Technical Assignment 3

Piez Hall Extension

Oswego, NY

Min Gao Li
Structural Option
Faculty Advisor: Dr. Thomas E. Boothby
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Executive Summary

In this technical report, a lateral system analysis of Piez hall extension was performed to determine the adequacy and the resistive strength of the system. Stiffness of lateral elements, center of rigidity, center of mass, direct and torsional shear strength, building torsion, serviceability, and overturning moment will be discussed throughout this report. A spot check on a selected column and shear wall was performed to verify the adequacy of the elements to resist all loads.

The lateral system in Piez Hall extension consists of ten shear walls and four lateral concrete braces. The braces that run from the second level to the fourth level are located along the side of the cantilevered portion of the building to enforce stability. The shear walls are orientated and located throughout the rest of the building to best resist lateral loads. All shear walls extent all the way from the ground level to the roof with a few openings for windows and doors in some of the walls.

Thirteen load cases were found to be applicable from ASEC7-10 after deciding to model Piez hall with lateral loads. Even with a factor of 1.6 applied to the wind loads, Seismic loads was found to be the controlling design in either direction. This was expected for a building with large base and low height. A final check of the four wind load cases given in chapter 27 of ASCE7-10 were performed for completion. It was found that case 1 for wind loads in either direction was the most severe out of the 4 cases but still not larger than seismic loads.

Through analysis, it was found that shear wall 3 and 10 had the highest relative stiffness. The stiffness of each lateral resisting element was used to distribute the direct shear and torsional shear to each wall from the controlling forces found in ETABS. Since seismic loads control in both N-S and E-W directions, they were used to calculate the direct and torsional shear of the walls in story four.

Building torsion calculation was also performed. A 5% accidental torsion and the torsion caused by the difference in center or rigidity and center of mass were accounted for. Accidental torsion was obtained by subtracting the torsion with zero assigned eccentricity from the torsion with an assigned 5% eccentricity found from the ETABS output. The other torsion was obtained by multiplying the story forces by the eccentricity from the ETABS output. Then the total building torsion was found.

Serviceability requirements were also checked using un-factored loads. The lateral displacements and story drift for each level were found for the most flexible shear wall using both seismic and wind loads in the E-W direction. For wind, an allowable drift of h/400 was compared to the displacement at each level as defined in ASCE7-10. For seismic, 1% of the story height was used to compared to the inter-story drift as described in ASCE7-10, also. It was determined that all serviceability requirements were met.

Overturning moment was also checked during this analysis. The overturning moment for the seismic load case was calculated and compared to the resisting moment. It was found that the resisting moment was way higher than the overturning moment.

Finally, spot check for a selected column and shear wall was performed. The column was checked for combined axial and bending strength with an interaction diagram. The shear wall was checked for shear strength. These elements were found to be adequate.

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 kinds of 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

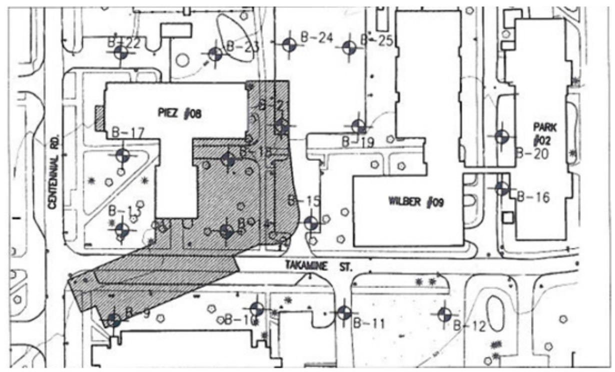


Figure 1: Site map showing existing Piez hall and the new extension (shaded area)

The Piez hall addition will add an expansion of approximately 155,000 square feet to the existing Piez hall. Snygg hall, which is next to the Piez hall, will be demolished as a result of 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.

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.

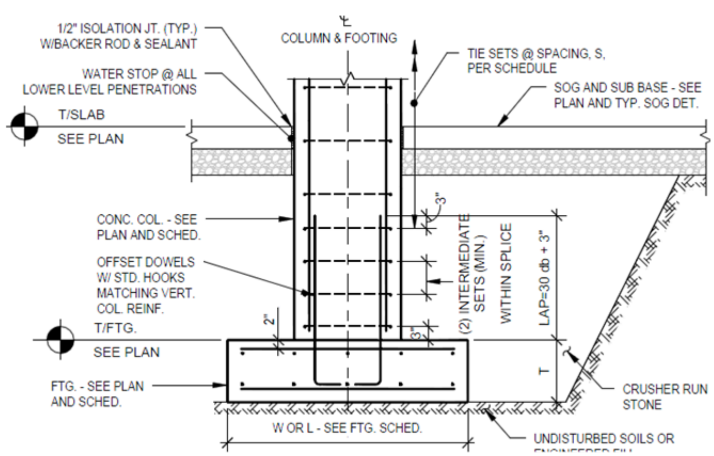


FIGURE 4: TYPICAL COLUMN FOOTING SHOWING REINFORCEMENT PLACEMENT

3. EVENLY SPACE FOOTING REINFORCEMENT. PROVIDE 3-INCHES CLEAR COVER UON.

Floor System

The typical floor structure of Piez Hall addition is a 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.

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 as well as resisting punching shear. Typical drop panels are 10.5'x10.5'x8" (see Figure 6)

In the conservatory 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 required spacing for temperature reinforcement. Typical steel reinforcement placement for the slab is given in figure 5.

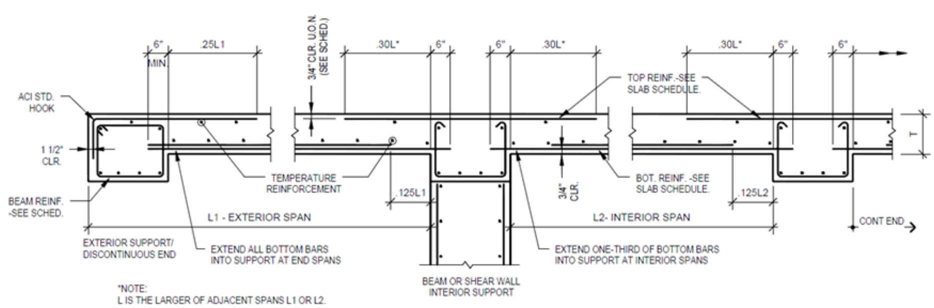


FIGURE 5: TYPICAL ONE WAY SLAB SHOWING REINFORCEMENT PLACEMENTS

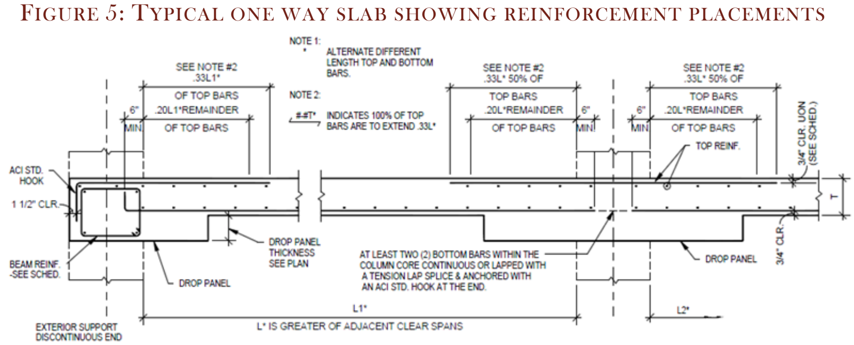


FIGURE 6: TYPICAL COLUMN STRIP DETAIL WITH DROP PANEL AND EDGE BEAM

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.

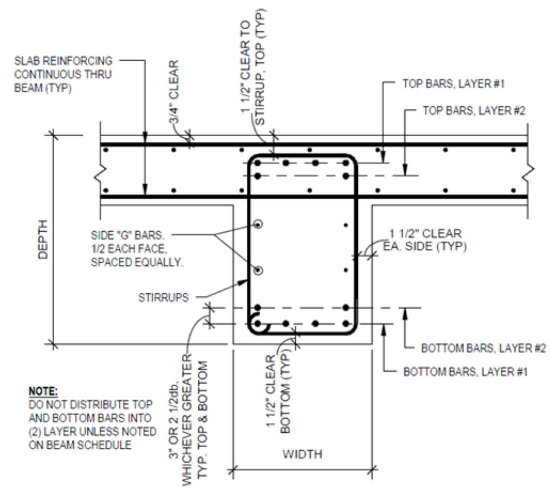


FIGURE 7: TYPICAL BEAM SECTION

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.

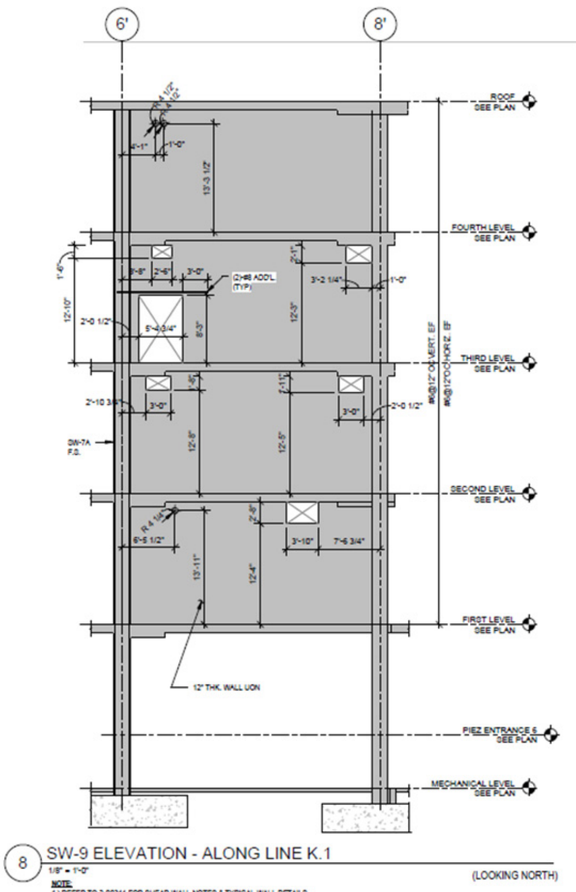


Figure 8: Typical concrete shear wall

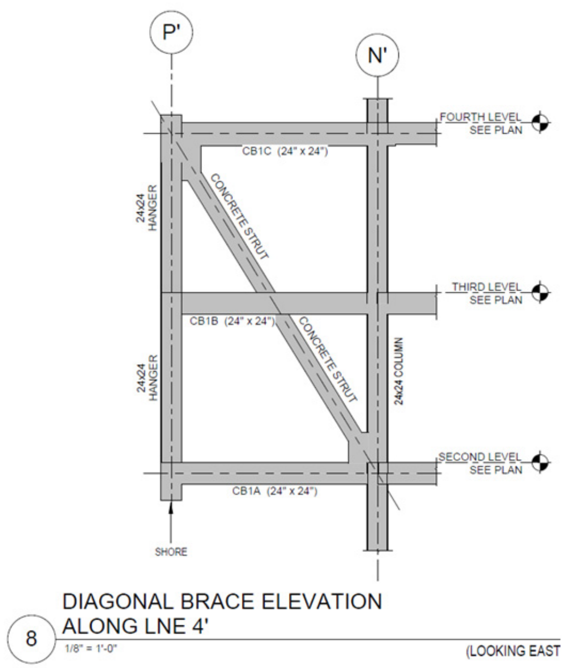


Figure 9: Typical concrete diagonal braces

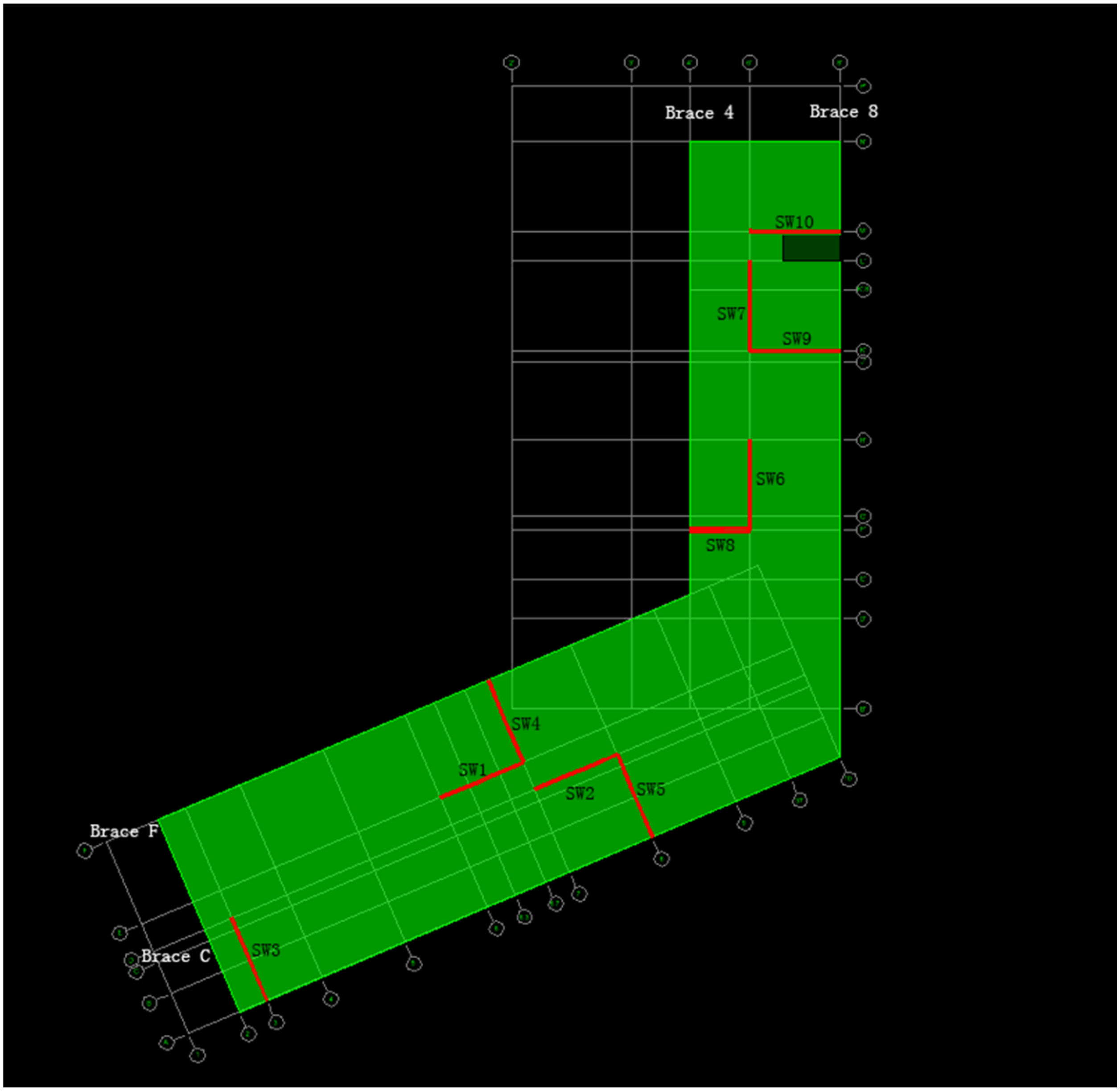


FIGURE 10: SHEAR WALL LOCATIONS OF A TYPICAL FLOOR

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

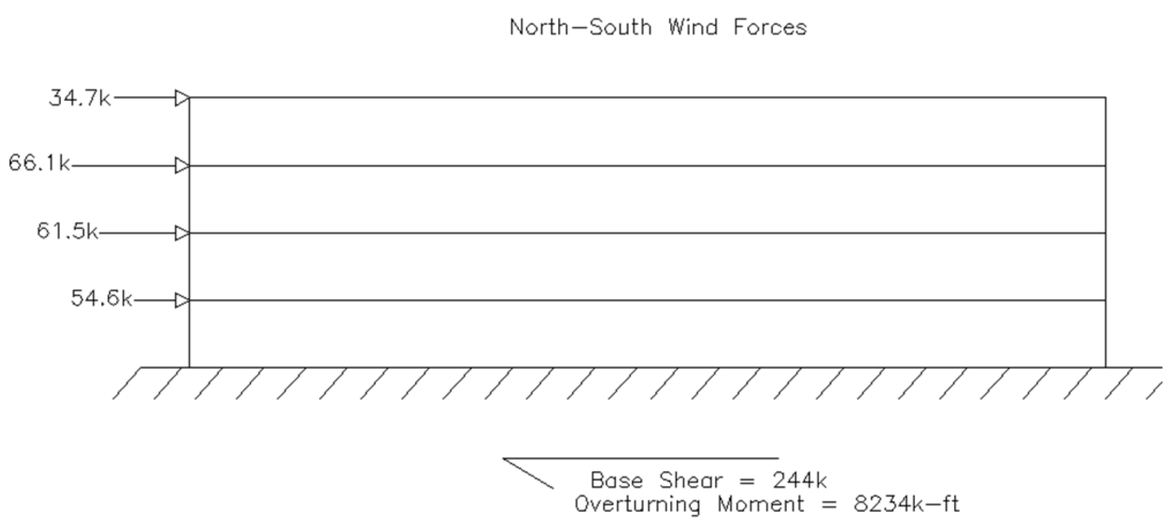


FIGURE 11: WIND FORCES DIAGRAM IN NORTH-SOUTH DIRECTION

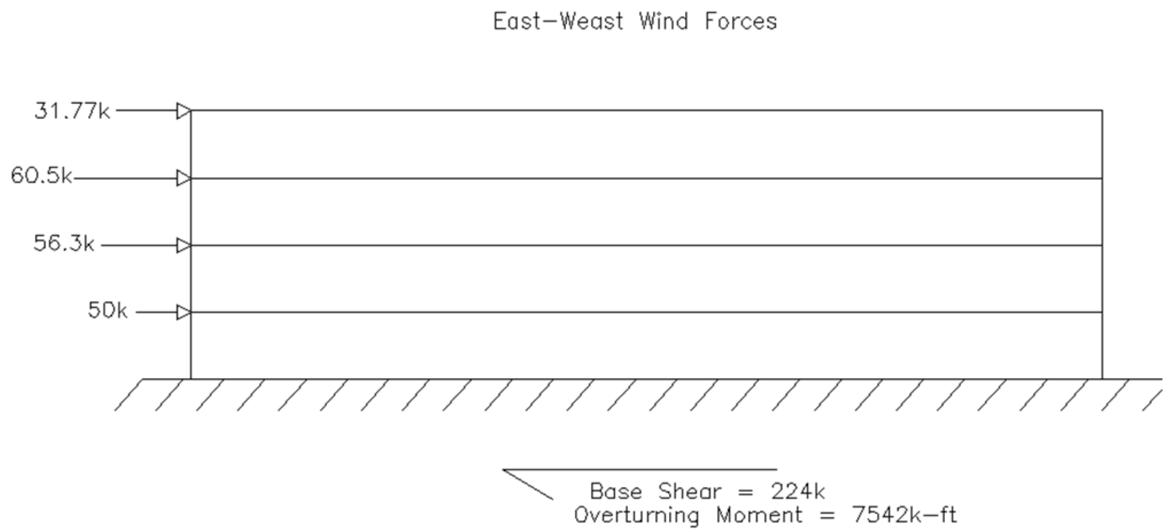


FIGURE 12: WIND FORCES DIAGRAM IN EAST-WEST DIRECTION

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

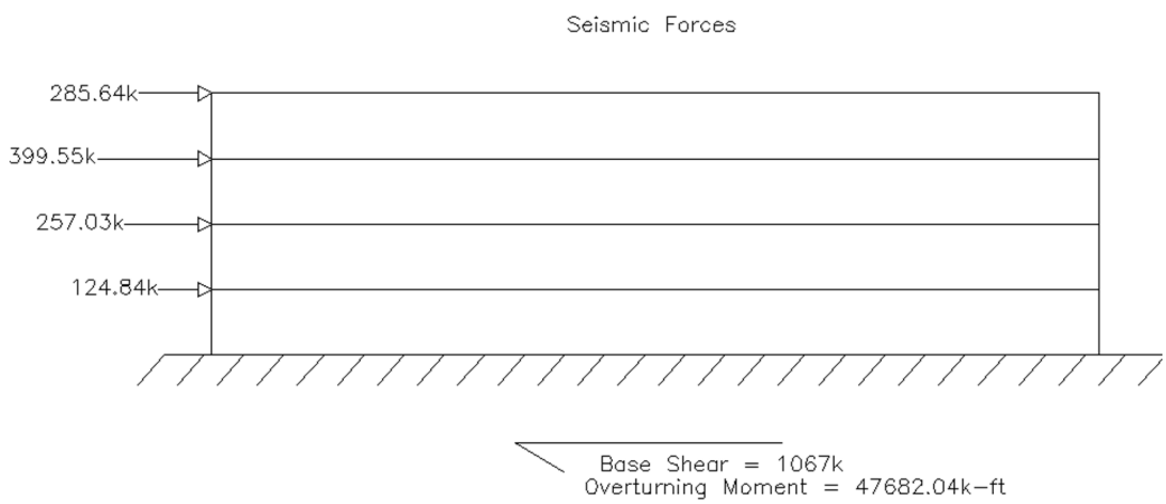


FIGURE 13: SEISMIC FORCES DIAGRAM IN EITHER DIRECTION

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. Even with the wind loads factored by 1.6 as permitted by ASCE 7-10 Section 2.3.2, the resulting base shear and overturning moment still would be less than the one caused by seismic loads. The results are summarized in the table below.

Comparison of Design Forces			
	N-S Wind (with 1.6 factor)	E-W Wind (with 1.6 factor)	Seismic
Base Shear (kip)	390.4	358.4	1067
Overturning Moment (k-ft)	13174.4	12067.2	47682

TABLE 9: COMPARISON OF WIND AND SEISMIC BASE SHEAR

Etabs Model

A model of the lateral system for Piez Hall addition was produced in ETABS. Since the planetarium and conservatory attached to the Piez' Hall did not have any lateral brace systems, they were neglected from the model. Although there are some moment connections near the edge of the conservatory and planetarium, the number of this type of connections were minimal and they are weak in comparison to the shear walls in the new Piez Hall addition. Thus, It was assumed that the conservatory and planetarium depends on the connections to the new Piez' Hall addition for resisting lateral loads. Also, the old Piez Hall was built around 50 years ago, the structural drawings for that portion of the building were long gone. Hence, modeling the lateral system in the old Piez Hall was impossible and was neglected in the model as well.

To model the lateral system in the new Hall, the stiffness of the lateral resisting elements need to incorporate the cracking of the concrete as defined in Section 8.8.2 of ACI 318-11. In order to do this, the code permits either a 50% factor to every gross section property for each concrete element or a certain percentage depended on the type of object. In this model, a 0.5 property modifier was assigned to the f_{22} direction of all concrete members. Another modification was each floor was modeled as a diaphragm. Since the diaphragm would include the mass for each floor, the self-mass was turned off for each material. Moreover, the diaphragm was modeled as rigid to accurately predict the behavior of the two-way flat slab with drop panels of Piez Hall extension.

In order to accurately model the connection of the cast-in-place beams and columns, all line members had to include a rigid-end offset of 0.5 to move the location of the beam ends to the column face. If not done, ETABS would assume a centerline modeling for the member connections, which would be too rigid for the concrete members. As for the shear walls, they were modeled as membranes, which carry shear in the line of direction but not out-of-plane shear.

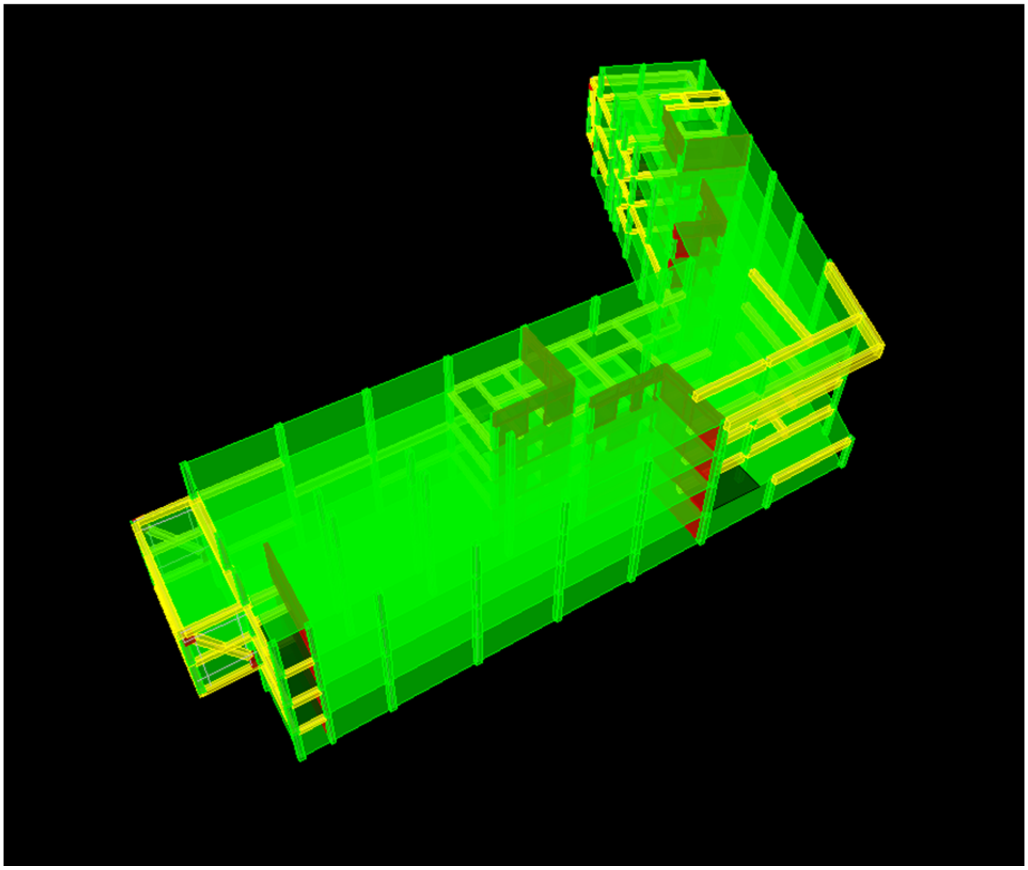


Figure 14: 3-D view of ETABS model

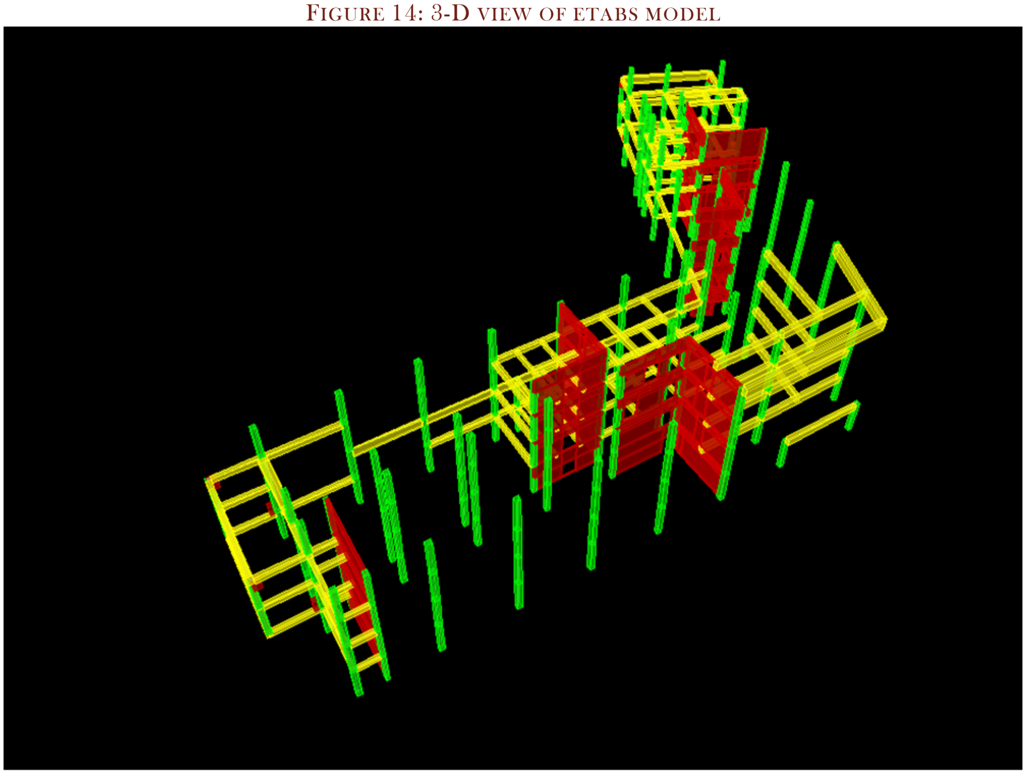


Figure 15: ETABS model showing shear walls, beams and columns

Center of Mass and Rigidity

In order to check the accuracy of the ETABS model, the center or rigidity and the center of mass for level four of the building were calculated by hand then compared to the outputs from the ETABS model. These outputs were given in the table 11.

The center or 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 must be first determined. Each shear wall was modeled individually in ETABS with an applied 1000k load at the top of the wall. The maximum horizontal displacement was then obtained for each wall and stiffness can be determined with the equation, $k=F/\delta$. Since the layout of the shear walls was not orthogonal to each other (see figure 10), the stiffness of each wall was further separated into X and Y components. By separating the k value of a single shear wall, it will be treated as two orthogonal walls that resist lateral loads in both X and Y direction. The X component of stiffness was obtained by multiplying the k value with the sine of 115 degree and the Y component was obtained by multiplying the k value with -cosine of 115 degree.

A center position of the wall was needed for the calculation of center of rigidity. The coordinates of the wall 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 wall can be located using the measure tool in the program. It was noted that a relative error of 4% was found for both the X and Y direction. This difference was probably due to the inaccuracy in determining the center position of the shear walls. Again, using AutoCAD to obtain the wall'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. The hand calculation for the center of mass produced a more accurate numbers in both the X and Y direction with a relative error both less than 1%. Overall, hand calculation of the center of mass and center of rigidity for Piez' Hall proves that the ETABS model was a fairly accurate model.

Stiffness and Coordinate Position of Shear Walls							
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)
SW 1	1000	2.11	473.26	469.09	62.64	1439.95	3263.88
SW 2	1000	2.47	404.33	400.77	53.51	1055.96	3263.88
SW 3	1000	1.49	671.72	88.90	665.81	2399.91	4031.86
SW 4	1000	1.52	657.13	86.97	651.35	1247.96	2975.89
SW 5	1000	1.52	657.13	86.97	651.35	767.97	3359.88
SW 6	1000	1.99	502.46	0.00	502.46	378.00	1682.00
SW 7	1000	2.27	440.74	0.00	440.74	378.00	926.00
SW 8	1000	4.40	227.09	227.09	0.00	505.00	1871.00
SW 9	1000	1.61	619.43	619.43	0.00	11.00	1115.00
SW 10	1000	1.49	671.72	671.72	0.00	189.00	611.00
Brace F	1000	1.05	474.94	470.76	62.86	2975.89	3455.88
Brace C	1000	1.05	474.94	470.76	62.86	2687.90	4031.86
Brace 4	1000	1.05	474.94	0.00	474.94	632.00	116.50
Brace 8	1000	1.05	474.94	0.00	474.94	0.00	116.50

TABLE 10: STIFFNESS AND COORDINATES POSITION OF SHEAR WALLS

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	1072.917	2324.325	1222.405	2299.537
3	1101.966	2298.883	1216.027	2258.935
4	1101.966	2298.883	1182.114	2232.062
Roof	1040.339	2374.683	1095.771	2237.524

TABLE 11: CENTER OF MASS AND CENTER OF RIGIDITY

Load Combinations

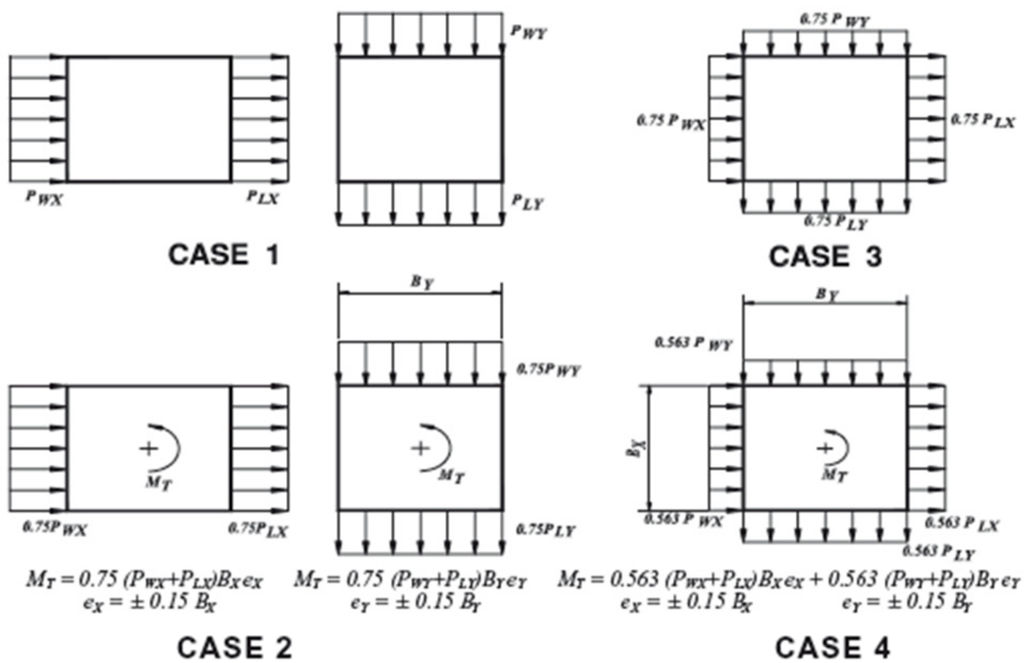
Several load combinations were accounted for during the modeling of the Piez Hall addition. These include the followings from ASCE 7-10 section 2.3.2

1. 1.4D
2. 1.2D + 1.6L +0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.5W)
4. 1.2D + 1.0W +L + 0.5(Lr or S or R)
5. 1.2D +1.0E + L + 0.2S
6. 0.9D +1.0W
7. 0.9D + 1.0E

To simplify the analysis and to avoid unnecessary errors, only load combination 4 to 7 was considered. Furthermore, since only lateral loads were considered in this study, the combination could be reduced to 1.0E and 1.6W for comparisons purposes.

In addition, the four wind load cases from chapter 27 of ASCE7-10 were also considered to find which one controlled. The next page shows the four wind load combinations that were considered and their respective factors.

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 16: FOUR WIND LOAD COMBINATIONS DEFINED IN ASCE7-10 CHAPTER 27

After checking each wind load combinations in excel, it was determined that Case 1 was the most severe out of the four wind cases. A detailed calculation can be found in Appendix C.

Horizontal and Vertical Irregularity

Piez Hall addition 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 δmax/δaverage equals to 1.025, which was less than 1.2 and concluded that torsional irregularity does not exists. Moreover, the Amplification of Accidental Torsional Moment does not apply to the Piez' Hall addition because the building 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 does not apply as well. By inspection, horizontal irregularity type 3 and 4 does not exist since the floor slab does not contain any large openings nor there are any offset shear walls.

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 should not be a concern.

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 17: HORIZONTAL IRREGULARITY TABLE FROM ASCE7-10

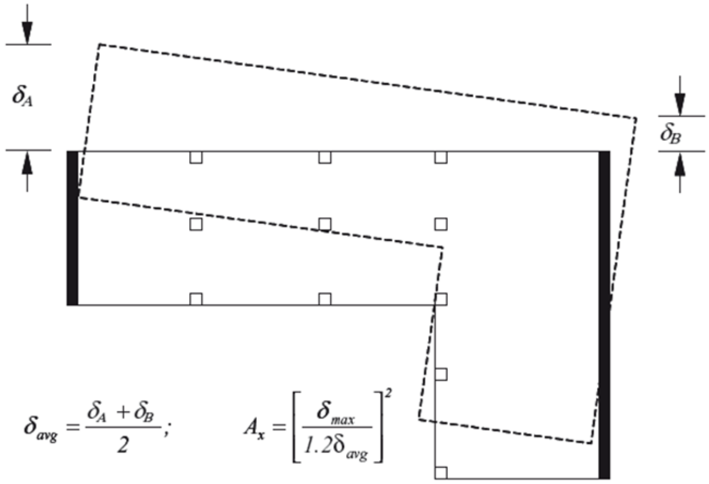


FIGURE 12.8-1 Torsional Amplification Factor, A_x

FIGURE 18: TORSIONAL AMPLIFICATION FACTOR FROM ASCE7-10

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 19: VERTICAL IRREGULARITY TABLE FROM ASCE7-10

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

Building Torsion

ETABS accounts 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 East-West direction, the building torsion was larger than the torsion in North-South direction. This was due to a greater building width in the East-West direction. Also notice that the first story was not accounted for in building torsion because it effectively act 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	124.84	2.07	257.88	1605.98	1863.86
3	257.03	3.33	855.66	3306.89	4162.55
4	399.55	5.57	2224.88	5139.51	7364.38
Roof	285.64	11.43	3264.85	3418.99	6683.84
				Σ=	**20074.64**

TABLE 12: 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	124.84	12.46	1555.20	2032.36	3587.56
3	257.03	9.51	2443.11	4184.81	6627.92
4	399.55	6.68	2668.61	6503.94	9172.56
Roof	285.64	4.62	1319.47	4372.38	5691.85
				Σ=	**25079.88**

TABLE 13: TOTAL BUILDING TORSION CAUSED BY STORY FORCES IN EAST-WEST DIRECTION

Lateral Load Distribution

Direct Shear

The direct shear was calculated for each shear walls and braces. The shear walls 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*dix²	Kx*diy²
SW 1	473.26	469.09	62.64	257.83	-1031.82	4163976.06	499422591.21
SW 2	404.33	400.77	53.51	-126.15	-1031.82	851637.12	426686203.45
SW 3	671.72	88.90	665.81	1217.80	-1799.79	987416769.40	287979173.23
SW 4	657.13	86.97	651.35	65.84	-743.83	2823662.63	48120809.56
SW 5	657.13	86.97	651.35	-414.14	-1127.82	111715785.18	110627150.74
SW 6	502.46	0.00	502.46	-804.11	550.06	324891957.98	0.00
SW 7	440.74	0.00	440.74	-804.11	1306.06	284983866.62	0.00
SW 8	227.09	227.09	0.00	-677.11	361.06	0.00	29604509.75
SW 9	619.43	619.43	0.00	-1171.11	1117.06	0.00	772936730.50
SW 10	671.72	671.72	0.00	-993.11	1621.06	0.00	1765165183.75
Brace F	474.94	470.76	62.86	1793.78	-1223.81	202257864.40	705068182.26
Brace C	474.94	470.76	62.86	1505.79	-1799.79	142526624.78	1524914720.47
Brace 4	474.94	0.00	474.94	-550.11	2115.56	143728580.49	0.00
Brace 8	474.94	0.00	474.94	-1182.11	2115.56	663676534.79	0.00
					$\sum(k*di^2)=$	9039562514.36	

TABLE 14: TORSIONAL RIGIDITY, J, REQUIRED TO OBTAIN THE SHEAR IN EACH WALL

Total Shear in Lateral Resisting Elements (North-South Direction ,Earthquake Controlling)				
Label	Lateral Force (kip)	Direct Shear (kip)	Torsional Shear (kip)	Total Shear (kip)
SW 1	400 ↑	-6.11	1.66	-4.45 ↓
SW 2	400 ↑	-5.22	1.49	-3.73 ↓
SW 3	400 ↑	-64.90	-2.31	-67.21↓
SW 4	400 ↑	-63.49	0.08	-63.42 ↓
SW 5	400 ↑	-63.49	1.30	-62.19 ↓
SW 6	400 ↑	-48.98	1.43	-47.55 ↓
SW 7	400 ↑	-42.96	1.26	-41.71 ↓
SW 8	400 ↑	0.00	-0.29	-0.29 ↓
SW 9	400 ↑	0.00	-2.45	-2.45 ↓
SW 10	400 ↑	0.00	-3.86	-3.86 ↓
Brace F	400 ↑	-6.13	1.64	-4.48 ↓
Brace C	400 ↑	-6.13	2.67	-3.46 ↓
Brace 4	400 ↑	-46.30	0.93	-45.37 ↓
Brace 8	400 ↑	-46.30	1.99	-44.31 ↓

TABLE 15: SHEAR 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)
SW 1	400 →	-52.23	1.38	-50.85 ←
SW 2	400 →	-44.62	1.24	-43.38 ←
SW 3	400 →	-9.90	-1.92	-11.82 ←
SW 4	400 →	-9.68	0.06	-9.62 ←
SW 5	400 →	-9.68	1.09	-8.60 ←
SW 6	400 →	0.00	1.19	1.19 →
SW 7	400 →	0.00	1.05	1.05 →
SW 8	400 →	-25.28	-0.24	-25.53 ←
SW 9	400 →	-68.97	-2.05	-71.02 ←
SW 10	400 →	-74.79	-3.22	-78.01 ←
Brace F	400 →	-52.42	1.37	-51.05 ←
Brace C	400 →	-52.42	2.23	-50.19 ←
Brace 4	400 →	0.00	0.77	0.77 →
Brace 8	400 →	0.00	1.66	1.66 →

TABLE 16: SHEAR FOR EACH LATERAL RESISTING ELEMENTS IN STORY FOUR

Allowable Story Drift

Since shear wall 8 was the most flexible of all the lateral resisting elements, its lateral displacement may be a concern. Therefore, shear wall 8 were 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 Piez' Hall extension was assigned as a category II building of masonry cantilever shear wall structures, 0.010hx was used for the allowable story drift. It was determined that all floor levels met the serviceability requirements for seismic as well. The result was expected because the building was a very stiff structure. In another words, serviceability problems such as drift should not be an issue here.

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

	Risk Category		
Structure	I or II	III	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 20: allowable story drift table from ASCE7-10

Story Drift, E-W Direction Seismic					
	Story	Displacement (in)	Story Drift (in)	Allowable Story Drift (in)	Adequacy
Shear Wall 8	2	0.015327	0.000080	0.16	ok
	3	0.042057	0.000139	0.16	ok
	4	0.072819	0.000160	0.16	ok
	Roof	0.099339	0.000138	0.16	ok

Table 17: story drift check for seismic load

Story Drift, E-W Direction Wind					
	Story	Displacement (in)	Story Drift (in)	Allowable Story Drift (in)	Adequacy
Shear Wall 8	2	0.005513	0.000029	0.48	ok
	3	0.013913	0.000044	0.48	ok
	4	0.022797	0.000046	0.48	ok
	Roof	0.030114	0.000038	0.48	ok

TABLE 18: STORY DRIFT CHECK FOR WIND LOAD

Overturning Moments

It was found that the seismic overturning moment controlled with a value of 47682 kips-ft. To determine the resisting moment, the weight of the structure is 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	1997.47
3	32	8225.02
4	48	19178.54
Roof	64	18281.01
Overturning Moment	∑=	**47682.04**
	Resisting Moment =	**1971801**

TABLE 19: OVERTURING AND RESISTING MOMENT FOR PIEZ HALL EXTENSION

Spot Checks

Spot checks were performed on one of the shear wall along gridline 3 for shear strength. A column located on the intersection of gridline A and 6 were checked for both axial load and bending capacity. It was determined through these analyses that the members were adequate.

Shear wall three was checked for shear strength. Vu was obtained from the controlling seismic load in the East-West direction from ETABS result. The reinforcement, dimensions, and material properties of the shear wall were obtained from the structural drawing. Shear capacity of the wall was computed and was found to be much greater than Vu, and there it is an adequate shear wall. It was believed that that wall was also designed to resist bearing loads, which is the reason why the wall's shear capacity is so large.

The same column A-6 was used to complete the column spot check in technical report one. To analyze the column, an interaction diagram was produced by hand. Three main points, the Pure Axial Strength, Pure Bending Strength, and Pure Tension of column A-6 was calculated and plotted onto a graph. Once the graph was completed, Vu was determined from technical report one and Mu was found from ETABS output by using the controlling seismic load cases in the E-W direction. The Vu and Mu point then was plotted into the interaction diagram. Since the point was within the interaction diagram, the column was proven to be adequate.

Conclusion

Through the lateral analysis on Piez Hall extension, it was determined that the lateral system provided adequate resistance to both seismic and wind forces in each direction. It also met serviceability requirements set forth by ASCE7-10. The analysis was carried out through both hand calculations and computer modeling, with the assistance of excels spreadsheets. The computer model information was checked and verified with hand calculation, and thus the model's output can be used for other complicated calculation.

Using ETABS, Piez hall addition's shear walls, lateral braces, rigid diaphragm, beams and columns were generated and analyzed. The data obtained was reviewed for stiffness, center of rigidity, center of mass, controlling load cases, direct and torsional shear, building torsion, serviceability, and overturning moments. A spot check was also performed on a column and shear wall to verify their adequacy to resist loads. It was found that the building were also sufficient to resist overturning moments in additional to gravity loads.

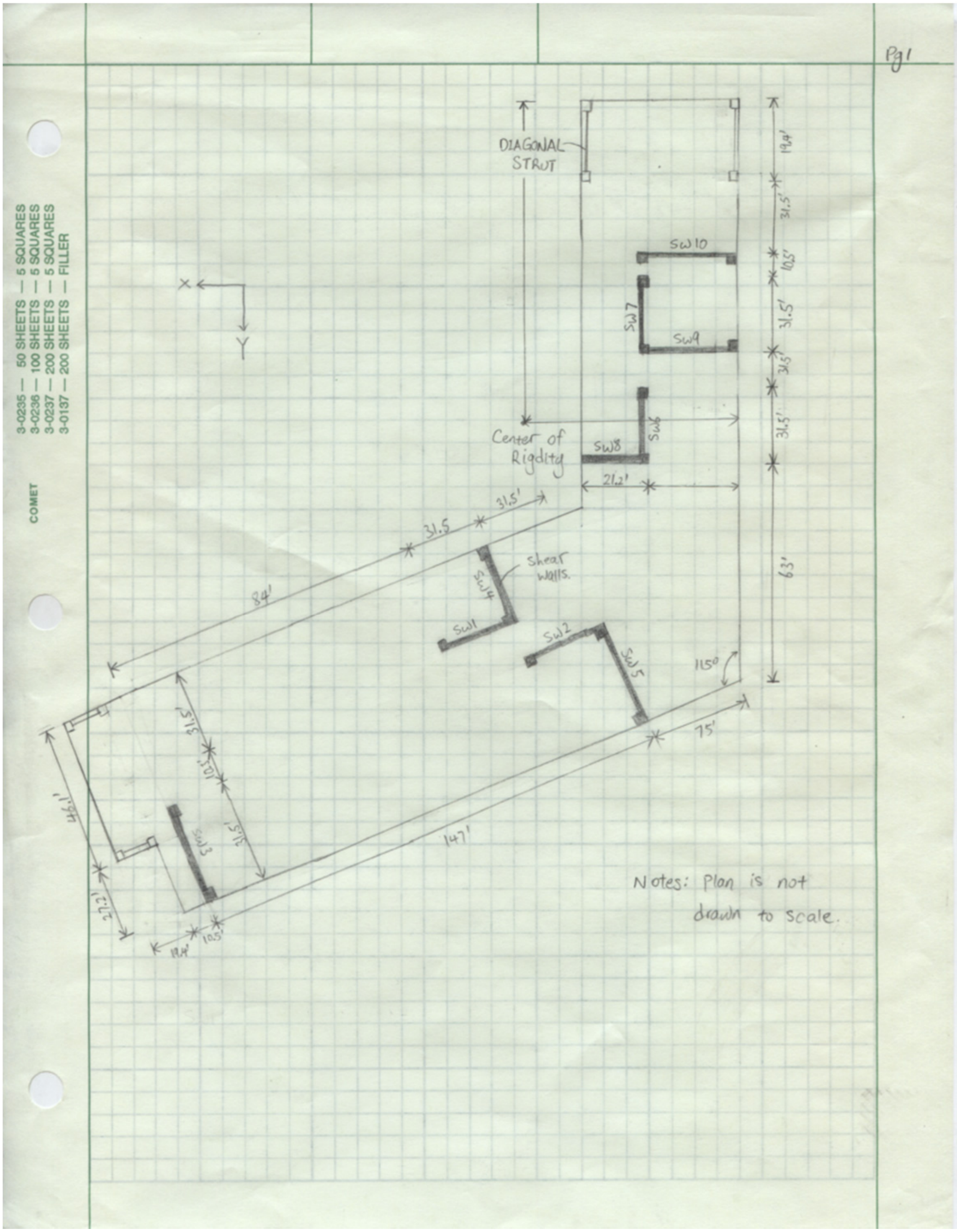
It was found that Seismic controlled in both North-South and East-West direction and for all floor level. These results were confirmed in technical report one during the initial wind and seismic analysis.

Although two horizontal irregularities were found in the Piez Hall addition, none of them were an issue due to the building's low risk seismic design category. Thus the building's torsion was calculated assuming an amplification factor of 1.0. Also, the building was found to have a resisting moment that was significantly larger than the overturning moments in either direction.

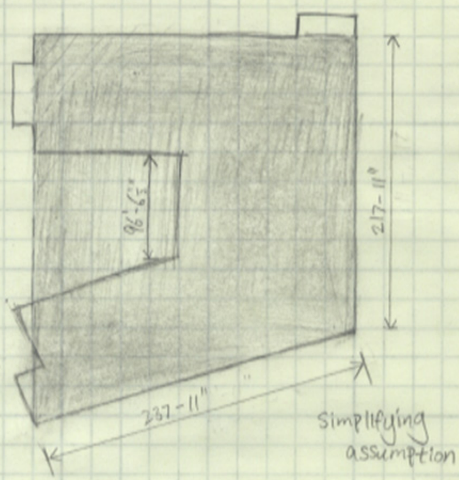
Finally, a spot check was performed to assure that the structural elements of the lateral system were designed with a capacity much greater than required to resist the lateral loads. Interaction diagram was developed for column checks and was concluded that the column was not oversize. Shear wall three was checked for its shear capacity. All in all, the lateral system of Piez hall addition was found to be adequately designed.

Appendices

Appendix A: Wind Analysis Calculations



Techn #1	Wind Load Calculation	MinGao Li	Pg 1/5



N-S Direction

$L = 217.92'$

$B = 237.92'$

E-W Direction

$L = 237.92'$

$B = 217.92'$

Use Method 2 ASCE 7-05 .

Wind Speed: $V = 90$ MPH

Wind directionality factor: $k_d = 0.85$ (table 6-4)

Occupancy category: III (table 6-1)

Importance factor: 1.15 (table 6-1)

Exposure Category: D (unobstracted Area)

Topographic Factor: $k_{zt} = 1.0$ (see §6.5.7)

Velocity Pressure Coefficients: k_z varies with height

$$q_z = 0.00256\, k_z\, k_{zt}\, k_d\, V^2 I \quad (\S 6.5.10)$$

Assume the structure is a rigid building since it is a concrete building. Then Gust factor $G = 0.85$ §6.5.8

Use $GC_{pi} = \pm 0.18$ for enclosed buildings (Fig 6-5)

Design Wind pressure are $P = qGC_p - q_i(GC_{pi})$

External Pressure Coefficient: (Fig 6-6)

Walls: Windward $\Rightarrow C_p = 0.8$

sideward $\Rightarrow C_p = -0.7$

Leeward:

$N-S: \; L/B = \frac{217.92}{237.92} = 0.916 \Rightarrow C_p = -0.5$

$E-W: \; L/B = \frac{237.92}{217.92} = 1.09 \Rightarrow C_p = -0.5$

Roof: $\theta = 0°$

$N-S: \; h/L = 64/217.92 = 0.294$

$E-W: \; h/L = 64/237.92 = 0.269$

$C_p = -0.9$ for dist 0 to h

$C_p = -0.5$ for dist h to 2h

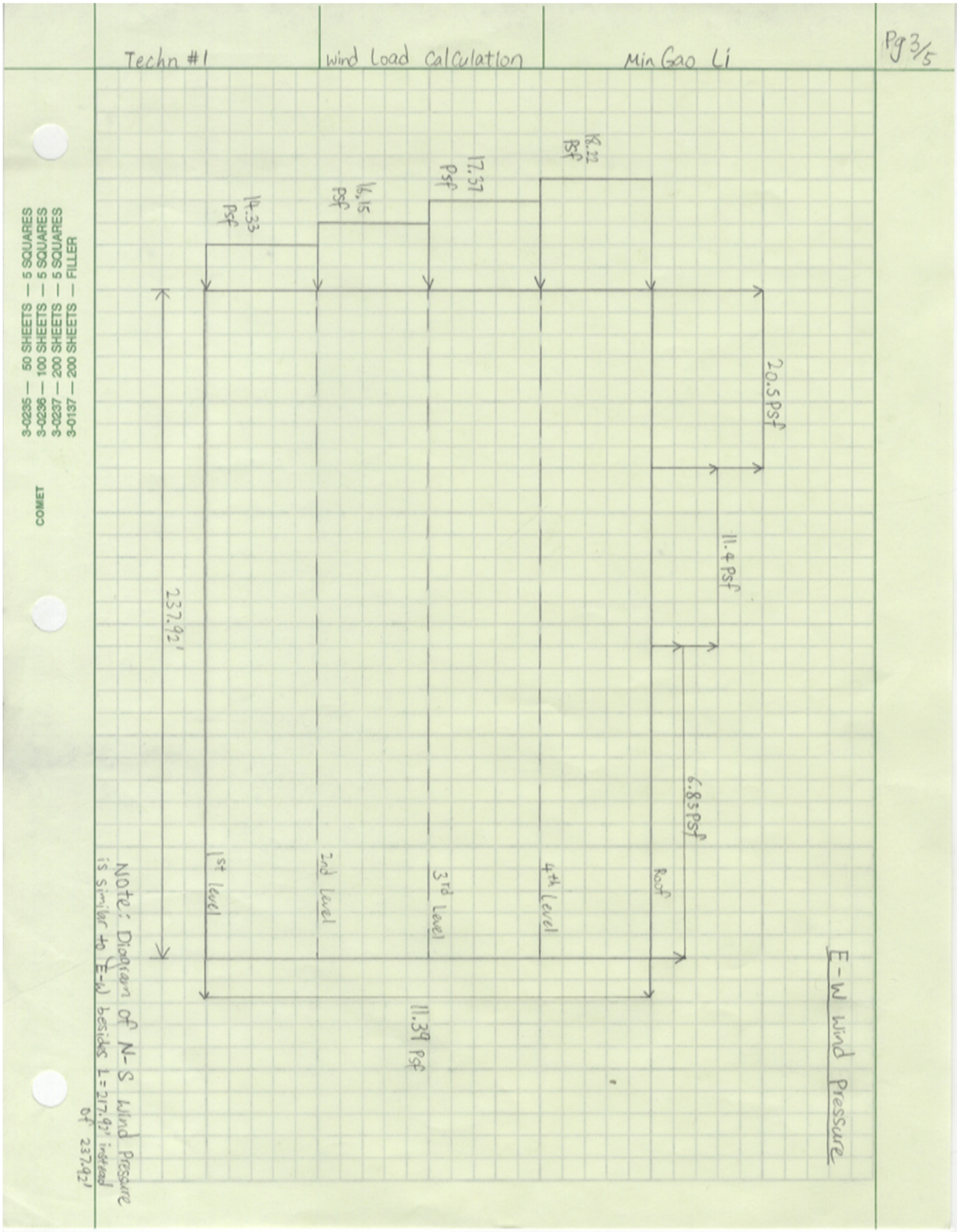
$C_p = -0.3$ for dist > 2h

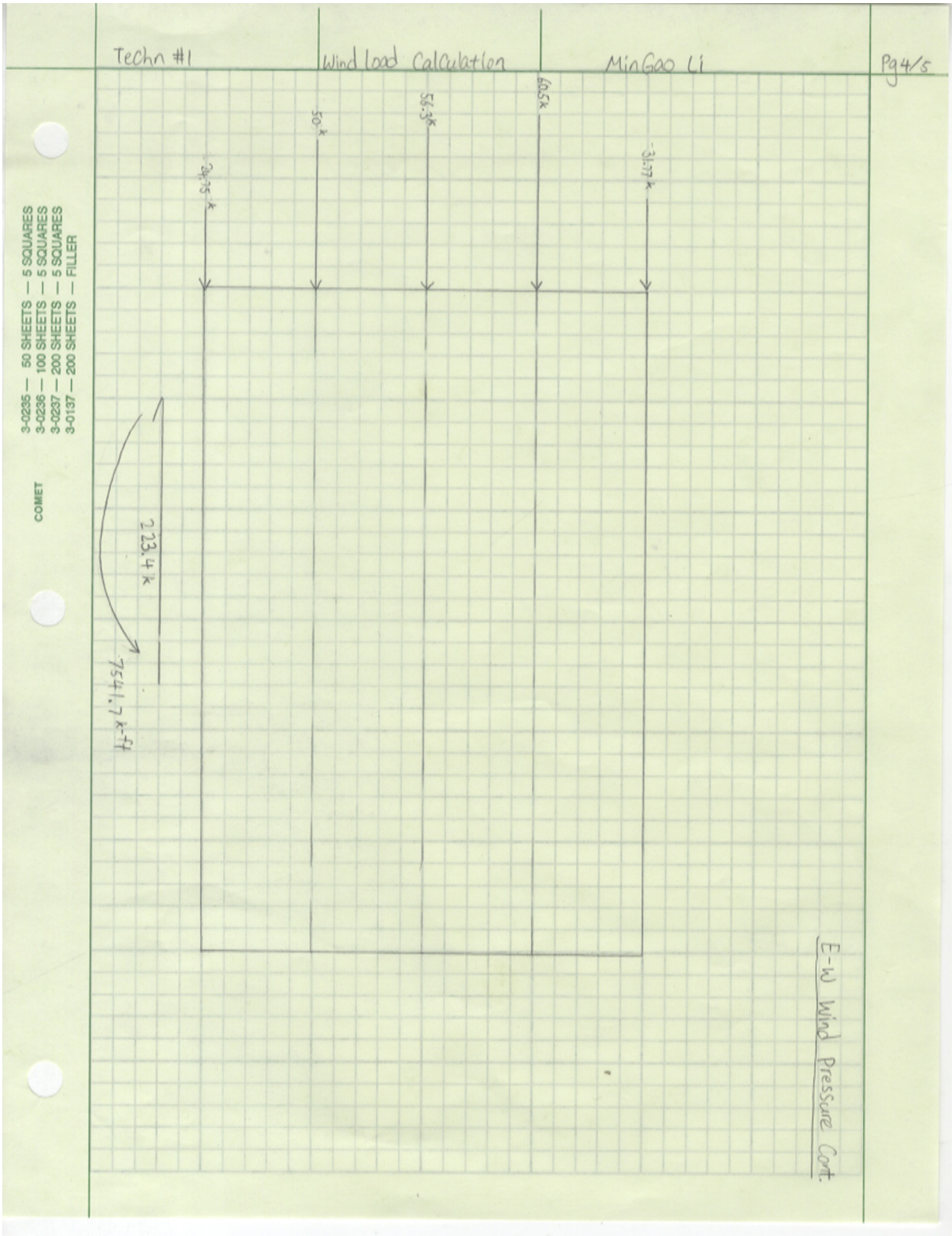
Design Wind Pressures

Windward: $P_z = q_z * GC_p - q_h(GC_{pi})$

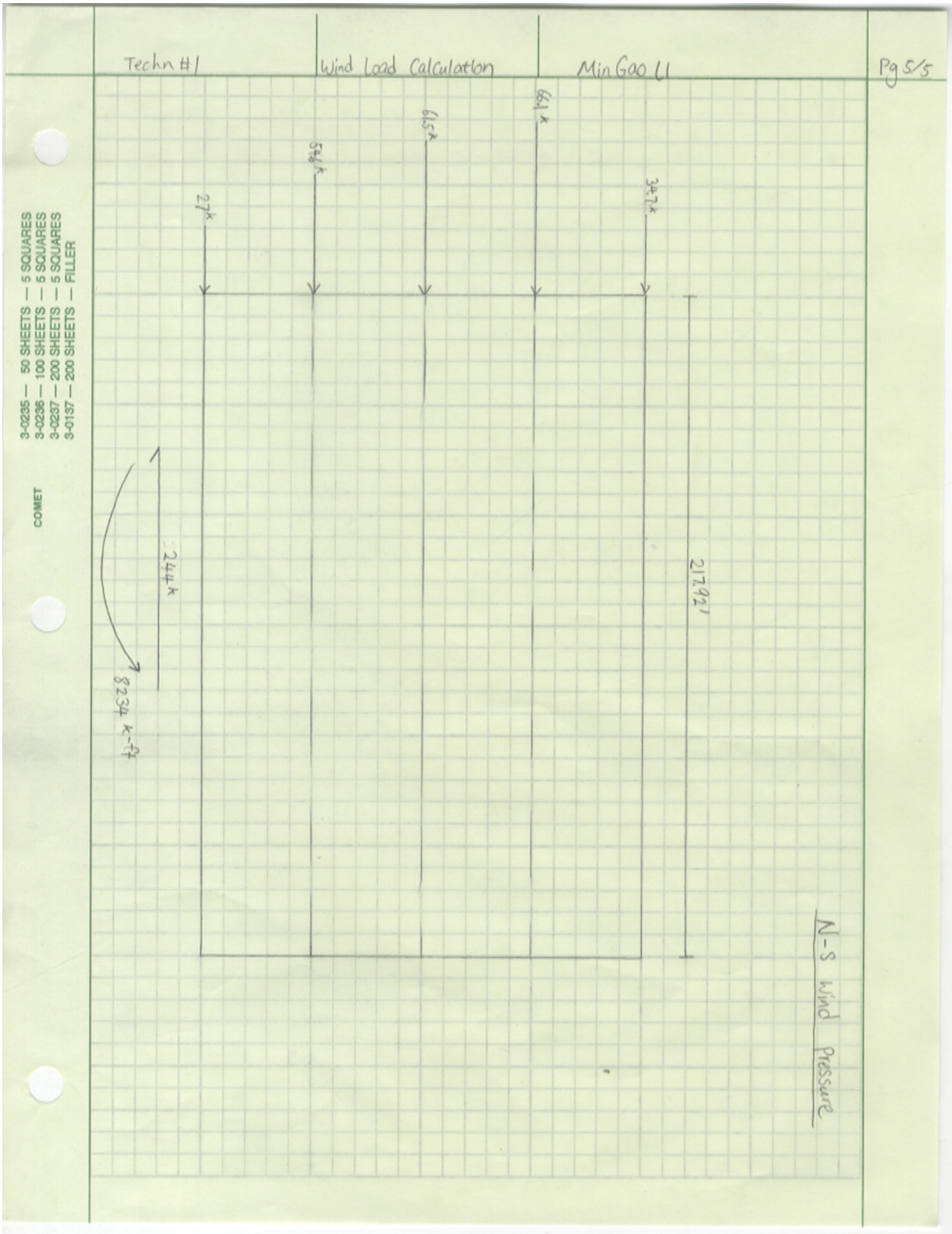
Leeward = sideward = Roof

$P_h = q_h(GC_p - GC_{pi})$


Techn #1
Wind Load Calculation
Min Gao Li
Pg 3/5
COMET
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
14.33 psf
16.15 psf
17.57 psf
18.72 psf
20.5 psf
11.4 psf
6.83 psf
237.92'
11.39 psf
1st level
2nd Level
3rd Level
4th Level
Roof
E-W Wind Pressure
Note: Diagram of N-S Wind Pressure is similar to E-W besides L=217.92' instead of 237.92'



Techn #1
Wind load Calculation
MinGao Li
Pg4/5
24.75 k
50 k
56.3 k
60.5 k
31.77 k
223.4 k
7541.7 k-ft
E-W Wind Pressure Cont.



Wind Calculation

Level	Elevation (ft)	Kz	qz (psf)
1st	0.00	1.03	20.88
2nd	16.00	1.04	21.08
3rd	32.00	1.17	23.76
4th	48.00	1.26	25.54
Roof	64.00	1.32	26.80

Level	Windward	Leeward	Side Wall
1st	14.20	-11.39	-15.94
2nd	14.33	-11.39	-15.94
3rd	16.15	-11.39	-15.94
4th	17.37	-11.39	-15.94
Roof	18.22	-11.39	-15.94

Roof	Cp	
0 to h	-0.90	-20.50
h to 2h	-0.50	-11.39
> 2h	-0.30	-6.83
Windward	0.80	
Leeward	-0.50	
Side Wall	-0.70	

Appendix B: Seismic Analysis Calculations

Techn #1	Seismic Load Cals	MinGao Li	Pg1

Analysis Procedure: Equivalent Lateral Force procedure (per ASCE7)

obtained from structural drawing 2-S001

Seismic importance factor, $I = 1.25$

$S_s = 0.194$ $F_a = 1.2$ (table 11.4-1)

$S_1 = 0.078$ $F_v = 1.7$ (table 11.4-2)

$S_{ms} = F_a \times S_s = 1.2 \times 0.194 = 0.2328$

$S_{m1} = F_v \times S_1 = 1.7 \times 0.078 = 0.1326$

$$\left.\begin{aligned} S_{DS} &= \tfrac{2}{3} S_{ms} = 0.1552 \\ S_{D1} &= \tfrac{2}{3} S_{m1} = 0.0884 \end{aligned}\right\} \text{Seismic Design Category B (Table 11.6-2)}$$

Approximate Fundamental Period (12.8.2.1)

$T = C_T h_N^x$

$C_T = 0.016$ & $x = 0.9$ for Concrete moment-resisting frames (Table 12.8-2)

$T = 0.016(64)^{0.9} = 0.676$ sec

For Oswego NY, from Fig 22-15 => $T_L = 6$ sec

For Ordinary Concrete Shear wall system

$R = 5$ (Table 12.2-1)

$$T \le \begin{cases} C_u T_a = 0.94645 & \text{(Cu refer to table 12.8.1)} \\ T_b = 0.3344 & \text{(from etab results.)} \end{cases}$$

$$C_s = \min \begin{cases} 0.1552/(5/1.25) = 0.0388 \\ 0.0884/(0.3344 \times 5/1.25) = 0.066 \\ 0.0884 \times 6/(0.3344^2 \times 5/1.25) = 1.186 \end{cases}$$

use a C_s of 0.0388

Seismic and weight of entire building

Façade Weight = 30 psf				
Level	Preimeter (ft)	Tributary Height (ft)	Area (ft^2)	Weight (kips)
1.00	944.00	8.00	7552.00	226.56
2.00	1024.00	16.00	16384.00	491.52
3.00	1024.00	16.00	16384.00	491.52
4.00	1024.00	16.00	16384.00	491.52
Roof	944.00	8.00	7552.00	226.56

Slab Weight			
Level	Floor Area (ft^2)	Slab Thickness (in)	Weight (kips)
1.00	26777.60	12.00	4016.64
2.00	29113.60	12.00	4367.04
3.00	28032.00	12.00	4204.80
4.00	28032.00	12.00	4204.80
Roof	26777.60	6.00	2008.32

Shear Wall Weight		
Level	Volume (ft^3)	Weight (kips)
1.00	1445.00	216.75
2.00	2886.00	432.90
3.00	2886.00	432.90
4.00	2886.00	432.90
Roof	1445.00	216.75

Superimposed Dead Load = 20psf		
Level	Floor Area (ft^2)	Weight (kips)
1.00	26777.60	535.55
2.00	29113.60	582.27
3.00	28032.00	560.64
4.00	28032.00	560.64
Roof	26777.60	535.55

Column Weight						
Level	Numer of column	Width or Dia (ft)	Depth (ft)	Tributary Height (ft)	Volume (ft^3)	Weight (kips)
1.00	62.00	2.00	2.00	8.00	1984.00	297.60
2.00	60.00	2.00	2.00	16.00	3840.00	576.00
3.00	58.00	2.00	2.00	16.00	3712.00	556.80
4.00	58.00	2.00	2.00	16.00	3712.00	556.80
Roof	58.00	2.00	2.00	8.00	1856.00	278.40
						2265.60

Total Weight per Level		
Level	Weight (kips)	Weight (psf)
1.00	5293.10	197.67
2.00	6449.73	221.54
3.00	6246.66	222.84
4.00	6246.66	222.84
Roof	3265.58	121.95
Total Weight	27501.74	
V	1067.07	

Appendix C: Load cases and Controlling Forces

Pg1

Case 1 NS

$e_x = 149.5$ in

$P = 400$ k

Case 1 EW

$e_y = 24.8$ in

$P = 400$ k

$e_x = \pm 0.15 B_x = 0.15 \times 2909'' = 436''$

$e_y = \pm 0.15 B_y = 3340.8 \times 0.15 = 501''$

Case 2 NS ± e

$+e = 149.5 + 436'' = 585.5''$

$P = 0.75 \times 400 = 300$ k

$-e = 436'' - 149.5 = 286.5''$

Case 2 EW ± e

$+e = 24.8 + 501 = 525.9''$

$P = 0.75 \times 400 = 300$ k

$-e = 501 - 24.8 = 476.2''$

Case 3

$P_{NS} = 300$ k $= P_{EW}$

$e_x = 149.5$ in

$e_y = 24.8$ in

Case 4

$P = 0.563 \times 400 = 225.2$ k

$+NS_e = 585.5''$ $+EW_e = 525.8''$

$-NS_e = 286.5''$ $-EW_e$ $476.2''$

Label	Stiffness K, (kip/in)	Kx (kip/in)	Ky (kip/in)	Dix (in)	Diy (in)	Ky*dix²	Kx*diy²
SW 1	473.26	469.09	62.64	257.83	-1031.82	4163976.06	499422591.21
SW 2	404.33	400.77	53.51	-126.15	-1031.82	851637.12	426686203.45
SW 3	671.72	88.90	665.81	1217.80	-1799.79	987416769.40	287979173.23
SW 4	657.13	86.97	651.35	65.84	-743.83	2823662.63	48120809.56
SW 5	657.13	86.97	651.35	-414.14	-1127.82	111715785.18	110627150.74
SW 6	502.46	0.00	502.46	-804.11	550.06	324891957.98	0.00
SW 7	440.74	0.00	440.74	-804.11	1306.06	284983866.62	0.00
SW 8	227.09	227.09	0.00	-677.11	361.06	0.00	29604509.75
SW 9	619.43	619.43	0.00	-1171.11	1117.06	0.00	772936730.50
SW 10	671.72	671.72	0.00	-993.11	1621.06	0.00	1765165183.75
Brace F	474.94	470.76	62.86	1793.78	-1223.81	202257864.40	705068182.26
Brace C	474.94	470.76	62.86	1505.79	-1799.79	142526624.78	1524914720.47
Brace 4	474.94	0.00	474.94	-550.11	2115.56	143728580.49	0.00
Brace 8	474.94	0.00	474.94	-1182.11	2115.56	663676534.79	0.00
					$\sum(k*di^2)$=	9039562514.36	

		Case 1 NS		
		p=	-400.00	kip ()
		ex=	80.15	in
	Torsional Shear x (k)	Torsional Shear y (k)	Direct Shear (k)	Total Shear (k)
Sw1	1.72	-0.06	-6.11	-4.45
Sw2	1.47	0.02	-5.22	-3.73
Sw3	0.57	-2.88	-64.90	-67.21
Sw4	0.23	-0.15	-63.49	-63.42
Sw5	0.35	0.96	-63.49	-62.19
Sw6	0.00	1.43	-48.98	-47.55
Sw7	0.00	1.26	-42.96	-41.71
Sw8	-0.29	0.00	0.00	-0.29
Sw9	-2.45	0.00	0.00	-2.45
Sw10	-3.86	0.00	0.00	-3.86
BraceF	2.04	-0.40	-6.13	-4.48

BraceC	3.00	-0.34	-6.13	-3.46
Brace4	0.00	0.93	-46.30	-45.37
Brace8	0.00	1.99	-46.30	-44.31

		Case 1 EW		
		p=	-400.00	kip (→)
		ey=	66.82	in
	Torsional Shear x (k)	Torsional Shear y (k)	Direct Shear (k)	Total Shear (k)
Sw1	1.43	-0.05	-52.23	-50.85
Sw2	1.22	0.02	-44.62	-43.38
Sw3	0.47	-2.40	-9.90	-11.82
Sw4	0.19	-0.13	-9.68	-9.62
Sw5	0.29	0.80	-9.68	-8.60
Sw6	0.00	1.19	0.00	1.19
Sw7	0.00	1.05	0.00	1.05
Sw8	-0.24	0.00	-25.28	-25.53
Sw9	-2.05	0.00	-68.97	-71.02
Sw10	-3.22	0.00	-74.79	-78.01
BraceF	1.70	-0.33	-52.42	-51.05
BraceC	2.51	-0.28	-52.42	-50.19
Brace4	0.00	0.77	0.00	0.77
Brace8	0.00	1.66	0.00	1.66

		Case 2 NS+e		
		p=	-300.00	kip ()
		ex=	516.50	in
	Torsional Shear x (k)	Torsional Shear y (k)	Direct Shear (k)	Total Shear (k)
Sw1	8.30	-0.28	-4.58	3.44
Sw2	7.09	0.12	-3.91	3.29
Sw3	2.74	-13.90	-48.68	-59.83
Sw4	1.11	-0.74	-47.62	-47.25
Sw5	1.68	4.62	-47.62	-41.31
Sw6	0.00	6.93	-36.73	-29.81
Sw7	0.00	6.07	-32.22	-26.15
Sw8	-1.41	0.00	0.00	-1.41
Sw9	-11.86	0.00	0.00	-11.86
Sw10	-18.67	0.00	0.00	-18.67
BraceF	9.88	-1.93	-4.60	3.35
BraceC	14.52	-1.62	-4.60	8.31

Brace4	0.00	4.48	-34.72	-30.24
Brace8	0.00	9.62	-34.72	-25.10

		Case 2 EW+e		
		p=	-300.00	kip (→)
		ex=	567.94	in
	Torsional Shear x (k)	Torsional Shear y (k)	Direct Shear (k)	Total Shear (k)
Sw1	9.12	-0.30	-39.17	-30.35
Sw2	7.79	0.13	-33.47	-25.55
Sw3	3.02	-15.28	-7.42	-19.69
Sw4	1.22	-0.81	-7.26	-6.85
Sw5	1.85	5.08	-7.26	-0.33
Sw6	0.00	7.62	0.00	7.62
Sw7	0.00	6.68	0.00	6.68
Sw8	-1.55	0.00	-18.96	-20.51
Sw9	-13.04	0.00	-51.73	-64.77
Sw10	-20.52	0.00	-56.09	-76.62
BraceF	10.86	-2.13	-39.31	-30.58
BraceC	15.97	-1.78	-39.31	-25.13
Brace4	0.00	4.92	0.00	4.92
Brace8	0.00	10.58	0.00	10.58

		Case 2 NS-e		
		p=	-300.00	kip ()
		ex=	356.20	in
	Torsional Shear x (k)	Torsional Shear y (k)	Direct Shear (k)	Total Shear (k)
Sw1	5.72	-0.19	-4.58	0.95
Sw2	4.89	0.08	-3.91	1.06
Sw3	1.89	-9.59	-48.68	-56.37
Sw4	0.76	-0.51	-47.62	-47.36
Sw5	1.16	3.19	-47.62	-43.27
Sw6	0.00	4.78	-36.73	-31.96
Sw7	0.00	4.19	-32.22	-28.03
Sw8	-0.97	0.00	0.00	-0.97
Sw9	-8.18	0.00	0.00	-8.18
Sw10	-12.87	0.00	0.00	-12.87
BraceF	6.81	-1.33	-4.60	0.88
BraceC	10.02	-1.12	-4.60	4.30
Brace4	0.00	3.09	-34.72	-31.63

Brace8	0.00	6.64	-34.72	-28.09

		Case 2 EW-e		
		p=	-300.00	kip ()
		ex=	434.30	in
	Torsional Shear x (k)	Torsional Shear y (k)	Direct Shear (k)	Total Shear (k)
Sw1	6.98	-0.23	-39.17	-32.43
Sw2	5.96	0.10	-33.47	-27.41
Sw3	2.31	-11.69	-7.42	-16.80
Sw4	0.93	-0.62	-7.26	-6.95
Sw5	1.41	3.89	-7.26	-1.96
Sw6	0.00	5.82	0.00	5.82
Sw7	0.00	5.11	0.00	5.11
Sw8	-1.18	0.00	-18.96	-20.15
Sw9	-9.97	0.00	-51.73	-61.70
Sw10	-15.69	0.00	-56.09	-71.79
BraceF	8.30	-1.63	-39.31	-32.63
BraceC	12.21	-1.36	-39.31	-28.46
Brace4	0.00	3.77	0.00	3.77
Brace8	0.00	8.09	0.00	8.09

Case 4 +NSe+EWe	Case 4 +NSe-EWe	Case 4 -NSe+EWe	Case 4 -Nse-EWe	Case 3
Total Shear				Total Shear (k)
-20.20	-21.76	-22.07	-23.63	-41.47
-16.71	-18.10	-18.38	-19.78	-35.33
-59.70	-57.53	-57.10	-54.93	-59.27
-40.61	-40.68	-40.70	-40.77	-54.78
-31.26	-32.49	-32.73	-33.95	-53.09
-16.66	-18.00	-18.27	-19.62	-34.76
-14.61	-15.79	-16.03	-17.21	-30.49
-16.45	-16.18	-16.12	-15.85	-19.36
-57.52	-55.22	-54.76	-52.46	-55.10
-71.53	-67.90	-67.18	-63.55	-61.40
-20.44	-21.98	-22.29	-23.83	-41.65
-12.63	-15.13	-15.63	-18.14	-40.24
-19.01	-19.88	-20.05	-20.92	-33.45
-10.90	-12.77	-13.14	-15.01	-31.98

Controlling Force		
Sw1	-50.85	Case 1 EW
Sw2	-43.38	Case 1 EW
Sw3	-67.21	Case 1 NS
Sw4	-63.42	Case 1 NS
Sw5	-62.19	Case 1 NS
Sw6	-47.55	Case 1 NS
Sw7	-41.71	Case 1 NS
Sw8	-25.53	Case 1 EW
Sw9	-71.02	Case 1 EW
Sw10	-78.01	Case 1 EW
BraceF	-51.05	Case 1 EW
BraceC	-50.19	Case 1 EW
Brace4	-45.37	Case 1 NS
Brace8	-44.31	Case 1 NS

Appendix E: Horizontal Irregularity

Pg1

horizontal Irregularity Check

Reentrant Corner Irregularity

$0.15 \times 880'' = 132'' < 183''$

$0.15 \times 3058'' = 458.64'' < 922.56''$

Irregularity does exsits. but since the Building is assigned to SDC "B", this irregularity can be omitted.

by inspection, horizontal irregularity 3, 4 does not exist. Type 5 exists.

Type 1a: Torsional Irregularity: for the x-direction

Without Torsion

Roof = 0.1032

Story 4 = 0.0748

With 5% accidental torsion:

Roof = 0.1021

Story 4 = 0.0744

$\delta_{max} = 0.1032 - 0.0748 = 0.284$

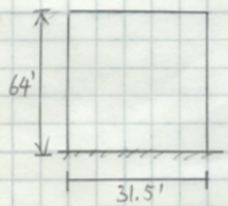
$\delta_{AVG} = 0.1021 - 0.0744 = 0.0277$

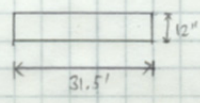
$\delta_{max} / \delta_{AVG} = 1.025 < 1.2 \rightarrow$ No Torsional Irregularity.

Appendix F: Spot Check Calculations

Pg1

Shear Wall 3 along line 3





#6 @ 18" O.C vertical reinf

#6 @ 12 O.C horizontal reinf

$V_U = 150$ kip
↳ from etabs.

<u>Max shear strength</u>

$V_U < \phi V_n = \phi 10 \sqrt{f'c}\, h \cdot d$

$d = 0.8\,(31.5 \times 12) = 302.4$

$\phi V_n = 0.75\,(10)\sqrt{5000}\,(12)(302.4) = 1924^k$

$V_U < \phi V_n \Rightarrow$ ok

Shear strength provided by V_c

$V_c = 2\sqrt{f'c}\; h\, d$

$= 2\sqrt{5000}\,(12)(302.4) = 513^k$

$V_c \leq 3.3\sqrt{f'c}\; h\, d = 3.3\sqrt{5000}\,(12)(302.4) = 847^k$

Horizontal reinf:

$P_t = \dfrac{A_v}{S_h} = \dfrac{(.44)(2)}{(12)(12)} = 0.0061 > 0.0025 \Rightarrow$ ok

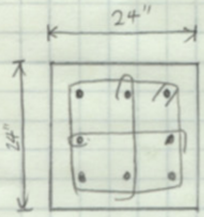
$V_s = \dfrac{A_v f_y d}{S} = \dfrac{2\,(0.44)(60)(302.4)}{12} = 1330.6\,k$

$V_U \leq \phi\,(V_c + V_s) = 0.75\,(847 + 1330.6) = 1633.2 \Rightarrow$ ok

Vertical reinforcement: $\rho = \dfrac{A_v}{S_h}$

$0.0025 + 0.5\left(2.5 - \dfrac{64}{31.5}\right)(0.00407 - 0.0025) = 0.00286 > 0.0025 \Rightarrow$ ok

∴ Wall is okay for shear.



Column A-6 check

Details from 2-S0302

(8) #8 bars. $f'c = 5000$ Psi

$A_s = 8 \times 0.79 = 6.32\ in^2$

Axial strength: $P_o = 0.85 f'c \cdot A_c + A_s f_y$

$P_o = 0.85(5)[24 \times 24 - 6.32] + 6.32(60)$

$P_o = 2800.34 = 2800$ kip

Compression Controlled Section: $\phi = 0.65$

$\phi P_n = 0.65 \times 2800 = 1820$ kips. $= \phi P_o$

Pure bending M_o: $d_i = 1.5 + \frac{1}{2} + \frac{1}{2} d_b = 2.5"$

Assume ε_{s1} & ε_{s2} doesn't yield

$f_{s1} = \frac{\varepsilon_u}{c}(c - d_i) E_s$ $\qquad B_1 = .85 - \frac{0.05}{1000}(5000 - 4000) = .8$

$f_{s1} = \frac{0.003}{c}(c - 2.5)(29000) \Rightarrow 87\left(\frac{c-2.5}{c}\right)$

$f_{s2} = \frac{0.003}{c}(c - 12)(29000) \Rightarrow 87\left(\frac{c-12}{c}\right)$

$f_{s3} = -60$ ksi

$\Sigma F = 0$: $0.85(f'c)(b)(B_1 c) + \Sigma A_{si} f_{si}$

$\Rightarrow 0.85(5)(24)(.8)c + (2.37) \times \frac{87(c-2.5)}{c}$

$+ (1.58) \times \frac{87(c-12)}{c} + (2.37)(-60)$

$\Rightarrow 81.6c^2 + 206.2c - 515.5 + 137.5c - 1649.5 - 142.2c$

MinGao Li | Coln spot check | Techn#3 | Pg 2/3

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

$$81.6C^2 + 201.5C - 2165 = 0$$

$$C = 4.06''$$

Vertify assumption: $f_{s2}: 87\left(\frac{4.06-12}{4.06}\right) = -170 > 60\text{ ksi (NG)}$

– Redo: $\Sigma F = 0: 87C^2 + \underbrace{206.2C - 515.5}_{f_{s1}A_{s1}} + \underbrace{(-60)(1.58)C}_{f_{s2}A_{s2}} - \underbrace{142.2C}_{A_{s3}f_{s3}} = 0$

$$87C^2 - 30.8C - 515.5$$

$$C = 2.62''$$

vertify assumption again:

$$87\left(\frac{2.62-2.5}{2.62}\right) = 4\text{ ksi} < 60\text{ ksi} \Rightarrow \text{ok}$$

$$\varepsilon_{s2} = \frac{0.003}{2.62}(2.62 - 12'') = -0.0107 < -0.00207 \Rightarrow \text{ok}$$

$$-\varepsilon_{s3} \text{ must } < -\varepsilon_{s2} \Rightarrow \text{ok}$$

$$M_o = 0.85(f'c)(b)(\beta_1)(c)\left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + \Sigma A_{si}f_{si}\left(\frac{h}{2} - d_i\right)$$

$$= 0.85(5)(24)(.8)(2.62)\left(12 - \frac{.8 \times 2.62}{2}\right) + (2.37)(4)(12-2.5)$$

$$+ \cancel{(1.58)(-60)(12-12)} \rightarrow 0 + (2.37)(-60)(12-21.5)$$

$$= 2341 + 90.06 + 1350.9 = 3782\text{ k·in} = 315.2\text{ k-ft}$$

$$\phi M_o = 284\text{ k-ft}$$

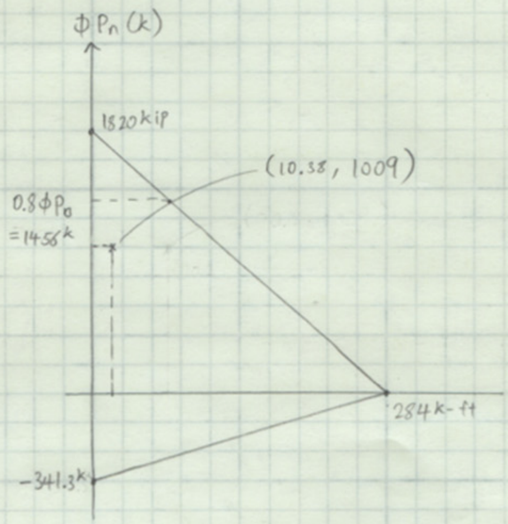
MinGao Li | Coln spot Check | Techn#3 | Pg 3/3

Pure Tension: $\phi T_o = \Sigma A_{si} f_y = (6.32)(-60) = 379.2\ kip \times .9 = 341.3\ kips$

Determine Axial Load: P

Using loads from techn#1: $P_u = 1009$ kips.

$M_u = 10.38$ (obtain in ETabs for seismic)

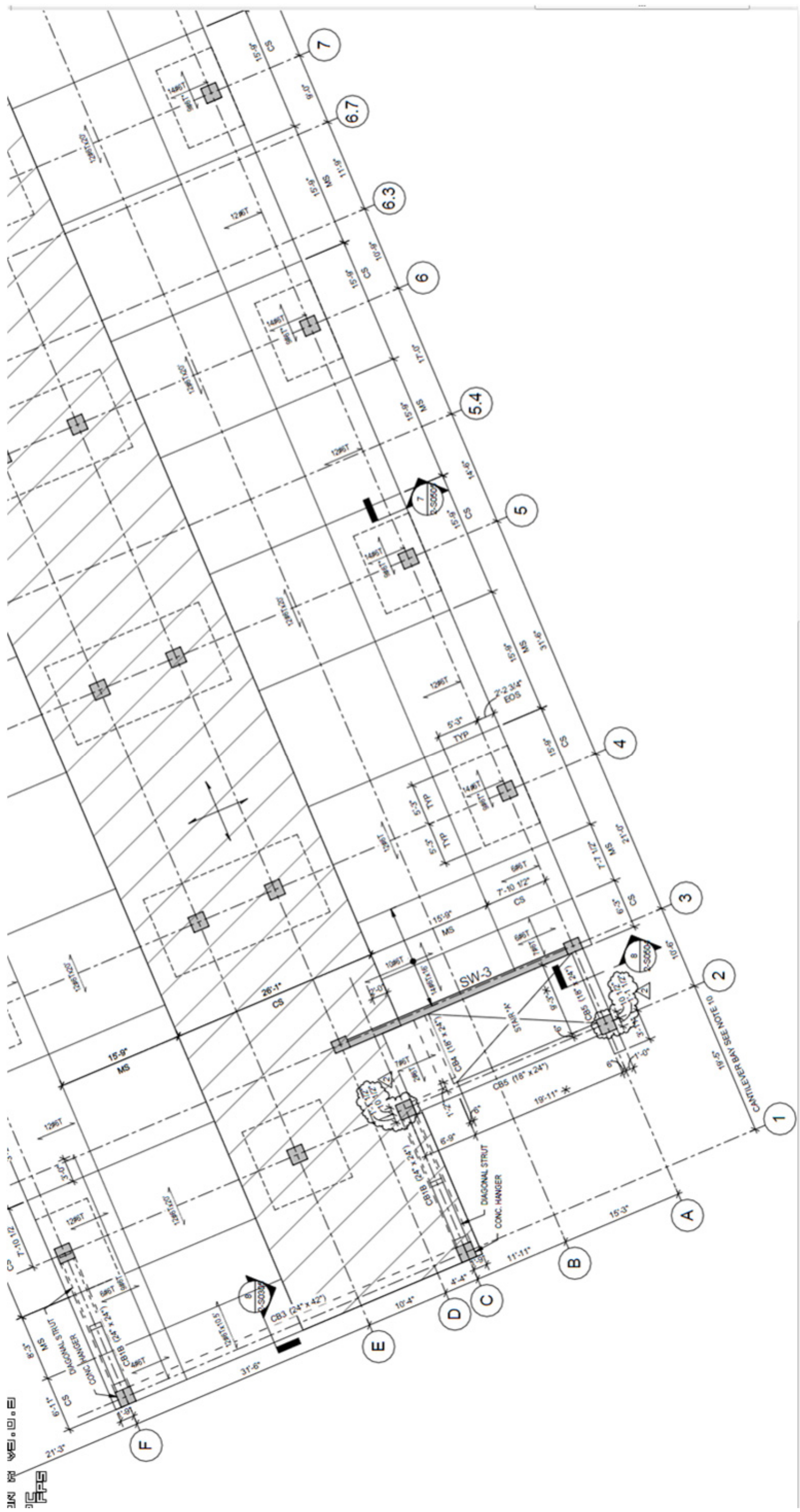


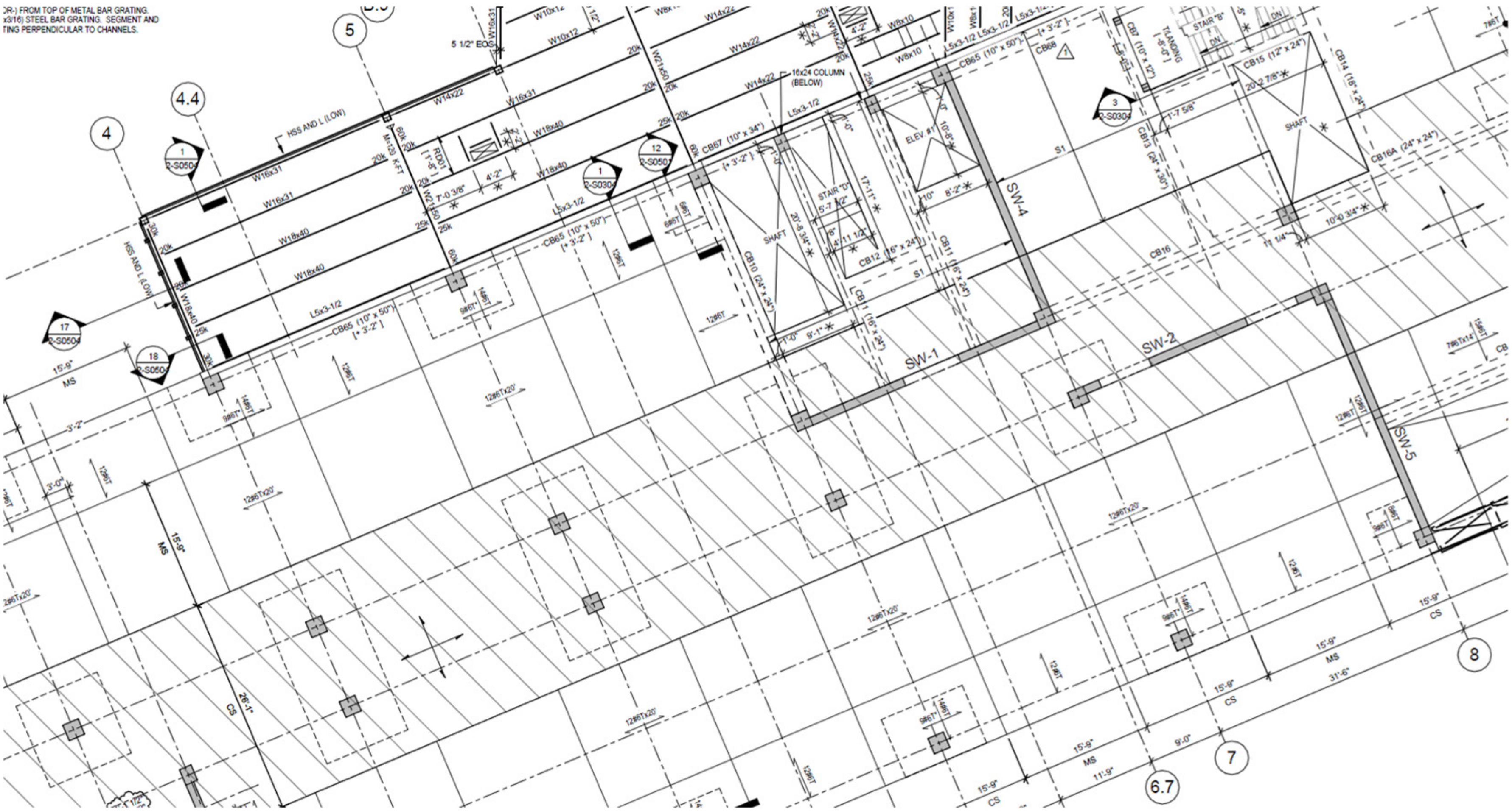
Mu, Pu falls within the interaction Diagram

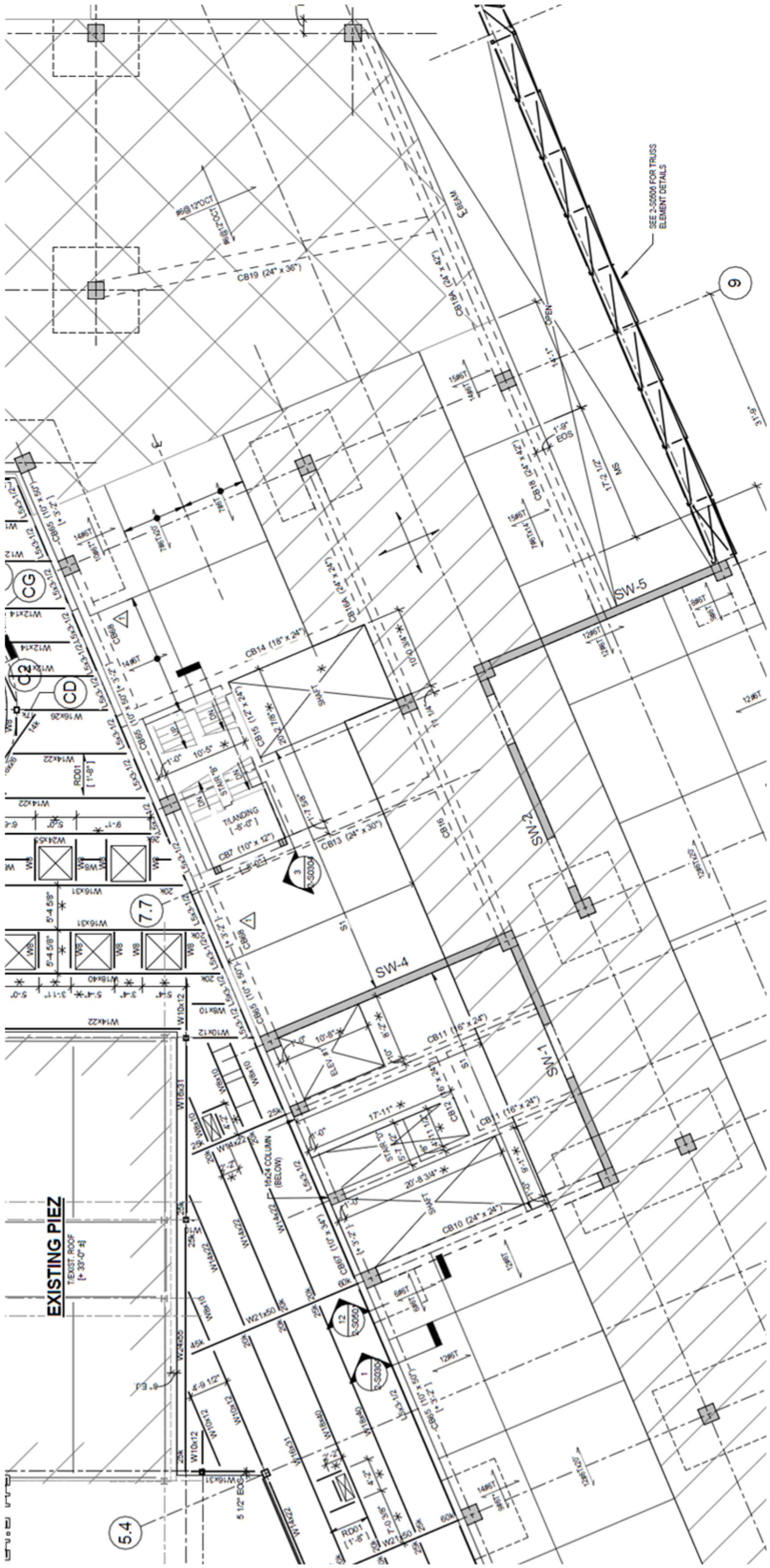
Column A-6 is Good.

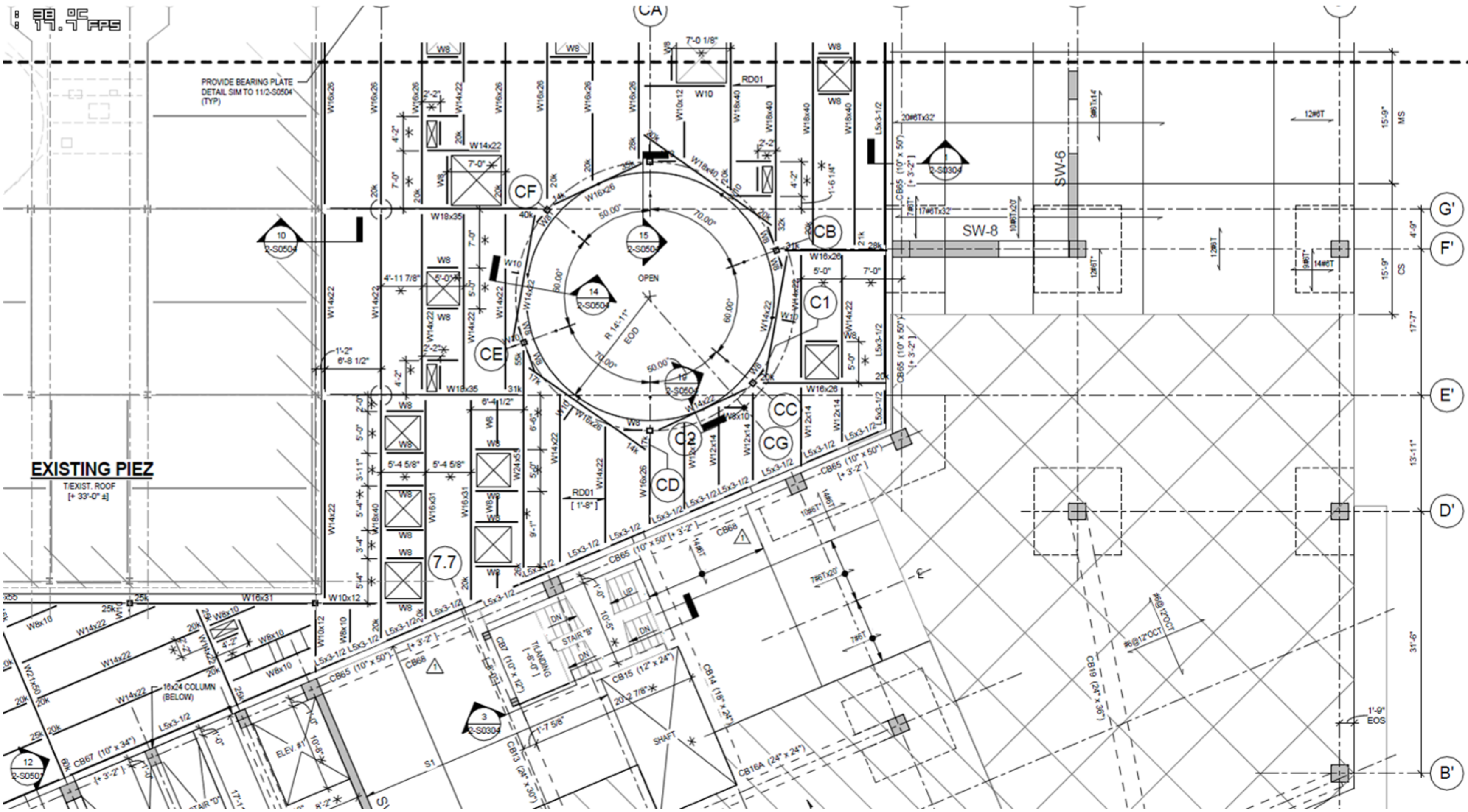
Appendix G: Typical Plans

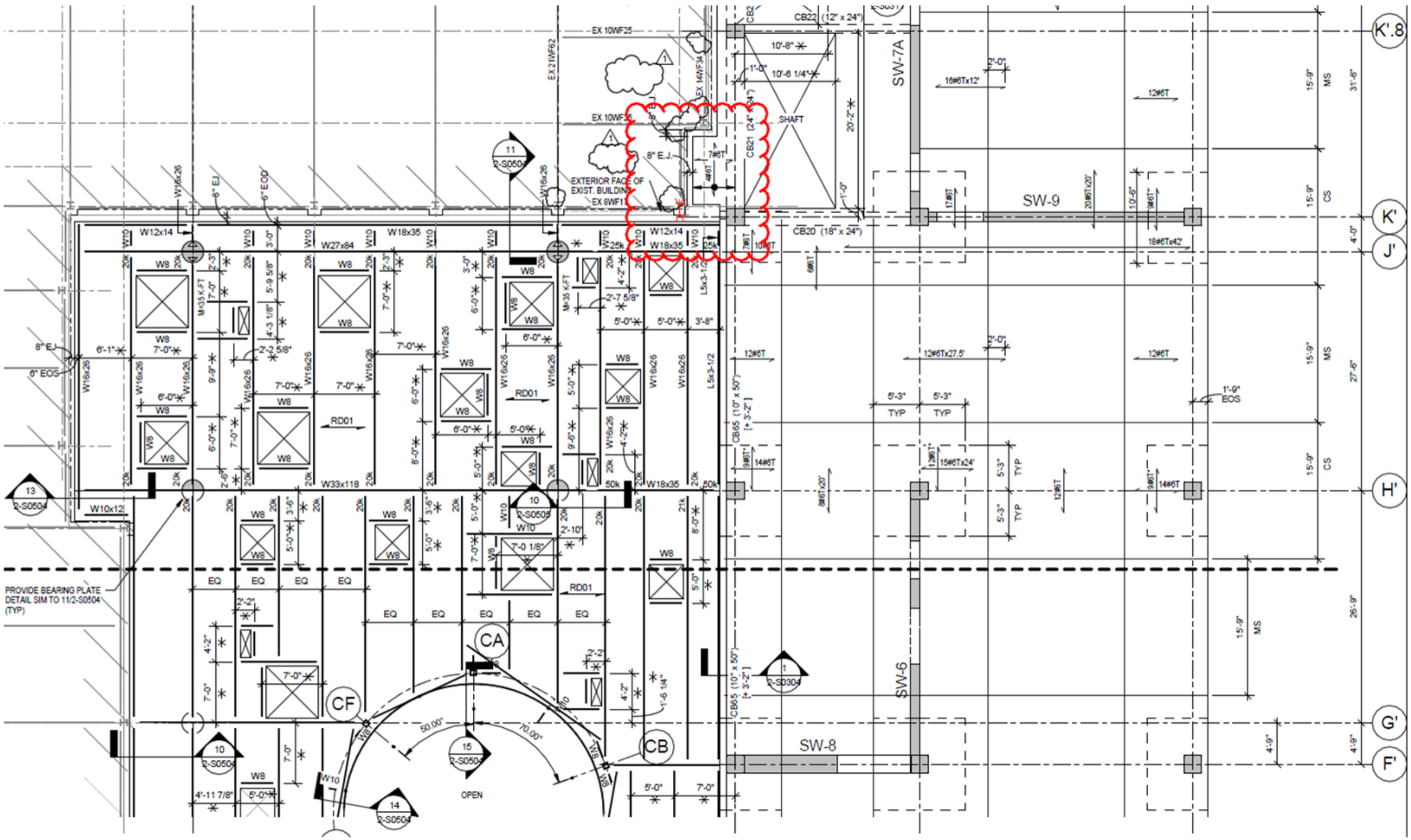
← N



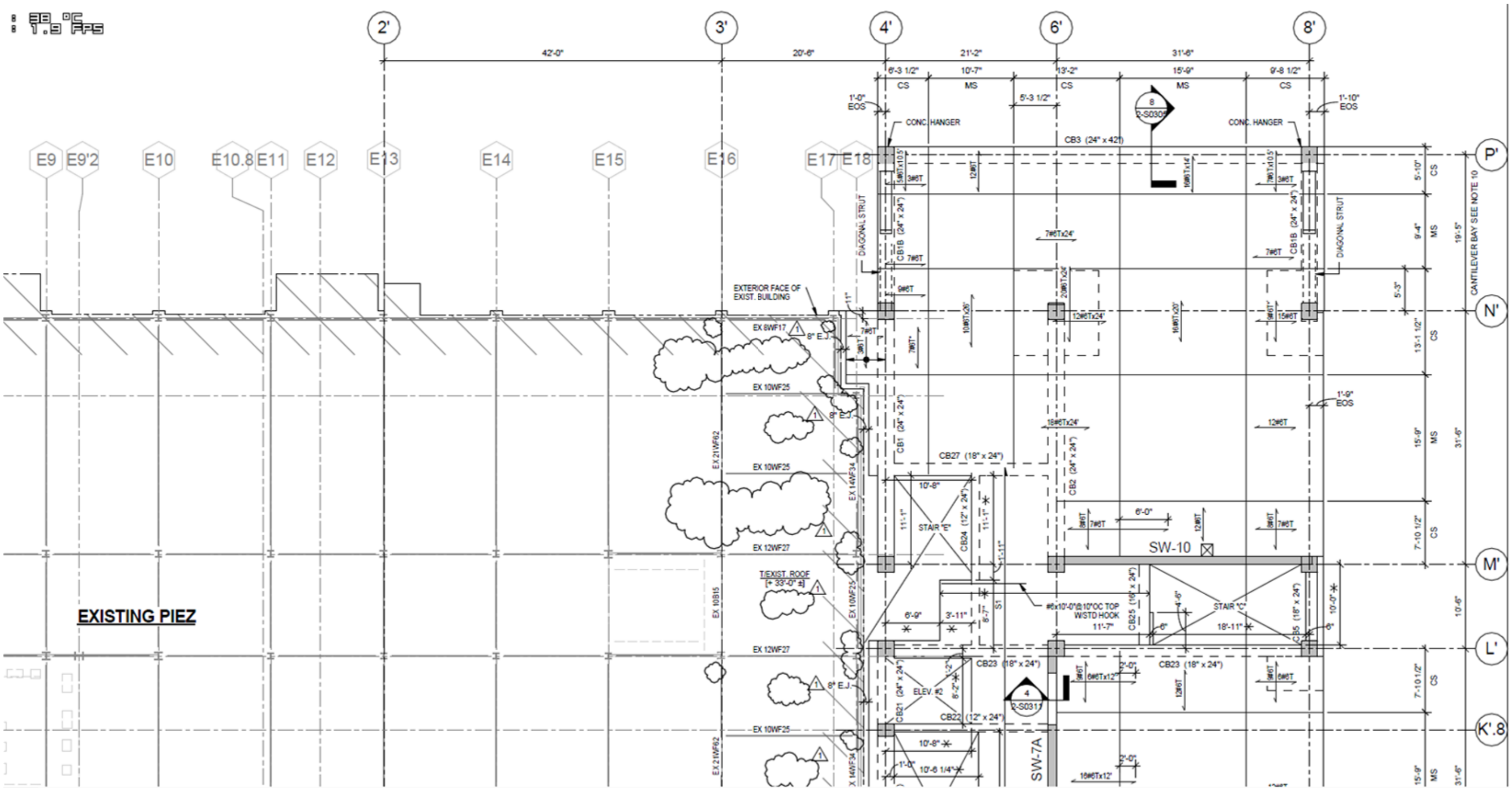

SW-1
SW-2
SW-4
SW-5
4
4.4
5
6.7
7
8

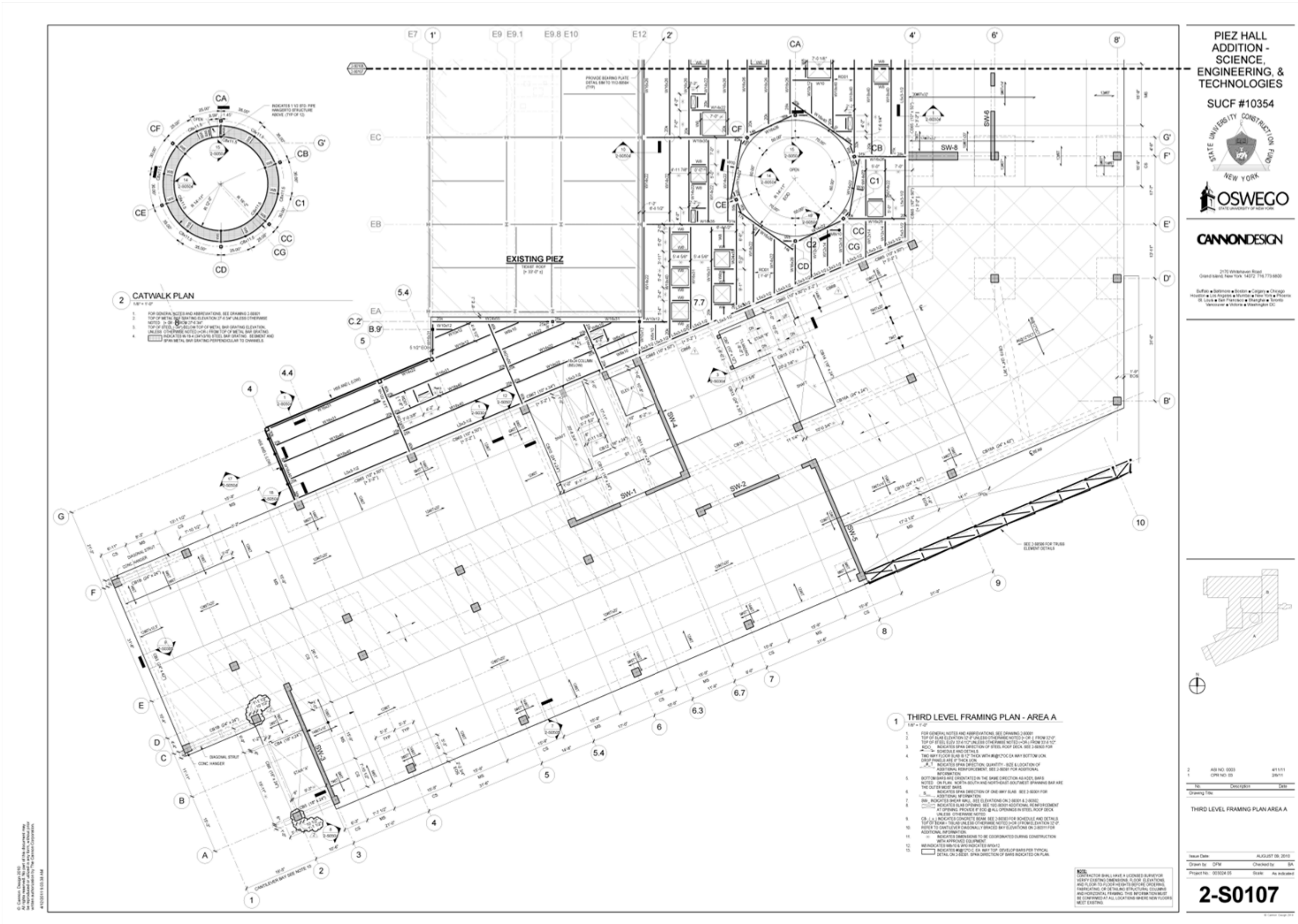

EXISTING PIEZ
SW-1
SW-2
SW-4
SW-5
SEE 2-S0006 FOR TRUSS ELEMENT DETAILS

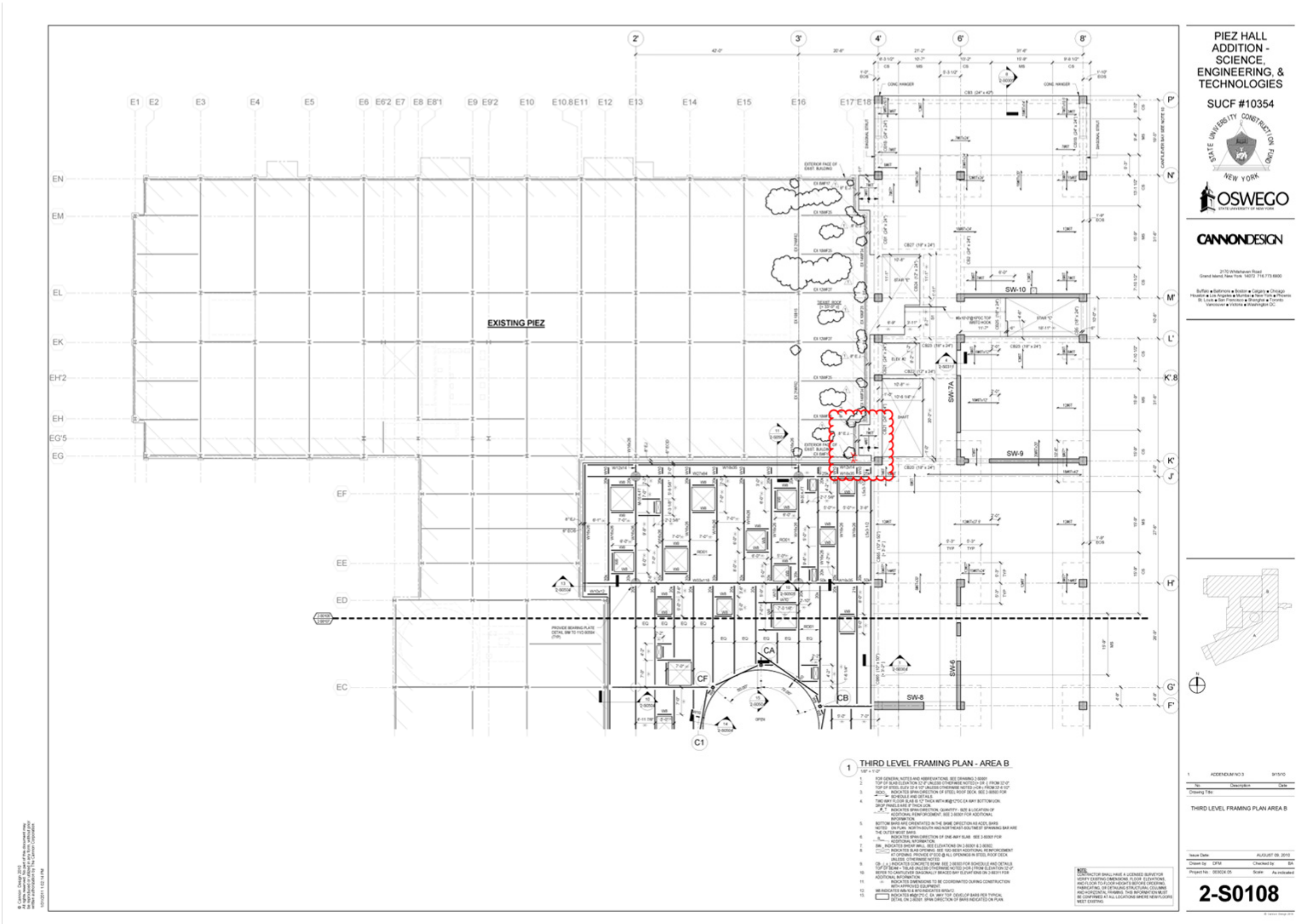

EXISTING PIEZ
T/EXIST. ROOF
[+ 33'-0" ±]
PROVIDE BEARING PLATE
DETAIL SIM TO 11/2-S0504
(TYP)
SW-6
SW-8
OPEN
EOD
R 14'-11"
CA
CB
CC
CD
CE
CF
CG
C1
C2
7.7
G'
F'
E'
D'
B'
RD01
SHAFT
ELEV
16x24 COLUMN
(BELOW)
CB19 (24" x 36")
CB14 (18" x 24")
CB16A (24" x 24")
CB15 (12" x 24")
CB13 (24" x 30")
CB7 (10" x 12")
CB65 (10" x 50")
CB67 (10" x 34")
15'-9"
4'-9"
17'-7"
13'-11"
31'-6"
1'-9"
EOS

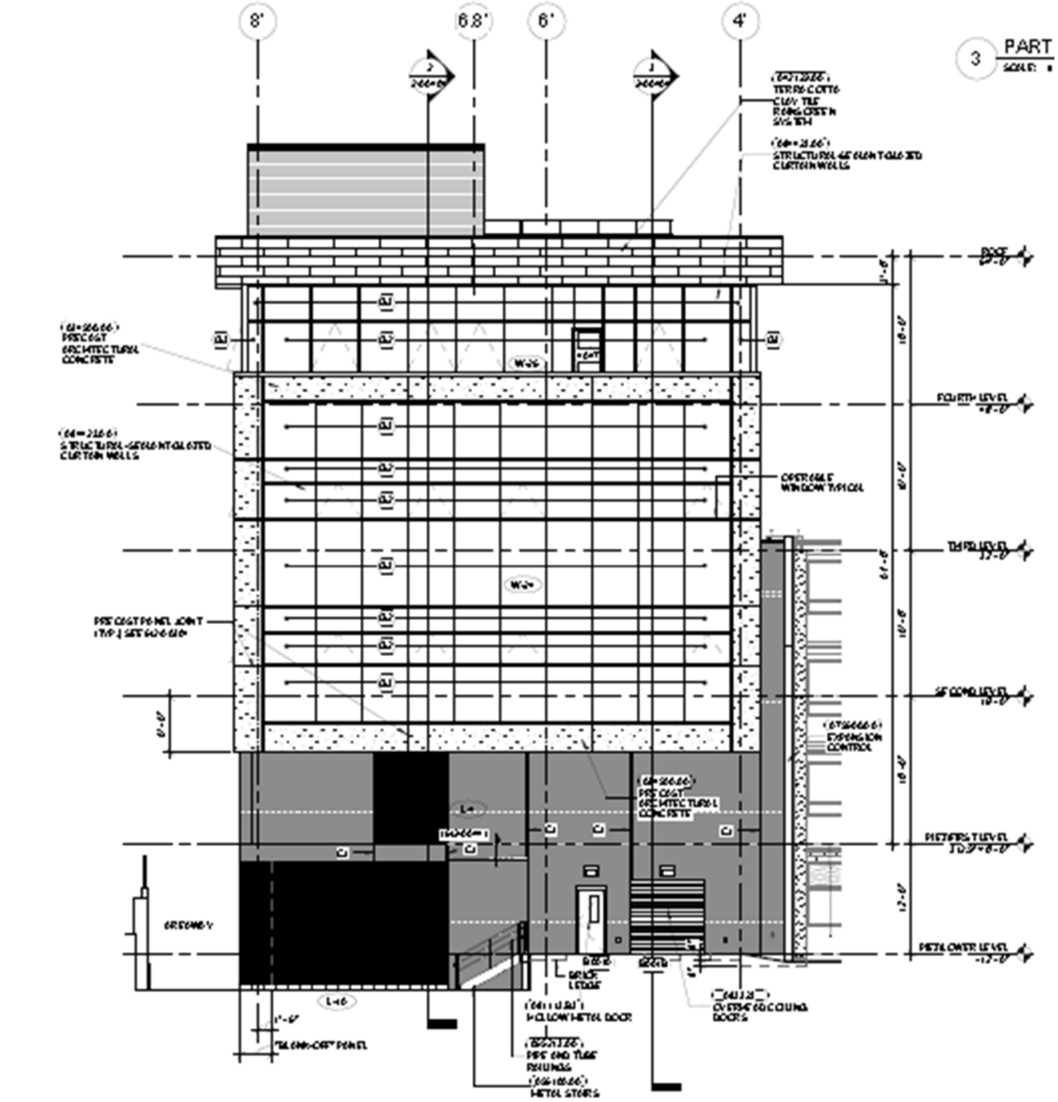


K'.8
K'
J'
H'
G'
F'
SW-7A
SW-9
SW-6
SW-8
SHAFT
CA
CB
CF
OPEN
EXTERIOR FACE OF EXIST. BUILDING
PROVIDE BEARING PLATE DETAIL SIM TO 11/2-S0504 (TYP)

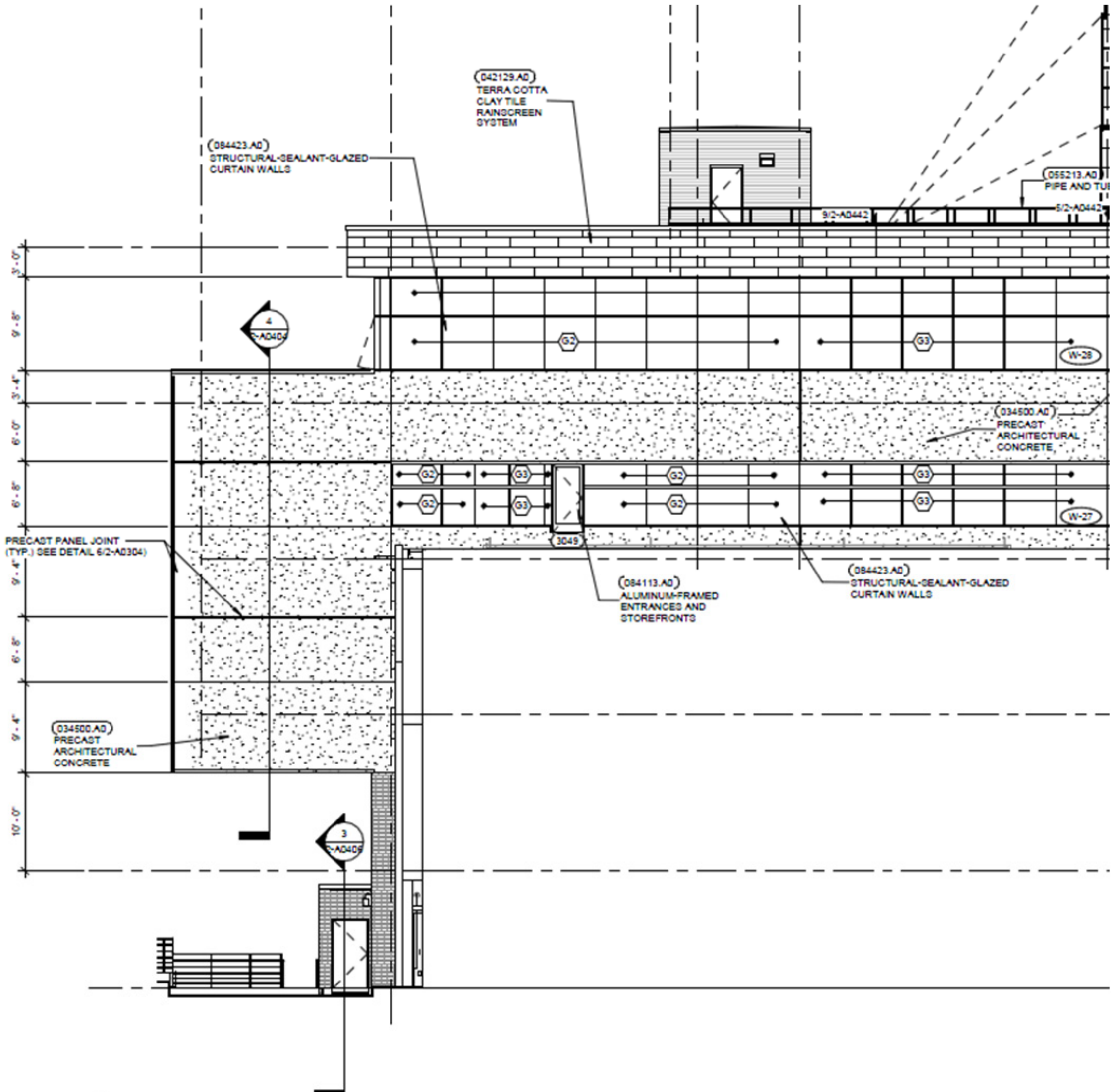

2'
3'
4'
6'
8'
P'
N'
M'
L'
K'.8
E9
E9'2
E10
E10.8
E11
E12
E13
E14
E15
E16
E17
E18
42'-0"
20'-6"
21'-2"
31'-6"
CONC. HANGER
CB3 (24" x 42")
DIAGONAL STRUT
EXTERIOR FACE OF EXIST. BUILDING
CB27 (18" x 24")
CB1 (24" x 24")
CB2 (24" x 24")
STAIR "E"
STAIR "C"
ELEV. #2
SW-10
SW-7A
CB23 (18" x 24")
CB25 (18" x 24")
CB21 (24" x 24")
CB22 (12" x 24")
CB24 (12" x 24")
CANTILEVER BAY SEE NOTE 10
T/EXIST. ROOF
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
EXISTING PIEZ
THIRD LEVEL FRAMING PLAN - AREA B
THIRD LEVEL FRAMING PLAN AREA B
2-S0108

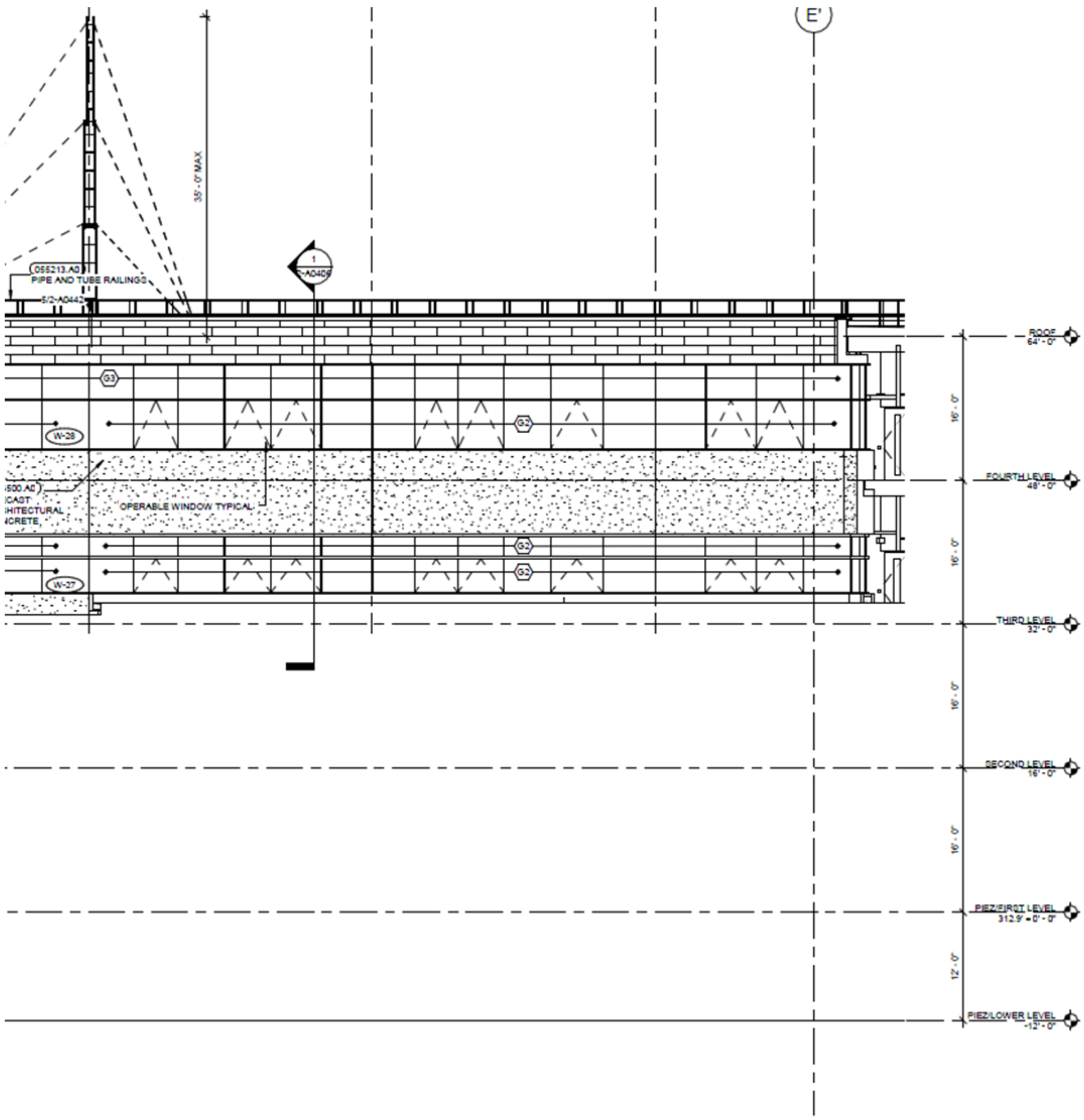


2 NORTH ELEVATION



1 PARTIAL WEST ELEVATION

SCALE: 1/8" = 1'-0"


E'
35' - 0" MAX
055213.A0
PIPE AND TUBE RAILINGS
5/2-A0442
G3
W-28
G2
OPERABLE WINDOW TYPICAL
W-27
ROOF
64' - 0"
FOURTH LEVEL
48' - 0"
THIRD LEVEL
32' - 0"
SECOND LEVEL
16' - 0"
PIEZ/FIRST LEVEL
312.9' = 0' - 0"
PIEZ/LOWER LEVEL
-12' - 0"
16' - 0"
12' - 0"