Existing Conditions (Tech 1)

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Executive Summary

The structural concepts and existing conditions report explains the physical existing conditions and the relative design concepts of the structure of Global Village Building 400. Global Village is a European-inspired complex that provides commercial and residential space for the campus at the Rochester Institute of Technology in Rochester, NY. Each location has been designed to incorporate themes and materials that represent different regions from around the world, including marble from Italy and wood siding from Denmark. Global Village is a four-story building that also supports a fifth story dedicated to mechanical equipment; making it rise to an overall height of 62.5 feet. The building is constructed of steel with metal deck and lightweight concrete at the first, second, and third floors while the other floors have wood framing. The building's main lateral-resisting system consists of concentrically braced frames in both directions.

Through the use of ASCE 7-10, gravity loads were found and compared to loads used by the original design team. If loads could not be found, a value frequently represented in textbooks was used. These loads were then used to spot check gravity members throughout the building. The results of the floor system: slab, beam, and girder were found to be close to what the design team used on the structure; only differing by the number of studs used on the beams and girders. An exterior and interior column were also examined and calculated to provide a more concise evaluation of the design loads. The results here matched the exact wide-flanges used on the structure. Through these calculations, the design loadings were therefore considered to be valid.

Lateral loadings, calculated by wind and seismic analyses, were also performed in accordance to ASCE 7-10. From the wind analysis, the average wind pressures start at 18 psf at the first floor and rise to 25 psf at the top of the structure. When comparing the base shear and overturning of wind to that of seismic, it was found that seismic loads control over wind load by a factor of almost 1.5. This is most likely due to the heavy load from the mechanical weight at the top floor or penthouse. The calculated value for base shear due to seismic loads averaged 330 kips with an average over-turning moment of 14,000 kip-ft.

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Purpose

The purpose of Technical Report 1 is to analyze and provide an understanding of the structure of Global Village. This report will compare calculated gravity loads and structural components with the existing loads and elements used by the design team. This report will also perform a seismic and wind analysis to examine lateral loads on the structure.

Introduction




Global Village is a mixed-use building that provides commercial and residential space for the campus at RIT. Global Village has achieved LEED Gold certification and has been designed to be community friendly. In total, the Global Village project provides 414 beds for on campus living and 24,000 square feet of commercial and retail space.

The $57.5 million dollar project consists of three independent structures on the campus at RIT. The main four-story Global Village building (Building 400) is 122,000 square feet and the two additional three-story Global Way buildings (Buildings 403 and 404) are 32,000 square feet each. The main project team includes RIT as the owner, Architectural Resources Cambridge as the architect, and The Pike Company as the CM-at-Risk. Eleven other firms were also employed to handle MEP, lighting, acoustics, and so forth.

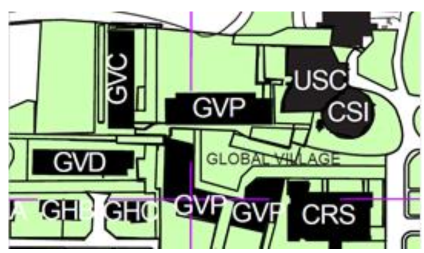


Figure 1: GVP is Building 400 (Global Village Building). GVC and GVD are Buildings 403 and 404 (Global Way Buildings). Courtesy of RIT.

Commercial space is located on the first and second floors, which consist of two dining facilities, a post office, salon, wellness center, sports outfitter, and a convenience store. Campus housing is located on the third and fourth floor which provides room for 210 beds. There is also a fifth floor; however, it is used primarily as a mechanical penthouse. Building 400's unique "U" shape creates a courtyard that features a removable stage, gas fireplace, and a glass fountain. See Figure 1 for a campus map of the Global Village complex. The area also includes outdoor seating with tables equipped with umbrellas.

The 28,000 square foot courtyard is also heated to extend its use during the winter and to minimize winter maintenance.

The façade of Building 400 is made up of a cement fiber board rain screen, brick masonry veneer, and flat seamed sheet metal with aluminum clad wood windows, and a coated extruded aluminum storefront.

Global Village Building 400 is a LEED Gold Certified Building. Green aspects include a green roof above the restaurant, daylight sensor lighting, and sensors to shut off mechanical equipment when windows are opened. Global Village is located on a sustainable site that is walk-able and transit oriented, encourages low-emitting vehicles, and reflects solar heat. The building reduces water consumption through water efficient landscaping and technologies such as high-efficiency toilets, faucets, and shower heads. Through the implementation of several energy efficient systems, the building is predicted to use 29.4% less energy. To encourage sustainable energy, seventy percent of the building's electricity consumption is provided from renewable sources (wind) through the engagement in a two-year renewable energy contract. Construction of Global Village included waste management recycling, air quality control, and low emitting materials. Along with regional materials, recycled content were also installed that constitute 20% of the total value of the materials in the project.

Global Village is a part of RIT's campus outreach program. The buildings not only provide student housing and retail space, but were also designed to be community friendly and to provide students with a global living experience. Global Village is LEED Gold certified and the courtyard created promotes outdoor activity.

Structural Overview

The structure of Global Village Building 400 consists of steel framing on a concrete foundation wall. The first, second, and third floor slabs use a lightweight concrete on metal decking system while the fourth floor, mechanical penthouse, and roof use wood framing. The lateral system consists of concentrically braced frames in both directions.

Foundation

In January 2009, Tierney Geotechnical Engineering, PC (TGE) provided a subsurface exploration and geotechnical investigation for Global Village. TGE performed 14 test borings and 2 test pits on the site of Building 400 and recommended foundation types and allowable bearing pressures along with seismic, floor slab, and lateral earth pressure design parameters.

In general, the borings and test pits encountered up to 8 inches of topsoil at the ground surface, or fill. The fill, generally consists of varying amounts of silt, sand, and gravel. At several locations, the fill also contained varying amounts of construction-type debris and deleterious material such as asphalt, topsoil, and wood. The fill was generally encountered to depths of approximately 4 to 8 feet. Below the fill, native soils with a very high compactness were encountered. Overall, most of the structure's foundation is on very compact glacial fill.

From these results, it was determined that the structure may then be supported on a foundation system consisting of isolated spread and continuous strip footings. TGE recommends an allowable bearing pressure of 7,500 psf to be used in the foundation design. It was also recommended by TGE that, due to lateral earth pressure, retaining walls are to be backfilled to a minimum distance of 2 feet behind the walls with an imported structural fill. To prevent storm run-off, permanent drains should also be installed behind all retaining walls.

Floor System

The first floor consists of a 6" concrete on grade slab. For the second and third floors, the floor system is comprised of 3¼" lightweight concrete slab on 3" composite metal (18-gage) decking. Individual steel deck panels are to be continuous over two or more spans except where limited by the structural steel layout. The rest of the floors are made up of wood framing with ¾" plywood sheathing. Shear stud connectors are welded to beams and girders where appropriate. See Figure 2 for details.

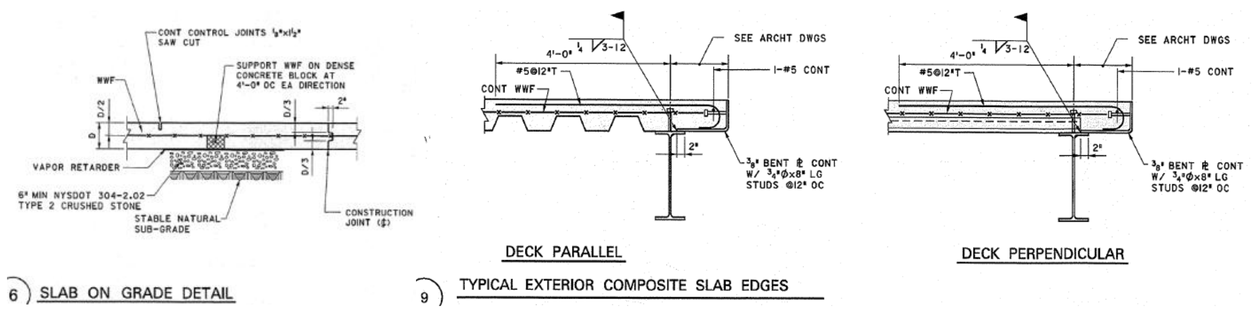


Figure 2: Typical composite slab details. Courtesy of RIT. Drawings not to scale.

Framing System

The framing grid that Global Village possesses is very unique and very complicated. The bay sizes on each floor vary dramatically and the beams don't line up on each side of the transfer girders. The framing is also not consistent between floors. There is no simple consistent grid except for a couple areas highlighted in Figure 3. In these highlighted areas, the beams vary from W18x35 to W16x31 while the transfer girders vary from W14x22 to W21x44. Column sizes also vary significantly throughout the structure where the majority is in between W10x54 to W12x106.

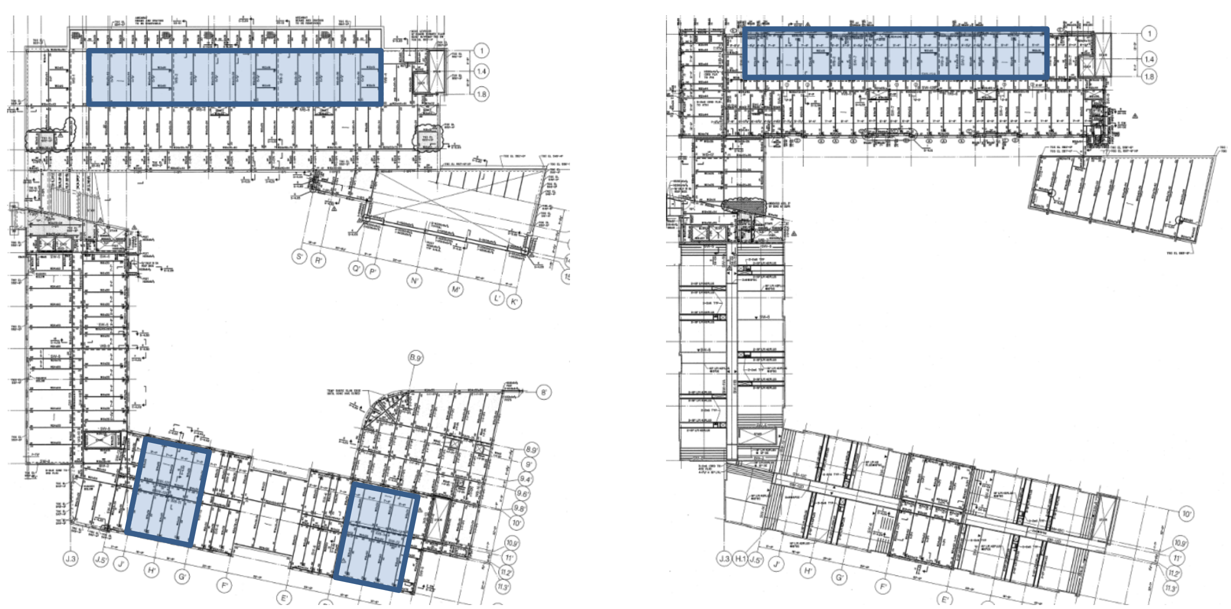
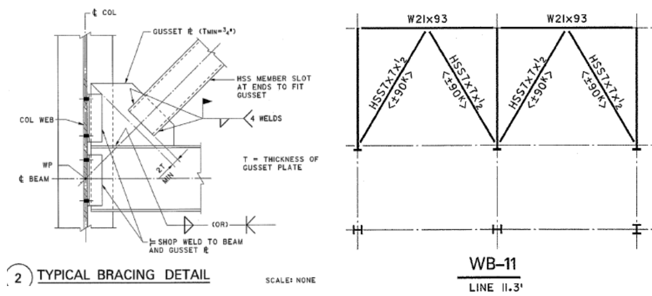


Figure 3: 2nd Floor (left) and 3rd Floor (right) framing plans. Typical bays on each level highlighted. Courtesy of RIT. Drawings not to scale.

Lateral System

The main lateral load resisting system consists of concentrically braced frames in both the N-S direction as well as the E-W direction. The lateral HSS bracing ranges in size where the majority is HSS7x7x½. See Figure 4 for details and placements.



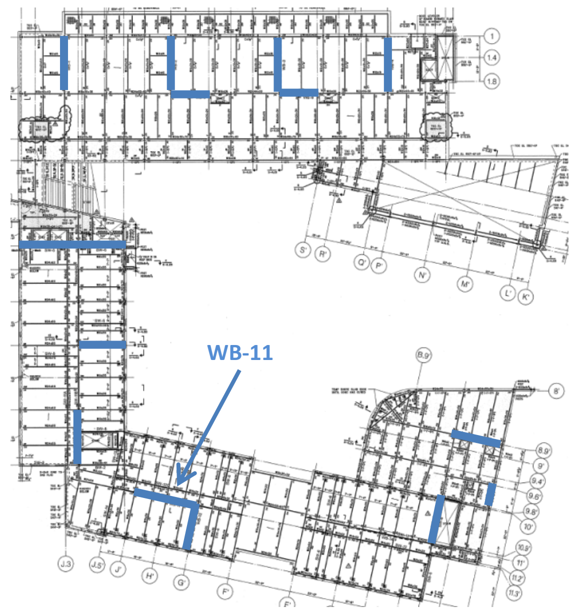


Figure 4: Typical bracing details and placement of bracing on 2nd Floor. Courtesy of RIT. Drawings not to scale.

Design Codes

Below is a list of codes and standards that the design team used on Global Village. As a comparison, codes and standards used for this report are given.

Design Codes

Design Codes:

- American Concrete Institute (ACI) 318-99, Building Code Requirements for Reinforced Concrete
- American Concrete Institute (ACI) 301-99, Specifications for Structural Concrete for Buildings
- ACI Detailing Manual-1994 (SP-66)
- CRSI Manual of Standard Practice (MSP 1-97)
- Structural Welding Code – Reinforced Steel (AWS DI.4-92)
- Code of Standard Practice for Steel Buildings & Bridges (AISC 1992)
- Part II published in the Timber Construction Manual (AITC 4th Edition)
- National Design Specification for Wood Construction (NF.PA, 1991 Edition)

Model Codes:

- 2007 Building Code of New York State / 2003 International Building Code
- 2007 Fire Code of New York State / 2003 International Fire Code
- Accessibility: BCNY Chapter 11, 2003 ICC/ANSI 117.1
- Electrical Code of New York, NFPA 70 2005
- 2007 Mechanical Code of New York State / 2003 International Mechanical Code
- 2007 Plumbing Code of New York State / 2003 International Plumbing Code

Standards:

- American Society of Civil Engineers (ASCE) 7-02, Minimum Design Loads for buildings and Other Structures

Thesis Codes

Design Codes:

- AISC Steel Construction Manual, 14th Edition

Standards:

- American Society of Civil Engineers (ASCE) 7-10, Minimum Design Loads for buildings and Other Structures

Material Properties

Listed below are materials and their strengths used in Global Village. These material strengths are followed best as possible in this report.

Steel

Unless Noted Otherwise	F_y = 50 ksi (A992 or A588 Grade 50)
Where Noted by (*) on Drawings	F_y = 36 ksi (A36)
Square and Rectangular HSS (Tubes)	F_y = 46 ksi (A500 Grade B)
Round HSS (Pipes)	F_y = 46 ksi (A500 Grade C)
Anchor Bolts (Unless Noted Otherwise)	F_y = 36 ksi (F1554)
High Strength Bolts (Unless Noted Otherwise)	F_u = 105 ksi (A325)
Metal Deck	F_y = 33 ksi (A653)
Weld Strength	F_y = 70 ksi (E70XX)

Concrete

Slabs-on-Grade	4000 psi (Normal Weight)
Walls, Piers	4000 psi (Normal Weight)
Concrete on Steel Deck	3000 psi (Light Weight)
Topping Slabs & Housekeeping Pads	3000 psi (Normal Weight)

Other

Bars, Ties, and Stirrups	60 ksi
Masonry	F'_m = 3000 psi
Wood	F_b = 1000 psi (Bending Stress)
	F_v = 70 psi (Shear Stress)

* Material strengths are based on American Society for Testing and Materials (ASTM) standard rating

* Other wood strengths are given in the structural drawings

Design Loads

Dead and Live Loads

Due to the fact that the structural drawings only gave a typical floor partition allowance of 20 psf as a dead load, other dead loads were found or assumed by using Vulcraft catalogs and textbooks on structural design. For a summary of assumed superimposed dead loads used, see Table 1.

Superimposed Dead Loads	
Description	Load (psf)
Framing	10
Superimposed DL	10
MEP Allowance	10
Partitions	20
Composite Decking	46
Roofing	60

Table 1: Summary of superimposed dead loads

Live loads, however, were provided in the structural drawings. These loads were compared to live loads found using Table 4-1 in ASCE 7-10 based on the usage of the spaces. The results are given in Table 2. Most live loads found match designer loads except for fan and mechanical equipment room loadings. Since these were not able to be found in ASCE 07-10, the loads were taken from the design team to be consistent.

Live Loads			
Space	Design Live Load (psf)	Live Load Used (psf)	Notes
Lobbies and Common Areas	100	100	ASCE 7-10: Residential
1st Floor Corridors	100	100	ASCE 7-10: Schools
Typical Floors	40	40	ASCE 7-10: Residential
Corridors above 1st Floor	80	80	ASCE 7-10: Schools
Stairways	100	100	ASCE 7-10: Stairways
Fan Room	80	80	Assumed
Mechanical Equipment Rooms	150	150	Assumed

Table 2: Comparison of design live loads and live loads used

Wind Loads

Winds loads were calculated using the Main Wind-Force Resisting System (Directional Procedure) outlined in Chapter 26 and 27 of ASCE 7-10. Before using this procedure, some simplifications were made by splitting the structure up into three separate rectangular buildings, see Figure 5. This was done due to the differing heights of the structure and some sections of the structure could be considered to be neglected (passageways). These separate buildings were then assumed to have constant heights and to contain no component and cladding effects.

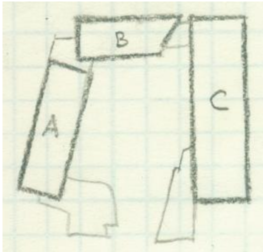


Figure 5: Simplifying building structure

Global Village was found to be categorized as a Type III Occupancy and Exposure Category C. General building dimensions, constants used, and calculation of gust factors for the direction normal to the long dimension (length) are given in Table 3. General building dimensions, constants used, and calculation of gust factors for the direction normal to the short dimension (width) are given in Table 5.

Calculations were done on Microsoft Excel to reduce calculation errors and save time. The wind pressure calculations in the long dimension are given in Table 4. The results can be found in Figure 6. The wind pressure calculations in the short dimension are given in Table 6. The results can be found in Figure 7. As a note, internal pressure was not included in the calculations because internal pressure can be considered self-cancelling unless there are large openings in the structure.

The structural sheets provide values to which the designer used but no overall base shear or wind pressures. The calculated values are similar to the values used in design except the designer's Basic Wind Speed is 90 mph where the value that was calculated was 120 mph. This is due to the different versions of ASCE 07. The designers used ASCE 7-02 where the values calculated for this report were from ASCE 7-10.

Normal to Long Dimension (Length)

Building Dimensions				Gust Factor Calculations				
Building	Length (ft)	Width (ft)	Height (ft)	z_{bar}	I_{zbar}	L_{zbar}	Q	G
A	165.500	52.800	51.830	31.098	0.202	494.099	0.853	0.852
B	136.330	52.800	62.500	37.500	0.196	512.948	0.862	0.857
C	223.000	52.800	62.500	37.500	0.196	512.948	0.835	0.844

Constants					
V (mph) =	120.000	$C_{p,windward}$ =	0.800	$C_{p,roof:<h/2}$ =	-1.300
k_d =	0.850	$C_{p,leeward}$ =	-0.500	$C_{p,roof:>h/2}$ =	-0.700
k_{zt} =	1.000	$C_{p,sides}$ =	-0.700		

Table 3: Building dimensions, gust factors, and constants

Building A								
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	18.145	-14.636	-20.490		
3rd	26.660	0.953	29.862	20.344	-14.636	-20.490		
Penthouse	37.330	1.024	32.086	21.859	-14.636	-20.490		
Roof	51.830	1.097	34.374	23.418	-14.636	-20.490	-38.054	-20.490

Building B								
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	18.262	-15.308	-21.431		
3rd	26.660	0.953	29.862	20.475	-15.308	-21.431		
4th	37.330	1.024	32.086	22.001	-15.308	-21.431		
Penthouse	48.000	1.080	33.841	23.204	-15.308	-21.431		
Roof	62.500	1.140	35.721	24.493	-15.308	-21.431	-39.801	-21.431

Building C								
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	17.979	-15.071	-21.099		
3rd	26.660	0.953	29.862	20.158	-15.071	-21.099		
4th	37.330	1.024	32.086	21.659	-15.071	-21.099		
Penthouse	48.000	1.080	33.841	22.844	-15.071	-21.099		
Roof	62.500	1.140	35.721	24.113	-15.071	-21.099	-39.184	-21.099

Table 4: Wind pressure loads normal to long dimension

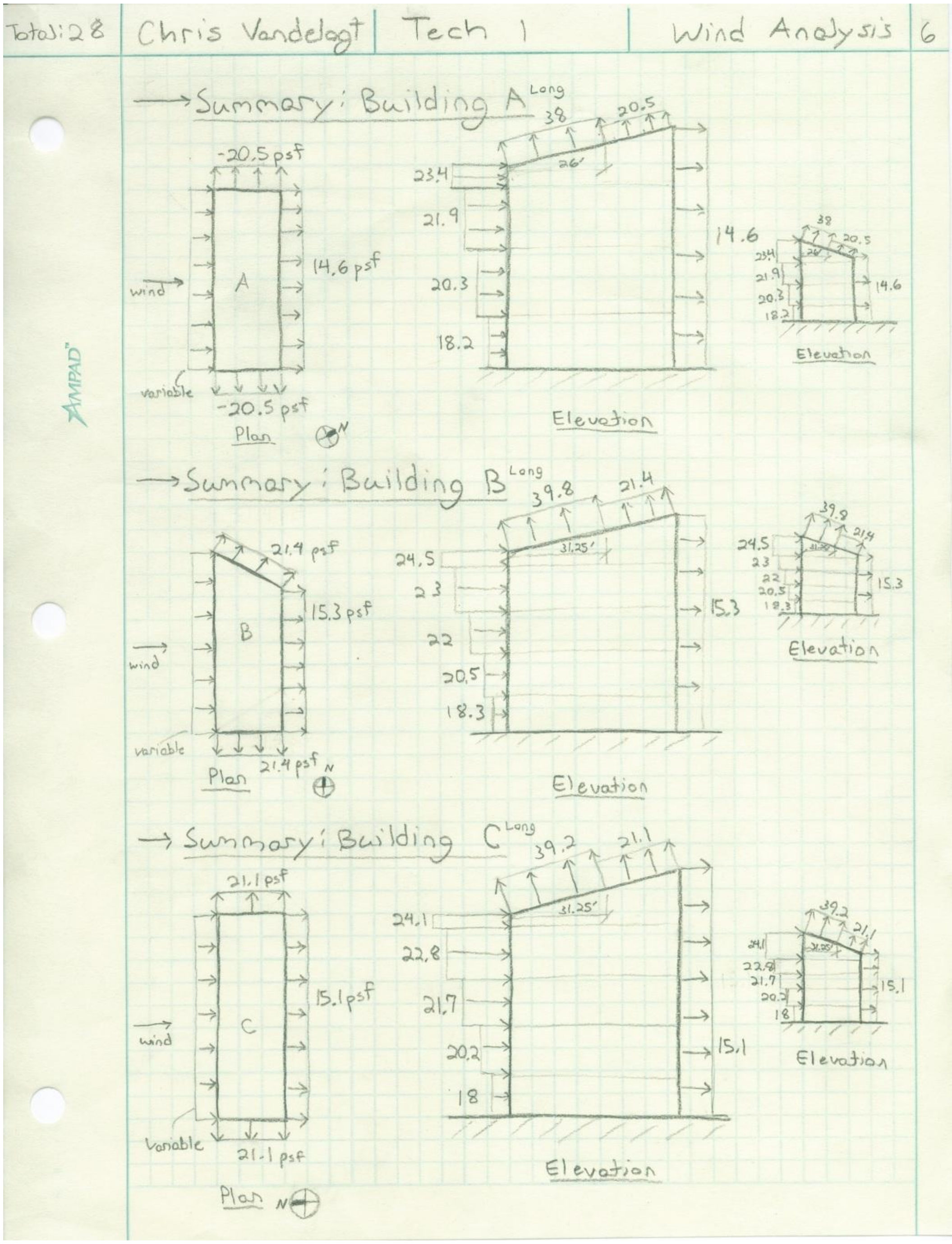


Figure 6: Summary of wind pressures normal to long dimension

Normal to Short Dimension (Width)

Building Dimensions				Gust Factor Calculations				
Building	Width (ft)	Length (ft)	Height (ft)	z_{bar}	I_{zbar}	L_{zbar}	Q	G
A	52.800	165.500	51.830	31.098	0.202	494.099	0.899	0.875
B	52.800	136.330	62.500	37.500	0.196	512.948	0.896	0.874
C	52.800	223.000	62.500	37.500	0.196	512.948	0.896	0.874

Constants					
V (mph) =	120.000	$C_{p,windward}$ =	0.800	$C_{p,roof:<h/2}$ =	-1.300
k_d =	0.850	$C_{p,leeward}$ =	-0.500	$C_{p,roof:>h/2}$ =	-0.700
k_{zt} =	1.000	$C_{p,sides}$ =	-0.700		

Table 5: Building dimensions, gust factors, and constants

Building A						
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.639	-15.034	-21.048
3rd	26.660	0.953	29.862	20.897	-15.034	-21.048
Penthouse	37.330	1.024	32.086	22.454	-15.034	-21.048
Roof	51.830	1.097	34.374	24.055	-15.034	-21.048

Building B						
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721	24.972	-15.608	-21.851

Building C						
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721	24.972	-15.608	-21.851

Table 6: Wind pressure loads normal to short dimension

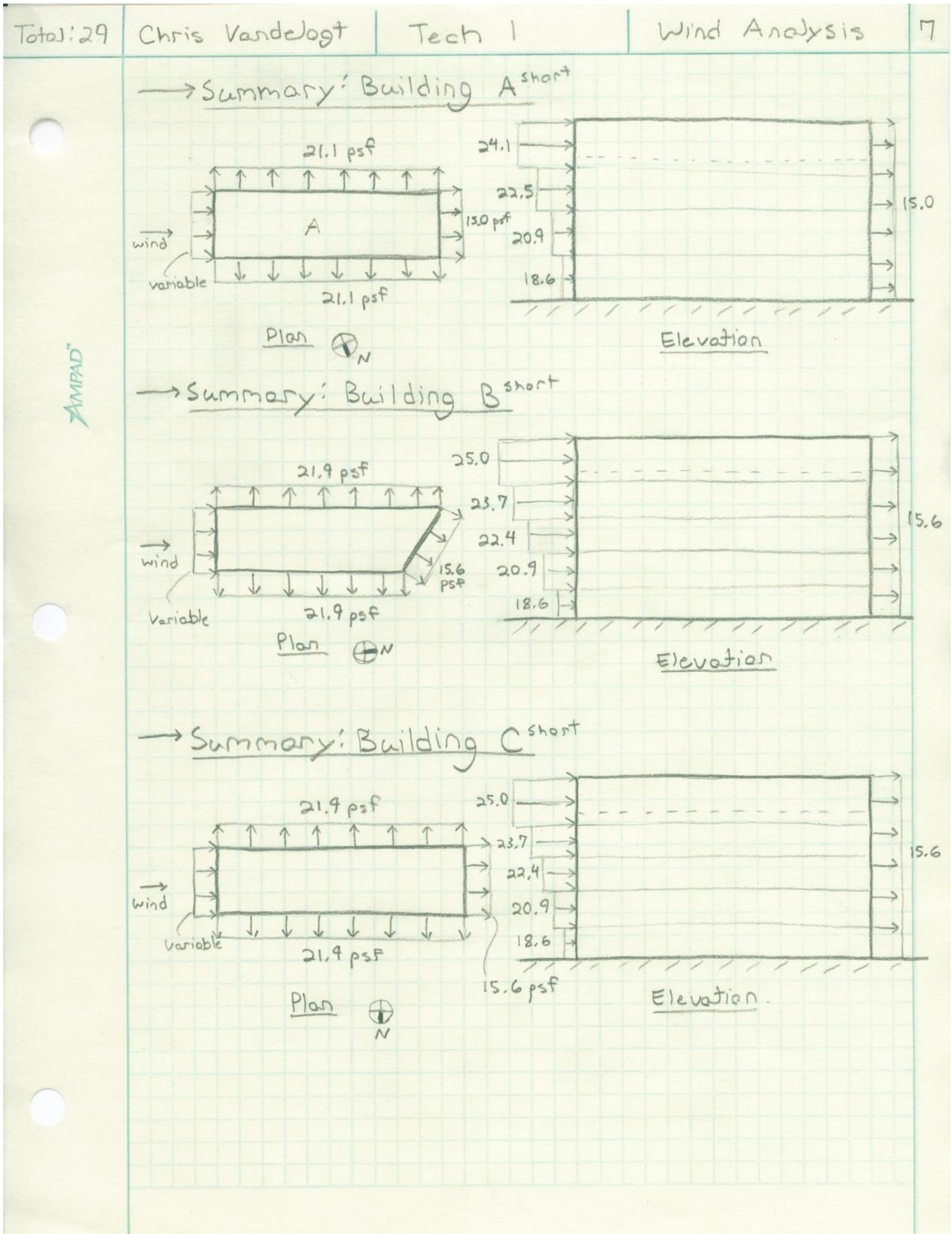


Figure 7: Summary of wind pressures normal to short dimension

Seismic Loads

Seismic Loads were calculated using the Equivalent Lateral Force Procedure outlined in Chapters 11 and 12 of ASCE 7-10. While performing the procedure, many seismic values were found which are noted in Table 7. As defined by the structural drawings, the building's lateral system is classified as a steel concentrically braced frame in both directions. This was used when finding the Response Modification Coefficient. Spectral Response Acceleration values were taken directly from the USGS website instead of using the ASCE maps to provide a more accurate result.

The structural drawings give a list of values that the design team used. Comparing these with the values calculated; it was found that all values were exact except for the Response Modification Coefficient. This difference could be from using different codes and standards. The calculated values are from ASCE 7-10 whereas the designer's values are from the 2007 Building Code of New York State.

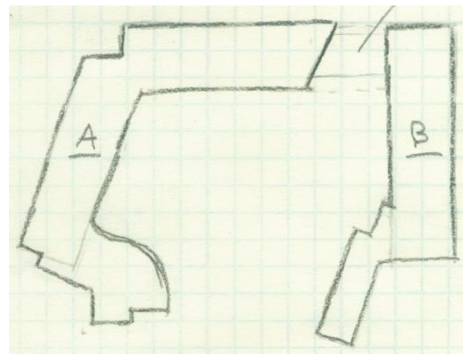


Figure 8: Simplifying building structure

Like in the wind analysis, the structure was split up and acted as different buildings. For the seismic analysis, the structure was considered to be two buildings since it was assumed that a passageway between the two sections would provide no effect on the structure in seismic, see Figure 8. The weight of each floor of each building was then computed using the dead loads listed in the gravity loads section of this report. See Table 8 for calculations and Figure 9 for a summary of forces on each building.

Seismic Variable	Value	Reference (ASCE 7-10)
I_e	1.25	Table 1.5-2
S_S	.21	USGS Website
S_1	.06	USGS Website
Site Class	C	Geotechnical Report
Occupancy Category	III	Table 1.5-1
S_{DS}	.168	Table 11.6-1
S_{D1}	.068	Table 11.6-2
Seismic Category	B	Table 11.6-1
R	3.25	Table 12.2-1
T_L	6 sec	Figure 22-12
C_t	.02	Table 12.8-2
x	.75	Table 12.8-2
T_a	.445 sec	
T	.7565 sec	
C_s	.035	Equation 12.8-2

Table 7: Seismic values

Building A							
Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	1833	0	0.00	0.00	0.00	341.85	0.00
2nd	1675	14	85277.05	0.08	26.18	341.85	366.45
3rd	1837	26.66	195745.76	0.18	60.08	315.67	1601.80
4th	1975	37.33	310557.05	0.28	95.32	255.59	3558.40
Penthouse	2003	48	419016.48	0.38	128.61	160.26	6173.45
Roof	444	62.5	103117.80	0.09	31.65	31.65	1978.20
Sum:	9767		1113714.13	1.00	341.85		
				√ ok	√ ok		
	Base Shear ($V=C_sW$) =		341.85		Total Overturning Moment=		13678.29

Building B							
Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	2641	0	0.00	0.00	0.00	327.04	0.00
2nd	1196	14	58315.01	0.06	18.04	327.04	252.58
3rd	1195	26.66	120501.43	0.11	37.28	309.00	993.91
4th	1071	37.33	155691.35	0.15	48.17	271.72	1798.11
Penthouse	2481	48	533460.01	0.50	165.04	223.55	7922.04
Roof	760	62.5	189109.52	0.18	58.51	58.51	3656.68
Sum:	9344		1057077.32	1.00	327.04		
				√ ok	√ ok		
	Base Shear ($V=C_sW$) =		327.04		Total Overturning Moment=		14623.33

Table 8: Seismic calculations

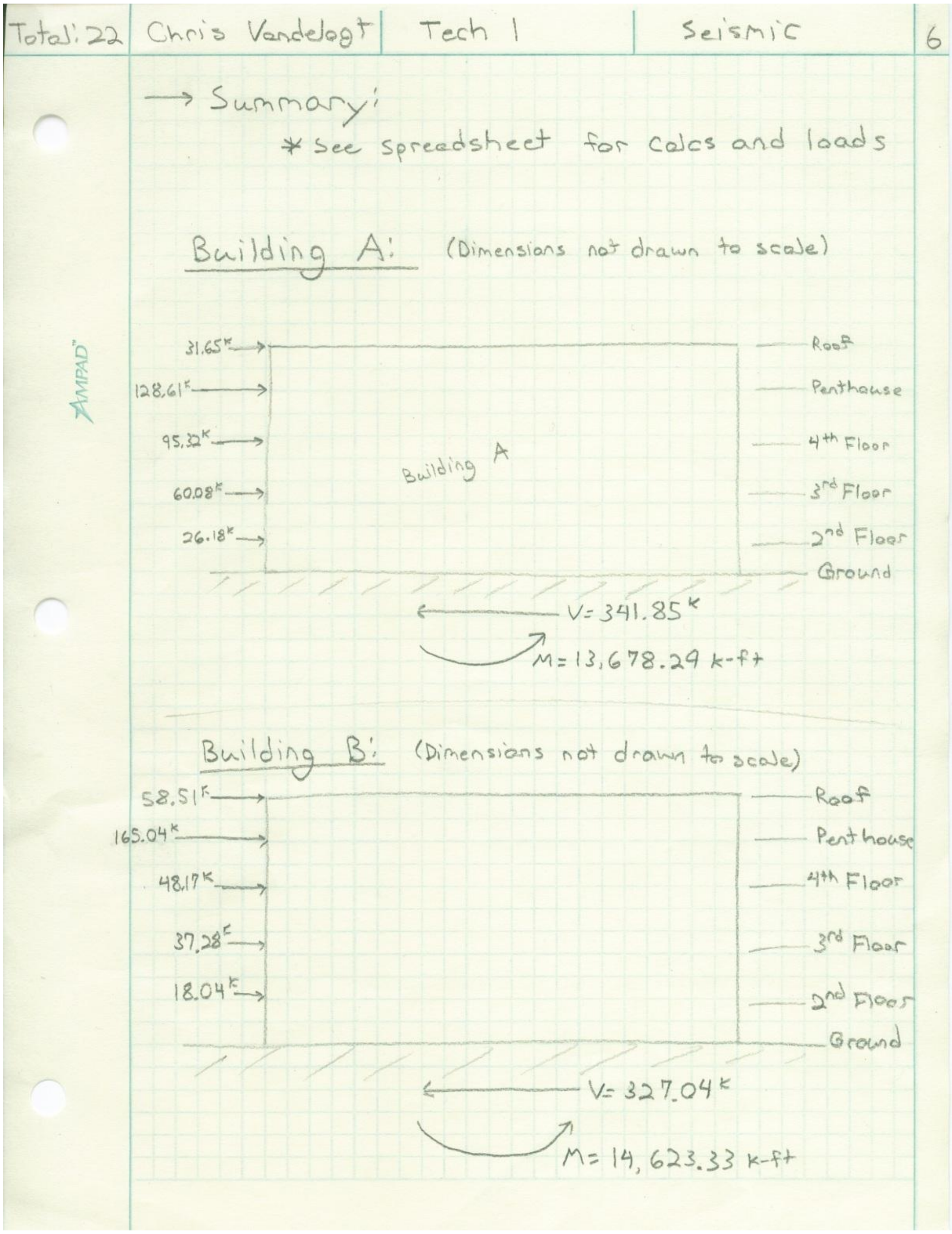


Figure 9: Summary of seismic loading

Snow Loads

The roof snow load was calculated in accordance to Chapter 7 of ASCE 7-10. The factors used to find the roof snow load can be found in Table 9. Using the flat roof procedure, the roof snow load was determined to be 30.8 psf where the snow load used by the design team was 39 psf. Since the factors used here match the factors listed on the structural drawings, the difference must be the equation used to calculate the flat roof snow load. On the structural sheet, the flat roof snow load procedure was used but in accordance with the "2007 Building Code of New York State." Therefore, it may be valid that the equations used to calculate roof snow load differ between ASCE 7-10 and the 2007 Building Code of New York State.

Flat Roof Snow Calculations	
Variable	Value
Ground Snow Load, p_g (psf)	40
Exposure Factor, C_e	1.0
Thermal Factor, C_t	1.0
Importance Factor, I_s	1.1
Flat Roof Snow Load, p_f (psf)	30.8

Table 9: Snow load factors

Due to different roof elevations, five locations needed to consider drift. See Figure 10 for snow drift locations and roof heights. Through calculating h_c/h_b for each location, it was determined all locations needed to consider drift. For calculations and results, see Table 10.

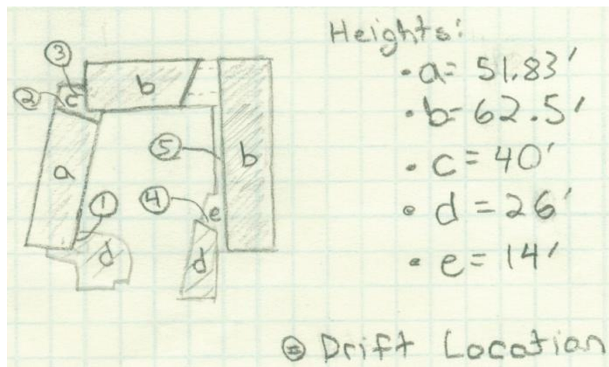


Figure 10: Snow drift locations

Pos.	Windward				Leeward				$p_{d,max}$ (psf)	Total Snow Load (psf)
	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	59.75	2.23	8.91	42.77	52.83	2.79	11.16	53.58	53.58	84.38
2	34.50	1.67	6.67	32.00	165.50	4.78	19.11	91.73	91.73	122.53
3	36.88	1.73	6.92	33.21	136.33	4.38	17.54	84.19	84.19	114.99
4	31.04	1.57	6.28	30.15	86.67	3.56	14.24	68.36	68.36	99.16
5	11.00	0.78	3.13	15.02	52.83	2.79	11.16	53.58	53.58	84.38

Table 10: Snow drift load calculations

Spot Checks

Composite Slab

The second and third floors of Global Village use a 3½" lightweight concrete slab on a 3" metal (18-gage) decking. The dead and live loads found in the gravity loads section of the report were used to test a typical bay on the 2nd floor, see Figure 11. The dead loads consisted of framing, superimposed, MEP, and a partition allowance while the live load in this region was considered to be a lobby. It was determined that a Vulcraft 3VLI18, with a capacity of 191 psf, would be sufficient in carrying the 110 psf loading. An unshored span check was also performed and proved to be adequate. From these results, the composite slab matches the designed slab's dimensions and has an overall weight of 46 psf.

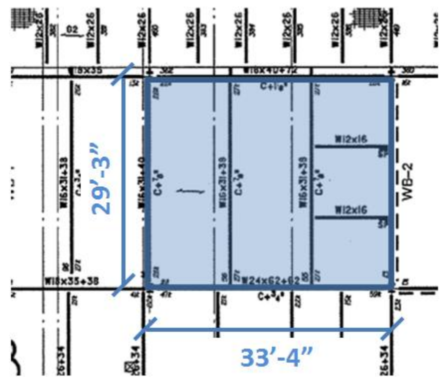


Figure 11: Typical bay examined. Courtesy of RIT.

Beam and Girder

Based on the spot check calculations of the beam, circled red in Figure 12, it was determined that the designer may have used different loads. The same W16x31 flange was found; however, the number of studs calculated (+24) is 14 studs lower than the designed number of studs (+38). The girder, circled in green, also has the correct W24x62 flange as used in the design but also is 12 studs lower (+50) than the design stud number (+62). A possible reason for this is that the space being designed is labeled as Academic Fit-out in the architectural drawings. Since the design team didn't know exactly what would be placed, they must have gone with a conservative live load. The loads used to spot check this beam consisted of framing, superimposed, MEP, composite decking, and a partition allowance with a live load for lobbies. These values, given in the gravity loads section, give a total dead load of 96 psf and a live load of 100 psf. Another reason for the difference could simply be that other dead loads were used.

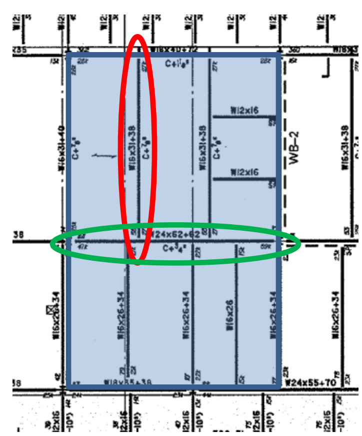


Figure 12: Beam analyzed in red. Girder analyzed in green. Courtesy of RIT.

Columns

The columns analyzed, circled in red in Figure 13, extend from the ground floor to the third floor. The rest of the floors are supported by load bearing walls. The procedure for estimating a column can still be used since the load above will eventually transfer from the load bearing walls to the columns below. Two columns; one interior and one exterior, were analyzed at the ground floor to get the maximum load on each column. After using the $P_{eq} = P_u + 24M_u/d$ equation, Table 4-1 in the AISC manual was used to find a column with the adequate capacity. From this analysis, both columns were calculated exactly as designed: W12x120 for the interior column and W10x54 for the exterior column.

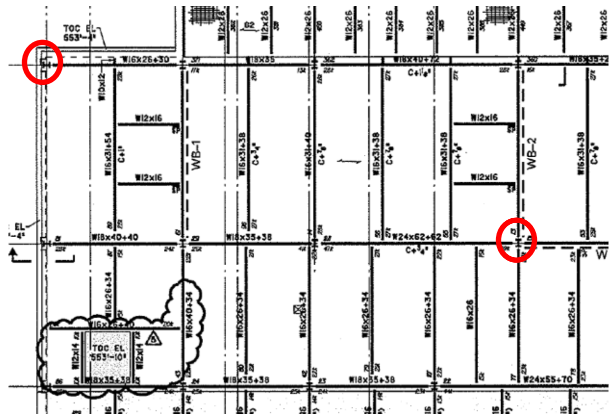


Figure 13: Columns analyzed. Courtesy of RIT.

Conclusion

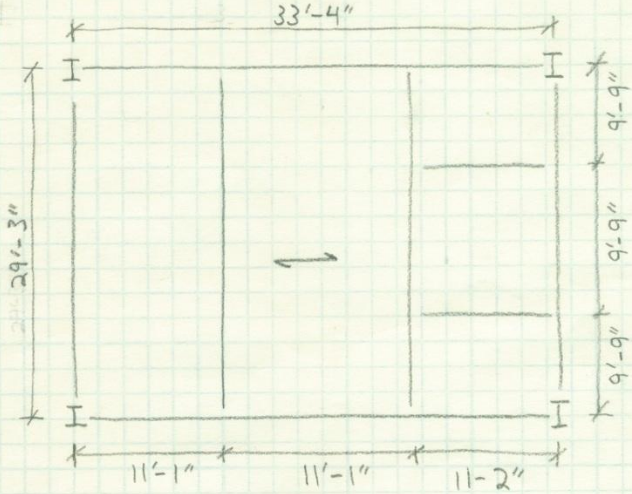
Technical Report 1 analyzed the existing structural conditions of Global Village Building 400 at RIT. An overview of the structure was examined, spot checks of gravity members were considered to be valid, and wind and seismic analyses were performed. Although different methods and standards were implemented by the designer and this report, the majority of the loads and the structural elements calculated were very similar.

The determination of dead and live loads relied on information provided by ASCE 7-10 as well as structural textbooks and class notes. These loads were then compared to values used by the design team. Overall, the loads found or estimated were very similar to the loads used on the structure.

These loads were used to check selected gravity members along with calculating the total building weight for a seismic analysis. The spot check of the floor system was very close to what was used. The correct wide flanges were found but the number of studs varied. However, the spot checks of an exterior and interior column were calculated exactly as the columns designed on Global Village. Although the structural drawings did not give a value for seismic, the calculated value for base shear averaged 330 kips with an average over-turning moment of 14,000 kip-ft. When compared to wind values, seismic loads govern by a factor of almost 1.5. Therefore, the seismic loads will control the lateral design of the building.

Appendix A: Gravity Load Calculations

Total: 1 | Chris Vandelogt | Tech 1 | Floor Sys: Steel Deck | 1



Location:
- Northwest End
- 2nd Floor

Loads:

→ Dead:
- Framing: 10 psf
- Super-imposed DL: 10 psf
- MEP: 10 psf
- Partition: 20 psf

→ Live: According to ASCE 7-10
- Office Buildings-Lobbies: 100 psf

since area is considered to be a lobby in the code occupancy types in arch drawings

For Floor system: Slab construction shall be 3¼" lightweight conc w/ WWF - W2.9xW2.9 on 3", 18 GA min. Total Thickness = 6¼". Composite Stl Deck.
- Span: 11'-1" ≈ 11'-6" (cons)
- Loading: 100 psf (LL) + 10 psf (super DL) = 110 psf
- Since 3" deep deck is needed, use Vulcraft 3VLI
- Use slab depth of 6.25" (t=3.25") 46 psf
- Per 18 GA min eq, use 3VLI18

Loading check at 11'-6": 191 psf > 110 psf ✓

Unshored Span check: 15' > 11'-1" ✓ (3-span)

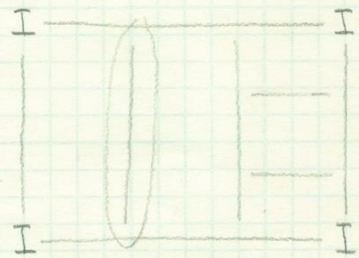
$\boxed{\text{Use Vulcraft 3VLI 18.}}$

Total: 2 | Chris Vandelogt | Tech 1 | Floor Sys: Beam | 1

→ Same Bay as used in steel deck analysis

→ Loads:

- Framing: 10 psf
- Super-imposed DL: 10 psf
- MEP: 10 psf
- Partitions: 20 psf
- LL: 100 psf
- Slab: 46 psf



$A_t = 11.083(29.25)$
$= 324.18\ ft^2$

LL Reduction

Live load is not reducible

$w_u = 1.2(96) + 1.6(100) = 275.2$

$w_u = 275.2(11.083) = 3.05\ k/ft$, $M_u = \frac{wl^2}{8} = \frac{3.05(29.25^2)}{8}$
$= 326.18$

From Table 3-19

assume $a \approx 1.5$ $\therefore Y_2 = 6.25 - \frac{1.5}{2} = 5.5$

Rough Economy

* 40(10) + 29.25(26) = 1160.5 — • W16×26: $\Sigma Q_n = 289$
$\phi M_n = 345$ ↳ $\frac{289}{14.6} = 19.8$ so 40 studs
(Table 3-21, 2 per rib, lightweight conc, deck perp, 3/4" studs)

→ * 1146.8 most economic — • W16×31: $\Sigma Q_n = 164$
$\phi M_n = 332$ ↳ $\frac{164}{14.6} = 11.2$ so 24 studs

* 1253 — • W16×36: $\Sigma Q_n = 133$
$\phi M_n = 353$ ↳ $\frac{133}{14.6} = 9.1$ so 20 studs

Check $a = \frac{164}{.85(3)(87.75)} = .73$ $\therefore Y_2 = 5.5''$ ok

$b_{eff} = \min \begin{cases} 2(29.25(12)/8) = 87.75 \leftarrow \text{controls} \\ 11.083(12) = 132.9 \end{cases}$

Total: 3	Chris Vandelogt	Tech 1	Floor Sys: Beam	2

→ Check unshored strength:

W16x31 $\phi_p M_p = 203'^k$

$w_u = 1.2(46)(11.083) + 1.2(31) + 1.6(20)(11.083) = 1.003$ klf

$w_u = 1.4(46)(11.083) + 1.4(31) = .757$ klf

$M_u = \dfrac{1.003(29.25)^2}{8} = 107.3'^k < 203'^k \therefore$ ok for shoring

→ Check wet conc deflection:

$w_{wc} = 46(11.083) + 31 = .541$ klf

$\Delta_{wc} = \dfrac{5(.541)(29.25^4)(1728)}{384(29000)(375)} = .778$ in

camber

$.8 \times .778 = .62 \approx .75''$

$\Delta_{wc\,max} = \dfrac{29.25(12)}{240} = 1.46$ in   since $.778 < 1.46$, ok

→ Check LL deflection:

$w_{LL} = 100(11.083) = 1.11$ From Table 3-20

$I_{LB} = 812$

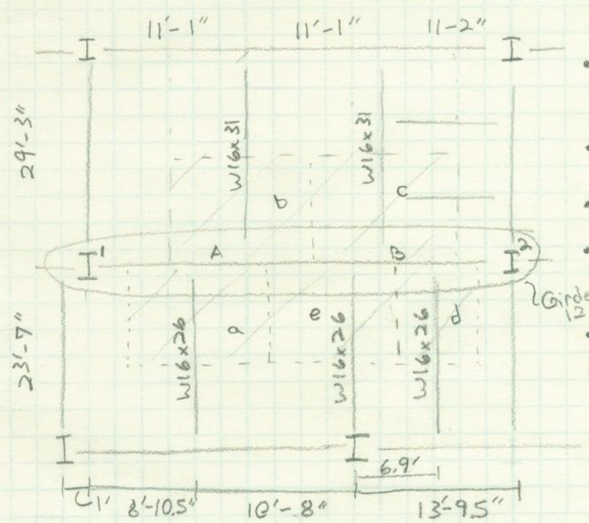
$\Delta_{LL} = \dfrac{5(1.11)(29.25^4)(1728)}{384(29000)(812)} = .776$

$\Delta_{LL\,max} = \dfrac{29.25(12)}{360} = .975$ since $.776 < .975$ ok

→ Summary:

$\boxed{\text{W16x31 w/ 24 studs \& 3/4'' Camber}}$

Total: 4 | Chris Vandelogt | Tech 1 | Floor Sys: Girder | 1



Areas:

- $a = \frac{9.875+10.66}{2}\left(\frac{23.583}{2}\right)$
  $= 121.1 \text{ ft}^2$
- $b = 11.083\left(\frac{29.25}{2}\right) = 162.1 \text{ ft}^2$
- $c = 11.125\left(\frac{29.25}{2}\right) = 162.7 \text{ ft}^2$
- $d = \left(\frac{13.79}{2}\right)\left(\frac{23.583}{2}\right)$
  $= 81.3 \text{ ft}^2$
- $e = \left(\frac{10.66+6.9}{2}\right)\left(\frac{23.583}{2}\right)$
  $= 103.5 \text{ ft}^2$

Since beams contact girder in 5 different locations, assume to take it as two unequal concentrated Loads unsymmetrically placed

→ Total Load on girder:

- Loads:
  - Super-imposed DL: 10 psf
  - MEP: 10 psf
  - Partitions: 20 psf
  - LL: 100 psf
  - Slab: 46 psf
  - Beam wt: Depends on beams
  - Girder wt Allowance: 50 lb/ft

Total: 5 | Chris Vandelogt | Tech 1 | Floor Sys: Girder | 2

→ Location A:

• From a: $P = 1.2(26(11.79) + (10+10+20+46)(121.1)) + 1.6(100(121.1))$
$= 32.24^{K}$

• From b: $P = 1.2(31(14.625) + (10+10+20+46)(162.1)) + 1.6(100(162.1))$
$= 43.21^{K}$

• Location: $\dfrac{8.875(32.24) + 11.083(43.21)}{(32.24 + 43.21)} = 10.14' \rightarrow$ in from col 1

→ Location B:

• From c: $P = 1.2(31(14.625) + (10+10+20+46)(162.7)) + 1.6(100(162.7))$
$= 43.37^{K}$

• From d: $P = 1.2(26(11.79) + (10+10+20+46)(81.3)) + 1.6(100(81.3))$
$= 21.77^{K}$

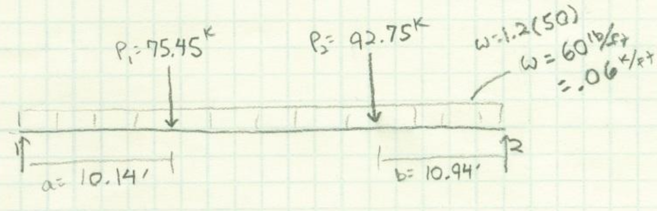
• From e: $P = 1.2(26(11.79) + (10+10+20+46)(103.5)) + 1.6(100(103.5))$
$= 27.61^{K}$

• Location: $\dfrac{11.16(43.37) + 6.89(21.77) + 13.79(27.61)}{(43.37 + 21.77 + 27.61)}$

$= 10.94' \leftarrow$ in from col 2

Result:

Girder 12:



Total: 6 | Chris Vandelogt | Tech 1 | Floor Sys: Girder | 3

→ Find Moment (From Table 3-23 in Manual)

$$R_2 = V_2 = \frac{P_1(l-a)+P_2 b}{l} = \frac{75.45(33.33-10.14)+92.75(10.94)}{33.33}$$

from point loads $= 82.94$

$M_{max} = R_2 b = 82.94(10.94) = 907.36'^k$

$M_{x=22.39} = \frac{.06(22.39)}{2}(33.33-22.39) = 7.35'^k$

$M_{max,tot} = 914.71'^k$

→ Find composite girder

$M_u = 914.71'^k$

Assume $Y_2 = 5.5$ (From Beam Analysis)
↳ assume $a \approx 1.5''$

Rough Economy

*2593 • W24×55: $\Sigma Q_n = 544$
$\phi M_n = 938$ ↳ $\frac{544}{14.6} = 37.3$ so 76 studs

→ *2566 most economic • W24×62: $\Sigma Q_n = 361$
$\phi M_n = 929$ ↳ $\frac{361}{14.6} = 24.7$ so 50 studs

*2626 • W24×68: $\Sigma Q_n = 251$
$\phi M_n = 933$ ↳ $\frac{251}{14.6} = 17.2$ so 36 studs

check $a = \frac{361}{.85(3)(100)} = 1.42''$ ∴ $Y_2 = 5.5''$ ok

$b_{eff} = \min \begin{cases} 2\left(\frac{33.33(12)}{8}\right) = 100 \leftarrow \text{controls} \\ 26.42(12) = 317.04 \end{cases}$

Total: 7	Chris Vandelogt	Tech 1	Floor Sys: Girder	4

→ Check unshored strength:

W24x62 $\phi_r M_p = 574'^k$

$$\omega_u = 1.2(46)(26.42) + 1.2(62) + 1.6(20)(26.4) = 2.38\ klf$$

$$\omega_u = 1.4(46)(26.42) + 1.4(62) = 1.78\ klf$$

$$M_u = \frac{2.38(33.33)^2}{8} = 330.5'^k < 574'^k \therefore \text{ ok for shoring}$$

→ Check wet conc deflection:

$$\omega_{wc} = 46(26.42) + 62 = 1.28\ klf$$

$$\Delta_{wc} = \frac{5(1.28)(33.33^4)(1728)}{384(29000)(1550)} = .79$$

Camber: $.8 \times .79 = .63 \approx .75''$

$$\Delta_{wcmax} = \frac{33.33(12)}{240} = 1.67 \text{ in} \quad \text{since } .79 < 1.67 \text{ ok}$$

→ Check LL deflection

$$\omega_{LL} = 100(26.42) = 2.64$$

$I_{LB} = 3110$ — From Table 3-20

$$\Delta_{LL} = \frac{5(2.64)(33.33^4)(1728)}{384(29000)(3110)} = .813$$

$$\Delta_{LLmax} = \frac{33.33(12)}{360} = 1.11 \quad \text{since } .813 < 1.11, \text{ ok}$$

→ Summary:

$\boxed{\text{W24x62 w/ 50 studs \& 3/4'' camber}}$

Total: 8	Chris Vandelogt	Tech 1	Snow Load	1

→ Ground Snow Load: $p_g = 40$ psf (from Fig 7-1)

→ Exposure Factor: $C_e = 1.0$ (from Table 7-2)

→ Thermal Factor: $C_t = 1.0$ (from Table 7-3)

→ Importance Factor: $I_s = 1.1$ (from Table 1.5-2)

→ Flat Roof Snow Load

$$p_f = .7\,C_e C_t I_s p_g$$
$$= .7(1)(1)(1.1)(40)$$
$$= \boxed{30.8 \text{ psf}} \leftarrow \text{Controls}$$

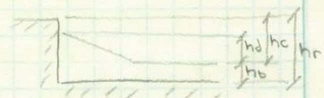
→ Snow Density

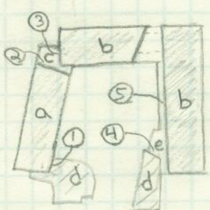
$$\gamma = .13p_g + 14 = .13(40) + 14 = 19.2 \text{ pcf} < \gamma_{max} = 30 \text{ pcf} \checkmark$$

→ Height of Balanced snow Load

$$h_b = p_f/\gamma = \frac{30.8}{19.2} = 1.6 \text{ ft}$$

→ Snow Drift





Heights:
- a = 51.83′
- b = 62.5′
- c = 40′
- d = 26′
- e = 14′

} Estimated

⊙ Drift Location

Total: 9	Chris Vandelogt	Tech 1	Snow Load	2

→ Location 1: Roof a and Roof d

$h_r = 51.83' - 26' = 25.83$

$h_c = h_r - h_b = 25.83 - 1.6 = 24.23$

$\frac{h_c}{h_b} = \frac{24.23}{1.6} = 15.14 > .2$, Drift must be calculated

• Windward

$\ell_u = 54.75'$ ℓ of low roof (for windward) for windward

$h_d = .75(.43 \sqrt[3]{\ell_u} \sqrt[4]{p_g + 10} - 1.5)$

$= .75(.43 \sqrt[3]{54.75} \sqrt[4]{40+10} - 1.5)$

$= 2.23\text{ ft} < h_c$ ok

$w_d = 4 h_d = 4(2.23) = 8.91\text{ ft}$

• Leeward

Controls

$\ell_u = 52.83\text{ft}$

$h_d = .43 \sqrt[3]{\ell_u} \sqrt[4]{p_g + 10} - 1.5$

$= .43 \sqrt[3]{52.83} \sqrt[4]{40+10} - 1.5$

$= 2.79\text{ ft} < h_c$ ok

$w_d = 4 h_d = 4(2.79) = 11.16\text{ ft}$

$p_d = 2.79(19.2\text{ pcf}) = 53.58$

Total Snow Load → 53.58 + 30.8

→ 84.38 psf

Additional Locations Calculated with spreadsheets

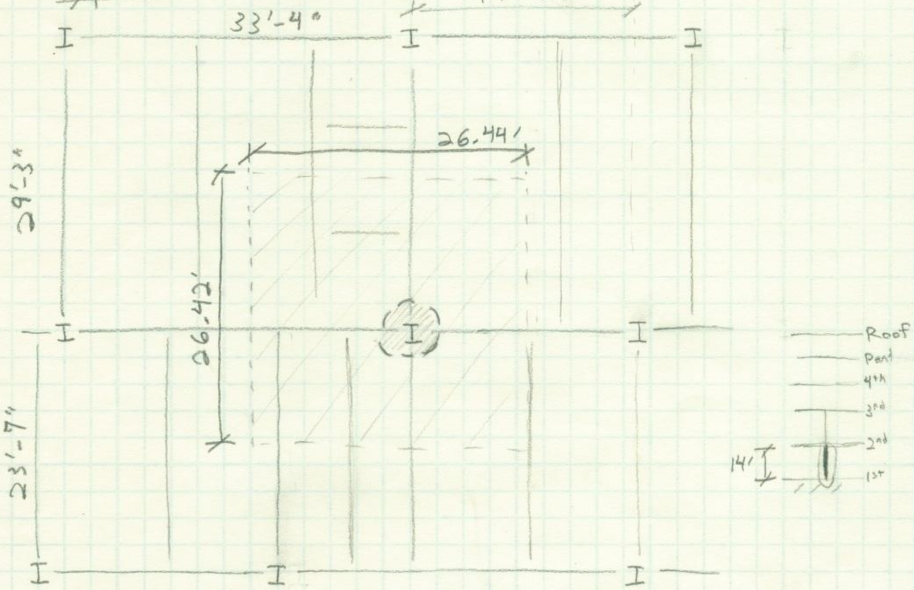
Total: 10 Chris Vandelogt Tech 1 Snow Load 3

Snow Drift Load Calculations

Drift Location	h_r (ft)	h_c (ft)	h_c/h_b	Windward				Leeward				$p_{d,max}$ (psf)	Total Snow Load (psf)
				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	25.83	24.23	15.14375	59.75	2.23	8.91	42.77	52.83	2.79	11.16	53.58	53.58	84.38
2	11.83	10.23	6.39375	34.50	1.67	6.67	32.00	165.50	4.78	19.11	91.73	91.73	122.53
3	22.5	20.9	13.0625	36.88	1.73	6.92	33.21	136.33	4.38	17.54	84.19	84.19	114.99
4	12	10.4	6.5	31.04	1.57	6.28	30.15	86.67	3.56	14.24	68.36	68.36	99.16
5	48.5	46.9	29.3125	11.00	0.78	3.13	15.02	52.83	2.79	11.16	53.58	53.58	84.38

Total: 11 | Chris Vandelogt | Tech 1 | Interior Column | 1

Typical Grid (2nd Floor)



→ Tributary area:

$$\left(\frac{23.583+24.25}{2}\right)\left(\frac{33.33+19.54}{2}\right) = 698.32\ ft^2$$

→ Loadings: Per ASCE 7-10 Residential,

- 2nd Floor: 100 psf
- non-red → Corridors: 100 psf
- reducible → Private Rooms: 40 psf
- 3rd + 4th Floor
- Mechanical: 150 psf (assumed)
- Pent
- Roof: Dead: 60 psf (assumed)
  - Snow: 30.8 psf — See Snow Load Calcs
- slab: 46 psf
- Framing: 10 psf
- Super-imposed: 10 psf
- MEP: 10 psf
- Partition: 20 psf

→ Corridor Area:

Width: 6.917'
Length: 26.44'

Trib Area: 182.89 ft^2

Total: 12 | Chris Vandelogt | Tech 1 | Interior Column | 2

→ Loads: LL red: $.25 + \frac{15}{\sqrt{4(698.32-182.89)}} = .58$

Level 2:

$P_L = 100(698.32) = 69.83^k$

$P_D = (10+10+10+46+20)(698.32) = 67.04^k$

Level 3:

$P_L = .58(40)(515.43) + (100)(182.89) = 30.25^k$

$P_D = (10+10+10+46+20)(698.32) = 67.04^k$

Level 4:

$P_L = .58(40)(515.43) + (100)(182.89) = 30.25^k$

No framing

$P_D = (10+10+46+20)(698.3) = 60.05^k$

Mech Pent:

P_L: none

P_D: $(10+10+46+150)(698.3) = 150.83^k$

Roof:

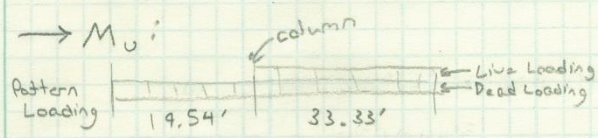
P_L: none

P_S: $30.8(698.32) = 21.51^k$

P_D: $60(698.32) = 41.9^k$

→ $P_u = 1.2(67.04+60.05+150.83+41.9+67.04)$
$+1.6(30.25+30.25+69.83) + .5(21.51^k)$
$= 683.52^k$

→ M_u:



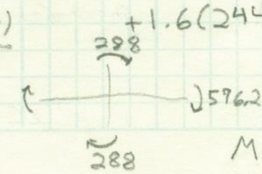
Live: $M_{uR} = \frac{(2.64)(33.33^2)}{12}$ — 100(26.42)
$= 244.4$

Dead: $\frac{w_u l^2}{12} = \frac{(2.54)(19.54^2)}{12}$ — 96(26.42')

$M_{uL} = 80.82'^k$

Dead: $M_{uR} = \frac{(2.54)(33.33^2)}{12}$
$= 235.14'^k$

$1.2(235.14 - 80.82)$
$+1.6(244.4) = 576.2$



$M_u = 288'^k$

Total: 13	Chris Vandelogt	Tech 1	Interior Column	3

→ Estimate Column sizes

$P_{eq} = P_u + 24 M_u / d$

$P_{eq} = 683.52 + 24\left(\frac{288}{12}\right)$

$= 1259.5^k$

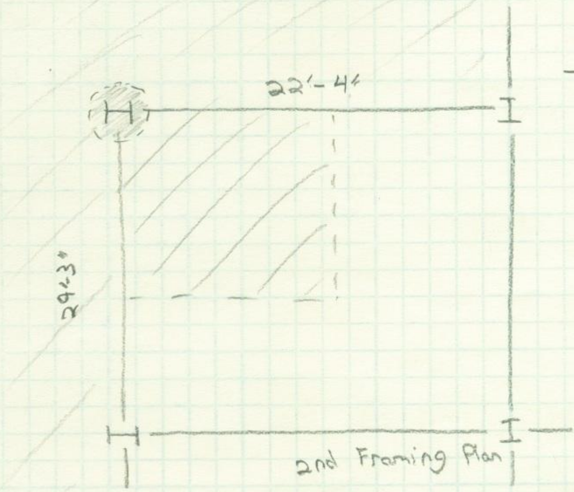
Assume $k = 1$
$L = 14'$
$d = 12''$

Using Table 4-1:

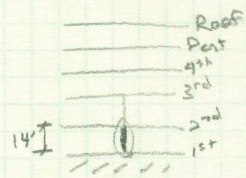
$\boxed{W12 \times 120}$, $\phi_c P_n = 1280^k > 1259.5^k$ ✓

AMPAD

Total: 14 | Chris Vandelogt | Tech 1 | Exterior Column | 1



→ Tributary area:
$\left(\frac{22.33}{2}\right)\left(\frac{29.25}{2}\right)$
$= 326.63\ ft^2$



→ Loadings: Per ASCE 7-10

- Live:
  - 2nd Floor: 100 psf (Office Lobbies)
  - 3rd / 4th Floor: 40 psf (Private Rooms)

Non-reducible since 326<400

- Dead / Snow
  - Mechanical: 150 psf (assumed)
  - Roof: Dead: 60 psf (assumed)
    Snow: 30.8 psf — see Snow load Calcs
  - Slab: 46 psf
  - Framing: 10 psf
  - Super-imposed: 10 psf
  - MEP: 10 psf
  - Partition: 20 psf

Total: 15 | Chris Vandelogt | Tech 1 | Exterior Column | 2

→ Loads:

Level 2:

$P_L = 100(326.63) = 32.66^k$

$P_D = (10+10+10+46+20)(326.63) = 31.36^k$

Level 3:

$P_L = 40(326.63) = 13.07^k$

$P_D = (10+10+10+46+20)(326.63) = 31.36^k$

Level 4:

$P_L = 40(326.63) = 13.07^k$

No framing

$P_D = (10+10+46+20)(326.63) = 28.09^k$

Mech Pent:

$P_L =$ none

$P_D = (10+10+46+150)(326.63) = 70.55^k$

Roof:

$P_L =$ none

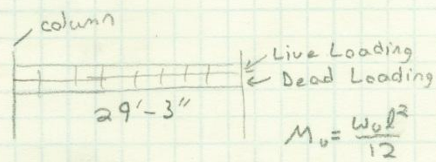
$P_S = 30.8(326.63) = 10.06^k$

$P_D = 60(326.63) = 19.6^k$

→ $P_u = 1.2(31.36+31.36+28.09+70.55+19.6)$
$+1.6(32.66+13.07+13.07)+.5(10.06)$

$= 316.26^k$

→ M_u:



$M_u = \dfrac{w_u l^2}{12}$

100(11,165)

Live: $M_u = \dfrac{(1.11)(29.25^2)}{16}$

$= 59.36$

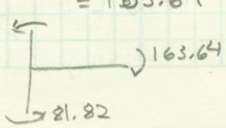
96(11,165)

Dead: $M_u = \dfrac{(1.07)(29.25^2)}{16}$

$= 57.22$

$M_u = 1.2(57.22) + 1.6(59.36)$

$= 163.64$



$M_u = 81.82'^k$

Total 116	Chris Vandelogt	Tech 1	Exterior Column	3

→ Estimate Column Sizes

$P_{eq} = P_u + 24M_u/d$

$P_{eq} = 316.26 + 24\left(\frac{81.82}{10}\right)$

$= \underline{512.63^k}$

Assume $K=1$
$L=14'$
$d=10''$

<u>Using Table 4-1:</u>

$\boxed{W10\times54}$, $\phi P_n = 519^k > 512.63^k$ ✓

Appendix B: Seismic Load Calcs

Total: 17 | Chris Vandelogt | Tech 1 | Seismic | 1

→ Importance Factor: $I_e = 1.25$ (From Table 1.5-2)
→ .2-Sec Spectral Response Acc: $S_s = .21$
1-Sec Spectral " " : $S_1 = .06$ (From USGS)

→ Site Class: C (From Geotechnical Report)

→ Site Coefficient: $F_a = 1.2$, $F_v = 1.7$ (From Table 11.4-1,2)

→ Modified SRA:

$\cdot S_{MS} = F_a S_s = 1.2(.21) = .252$

$\cdot S_{M1} = F_v S_1 = 1.7(.06) = .102$

→ Design SRA:

Importance Cat III

$\cdot S_{DS} = \frac{2}{3} S_{MS} = .168$ Design Cat B

$\cdot S_{D1} = \frac{2}{3} S_{M1} = .068$ Design Cat B

} From Table 11.6-1,2

Use Seismic Design Category B

→ Response Modification Coefficient (R): From Table 12.2-1
$R = 3.25$ (Steel ordinary concentrically Braced Frame)

→ Long Period Transition Period: $T_L = 6$ sec (From Fig 22-12)

Equivalent Lateral Force Procedure

→ Approx. Fundamental Period (T_a):

$T_a = C_t h_n^x$

$C_t = .02$, $x = .75$ } Table 12.8-2 "other structures"

$T_a = .02(62.5)^{.75}$

$= .445$ sec

→ Fundamental Period

$T = T_a (C_u)$ $C_u = 1.7$ (From Table 12.8-1)

$T = .445(1.7)$

$= .7565$ sec

Total: 18 | Chris Vandelogt | Tech 1 | Seismic | 2

→ Calculation of Seismic Response Coefficient (C_s)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{.168}{\left(\frac{3.25}{1.25}\right)} = .067$$

since $T \le T_L$, $.7565 \le 6$

$$C_{s,max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{.068}{.7565\left(\frac{3.25}{1.25}\right)} = .035 < .067$$

controls

$$C_{s,min} = \overset{= .00924}{.044 S_{DS} I_e} \geq .01 \quad = .01$$

$C_s = .035$

→ Loads of Building

Deads Loads:

→ Floors 1-3: 96 psf
- Framing: 10 psf
- Super-imposed: 10 psf
- MEP: 10 psf
- Partitions: 20 psf
- Slab: 46 psf

→ Floor 4: 86 psf
- Same as floors 1-3 except framing

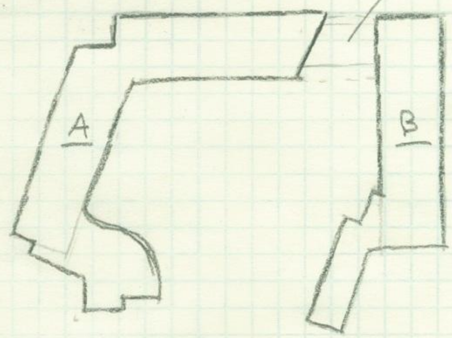
→ Penthouse: 216 psf
- Super-imposed: 10 psf
- MEP: 10 psf
- Slab: 46 psf
- MEP Equip.: 150 psf

→ Roof: 66.16
- Framing: 60 psf
- Snow: since 30.8 > 30 → .2(30.8) = 6.16

*Note: There are some places where mechanical equipment is located on floors 1-4

Total: 14 | Chris Vandelogt | Tech 1 | Seismic | 3

→ Building Plan



Due to this assume building would act as two separate buildings in seismic

→ Weight of Building A

Weight:

- Floor 1
  - $A_{Mech} = 1446\ ft^2 \cdot 150\ psf =$ 217^k
  - $A_{other} = 16832\ ft^2 \cdot 96\ psf =$ 1616^k
- Floor 2
  - $A_{Tot} = 17451\ ft^2 \cdot 96\ psf =$ 1675^k
- Floor 3
  - $A_{Tot} = 19130\ ft^2 \cdot 96\ psf =$ 1837^k
- Floor 4
  - $A_{Mech} = 8391\ ft^2 \cdot 150\ psf =$ 1259^k
  - $A_{other} = 8322\ ft^2 \cdot 86\ psf =$ 716^k
- Penthouse
  - $A_{Tot} = 6704\ ft^2 \cdot 216\ psf =$ 1448^k
  - $A_{roof} = 8391\ ft^2 \cdot 66.16\ psf =$ 555^k
- Roof
  - $A_{Tot} = 6704\ ft^2 \cdot 66.16\ psf =$ 444^k

Total Weight: 9767^k

Total: 20 | Chris Vandelogt | Tech 1 | Seismic | 4

→ Weight of Building B

- Floor 1 — Weight:
  - $A_{Mech} = 10513\ ft^2 \cdot 150\ psf =$ 1577^k
  - $A_{other} = 1108\ ft^2 \cdot 96\ psf =$ 1064^k
- Floor 2
  - $A_{Tot} = 12456\ ft^2 \cdot 96\ psf =$ 1196^k
- Floor 3
  - $A_{Tot} = 12451\ ft^2 \cdot 96\ psf =$ 1195^k
- Floor 4
  - $A_{Tot} = 12456\ ft^2 \cdot 86\ psf =$ 1071^k
- Penthouse
  - $A_{Tot} = 11487\ ft^2 \cdot 216\ psf =$ 2481^k
- Roof
  - $A_{Tot} = 11487\ ft^2 \cdot 66.16\ psf =$ 760^k

Total Weight: 9344^k

→ Building Analysis

- Base Shear: $V = C_s W$
- Vertical Distribution of Seismic Forces

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\Sigma w_x h_x^k}$$

$$k = 1 + \frac{.7565 - .5}{2} = 1.1283$$

* See Spreadsheet for Calculations

Total: 21 Chris Vandelogt Tech 1 Seismic 5

Seismic Forces

Building A

Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	1833	0	0.00	0.00	0.00	341.85	0.00
2nd	1675	14	85277.05	0.08	26.18	341.85	366.45
3rd	1837	26.66	195745.76	0.18	60.08	315.67	1601.80
4th	1975	37.33	310557.05	0.28	95.32	255.59	3558.40
Penthouse	2003	48	419016.48	0.38	128.61	160.26	6173.45
Roof	444	62.5	103117.80	0.09	31.65	31.65	1978.20
Sum:	9767		1113714.13	1.00 √ok	341.85 √ok		
Base Shear ($V=C_sW$) =			341.85		Total Overturning Moment =		13678.29

Building B

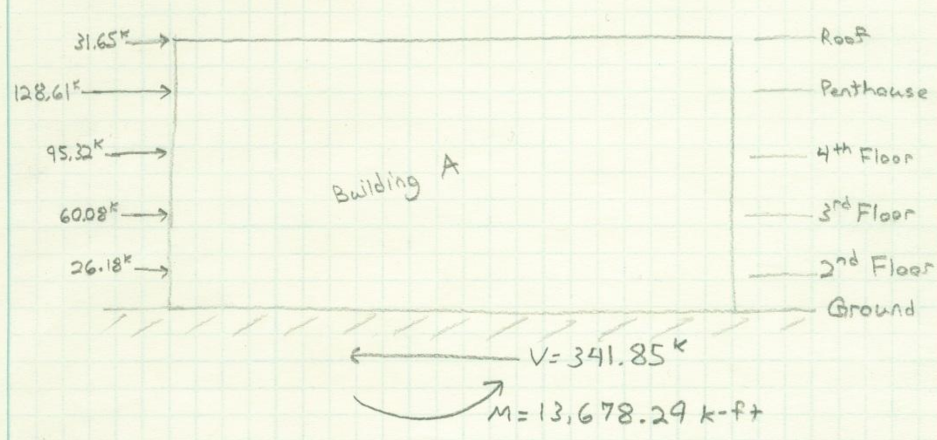
Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	2641	0	0.00	0.00	0.00	327.04	0.00
2nd	1196	14	58315.01	0.06	18.04	327.04	252.58
3rd	1195	26.66	120501.43	0.11	37.28	309.00	993.91
4th	1071	37.33	155691.35	0.15	48.17	271.72	1798.11
Penthouse	2481	48	533460.01	0.50	165.04	223.55	7922.04
Roof	760	62.5	189109.52	0.18	58.51	58.51	3656.68
Sum:	9344		1057077.32	1.00 √ok	327.04 √ok		
Base Shear ($V=C_sW$) =			327.04		Total Overturning Moment =		14623.33

Total: 22 | Chris Vandelogt | Tech 1 | Seismic | 6

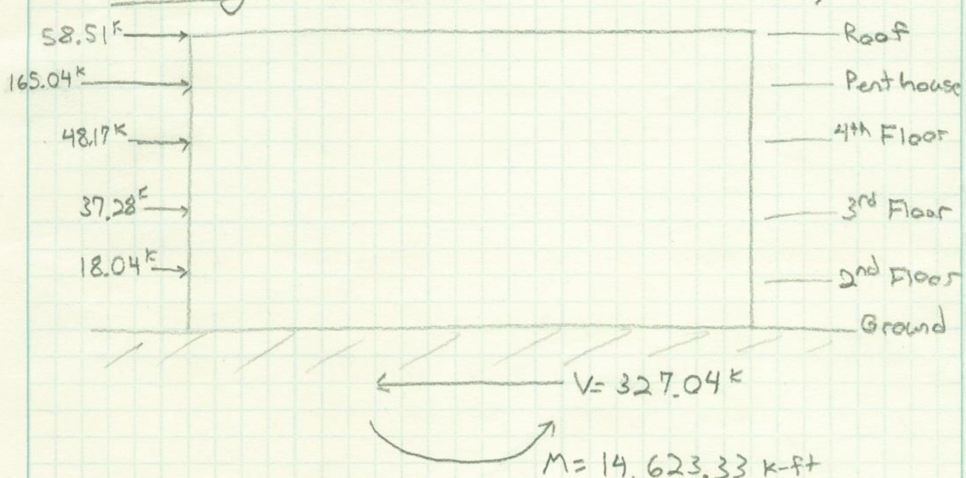
→ Summary:

* see spreadsheet for calcs and loads

Building A: (Dimensions not drawn to scale)

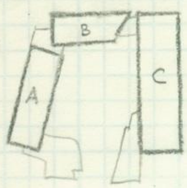


Building B: (Dimensions not drawn to scale)



Appendix B: Wind Load Calcs

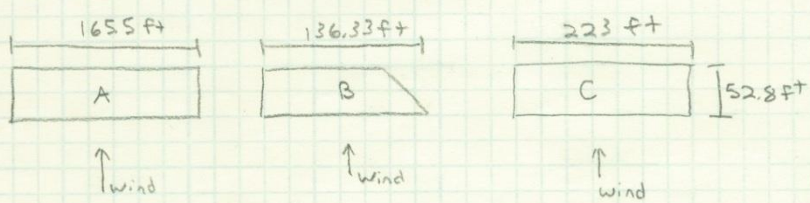
Total: 23	Chris Vandelogt	Tech 1	Wind Analysis	1



- Thick outline represents the tallest height of each section of the structure
- To simplify, analyze the structure as 3 different buildings (outlined and labeled a, b, and c)

→ Dimensions

- Building A
  - Length: 165.5 ft
  - Width: 52.8 ft
  - Height: 51.83 ft
- Building B
  - Length: 136.33 ft
  - Width: 52.8 ft
  - Height: 62.5 ft
- Building C
  - Length: 223 ft
  - Width: 52.8 ft
  - Height: 62.5 ft



Total: 24	Chris Vandelogt	Tech 1	Wind Analysis	2

→ Basic Wind Speed: From Figure 26.5-1B ASCE 7-10
$V = 120$ mph

→ Wind Directionality Factor: From Table 26.6-1
$k_d = .85$

→ Occupancy Category III

→ Exposure Category: C From Section 26.7.3

→ Topography Factor: From Section 26.8.2
$K_{zt} = 1.0$

→ Frequency: From Sect 26.9.2.1

$$L_{eff} = \frac{\Sigma h_i L_i}{\Sigma h_i} = 52.8$$

$h = 62.5 < 4(52.8)$ ← Allowed to use approx Freq

$$n_a = \frac{75}{h} \quad \text{(Equation 26.9-4)}$$

$$= \frac{75}{62.5} \quad \text{or} \quad \frac{75}{50}$$

$$= 1.2 \quad \text{or} \quad 1.5 > 1.0 \therefore \text{Rigid}$$

→ Gust Factor: From Sect 26.9

$$G = .925\left(\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

where: $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$

- $\bar{z} = .6h > z_{min} = 15'$ (Table 26.9-1) ✓ok

- $c = .2$ (Table 26.9-1)

g_Q and $g_v = 3.4$

*See spreadsheet for calculations

Total: 25 | Chris Vandelogt | Tech 1 | Wind Analysis | 3

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$$

- $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}}$
  - $\ell = 500$
  - $\bar{\varepsilon} = 1/5$

→ *Note: Ignore internal pressure since net addition is zero and no large openings are located in the building

→ Velocity Pressure Exposure: From Table 27.3-1

k_z @ 14' = .85 $\quad$ k_z @ 37.33' = 1.024
k_z @ 26.66' = .953 $\quad$ k_z @ 51.83' = 1.097
k_z @ 48' = 1.08 $\quad$ k_z @ 62.5 = 1.14

→ Velocity Pressure: From Sect 27.3.2

$$q_z = .00256\, K_z K_{zt} K_d V^2$$

* See spreadsheet for Calculations

→ Wind Loads: From Section 27.4.1

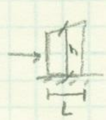
$$p = qGC_p$$

q_z for windward
q_h for sides and leeward

where $C_p = \begin{cases} .8 \text{ windward} \\ -.5 \text{ leeward} \\ -.7 \text{ sides} \end{cases}$ From Fig 27.4-1

$L/B < 1.0$

since roofs are monoslope:



use $h/L \geq 1.0$

$\theta < 10°$

$C_p = \begin{cases} 0 \text{ to } h/2 : -1.3, -.18 \\ > h/2 : -.7, -.18 \end{cases}$ Worst cases

From Fig 27.4-1

* See spreadsheet for Calculations

Total: 26 Chris Vandelogt Tech 1 Wind Analysis 4

Wind Analysis -Wind Normal to Long Dimension (Length)

Building Dimensions				Gust Factor Calculations				
Building	Length (ft)	Width (ft)	Height (ft)	z_{bar}	I_{zbar}	L_{zbar}	Q	G
A	165.500	52.800	51.830	31.098	0.202	494.099	0.853	0.852
B	136.330	52.800	62.500	37.500	0.196	512.948	0.862	0.857
C	223.000	52.800	62.500	37.500	0.196	512.948	0.835	0.844

Constants

V (mph) =	120.000	$C_{p,windward}$ =	0.800	$C_{p,roof<h/2}$ =	-1.300
k_d =	0.850	$C_{p,leeward}$ =	-0.500	$C_{p,roof>h/2}$ =	-0.700
k_{zt} =	1.000	$C_{p,sides}$ =	-0.700		

Building A

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	18.145	-14.636	-20.490		
3rd	26.660	0.953	29.862	20.344	-14.636	-20.490		
Penthouse	37.330	1.024	32.086	21.859	-14.636	-20.490		
Roof	51.830	1.097	34.374	23.418	-14.636	-20.490	-38.054	-20.490

Building B

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	18.262	-15.308	-21.431		
3rd	26.660	0.953	29.862	20.475	-15.308	-21.431		
4th	37.330	1.024	32.086	22.001	-15.308	-21.431		
Penthouse	48.000	1.080	33.841	23.204	-15.308	-21.431		
Roof	62.500	1.140	35.721	24.493	-15.308	-21.431	-39.801	-21.431

Building C

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)	$p_{roof<h/2}$ (lb/ft^2)	$p_{roof>h/2}$ (lb/ft^2)
2nd	14.000	0.850	26.634	17.979	-15.071	-21.099		
3rd	26.660	0.953	29.862	20.158	-15.071	-21.099		
4th	37.330	1.024	32.086	21.659	-15.071	-21.099		
Penthouse	48.000	1.080	33.841	22.844	-15.071	-21.099		
Roof	62.500	1.140	35.721	24.113	-15.071	-21.099	-39.184	-21.099

Total: 27 Chris Vandelogt Tech 1 Wind Analysis 5

Wind Analysis -Wind Normal to Short Dimension (Width)

Building Dimensions				Gust Factor Calculations				
Building	Width (ft)	Length (ft)	Height (ft)	z_{bar}	I_{zbar}	L_{zbar}	Q	G
A	52.800	165.500	51.830	31.098	0.202	494.099	0.899	0.875
B	52.800	136.330	62.500	37.500	0.196	512.948	0.896	0.874
C	52.800	223.000	62.500	37.500	0.196	512.948	0.896	0.874

Constants					
V (mph) =	120.000	$C_{p,windward}$ =	0.800	$C_{p,roof<h/2}$ =	-1.300
k_d =	0.850	$C_{p,leeward}$ =	-0.500	$C_{p,roof>h/2}$ =	-0.700
k_{zt} =	1.000	$C_{p,sides}$ =	-0.700		

Building A

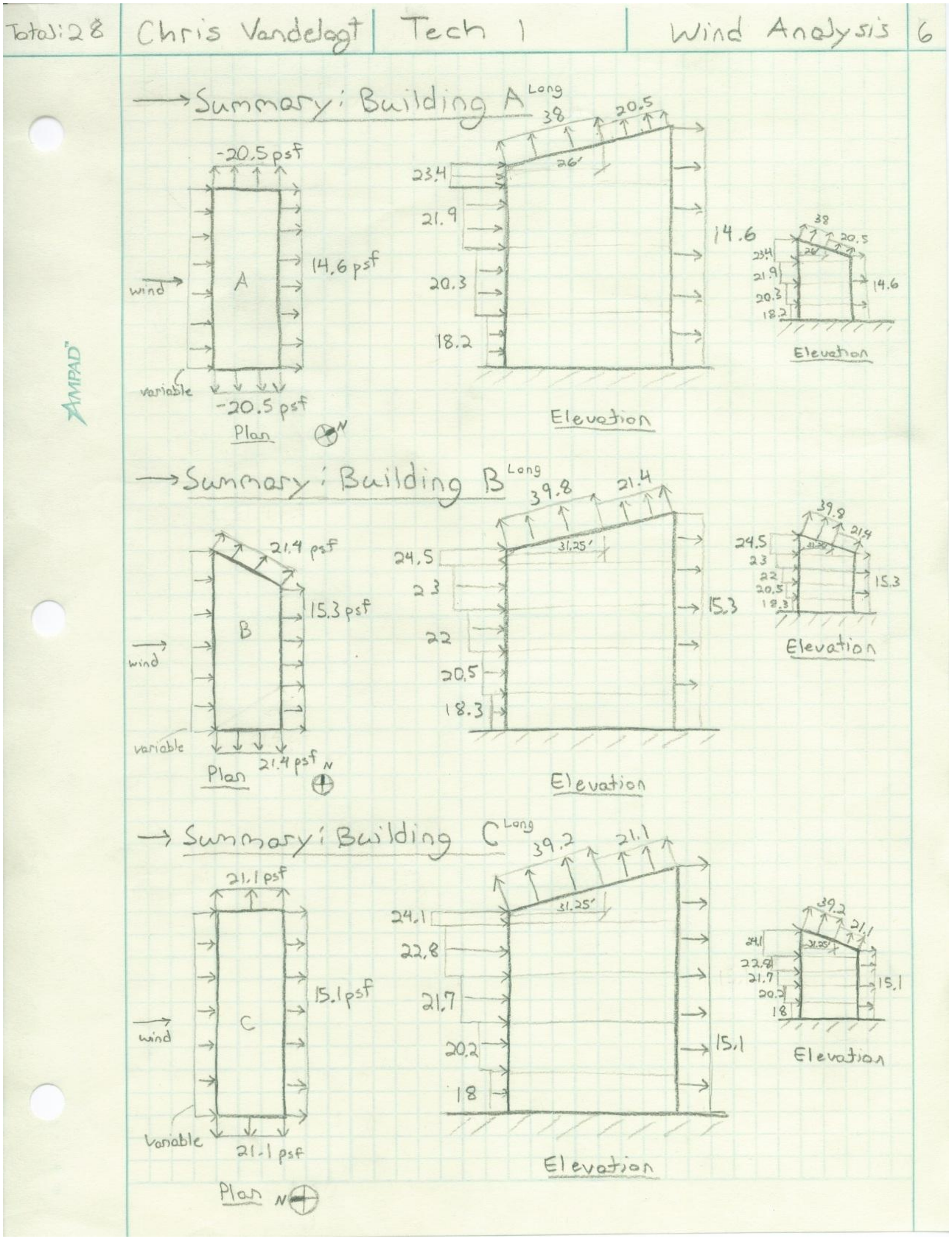
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.639	-15.034	-21.048
3rd	26.660	0.953	29.862	20.897	-15.034	-21.048
Penthouse	37.330	1.024	32.086	22.454	-15.034	-21.048
Roof	51.830	1.097	34.374	24.055	-15.034	-21.048

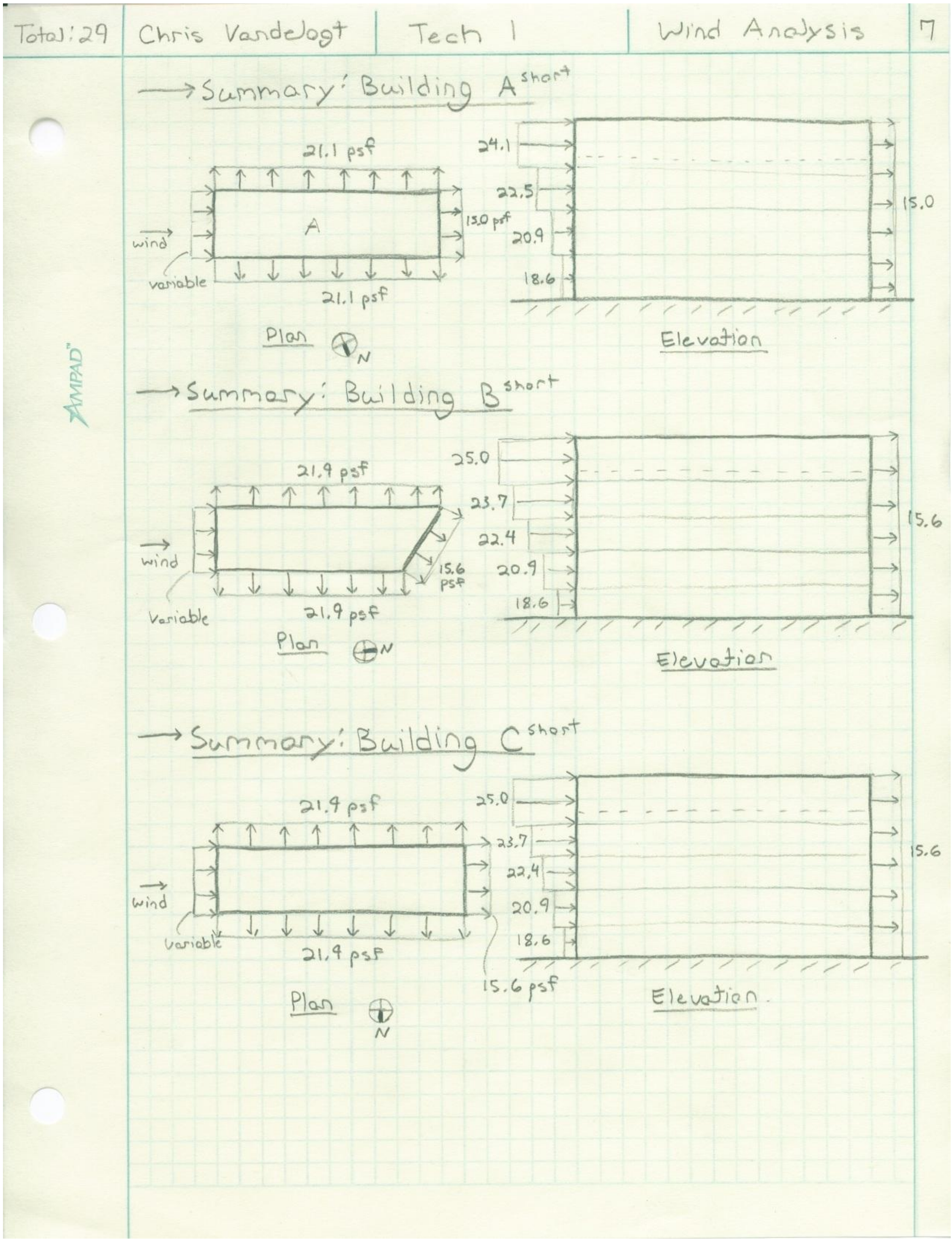
Building B

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721	24.972	-15.608	-21.851

Building C

Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721	24.972	-15.608	-21.851


Total: 28
Chris Vandelogt
Tech 1
Wind Analysis
6
Summary: Building A Long
-20.5 psf
wind
A
14.6 psf
variable
-20.5 psf
Plan
N
38
20.5
26'
23.4
21.9
20.3
18.2
14.6
Elevation
Summary: Building B Long
21.4 psf
15.3 psf
B
wind
variable
21.4 psf
Plan
N
39.8
21.4
31.25'
24.5
23
22
20.5
18.3
15.3
Elevation
Summary: Building C Long
21.1 psf
15.1 psf
C
wind
Variable
21.1 psf
Plan
N
39.2
21.1
31.25'
24.1
22.8
21.7
20.2
18
15.1
Elevation



Total: 29
Chris Vandelogt
Tech 1
Wind Analysis
7
Summary: Building A short
21.1 psf
A
wind
variable
13.0 psf
21.1 psf
Plan
N
24.1
22.5
20.9
18.6
15.0
Elevation
Summary: Building B short
21.9 psf
wind
Variable
15.6 psf
21.9 psf
Plan
N
25.0
23.7
22.4
20.9
18.6
15.6
Elevation
Summary: Building C short
21.9 psf
wind
variable
21.9 psf
15.6 psf
Plan
N
25.0
23.7
22.4
20.9
18.6
15.6
Elevation