Angela Mincemoyer
Structural Option
Advisor: Dr. Boothby
Peggy Ryan Williams Center
Ithaca, New York
20 November 2013

Peggy Ryan Williams Center

Technical Report 4

Angela Mincemoyer
Structural Option
November 20, 2013

Dr. Boothby
Advisor
Penn State University

Dear Dr. Boothby,

The following Technical Report 4 was prepared for AE 481W. The purpose of the report was to determine if the building's lateral system is adequate for the wind and seismic loads according to industry standard serviceability and strength considerations. In order to answer this question, a 3D structural computer model was constructed using ETABS. Once the model was verified, it was used to determine the member forces and drifts of various wind and earthquake load cases. The results were then interpreted to determine if the lateral system was in fact adequate.

The contents of this report include the wind and seismic loads that were input into ETABS, a 3D view of the ETABS model, member forces determined from the model, determination of worst load case, determination of the controlling load combination, and strength and serviceability checks of the braced frames. Various calculations performed in excel detail all of the necessary calculations. It is important to note that the following calculations are based on the gravity, wind, and seismic loads which may be seen in the appendices.

Thank you in advance for taking the time to review the following report.

Sincerely,

Angela Mincemoyer

Enclosed: Technical Report 4

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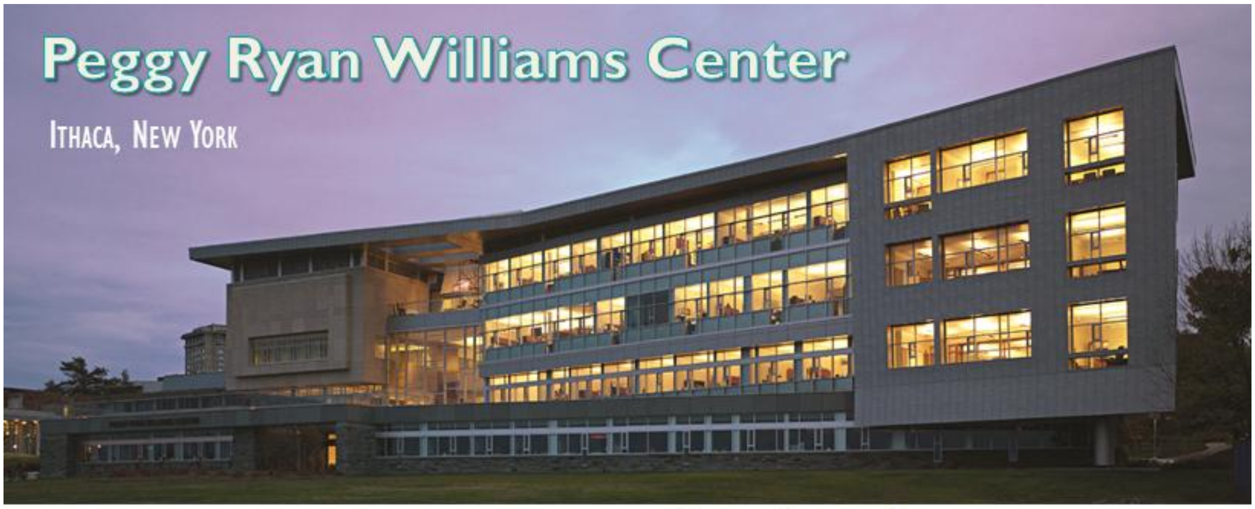


Primary Project Team:

Owner | Ithaca College
Architect | Holt Architects
Structural Engineer | Ryan-Biggs Associates
Mechanical & Electrical Engineer | Delta Engineers
General Contractor | Christa Construction

General Building Data:

Building Occupant | Ithaca College
Occupancy | Office Use
Size | 58,200 gross square feet
Stories | 4 stories above grade
Substantial Completion | March 2010
Cost of Construction | approx. $19.3 million
Project Delivery Method | Design-Bid-Build

Architecture:

- Various aspects were driven by desire to be eco-friendly
- Large areas of glass provide views of Cayuga Lake
- Façade consists of zinc panels, blue stone veneer, composite aluminum panels, and limestone panels
- Pedestrian bridge connects PRWC to adjacent building

Sustainability:

- Awarded LEED Platinum
- "V" shaped roof aids in rain water collection
- Day lighting made possible by large areas of glass
- Intensive Green Roof
- Atrium promotes natural ventilation

Structure:

- *Foundation*
  - Slab-on-grade, foundation walls, footings, various grade beams, piers and drilled piers
- *Framing System*
  - All floors are composed of composite steel decking
  - Steel framing consists of wide flange beams, girders, and columns
- *Lateral System*
  - Concentrically braced structural steel frames in both the North-South and East-West directions

MEP:

- *Mechanical*
  - Main heating and cooling source is geothermal via a closed loop system adjacent to the building
  - Two dedicated outdoor air units (DOA) will utilize water to water heat pumps
- *Electrical*
  - Primary Service: 12.5 KV primary fused switches, 500 KVA transformer, 480/277 Volt Distribution Switchboard
  - Secondary Distribution: 150 KVA, 480V to 120/208 Volt transformer and (1) 120/208 Volt Main power panel
- *Plumbing*
  - Collect and store rainwater for gray water use
  - (3) rainwater collections tanks

Angela Mincemoyer | Structural | http://www.engr.psu.edu/ae/thesis/portfolios/2014/ahm5066/index.html

Executive Summary

The Peggy Ryan Williams Center, formerly known as "The Gateway Building," is a four story office building located on the Ithaca College campus, Ithaca, New York. The building was originally known as "The Gateway Building" because the college saw the building as a gateway to the campus. At the time, the college was moving into a new era of sustainability and they wanted to show their prospective students, employees, and visitors the strides that they were making towards their goal.

Sustainability and a desire to connect with nature were both driving forces for the building's architectural features. The large areas of glass, offering vistas to Cayuga Lake, allow the occupants to feel like they are part of the nature around them. Other eco-friendly architectural features include the "V" shaped roof which aids in rainwater collection, and the large atrium which extends through the building to promote natural ventilation.

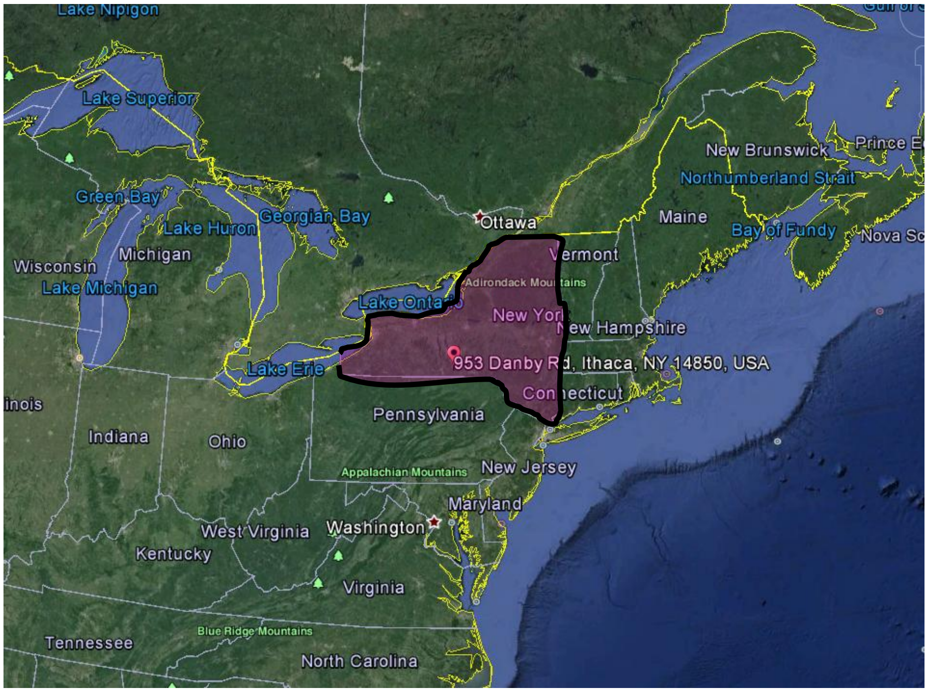
The structural system components are fairly common; however, their placement and size variations make the framing very irregular. The roof of the building is constructed of roof decking, which spans perpendicular to the beams, girders, and columns. The floor of Level 1 through Level 3 consists of composite decking and wide flanged beams, girders, and columns. Various beams and girders are provided with shear studs for composite action. Sizes and spans of the wide flanges vary greatly throughout the building and even throughout a single floor framing system. At locations where the building cantilevers, moment connections and larger beam/girder sizes make the cantilevers possible.

Columns, piers, and drilled piers support the foundation for the PRWC. The drilled piers range from resting on top of bedrock, to being drilled down 4'-0" below competent bedrock, depending on their location and loading.

Another distinctive feature of the Peggy Ryan Williams Center is the pedestrian bridge, which connects the building to the adjacent Dillingham Center. The bridge is a box truss supported in a double cantilever configuration with a 2" expansion joint on either end. I am eager to explore ways to improve the existing design for the bridge.

Due to its location, the PRWC was designed following the 2002 Building Code of New York State (BCNYS) which adopted the 2000 International Building Code (IBC). In addition to the BCNYS, additional loading and design requirements from American Society of Civil Engineering (ASCE) 7-98 are incorporated by reference into the IBC. In addition, various other codes were used in the design and are discussed in further detail in the following report.

Site Plan and Location Plan



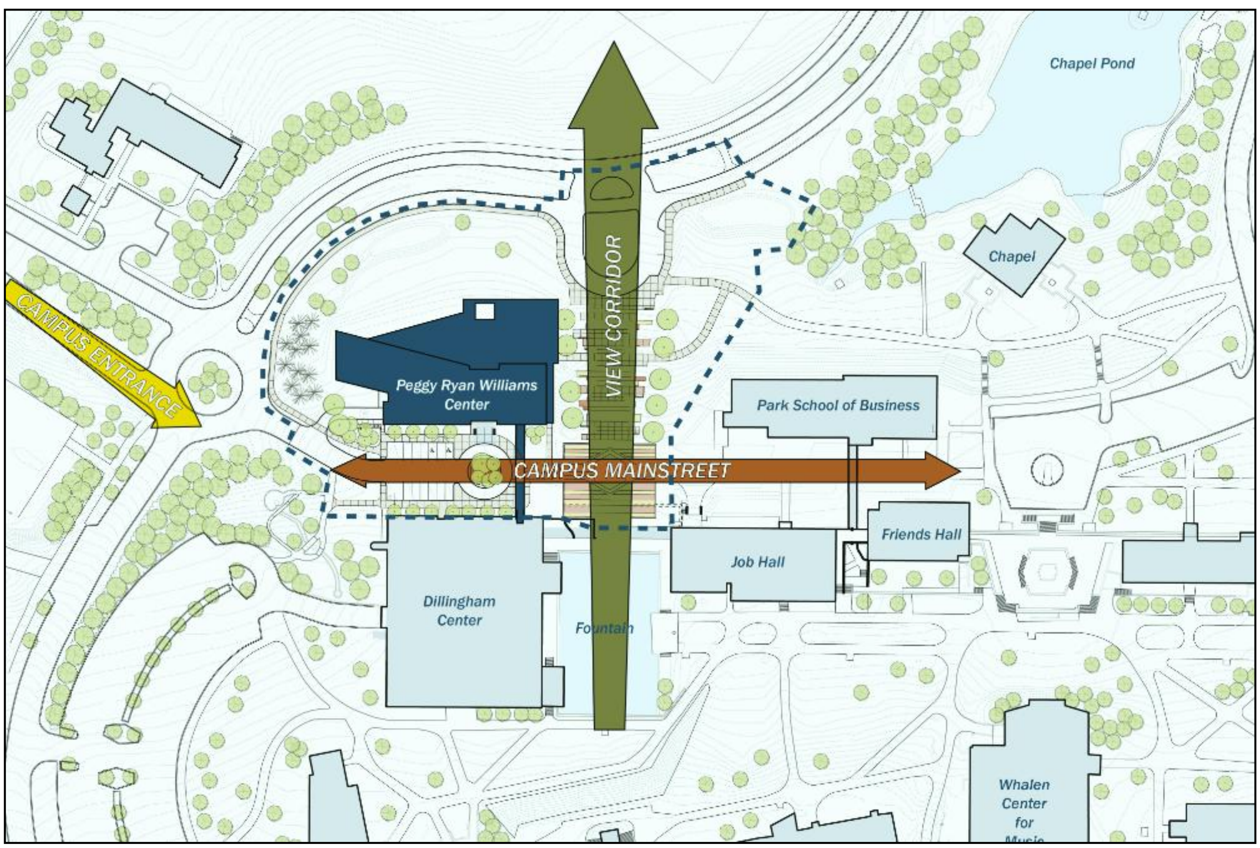


Photo provided courtesy of Holt Architects

Documents Used in Preparation of this Report

- Building Code of New York State
  - 2002 BCNYS (IBC 2000 adopted)

- International Building Code
  - IBC 2009

- American Society of Civil Engineers
  - ASCE 7-98: Minimum Design Loads for Buildings and Other Structures

- Vulcraft Deck Catalog

- American Concrete Institute
  - ACI 318-11

- American Institute of Steel Construction
  - AISC 14^{th} edition

- American Wood Council
  - National Design Specification (NDS): Design Values for Wood Construction

- Boise Cascade
  - Engineered Wood Products: Boise Glulam Beam and Column Specifier Guide

- Reed Construction Data
  - RS Means: Square Foot Cost 2013
  - RS Means: Facilities, Maintenance, and Repair 2013

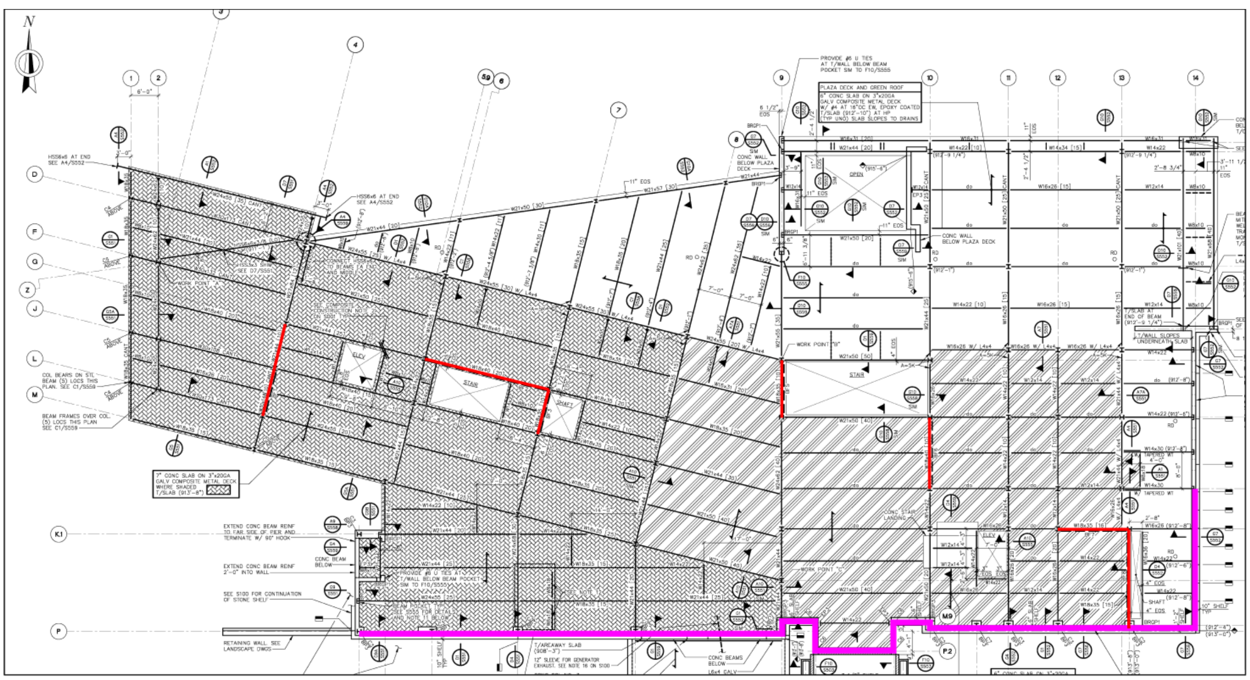
- UC Berkley's Industrial Engineering and Operations Research Center

- EFCO Corporation's Catalog

- Common Wealth Curb Appeal Bluestone Guide

Brief Overview of the Lateral System

The lateral system of the Peggy Ryan Williams Center consists primarily of concentrically braced structural steel frames. The north-south direction consists of various frames located throughout the building footprint. However, the east-west direction has fewer effective frames. The lack of effective east-west frames will allow more torsion to exist throughout the building. On the ground level of the building, a foundation wall is introduced which resists the soil loads. This foundation wall aids in the wind and seismic lateral loads as well. This causes some of the braced frames to carry more loads on story 2 than on story 1. Locations of the braced frames and foundation walls may be viewed below in Figure 1.



FOUNDATION WALL

BRACED FRAMES

Figure 1: First Floor Framing Plan Showing Locations of Braced Frames and Foundation Walls
Drawing S101

Computer Modeling Process

Due to my familiarity with ETABS, from the AE 530 Computer Modeling class, I decided to model the building in ETABS. However, as may be seen in Figure 1 above, the framing of the Peggy Ryan Williams Center has irregular geometries. Therefore, the layout lent itself to first be drawn in AutoCAD and then exported to ETABS. I began by drawing the grid and slabs in AutoCAD. I then exported these files into ETABS and converted them to ETABS grids and slabs, respectively. Upon completing the layout, I added the various braced frames to the entire building, as well as the foundation walls. While modeling, I made the decision to use the worst case roof height, which is conservative. I also assumed that the foundation walls would crack, per ACI 318. I modeled all of the columns of the braced frames to have a pinned base condition. I modeled these as pinned because a typical column detail illustrates the pinned connection which allows no moment transfer to the pier below. I modeled the foundation walls to have a fixed base because they are supported by 6’-0” wide footings. I then assigned a rigid diaphragm to each slab of the building. Once I had the elements modeled, I began to verify my model.

At this point, I ran into numerous problems with the model. First, I noticed that the joints on the braced frames were deflecting up and down. To begin troubleshooting, I made sure that all of the beams were fixed-fixed and that all of the braces were pinned-pinned. I then redrew many braces and beams to see if that would restrain the joints. None of these solutions seemed to fix the problem. I then removed the mass from all of the materials of the model. This appeared to fix the problem of the moving joints. It was decided that the materials of the model could not be modeled with mass since the model was only intended to be a lateral model, not a gravity model. Since the problem appeared to be fixed, I continued to verify the model by adding a 1000 kip test load to the model. Upon running the load, I noticed that neither the displacements nor the center of rigidity made sense. The displacements were much higher than expected. This led me to believe that the diaphragms and frames were not interacting. The center of rigidity was far away from the expected location. I attempted to fix the issue by removing the openings in the floors and temporarily removing the foundation walls to see if the center of rigidity would move and be more reasonable. However, neither of these solutions solved the problems.

At this point, it was determined that I needed to restart my model in ETABS. I learned that I needed to start simple and work my way up to a more complicated model. I started by simply modeling one frame at a time (on the west end of the building) and seeing how it reacted to a 100 kip test load. Once I verified that the model appeared to be behaving properly I would add another frame. Because the building changes geometry, I chose to only start by modeling the west end of the building, the orthogonal portion of the building. Once I had the west end of the building completed, I decided that I needed to move forward with the modeling process and further verify the model. Therefore, my model only consists of the west end of the building.

Since my ETABS model appeared to be behaving properly at this point in time, I began to hypothesize why my original model did not work. Two conclusions were drawn. First, by drafting both the grid and the floor slabs in AutoCAD and then importing into ETABS, the slabs were not snapped to the grid. Therefore, when I assigned the diaphragm to the slab and drew the frames (which snapped to the grid) the frames and diaphragms were not interacting. Second, the order in which the elements were drawn/assigned was not the correct order. It was hypothesized that in order to have the diaphragm interact with the frames, it not only had to be snapped to the grid, but also drawn/assigned after the frames were in place. Because I had drawn the slab before my frames in my original model, this was most likely also a source of error. Therefore, when I modeled the west end of the building, I made sure that my slab (and in turn the assigned diaphragm) attached to the grid and that the slabs were drawn after the frames were in place.

The following Figures (Figures 2 and 3) illustrate the portion of the lateral system that was modeled.

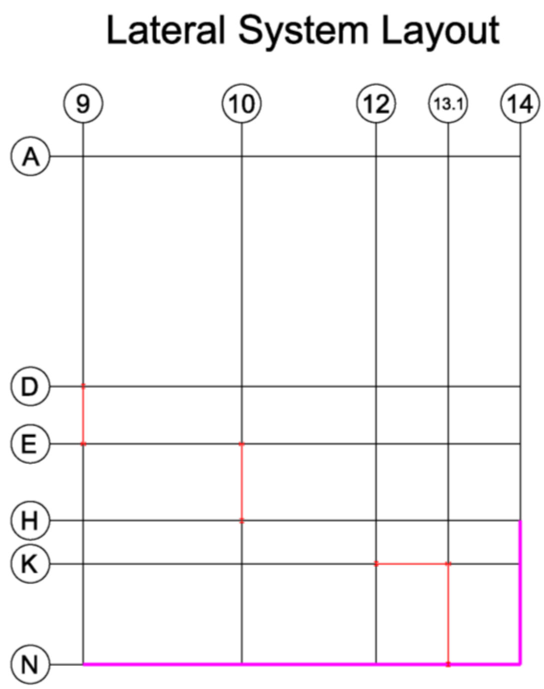


Figure 2: Plan View of Lateral System

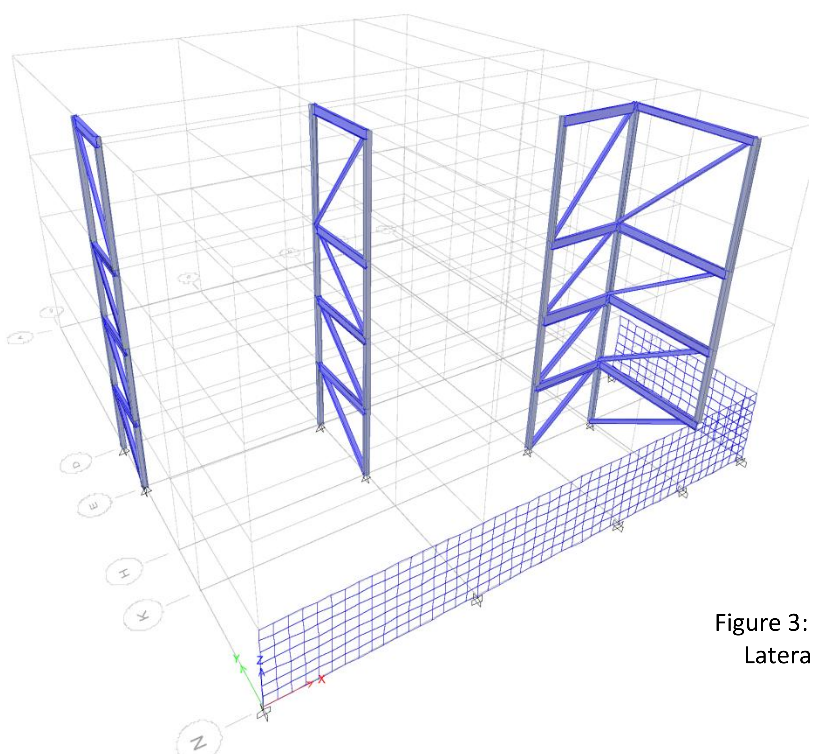


Figure 3: 3D View of Lateral System

Verifying the Model

Now that I had a working model, I continued to verify that the model was behaving properly by adding a 100 kip test load (in each the x and the y direction) and observing the behaviors. First, I checked to see if the ETABS generated centers of mass and centers of rigidity appeared to be reasonable. It can be seen on Figure 4 below that these centers do appear to be reasonable with respect to the simplified model. It may be important to note that the center of rigidity for the first story is on the foundation wall, which makes sense due to the foundation wall having a high rigidity. On stories 2-4, the center of rigidity is much closer to the centers of mass due to the foundation wall discontinuing after the first story.

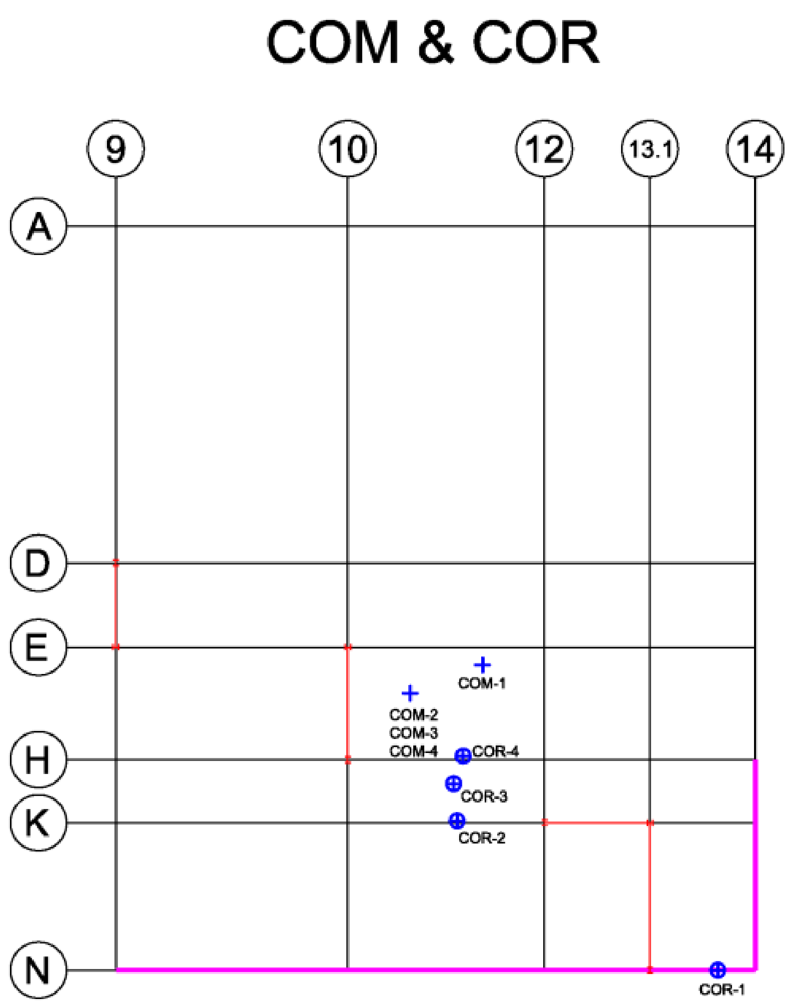


Figure 4: Centers of Mass and Centers of Rigidity

Story	COM		COR	
	X	Y	X	Y
1	52.2	43.5	85.661	0.0057
2	41.9091	39.4545	48.5779	21.2733
3	41.9091	39.4545	48.0732	26.562
4	41.9091	39.4545	49.4287	30.5032

Next, I verified that the deflected shape of the frames was reasonable under the 100 kip test load. Finally, I looked at the base reactions of each of the elements and ensured that when each of the 100 kip loads were applied that the sum of the applied forces and base reactions equaled zero. This verification may be seen in Figures 5 and 6 below. From the two Figures, it is evident that the foundation walls do a lot of the work in resisting the applied loads.

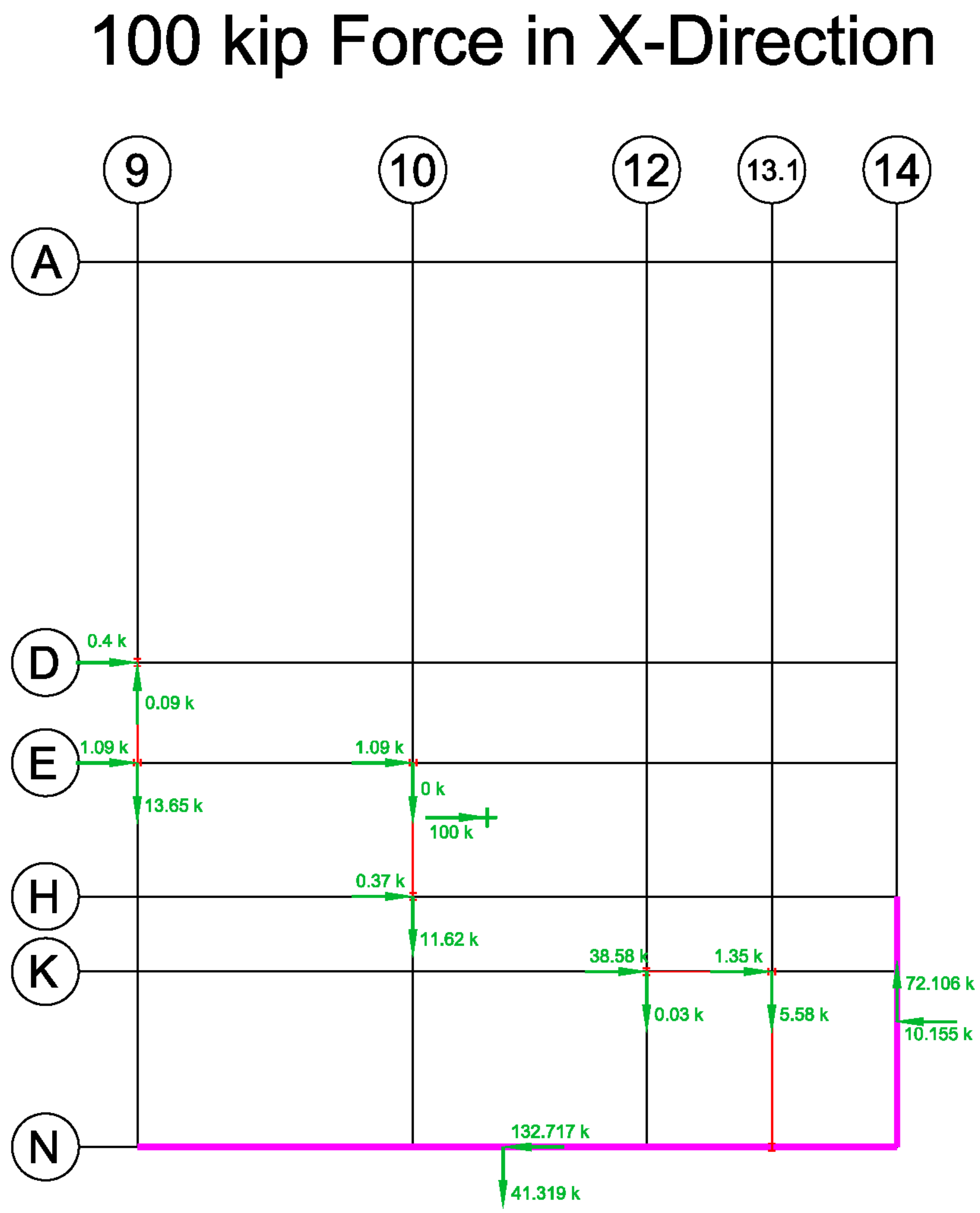


Figure 5: 100 kip Force in X-Direction to Verify Sum of Forces Equals Zero

100 kip Force in Y-Direction

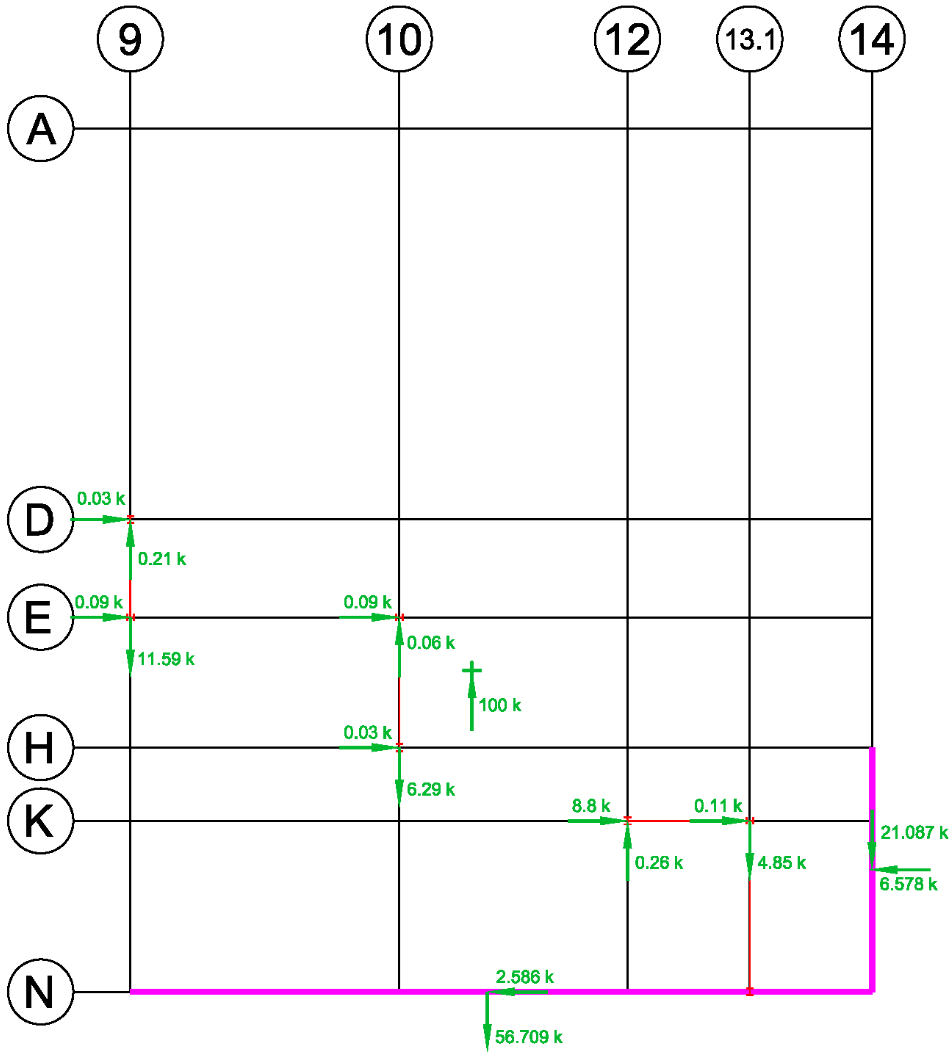


Figure 6: 100 kip Force in Y-Direction to Verify Sum of Forces Equals Zero

As I was verifying the model, I noticed that members of frames which were not in plane with the forces exhibited axial forces. These axial forces are due to the effects of torsion. Torsion is induced on the building because there is eccentricity due to the centers of mass and centers of rigidity not sharing the same locations. The torsion is then resisted by all of the frames, causing axial forces in members which are not in plane with the force.

Determination of Loads

Wind Load Cases

The four wind cases illustrated in ASCE7-98 Figure 6-9 were used to calculate the applied wind forces. The wind forces were distributed vertically by multiplying the calculated distributed load (psf) by the tributary height of each story to obtain a linear load for the diaphragm edge. The linear load was then multiplied by the tributary width of the story to obtain a point load. These calculations may be seen below. These forces were then horizontally distributed by applying each force to each stories' center of pressure. In order to further simplify the ETABS input (and avoid more human error) the windward and leeward forces were added together to obtain one force to apply to the stories' center of pressure. The resultant forces and locations may be seen below.

Wind Case 1

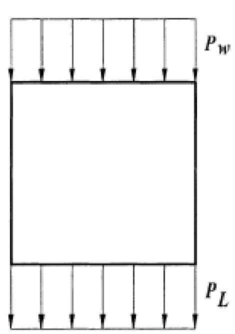
North-South Direction

WINDWARD P_W

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.73	*	13.33	=	103.20	*	98.0	=	10.2
Level 2	9.18	*	13.33	=	122.40	*	88.0	=	10.8
Level 3	10.17	*	17.17	=	174.60	*	79.5	=	13.9
Roof	14.86	*	20.50	=	304.80	*	83.5	=	25.5

LEEWARD P_L

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-7.26	*	13.33	=	-96.90	*	98.0	=	-9.5
Level 2	-7.26	*	13.33	=	-96.90	*	88.0	=	-8.6
Level 3	-7.26	*	17.17	=	-124.70	*	79.5	=	-10
Roof	-10.51	*	20.50	=	-215.50	*	83.5	=	-18



Diaphragm	Force to Apply P	Location x	Location y
Level 1	19.7	49.0	56.5
Level 2	19.4	44.0	37.3
Level 3	23.9	39.8	37.8
Roof	43.5	41.8	40.0

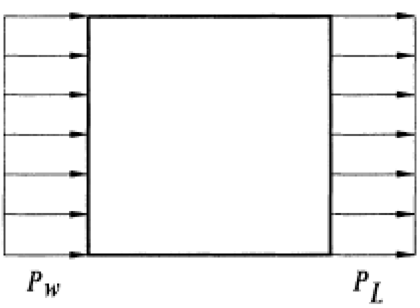
East-West Direction

WINDWARD P_W

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.72	*	13.33	=	103.00	*	113.0	=	11.7
Level 2	9.31	*	13.33	=	124.20	*	74.5	=	9.3
Level 3	10.39	*	17.17	=	178.40	*	75.5	=	13.5
Roof	15.05	*	20.50	=	308.60	*	80.0	=	24.7

LEEWARD P_L

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-4.29	*	13.33	=	-57.30	*	113.0	=	-6.5
Level 2	-4.29	*	13.33	=	-57.30	*	74.5	=	-4.3
Level 3	-4.29	*	17.17	=	-73.80	*	75.5	=	-5.6
Roof	-7.50	*	20.50	=	-153.80	*	80.0	=	-12.4



Diaphragm	Force to Apply P	Location x	Location y
Level 1	18.2	49.0	56.5
Level 2	13.6	44.0	37.3
Level 3	19.1	39.8	37.8
Roof	37.1	41.8	40.0

Wind Case 2

For wind case 2, because the effects of an applied moment are being calculated, the worst case for the applied forces had to be determined. The worst case may be seen below.

North-South Direction

WINDWARD P_W

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.73	*	13.33	=	103.20	*	49.0	=	5.1
Level 2	9.18	*	13.33	=	122.40	*	44.0	=	5.4
Level 3	10.17	*	17.17	=	174.60	*	39.8	=	7
Roof	14.86	*	20.50	=	304.80	*	41.8	=	12.8

WINDWARD $0.75\ P_W$

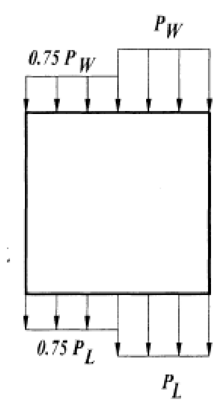
Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.73	*	13.33	*	0.75	=	77.40	*	49.0	=	3.8
Level 2	9.18	*	13.33	*	0.75	=	91.80	*	44.0	=	4.1
Level 3	10.17	*	17.17	*	0.75	=	131.00	*	39.8	=	5.3
Roof	14.86	*	20.50	*	0.75	=	228.60	*	41.8	=	9.6

LEEWARD P_L

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-7.26	*	13.33	=	-96.90	*	49.0	=	-4.8
Level 2	-7.26	*	13.33	=	-96.90	*	44.0	=	-4.3
Level 3	-7.26	*	17.17	=	-124.70	*	39.8	=	-5
Roof	-10.51	*	20.50	=	-215.50	*	41.8	=	-9

LEEWARD $0.75\ P_L$

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-7.26	*	13.33	*	0.75	=	-72.7	*	49.0	=	-3.6
Level 2	-7.26	*	13.33	*	0.75	=	-72.7	*	44.0	=	-3.2
Level 3	-7.26	*	17.17	*	0.75	=	-93.5	*	39.8	=	-3.8
Roof	-10.51	*	20.50	*	0.75	=	-161.6	*	41.8	=	-6.8



Diaphragm	Force to Apply P	Location x	Location y
Level 1	9.9	24.5	56.5
Level 2	9.7	22.0	37.3
Level 3	12	19.9	37.8
Roof	21.8	20.9	40.0

Diaphragm	Force to Apply 0.75 P	Location x	Location y
Level 1	7.4	73.5	56.5
Level 2	7.3	66.0	37.3
Level 3	9.1	59.6	37.8
Roof	16.4	62.6	40.0

East-West Direction

WINDWARD P_W

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.72	*	13.33	=	103.00	*	56.5	=	5.9
Level 2	9.31	*	13.33	=	124.20	*	37.3	=	4.7
Level 3	10.39	*	17.17	=	178.40	*	37.8	=	6.8
Roof	15.05	*	20.50	=	308.60	*	40.0	=	12.4

WINDWARD 0.75 P_W

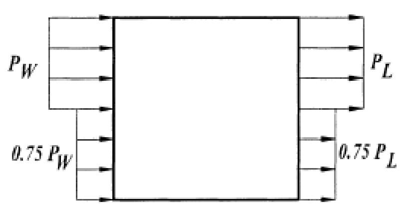
Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.72	*	13.33	*	0.75	=	77.3	*	56.5	=	4.4
Level 2	9.31	*	13.33	*	0.75	=	93.2	*	37.3	=	3.5
Level 3	10.39	*	17.17	*	0.75	=	133.8	*	37.8	=	5.1
Roof	15.05	*	20.50	*	0.75	=	231.5	*	40.0	=	9.3

LEEWARD P_L

Diaphragm	p (psf)	*	Height (ft)	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-4.29	*	13.33	=	-57.30	*	56.5	=	-3.3
Level 2	-4.29	*	13.33	=	-57.30	*	37.3	=	-2.2
Level 3	-4.29	*	17.17	=	-73.80	*	37.8	=	-2.8
Roof	-7.50	*	20.50	=	-153.80	*	40.0	=	-6.2

LEEWARD 0.75 P_L

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-4.29	*	13.33	*	0.75	=	-43.00	*	56.5	=	-2.5
Level 2	-4.29	*	13.33	*	0.75	=	-43.00	*	37.3	=	-1.7
Level 3	-4.29	*	17.17	*	0.75	=	-55.30	*	37.8	=	-2.1
Roof	-7.50	*	20.50	*	0.75	=	-115.30	*	40.0	=	-4.7



Diaphragm	Force to Apply P	Location x	Location y
Level 1	9.2	49.0	84.8
Level 2	6.9	44.0	55.9
Level 3	9.6	39.8	56.6
Roof	18.6	41.8	60.0

Diaphragm	Force to Apply 0.75 P	Location x	Location y
Level 1	6.9	49.0	28.3
Level 2	5.2	44.0	18.6
Level 3	7.2	39.8	18.9
Roof	14.0	41.8	20.0

Wind Case 3

North-South Direction

WINDWARD $0.75\ P_W$

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.73	*	13.33	*	0.75	=	77.40	*	98.0	=	7.6
Level 2	9.18	*	13.33	*	0.75	=	91.80	*	88.0	=	8.1
Level 3	10.17	*	17.17	*	0.75	=	131.00	*	79.5	=	10.5
Roof	14.86	*	20.50	*	0.75	=	228.60	*	83.5	=	19.1

LEEWARD $0.75\ P_L$

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-7.26	*	13.33	*	0.75	=	-72.70	*	98.0	=	-7.2
Level 2	-7.26	*	13.33	*	0.75	=	-72.70	*	88.0	=	-6.4
Level 3	-7.26	*	17.17	*	0.75	=	-93.50	*	79.5	=	-7.5
Roof	-10.51	*	20.50	*	0.75	=	-161.60	*	83.5	=	-13.5

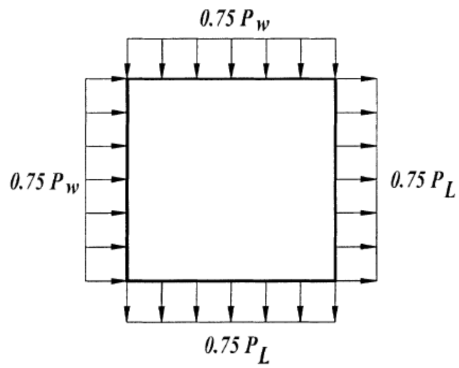
East-West Direction

WINDWARD $0.75\ P_W$

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.72	*	13.33	*	0.75	=	77.30	*	113.0	=	8.8
Level 2	9.31	*	13.33	*	0.75	=	93.20	*	74.5	=	7.0
Level 3	10.39	*	17.17	*	0.75	=	133.80	*	75.5	=	10.2
Roof	15.05	*	20.50	*	0.75	=	231.50	*	80.0	=	18.6

LEEWARD $0.75\ P_L$

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-4.29	*	13.33	*	0.75	=	-43.00	*	113.0	=	-4.9
Level 2	-4.29	*	13.33	*	0.75	=	-43.00	*	74.5	=	-3.3
Level 3	-4.29	*	17.17	*	0.75	=	-55.30	*	75.5	=	-4.2
Roof	-7.50	*	20.50	*	0.75	=	-115.30	*	80.0	=	-9.3



Diaphragm	Force to Apply P	Location x	Location y
Level 1	14.8	49.0	56.5
Level 2	14.5	44.0	37.3
Level 3	18.0	39.8	37.8
Roof	32.6	41.8	40.0

Diaphragm	Force to Apply P	Location x	Location y
Level 1	13.7	49.0	56.5
Level 2	10.3	44.0	37.3
Level 3	14.4	39.8	37.8
Roof	27.9	41.8	40.0

Wind Case 4

For wind case 4, because the effects of an applied moment are being calculated, the worst case for the applied forces had to be determined. The worst case may be seen below.

North-South Direction

WINDWARD $0.56\ P_W$

Diaphragm	p (psf)	*	Height (ft)	*	0.56	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.73	*	13.33	*	0.56	=	57.80	*	49.0	=	2.9
Level 2	9.18	*	13.33	*	0.56	=	68.60	*	44.0	=	3.1
Level 3	10.17	*	17.17	*	0.56	=	97.80	*	39.8	=	3.9
Roof	14.86	*	20.50	*	0.56	=	170.70	*	41.8	=	7.2

WINDWARD $0.75\ P_W$

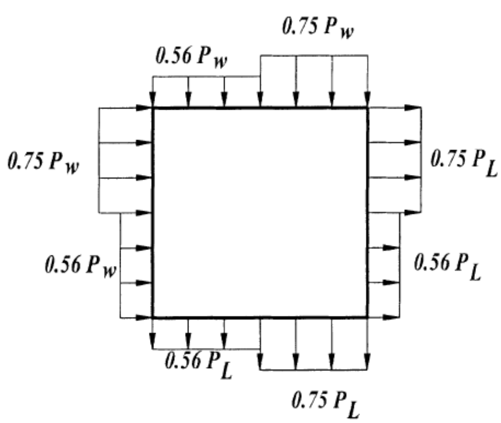
Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.73	*	13.33	*	0.75	=	77.40	*	49.0	=	3.8
Level 2	9.18	*	13.33	*	0.75	=	91.80	*	44.0	=	4.1
Level 3	10.17	*	17.17	*	0.75	=	131.00	*	39.8	=	5.3
Roof	14.86	*	20.50	*	0.75	=	228.60	*	41.8	=	9.6

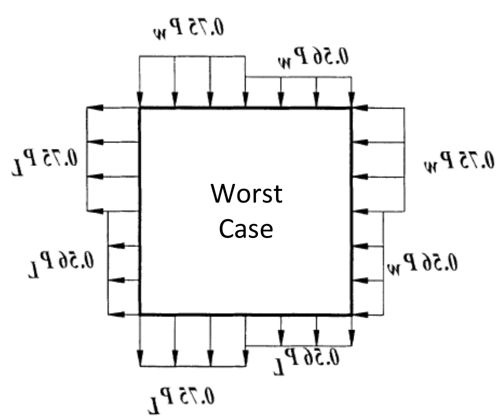
	Diaphragm	p (psf)	*	Height (ft)	*	0.56	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
LEEWARD 0.56 P_L	Level 1	-7.26	*	13.33	*	0.56	=	-54.30	*	49.0	=	-2.7
	Level 2	-7.26	*	13.33	*	0.56	=	-54.30	*	44.0	=	-2.4
	Level 3	-7.26	*	17.17	*	0.56	=	-69.90	*	39.8	=	-2.8
	Roof	-10.51	*	20.50	*	0.56	=	-120.70	*	41.8	=	-5.1

	Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
LEEWARD 0.75 P_L	Level 1	-7.26	*	13.33	*	0.75	=	-72.70	*	49.0	=	-3.6
	Level 2	-7.26	*	13.33	*	0.75	=	-72.70	*	44.0	=	-3.2
	Level 3	-7.26	*	17.17	*	0.75	=	-93.50	*	39.8	=	-3.8
	Roof	-10.51	*	20.50	*	0.75	=	-161.60	*	41.8	=	-6.8

Diaphragm	Force to Apply 0.56 P	Location x	y
Level 1	5.6	24.5	56.5
Level 2	5.5	22.0	37.3
Level 3	6.7	19.9	37.8
Roof	12.3	20.9	40.0

Diaphragm	Force to Apply 0.75 P	Location x	y
Level 1	7.4	73.5	56.5
Level 2	7.3	66.0	37.3
Level 3	9.1	59.6	37.8
Roof	16.4	62.6	40.0





East-West Direction

WINDWARD 0.56 P_W

Diaphragm	p (psf)	*	Height (ft)	*	0.56	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.72	*	13.33	*	0.56	=	57.70	*	56.5	=	3.3
Level 2	9.31	*	13.33	*	0.56	=	69.60	*	37.3	=	2.6
Level 3	10.39	*	17.17	*	0.56	=	99.90	*	37.8	=	3.8
Roof	15.05	*	20.50	*	0.56	=	172.80	*	40.0	=	7

WINDWARD 0.75 P_W

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	7.72	*	13.33	*	0.75	=	77.30	*	56.5	=	4.4
Level 2	9.31	*	13.33	*	0.75	=	93.20	*	37.3	=	3.5
Level 3	10.39	*	17.17	*	0.75	=	133.80	*	37.8	=	5.1
Roof	15.05	*	20.50	*	0.75	=	231.50	*	40.0	=	9.3

LEEWARD 0.56 P_L

Diaphragm	p (psf)	*	Height (ft)	*	0.56	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-4.29	*	13.33	*	0.56	=	-32.10	*	56.5	=	-1.9
Level 2	-4.29	*	13.33	*	0.56	=	-32.10	*	37.3	=	-1.2
Level 3	-4.29	*	17.17	*	0.56	=	-41.30	*	37.8	=	-1.6
Roof	-7.50	*	20.50	*	0.56	=	-86.10	*	40.0	=	-3.5

LEEWARD 0.75 P_L

Diaphragm	p (psf)	*	Height (ft)	*	0.75	=	W (plf)	*	Wall Length (ft)	=	Pw (kip)
Level 1	-4.29	*	13.33	*	0.75	=	-43.00	*	56.5	=	-2.5
Level 2	-4.29	*	13.33	*	0.75	=	-43.00	*	37.3	=	-1.7
Level 3	-4.29	*	17.17	*	0.75	=	-55.30	*	37.8	=	-2.1
Roof	-7.50	*	20.50	*	0.75	=	-115.30	*	40.0	=	-4.7

Diaphragm	Force to Apply 0.56 P	Location x	Location y
Level 1	5.2	49.0	28.3
Level 2	3.8	44.0	18.6
Level 3	5.4	39.8	18.9
Roof	10.5	41.8	20.0

Diaphragm	Force to Apply 0.75 P	Location x	Location y
Level 1	6.9	49.0	84.8
Level 2	5.2	44.0	55.9
Level 3	7.2	39.8	56.6
Roof	14.0	41.8	60.0

Seismic Load Cases

Four seismic load cases were used to calculate the applied seismic forces. Two of these load cases were in the North-South direction, accounting for positive and negative accidental torsion, and two were in the East-West direction, accounting for accidental torsion in that direction. The seismic forces were calculated for each floor of the building and then applied to the center of mass of each floor. Because the seismic forces which were originally calculated in Technical Report 2 included the mass of the entire building, each seismic story force was adjusted to account for the decrease in the building's mass (since the model only included the west portion of the building). The accidental torsional moment was calculated by multiplying 5% of the Bx dimension (the length of the building face perpendicular to the force) by the story force. The seismic force calculations may be viewed below.

North-South Direction

Diaphragm	Story Force (kips)	Adjustment	Adj Story Force (kips)	Story Shear (V_i) (kips)	Bx (ft)	5% Bx (ft)	Ax	Mz (ft-kip)
Level 1	25.77	0.53	13.62	13.62	98.00	4.9	1.0	66.8
Level 2	15.42	0.35	5.45	19.07	88.00	4.4	1.0	24.0
Level 3	18.49	0.41	7.50	26.57	79.50	3.975	1.0	29.9
Roof	8.79	0.40	3.49	30.07	83.50	4.175	1.0	14.6

East-West Direction

Diaphragm	Story Force (kips)	Adjustment	Adj Story Force (kips)	Story Shear (V_i) (kips)	By (ft)	5% By (ft)	Ax	Mz (ft-kip)
Level 1	25.77	0.53	13.62	13.62	113.00	5.65	1.0	77.0
Level 2	15.42	0.35	5.45	19.07	74.50	3.725	1.0	20.4
Level 3	18.49	0.41	7.50	26.57	75.50	3.775	1.0	28.4
Roof	8.79	0.40	3.49	30.07	80.00	4	1.0	14.0

Distribute Lateral Forces to Beams

Due to the way in which ETABS performs its calculations and the way the frames interact with the diaphragm, the program would not provide the beam forces. Therefore, the story forces were distributed to the beams by hand using a relative stiffness method. The calculation of the beam forces may be viewed below.

Equations Used

Beam Axial Stiffness: $k = \frac{12E\,(I_1+I_2)}{h^3}$

Shear Wall Stiffness: $k = \frac{3\,E\,I}{h^3}$

Distribution of Forces: $V_i = \frac{k_i * V}{\sum k_i}$

FRAME K

W 18x35 Beam

Resists forces in EW direction

E =	29000	ksi
I_1 =	93.4	in^4
I_2 =	272	in^5
h =	159.96	in
k =	31.07	k/in
$k_i/\sum ki$ =	0.0002	

Story	Load Case/Combo	Story Force (kip)	P (kip)	M max (ft-kip)
Story1	Wind - Case 1 - EW	-18.2	-0.0034	-2.3079
Story1	Wind - Case 3 - NS EW	-13.7	-0.0025	-1.8824
Story1	Earthquake -- EW	-25.77	-0.0048	-1.9567
Story1	Wind - Case 2 -- EW	-16.1	-0.0030	-2.0718
Story1	Wind - Case 4 -- NS EW	-12.1	-0.0023	-1.6422

SOUTH FOUNDATION WALL

Resists forces in EW direction

E =	3605.0	ksi
t =	20.5	in
b =	1092	in
h =	159.96	in
k =	166975.8	k/in
$k_i/\sum ki$ =	0.9998	

FRAME 9

W 18x35 Beam

Resists forces in NS direction

E =	29000	ksi
I_1 =	93.4	in^4
I_2 =	272	in^5
h =	159.96	in
k =	31.07	k/in
$k_i/\sum ki$ =	0.0006	

Story	Load Case/Combo	Story Force (kip)	P (kip)	M max (ft-kip)
Story1	Wind - Case 1 - NS	-19.7	-0.0117	-2.8319
Story1	Wind - Case 3 - NS EW	-14.8	-0.0088	-3.6253
Story1	Earthquake -- NS	-25.77	-0.0154	-1.3853
Story1	Wind - Case 2 -- NS	-17.3	-0.0103	-2.7217
Story1	Wind - Case 4 -- NS EW	-13.0	-0.0078	-3.1441

FRAME 10

W 18x40 Beam

Resists forces in NS direction

$E =$ 29000 ksi

$I_1 =$ 93.4 in^4

$I_2 =$ 272 in^5

$h =$ 159.96 in

$k =$ 31.07 k/in

$k_i / \sum ki =$ 0.0006

Story	Load Case/Combo	Story Force (kip)	P (kip)	M max (ft-kip)
Story1	Wind - Case 1 - NS	-19.7	-0.0117	2.8167
Story1	Wind - Case 3 - NS EW	-14.8	-0.0088	2.8469
Story1	Earthquake -- NS	-25.77	-0.0154	1.4774
Story1	Wind - Case 2 -- NS	-17.3	-0.0103	2.5823
Story1	Wind - Case 4 -- NS EW	-13.0	-0.0078	2.4872

FRAME 13.1

W 18x35 Beam
Resists forces in NS direction

$E =$ 29000 ksi
$I_1 =$ 93.4 in^4
$I_2 =$ 272 in^5
$h =$ 159.96 in

$k =$ 31.07 k/in
$k_i/\sum ki =$ 0.3333

Story	Load Case/Combo	Story Force (kip)	P (kip)	M max (ft-kip)
Story2	Wind - Case 1 - NS	-19.4	-6.47	6.6833
Story2	Wind - Case 3 - NS EW	-14.5	-4.83	0.5345
Story2	Earthquake -- NS	-15.42	-5.14	3.65
Story2	Wind - Case 2 -- NS	-17.0	-5.67	5.1921
Story2	Wind - Case 4 -- NS EW	-12.8	-4.27	0.6033

EAST FOUNDATION WALL

Resists forces in NS direction

$E =$ 3605.0 ksi
$t =$ 20.5 in
$b =$ 360 in
$h =$ 159.96 in

$k =$ 52017.38 k/in
$k_i/\sum ki =$ 0.9982

Determination of Worst Case Wind/Seismic

The following shows a compilation of the ETABS output which illustrates the axial forces and moments of each member for each frame. Both the worst case for the axial force and for the moment was determined. Ultimately, the axial force will dictate the worst case lateral force for each frame because the magnitudes of the axial forces are much larger and in turn more significant than those of the moments.

NOTE: Positive numbers denote tension and negative numbers denote compression.

FRAME K

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	K12	Wind - Case 1 - NS	-0.163	-0.1379	2.3269
Story1	K12	Wind - Case 1 - EW	158.682	-2.2618	-0.4562
Story1	K12	Wind - Case 3 - NS EW	119.336	-1.8064	1.4005
Story1	K12	Earthquake -- NS +moment	-0.028	-0.052	1.1484
Story1	K12	Earthquake -- EW +moment	72.448	-1.101	-0.2764
Story1	K12	Earthquake -- NS -moment	72.468	-1.0345	-0.5191
Story1	K12	Earthquake -- EW -moment	72.468	-1.0345	-0.5191
Story1	K12	Wind - Case 2 -- NS	-0.127	-0.0708	1.8818
Story1	K12	Wind - Case 2 -- EW	139.607	-1.9506	-0.5252
Story1	K12	Wind - Case 4 -- NS EW	104.674	-1.592	1.2627

WORST CASE (magnitude)		
Axial	158.682	Wind - Case 1 - EW
Moment	2.3269	Wind - Case 1 - NS

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	K13.1	Wind - Case 1 - NS	-51.851	0.6939	0.5855
Story1	K13.1	Wind - Case 1 - EW	-108.165	0.2385	9.1857
Story1	K13.1	Wind - Case 3 - NS EW	-120.322	0.6996	7.3574
Story1	K13.1	Earthquake -- NS +moment	-23.18	0.3508	0.2226
Story1	K13.1	Earthquake -- EW +moment	-52.862	0.1075	4.8237
Story1	K13.1	Earthquake -- NS -moment	-48.548	-0.0494	4.6112
Story1	K13.1	Earthquake -- EW -moment	-48.548	-0.0494	4.6112
Story1	K13.1	Wind - Case 2 -- NS	-41.481	0.4935	0.3487
Story1	K13.1	Wind - Case 2 -- EW	-92.005	0.1207	7.9561
Story1	K13.1	Wind - Case 4 -- NS EW	-106.307	0.6349	6.4772

WORST CASE (magnitude)		
Axial	120.322	Wind - Case 3 - NS EW
Moment	9.1857	Wind - Case 1 - EW

WORST CASE (magnitude)		
Axial	158.6820	Wind - Case 1 - EW

Story	Brace	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	D1	Wind - Case 1 - NS	-4.43	0	0
Story1	D1	Wind - Case 1 - EW	-23.631	0	0
Story1	D1	Wind - Case 3 - NS EW	-21.108	0	0
Story1	D1	Earthquake -- NS +moment	-1.673	0	0
Story1	D1	Earthquake -- EW +moment	-10.216	0	0
Story1	D1	Earthquake -- NS -moment	-7.454	0	0
Story1	D1	Earthquake -- EW -moment	-7.454	0	0
Story1	D1	Wind - Case 2 -- NS	-1.848	0	0
Story1	D1	Wind - Case 2 -- EW	-19.173	0	0
Story1	D1	Wind - Case 4 -- NS EW	-18.829	0	0

WORST CASE (magnitude)		
Axial	-23.631	Wind - Case 1 - EW
Moment	0	-

Beam W18x35

WORST CASE (magnitude)		
Axial	-0.0048	Earthquake -- EW
Moment	-2.31	Wind - Case 1 - EW

→Frame K worst load case was determined to be Wind Case 1 in the East-West direction.

FRAME 9

FRAME 9

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	E9	Wind - Case 1 - NS	62.8	0.1929	0.3644
Story1	E9	Wind - Case 1 - EW	31.478	-0.1163	7.638
Story1	E9	Wind - Case 3 - NS EW	70.803	0.0568	6.0271
Story1	E9	Earthquake -- NS +moment	19.387	-0.1027	-0.0847
Story1	E9	Earthquake -- EW +moment	10.959	-0.2084	4.0475
Story1	E9	Earthquake -- NS -moment	18.475	-0.168	4.1411
Story1	E9	Earthquake -- EW -moment	18.475	-0.168	4.1411
Story1	E9	Wind - Case 2 -- NS	61.775	0.2366	0.3948
Story1	E9	Wind - Case 2 -- EW	32.766	-0.0537	6.7765
Story1	E9	Wind - Case 4 -- NS EW	61.195	0.0376	5.2678

WORST CASE (magnitude)		
Axial	70.803	Wind - Case 3 - NS EW
Moment	7.638	Wind - Case 1 - EW

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	D9	Wind - Case 1 - NS	-79.368	-0.1005	1.1332
Story1	D9	Wind - Case 1 - EW	-46.702	-2.7714	0.0979
Story1	D9	Wind - Case 3 - NS EW	-94.704	-2.1632	0.9231
Story1	D9	Earthquake -- NS +moment	-29.882	0.0786	0.0056
Story1	D9	Earthquake -- EW +moment	-21.827	-1.4545	-0.3208
Story1	D9	Earthquake -- NS -moment	-30.934	-1.5266	-0.12
Story1	D9	Earthquake -- EW -moment	-30.934	-1.5266	-0.12
Story1	D9	Wind - Case 2 -- NS	-76.626	-0.1461	1.2314
Story1	D9	Wind - Case 2 -- EW	-46.469	-2.4817	0.2585
Story1	D9	Wind - Case 4 -- NS EW	-82.198	-1.885	0.7684

WORST CASE (magnitude)		
Axial	-94.704	Wind - Case 3 - NS EW
Moment	-2.7714	Wind - Case 1 - EW

WORST CASE (magnitude)		
Axial	70.8030	Wind - Case 3 - NS EW

Story	Brace	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	D4	Wind - Case 1 - NS	22.293	0	0
Story1	D4	Wind - Case 1 - EW	20.485	0	0
Story1	D4	Wind - Case 3 - NS EW	32.158	0	0
Story1	D4	Earthquake -- NS +moment	14.121	0	0
Story1	D4	Earthquake -- EW +moment	14.623	0	0
Story1	D4	Earthquake -- NS -moment	16.763	0	0
Story1	D4	Earthquake -- EW -moment	16.763	0	0
Story1	D4	Wind - Case 2 -- NS	19.982	0	0
Story1	D4	Wind - Case 2 -- EW	18.438	0	0
Story1	D4	Wind - Case 4 -- NS EW	28.26	0	0

WORST CASE (magnitude)		
Axial	32.158	Wind - Case 3 - NS EW
Moment	0	-

Beam W 18x35

WORST CASE (magnitude)		
Axial	-0.0154	Wind - Case 1 - NS
Moment	-3.63	Wind - Case 3 - NS EW

→Frame 9 worst load case was determined to be Wind Case 3.

FRAME 10

FRAME 10

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	H10	Wind - Case 1 - NS	62.844	-0.1606	1.3537
Story1	H10	Wind - Case 1 - EW	22.181	-2.5714	-0.2308
Story1	H10	Wind - Case 3 - NS EW	63.834	-2.0575	0.8409
Story1	H10	Earthquake -- NS +moment	22.518	-0.0333	0.3811
Story1	H10	Earthquake -- EW +moment	8.689	-1.3882	-0.3097
Story1	H10	Earthquake -- NS -moment	11.243	-1.3689	-0.3579
Story1	H10	Earthquake -- EW -moment	11.243	-1.3689	-0.3579
Story1	H10	Wind - Case 2 -- NS	57.548	-0.1254	1.2014
Story1	H10	Wind - Case 2 -- EW	21.306	-2.251	-0.1985
Story1	H10	Wind - Case 4 -- NS EW	55.774	-1.8058	0.7364

WORST CASE (magnitude)		
Axial	63.834	Wind - Case 3 - NS EW
Moment	2.5714	Wind - Case 1 - EW

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	E10	Wind - Case 1 - NS	-71.342	0.2703	0.3638
Story1	E10	Wind - Case 1 - EW	-31.469	-0.1592	7.6271
Story1	E10	Wind - Case 3 - NS EW	-77.207	0.0826	6.0185
Story1	E10	Earthquake -- NS +moment	-27.946	0.0225	-0.0846
Story1	E10	Earthquake -- EW +moment	-14.934	-0.1579	4.0418
Story1	E10	Earthquake -- NS -moment	-18.981	-0.1899	4.1352
Story1	E10	Earthquake -- EW -moment	-18.981	-0.1899	4.1352
Story1	E10	Wind - Case 2 -- NS	-65.681	0.2334	0.3942
Story1	E10	Wind - Case 2 -- EW	-30.037	-0.1451	6.7668
Story1	E10	Wind - Case 4 -- NS EW	-67.443	0.0728	5.2603

WORST CASE (magnitude)		
Axial	-77.207	Wind - Case 3 - NS EW
Moment	7.6271	Wind - Case 1 - EW

Story	Brace	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story1	D2	Wind - Case 1 - NS	13.276	0	0
Story1	D2	Wind - Case 1 - EW	14.51	0	0
Story1	D2	Wind - Case 3 - NS EW	20.893	0	0
Story1	D2	Earthquake -- NS +moment	8.481	0	0
Story1	D2	Earthquake -- EW +moment	9.757	0	0
Story1	D2	Earthquake -- NS -moment	12.089	0	0
Story1	D2	Earthquake -- EW -moment	12.089	0	0
Story1	D2	Wind - Case 2 -- NS	12.707	0	0
Story1	D2	Wind - Case 2 -- EW	13.641	0	0
Story1	D2	Wind - Case 4 -- NS EW	18.23	0	0

WORST CASE (magnitude)		
Axial	20.893	Wind - Case 3 - NS EW
Moment	0	-

Beam W 18x40

WORST CASE (magnitude)		
Axial	-0.0154	Earthquake -- NS
Moment	2.85	Wind - Case 3 - NS EW

WORST CASE (magnitude)		
Axial	63.8340	Wind - Case 3 - NS EW

→Frame 10 worst load case was determined to be Wind Case 3.

FRAME 13.1

FRAME 13.1

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story2	N13.1	Wind - Case 1 - NS	54.802	-0.5153	6.8169
Story2	N13.1	Wind - Case 1 - EW	-34.57	-5.2862	-3.5117
Story2	N13.1	Wind - Case 3 - NS EW	15.079	-4.3685	2.4648
Story2	N13.1	Earthquake -- NS +moment	24.224	-0.3516	3.7804
Story2	N13.1	Earthquake -- EW +moment	-12.682	-2.9568	-1.7381
Story2	N13.1	Earthquake -- NS -moment	-18.845	-2.6911	-2.8976
Story2	N13.1	Earthquake -- EW -moment	-18.845	-2.6911	-2.8976
Story2	N13.1	Wind - Case 2 -- NS	42.703	-0.2392	5.1691
Story2	N13.1	Wind - Case 2 -- EW	-34.644	-4.493	-3.7091
Story2	N13.1	Wind - Case 4 -- NS EW	14.203	-3.8672	2.3253

WORST CASE (magnitude)		
Axial	54.802	Wind - Case 1 - NS
Moment	6.8169	Wind - Case 1 - NS

Story	Column	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story2	K13.1	Wind - Case 1 - NS	-33.58	2.7824	0.8566
Story2	K13.1	Wind - Case 1 - EW	-134.886	-2.1703	12.7716
Story2	K13.1	Wind - Case 3 - NS EW	-126.739	0.4517	10.2651
Story2	K13.1	Earthquake -- NS +moment	-12.229	1.5825	0.3252
Story2	K13.1	Earthquake -- EW +moment	-64.974	-1.0449	7.4116
Story2	K13.1	Earthquake -- NS -moment	-61.738	-1.4191	7.2478
Story2	K13.1	Earthquake -- EW -moment	-61.738	-1.4191	7.2478
Story2	K13.1	Wind - Case 2 -- NS	-26.088	2.1853	0.6111
Story2	K13.1	Wind - Case 2 -- EW	-115.955	-2.1036	11.1385
Story2	K13.1	Wind - Case 4 -- NS EW	-111.722	0.4511	9.0213

WORST CASE (magnitude)		
Axial	-134.886	Wind - Case 1 - EW
Moment	12.7716	Wind - Case 1 - EW

WORST CASE (magnitude)		
Axial	54.8020	Wind - Case 1 - EW

Story	Brace	Load Case/Combo	P kip	M2 kip-ft	M3 kip-ft
Story2	D5	Wind - Case 1 - NS	-39.251	0	0
Story2	D5	Wind - Case 1 - EW	20.705	0	0
Story2	D5	Wind - Case 3 - NS EW	-13.835	0	0
Story2	D5	Earthquake -- NS +moment	-22.313	0	0
Story2	D5	Earthquake -- EW +moment	10.179	0	0
Story2	D5	Earthquake -- NS -moment	15.616	0	0
Story2	D5	Earthquake -- EW -moment	15.616	0	0
Story2	D5	Wind - Case 2 -- NS	-30.718	0	0
Story2	D5	Wind - Case 2 -- EW	21.048	0	0
Story2	D5	Wind - Case 4 -- NS EW	-12.925	0	0

WORST CASE (magnitude)		
Axial	-39.251	Wind - Case 1 - NS
Moment	0	-

Beam W 18x35

WORST CASE (magnitude)		
Axial	-6.47	Wind - Case 1 - NS
Moment	6.68	Wind - Case 1 - NS

→Frame 13.1 worst load case was determined to be Wind Case 1 in the East-West direction.

Determination of Controlling Load Combination

In order to determine the controlling load combination, each frame was modeled in RISA and three separate types of loadings were applied, the dead load, live load, and snow load. An excel sheet was then constructed using both the axial forces and the moments in order to determine the controlling load combination. The load combinations tested were taken from ASCE7-98 Section 2.3.2 and may be viewed below. By inspection, it was determined that only load combinations 2, 3, and 4 had to be calculated. As shown by the calculations below, the controlling load combination for every frame was determined to be 1.2 D + 1.6 W + 0.5 L + 0.5 S.

2.3.2 Basic Combinations

Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

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

FRAME K

Frame K
at Level 1

	Member Reaction Due to: Dead Load	Live Load	Snow	Wind/Earthquake	1.2 D + 1.6 L + 0.5 S	1.2 D + 1.6 S + 0.5 L	1.2 D + 1.6 S + 0.8 W	1.2 D + 1.6 W + 0.5 L + 0.5 S
Column K12				Wind - Case 1 - EW				
Axial Force (kip)	-48.159	-35.929	-7.215	-158.682	-118.9	-87.3	-196.3	-333.3
Moment (ft-kip)	1.618	1.554	-0.062	2.2618	4.4	2.6	3.7	6.3
Column K13.1								
Axial Force (kip)	-26.231	-28.519	-6.214	-108.165	-80.2	-55.7	-128.0	-221.9
Moment (ft-kip)	12.592	11.468	-0.122	-9.1857	33.4	20.6	7.6	6.1
Beam W18 x 35								
Axial Force (kip)	0.834	0.718	0.041	0.0034	2.2	1.4	1.1	1.4
Moment (ft-kip)	-12.084	-11.14	-0.13	-2.3079	-32.4	-20.3	-16.6	-23.8
Brace								
Axial Force (kip)	-0.333	-0.288	-0.015	-23.631	-0.9	-0.6	-19.3	-38.4
Moment (ft-kip)	0	0	0	0	0.0	0.0	0.0	0.0

FRAME 9

Frame 9
at Level 1

	Member Reaction Due to: Dead Load	Live Load	Snow	Wind/Earthquake	1.2 D + 1.6 L + 0.5 S	1.2 D + 1.6 S + 0.5 L	1.2 D + 1.6 S + 0.8 W	1.2 D + 1.6 W + 0.5 L + 0.5 S
Column E9				Wind - Case 3 - NS EW				
Axial Force (kip)	-116.696	-93.373	-11.838	-70.803	-295.4	-205.7	-215.6	-305.9
Moment (ft-kip)	0.52	0.603	-0.119	6.0271	1.5	0.7	5.3	10.5
Column D9								
Axial Force (kip)	-88.54	-59.58	-18.96	-94.704	-211.1	-166.4	-212.3	-297.0
Moment (ft-kip)	-6.329	-5.612	-0.144	-2.1632	-16.6	-10.6	-9.6	-13.9
Beam W18 x 35								
Axial Force (kip)	0.989	0.805	0.085	0.0088	2.5	1.7	1.3	1.6
Moment (ft-kip)	13.635	11.615	0.724	3.6253	35.3	23.3	20.4	28.3
Brace								
Axial Force (kip)	-0.651	-0.562	-0.029	-32.158	-1.7	-1.1	-26.6	-52.5
Moment (ft-kip)	0	0	0	0	0.0	0.0	0.0	0.0

FRAME 10

Frame 10
at Level 1

	Member Reaction Due to:							
	Dead Load	Live Load	Snow	Wind/Earthquake	1.2 D + 1.6 L + 0.5 S	1.2 D + 1.6 S + 0.5 L	1.2 D + 1.6 S + 0.8 W	1.2 D + 1.6 W + 0.5 L + 0.5 S
Column H10				Wind - Case 3 - NS EW				
Axial Force (kip)	-64.894	-52.986	-5.673	-63.834	-165.5	-113.4	-138.0	-209.3
Moment (ft-kip)	14.115	12.964	-0.067	2.0575	37.6	23.3	18.5	26.7
Column E10								
Axial Force (kip)	-57.21	-43.24	-8.07	-77.207	-141.9	-103.2	-143.3	-217.8
Moment (ft-kip)	-7.484	-6.791	-0.039	-6.0185	-19.9	-12.4	-13.9	-22.0
Beam W18 x 40								
Axial Force (kip)	0.313	0.261	0.021	-0.0088	0.8	0.5	0.4	0.5
Moment (ft-kip)	-47.963	-42.85	-0.208	-2.8469	-126.2	-79.3	-60.2	-83.6
Brace								
Axial Force (kip)	0.647	0.603	-0.01	-20.893	1.7	1.1	-16.0	-32.4
Moment (ft-kip)	0	0	0	0	0.0	0.0	0.0	0.0

FRAME 13.1

Frame 13.1
at Level 2

	Member Reaction Due to: Dead Load	Live Load	Snow	Wind/Earthquake	1.2 D + 1.6 L + 0.5 S	1.2 D + 1.6 S + 0.5 L	1.2 D + 1.6 S + 0.8 W	1.2 D + 1.6 W + 0.5 L + 0.5 S
Column N13.1				Wind - Case 1 - EW				
Axial Force (kip)	-28.554	-20.709	-4.872	-34.57	-69.8	-52.4	-69.7	-102.4
Moment (ft-kip)	11.975	10.678	0.271	5.2862	31.6	20.1	19.0	28.3
Column K13.1								
Axial Force (kip)	-26.231	-28.519	-6.214	-134.886	-80.2	-55.7	-149.3	-264.7
Moment (ft-kip)	12.592	11.468	-0.122	12.7716	33.4	20.6	25.1	41.2
Beam W18 x 35								
Axial Force (kip)	0.952	1.013	-0.128	0	2.7	1.4	0.9	1.6
Moment (ft-kip)	25.245	23.261	-0.162	2.12	67.4	41.7	31.7	45.2
Brace								
Axial Force (kip)	-0.78	-0.675	-0.036	-20.705	-2.0	-1.3	-17.6	-34.4
Moment (ft-kip)	0	0	0	0	0.0	0.0	0.0	0.0

Check Frame Member Strengths

Each frame's elements were checked for strength adequacy. For frames K, 9, and 10, the critical case was found to be at Story 1. For frame 13.1, the critical case was found to be at Story 2 due to one of its columns discontinuing at Level 2, and the foundation wall picking up the load. Each of the columns was checked using the interaction beam-column equations shown below. Due to the small tension forces in the beams, the beams were only checked for their moment strength capacity. Each of the braces was checked for their compression strength. All of the ΦPn and ΦMn values were determined using tables found in the Steel Manual. As indicated below, all of the members passed for strength.

Equations Used

Pu/ΦPn ≥ 0.2 → $$\frac{P_u}{\varphi P_n} + \frac{8M_u}{9\varphi M_n} \leq 1.0$$

Pu/ΦPn < 0.2 → $$\frac{P_u}{2\varphi P_n} + \frac{M_u}{\varphi M_n} \leq 1.0$$

FRAME K

At Story 1	Member	Pu	ΦPn	Mu	ΦMn	Pu/ΦPn	Pu/ΦPn + 8Mu/9ΦMn	PASS/FAIL
Column K12	W10x49	-333.3	492.0	6.3	106	0.68	0.73	PASS
Column K13.1	W10x49	-221.9	492.0	6.1	227	0.45	0.47	PASS
Beam	W18x35	1.4	-	-23.8	249			PASS
Brace	HSS8x8x3/8	-38.4	363.0	0.0	-			PASS

FRAME 9

At Story 1	Member	Pu	ΦPn	Mu	ΦMn	Pu/ΦPn	Pu/ΦPn + 8Mu/9ΦMn	PASS/FAIL
Column E9	W10x49	-305.9	492.0	10.5	106	0.62	0.71	PASS
Column D9	W10x49	-297.0	492.0	-13.9	227	0.60	0.66	PASS
Beam	W18x35	1.6	-	28.3	249			PASS
Brace	HSS8x8x3/8	-52.5	363.0	0.0	-			PASS

FRAME 10

At Story 1	Member	Pu	ΦPn	Mu	ΦMn	Pu/ΦPn	Pu/ΦPn + 8Mu/9ΦMn	PASS/FAIL
Column H10	W10x49	-209.3	492.0	26.7	227	0.43	0.53	PASS
Column E10	W10x49	-217.8	492.0	-22.0	106	0.44	0.63	PASS
Beam	W18x40	0.5	-	-83.6	294			PASS
Brace	HSS8x8x3/8	-32.4	363.0	0.0	-			PASS

FRAME 13.1

	Member	Pu	ΦPn	Mu	ΦMn	Pu/ΦPn	Pu/ΦPn + 8Mu/9ΦMn	PASS/FAIL
Column N13.1	W10x49	-102.4	492.0	28.3	227	0.21	0.32	PASS
Column K13.1	W10x49	-264.7	492.0	41.2	106	0.54	0.88	PASS
Beam	W18x35	1.6	-	45.2	249			PASS
Brace	HSS8x6x3/8	-34.4	278.0	0.0	-			PASS

Check Drift

Seismic Story Drift

Story drift in each direction was checked against the allowable story drift per ASCE7-98 Table 9.5.2.8. As shown below, the story drifts were compared to the value calculated using the equation $0.015h_{sx}$. Each of the directions passed with a large safety margin in respects to the allowable story drift and the check may be viewed below. Accidental torsion was accounted for by checking the story drifts for both the positive and negative moments induced by the accidental torsion. Because the building is classified as Seismic Design Category A, torsional irregularities did not need to be considered.

TABLE 9.5.2.8. Allowable Story Drift, Δ_a[a]

Structure	Seismic Use Group I	II	III
Structures, other than masonry shear wall or masonry wall frame structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonary cantilever shear wall structures[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 9.5.2.8 is not waived.
[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

North-South Direction

Positive Moment (due to Accidental Torsion)

Story	Maximum Allowable Drift	Drift - X dirn	Pass/Fail	Drift - Y dirn	Pass/Fail
Level 1	0.19995	0.000106	PASS	0.000174	PASS
Level 2	0.19995	0.000076	PASS	0.000269	PASS
Level 3	0.19995	0.000137	PASS	0.000311	PASS
Roof	0.30000	0.000207	PASS	0.000292	PASS

Negative Moment (due to Accidental Torsion)

Story	Maximum Allowable Drift	Drift - X dirn	Pass/Fail	Drift - Y dirn	Pass/Fail
Level 1	0.19995	0.000013	PASS	0.000019	PASS
Level 2	0.19995	0.000089	PASS	0.000053	PASS
Level 3	0.19995	0.000094	PASS	0.000059	PASS
Roof	0.30000	0.000077	PASS	0.000053	PASS

East-West Direction

Positive Moment (due to Accidental Torsion)

Story	Maximum Allowable Drift	Drift - X dirn	Pass/Fail	Drift - Y dirn	Pass/Fail
Level 1	0.19995	0.000112	PASS	0.000156	PASS
Level 2	0.19995	0.000994	PASS	0.000178	PASS
Level 3	0.19995	0.000999	PASS	0.000178	PASS
Roof	0.30000	0.000758	PASS	0.000156	PASS

Negative Moment (due to Accidental Torsion)

Story	Maximum Allowable Drift	Drift - X dirn	Pass/Fail	Drift - Y dirn	Pass/Fail
Level 1	0.19995	0.000137	PASS	0.000194	PASS
Level 2	0.19995	0.001160	PASS	0.000277	PASS
Level 3	0.19995	0.001178	PASS	0.000291	PASS
Roof	0.30000	0.000888	PASS	0.000242	PASS

Drift Due to Wind

Drift due to each of the four wind load cases was checked against the industry accepted standard of L/400. Each of the load cases passed. However, their safety margin was less than that of the seismic story drifts. It was determined that the wind load case 1 in the East-West direction is critical for drift. This makes sense because this case was also critical for member strength.

Case 1

North-South Direction

Maximum Displacement (in)	Maximum Allowable Displacement L/400 (in)	Pass/Fail
0.596817	1.8	PASS

East-West Direction

Maximum Displacement (in)	Maximum Allowable Displacement L/400 (in)	Pass/Fail
1.174024	1.8	PASS

Case 2

North-South Direction

Maximum Displacement (in)	Maximum Allowable Displacement L/400 (in)	Pass/Fail
0.581527	1.8	PASS

East-West Direction

Maximum Displacement (in)	Maximum Allowable Displacement L/400 (in)	Pass/Fail
1.057958	1.8	PASS

Case 3

North-South & East-West Direction

Maximum Displacement (in)	Maximum Allowable Displacement L/400 (in)	Pass/Fail
1.081247	1.8	PASS

Case 4

North-South & East-West Direction

Maximum Displacement (in)	Maximum Allowable Displacement L/400 (in)	Pass/Fail
0.944237	1.8	PASS

Impact on Foundation

The overturning moment due to each wind and seismic was calculated and compared to the resisting moment of the building. Because the controlling overturning moment was in the x direction and the x direction provides the smallest moment arm (thus the smallest resisting moment) for the building, only that overturning moment was checked. It was determined by computing the below calculations that the building's resisting moment is enough to resist the overturning moment induced on the building. Therefore, the foundation design would not need to be altered to resist the overturning moment. Further investigation would need to be done on the foundation to determine if it is adequate to carry the soil loads.

Overturning Moment Due to Wind

Case	Overturning Moment in X-dirn (ft-kip)	Overturning Moment in Y-dirn (ft-kip)
Wind - Case 1 - NS	4345.1	0.0
Wind - Case 1 - EW	0.0	-3594.6
Wind - Case 2 - NS	3819.2	0.0
Wind - Case 2 - EW	0.0	-3164.7
Wind - Case 3 - NS EW	3259.3	-2706.8
Wind - Case 4 - NS EW	2868.1	-2374.9

Overturning Moment Due to Seismic

Case	Overturning Moment in X-dirn (ft-kip)	Overturning Moment in Y-dirn (ft-kip)
Earthquake -- NS +moment	2021.3	0.0
Earthquake -- NS -moment	0.0	-2021.3
Earthquake -- EW +moment	0.0	-2021.3
Earthquake -- EW -moment	0.0	-2021.3

Check Overturning Moment

Controlling Overturning Moment is Due to Wind = 4345.1 ft-kip

Resisting Moment

Building Weight	6257	kip
Moment Arm	39.5	ft

Resisting Moment	247151.5	ft-kip

Factor Safety for Overturning Moment

Factor of Safety = 56.9

This is greater than the 1.5 factor of safety required by code so it passes

Conclusion

The purpose of Technical Report 4 was to determine if the lateral system of the Peggy Ryan Williams Center at Ithaca College is acceptable according to industry standard serviceability and strength considerations. Because of the problems previously discussed in this report, only the west portion of the building's lateral system was created in ETABS. That portion of the building consists of four concentrically braced steel frames and a foundation wall on the east and south sides of the building. Once the model was complete, it was verified by checking its behavior when a 100 kip load was applied in each the x direction and the y direction. Once the model was confirmed, it was used to distribute the lateral forces to each element of the lateral system.

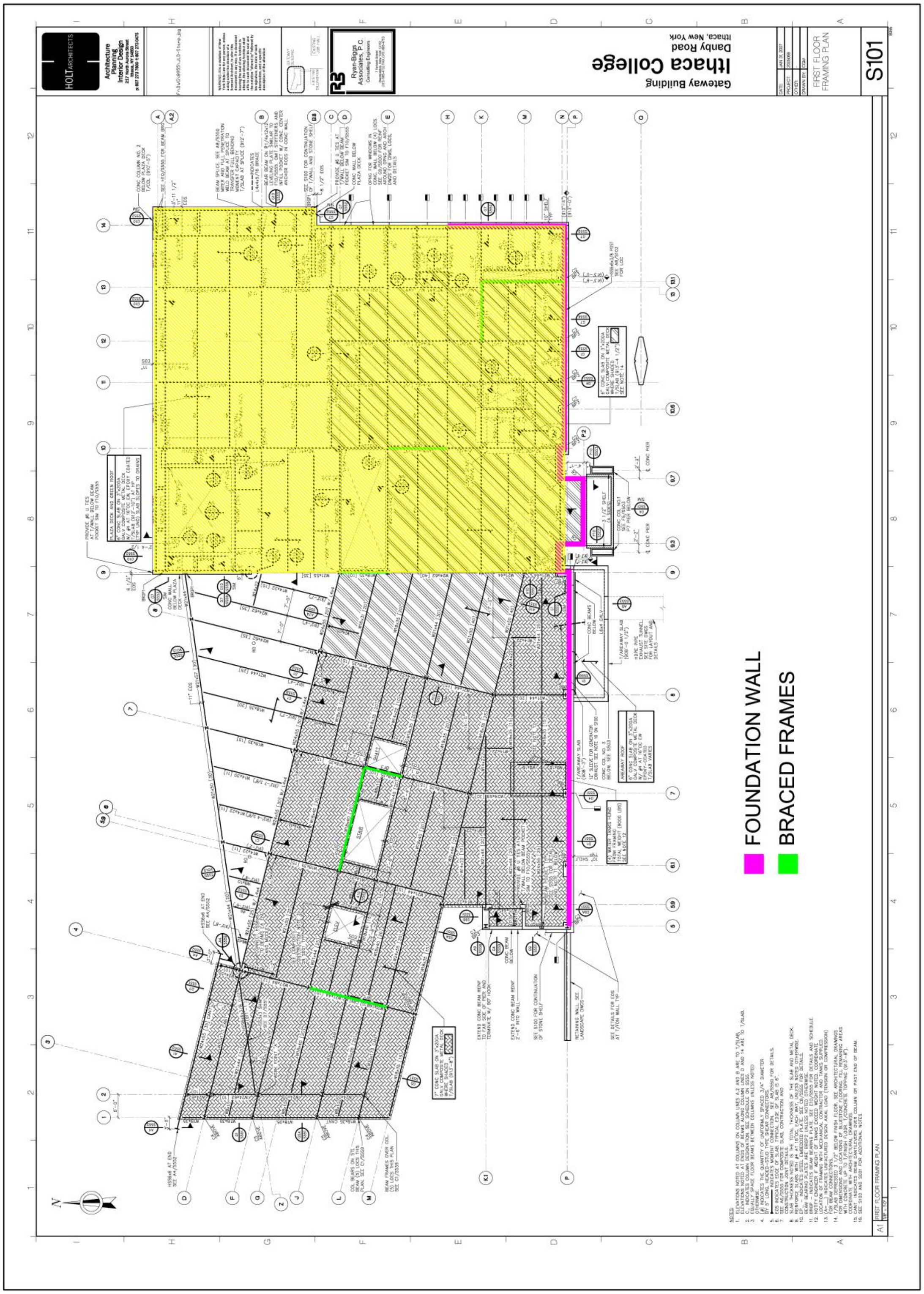
The lateral system was checked for the wind and seismic load cases as described in ASCE7-98. If was determined that the critical load case for all of the frames was one of the wind load cases. This makes sense because the building is located in Ithaca, New York. The critical case for two of the frames was wind load case 1 in the east and west direction. The remaining frames' critical load case was the wind load case 3. These results were then combined with live, dead, and snow loading results from RISA in order to conclude that the controlling load combination was 1.2 D + 1.6 W + 0.5 L + 0.5 S.

Each frame was checked at its critical level in order to determine if it has adequate strength. It was found that each member of each frame's critical level is adequate with regards to strength. Drift due to both wind and seismic loading was also checked. The lateral system was found to be adequate with respect to industry standard serviceability. Finally, overturning moments and impact on the foundation was considered. A large factor of safety for the overturning moment on the foundation was found. Therefore, the foundation was determined to be adequate. However, the foundation would also need to be checked for soil loads.

Upon completion of the lateral system analysis of the Peggy Ryan Williams Center, it has been determined that the building's lateral system is acceptable according to industry standard serviceability and strength conditions.

Appendix A

First Floor Framing Plan – Highlighting the Region of the Building Modeled and the Lateral Elements



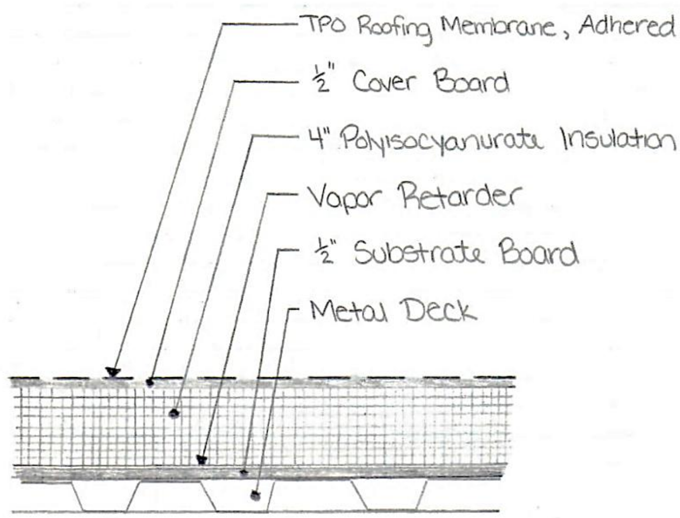
Appendix B

Gravity Loads from Technical Report 2

Angela Mincemoyer	Gravity Loads	Tech Report 2	7/43

TYPICAL ROOF BAY LOADING

Sketch of detail H5 Membrane Roofing RS-1 page A001



DEAD LOADS:

TPO Roofing membrane, adhered = 2 psf

½" cover board = 2 psf

4" Polyisocyanurate insulation = 6 psf

Vapor retarder = 1 psf

½" substrate board = 2 psf

Metal deck = 2.2 psf

Misc. & Superimposed:

mechanical equipment & piping = 5 psf
sprinklers = 10 psf
lighting = 5 psf
suspended ceiling = 3 psf
framing allowance = 10 psf

→ Total roof dead load = 48.2 psf

(25 psf roof dead load was used in design)

Angela Mincemoyer	Gravity Loads	Tech Report 2	8/43

LIVE LOADS:

per ASCE7-98 section 4.9 Minimum Roof Live Loads required the use of equation 4-2 ($L_r = 20R_1R_2$, where $12 \le L_r \le 20$).
The variable R_1 & R_2 rely on tributary areas. Because I am looking for a typical psf (I do not have a tributary area) I will be conservative and set $L_r = 20$ psf.

$\boxed{\text{(ASCE7-98) roof live load} = 20 \text{ psf}}$

- No design roof live load was provided.
  → roof snow load most likey controlled

SNOW LOADS:

uniform ground snow load (p_g) = 45 psf

$p_f = 0.7\, C_e C_t I p_g$

min $p_f = 20 \cdot I$

Exposure Factor (C_e) = 1.0
- partially exposed
- exposure B

Thermal Factor (C_t) = 1.0

Importance Factor (I) = 1.1
- category III

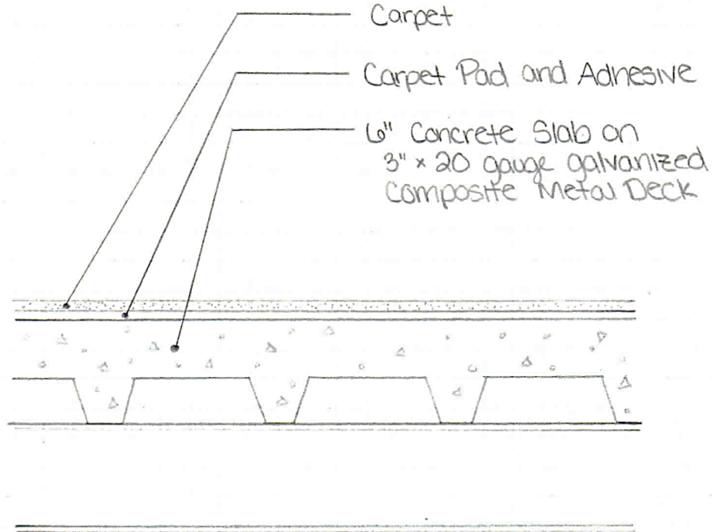
→ $p_f = 0.7(1.0)(1.0)(1.1)(45) = 34.65$ psf

check min p_f:
min $p_f = 20 \cdot I = 20(1.1) = 22$ psf < 34.65 psf ✓

→ $\boxed{p_f = 35 \text{ psf}}$

→ design uniform flat-roof snow load = 35 psf
→ the design matches the code minimum

TYPICAL FLOOR BAY LOADING:



DEAD LOADS:

Carpet = 1 psf

carpet pad & adhesive = 1.5 psf

6" concrete slab on 3" × 20 gauge galvanized composite metal deck = 57 psf

Misc & Superimposed:

mechanical equipment & piping = 5 psf
sprinklers = 10 psf
lighting = 5 psf
suspended ceiling = 3 psf
framing allowance = 10 psf

→ Total floor dead load = 92.5 psf

(for interior floors, a 80 psf dead load was used in design)

Angela Mincemoyer	Gravity Loads	Tech Report 2	10/43

LIVE LOADS:

$\boxed{\text{Live load} = 80\ psf}$

(corridors above first floor used for flexibility)

→ design interior floor live load = 80 psf

→ the design matches the code minimum

NON-TYPICAL LOADS:

Green Roof: (detail H7 - roof type RS-2)

DEAD LOADS: vegetation & planting medium = 70 psf

water retention composite = 2 psf

moisture mat = 2 psf

4" extruded polystyrene insulation = 1 psf

drainage composite = 2 psf

root barrier = 2 psf

hot fluid applied roofing = 2 psf

6" concrete slab on 3" × 20 ga galvanized composite metal deck = 57 psf (pg. 54 of Vulcraft catalog)

misc & superimposed:

same as typical roof bay = 33 psf

$\boxed{\text{Total Roof Dead Load} = 171 \text{ psf}}$

(Design dead load = 120 psf)

LIVE LOADS: $\boxed{\text{Live Load} = 100 \text{ psf}}$ (ASCE 7-98)

(Design live load = 100 psf)

Angela Mincemoyer	Gravity Loads	Tech Report 2	12/43

Roof System RS-3 : used on first floor roof (deck area)

DEAD LOADS :

2" bluestone paver = 26 psf
→ per Bluestone Guide from Braen Supply Inc)

pedestal system = 3 psf

4" extruded polystyrene insulation = 1 psf

drainage composite = 2 psf

hot liquid applied roofing = 2 psf

6" concrete slab on 3" × 20 ga = 57 psf (pg. 54 of Vulcraft catalog)
galvanized composite metal deck

misc & superimposed :

same as typical roof bay = 33 psf

$\boxed{\text{Total Roof Dead Load} = 124 \text{ psf}}$

(Design dead load = 120 psf)

LIVE LOADS :

$\boxed{\text{Live Load} = 100 \text{ psf}}$ (ASCE 7-98)

(Design live load = 100 psf)

Misc. Floors:

first floor:

DEAD LOAD: 7" concrete slab on 3" × 20 ga. galvanized composite metal deck = 69 psf (pg. 54 of Vulcraft catalog)

* this composite deck weighs 12 psf more than the typical floor bay loading.

→ For areas of 7" concrete slab on 3" × 20 ga. galvanized composite metal deck, an addition 12 psf should be added to the typical floor bay loading.

→ This will result in a total dead load = 104.5 psf

(Design dead load = 80 psf)

Interior floor with Bluestone:

DEAD LOAD: 2" bluestone paver = 26 psf
→ per Bluestone Guide from Braen Supply, Inc)

pedestal system = 3 psf

in addition to typical floor bay loading = 90 psf
(carpet, carpet pad & adhesive are not included in this load)

Total floor dead load = 119 psf

(design dead load = 120 psf)

LIVE LOAD: In areas where bluestone flooring is present, an additional 20 psf live live should be added to the typical floor bay live load. Resulting in a total floor live load of 100 psf in those areas.

The additional 20 psf is added to account for any repairs that may be required in the future, such as broken/cracked sections needing to be replaced.

(design live load = 100 psf)

Angela Mincemoyer	Gravity Load	Tech Report 2	14/43

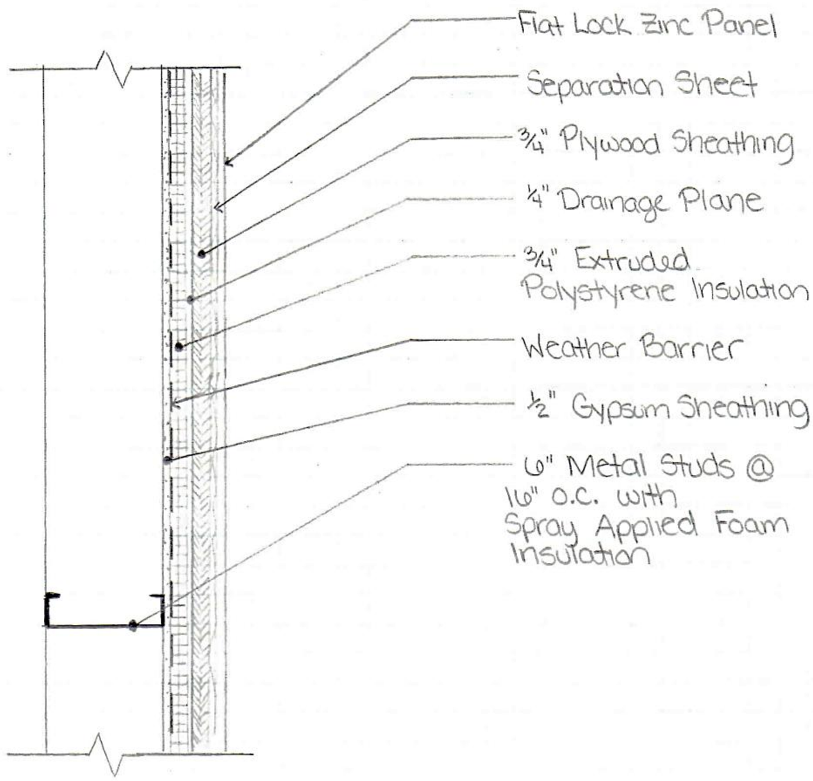
Mechanical Room:

Live load = 150 psf
(industry standard)

Stairs:

Live load = 100 psf (ASCE7-98)

TYPICAL EXTERIOR WALL DETAIL: Zinc Panels (EW-4)



DEAD LOADS:

Flat Lock Zinc Panel = 2 psf
Separation sheet = 1 psf
3/4" Plywood Sheathing = 2.4 psf (ASCE 7-10)
3/4" Extruded Polystyrene Insulation = 0.5 psf
Weather Barrier = 1 psf
1/2" gypsum sheathing = 2 psf (ASCE 7-10)
6" metal studs @ 16" o.c. = 4 psf
Spray foam insulation = 1 psf

$\boxed{\text{Total dead load} = 13.9 \text{ psf}}$

Angela Mincemoyer	Gravity Load	Tech Report 2	16/43

LOAD PATH DESCRIPTION:

The exterior wall facade load is carried by a grid of 6" metal studs @ 16" O.C. The load is first transferred to the horizontal 6" metal studs and then into the vertical 6" metal studs. The metal studs then transfer the load into the foundation.

Adjusted Gravity Loads from Technical Report 2 Amendment

Angela Mincemoyer | Adjusted Gravity Load | Tech Report 2

Misc. & Superimposed:

sprinklers = 5 psf

→ Total roof dead load = 43.2 psf (typical)

→ Total floor dead load = 87.5 psf (typical)

A. Mincemoyer | Gravity Load | Tech Report 2 Amendment

EXTERIOR WALL LOADS:

Zinc Panels (EW-4)
from page 15 of Tech Report 2
→ total dead load = 13.9 psf

Aluminum Storefronts

EFCO Corporation's System 960 Wall

→ total dead load = 12.0 psf

Composit Aluminum Panel (EW-3)

Composite aluminum wall panel = 2 psf
weather barrier = 1 psf
3/4" plywood sheathing = 2.4 psf (ASCE 7-10)
6" metal studs @ 16" o.c. = 4 psf
Spray foam insulation = 1 psf
5/8" gypsum board = 2.5 psf

→ total dead load = 12.9 psf

Limestone Panel (EW-2)

1¼" limestone panel = 15 psf
3/4" extruded polystyrene insulation = 0.5 psf
weather barrier = 1 psf
½" gypsum sheathing = 2 psf (ASCE 7-10)
Stainless steel stone anchor = 2 psf
6" metal studs @ 16" o.c. = 4 psf
Spray foam insulation = 1 psf
5/8" Gypsum board = 2.5 psf

→ total dead load = 28 psf

Blue Stone Veneer (EW-1)

5" blue stone veneer = (160 pcf)(5/12) = 67 psf
→ Common Wealth Curb Appeal - Bluestone Guide
1" cavity drainage mat = 2 psf
3" extruded polystyrene insulation = 2 psf
8" concrete foundation wall = (150 pcf)(8/12) = 100 psf
1½" polyisocyanurate insulation = 2 psf
5/8" gypsum board = 2.5 psf
sheet water proofing = 1 psf

→ total dead load = 176.5 psf

Appendix C

Wind Loads from Technical Report 2

Angela Mincemoyer	Wind Load	Tech Report 2	20/43

Per ASCE7-98 Chapter 6:
METHOD 1: BCNYS Section 1609.6

- according to IBC 2000 section 1609.6, the mean roof height must not exceed the least horizontal dimension, in order to use the simplified method

→ least horizontal dimension ≈ 29 ft

→ mean roof height > 29 ft

⇒ Method 1 (simplified method) may not be used

METHOD 3:

- wind tunnel tests must be completed in order to determine design wind loads

⇒ Method 3 is not feasible for this report

METHOD 2:

- I will use this method to calculate the wind loads on the Peggy Ryan Williams Center.

(assuming no irregular geometries)

Angela Mincemoyer	Wind Load	Tech Report 2	21/43

ASCE7-98: Section 6.5.3 - Design Procedure

STEPS:

#1

Basic Wind speed, V (Per Fig 6-1) → V = 90 mph

wind directionality factor, Kd

→ Buildings → Main Wind Force Resisting System (Per table 6-6) → Kd = 0.85

#2 Importance Factor, I

per Table 1-1 → Category III
- "Buildings or other structures with a capacity greater than 500 for colleges or adult education facilities"

per Table 6-1 → I = 1.15

#3

Exposure Category

per section 6.5.6.1 → Exposure B

Velocity Pressure Exposure Coefficient

per Table 6-5:

mean roof height > least horizontal dimension → NOT a low-rise building

→ must use case 2

North-South Direction - Main Roof: (Table 6-5)

Garden Level:

$z = 4 \text{ ft} \rightarrow K_{z_G} = 0.57$

Level 1:

$z = 17.25 \text{ ft} \rightarrow K_{z_1} = 0.5925$

Level 2:

$z = 30.5 \text{ ft} \rightarrow K_{z_2} = 0.703$

Level 3:

$z = 43.75 \text{ ft} \rightarrow K_{z_3} = 0.779$

Roof:

$z = 65 \text{ ft} \rightarrow K_{z_R} = 0.87$

North-South Direction - Atrium: (Table 6-5)

Garden Level: $K_{z_G} = 0.57$

Level 1: $K_{z_1} = 0.5925$

Level 2: $K_{z_2} = 0.703$

Level 3: $K_{z_3} = 0.779$

Roof:

$z = 70 \text{ ft} \rightarrow K_{z_R} = 0.89$

East-West Direction - Main Roof: (Table 6-5)

Garden Level:

$z = 2.25\text{ ft} \rightarrow K_{z_G} = 0.57$

Level 1:

$z = 15.5\text{ ft} \rightarrow K_{z_1} = 0.5725$

Level 2:

$z = 28.75\text{ ft} \rightarrow K_{z_2} = 0.69$

Level 3:

$z = 42\text{ ft} \rightarrow K_{z_3} = 0.77$

Roof:

$z = 60\text{ ft} \rightarrow K_{z_R} = 0.85$

East-West Direction - Atrium: (Table 6-5)

Garden Level: $K_{z_G} = 0.57$

Level 1: $K_{z_1} = 0.5725$

Level 2: $K_{z_2} = 0.69$

Level 3: $K_{z_3} = 0.77$

Roof:

$z = 67\text{ ft} \rightarrow K_{z_R} = 0.878$

#4 Topographic Factor, K_{zt} (section 6.5.7.2)

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

→ section 6.5.7.1
$H < 60$ ft → $K_{zt} = 1.0$

#5 Gust Effect Factor, G

- ASCE 7-98 does not have a specific formula for the fundamental frequency
- I will estimate the fundamental frequency by setting it equal to $1/T_a$
  - where T_a = approximate fundamental period (per section 9.5.3.3)

$$T_a = C_T h_n^{3/4} \quad \text{(eqn 9.5.3.3-1)}$$

$C_T = 0.02$
$h_n = 70$ ft

$$T_a = (.02)(70)^{3/4} = 0.484$$

$$\rightarrow n_1 = 1/T_a = \frac{1}{0.484} \quad \rightarrow n_1 = 2.07 \text{ Hz}$$

$n_1 = 2.07$ Hz > 1.0 Hz → Rigid Building
(per 6.2 Definitions)

Section 6.5.8.1 Rigid Structures:

$$G = 0.925\left(\frac{(1 + 1.7 g_Q I_{\bar{z}} Q)}{1 + 1.7 g_v I_{\bar{z}}}\right) \quad \text{(eqn 6-2)}$$

$$I_{\bar{z}} = c(33/\bar{z})^{1/6} \quad \text{(eqn 6-3)}$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad \text{(eqn 6-4)}$$

$$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}} \quad \text{(eqn 6-5)}$$

Angela Mincemoyer | Wind Load | Tech Report 2 | 25/43

North-South Direction - Main Roof :

$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\varepsilon}}$

$\bar{z} = 0.6h > z_{min}$
$= 0.6(65) = 39 > 30 \checkmark \rightarrow \bar{z} = 39$

$\ell = 320$, $\bar{\varepsilon} = 1/3$ } Table 6-4

$\rightarrow L_{\bar{z}} = 320\left(\frac{39}{33}\right)^{1/3} \rightarrow L_{\bar{z}} = 338.32$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$ B = 245 ft, h = 65 ft

$= \sqrt{\frac{1}{1+0.63\left(\frac{245+65}{338.32}\right)^{0.63}}} \rightarrow Q = 0.791$

$I_{\bar{z}} = c(33/\bar{z})^{1/6}$ C = 0.30 (Table 6-4), $\bar{z} = 39$

$= (0.30)\left(33/39\right)^{1/6}$

$\rightarrow I_{\bar{z}} = 0.292$

$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right)$ $g_Q = 3.4$, $g_v = 3.4$ } Section 6.5.8.1

$= 0.925\left(\frac{(1+1.7(3.4)(0.292)(0.791))}{1+1.7(3.4)(0.292)}\right)$

$\rightarrow G = 0.804$

Angela Mincemoyer	Wind Load	Tech Report 2	26/43

North-South Direction- Atrium:

$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}}$

$\bar{z} = 0.6h > z_{min}$
$= 0.6(70) = 42 > 30$ ✓ $\rightarrow \bar{z} = 42$

$\ell = 320$, $\bar{\epsilon} = 1/3$ } Table 6-4

$\rightarrow L_{\bar{z}} = 320(42/33)^{1/3}$ $\rightarrow L_{\bar{z}} = 346.79$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1+0.63\left(\frac{245+70}{346.79}\right)^{0.63}}} \rightarrow Q = 0.792$$

$I_{\bar{z}} = c(33/\bar{z})^{1/6}$ $c = 0.30$ (Table 6-4), $\bar{z} = 42$

$= (0.30)(33/42)^{1/6}$

$\rightarrow I_{\bar{z}} = 0.288$

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right)$$

$g_Q = 3.4$, $g_v = 3.4$ } section 6.5.8.1

$$= 0.925\left(\frac{(1+1.7(3.4)(.288)(.792))}{1+1.7(3.4)(.288)}\right)$$

$\rightarrow G = 0.805$

Angela Mincemoyer | Wind Load | Tech Report 2 | 27/43

East-West Direction - Main Roof:

$$L_{\bar{z}} = \ell (\bar{z}/33)^{\bar{\epsilon}}$$

$$\bar{z} = 0.6h > z_{min}$$
$$= 0.6(60) = 36 > 30 \checkmark \quad \rightarrow \bar{z} = 36$$

$$\left.\begin{array}{l} \ell = 320 \\ \bar{\epsilon} = 1/3 \end{array}\right\} \text{Table 6-4}$$

$$\rightarrow L_{\bar{z}} = 320(36/33)^{1/3} \quad \rightarrow L_{\bar{z}} = 329.42$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1+0.63\left(\frac{110+60}{329.42}\right)^{0.63}}} \quad \rightarrow Q = 0.841$$

$$I_{\bar{z}} = C(33/\bar{z})^{1/6} \qquad \left.\begin{array}{l} C = 0.30 \\ \bar{z} = 36 \end{array}\right. \text{(Table 6-4)}$$

$$= (0.30)(33/36)^{1/6}$$

$$\rightarrow I_{\bar{z}} = 0.296$$

$$G = 0.925\left(\frac{(1+1.7 g_Q I_{\bar{z}} Q)}{1+1.7 g_v I_{\bar{z}}}\right) \qquad \left.\begin{array}{l} g_Q = 3.4 \\ g_v = 3.4 \end{array}\right\} \text{section 6.5.8.1}$$

$$= 0.925\left(\frac{(1+1.7(3.4)(.296)(.841))}{1+1.7(3.4)(.296)}\right)$$

$$\rightarrow G = 0.832$$

Angela Mincemoyer	Wind Load	Tech Report 2	28/43

East-West Direction - Atrium:

$$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\varepsilon}}$$

$$\bar{z} = 0.6h > z_{min}$$
$$= 0.6(67) = 40.2 > 30 \checkmark \quad \rightarrow \bar{z} = 40.2$$

$$\left.\begin{aligned}\ell &= 320\\ \bar{\varepsilon} &= 1/3\end{aligned}\right\} \text{Table 6-4}$$

$$\rightarrow L_{\bar{z}} = 320(40.2/33)^{1/3} \quad \rightarrow \quad L_{\bar{z}} = 341.76$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1+0.63\left(\frac{110+67}{341.76}\right)^{0.63}}} \quad \rightarrow Q = 0.840$$

$$I_{\bar{z}} = c(33/\bar{z})^{1/6}$$

$$= (0.30)(33/40.2)^{1/6} \qquad c = 0.30 \text{ (Table 6-4)}, \quad \bar{z} = 40.2$$

$$\rightarrow I_{\bar{z}} = 0.290$$

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right)$$

$$\left.\begin{aligned}g_Q &= 3.4\\ g_v &= 3.4\end{aligned}\right\} \text{section 6.5.8.1}$$

$$= 0.925\left(\frac{(1+1.7(3.4)(.290)(.840))}{1+1.7(3.4)(.290)}\right)$$

$$\rightarrow G = 0.832$$

#6 Enclosure Classification

→ enclosed per Section 6.5.9

#7 Internal pressure coefficient GC_{pi}

$GC_{pi} = +0.18$ / -0.18 per section 6.5.11.1 Table 6-7

#8 External pressure coefficient, C_p per Section 6.5.11.2.1

per Fig. 6-3: North-South

$\frac{L}{B} = \frac{110}{245} = 0.45$ → windward wall $C_p = 0.8$ (use with q_z)
leeward wall $C_p = -0.5$ (use with q_h)

East West:

$\frac{L}{B} = \frac{245}{110} = 2.2$ → windward wall $C_p = 0.8$ (use with q_z)
leeward wall $C_p = -0.29$ (use with q_h)
↑ found using linear interpolation

#9 Velocity Pressure, q_z, q_h per section 6.5.10 eqn 6-13

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (lb/ft^2)

North-South Direction - Main Roof:

Garden Level:

$q_{z_G} = 0.00256(0.57)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_G} = 11.55$ psf

Level 1:

$q_{z_1} = 0.00256(0.5925)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_1} = 12.01$ psf

Level 2:

$q_{z_2} = 0.00256(0.703)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_2} = 14.25$ psf

Level 3:

$$q_{z_3} = 0.00256(0.779)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_3} = 15.79 \text{ psf}$$

Roof:

$$q_{z_R} = 0.00256(0.87)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_R} = 17.03 \text{ psf} = q_h$$

North-South Direction - Atrium:

Garden Level:

$$q_{z_G} = 11.55 \text{ psf}$$

Level 1:

$$q_{z_1} = 12.01 \text{ psf}$$

Level 2:

$$q_{z_2} = 14.25 \text{ psf}$$

Level 3:

$$q_{z_3} = 15.79 \text{ psf}$$

Roof:

$$q_{z_R} = 0.00256(0.89)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_R} = 18.04 \text{ psf} = q_h$$

East-West Direction - Main Roof:

Garden Level:

$$q_{z_G} = 0.00256(0.57)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_G} = 11.55 \text{ psf}$$

Level 1:

$$q_{z_1} = 0.00256(0.5725)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_1} = 11.60 \text{ psf}$$

Level 2:

$$q_{z_2} = 0.00256(0.69)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_2} = 13.99 \text{ psf}$$

Level 3:

$$q_{z_3} = 0.00256(0.77)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_3} = 15.61 \text{ psf}$$

Roof:

$$q_{z_R} = 0.00256(0.85)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_R} = 17.23 \text{ psf} = q_h$$

East - West Direction - Atrium:

Garden Level:

$$q_{z_G} = 11.55 \text{ psf}$$

Level 1:

$$q_{z_1} = 11.60 \text{ psf}$$

Level 2:

$$q_{z_2} = 13.99 \text{ psf}$$

Level 3:

$$q_{z_3} = 15.61 \text{ psf}$$

Roof:

$$q_{z_R} = 0.00256(.878)(1.0)(0.85)(90^2)(1.15) \rightarrow q_{z_R} = 17.80 \text{ psf} = q_h$$

#10 - Design Wind Load, P per section 6.5.12.2.1

see excel pages that follow

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Design Wind Load, P - per section 6.5.12.2.1

$p = qGC_p$ (psf)

North-South Direction - Main Roof:

		q	*	G	*	C_p	=	p (psf)	*	Area (sf)	=	Force (k)
WINDWARD	Garden Level	11.55	*	0.804	*	0.8	=	7.43	*	1942	=	14.4
	Level 1	12.01	*	0.804	*	0.8	=	7.72	*	2986	=	23.1
	Level 2	14.25	*	0.804	*	0.8	=	9.17	*	2986	=	27.4
	Level 3	15.79	*	0.804	*	0.8	=	10.16	*	3734	=	37.9
	Roof	17.63	*	0.804	*	0.8	=	11.34	*	2240	=	25.4
LEEWARD	Garden Level	17.63	*	0.804	*	-0.5	=	-7.09	*	1942	=	-13.8
	Level 1	17.63	*	0.804	*	-0.5	=	-7.09	*	2986	=	-21.2
	Level 2	17.63	*	0.804	*	-0.5	=	-7.09	*	2986	=	-21.2
	Level 3	17.63	*	0.804	*	-0.5	=	-7.09	*	3734	=	-26.5
	Roof	17.63	*	0.804	*	-0.5	=	-7.09	*	2240	=	-15.9

Wind Load Base Shear

	Force (k)
Garden Level	28.2
Level 1	44.2
Level 2	48.5
Level 3	64.4
Roof	41.3
Total	226.6

North-South Direction - Atrium:

		q	*	G	*	C_p	=	p (psf)	*	Area (sf)	=	Force (k)
WINDWARD	Garden Level	11.55	*	0.805	*	0.8	=	7.44	*	182	=	1.4
	Level 1	12.01	*	0.805	*	0.8	=	7.73	*	280	=	2.2
	Level 2	14.25	*	0.805	*	0.8	=	9.18	*	280	=	2.6
	Level 3	15.79	*	0.805	*	0.8	=	10.17	*	455	=	4.6
	Roof	18.04	*	0.805	*	0.8	=	11.62	*	315	=	3.7
LEEWARD	Garden Level	18.04	*	0.805	*	-0.5	=	-7.26	*	182	=	-1.3
	Level 1	18.04	*	0.805	*	-0.5	=	-7.26	*	280	=	-2.0
	Level 2	18.04	*	0.805	*	-0.5	=	-7.26	*	280	=	-2.0
	Level 3	18.04	*	0.805	*	-0.5	=	-7.26	*	455	=	-3.3
	Roof	18.04	*	0.805	*	-0.5	=	-7.26	*	315	=	-2.3

Wind Load Base Shear

	Force (k)
Garden Level	2.7
Level 1	4.2
Level 2	4.6
Level 3	7.9
Roof	5.9
Total	25.4

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East-West Direction - Main Roof:

		q	*	G	*	C_p	=	p (psf)	*	Area (sf)	=	Force (k)
WINDWARD	Garden Level	11.55	*	0.832	*	0.8	=	7.69	*	667	=	5.1
	Level 1	11.6	*	0.832	*	0.8	=	7.72	*	1333	=	10.3
	Level 2	13.99	*	0.832	*	0.8	=	9.31	*	1333	=	12.4
	Level 3	15.61	*	0.832	*	0.8	=	10.39	*	1697	=	17.6
	Roof	17.23	*	0.832	*	0.8	=	11.47	*	1030	=	11.8
LEEWARD	Garden Level	17.23	*	0.832	*	-0.29	=	-4.16	*	667	=	-2.8
	Level 1	17.23	*	0.832	*	-0.29	=	-4.16	*	1333	=	-5.5
	Level 2	17.23	*	0.832	*	-0.29	=	-4.16	*	1333	=	-5.5
	Level 3	17.23	*	0.832	*	-0.29	=	-4.16	*	1697	=	-7.1
	Roof	17.23	*	0.832	*	-0.29	=	-4.16	*	1030	=	-4.3

Wind Load Base Shear

	Force (k)
Garden Level	7.9
Level 1	15.8
Level 2	18.0
Level 3	24.7
Roof	16.1
Total	82.5

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East-West Direction - Atrium:

		q	*	G	*	C_p	=	p (psf)	*	Area (sf)	=	Force (k)
WINDWARD	Garden Level	11.55	*	0.832	*	0.8	=	7.69	*	67	=	0.5
	Level 1	11.6	*	0.832	*	0.8	=	7.72	*	133	=	1.0
	Level 2	13.99	*	0.832	*	0.8	=	9.31	*	133	=	1.2
	Level 3	15.61	*	0.832	*	0.8	=	10.39	*	217	=	2.3
	Roof	17.8	*	0.832	*	0.8	=	11.85	*	150	=	1.8
LEEWARD	Garden Level	17.8	*	0.832	*	-0.29	=	-4.29	*	67	=	-0.3
	Level 1	17.8	*	0.832	*	-0.29	=	-4.29	*	133	=	-0.6
	Level 2	17.8	*	0.832	*	-0.29	=	-4.29	*	133	=	-0.6
	Level 3	17.8	*	0.832	*	-0.29	=	-4.29	*	217	=	-0.9
	Roof	17.8	*	0.832	*	-0.29	=	-4.29	*	150	=	-0.6

Wind Load Base Shear

	Force (k)
Garden Level	0.8
Level 1	1.6
Level 2	1.8
Level 3	3.2
Roof	2.4
Total	9.8

Roof Uplifts: - per Figure 6-3

$p = qGC_p$ (psf)

North-South Direction - Main Roof: ***h = 65*** h/L = 65/110 = 0.591

	q	*	G	*	C_p	=	p (psf)
0 to h/2	17.63	*	0.804	*	-0.925	=	-13.11
h/2 to h	17.63	*	0.804	*	-0.864	=	-12.25
h to 2h	17.63	*	0.804	*	-0.536	=	-7.60

North-South Direction - Atrium: ***h = 70*** h/L = 70/110 = 0.636

	q	*	G	*	C_p	=	p (psf)
0 to h/2	18.04	*	0.805	*	-0.988	=	-14.35
h/2 to h	18.04	*	0.805	*	-0.846	=	-12.29

East-West Direction - Main Roof: ***h = 60*** h/L = 60/245 = .245

	q	*	G	*	C_p	=	p (psf)
0 to h/2	17.23	*	0.832	*	-0.9	=	-12.90
h/2 to h	17.23	*	0.832	*	-0.9	=	-12.90
h to 2h	17.23	*	0.832	*	-0.5	=	-7.17
>2h	17.23	*	0.832	*	-0.3	=	-4.30

East-West Direction - Atrium: ***h = 67*** h/L = 67/245 = .273

	q	*	G	*	C_p	=	p (psf)
h/2 to h	17.8	*	0.832	*	-0.9	=	-13.33
h to 2h	17.8	*	0.832	*	-0.5	=	-7.40

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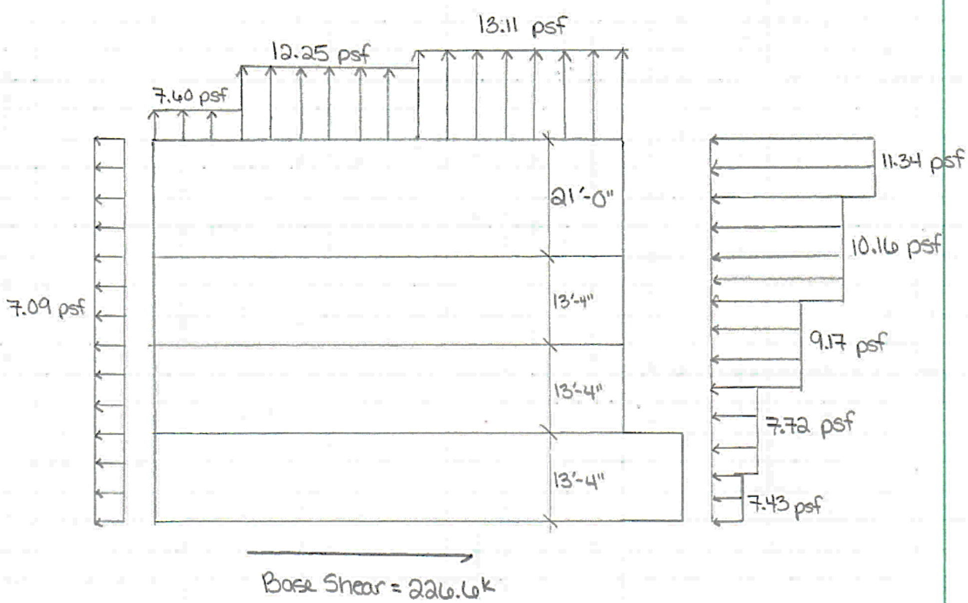
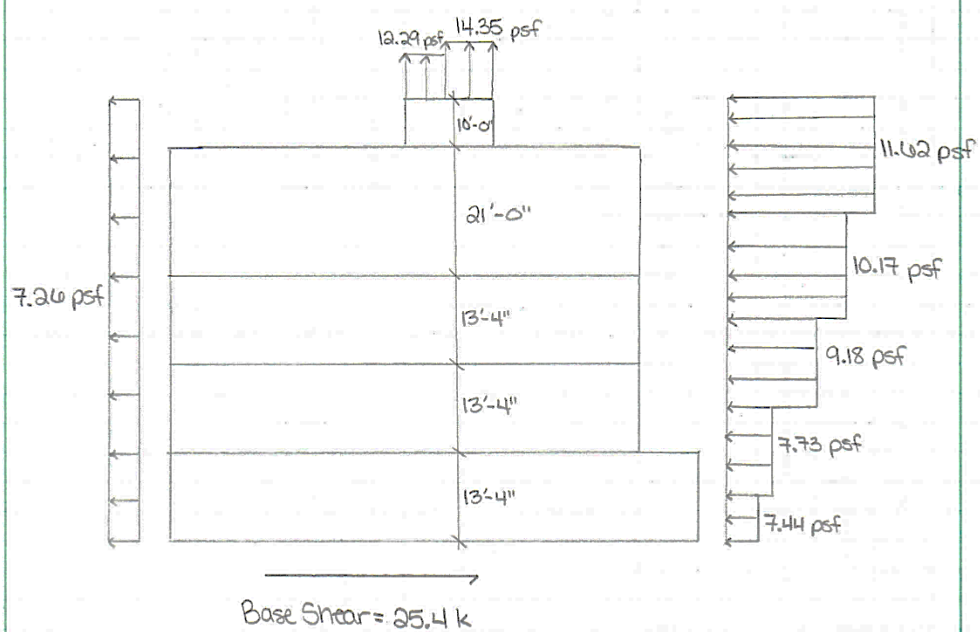
North-South Direction - Main Roof:



North-South Direction- Atrium:



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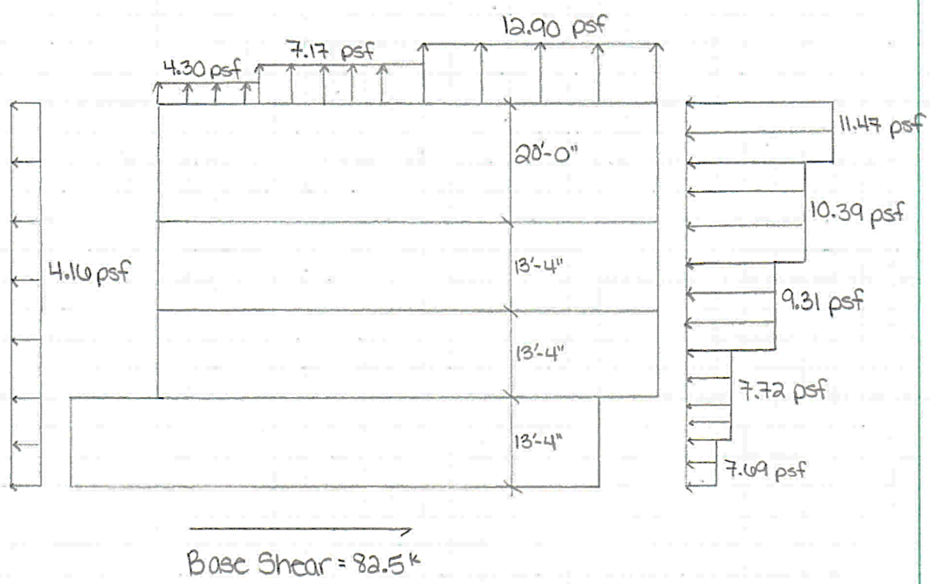
East West Direction - Main Roof



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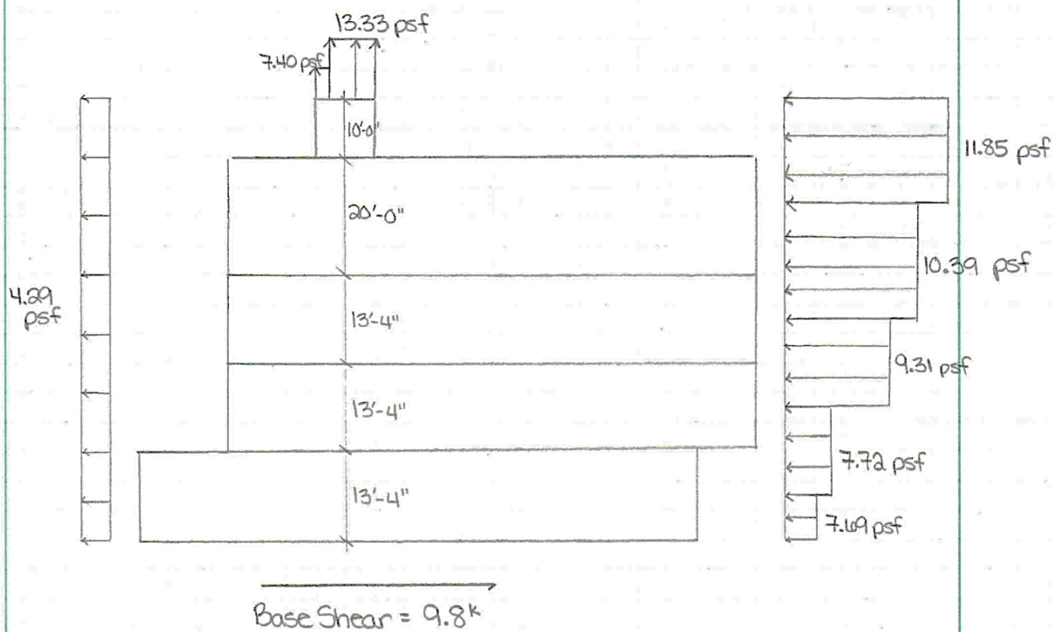
East West Direction - Atrium:



Adjusted Wind Loads from Technical Report 2 Amendment

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Atrium:

Per ASCE 7-98 Section 6.3

A_o = total area of openings in a wall that receives positive external pressure

- openings are defined as apertures or holes in the building envelope which allow air to flow through the building envelope...

- The only opening in my building is the atrium roof. Therefore, $A_o = 0$ for the PRWC.
  → thus classifying the building as enclosed

- In order to account for the atrium opening, I will calculate the wind pressure on the atrium wall above the roof using external pressure and internal suction. (using the GC_{pi} calculated on page 29)

$p = qGC_p - q_i(GC_{pi})$

North-South Direction - Atrium:

		q	*	G	*	C_p	)	-	(	q_i	*	GC_{pi}	)	=	p (psf)	*	Area (sf)	=	Force (k)
WINDWARD	Garden Level	11.55	*	0.805	*	0.8	)	-	(	-	*	-	)	=	7.44	*	182	=	1.4
	Level 1	12.01	*	0.805	*	0.8	)	-	(	-	*	-	)	=	7.73	*	280	=	2.2
	Level 2	14.25	*	0.805	*	0.8	)	-	(	-	*	-	)	=	9.18	*	280	=	2.6
	Level 3	15.79	*	0.805	*	0.8	)	-	(	-	*	-	)	=	10.17	*	455	=	4.6
	Roof	18.04	*	0.805	*	0.8	)	-	(	18.04	*	-0.18	)	=	14.86	*	315	=	4.7
LEEWARD	Garden Level	18.04	*	0.805	*	-0.5	)	-	(	-	*	-	)	=	-7.26	*	182	=	-1.3
	Level 1	18.04	*	0.805	*	-0.5	)	-	(	-	*	-	)	=	-7.26	*	280	=	-2.0
	Level 2	18.04	*	0.805	*	-0.5	)	-	(	-	*	-	)	=	-7.26	*	280	=	-2.0
	Level 3	18.04	*	0.805	*	-0.5	)	-	(	-	*	-	)	=	-7.26	*	455	=	-3.3
	Roof	18.04	*	0.805	*	-0.5	)	-	(	18.04	*	0.18	)	=	-10.51	*	315	=	-3.3

Wind Load Base Shear

	Force (k)
Garden Level	2.7
Level 1	4.2
Level 2	4.6
Level 3	7.9
Roof	8.0
Total	27.4

East-West Direction - Atrium:

		q	*	G	*	C_p	) - (	q_i	*	GC_{pi}	)	=	p (psf)	*	Area (sf)	=	Force (k)
WINDWARD	Garden Level	11.55	*	0.832	*	0.8	) - (	-	*	-	)	=	7.69	*	67	=	0.5
	Level 1	11.6	*	0.832	*	0.8	) - (	-	*	-	)	=	7.72	*	133	=	1.0
	Level 2	13.99	*	0.832	*	0.8	) - (	-	*	-	)	=	9.31	*	133	=	1.2
	Level 3	15.61	*	0.832	*	0.8	) - (	-	*	-	)	=	10.39	*	217	=	2.3
	Roof	17.8	*	0.832	*	0.8	) - (	17.8	*	-0.18	)	=	15.05	*	150	=	2.3
LEEWARD	Garden Level	17.8	*	0.832	*	-0.29	) - (	-	*	-	)	=	-4.29	*	67	=	-0.3
	Level 1	17.8	*	0.832	*	-0.29	) - (	-	*	-	)	=	-4.29	*	133	=	-0.6
	Level 2	17.8	*	0.832	*	-0.29	) - (	-	*	-	)	=	-4.29	*	133	=	-0.6
	Level 3	17.8	*	0.832	*	-0.29	) - (	-	*	-	)	=	-4.29	*	217	=	-0.9
	Roof	17.8	*	0.832	*	-0.29	) - (	17.8	*	0.18	)	=	-7.50	*	150	=	-1.1

Wind Load Base Shear

	Force (k)
Garden Level	0.8
Level 1	1.6
Level 2	1.8
Level 3	3.2
Roof	3.4
Total	10.8

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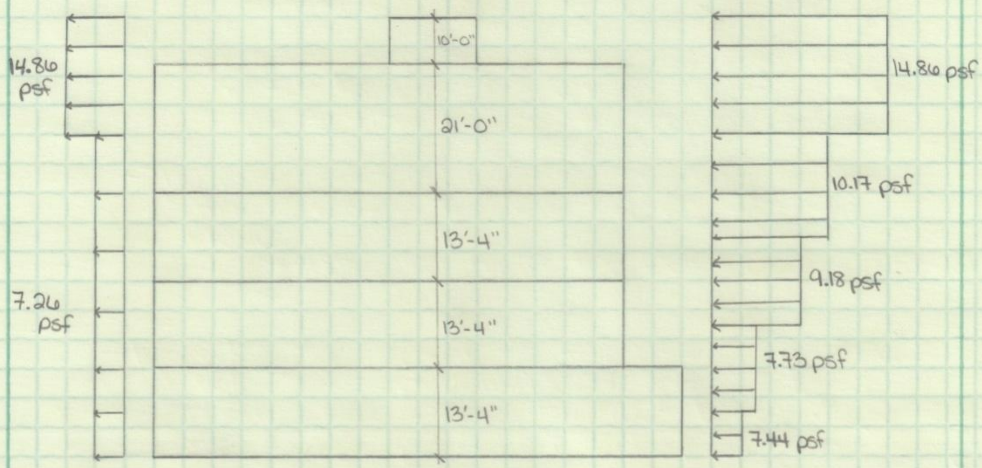
North-South Direction - Atrium:



Base Shear = 27.4 k

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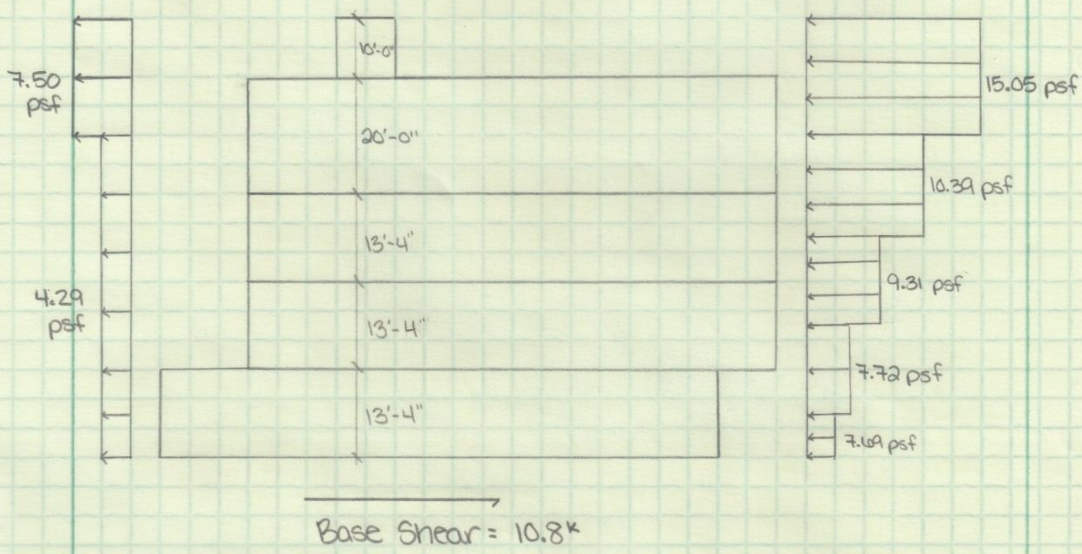
East-West Direction - Atrium:



Appendix D

Seismic Loads from Technical Report 2

Angela Mincemoyer	Seismic Load	Tech Report 2	41/43

#1 Exempt?

Occupancy Category : III → per Table 9.1.3
Seismic Use Group = II

Site Class : B

$S_s = 18.7\%g \rightarrow 0.187\,g$
↳ per Figure 9.4.1.1 (a)

$S_1 = 6.3\%g \rightarrow 0.063\,g > 0.04g$ → not exempt
↳ per Figure 9.4.1.1 (b)

#2 Occupancy Importance Factor
$I = 1.25$ → per Table 9.1.4

#3 Adjust for site class: → per Section 9.4.1.2.4

$S_{MS} = F_a S_s$ $F_a = 1.0$ (Table 9.4.1.2.4a)

$S_{MS} = (1.0)(0.187) \rightarrow S_{MS} = 0.187$

$S_{M1} = F_v S_1$ $F_v = 1.0$ (Table 9.4.1.2.4b)

$S_{M1} = (1.0)(0.063) \rightarrow S_{M1} = 0.063$

#4 Spectral Response Acceleration Parameters: (Section 9.4.1.2.5)

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{DS} = \frac{2}{3}(0.187) \rightarrow S_{DS} = 0.125$

$S_{D1} = \frac{2}{3} S_{M1}$

$S_{D1} = \frac{2}{3}(0.063) \rightarrow S_{D1} = 0.042$

#5 Seismic Design Category (Table 9.4.2.1a)

Seismic Design Category A

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#6 Select Procedure:

Due to the PRWC classifying as seismic design category A, per section 9.5.2.5.1:

the building "shall be analyzed for minimum lateral forces given by eqn. 9.5.2.5.1, applied independently in each of two orthogonal directions"

eqn 9.5.2.5.1 $F_x = 0.01\ W_x$

per Table 9.5.2.2

Structural Steel Systems Not Specifically Detailed for Seismic Resistance

$R = 3$

	Area (SF)	W (psf)	W_x (K)
Level 1			
typ. Floor	14,682	92.5	1,358
Green Roof	3,157	171	540
Deck	3,942	124	489
Level 2			
typ. Floor	15,257	92.5	1,412
Level 3			
typ. Floor	12,785	92.5	1,183
Green Roof	2,899	171	496
Roof			
typ Roof	15,936	48.2	769
Atrium			
typ Roof	204	48.2	10

* W - per section 9.5.3.2 is the dead load only
- I will use the previously calculated typical floor bay loading, typical roof bay, green roof, and deck area loadings.

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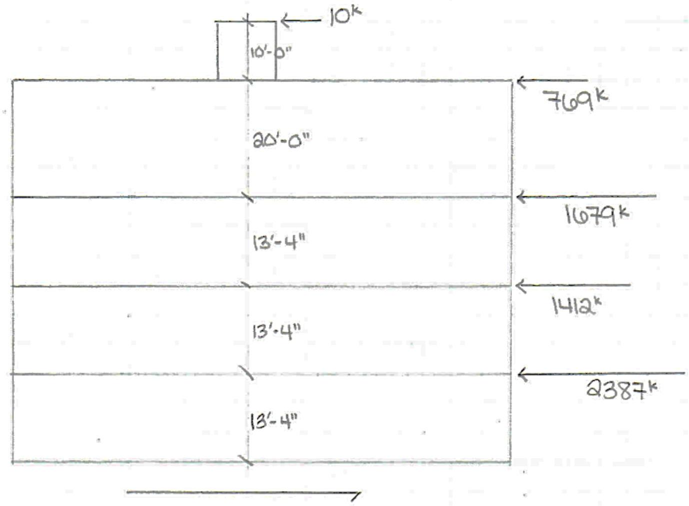
$W_1 = 1358 + 540 + 489 \rightarrow W_1 = 2387^k$

$W_2 = 1412^k$

$W_3 = 1183 + 496 \rightarrow W_3 = 1679^k$

$W_R = 769^k$ $\quad W_{R\,atrium} = 10^k$

North-South and East-West Seismic Forces:



Base Shear = 6257^k

Adjusted Seismic Loads from Technical Report 2 Amendment

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* continuation of table on page 42 *

$F_x = 0.01 W_x$

	W_x (K)	F_x (K)
Level 1		
typ Floor	1358	13.58
Green Roof	540	5.4
Deck	489	4.89
Level 2		
typ floor	1412	14.12
Level 3		
typ floor	1183	11.83
Green Roof	496	4.96
Roof		
typ Roof	769	7.69
Atrium		
typ Roof	10	0.10

$W_1 = 13.58 + 5.4 + 4.89 \rightarrow W_1 = 23.87^k$

$W_2 = 14.12^k$

$W_3 = 11.83 + 4.96 \rightarrow W_3 = 16.79^k$

$W_R = 7.69^k$

$W_{Atrium} = 0.10^k$

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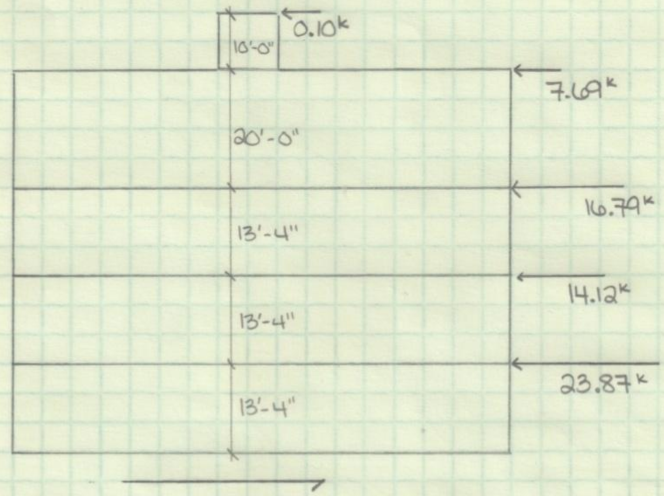
North-South and East-West Seismic Forces:



Base Shear = 62.59k

Seismic Forces Due to Exterior Walls

	Elevation	Zinc Panel Area (SF)	Aluminum Storefront Area (SF)	Aluminum Panel Area (SF)	Limestone Panel Area (SF)	Bluestone Veneer Area (SF)	Wx (kip)	Fx (kip)
Level 1								
	North	650.0	1766.0	674.8	0.0	32.5	44.7	0.447
	East	485.5	124.9	303.4	203.3	252.9	62.5	0.625
	South	538.7	634.2	307.2	454.3	85.0	46.8	0.468
	West	260.8	482.3	0.0	207.8	134.8	39.0	0.390
							w_1 =	1.9
Level 2								
	North	650.0	1569.2	371.9	490.8	0.0	46.4	0.5
	East	0.0	486.3	110.5	711.0	0.0	27.2	0.3
	South	502.1	1978.3	392.4	190.8	0.0	41.1	0.4
	West	271.0	482.3	0.0	342.6	0.0	19.1	0.2
							w_2 =	1.3
Level 3								
	North	791.9	1147.6	849.6	939.4	0.0	62.0	0.6
	East	0.0	658.4	0.0	924.2	0.0	33.8	0.3
	South	559.4	2503.5	144.9	489.1	0.0	53.4	0.5
	West	348.9	807.4	0.0	197.6	0.0	20.1	0.2
							w_3 =	1.7
Roof								
	North	1742.6	0.0	0.0	467.8	0.0	37.3	0.4
	East	597.1	0.0	0.0	177.2	0.0	13.3	0.1
	South	1749.8	159.4	0.0	21.9	0.0	26.8	0.3
	West	713.2	0.0	0.0	50.3	0.0	11.3	0.1
							w_{roof} =	0.9
Atruim								
	North	0.0	0.0	0.0	105.5	0.0	3.0	0.0
	East	0.0	0.0	0.0	42.0	0.0	1.2	0.0
	South	0.0	60.0	0.0	49.3	0.0	2.1	0.0
	West	0.0	0.0	0.0	50.3	0.0	1.4	0.0
							w_{atrium} =	0.1

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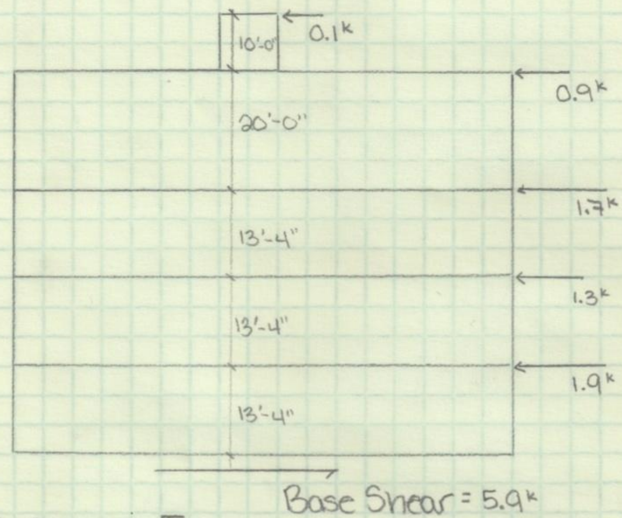
Seismic Forces Due to Exterior Walls:



Total Seismic Forces: