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

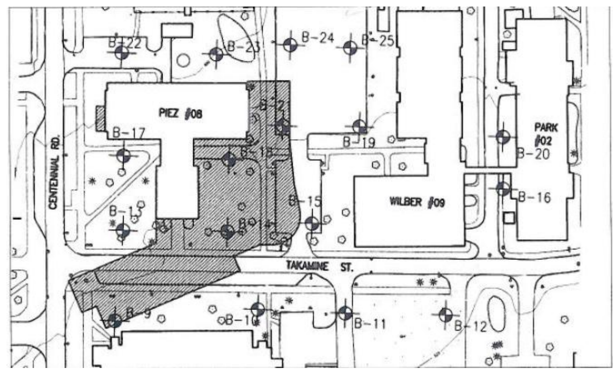


Figure 1: Site map showing existing Piez hall and the new extension (shaded area)

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.

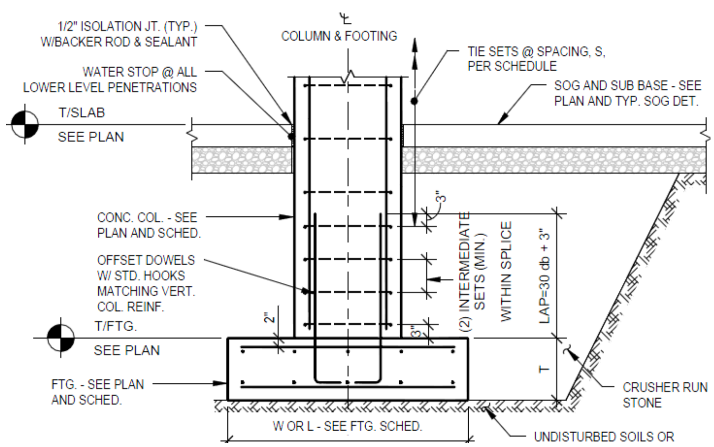


FIGURE 4: TYPICAL COLUMN FOOTING SHOWING REINFORCEMENT PLACEMENT

2. S = COLUMN TIE SPACING - SEE COLUMN SCHEDULE.
3. EVENLY SPACE FOOTING REINFORCEMENT. PROVIDE 3-INCHES CLEAR COVER UON.

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.

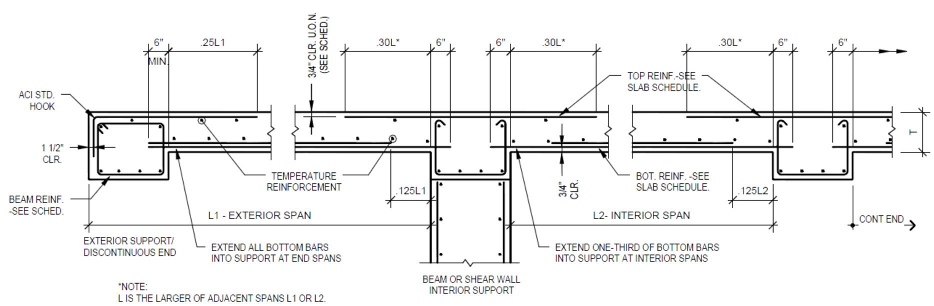


FIGURE 5: TYPICAL ONE WAY SLAB SHOWING REINFORCEMENT PLACEMENTS

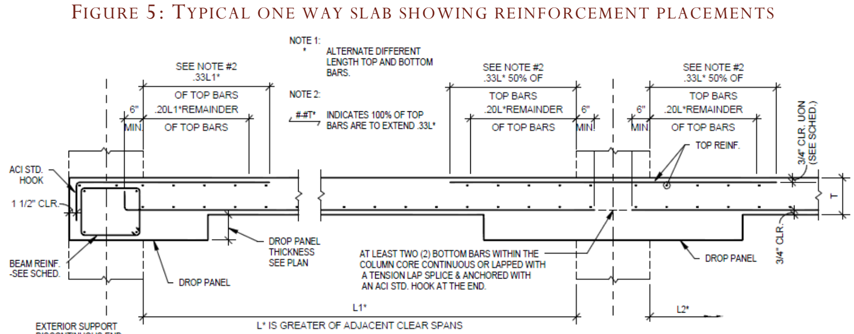


FIGURE 6: TYPICAL COLUMN STRIP DETAIL WITH DROP PANEL AND EDGE BEAM

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.

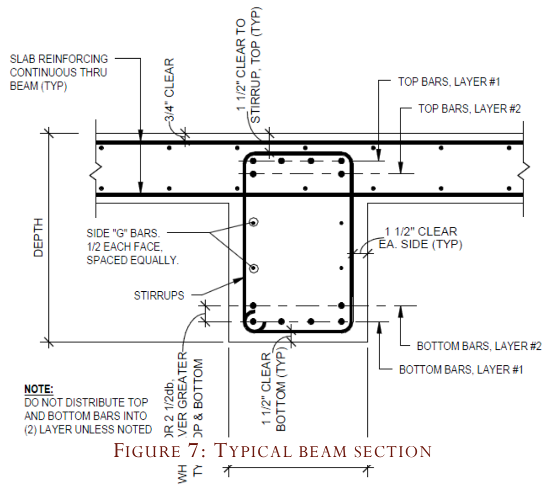


FIGURE 7: TYPICAL BEAM SECTION

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.

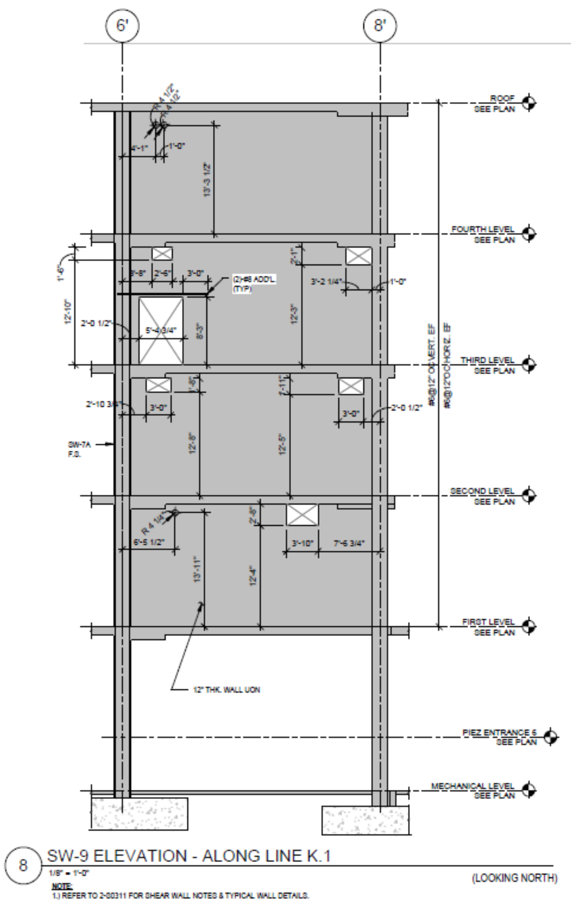


Figure 8: Typical concrete shear wall

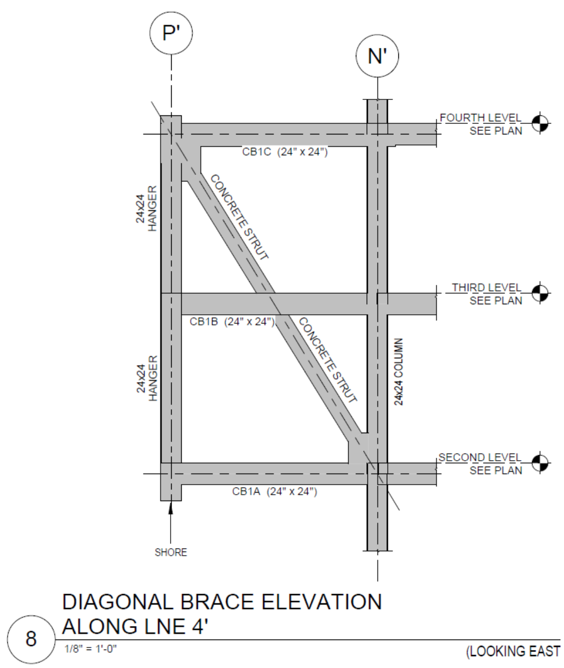


Figure 9: Typical concrete diagonal braces

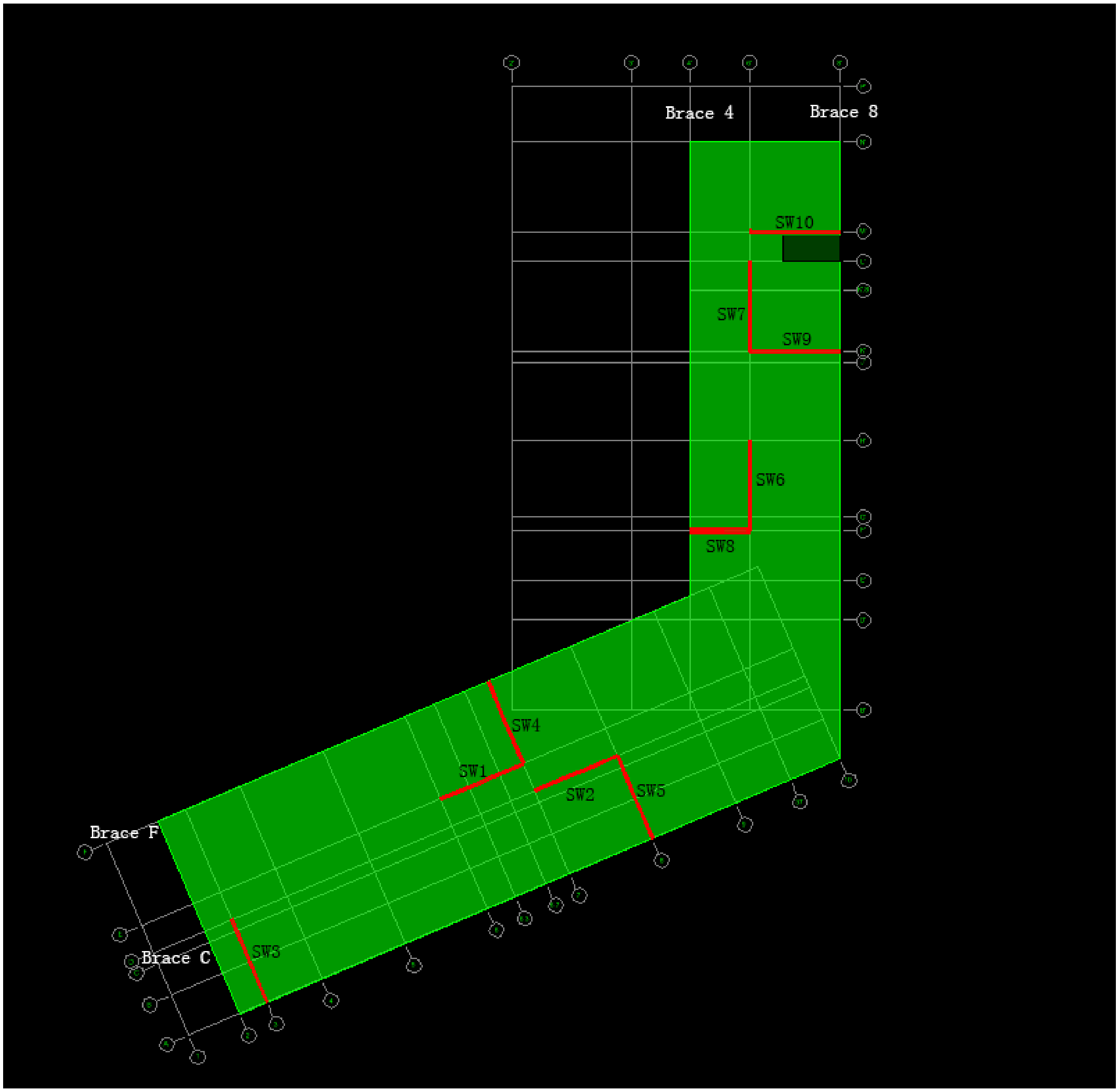


Figure 10: shear wall locations of a typical floor

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 14th 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

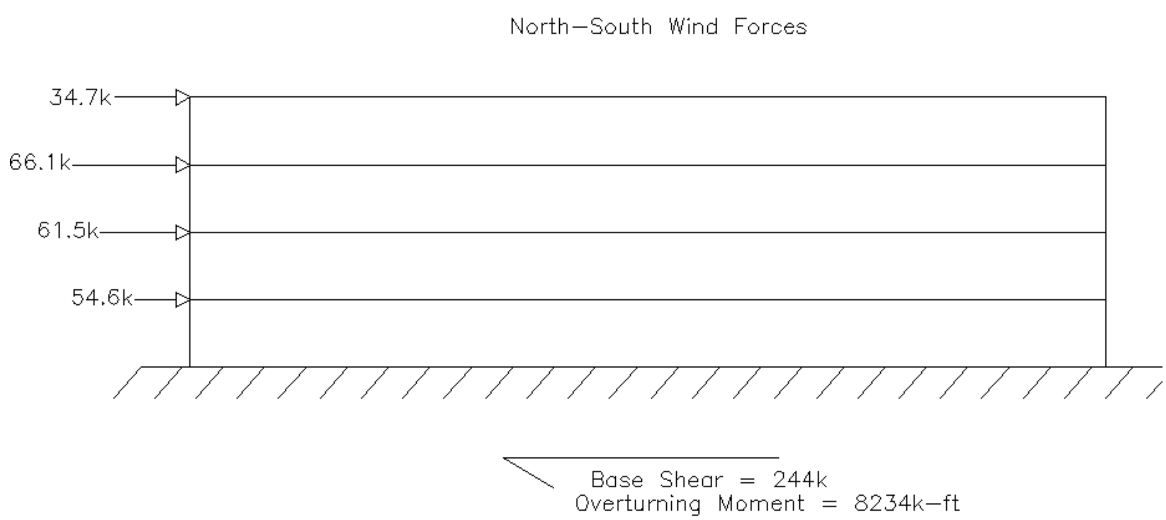


FIGURE 11: WIND FORCES DIAGRAM IN NORTH-SOUTH DIRECTION

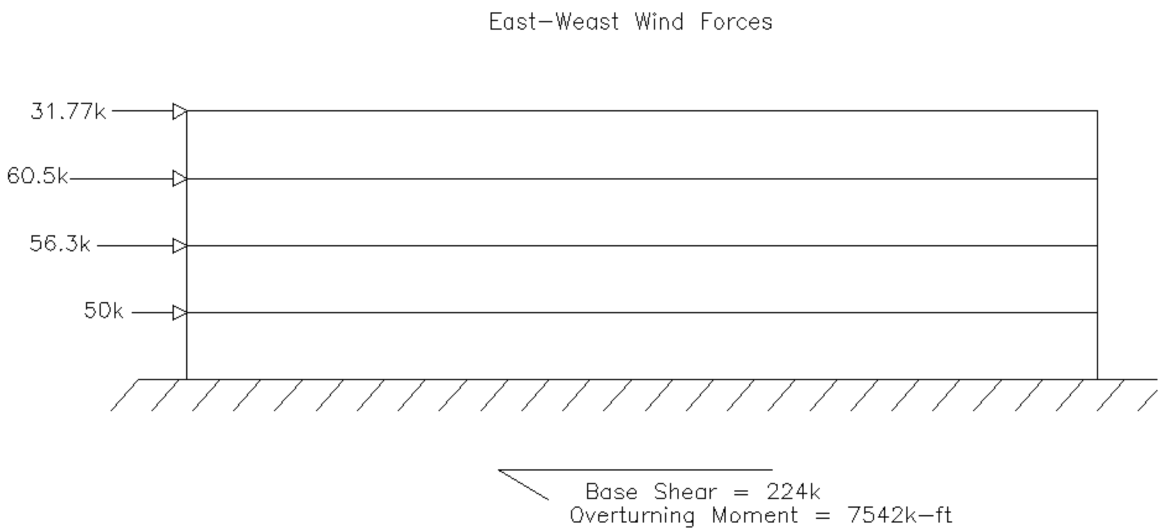


FIGURE 12: WIND FORCES DIAGRAM IN EAST-WEST DIRECTION

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

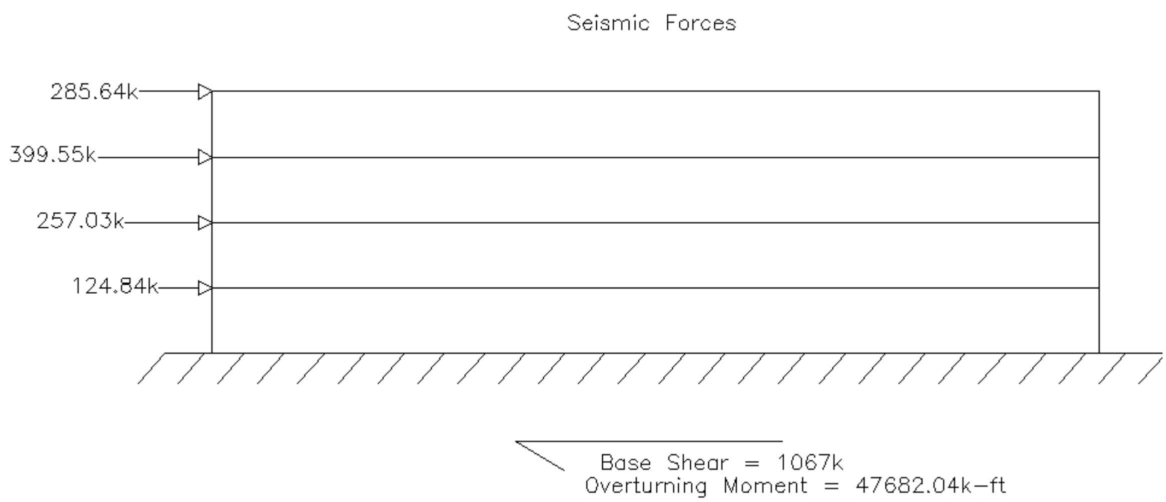


FIGURE 13: SEISMIC FORCES DIAGRAM IN EITHER DIRECTION

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**

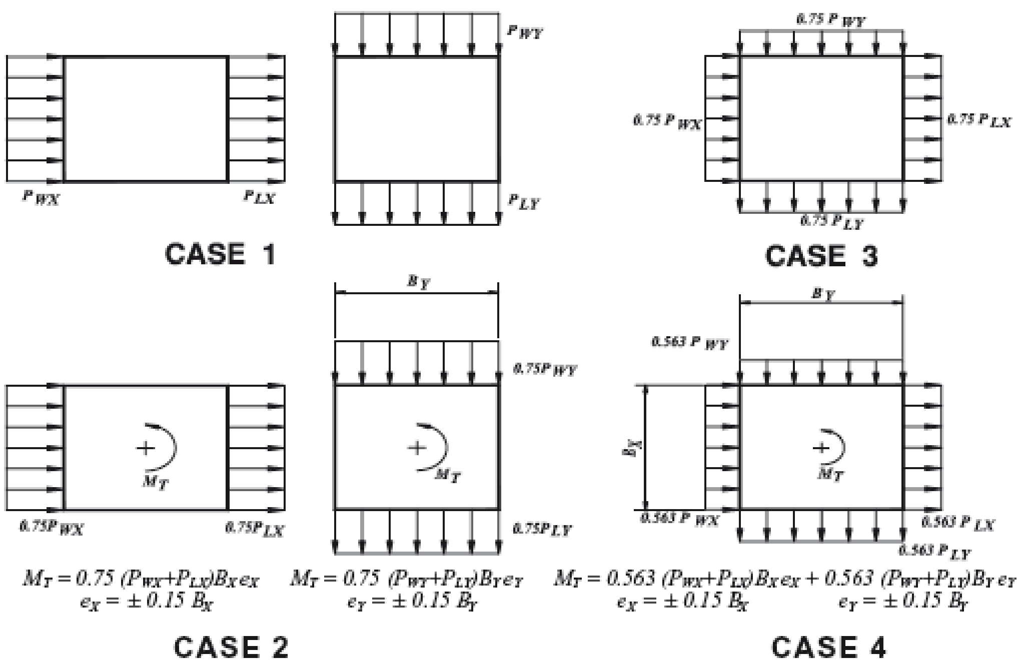
TABLE 12: NORTH SOUTH DIRECTION WIND FORCES FOR PROPOSED DESIGN

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 14th 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.

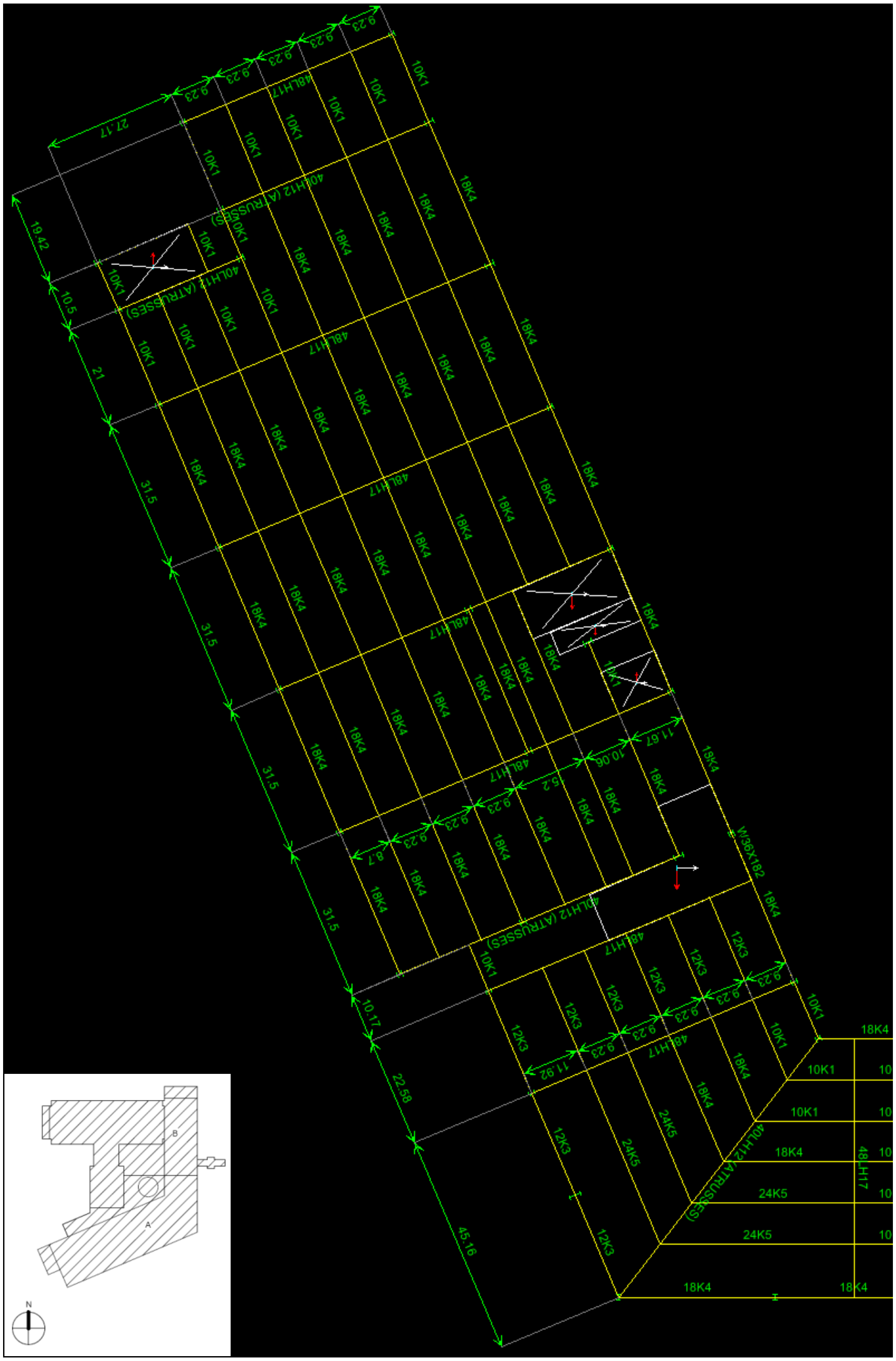


Figure 15: typical floor plan with member sizes

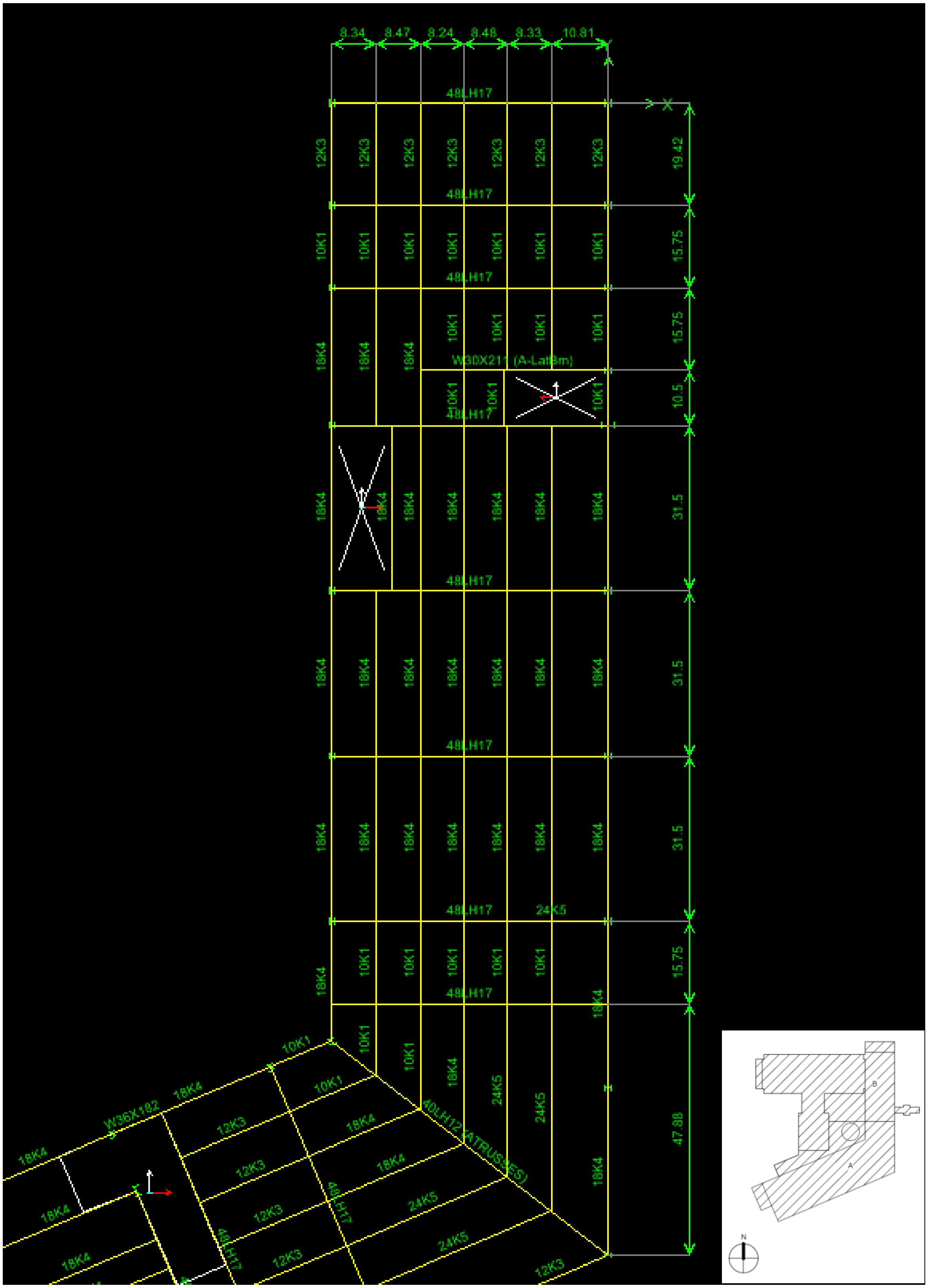


Figure 16: typical floor plan with member sizes

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.

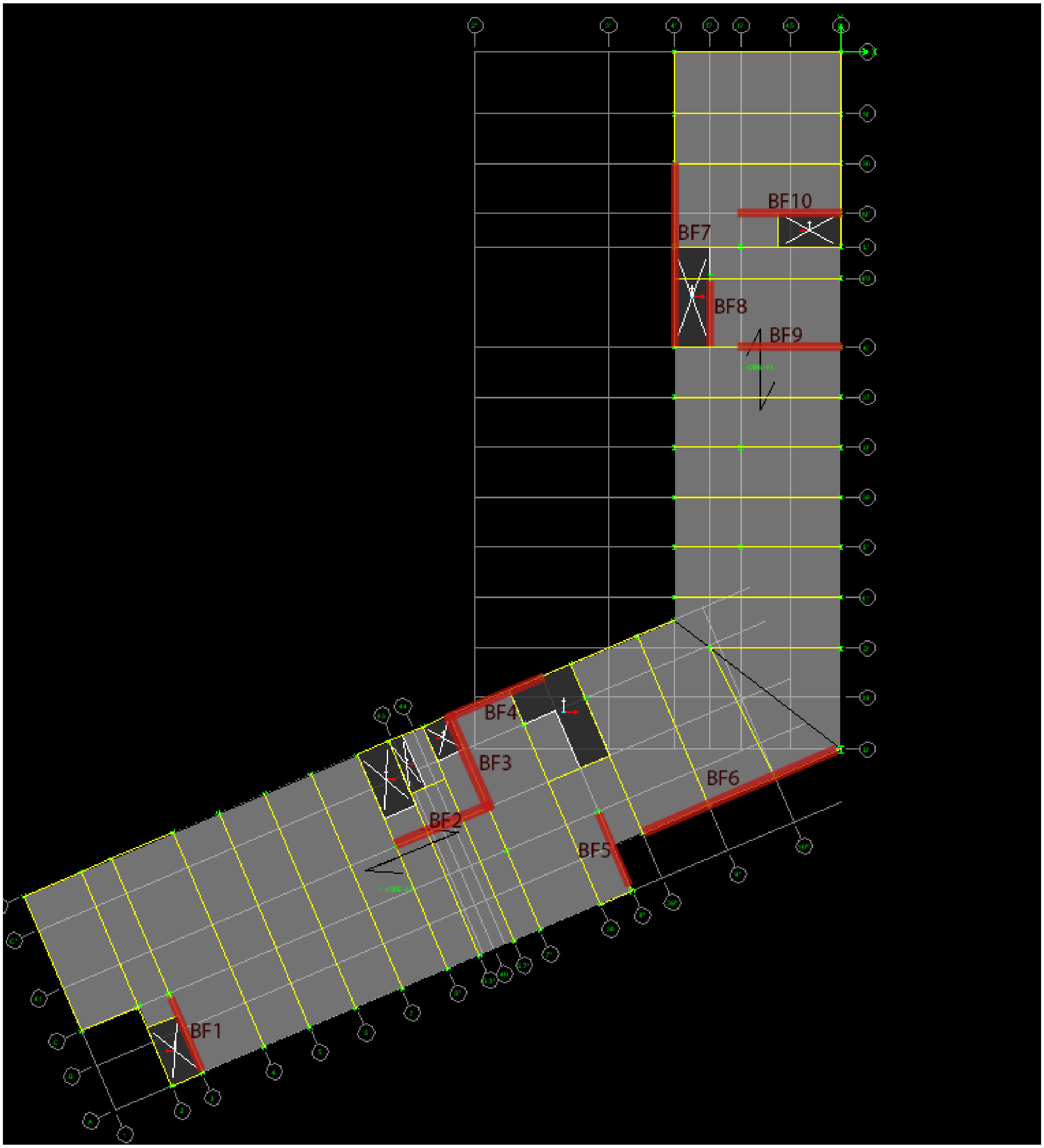


FIGURE 17: BRACED FRAMES LOCATION FOR THE PROPOSED DESIGN

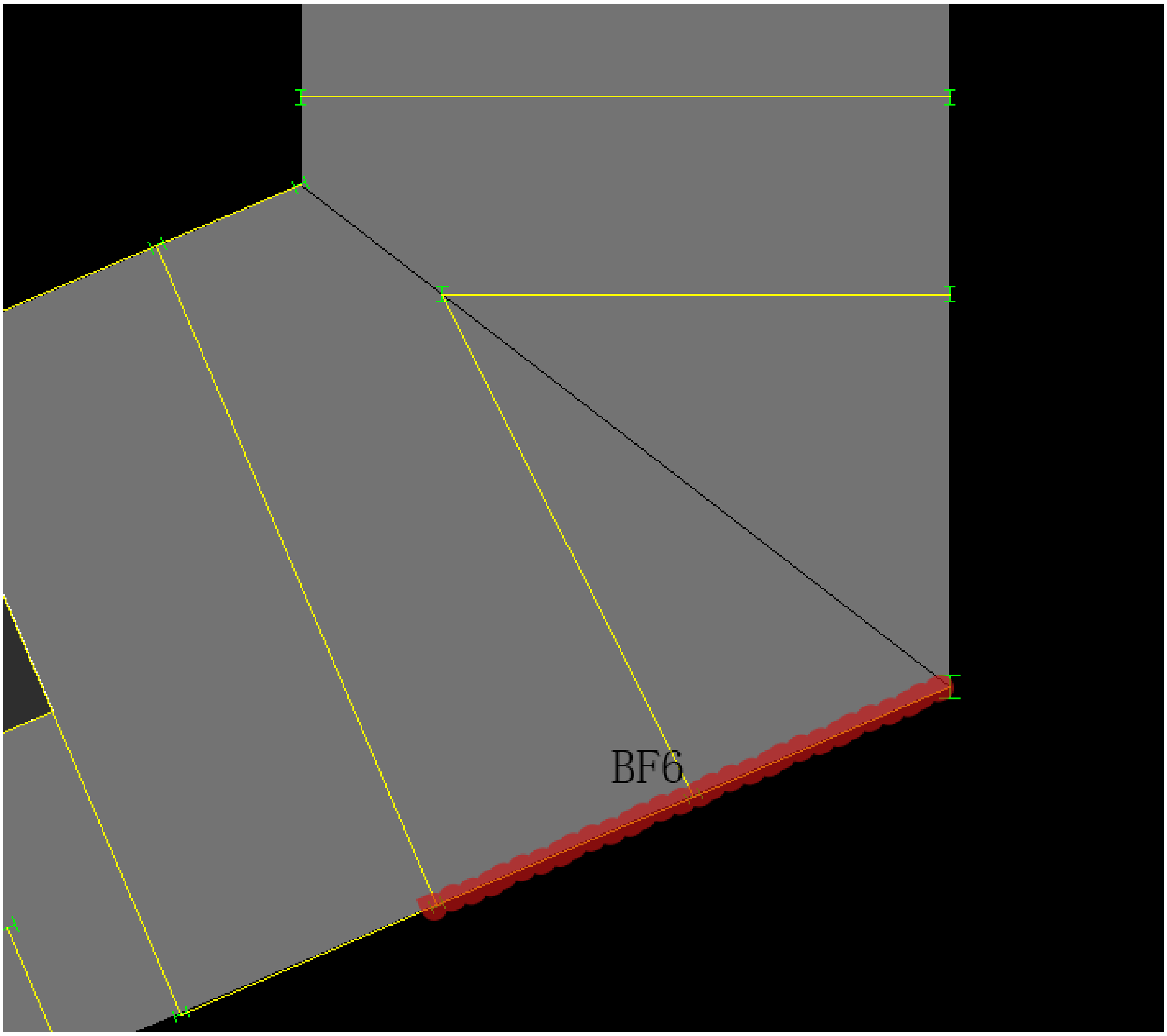
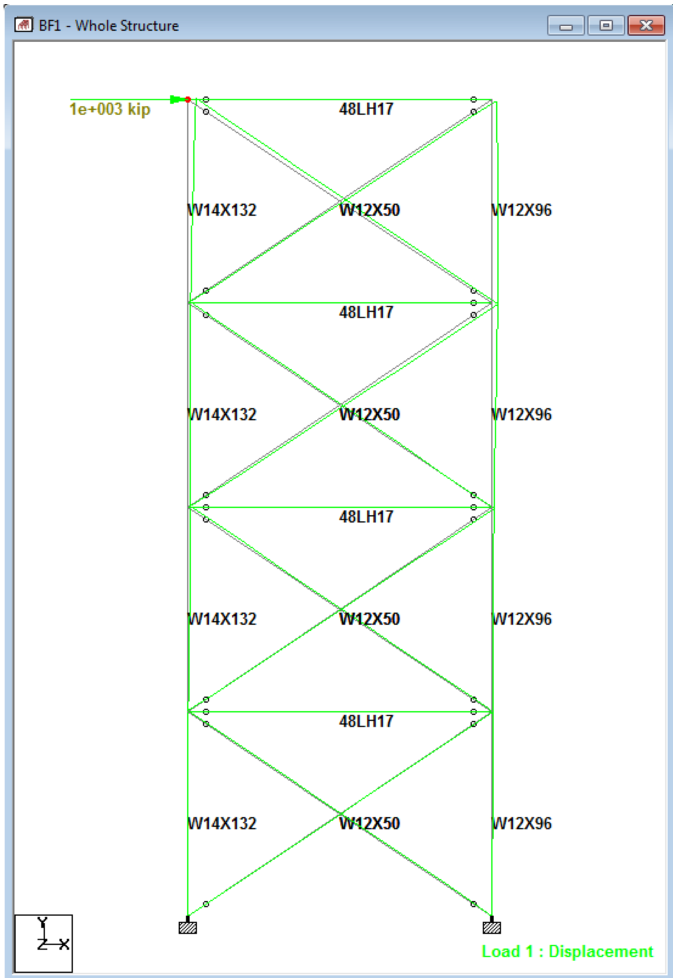


Figure 18: braced frame 6 of the proposed design



BF1 - Node Displacements:

All | Summary

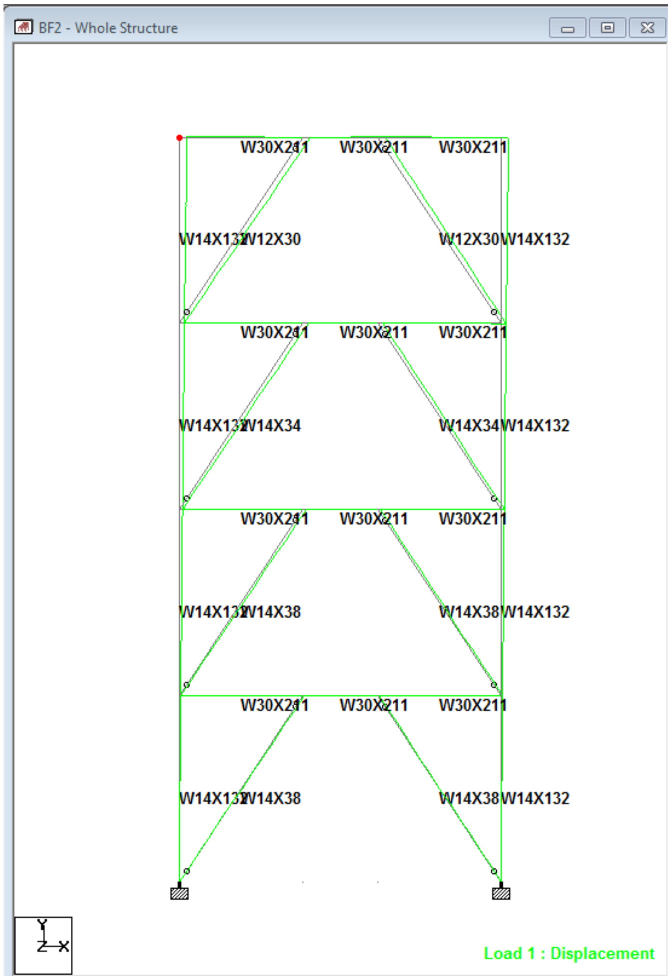
		Horizontal	Vertical	Horizontal	Resultant	
Node	L/C	X in	Y in	Z in	in	rX rad
1	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
2	1 LOAD CAS	0.661	0.426	0.000	0.786	0.000
3	1 LOAD CAS	3.399	0.788	0.000	3.489	0.000
4	1 LOAD CAS	3.453	0.933	0.000	3.577	0.000
5	1 LOAD CAS	8.979	1.058	0.000	9.041	0.000
6	1 LOAD CAS	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



BF2 - Node Displacements:

All / Summary

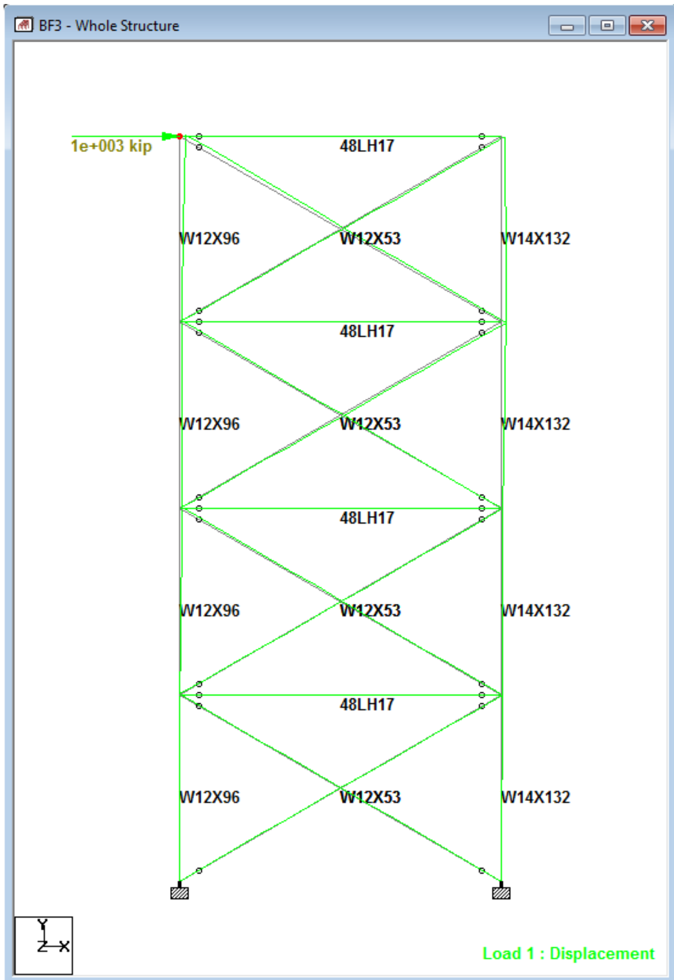
		Horizontal	Vertical	Horizontal	Resultant	
Node	L/C	X in	Y in	Z in	in	rX rad
4	1 LOAD CAS	5.961	0.589	0.000	5.990	0.000
5	1 LOAD CAS	8.664	0.564	0.000	8.683	0.000
6	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
7	1 LOAD CAS	1.553	-0.309	0.000	1.584	0.000
8	1 LOAD CAS	3.560	-0.505	0.000	3.595	0.000
9	1 LOAD 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

BF2 - 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	0.200	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

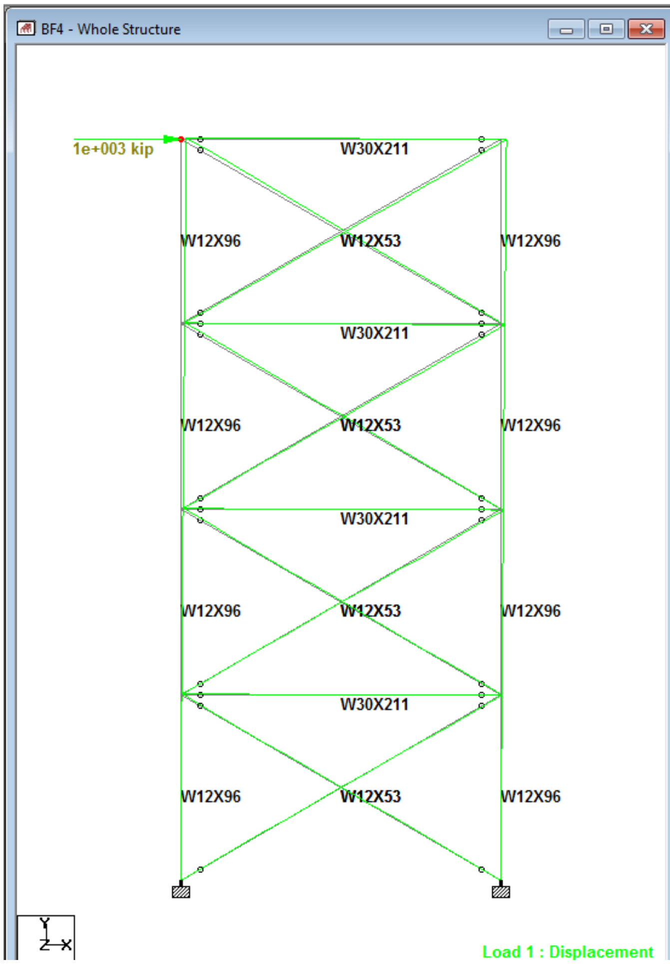
		Horizontal	Vertical	Horizontal	Resultant	
Node	L/C	X in	Y in	Z in	in	rX rad
1	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
2	1 LOAD CAS	0.617	0.504	0.000	0.797	0.000
3	1 LOAD CAS	2.986	0.935	0.000	3.129	0.000
4	1 LOAD CAS	2.721	1.106	0.000	2.937	0.000
5	1 LOAD CAS	7.648	1.257	0.000	7.750	0.000
6	1 LOAD CAS	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

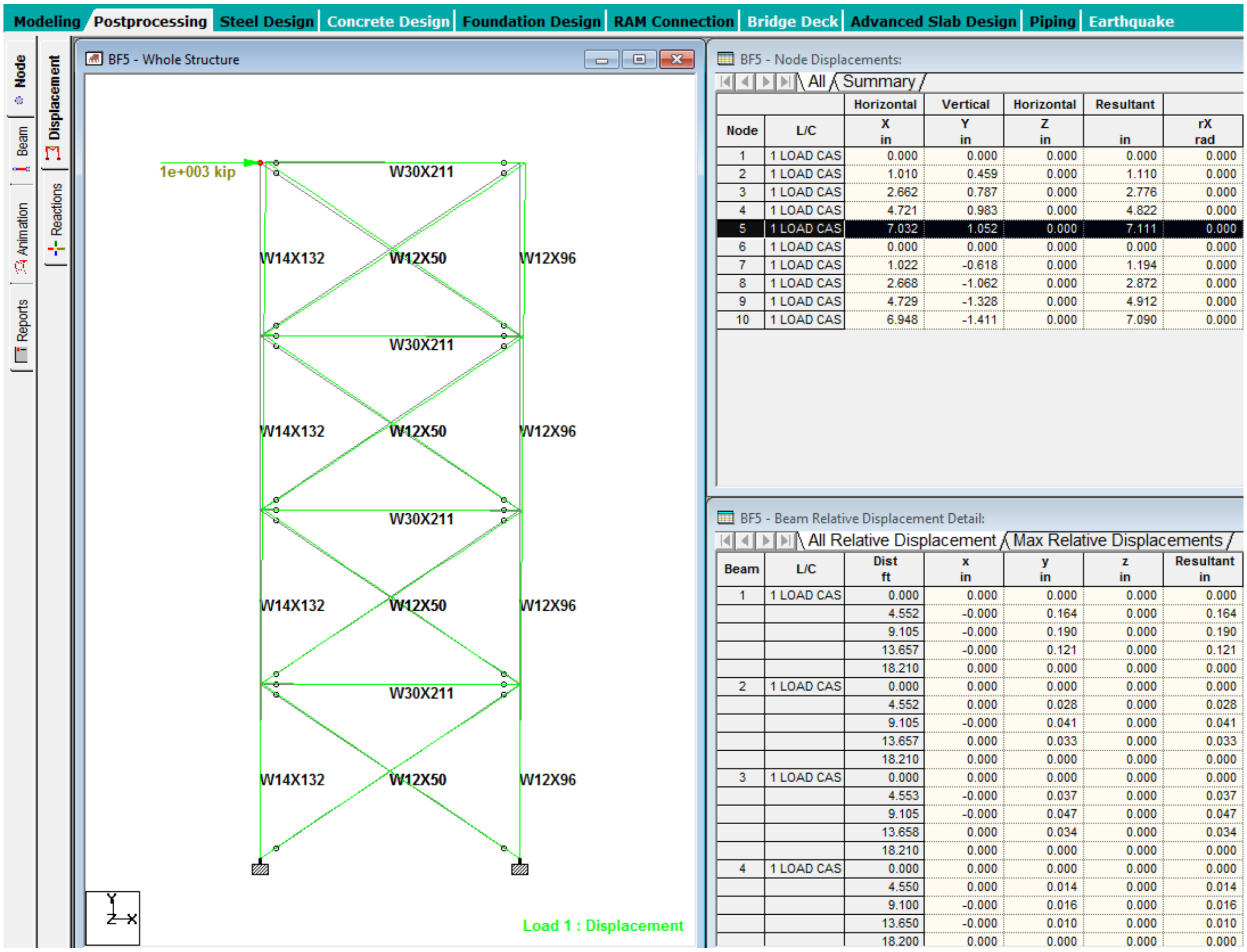
		Horizontal	Vertical	Horizontal	Resultant	
Node	L/C	X in	Y in	Z in	in	rX rad
1	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
2	1 LOAD CAS	0.948	0.540	0.000	1.090	0.000
3	1 LOAD CAS	2.440	0.925	0.000	2.609	0.000
4	1 LOAD CAS	4.283	1.157	0.000	4.436	0.000
5	1 LOAD CAS	6.360	1.239	0.000	6.479	0.000
6	1 LOAD CAS	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

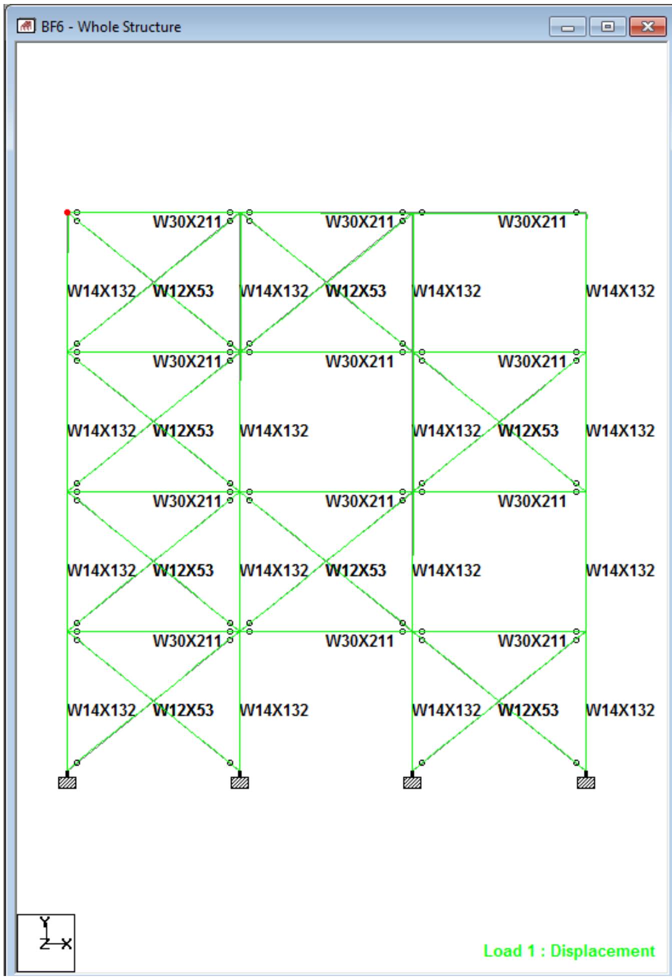
FIGURE 22: BRACED FRAME 4 OF PROPOSED DESIGN



		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

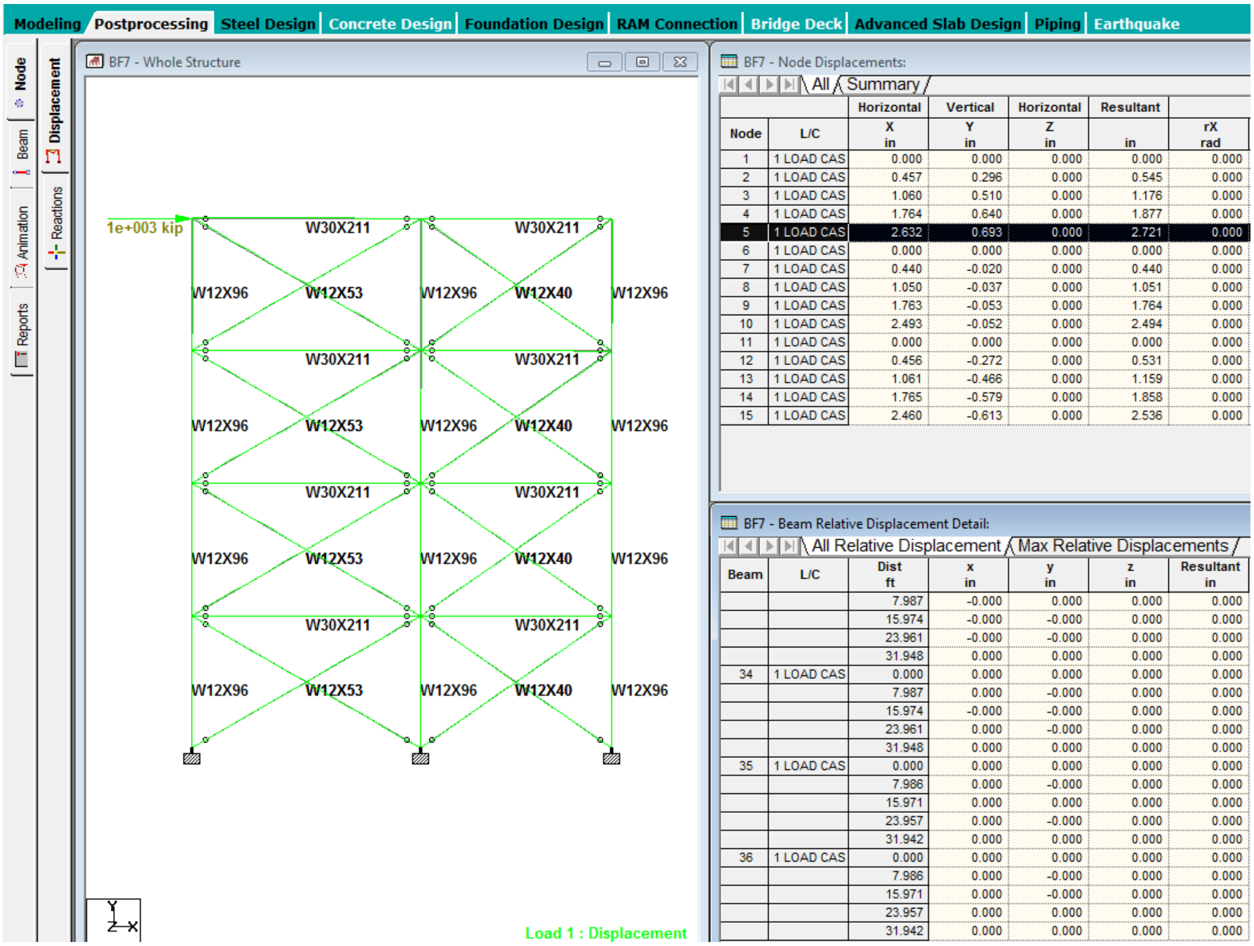
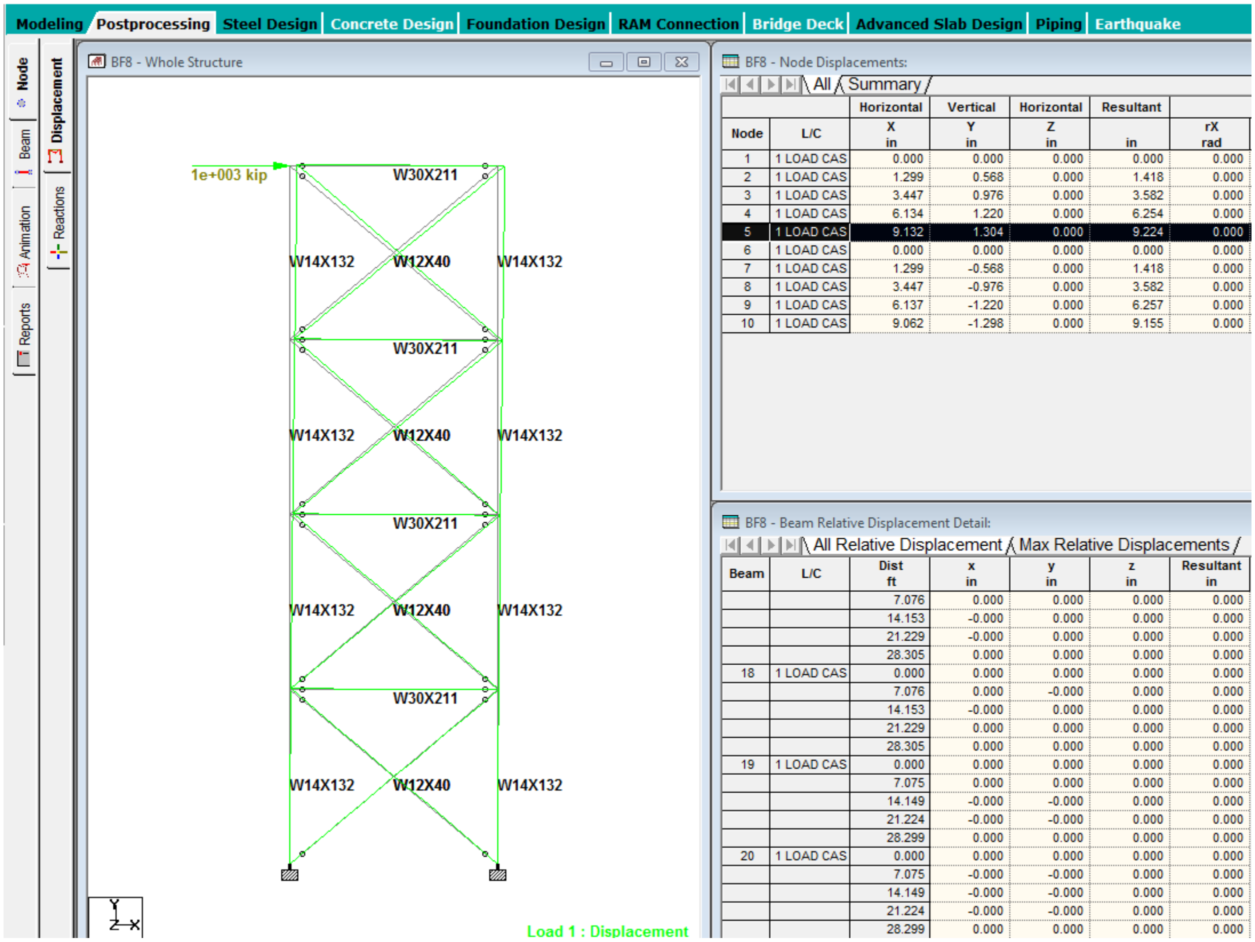


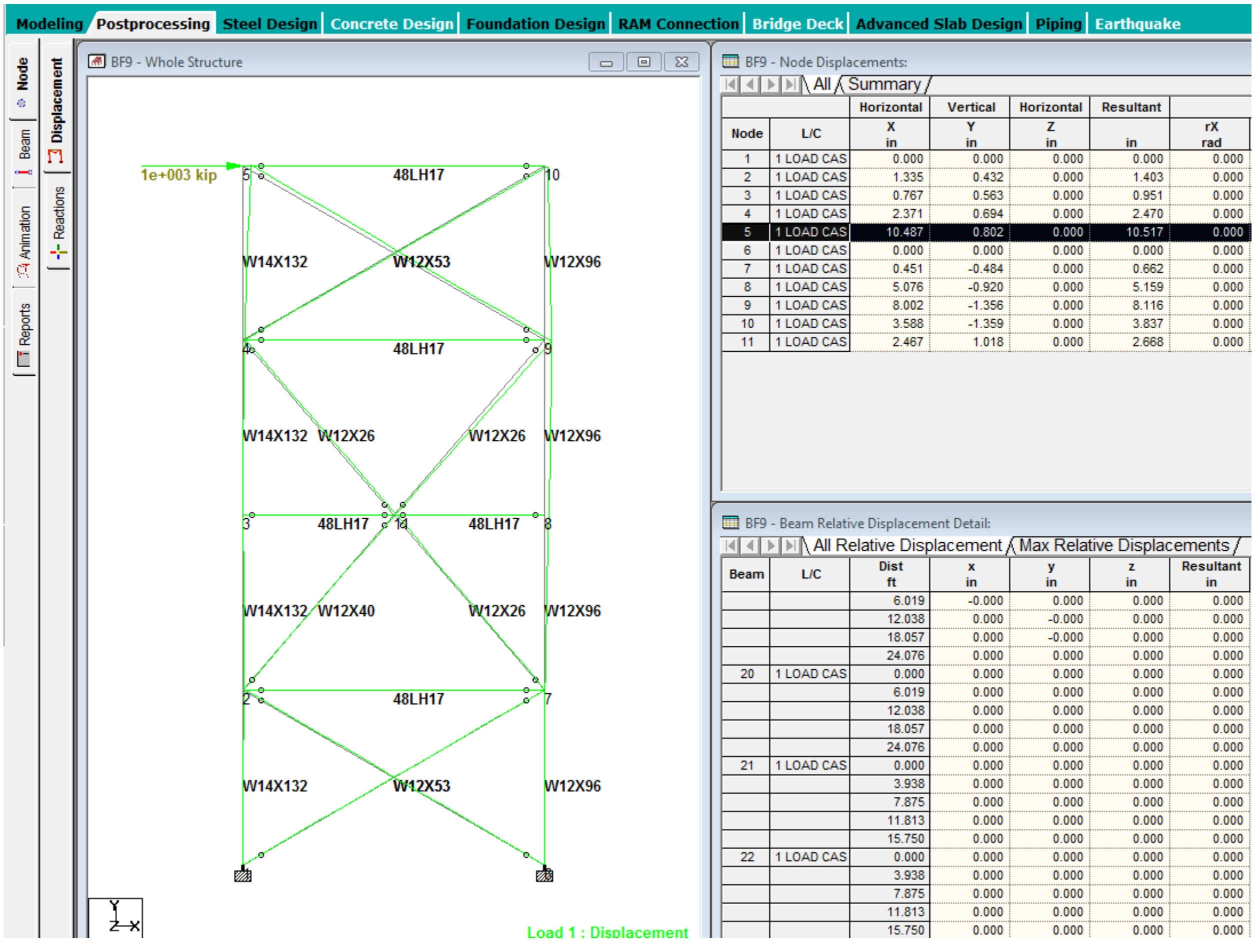
Figure 25: Braced Frame 7 of Proposed Design



		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

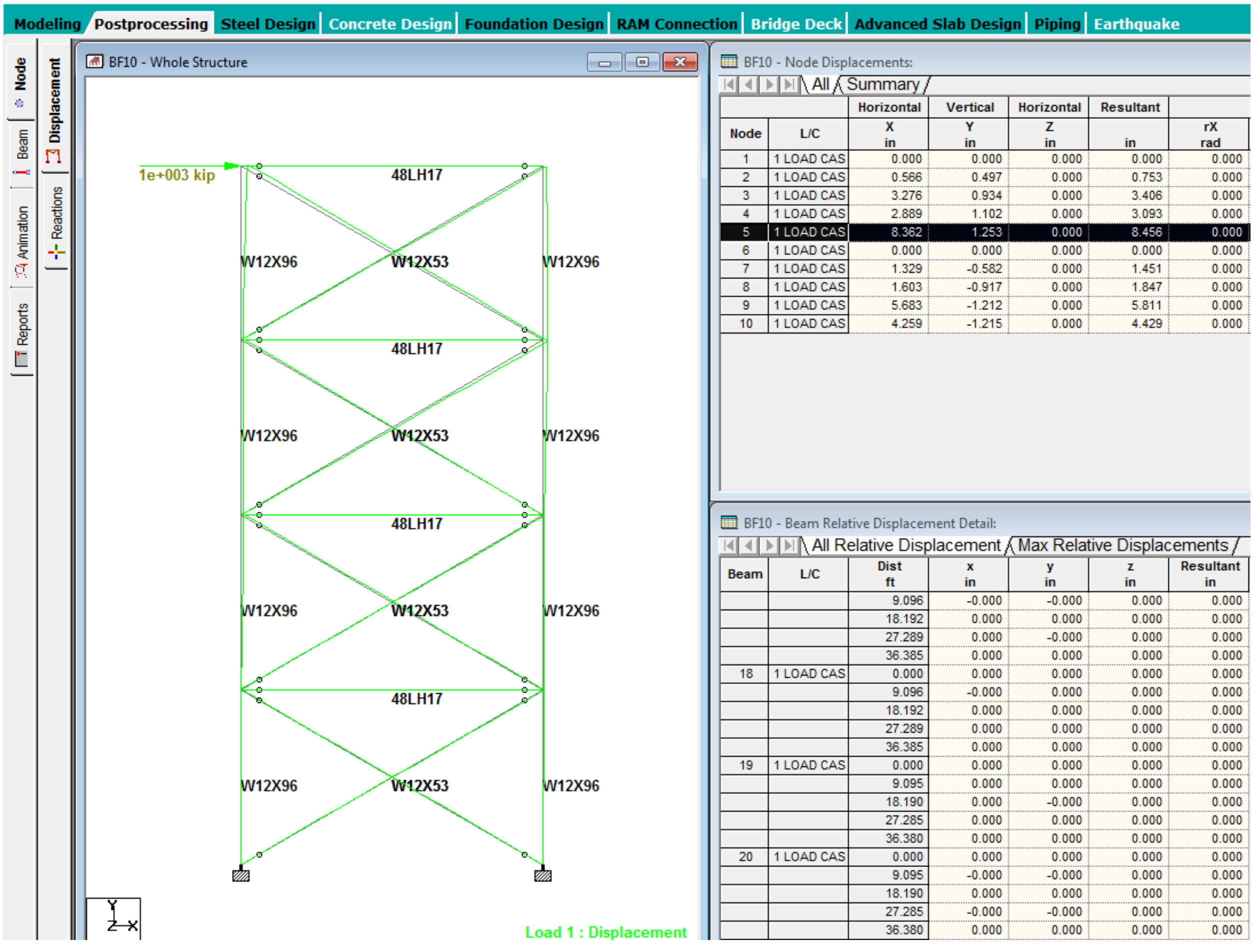


FIGURE 28: BRACED FRAME 10 OF PROPOSED DESIGN

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.

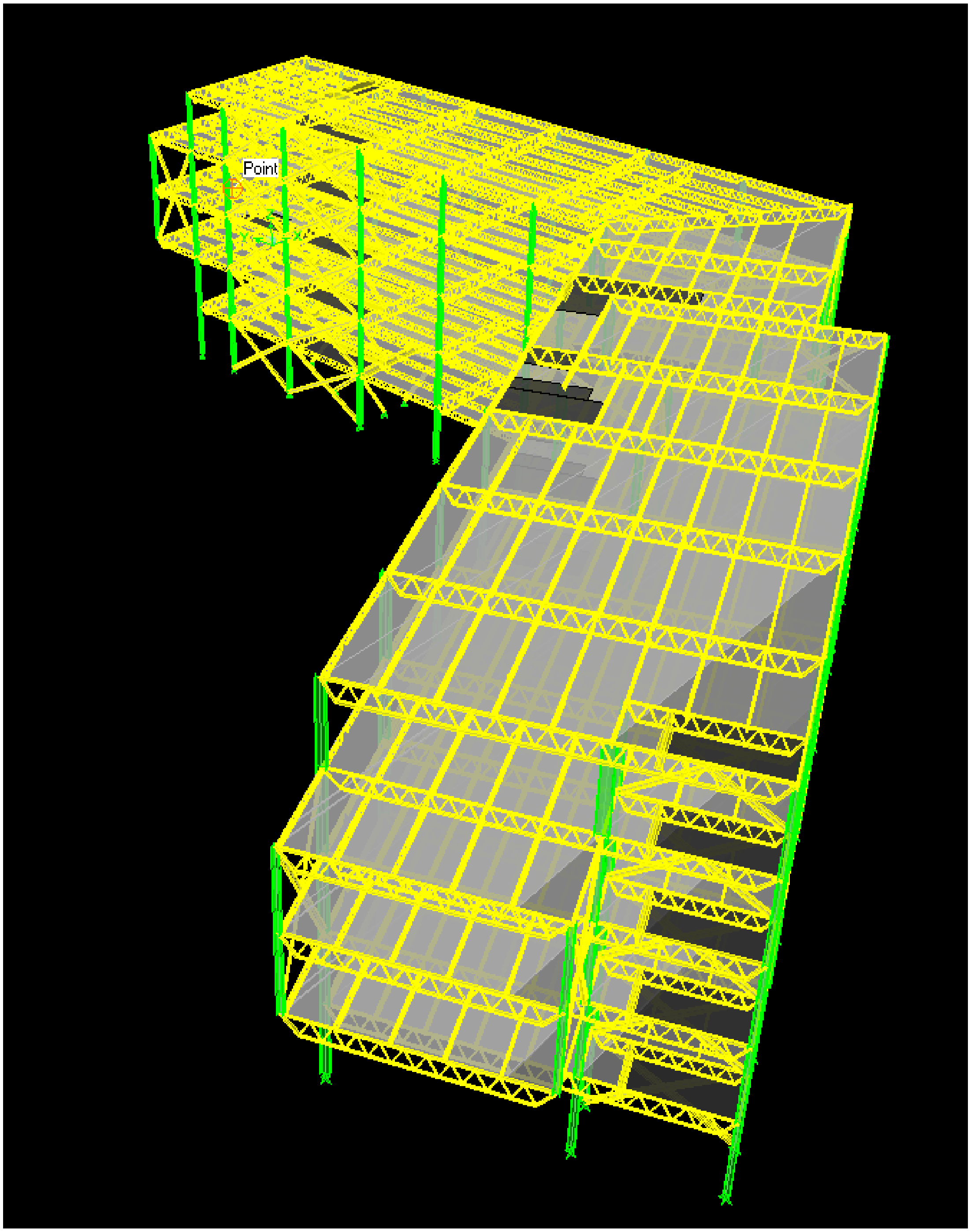


FIGURE 29: ETABS MODEL OF PROPOSED DESIGN

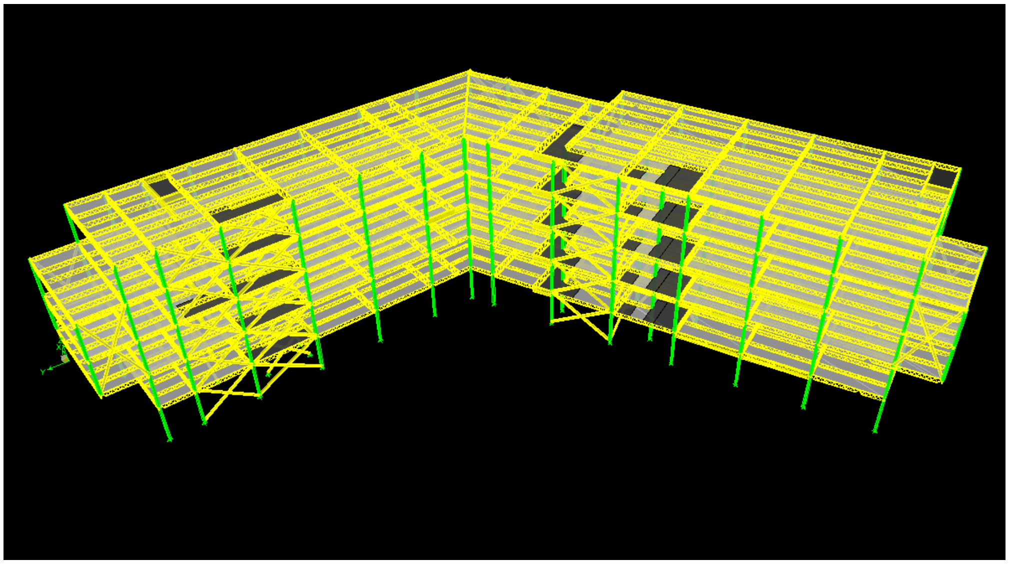


FIGURE 30: ETABS MODEL OF PROPOSED DESIGN

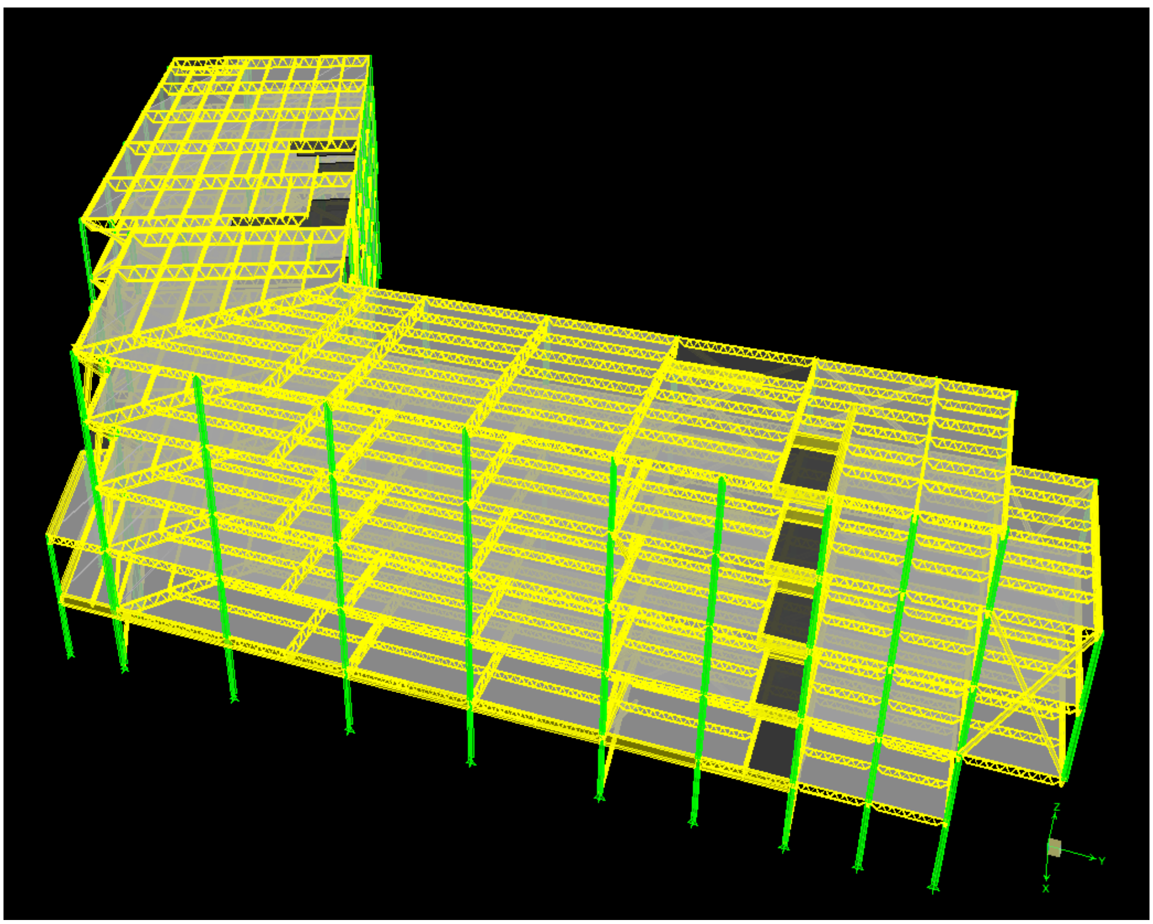


FIGURE 31: ETABS MODEL OF PROPOSED DESIGN

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.

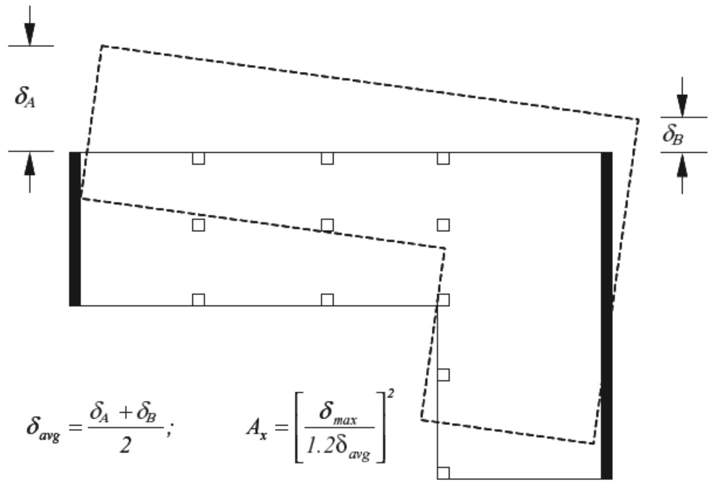


FIGURE 12.8-1 Torsional Amplification Factor, A_x

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 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F 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 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D 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 Table 12.6-1	D, E, and F 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 Table 12.6-1	D, E, and F 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 12.3.3.4 12.7.3 Table 12.6-1 Section 16.2.2	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F 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 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F 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 Table 12.6-1	E and F 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 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F 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 Table 12.6-1	E and F 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 12.3.3.2 Table 12.6-1	D, E, and F B and C 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.

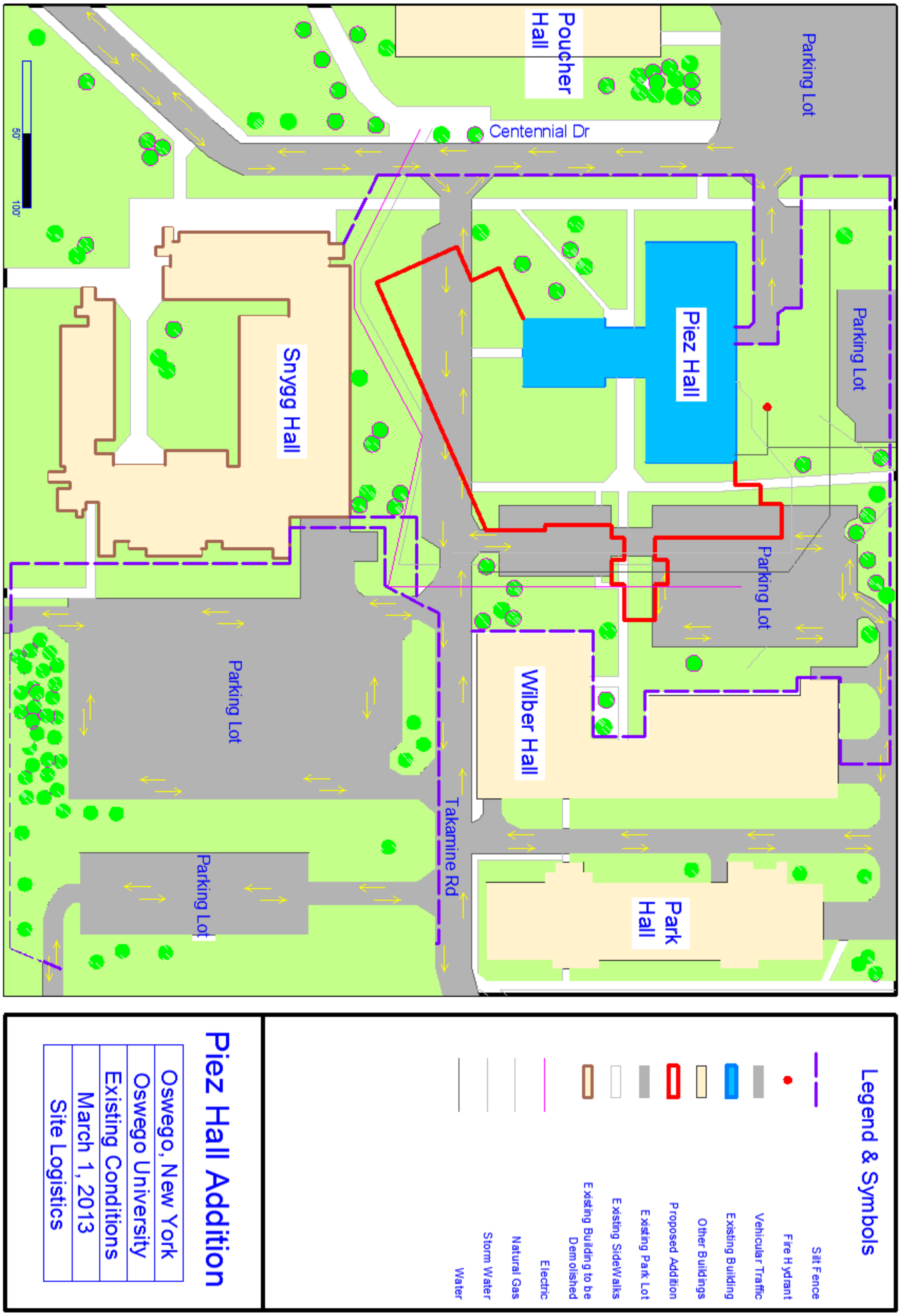


Figure 38: existing condition of construction site

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.

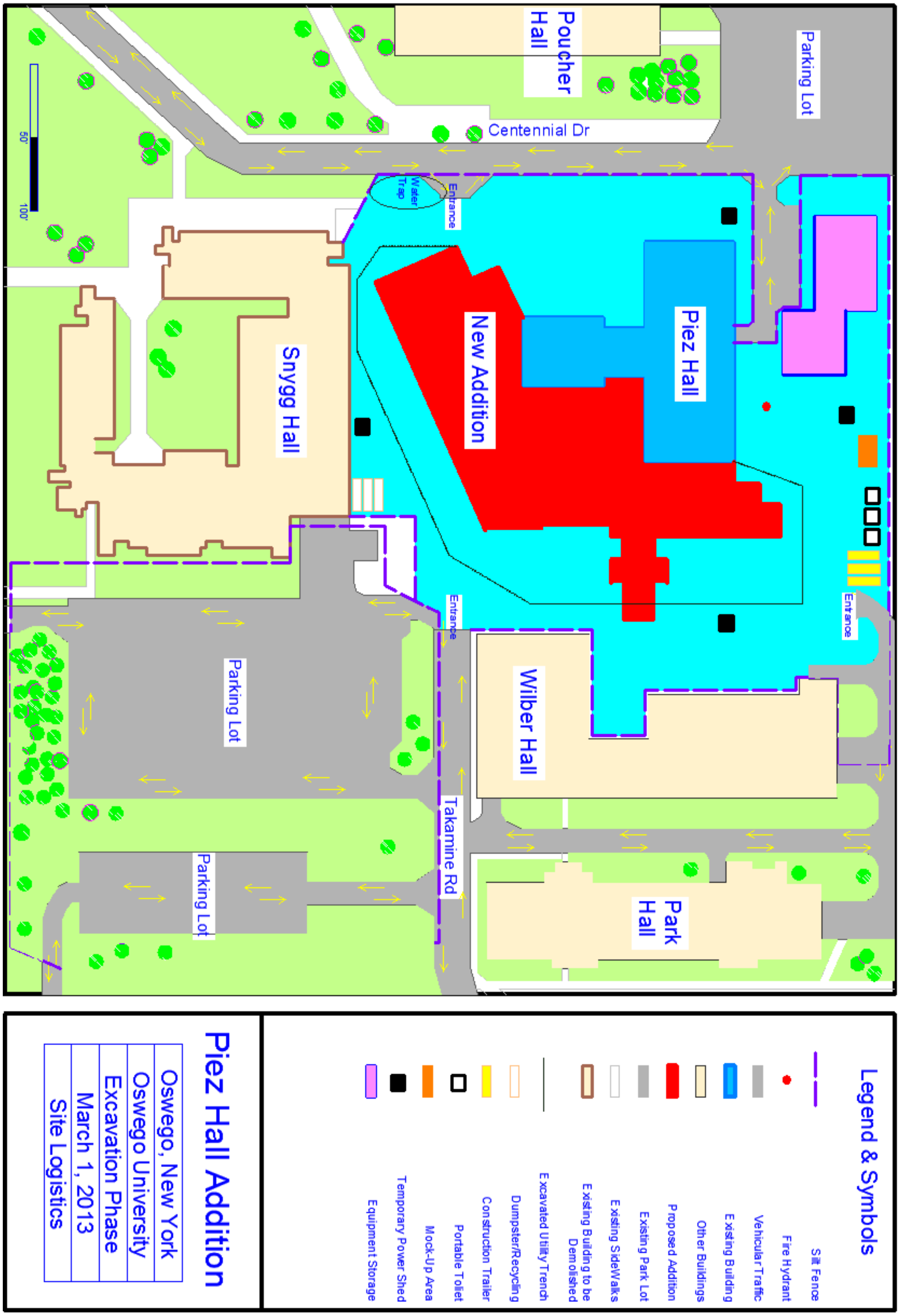


FIGURE 39: EXCAVATION AND MOBILIZATION PHASE OF PROPOSED DESIGN

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.

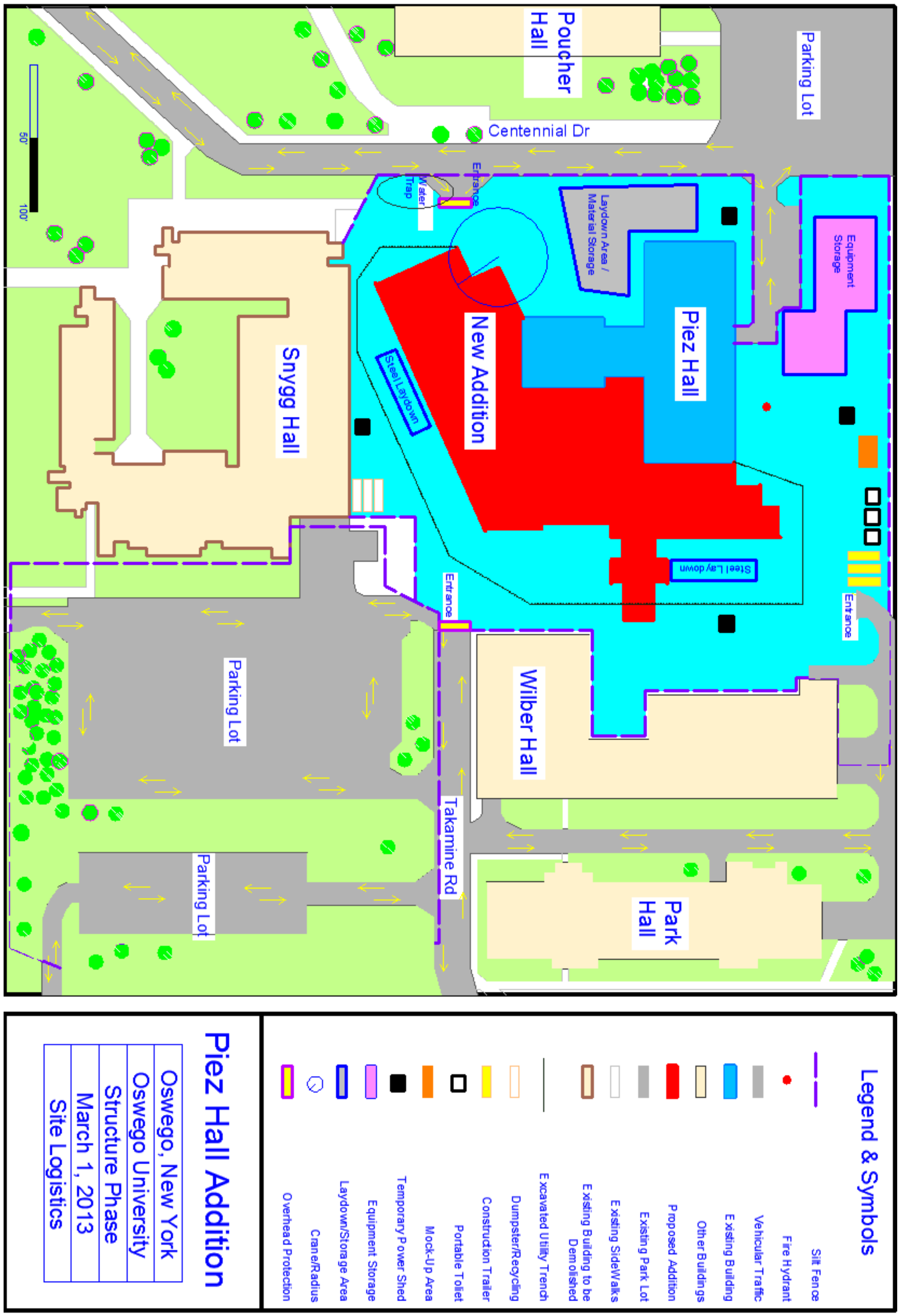


FIGURE 40: STRUCTURE PHASE OF PROPOSED DESIGN

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.

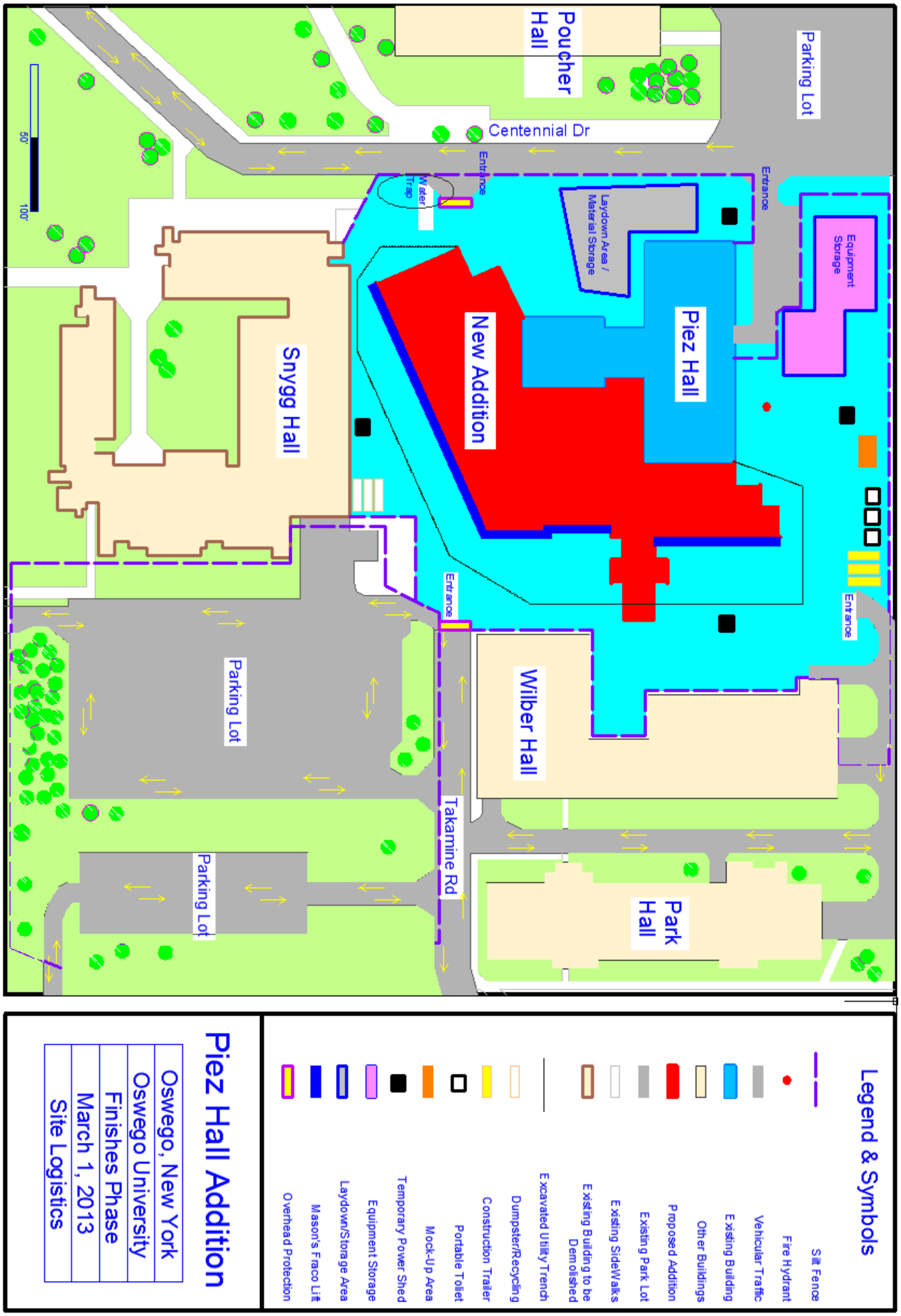


FIGURE 41: FINISHES PHASE OF PROPOSED DESIGN

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.

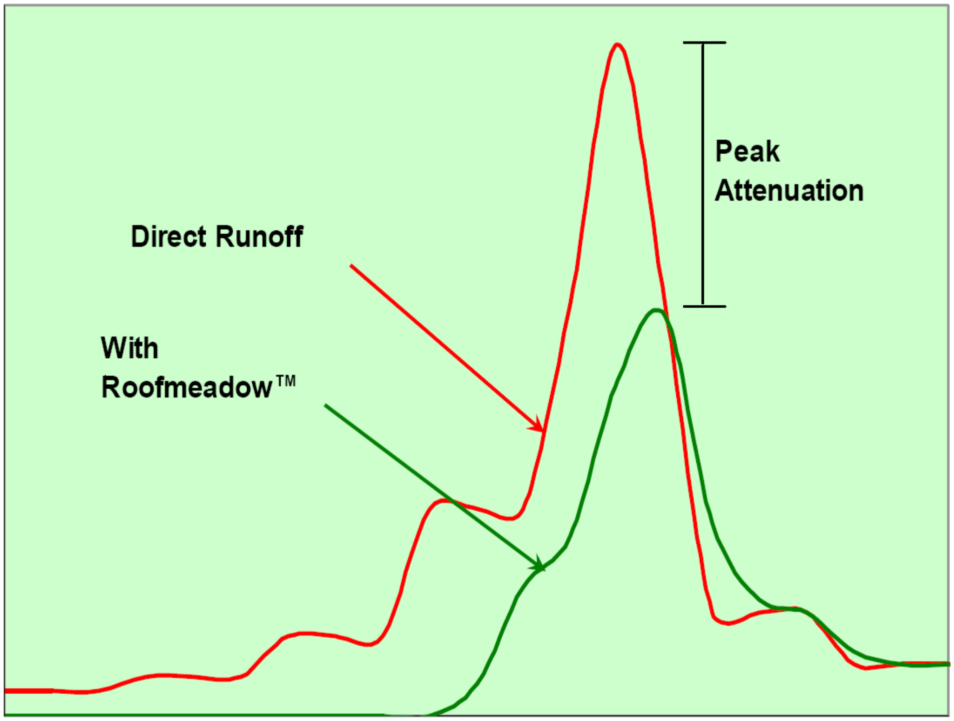


FIGURE 43: STORM WATER RUNOFF RETENTION (PASCHAL, 2006)

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.

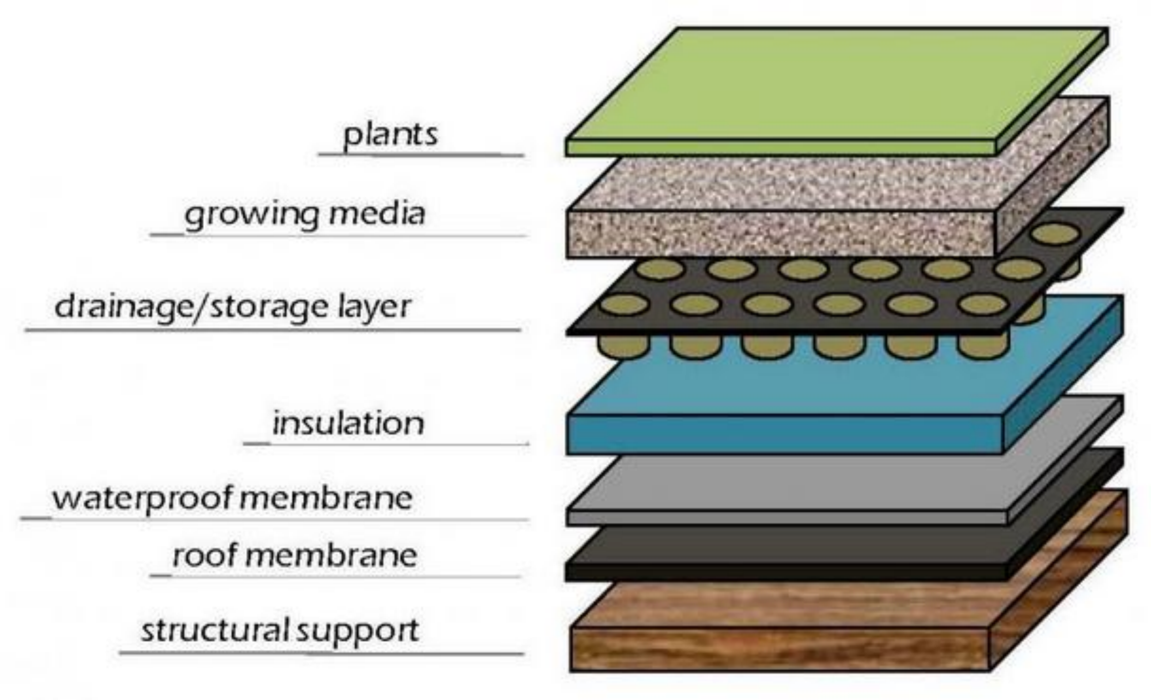


Figure 44: layers of green roof assembly (J-Drain, 2013)

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) → Solution 2: 1-5°	
Special features	→ **Especially cost effective roof greening.** → **Low maintenance green roof design.** → **Relatively low biodiversity.**	
Technical data	Weight:	**Solution 1: 90-140 kg/m² or 0.9-1.4 KN/m² *** **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)

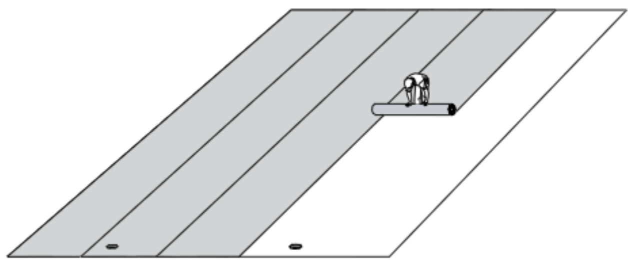


FIGURE 47: INSTALLATION OF WATERPROOF MEMBRANE (CONSERVATION TECHNOLOGY, INC, 2013)

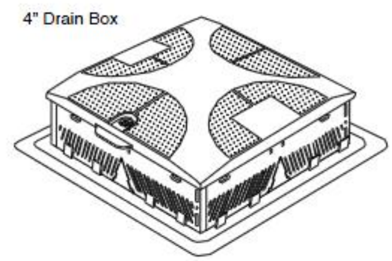


FIGURE 48: DRAIN BOX (CONSERVATION TECHNOLOGY, INC, 2013)

- 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.

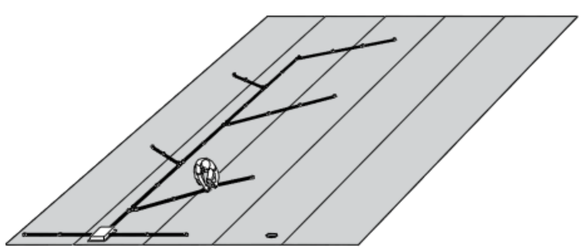


FIGURE 49: SAMPLE LAYOUT FOR CHANNEL PLACEMENT THAT CONNECTS THE DRAIN BOX (CONSERVATION TECHNOLOGY, INC, 2013)

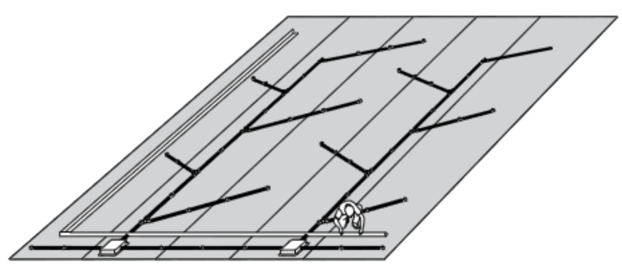


FIGURE 50: INSTALLATION OF RETAINING EDGE (CONSERVATION TECHNOLOGY, INC, 2013)

- 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.

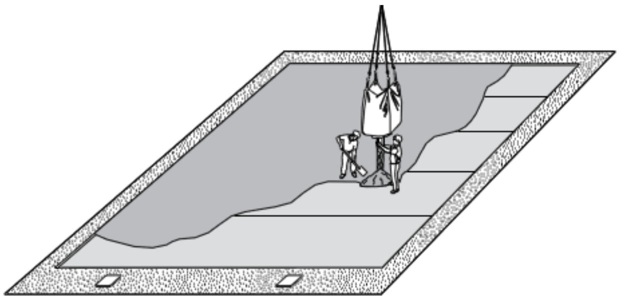


FIGURE 51: SPREAD OF GRAVEL AND SOIL (CONSERVATION TECHNOLOGY, INC, 2013)

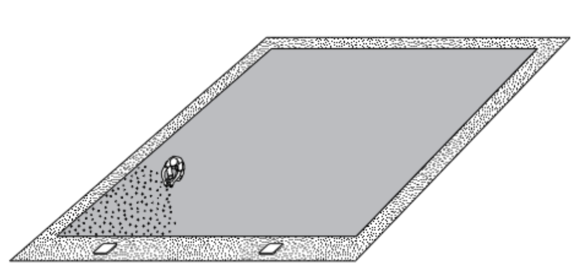


FIGURE 52: PLANT AND COMPLETE THE EXTENSIVE GREEN ROOF SYSTEM (CONSERVATION TECHNOLOGY, INC, 2013)

- 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

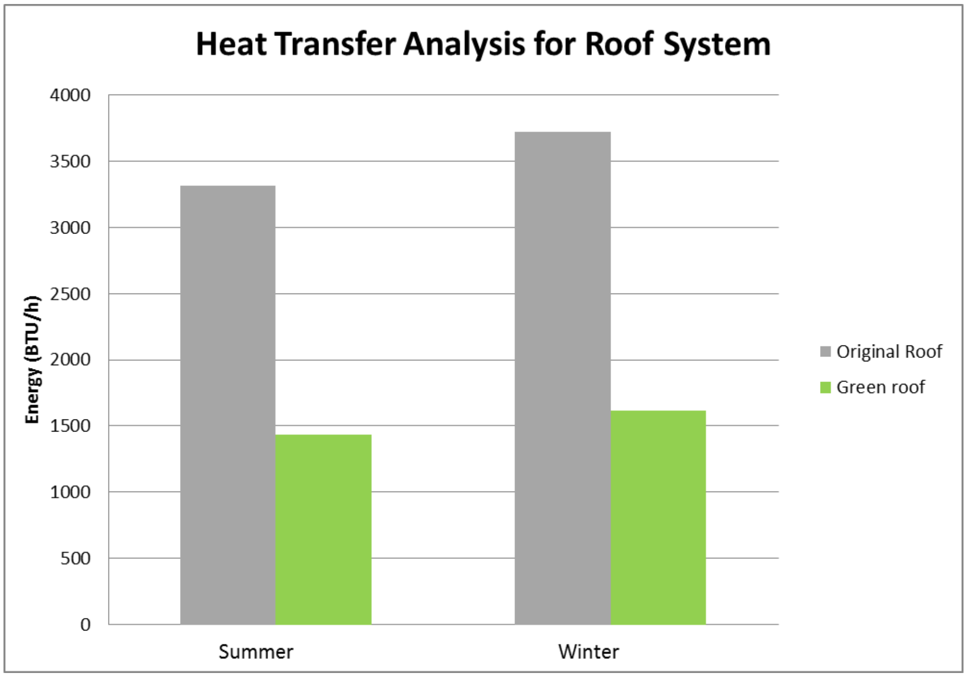


FIGURE 54 HEAT TRANSFER ANALYSIS FOR ROOF SYSTEM

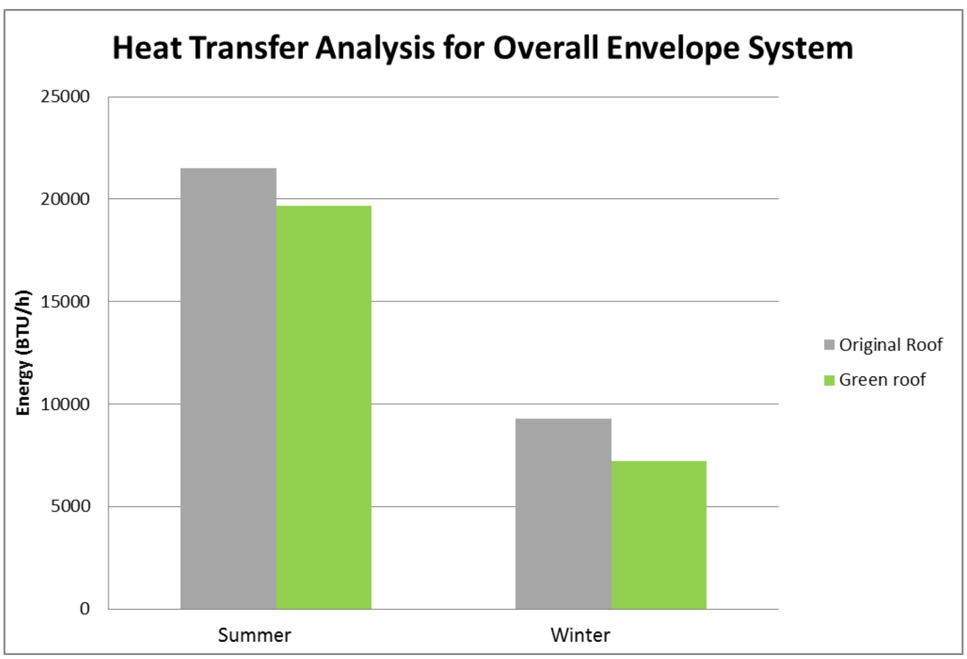


FIGURE 55 HEAT TRANSFER ANALYSIS FOR OVERALL ENVELOPE SYSTEM

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 ➢ 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 ➢ Decrease 25% heating load in winter

TABLE 27 THERMAL RESISTANCE COMPARISON

Green Roof System		
Main Components	➢ structural support ➢ roof membrane ➢ waterproof membrane ➢ insulation	➢ drainage/storage layer ➢ growing media ➢ plants
Drainage System	➢ Channels direct water to drain box. Drain box connects to existing drainage system and to the sewerage system. ➢ 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

Appendix A: Wind Load Calculations

Level	Elevation (ft)	Kz	qz (psf)
1st	0.00	1.03	20.88
2nd	18.21	1.06	21.49
3rd	36.42	1.20	24.32
4th	54.63	1.29	26.15
Roof	72.83	1.35	27.36

Level	Windward	Leeward	Side Wall
1st	14.20	-11.63	-16.28
2nd	14.61	-11.63	-16.28
3rd	16.54	-11.63	-16.28
4th	17.78	-11.63	-16.28
Roof	18.61	-11.63	-16.28

Roof	Cp	
0 to h	-0.90	-20.93
h to 2h	-0.50	-11.63
> 2h	-0.30	-6.98
Windward	0.80	
Leeward	-0.50	
Side Wall	-0.70	

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		0 to h	-20.93	4.93	-4.93	-25.86	-16.01
		h to 2h	-11.63	4.93	-4.93	-16.56	-6.70
		> 2h	-6.98	4.93	-4.93	-11.90	-2.05

Wind Forces N-S direction							
Floor	Elevation	Length (ft)	Tributary Heigh	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		
				Overturning Moment =		10904.93	

Wind Forces E-W direction							
Floor	Elevation	Length (ft)	Tributary Heigh	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		
				Overturning Moment =		9988.24	

Appendix B: Seismic Load Calculations

Seismic Forces

Level	Story Weight, Wx (k)	Story Height, hx (ft)	W*hx^k	Cvx	Story Force (k)	Story Shear (k)	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

k= 1.09 for T = 0.676 (eq 12.8-12)

Façade Weight = 30 psf

Level	Preimeter (ft)	Tributary Height (ft)	Area (ft^2)	Weight (kips)
1.00	1028.70	8.00	8229.60	246.89
2.00	1028.70	16.00	16459.20	493.78
3.00	1028.70	16.00	16459.20	493.78
4.00	795.60	16.00	12729.60	381.89
Roof	1028.70	8.00	8229.60	246.89

Column Weight — Assume W14x44 — 44.00 plf

Level	Numer of column	Tributary Height (ft)	Weight (kips)
1.00	64.00	8.00	22.53
2.00	64.00	16.00	45.06
3.00	64.00	16.00	45.06
4.00	64.00	16.00	45.06
Roof	64.00	8.00	22.53
			180.22

Braces Weight

Level	11 Braced frames	Weight (kips)	Assume 26x60 HSS
1.00	11.00	52.80	
2.00	11.00	52.80	
3.00	11.00	52.80	
4.00	11.00	52.80	
Roof	11.00	52.80	

Slab Weight — 35.00 psf

Level	Floor Area (ft^2)	Slab Wight =35	Weight (kips)
1.00	33964.80	35.00	1188.77
2.00	33964.80	35.00	1188.77
3.00	33964.80	35.00	1188.77
4.00	18631.20	35.00	652.09
Roof	33964.80	56.00	1902.03

Superimposed Dead Load = 20psf

Level	Floor Area (ft^2)	Weight (kips)
1.00	33964.80	679.30
2.00	33964.80	679.30
3.00	33964.80	679.30
4.00	18631.20	372.62
Roof	33964.80	679.30

Total Weight per Level

	Level	Weight (kips)	Weight (psf)
	1.00	2190.28	64.49
	2.00	2459.70	72.42
	3.00	2459.70	72.42
	4.00	1504.46	80.75
	Roof	2903.54	85.49
	Total Weight	11517.67	
	V	446.89	
Existing	V	1040	

Appendix C: Center of Rigidity and Center of Mass Calculations

	δ (in)	k (kip/in)	kx	ky	X position (in)	Y position (in)	ki*xi	ki*yi	Weight	Wi*xi	Wi*yi
Brace1	8.98	111.37	43.43	102.46	2759.90	4031.86	282782.58	175122.55	16644.00	45935717.35	67106277.84
Brace2	8.66	115.42	106.19	45.01	1655.94	3263.88	74540.35	346580.06	16644.00	27561506.97	54324018.72
Brace3	7.65	130.75	50.99	120.29	749.05	1247.96	90105.69	63638.13	16644.00	12467229.81	20771046.24
Brace4	6.36	157.23	144.65	61.32	1221.24	2885.88	74887.51	417454.34	16644.00	20326360.17	48032586.72
Brace5	7.03	142.21	55.46	130.83	883.17	3108.26	115544.97	172386.43	16644.00	14699406.58	51733879.44
Brace6	2.08	481.23	442.73	187.68	693.16	2793.37	130093.06	1236718.19	32105.00	22253982.06	89681143.85
Brace7	2.63	379.94	0.00	379.94	726.80	737.00	276139.82	0.00	24966.00	18145288.80	18399942.00
Brace8	7.03	142.25	0.00	142.25	567.81	985.00	80769.91	0.00	16644.00	9450671.25	16394340.00
Brace9	9.13	109.51	109.51	0.00	217.35	1115.00	0.00	122098.12	16644.00	3617573.40	18558060.00
Brace10	8.36	119.59	119.59	0.00	217.35	611.00	0.00	73068.64	16644.00	3617573.40	10169484.00
Total			1072.56	1169.79			1124863.89	2607066.46	5310023.00	6076084909.79	12436940378.81

Center of Rigdity (Top Left of Brace 8 as reference point)		relative error
x=	1048.77 in	2.05
y=	2228.67 in	1.80
ETABS x=	1070.25 in	
ETABS y=	2268.76 in	
11.62	96.00	

Center of Mass (Top Left of Brace as reference point)		e (ft)	
x=	1144.27 in	-4.10	
y=	2342.16 in	1.83	
ETABS x=	1097.34 in		-2.26
ETABS y=	2300.09 in		2.61
Slab Weight	5119800.00		
x=	1152.00		
y=	2352.00		

Appendix E: Wind Load Distribution Calculation

Torsion Calculation

	k (kip/in)	kx	ky	dix	diy	kiy*dix^2	kix*diy^2
Brace1	111.37	43.43	102.46	1689.64	-1763.10	292515918.88	135017337.95
Brace2	115.42	106.19	45.01	585.69	-995.12	15441146.98	105152244.60
Brace3	130.75	50.99	120.29	-321.20	1020.80	12410665.71	53137332.90
Brace4	157.23	144.65	61.32	150.99	-617.12	1397961.57	55089285.51
Brace5	142.21	55.46	130.83	-187.09	-839.50	4579342.76	39086346.40
Brace6	481.23	442.73	187.68	-377.09	-524.61	26687786.22	121846231.65
Brace7	379.94	0.00	379.94	-343.45	1531.76	44817876.18	0.00
Brace8	142.25	0.00	142.25	-502.44	1283.76	35910022.89	0.00
Brace9	109.51	109.51	0.00	-852.90	1153.76	0.00	145769464.81
Brace10	119.59	119.59	0.00	-852.90	1657.76	0.00	328650424.38
Total		1072.56	1169.79			1417509389.38	

		Case 1 NS		
		p=	-86.18	kip (↑)
		ex=	-27.09	in
	Torsional Shear x (k	Torsional Shear y (k	Direct Shear (k)	Total Shear (k)
Brace1	-0.13	0.29	-7.55	-7.39
Brace2	-0.17	0.04	-3.32	-3.45
Brace3	0.09	-0.06	-8.86	-8.84
Brace4	-0.15	0.02	-4.52	-4.65
Brace5	-0.08	-0.04	-9.64	-9.76
Brace6	-0.38	-0.12	-13.83	-14.33
Brace7	0.00	-0.21	-27.99	-28.21
Brace8	0.00	-0.12	-10.48	-10.60
Brace9	0.21	0.00	0.00	0.21
Brace10	0.33	0.00	0.00	0.33

		Case 1 EW		
		p=	-86.18	kip (→)
		ey=	31.33	in
	Torsional Shear x (k	Torsional Shear y (k	Direct Shear (k)	Total Shear (k)
Brace1	0.15	-0.33	-3.49	-3.67
Brace2	0.20	-0.05	-8.53	-8.38
Brace3	-0.10	0.07	-4.10	-4.12
Brace4	0.17	-0.02	-11.62	-11.47
Brace5	0.09	0.05	-4.46	-4.32
Brace6	0.44	0.13	-35.57	-35.00
Brace7	0.00	0.25	0.00	0.25
Brace8	0.00	0.14	0.00	0.14
Brace9	-0.24	0.00	-8.80	-9.04
Brace10	-0.38	0.00	-9.61	-9.99

		Case 2 NS+e		
		p=	-86.18	kip (↑)
		ex=	409.26	in
	Torsional Shear x (k	Torsional Shear y (k	Direct Shear (k)	Total Shear (k)
Brace1	1.91	-4.31	-7.55	-9.95
Brace2	2.63	-0.66	-3.32	-1.34
Brace3	-1.30	0.96	-8.86	-9.20
Brace4	2.22	-0.23	-4.52	-2.53
Brace5	1.16	0.61	-9.64	-7.87
Brace6	5.78	1.76	-13.83	-6.29
Brace7	0.00	3.25	-27.99	-24.74
Brace8	0.00	1.78	-10.48	-8.70
Brace9	-3.14	0.00	0.00	-3.14
Brace10	-4.93	0.00	0.00	-4.93

		Case 2 EW+e		
		p=	-86.18	kip (→)
		ex=	532.45	in
	Torsional Shear x (k	Torsional Shear y (k	Direct Shear (k)	Total Shear (k)
Brace1	2.48	-5.60	-3.49	-6.62
Brace2	3.42	-0.85	-8.53	-5.96
Brace3	-1.69	1.25	-4.10	-4.53
Brace4	2.89	-0.30	-11.62	-9.03
Brace5	1.51	0.79	-4.46	-2.16
Brace6	7.52	2.29	-35.57	-25.76
Brace7	0.00	4.22	0.00	4.22
Brace8	0.00	2.31	0.00	2.31
Brace9	-4.09	0.00	-8.80	-12.89
Brace10	-6.42	0.00	-9.61	-16.03

		Case 2 NS-e		
		p=	-86.18	kip (↑)
		ex=	463.44	in
	Torsional Shear x (k	Torsional Shear y (k	Direct Shear (k)	Total Shear (k)
Brace1	2.16	-4.88	-7.55	-10.27
Brace2	2.98	-0.74	-3.32	-1.08
Brace3	-1.47	1.09	-8.86	-9.24
Brace4	2.52	-0.26	-4.52	-2.26
Brace5	1.31	0.69	-9.64	-7.64
Brace6	6.54	1.99	-13.83	-5.29
Brace7	0.00	3.68	-27.99	-24.31
Brace8	0.00	2.01	-10.48	-8.47
Brace9	-3.56	0.00	0.00	-3.56
Brace10	-5.59	0.00	0.00	-5.59

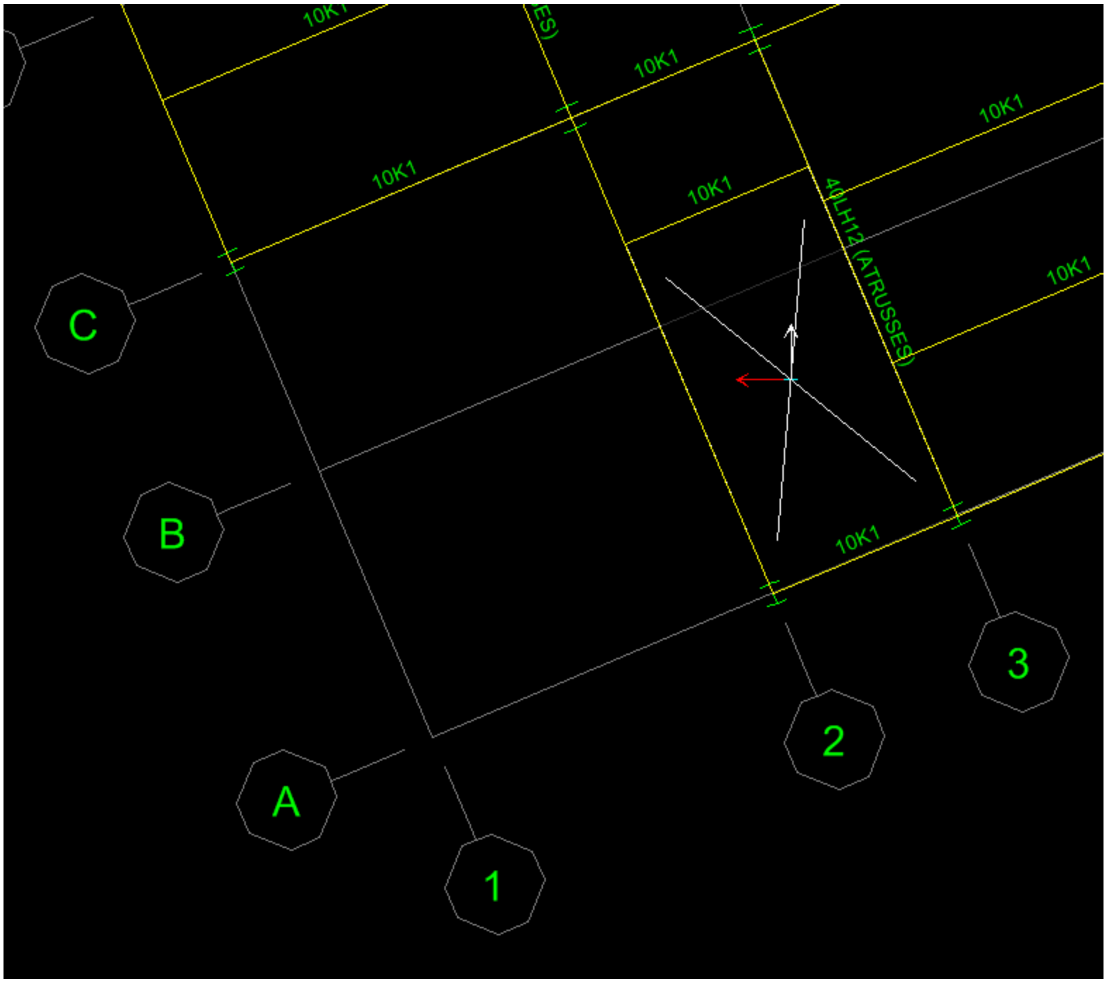
		Case 2 EW-e		
		p=	-86.18	kip (↑)
		ex=	469.79	in
	Torsional Shear x (k	Torsional Shear y (k	Direct Shear (k)	Total Shear (k)
Brace1	2.19	-4.94	-3.49	-6.25
Brace2	3.02	-0.75	-8.53	-6.27
Brace3	-1.49	1.10	-4.10	-4.48
Brace4	2.55	-0.26	-11.62	-9.34
Brace5	1.33	0.70	-4.46	-2.43
Brace6	6.63	2.02	-35.57	-26.92
Brace7	0.00	3.73	0.00	3.73
Brace8	0.00	2.04	0.00	2.04
Brace9	-3.61	0.00	-8.80	-12.41
Brace10	-5.66	0.00	-9.61	-15.27

Case 4 +NSe+EWe	Case 4 +NSe-EWe	Case 4 -NSe+EWe	Case 4 -Nse-EWe	Case 3
Total Shear				Total Shear (k)
-12.44	-12.16	-12.67	-12.40	-8.30
-5.49	-5.71	-5.29	-5.52	-8.87
-10.30	-10.27	-10.34	-10.30	-9.72
-8.68	-8.91	-8.48	-8.71	-12.09
-7.53	-7.73	-7.35	-7.55	-10.56
-24.06	-24.93	-23.31	-24.18	-36.99
-15.40	-15.78	-15.08	-15.45	-20.97
-4.80	-5.00	-4.62	-4.82	-7.85
-12.03	-11.67	-12.35	-11.99	-6.62
-15.73	-15.17	-16.22	-15.66	-7.25

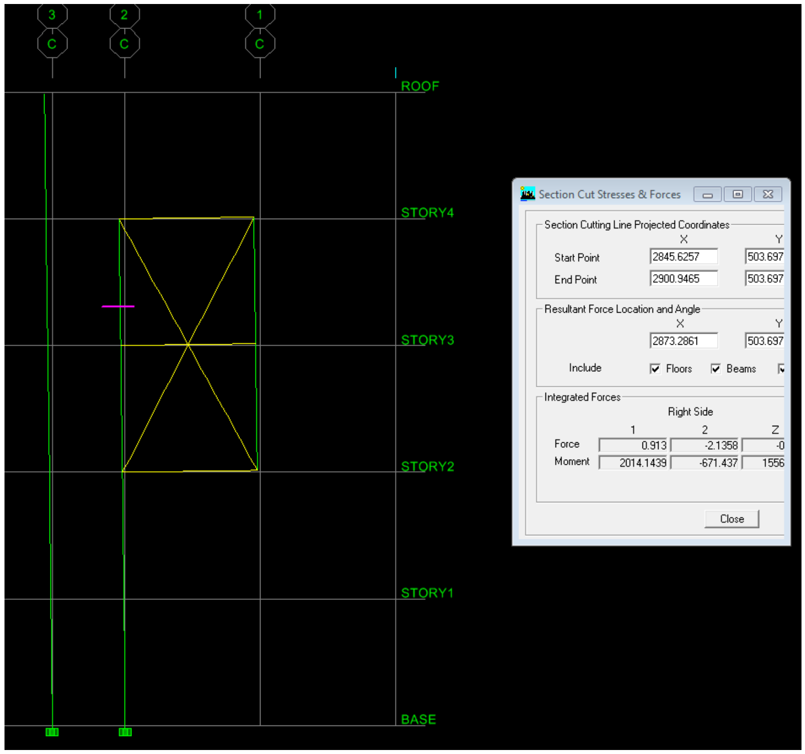
Controlling Force		
Brace1	-3.67	Case 1 EW
Brace2	-8.38	Case 1 EW
Brace3	-8.84	Case 1 NS
Brace4	-4.65	Case 1 NS
Brace5	-9.76	Case 1 NS
Brace6	-14.33	Case 1 NS
Brace7	-28.21	Case 1 NS
Brace8	0.14	Case 1 EW
Brace9	-9.04	Case 1 EW
Brace10	-9.99	Case 1 EW

Appendix D: Spot checks

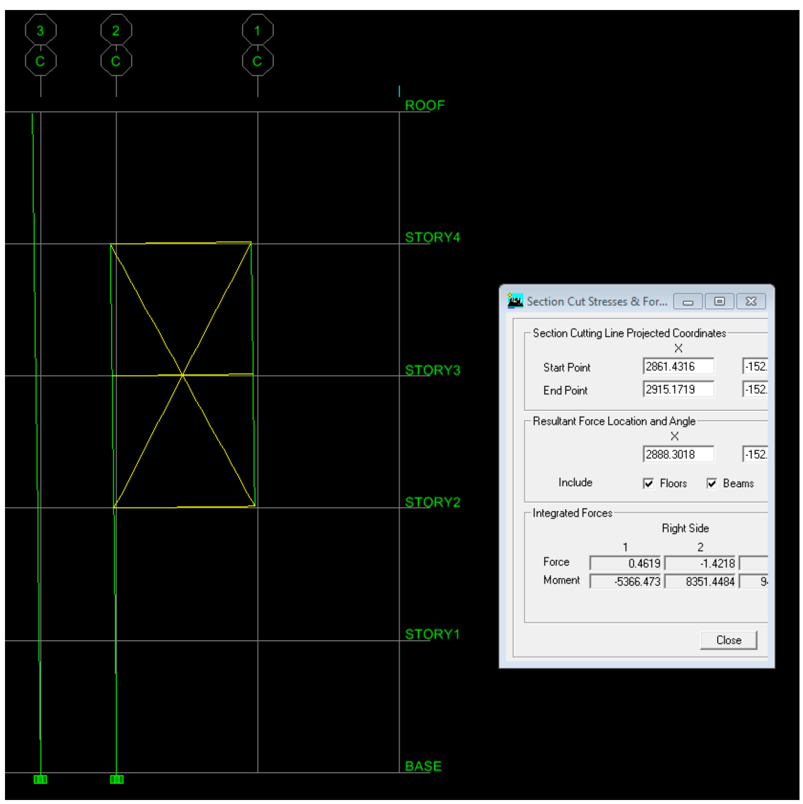
Column C-2 check



Pu of Column C-2



Mu of column C-2



Column C-2 Check | | MinGao Li | Pg1

Selected member is a W14×132

$P_U = 913^{k}$
$M_U = 5367$ kip-in
} Obtained from ETABS.

From AISC 14th Edition

$\phi P_n = 1370^k$ $\quad r_x/r_y = 1.67$ $\quad r_y = 3.76$ in

LC: 1.2D + 1.6W

$$P_U/\phi P_n = \frac{.913 \times 1.2}{1370} < 0.2$$

$$B_1 = C_m(1 - \alpha P_r/P_{e1})^{-1}$$

$$P_{e1} = \frac{\pi^2 \times 29000}{(kL)^2}$$

$$= \frac{43800 \times 10^4}{(18.2 \times 12)^2} = 9183$$

Assume $C_m = 1.0$
$\alpha = 1.0$

$$B_1 = 1\left(1 - {}^{1.1}/_{9183}\right) = 0.99$$

$$\frac{P_r}{2P_c} + \left(\frac{M_r}{M_c}\right) \le 1.0 \quad ; \; M_c = 840\text{k-ft}$$ Table 3-10

$M_U = 5367/12 \times 1.6 = 715.6$ k-ft

$$\frac{1.1}{2(1370)} + \left(\frac{715.6}{840}\right) = 0.85 \le 1.0$$

Column C-2 is adequate.

COMET 3-0235 — 50 SHEETS — 5 SQUARES 3-0236 — 100 SHEETS — 5 SQUARES 3-0237 — 200 SHEETS — 5 SQUARES 3-0137 — 200 SHEETS — FILLER

Appendix E: Construction Cost Estimation

MF-2004	Description	Quantity Take off	Labor Cost/Unit	Labor Amount	Material Cost/Unit	Material Amount	Other Cost/Unit	Other Amount	Total Cost/Unit	Total Amount
01-00-00	**General Requirements**									
	General Requirements	**159000**							**4.5 /GSF**	**715500**
03-00-00	**Concrete**									
	Formwork Wall Footings	4167.41 /SF	5.17 /SF	21546	1.52 /SF	6334			6.69 /SF	27879.9729
	Formwork Spread Footings	7456.31 /SF	5.99 /SF	44663	1.52 /SF	11334			7.51 /SF	55996.8881
	Formwork Foundaion Wall	45489.46 /SF	5.81 /SF	264294	1.52 /SF	69144			7.33 /SF	333437.7418
	Formwork Exterior Cip Wall	906.69 /SF	9.51 /SF	8623	2.59 /SF	2348			12.1 /SF	10970.949
	Formwork Lightweight Slab	142878.5								
	Cip Floor Edgeform	11303.15	3.93	44421	1.73	19554			5.66	63975.829
	Slab On Grade Edgeform	3123.01	3.93	12273	1.73	5403			5.66	17676.2366
	Control Joint Filler	3646.61	0.69	2723	0.9	3552			1.59	5798.1099
	Strip Footer WaterStop - PVC	1867.51	2.01	3754	8	14940			10.01	18693.7751
	Foundation Wall Waterstop - PVC	454.9	2.01	914	8	3639			10.01	4553.549
	Rebar Wall Footings	21.09	773.12	16305	983.25	20737			1756.37	37041.8433
	Rebar Spread Footings	24.09	773.12	18624	983.25	23686			1756.37	42310.9533
	Rebar Foundation Wall	129.83	773.12	100374	983.25	127655			1756.37	228029.5171
	Rebar Exterior Cip Wall	0.69	773.12	533	983.26	678			1756.38	1211.9022
	Rebar Lightweight Slab									
	Rebar S.O.G	0.08	773.1	62	983.3	79			1756.4	140.512
	Fiber Reinforcement - S.O.G	17699.98			1.18	20886			1.18	20885.9764
	Fiber Reinforcement - Lightweight Slab				1.18	1647			1.18	
	Monolithic Interior Stair	92.5	66.34	6136	45.64	4222			111.98	10358.15
	C.I.P Interior Stairs	390.65	66.34	25916	45.64	17829			111.98	43744.987
	Tiered Floors	940	61.21	57537	45.64	42902			106.85	100439
	Normal Weight Fill @ Metal Floor Deck	58.17 CY	133.5 /CY	7766	92 /CY	5352			225.5	13117.335
	Concrete Fill Metal Pan Stair	5.03	556.06	2797	90	453			646.06	3249.6818
	Sawcut Concrete Control Joint	3946.61	0.79	3118	0.36	1421			1.15	4538.6015
	Concrete Wall Footings	337.43	48.91	16504	90	31369			138.91	46872.4013
	Concrete Spread Footings	926.41	44.81	41512	90	83377			134.81	124889.3321
	Concrete Foundation Wall	1180.25	58.07	68537	90	106223			148.07	174759.6175
	Concrete Exterior Cip Wall	7.93	73.75	585	90	714			163.75	1298.5375
	Concrete S.O.G	740.57	45.31	33555	90	66651			135.31	100206.5267
	Concrete Lightweight Slab	CY								
	Concrete Sealer - Slab on Grade	39466.11	0.19	7499	0.5	19733			0.69	27231.6159
	Concrete Sealer - Lightweight Slab	142878.5								
	Finish Concrete Slab	39466.11	0.59	23285					0.59	23285.0049
	Finish Metal Pan Stair	814.68	0.79	644					0.79	643.5972
	Finish Lightweight Slab	142878.5								
	Cure and Protect Concrete	39466.11	0.48	18944	0.24	9472			0.72	28415.5992
	Cure and Protect Conc to Stairs	814.68	0.91	741	0.3	244			1.21	985.7628
	PC Wall Panel	15656.84	24.51	383749	12.15	190231			36.66	573979.7544
	Precast Concrete Soffit	2238.46	19.63	43941	18.94	42396			38.57	86337.4022
	SFC - 1: Sherwin Williams - Armorseal 1000HS	337	10.73	3616	3	1011			13.73	4627.01
03-00-00	**Concrete**									**2237583.673**
04-00-00	**Masonry**	**159000**	**3.89**	**618168**	**7.18**	**1141364**			**11.07**	**1760130**

05-00-00	Metals									
	Misc Meatal Allowance	159000					0.5	79500	0.5	79500
	WF Steel Column	2.47 TN	710.95 /TN	1756	2173.5 /TN	5369			2884.45 /TN	7125
	HSS Steel Column	13.14 TN	755.46 /TN	9927	2449.63 /TN	32188			3205.09 /TN	42115
	Steel Pipe Column	0.21 LF	26.1 /LF	5	129.86 /LF	27			155.96	33
	Struct Framing	1304.018 TN	621.03 /TN	60302	2173.5 /TN	211047			2794.53	3644119
	Misc. Connections	195.6028 TN	821.6 /TN	11971	2277 /TN	33176			3098.6	606095
	Closure angel At Floor Deck	453.03 LF	5.03 /LF	2279	11.96 /LF	5418			16.99	7697
	Structural Reinforcement at Glazing	20.15 TN	2079.18 /TN	41895	2906.28 /TN	58562			4985.46	100457
	1 1/2"x18ga. Steel Roof Deck	8637.58 SF	0.48 /SF	4146	2.08 /SF	17966			2.56	22112
	3"x18ga. Steel Roof Deck	2671.46 SF	0.48 /SF	1282	2.08 /SF	5557			2.56	6839
	Metal Stud	159000 SF	2.41 /SF	129120	3.8 /SF	203591			6.21	987390
	Support-Toilet Part	29 EA	129.94 /EA	3768	108 /EA	3132			237.94	6900
	Support-Projector	32 LF	21.18 /LF	678	15.75 /LF	504			36.93	1182
	Support-Projector Screen	278 LF	21.18 /LF	5888	15.75 /LF	4379			36.93	10267
	Catwalk Stairs	55 RI	66.41 /RI	3653	148 /RI	8140			214.41	11793
	Metal Pan Stairs	207 RI	155.83 /RI	32257	288 /RI	59616			443.83	91873
	Metal Pan Landings (Incl Conc)	347.78 SF	25.03 /SF	8705	49.75 /SF	17302			74.78	26007
	Wall Rail 1-1/2"	518.67 LF	7.79 /LF	4040	21.56 /LF	11183			29.35	15223
	2 Line HandRail W/ Cip Base at Roof	837 LF	40.3 /LF	33731	111.51 /LF	93334			151.81	127065
	Guardrail - Pipe	573.34 LF	15.58 /LF	8933	43.13 /LF	24728			58.71	33661
	Handrail Extensions	82.64 LF	15.58 /LF	1288	43.13 /LF	3564			58.71	4852
	Guardrail - 2 Pipes	889.61 LF	15.58 /LF	13860	43.13 /LF	38369			58.71	52229
	Handrail - Pipe Planetarium	75.92 LF	15.58 /LF	1183	43.13 /LF	3274			58.71	4457
	2 Line Handrail W/ Cip Base at Wood Pavers	157 LF	40.3 /LF	6327	111.51 /LF	17507			151.81	23834
	Handrail - Pipe	206.02 LF	25.97 /LF	5350	93.44 /LF	19251			119.41	24601
	Steel Grating	240.09 SF	14.32 /SF	3438	14.16 /SF	3400			28.48	6838
	Handrail - Rectangular	13.67 LF	25.97 /LF	355	97.44 /LF	1332			123.41	1687
	Handrail W/ Glass Panels	218.43 LF	30.49 /LF	6660	105.44 /LF	23031			135.93	29691
	Guardrail - Glass Railiing	48.67 LF	31.12 /LF	1515	360 /LF	17521			391.12	19036
05-00-00	Metals									6340051
06-00-00	Wood, Plastics & Composites	159000	1.45 /GSF	230634	3.98 /GSF	632439			5.43 /GSF	863072
07-00-00	Thermal & Moisture Protection	159000	3.04 /GSF	483821	3.65 /GSF	580800			6.7 /GSF	1064621
08-00-00	Openings	159000	3.79 /GSF	602380	13.74 /GSF	2185199			17.53 /GSF	2787579
09-00-00	finishes	159000	13.57 /GSF	2158096	11.6 /GSF	1844787			25.18 /GSF	4002883
10-00-00	Specialties	159000	0.4	63903	1	159342	0.34	53850	1.74	277095
11-00-00	Equipment	159000	3.55	563837	15.94	2534022	1.16	184050	20.64	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	Electrial									3968507
27-00-00	Communications									117659
28-00-00	Electricon Safety & Security									420906
31-00-00	Earthwork									591027
	Total									40468516

Appendix F: Construction Schedule

ID	Task Mode	Task Name	Duration	Start	Finish	2011 2012 2013
1		Post Abatement Notifications	10 days	Mon 1/3/11	Fri 1/14/11	
2		Contract Execution	0 days	Mon 1/3/11	Mon 1/3/11	1/3
3		Abate Steam Tunnel - Chemical Storage	7 days	Mon 1/17/11	Tue 1/25/11	
4		Perform Abatement - Chemical Storage	8 days	Mon 1/17/11	Wed 1/26/11	
5		Abate Steam Tunnel - Green House	7 days	Mon 1/17/11	Tue 1/25/11	
6		Perform Abatement - Green house	10 days	Mon 1/17/11	Fri 1/28/11	
7		Abate/Remove STM UTL and Structures - North	5 days	Thu 1/20/11	Wed 1/26/11	
8		Demo Chemical Storage (Post Abatement)	5 days	Thu 1/20/11	Wed 1/26/11	
9		Demo Green House (Post Abatement)	5 days	Wed 1/26/11	Tue 2/1/11	
10		Abate/Remove STM UTL and Structures - East	5 days	Wed 1/26/11	Tue 2/1/11	
11		Abate Underground electric Utilities - East	2 days	Thu 1/27/11	Fri 1/28/11	
12		Abatement of Green House & Storage Tank	2 days	Thu 1/27/11	Fri 1/28/11	
13		Abate/Remove windows - south elevation	0 days	Mon 1/31/11	Mon 1/31/11	1/31
14		abate/remove stm utl and structures - south	4 days	Mon 1/31/11	Thu 2/3/11	
15		abate underground electric Utilities - South	5 days	Mon 1/31/11	Fri 2/4/11	
16		Backfill/Grade at Green house & chemical	5 days	Wed 2/2/11	Tue 2/8/11	
17		Abate/remove windows - east elevation	5 days	Fri 2/4/11	Thu 2/10/11	
18		demo of green house & storage tank	4 days			
19		foundation underpinning	0 days	Wed 2/9/11	Wed 2/9/11	2/9
20		basement mass excavation	30 days	Wed 2/9/11	Tue 3/22/11	
21		perform asbestos abatement at existing	30 days	Thu 2/10/11	Wed 3/23/11	
22		demo lower level enties and foundations	4 days	Thu 2/10/11	Tue 2/15/11	
23		install temporary protection at lower level	4 days	Thu 2/10/11	Tue 2/15/11	
24		remove exterior wall/brick at south atrium	2 days	Mon 2/14/11	Tue 2/15/11	
25		abatement/demo of Piez Hall	5 days			
26		submission of submittal schedule	0 days			2/25
27		strip footing excavation	0 days	Wed 3/9/11	Wed 3/9/11	3/9
28		FRP column footings	10 days	Wed 3/16/11	Tue 3/29/11	

Project: Proposed Redesign
Date: Wed 4/3/13

Task		Inactive Summary	
Split		Manual Task	
Milestone		Duration-only	
Summary		Manual Summary Rollup	
Project Summary		Manual Summary	
External Tasks		Start-only	
External Milestone		Finish-only	
Inactive Task		Deadline	
Inactive Milestone		Progress	

Page 1

ID	Task Mode	Task Name	Duration	Start	Finish	2011 2012 2013
29		slab stone subbase	12 days	Fri 4/1/11	Mon 4/18/11	
30		underslab plumbing rough - in	13 days	Wed 4/20/11	Fri 5/6/11	
31		underslab electrical rough - in	10 days	Wed 5/4/11	Tue 5/17/11	
32		FRP interior slab on Grade	4 days	Mon 5/9/11	Thu 5/12/11	
33		footings/SOG/Foundations complete	10 days	Mon 5/9/11	Fri 5/20/11	
34		**form suspended slab - pour1**	10 days	Mon 5/23/11	Fri 6/3/11	
35		**form suspended slab - pour2**	10 days			
36		Base Mat Slab rebar - pour 1	0 days	Mon 6/6/11	Mon 6/6/11	6/6
37		MEP Slab rough-in first level - pour 1	6 days	Tue 6/14/11	Tue 6/21/11	
38		**top mat slab rebar - pour 1**	6 days	Tue 6/14/11	Tue 6/21/11	
39		**base mat slab rebar - pour 2**	2 days	Thu 6/16/11	Fri 6/17/11	
40		**form suspended slab - pour 3**	3 days	Tue 6/21/11	Thu 6/23/11	
41		place and finish slab pour 1	2 days	Wed 6/22/11	Thu 6/23/11	
42		mep slab rough- in - first level - pour 2	2 days	Wed 6/22/11	Thu 6/23/11	
43		**frp columns/shear walls - first level - pour 1**	6 days	Thu 6/23/11	Thu 6/30/11	
44		**top mat slab rebar - pour 2**	2 days	Fri 6/24/11	Mon 6/27/11	
45		**form suspended slab pour 4**	3 days	Mon 6/27/11	Wed 6/29/11	
46		**base mat slab rebar - pour 3**	8 days	Wed 6/29/11	Fri 7/8/11	
47		place and finish slab - pour 2	2 days	Thu 6/30/11	Fri 7/1/11	
48		mep slab rough-in - first level pour 3	6 days	Thu 6/30/11	Thu 7/7/11	
49		**frp columns/shear walls - first level - pour 2**	2 days	Fri 7/1/11	Mon 7/4/11	
50		**top mat slab rebar - pour 3**	2 days	Mon 7/4/11	Tue 7/5/11	
51		**form suspended slab - pour 1**	3 days	Tue 7/5/11	Thu 7/7/11	
52		**base mat slab rebar - pour 4**	8 days	Thu 7/7/11	Mon 7/18/11	
53		place and finish slab - pour 3	2 days	Thu 7/7/11	Fri 7/8/11	
54		mep slab rough - in - first level - pour 4	6 days	Fri 7/8/11	Fri 7/15/11	
55		**frp columns/shear walls - first level pour 3**	2 days	Mon 7/11/11	Tue 7/12/11	
56		**top mat slab rebar - pour 4**	2 days	Tue 7/12/11	Wed 7/13/11	

Project: Proposed Redesign
Date: Wed 4/3/13

Task		Inactive Summary	
Split		Manual Task	
Milestone		Duration-only	
Summary		Manual Summary Rollup	
Project Summary		Manual Summary	
External Tasks		Start-only	
External Milestone		Finish-only	
Inactive Task		Deadline	
Inactive Milestone		Progress	

Page 2

ID	Task Mode	Task Name	Duration	Start	Finish
57		form suspended slab - pour 2	3 days	Wed 7/13/11	Fri 7/15/11
58		base mat slab rebar - pour 1	8 days	Fri 7/15/11	Tue 7/26/11
59		place and finish slab - pour 4	2 days	Fri 7/15/11	Mon 7/18/11
60		mep slab rough - in - second lvel - pour 1	6 days	Fri 7/15/11	Fri 7/22/11
61		frp columns/shear walls - first level - pour 4	2 days	Tue 7/19/11	Wed 7/20/11
62		top mat slab rebar - pour 1	2 days	Tue 7/19/11	Wed 7/20/11
63		base mat slab rebar - pour 2	3 days	Thu 7/21/11	Mon 7/25/11
64		form suspended slab - pour 3	8 days	Fri 7/22/11	Tue 8/2/11
65		place and finish slab - pour 1	2 days	Mon 7/25/11	Tue 7/26/11
66		mep slab rough - in - second level - pour 2	2 days	Mon 7/25/11	Tue 7/26/11
67		frp coumns/shear walls - second level - pour 1	6 days	Tue 7/26/11	Tue 8/2/11
68		top mat slab rebar - pour 2	2 days	Wed 7/27/11	Thu 7/28/11
69		form suspended slab - pour 4	3 days	Thu 7/28/11	Mon 8/1/11
70		base mat slab rebar - pour 3	8 days	Mon 8/1/11	Wed 8/10/11
71		place and finish slab - pour 2	2 days	Tue 8/2/11	Wed 8/3/11
72		mep slab rough-in second level - pour 3	6 days	Tue 8/2/11	Tue 8/9/11
73		frp columns/shearwalls - second level pour 2	2 days	Wed 8/3/11	Thu 8/4/11
74		top mat slab rebar - pour 3	2 days	Thu 8/4/11	Fri 8/5/11
75		form supeded slab - pour 1	3 days	Fri 8/5/11	Tue 8/9/11
76		base mat slab rebar - pour 4	8 days	Tue 8/9/11	Thu 8/18/11
77		place and finish slab - pour 3	2 days	Tue 8/9/11	Wed 8/10/11
78		mep slab rough - in - second level - pour 4	6 days	Wed 8/10/11	Wed 8/17/11
79		frp columns/shear walls - second level - pour 3	2 days	Thu 8/11/11	Fri 8/12/11
80		top mat slab rebar - pour 4	2 days	Fri 8/12/11	Mon 8/15/11
81		form suspended slab - pour 4	3 days	Mon 8/15/11	Wed 8/17/11
82		form suspended slab - pour 2	8 days	Wed 8/17/11	Fri 8/26/11
83		place and finish slab - pour 4	2 days	Wed 8/17/11	Thu 8/18/11

Project: Proposed Redesign
Date: Wed 4/3/13

Task		Inactive Summary	
Split		Manual Task	
Milestone		Duration-only	
Summary		Manual Summary Rollup	
Project Summary		Manual Summary	
External Tasks		Start-only	
External Milestone		Finish-only	
Inactive Task		Deadline	
Inactive Milestone		Progress	

Page 3

ID	Task Mode	Task Name	Duration	Start	Finish
84		frp columns/shear walls - second level - pour 4	6 days	Fri 8/19/11	Fri 8/26/11
85		form suspended slab - pour 3	2 days	Tue 8/23/11	Wed 8/24/11
86		form suspended slab - pour 4	8 days	Thu 8/25/11	Mon 9/5/11
87		base mat slab rebar - pour 4	6 days	Fri 9/2/11	Fri 9/9/11
88		mep slab rough-in third level - pour 4	6 days	Mon 9/12/11	Mon 9/19/11
89		top mat slab rebar - pour 4	2 days	Wed 9/14/11	Thu 9/15/11
90		place and finish slab - pour 4	3 days	Mon 9/19/11	Wed 9/21/11
91		cure suspended slab - pour 4	2 days	Wed 9/21/11	Thu 9/22/11
92		reshore slab - third level - pour 4	2 days	Fri 9/23/11	Mon 9/26/11
93		structural steel framing - second level, area A	7 days	Tue 10/4/11	Wed 10/12/11
94		Structural steel framing - exist level 3, area A	2 days	Thu 10/6/11	Fri 10/7/11
95		structural steel framing - third level, area "a"	10 days	Thu 10/20/11	Wed 11/2/11
96		structural steel framing - third level, atrium	10 days	Thu 11/3/11	Wed 11/16/11
97		structural steel framing - third level, area "b"	10 days	Thu 11/17/11	Wed 11/30/11
98		steel decking third level, area "b" existing	10 days	Thu 12/1/11	Wed 12/14/11
99		mep hangers	10 days	Thu 12/15/11	Wed 12/28/11
100		spray fireproofing - third level area B	10 days	Thu 12/29/11	Wed 1/11/12
101		Spray fireproofing - third level area A	10 days	Thu 1/12/12	Wed 1/25/12
102		HVAC Duct above ceiling rough - in	10 days	Thu 1/26/12	Wed 2/8/12
103		sprinkler above ceiling rough - in	10 days	Thu 2/9/12	Wed 2/22/12
104		chemical waste and ven poping above ceiling	15 days	Thu 3/1/12	Wed 3/21/12
105		Plumbing Piping Above Ceiling Rough-In	10 days	Thu 3/15/12	Wed 3/28/12
106		Electrical Above Ceiling Rough In	20 days	Thu 3/29/12	Wed 4/25/12
107		Temp Control Above Ceiling Rough-In	25 days	Thu 4/12/12	Wed 5/16/12
108		Interior Partition Framing	10 days	Thu 5/10/12	Wed 5/23/12
109		Permenant HVAC Systems for Construction Use	0 days	Thu 6/14/12	Thu 6/14/12
110		HVAC Duct In-Wall Rough-In	10 days	Thu 6/21/12	Wed 7/4/12
111		Chemical Waste and Vent Piping in-wall Rough-in	10 days	Thu 6/28/12	Wed 7/11/12

Project: Proposed Redesign
Date: Wed 4/3/13

Task		Inactive Summary	
Split		Manual Task	
Milestone		Duration-only	
Summary		Manual Summary Rollup	
Project Summary		Manual Summary	
External Tasks		Start-only	
External Milestone		Finish-only	
Inactive Task		Deadline	
Inactive Milestone		Progress	

Page 4

ID	Task Mode	Task Name	Duration	Start	Finish
112		Electrical in Wall-Rough-in	10 days	Thu 7/12/12	Wed 7/25/12
113		Temp control in-wall rough-in	18 days	Thu 7/26/12	Mon 8/20/12
114		GYP board Walls	30 days	Thu 8/9/12	Wed 9/19/12
115		GYP board Walls	15 days	Tue 9/4/12	Mon 9/24/12
116		GYP board Walls	7 days	Tue 10/16/12	Wed 10/24/12
117		Tape/finsih Walls	15 days	Tue 11/6/12	Mon 11/26/12
118		Top of wall firestoping	9 days	Thu 11/15/12	Tue 11/27/12
119		GYP board ceiliings	3 days	Thu 12/6/12	Mon 12/10/12
120		Tape/finish ceiling	2 days	Wed 12/19/12	Thu 12/20/12
121		prime/first coat ceilings	6 days	Mon 12/24/12	Mon 12/31/12
122		prime/first coat walls	4 days	Wed 12/26/12	Mon 12/31/12
123		prime/first coat HM frames	6 days	Thu 1/3/13	Thu 1/10/13
124		Lab casework	6 days	Wed 1/9/13	Wed 1/16/13
125		fume hoods	10 days	Thu 1/17/13	Wed 1/30/13
126		fume hood controls	10 days	Fri 1/25/13	Thu 2/7/13
127		fume hood testing	3 days	Fri 2/8/13	Tue 2/12/13
128		controls testing	20 days	Fri 2/22/13	Thu 3/21/13
129		substantial completion/c of o	25 days	Wed 2/27/13	Tue 4/2/13
130		punch list	0 days	Wed 2/27/13	Wed 2/27/13
131		systems training	20 days	Wed 4/3/13	Tue 4/30/13
132		final completion	20 days	Wed 4/3/13	Tue 4/30/13
133		campus ff&e complete	0 days	Wed 5/1/13	Wed 5/1/13
134		project mobilization	0 days		
135		install temporary protection at windows	0 days		
136		testing training and commissioning complete	6 days	Mon 1/31/11	Mon 2/7/11
137		equipment & specialties complete	0 days		
138		FRP columns - lower level	0 days	Wed 4/20/11	Wed 4/20/11
139		FRP interior foundation walls	10 days	Wed 4/20/11	Tue 5/3/11

Project: Proposed Redesign
Date: Wed 4/3/13

Task	Inactive Summary
Split	Manual Task
Milestone	Duration-only
Summary	Manual Summary Rollup
Project Summary	Manual Summary
External Tasks	Start-only
External Milestone	Finish-only
Inactive Task	Deadline
Inactive Milestone	Progress

Page 5

ID	Task Mode	Task Name	Duration	Start	Finish
140		Top track/layout	10 days	Tue 10/4/11	Mon 10/17/11
141		Top track/layout	5 days	Tue 10/11/11	Mon 10/17/11
142		Top track/layout	5 days	Tue 10/18/11	Mon 10/24/11
143		Top track/layout	2 days	Thu 10/20/11	Fri 10/21/11
144		HVAC duct above ceiling rough - in	5 days	Thu 10/27/11	Wed 11/2/11
145		Sprinkler above ceiling rough-in	15 days	Thu 11/17/11	Wed 12/7/11
146		chemical waste and vent piping above ceiling	10 days	Thu 12/1/11	Wed 12/14/11
147		plumbing piping above ceiling rough-In	10 days	Thu 12/15/11	Wed 12/28/11
148		electrical above ceiling rough-in	10 days	Thu 12/29/11	Wed 1/11/12
149		temp control above ceiling rough-in	20 days	Thu 1/26/12	Wed 2/22/12
150		interior partition framing	25 days	Thu 3/1/12	Wed 4/4/12
151		HVAC duct in-wall rough-in	10 days	Thu 3/15/12	Wed 3/28/12
152		chemical waste and vent piping in-wall rough-in	10 days	Thu 3/29/12	Wed 4/11/12
153		plumbing piping in-wall rough-in	10 days	Thu 4/12/12	Wed 4/25/12
154		temp control in-wall rough-in	10 days	Thu 4/26/12	Wed 5/9/12
155		electrical in-wall rough-in	20 days	Thu 4/26/12	Wed 5/23/12
156		GYP board walls	18 days	Thu 5/24/12	Mon 6/18/12
157		GYP board walls	7 days	Thu 6/14/12	Fri 6/22/12
158		Tape/finish walls	6 days	Mon 6/25/12	Mon 7/2/12
159		top of wall firestoping	2 days	Tue 7/3/12	Wed 7/4/12
160		GYP board ceilings	1 day	Thu 7/5/12	Thu 7/5/12
161		tape/finish ceilings	3 days	Fri 7/6/12	Tue 7/10/12
162		prime/first coat ceilings	2 days	Wed 7/11/12	Thu 7/12/12
163		prime/first coat walls	4 days	Fri 7/13/12	Wed 7/18/12
164		accoustical ceiling grid	6 days	Thu 7/19/12	Thu 7/26/12
165		sprinkler heads	10 days	Fri 7/27/12	Thu 8/9/12
166		install lighting fixtures	10 days	Fri 8/10/12	Thu 8/23/12
167		lighting controls - elec RM 1057	10 days	Fri 8/24/12	Thu 9/6/12

Project: Proposed Redesign
Date: Wed 4/3/13

Task	Inactive Summary
Split	Manual Task
Milestone	Duration-only
Summary	Manual Summary Rollup
Project Summary	Manual Summary
External Tasks	Start-only
External Milestone	Finish-only
Inactive Task	Deadline
Inactive Milestone	Progress

Page 6

ID	Task Mode	Task Name	Duration	Start	Finish
168		install lighting controls	7 days	Fri 8/24/12	Mon 9/3/12
169		finish paint walls	3 days	Fri 9/7/12	Tue 9/11/12
170		water coolers	5 days	Wed 9/12/12	Tue 9/18/12
171		water coolers	10 days	Wed 9/19/12	Tue 10/2/12
172		water coolers	10 days	Wed 10/3/12	Tue 10/16/12
173		plumbing fixture carriers - toilet 3083	10 days	Wed 10/17/12	Tue 10/30/12
174		submit/review environmental & abatement	15 days	Tue 12/28/10	Mon 1/17/11
175		subit/review concrete submittals	15 days	Mon 1/3/11	Fri 1/21/11
176		submission of critical submittals	0 days		
177		GYP board soffits	5 days	Mon 6/25/12	Fri 6/29/12
178		tape/finish soffits	6 days	Mon 7/2/12	Mon 7/9/12
179		building mass excavation complete	0 days		
180		install R-2 roofing system - main roof	25 days	Thu 3/1/12	Wed 4/4/12
181		install skylights - main roof	15 days	Thu 4/5/12	Wed 4/25/12
182		building enclosure/watertight	0 days		
183		accoustical ceiling grid	8 days	Thu 1/17/13	Mon 1/28/13
184		sprinkler heads	10 days	Sun 1/29/12	Thu 2/9/12
185		install lighting fixtures	20 days	Tue 2/12/13	Mon 3/11/13
186		install lighting controls	10 days	Tue 3/12/13	Mon 3/25/13
187		GYP board soffits	5 days	Thu 12/6/12	Wed 12/12/12
188		acid neutralization pits	2 days	Mon 5/23/11	Tue 5/24/11
189		sewage ejector pit	2 days	Mon 5/23/11	Tue 5/24/11
190		sump pit	2 days	Mon 5/23/11	Tue 5/24/11
191		base mat slab rebar - pour 1	2 days	Wed 8/17/11	Thu 8/18/11
192		mep slab rough-in third level - pour 1	3 days	Fri 8/19/11	Tue 8/23/11
193		top mat slab rebar - pour 1	2 days	Wed 8/24/11	Thu 8/25/11
194		base mat slab rebar - pour 2	2 days	Thu 8/25/11	Fri 8/26/11
195		place and finish slab - pour 1	2 days	Sun 8/28/11	Mon 8/29/11

Project: Proposed Redesign
Date: Wed 4/3/13

Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | Deadline | Progress

Page 7

ID	Task Mode	Task Name	Duration	Start	Finish
196		mep slab rough-in third level - pour 2	3 days	Mon 8/29/11	Wed 8/31/11
197		frp column/shear walls - third level - pour 1	8 days	Tue 8/30/11	Thu 9/8/11
198		top mat slab rebar - pour 2	2 days	Thu 9/1/11	Fri 9/2/11
199		base mat slab rebar - pour 3	2 days	Fri 9/2/11	Mon 9/5/11
200		place and finish slab - pour 2	2 days	Mon 9/5/11	Tue 9/6/11
201		mep slab rough-in - third level pour 3	3 days	Tue 9/6/11	Thu 9/8/11
202		frp columns/shear walls - third level - pour 2	8 days	Wed 9/7/11	Fri 9/16/11
203		top mat slab rebar - pour 3	2 days	Fri 9/9/11	Mon 9/12/11
204		form suspended slab - pour 1	6 days	Fri 9/9/11	Fri 9/16/11
205		place and finish slab - pour 3	2 days	Tue 9/13/11	Wed 9/14/11
206		frp solumns/shear walls - third level - pour 3	8 days	Thu 9/15/11	Mon 9/26/11
207		form suspended slab - pour 2	6 days	Mon 9/19/11	Mon 9/26/11
208		frp column/shear walls - third level - pour 4	8 days	Fri 9/23/11	Tue 10/4/11
209		form suspended slab - pour 3	6 days	Tue 9/27/11	Tue 10/4/11
210		form suspended slab - pour 4	6 days	Wed 10/5/11	Wed 10/12/11
211		base mat slab rebar - pour 4	2 days	Thu 10/13/11	Fri 10/14/11
212		mep slab rough-in fourth level - pour 4	3 days	Mon 10/17/11	Wed 10/19/11
213		top mat slab rebar - pour 4	2 days	Thu 10/20/11	Fri 10/21/11
214		place and finish slab -pour 4	2 days	Mon 10/24/11	Tue 10/25/11
215		cure suspended slab - pour 4	7 days	Wed 10/26/11	Thu 11/3/11
216		reshore slab - fourth level - pour 4	2 days	Fri 11/4/11	Mon 11/7/11
217		architctural precast - north elevation	5 days	Tue 11/8/11	Mon 11/14/11
218		architectural precast - east elevation	10 days	Tue 11/15/11	Mon 11/28/11
219		spray insulation - east elevation	5 days	Tue 11/29/11	Mon 12/5/11
220		exterior cold metal framing - east elevation	20 days	Thu 10/6/11	Wed 11/2/11
221		exterior cold metal framing - south elevation	30 days	Tue 1/3/12	Mon 2/13/12
222		exterior cold metal framing - west elevation	20 days	Tue 2/14/12	Mon 3/12/12
223		spray insulation masonry - west elevation	5 days	Tue 3/13/12	Mon 3/19/12

Project: Proposed Redesign
Date: Wed 4/3/13

Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | Deadline | Progress

Page 8

ID		Task Mode	Task Name	Duration	Start	Finish
224			terra cotta - west elevation	25 days	Tue 3/20/12	Mon 4/23/12
225			metal pan ceiling grid	6 days	Thu 1/17/13	Thu 1/24/13
226			telecomm in-wall rough-in	15 days	Thu 4/26/12	Wed 5/16/12
227			metal pan ceiling grid	1 day	Thu 7/19/12	Thu 7/19/12
228			tape/finish soffits	6 days	Thu 12/13/12	Thu 12/20/12
229			excavation protection	25 days	Wed 2/23/11	Tue 3/29/11
230			base mat slab rebar - pour 1	2 days	Mon 9/19/11	Tue 9/20/11
231			mep slab rough-in fourth level - pour 1	3 days	Wed 9/21/11	Fri 9/23/11
232			top mat slab rebar - pour 1	2 days	Mon 9/26/11	Tue 9/27/11
233			place and finish slab - pour 1	2 days	Wed 9/28/11	Thu 9/29/11
234			frp columns/shear walls - fourth level - pour 1	8 days	Fri 9/30/11	Tue 10/11/11
235			form suspended slab - pour 1	8 days	Wed 10/12/11	Fri 10/21/11
236			form suspended slab - pour 2	8 days	Mon 10/24/11	Wed 11/2/11
237			form suspended slab - pour 3	8 days	Thu 11/3/11	Mon 11/14/11
238			form suspended slab - pour 4	8 days	Tue 11/15/11	Thu 11/24/11
239			base mat slab rebar - pour 4	2 days	Fri 11/25/11	Mon 11/28/11
240			top mat slab rebar - pour 4	2 days	Tue 11/29/11	Wed 11/30/11
241			steel decking - third level, atrium	10 days	Thu 12/1/11	Wed 12/14/11
242			place and finish slab - pour 4	2 days	Thu 12/1/11	Fri 12/2/11
243			cure suspended slab - pour 4	7 days	Mon 12/5/11	Tue 12/13/11
244			reshore slab - roof level - pour 4	1 day	Wed 12/14/11	Wed 12/14/11
245			structural steel framing - truss	10 days	Thu 12/15/11	Wed 12/28/11
246			telecom & security above ceiling rough-in	15 days	Thu 5/10/12	Wed 5/30/12
247			fire alarm above ceiling rough-in	15 days	Thu 5/10/12	Wed 5/30/12
248			GYP board walls	7 days	Thu 12/6/12	Fri 12/14/12
249			tape/finish walls	6 days	Mon 12/17/12	Mon 12/24/12
250			top of wall firestoping	2 days	Tue 12/25/12	Wed 12/26/12
251			GYP board ceilings	1 day	Thu 12/27/12	Thu 12/27/12

Project: Proposed Redesign
Date: Wed 4/3/13

Task
Split
Milestone
Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress

Page 9

ID		Task Mode	Task Name	Duration	Start	Finish
252			tape/finish ceilings	3 days	Fri 12/28/12	Tue 1/1/13
253			prime/first coat ceilings	2 days	Wed 1/2/13	Thu 1/3/13
254			prime/fire coat walls	4 days	Fri 1/4/13	Wed 1/9/13
255			prime/first coat HM frames	4 days	Thu 1/10/13	Tue 1/15/13
256			lab casework	10 days	Wed 1/16/13	Tue 1/29/13
257			fume hoods	5 days	Wed 1/30/13	Tue 2/5/13
258			fume hood controls	2 days	Wed 2/6/13	Thu 2/7/13
259			tape/finish walls	9 days	Tue 11/6/12	Fri 11/16/12
260			top of wall firestoping	3 days	Mon 11/19/12	Wed 11/21/12
261			GYP board ceilings	2 days	Thu 11/22/12	Fri 11/23/12
262			tape/finish ceilings	6 days	Mon 11/26/12	Mon 12/3/12
263			prime/first coat ceilings	4 days	Tue 12/4/12	Fri 12/7/12
264			prime/first coat walls	6 days	Mon 12/10/12	Mon 12/17/12
265			GYP board soffits	5 days	Mon 12/17/12	Fri 12/21/12
266			accoustical ceiling grid	8 days	Tue 12/18/12	Thu 12/27/12
267			tape/finish soffits	6 days	Mon 12/24/12	Mon 12/31/12
268			sprinkler heads	10 days	Fri 12/28/12	Thu 1/10/13
269			install lighting fixtures	20 days	Fri 1/11/13	Thu 2/7/13
270			install lighting controls	10 days	Fri 2/8/13	Thu 2/21/13
271			install lighting controls	7 days	Fri 2/22/13	Mon 3/4/13
272			cure suspended slab - pour 1	7 days	Mon 6/27/11	Tue 7/5/11
273			cure suspended slab - pour 2	7 days	Tue 7/5/11	Wed 7/13/11
274			reshore slab - first level - pour 1	2 days	Wed 7/6/11	Thu 7/7/11
275			cure suspended slab - pour 3	7 days	Wed 7/13/11	Thu 7/21/11
276			reshore slab - first level - pour 2	2 days	Thu 7/14/11	Fri 7/15/11
277			cure suspended slab - pour 4	7 days	Thu 7/21/11	Fri 7/29/11
278			reshore slab - first level - pour 3	2 days	Fri 7/22/11	Mon 7/25/11
279			reshore slab - first level - pour 4	2 days	Mon 8/1/11	Tue 8/2/11

Project: Proposed Redesign
Date: Wed 4/3/13

Task
Split
Milestone
Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress

Page 10

ID		Task Mode	Task Name	Duration	Start	Finish
280			**base mat slab rebar - pour 2**	2 days	Tue 9/27/11	Wed 9/28/11
281			mep slab rough-in fourth-level - pour 2	3 days	Thu 9/29/11	Mon 10/3/11
282			**top mat slab rebar - pour 2**	2 days	Tue 10/4/11	Wed 10/5/11
283			place and finish slab - pour 2	2 days	Thu 10/6/11	Fri 10/7/11
284			**frp columns/shear walls - fourth level - pour 2**	8 days	Mon 10/10/11	Wed 10/19/11
285			GYP board walls	15 days	Mon 6/25/12	Fri 7/13/12
286			GYP board walls	7 days	Mon 7/16/12	Tue 7/24/12
287			tape/finish walls	6 days	Wed 7/25/12	Wed 8/1/12
288			top of wall firestoping	2 days	Thu 8/2/12	Fri 8/3/12
289			GYP board ceilings	1 day	Mon 8/6/12	Mon 8/6/12
290			tape/finish ceilings	3 days	Tue 8/7/12	Thu 8/9/12
291			prime/first coat ceilings	2 days	Fri 8/10/12	Mon 8/13/12
292			prime/first coat walls	4 days	Tue 8/14/12	Fri 8/17/12
293			prime/first coat HM frames	4 days	Mon 8/20/12	Thu 8/23/12
294			Lab casework	10 days	Fri 8/24/12	Thu 9/6/12
295			lab plumbing fixtures	10 days	Fri 9/7/12	Thu 9/20/12
296			lab plumbing fixtures	10 days	Fri 9/21/12	Thu 10/4/12
297			plumbing fixtures carriers - toilet 3081	10 days	Fri 10/5/12	Thu 10/18/12
298			resillient flooring	3 days	Fri 2/8/13	Tue 2/12/13
299			Carpeting	3 days	Wed 2/13/13	Fri 2/15/13
300			metal pan ceilings tiles	3 days	Tue 3/12/13	Thu 3/14/13
301			air balancing	1 day	Fri 3/15/13	Fri 3/15/13
302			GYP board soffits	5 days	Wed 7/25/12	Tue 7/31/12
303			tape/finish soffits	6 days	Wed 8/1/12	Wed 8/8/12
304			metal pan ceiling grid	6 days	Tue 12/18/12	Tue 12/25/12
305			accoustical ceiling tiles	3 days	Tue 3/12/13	Thu 3/14/13
306			frp strip footings	24 days	Wed 3/16/11	Mon 4/18/11
307			**base mat slab rebar - pour 3**	2 days	Wed 10/5/11	Thu 10/6/11

Project: Proposed Redesign
Date: Wed 4/3/13

Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | Deadline | Progress

Page 11

ID		Task Mode	Task Name	Duration	Start	Finish
308			mep slab rough-in - fourth level - pour 3	3 days	Fri 10/7/11	Tue 10/11/11
309			**top mat slab rebar - pour 3**	2 days	Wed 10/12/11	Thu 10/13/11
310			place and finish slab - pour 3	2 days	Fri 10/14/11	Mon 10/17/11
311			**frp columns /shear walls - fourth level - pour 3**	8 days	Tue 10/18/11	Thu 10/27/11
312			telecom & security above ceiling rough-in	15 days	Thu 1/26/12	Wed 2/15/12
313			fire alarm above ceiling rough-in	15 days	Thu 1/26/12	Wed 2/15/12
314			plumbing fixture carriers - toilet 1016	10 days	Thu 3/15/12	Wed 3/28/12
315			plumbing fixture carriers - toilet 1015	10 days	Thu 3/15/12	Wed 3/28/12
316			fire alarm in-wall rough-in	10 days	Thu 4/26/12	Wed 5/9/12
317			prime/first coat HM frames	4 days	Thu 7/19/12	Tue 7/24/12
318			lab casework	10 days	Wed 7/25/12	Tue 8/7/12
319			lab plumbing fixtures	10 days	Wed 8/8/12	Tue 8/21/12
320			lab plumbing fixtures	10 days	Wed 8/22/12	Tue 9/4/12
321			tape/finish walls	9 days	Mon 7/16/12	Thu 7/26/12
322			top of wall firestoping	3 days	Fri 7/27/12	Tue 7/31/12
323			GYP board ceilings	2 days	Wed 8/1/12	Thu 8/2/12
324			tape/finish walls	6 days	Fri 8/3/12	Fri 8/10/12
325			prime/first coat ceilings	4 days	Mon 8/13/12	Thu 8/16/12
326			prime/first coat walls	6 days	Fri 8/17/12	Fri 8/24/12
327			prime/first coat HM frames	6 days	Mon 8/27/12	Mon 9/3/12
328			lab casework	10 days	Tue 9/4/12	Mon 9/17/12
329			telecomm in-wall rough-in	15 days	Tue 9/4/12	Mon 9/24/12
330			lab plumbing fixtures	10 days	Wed 1/30/13	Tue 2/12/13
331			lab plumbing fixtures	10 days	Wed 2/13/13	Tue 2/26/13
332			lab gas testing	10 days	Wed 2/27/13	Tue 3/12/13
333			**frp columns/shear walls - fourth level - pour 4**	8 days	Wed 10/26/11	Fri 11/4/11
334			spray insulation precast - north elevation	2 days	Tue 11/15/11	Wed 11/16/11
335			exterior cold metal framing - north elevation	5 days	Thu 11/17/11	Wed 11/23/11

Project: Proposed Redesign
Date: Wed 4/3/13

Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | Deadline | Progress

Page 12

ID		Task Mode	Task Name	Duration	Start	Finish
336			accoustical ceiling grid	6 days	Thu 1/10/13	Thu 1/17/13
337			sprinkler heads	10 days	Fri 1/18/13	Thu 1/31/13
338			install lighting fixtures	10 days	Fri 2/1/13	Thu 2/14/13
339			tape/finish walls	9 days	Thu 6/14/12	Tue 6/26/12
340			top of wall firestoping	3 days	Wed 6/27/12	Fri 6/29/12
341			GYP board ceilings	2 days	Mon 7/2/12	Tue 7/3/12
342			tape/finish ceilings	6 days	Wed 7/4/12	Wed 7/11/12
343			prime/first coat ceilings	4 days	Wed 7/18/12	Mon 7/23/12
344			prime/first coat walls	6 days	Thu 7/26/12	Thu 8/2/12
345			prime/first coat HM frames	6 days	Fri 8/3/12	Fri 8/10/12
346			Lab casework	10 days	Tue 11/29/11	Mon 12/12/11
347			architectural precast - south elevation	5 days	Tue 12/6/11	Mon 12/12/11
348			spray insulation - south elevation	10 days	Mon 8/27/12	Fri 9/7/12
349			architectural woodwork	10 days	Mon 9/10/12	Fri 9/21/12
350			finish paint walls	3 days	Tue 11/6/12	Thu 11/8/12
351			GYP board soffits	5 days	Tue 12/18/12	Mon 12/24/12
352			prime/first coat HM frames	6 days	Wed 12/26/12	Wed 1/2/13
353			lab casework	10 days	Wed 1/9/13	Tue 1/22/13
354			fume hoods	10 days	Wed 1/23/13	Tue 2/5/13
355			fume hood controls	2 days	Wed 3/23/11	Thu 3/24/11
356			**frp shear walls - lower level**	10 days	Wed 3/23/11	Tue 4/5/11

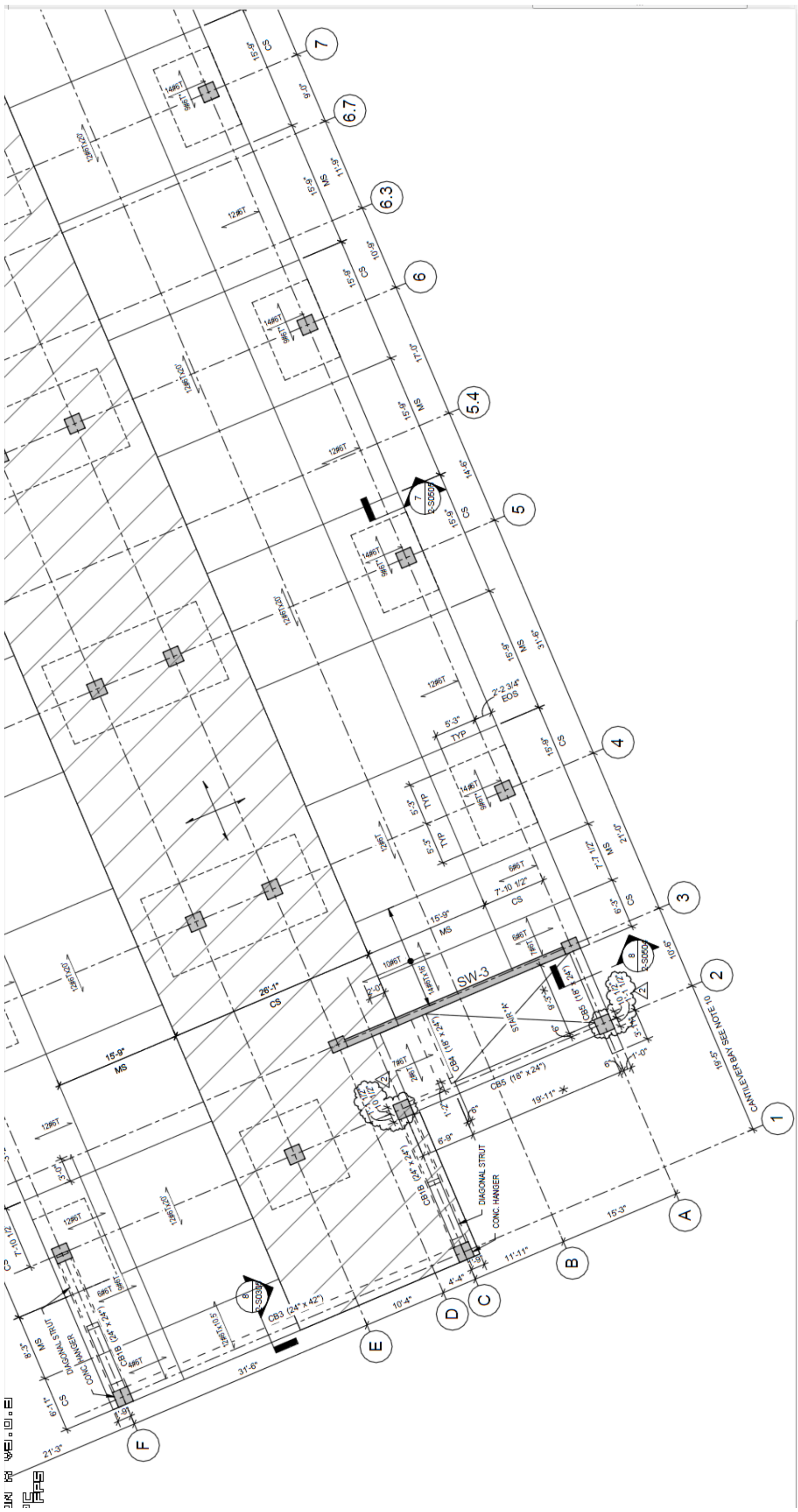
Project: Proposed Redesign
Date: Wed 4/3/13

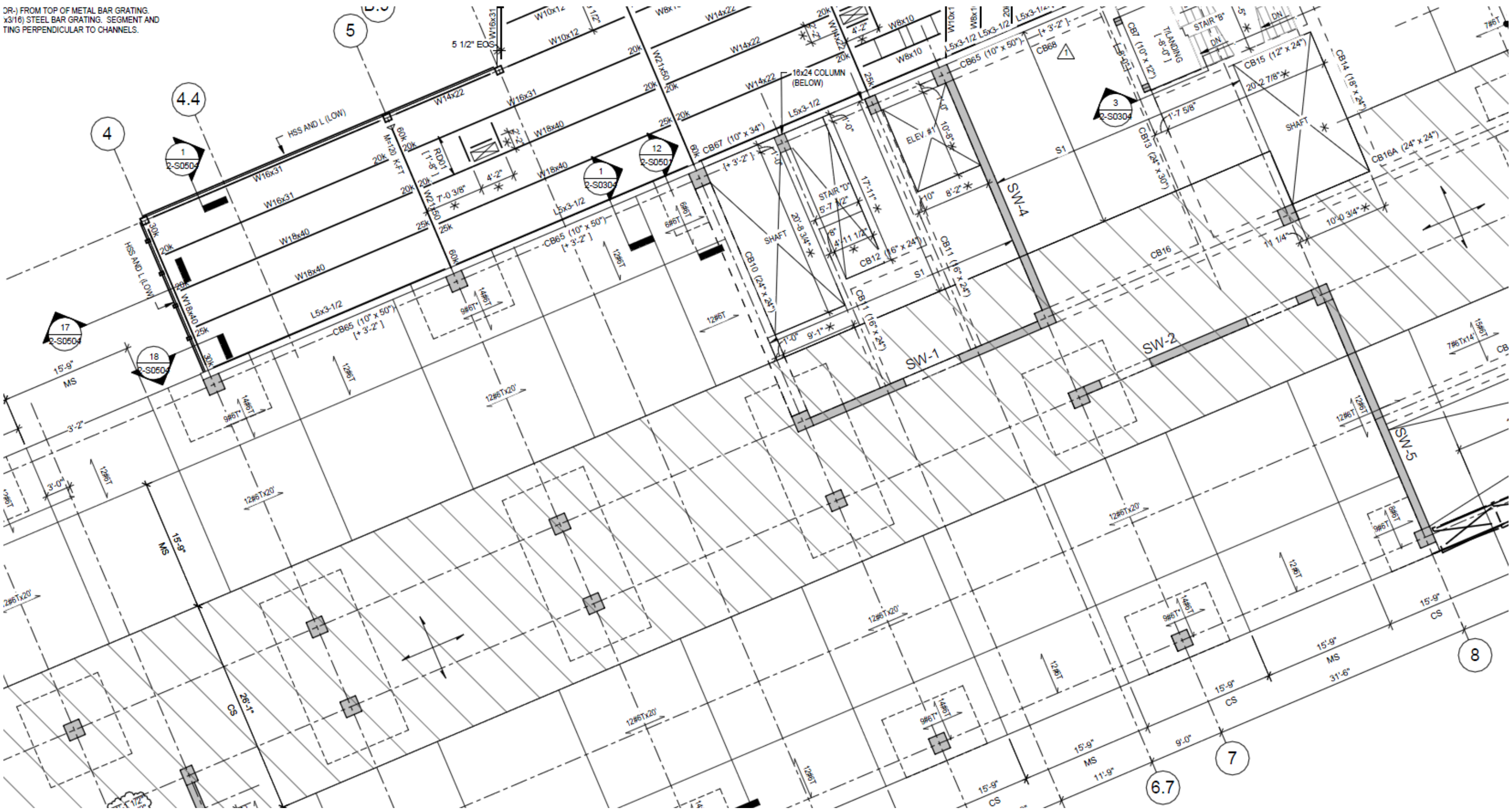
Task		Inactive Summary	
Split		Manual Task	
Milestone		Duration-only	
Summary		Manual Summary Rollup	
Project Summary		Manual Summary	
External Tasks		Start-only	
External Milestone		Finish-only	
Inactive Task		Deadline	
Inactive Milestone		Progress	

Page 13

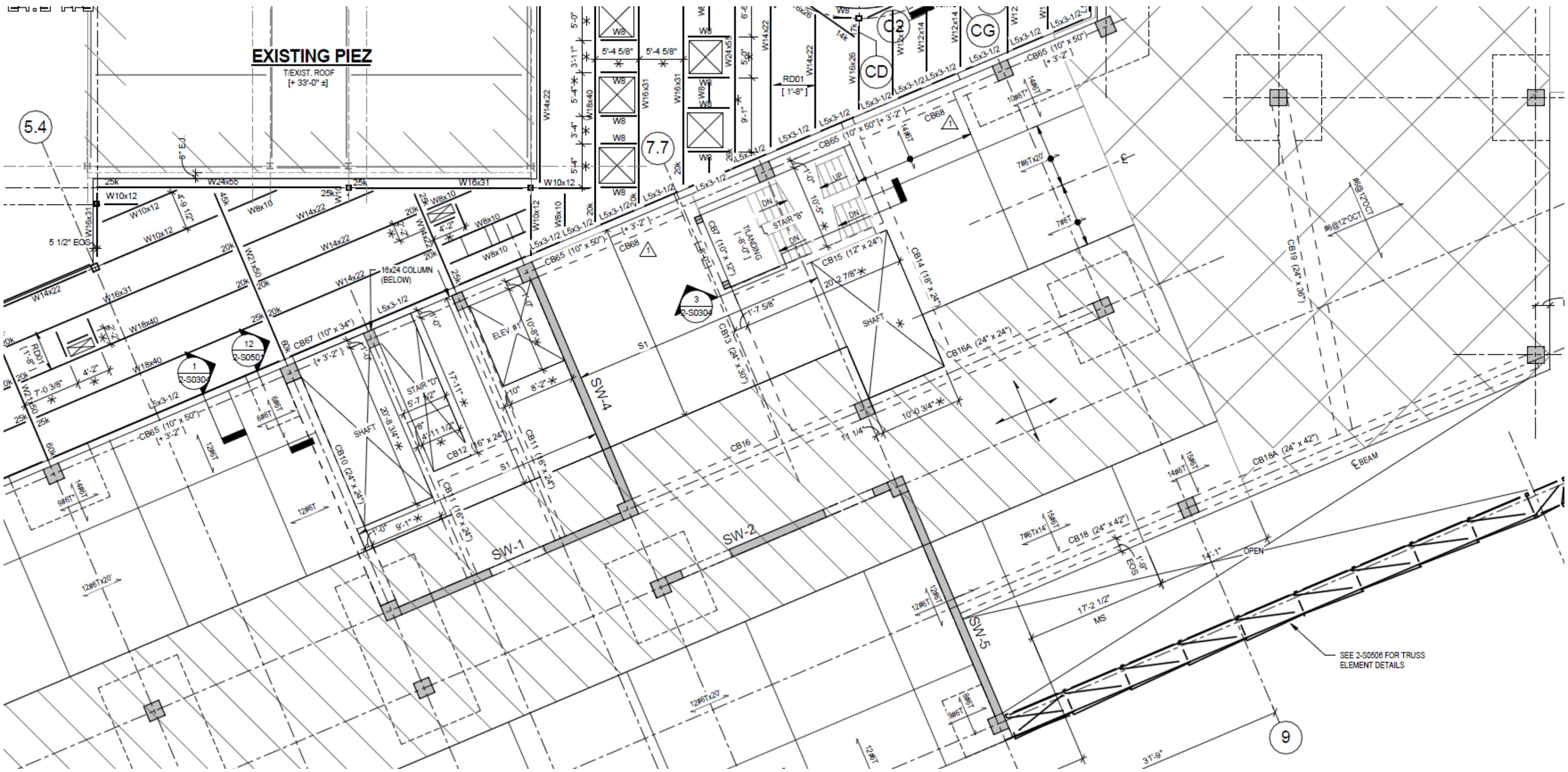
Appendix G: Typical Floor Plans

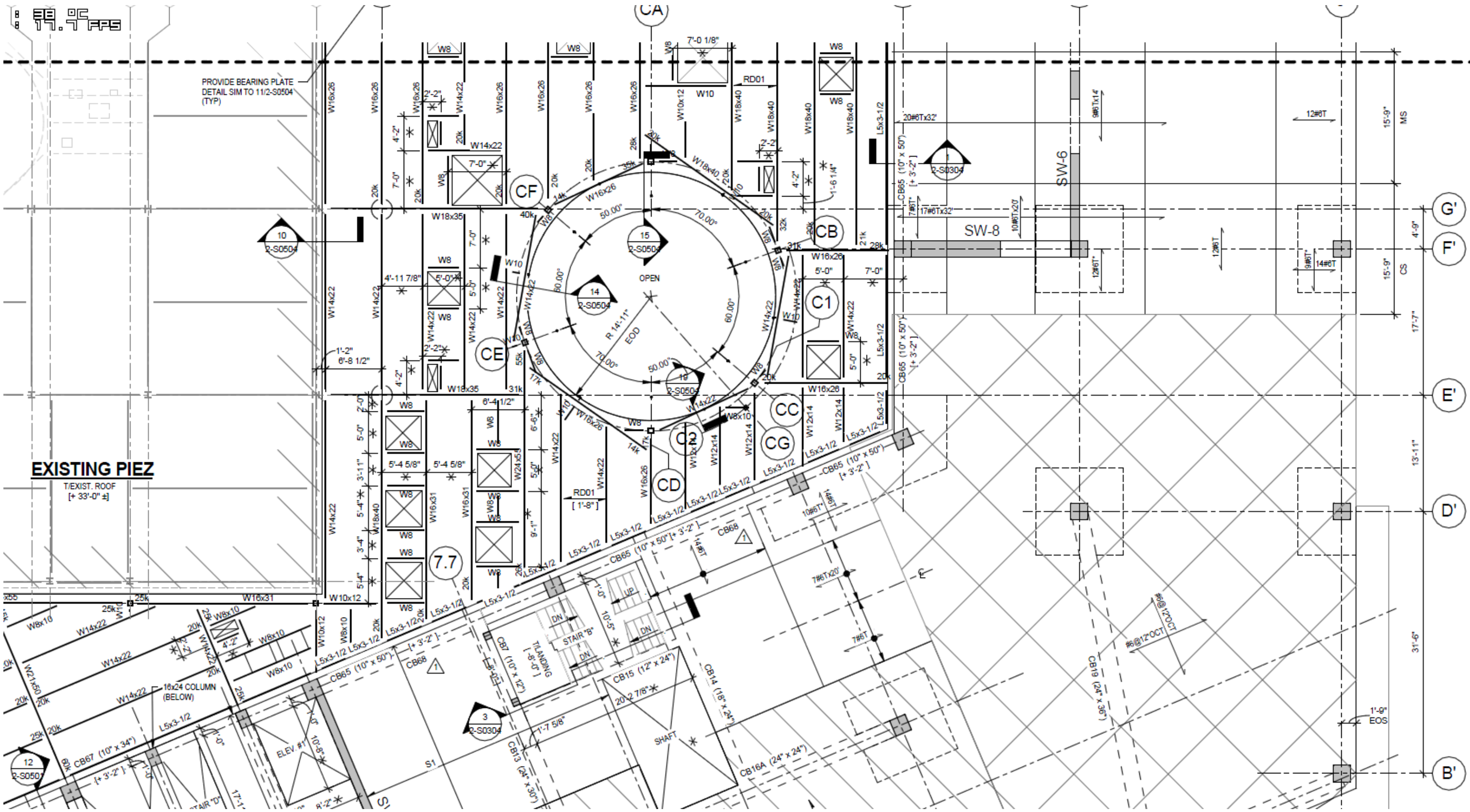
← N

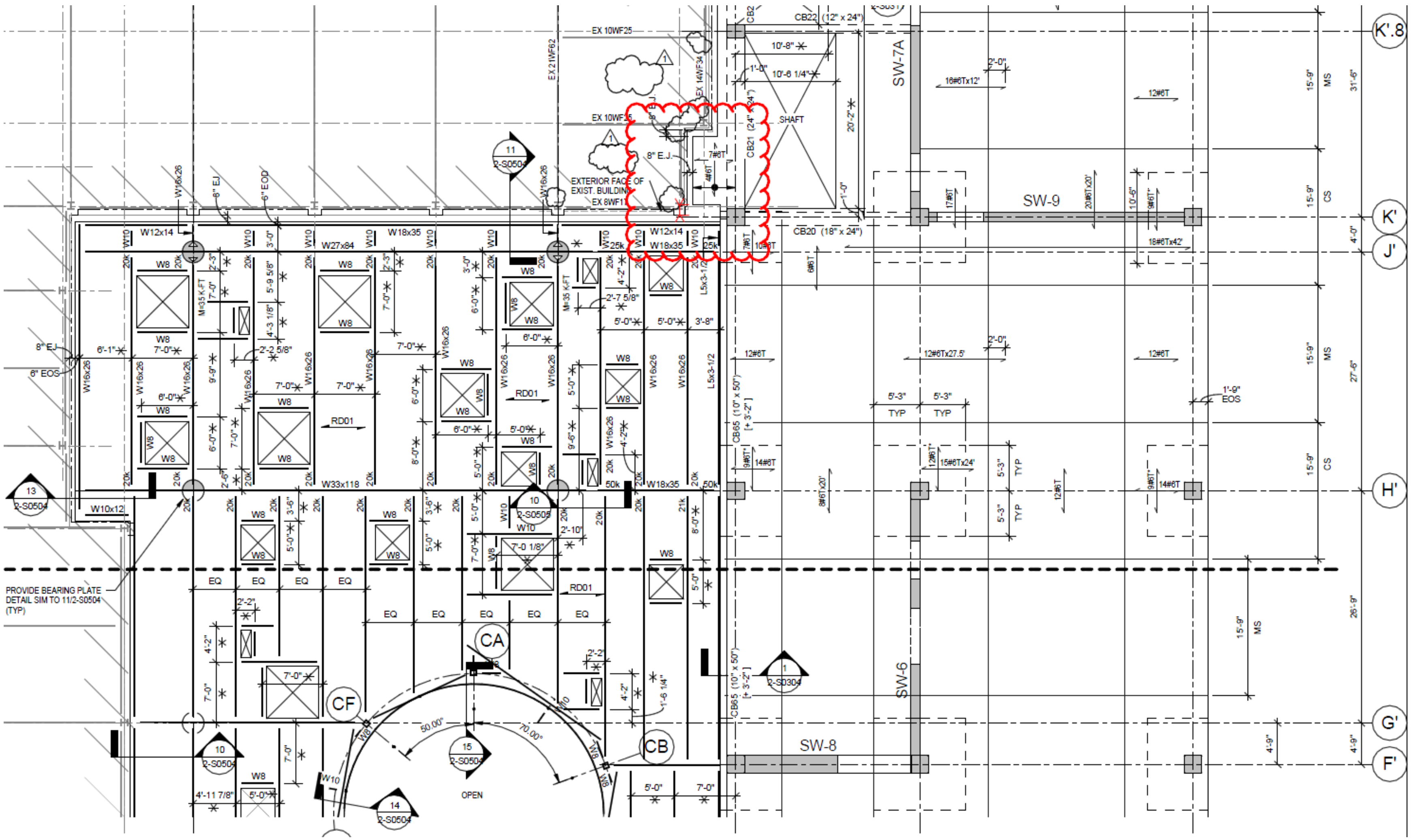


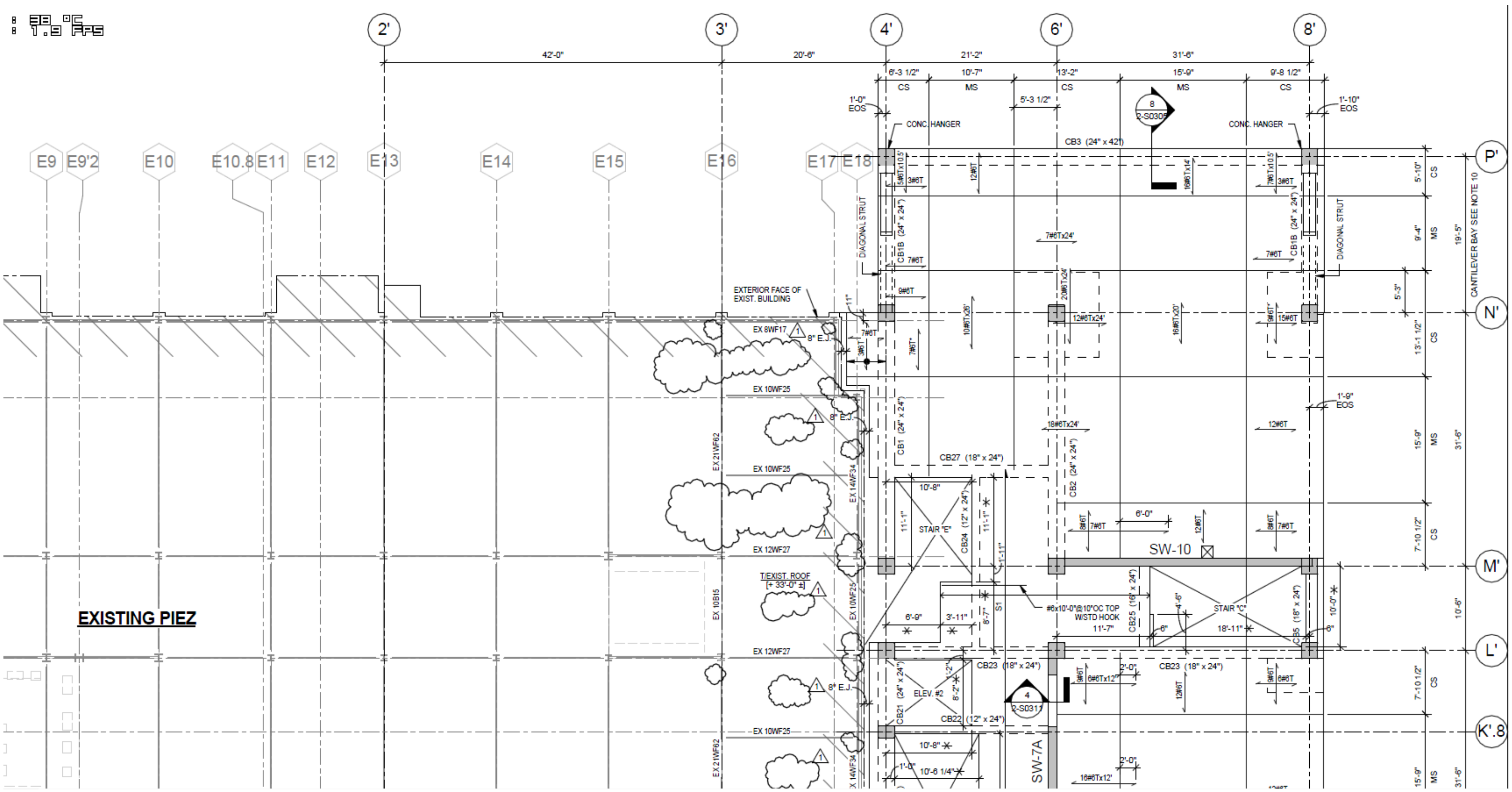


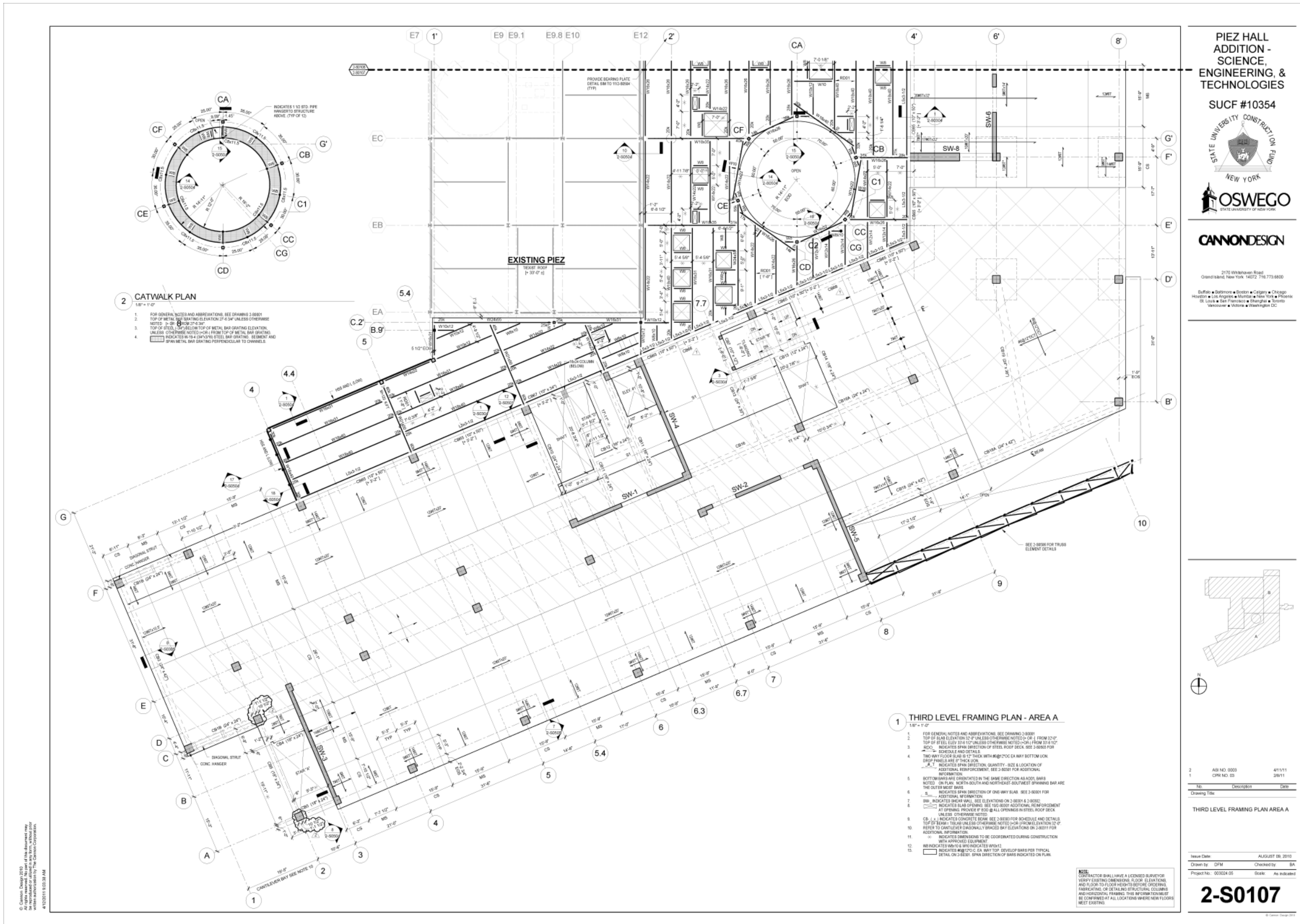
OR-) FROM TOP OF METAL BAR GRATING.
x3/16) STEEL BAR GRATING. SEGMENT AND
TING PERPENDICULAR TO CHANNELS.
SW-1
SW-2
SW-4
SW-5
SHAFT
STAIR "D"
ELEV #1
16x24 COLUMN (BELOW)
HSS AND L (LOW)
CB65 (10" x 50")
CB67 (10" x 34")
CB10 (24" x 24")
CB11 (16" x 24")
CB12 (16" x 24")
CB13 (24" x 30")
CB14 (18" x 24")
CB15 (12" x 24")
CB16A (24" x 24")
CB7 (10" x 12")
4
4.4
5
6.7
7
8

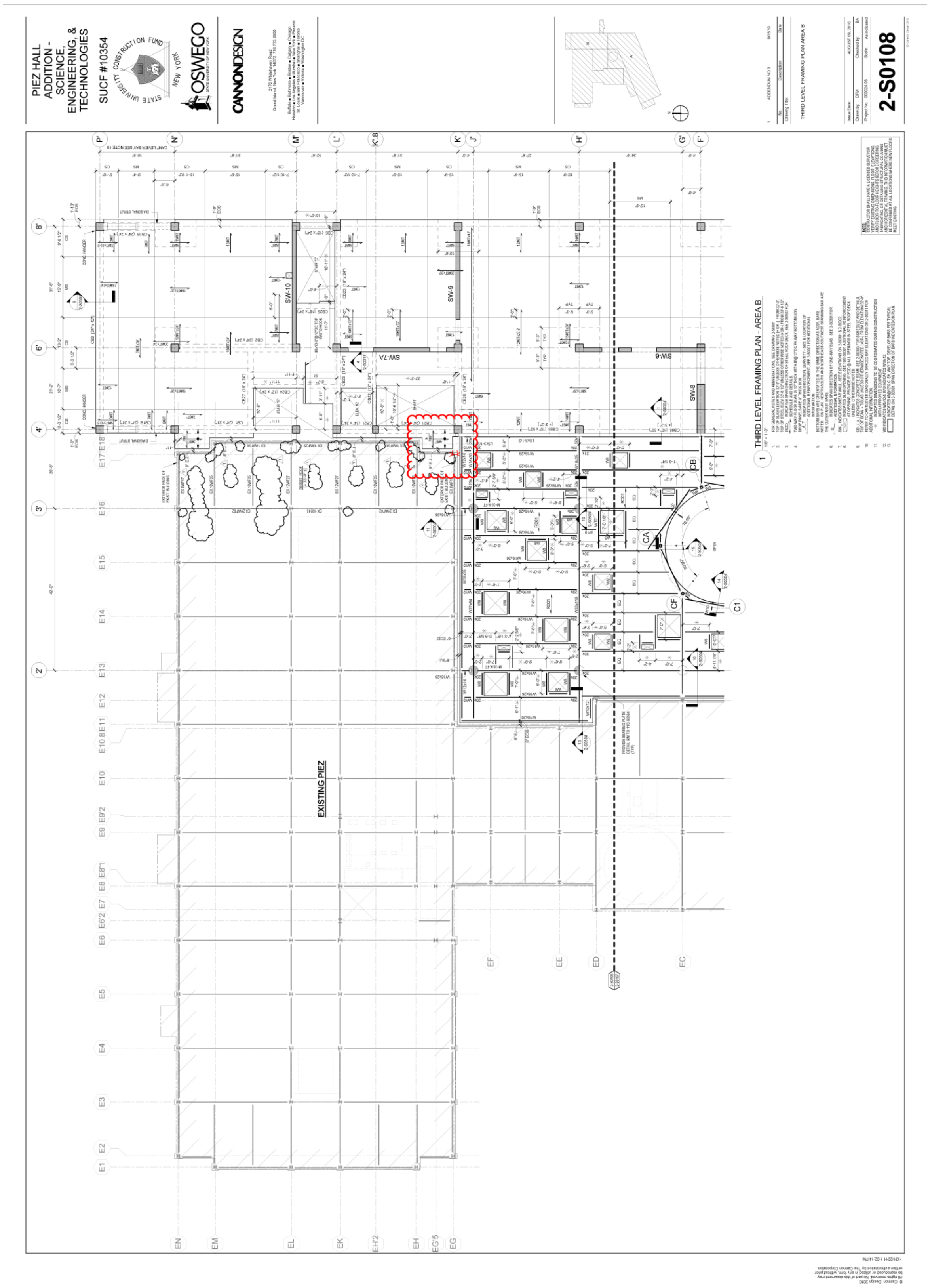

EXISTING PIEZ
T/EXIST. ROOF
[+ 33'-0" ±]
SW-1
SW-2
SW-4
SW-5
STAIR "B"
STAIR "D"
SHAFT
ELEV. #1
SEE 2-S0506 FOR TRUSS
ELEMENT DETAILS
5.4
7.7
9
CD
CG

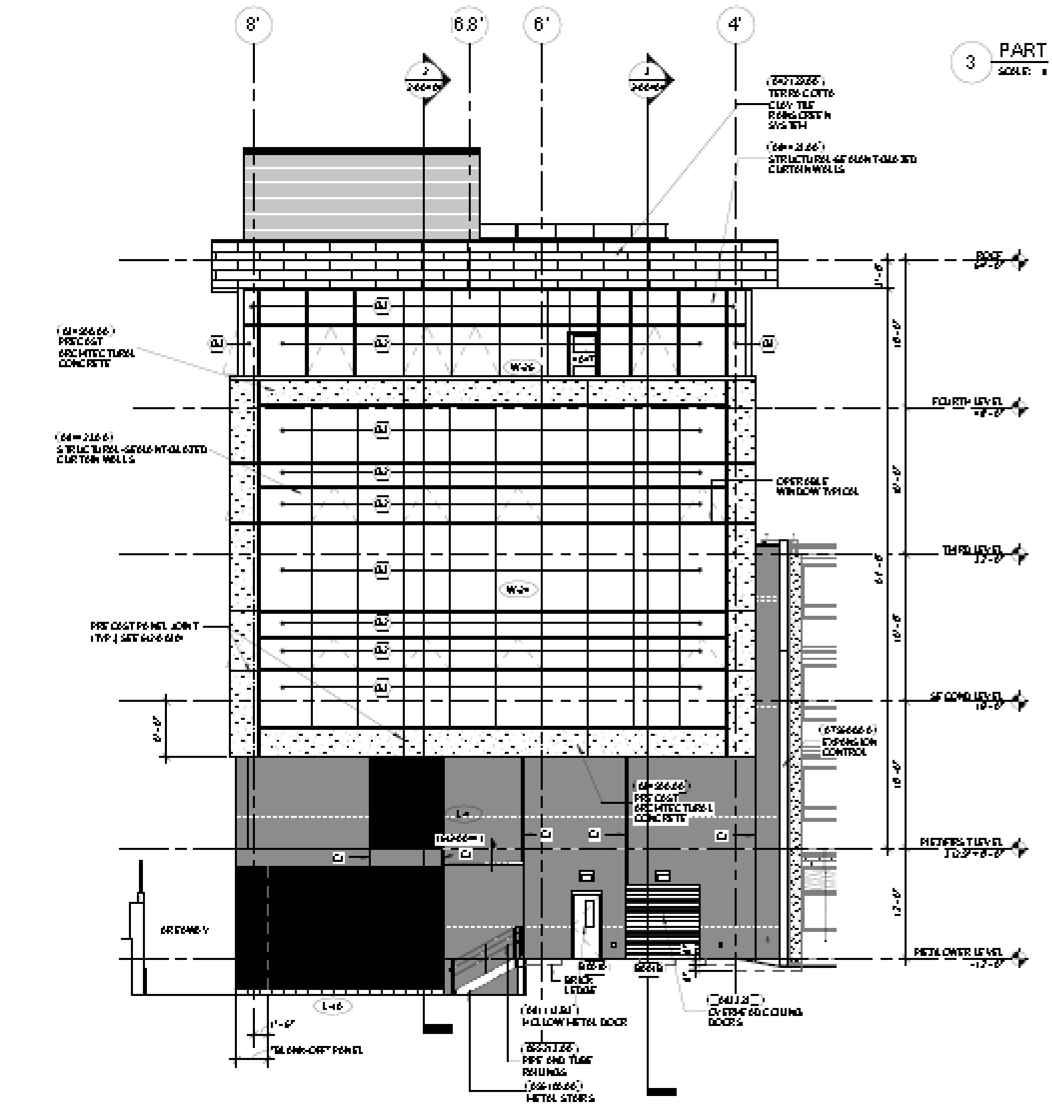

EXISTING PIEZ
T/EXIST. ROOF
[+ 33'-0" ±]
PROVIDE BEARING PLATE DETAIL SIM TO 11/2-S0504 (TYP)
SW-6
SW-8
OPEN
16x24 COLUMN (BELOW)



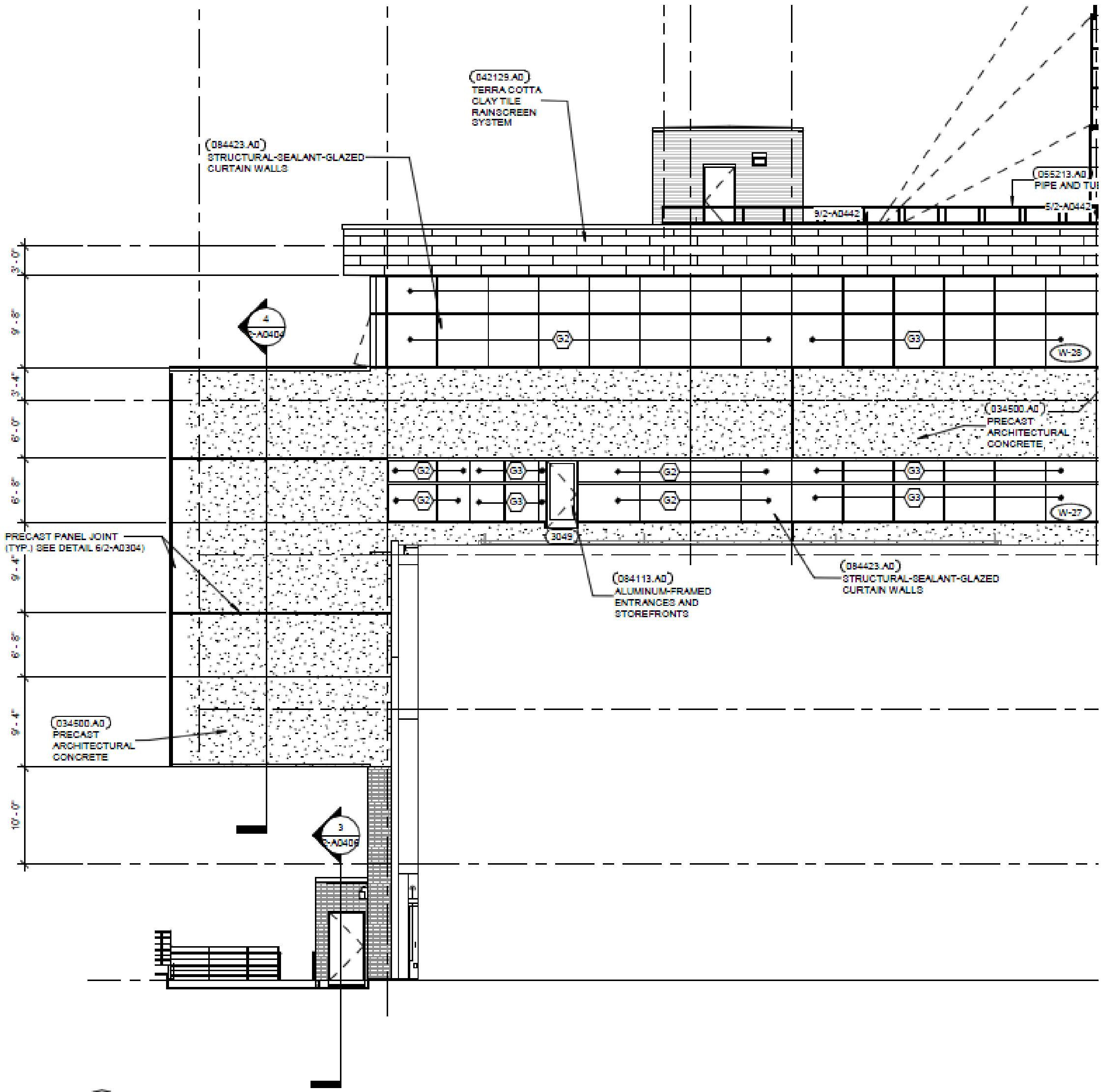

2'
3'
4'
6'
8'
42'-0"
20'-6"
21'-2"
31'-6"
E9
E9'2
E10
E10.8
E11
E12
E13
E14
E15
E16
E17
E18
P'
N'
M'
L'
K'.8
CONC. HANGER
DIAGONAL STRUT
CB3 (24" x 42")
CB1B (24" x 24")
CB1 (24" x 24")
CB2 (24" x 24")
CB27 (18" x 24")
CB24 (12" x 24")
CB25 (18" x 24")
CB23 (18" x 24")
CB22 (12" x 24")
CB21 (24" x 24")
CB5 (18" x 24")
STAIR "E"
STAIR "C"
ELEV. #2
SW-10
SW-7A
EXTERIOR FACE OF EXIST. BUILDING
EX 8WF17
EX 10WF25
EX 12WF27
EX 21WF62
EX 14WF34
EX 10B15
T/EXIST. ROOF [+ 33'-0" ±]
8" E.J.
CANTILEVER BAY SEE NOTE 10
EXISTING PIEZ


PIEZ HALL ADDITION - SCIENCE, ENGINEERING, & TECHNOLOGIES
SUCF #10354
STATE UNIVERSITY CONSTRUCTION FUND
NEW YORK
OSWEGO
CANNONDESIGN
CATWALK PLAN
EXISTING PIEZ
THIRD LEVEL FRAMING PLAN - AREA A
THIRD LEVEL FRAMING PLAN AREA A
2-S0107

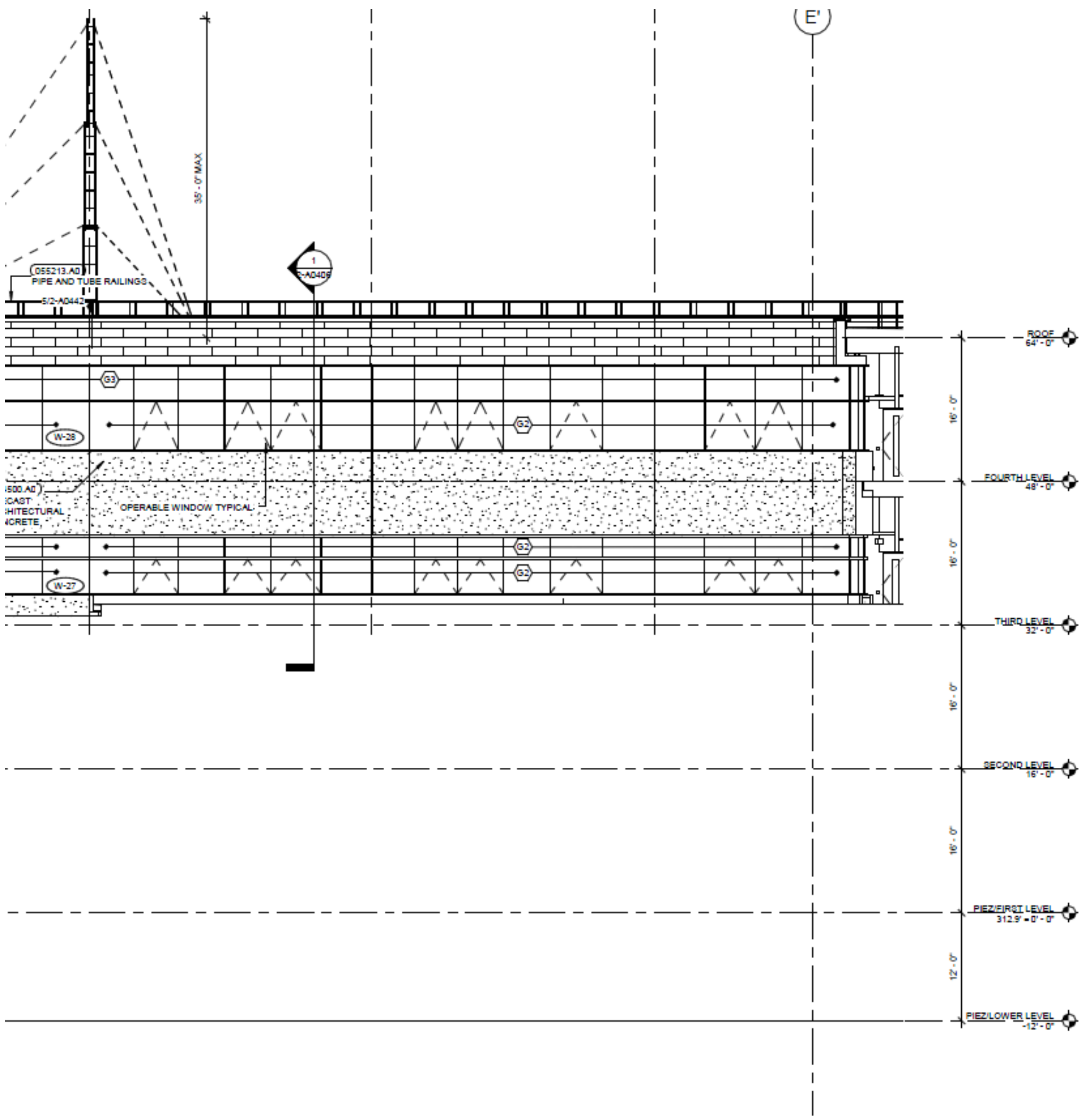

PIEZ HALL ADDITION - SCIENCE, ENGINEERING, & TECHNOLOGIES
SUCF #10354
OSWEGO
CANNONDESIGN
THIRD LEVEL FRAMING PLAN - AREA B
2-S0108
EXISTING PIEZ



2 NORTH ELEVATION



1 PARTIAL WEST ELEVATION
SCALE: 1/8" = 1'-0"


E'
35' - 0" MAX
055213.A0
PIPE AND TUBE RAILINGS
6/2-A0442
1
2-A0406
G3
W-28
G2
500.A0
CAST
HITECTURAL
NCRETE,
OPERABLE WINDOW TYPICAL
W-27
ROOF
64' - 0"
16' - 0"
FOURTH LEVEL
48' - 0"
16' - 0"
THIRD LEVEL
32' - 0"
16' - 0"
SECOND LEVEL
16' - 0"
16' - 0"
PIEZ/FIRST LEVEL
312.9' = 0' - 0"
12' - 0"
PIEZ/LOWER LEVEL
-12' - 0"

References

- Conservation Technology, Inc. (2013). Green roof handbook. Retrieved from www.conservationtechnology.com

- J-Drain. (2013). Green roof drainage systems. Retrieved from www.j-drain.com

- Paschal, H. (2006). Green roof systems achieving high performance protection. In ***Henry Company***. doi:www.henry.com

- vegetali.D. (2013). Hydropack booklet. Retrieved from www.vegetalid.com

Special Thanks to Rachel Chicchi
from Cannon Design