THE UNIVERSITY SCIENCES BUILDING

NORTHEASTERN, USA

Technical Report 3

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Executive Summary

The purpose of Technical Report 3 is to analyze and evaluate the effectiveness of the existing lateral system of the University Sciences Building (USB). The USB is a 209,000 square foot dual sciences building located in Northeast, USA. It houses 300+ offices, multiuse classrooms, laboratories, and open collaborative spaces. The USB took nearly 3 ½ years to construct under a construction manager at risk method. The building's atriums are the main focus in the core design of the building, including a 3 story helical ramp atrium. The USB's unique materials and one of kind cantilevers provide an interesting appeal.

This report includes the analysis of dead, live, snow loads that were provided on the structural drawings. Also, wind and seismic loads were calculated per chapters 6 and 12 of ASCE 7-05. It was determined that the seismic loads control by a factor of 1.6.

Furthermore, a computer model of the lateral system was constructed using ETABS. The model included line elements representing columns, beams and bracing members. Meshed areas were used to represent the walls in the lateral system. Rigid diaphragms were also put into the model, introducing mass at each level.

After the completion of the model, modal information was documented and determined to be accurate. Eight braced frames were then analyzed to check the adequacy of their design. The relative stiffness of each frame was found to understand how much load each frame experiences compared to the others. Direct, torsional, and total shear were calculated for both wind and seismic in the North-South and East-West directions. Next, story displacements and drifts were calculated under the same loading conditions. These values were then compared to the industry standard H/400 maximum drift value. Building torsion, inherent and accidental, was also calculated to gain an understanding of how much torsion the building may experience. Finally, three members from different braces frames were analyzed through hand calculations. These members include a HHS diagonal brace, a wide flanged column, and a concrete column.

With an accurate ETABS model with verified hand calculations, it can be determined that the lateral system is capable of resisting the lateral loads. As the thesis procedure continues, this technical report provides valuable information in considering a redesign proposal.

Building Introduction

The University Sciences Building is a pioneering sciences facility pushing the envelope on innovative research and education. The 209,000 square foot dual building is strategically nested on a 5.6 acre site on the urban university in Northeastern, USA. The building includes 300+ offices, state-of-the-art laboratories, classrooms, lecture halls, a 250 seat auditorium, and a 147 space parking garage. The University's standard building aesthetics include a symmetrical layout and typically a beige brick veneer. The USB's extravagant cantilevers and complex building enclosures express the University's commitment to innovative architecture and sustainability.


Figure 1 – Google Maps aerial view of site

The building was designed around the common idea of atrium space and the majority of other open spaces exposed to light, predominately through curtain wall systems. The intent was to let these open areas serve as collaborative spaces for interaction among students, researchers, and professors. The featured atrium of the building is its 3 story helical structure, which serves as a ramp to levels 3–5 with classrooms intermediately located through its core (Figure 2).


Figure 2 – Helical ramp

The sophisticated and 'edgy' design of the façade expresses the University's movement to push the envelope for not only the sciences but also its architecture. The material used to clad the building is a unique zinc material. Both the black zinc molded squares and the sliver aluminum window trim give the building a different and uneven appearance which sparks interest towards the building.


Figure 3 – South Cantilever

Each floor's different floor plans presents one of a kind overhangs and cantilevers which really express the structure of the building (Figure 3). The placement of key structural components are carefully placed to preserve optimal structural function from floor to floor.

Structural Overview

The University Sciences Building sits upon a Site Class C (Geotechnical Report verified with ASCE 7-05 Chapter 11) with drilled 30'' caissons, caisson caps, spread, continuous, stepped footings, grade beams and column footings. Levels 1-3 of Building 1 and level 4 of Building 2 use concrete beams and slabs with a combination of concrete columns and steel encased columns. The upper floors of both buildings use a composite beam/slab system and continue with steel and encased columns. The lateral systems consists of shear walls and braced steel frames. The shear/retaining walls start from the grade and end at various heights around the building. The braced frames are composed of wide flange chords with HSS diagonals that also reach various heights.

Foundations

The design and analysis of foundations are in accordance with the geotechnical report provided by Construction Engineering Consultants, Inc and ASCE 7-05. Schematic and design development stages were conducted with a safe assumpiton that the soil class was solid rock. The majority of the University's soil has been geologic lly tested to show this. As time proceeded and the geotechincal report was released, it was found that the site class was different than anticipated, was a site class C was determined appropreiate. This induced a complete redesign of Building 2's foundation along with using a new 'flowable fill' for backfill for Building 1. Flowable fill is entrained with fly ash, cement, and other agents to generate negliable lateral pressure on surrounding foundation walls but maintains a compressive strength of 500 psi (Calculations for this are not provided in this technical report).

In has been concluded from the structural drawings that the allowable soil/rock bearing pressures for spread footings on weathered shale are 6000 psf. Likewise for siltstone/sandstone allowable pressures are 12000 psf. In addition, caissons socketed 5' into siltstone/sandy stone are to have an allowable pressure of 50 ksf.

The building load path initiated from the floor systems to columns and then to their respective caissons or interior column footings. For exterior perimeter caissons, they are connected with grade beams to interior caissons or grade column foundations. The slab on grade (SOG) is to be poured onto compacted soil to withstand 500 psf and a minimum of 6" of compacted Penn DOT 2A or 2B material. Furthermore, the fill must be compacted to 95% of the dry density per ASTM D 1557. A vapor barrier is then required to be placed between the fill and the slab.

Expansion joints should be used between the footings and floor slabs to minimize differential settlement stresses. The slab on grade is designed to have an f’c of 4500 psi of normal weight concrete and a mix class C.

Floor Systems

Due to the complexity of the floor layouts, typical bays occur irregularly and are comprised of a variety of beam sizes and lengths (Refer to appendix E for floor plans). In Building 1, floors 1 - 3 utilize concrete reinforced beams that range in size from 50”x24” to 10”x12”, integral with formed 6” reinforced slabs. The upper floors utilize composite and non-composite beam construction. These floor systems range from 1” x 20 gauge metal deck with 5” reinforced concrete topping to 2” x 18 gauge metal deck with 4.5” reinforced concrete topping. The most recurring slab is a composite 2”x18 GA deck with 4.5” normal weight concrete topping, which is found in both building 1 and 2 on floor 4-roof. Areas on levels 4 and 5 of Building 1 brace the metal decking between beams and girders with L4x4x3/8”.

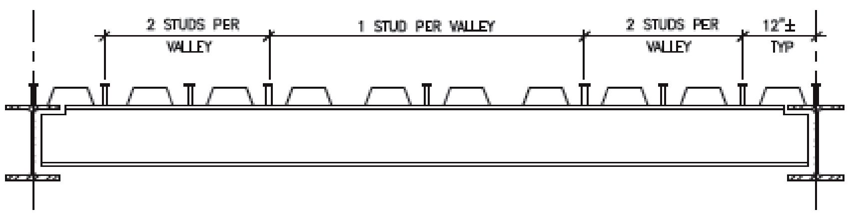


Figure 4. Perpendicular Decking Section – Case 3

The composite and non-composite decks are placed with the ribs of the deck perpendicular to the infill beams to maintain the rigidity of the system. This proved to be a conflict to construct with the placement of shear studs. Where it is efficient to place studs along the length of the beam uniformly normal to the valley and peaks of the deck, it was extremely difficult to maintain this layout with the odd angling placement of particular beams (Figure 4).

Framing System

The USB has three different types of columns, reinforced concrete, encased A992 steel with concrete, and A992 wide flange steel. Reinforced concrete columns vary in size from 24” to 18” diameter circular columns and 16”x18” to 33”x37” rectangular columns. Also, wide flange columns range from W12x40 to W21x210. Levels 1 and 2 of Building 1 have both circular and rectangular concrete columns. Level 3 of Building 1 uses circular/rectangular encased steel and circular reinforced doesn’t hold true for three shear walls that start with a connection to a caisson cap at grade and rise 72’ to columns, likewise with Building 2. Framing girders are then connected to these columns with simple and complex connections. (e.g. pin-pin, moment). The layout of the girders and beams have been arranged with much complexity and provide a challenge for analysis. This complexity not only produced adversity for the fabricators and erectors, increased the price of the building, but also delayed the floor to floor connection schedule. The most nearly identified typical bay has 30'x27' dimensions. .

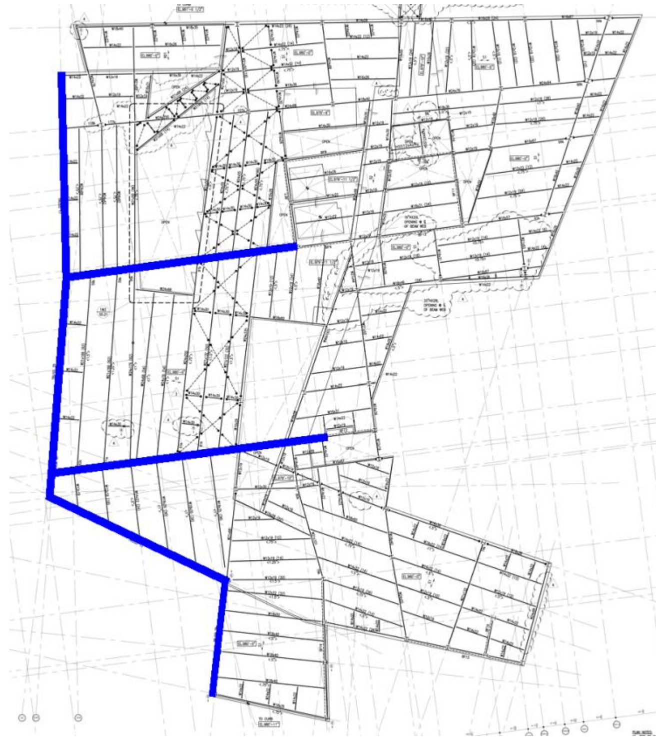


Figure 5. Highlighted truss elements from Building 1 Level 8.

An intricate and vital part of this structural framing system is the truss system in Building 1 which varies in height from Level 6 to the roof (Figure 5). These trusses are comprised of chord sizes as big as W30x292 and intermediate bracing elements as small as W14x53. Due to the complex cantilevers and floor plans, a system needed to be implemented to handle the buildings loads. The system is well hidden in the building and parts where it can be seen (through some windows) presents and interesting look for the building.

Lateral System

The most common lateral force resisting system in The USB is braced frames. The USB utilizes 16 different braced frames between the two buildings. The majority of these are framed within a single bay. Others are 'Chevron' braced frames between two bays and a few span through 3 or more bays.

In Building 1 these braced frames are connected to shear walls were the load is taken from steel elements to concrete elements. These concrete elements are generated from the formed concrete walls lining the 147 parking spot garage. This adds a considerable weight to the building. All shear/retaining walls employed in

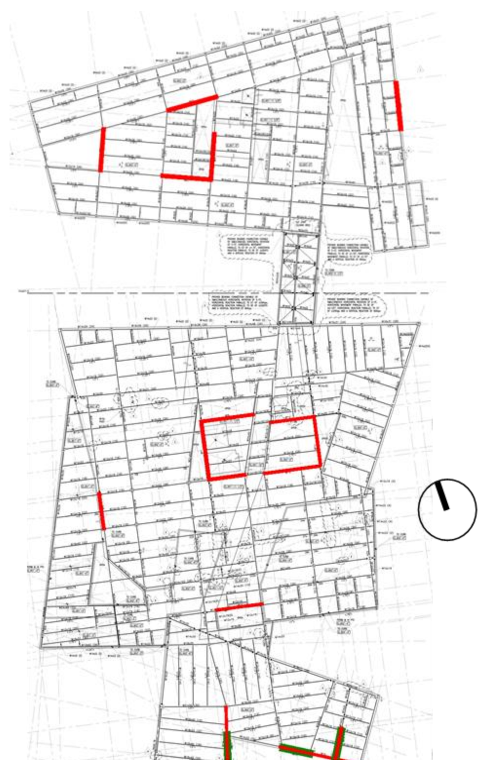
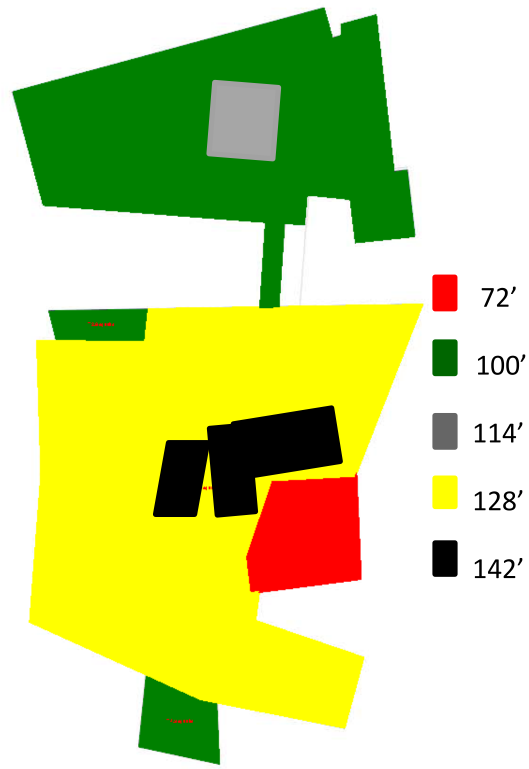


Figure 6. Level 6 Braced Frames and Shear walls

building are kept on the lower floors, which has been assumed to

level 6. Refer to Figure 6 for the layout of brace frames (red) and shear walls (green) on Level 6. The challenge for Technical Report 3 will be to figure out how these lateral force resisting systems receive force on all floors of the building.

Roof System



This dual building system has 5 different roof heights which take into account mechanical penthouses. Figure 7
gives a discription of these varying heights in reference to grade elevation of 0'-0" (+880'). The framing of the roof is composed of wide flange framing with a 3" x 18 GA metal roof deck. The construction of the roof includes a modified bituminous roof system. This systems ranges in size from 3" to 12". This system is to undergo a flood test with 2" of ponding water for 24 hours to test for adaquacy.

Design Codes

In accordance with the specifications of structural drawing S0.01 the original design is to comply with the following codes:

- 2006 International Building Code with local amendments (IBC 2006)
- 2006 International Fire Code with local amendments (IFC 2006)
- Minimum Design Loads for Building and other structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318)
- AISC Manual of Steel Construction LRFD 3rd Edition

These codes were also used in hand calculations and verifications in this Technical Report and those forthcoming.

Materials Used

The materials used for the construction of The USB are described in the following tables including relevant specifications:

Structural Steel			
Type	ASTM Standard	Grade	F_y (ksi)
Wide Flange	A992	50	50
Channels	A572	50	50
Rectangular and Round HSS	A500	B	46
Pipes	A53	E	35
Angles	A572	50	50
Plates	A572	50	50
Tees	A992	50	50

Concrete			
Location in the Structure	f'_c	Weight	Mix Class
Footings, Caissons, Grade Beams	4000	Normal	A
Slab On Grade	4500	Normal	C
Walls and Columns	4500	Normal	C
Beams and Slabs	4500	Normal	C
Slab on Metal Deck	4000	Normal	C
Equipment Pads and Curbs	4000	Normal	B
Lean Concrete	3000	Normal	E

- f'c is the concrete compressive strength at 28 days or at 7 days for high early strength concrete.
- Mix class as defined by project specifications

Aggregate	
Type	ASTM Standard
Normal Weight	C33
Light Weight	C330 and C157

Figure 8. Summary of Materials used on The USB Project with applicable specifications

Gravity Loads

Per the requirements of Technical Report 1, dead, live, and snow loads are to be calculated and verified to those provided on the structural drawings. Alongside these calculations and verifications spot check calculations of gravity members for adequacy are also provided. These calculations can be found in appendix A.

Dead and Live Loads

The structural drawings provide a schedule of superimposed dead and live loads for particular areas (Figure 9). Calculations of certain loads verify those provided in the table and in some cases are found to be conservative. This was perhaps a consideration due the complexity of the floor layout. Self-weights were also calculated to be applied in addition to the given dead and live loads.

Provided Superimposed Dead Loads and Live Loads		
Locations	Superimposed Dead Load (psf)	Live Loads (psf)
Garage	35	50
Planetary Robotics	15	150
Loading Dock	5	250
Storage	35	125
Classroom	35	40
Halls, Assembly, Public Areas	35	80
Office, Meetings Rooms	35	50
Mechanical and Machine Room	75	100
Roof	35	30
Green Roof 1	35	30
Garage Roof	200	100
Green Roof 2	200	30
Mechanical Roof	35	50
Bridge 1	75	100
Roof Pavers	50	100
Roof River Rocks	55	30

Figure 9. Table of provided superimposed dead loads and live loads

Building Weight

The building weight was calculated considering superimposed dead loads, self-weights of columns, shear walls, braced frames, roofs, and exterior wall loads. This section is intended to provide weights for seismic calculations to generate total base shear. This value is then compared to the value provided on the drawings (See Seismic Section). Without the assistance of computer software to generate accurate weights, overall assumptions had to be made. First, from the provided schedules, pounds per square foot of reinforced concrete beams were tabulated considering weight of normal weight concrete (145 pcf) and supplemental reinforcement bars. Secondly, formed slab and metal deck slab pounds per square foot were calculated. Next linear takeoffs of steel beams were tabulated on floors 3-6 of building 1. This process reoccurred for floors 5-6 in building 2. Also counts of columns from the column schedule were made. A weight per lineal foot was noted per column. Next, the building enclosure is broken up into two groups; curtain walls and stud build out system. From assembly weight estimates it was assumed 15 psf for the curtain wall and 30 psf for the stud build out. Finally, the provided superimposed dead loads was summated and yielded a total pound per square foot for the floor. With all of the slabs, concrete beams, steel beams, columns, façade, and superimposed dead loads calculated to either a pound per square foot or linear foot, they are ready to be multiplied by its respective dimensions to result a total kilo pound per floor.

With a weight of kips per floor, it was then divided by that floor's square footage resulting in a kip per square foot (ksf) for that floor. As stated before, level 3-6 in building 1 and levels 5-6 in building 2 were calculated with detailed member calculation. After investigation and grouping of these numbers per their typical floor layout, an average ksf was calculated to be applied to similar levels. This ksf was then applied to the remaining floors square footage once again resulting in kips per floor. The individual kips per floor were then summed to yield a total building weight. The following tables show numerical calculation. It is important to note that Technical Report 3 with provide a more detailed calculation of the building weight.

Building 1			
Level	~ Square Footage	Weight (K)	KSF
3	33,676	5,180.689	0.153839
4	20,983	2,644.86	0.126048
5	22,359	3,190.55	0.142697
6	27,633	3795.15	0.137342
7	21,018	2,592.60	0.123352
8	25,697	3,455.30	0.134463
9	21,970	2,954.15	0.134463
Total	**173,336**	**23,813.32**	**0.137382**

Building 2			
Level	~ Square Footage	Weight (K)	KSF
5	13413	1,654.52	0.1234 *
6	14,103	1,739.609	0.1234
7	13,438	1,657.604	0.1234
8	14,492	1,787.617	0.1234
Roof	14,915	1,839.795	0.1234
Total	**70,361**	**8,679**	**0.1234**

Figure 10. Table of floor approximate square footage, weights (K), and KSF.

* Note: Level 5 of Building 2 was calculated with member weight accuracy and its respective KSF was used as an average for the remaining floors.

From the structural loading diagrams, Live Loads were noted and compared to those provided in ASCE 7-05. Most of these values were verified by the code and others were found to be very conservative. A summary of these results can be found in Figure 11.

Live Loads			
Location	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Garage	50	40	May be from storage during construction
Planetary Robotics	150	200	N/A
Loading Dock	250	N/A	N/A
Storage	125	125	Anticipated light storage
Classroom	40	40	N/A
Halls, Assembly, Public Areas	80	80	N/A
Office, Meetings Rooms	50 (+20)	50 (+20)	+20 for Partition load
Mechanical and Machine Room	100	100	N/A
Roof	30	20	N/A
Green Roof 1	100	100	N/A
Garage Roof	30	30	N/A
Green Roof 2	50	60	Project green roof specifications may cause discrepancy
Mechanical Roof	100	N/A	N/A
Bridge	100	100	Serves as a corridor
Roof Pavers	100	100	N/A
Roof River Rocks	30	N/A	N/A

Figure 11. Comparison table of live loads from design documents and ASCE 7-05

Snow Loads

Snow loads were calculated in accordance with Chapter 7 of ASCE 7-05. This section highlights design criteria for The USB's location and design procedures. All design criteria and loads are summarized in Figure 12.

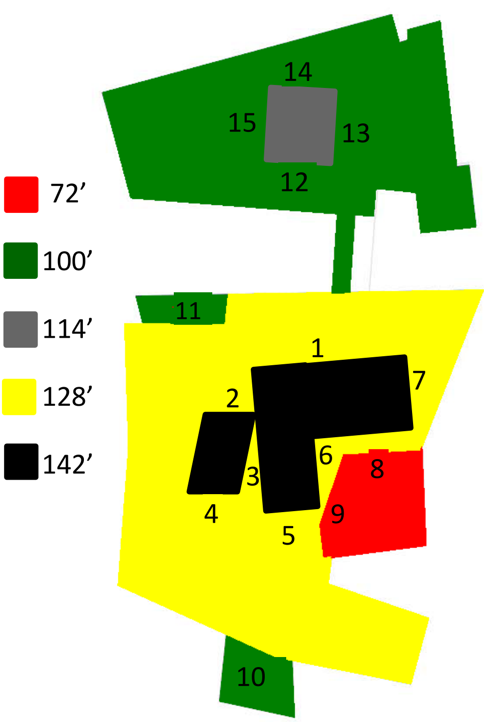
Flat Roof Snow Load Criteria			
Variable	Design Value	ASCE 7-05	Notes
Ground Snow Load, p_g (psf)	30	25	Fig -1 Conservative approach
Snow Exposure Factor, C_e	1.0	1.0	Table 7-2.
Snow Load Importance Factor, I_s	1.1	1.1	Table 7-4, Category III
Thermal Factor, C_t	1.0	1.0	Table 7-3, All other structures
Flat Roof Snow Load, p_f (psf)	27	23.1 (=$0.7C_eC_tIp_g$)	Eq 7-1, Conservative Approach
Snow Specific Gravity γ (pcf)	N/A	18	Eq 7-3
Base Snow Accumulation Height, h_b	N/A	1.3	N/A

Figure 12. Comparison table of snow load criteria from design documents and ASCE 7-05

The structural drawings provide design criterion that is accurate, but conservative in two locations. Figure 7-1 from ASCE 7-05 clearly shows that the building location should be designed with a 25 psf ground snow load. This difference is only slightly conservative. Likewise, the flat roof load calculation, with using a p_g of 30 psf, should yield 23.1 psf and not 27 psf. Once again this is a conservative approach but throughout this technical report and those forthcoming, a p_f of 23.1 psf will be used. Snow drift calculations were also performed for 15 potential locations on 5 different roof heights. Figure 13 shows snow drift calculations, along with Figure 14 and 15 providing a plan and elevation to assist drift calculations.

Snow Drift Calculations											
	General			Windward				Leeward			
Location	h_r	h_c	h_c/h_b	$L_{u\,(ft)}$	h_d (ft)	w_d (ft)	p_d (psf)	$L_{u\,(ft)}$	h_d (ft)	w_d (ft)	p_d (psf)
1	14	12.71	9.85	25	1.25	4.99	22.3	28.5	1.35	5.41	**24.2**
2	14	12.71	9.85	26.75	1.30	5.20	**23.3**	25	1.25	4.99	22.3
3	14	12.71	9.85		VOID				VOID		
4	14	12.71	9.85	68	2.19	8.74	**39.1**	25	1.25	4.99	22.3
5	14	12.71	9.85	25	1.25	4.99	22.3	39.5	1.64	6.55	**29.3**
6	14	12.71	9.85	25	1.25	4.99	**22.3**	25	1.25	4.99	**22.3**
7	14	12.71	9.85	25	1.25	4.99	22.3	54.75	1.95	7.82	**35.0**
8	56	54.71	42.39	35.25	1.53	6.14	27.5	41	1.67	6.69	**29.9**
9	56	54.71	42.39	37	1.58	6.31	28.2	70	2.22	8.87	**39.7**
10	28	26.71	20.70	25	1.25	4.99	22.3	35.25	1.53	6.14	**27.5**
11	28	26.71	20.70	25	1.25	4.99	22.3	99.5	2.63	10.53	**47.1**
12	14	12.71	9.85	25	1.25	4.99	**22.3**	25	1.25	4.99	**22.3**
13	14	12.71	9.85	43.75	1.73	6.93	**31.0**	25	1.25	4.99	22.3
14	14	12.71	9.85	25	1.25	4.99	**22.3**	25	1.25	4.99	**22.3**
15	14	12.71	9.85	58.5	2.02	8.09	**36.2**	25	1.25	4.99	22.3

Figure 13. Table of Snow Drift Calculations. Note: Snow Drift Loads are in addition to flat roof snow load. Total Snow @ max drift location = 23.1 psf + 47.1 psf = 70.2 psf



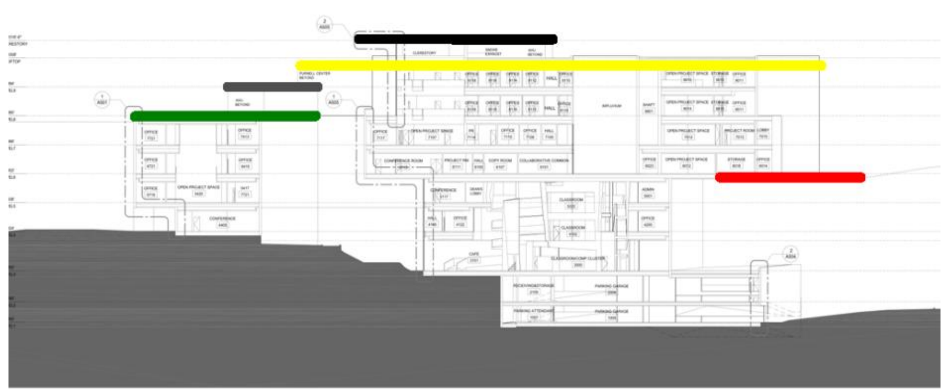


Figure 15. Elevation looking NE detailing roof elevations

Figure 14. Plan of varying roof elevations with potential drift locations

Lateral Loads

As part of technical report 1, wind and seismic loads were calculated to retain a better understanding of the lateral systems to be further elaborated in Technical report 3. Without the assistance of modeling the whole structure in a structural software, it is uncertain to evaluate how much force is being distributed among the different lateral components. Assumptions were made to provide a simplified basis for calculations. For this Technical Report, hand calculations were performed in accordance with ASCE 7-05 and can be found in the Appendices B (wind) and C (seismic).

Wind Loads

Wind load calculations were conducted in accordance with Method 2 Main Wind Force Resisting System (MWRFS) procedure from Chapter 6 of ASCE 7-05. Once again, due to the complexity of floor plans and elevations which produce an undulating façade, assumptions have been made in order to perform basic calculations. Building 1 was simplified by taking the most extreme dimensions (length, base, and height) and using them to generate a box building. This allowed wind to be analyzed on a planar surface normal to the wind in both the North-South and East-West directions of Building 1. This initially would trigger the belief of a conservative approach but further investigation in Technical Report 3 may show otherwise. It is to be noted that for N-S wind, the south wind will be conservative for its elevation changes. Similarly, E-W wind has a gradual change in grade but these calculations have implemented the conservative approach.

The wind follows are particular load path which essentially drives the design of the lateral systems. The wind encounters the components and cladding of the façade which are then taken by the floor slabs. Next, the slabs carry the load to the shear walls and brace frames which deliver the load to the foundation of the building. The following tables (Figures 18-23) show resulting wind pressures and forces in both the North-South and East-West directions of Building 1.

Wind Pressures - N-S Direction							
Type	Floor	Height	Wind Pressure (psf)	Internal Pressure		Net Pressure	
				(+)	(-)	(+)	(-)
Windward	1	0	7.80	3.74	-3.74	11.54	4.06
	2	10	7.80	3.74	-3.74	11.54	4.06
	3	25	9.03	3.74	-3.74	12.77	5.29
	4	44	10.68	3.74	-3.74	14.42	6.94
	5	58	11.52	3.74	-3.74	15.26	7.78
	6	72	12.07	3.74	-3.74	15.81	8.33
	7	86	12.97	3.74	-3.74	16.71	9.23
	8	100	13.55	3.74	-3.74	17.29	9.81
	9	114	14.03	3.74	-3.74	17.77	10.29
	10	128	14.51	3.74	-3.74	18.25	10.77
	11	142	14.97	3.74	-3.74	18.71	11.23
Leeward	All Floors		-8.83	3.74	-3.74	-5.09	-12.57
Side Walls	All Floors		-13.10	3.74	-3.74	-9.36	-16.84
Roof		0-57	-16.84	3.74	-3.74	-13.10	-20.58
		57-144	-16.84	3.74	-3.74	-13.10	-20.58
		144-228	-9.36	3.74	-3.74	-5.62	-13.10
		>228	-5.61	3.74	-3.74	-1.87	-9.35

Figure 18. Tabulations of North-South Wind Pressures on Building 1

Wind Forces N-S Direction							
Level	Elevation (ft)	Floor Height(ft)	Base (ft)	Wind Pressure (psf)	Resultant Force (k)	Story Shear (k)	Overturning Moment (ft-k)
1	0	0	200	7.80	7.8	321.6	0.00
2	10	10	200	7.80	15.6	313.8	156.02
3	25	15	200	9.03	25.3	298.2	631.26
4	44	19	200	10.68	37.4	272.9	1,647.57
5	58	14	200	11.52	31.1	235.5	1,802.52
6	72	14	200	12.07	33.0	204.4	2,378.33
7	86	14	200	12.97	35.1	171.4	3,015.45
8	100	14	200	13.55	37.1	136.3	3,713.27
9	114	14	200	14.03	38.6	99.2	4,401.31
10	128	14	200	14.51	39.9	60.6	5,113.50
11	142	14	200	14.97	20.6	20.6	2,930.26
Total Base Shear						321.6	N/A
Total Over Turing Moment						N/A	**25,789.49**

Figure 19. Tabulations of North-South Wind Resultant Forces on Building 1

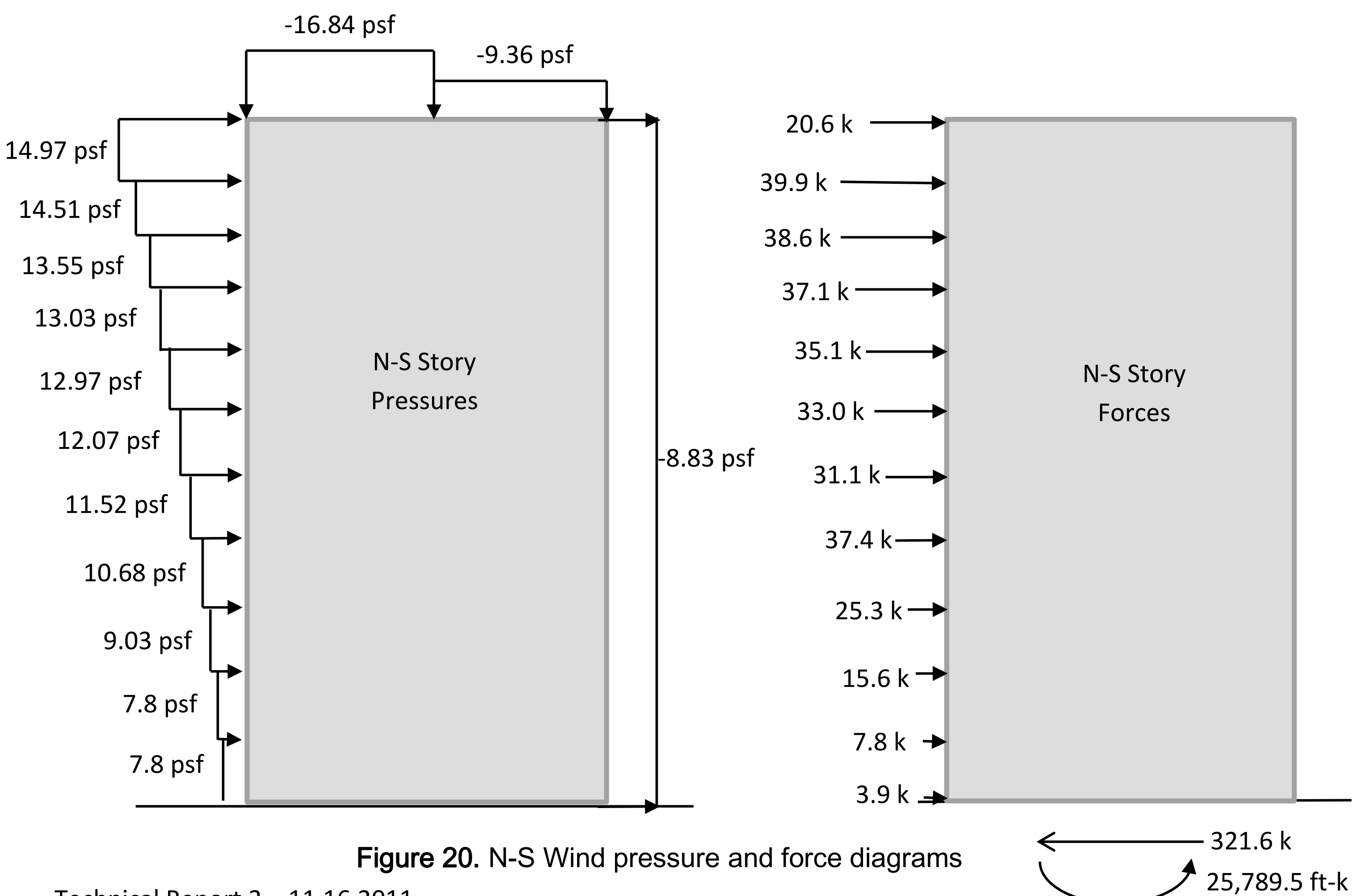


Figure 20. N-S Wind pressure and force diagrams

Similar calculations were performed for wind in the East-West direction (Figure 20). As the elevation and grade vary on the west and east elevations, it has been assumed to simplify this by using floors 3 to 11 (penthouse roof) in the calculations. The West Elevation incorporates elaborate overhangs which will be an interesting topic of investigation in Technical Report 3. The overall assumptions of a planar elevation are intuitive at this point to be conservative but suction and lift may prove to increase the wind pressures over the initial assumptions.

Wind Pressures - E-W Direction							
Type	Floor	Height	Wind Pressure (psf)	Internal Pressure		Net Pressure	
				(+)	(-)	(+)	(-)
Windward	3	25	8.99	3.74	-3.74	12.73	5.25
	4	44	10.62	3.74	-3.74	14.36	6.88
	5	58	11.47	3.74	-3.74	15.21	7.73
	6	72	12.01	3.74	-3.74	15.75	8.27
	7	86	12.91	3.74	-3.74	16.65	9.17
	8	100	13.48	3.74	-3.74	17.22	9.74
	9	114	13.96	3.74	-3.74	17.70	10.22
	10	128	14.44	3.74	-3.74	18.18	10.70
	11	142	14.90	3.74	-3.74	18.64	11.16
Leeward	All Floors		-9.31	3.74	-3.74	-5.57	-13.05
Side Walls	All Floors		-13.04	3.74	-3.74	-9.30	-16.78
Roof		0-57	-16.76	3.74	-3.74	-13.02	-20.50
		57-144	-16.76	3.74	-3.74	-13.02	-20.50
		144-228	-9.31	3.74	-3.74	-5.57	-13.05
		>228	-5.59	3.74	-3.74	-1.85	-9.33

Figure 21. Tabulations of East-West Wind Pressures on Building 1

Wind Forces E-W Direction							
Level	Elevation (ft)	Floor Height(ft)	Base (ft)	Wind Pressure (psf)	Resultant Force (k)	Story Shear (k)	Overturning Moment (ft-k)
1	0	0	228	7.76	8.9	379.4	0.00
2	10	10	228	7.76	22.1	370.6	1,358.95
3	25	15	228	8.99	34.8	348.5	1,757.22
4	34	19	228	10.62	40.0	313.6	2,377.57
5	48	14	228	11.47	36.6	273.7	3,544.71
6	62	14	228	12.01	38.3	237.0	4,304.37
7	86	14	228	12.91	41.2	198.7	5,080.46
8	100	14	228	13.48	43.0	157.5	5,899.15
9	114	14	228	13.96	44.6	114.4	2,782.58
10	128	14	228	14.44	46.1	69.9	5,899.15
11	117	14	228	14.90	23.8	23.8	2,782.58
Total Base Shear						379.4	N/A
Total Over Turing Moment						N/A	**27,105.01**

Figure 22. Tabulations of East-West Wind Story Forces on Building 1

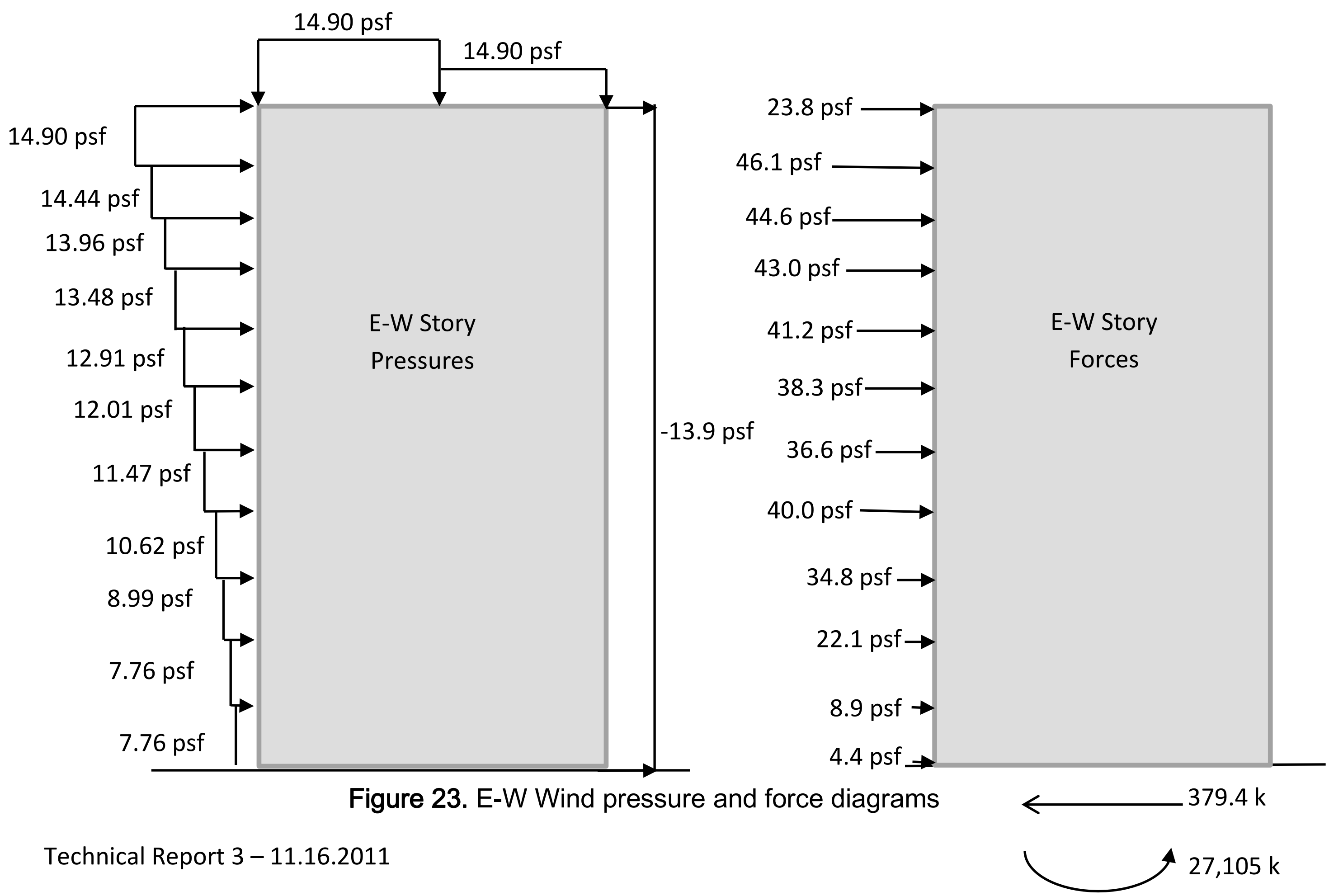


Figure 23. E-W Wind pressure and force diagrams

Seismic Loads

The seismic loads calculated in Technical Report 1 comply with the Equivalent Lateral Force Procedure in Chapters 11 and 12 from ASCE 7-05. Similar to the wind calculations, assumptions were made to generate proper calculations without modeling the building in structural software. Seismic loads are dependent on the building weight, which is more accurate, whereas wind assumptions are based on the dependency of the footprint and surface areas. Therefore, the seismic calculations represent a more accurate depiction of the actual structure. The structural drawings provide design criteria for this structure which can be found in Figure 23. The intent of these calculations was to compare base shears of Building 1 and Building 2 from the structural drawings with those calculated. All provided criteria was noted and found to be adequate in accordance with ASCE 7-05. The only discrepancy was the Seismic Response Coefficient, $C_{s.}$ The drawings provide this value as 0.0265. Under the code, the calculated value of C_s was found to be 0.0256, which will be used to calculate the base shear in this technical report and those to follow. The approximate building period and frequency were calculated to gain an understanding of buildings characteristics.

General Seismic Information	
Site Class	D
Importance Factor (I_e)	1.25
Short Spectral Response Acceleration	0.128
1 Sec Spectral Response Acceleration	0.06
Site Coefficient (F_a)	1.2
Site Coefficient (Fv)	1.7
Response Modification Coefficient	5
Long Period (seconds)	12
Modified Short S.R.A - S_{MS}	0.1536
Modified 1 Sec S.R.A. - S_{M1}	0.1020
Design Short S.R.A. - S_{DS}	0.1024
Design 1 Sec S.R.A. - S_{D1}	0.0680
Seismic Design Category	B

Figure 23. Seismic Design Criterion

The concept of how seismic loads impact a building structure is vital to the understanding of how to employ lateral force resisting systems. The weight of the building is a direct correlation of what the building experiences during seismic activity. The weight of each floor is transferred into lateral structural elements which form into the foundations. All structural components in the ground (below grade) are assumed to be rigid with the ground itself, resulting with only the weight above grade impacting base shear (refer to the Building Weights section for representative building weights). It is to be noted that level 3 of building 1 has 50% of its floor weight below grade which means 50% of level 3’s building weight was considered for the total weight of the building above grade. This is the same logic noted in Wind for the East-West direction The following diagrams summarize the seismic calculations.

Distribution of Seismic Forces								
Level	H (ft)	Elevation (ft)	Weight (k)	wh^k	C_{vx}	f_i (k)	V_i (k)	Overturning Moment (ft-k)
3	19	19	33,676	794,443	.057	35	610	662
4	14	33	20,938	893,429	.064	39	575	1,292
5	14	47	22,539	1,405,826	.101	62	536	2,896
6	14	61	27,633	2,280,235	.164	100	474	6,097
7	14	75	21,018	2,171,239	.156	95	374	7,138
8	14	89	25,697	3,180,919	.229	139	279	12,409
9	14	103	21,970	3,181,345	.229	139	139	14,363
Total Story Forces (Base Shear, $V=C_sW$)						610	N/A	N/A
Total Overturning Moment								**44,857**

Figure 24. Table of Distributed Floor Seismic Forces

NOTE: Seismic loading controls on base shear and overturning moment.

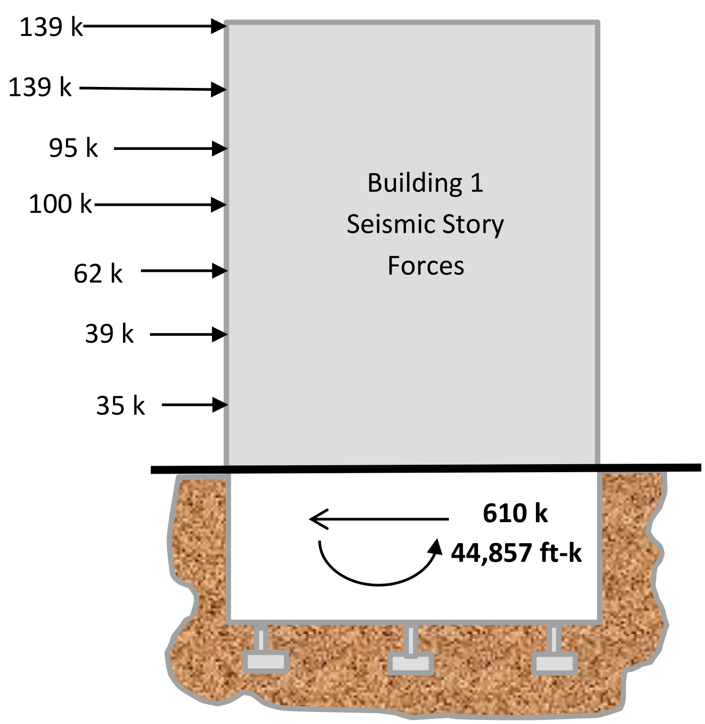


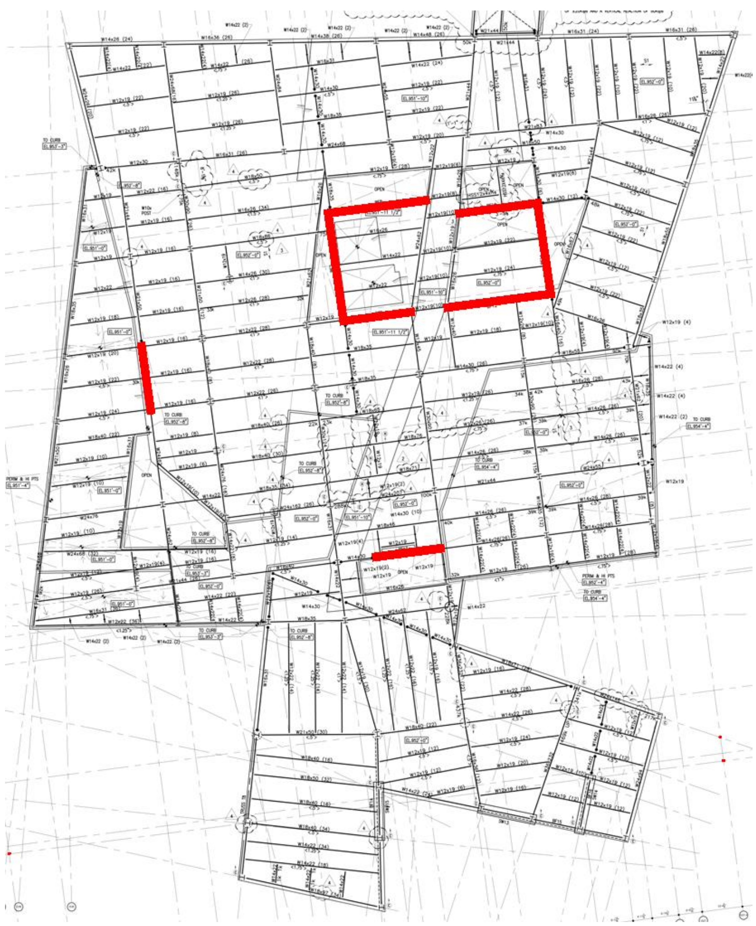
Figure 25. Seismic Force Distribution Loading Diagram

Lateral Load Distribution

The lateral loads are resisted by the combination of the steel braced frames and shear walls. The shear walls are more commonly found in the lower levels and the braced frames rise through the height of the building. In this report, the floor diaphragms were modeled as rigid diaphragms in ETABS. The lateral loads are transferred through the façade to the floor systems and then to the lateral system. These systems will ultimately take the loads to the foundation of the building. In the interest of this providing an accurate technical report with respect to the complexity of the building, the braced frames of interest in this section are the ones highlighted below. From these frames the stiffness' are found from applying a 100 kip load at the top of each frame. After compiling that information, a ratio of each stiffness to the total stiffness is found to define a relative stiffness of each frame. This again was accomplished by applying a 100 kip load to the top of each frame. ETABS generated the following relative stiffness's (Figure 26)

Braced Frame Stiffness			
Frame	Displacement	K (k/in)	Relative Stiffness K
BF6	1.513373	66.08	18.69
BF7	0.959372	104.23	29.49
BF8	2.109039	47.41	13.41
BF9	6.204556	16.12	4.56
BF10	2.185491	45.76	12.94
BF11	3.801471	26.31	7.44
BF12	4.786888	20.89	5.91
BF13	3.744502	26.71	7.55

Figure 26. Table of relative stiffness of highlighted braced frames



Of these eight braced frames, hand calculations, supplemented with excel spreadsheet calculations were performed to determine the distribution of the lateral loads in the particular frames. These calculations included wind loads in both the North-South and East-West directions and likewise with seismic loads. Direct and torsional shear were calculated under these conditions which yielded a total shear for each braced frame. The torsional shear was calculated per the eccentricity generated between the offset of the center of mass and rigidity with respect to the loading direction. For simplicity and conservation, the eccentricity was calculated at the 8^{th} level, of which all of the brace frames exist. Furthermore, as explained earlier, only these eight braced frames were evaluated for because they were either normal or parallel to the loading directions, the others were at odd angles and not evaluated in this report. These calculations can be found below, as well in the Appendix C.

E-W Wind Load Distribution to Braced Frames								
Frame	K (k/in)	Total Lateral Load	e (ft)	d (ft)	k*d^2	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
BF6	66.08	379.4	1.921	11.214	8309.811	0	1.12	1.12
BF7	104.23	379.4	1.921	37.9432	150058.5	0	6.09	6.09
BF8	47.41	379.4	1.921	51.6307	126382.2	114.65	3.70	118.35
BF9	16.12	379.4	1.921	23.714	9065.143	38.98	0.58	39.56
BF10	45.76	379.4	1.921	46.938	100817.3	110.66	3.25	113.91
BF11	26.31	379.4	1.921	37.9432	37878.15	0	1.51	1.51
BF12	20.89	379.4	1.921	23.714	11747.57	50.52	0.75	51.27
BF13	26.71	379.4	1.921	-37.536	37633.09	64.59	-1.52	63.08

N-S Wind Load Distribution to Braced Frames								
Frame	K (k/in)	Total Lateral Load	e (ft)	d (ft)	k*d^2	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
BF6	66.08	321.6	15.611	3.095	632.982	108.08	4.27	112.35
BF7	104.23	321.6	15.611	21.3242	47395.62	170.48	46.42	216.91
BF8	47.41	321.6	15.611	35.012	58117.08	0	34.67	34.67
BF9	16.12	321.6	15.611	7.095	811.4651	0	2.39	2.39
BF10	45.76	321.6	15.611	30.319	42064.5	0	28.98	28.98
BF11	26.31	321.6	15.611	21.3242	11963.72	43.03	11.72	54.75
BF12	20.89	321.6	15.611	7.095	1051.582	0	3.10	3.10
BF13	26.71	321.6	15.611	-54.155	78334.13	0	-30.21	-30.21

E-W Wind Load Distribution to Braced Frames								
Frame	K (k/in)	Total Lateral Load	e (ft)	d (ft)	k*d^2	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
BF6	66.08	610	1.921	11.214	8309.811	0	1.80	1.80
BF7	104.23	610	1.921	37.9432	150058.5	0	9.79	9.79
BF8	47.41	610	1.921	51.6307	126382.2	184.33	5.95	190.29
BF9	16.12	610	1.921	23.714	9065.143	62.68	0.93	63.61
BF10	45.76	610	1.921	46.938	100817.3	177.92	5.22	183.14
BF11	26.31	610	1.921	37.9432	37878.15	0	2.43	2.43
BF12	20.89	610	1.921	23.714	11747.57	81.22	1.20	82.43
BF13	26.71	610	1.921	-37.536	37633.09	103.85	-2.44	101.41

N-S Wind Load Distribution to Braced Frames								
Frame	K (k/in)	Total Lateral Load	e (ft)	d (ft)	k*d^2	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)
BF6	66.08	610	15.611	3.095	632.982	205.01	8.10	213.11
BF7	104.23	610	15.611	21.3242	47395.62	323.37	88.05	411.42
BF8	47.41	610	15.611	35.012	58117.08	0	65.76	65.76
BF9	16.12	610	15.611	7.095	811.4651	0	4.53	4.53
BF10	45.76	610	15.611	30.319	42064.5	0	54.96	54.96
BF11	26.31	610	15.611	21.3242	11963.72	81.62	22.23	103.85
BF12	20.89	610	15.611	7.095	1051.582	0	5.87	5.87
BF13	26.71	610	15.611	-54.155	78334.13	0	-57.30	-57.30

Figure 27: Wind and seismic distribution to 8 braced frames

ETABS Computer Model

As previously stated, due to the complexity of the building, the scope and time table of the report, only building 1 was modeled in ETABS. This is a feasible action because the connection of the two buildings is not adequate to distribute loads between buildings. Building 1 includes 11 braced frames, 3 multistory (4+ stories) shear walls and other foundation shear walls in the basement, all of which were assigned material properties specific to the project. The braced frames were modeled as line elements that per AISC edition 13. The braced frames included wide flange columns and beams and HHS tubes for diagonal bracing. The 4500 psi f'c shear walls were modeled as shell elements and meshed into areas of 24"x24". All concrete walls were assigned a rigid end offset of 0.5 to ensure a rigid connection among members. Diaphragms were also modeled as rigid members with assigned mass to account for its weight. The following figures represent the ETABS Model.

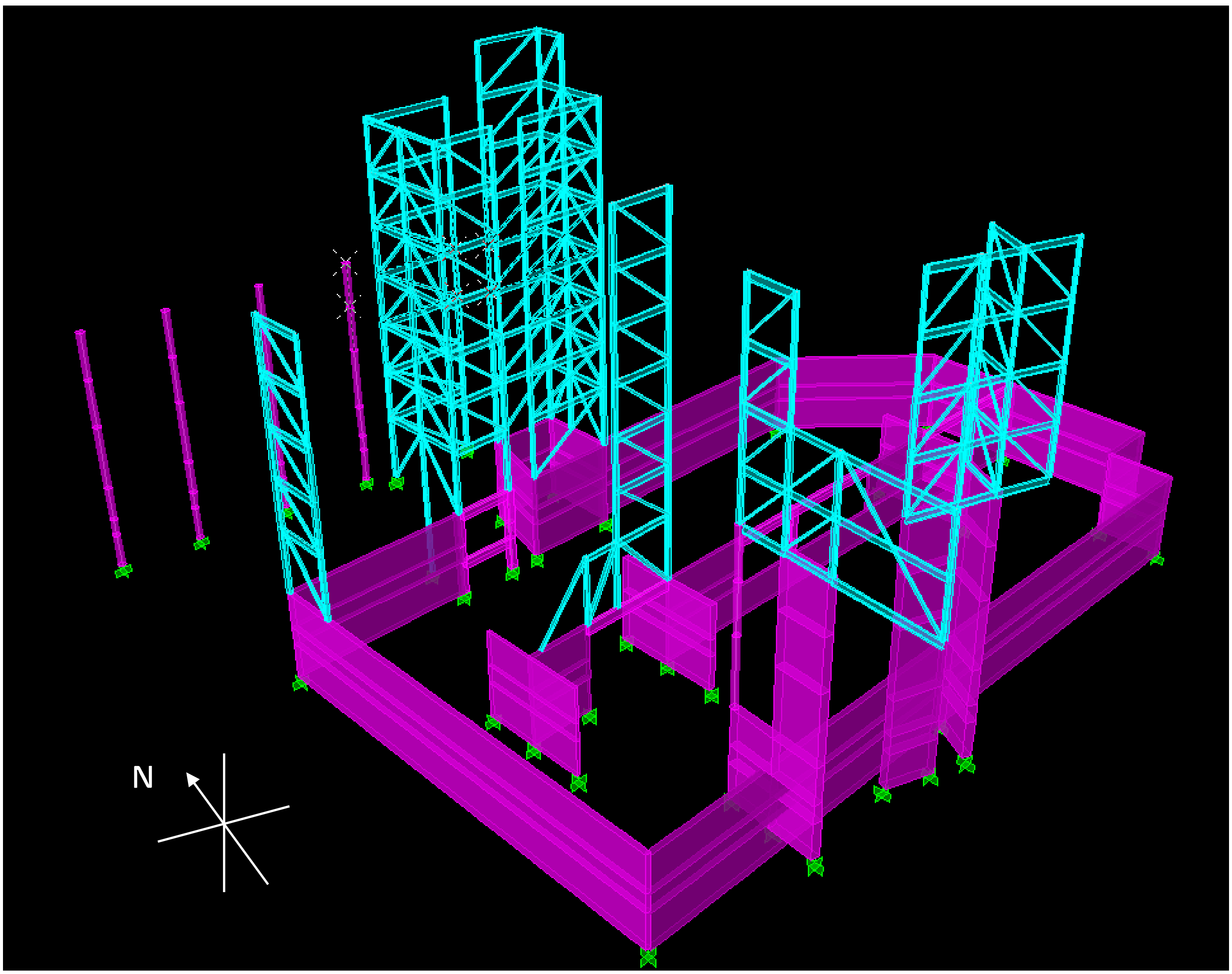


Figure 28: 3D ETABS Model showing braced frames and shear walls

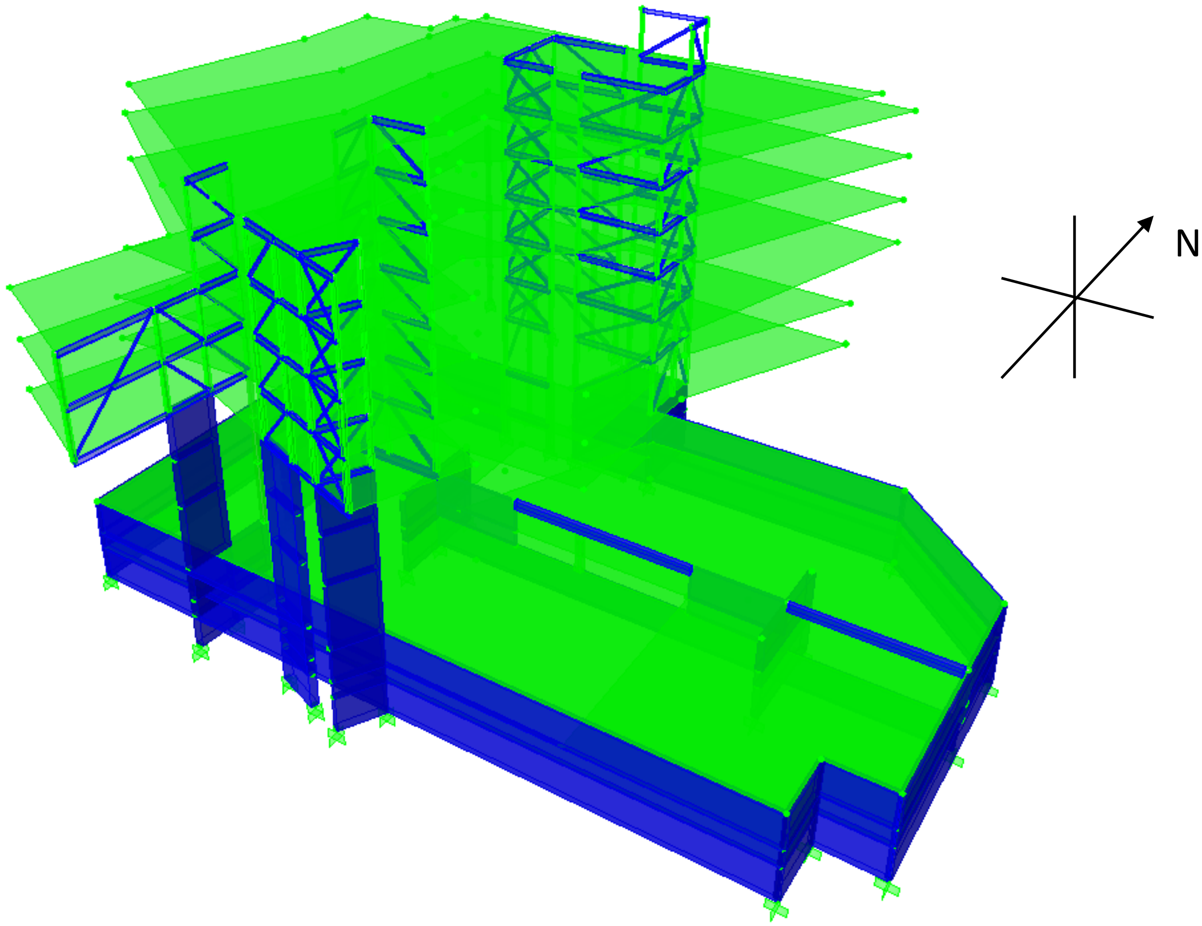
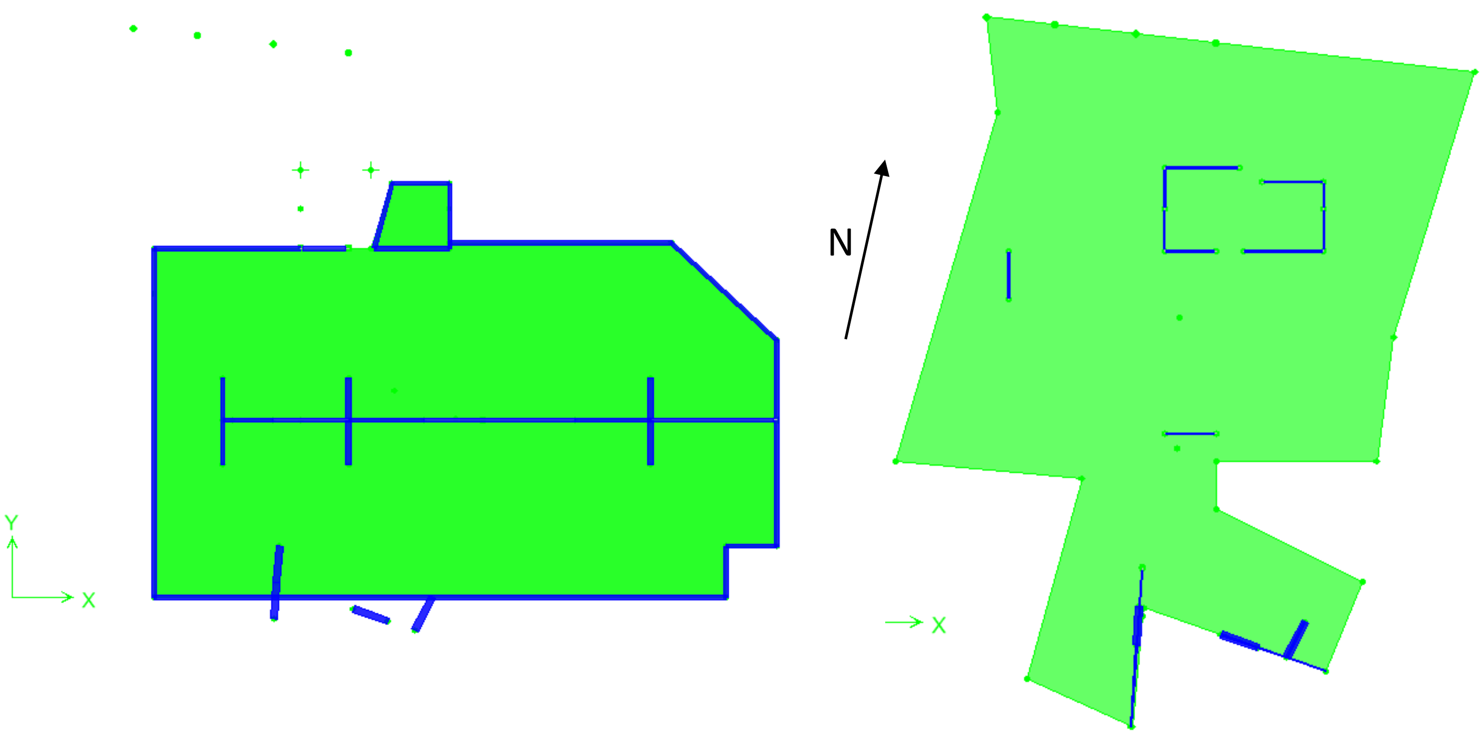


Figure 29: Top: 3D ETABS model with diaphragms

Bottom: Level 3 and Level 7 floor plans



Load Combinations

ASCE 7-05 section 2.3 designates load combinations per strength design, which were considered for this report. The combinations include both lateral and gravity loads. These loads were imported into ETABS and evaluated per the displacement and forces of particular members. It is important to note that different load cases govern depending on which member was analyzed. Therefore, the worst case scenario for deflections and displacement was used in evaluating member forces and displacements. The following are the combinations defined by ASCE 7-05.

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

The following charts display the displacement and story drifts for wind and seismic in both the North-South and East-West directions.

E-W Wind Displacements and Story Drifts			
Level	Displacement (in)	Allowable Displacement (in)	Ok?
Roof	1.066	3.9	Yes
9	0.89	3.48	Yes
8	0.748	3.06	Yes
7	0.541	2.64	Yes
6	0.319	2.22	Yes
5	0.155	1.8	Yes
4	0.073	1.38	Yes
3	0.0015	0.75	Yes
2'	0.0006	0.45	Yes
2	0.0003	0.3	Yes

Figure 30: Wind and seismic displacements and story drifts

N-S Wind Displacements and Story Drifts			
Level	Displacement (in)	Allowable Displacement (in)	Ok?
Roof	0.374	3.9	Yes
9	0.323	3.48	Yes
8	0.271	3.06	Yes
7	0.215	2.64	Yes
6	0.158	2.22	Yes
5	0.115	1.8	Yes
4	0.0618	1.38	Yes
3	0.0026	0.75	Yes
2'	0.0008	0.45	Yes
2	0.0004	0.3	Yes

E-W Seismic Displacements and Story Drifts				
Level	Displacement (in)	Story Drift (in)	Allowable Displacement (in)	Ok?
Roof	2.3447	0.2261	3.9	Yes
9	2.1186	0.2998	3.48	Yes
8	1.8189	0.2655	3.06	Yes
7	1.5534	0.3865	2.64	Yes
6	1.1668	0.2812	2.22	Yes
5	0.8856	0.3126	1.8	Yes
4	0.5730	0.3864	1.38	Yes
3	0.1866	0.1123	0.75	Yes
2'	0.0743	0.0536	0.45	Yes
2	0.0208	0.0208	0.3	Yes

N-S Seismic Displacements and Story Drifts				
Level	Displacement (in)	Story Drift (in)	Allowable Displacement (in)	Ok?
Roof	1.73	0.09	3.9	Yes
9	1.64	0.22	3.48	Yes
8	1.42	0.28	3.06	Yes
7	1.15	0.35	2.64	Yes
6	0.80	0.11	2.22	Yes
5	0.69	0.34	1.8	Yes
4	0.34	0.24	1.38	Yes
3	0.10	0.06	0.75	Yes
2'	0.04	0.03	0.45	Yes
2	0.01	0.01	0.3	Yes

Building Torsion

Due to the resulting difference in locations of the center of the mass and the center of rigidity, the lateral seismic loads will act at the center of mass generating torsion with a moment arm as the eccentricity between the center of mass and rigidity. ETABS has calculated the center of mass and center of rigidity for each floor allowing for accurate calculations of the building's torsion. Furthermore, the building has been modeled with the applied seismic loads accounting for 5% accidental torsion that may occur in the building. These calculations are important to keep in mind while considering a redesign, in that a slight variation can change the torsion the building experiences. The following calculations show the building torsion from both inherent and accidental torsion in the North-South and East-West directions.

Building Torsion E-W Direction - Seismic Loading							
Level	Story Force (k)	COR	COM	$e_{x\,(ft)}$	M_t (ft-k)	M_a (ft-k)	M_{total} (ft-k)
Roof	139	105.396	87.283	18.11	2517.707	1584.6	4102.307
9	139	103.793	109.275	5.48	761.998	1584.6	2346.598
8	95	100.786	102.707	1.92	182.495	1083	1265.495
7	100	95.562	105.519	9.96	995.7	1140	2135.7
6	62	86.193	102.046	15.85	982.886	706.8	1689.686
5	39	99.116	109.6	10.48	408.876	444.6	853.476
4	35	101.924	109.6	7.68	268.66	399	667.66
3	Below Grade						
2							
2'							
						Total	13060.92

Building Torsion N-S Direction - Seismic Loading							
Level	Story Force (k)	COR	COM	$e_{x\,(ft)}$	M_t (ft-k)	M_a (ft-k)	M_{total} (ft-k)
Roof	139	116.196	109	7.20	1000.244	1390	2390.244
9	139	116.751	103.286	13.47	1871.635	1390	3261.635
8	95	117.405	101.794	15.61	1483.045	950	2433.045
7	100	119.626	109.629	10.00	999.7	1000	1999.7
6	62	123.908	108.168	15.74	975.88	620	1595.88
5	39	116.499	102.798	13.70	534.339	390	924.339
4	35	111.995	102.798	9.20	321.895	350	671.895
3	Below Grade						
2							
2'							
						Total	13276.74

Figure 31: Building Torsion charts.

Member Checks

Three individual member calculation checks were performed to verify that the members are adequate for the gravity and lateral loads. The loads of each particular member were obtained from the ETABS model. Each member had a different load case that gave it is worst case scenario. The three members considered were a HSS diagonal brace, a wide flange column, and a concrete column (verified in spColumn). The following diagrams show these members.

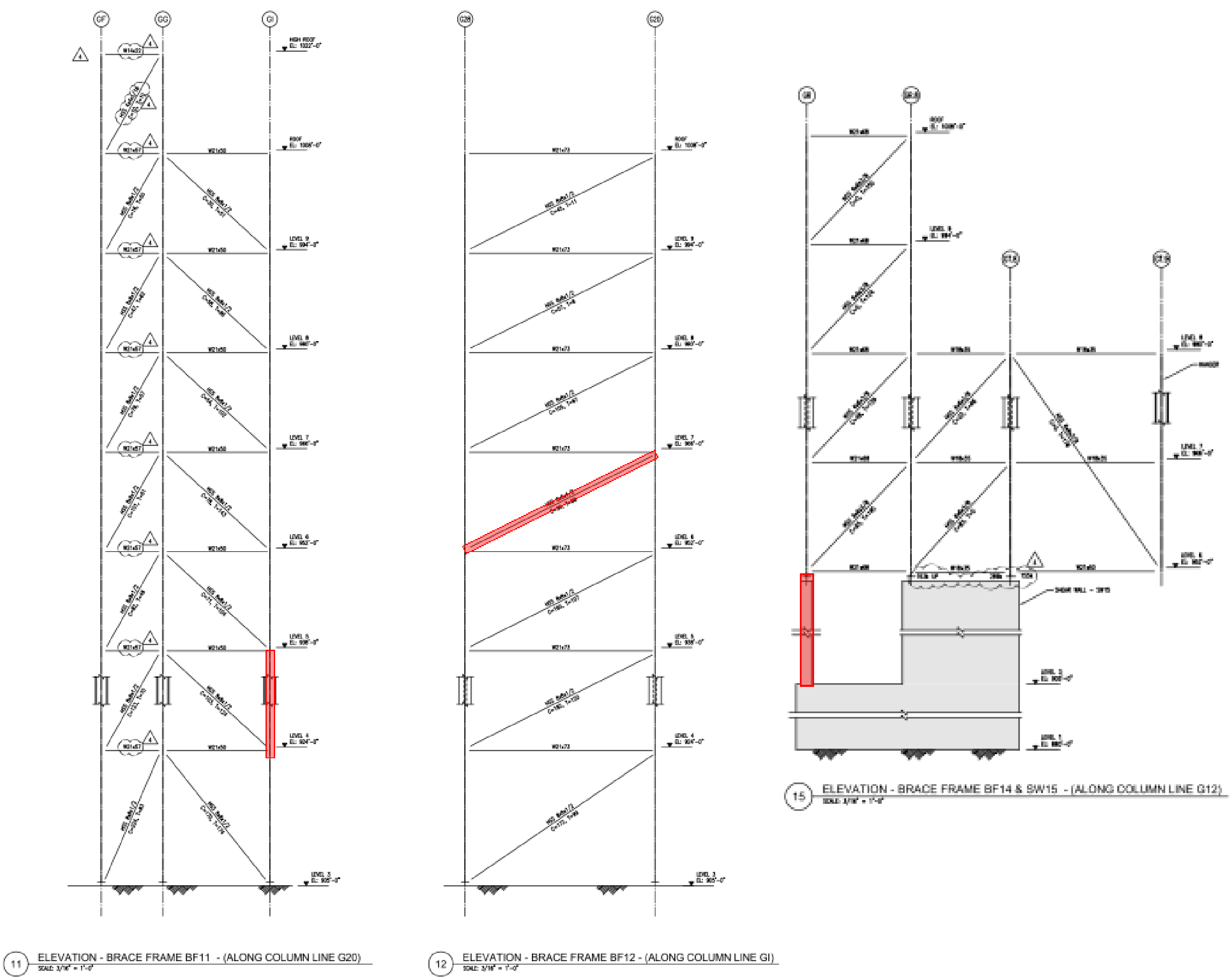


Figure 32: Highlight members of braced frames that are checked for adequacy

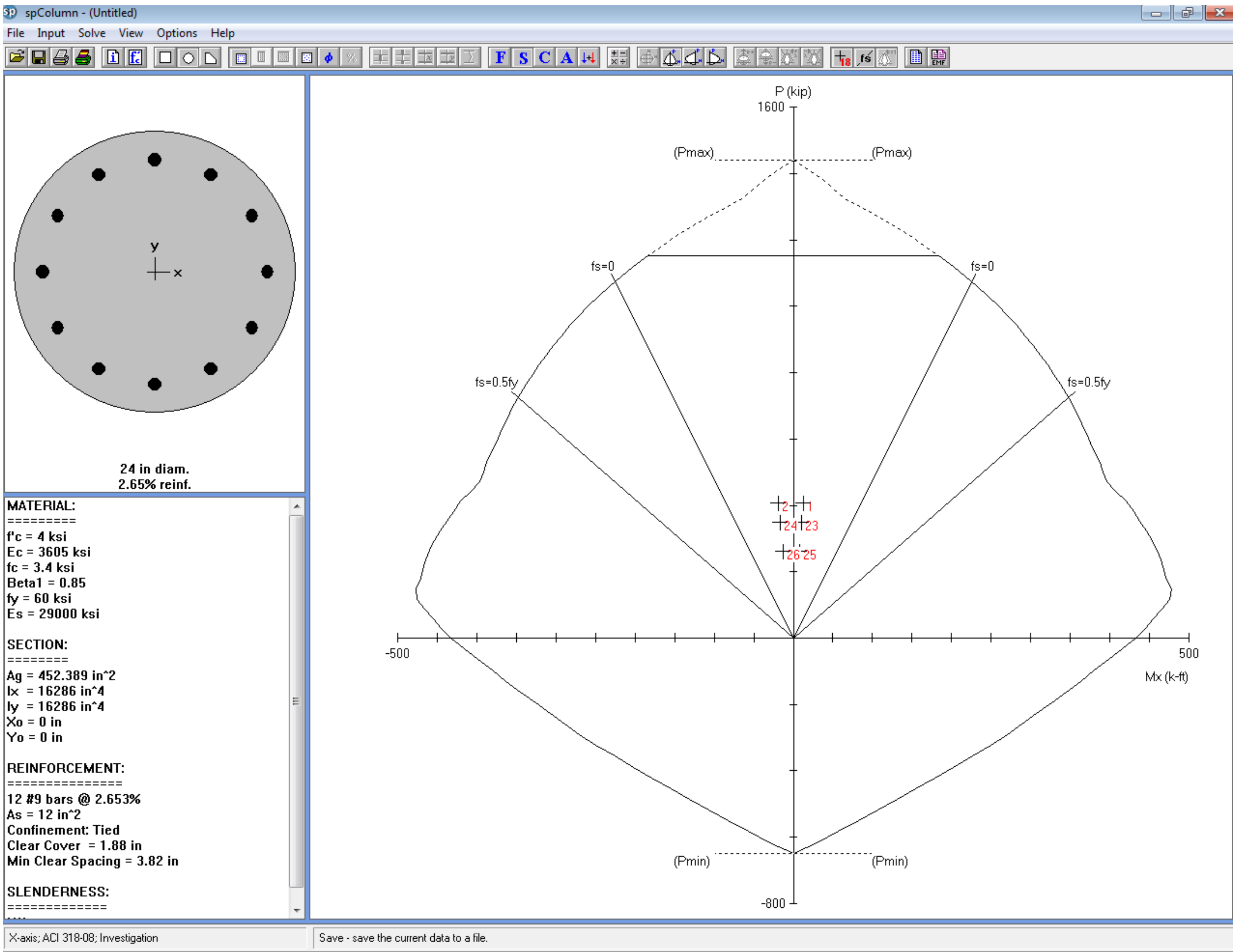


Figure 33: spColumn output for the concrete column

Conclusion

Technical Report 3 was a thorough investigation into the lateral system of the University Sciences Building (USB). Upon the analysis of multiple elements of this system, it has been determined that the lateral system is adequate to carry the lateral loads it is likely to experience. This can be concluded from the combination of an elaborate ETABS computer model and supplemental hand calculations to verify the models accuracy. The wind forces were calculated using the Main Wind Force Resisting System method from ASCE 7-05. Similarly, the seismic forces were calculated by Equivalent Lateral Force method. It has been determined that seismic loads will control by nearly 38% over wind but both wind and seismic were both considered in the lateral system to produce the worst case loading scenario.

Due to the complexity of the building and the scope of this technical report, only one building was model. This was determined feasible as the connection of the two buildings is not adequate to transfer loads. The modeled consisted to line elements representing columns, beams and bracing. In addition, area meshes were utilized to represent shear walls accurately. Rigid diaphragms were also inserted to represent the floor systems and to induce mass into the system. After implementing multiple load combinations, the system was found to adequately carry the loads. Furthermore, hand calculations were performed and verified the models output.

Eight individual brace frames were analyzed to determine their relative stiffness. This was then used to help determine how much each frame received direct, torsional shear, and total shear. In addition, story displacements and drifts were calculated with respect to the controlling load cases.

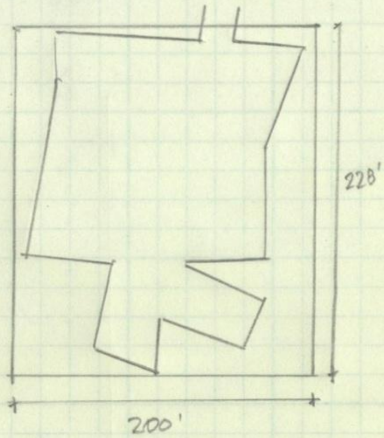
Finally three individual members of the lateral system were checked through hand calculations to verify their adequacy. These members included a HHS diagonal bracing member, a wide flange column, and a concrete column. All three members were found to be more than adequate to carry the appropriate loads.

Technical Report 3 has valuable information about the University Sciences Building that will be useful in writing a proposal for the redesign process.

Appendix

Appendix A: Wind Calculations

1 of 2 | Chris Dunlay | Technical Report #1 | Wind Calcs - Building 1



Building 1

- Base & Length
  - ↳ N-S → L = 228' B = 200'
  - ↳ E-W → L = 200' B = 228'
- This building has three diff. heights. The larest height will be taken as the uniform height = 142'

- Method 2 - Analytical Procedure
  - ↳ Basic Wind Speed, V = 90 mph (Fig 6-1)
  - ↳ Wind Directionality Factor, K_d = 0.85 (Table 6-1)
  - ↳ Occupancy Category - III (Table 1-1)
  - ↳ Importance Factor = 1.15 (Table 6-1) Non-Hurricane
  - ↳ Exposure Category = B (§ 6.5.6.3)
  - ↳ Topographic Factor, K_{zt} = 1.0 (§ 6.5.7) Homogeneous Topo
  - ↳ Internal Pressure = ±0.18 (Fig 6.5) Enclosed Bldg
  - ↳ Velocity Pressure Coefficent = $K_z - K_h$
  - ↳ Velocity Pressures
    - $q_z = 0.00256 K_z K_{zt} K_d V^2 I$
    - $q_h = 0.00256 K_h K_{zt} K_d V^2 I$

  } See Excel

2 of 2 | CHRIS DUNLAY | TECHNICAL REPORT #1 | WIND CALCULATIONS

↳ GUST FACTOR

- $T_a = C_t h_n^x = 0.02(142)^{0.75} = 0.823\ s$
- $f = 1/T_a = 1/0.823 = 1.21 > 1.0 \rightarrow G = 0.85$

↳ CALCULATE ACTUAL GUST FACTOR

- NORTH-SOUTH
  - $\bar{z} = 0.6(142) = 85.2' > 30'$ OK! ✓
  - $c = 0.3$
  - $l = 320\ ft$
  - $\bar{\varepsilon} = 1/3.0$
  - $L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}} = 320\left(\frac{85.2}{33}\right)^{1/3} = 439$
  - $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{85.2}\right)^{1/6} = 0.143$
  - $Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{200+142}{439}\right)^{0.63}}}$

    $= 0.806$
  - $G = 0.925\left[\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right] = 0.925\left[\frac{1+1.7(3.4)(0.143)(0.806)}{1+1.7(3.4)(0.143)}\right]$

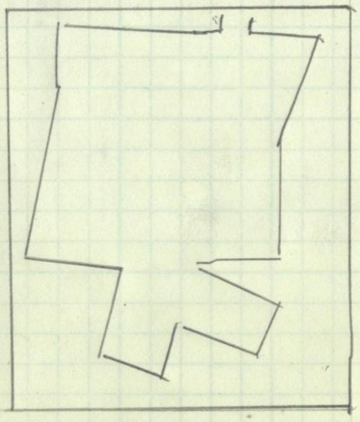
    $\boxed{= 0.844}$

- EAST-WEST
  - $Q = \sqrt{\frac{1}{1+0.63\left(\frac{228+142}{439}\right)^{0.63}}} = 0.799$
  - $G = 0.925\left[\frac{1+1.7(3.4)(0.143)(0.799)}{1+1.7(3.4)(0.143)}\right]$

    $\boxed{= 0.840}$

Appendix B: Seismic Calculations

1 OF 2 CHRIS DUNLAY TECHNICAL REPORT #1 SEISMIC CALCULATIONS



SITE CLASS - C (FROM GEOTECHNICAL REPORT)

OCCUPANCY CATEGORY - III (TABLE 1-1)

IMPORTANCE FACTOR - 1.25 (TABLE 11.5-1)

$S_s = 0.128$ (FROM USGS)

$S_1 = 0.06$

SITE COEFFICENTS:

$F_a = 1.2$ (TABLE 11.4-1)

$F_v = 1.7$ (TABLE 11.4-2)

- SHORT SPECTRAL RESPONSE ACCEL - $S_{ms} = F_a S_s = (1.2)(0.128) = 0.1536$
- 1-SECOND SPECTRAL RESPONSE ACCEL - $S_{m1} = F_v S_1 = (1.7)(0.06) = 0.102$
- DESIGN SRA - $S_{DS} = \frac{2}{3} S_{ms} = (2/3)(0.1536) = 0.1024$
- DESIGN SRA - $S_{D1} = \frac{2}{3} S_{m1} = (2/3)(0.102) = 0.068$

↳ SEISMIC DESIGN CATEGORY (TABLE 11.6-1)

$S_{DS} = 0.1024$ w/ Occ. Cat = III

$S_{DS} < 0.167$ → CATEGORY A

- RESPONSE MODIFICATION COEFFICIENT, $R = 5$ (TABLE 12.2-1)

USE EQUIVALENT LATERAL FORCE ANALSIS (

↳ APPROXIMATE PERIOD (TABLE 12.8-2)

- $T_a = C_T h_n^x$ → STRUCTURE TYPE "ALL OTHER STRUCTURAL SYSTEMS"

$= (0.02)(103)^{0.75} = 0.647$

- FREQUENCY → $f = 1/T_a = 1.55$

↳ SEISMIC RESPONSE COEFFICENT.

- $C_u = 1.7$ → (TABLE 12-8-1 $S_{D1} \le 0.1$)

2 OF 2 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SEISMIC CALCULATIONS

- $C_{s,CALC} = \frac{S_{DS}}{(R/I)} = \frac{0.1024}{(5/1.25)} = 0.0256$

- $C_{s,MAX} = \begin{cases} \frac{S_{D1}}{T(R/I)} & ; \text{ FOR } T \leq T_L \\ \frac{T_L S_{D1}}{T^2(R/I)} & ; \text{ FOR } T > T_L \end{cases}$ $(T_L = 12 \rightarrow \text{TBL } 22\text{-}15)$

  T=

  $\hookrightarrow$ SEE CALC

- BASE SHEAR

  $V = C_s W$; W CAN BE FOUND IN EXCEL SHEET

  $= (0.0256)(23{,}812\text{ K}) = 609.6^K$

  $\% = 1 - \left(\frac{609.6^K}{620.6^K}\right)\Big] \times 100 = 1.77\%$ DIFFERENCE

- SEE EXCEL SHEET FOR REMAING CALCS & BUILDING 2

Appendix C: Stiffness Calculations

1 of 2 | Chris Dunlay | Technical Report #3 | Stiffness Calculations

$K_6 = 66.08\ ^{k}/_{in}$

$K_7 = 104.23\ ^{k}/_{in}$

$K_8 = 47.41\ ^{k}/_{in}$

$K_9 = 16.12\ ^{k}/_{in}$

$K_{10} = 45.76\ ^{k}/_{in}$

$K_{11} = 26.31\ ^{k}/_{in}$

$K_{12} = 20.89\ ^{k}/_{in}$

$K_{13} = 26.71\ ^{k}/_{in}$

DIRECT LOADS N-S

$$F_6 = \frac{66.08}{66.08 + 104.23 + 26.31} V_{N-S} = \underline{\underline{0.336\ V_{N-S}}}$$

$$F_7 = \frac{104.23}{66.08 + 104.23 + 26.31} V_{N-S} = \underline{\underline{0.503\ V_{N-S}}}$$

$$F_{11} = \frac{26.31}{66.08 + 104.23 + 26.31} V_{N-S} = \underline{\underline{0.134\ V_{N-S}}}$$

DIRECT LOADS E-W

$$F_8 = \frac{47.41}{47.41 + 16.12 + 45.76 + 20.89 + 26.71} V_{E-W} = \underline{\underline{0.302\ V_{E-W}}}$$

$$F_9 = \frac{16.12}{47.41 + 16.12 + 45.76 + 20.89 + 26.71} V_{E-W} = \underline{\underline{0.103\ V_{E-W}}}$$

$$F_{10} = \frac{45.76}{47.41 + 16.12 + 45.76 + 20.89 + 26.71} V_{E-W} = \underline{\underline{0.292\ V_{E-W}}}$$

$$F_{12} = \frac{20.89}{47.41 + 16.12 + 45.76 + 20.89 + 26.71} V_{E-W} = \underline{\underline{0.133\ V_{E-W}}}$$

$$F_{13} = \frac{26.71}{47.41 + 16.12 + 45.76 + 20.89 + 26.71} V_{E-W} = \underline{\underline{0.170\ V_{E-W}}}$$

2 of 2 | CHRIS DUNLAY | TECHNICAL REPORT #3 | STIFFNESS CALCS

TORSIONAL LOADS N-S

$$e = 117.405' - 101.794' = 15.611'$$

$$F_{T6} = \frac{(66.08)(3.095)V_{N-S}(15.611)}{632.982 + 47,395 + 58117 + 811 + 42664 + 11963 + 1051 + 78334}$$

$$= 0.0133\,V_{N-S}$$

F_{T7} =
F_{T8} =
F_{T9} =
F_{T10} =
F_{T11} =
F_{T12} =
F_{T13} =

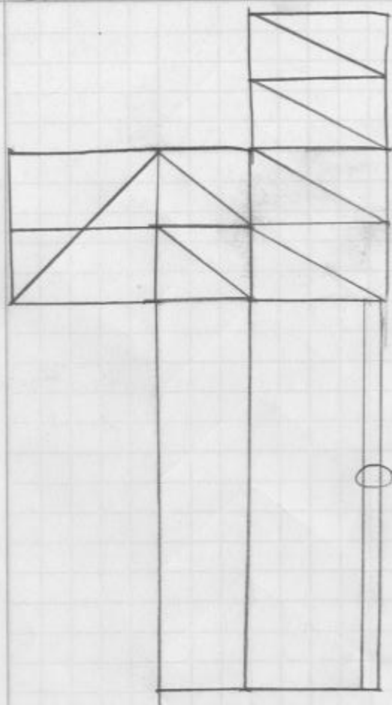
SEE EXCEL SHEET

TORSIONAL LOADS E-W

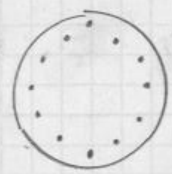
F_{T7}
F_{T8}
F_{T9}
F_{T10}
F_{T11}
F_{T12}
F_{T13}

SEE EXCEL SHEET

Appendix D: Member Checks



24" DIAMETER CONCRETE COL.



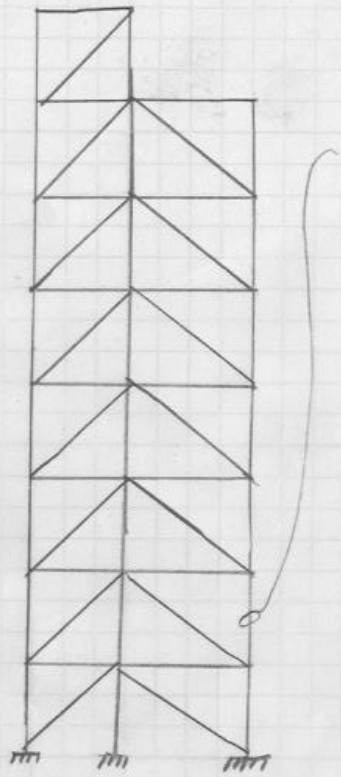
(12) # 9 BARS

FROM ETABS

- CALCULATIONS WERE PERFORMED IN SP COLUMN.

↳ SEE NEXT PAGE.

2 of 3 | Chris Dunlay | Tech Report # 3 | Member Check

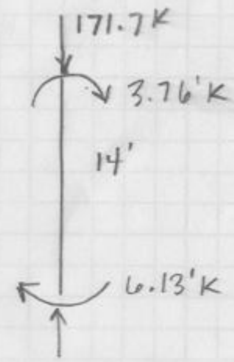


W12 x 210

$F_y = 50$ ksi

$A_g = 61.8 \text{ in}^2$

$I = 2140 \text{ in}^4$



$p = 0.424E-3$

$b_x = 0.686E-3$

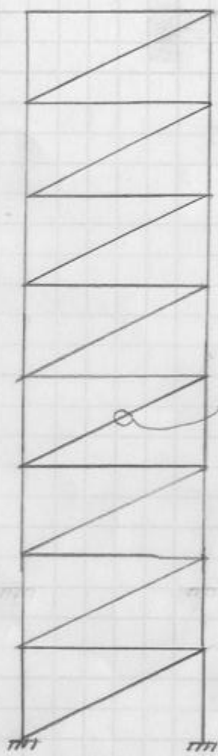
$P_r = 0.424E-3\,(171.7) = 0.073 < 0.2$

$\rightarrow \frac{1}{2} P_r + \frac{9}{8}\left(b_x M_{rx} + b_x M_{ry}\right) \le 1.0$

$= \frac{1}{2}(0.073) + \frac{9}{8}\left[(0.686E-3)(3.76) + (1.49E-3)(6.13)\right]$

$= 0.0497 < 1.0$ OK! ✓

3 of 3 | Chris Dunlay | Tech Report #3 | Member Check



HSS 8 x 8 x 1/2

$A_g = 13.5\ in^2$

$I = 125\ in^4$

$\phi P_N = 559\ K$ → Table 5-5 AISC Manual

↳ $= \phi F_y A_g$

$= 0.9(46\ ksi)(13.5\ in^2) = \underline{558.9\ K}$

↳ Tension

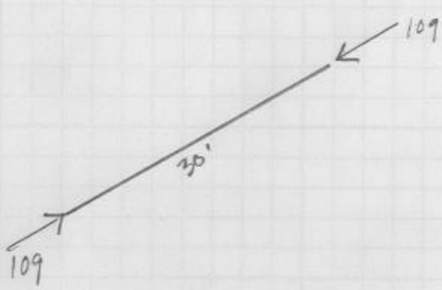
From ETABS → $109\ K$

$\phi P_N = 559^K > P_u = 109$ OK! ✓

↳ Compression

$$P_{CR} = \frac{\pi^2 E I}{L^2} = \frac{\pi^2 (29000)(125)}{(30 \times 12)^2}$$

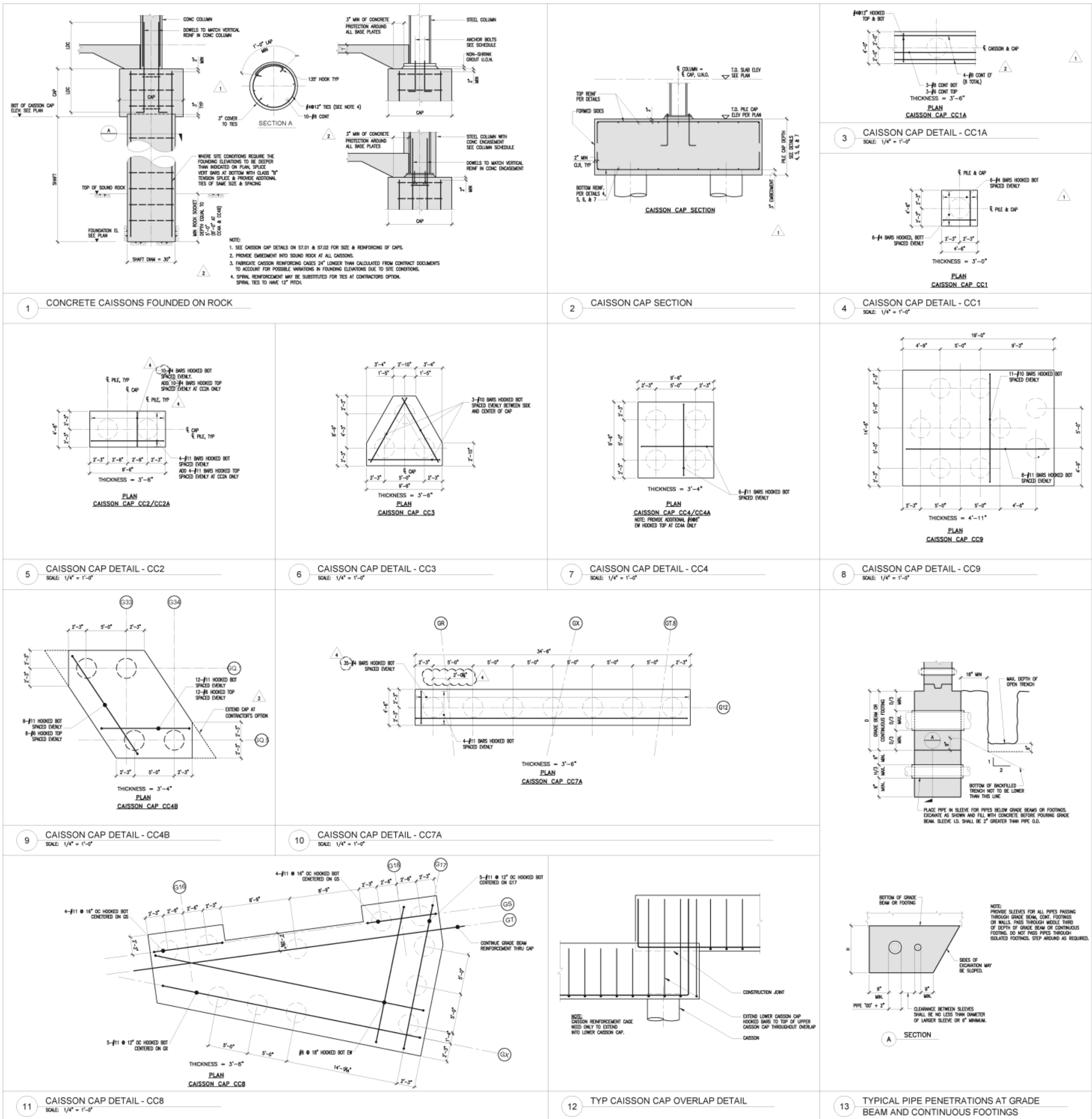
$$= 276\ K$$



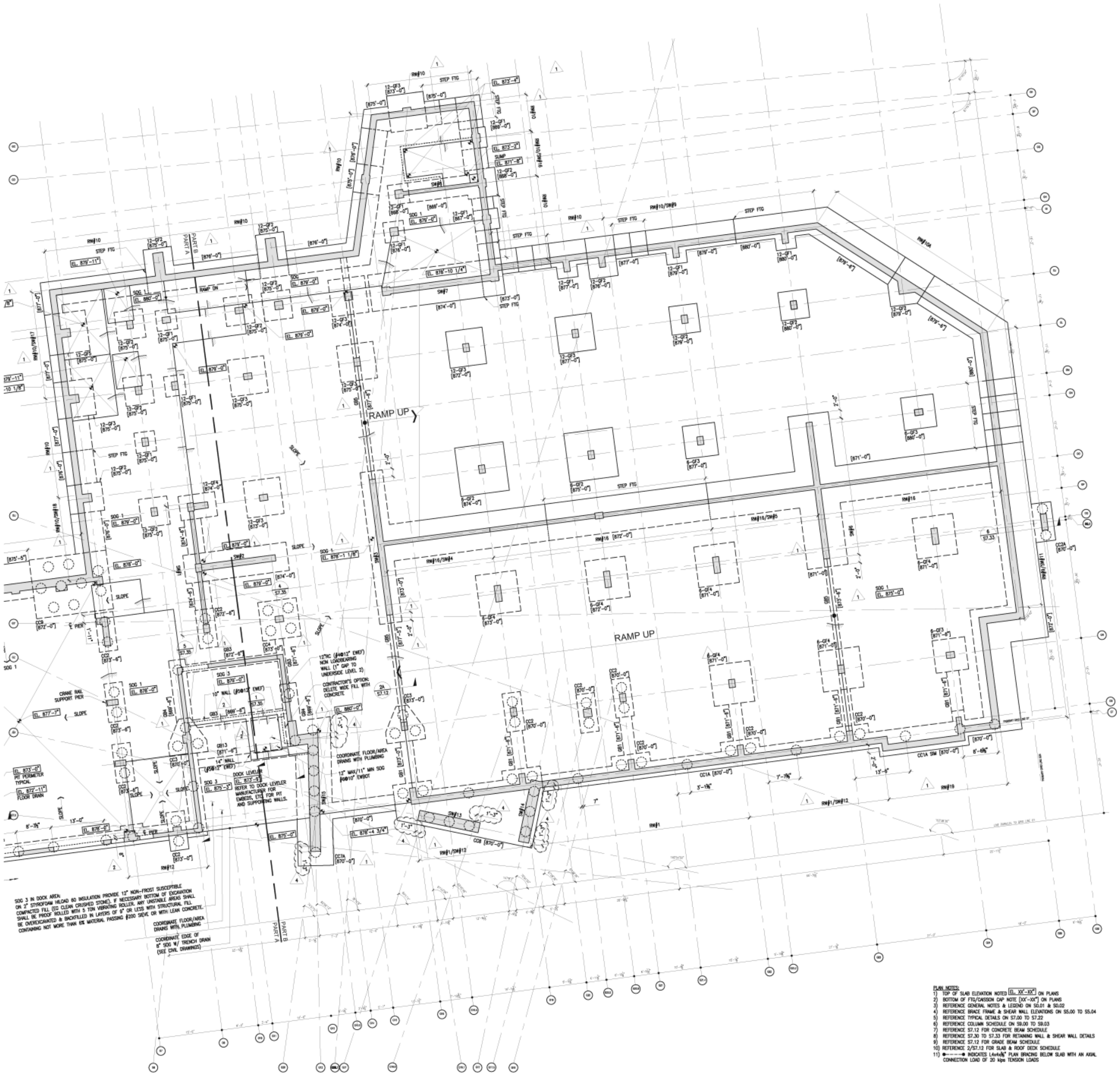
$P_{CR} = 276^K > P_u = 109^K$ OK! ✓

Appendix E: Typical Plans

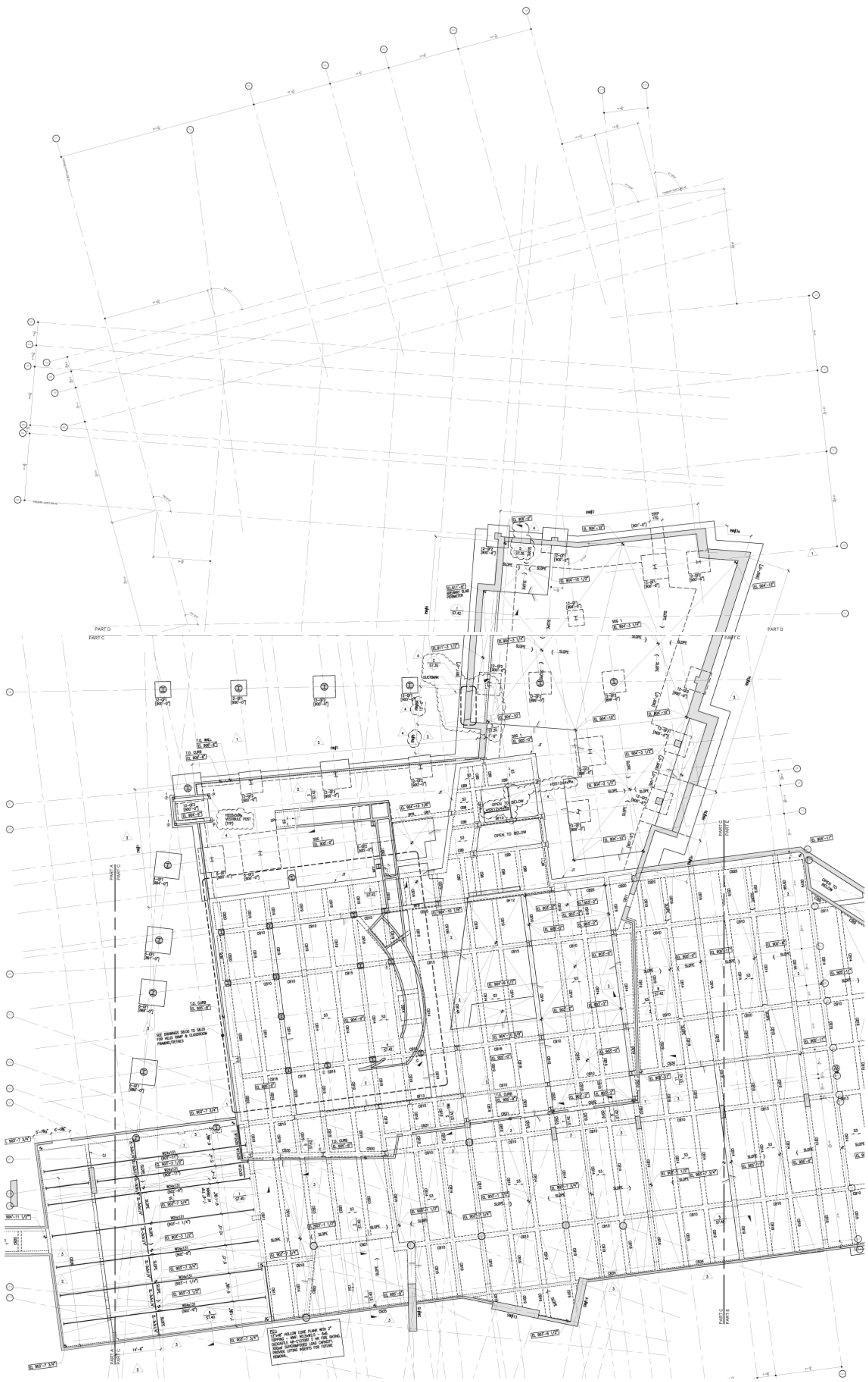
Foundations



Level 1 Foundation Plan

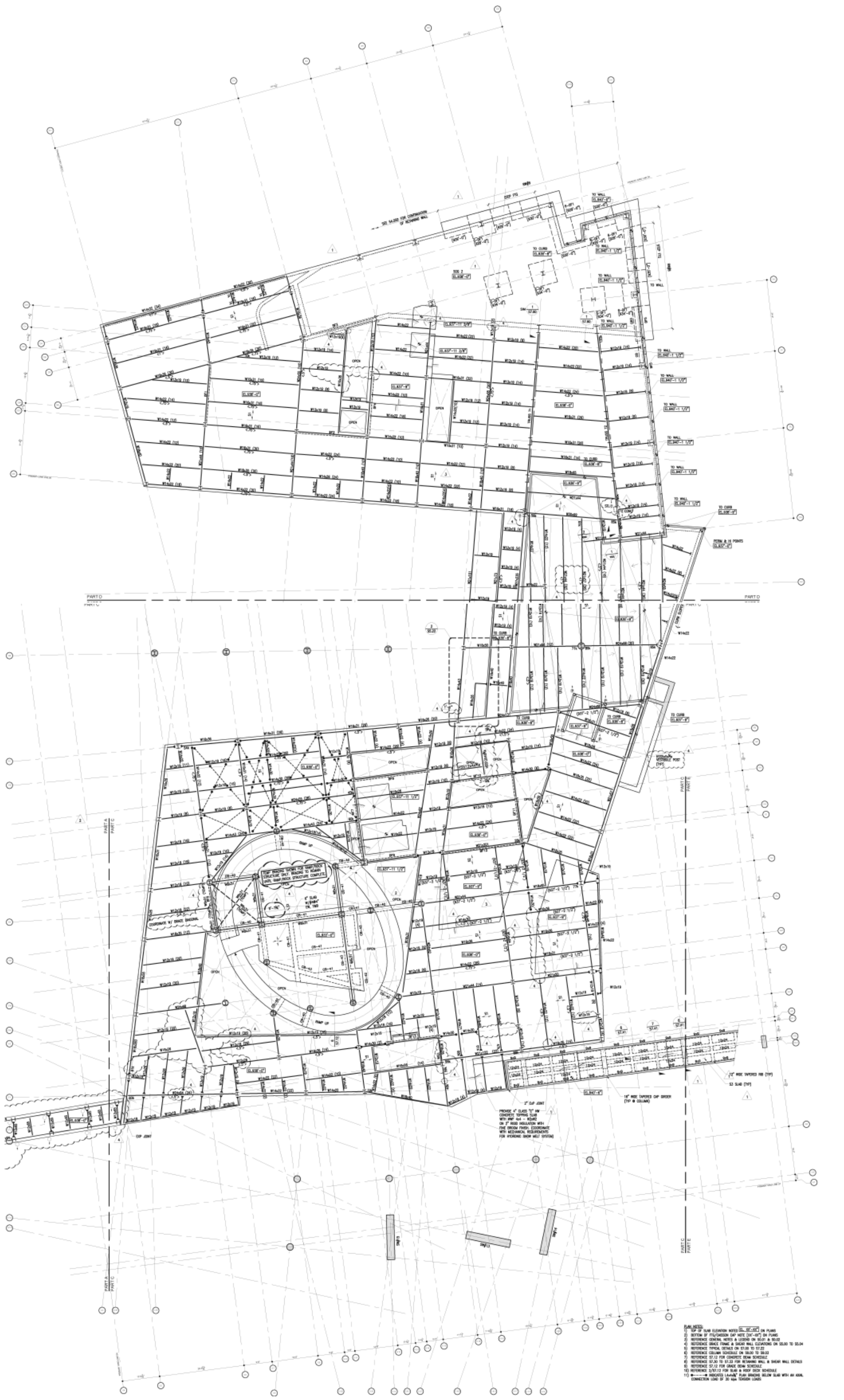


Level 3



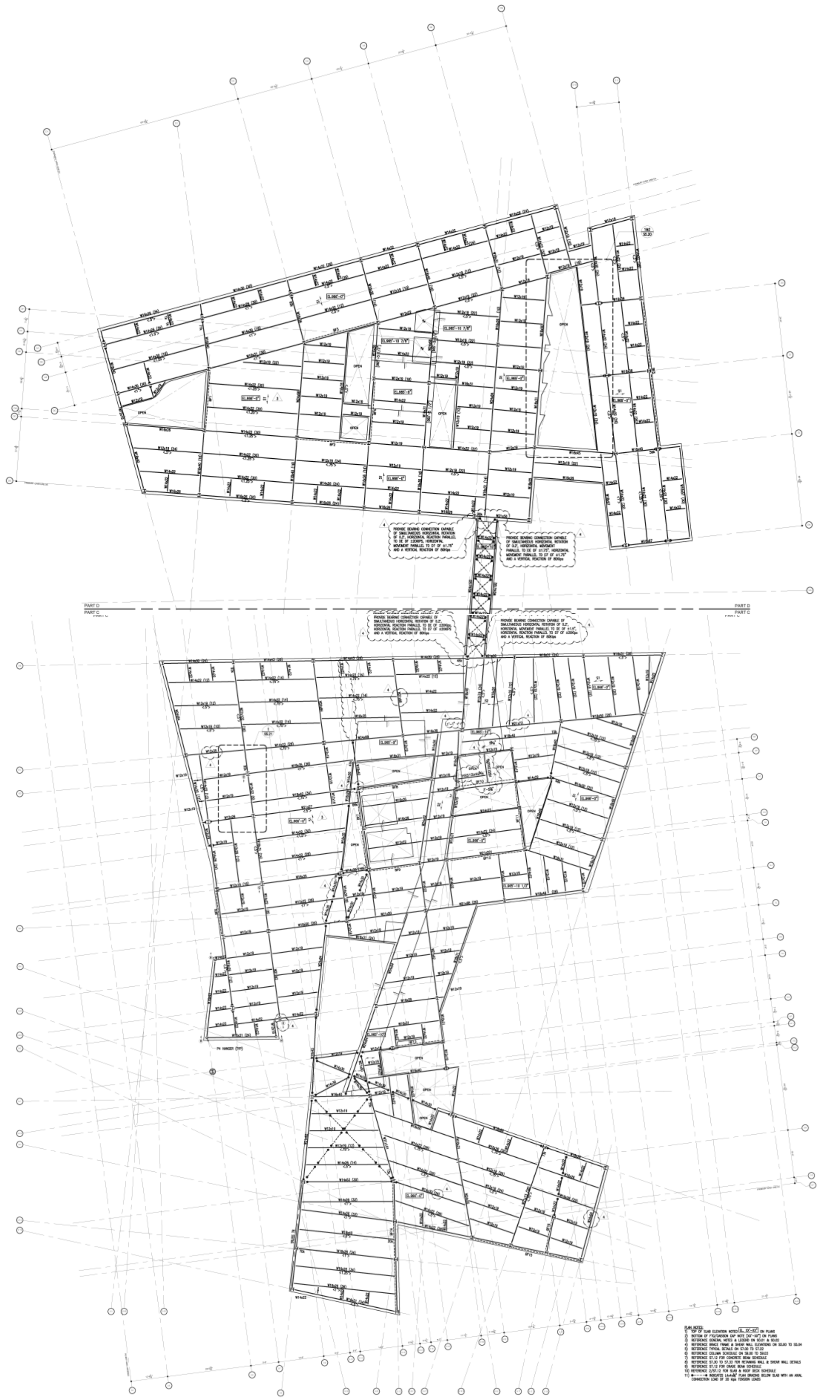
Level 4

Level 5

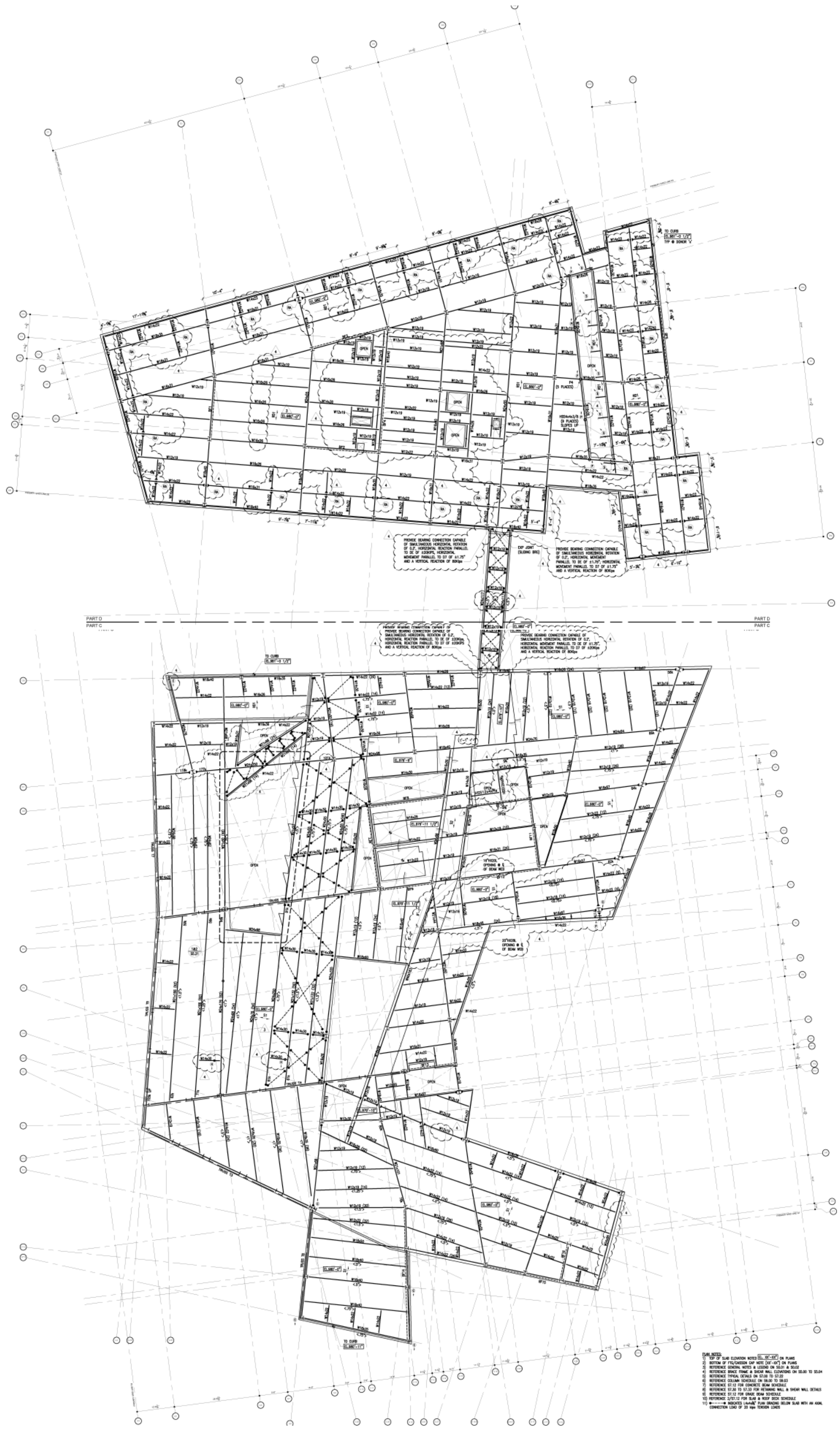


Level 6

Level 7



Level 8



Level 9