Lateral System Analysis (Tech 3)

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Executive Summary

The lateral system analysis report explains the lateral loads that act upon the structure of Global Village and the lateral systems that resist them. 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 lateral system consists of concentrically braced frames and wood shear walls in both the N-S direction as well as the E-W direction. For the purposes of this report, only the north leg of Global Village will be analyzed. In this leg, six braced frames are used between the ground and the third floor while shear walls are placed on the third, fourth, and fifth floors.

For this technical report, the lateral system was analyzed under eight different load cases. Two come from seismic forces acting in the N-S and E-W Directions. The other six are for various wind load cases as described in Figure 27.4-8 of ASCE 7-10. An ETABS model was then used to analyze these different load cases. Some assumptions were made to the model due to lack of knowledge of the program or to simplify calculations. The major assumption was the use of only braced frames since it was unknown how to model wood shear walls. The braced frames were then assumed to replicate up to the roof. The results of this model were used to calculate and check drift, overturning, torsion, direct shear, and member strengths.

Story drifts were found directly from the ETABS model. The worst case in each direction, for both wind and seismic were considered. These values were then compared to allowable seismic and wind story drifts as outlined in ASCE 7-10. The maximum drifts in both the N-S and E-W Directions were controlled by loads due to seismic. The total drift from ETABS in the N-S Direction is .463” and .798” in the E-W Direction; which are well below the allowed 10.513”. As a note, a maximum total drift of .426” caused by wind in the N-S Direction is below the allowable 1.752”. As a result, the lateral system is adequate for drift.

The overturning moment was controlled by seismic loads which produce a moment of 15,876.8$^{ft\text{-}k}$. The self-weight of the building creates a resisting moment of 246,681.68$^{ft\text{-}k}$ which is far above the overturning moment. Therefore, overturning is not an issue.

The penthouse level was considered for torsion due to seismic loads. Seismic forces were chosen because of a greater eccentricity than wind. The penthouse level was then selected since it had the largest story force. The maximum torsional shear was applied to frame WB-4 with a magnitude of 10.5^{k}

in the E-W Direction and 7.3^k in the N-S Direction. This may have an impact on the structure and should be considered further.

Direct shear values show how the loads are distributed between the braced frames in each direction. The amount of force that each load receives depends on the stiffness of each frame compared to the total stiffness in that direction. In the E-W Direction, two identical braced frames with equal stiffness receive half of the load applied in the E-W Direction. In the N-S Direction, frames WB-2, 3, 4 receive around 33% of the load applied and WB-1 only receives 1%. This is predominantly due to WB-1 having no cross bracing on the bottom floor.

Lastly, an HSS9x9x½ cross brace member and a W12x120 column were checked for strength. It was found that the ground floor HSS member in frame WB-2 had the largest axial force of 195.51^k. This was due to load Case 1 acting in the N-S Direction. Using the AISC Manual, this member has a capacity of 365^k and is therefore sufficient to support the load. The W12x120 column in frame WB-1 was checked using the combined flexure and compression equation on page 6-2 of the AISC Manual. The outcome of the equation came out to be .15 which is much lower than 1.0 and is therefore adequate.

Table of Contents

Purpose

The purpose of Technical Report 3 is to analyze the lateral system of Global Village. This report will examine lateral loads, due to wind and seismic, and the systems that resist them. This report will also explain how these loads are distributed and check drift, overturning, torsion, direct shear, and member strengths.

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.

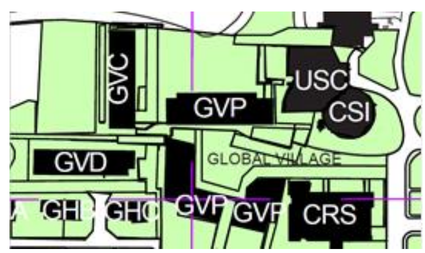


Figure 1: GVP is Building 400 (Global Village Building). GVC and GVD are Buildings 403 and 404 (Global Way Buildings). Courtesy of RIT.

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.

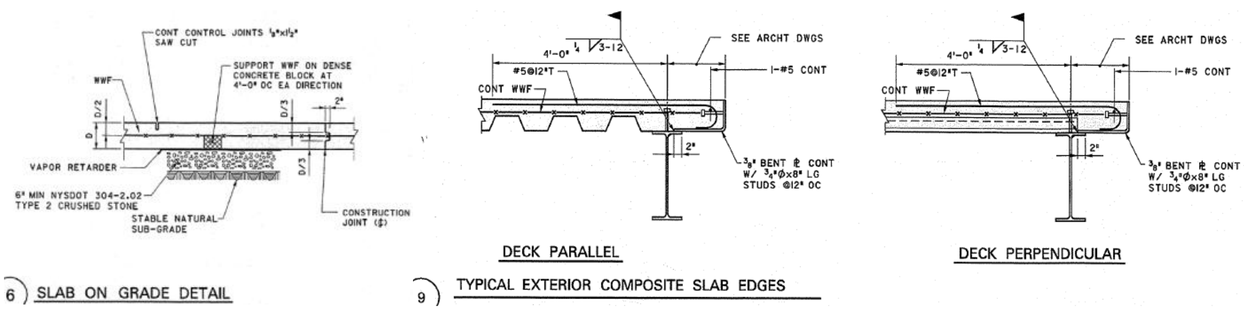


Figure 2: Typical composite slab details. Courtesy of RIT. Drawings not to scale.

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.

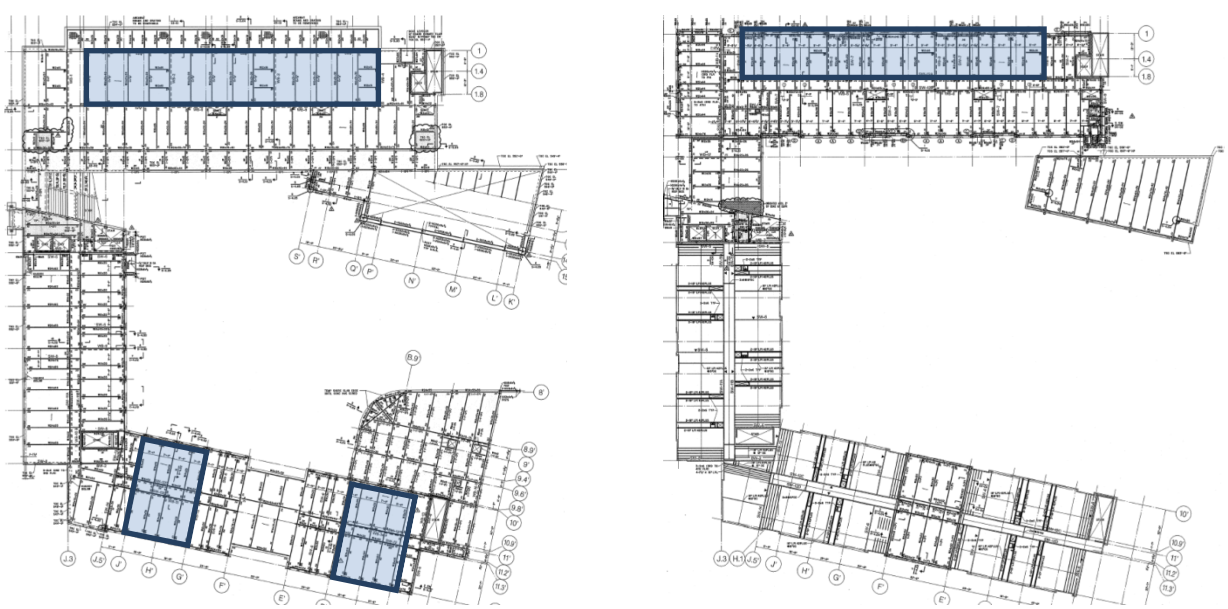


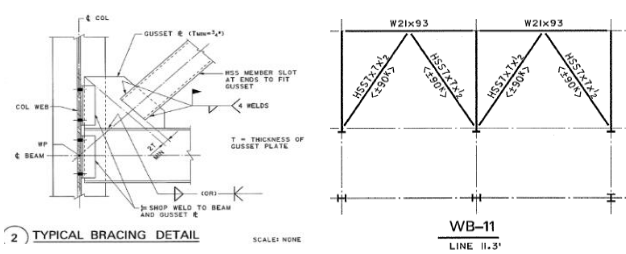
Figure 3: 2nd Floor (left) and 3rd Floor (right) framing plans. Typical bays on each level highlighted. Courtesy of RIT. Drawings not to scale.

Lateral System

The lateral load resisting system consists of concentrically braced frames and wood shear walls, each acting on separate floors. Braced frames are used between the ground and the third floor while shear walls are placed on the third, fourth, and fifth (penthouse) floors.

The lateral HSS bracing ranges in size where the majority is HSS7x7x½. See **Figure 4** for details and placements of the braced framing used on the second floor. The shear walls are made of wood blocking, consisting of 2x4's, and sheathing. These wood shear walls are used due to the use of wood structuring above the third floor. For placements and details, see **Figure 5**.

For the purposes of this report, only the north leg of Global Village will be analyzed. Reasoning behind this decision was due to greater wind and seismic loadings which will be explained further later on. The rest of this report will also explain the lateral system in more detail; including load paths and distribution, torsion, drift, and overturning moments. An ETABS model was also produced to compute results and compare them to hand calculations.



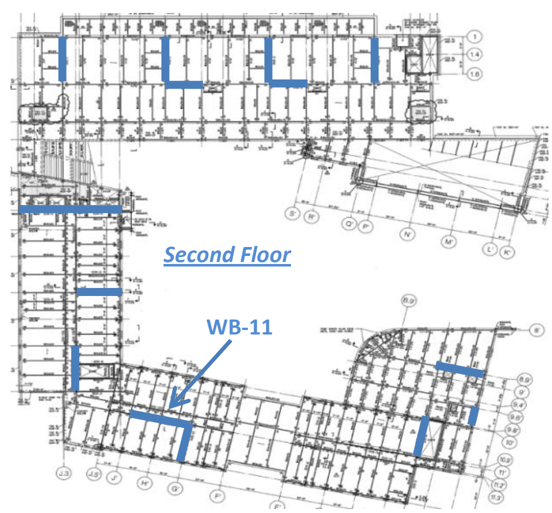
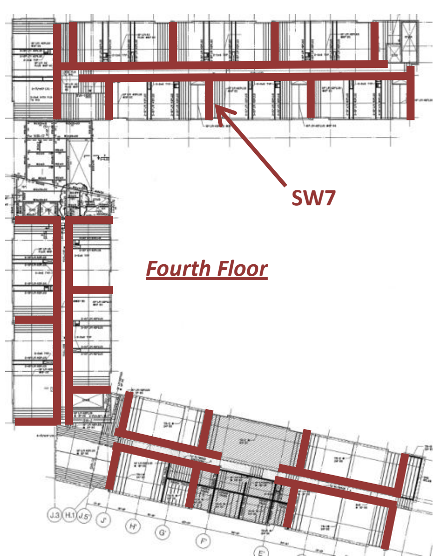


Figure 4: Typical bracing details and placement of bracing on 2nd Floor. Courtesy of RIT. Drawings not to scale.



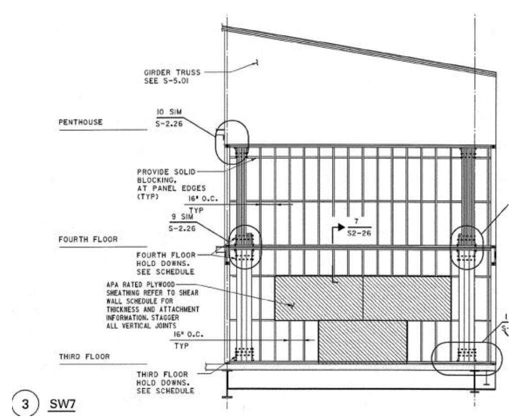


Figure 5: Typical shear wall details and placements used on the 4th Floor. Courtesy of RIT. Drawings not to scale.

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

Gravity 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

Lateral Loads

In order to analyze the lateral system of Global Village, wind and seismic loads were calculated for this report. Wind loads were calculated using the MFRS (Directional) Procedure and seismic loads were calculated using the Equivalent Lateral Force Procedure outlined in ASCE 7-10. A summary of the story forces for both wind and seismic can be found at the end of this section.

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 6**. This was done because of the differing heights in the structure and some sections could be considered neglected (passageways). These separate buildings were then assumed to have constant heights and to contain no component and cladding effects.

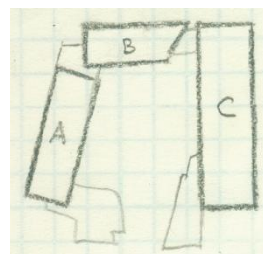


Figure 6: Simplifying building structure

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 7**. The wind pressure calculations in the short dimension are given in **Table 6**. The results can be found in **Figure 8**. 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		
Pent	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		
Pent	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		
Pent	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

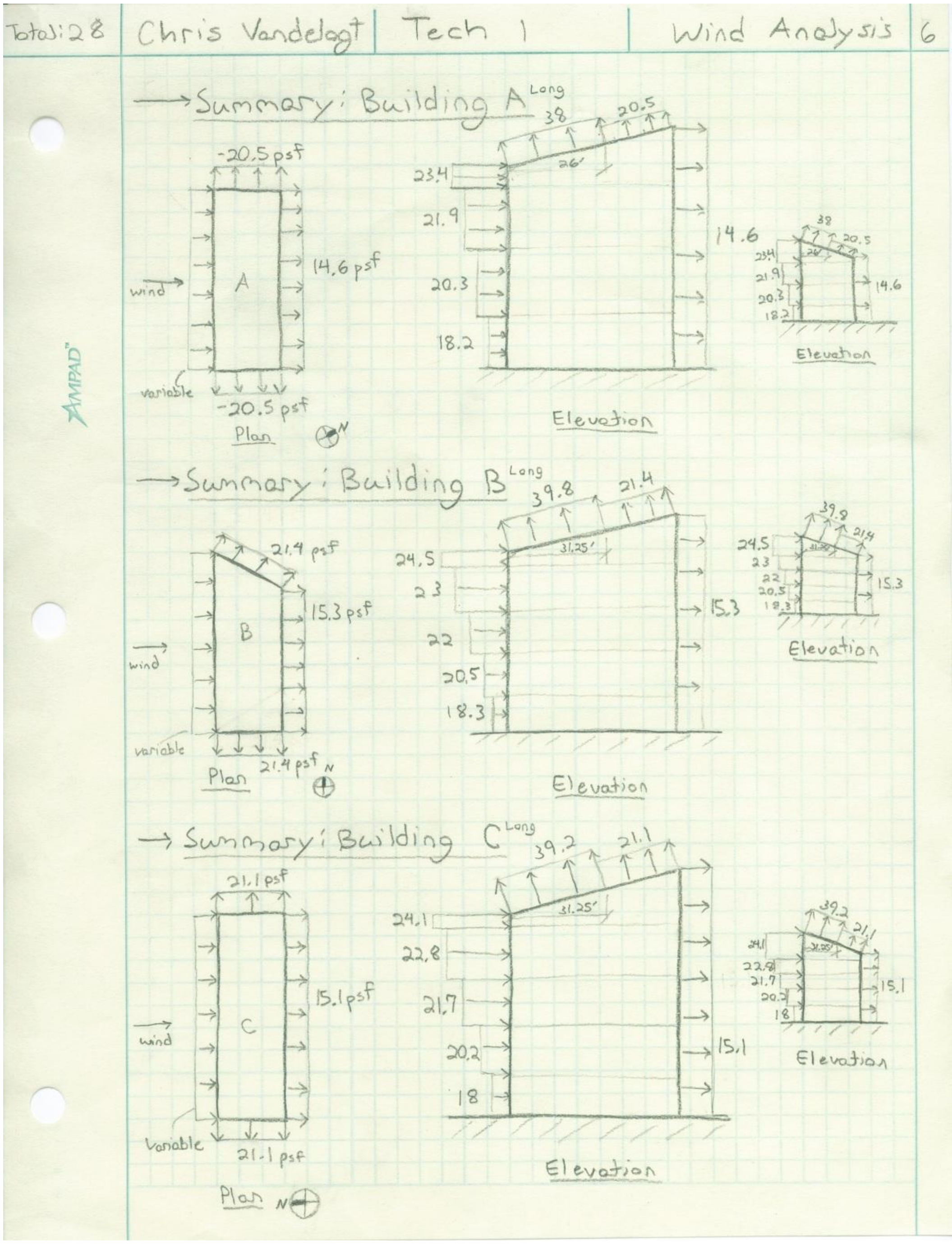


Figure 7: Summary of wind pressures normal to long dimension

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

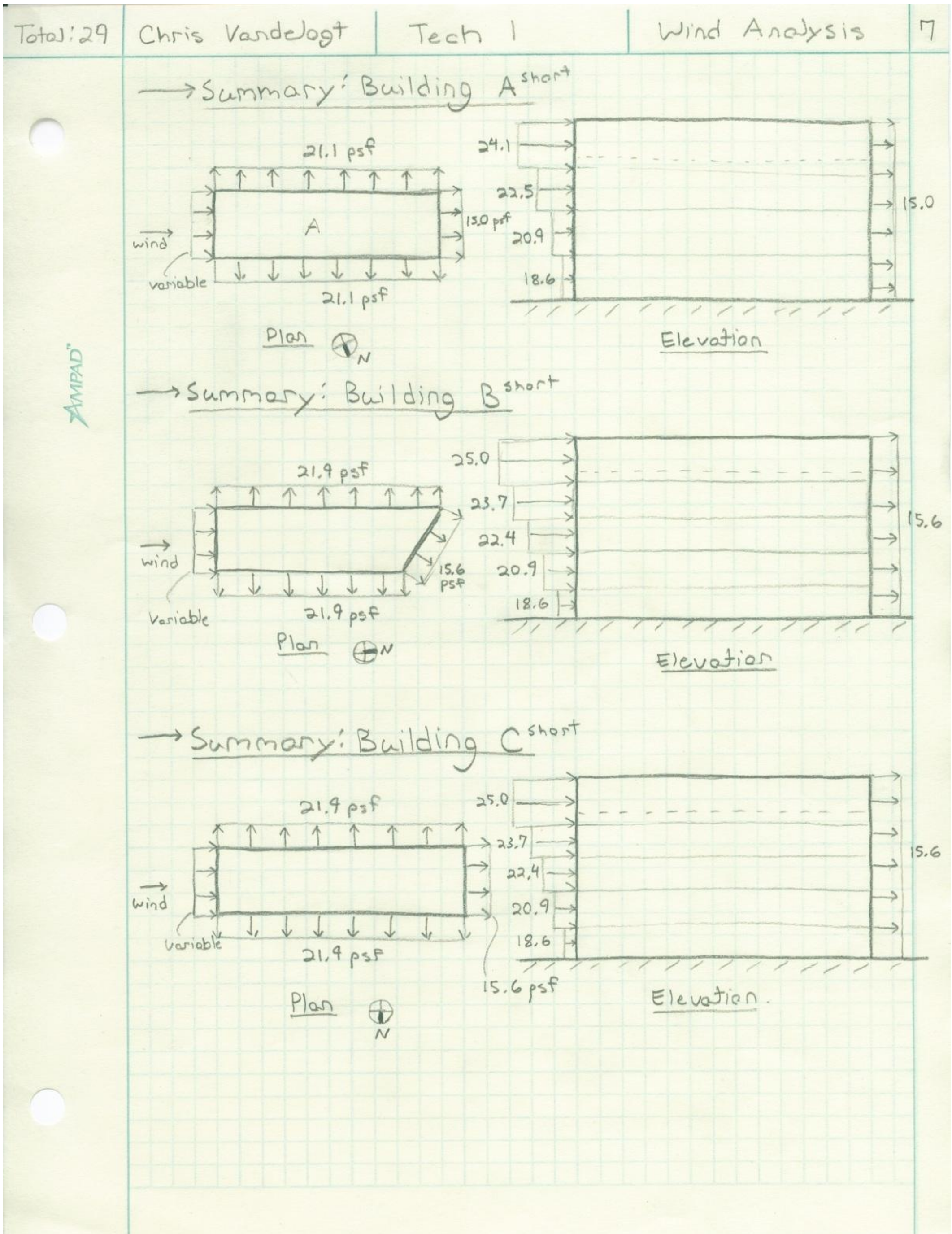


Figure 8: Summary of wind pressures normal to short dimension

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.

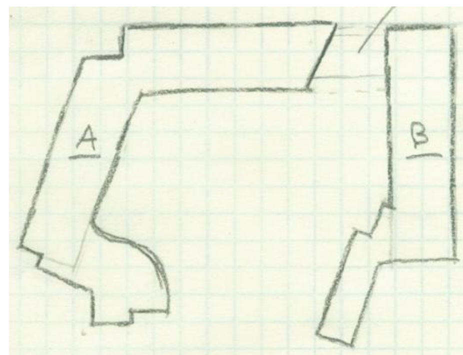


Figure 9: Simplifying building structure

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 9. 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 10 for a summary of forces on each building.

Seismic Variable	Value	Reference (ASCE 7-10)	Drawings
I_e	1.25	Table 1.5-2	-
S_S	.21	USGS Website	.21
S_1	.06	USGS Website	.06
Site Class	C	Geotechnical Report	C
Occupancy Category	III	Table 1.5-1	-
S_{DS}	.168	Table 11.6-1	.17
S_{D1}	.068	Table 11.6-2	.06
Seismic Category	B	Table 11.6-1	B
R	3.0	Table 12.2-1	5.0
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	.038	Equation 12.8-2	.038

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 (ft-k)
Ground	1833	0	0.00	0.00	0.00	371.15	0.00
2nd	1675	14	85277.05	0.08	28.42	371.15	397.86
3rd	1837	26.66	195745.76	0.18	65.23	342.73	1739.10
4th	1975	37.33	310557.05	0.28	103.49	277.49	3863.41
Pent	2003	48	419016.48	0.38	139.64	174.00	6702.60
Roof	444	62.5	103117.80	0.09	34.36	34.36	2147.75
Sum:	9767		1113714.1	1.00	371.15		
				√ ok	√ ok		
	Base Shear ($V=C_sW$) =		371.15		Total Overturning Moment=		14850.72

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 (ft-k)
Ground	2641	0	0.00	0.00	0.00	355.07	0.00
2nd	1196	14	58315.01	0.06	19.59	355.07	274.23
3rd	1195	26.66	120501.43	0.11	40.48	335.48	1079.10
4th	1071	37.33	155691.35	0.15	52.30	295.01	1952.24
Pent	2481	48	533460.01	0.50	179.19	242.71	8601.08
Roof	760	62.5	189109.52	0.18	63.52	63.52	3970.11
Sum:	9344		1057077.3	1.00	355.07		
				√ ok	√ ok		
	Base Shear ($V=C_sW$) =		355.07		Total Overturning Moment=		15876.76

Table 8: Seismic calculations

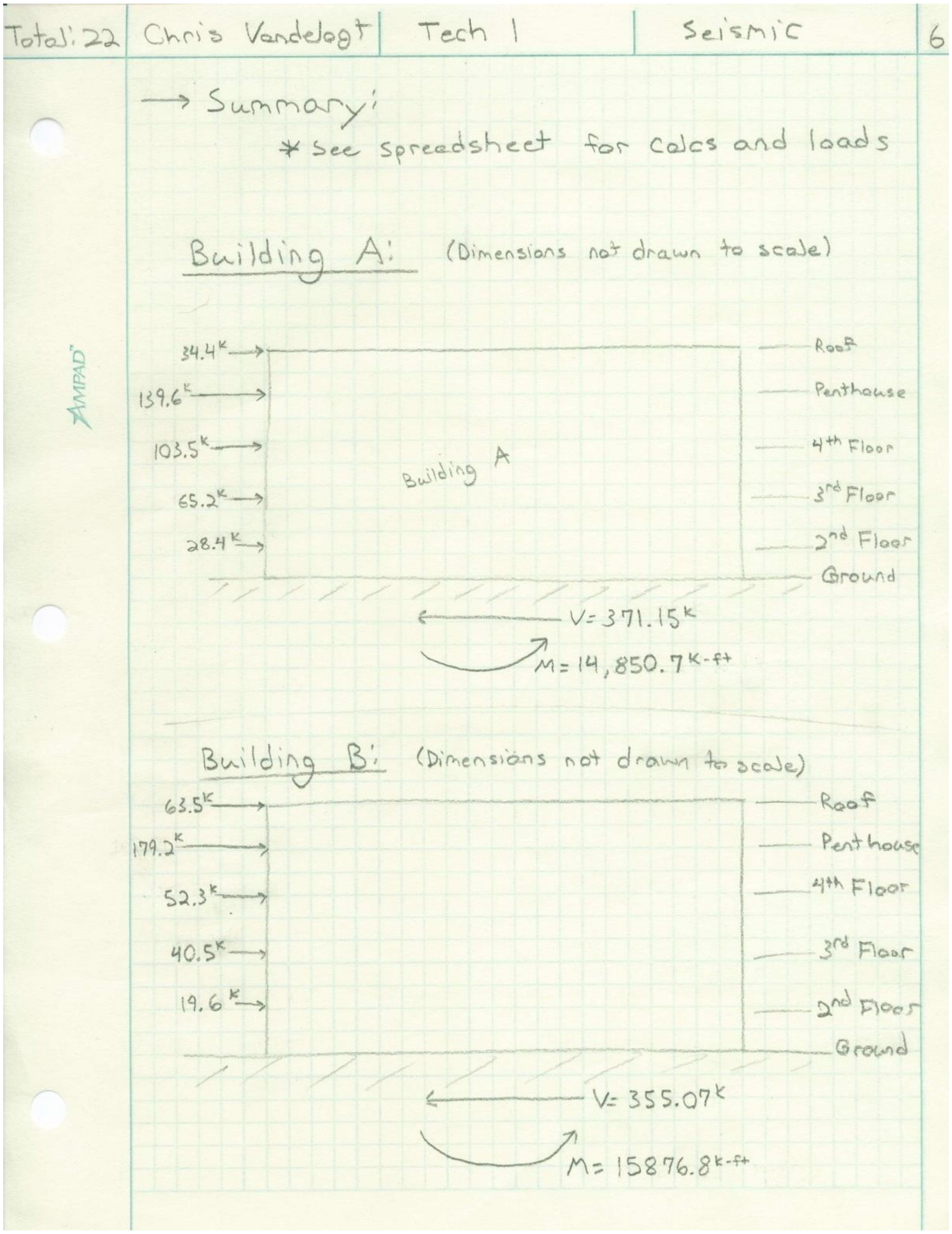


Figure 10: Summary of seismic loading

Lateral Load Summary

From the results above, Building C for wind and Building B for seismic control. Both of these sections correspond to the north leg of Global Village, see Figure 11. From here on, only this section will be examined for this report. Typical bracing details are given below and specific details for each braced frame can be found in **Appendix A**. For a summary of the wind and seismic loads acting upon this section, see Figure 12. These forces would then be input into ETABS model for a lateral system analysis.

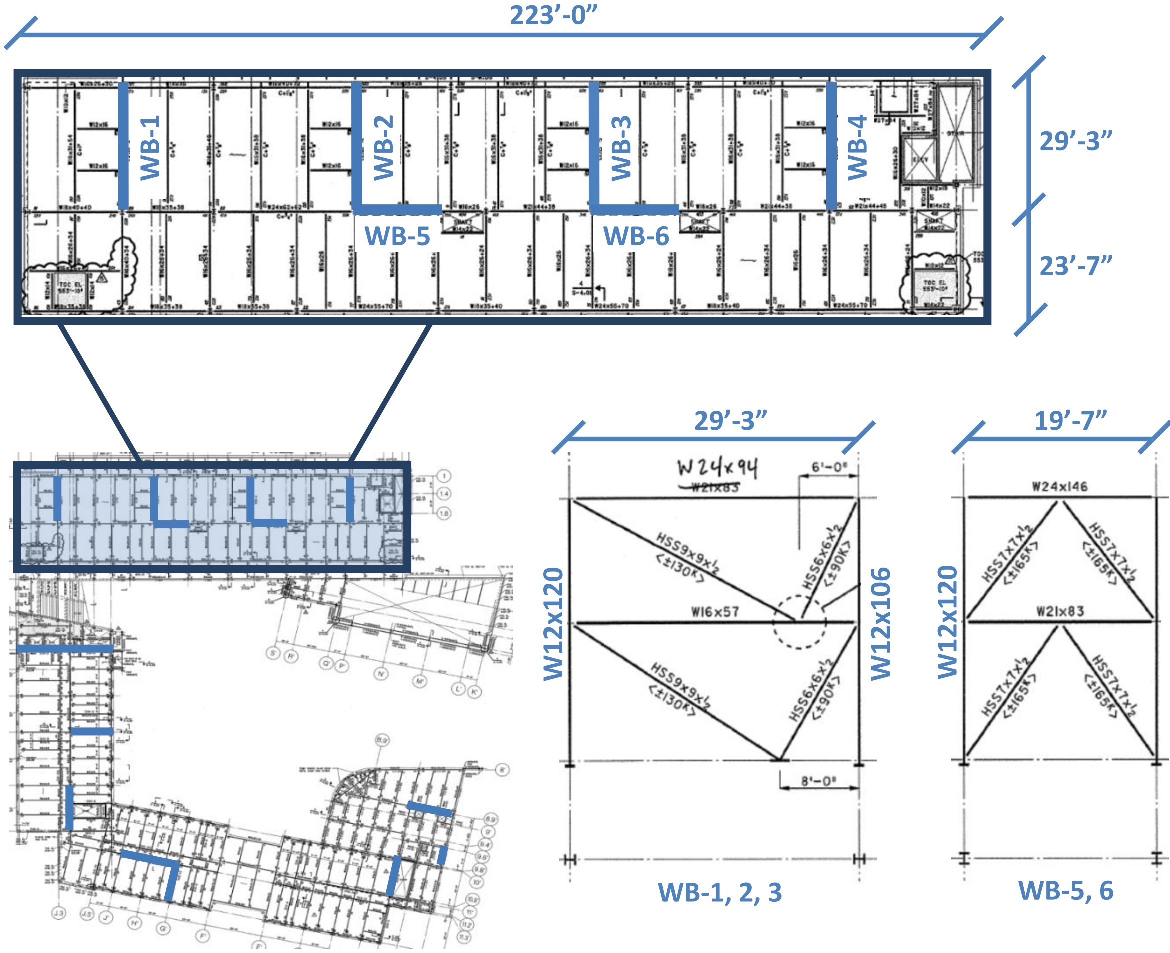


Figure 11: Typical bracing details and placement of bracing on 2nd Floor of the north leg. Courtesy of RIT. Drawings not to scale.

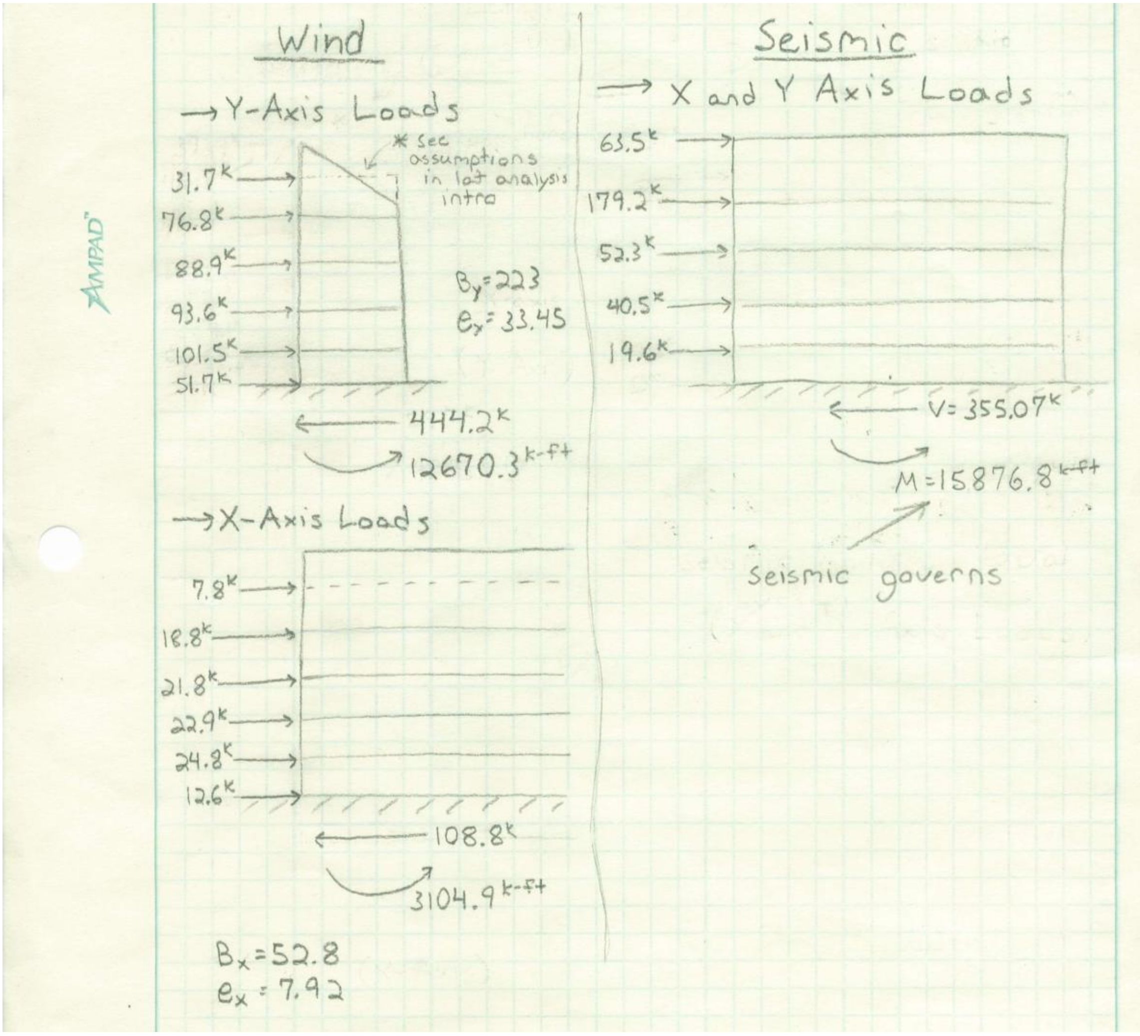


Figure 12: Summary of loads acting upon north leg of Global Village

Lateral Analysis

As explained above, only the north leg of Global Village will be examined for the remainder of this report. To analyze the lateral system of Global Village, a model was built using ETABS. Some assumptions were made during the process due to insufficient knowledge of the program or to make framing the building easier.

The geometry of the building was assumed to be a rectangular prism with dimensions: 223’-0” long by 52’-10” wide by 58’-5” high. The 11’-0” protrusions on either side of the ground floor, affecting the width of the building, were neglected. This decision was made since it would have little effect on the lateral system being that the protrusions only take place between the ground and second floor. The height of the building was also changed to a flat roof mainly because of a lacking knowledge of ETABS to make a sloped roof. A height of 58’-5” was chosen since this is the tallest point that the braced frames would reach, another assumption which is explained below, if the roof was sloped.

The building model did not take into account the 14’-0” grade level change from one side of the building to the other. Instead, the model was designed to have the same ground to roof height on each side. This decision may affect the braced frames used on the ground floor as a result of greater wind loads between the ground and second floor.

The largest assumption or change applied to the model was the use of braced frames only. Unlike the combination of braced frames and shear walls used in the existing structure, the model assumes that the braced frames extend to the roof and the shear walls are neglected. The elements between the second and third floor of the existing braced frame were replicated up to the roof to accommodate this assumption. This was again due to lack of knowledge of ETABS to make wood framed shear walls.

ETABS Model

The resulting model built using ETABS is shown in Figure 13. Using this program, relative story drifts were obtained and then compared to accepted values which will be explained later in this report. This program was also used to obtain the relative stiffness of each braced frame, member forces, and the centers of mass and rigidity. As a note, only the lateral members were modeled since gravity members do not resist lateral forces. Also, the X-Direction corresponds to the 223’-0” length and the Y-Direction corresponds to the 52’-10” width.

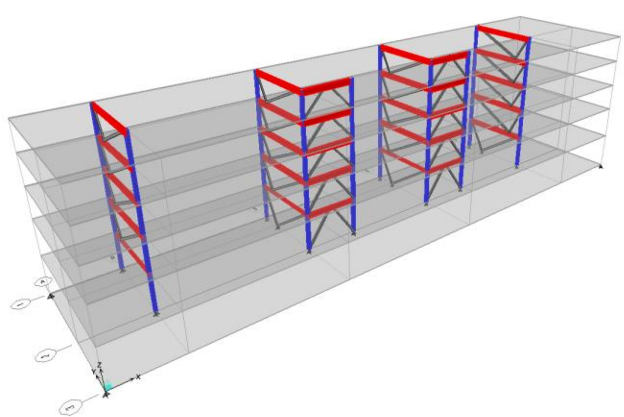


Figure 13: North leg of Global Village modeled in ETABS

X-Frame

As explained earlier in this report, the braced frames used in Global Village are assumed to extend to the roof while the shear walls used above the third floor are neglected. Based on this assumption, two braced frames are modeled in the X-Direction, see Figure 14. Both of these frames most commonly consist of HSS7x7x½ cross bracing, W24x146 beams, and W12x120 columns.

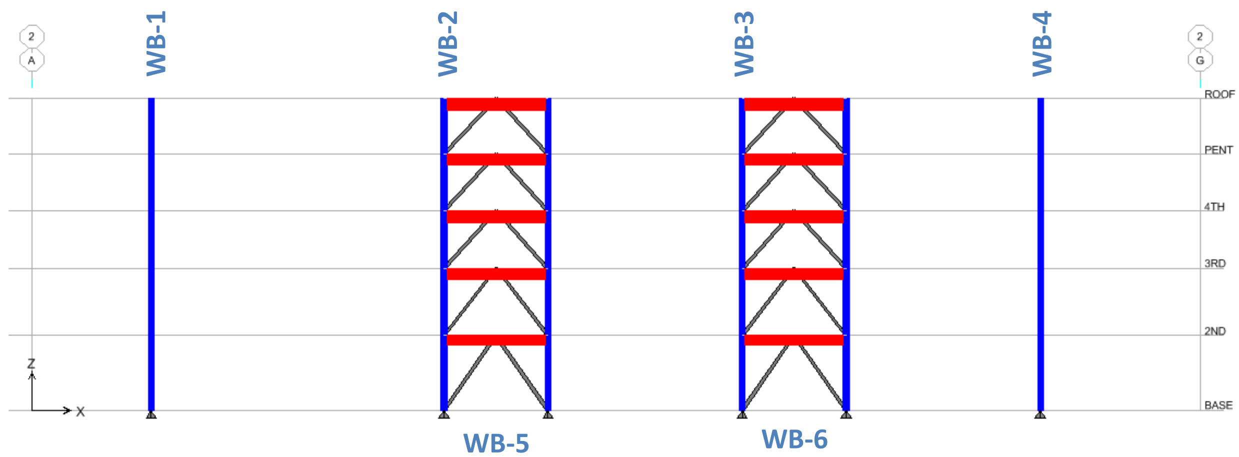


Figure 14: Elevation of braced frames in the X-Direction modeled in ETABS

Y-Frame

For the Y-Direction, four braced frames are used to resist the increased wind loads due to a larger wall tributary area, see Figure 15. All of these frames most commonly use HSS6x6x½ and HSS9x9x½ cross bracing, W24x146 beams, and W12x120 columns.

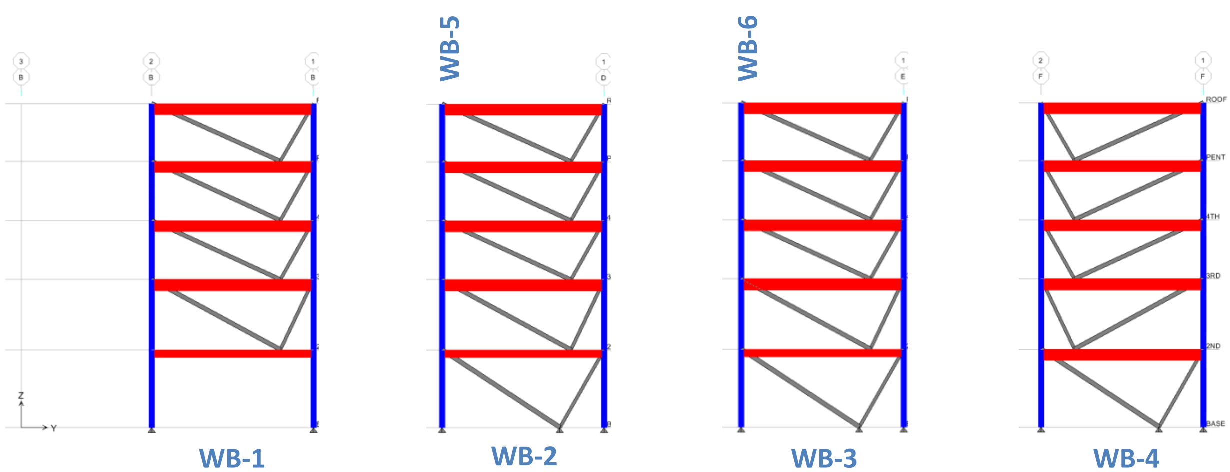


Figure 15: Elevations of braced frames in the Y-Direction modeled in ETABS

Applied Loads

After the braced frames were modeled, story diaphragms were defined and the calculated story masses were added to model the slabs. Eight different load cases were then input into ETABS, two of which are for seismic forces acting in the X and Y-Directions. The other six are for the various wind load cases described in Figure 27.4-8 of ASCE 7-10 or in Figure 16 below. For the story forces acting in the X and Y-Directions due to wind and seismic, see Figure 12 on page 23.

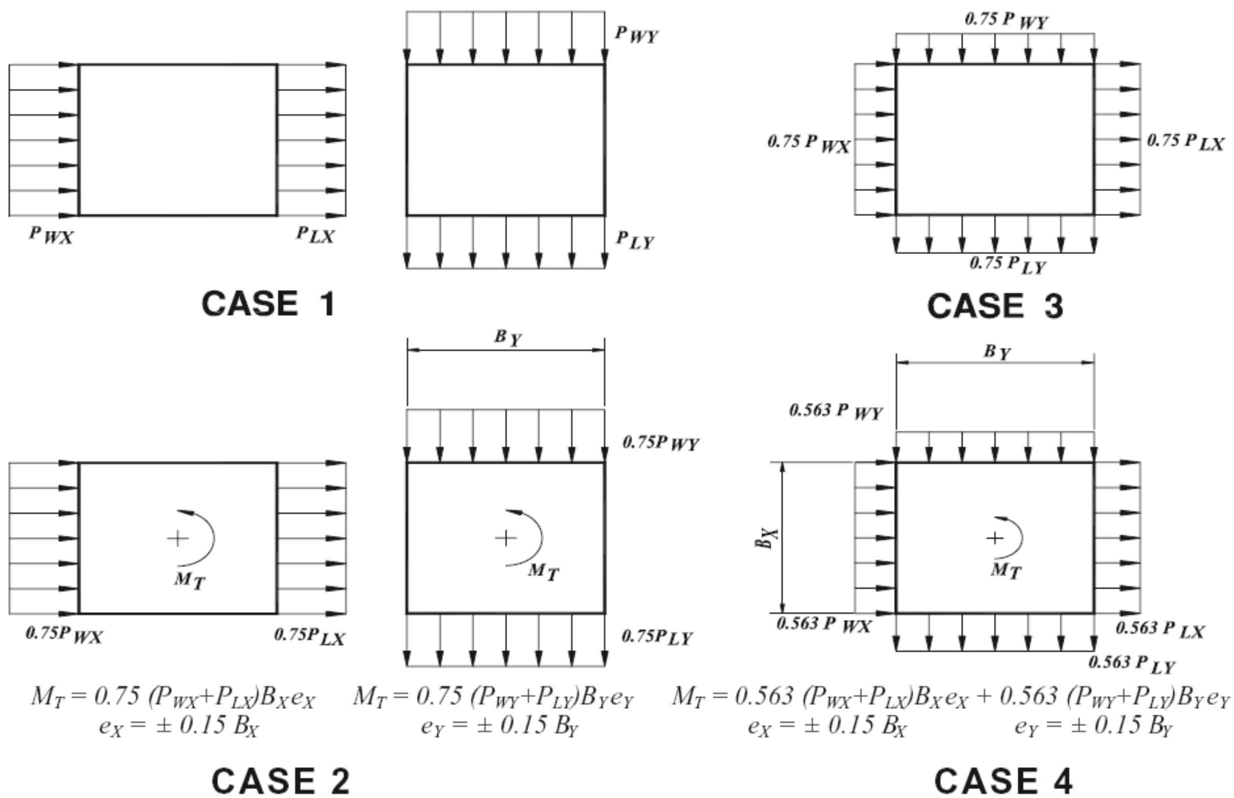


Figure 16: Wind load cases used in ETABS. Courtesy of ASCE 7-10 Figure 27.4-8.

Load Path and Distribution

As the façade collects the forces due to wind, they are transferred to the slabs of the building. The slab forces are then transferred to the braced frames that run parallel to the load. As shown in **Figure 17**, this load is then resisted by the beam and HSS cross bracing. The blue arrow represents the lateral load acting on the braced frame while the red arrows show the load within the members.

Seismic loads originate from the mass of the structure itself. These loads are created predominantly from the slabs of the structure. When seismic loads are created by ground motion, the braced frames incur the forces from the slabs and transfer them to the foundation and thus to grade.

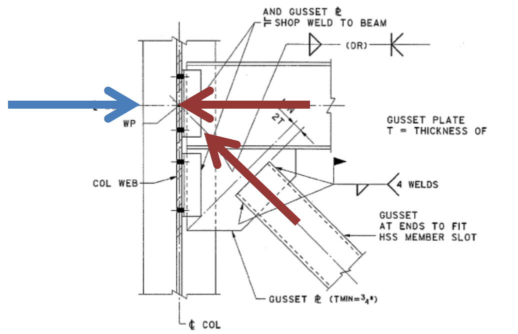


Figure 17: Lateral load path through a HSS cross braced connection. Courtesy of RIT.

Lateral Movement

Story Drift is a serviceability consideration and is defined as the displacement of one level with respect to the level below it. ETABS was used to find the maximum story drift caused by both wind and seismic forces in the X and Y-Directions. These values were then compared to allowable values outlined in ASCE 7-10. For seismic, Table 12.12-1 in ASCE 7-10 was used to find an allowable story drift of $0.015h_{sx}$. For wind, an allowance of $h_{sx}/400$ was used. As shown in **Table 9**, the maximum story drifts for both seismic and wind in the X and Y-Directions are well below the allowable values proving that this lateral system is acceptable for drift.

Story Drifts (in)						
Level	Seismic			Wind		
	$\Delta_{X\text{-}Frame}$	$\Delta_{Y\text{-}Frame}$	$\Delta_{Allowable}$	$\Delta_{X\text{-}Frame}$	$\Delta_{Y\text{-}Frame}$	$\Delta_{Allowable}$
Roof	0.109	0.059	1.873	0.026	0.027	0.312
Pent	0.156	0.095	1.921	0.029	0.038	0.320
4th	0.164	0.103	1.921	0.031	0.049	0.320
3rd	0.190	0.122	2.279	0.038	0.072	0.380
2nd	0.179	0.083	2.520	0.060	0.240	0.420
Total Drift	**0.798**	**0.463**	**10.513**	**0.184**	**0.426**	**1.752**
	√ ok	√ ok		√ ok	√ ok	

Table 9: Maximum story drifts found using ETABS

Overturning Moment

From Figure 12 in the lateral load summary section, seismic loads control the overturning moment of the building. The seismic forces result in an overturning moment, M_o, of 15,876.8$^{ft\text{-}k}$. The critical moment occurs in the direction with the least depth, corresponding to the Y-Direction of the model or the width of the building.

To resist this moment, the building weight is multiplied by the moment arm. The moment arm in this case is half the building width. The resisting moment, M_R, calculates out to 246,681.6$^{ft\text{-}k}$ which is much greater than M_o. Therefore, the building has the capacity to withstand the overturning moment due to seismic loads.

Torsion

When the center of pressure or rigidity is different than the center of mass, the building induces a torsional effect. This torsional effect is due to lateral loads being concentrated at the center of rigidity for seismic, or center of pressure for wind. The eccentricity from the center of mass is then multiplied by the lateral force which creates a moment on the building. Since the center of rigidity is further from the center of mass, torsional effects caused by seismic forces will be analyzed. ETABS was used to find the centers of mass and rigidity at each level, see Table 10.

Level	Center of Mass (in)		Center of Rigidity (in)	
	X	Y	X	Y
Roof	1315.522	318.399	1359.603	267.186
Pent	1315.583	318.036	1388.017	267.34
4th	1315.475	318.907	1428.15	268.17
3rd	1315.485	318.866	1486.811	271.848
2nd	1316.543	318.702	1594.849	287.362

Table 10: Center of mass and rigidity values found using ETABS

To calculate torsional shear on each braced frame, the stiffness of each needs to be determined. The stiffness, k, is found by using the equation:

$$k = {}^{P}\!/_{\delta}$$

ETABS was used to find the displacement, δ, that a 100^k force, P, would produce for each frame. The relative stiffness, k_{rel}, was then computed using the equation:

$$k_{rel} = {}^{k}\!/_{\sum k}$$

These calculated values are shown in **Table 11**.

Frame Stiffness									
X-Direction					Y-Direction				
Frame	Δ	k	k_{rel}	k_{dir}	Frame	Δ	k	k_{rel}	k_{dir}
WB-5	0.66	150.75	0.20	0.50	WB-1	16.12	6.20	0.01	0.01
WB-6	0.66	150.75	0.20	0.50	WB-2	0.68	146.28	0.20	0.33
	Sum:	301.50			WB-3	0.68	146.28	0.20	0.33
					WB-4	0.69	145.49	0.20	0.33
						Sum:	444.25		

Table 11: Calculated k and k_{rel} values for each braced frame

Torsional shear is calculated using the equation:

$$T = \frac{V_s \cdot e \cdot d_i \cdot R_i}{J}$$

where:

V_s = story shear
e = distance from center of mass to center of rigidity
d_i = distance from center of frame to center of rigidity
R_i = relative stiffness of frame (k_{rel})
J = torsional moment of inertia [$\sum(R_i \cdot d_i^2)$]

The torsional shear for the penthouse is shown in **Table 12**. The penthouse level was chosen because of a larger story shear. From these values, torsion doesn't affect the building significantly but should be considered for braced frames WB-2 and WB-4.

Torsional Shear - Penthouse Floor									
	Frame	e	X	Y	di	k*d²	k*d	Coeff.	Torsional Shear (k)
X-Direction	WB-1	72.4	268.0	458.5	1120.0	10432.0	9.3	0.0	0.6
	WB-2	72.4	927.5	458.5	460.6	41606.6	90.3	0.0	5.5
	WB-3	72.4	1599.5	458.5	211.4	8768.9	41.5	0.0	2.5
	WB-4	72.4	2271.4	458.5	883.4	152260.1	172.4	0.1	10.5
	WB-5	72.4	1044.7	283.0	15.7	49.6	3.2	0.0	0.2
	WB-6	72.4	1716.7	283.0	15.7	49.6	3.2	0.0	0.2
	Frame	e	X	Y	di	k*d²	k*d	Coeff.	Torsional Shear (k)
Y-Direction	WB-1	50.7	268.0	458.5	1120.0	10432.0	9.3	0.0	0.4
	WB-2	50.7	927.5	458.5	460.6	41606.6	90.3	0.0	3.9
	WB-3	50.7	1599.5	458.5	211.4	8768.9	41.5	0.0	1.8
	WB-4	50.7	2271.4	458.5	883.4	152260.1	172.4	0.0	7.3
	WB-5	50.7	1044.7	283.0	15.7	49.6	3.2	0.0	0.1
	WB-6	50.7	1716.7	283.0	15.7	49.6	3.2	0.0	0.1

Table 12: Torsional shear for the penthouse

J = 213166.713

Direct Shear

Direct shear is the force that each frame incurs as the lateral loads are applied to the building. This is calculated by multiplying the story force by the relative stiffness:

$$V_x = V_{Story} \cdot k_{dir}$$

The relative stiffness in this equation is the stiffness of the frame divided by the total stiffness acting in the direction of the force. Unlike torsion, the total stiffness is different for the X and Y-Directions. The relative stiffness, k_{dir}, of each frame can be found in **Table 11** above. Seismic loads were used to calculate direct story shears and are shown in **Table 13**. These direct shear values show how much force each frame sustains versus other frames in the same direction.

Direct Shear (k)						
Y-Direction	Level	Story Force (V)	WB-1	WB-2	WB-3	WB-4
	Roof	63.5	0.89	20.91	20.91	20.80
	Pent	179.2	2.50	59.01	59.01	58.69
	4th	52.3	0.73	17.22	17.22	17.13
	3rd	40.5	0.57	13.34	13.34	13.26
	2nd	19.6	0.27	6.45	6.45	6.42
X-Direction	Level	Story Force (V)	WB-5	WB-6		
	Roof	63.5	31.75	31.75		
	Pent	179.2	89.6	89.6		
	4th	52.3	26.15	26.15		
	3rd	40.5	20.25	20.25		
	2nd	19.6	9.8	9.8		

Table 13: Direct shear due to seismic loads

Member Strengths

Spot checks for strength were done on a lateral bracing member and a column which can be seen in **Appendix X**. ETABS was used to find the largest force on these members. It was found that the ground floor HSS9x9x½ member in frame WB-2 had the largest axial force, 195.51^k, due to Case 1 (Y-Direction) forces. Table 4-4 in the AISC Manual was used to find the capacity of an HSS9x9x½, which came out to be 365^k. ϕP_{cr} was also checked and it was determined that the member was adequate.

In frame WB-1, the W12x120 column was checked using Table 6-1 and the combined flexure and compression equation on page 6-2 in the AISC Manual. Since the outcome of the equation was lower than 1.0, the column was determined adequate.

Conclusion

Technical Report 3 analyzed the lateral system of Global Village Building 400 at RIT. Lateral loads, due to wind and seismic were calculated, and an analysis was done using these loads on a model in ETABS. Although the model assumed that the braced frames extended to the roof and the shear walls were neglected, the lateral system was proven to be adequate.

Story drift values were taken directly from ETABS and compared to allowable values outlined in ASCE 7-10. The maximum story drift that the lateral frame induced was .798" in the E-W Direction as a result of seismic loads. This is much less than the allowable 10.5". As a note, the maximum wind drift of .426" is also below the allowable 1.75" for wind loads.

The overturning moment produced by lateral loads were controlled by seismic forces. These forces create a moment of 15,876.8^{k}. To resist this moment, the self-weight of the building is multiplied by half of its width producing a moment of 246,681.6$^{ft\text{-}k}$. Therefore, the building has the capacity to withstand the overturning moment caused by seismic forces.

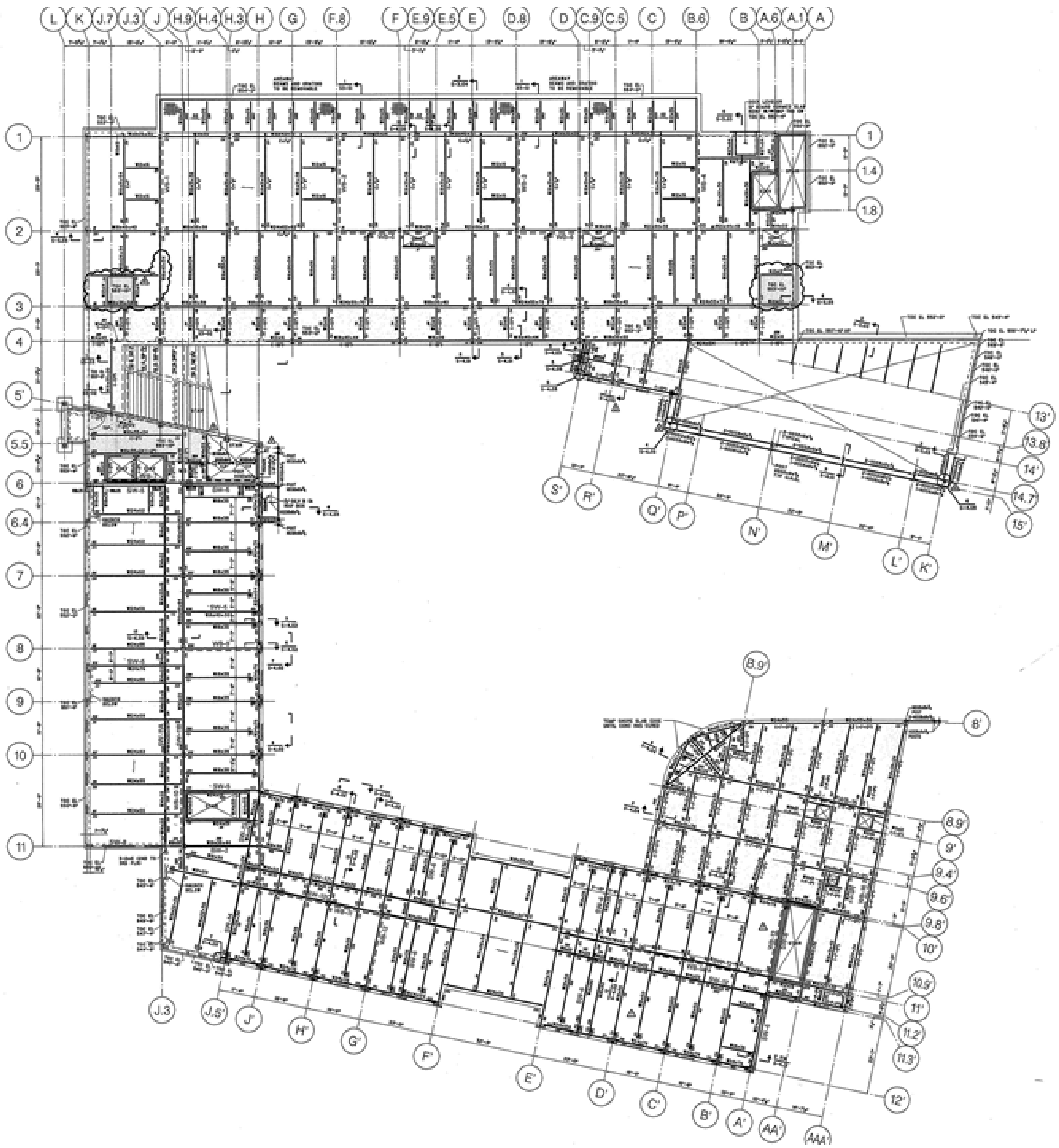
Seismic forces for torsion were chosen because of a greater eccentricity than wind. The penthouse level was then selected since it had the largest story force. The largest amount of shear added as a result of torsion was 10.5^{k} on frame WB-4 in the E-W Direction. This force may have an impact on the structure and should be considered further.

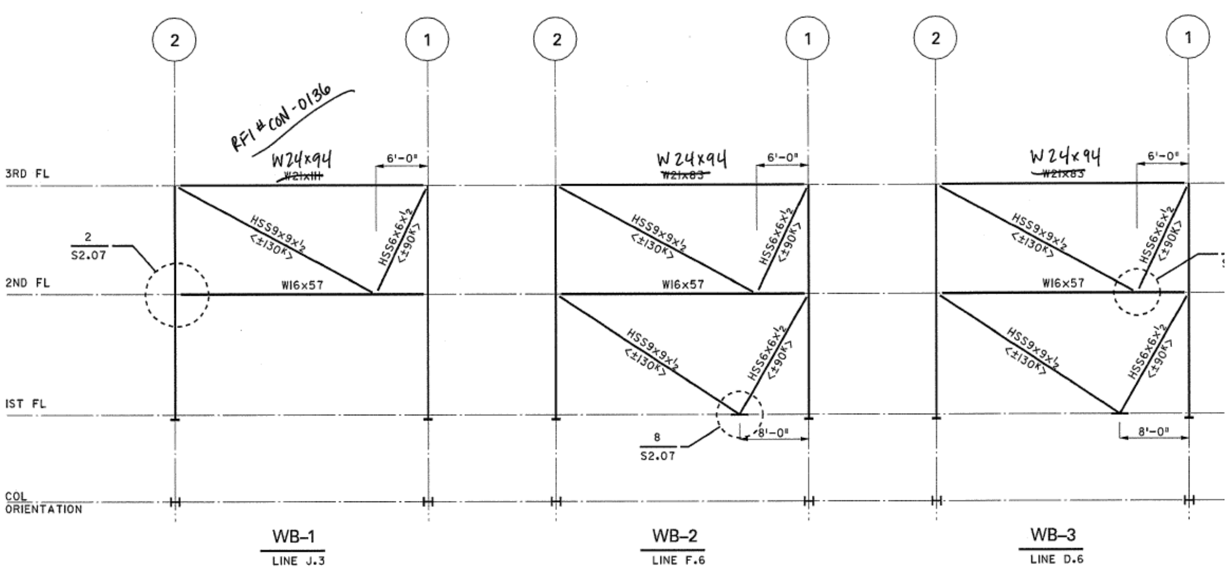
Direct shear calculations show how the loads are distributed between the frames in each direction. In the E-W Direction, WB-5 and WB-6 each receive half of the lateral force induced on them in this direction. This is because the frames are exact and thus have the same stiffness. In the N-S Direction, WB-2, 3, 4 receive 33% of the lateral load applied and WB-1 receives 1%. This is because the ground level of WB-1 doesn't have cross bracing and therefore drifts considerably when a force acts upon the frame.

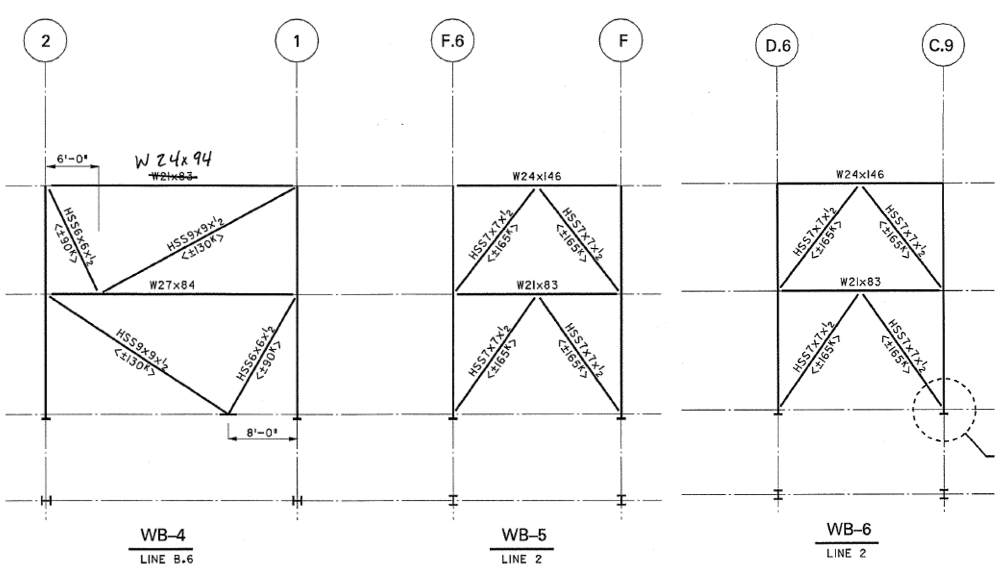
The largest axial force occurred in the HSS9x9x½ cross brace member on the ground level of frame WB-2. This axial force of 195.51^{k} was due to a Case 1 loading in the N-S Direction. By using the AISC Manual, it was determined that the member had a capacity of 365^{k} and is therefore adequate.

The assumed lateral system of Global Village performed well with the applied wind and seismic loads. Member strengths, drift values, and overturning moments were proven to be adequate. The only concern would be the effect of torsional shear on frame WB-4 and should be investigated further.

Appendix A: Typical Plans and Details




WB-1
LINE J.3
WB-2
LINE F.6
WB-3
LINE D.6
RFI # CON-0136
W24x94
3RD FL
2ND FL
IST FL
COL ORIENTATION
HSS9x9x½
HSS6x6x½
W16x57
S2.07


WB-4
LINE B.6
WB-5
LINE 2
WB-6
LINE 2
W 24x94
W27x84
W24x146
W21x83
HSS7x7x½

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:
- $S_{MS} = F_a S_s = 1.2(.21) = .252$
- $S_{M1} = F_v S_1 = 1.7(.06) = .102$

→ Design SRA: Importance Cat III
- $S_{DS} = \frac{2}{3} S_{MS} = .168$ Design Cat B
- $S_{D1} = \frac{2}{3} S_{M1} = .068$ Design Cat B

From Table 11.6-1,2

Use Seismic Design Category B

→ Response Modification Coefficient (R)
$R = 3.0$ (Steel ordinary Concentrically Braced Frame)

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

<u>Equivalent Lateral Force Procedure</u>

→ 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)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{.168}{\left(\frac{3.0}{1.25}\right)} = .07$$

controls

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

$$C_{s,max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{.068}{.7565\left(\frac{3.0}{1.25}\right)} = .038 < .067$$

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

$\therefore .00924$

$$\underline{C_s = .038}$$

→ 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

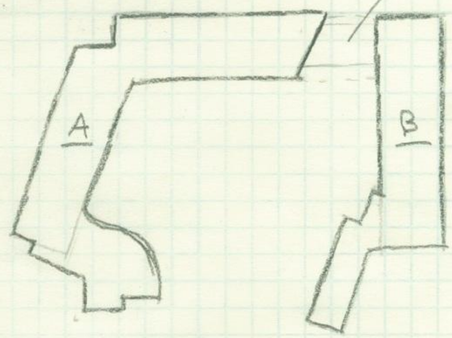
→ Roof: 66.16
- Framing: 60 psf
- Snow: since 30.8 > 30 → .2(30.8) = 6.16

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

Total: 14	Chris Vandelogt	Tech I	Seismic	3

→ Building Plan

Passage way



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

	Weight:
• Floor 1	
$A_{mech} = 10513\ ft^2 \cdot 150\ psf =$	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 Spread sheet 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	371.15	0.00
2nd	1675	14	85277.05	0.08	28.42	371.15	397.86
3rd	1837	26.66	195745.76	0.18	65.23	342.73	1739.10
4th	1975	37.33	310557.05	0.28	103.49	277.49	3863.41
Penthouse	2003	48	419016.48	0.38	139.64	174.00	6702.60
Roof	444	62.5	103117.80	0.09	34.36	34.36	2147.75
Sum:	9767		1113714.13	1.00 √ ok	371.15 √ ok		
	Base Shear ($V=C_sW$) =		371.15		**Total Overturning Moment =**		14850.72

Building B

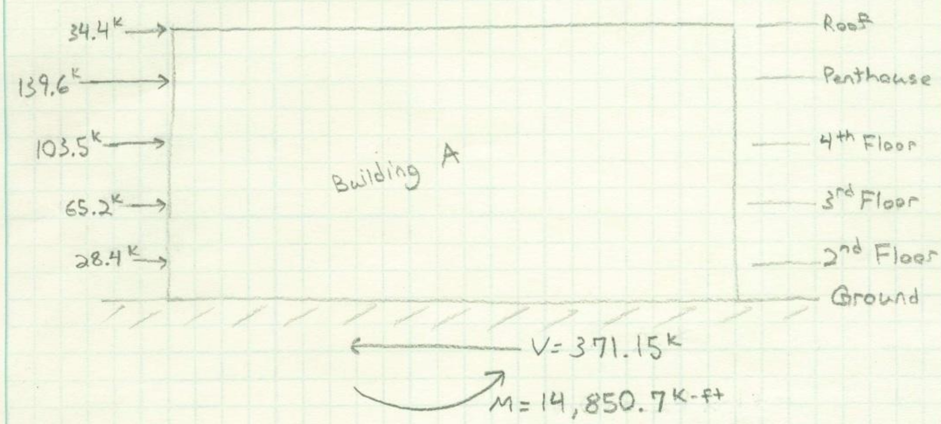
Floor	Floor Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	2641	0	0.00	0.00	0.00	355.07	0.00
2nd	1196	14	58315.01	0.06	19.59	355.07	274.23
3rd	1195	26.66	120501.43	0.11	40.48	335.48	1079.10
4th	1071	37.33	155691.35	0.15	52.30	295.01	1952.24
Penthouse	2481	48	533460.01	0.50	179.19	242.71	8601.08
Roof	760	62.5	189109.52	0.18	63.52	63.52	3970.11
Sum:	9344		1057077.32	1.00 √ ok	355.07 √ ok		
	Base Shear ($V=C_sW$) =		355.07		**Total Overturning Moment =**		15876.76

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

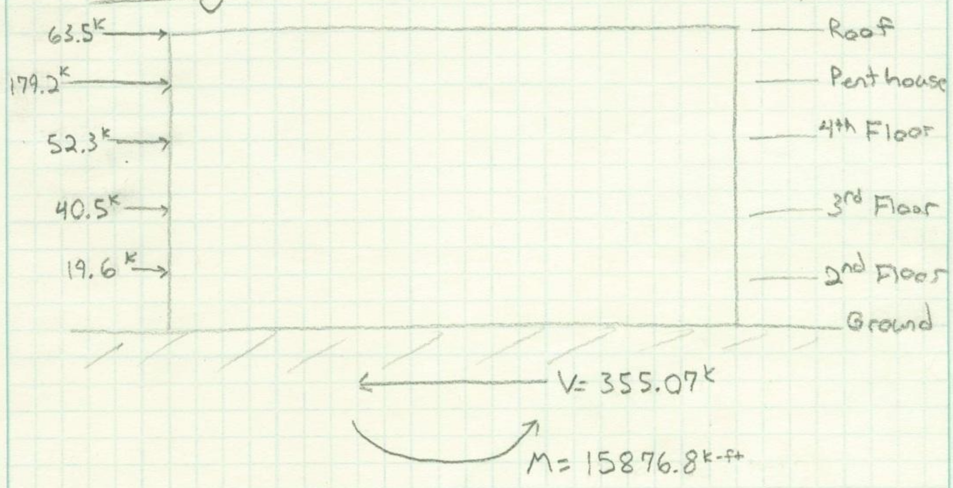
→ Summary:

* see spreadsheet for calcs and loads

Building A: (Dimensions not drawn to scale)

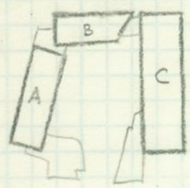


Building B: (Dimensions not drawn to scale)



Appendix C: Wind Load Calcs

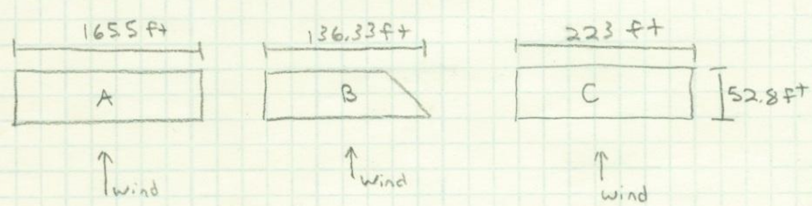
Total: 23 | Chris Vandelogt | Tech 1 | Wind Analysis | 1



- 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

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

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

$= 1.2$ or $1.5 > 1.0 \therefore$ 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 I | 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 k_z @ 37.33' = 1.024
k_z @ 26.66' = .953 k_z @ 51.83' = 1.097
k_z @ 48' = 1.08 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

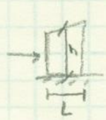
$p = qGC_p$

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 $h/L \geq 1.0$ $C_p = \begin{cases} 0 \text{ to } h/2: & -1.3, -.18 \\ > h/2: & -.7, -.18 \end{cases}$ worst cases

$\theta < 10°$

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

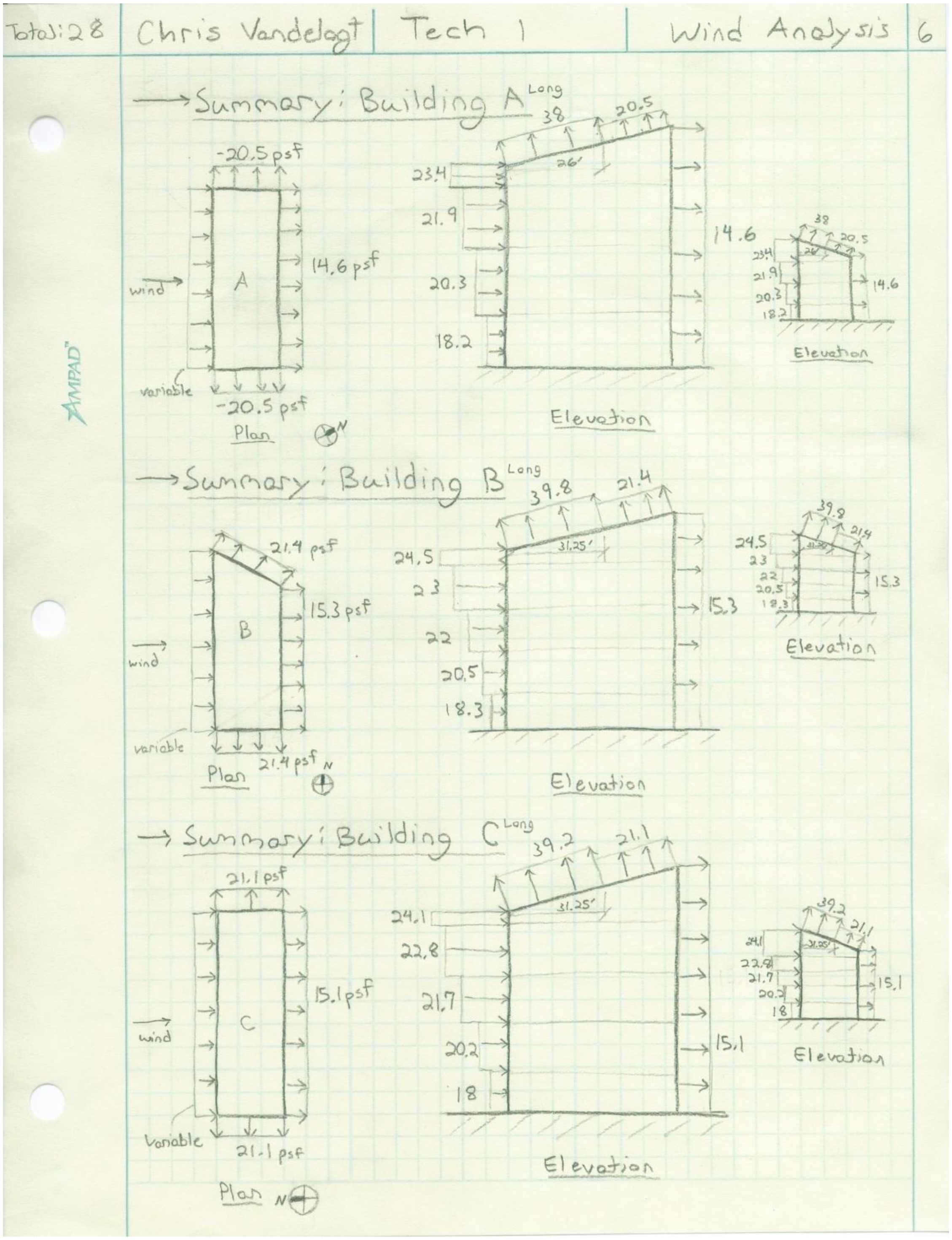
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.639	-15.034	-21.048
3rd	26.660	0.953	29.862	20.897	-15.034	-21.048
Penthouse	37.330	1.024	32.086	22.454	-15.034	-21.048
Roof	51.830	1.097	34.374	24.055	-15.034	-21.048

Building B

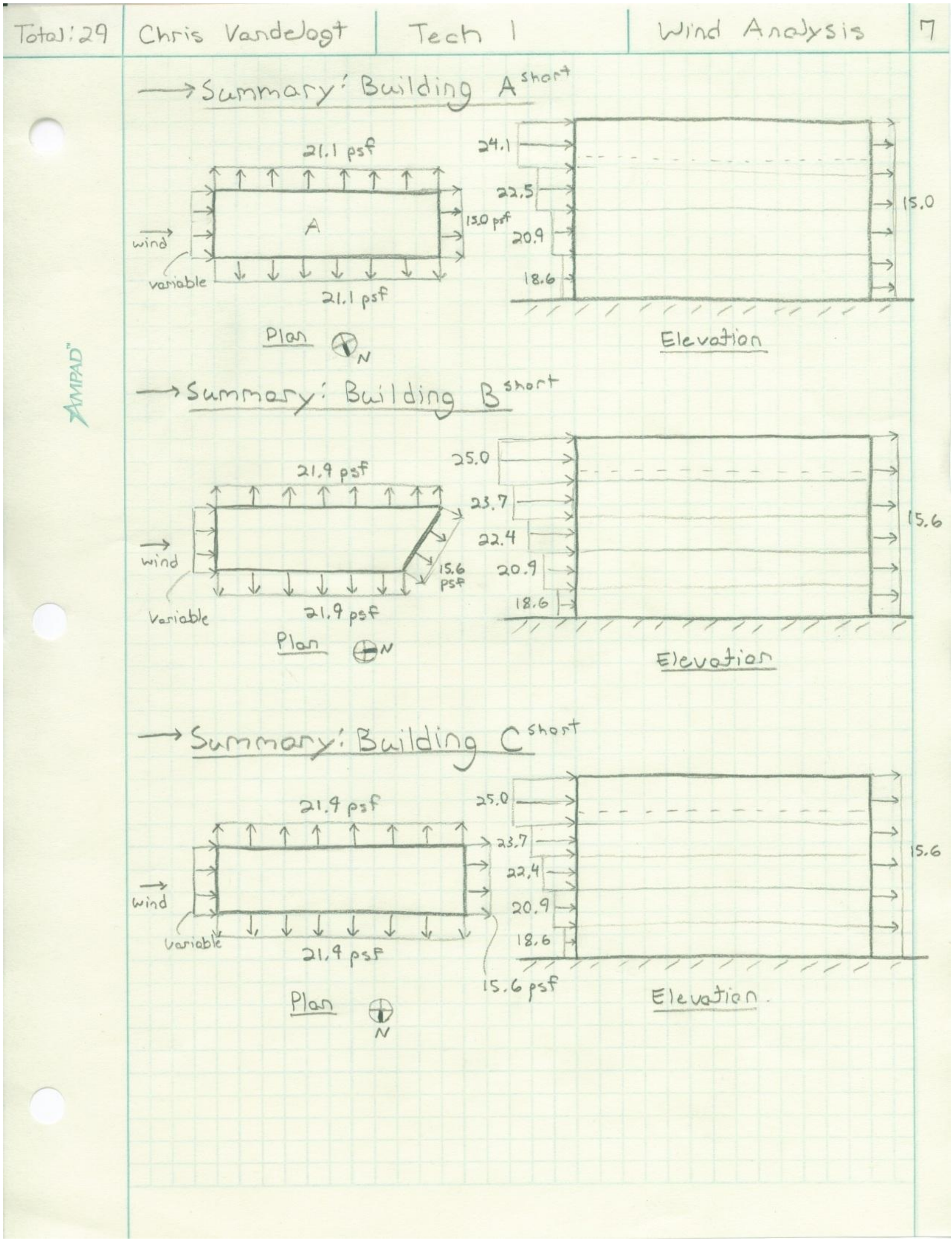
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721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

Building C

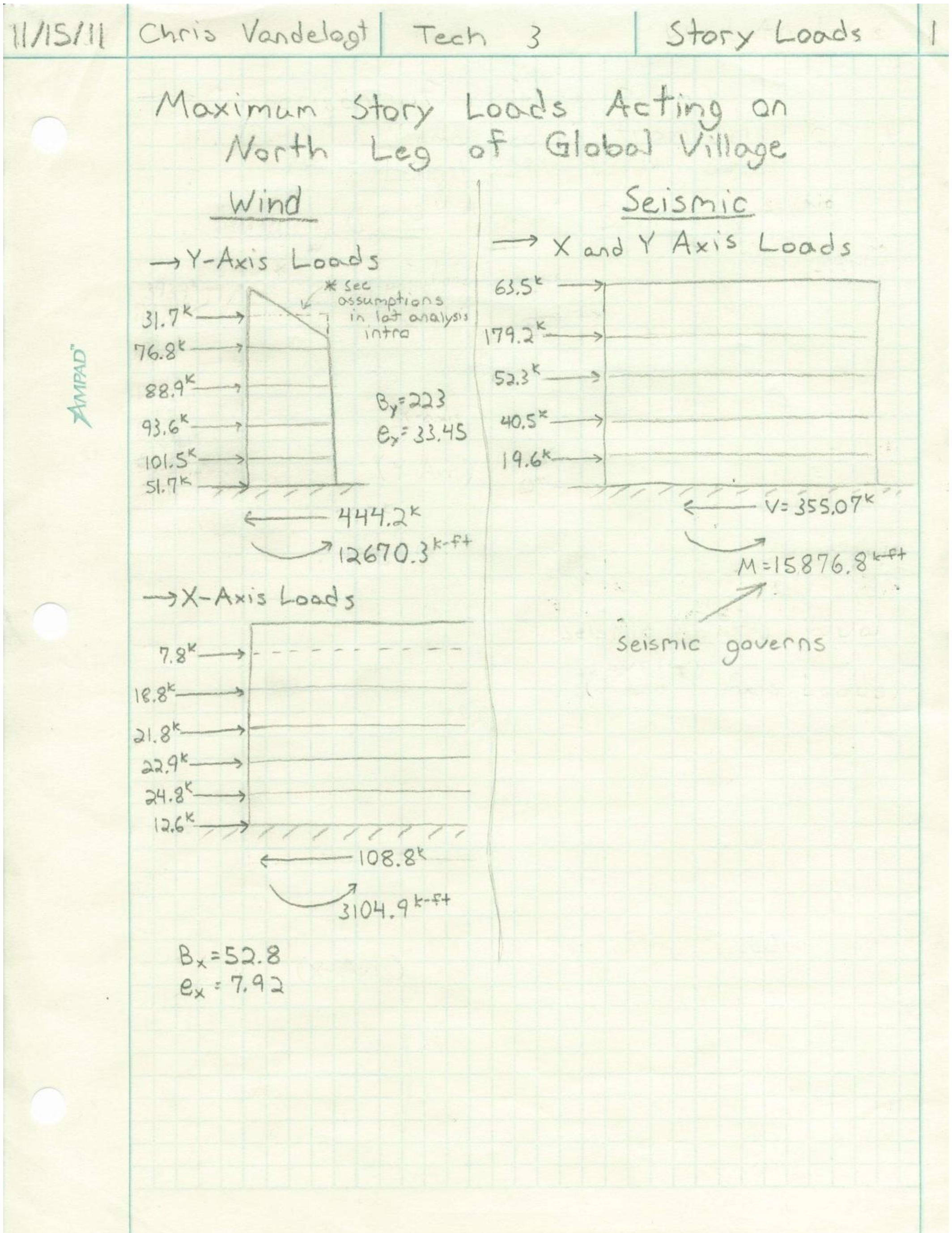
Floor	Height	k_z	q_z (lb/ft^2)	p_{wind} (lb/ft^2)	p_{lee} (lb/ft^2)	p_{side} (lb/ft^2)
2nd	14.000	0.850	26.634	18.620	-15.608	-21.851
3rd	26.660	0.953	29.862	20.876	-15.608	-21.851
4th	37.330	1.024	32.086	22.431	-15.608	-21.851
Penthouse	48.000	1.080	33.841	23.658	-15.608	-21.851
Roof	62.500	1.140	35.721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



Total: 28
Chris Vandelogt
Tech 1
Wind Analysis
6
Summary: Building A Long
-20.5 psf
14.6 psf
wind
A
variable
-20.5 psf
Plan
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
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
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

Appendix D: Story Loads



11/15/11 | Chris Vandelogt | Tech 3 | Story Loads | 1

Maximum Story Loads Acting on North Leg of Global Village

Wind

→ Y-Axis Loads

31.7k, 76.8k, 88.9k, 93.6k, 101.5k, 51.7k

* see assumptions in lat analysis intro

$B_y = 223$
$e_x = 33.45$

444.2k
12670.3 k-ft

→ X-Axis Loads

7.8k, 18.8k, 21.8k, 22.9k, 24.8k, 12.6k

108.8k
3104.9 k-ft

$B_x = 52.8$
$e_x = 7.92$

Seismic

→ X and Y Axis Loads

63.5k, 179.2k, 52.3k, 40.5k, 19.6k

V = 355.07k
M = 15876.8 k-ft

Seismic governs

Appendix E: ETABS Model Calcs

11/15/11	Chris Vandelogt	Tech 3	Lateral Model	1

→ Mass of Each Floor $\left(\frac{\text{psf of floor}}{1000 \cdot 32.2}\right)$

- Floor 1 = .0038
- Floor 2 = .003
- Floor 3 = .003
- Floor 4 = .0027
- Penthouse = .0067
- Roof = .0021

11/15/11 Chris Vandelogt Tech 3 Story Drift 1

Story Drifts

Level	Seismic			Wind		
	EQX	EQY	Max	WX	WY	Max
Roof	0.109	0.059	1.873	0.026	0.027	0.312
Pent	0.156	0.095	1.921	0.029	0.038	0.320
4th	0.164	0.103	1.921	0.031	0.049	0.320
3rd	0.190	0.122	2.279	0.038	0.072	0.380
2nd	0.179	0.083	2.520	0.060	0.240	0.420
Σ	**0.798**	**0.463**	**10.513**	**0.184**	**0.426**	**1.752**

11/15/11 | Chris Vandelogt | Tech 3 | Overturning Moment | 1

→ Resisting Moment (M_R)

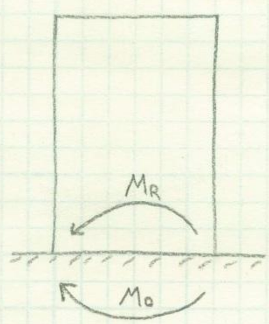
M_R = Building Weight × Moment Arm

↳ $\frac{\text{Building Width}}{2}$

$M_R = 9344^k \left(\frac{52.8'}{2}\right) = 246,681.6'^k$

* From Comparison page, seismic governs in which the moment, M_o, produced is $15,876.8'^k$

$M_R >> M_o \therefore \checkmark ok$



11/15/11	Chris Vandelogt	Tech 3	Torsion/Direct Shear	1

→ Penthouse Floor

	x (in)	y (in)
Center of Mass	1315.6	318.04
Center of Rigidity	1388.02	267.34

→ Direct Shear

X-Dir: $F_{ix} = \frac{k_{ix}}{\Sigma k_{ix}} V_x$ where V is the story Force

Y-Dir: $F_{iy} = \frac{k_{iy}}{\Sigma k_{iy}} V_y$ See spreadsheet for calculations

Example: WB-5 (X-Dir) at Penthouse

$$F = \frac{150.75}{301.5}(179.2) = 89.6^k \leftarrow \tfrac{1}{2} \text{ of } V$$

*This makes sense since WB-5 and WB-6 are framed the same

→ Torsional Shear

$$T_{ix} = \frac{V \cdot e \cdot d_i \cdot R_i}{J}$$

where V = story shear

e = distance from center of mass to center of rigidity

d_i = distance from center of frame to center of rigidity

J = torsional moment of inertia ($\Sigma R_i d_i^2$)

See Spreadsheet for Calculations

11/15/11 Chris Vandelogt Tech 3 COM, COR, K 1

Level	Center of Mass (in)		Center of Rigidity (in)	
	X	Y	X	Y
Roof	1315.522	318.399	1359.603	267.186
Pent	1315.583	318.036	1388.017	267.34
4th	1315.475	318.907	1428.15	268.17
3rd	1315.485	318.866	1486.811	271.848
2nd	1316.543	318.702	1594.849	287.362

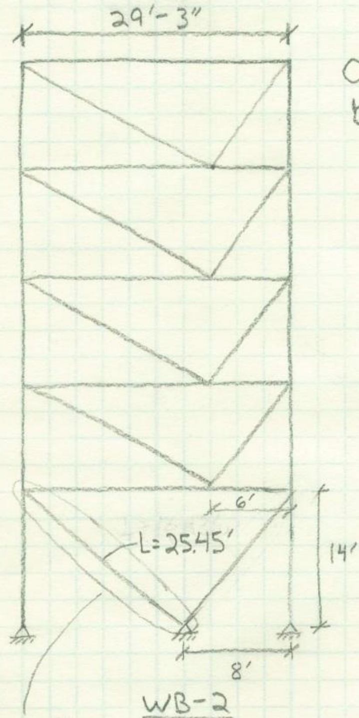
Frame Stiffness									
X-Direction					Y-Direction				
Frame	Δ	k	krel	kdir	Frame	Δ	k	krel	kdir
WB-5	0.66	150.75	0.20	0.50	WB-1	16.12	6.20	0.01	0.01
WB-6	0.66	150.75	0.20	0.50	WB-2	0.68	146.28	0.20	0.33
	Sum:	301.50			WB-3	0.68	146.28	0.20	0.33
					WB-4	0.69	145.49	0.20	0.33
						Sum:	444.25		

11/15/11 Chris Vandelogt Tech 3 Torsion/Direct Shear k

	Torsional Shear - Penthouse Floor								
	Frame	e	X	Y	di	k*d^2	k*d	Coeff.	Torsional Shear (k)
X-Direction	WB-1	72.4	268.0	458.5	1120.0	10432.0	9.3	0.0	0.6
	WB-2	72.4	927.5	458.5	460.6	41606.6	90.3	0.0	5.5
	WB-3	72.4	1599.5	458.5	211.4	8768.9	41.5	0.0	2.5
	WB-4	72.4	2271.4	458.5	883.4	152260.1	172.4	0.1	10.5
	WB-5	72.4	1044.7	283.0	15.7	49.6	3.2	0.0	0.2
	WB-6	72.4	1716.7	283.0	15.7	49.6	3.2	0.0	0.2
	Frame	e	X	Y	di	k*d^2	k*d	Coeff.	Torsional Shear (k)
Y-Direction	WB-1	50.7	268.0	458.5	1120.0	10432.0	9.3	0.0	0.4
	WB-2	50.7	927.5	458.5	460.6	41606.6	90.3	0.0	3.9
	WB-3	50.7	1599.5	458.5	211.4	8768.9	41.5	0.0	1.8
	WB-4	50.7	2271.4	458.5	883.4	152260.1	172.4	0.0	7.3
	WB-5	50.7	1044.7	283.0	15.7	49.6	3.2	0.0	0.1
	WB-6	50.7	1716.7	283.0	15.7	49.6	3.2	0.0	0.1
					J =	213166.713			

	Direct Shear (k)					
	Level	V Story	WB-1	WB-2	WB-3	WB-4
Y-Direction	Roof	63.5	0.89	20.91	20.91	20.80
	Pent	179.2	2.50	59.01	59.01	58.69
	4th	52.3	0.73	17.22	17.22	17.13
	3rd	40.5	0.57	13.34	13.34	13.26
	2nd	19.6	0.27	6.45	6.45	6.42
	Level	V Story	WB-5	WB-6		
X-Direction	Roof	63.5	31.75	31.75		
	Pent	179.2	89.6	89.6		
	4th	52.3	26.15	26.15		
	3rd	40.5	20.25	20.25		
	2nd	19.6	9.8	9.8		

11/15/11 | Chris Vandelogt | Tech 3 | Member Checks | 1



HSS 9x9x½
$A_g = 15.3\,in^2$
$I_x = 183\,in^4$

Check circled HSS cross bracing member (HSS9x9x½)

→ From ETABS output of axial force (worst case: Case 1 Y-Dir)

$P_u = 195.51^k$

→ Using AISC Table 4-4 (Using L=26')

$\Phi P_n = 365^k > 195.51^k$

✓ok

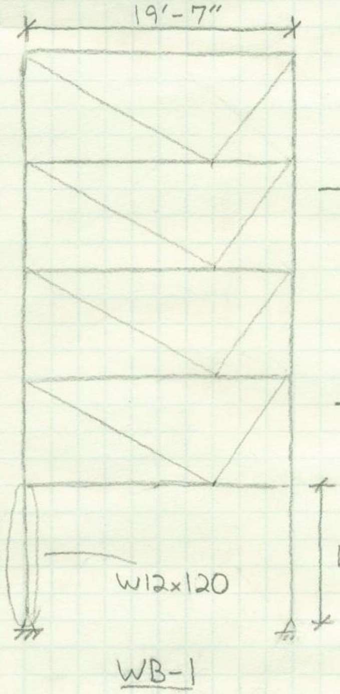
$$P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2(29000)(183)}{(25.45 \times 12)^2} = 561.6^k$$

$$\Phi P_n = .9(561.6) = 505.4^k > 195.51^k$$

✓ok

$\boxed{\text{Member is Adequate}}$

11/15/11 | Chris Vandelogt | Tech 3 | Member Checks | 2



Check circled column
(W12x120)

→ From ETABS output
Worst case: Case 1 Y-Dir

$P_u = 64.27^k$

$M_u = 84.46$

→ Using AISC Table 6-1

$p = .779$

$b_x = 1.31$

$pP_r = \left(\frac{.779}{1000}\right)(64.27)$

$= .0501 < .2$

$\frac{1}{2}pP_r + \frac{9}{8}(b_x M_{rx}) \leq 1.0$

$\frac{1}{2}(.0501) + \frac{9}{8}\left(\frac{1.31}{1000}\right)(84.46) \leq 1.0$

$.02505 + .1245 \leq 1.0$

$.14955 \leq 1.0$ ✓ ok

$\boxed{\text{Member is Adequate}}$