.25 | 567.81 | 985.00 |
| BF 9 | 1000 | 9.13 | 109.51 | 109.51 | 0.00 | 217.35 | 1115.00 |
| BF 10 | 1000 | 8.36 | 119.59 | 119.59 | 0.00 | 217.35 | 611.00 |

Table 15: stiffness and coordinate of braced frames

| Center of Mass and Center of Rigidity | | | | |
|---|---|---|---|---|
| **Story** | **Center of Mass X (in)** | **Center of Mass Y (in)** | **Center of Rigidity X (in)** | **Center of Rigidity Y (in)** |
| 2 | 1071.106 | 2304.073 | 1088.461 | 2257.057 |
| 3 | 1097.344 | 2300.094 | 1070.254 | 2268.762 |
| 4 | 1090.824 | 2290.011 | 1068.785 | 2262.289 |
| Roof | 1055.822 | 2341.231 | 1056.866 | 2233.891 |

Table 16: center of mass and rigidity of proposed design

## Horizontal and Vertical Irregularity

The proposed design was checked for both horizontal and vertical irregularities. Torsional irregularity was checked for story three and four for the Y direction seismic loading using the displacement from ETABS outputs. It was found that $\delta max/\delta average$ was less than 1.2 and concluded that torsional irregularity did not exists. Moreover, the amplification of accidental torsional moment did not apply to the Piez' Hall addition because the building location was in seismic design category "B" as defined in Section 12.8.4.3 of ASCE7-10. Because of this reason, the reentrant corner irregularity in story four did not apply as well. By inspection, horizontal irregularity type 3 and 4 did not exist too since the floor slab did not contain any large openings nor there were any offset braced frames.

However, it was obvious that horizontal irregularity type 5 exist in either direction as described in table 12.3-1 of ASCE7-10. Thus, the building must comply with Section 12.7.3 and 16.2.2 of the code. Since both of these sections stated that a 3-D model of the building was required to determine member forces and structure displacements, the ETABS model met this requirement and horizontal irregularity type 5 was addressed.

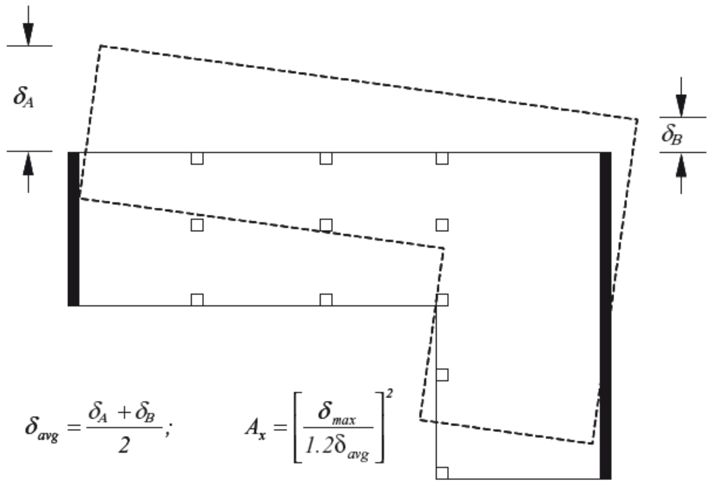



**FIGURE 12.8-1 Torsional Amplification Factor, $A_x$**

FIGURE 32: TORSIONAL AMPLIFICATION FACTOR FROM ASCE 7-10

**Table 12.3-1 Horizontal Structural Irregularities**

| Type | Description | Reference Section | Seismic Design Category Application |
|---|---|---|---|
| 1a. | **Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid. | 12.3.3.4<br>12.7.3<br>12.8.4.3<br>12.12.1<br>Table 12.6-1<br>Section 16.2.2 | D, E, and F<br>B, C, D, E, and F<br>C, D, E, and F<br>C, D, E, and F<br>D, E, and F<br>B, C, D, E, and F |
| 1b. | **Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid. | 12.3.3.1<br>12.3.3.4<br>12.7.3<br>12.8.4.3<br>12.12.1<br>Table 12.6-1<br>Section 16.2.2 | E and F<br>D<br>B, C, and D<br>C and D<br>C and D<br>D<br>B, C, and D |
| 2. | **Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction. | 12.3.3.4<br>Table 12.6-1 | D, E, and F<br>D, E, and F |
| 3. | **Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next. | 12.3.3.4<br>Table 12.6-1 | D, E, and F<br>D, E, and F |
| 4. | **Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements. | 12.3.3.3<br>12.3.3.4<br>12.7.3<br>Table 12.6-1<br>Section 16.2.2 | B, C, D, E, and F<br>D, E, and F<br>B, C, D, E, and F<br>D, E, and F<br>B, C, D, E, and F |
| 5. | **Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system. | 12.5.3<br>12.7.3<br>Table 12.6-1<br>Section 16.2.2 | C, D, E, and F<br>B, C, D, E, and F<br>D, E, and F<br>B, C, D, E, and F |

FIGURE 33: HORIZONTAL IRREGULARITY TABLE FROM ASCE 7-10

**Table 12.3-2 Vertical Structural Irregularities**

| Type | Description | Reference Section | Seismic Design Category Application |
|---|---|---|---|
| 1a. | **Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. | Table 12.6-1 | D, E, and F |
| 1b. | **Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above. | 12.3.3.1<br>Table 12.6-1 | E and F<br>D, E, and F |
| 2. | **Weight (Mass) Irregularity:** Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered. | Table 12.6-1 | D, E, and F |
| 3. | **Vertical Geometric Irregularity:** Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story. | Table 12.6-1 | D, E, and F |
| 4. | **In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:** In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab. | 12.3.3.3<br>12.3.3.4<br>Table 12.6-1 | B, C, D, E, and F<br>D, E, and F<br>D, E, and F |
| 5a. | **Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration. | 12.3.3.1<br>Table 12.6-1 | E and F<br>D, E, and F |
| 5b. | **Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:** Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration. | 12.3.3.1<br>12.3.3.2<br>Table 12.6-1 | D, E, and F<br>B and C<br>D, E, and F |

FIGURE 34: VERTICAL IRREGULARITY TABLE FROM ASCE 7-10

From table 12.3-2, only vertical irregularity type 4 and 5 needed to be checked for buildings in SDC "B". Since the braced frames were continuous for the full building height, both of these irregularities did not exist.

## Building Torsion

ETABS accounted for incidental torsion, but it was not accounted for the torsion caused by the difference in the center or rigidity and the center or mass. In the model, a 5% eccentricity was used to account for accidental torsion. In order to get the total torsion of the building, all three of these factors must be considered together.

In the tables below, torsional moment was obtained by multiplying the eccentricity by the story force. The accidental torsion was obtained by subtracting the torsion with zero assigned eccentricity from the torsion with an assigned 5% eccentricity found from the ETABS model. Then, the total torsion for each floor was found by adding the two moments together and the total torsion for the building was the sum of the total torsion for each floor. In the north-south direction, the building torsion was found larger than the torsion in east-west direction. Also notice that the first story was not accounted for in building torsion because it effectively acted as a ground floor and therefore would not have torsion effects on the building.

| Building Torsion, N-S Direction (Earthquake Controlling) | | | | | |
|---|---|---|---|---|---|
| **Story** | **Story Force (kip)** | **Eccentricity (ft)** | **Torsional Moment, Mt (kip-ft)** | **Accidental Torsion, Ma (kip-ft)** | **Total Torsion, Mt (kip-ft)** |
| 2 | 42.64 | 3.918 | 167.06 | 533 | 700.06 |
| 3 | 90.64 | 2.611 | 236.66 | 1133 | 1369.66 |
| 4 | 86.18 | 2.31 | 199.07 | 1077.25 | 1276.32 |
| Roof | 227.42 | 8.945 | 2034.27 | 2842.75 | 4877.02 |
| | | | | ∑= | **8223.06** |

TABLE 17: TOTAL BUILDING TORSION CAUSED BY STORY FORCES IN NORTH-SOUTH DIRECTION

| Building Torsion, E-W Direction (Earthquake Controlling) | | | | | |
|---|---|---|---|---|---|
| **Story** | **Story Force (kip)** | **Eccentricity (ft)** | **Torsional Moment, Mt (kip-ft)** | **Accidental Torsion, Ma (kip-ft)** | **Total Torsion, Mt (kip-ft)** |
| 2 | 42.64 | 1.446 | 61.66 | 533 | 594.66 |
| 3 | 90.64 | 2.2575 | 204.62 | 1133 | 1337.62 |
| 4 | 86.18 | 1.837 | 158.31 | 1077.25 | 1235.56 |
| Roof | 227.42 | .087 | 19.78 | 2842.75 | 2862.53 |
| | | | | ∑= | **6030.37** |

FIGURE 35: TOTAL BUILDING TORSION CAUSED BY STORY FORCES IN EAST-WEST DIRECTION

The existing design had an overall building torsion of 20074.64kip-ft in the N-S direction and 25079.88kip-ft in the E-W direction. In comparison, the proposed designed had reduced overall building torsion by 60% in the N-S direction and as much as 75% in the E-W direction.

## Lateral Load Distribution

### Direct Shear

The direct shear was calculated for each brace. The braces that were not parallel to either the X or Y axis were treated by separating its stiffness (k) value into X and Y components, and thus resisting lateral loads in both X and Y directions.

### Torsional Shear

Torsional shear was also included for the lateral analysis. The torsional shear resulting from a difference in the center of mass and the center of rigidity was calculated using ETABS output and Excel spreadsheet for level four of the building.

| Torsional Rigidity | | | | | | | |
|---|---|---|---|---|---|---|---|
| Label | Stiffness K, (kip/in) | Kx (kip/in) | Ky (kip/in) | Dix (in) | Diy (in) | $Ky*dix^2$ | $Kx*diy^2$ |
| BF 1 | 111.37 | 43.43 | 102.46 | 1689.64 | -1763.10 | 292515918.88 | 135017337.95 |
| BF 2 | 115.42 | 106.19 | 45.01 | 585.69 | -995.12 | 15441146.98 | 105152244.60 |
| BF 3 | 130.75 | 50.99 | 120.29 | -321.20 | 1020.80 | 12410665.71 | 53137332.90 |
| BF 4 | 157.23 | 144.65 | 61.32 | 150.99 | -617.12 | 1397961.57 | 55089285.51 |
| BF 5 | 142.21 | 55.46 | 130.83 | -187.09 | -839.50 | 4579342.76 | 39086346.40 |
| BF 6 | 481.23 | 442.73 | 187.68 | -377.09 | -524.61 | 26687786.22 | 121846231.65 |
| BF 7 | 379.94 | 0.00 | 379.94 | -343.45 | 1531.76 | 44817876.18 | 0.00 |
| BF 8 | 142.25 | 0.00 | 142.25 | -502.44 | 1283.76 | 35910022.89 | 0.00 |
| BF 9 | 109.51 | 109.51 | 0.00 | -852.90 | 1153.76 | 0.00 | 145769464.81 |
| BF 10 | 119.59 | 119.59 | 0.00 | -852.90 | 1657.76 | 0.00 | 328650424.38 |
| | | | | | $\sum(k*di^2)=$ | **1417509389.38** | |

Table 18: torsional rigidity required to obtain shear forces in the braced frames

| Total Shear in Lateral Resisting Elements (North-South Direction ,Earthquake Controlling) | | | | |
|---|---|---|---|---|
| Label | Lateral Force (kip) | Direct Shear (kip) | Torsional Shear (kip) | Total Shear (kip) |
| BF 1 | 86.18 ↑ | -7.55 | 0.29 | -7.39 ↓ |
| BF 2 | 86.18 ↑ | -3.32 | 0.04 | -3.45 ↓ |
| BF 3 | 86.18 ↑ | -8.86 | -0.06 | -8.84↓ |
| BF 4 | 86.18 ↑ | -4.52 | 0.02 | -4.65 ↓ |
| BF 5 | 86.18 ↑ | -9.64 | -0.04 | -9.76 ↓ |
| BF 6 | 86.18 ↑ | -13.83 | -0.12 | -14.33↓ |
| BF 7 | 86.18 ↑ | -27.99 | -0.21 | -28.2 ↓ |
| BF 8 | 86.18 ↑ | -10.48 | -0.12 | -10.60↓ |
| BF 9 | 86.18 ↑ | 0.00 | 0.00 | -0.21 ↓ |
| BF 10 | 86.18 ↑ | 0.00 | 0.00 | -0.33 ↓ |

Table 19: shear forces for each lateral resisting elements in story four

| Total Shear in Lateral Resisting Elements (East-West Direction, Earthquake Controlling) | | | | |
|---|---|---|---|---|
| **Label** | **Lateral Force (kip)** | **Direct Shear (kip)** | **Torsional Shear** | **Total Shear (kip)** |
| BF 1 | 86.18 → | -3.49 | -0.33 | -3.67 ← |
| BF 2 | 86.18 → | -8.53 | -0.05 | -8.38 ← |
| BF 3 | 86.18 → | -4.10 | 0.07 | -4.12 ← |
| BF 4 | 86.18 → | -11.62 | -0.02 | -11.47 ← |
| BF 5 | 86.18 → | -4.46 | 0.05 | -4.32 ← |
| BF 6 | 86.18 → | -35.57 | 0.13 | -35.00 ← |
| BF 7 | 86.18 → | 0.00 | 0.25 | 0.25 → |
| BF 8 | 86.18 → | 0.00 | 0.14 | 0.14 → |
| BF 9 | 86.18 → | -8.80 | 0.00 | -9.04 ← |
| BF 10 | 86.18 → | -9.61 | 0.00 | -9.99 ← |

TABLE 20: SHEAR FOR EACH LATERAL RESISTING ELEMENTS IN STORY FOUR

## Allowable Story Drift

Since braced frame 9 was the most flexible of all the lateral resisting elements, its lateral displacement might be a concern. Therefore, braced frame 9 was checked against the allowable story drift for both wind and seismic load cases. Lateral displacements and drift were obtained from ETABS. The total displacement at each floor was checked against the allowable displacement h/400. All story levels were found to meet serviceability requirements for wind. For seismic, the inter-story drift were found from ETABS and were compared to the allowable inter-story drift given in Table 12.12-1 of ASCE7-10. Since the proposed design was assigned as a category II building of all other structures, 0.020hx was used for the allowable story drift limit. It was determined that all floor levels met the serviceability requirements for seismic as well. The result was expected because the building had a fundamental period of 0.7 seconds, which it is not too flexible. In another words, serviceability requirements such as horizontal drift was met.

**Table 12.12-1 Allowable Story Drift, $\Delta_a{}^{a,b}$**

| Structure | Risk Category I or II | Risk Category III | Risk Category IV |
|---|---|---|---|
| Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, 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 $x$.
[b] For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

FIGURE 36: ALLOWABLE STORY DRIFT TABLE FROM ASCE 7-10

**Story Drift, E-W Direction (Seismic)**

| | Story | Displacement (in) | Story Drift (in) | Allowable Story Drift (in) | Adequacy |
|---|---|---|---|---|---|
| Braced Frame 9 | 2 | 0.62 | 0.034 | 0.36 | ok |
| | 3 | 1.04 | 0.023 | 0.36 | ok |
| | 4 | 1.47 | 0.024 | 0.36 | ok |
| | Roof | 1.82 | 0.019 | 0.36 | ok |

TABLE 21: STORY DRIFT CHECK FOR SEISMIC LOAD

**Story Drift, E-W Direction (Wind)**

| | Story | Displacement (in) | Story Drift (in) | Allowable Story Drift (in) | Adequacy |
|---|---|---|---|---|---|
| Braced Frame 9 | 2 | 0.336 | 0.0185 | 0.546 | ok |
| | 3 | 0.523 | 0.0103 | 0.546 | ok |
| | 4 | 0.682 | 0.00874 | 0.546 | ok |
| | Roof | 0.787 | 0.00583 | 0.546 | ok |

TABLE 22: STORY DRIFT CHECK FOR WIND LOAD

## Overturning Moment

It was found that the seismic overturning moment controlled with a value of 25348.93 kips-ft. To determine the resisting moment, the weight of the structure was multiplied by half of the least dimension of the building (moment arm). Then, a factor of safety was applied to assure that 2/3 Mr > Mo. Even with the additional factor of safety, the resisting moment capacity still exceeded the overturning moment by a large portion. However, a further investigation of the foundation will have to be performed in order to determine any area of concern. As of now, the foundation appears to be adequate for the overturning moment.

| Overturning and Resisting Moments | | |
|---|---|---|
| **Story** | **Height (ft)** | **Moments (k-ft)** |
| 2 | 16 | 776.46 |
| 3 | 32 | 3301.19 |
| 4 | 48 | 4708.13 |
| Roof | 64 | 16563.15 |
| **Overturning Moment** | $\sum=$ | **25348.93** |
| | **Resisting Moment =** | **767844.67** |

TABLE 23: OVERTURNING AND RESISTING MOMENT FOR PROPOSED DESIGN

## Spot Checks

Spot checks were performed on column C-2 for both axial load and bending capacity. The column was a W14x132. To analyze column C-2, table 3-10 of AISC 14th edition was used to find the moment capacity for an un-braced length of 18.2'. Table 4-1 was used to find the axial load capacity. Both of the internal axial load and moment was found in ETABS output by setting the controlling seismic load cases in the N-S direction. After knowing $P_u/\phi P_c < 0.2$, the interaction equation of $0.5P_u/P_c + M_u/M_c$ was used to determine the adequacy of the column. The B1 factor was accounted for and determined to equal to 1. After solving the interaction equation, the answer was 0.85 which was less than 1. It was determined through these analyses that the members were adequate.

# Construction Breadth

The construction breadth was done to determine the impact of the proposed structural system would have on Piez Hall addition in terms of cost, time and site logistics. The current concrete construction cost was compared to the cost estimate of the redesigned system. The construction schedule to build the current concrete system was compared to the new steel system. Finally, site logistics for the proposed design was developed. The new system also featured less interior columns and wider floor spaces in the cost of additional building height.

## Cost Estimation

With the change from concrete to steel, a cost analysis was completed to compare the redesigned system cost with the existing concrete system. RS Means 2012 was used to determine the cost of the new system. Table 24 and 25 showed a summary of each system's cost. The analysis showed that the new structural system would cost 602,919 USD more than the existing superstructure. A detailed superstructure cost calculations can be found in Appendix E.

| Cost Estimation for Proposed Design | | |
|---|---|---|
| MF-2004 | **Description** | **Cost** |
| 01-00-00 | General Requirements | 715500 |
| 03-00-00 | Concrete | 2237584 |
| 04-00-00 | Masonry | 1760130 |
| 05-00-00 | Metals | 7042970 |
| 06-00-00 | Wood, Plastics & Composites | 863072 |
| 07-00-00 | Thermal & Moisture Protection | 1064621 |
| 08-00-00 | Openings | 2787579 |
| 09-00-00 | Finishes | 4002883 |
| 10-00-00 | Specialties | 277095 |
| 11-00-00 | Equipment | 3281909 |
| 12-00-00 | Furnishings | 264626 |
| 13-00-00 | Special Construction | 136842 |
| 14-00-00 | Conveying Equipment | 516183 |
| 21-00-00 | Fire Suppression | 1188286 |
| 22-00-00 | Plumbing | 1724745 |
| 23-00-00 | HVAC | 8209311 |
| 26-00-00 | Electrical | 3968507 |
| 27-00-00 | Communications | 117659 |
| 28-00-00 | Electrician Safety & Security | 420906 |
| 31-00-00 | Earthwork | 591027 |
| | **Total** | **41171435** |

TABLE 24: COST ESTIMATION FOR PROPOSED DESIGN

| Cost Estimation for Existing Design | | |
|---|---|---|
| MF-2004 | **Description** | **Cost** |
| 01-00-00 | General Requirements | 715500 |
| 03-00-00 | Concrete | 7772477 |
| 04-00-00 | Masonry | 1760130 |
| 05-00-00 | Metals | 1406278 |
| 06-00-00 | Wood, Plastics & Composites | 863072 |
| 07-00-00 | Thermal & Moisture Protection | 1064621 |
| 08-00-00 | Openings | 2787579 |
| 09-00-00 | Finishes | 4002883 |
| 10-00-00 | Specialties | 277095 |
| 11-00-00 | Equipment | 3281909 |
| 12-00-00 | Furnishings | 264626 |
| 13-00-00 | Special Construction | 136842 |
| 14-00-00 | Conveying Equipment | 516183 |
| 21-00-00 | Fire Suppression | 704926 |
| 22-00-00 | Plumbing | 1724745 |
| 23-00-00 | HVAC | 8209311 |
| 26-00-00 | Electrical | 3968507 |
| 27-00-00 | Communications | 117659 |
| 28-00-00 | Electrician Safety & Security | 420906 |
| 31-00-00 | Earthwork | 591027 |
| | **Total** | **40585679** |

TABLE 25: COST ESTIMATION FOR PROPOSED DESIGN

In table 26, the total amount of beams, columns, braces are counted from the ETABS model. The total weight of steel required was obtained by the sum of the member's length multiply by their designated weight per linearly feet. Then, the cost for steel was found by multiplying the total weight of steel in tons by the cost per ton (2794.53 USD/ton). The cost for connections was also accounted for in the cost estimation. The amount of steel connection required was assumed to be 15% of the total steel weight and the cost of connections was 3099 USD/ton. In addition, the cost of sprayed fireproofing was obtained from RS means and included in the cost estimation. The original concrete system cost estimation was obtained from Cannon Design.

## Construction Schedule

The construction schedule of the proposed design was compared to the original schedule. The study was conducted to determine whether or not construction time could be reduced. In the original schedule obtained from Cannon Design, the amount of days used to shore, cure, place formwork and rebar of the concrete system was calculated to be approximately 267 days. Table 26 showed 176 days were needed to erect and install the steel system based on RS means 2012. With everything else (e.g. interior finishes, masonry work, MEP system) being the same, the proposed design could potentially shorten the construction time of Piez Hall addition by 100 days which is about 3 months

| Schedule for Proposed Design | | | | | |
|---|---|---|---|---|---|
| **Member Size** | Number of Members | Lengths (ft) | Weight (lb) | Labor Hour/Crew(s)/L.F | Total Hours |
| 12x14 | 7 | 18.7 | 1832.6 | 0.064 | 8.3776 |
| 12x26 | 18 | 18.7 | 8751.6 | 0.064 | 21.5424 |
| 12x40 | 61 | 28 | 68320 | 0.069 | 117.852 |
| 12x96 | 21 | 16 | 32256 | 0.088 | 29.568 |
| 12x106 | 3 | 16 | 5088 | 0.088 | 4.224 |
| 14x68 | 210 | 16 | 228480 | 0.07 | 235.2 |
| 14x132 | 57 | 16 | 120384 | 0.078 | 71.136 |
| 14x159 | 5 | 16 | 12720 | 0.08 | 6.4 |
| 14x193 | 2 | 16 | 6176 | 0.083 | 2.656 |
| 14x283 | 1 | 16 | 4528 | 0.087 | 1.392 |
| 27x336 | 4 | 16 | 21504 | 0.075 | 4.8 |
| 33x130 | 25 | 11 | 35750 | 0.071 | 19.525 |
| 33x201 | 62 | 15.75 | 196276.5 | 0.073 | 71.2845 |
| 40x593 | 74 | 20 | 877640 | 0.09 | 133.2 |
| 48LH17 | 136 | 53 | 302736 | 0.036 | 259.488 |
| 24K5 | 684 | 22 | 331056 | 0.026 | 391.248 |
| | | | | **Total** | **1406 hours** |
| | | | | | **176 days** |

TABLE 26: TIME TABLE FOR PROPOSED DESIGN

## Construction Logistics

The author also considered the constructability of the new design. A proposed site logistics was developed to show the different construction phases.


FIGURE 37: SITE MAP FOR PIEZ HALL (PHOTO TAKEN FROM: BING.COM)

## Existing Conditions

Figure 36 showed a map of the site. The Piez Hall addition was found in Oswego, NY 13126. This building was less than two miles away from the Oswego Harbor Power Oil Plant. Directly to the south of the site was West Seneca St, route 104. These major roadways allow for several means of access to the site in a short distance. The site was surrounded by roadways. Directly to the south was Takamine street. Directly to the west was Centennial Dr. These two roads would be the major access to the site.

The site was part of a university campus. The buildings in this area have large base and short hieght, ranging from two to four stories. The Piez Hall addition was actually one of the tallest buildings at 73 feet. Majority of the spaces in the site was parking lot as showed in figure 37.

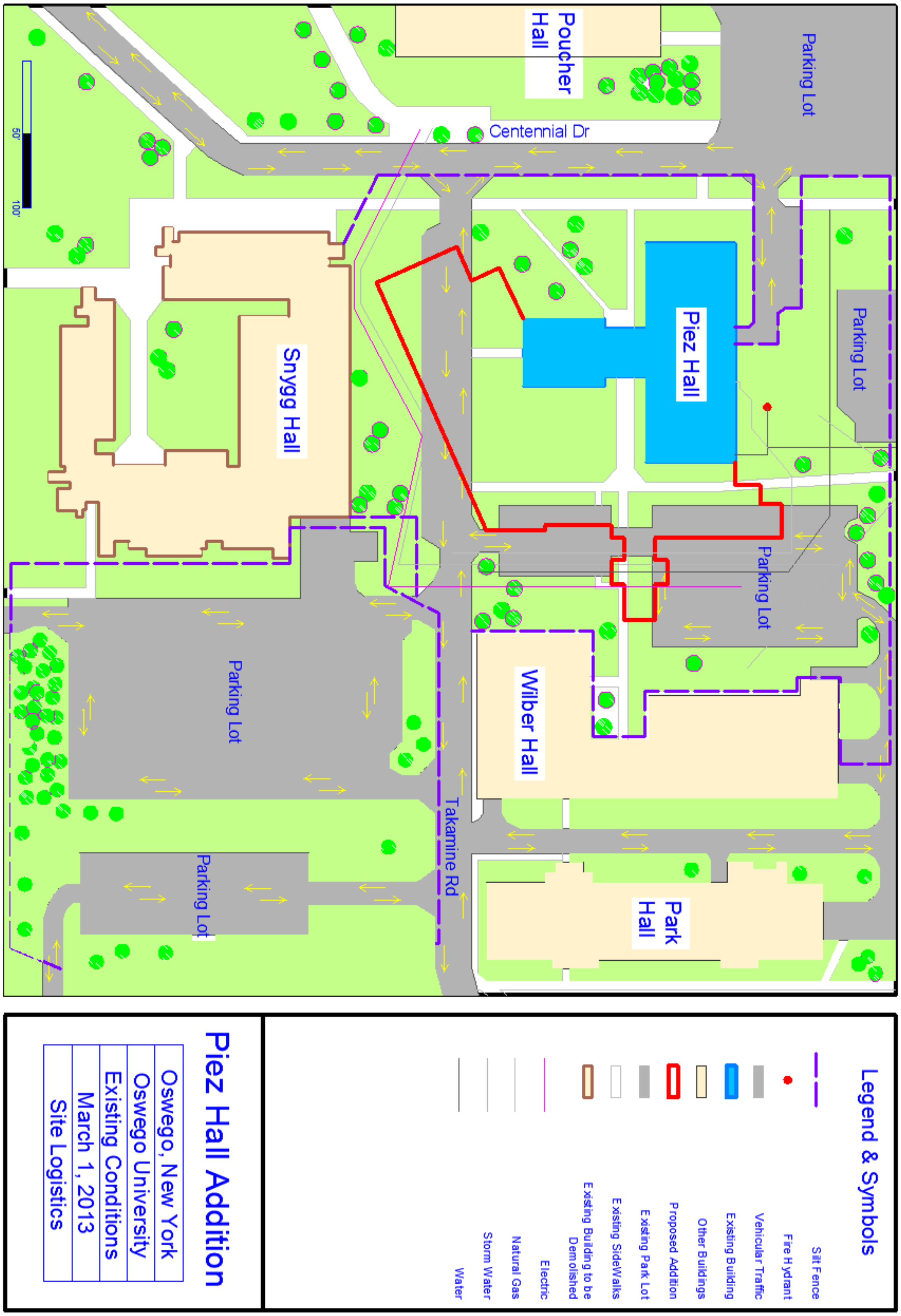



FIGURE 38: EXISTING CONDITION OF CONSTRUCTION SITE

## Excavation and Mobilization Phase

Snygg Hall had to be demolished for the new building footprint. Also, part of the Takamine street had to be modified as showed in figure 38. A temporary water runoff trap was located in the west of the new addition to catch excess muddy water since the site slopes down from east to west. Parking was available for the project team, roughly 60 spaces near the site trailers, throughout the construction period. Temporary power would enter the site from underground which connected to four power shed located in the north, south, east and west of the addition. This would remain in place for the duration of construction. Permanent power would be placed later during construction. Although underground utilities already had been placed, the connection would be established near the future. Other utilities can be seen around the building in their respective trench locations.

There are plenty of spaces in the site, the north parking lot were occupied to store construction and temporary equipment for the new addition. Excavation began near the north of the existing building and moved toward south of the site. Dumpsters were placed south of the site. During the construction, Takamine street would be barricaded since the street would be modified. However pedestrians could still walk around the fence while vehicles had to take either Rudolph road or George Washington Blvd. to travel from Sheldon Avenue to Centennial drive. A mock-up area would be designated in the north of the addition. Here, a mock-up of brick veneer including a small part of the curtain wall system will be built to demonstrate the correct way to joint and connect the two pieces.

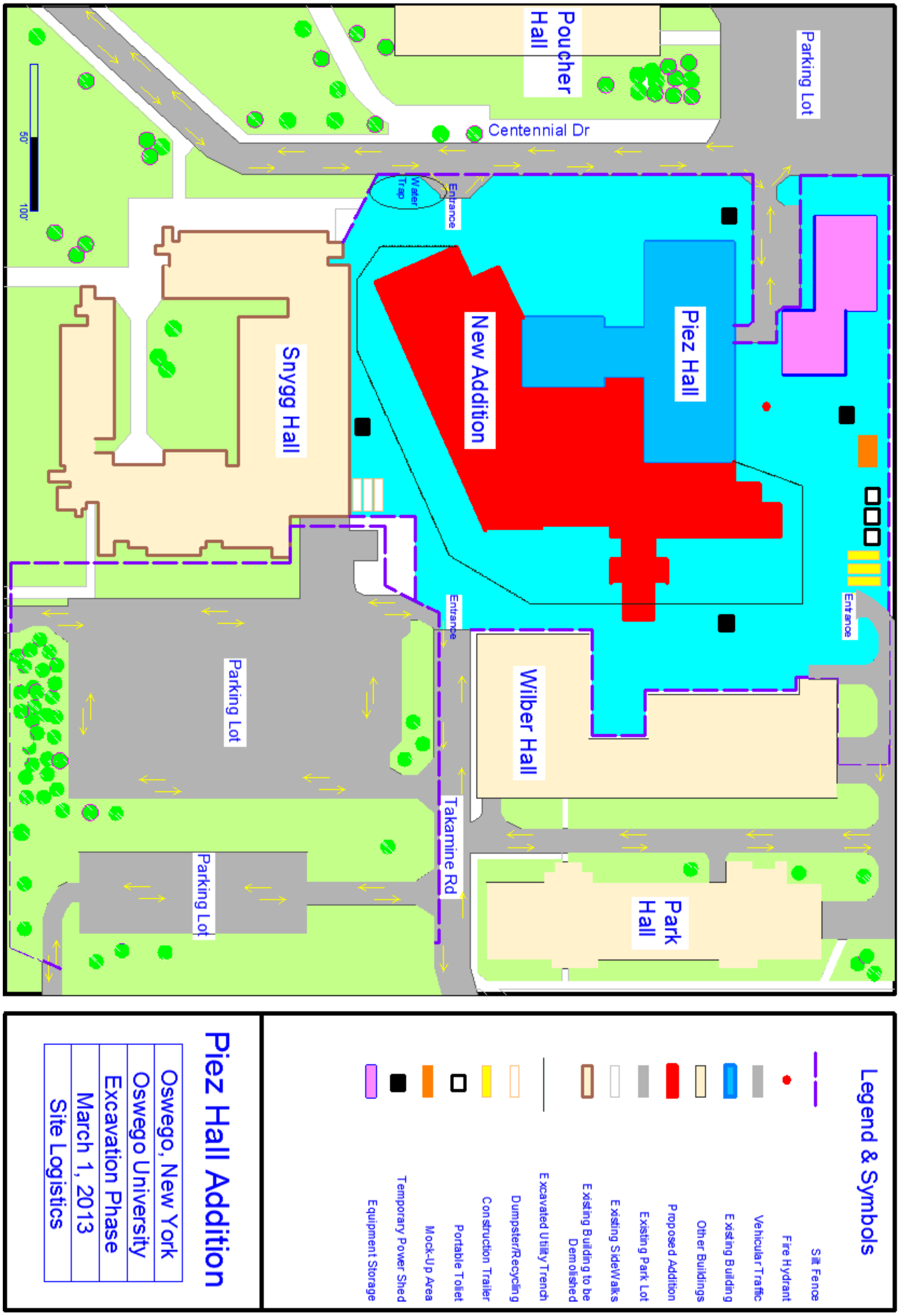



FIGURE 39: EXCAVATION AND MOBILIZATION PHASE OF PROPOSED DESIGN

### Structure Phase

Access to the construction site and the equipment storage parking lot were available from Takamine Street or the road north of Takamine. There was another entrance to the site from Centennial Drive, a temporary bypass route was established for this purpose. Additional parking was provided for construction workers in the neighboring parking lots. A 60-ton crawler crane was used for steel erection. The laydown area for steel was showed in figure 39. Steel will be taken from the truck and placed in the laydown area where materials could be easily picked up by the crane. Temporary bracing for trusses installation were provided to prevent the trusses from rotating. Overhead protection was required by OSHA standards, and it was located near every site entrance due to crane picks.

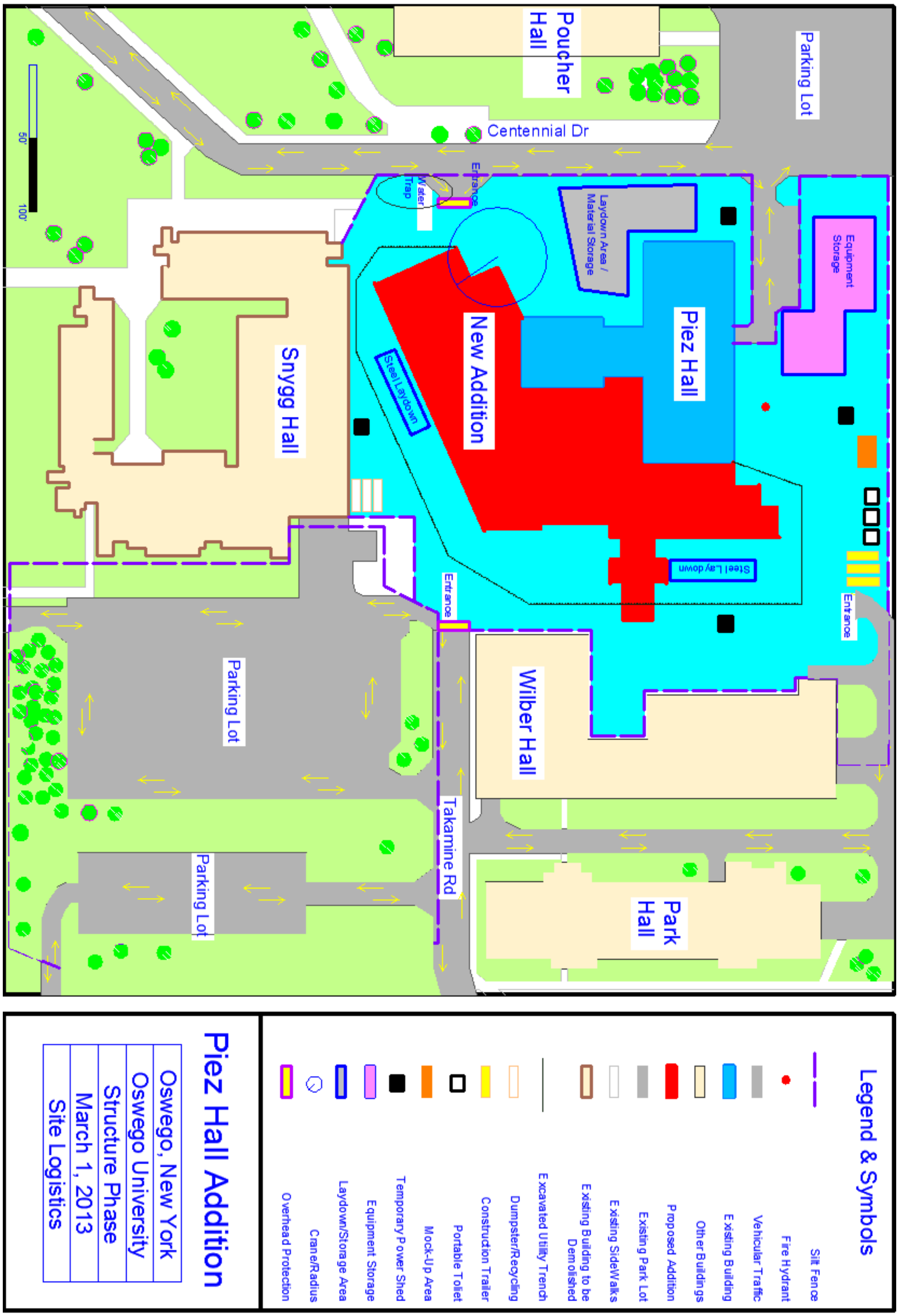



Figure 40: structure phase of proposed design

## Finishes Phase

By this phase, the trusses bracings had been removed since the building had already developed its full structural strength. Masonry had been included on the finishes plan due to the masonry façade work happening simultaneously with interior work. Fraco lifts were shown along the east and south facade of the addition to provide the most efficient brick placement possible. The interior installation would follow the same pattern as the foundations and structure, from the north near the existing building to the south. Two façade locations would be left open intentionally to allow materials to be lifted into and removed from the building. The openings at the south and east side serves as a pedestrian access to the first level and overhead protection were required at these locations. All-terrain forklifts would be used to lift masonry materials to scaffolding and deliver MEP materials to the upper floors. The finishes plan was shown in figure 40.

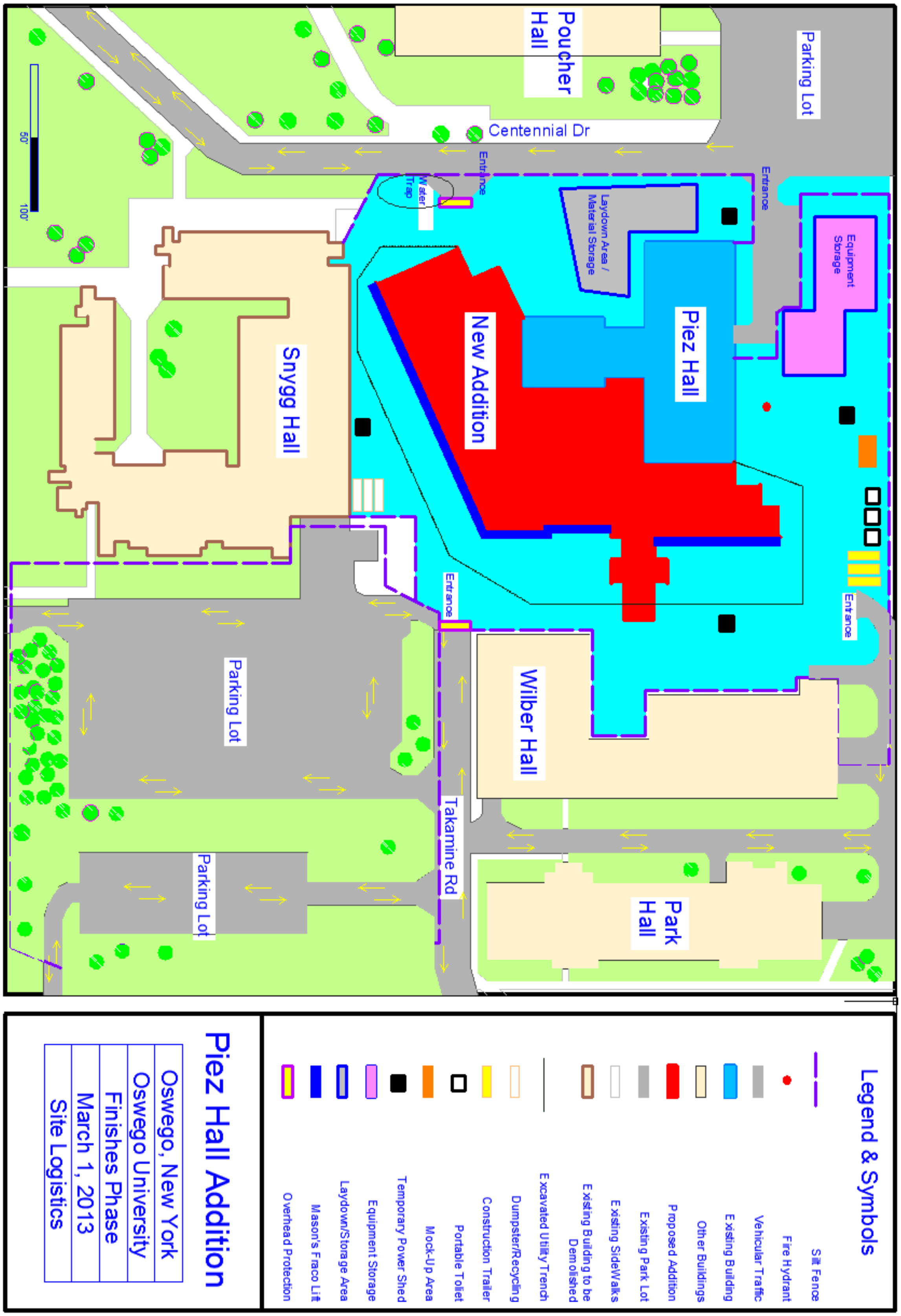



FIGURE 41: FINISHES PHASE OF PROPOSED DESIGN

# Sustainability Breadth

The Piez Hall Extension was selected to conduct the case study for the energy analysis. The location, approximated floor area, construction cost, and sustainability feature of the building is included. Laboratory 4011 on the fourth level was chosen to do the energy model analysis. To begin the analysis, the R-value of the existing roof and wall assembly were calculated in Microsoft Excel. The R-value of an extensive green roof system was found on the ASHRAE handbook. The assembly of the green roof, constructability, and drainage system was also explained and detailed. Next, two energy models were created using Trace 700; one for the existing roof system and one for the green roof assembly. The result of the analysis showed that the extensive green roof system will reduce the cooling load by 10% in summer and will decrease the heating load by 25% in winter.

## Existing Condition

- Laboratory 4011 was selected to be the subject of study in this project
- There are 4 workstations as shown in figure 5
- Since Piez Hall is certified LEED Gold, the existing R-value of its envelope system is high.


FIGURE 42ENLARGED VIEW OF LABORATORY 4011

## Existing Roof System

To begin the analysis, the R value of the exterior wall and roof assembly was calculated. According to the drawing set, the roof is R-1 assemblies which consist of white ballast in white adhesive over built-up asphalt roofing over cover board over average. R-24 polyisocyanurate insulation (tappered) over manufacture recommended vapor retarder, all fully adhered to concrete. The overall R value of the existing roof was calculated to be 24.

## Existing Wall System

The assembly of the existing wall was separated into top and bottom wall with an 80" tall and 1" thick low-E double pane glass window (R value of 0.33). The top wall consists of 1.8" terra cotta clay tile, 1.2" air space, 0.36 exterior gypsum, sprayed polyurethane foam insulation and air barrier, 0.625" sheathing, 0.625" glass fiber in cold framed metal frame. The bottom wall is made up of 6" precast concrete, sprayed polyurethane foam insulation, air barrier, 12" air space and 0.625" gypsum. The overall R value for the top wall is 20.5 while the bottom wall has an overall R value of 16.

## Proposed Roof System (Green Roof System)

In the study done by Henry Inc., an extensive green roof with 4" of growing media can reduce the storm water runoff by 95 percent and substantially reduce the peak flow rates.

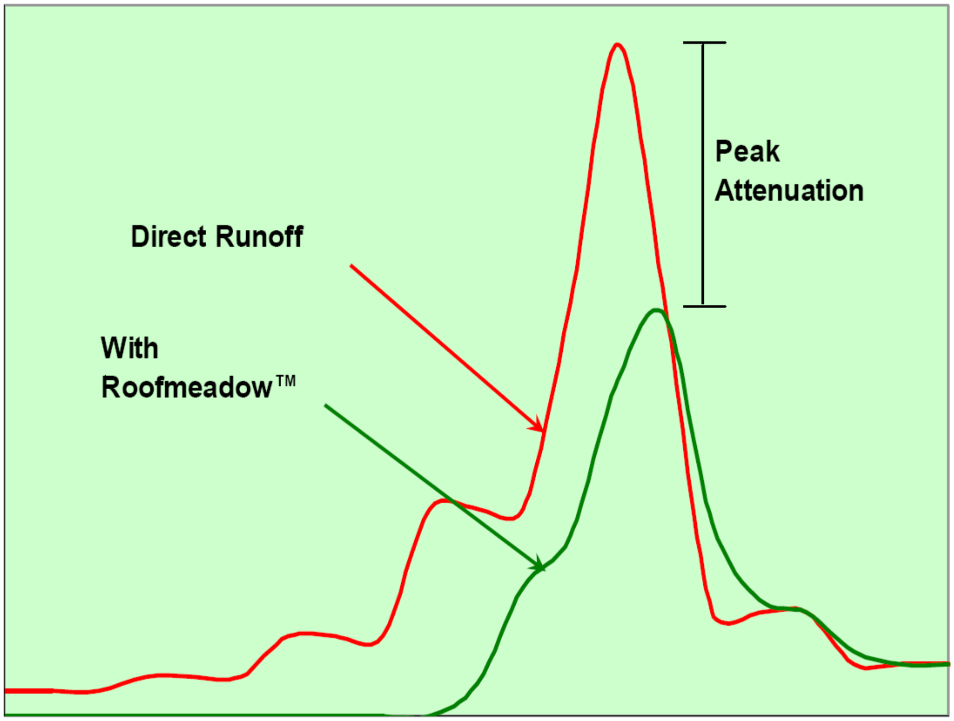



FIGURE 43: STORM WATER RUNOFF RETENTION (PASCHAL, 2006)

Moreover, a green roof serves as a shield to protect the roof assembly from the sun's harmful UV rays and reduces the expansion and contraction effect caused by daily variable temperature. In short, a vegetated roof typically has a service life span of 1.5 times greater than the life span of a regular roof.

A green roof also has the ability to reduce heat island effect [1] in cities. This is because the evaporation of water takes heat from the roof in order to escape into vapor. In turn, it cools the roof and saves 8000BTUs of energy for every gallon of water that is evaporated.

[1] Heat Island Effect: The increase of temperature in cities due to stored heat from masonry, concrete sidewalks, routs and pavement.

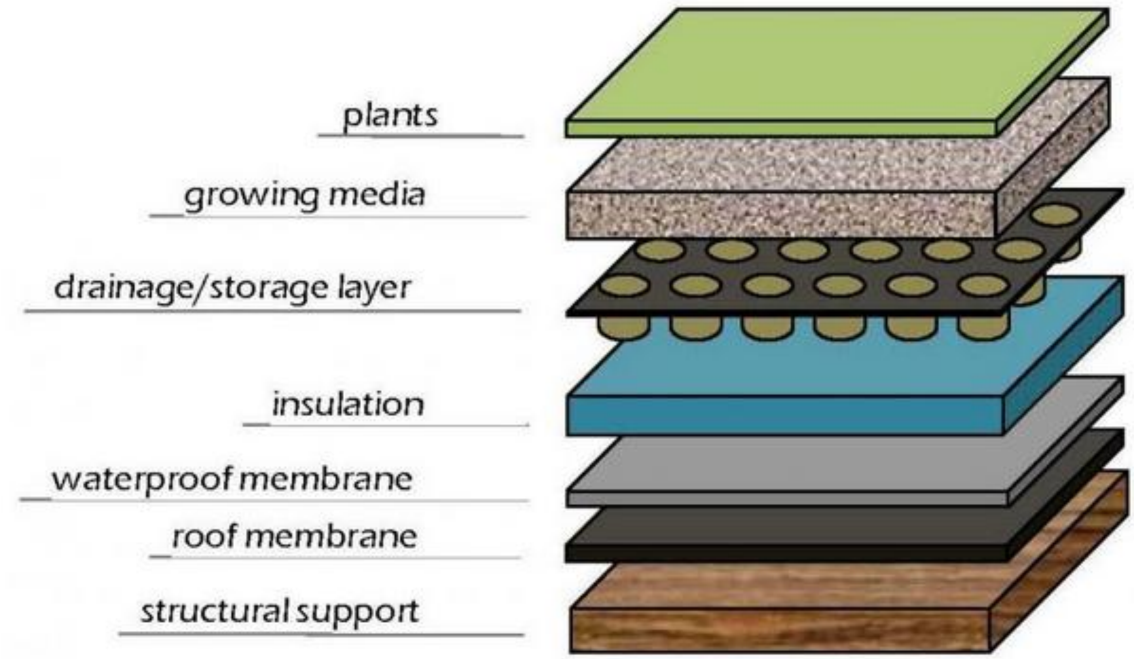



FIGURE 44: LAYERS OF GREEN ROOF ASSEMBLY (J-DRAIN, 2013)

A green roof is made up of 5 major components; roof structure, membrane/insulation, drainage layer & filter, plant media, and vegetation.

A typical green roof adds 25 to 100 psf more load than conventional flat roofs. This will require Piez Hall addition to have structural reinforcement. On top of the structural support, a layer of waterproofing membrane will be needed. Since leaks in green roof can be quite difficult to identify, the repair cost will be expensive. It is important for the membrane to resist building movement, ponded water, and root penetration as well as being non-biodegradable. A secondary membrane will be used to prevent root penetration if the primary waterproofing system is not sufficient. During construction, a tough protection mat might be used to protect the waterproofing or root barrier from mechanical damage. On top of all these insulations/membranes, a water-storing drainage layer will be needed. Stored water in green roof helps plants to survive through periods of dry weather. However, excessive water will kill the plants if the water is not drained properly from the storage layer.  In addition, a non-clogging separation fabric is important to keep the water storage and drainage layer freed from soil in order for them to function properly. The growing media consists of engineered soil, which is lightweight; has good water storage characteristics, low organic content and a good distribution of particle sizes. Finally, the appropriate plants must be identified. These plants must be able to survive under hot and dry climate.

## Potential LEED Points

By incorporating a green roof, 6 potential LEED points could be earned

- Reduce Heat Islands: 1 Point
- Reflectance Reduction: 1 Point
- Limit Use of Potable Water for Irrigation 1 Point
- Only Graywater for Irrigation/no irrigation System 1 Point
- Decrease Stormwater Runoff 1 Point
- Recycled Content 1 Point

According to the USGBC LEED 2009 for New Construction and Major Renovations

**LEED 2009 for New Construction and Major Renovations**

100 base points; 6 possible Innovation in Design and 4 Regional Priority points

| | |
|---|---|
| Certified | 40–49 points |
| Silver | 50–59 points |
| Gold | 60–79 points |
| Platinum | 80 points and above |

FIGURE 45: USGBC LEED 2009 LEED CHART

20 points is required to go from LEED gold to platinum. The potential 6 points from the green roof might be able to get Piez Hall addition to LEED platinum if the building is already at 74 points.

## Constructability

For the Piez Hall Extension, Optigreen's Economy Roof System is recommended. The extensive system features lightweight (approximately 30psf), fast installation, cost effective, and low maintenance. The general cost for the green roof is about 15 USD per sq. foot. The structural system of Piez Hall should be reinforced with either more rebar or thicker members to account for the additional 30 psf load on the roof. Due to the limited amount of time, the structural member was not resized in this report. However, it is still very important to note that the roof system requires additional structural reinforcement and to adhere to all codes.

**Green Roof "Economy Roof"**

| | | |
|---|---|---|
| Specifications | ⇢ Solution 1: 0-5° (Please select a variant)<br>⇢ Solution 2: 1-5° | |
| **Special features** | **⇢ Especially cost effective roof greening.**<br>**⇢ Low maintenance green roof design.**<br>**⇢ Relatively low biodiversity.** | |
| **Technical data** | Weight: | **Solution 1: 90-140 kg/m² or 0.9-1.4 KN/m² ***<br>**Solution 2: 100-140 kg/m² or 1.0-1.4 KN/m² *** |
| | Depth: | **80 mm** |
| | Roof pitch: | **0 - 5° (0-9%)** |
| | Vegetation form: | **Sedum/herbs/grasses** |
| | Water retention: | **50 - 60 %** |
| | Discharge coefficient: | **0.5 - 0.4** |
| | Water storage: | **ca. 25 l/m²** |
| | Ecological value: | ■■□□□ |
| | Maintenance and care: | ■□□□□ |
| | Cost factor: ** | ■□□□□ |

* Weight details relate to water-saturated conditions; weight when dry is equal to ca. 60-70 % of weight when wet.
** Price varies regionally.

FIGURE 46: EXTENSIVE GREEN ROOF DETAILS (PHOTO TAKEN FROM: OPTIGREEN.COM)

## Installation Process

- To begin the installation of the green roof system, place the roof waterproof membrane and insulation (polyisocyanurate in this case) on top of the concrete support. Make sure each sheet was overlapping adjacent sheet at least six inches. Cut openings for roof drain at the edges of the roof. (Drain box was a good choice to avoid clogging of drain and prevents freeze thaw)

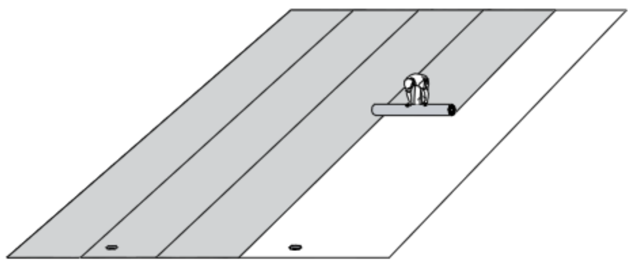


FIGURE 47: INSTALLATION OF WATERPROOF MEMBRANE (CONSERVATION TECHNOLOGY, INC, 2013)

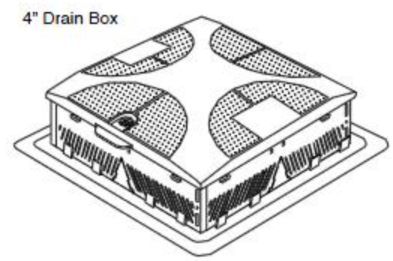



FIGURE 48: DRAIN BOX (CONSERVATION TECHNOLOGY, INC, 2013)

- Install drainage system by first placing the water drainage/storage layer on top of the insulation. Then, add drainage channel to direct water to the drain box. Each joining point should be two channel lengths.
- The drainage box should be connected to the existing drainage system and directed to the sewerage system.
- The piping system of the drainage can be made smaller due to water retention nature of the green roof. Hence, there will be potential savings for using smaller pipes.

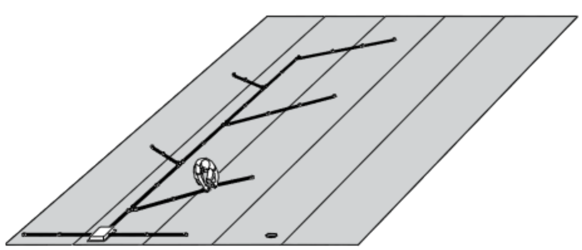


FIGURE 49: SAMPLE LAYOUT FOR CHANNEL PLACEMENT THAT CONNECTS THE DRAIN BOX (CONSERVATION TECHNOLOGY, INC, 2013)

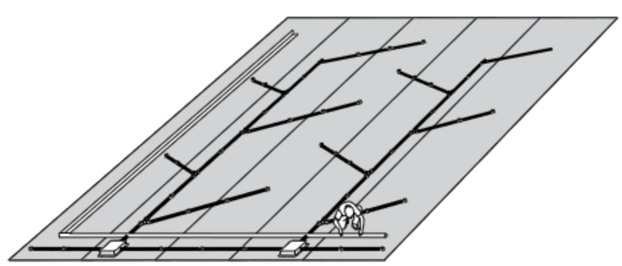


FIGURE 50: INSTALLATION OF RETAINING EDGE (CONSERVATION TECHNOLOGY, INC, 2013)

- Install retaining edge to separate the soil and the gravel perimeter.
- Lay the separation fabric and make sure every sheet overlapped the adjacent sheet at least six inches. Trim the fabric carefully at the edges

➢ Spread the gravel perimeter and the soil. Gravel particles should have a minimum of 3/8" diameter size. The gravels and soil should be dispensed from the super sack from the crane to minimize potential damage.

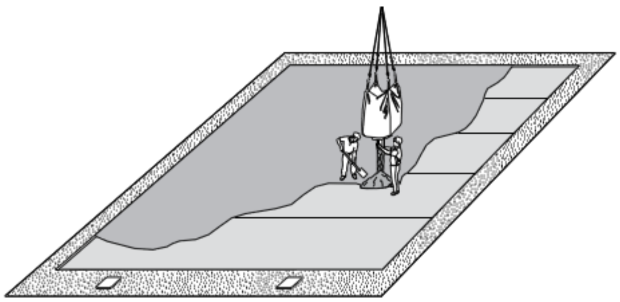


Figure 51: spread of gravel and soil (Conservation Technology, Inc, 2013)

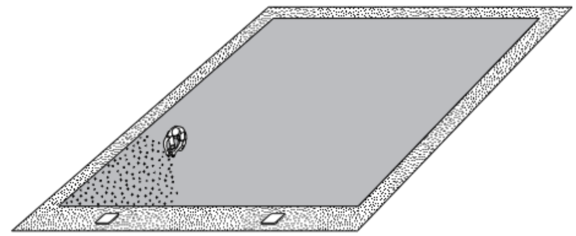


Figure 52: plant and complete the extensive green roof system (Conservation Technology, Inc, 2013)

➢ Plant herbs, shrubs, and grasses on the roof in a random pattern. Water them thoroughly after installation and during extended dry periods for the first two years. Release fertilizer twice yearly.

Figure 53: example of a completed extensive green roof system (paschal, 2006)

## Trace model

After obtaining an R value of 24 for the existing roof system and an R value of 60 for the green roof system, two trace models were developed. The outputs of these models were used to compare the thermal impact between the original roofing system and the green roof assembly. Several assumptions were made in order to create the energy model and are listed in the following:

- Location: LaGuardia, New York
- Room temperature: 75 °F and 50 % RH
- Air supply: 100% outdoor due to the room being a lab
- Mechanical system: geothermal system with back up boiler
- There are couple workstations in the laboratory, each workstation was assumed to have a capacity of 4 people

Results and comparison

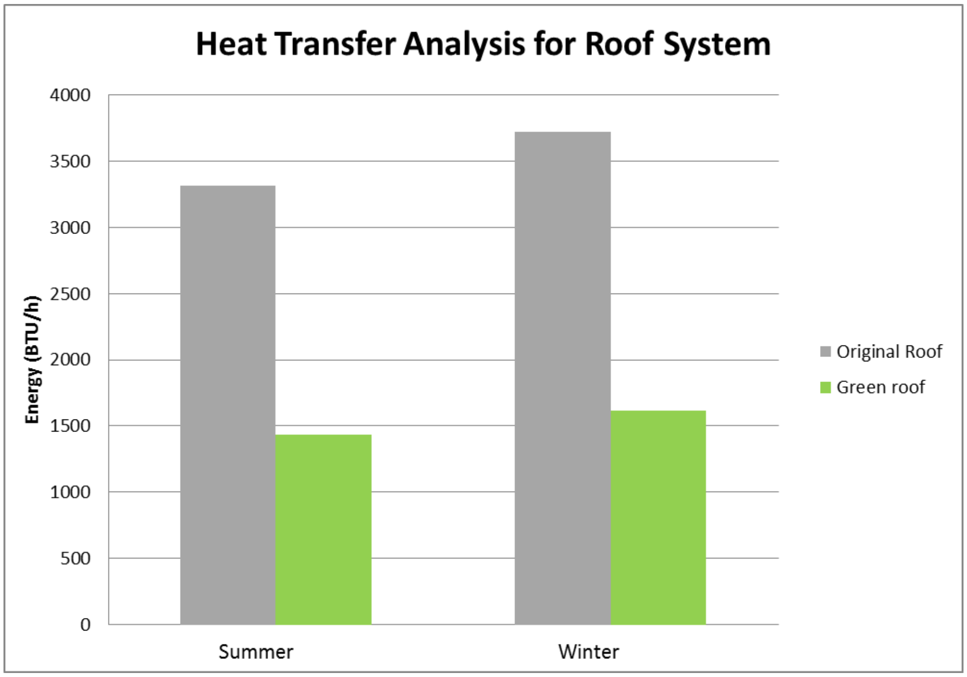



FIGURE 54 HEAT TRANSFER ANALYSIS FOR ROOF SYSTEM

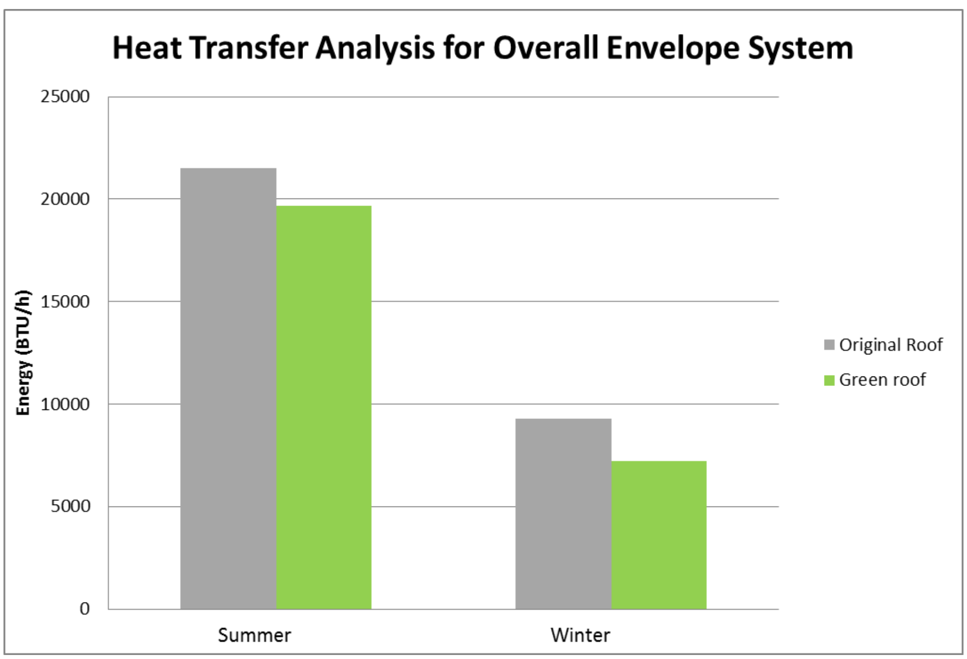



FIGURE 55 HEAT TRANSFER ANALYSIS FOR OVERALL ENVELOPE SYSTEM

In the charts above, the green roof will reduce the cooling load by about 10% in summer and will decrease the heating load by about 25% in winter.

Summary

| Thermal Resistance Comparison | |
|---|---|
| **R-value** | ➢ The existing wall system had an overall R-value of 16<br>➢ The existing roof system had an overall R-value of 24 |
| | ➢ The green roof system will have an overall R-value of 60 |
| **Thermal Transfer** | ➢ Reduce 10% cooling load in summer<br>➢ Decrease 25% heating load in winter |

TABLE 27 THERMAL RESISTANCE COMPARISON

| Green Roof System | | |
|---|---|---|
| **Main Components** | ➢ structural support<br>➢ roof membrane<br>➢ waterproof membrane<br>➢ insulation | ➢ drainage/storage layer<br>➢ growing media<br>➢ plants |
| **Drainage System** | ➢ Channels direct water to drain box. Drain box connects to existing drainage system and to the sewerage system.<br>➢ Potential savings on smaller pipes | |
| **Structural Information** | ➢ Additional 30 psf | |
| **Cost Information** | ➢ Additional 15 USD per sq. foot | |

TABLE 28 GENERAL INFORMATION OF GREEN ROOF SYSTEM

# Conclusion

The author decided to choose a composite system with long span trusses, K-series joists, and braced frames to achieve the goals. The reason to use a steel system was because seismic loads were the controlling forces in the existing design. A steel system would greatly reduce the weight of the building as well as the seismic loads. In the depth study, it was proved that the weight of the building was reduced by 3 times and the resulting seismic loads were reduced by 2 folds. However, it was found that seismic forces were still the dominant lateral loads compared to wind forces. This was probably due to the building having a large base and relatively short height. The foundation system was remained unchanged due to the time permitted. Therefore, a shallow spread footing foundation system was still in use for the proposed design.

A steel framing layout for the floors of Piez Hall extension was created. In this layout, the numbers of interior columns and shear walls were eliminated to achieve more open interior spaces. Ten braced frames were placed throughout the building to best resist lateral loads and to minimize building torsion. Most of the braced frames locations were either the same as the existing shear walls location or placed around elevator shafts and stairwell cores. The braces of the lateral elements were designed to account for opening along the building elevation.

ETABS was used to size the members for the proposed design. The model was then check for its accuracy by performing hand calculation for the center of rigidity and the center of mass. The member sized selected by ETABS were checked by hand calculations. Upon the completion ETABS model, the outputs were used to determine building torsion, lateral load distribution, allowable story drift, and overturning moments. These values were then compared to the values of the existing concrete design as determined in technical report 3. It was determined that building torsion in the new redesign was reduced by 60% in the north-south direction and 75% in the east-west direction. This makes sense because the controlling seismic loads were greatly reduced in for the new redesign.

Another reason to choose a steel system was because Piez Hall's location at New York. Skilled labors and contractors in steel construction would decrease the construction time. Furthermore, construction confusions would also be minimized since the numbers of sub-contractors in the project were reduced. To support these claims, a construction breadth study was conducted. It was determined that although the construction cost is increased by 600,000 USD, but the construction time is shortened by three months. Construction site logistics were also developed to map out the existing conditions, excavation/mobilization, structure, and finishes phases of the project.

An extensive green roof system from Optigreen Inc. was selected to incorporate into the proposed design in order to reduce annual energy load (10% in summer and 25% in winter), storm water run-off time, and to improve acoustic performances. The chosen green roof featured fast installation, cost effective, and low maintenance and weight only 30psf. LEED and installation process of the green were also discussed in the sustainability breadth.

With the proposed design, the following objectives were achieved

- Reduce construction schedule
- Reduce construction confusions
- Increase bay sizes to allow more open interior spaces
- Reduce building's overall weight and torsion
- Reduce seismic load
- Reduce annual energy cost

# Appendices