THE UNIVERSITY SCIENCES BUILDING

NORTHEASTERN, USA

Chris Dunlay – Structural

Faculty Consultant: Dr. Boothby

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Table of Contents

Executive Summary

Technical Report 1 is targeted to provide detailed information on the current structural system of The University Sciences Building (USB). This report is a compilation of descriptive calculations and figures detailing foundations, floor systems, framing systems, lateral systems, roof systems, design codes, and materials used for the construction of The USB.

Individual calculated structural element self-weights, along with provided superimposed dead loads were used to determine the overall weight of the building. Three designed gravity members were analyzed with applicable dead, superimposed dead, and live loads and were determined adequate. These members consisted of an interior column, slab, and beam,

Lateral loads were also calculated in accordance with ASCE 7 – 05 codes. In order to perform such calculations with the means necessary for this technical report, many assumptions were made under engineering judgment and are to be checked/compared in further technical reports. It was assumed that the layout of the building could be broken into two buildings. This was determined because the passage between the two would not be adequate to maintain rigidity of both buildings. The report designates the southernmost structure as Building 1 and the northern most as Building 2. Furthermore, the complexity of floor plans and elevations premised the assumption for simplified dimensions in wind calculations for both Building 1 and Building 2.

For seismic, Building 1 considered levels 3 – 9 (roof) for its weight with 50% assumed for level 3 (50% considered below grade). Building 2 considered floors 5 – 8 (roof) for its weight. Individual element weights (beams, columns, slabs, Façade) were calculated (only for Building 1 levels 3 – 6, Building 2 level 5). For simplification, a square footage weight (KSF) was found for those respective floors. This weight was then applied to the remaining floor's square footage to yield its individual floor weight (KIPS).

The drawings provide design base shears of 620.6 kips for building 1 and 176.3 kips for building 2. These values were compared to this reports analysis of a base shear; 609.6 kip for Building 1 and 207.1 kips for building 2. As noted in the report, the difference in the Seismic Response Coefficient and overall simplifying assumptions for building weight may have been the cause of any discrepancy. The calculations concluded that seismic would control over wind by a factor 1.65 and would serve as the basis for analyzing the lateral system in Technical Report 3.

The appendices provide hand calculations of member spot checks, wind, seismic, snow and typical drawings.

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 Universities standard building aesthetics revolve around a symmetrical layout and typically a beige brick veneer. The USB's extravagant cantilevers and complex building enclosures express the universities commitment to innovative architecture and sustainability but maintains a tasteful respect to surrounding buildings.


Figure 1 – Google Maps aerial view of site

The building was designed around the common idea of atriums 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 helix structure, which serves as a ramp to levels 3–5 with classrooms intermediately located through its core (Figure 2).


Figure 2 – Helix ramp

The sophisticated and 'edgy' design of the façade expresses the universities 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 floors different floor plan present's unique overhangs and cantilevers which really express the structure of the building (Figure 3). The placement of key structural components are intricately placed to preserve optimal function from floor to floor.

Structural Overview

The University Sciences Building sits upon a Site Class C (ASCE 7-05 Chapter 11) with drilled 30'' caissons, with caisson caps, spread, continuous, stepped, 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 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 foudations 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 universities soil has been geologically tested to show this. As time proceeded and the geotechincal report was released, it was found that the site class was actually C. 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 water which negates lateral pressure on surrounding foundation walls but maintains a compressive strength of 500 psi.

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.

Under these conditions it was determined that the building loads be carried from concrete columns to their respective caissons and interior column footings. For exterior perimeter caissons, they shall be 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 grade. Furthermore, the fill must be compacted to 95% of the dry density per ASTM D 1557. A vapor barrier is then required to lie 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 grades should 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). The lower floors that utilize concrete reinforced beams range in size from 50"x24" to 10"x12" 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 reoccurring slab is a composite 2"x18 GA deck with 4.5" normal weight concrete topping. Areas in levels 4 and 5 of Building 1 brace the metal decking between beams and girders with L4x4x3/8" with an axial tension connection load of 20 kips.

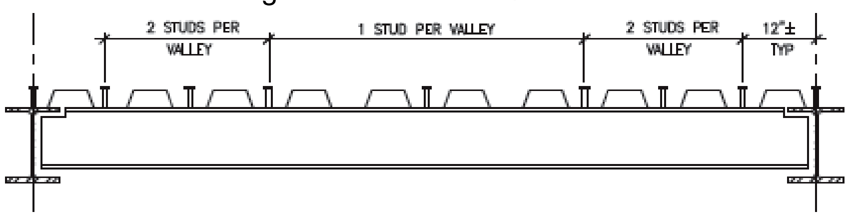


Figure 4. Perpendicular Decking Section – Case 3

The composite and non-composite slabs are placed with the ribs of the deck perpendicular to the infill beams within the bays. This proved to be a conflict in constructability 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 genres of columns, reinforced concrete, encased steel, and A572 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 concrete columns. This trend dissipates as you transverse up the building converting to steel 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. .

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.

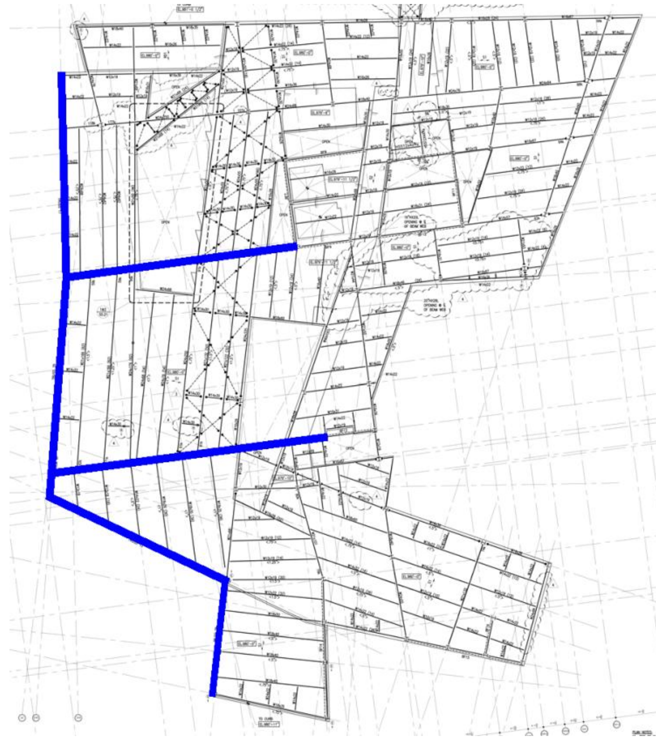


Figure 5. Highlighted truss elements from Building 1 Level 8.

Lateral System

The most common lateral force resisting system in The USB is braced frames. The USB utilizes 16 different brace 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.

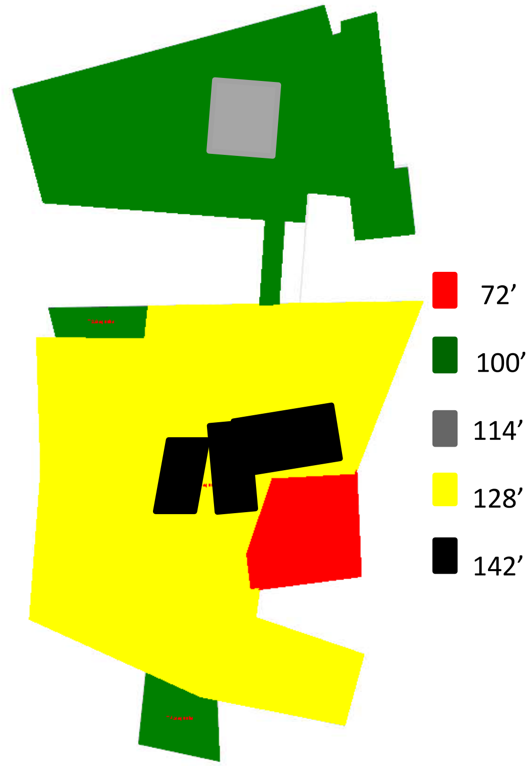
In Building 1 these braced frames are connected to shear walls were the load is taken from steel elements to concrete elements. These concrete elements are generated from the formed concrete walls lining the 147 parking spot garage. This provides a considerable weight for the building. All shear/retaining walls employed in building are kept on the lower floors, which has been assumed to retain the majority of the weight on a lower elevation.

Figure 6. Level 6 Braced Frames and Shear walls

This doesn’t hold true for three shear walls that start with a connection to a caisson cap at grade and rise 72’ 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 build 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 Mechanical Code with local amendments (IMC 2006)
- 2006 International Electrical Code with local amendments (IEC 2006)
- 2006 International Fuel Gas Code with local amendments (IFGC 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
Fly Ash	C618

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 was aimed 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 floors 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 member accuracy. 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 summated to yield a total building weight. The following tables give numerical description.

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	N/A	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	N/A	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.

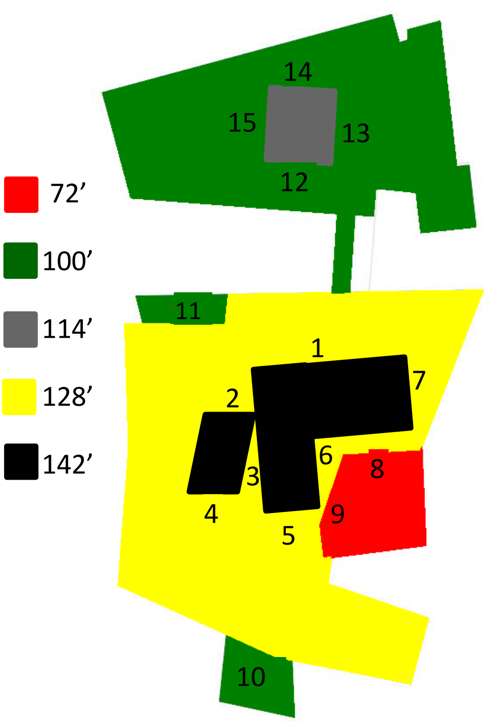
Flat Roof Snow Load Criteria			
Variable	Design Value	ASCE 7-05	Notes
Ground Snow Load, p_g (psf)	30	25	Fig -1 Conservative approach
Snow Exposure Factor, C_e	1.0	1.0	Table 7-2.
Snow Load Importance Factor, I_s	1.1	1.1	Table 7-4, Category III
Thermal Factor, C_t	1.0	1.0	Table 7-3, All other structures
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 Heighg, 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 isn't necessarily bad as it is 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



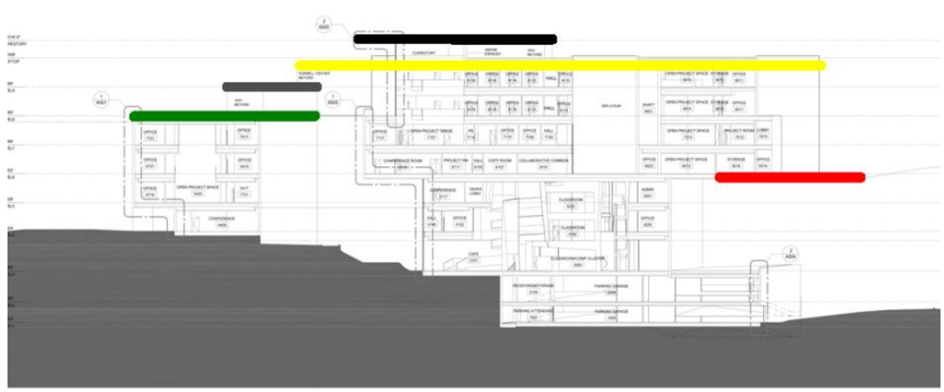


Figure 15. Elevation looking NE detailing roof elevations

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

Column Gravity Check

Due to the complexity of the floor framing layout the column chosen to analyze was along column lines GN and G18 which sits in a symmetrical bay (Figure 16). It was important to choose a column in a location in which the loads, for the most part, would be taken by this particular column. With the assumption of a pin pin connection on this column and the fact that it does not participate in the lateral force resisting system, it was found that second order effects were dismissed from the calculations. This particular column is a W12x72 and supports a tributary area of 715.5 SF. It initiates at level 3 of building 1 and rises to the top of level 5 where it concludes to a Green Roof. Loads considered in this calculation are superimposed dead loads, reduced live loads, self-weights of beams, slab/deck framed within the tributary area, and the column itself. The floor to floor heights of 14’ were used as its length. Refer to Appendix A for hand calculations.

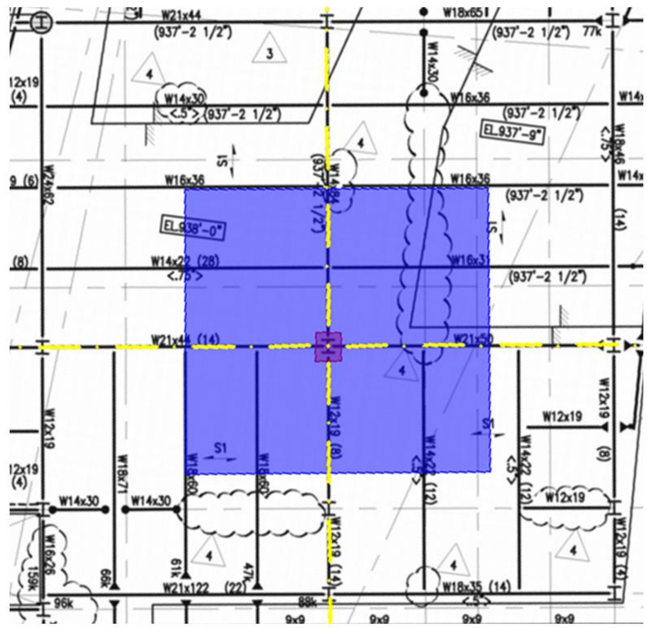


Figure 16. W12x72 Column and its tributary area

It was determined that the W12x72 column was adequate to carry the mentioned gravity loads. It is to be noted that design was highly based around the 200 psf superimposed dead load from the dead load (15” green roof) that is applied at the top of level 6. The AISC LRFD Steel Construction manual was used to carry out all appropriate calculations.

Composite Beam Gravity Check

In the interest of performing a beam gravity check in the complexity of the floor plans, a beam was chosen in the same typical bay that the column was. A pin pin connected W14x22 composite beam with a 2VLI18 Vulcraft deck (6.5” slab with 4.5” topping) was analyzed (Figure 17). Self-weights, superimposed dead loads, and live loads were used as the applicable loading on this particular beam with a tributary width of 7’ 3/8”.

Calculations were performed to check deck spans, unshored construction, flexure under construction load, composite design under full gravity load, shear stud allowance, live load deflection, and construction load construction, along with all necessary deflections (See Appendix A) . All of these checks proved the initial design to be adequate. Checking for composite action under full gravity load showed that the beam is more than appropriate for strength. A discrepancy in design moments way have resulted from constructability concerns. The required strength under construction loads controls the design of using a composite beam.

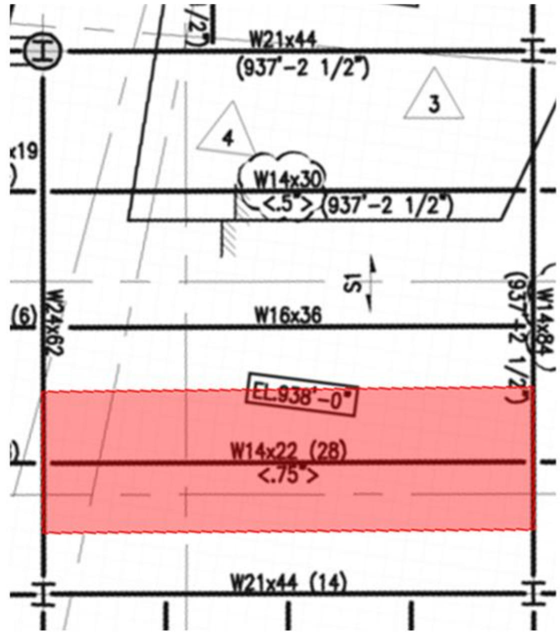


Figure 17. W14x22 Composite Beam and its tributary width

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

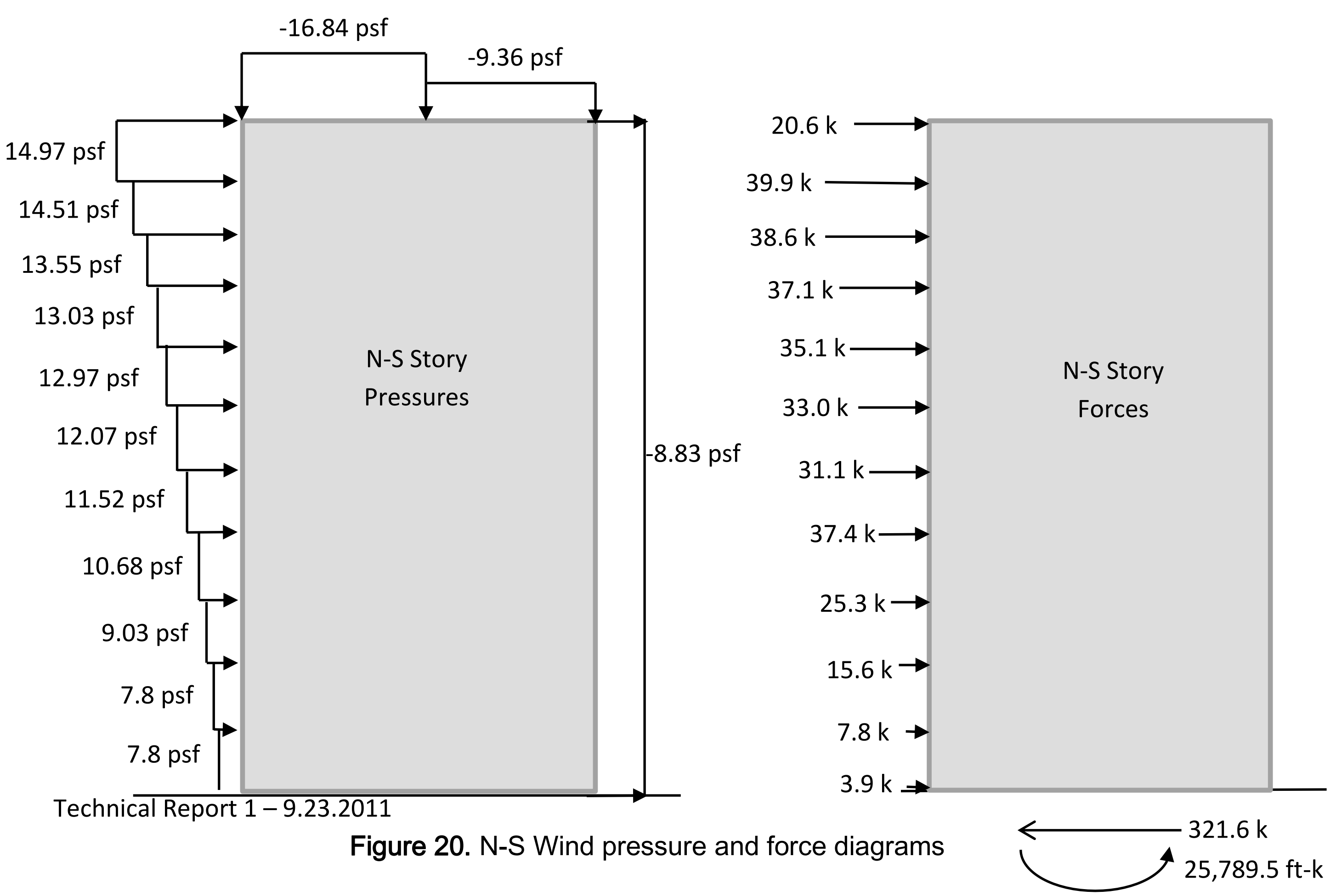


Figure 20. N-S Wind pressure and force diagrams

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

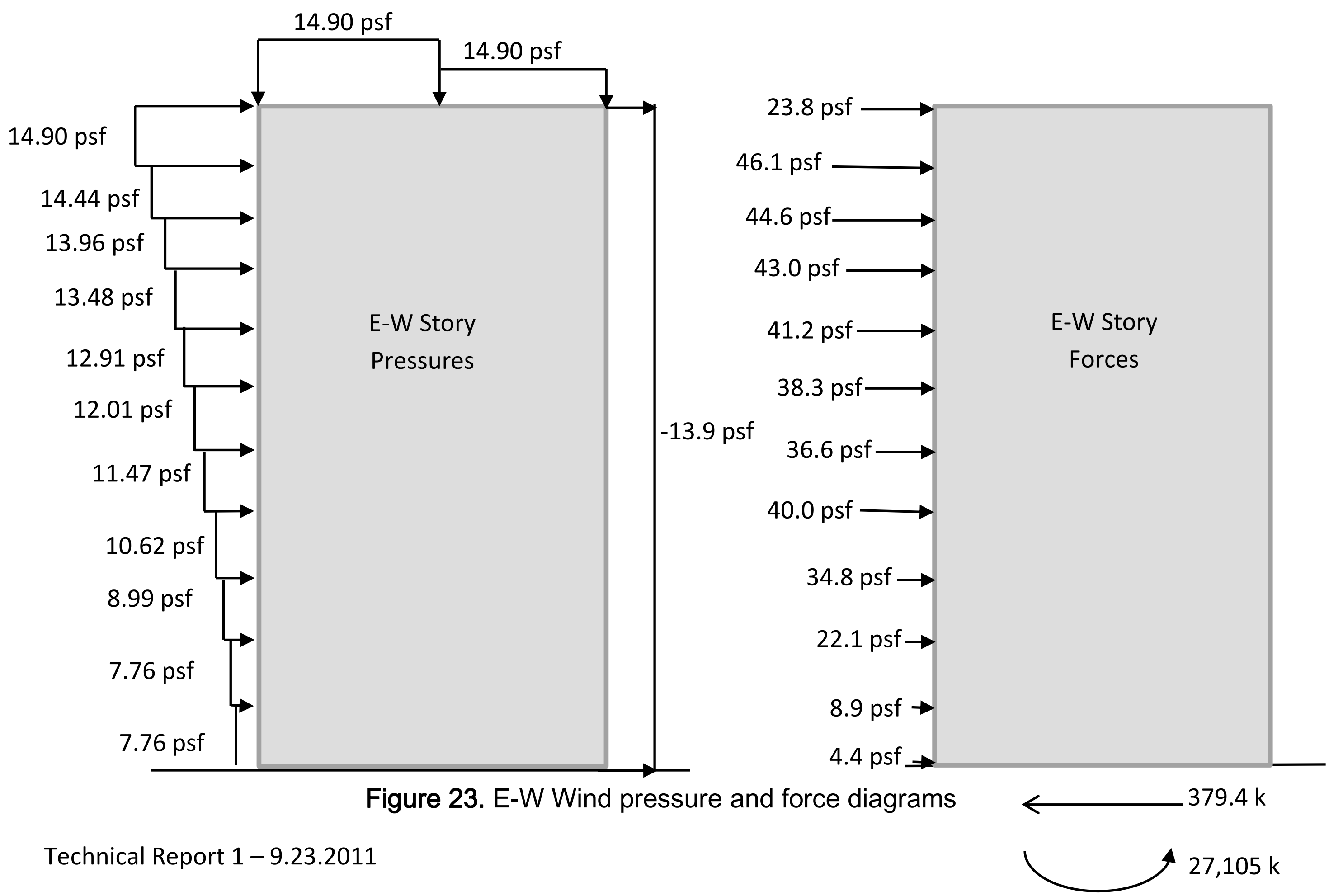


Figure 23. E-W Wind pressure and force diagrams

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

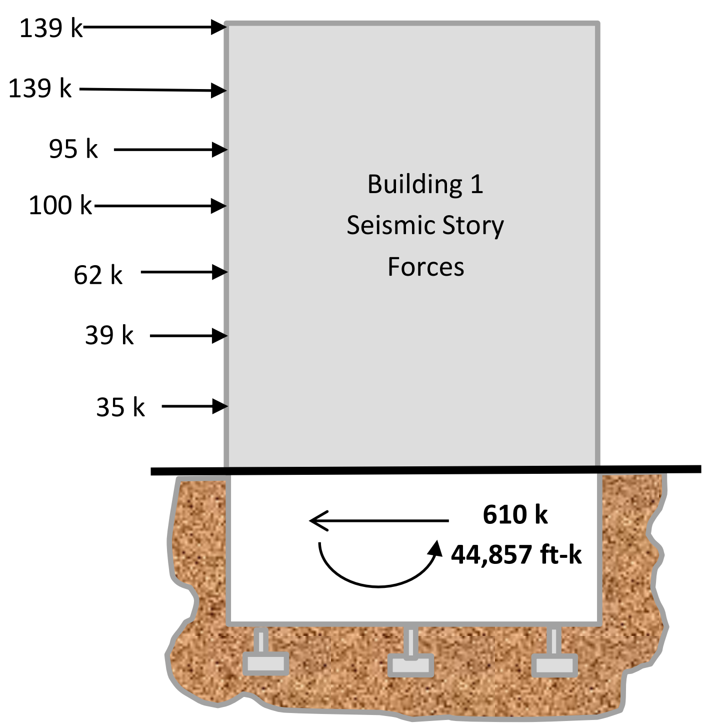


Figure 25. Seismic Force Distribution Loading Diagram

Conclusion

Technical Repot 1 was an accumulation of investigations on the structural system of The University Sciences Building. These systems included foundations, floor systems, framing systems, lateral systems, and roof systems, which were all summarized using detailed descriptions and figures to fully convey the purpose of each system. The complexity of floor plans and interaction between structural components made for an interesting investigation but premised assumptions to simplify proper calculations in this report, but is to be elaborated in further reports.

Alongside detailed descriptions and figures, calculations were provided to assist and determine the adequacy of particular gravity members. These members included a composite beam in a typical bay, a composite slab on metal deck, and a typical interior column. All of the calculations that were to comply with code were done so in reference with ASCE 7-05 and AISC Steel Construction Manual. All gravity member checks were not only calculated by self-weights, but also with dead, superimposed dead, and live loads. The superimposed dead and live loads that were provided on the structural drawings were compared and verified with ASCE 7-05 Chapters 3 and 4. All three members yielded results to verify their adequacy per the original design.

In addition to the gravity member calculations, wind and seismic loads were investigated. Once again, ASCE 7-05 was used to perform these calculations. For wind, overall assumptions were made to simplify calculations performed at this stage of the technical reports. Likewise, seismic calculations assumptions were made for simplifying the overall structural analysis. Technical Report 3 will branch into more detail pertaining wind and seismic loads and the lateral force resisting system. It was determined that seismic loads would control over wind by a factor of 1.65. There is no doubt that the seismic loads will control but the factor of which it does may change in further technical reports.

The information provided in Technical Report 1 is vital information for further exploration into the characteristics and behaviors of this structure and are to be elaborated as the analytical procedure continues.

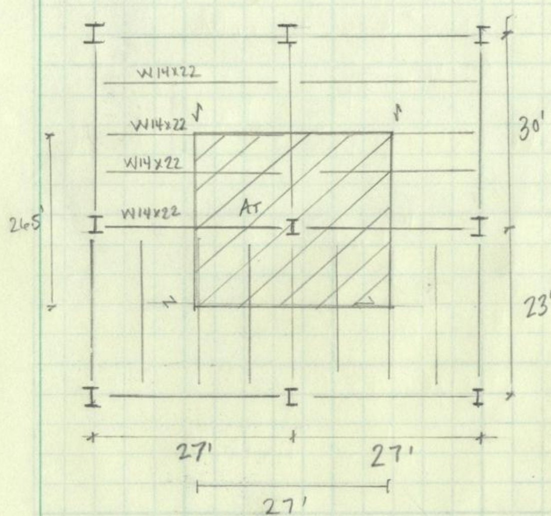
Appendix

Appendix A: Gravity Checks

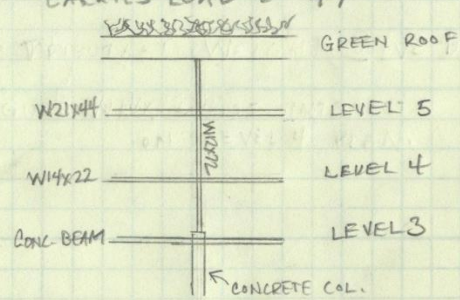
Column

1 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS - COLUMN

SPOT CHECK # 2 - INTERIOR COLUMN



- COLUMN - W12X72
- COLUMN SPANS FROM THE BASE OF 3 TO TOP OF 5
- A_T = 715.5 SF
- HEIGHT OF WHICH COLUMN CARRIES LOAD = 47'



- WEIGHT OF LEVEL 4
  - ↳ W14X22 BEAMS: 22 PLF x (117 LF) = 2.57^K
  - ↳ W12X72 COLUMN: 72 PLF x 14 LF + $\frac{14}{2}$ LF = 1.51^K
  - ↳ SLAB & SDL: 69 PSF + 35 PSF = 104 PSF
    - = 104 PSF x (715.5 SF) = 74.4^K
  - ↳ LIVE LOAD: 40 PSF $K_{LL} A_T = 4(715.5) = 2862$ SF

$$\text{REDUCTION} = \max \begin{cases} 0.4 \\ \left(0.25 + \frac{15}{\sqrt{2862}}\right) = 0.53 \rightarrow \text{CONTROLS} \end{cases}$$

LL = 40 PSF x 0.53 = 21.2 PSF

WEIGHT = 21.2 PSF x 715.5 SF = 15.2^K

↳ $P_{u4} = 1.2D + 1.6L = 1.2(2.57 + 1.51 + 74.4) + 1.6(15.2)$

$= 118.5^K$

2 of 4 | Chris Dunlay | Technical Report #1 | Spot Checks – Column

- Weight of Level 5
  - ↳ W14x22 beams: 22 PLF x 25 LF = 0.55^K
  - ↳ W16x36 beams: 36 PLF x 13.5 LF = 0.486^K
  - ↳ W16x31 beams: 31 PLF x 13.5 LF = 0.42^K
  - ↳ W21x44 beams: 44 PLF x 13.5 LF = 0.59^K
  - ↳ W21x50 beams: 50 PLF x 13.5 LF = 0.675^K
  - ↳ W18x40 beams: 40 PLF x 11.5 LF = 0.69^K
  - ↳ W12x19 beams: 19 PLF x 11.5 LF = 0.22^K

  3.63^K

  - ↳ Slab & SDL: (69 PSF + 35 PSF) x (715.5 SF) = 74.4^K
  - ↳ W12x72 column: 72 PLF x 14' = 1.01^K
  - ↳ Live load: 15.2^K
  - ↳ $Pu_5 = 1.2(3.63 + 74.4 + 1.01) + 1.6(15.2)$

    $= 119.2^K$

- Weight of Level 6 (15" Green Roof)
  - ↳ W14x26: 26 PLF x 67.5 LF = 1.76^K
  - ↳ W21x44: 44 PLF x 28.5 LF = 1.25^K
  - ↳ W24x55: 55 PLF x 13.5 LF = 0.742^K
  - ↳ W12x72 column: 72 PLF x $\frac{14}{2}$ LF = 0.5^K
  - ↳ Slab & SDL: (69 PSF + 200 PSF) x 715.5 SF = 192.5^K
  - ↳ Snow: 30 PSF x 715.5 SF = 21.5^K
  - ↳ Live load = 30 PSF x 0.53 = 15.9
  - ↳ $Pu_6 = 1.2(218.3) + 1.6(15.9) = 286.2\ K$

3 of 4 | Chris Dunlay | Technical Report #1 | Spot Checks – Column

- $P_{u,TOTAL} = 118.5^K + 119.2^K + 286.2^K$
  $= 524^K$
- From Table 4-1 AISC
  ↳ $A_g = 21.1 \text{ in}^2$
  $I_x = 597 \text{ in}^4$
  $I_y = 195 \text{ in}^4$
  $r_y = 3.04 \text{ in}$
  $r_x/r_y = 1.75$
  $r_x = 5.32''$

Assume: $K = 1.0$ → Conservative

Pin - Pin Connections

Check 1

- $\dfrac{KL}{r_x/r_y} = \dfrac{(1.0)(14')}{1.75} = 8.0$
- W12X72 @ 14' in Table 4-1
  $\phi P_N = 761^K > 524^K$

Check 2

- $\dfrac{KL}{r_x} = \dfrac{(1.0)(14)(12)}{5.32} = 31.6 \qquad \dfrac{KL}{r_y} = \dfrac{(1.0)(14)(12)}{3.04} = 55.3$ ↳ Controls
- $4.71\sqrt{\dfrac{E}{F_y}} = 4.71\sqrt{\dfrac{29000}{50}} = 113.4$
- $\dfrac{KL}{r_y} = 50.33 < 4.71\sqrt{\dfrac{E}{F_y}} = 113.4 \longrightarrow$ Inelatic Behavior
- Find Flexural Buckling Stress
  ↳ $F_e = \dfrac{\pi^2(29000)}{(55.3)^2} = 93.6 \text{ ksi}$
  ↳ $F_{CR} = \left[0.658^{(F_y/F_e)}\right]F_y = \left[0.658^{(50/93.6)}\right]50 = 39.98 \text{ ksi}$

4 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS – COLUMN

- NOMINAL STRENGTH

↳ $P_N = F_{CR} A = (39.98)(21.1 in^2) = 843.6 K$

$\phi P_N = (0.9)(843.6) = 759.3^K$

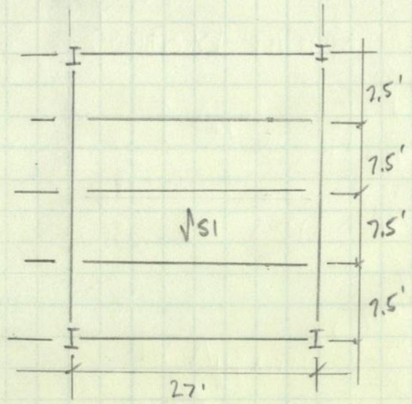
$P_u = 254^K < 759.3^K$ <u>OK!</u> ✓

↳ TABLE 4-1 AISC SHOWS THAT WITH THE GIVEN LOADING @ 14' AND A CALCULATE DESIGN NOMINAL STRENGTH OF 759.3^K, THE MOST APPLICABLE SELECTION IS A W12X72 WITH AN AVAILABLE STRENGTH OF 761^K.

$761^K \approx 759.3$ <u>GOOD!</u> ✓

Slab on Metal Deck

1 OF 1 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK - SLAB



SLAB ON METAL DECK REQUIREMENTS

↳ 20 GAUGE MINIMUM

↳ VULCRAFT

↳ SPAN NO GREATER THAN 9'-8"

↳ 3-SPANS WHERE POSSIBLE

↳ S1 SPECIFIES A 2"x18 6½" (4½ TOPPING) NORMAL WEIGHT CONCRETE DECK

↳ USE 2VLI18

↳ 2HR UNPROTECTED

- TOTAL UNFACTORED LOAD
  - ↳ DEAD = 35 PSF
  - ↳ LIVE = 40 PSF

  } TOTAL = 75 PSF

VULCRAFT 2008 MANUAL

- SDI MAX UNSHORED CLEAR SPAN
  - ↳ 3-SPAN FOR 2VLI18

    = 10'6" > 7'6" OK! ✓

- SUPERIMPOSE LOADS CLEAR SPAN
  - ↳ 2VLI18 @ 7'6" CLEAR SPAN.

    = 400 PSF

    400 PSF > 75 PSF OK! ✓

- FIRE RATING
  - ↳ 2HR UNPROTECTED DECK w/ 4½" TOPPING.

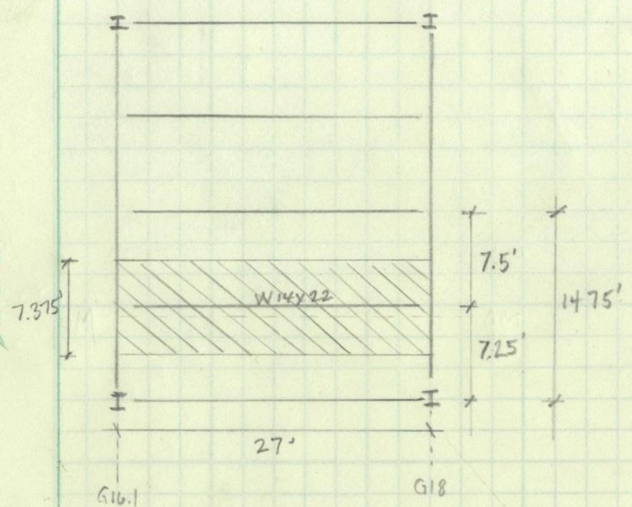
    2VLI IS AVAILABLE OK! ✓

Composite Beam

1 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS–BEAM

SPOT CHECK #1: COMPOSITE BEAM

↳ LOCATION: BUILDING 1, LEVEL 5, BW G16.1 & G18 ON GM.



LOADING:

↳ w_D – S.I. DL = 35 PSF

– S.W. = 69 PSF

↳ 2VLI18 VULCRAFT COMP. DECK

\+ ______

104 PSF

↳ w_L = 40 PSF

LIVE LOAD REDUCTION:

↳ $A_T = 27' \times 7.375' = 199\ ft^2$

↳ $K_{LL} = 2$ (INT. BEAM)

↳ $K_{LL}A_T = 398\ ft^2 < 400\ ft^2$

↳ NOT NEEDED!

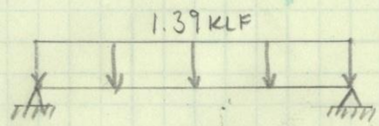
↳ $w_u = 1.2D + 1.6L$ → CONTROLING LOAD COMBO

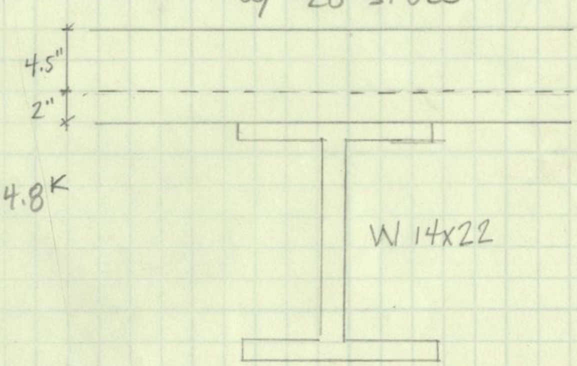
$= 1.2(104) + 1.6(40) = 188.8$ psf → CLEAR SPAN of 9.5' PERMISSES

$= \frac{(188.8)(7.375)}{1000} = 1.39$ KLF

243 psf > 188.8 psf OK! ✓

CONFIRM SELECTION OF W14x22 w/ 28 STUDS





$V_u = w_u\left(\frac{l}{2}\right) = (1.39)\left(\frac{27}{2}\right)$

$\boxed{= 18.8^K} < \phi V_n = 94.8^K$

$M_u = \frac{w_u l^2}{8} = \frac{(1.39)(27)^2}{8}$

$\boxed{= 126.7\ ft\text{-}K}$

2 of 4 | Chris Dunlay | Technical Report #1 | Spot Checks - Beam

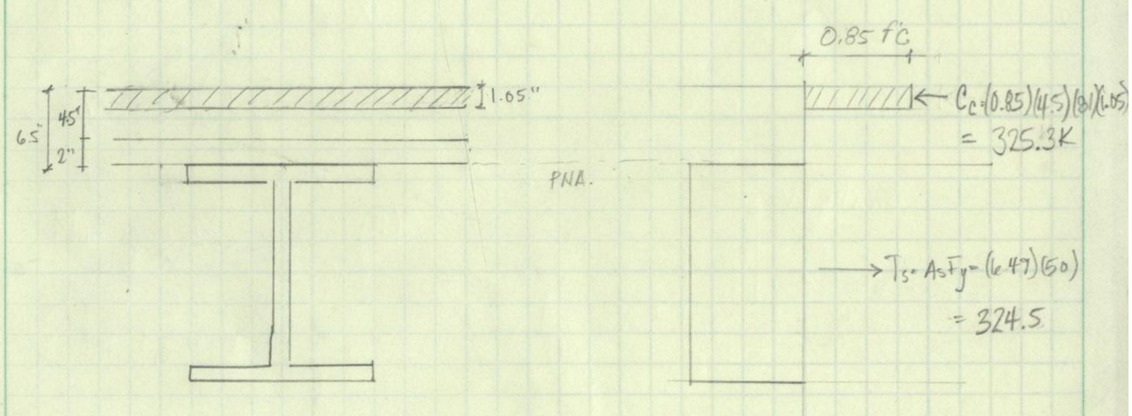
↳ $b_{eff} =$ MIN
- $\frac{\text{SPAN}}{4} = \frac{(27)(12)}{4} = 81''$ ⟶ CONTROLS
- $\frac{\text{BM SPACING}}{2} = \frac{(14.75)(12)}{2} = 88.5''$

↳ REQUIRED STRENGTH UNDER UNSHORED CONSTRUCT. (D+L) (NON-COMP)

- $w_u = 1.2[(69\text{ psf} \times 7.5') + 22\text{ PLF}] + 1.6[(20\text{ psf} \times 7.5')] = 0.89$ PLF
- $M_u = \frac{(0.89)(27)^2}{8} = 81$ ft-K
- $\phi M_b = 125'K > 81$ ft-K → DOES NOT NEED SHORING! ✓

↳ CONFIRM COMPOSITE DESIGN @ FULL GRAVITY LOAD

- $C_c = 0.85 f'_c b_{eff} t = (0.85)(4.5)(81)(4.5) = 1394^K$
- $T_s = A_s F_y = (6.49)(50) = 324.5^K$ → CONTROLS ΣQ_N
  PNA is in Concrete
- $a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{325}{(0.85)(4.5)(81)} = 1.05''$
- $Y_2 = 6.5'' - \frac{1.05}{2} = 5.98'' \approx 6.0''$ ⟶
- $A_{s-c} = \frac{324.5 - 325.3}{2(50)} = 0$ ⟶ PNA IS TFL
- $\phi M_n = 313$ ft-K > 126.7 ft-K



3 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS-BEAM

↳ CONFIRM STUDS

- (28) 3/4" ϕ SHEARS ACROSS 27' (1 PER RIB)
- $Q_N = 0.5 A_{sc}\sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u$; $A_{sc} = \frac{\pi(3/4)^2}{4} = 0.442\ in^2$

  $0.5(0.442)\sqrt{4.5(3704)} \leq (1.0)(1.0)(0.44)(65)$ $E_c = 145^{1.5}\sqrt{4.5} = 3704\ ksi$

  $= 28.53 \leq 28.73$ ✓ OK!
- # OF STUDS = $\frac{\Sigma Q_N}{Q_N} \times 2 = \frac{324.5}{28.53} = 11.3 \rightarrow 12 \times 2 = 24$ STUDS

  ↳ DESIGN REQUIRES 24 STUDS ALONG ITS LENGTH BUT STRUCTURAL DWGS CALL FOR 28. THIS AMOUNT WAS CHOOSEN SO A STUD CAN BE PLACED IN ONE RIB (12") OVER ITS 27' SPAN.

  OK! ✓

↳ CHECK LIVE LOAD DEFLECTIONS

- $\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E_c I_{LB}}$; $I_{LB} = 737\ in^4 \rightarrow$ TABLE 3-20 AISC

  $E_c = 29,000\ ksi$

  $w_{LL} = \frac{(40)(7.375)}{1000} = 0.295\ k/ft$

  $= \frac{5(0.295)(27)^4}{384(29000)(737)}(1728)$

  $= 0.165\ in$
- $\Delta_{LL,\text{ALLOWABLE FLOORS}} = \frac{L}{360} = \frac{(27)(12)}{360} = 0.9\ in$

  $0.165\ in < 0.9\ in$ OK! ✓

↳ CHECK CONSTRUCTION LOAD DEFLECTIONS

- $w_D = 69$ PSF x $(7.375') = 508.9$ PLF SLAB WEIGHT

  + 22 PLF → BEAM WEIGHT

  530.9 PLF
- $w_{LL} = 20$ PSF x $(7.375') = 147.5$ PLF

4 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECKS - BEAMS

- $w_u = 1.2(0.531) + 1.6(0.147) = 0.872$ plf
- $M_u = \dfrac{w_u l^2}{8} = \dfrac{(0.872)(27)^2}{8} = 79.5$ ft-K

↳ $\phi_b M_p = 125$ ft-K > 79.5 ft-K <u>OK!</u> ✓

- $\Delta_{DL} = \dfrac{5 w_D L^4}{384 E I}$ $E = 29000$ ksi, $I = 199$ in^4 → FROM TABLE 1-1 AISC

$$= \frac{5(0.531)(27)^4}{384(29000)(199)}(1728) = 1.1'' - \underset{\text{CAMBER}}{0.75''} = 0.35''$$

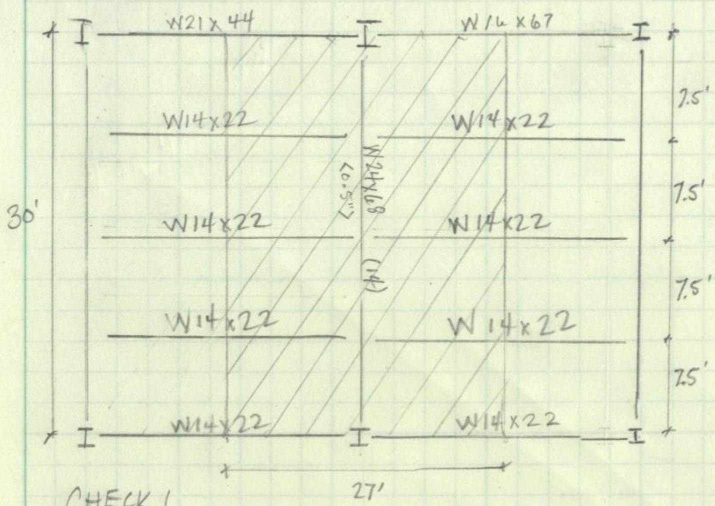
- $\Delta_{DL,\,ALLOW} = L/180 = \dfrac{(27)(12)}{180} = 1.8''$

↳ $1.8'' > 1.1'' > 0.35''$ <u>OK!</u> ✓

<u>CONCLUSION:</u> THESE CALCULATIONS CONFIRM THAT A W 14x22 COMPOSITE BEAM WITH 28 STUDS ACROSS ITS 27' SPAN IS ADEQUTE.

Girder

1 of 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK - GIRDER



CHECK 1

- DEAD LOAD ON W14x22 BEAMS
  - ↳ SUPERIMPOSED = 35 PSF × (27') × (7.5') = 7.09^K
  - ↳ BEAM S.W. = 22 PLF × (27') = 0.59^K
  - ↳ SLAB = 69 PSF × (27') × (7.5') = 13.97^K
- LIVE LOAD
  - ↳ K_{LL} = 2 (INTERIOR BEAMS
  - ↳ A_T = (27)(7.5) = 810 ◻' } $K_{LL}A_T$ = 405 ◻'
  - ↳ $L = 40\,\text{psf}\left[0.25 + \dfrac{15}{\sqrt{405}}\right] = 39.8\ \text{PSF} \times (27') \times (7.5') = 8.06^K$
- $P_u = 1.2(7.09^K + 0.59^K + 13.97^K) + 1.6(8.06^K) = \boxed{38.9^K}$

3 OF 4 | CHRIS DUNLAY | TECHNICAL REPORT #1 | SPOT CHECK – GIRDER

• SOLVE FOR a

$$a = \frac{\Sigma Q_N}{0.85 f'_c b_{eff}} \quad , \quad b_{eff} = \min \begin{cases} \frac{(30)(12)}{4} = 90'' \\ \frac{7.5' + 7.5'}{2}(12) = 90'' \rightarrow \text{USE} \end{cases}$$

$$= \frac{199.7}{0.85(4.5)(90)} = 0.58''$$

• $Y_2 = 6.5'' - \frac{0.58''}{2} = 6.21''$

TOTAL SLAB DEPTH

• $M_N = 199.7\left(\frac{23.7}{2}\right) + 199.7\left(6.5 - \frac{0.58}{2}\right)$

$= 3606.$ IN-KIPS $= 300.5$ ft-K

$\phi M_N = 0.9(300.5 \text{ ft-K}) = 270$ ft-K

$\boxed{270 \text{ ft-K} \ll 664 \text{ ft-K}}$ OK! ✓

CHECK 2

• UNSHORED CONSTRUCTION

↳ $M_u = 587.5$ ft-K (FROM PAGE 2)

↳ $\phi M_p = 664$ ft-K (TABLE 3-19)

587.5 ft-K < 664 ft-K OK! ✓

• UNDER FULL GRAVITY LOAD

↳ $C_c = 0.85 f'_c b_{eff} t = 0.85(4.5)(90)(4.5) = 1550$ K

↳ $T_s = A_s F_y = (20.1)(50) = 1005$ K → CONTROLS ΣQ_N

• PNA IS CONC.

• TFL

↳ $a = \frac{\Sigma Q_N}{0.85 f'_c b_{eff}} = \frac{1005}{0.85(4.5)(90)} = 2.92''$

↳ $Y_2 = 6.5 - \frac{2.92}{2} = 5.04 \approx 5.00$

↳ WITH $Y_2 = 5.00$ & TFL → $\phi M_n = 1270$ ft-K $\gg 587$ ft-K

OK! ✓

- CHECK LIVE LOAD DEFLECTIONS

$\hookrightarrow \Delta_{LL} = \dfrac{5 w_{LL} L^4}{384 E I_{LB}}$; $I_{LB} = 4680 \text{ in}^4$ (TABLE 3-20 AISC)

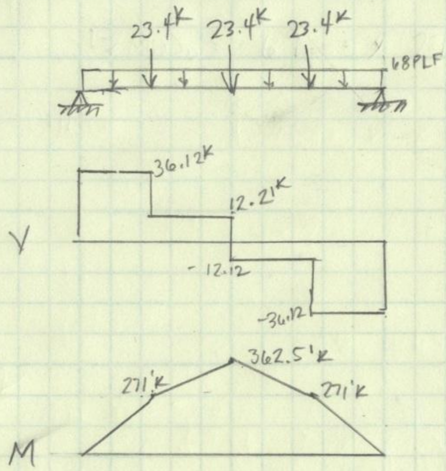
$w_{LL} = \dfrac{(40)(27')}{1000} = 1.08$ KLF

$= \dfrac{5(1.08)(30)^4}{384(29000)(4680)}(1728) = 0.145''$

$\hookrightarrow \Delta_{LL, ALLOW} = \dfrac{l}{360} = \dfrac{(30)(12)}{360} = 1.00''$

$\underline{0.145'' < 1.00''}$ OK! ✓

- CHECK CONSTRUCTION LOAD DEFLECTIONS



$P_D = 69 \text{ PSF} \times 7.5' \times 27' = 13.97^K$

$w_D = 68$ PLF

$P_L = 20 \text{ PSF} \times 7.5' \times 27' = 4.05^K$

$P_u = 1.2(13.97) + 1.6(4.05) = 23.24^K$

$M_u = 362.5$ ft-K

$\phi M_p = 664'K$

$\underline{664 \text{ ft-K} > 362.5 \text{ ft-K}}$ OK! ✓

$\hookrightarrow W_{DL} = \underbrace{(0.0198 \text{ PLF})}_{\text{W14x22 BEAMS}} + (69\text{psf})(27') + 0.068 \text{ PLF} = 1.86$ PLF

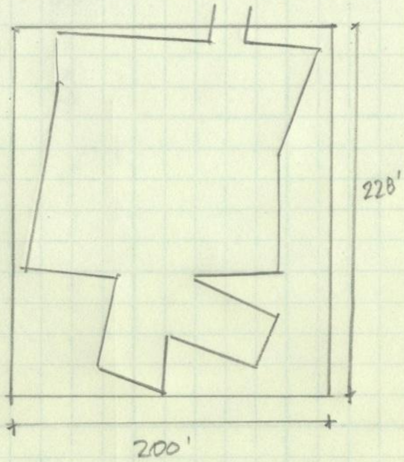
$\hookrightarrow \Delta_{DL} = \dfrac{5 w_{DL} L^4}{384 EI}$; $E = 29000$ KSI ; $I = 1830 \text{ in}^4$

$= \dfrac{5(1.86)(30)^4}{384(29000)(1680)}(1728) = 0.63''$

$\hookrightarrow \Delta_{DL, ALLOW} = \dfrac{l}{180} = \dfrac{(30)(12)}{180} = 2'' \rightarrow 2'' > 0.63''$ OK! ✓

Appendix B: 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

$$\left.\begin{array}{l} \circ\ q_z = 0.00256 K_z K_{zT} K_d V^2 I \\ \circ\ q_h = 0.00256 K_h K_{zT} K_d V^2 I \end{array}\right\} \text{SEE EXCEL}$$

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

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

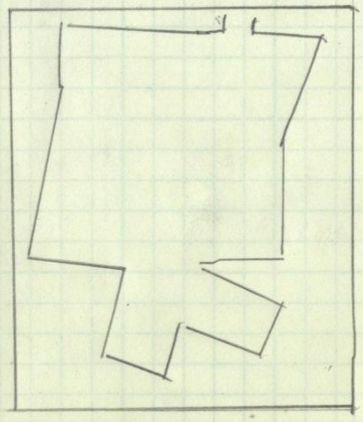
    $\boxed{= 0.844}$

- 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 C: 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

$S_{DS} < 0.167 \rightarrow$ CATEGORY A

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

USE EQUIVALENT LATERAL FORCE ANALSIS (

↳ APPROXIMATE PERIOD (TABLE 12.8-2)

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

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

- FREQUENCY $\rightarrow f = 1/T_a = 1.55$

↳ SEISMIC RESPONSE COEFFICENT.

- $C_u = 1.7 \rightarrow$ (TABLE 12.8-1 $S_{D1} \leq 0.1$)

2 OF 2 CHRIS DUNLAY TECHNICAL REPORT #1 SEISMIC CALCULATIONS

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

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

  T=

  $\rightarrow$ SEE CALC

- BASE SHEAR

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

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

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

- SEE EXCEL SHEET FOR REMAING CALCS & BUILDING 2

Appendix D: Snow Calculations

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

↳ GROUND SNOW LOAD, p_g → 30 psf (FIGURE 7-1)

↳ EXPOSURE FACTOR, C_e → 1.0 (TABLE 7-2)

↳ THERMAL FACTOR, C_t → 1.0 (TABLE 7-3)

↳ IMPORTANCE FACTOR, I → 1.1 (TABLE 7-4)

↳ FLAT ROOF SNOW LOAD

- $p_f = 0.7\, C_e C_t I\, p_g$

$= (0.7)(1.0)(1.0)(1.1)(30)$

$= \underline{\underline{23.1}}$ psf → STRUCTURAL DRAWING USE $\underline{\underline{27}}$

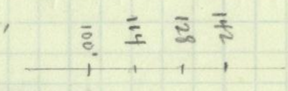
↳ SNOW SPECIFIC GRAVITY

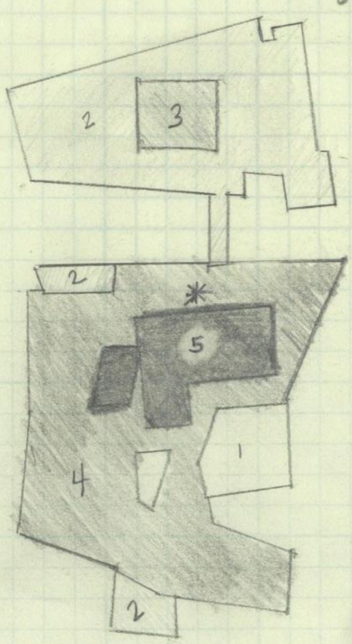
- $\gamma = 0.13 p_g + 14$

$= (0.13)(30) + 14 = \boxed{17.9 \text{ pcf}}$

↳ BASE SNOW ACCUMULATION HT

- $h_b = \dfrac{p_f}{\gamma} = \dfrac{23.1}{17.9} = 1.29'$





CALCULATION FOR "✱"

ROOF 5 → 4

$h_r = 142' - 128' = 14'$

$h_c = h_r - h_b = 14' - 1.29' = 12.71'$

$\dfrac{h_c}{h_b} = \dfrac{12.71'}{1.29} = 9.85' < 25'$

USE $\underline{\underline{25'}}$

CHRIS DUNLAY | TECHNICAL REPORT #1 | SNOW CALCULATIONS

WINDWARD — LOWER ROOF DIST

$l_u = 18' < 25 \rightarrow \text{USE } 25'$

$h_d = 0.75\left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5\right)$

$= 0.75\left(0.43\sqrt[3]{25}\sqrt[4]{30+10} - 1.5\right)$

$= 1.25' < h_c$ GOOD!

$w_d = 4h_d = 4(1.25) = 5.00$

$p_d = 1.25(17.9) = 22.4$

LEEWARD — TALLER ROOF DIST

$l_u = 28.5' > 25'$ OK!

$h_d = 0.75\left(0.43\sqrt[3]{28.5}\sqrt[4]{40} - 1.5\right)$

$= 1.35 < h_c$ GOOD!

$w_d = 4(1.35) = 5.4' < 14'$ OK!

$p_d = 1.35(17.9) = 24.2 \text{ psf}$

TOTAL SNOW LOAD

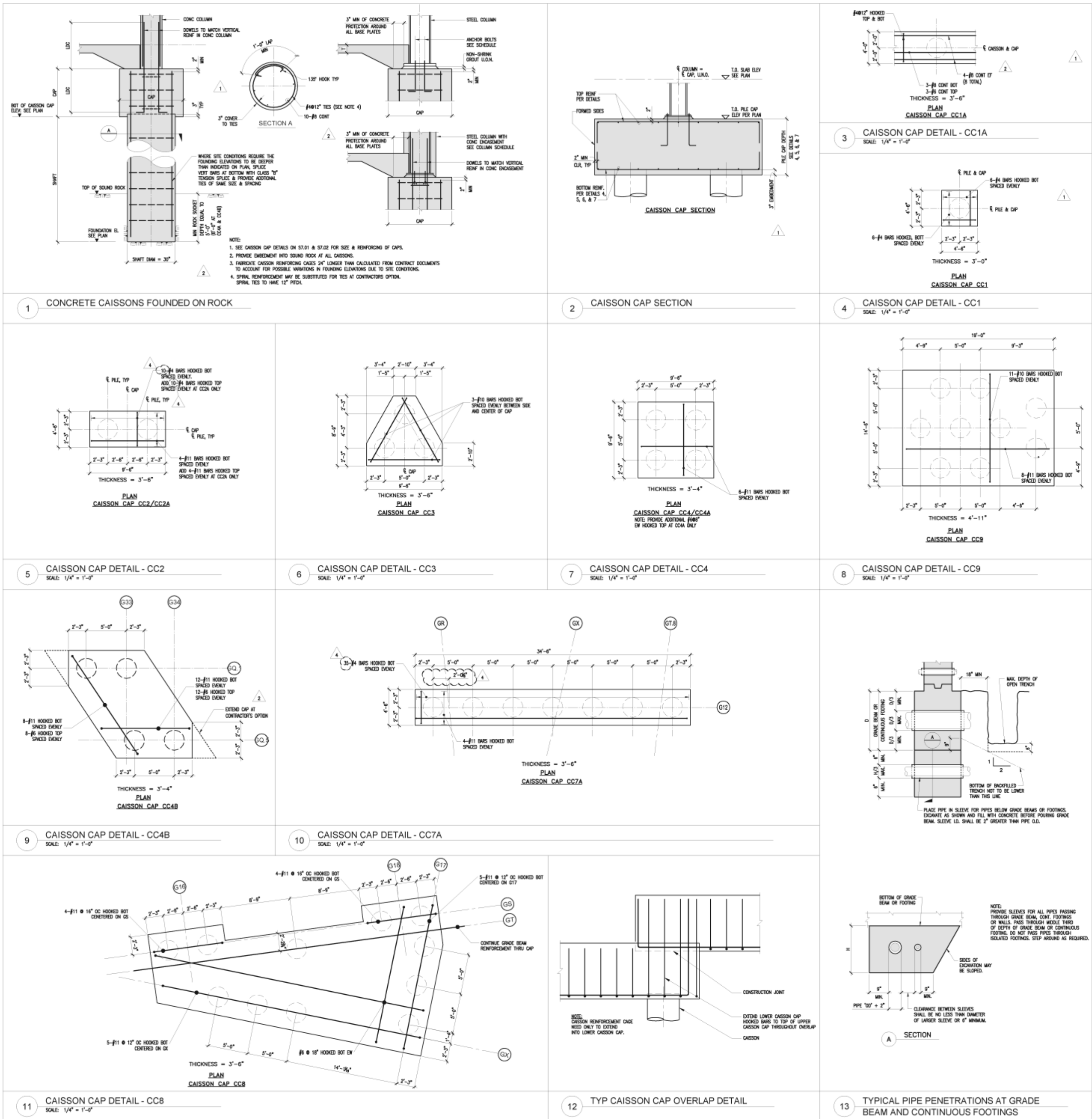
$= 24.2 \text{ psf} + 23.1 \text{ psf}$

$= 47.3 \text{ psf}$

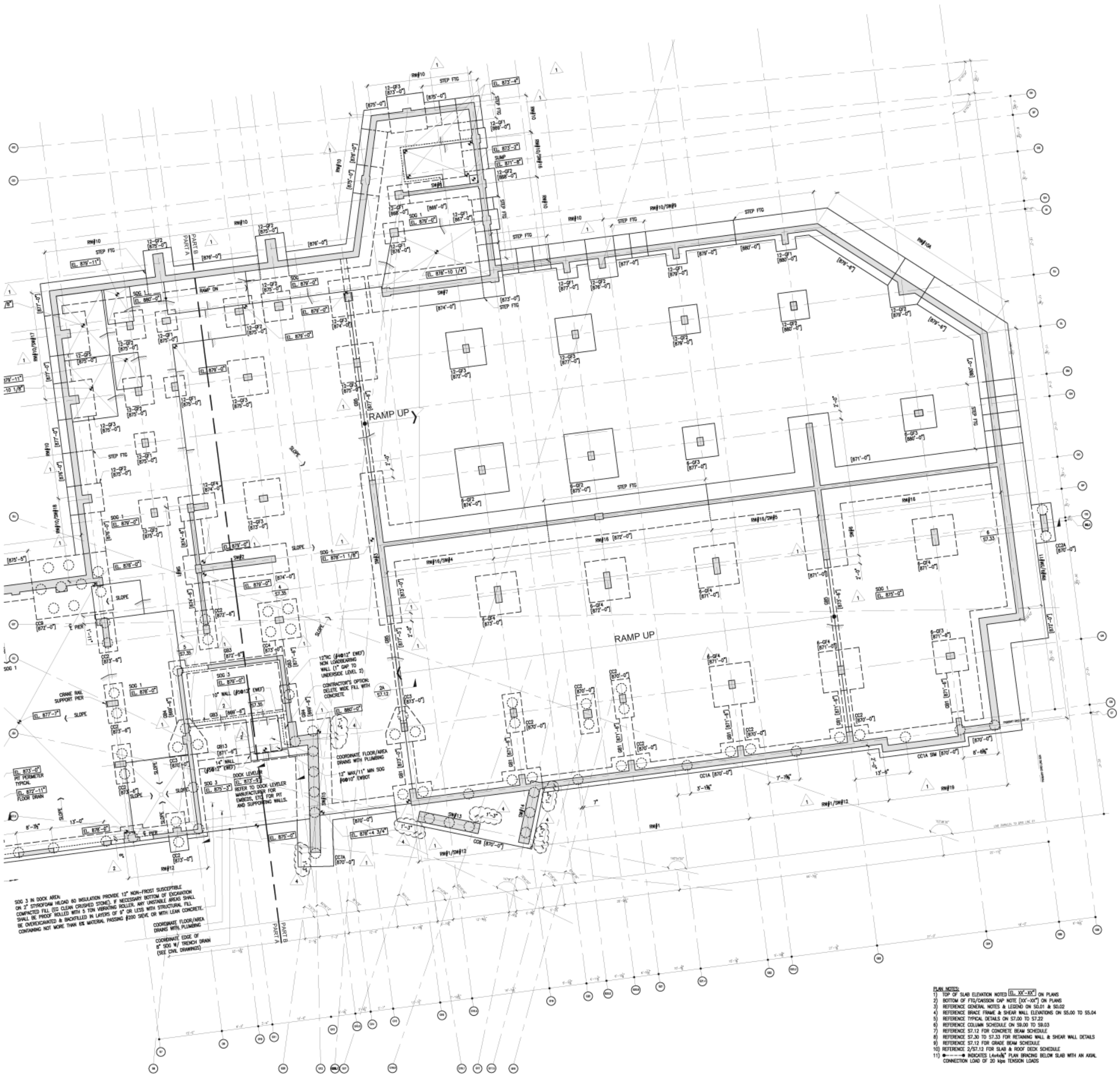
ALL OTHER DRIFT LOCATIONS ARE CALCULATED ON SPREADSHEET.

Appendix E: Typical Plans

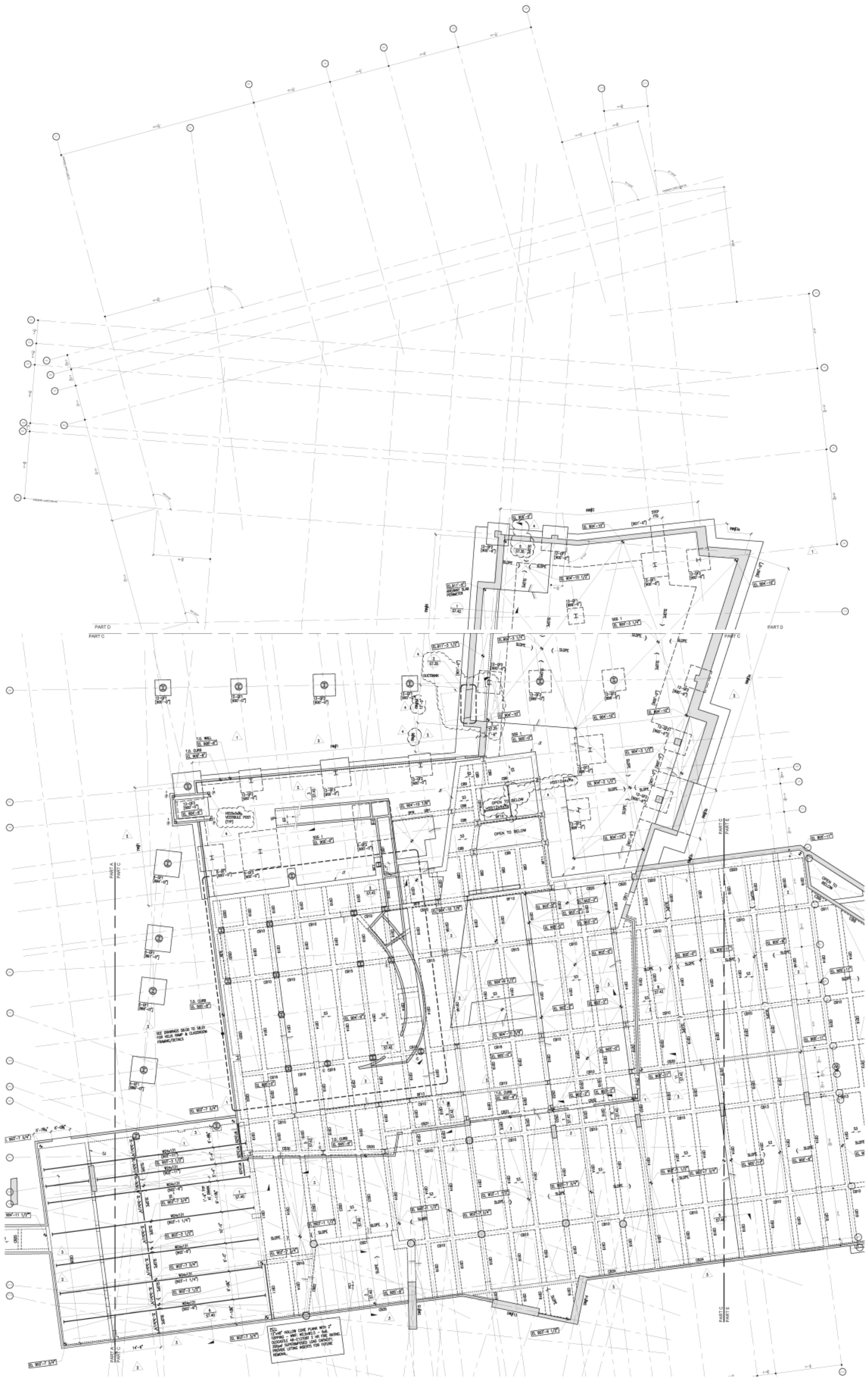
Foundations



Level 1 Foundation Plan



Level 3

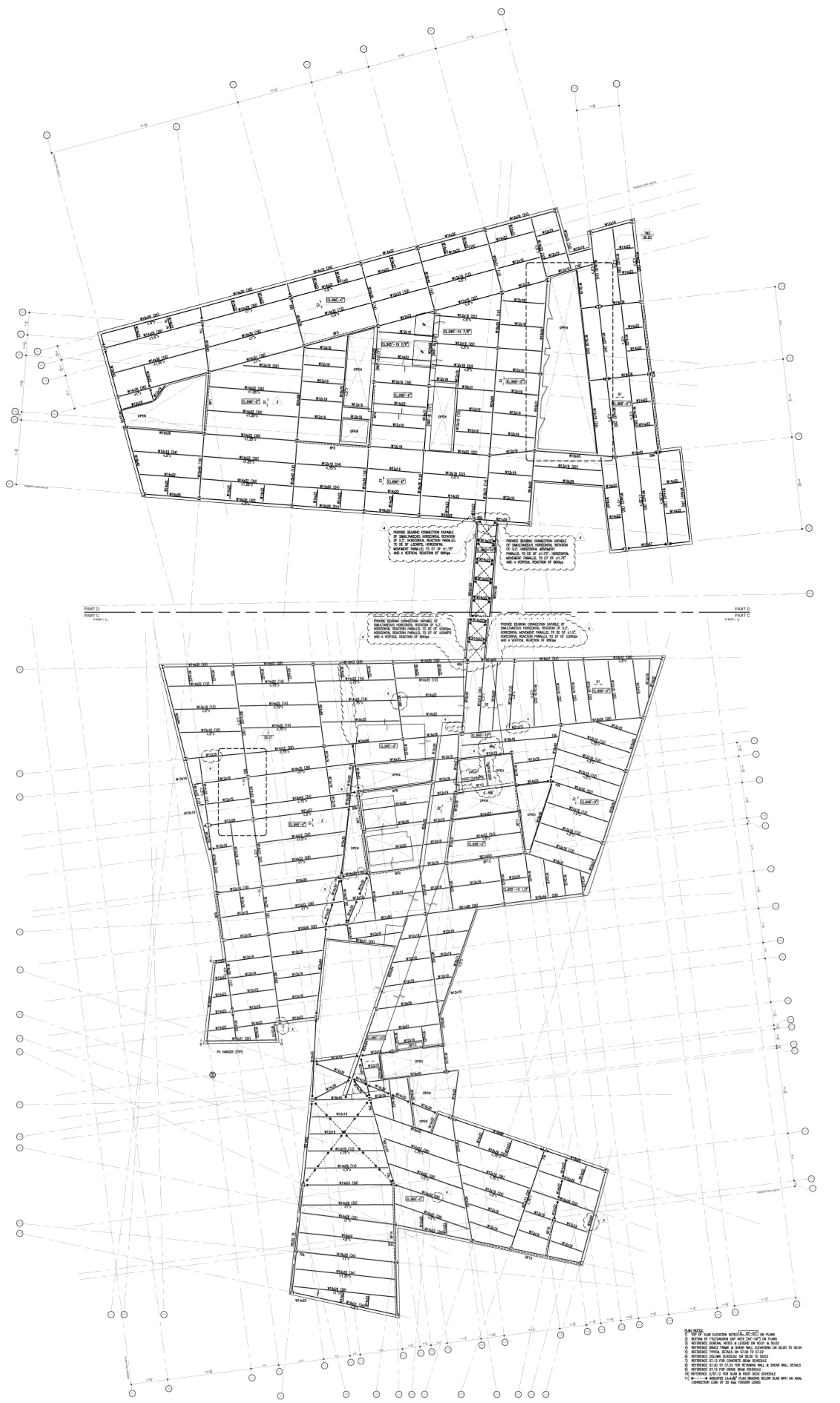


Level 4

Level 5

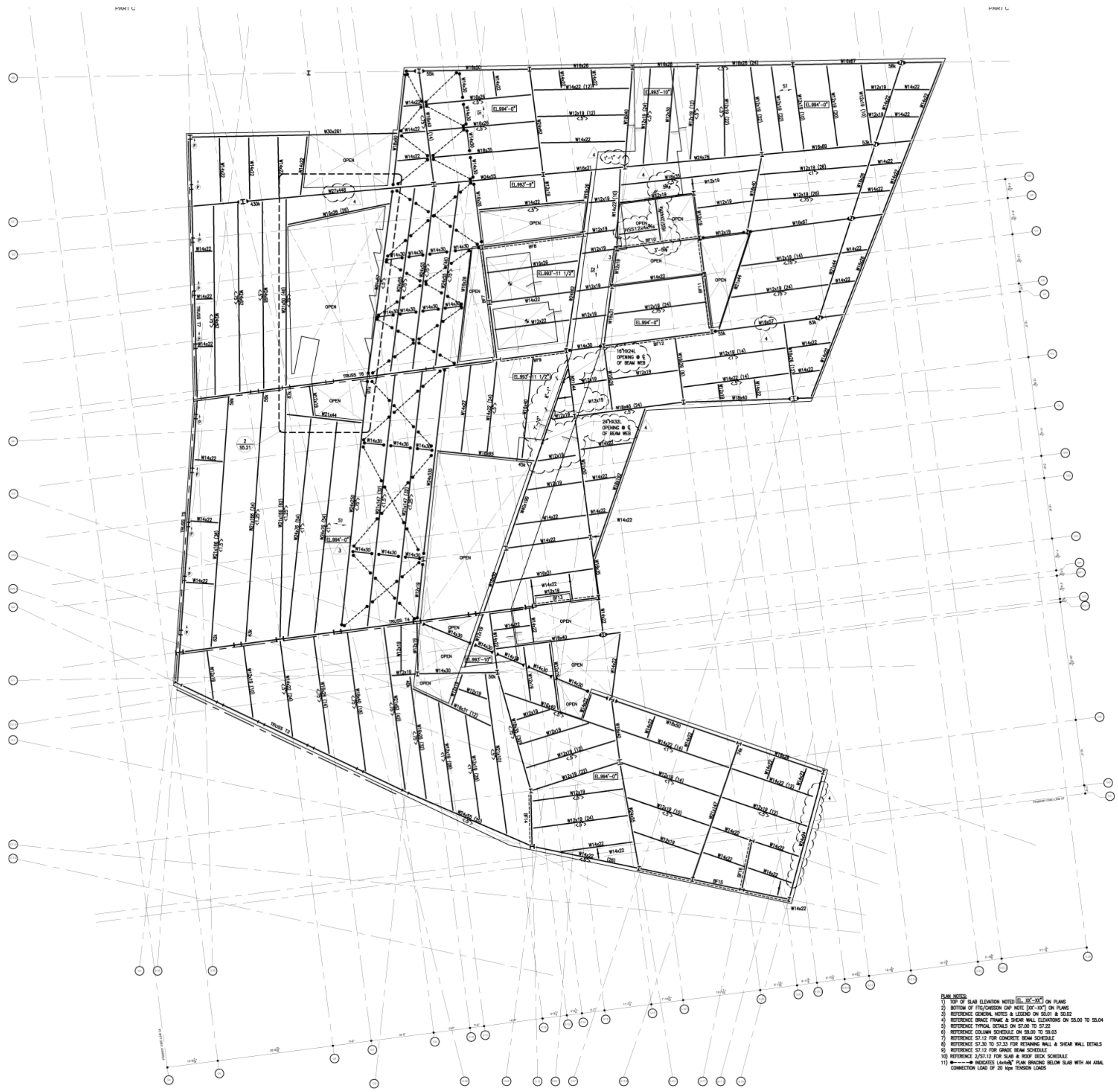
Level 6

Level 7



Level 8

Level 9


PLAN NOTES:
1) TOP OF SLAB ELEVATION NOTED EL. XX'-XX" ON PLANS
2) BOTTOM OF FTG/CAISSON CAP NOTE [XX'-XX"] ON PLANS
3) REFERENCE GENERAL NOTES & LEGEND ON S0.01 & S0.02
4) REFERENCE BRACE FRAME & SHEAR WALL ELEVATIONS ON S5.00 TO S5.04
5) REFERENCE TYPICAL DETAILS ON S7.00 TO S7.22
6) REFERENCE COLUMN SCHEDULE ON S9.00 TO S9.03
7) REFERENCE S7.12 FOR CONCRETE BEAM SCHEDULE
8) REFERENCE S7.30 TO S7.33 FOR RETAINING WALL & SHEAR WALL DETAILS
9) REFERENCE S7.12 FOR GRADE BEAM SCHEDULE
10) REFERENCE 2/S7.12 FOR SLAB & ROOF DECK SCHEDULE
11) INDICATES L4x4x3/8" PLAN BRACING BELOW SLAB WITH AN AXIAL CONNECTION LOAD OF 20 kips TENSION LOADS