Technical Assignment 1

Piez Hall Extension

Oswego, NY

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Executive Summary

The goal of technical assignment one was to gain a better understand of the structural system of Piez hall extension. This was accomplished through careful examination of the foundation, floor, framing, lateral, and roof system of the building. Also, research and spot check calculations of Piez Hall were included in this technical report. Gravity loads were analyzed in order to perform spot checks on typical column, beam and floor slab. A typical interior column labeled D-6 on the structural drawing was checked for its compressive load carrying capacity. A typical beam labeled CB2 and a typical 31.5'x31.5' bay of the floor system were checked against deflection, shear and flexural requirements. All members were found to be adequately designed for gravity loads. However, these structural members were not checked for their lateral loads carrying capacity due to the time permitted in this technical report. A thorough check on these members for both their gravity and lateral loads carrying capacity will be performed in technical report 3.

Also, the overall weight of the building was obtained in order to calculate seismic loads. The author followed the procedure from ASCE 7-10 to obtain the wind and seismic loads of Piez Hall. Many simplifications were made throughout the process in order to reach the conclusion of this report. For instance, the geometric shape of Piez hall was modified in order to simplify the use of equivalent lateral force procedure as defined by ASCE 7-10. For the seismic loads calculation, it was found that the base shear of the building was 1067kips, which was less than 3% difference from the 1040kips listed in the structural drawings. This minor difference was probably due to the error in obtaining the area of the floor plan. In conclusion, it was determined that seismic forces control over wind forces in all directions. Although wind loads effect on component and cladding of the facade must be taken into consideration, this was not included in this technical report because of the limited amount of time. Component and cladding of the façade will be investigated in the future.

In addition, appendices that contains all hand calculations, diagrams, charts and typical structural plans were included in this technical report.

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

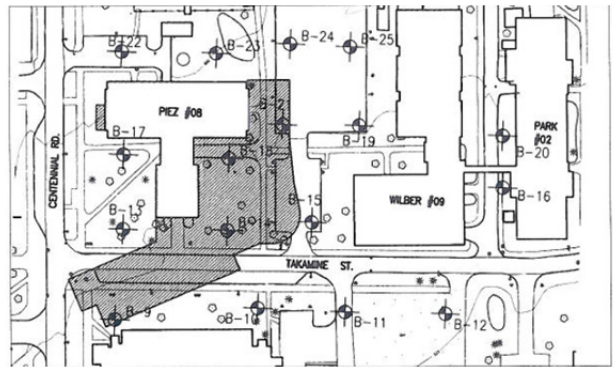


Figure 1: Site map showing existing Piez hall and the new extension (shaded area)

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.

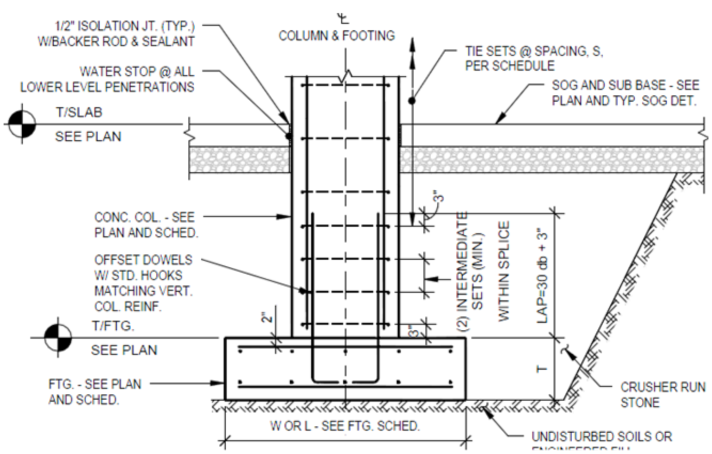


FIGURE 4: TYPICAL COLUMN FOOTING SHOWING REINFORCEMENT PLACEMENT

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 and 11.

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.

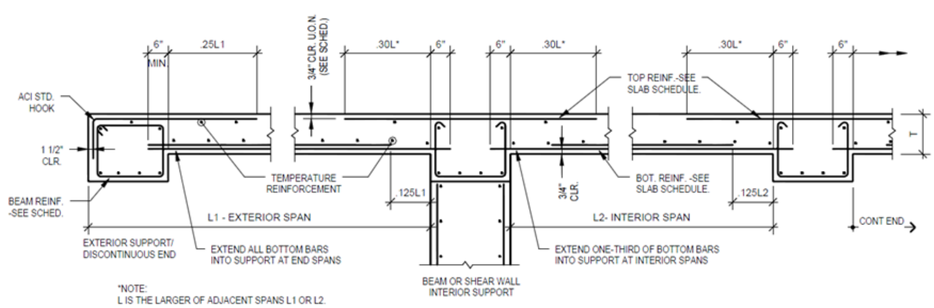


Figure 5: Typical one way slab showing reinforcement placements

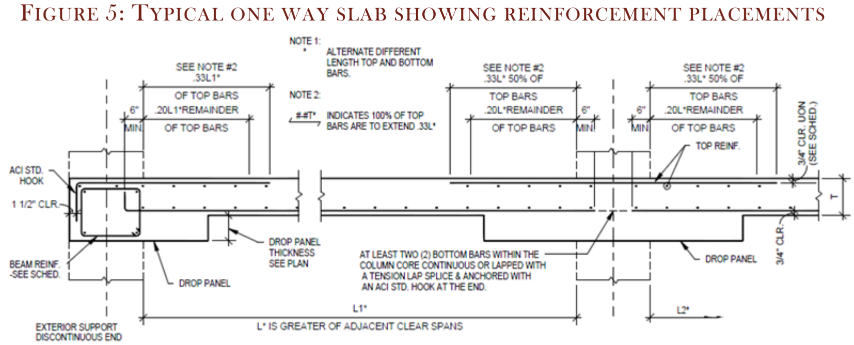


Figure 6: Typical column strip detail with drop panel and edge beam

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.

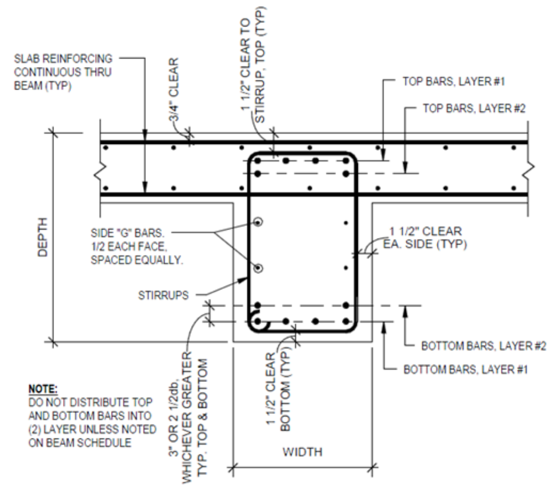


FIGURE 7: TYPICAL BEAM SECTION

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.

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.

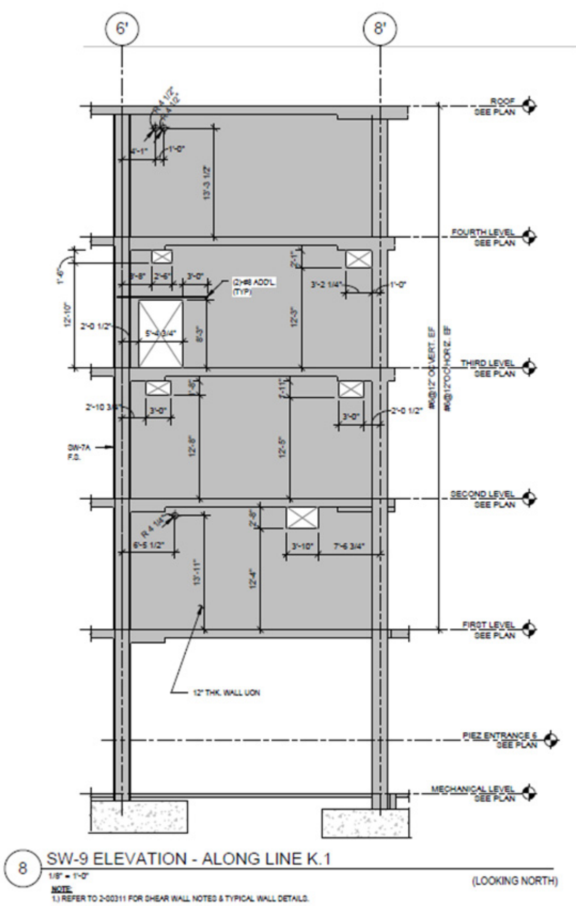


Figure 8: Typical concrete shear wall

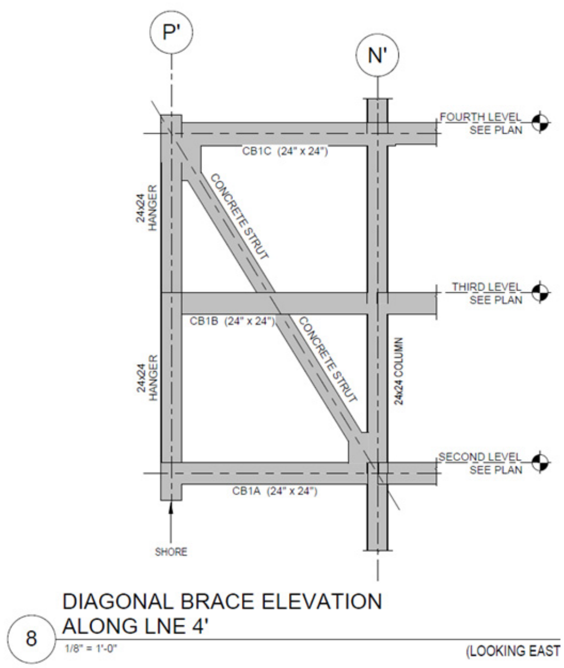


Figure 9: Typical concrete diagonal braces

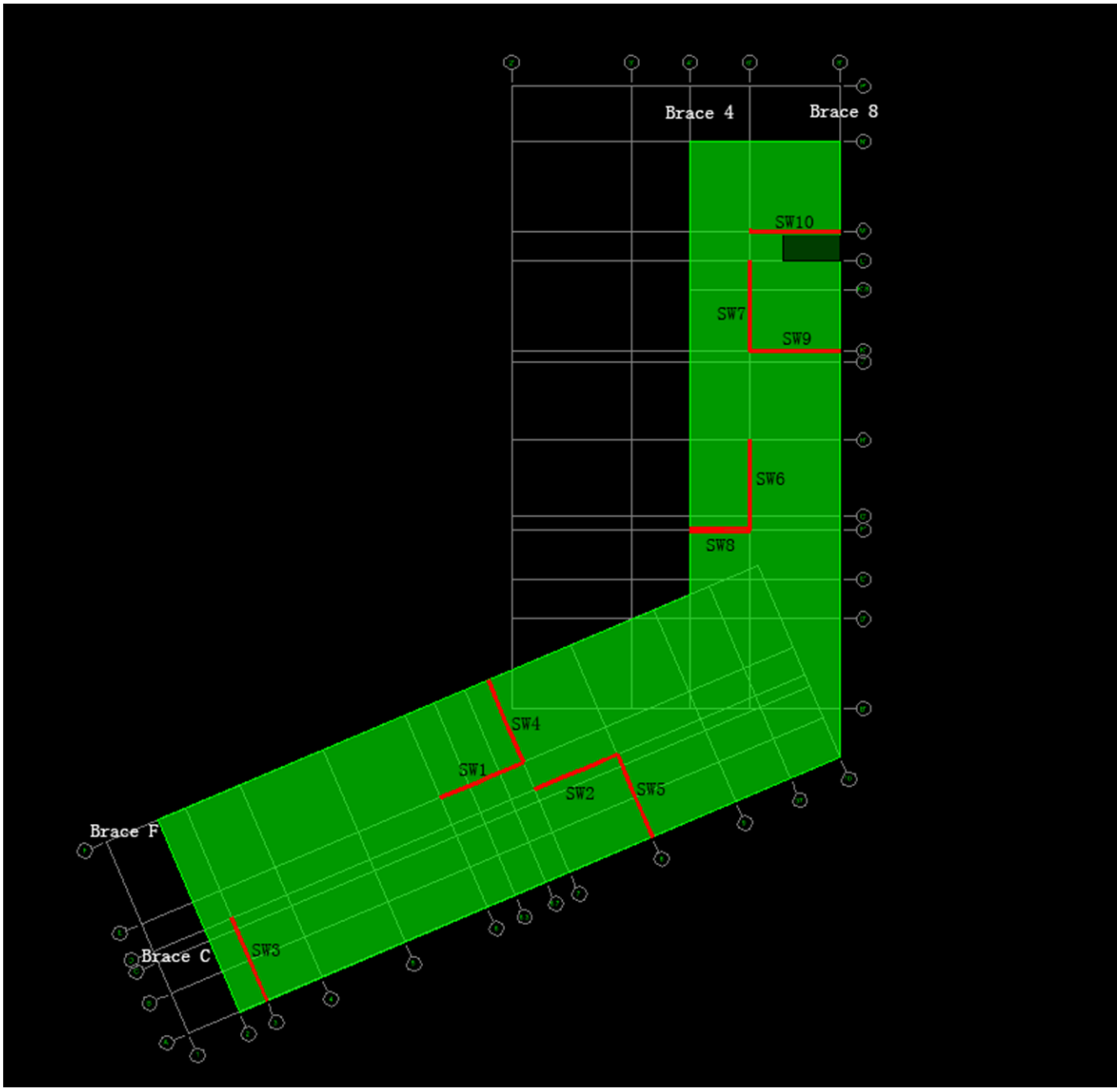


FIGURE 10: SHEAR WALL LOCATIONS OF A TYPICAL FLOOR

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.

Column Gravity Check

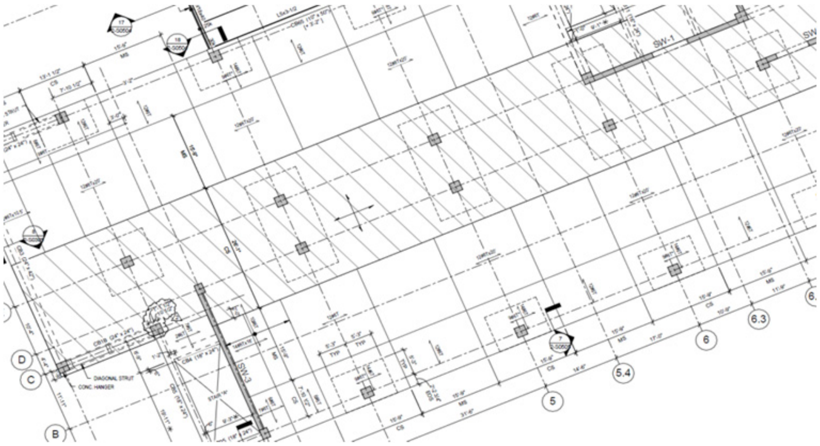


Figure 11: Typical framing plan showing beam CB2

Column D-6 was chosen to do a spot check because it was an interior column. In another words, only the compressive strength of the column was needed to check. This greatly reduced the time it will take to determine second order effects introduced by lateral forces. The column was a 24”x24” square reinforced column in a 21’x31.5’ bay with eight #8 bar reinforcement and #3 ties at 15” spacing. When calculating the gravity loads of the column, roof live load was not reduced in order to be conservative. Live loads in all other floors were 100psf and reduced accordingly. It was found that D-6 had a strength capacity way exceeded the applied gravity loads. Detailed calculation can be found in Appendix A for gravity load calculations.

Beam Gravity Check

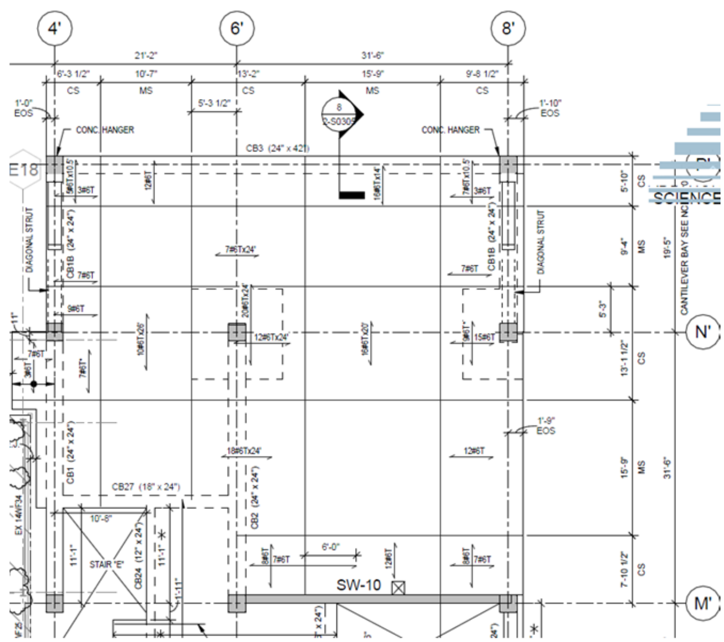


Figure 12: Typical framing plan showing column D-6

Beam CB2 spanned along line 6’ and between lines N’ and M’. This beam was a 24”x24” doubly reinforced beam with a length of 31.5’. The top reinforcements were three #6 bars, the bottom reinforcements were five #8 bars and #4 stirrups were at 10” spacing. The beam was framed into the floor slab to from a T beam with h=12”. Live load reduction was applied. The maximum design moment was determined using ACI moment coefficient from chapter 8.3. The beam was found to be adequately designed to resist both bending and shear. Also, deflection of the beam was properly checked against Table 9.5a of ACI318-11 and no issue was found. Again, detailed calculations can be found in Appendix A for gravity load calculations.

Slab Gravity Check

A spot check was performed in an exterior 31.5'x31.5' bay enclosed by line A,D,5 and 6 for a typical floor (see Figure 11). The slab was 12" thick with 5000psi strength concrete. The slab was checked against ACI 318-11 table 9.5c for minimum slab thickness. Since the adjacent clear span had a length of 10.33', it was less than 2/3 of 31.5', which means the direct design method was not allowed to use here. Thus the equivalent frame method was needed to determine the moments in the column and middle strip as shown in table 5. ACI 318-11 section 11.11 provides guidelines for punching shear failure checks. The slab was checked to be adequate for deflection.

	B1		B2			B3			B4	
	col	slab	slab	col	slab	slab	col	slab	slab	col
COF		0.508			0.507			0.508		
DF	0.398	0.602	0.209	0.138	0.652	0.652	0.138	0.209	0.602	0.398
FEM (kips-ft)	0	1040.6	-1040.6	0	112.0	-112.0	0	1040.6	-1040.6	0
B1	-414.3	-626.3	194.2	128.5	605.9	-605.9	-128.5	-194.2	626.3	414.3
C1		98.7	-318.2		-307.2	307.2		318.2	-98.7	
B2	-39.3	-59.4	130.8	86.5	408.0	-408.0	-86.5	-130.8	59.4	39.3
C2		66.5	-30.2		-206.9	206.9		30.2	-66.5	
B3	-26.5	-40.0	49.6	32.8	154.7	-154.7	-32.8	-49.6	40.0	26.5
C3		25.2	-20.3		-78.4	78.4		20.3	-25.2	
B4	-10.0	-15.2	20.7	13.7	64.4	-64.4	-13.7	-20.7	15.2	10.0
C4		10.5	-7.7		-32.7	32.7		7.7	-10.5	
B5	-4.2	-6.3	8.4	5.6	26.3	-26.3	-5.6	-8.4	6.3	4.2
C5		4.3	-3.2		-13.4	13.4		3.2	-4.3	
B6	-1.7	-2.6	3.5	2.3	10.8	-10.8	-2.3	-3.5	2.6	1.7
SUM	**-495.9**	**495.9**	**-1013.0**	**269.3**	**743.7**	**-743.7**	**-269.3**	**1013.0**	**-495.9**	**495.9**
Sum at joint	**0.0**			**0.0**			**0.0**			**0.0**

TABLE 5: MOMENT DISTRIBUTION

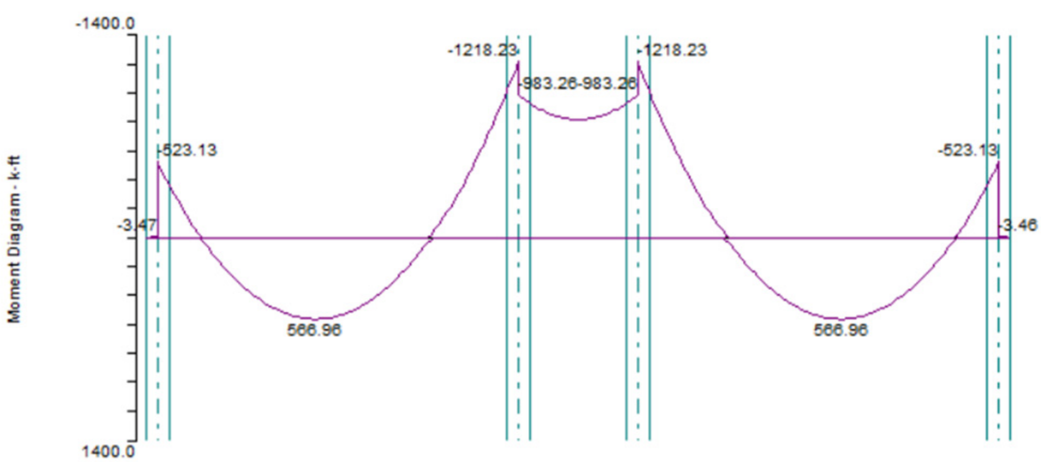


FIGURE 13: MOMENT DISTRIBUTION FROM SP SLAB

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 6. 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 14 and 15 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 B 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 6: 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 7: 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 8: WIND FORCES IN EAST-WEST DIRECTION

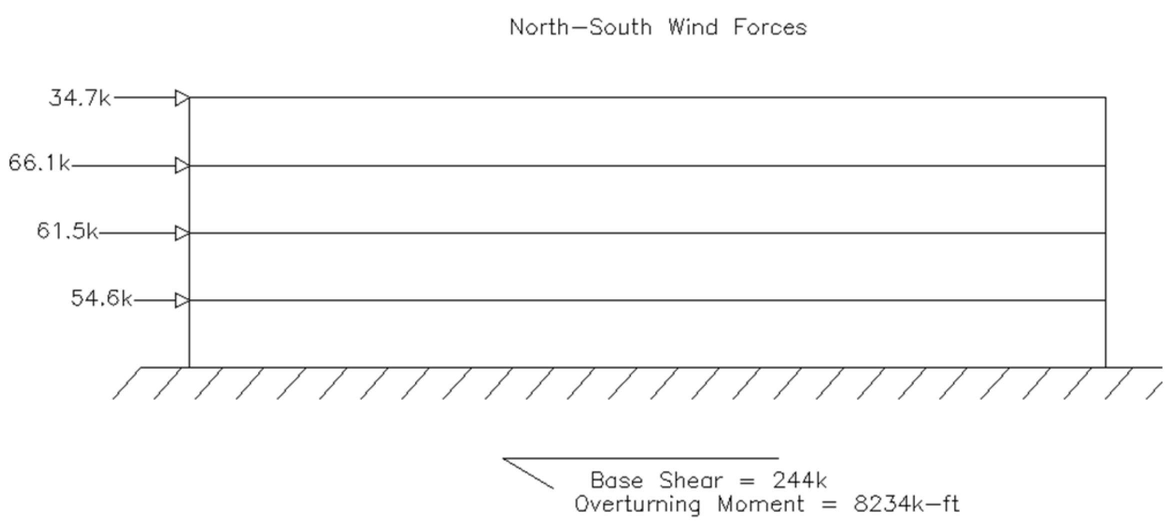


FIGURE 14: WIND FORCES DIAGRAM IN NORTH-SOUTH DIRECTION

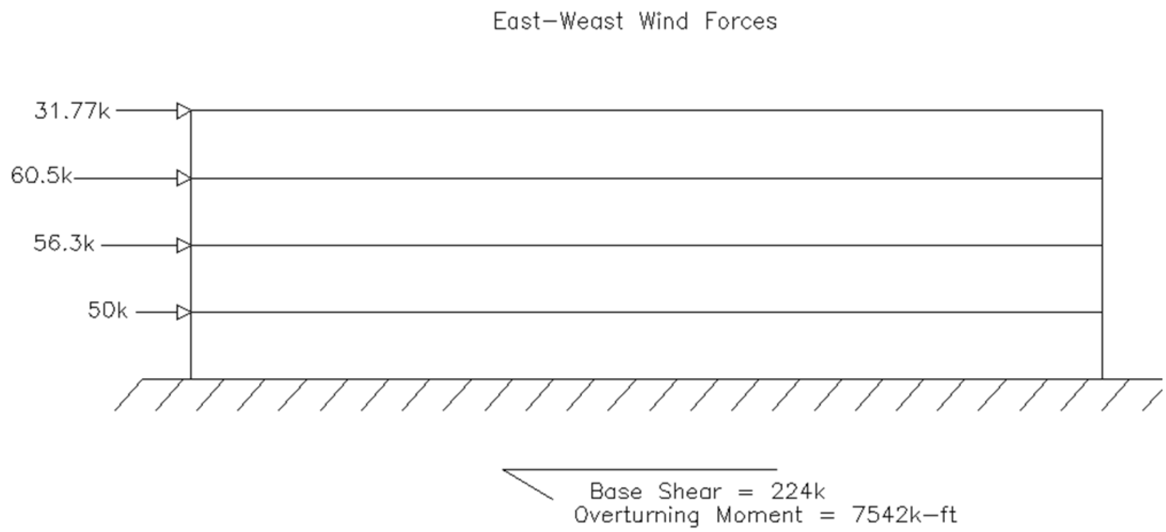


FIGURE 15: WIND FORCES DIAGRAM IN EAST-WEST DIRECTION

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 16, 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 9: SEISMIC FORCES DISTRIBUTION

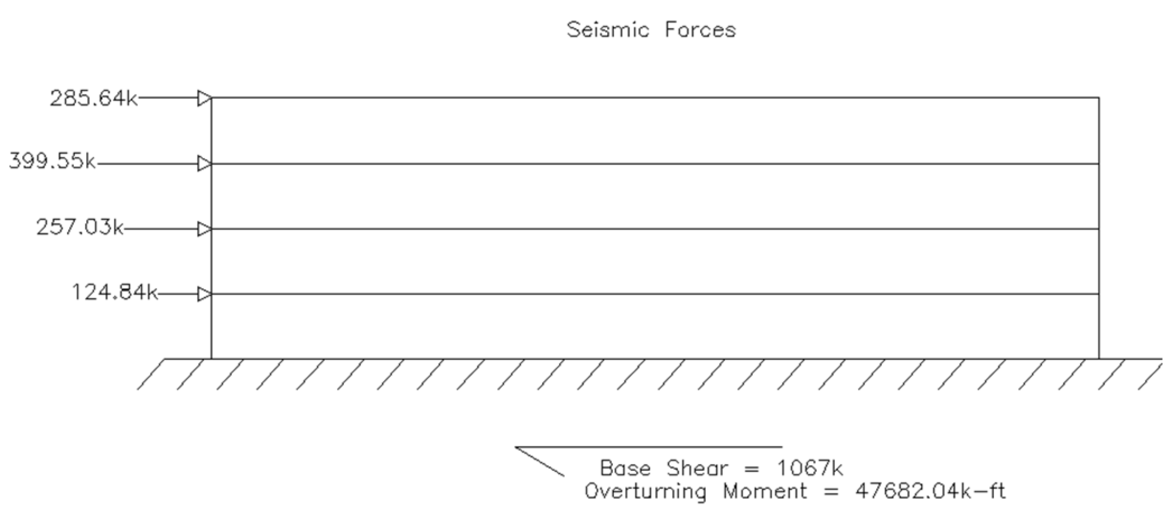


FIGURE 16: SEISMIC FORCES DIAGRAM IN EITHER DIRECTION

Conclusion

The task of technical assignment one was to analyze the existing structural condition of Piez Hall extension. By examining the component and details of the building, a better understanding of the overall structural system as a whole was gained. Through spot checks, it was determined that the building was adequate to carry all the gravity loads. According to ACI 318-11, beam and slab were found to have no problems in deflection and shear failure.

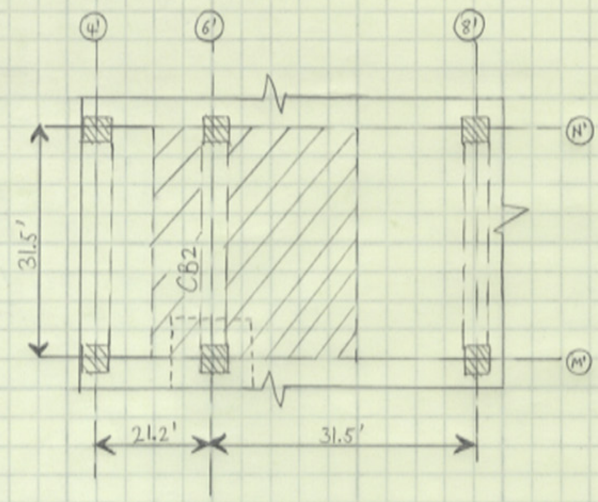
Superimposed dead loads were assumed to be 20psf in the calculation for overall weight of the building. Live loads given in the structural drawings were checked against ASCE 7-10 and the differences are explained and discussed.

Various kinds of lateral loads were also determined per ASCE 7-10 and included in this report. Wind and seismic loads were both calculated in order to obtain the base shear and overturning moment of the building produced by these loads. Throughout the process, many simplification and assumptions were made; especially the geometry of the building was modified in order to simplify the calculation of wind loads. All in all, it was determined that seismic loads will produce the greatest overturning moment and base shear in all directions. This was expected since Piez Hall was a mid-rise building with a large base. Only seismic loads needed to be taken into consideration when designing the lateral force resisting system of the building. In technical report 3, the transfer of lateral loads through the resisting system to the foundation will be examined in detail.

Appendices

Appendix A: Gravity Load Calculation

Techn #1 | Spot Check for Beam | MinGao Li | Pg 1/3



Beam CB2 is 24"x24"

$f'c = 5000\,psi$

$DL = 150\,psf$

$SID = 20\,psf$

$LL = 100\,psf$

$LC: 1.6L_L + 1.2D$

$$\text{Tributary Area} = (31.5 + 21.2)\tfrac{1}{2} \times 31.5 = 830\,ft^2$$

$$\text{Influence Area} = (21.2 + 31.5) \times 31.5 = 1660.\,ft^2$$

Live Load reduction

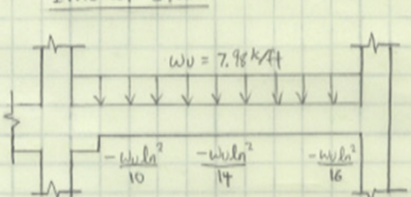
$$\min \begin{cases} 0.4 \text{ for 2 or more floors above} \\ 0.25 + \dfrac{15}{\sqrt{1660}} = 0.618 \end{cases}$$

$$\text{Live Load} = 0.618 \times 100\,psf = 61.8\,psf$$

$$W_u = \left[1.2(150+20) + 1.6(61.8)\right](21.2' + 31.5')\tfrac{1}{2}$$

$$W_u = 7.98\,k/ft$$

Interior span



Using ACI 8.3 to determine Mu

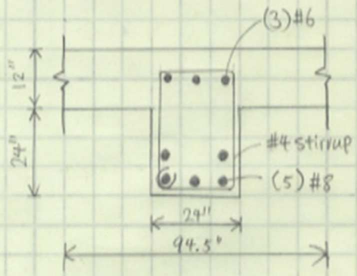
$$l_n = 31.5' - 2' = 29.5'$$

$$M_u^- = \frac{7.98 \times 29.5^2}{10} = 694.5\,k\text{-}ft$$

$$M_u^+ = \frac{7.98 \times 29.5^2}{14} = 496\,k\text{-}ft$$

Techn #1 | Spot Check for Beam | Min Gao Li | Pg 2/3

Typical Beam section : 2-S0303



$A'_s = (3).44 = 1.32\ in^2$

$A_s = (5).79 = 3.95\ in^2$

$\beta_1 = 0.8$ for $f'c = 5000$ psi

$d' = 2.5''$

$d = 36 - 2.5 = 33.5''$

$$b_{eff} = \min\left|\begin{array}{l} b_w + 16h_f \\ \text{Tributary width} \\ \tfrac{1}{4}\text{ span length} \end{array}\right. \Rightarrow \min\left|\begin{array}{l} 24'' + 16\times 12'' \\ (21.2+31.5)\tfrac{1}{2}\times 12 \\ \tfrac{1}{4}\times 31.5'\times 12 \end{array}\right. \Rightarrow \min\left|\begin{array}{l} 216'' \\ 316.2'' \\ 94.5'' \Rightarrow \text{Control} \end{array}\right.$$

Assume Case 2 ($\varepsilon_s < \varepsilon_y$ & $a < h_f$)

Solve c from T=C

$$0.85f'c(\beta_1 c)b_{eff} + A'_s\left(\frac{0.003}{c}\right)(c-d')E_s = A_s f_y$$

Simplifies to $(0.85f'c\, b_{eff}\, \beta_1)c^2 + (0.003A'_sE_s - A_sf_y)c - 0.003A'_s d' E_s = 0$

$\Rightarrow 321.3c^2 - 122.16c - 287.1 = 0$

$c = 1.154''$

$a = \beta_1 \times c = 0.9232'' < h_f = 12'' \Rightarrow ok$

$\varepsilon'_s = \frac{0.003}{c}(c-d') = \frac{0.003}{1.154}(1.154-2.5) = -0.0035 < \varepsilon_y = 0.00207$
$\Rightarrow ok$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{1.154}(33.5-1.154) = 0.084 > 0.005$

$\phi = 0.9$

Techn #1	Spot Check for Beam	MinGao Li	Pg 3/3

$$\phi M_n = 0.9\left[A's f_y (d-d') + 0.85 f'c\, a \times b_{eff}\left(d - \frac{a}{2}\right)\right]$$

$$= 0.9\left[1.32(60)(33.5-2.5) + 0.85(5)(0.9232)(94.5)\left(33.5 - \frac{.9232}{2}\right)\right]$$

$$= 13234.7^{k\text{-}in}$$

$$= 1102.9^{k\text{-}ft} > M_u^- = 694.5^{k\text{-}ft} \Rightarrow ok$$

Check beam shear reinforcement

$$V_u = \frac{W_u \times l_n}{2} = \frac{7.98 \times 29.5}{2} = 117.7 \text{ kips}$$

$$\phi V_n = \phi V_c + \phi V_s$$

$$= (2\lambda\sqrt{f'c}\, b_w \cdot d + A_v f_{yt} d/s)\, 0.75$$

The Structural Engineer has provided #4 stirrup @ 10" O.C

$$\phi V_n = \left[2(\sqrt{5000})(24)(33.5) + (0.2)(60000)(33.5)/10\right] 0.75/1000$$

$\phi V_n = 115.4^{kips} \approx V_u = 117.7^{kips} \rightarrow$ (with only 2% difference)

Notes: V_u is taken as "d" distance away from the face of support. Therefore, $V_u = 117.7^k$ is an overestimated Load.

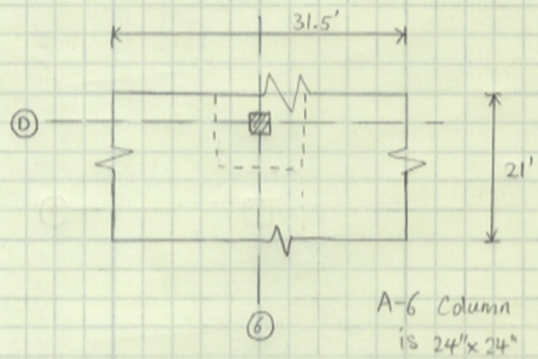
Check deflection

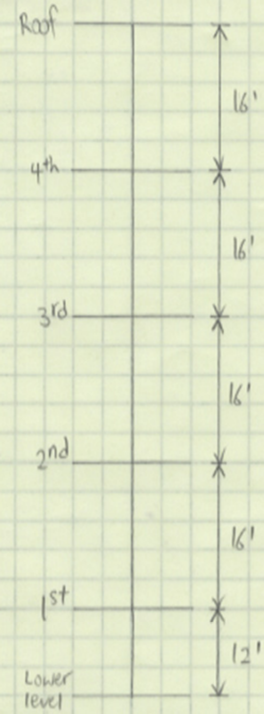
from ACI Table 9.5(a)

minimum $h \geq l/18.5$ for one end continuous beam

$$l/18.5 = \frac{31.5}{18.5} \times 12 = 20.43''$$

$h = 24'' > 20.43'' \rightarrow$ deflection is ok.





A-6 Column
is 24"x 24"
f'c = 5000 psi

Tributary Area = 31.5' x 21'
= 659 ft^2

Influence Area = 31.5' x (31.5 + 10.33)' x 2
= 2635.3 ft^2

Roof: LL = 100 psf (unpredictable)
DL = (150 pcf x 6.5/12) = 81.25 psf
SID = 20 psf
Snow = 46 psf

Floor: LL = 100 psf
DL = (150 x 12/12) = 150 psf
SID = 20 psf

Use LRFD Load Combination: 1.2 Dead + 1.6 Live + ½ Snow

Techn #1 | Spot check for Column | MinGaoLi | Pg 2/3

Live Load Reduction: $100 \times \max \begin{cases} 0.4 & \text{for two or more floors above} \\ 0.25 + \dfrac{15}{\sqrt{2635.3}} = 0.542 \end{cases}$

Live load: $100 \times 0.542 = 54.2$ psf

Roof P_u

$$P_u = \Big[1.2(\overset{\text{DL}}{81.25} + \overset{\text{SID}}{20}) + \tfrac{1}{2}(\overset{\text{Snow}}{46}) + 1.6(\underset{\text{NO reduction b/c it is unpredictable}}{100})\Big]\overset{\text{trib Area}}{659 \text{ ft}^2}$$

$P_u = 200.7^k$

Floor P_u

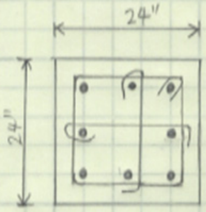
$$P_u = \Big[1.2(150+20) + 1.6(54.2)\Big] 659 \text{ ft}^2 \times \overset{\text{4 flrs}}{4}$$

$= 766.3^k$

$$\text{Total } P_u = 200.7 + 766.3 + \overbrace{.15\text{kcf} \times 2' \times 2' \times (4 \times 16 + 13/2)}^{\text{self weight}}$$

$P_u = 1009$ kip

Check Column Reinforcement



Details from 2-S0302

(8) #8 bars

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

$\phi P_o = \phi(0.85 f'_c A_c + A_s f_y)$

$= 0.65\left[0.85(5)(24\times 24 - 6.32) + 6.32\times 60\right]$

$= 1820.2^k$

$\phi P_n = \alpha\phi P_o = 0.8 \times 1820.2^k = 1456.2^k$

$\phi P_n = 1456.2^k > P_u = 1009\ kips \Rightarrow Ok$

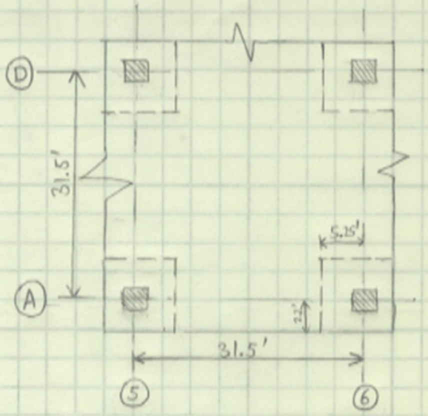
Minimum reinforcement

$\rho = \frac{A_s f_y}{bd} = \frac{6.32}{24\times 24} = 0.011 > 0.01 \Rightarrow ok$

Column is okay for Compression. & it is no need to check bending b/c it is an interior Column.

Techn #1	Spot Check for Slab	MinGao Li	Pg 1/3

Typical bay @ 1 to 4 level



Column size are 24" x 24"

Live Load: 100 psf

$F_y = 60ksi$

$f'c = 5000psi$

$l_n = 31.5' - 2' = 29.5'$

ACI 318-11

Table 9.5 (c) for drop panels without edge beams & interior beams.

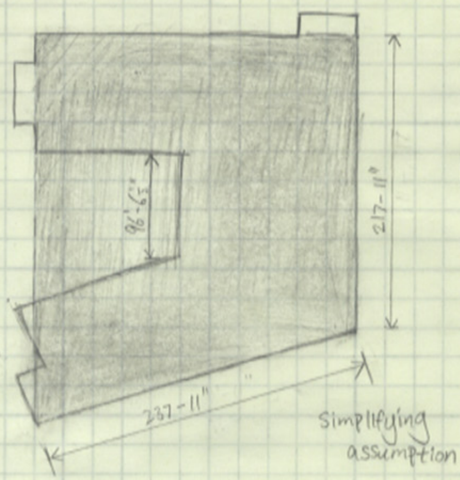
$$l_n/33 = \frac{29.5}{33} \times 12 = 10.73''$$

The structural engineer used a slab thickness of 12"

12" > 10.73" ⇒ okay for deflection.

Appendix B: Wind Load Calculation

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

Techn #1 | Wind Load Calculation | Min Gao Li | Pg 2/5

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 $> 2h$

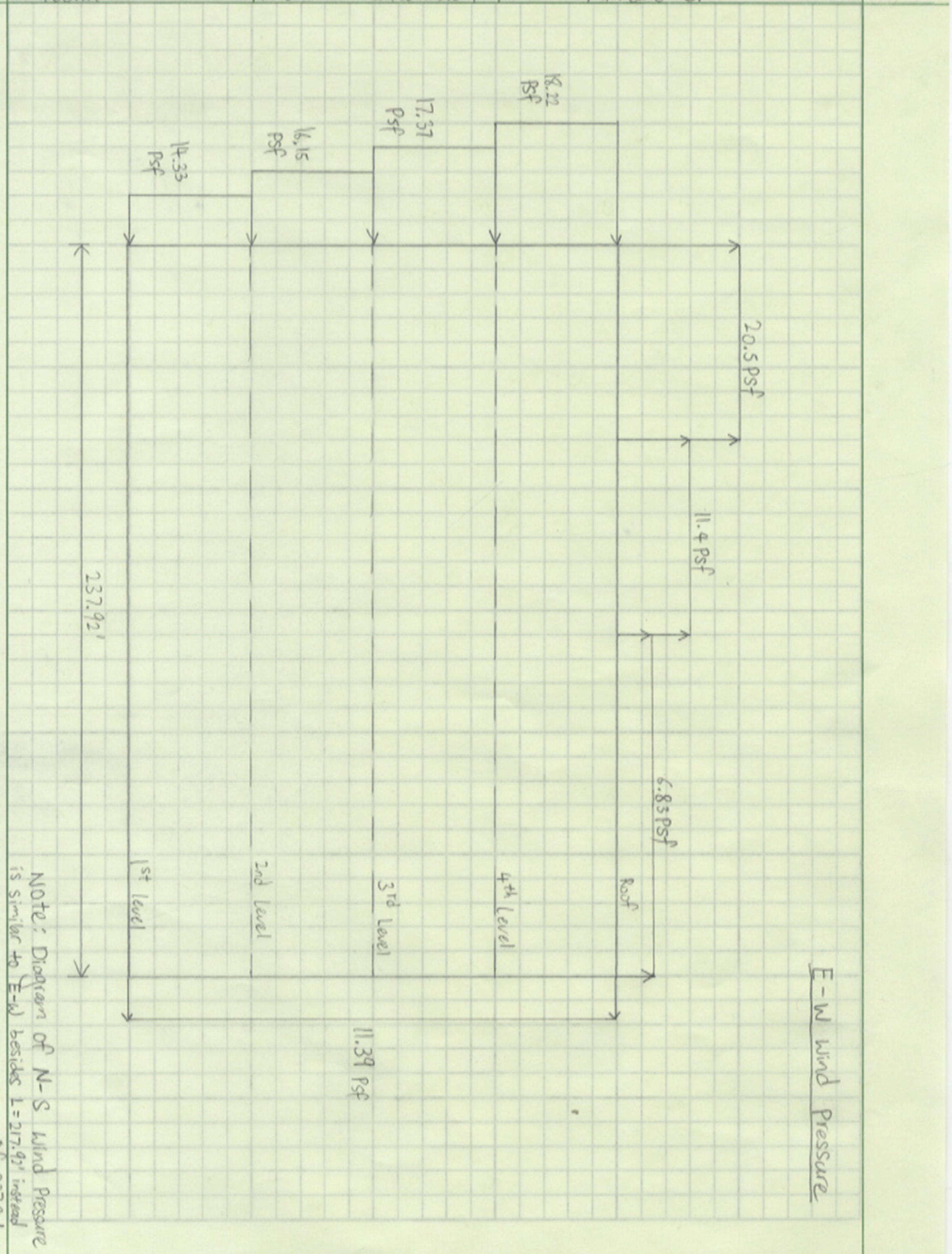
Design Wind Pressures

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

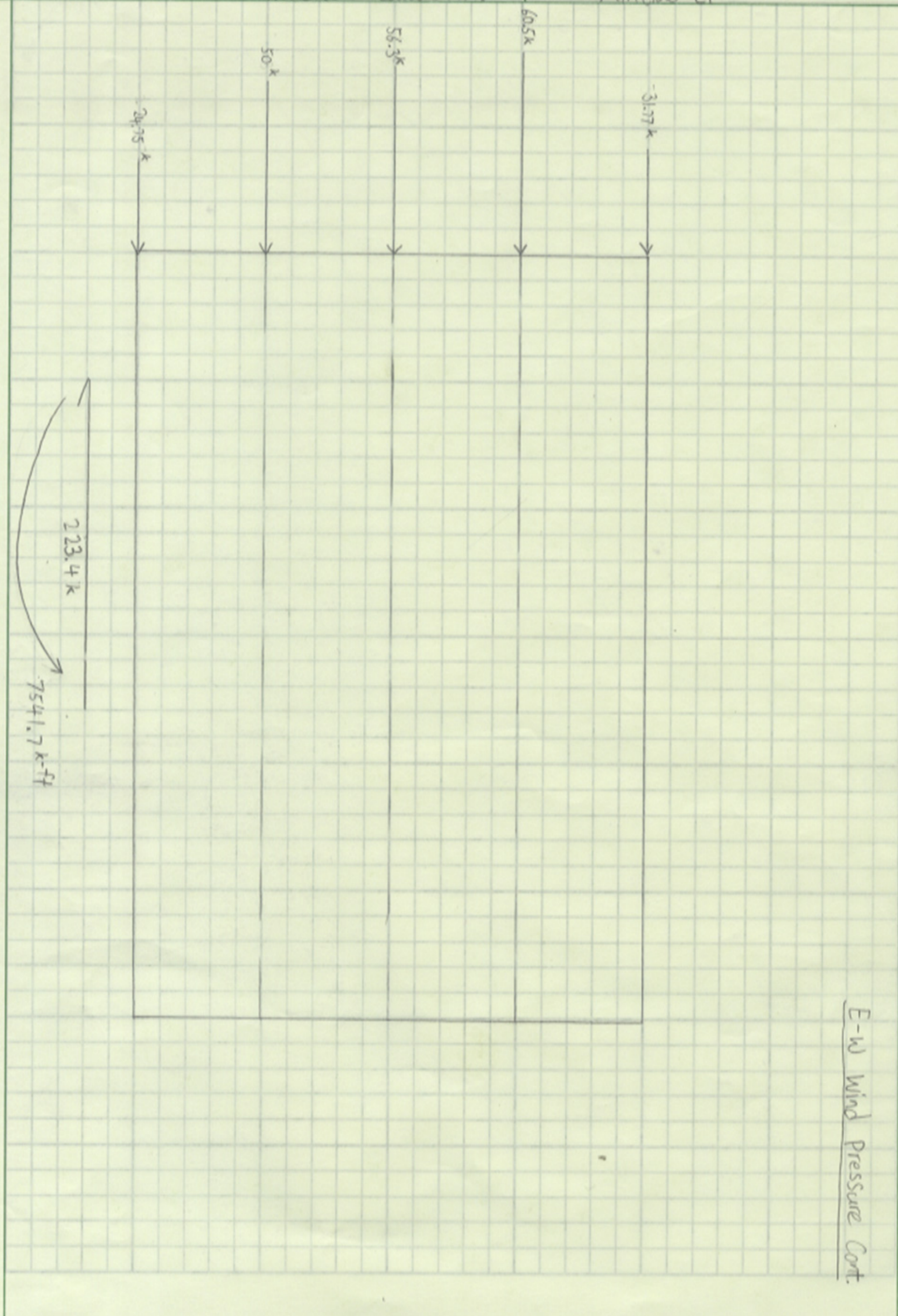
Leeward = sideward = Roof

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

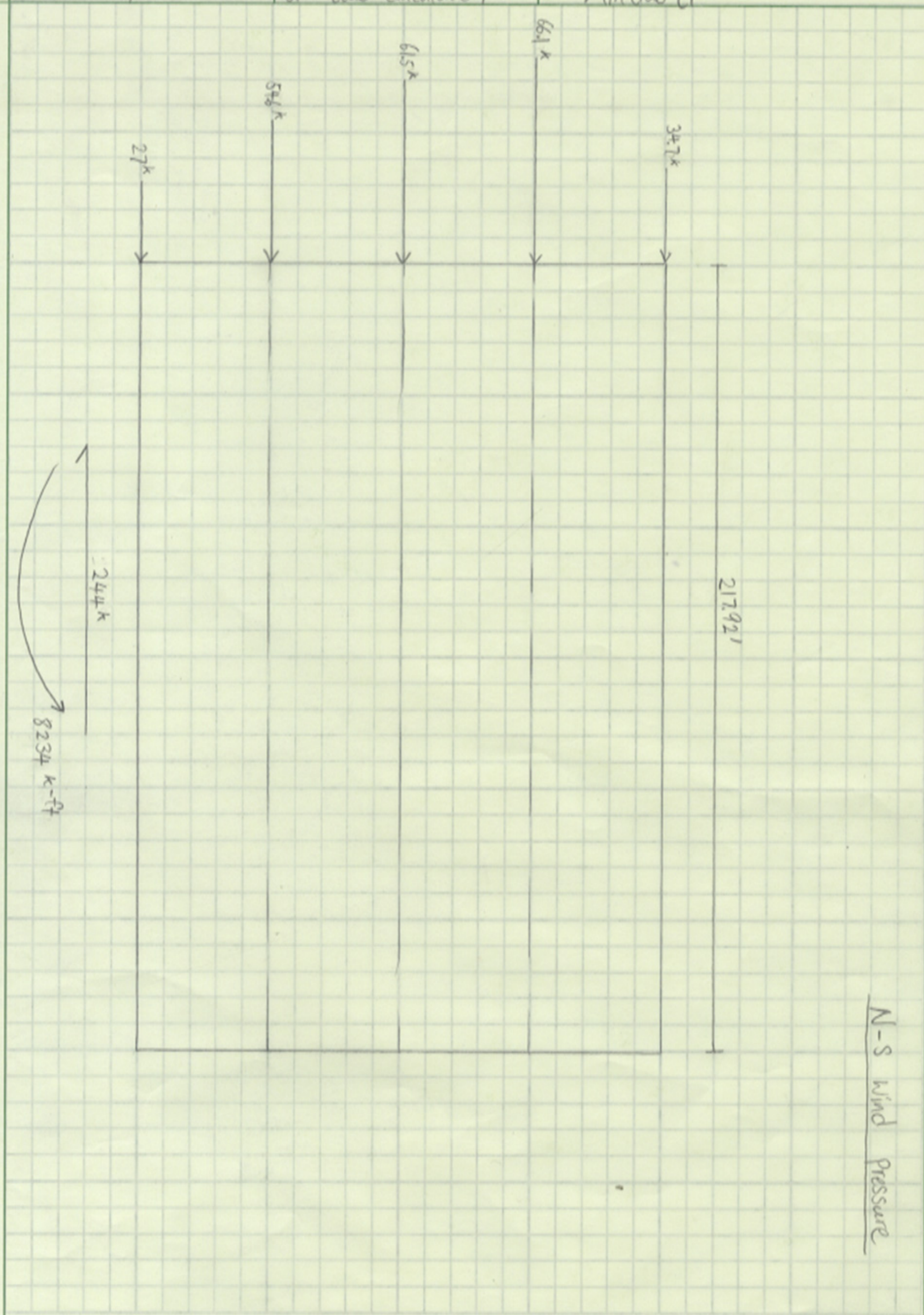
Techn #1 | Wind Load Calculation | Min Gao Li | Pg 3/5



Techn #1 | Wind Load Calculation | MinGao Li | Pg 4/5



Techn #1 | Wind Load Calculation | Min Gao LI | Pg 5/5



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 C: Seismic Load Calculation

Techn #1 | Seismic Load Cals | MinGao Li | Pg1

Analysis Procedure: Equivalent Lateral Force procedure (per ASCE7)

Seismic importance factor, $I = 1.25$

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

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

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

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

$S_{DS} = \frac{2}{3} S_{ms} = 0.1552$

$S_{D1} = \frac{2}{3} S_{m1} = 0.0884$

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 $\Rightarrow T_L = 6$ sec

For Ordinary Concrete Shear wall system

$R = 5$ (Table 12.2-1)

$C_s = \frac{S_{DS}}{R/I} = \frac{0.1552}{(5/1.25)} = 0.0388$

For $T = 0.676$ sec $< T_L = 6$ sec

C_s should be $< \frac{S_{D1}}{(R/I)T} = \frac{0.0884}{(5/1.25) \times 0.676} = 0.033$

OK since S_{D1} reduced by $\frac{2}{3}$

C_s should be > 0.01

use $C_s = 0.0388$

Seismic and weight of entire building

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

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	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

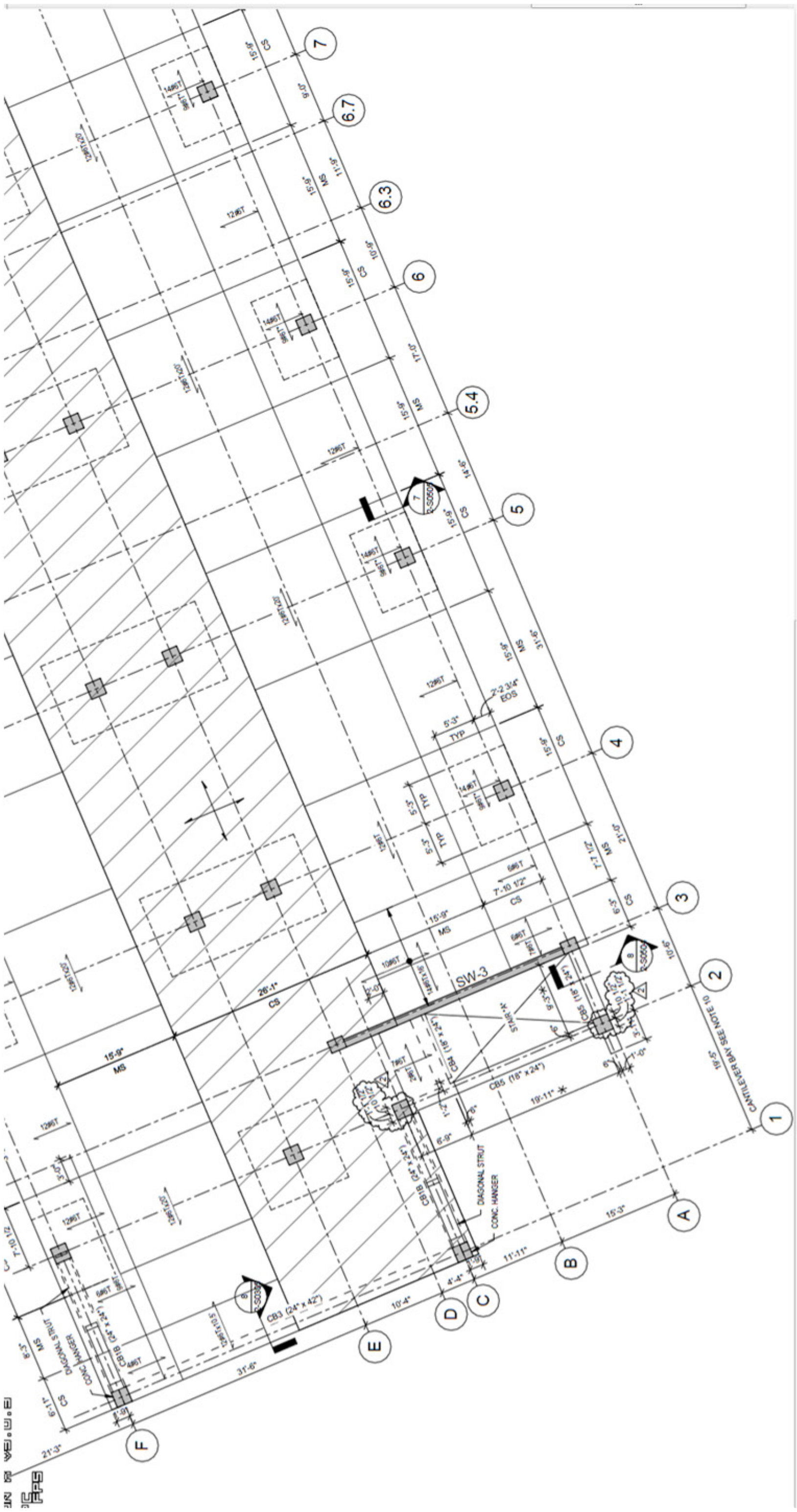
Column Weight						
Level	Numer of column	Width or Dia (ft)	Depth (ft)	Tributary Height (ft)	Volume (ft^3)	Weight (kips)
1.00	62.00	2.00	2.00	8.00	1984.00	297.60
2.00	60.00	2.00	2.00	16.00	3840.00	576.00
3.00	58.00	2.00	2.00	16.00	3712.00	556.80
4.00	58.00	2.00	2.00	16.00	3712.00	556.80
Roof	58.00	2.00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

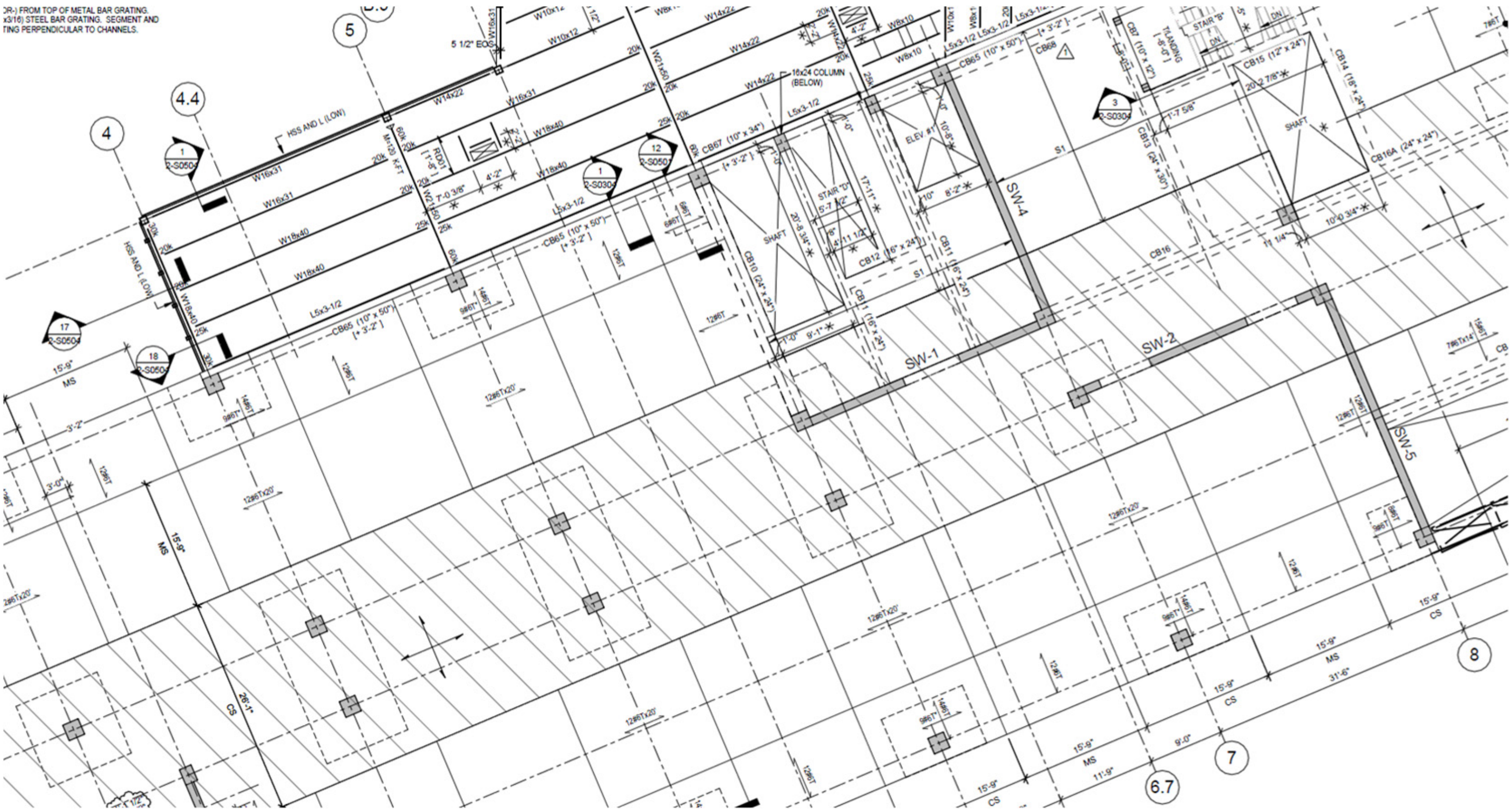
Slab Weight			
Level	Floor Area (ft^2)	Slab Thickness (in)	Weight (kips)
1.00	33964.80	12.00	5094.72
2.00	33964.80	12.00	5094.72
3.00	33964.80	12.00	5094.72
4.00	18631.20	12.00	2794.68
Roof	33964.80	6.00	2547.36

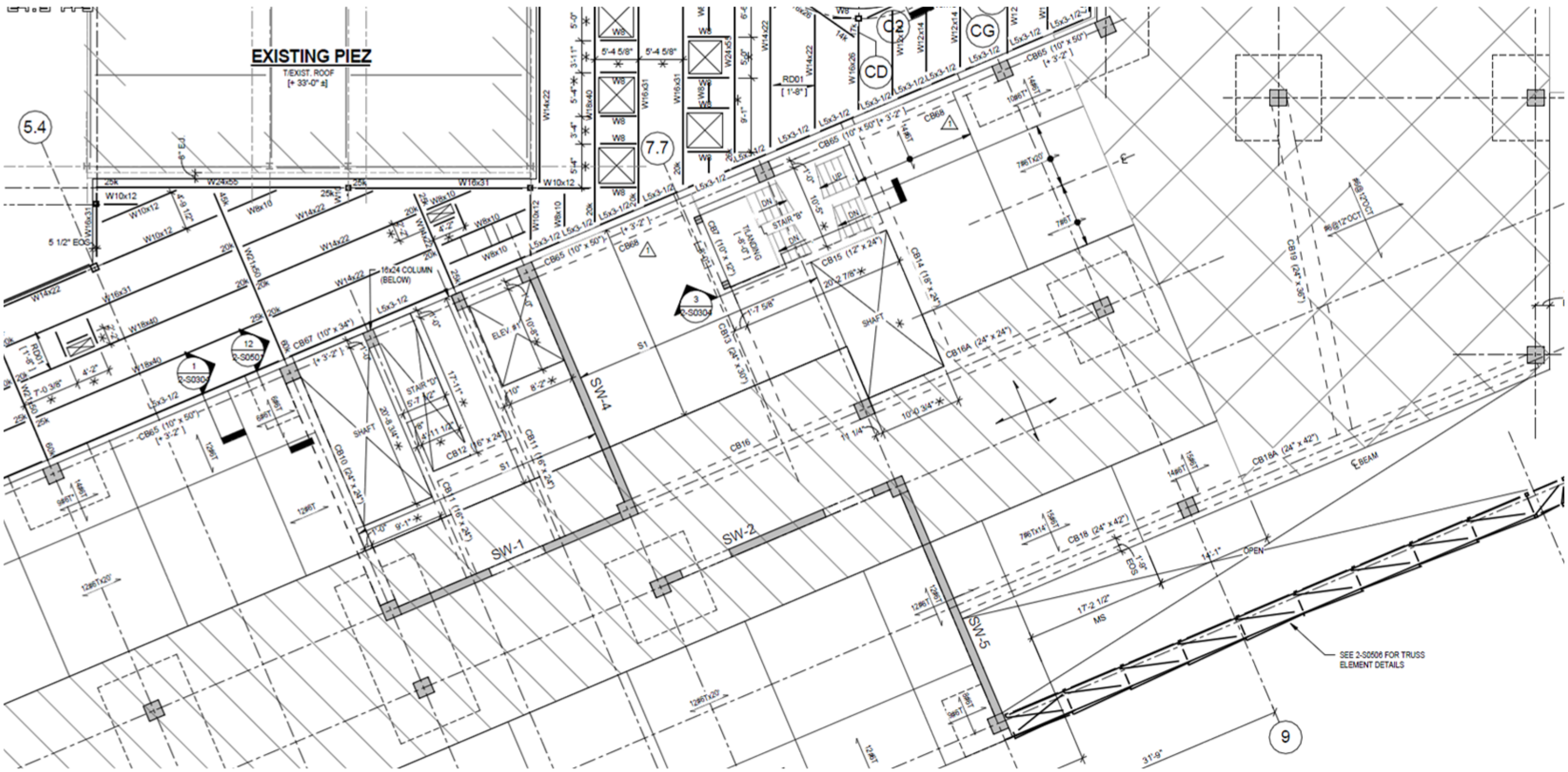
Total Weight per Level	
Level	Weight (kips)
1.00	6535.25
2.00	7276.69
3.00	7257.49
4.00	4538.89
Roof	3968.69
Total Weight	29577.02
V	1147.59
V	1040

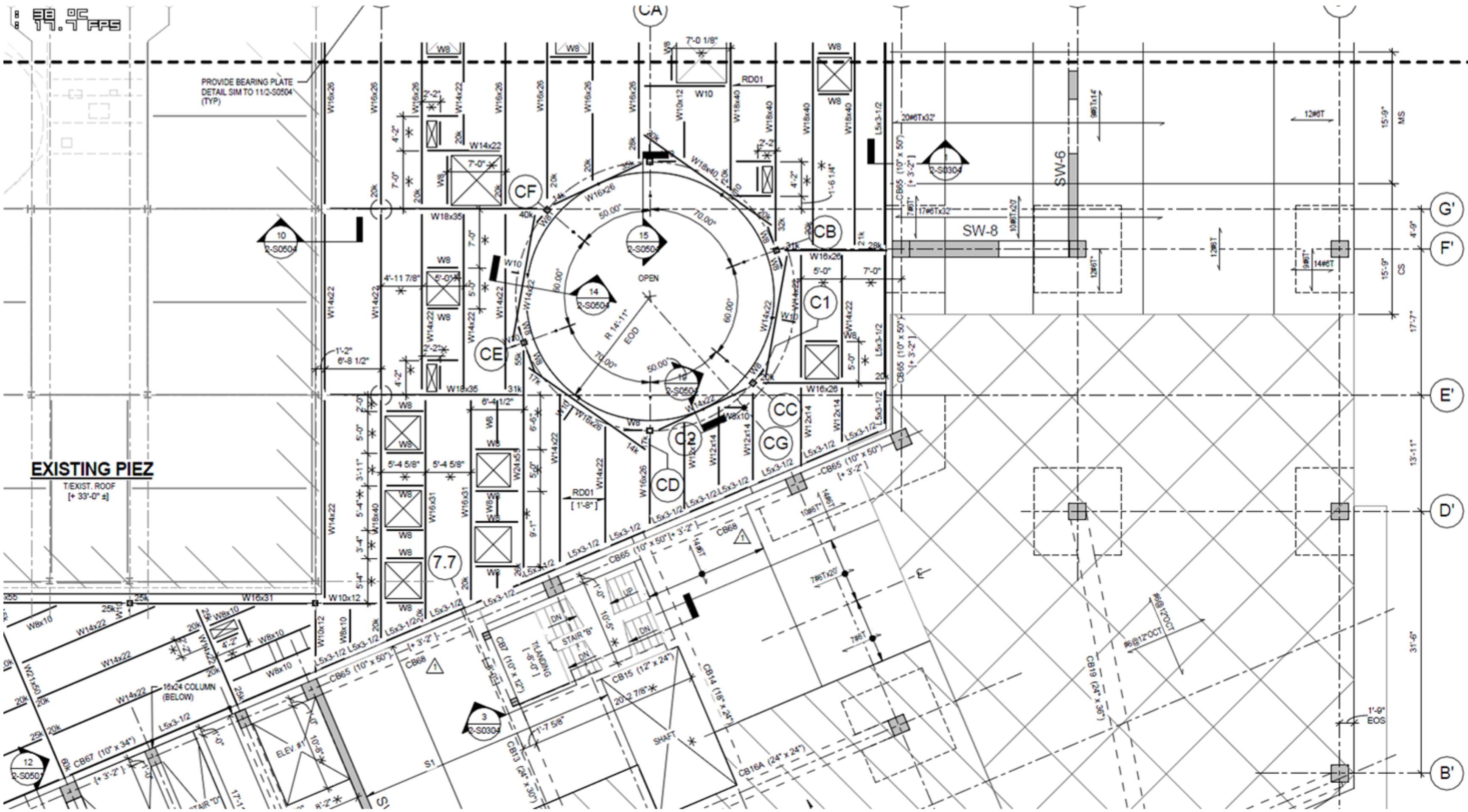
Appendix D: Typical Floor Plans

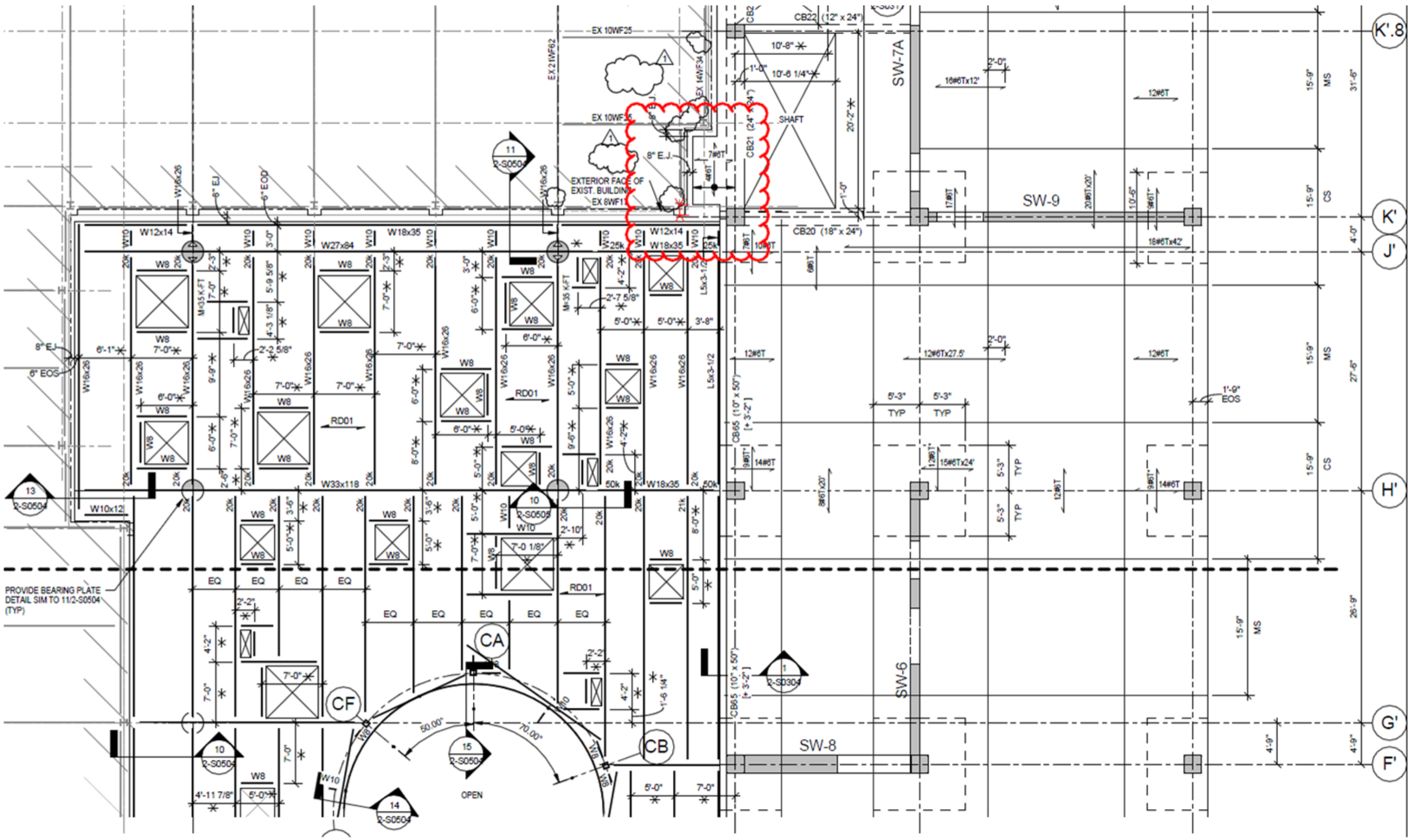
← N

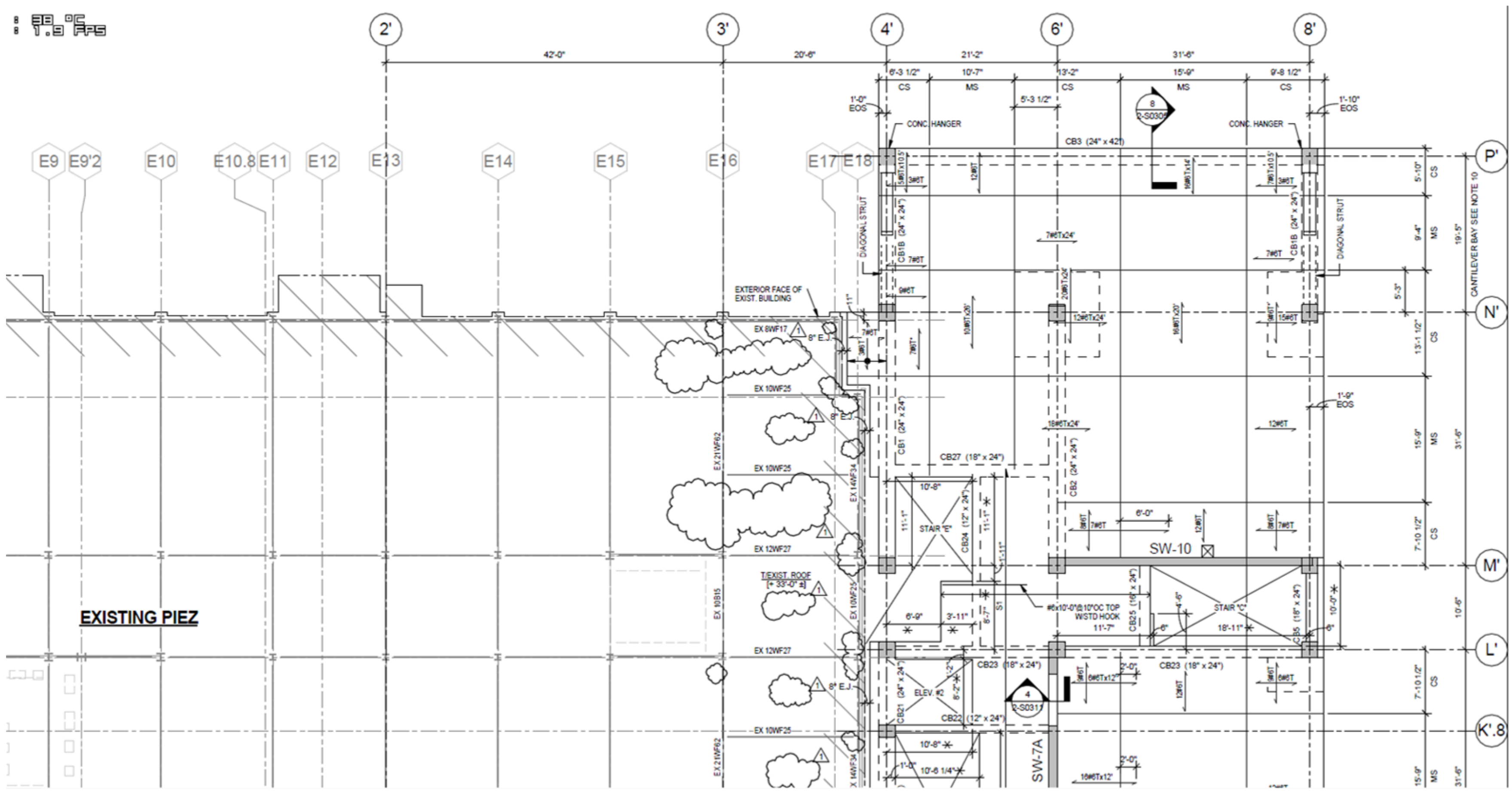


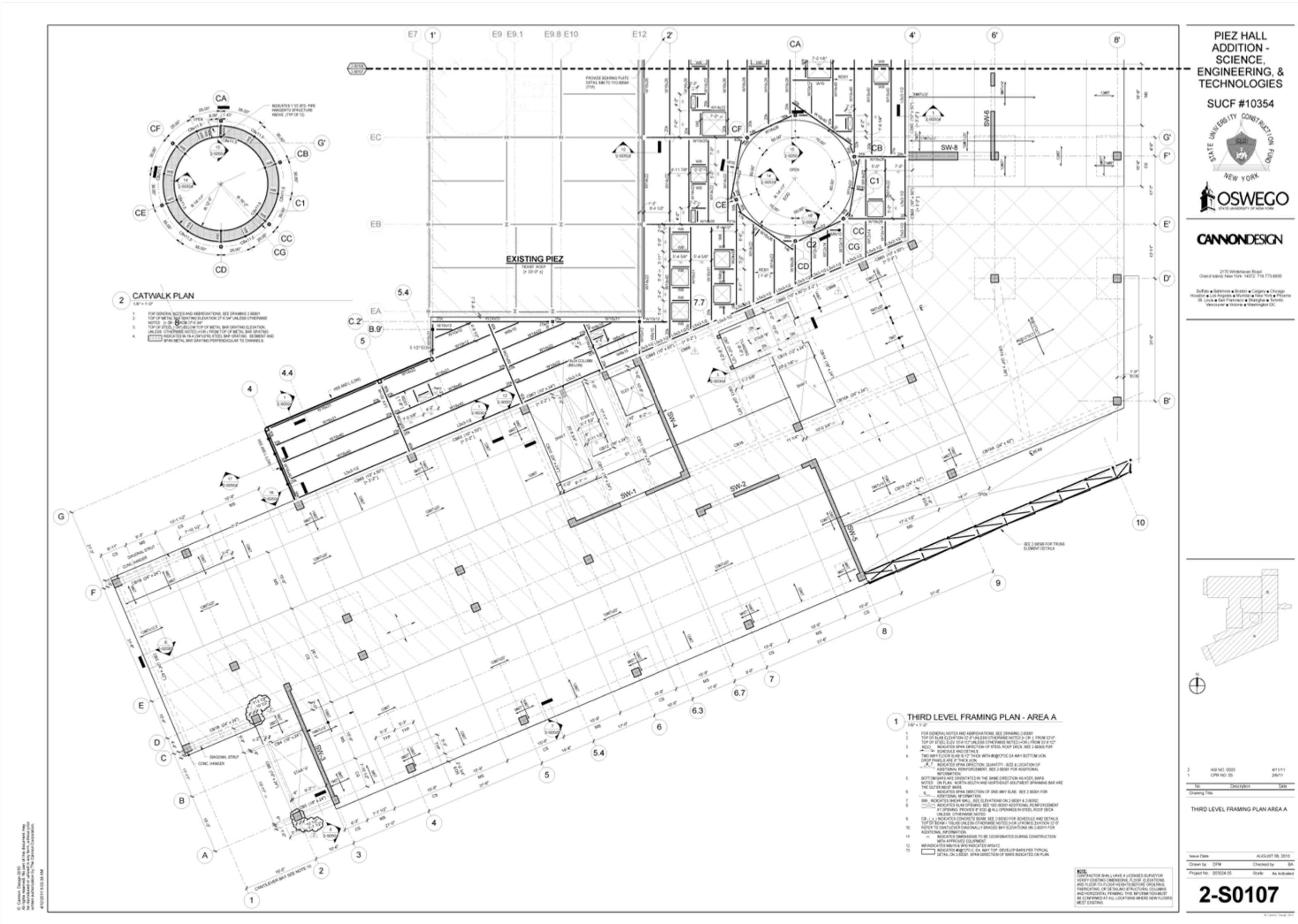

SW-1
SW-2
SW-4
SW-5
SHAFT
ELEV #1
STAIR "D"
STAIR "B"
CB10 (24" x 24")
CB11 (16" x 24")
CB12 (16" x 24")
CB13 (24" x 30")
CB14 (16" x 24")
CB15 (12" x 24")
CB16A (24" x 24")
CB65 (10" x 50")
CB67 (10" x 34")
16x24 COLUMN (BELOW)
HSS AND L (LOW)
4
4.4
5
6.7
7
8

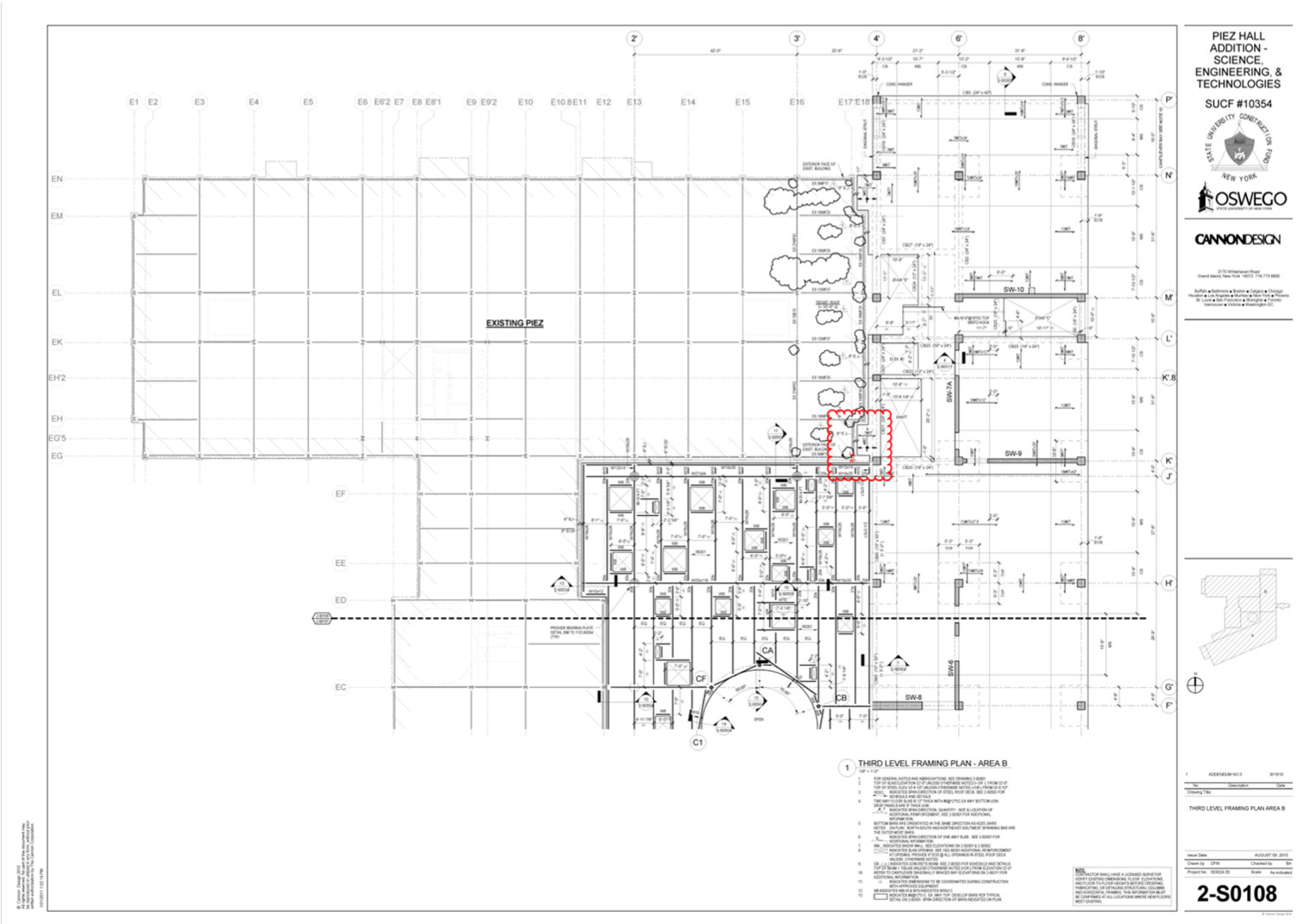

EXISTING PIEZ
T/EXIST. ROOF
5.4
7.7
CD
CG
SW-1
SW-2
SW-4
SW-5
SEE 2-S0506 FOR TRUSS ELEMENT DETAILS
9


EXISTING PIEZ
T/EXIST. ROOF
[+ 33'-0" ±]
PROVIDE BEARING PLATE DETAIL SIM TO 11/2-S0504 (TYP)
OPEN
SW-6
SW-8
STAIR "B"
SHAFT
ELEV #1
16x24 COLUMN (BELOW)

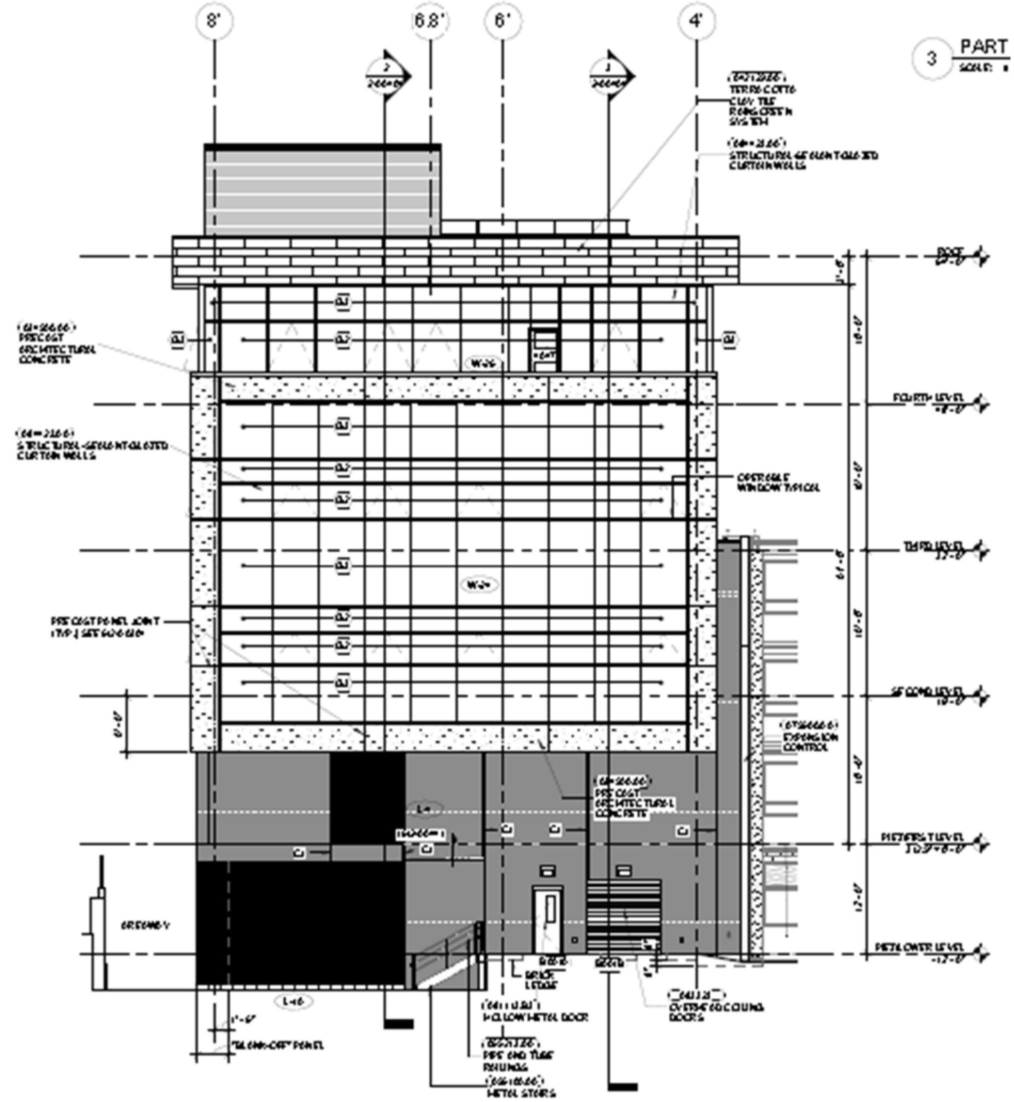

K'.8
K'
J'
H'
G'
F'
SW-7A
SW-9
SW-6
SW-8
SHAFT
CA
CB
CF
OPEN
EXTERIOR FACE OF EXIST. BUILDING
PROVIDE BEARING PLATE DETAIL SIM TO 11/2-S0504 (TYP)


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")
STAIR "E"
STAIR "C"
SW-10
SW-7A
ELEV. #2
CB23 (18" x 24")
CB21 (24" x 24")
CB22 (12" x 24")
CB24 (12" x 24")
CB25 (18" x 24")
CB2 (24" x 24")
CB1 (24" x 24")
CANTILEVER BAY SEE NOTE 10
EXISTING PIEZ
T/EXIST. ROOF

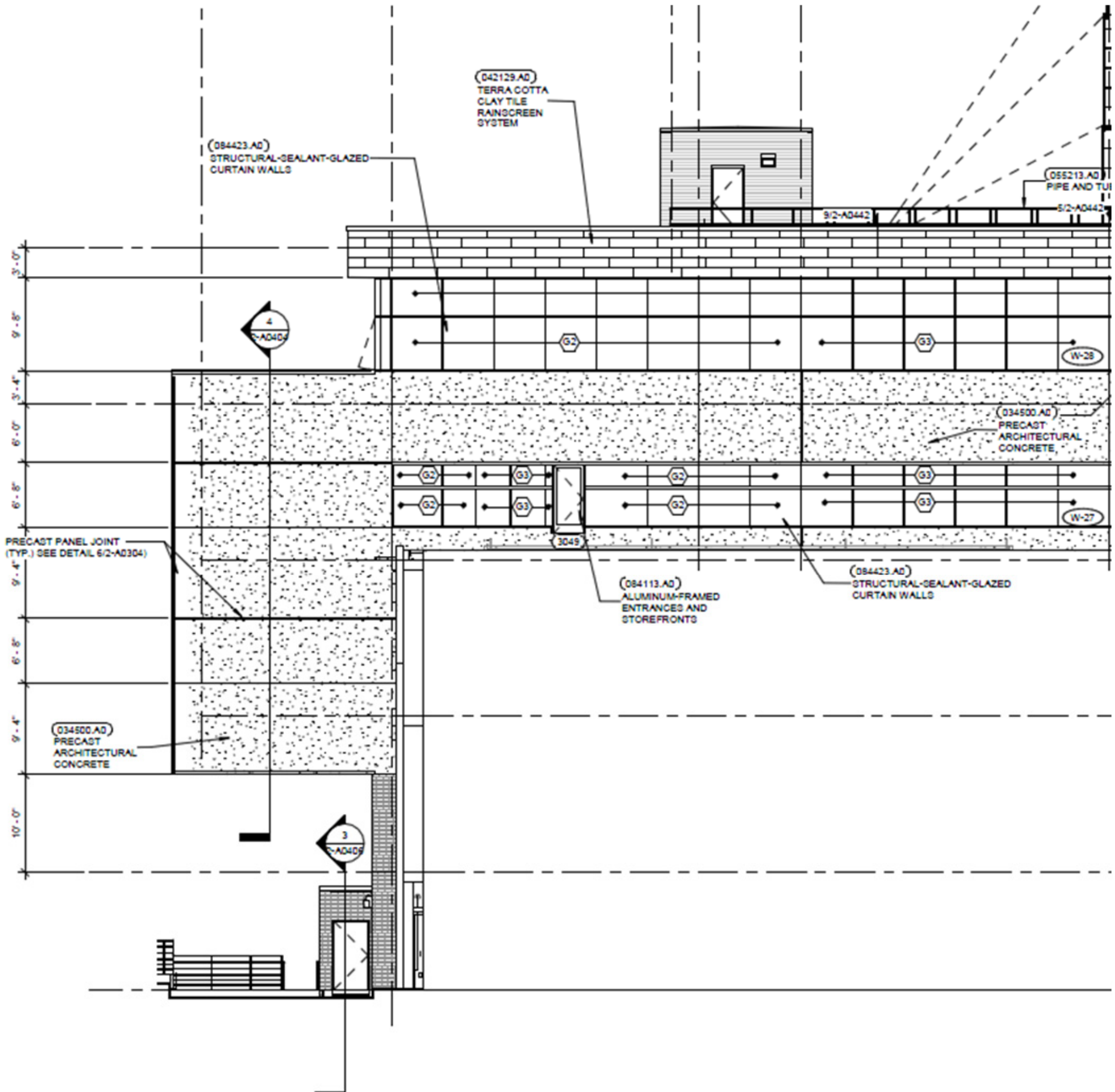

PIEZ HALL ADDITION - SCIENCE, ENGINEERING, & TECHNOLOGIES
SUCF #10354
STATE UNIVERSITY CONSTRUCTION FUND
NEW YORK
OSWEGO
CANNONDESIGN
EXISTING PIEZ
CATWALK PLAN
THIRD LEVEL FRAMING PLAN - AREA A
THIRD LEVEL FRAMING PLAN AREA A
2-S0107



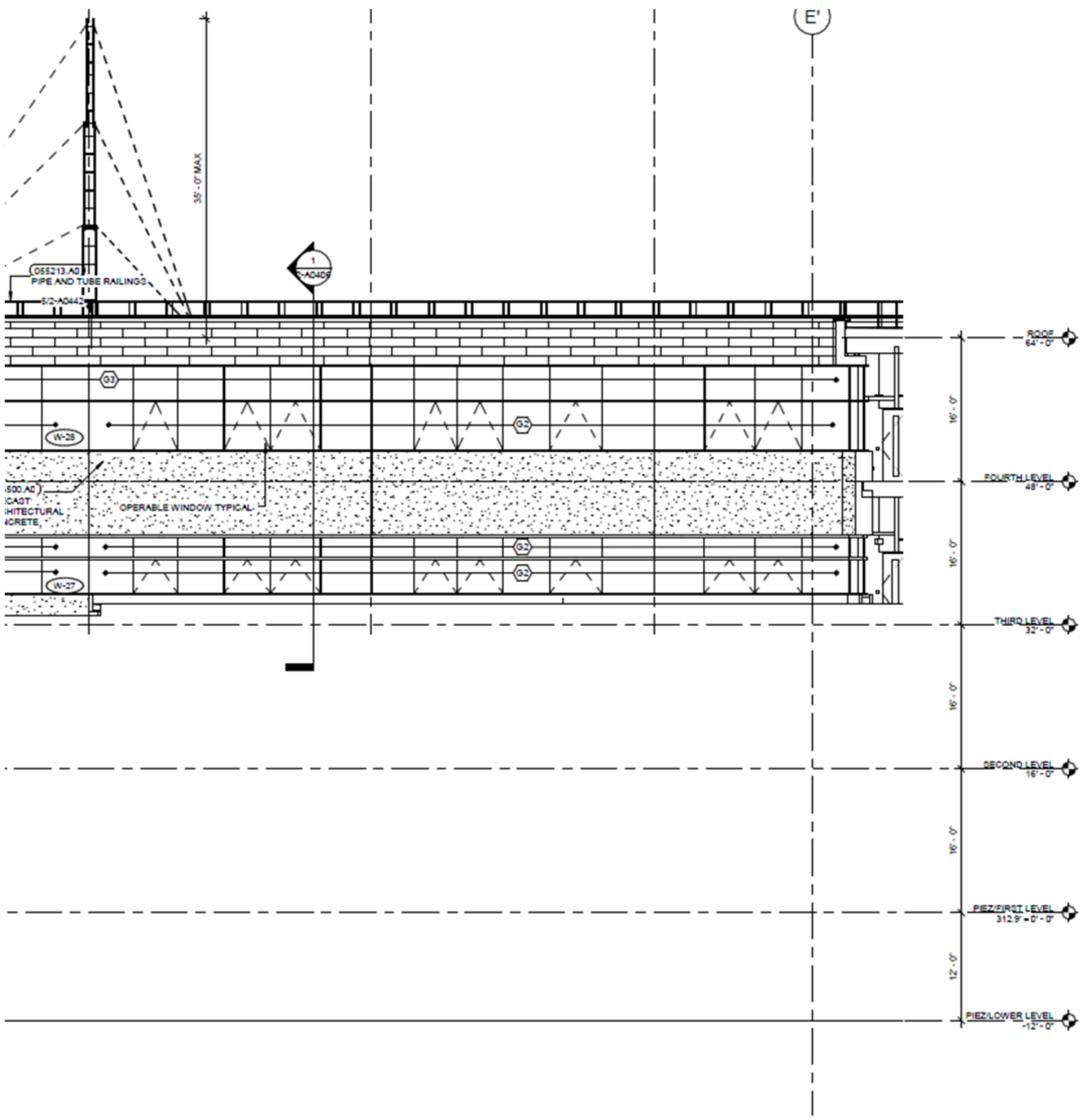
PIEZ HALL ADDITION - SCIENCE, ENGINEERING, & TECHNOLOGIES
SUCF #10354
STATE UNIVERSITY CONSTRUCTION FUND
NEW YORK
OSWEGO
CANNONDESIGN
EXISTING PIEZ
THIRD LEVEL FRAMING PLAN - AREA B
THIRD LEVEL FRAMING PLAN AREA B
2-S0108



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
G2
W-28
W-27
OPERABLE WINDOW TYPICAL
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"