800 NORTH GLEBE

Arlington, VA

Technical Report 3

Ryan Johnson

Structural Option

Advisor: Dr. Linda Hanagan

December 1, 2009

Table of Contents

Executive Summary

Technical report III is a lateral system analysis and design confirmation report of the existing lateral system of 800 North Glebe. The purpose of this report was to gain a broader understanding of the lateral system by determining which lateral loads will control the design, how the lateral loads are distributed among load resisting elements in a logical load path, and verify the lateral load resisting system have been sufficiently designed for strength and serviceability. Since the engineer had designed the lateral system with the intent of having the shear walls being capable of supporting the majority of the lateral load, the single shearwall core was the primary system analyzed. However, the entire was partially investigated to compare possible strength, displacement, story drift, and overturning differences.

Preliminary hand calculations were performed to investigate and determine the relative stiffness of each lateral load resisting shear wall. It was concluded that each shearwall distributed the forces uniformly in each respective direction. Shearwall relative stiffness was then used to calculate the structural center-of-rigidity (COR). This point was placed on the structure and it was concluded that since it did not lie in the direct location on the slab center-of-mass (COM), there was a building eccentricity. Eccentricity led to torsional effects having an impact on the structural elements when added to direct shear.

Two computer models were created in ETABS to compare and verify hand calculations, one with only the shearwalls and one with the entire structure modeled. Wind and seismic loads were applied to the building and due to the nonuniform and unique smooth curved shape of 800 North Glebe, it was found that when looking at strength design, wind created greater loads, and when looking at serviceability issues, seismic created greater concerns. Therefore, because of the differences among which loading condition was greater for strength and serviceability, no single load case would control the entire building design. However, thesis calculations were performed with the assumption that wind loading would play a greater role in lateral system design because of the significant surface area of the façade. This led to ASCE 7-05 load case 6 (0.9D + 1.6W) being used for analysis.

It was found that no major concerns were found in the lateral design of 800 North Glebe with regard to torsion, shear, drift and displacement and overturning when only lateral loads were applied. However, future analysis will be performed with all loads, both gravity and lateral being applied, and variations may arise that alter controlling load cases or critical member checks.

Introduction

Located in downtown Arlington, VA, 800 North Glebe offers class-A mixed-use office space and one level of public space. Three levels of below grade parking are shared between 800 N. Glebe and 900 N. Glebe, Virginia Tech's new research building. Vertical transportation of stairways and elevators bring you from the garage to the large open retail and gathering space. Levels two through ten provide open plan office space. Column spacing of 30' x 46' allows for 30,000 square foot floor plates with 9'-0" floor-to-ceiling heights. Building setbacks are located at levels four, six, and eight to aesthetically vary the building and offer different office layouts as seen in figures 1 through 4.

The purpose of Technical report III, *Lateral System Analysis and Confirmation Design*, is to gain a better understanding of the current lateral system and explore how it differs from assuming the entire structure participates. Upon completion of the lateral system analysis, conclusions will be found on the means to which both systems handle lateral loading.

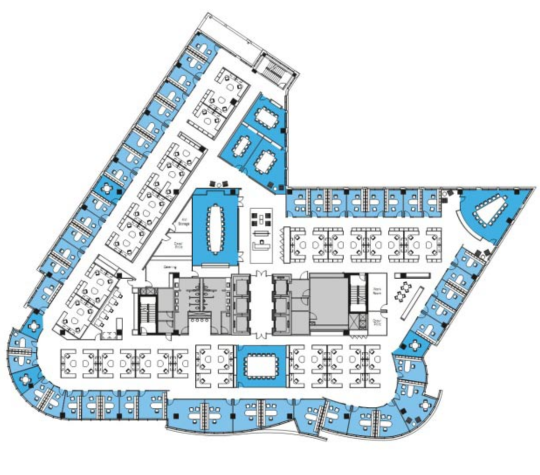


Figure 1: Floor Level 3

Figure 2: Floor level 5

Figure 3: Floor Level 8

Figure 4: Floor Level 10

Architectural Overview

800 North Glebe is a 10-story 316,000 square-foot mixed-use office building. Retail and public gathering spaces are located at street level in the 2-story lobby of the building. The remaining nine levels will provide class-A mixed-use offices. 800 North Glebe was designed for LEED Gold Certification by utilizing numerous strategies to minimize its carbon footprint.

Figure 3: South East Face

Innovative sustainable and responsible design practices are one of the designer's primary goals. Integration of sustainability and every day design by minimizing the carbon footprint, balancing energy, resources and feasibility all went into design on 800 North Glebe. In accordance with the U.S. Green Building Council's Leadership in Energy and Environmental Design, the owner has a goal to achieve LEED Gold Certification, which the designers fulfilled. LEED Gold Certification requires the design to attain at least 34 out of 61 possible points.

The 10-story façade, created by three sail-like sweeping glass curtain walls, accentuate the sight lines of the building. Radial lines and circles were widely used to define the crown and drum feature of level one and the sail feature of the remaining levels. Refer to figure 5,6 and 7 for visual representation of façade features.

Figure 4: Sail Feature

Retail and community spaces on the ground level offer 14'-6" ceiling heights with floor-to-ceiling glazing. Over the main building entrance, there is a diamond expression decorative composite metal canopy with a plaster soffit and sunguard ultrawhite laminated backlit glass as shown in figures 6 and 7.

Offices on the remaining levels of the structure offer 9’-0” floor-to-ceiling heights.


Figure 5: Front View

Three types of Architectural precast panels, metal cladding and glazing will adorn 800 North Glebe’s façade. The large sail-like curtain wall consists of Viracon VRE 1-46 on insulated heat strengthened vision and spandrel glass with PVD finished custom color composite metal mullions. Along the street level, one will find a variety of stone, metal and glazing. These include Oconee granite with a polished finish at the base, insulated spandrel glass, precast concrete panels with a light sandblast finish and PVDF finished aluminum louvers.

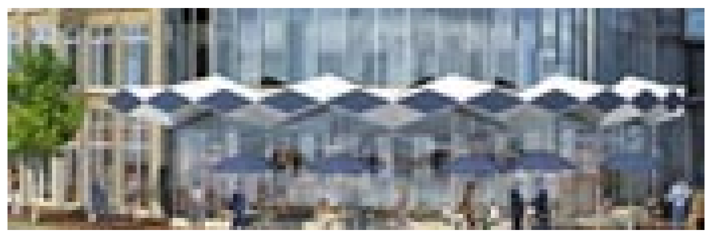


Figure 6: Canopy Over Main Entrance

Vertical bands rising up the building are made of precast concrete panels with a medium sandblast finish while horizontal bands consist of exposed aggregate finished panels. Other glazing found on the building is sunguard supernatural-68 on ultrawhite insulated glass and Viracon VRE 1-46 on insulated punch vision glass.

Protection from the elements on the roof is provided by the composite roof membrane. The composite consists of R-19 high density rigid insulation, protection board, and fully adhered 60 mil TPO membrane on top of a structural concrete slab. Where the roof system terminates at a curtain wall, fluid applied waterproofing is placed atop drainage board.

Existing System Overview

Foundation

Geotechnical studies performed by ATC Associated Inc., reported site and subsurface conditions encountered and the following information details their geotechnical recommendations for the project. Three levels of parking make up the substructure of 800 N. Glebe, at roughly thirty feet below existing grade. Groundwater levels were encountered at depths ranging from approximately 22' to 37' below the existing ground surface.

Gravel, sand, silt and clay comprise the underlain site between existing elevation and bedrock, located 35.7' to 58.8' below existing ground surfaces. The analysis indicated that spread footing foundations bearing on the dense residual soil would be feasible for a majority of the structure. However, under interior wall, the foundation shall be designed with minimum widths of 18" to 24", where many are designed to be 12'x12'x6'. Below the ground level lobby area, caissons needed to be a minimum diameter of 60" and a mat foundation would be sufficient when designed for a maximum allowable bearing pressure of 3.5 ksf.

3 ksi normal-weight concrete (NWC) is used for the foundations and interior slab on grade, the garage slab-on-grade (SOG) uses 4.5 ksi NWC and the cellar columns are composed of 4 ksi and 8 ksi. Reinforcing varies in size throughout the footings and caissons, depending on thickness. A large mat foundation is located below the shearwalls at a thickness of 6'-0".

Superstructure

A 4" thick SOG is located near the main entrance of the retail lobby. A 24" wide x 30" deep turndown, reinforced with #5s, surrounds the perimeter of the SOG. The ground level retail includes a 10" thick one-way slab with 10'-0"x10'-0"x5.5" drop panels support around the columns for punching shear resistance. Plaza slabs are 12" thick with 10'-0"x10'-0"x12" drop panels. Concrete strengths for the ground level include 3 ksi (SOG), 5 ksi (plaza slabs and framed interior slabs) and 4, 6 & 8 ksi (superstructure columns). Reinforcement for the SOG includes 6x6-10/10 welded-wire-fabric, while the one-way slab is reinforced with #5, #6 and #7s.

The remaining levels of the superstructure employ a one-way slab over post tensioned girders for the majority of the slab area which is represented as yellow in Figure 9. Girders range in size from 48" wide x 18" thick to 72" wide x 20" deep. Post tension tendons are ½" diameter with .153 square in. area low-relaxation strands with an ultimate strength of 270 ksi. A minimum of two post tension cables pass through the column reinforcement in the direction of the girder. This allows for continuous force distribution from one span to another, spanning the East/West directions. For levels two through six, two-way mildly reinforced slabs, colored cyan in Figure 9.

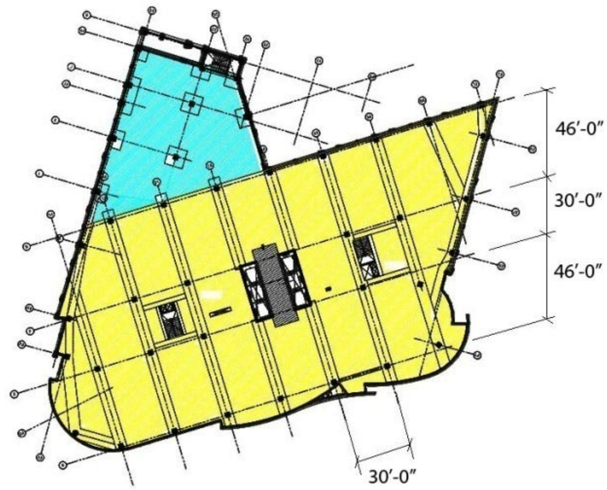


Figure 7: Slab Type Layout

Two-way slabs are 10.5" thick and are generally reinforced with #5 @ 10" in both directions. Drop panels in these areas are typically 10'-0"x10'-0"x7.5" to alleviate punching shear at the columns. Slabs over the 36" diameter column are 12" thick with #5 @ 12" parallel to the girder and #6 @10" perpendicular to the girders, due to the cantilever action.

Though the primary supporting material is concrete, steel shapes are used throughout the building for additional support. Elevator openings are supported by S8x18.4. HSS 6x3x1/4 were used as beams for additions support of shaft walls and W12x16s were used as elevator safety beams below the slabs. Steel allows for easy attachment of elevator rails and differential shaft openings.

Lateral System

Shear walls in the core of the building provide the entire lateral support, as designed by the engineer (Figure 8). However, since the building primarily consists of reinforced concrete columns and post-tensioned concrete beams, part of the lateral forces could be distributed through these members, as seen in Figure 9 where columns are red and beams cyan.

Two 12"thick "C" shaped walls, 31.83' long East/West and 9.58' long North/South per each "C", encase the elevator banks and are reinforced with #4 horizontally and #5 vertically. From the sixth floor down, walls running North/South are specially reinforced three feet from each end with #7 and #8 rebar. All of the shear walls use concrete with a compressive strength of f'_c= 6 ksi. Building drift criteria for wind loads is L/400 or 3/8" inter-story drift at typical floors (12'-9" floor-to-floor) and for seismic loads is L/76 or 2" inter-story drift at typical floors (12'-9" floor-to-floor).

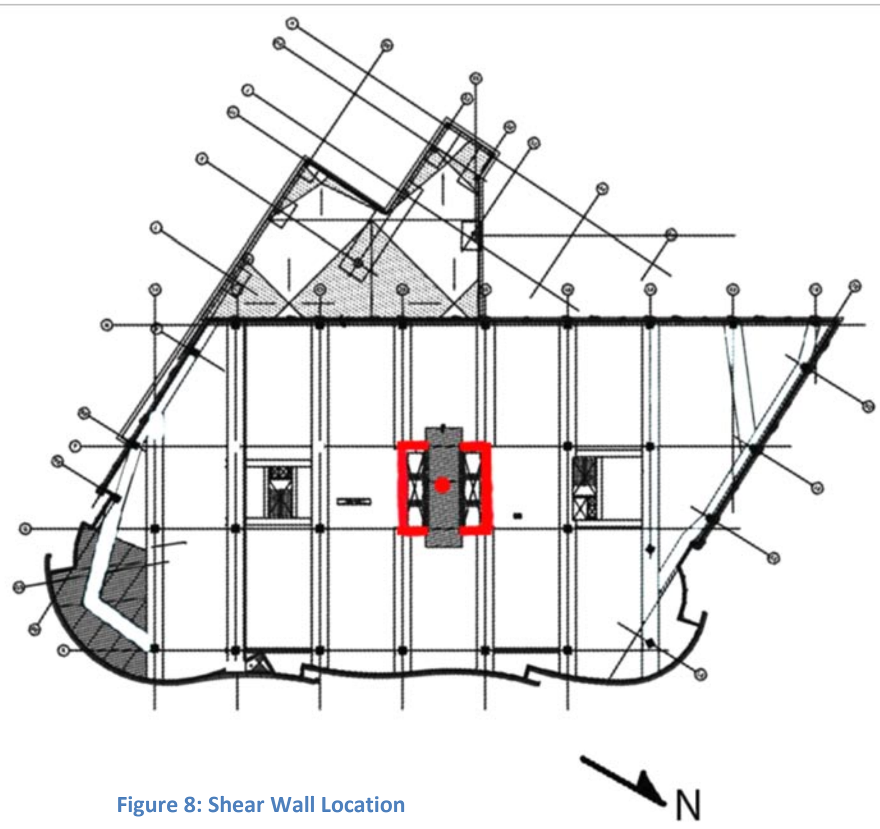


Figure 8: Shear Wall Location

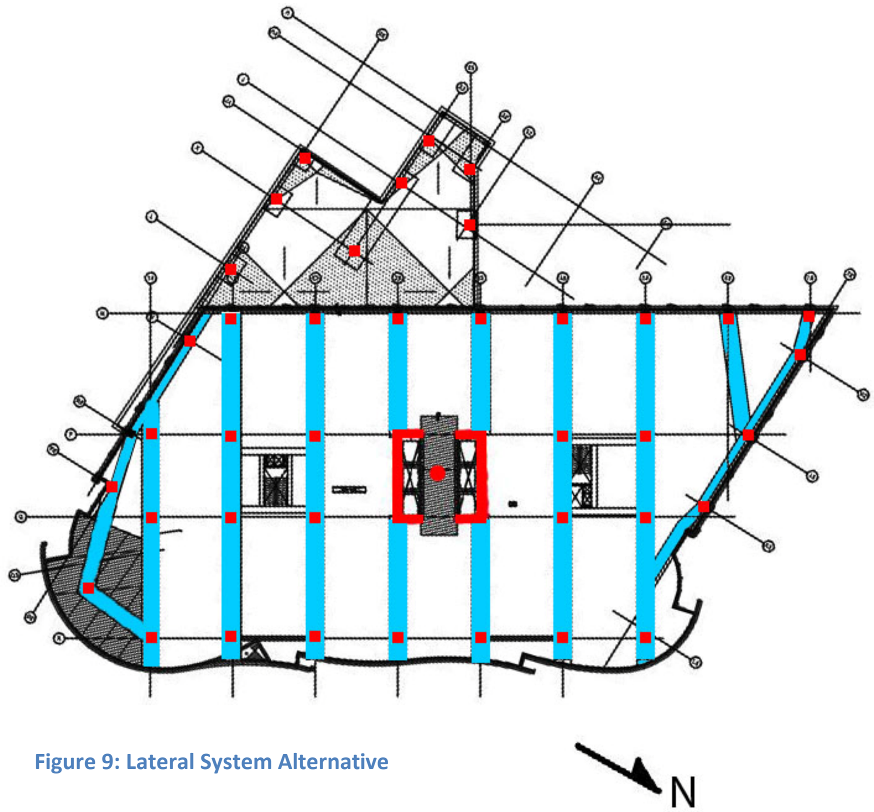


Figure 9: Lateral System Alternative

The columns throughout the building are primarily 30”x30” with 72” wide by 18” deep post-tensioned beams tying into them. Though these members were not designed to take the primary lateral force, they will transfer loads through themselves, and therefore have some affect on the lateral system. A 9” normally reinforced concrete slab transfers loads to the post-tensioned beams and act as a rigid diaphragm for the structure. Also, post-tensioned tendons surround the building slab edges to reduce slab deflection, but will also help transfer lateral forces. These are not marked above but are around the entire one-way slab perimeter.

Design Codes and Standards

Thesis design had been performed with the most up to date codes and standard available. These may differ from the original design, resulting in possible calculation variations.

Original Design:

- International Building Code, 2003
- Virginia Uniform Building Code, 2003
- American Society of Civil Engineers (ASCE)
  - ASCE 7-02, Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
  - Building Code Commentary 318-02
  - Structural Concrete for Buildings, ACI 301
- America Institute of Steel Construction (AISC)
  - Manual of Steel Construction, Thirteenth Edition, 2005

Thesis Design with Additional References:

- International Building Code, 2006
- Virginia Uniform Building Code, 2003
- American Society of Civil Engineers (ASCE)
  - ASCE 7-05, Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
  - Building Code Commentary 318-08
- America Institute of Steel Construction (AISC)
  - Manual of Steel Construction, Thirteenth Edition, 2005
- Precast / Prestressed Concrete Institute
  - PCI Manual for the Design of Hollow Core Slabs, Second Edition, 1998

Deflection Criteria

Horizontal Framing Deflections:

- **Live Load**
  - **< L/600 or ½”**

- **Total Load Excluding Self Weight**
  - **< L/480 or ¾”**

*Horizontal framing deflections are strictly set because of all the brittle finishes being supported by the slabs. The curtain wall system has a lot of dependency on how much the slabs move.

Lateral Drift:

- **Wind Loads**
  - **< L/400 or 3/8” interstory drift at typical floors (12’-9”)**

- **Seismic Loads**
  - **< L/76 or 2” interstory drift at typical floors (12’-9”)**

Main Structural Elements Supporting Components and Cladding:

- **At Screenwalls**
  - **< L/240 or ¾”**

- **At Floors Supporting Curtainwalls**
  - **< L/600 or ½”**

- **At Roof Parapet Supporting Curtainwalls**
  - **< L/600 or ½”**

- **At Non-Brittle Finishes**
  - **< L/240**

Materials

Steel:

Wide Flange	**50 ksi (A992)**
Plates, Channels, Angles and Bars	**36 ksi (A36)**
Round Pipes	**42 ksi (A53 Grade B)**
HSS Rectangular or Square Tubing	**46 ksi (A500 Grade B)**
HSS Round Tubing	**42 ksi (A500 Grade B)**
Bolts	**36/45 ksi (A325 or A490)**
Anchor Rods	**(F1554 Grade 55)**
Weld Strength	**70 ksi (E70XX)**

Concrete:

Foundations, Int. Slab on Grade	**f’c = 3000 psi**
Interior Walls	**f’c = 5000 psi**
Ext. Slab of Grade, Pads, Garage SOG	**f’c = 4,500 psi**
Garage and Plaza Slabs, Framed Int. Slabs	**f’c = 5000 psi**
Ext. Walls, Beams, Basement Walls	**f’c = 4000 & 5000 psi**
Deck Supported Slabs	**f’c = 3500 psi**
Cellar Columns	**f’c = 4000 & 8000 psi**
Superstructure Columns	**f’c = 4000, 8000 & 6000 psi**
Shear Walls	**f’c = 6000 psi**
Masonry	**f’m = 1500 psi**

Reinforcement:

Longitudinal Bars	**60 ksi (A615)**
Deformed Bars (Ties)	**60 ksi (A615)**
Welded Wire Mesh	**(A185)**

Post Tensioning:

Tendons	**270 ksi (A416)**

Cold Formed Steel:

20 Gage	**33 ksi (A653)**
18 Gage	**33 ksi (A653)**
16 Gage	**50 ksi (A653)**

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

Building Loads

Gravity - Live Loads

ASCE 7-05, *Minimum Design Loads for Buildings and other Structures,* was the main reference for determination of loads in this project for 800 North Glebe. These loads were compared to the loads specified by the designer per IBC 2003 and the 2003 Virginia Uniform State Building Code which references ASCE 7-02. A few loadings used by the designer were seen to be greater, i.e. garage entry, and therefore the larger value was used because of the significant increase. These values are outlined in table 1 below.

Live Loads				
Description	Location	Designer Loads	(ASCE 7-05)	Thesis Loads
Parking	P3	40	40	40
Stairs	P3	100	100	100
Parking	P2	40	40	40
Stairs	P2	100	100	100
Parking	P1	40	40	40
Stairs	P1	100	100	100
Garage Entry	Level 1	250	50	250
Main Retail/Assembly	Level 1	100 125 250	100	100
Elevator Lobby	Level 1	100	100	100
Entry	Level 1	100	100	100
Loading Dock	Level 1	350		350
Yards and Terraces	Level 1	100	100	100
Marquees and Canopies	Level 2	75	75	75
Corridors Above First Floor	Level 2-10	100	80	80
Walkways and Elevated Platforms		60	60	60
Mechanical	Penthouse	150	125	125
Roof	Roof	30	20	20
Live loads reduction has not been used				

Table1: Building Live Loads

Gravity - Dead Loads

Building dead loads and their general description are laid out in table 2 below. A more detailed description of how the dead loads were calculated can be found in the Appendix. Slab areas were taken from CAD floor plans provided by the designer and varied by floor because of the curves and the major setback at levels four, six and eight. Four slab thicknesses of 7 ½”, 9”, 10 ½” and 12” are used per floor depending on the location and usage. The 7 ½” slab thickness is located between the elevator banks, primarily because the area is minimal. Two-way mildly reinforced slabs located on levels two though six have slab thicknesses of 10 ½” with 7” thick drop panels to reduce the punching shear around the columns. Across the Post tensioned (PT) girders is the 9” one-way slab. Located at the main entrance is a 36” diameter column rising from the ground to the top of the building with a 12” cantilevered slab. The 12” slab was needed because of the increased moment the cantilevered section caused over the beam.

Dead Loads				
Description	Location	Designer	Superimposed Dead Load	Thesis Loads
Concrete	All Levels	150 pcf		150 pcf
Partitions, Finishes	All Levels		20 psf	20 psf
MEP	All Levels		5 psf	5 psf
Precast Panels	Curtain Wall		35 psf	20 psf*
Curtain Glass	Curtain Wall		15 psf	

Table 2: Building Dead Loads

*Assume the façade is composed of 20% precast and 80% glazing.

Wind Loads

ASCE 7-05 was the governing resource for wind load calculations. Section 6.5 describes *Method 2 – Analytical Procedure* for main wind-force resisting systems (MWRS) of enclosed buildings. Exposure, height, topographic effects, wind direction and wind velocity all played a part in determining velocity pressures. In conjunction with gust effect factors, external and internal pressure coefficients, and force coefficients it was eventually determine the base shear for the building. Section four outlines four cases in which wind loads should be applied to determine the greatest story forces. These cases would entered into a computer model and it was found that case one, full wind loads applied to the primary axis without eccentricity effects, produced greater forces on the structure. Table 4 outlines the variables used in analysis, and the calculations are shown in the Appendix.

A box was drawn around the building shape, along the principle lateral system axis, as seen in Figure 10. The size of the box was approximated to enclose a majority of the building and to determine the center-of-pressure. It can be seen that the lower side of the building is perpendicular to the applied wind load. Because of this, the wind forces in this direction are larger than the wind forces acting on the left side of the building, but both faces experience significantly large wind pressures. Lateral load calculations discussed later will determine the extent of the forces increase.

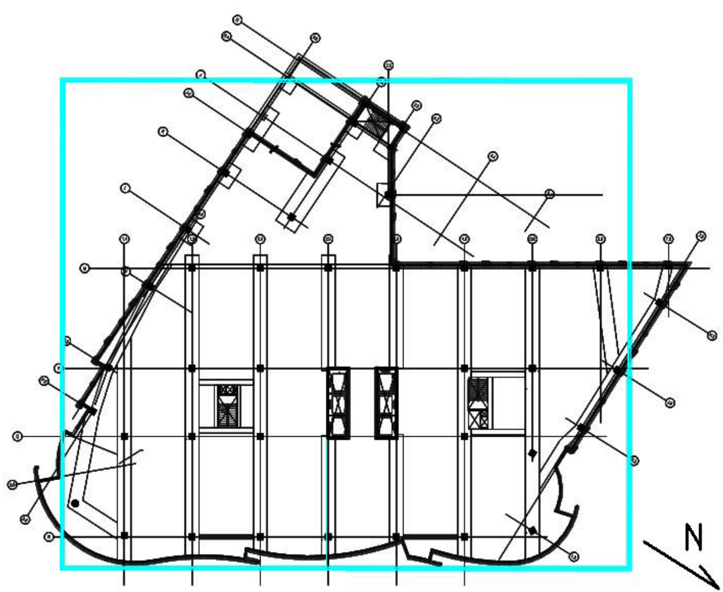


Figure 10: Generalized Building Shape Diagram

Wind Loads			
Category			Reference
Basic Wind Speed (mph)	V_{3s}	90	Figure 6-1
Importance Factor	I	1.0	Table 6-1
Exposure Category	-	B	6.5.6.3
Directionality Factor	K_d	0.85	Table 6-4
Topographic Factor	K_{zt}	1.00	6.5.7.1
Intensity of Turbulence	I_z	Varies	Eq. 6-5
Integral Length Scale of Turbulence	L_z	Varies	Eq. 6-7
Background Response Factor (North/South)	Q	0.780	Eq. 6-6
Background Response Factor (East/West)	Q	0.778	Eq. 6-6
Gust Effect Factor (N/S)	G_f	0.8191	6.5.8.1
Gust Effect Factor (E/W)	G_f	0.8175	6.5.8.1
	GC_{pi}	0.18	Figure 6-5
	GC_{pi}	-0.18	Figure 6-5
Windward Pressure	C_p	0.8	Figure 6-6
Leeward Pressure (E/W)	C_p	-0.5	Figure 6-6
Leeward Pressure (N/S)	C_p	-0.45	Figure 6-6 (interpolated)
Velocity Pressure Exposure Coefficient Evaluated at Height z	K_z	Varies	Table 6-3
Velocity Pressure at Height z	q_z	Varies	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	19.70	Eq. 6-15

Table 4: Building Wind Load Variables

Tables 5 and 6 show how the forces act on the building in the North/South direction while tables 7 and 8 show the forces acting in the East/West directions respectively. Figures 11 and 12 depiction how these pressures act on the building at each level. The figures and tables are based off of the MWRS calculations and are the forces used in the computer model.

Wind Loads N/S (Short Walls Resisting)								
Floor	Story Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)		Force of Windward Pressure (k)	Story Shear Windward (k)
					Windward	Leeward		
PH Roof	0	153.75	1.12	19.70	12.91	-7.26	24.75	0.00
PH	18.5	135.25	1.08	18.99	12.44	-7.26	42.58	24.75
10	13.75	121.5	1.04	18.41	12.07	-7.26	35.07	67.33
9	13.75	107.75	1.01	17.79	11.66	-7.26	32.65	102.40
8	12.75	95	0.97	17.16	11.25	-7.26	30.21	135.05
7	12.75	82.25	0.93	16.47	10.79	-7.26	28.89	165.26
6	12.75	69.5	0.89	15.70	10.29	-7.26	27.41	194.16
5	12.75	56.75	0.84	14.82	9.71	-7.26	25.68	221.56
4	12.75	44	0.78	13.78	9.03	-7.26	23.59	247.24
3	12.75	31.25	0.71	12.49	8.19	-7.26	20.88	270.84
2	12.75	18.5	0.61	10.76	7.05	-7.26	9.66	291.72
1	18.5	0	0.00	0.00	0.00	0.00	0.00	301.38
					Σ Windward Story Shear (k)=			301.38

Table 1: N/S Windward Pressures

Wind Loads N/S (Short Walls Resisting)					
Floor	Story Height (ft)	Height Above Ground (ft)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)
PH Roof	0	153.75	20.17	39.18	0.00
PH	18.5	135.25	19.70	67.75	39.18
10	13.75	121.5	19.33	56.53	106.94
9	13.75	107.75	18.92	53.33	163.47
8	12.75	95	18.51	50.11	216.80
7	12.75	82.25	18.05	48.80	266.92
6	12.75	69.5	17.55	47.31	315.71
5	12.75	56.75	16.97	45.58	363.02
4	12.75	44	16.29	43.50	408.60
3	12.75	31.25	15.45	40.78	452.10
2	12.75	18.5	14.31	19.61	492.88
1	18.5	0	0.00	0.00	512.49
		Σ Total Story Shear (k)=			512.49

Table 2: N/S Total Pressures

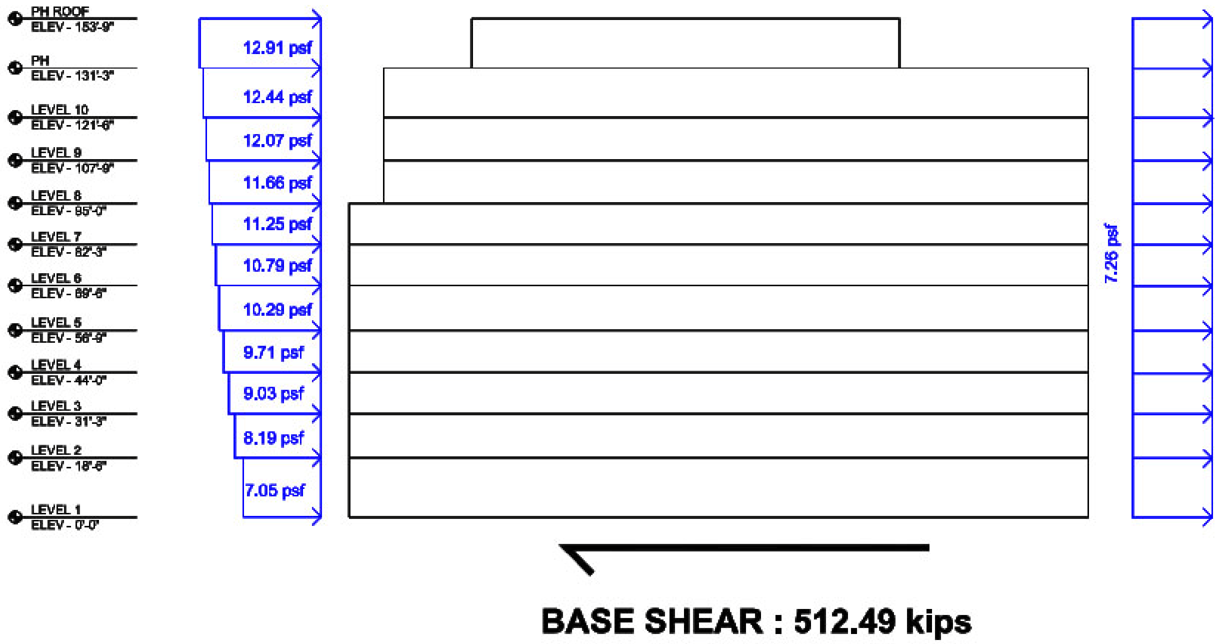


Figure 11: N/S Wind Load Diagram

Wind Loads E/W (Long Walls Resisting)								
Floor	Story Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)		Force of Windward Pressure (k)	Story Shear Windward (k)
					Windward	Leeward		
PH Roof	0	153.75	1.12	19.70	12.88	-8.05	28.72	0.00
PH	18.5	135.25	1.08	18.99	12.42	-8.05	49.41	28.72
10	13.75	121.5	1.04	18.41	12.04	-8.05	40.70	78.13
9	13.75	107.75	1.01	17.79	11.64	-8.05	37.89	118.83
8	12.75	95	0.97	17.16	11.23	-8.05	35.06	156.72
7	12.75	82.25	0.93	16.47	10.77	-8.05	33.53	191.78
6	12.75	69.5	0.89	15.70	10.27	-8.05	31.80	225.31
5	12.75	56.75	0.84	14.82	9.69	-8.05	29.80	257.12
4	12.75	44	0.78	13.78	9.01	-8.05	27.38	286.92
3	12.75	31.25	0.71	12.49	8.17	-8.05	24.23	314.30
2	12.75	18.5	0.61	10.76	7.03	-8.05	11.21	338.53
1	18.5	0	0.00	0.00	0.00	0.00	0.00	349.74
					Σ Windward Story Shear (k)=			349.74

Table 3: E/W Windward Pressures

Wind Loads E/W (Long Walls Resisting)					
Floor	Story Height (ft)	Height Above Ground (ft)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)
PH Roof	0	153.75	20.93	47.33	0.00
PH	18.5	135.25	20.47	81.87	47.33
10	13.75	121.5	20.09	68.37	129.20
9	13.75	107.75	19.69	64.56	197.57
8	12.75	95	19.28	60.72	262.13
7	12.75	82.25	18.82	59.19	322.85
6	12.75	69.5	18.32	57.46	382.05
5	12.75	56.75	17.74	55.46	439.51
4	12.75	44	17.06	53.04	494.97
3	12.75	31.25	16.22	49.89	548.01
2	12.75	18.5	15.08	24.04	597.91
1	18.5	0	0.00	0.00	621.95
				Σ Total Story Shear (k)=	621.95

Table 4: E/W Total Pressures

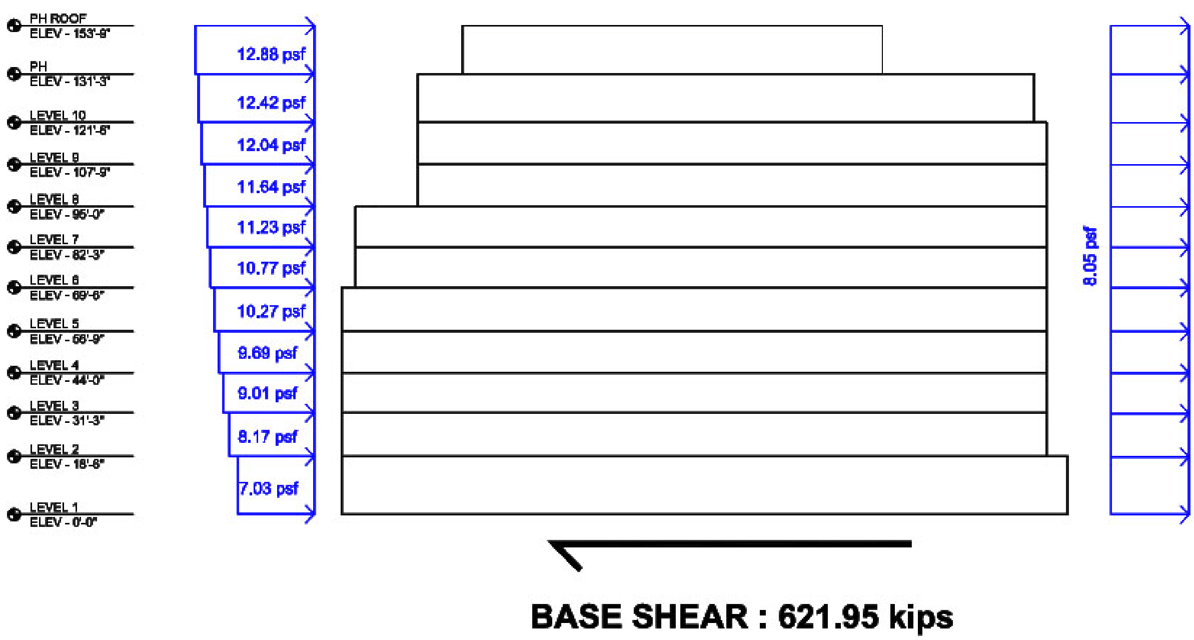


Figure 12: E/W Wind Load Diagram

Seismic Loads

Seismic calculations of 800 North Glebe were based upon ASCE 7-05 for thesis design. The engineering firm had used ASCE 7-02 / IBC 2003 and the 2003 Virginia Uniform Statewide Building Code to calculate the base shear from the equivalent lateral force analysis procedure. A difference among the calculated base shears was found and discussed with Structura and my consultant. It was determined that variations in base shear may be observed because of the building weight assumptions used for thesis calculations compared to those of Structura, whom had used exact weights determine by their RAM model. Floor weight calculation used for thesis can be found in the Appendix.

Design criteria variables used for thesis analysis can be found below in table 6. Design criteria variables were used to determine story forces at each level, story shear at each level, and base shear, where the output is located in table 7. Figure 13 was constructed to display how these forces acted on the building, while calculations to support the excel graph below are located in the Appendix.

The model output for maximum modal period of vibration was found to be 5.6079 seconds, as compared to Structura's value of 5.897 seconds. However, this value was not used as the fundamental period because it means the structure is more flexible than what value the code permits for fundamental period of vibration, T_aC_u = 1.868s. A lower period of vibration being used for design assumes the lateral resisting structural elements are more rigid and therefore, must be designed for the larger forces. The period of vibrations for the structure are found in table 5 below. When only the shear walls are analyzed compared to the entire structure, a larger period was found, meaning the structure is less stiff. The largest difference can be found in the building rotation (torsion). Since the lateral shearwall core is centrally located with the majority of the building spread over a large slab area causing the building to significantly rotate. The columns and beams are spread throughout the structure, increasing the stiffness and reducing the torsional effects.

	X-Translation	T-Translation	Z-Rotation
Shear Walls Only	5.6079	1.494	6.1358
Entire Structure	3.0614	1.0138	1.5452
Structural Design	5.897	2.198	NA

Table 5: Structure Period of Vibration

Design Criteria Variables				Structura
Seismic Use Group		Group II		Group II
Site Class		D	Geotech Report	D
Importance Factor	I_e	1.00	Table 11.5-1	1.0
Spectral Response Acceleration, Short	S_s	0.179	USGS	0.179
Spectral Response Acceleration, 1s	S_1	0.063	USGS	0.063
Site Coefficient	F_a	1.6	Table 11.4-1	
Site Coefficient	F_v	2.4	Table 11.4-2	
Soil Modified Acceleration	S_{MS}	0.2864		
Soil Modified Acceleration	S_{M1}	0.1512		
Design Spectral Response, short	S_{ds}	0.191	USGS	0.191
Design Spectral Response, 1s	S_{d1}	0.101	USGS	0.101
Response Modification Coefficient	R	5.5	Table 12.2-1	5.5
Seismic Design Category		B	Table 11.6-1	B
Approx. Period Parameter	C_t	0.02	Table 12.8-2	
Building height (above grade)	h_n	153.75		
Approx. Period Parameter	x	0.75	Table 12.8-2	
Approx. Fundamental Period	T_a	1.100	Eq. 12.8-8	1.1
T-Used	T_aC_u	1.868	12.8.2	
	T_s	0.528	11.4.5	
Calculated Period Upper Limit Coefficient	C_u	1.7	Table 12.8-1	1.698
Seismic Response Coefficient	C_s	0.0347	Eq. 12.8-2	
	C_s	0.0098		
Structural Period Exponent	k	1.684	12.8.3	1.684
Long Period Transition Period	T_L	6	Figure 22-15	
Building Weight (k)	W	59780		73181.57
Base Shear: Cs x W=	V	597.80		578
Basic Seismic-Force Resisting System		Ordinary Reinforced Concrete Shear Walls	Structural Plans	
Effects from the below grade parking garages were not taken into account for calculations				

Table 6: Seismic Design Criteria Variables

Floor	Story Height (ft)	Height Above Ground h_x (ft)	Story Weight w_x (kips)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Story Force F_x (kips)	Story Shear V_x (kips)
PH Roof	0	153.75	698.05	4812.58	3359440.72	0.04	21.01	0.00
Main Roof	18.5	135.25	5304.52	3878.13	20571585.89	0.22	128.67	21.01
10	13.75	121.5	5210.83	3237.56	16870380.75	0.18	105.52	149.68
9	13.75	107.75	5165.42	2644.77	13661341.03	0.1429	85.45	255.20
8	12.75	95	5417.99	2139.38	11591169.80	0.12	72.50	340.65
7	12.75	82.25	5597.77	1678.41	9395329.51	0.10	58.77	413.15
6	12.75	69.5	6177.61	1263.92	7807983.31	0.08	48.84	471.92
5	12.75	56.75	6221.57	898.47	5589889.80	0.06	34.96	520.76
4	12.75	44	6353.88	585.34	3719196.47	0.04	23.26	555.72
3	12.75	31.25	6250.91	328.99	2056464.72	0.02	12.86	578.98
2	12.75	18.5	6996.27	136.08	952044.06	0.01	5.95	591.85
1	18.5	0	389.36	0.00	0.00	0.00	0.00	597.80
				Total	95574826.07		597.80	

Table 7: Seismic Design Loads

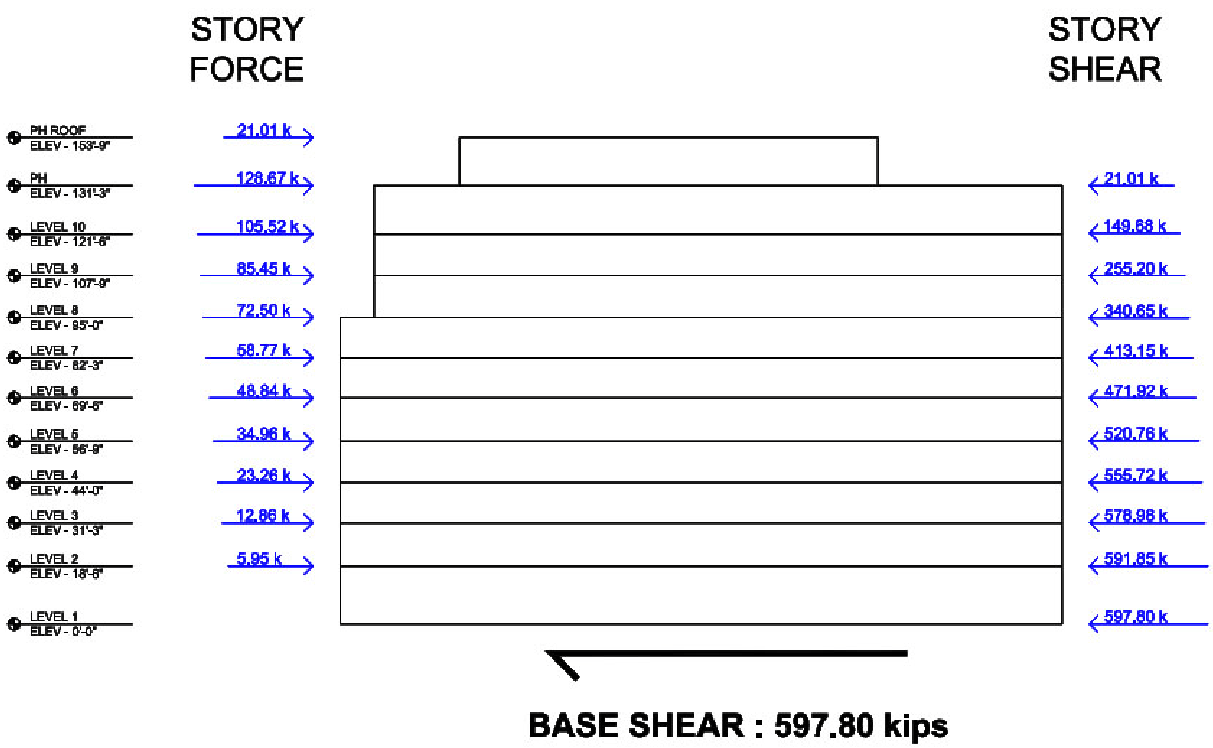


Figure 13: Seismic Force Diagram

ETABS Model

Two computer models were created using ETABS, Computer and Structures Inc. structural modeling and analysis program. The first model, model A, only included the central-core shear walls as the lateral resisting system, as designed by the engineer, seen in figure 14. The second model, model B, included the entire structural system of columns, beams and shear walls because their stiffness would participate in transferring lateral forces, seen in figure 15. Results from model A were compared to hand calculations performed to determine the center-of-rigidity and elements' stiffness and story displacements. Load combinations were entered manuals into ETABS based on AISC 7-05. Information such as which load case controlled was determined from the computer model. Most of the lateral analysis was performed on model A because this is the system the engineer had designed to resist the lateral forces. Model B was created for comparison purposes and results will be discussed on the variations between the models in certain aspects of the lateral system.

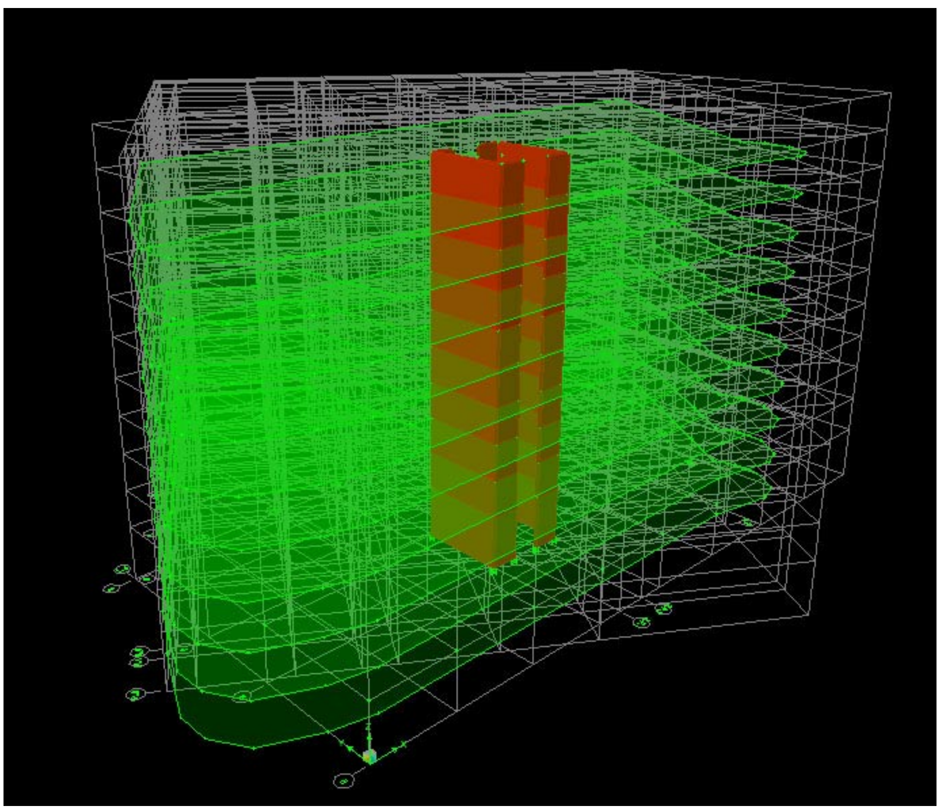


Figure 14: ETABS Model A

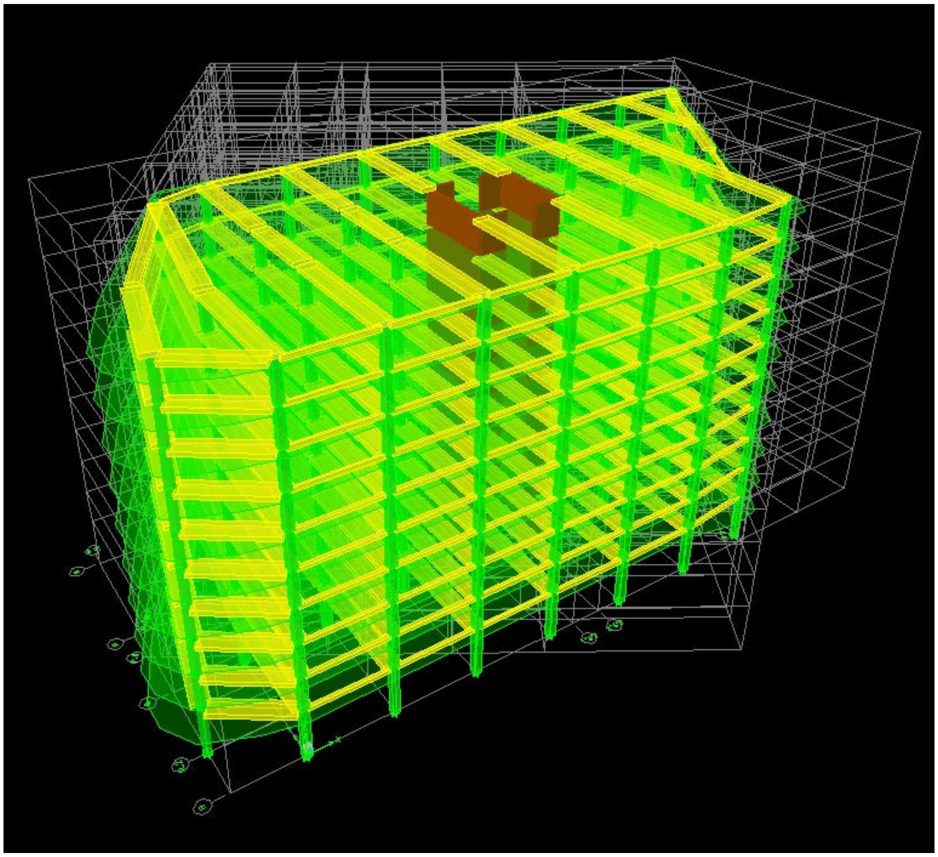


Figure 15: ETABS Model B

Both models include rigid diaphragms at each floor level, to which additional area masses were added based on uniform concrete slabs to simplify the addition of gravity load. Other analysis assumptions that were included in the ETABS model include, but are not limited to:

- All restraints on the bottom level were modeled as fixed.
- Structural members were modeled without their material properties mass per unit area.
- Shear walls modeled as shell elements meshed into areas with a maximum dimension of 24”x24” to allow for the walls to act as a rigid unit.
    - Shell element resistance properties were manual reduced to minimize the walls capabilities of taking out-of-plane bending.
- Beams and columns of model B were modeled as line elements with specified dimensions to match the existing structural plans.
- The moment of inertias of columns and portions of the shears walls were reduced to $0.7I_g$. This is done to account for inelastic response of members and the decrease in effective stiffness.
    - The portions of the shear walls, on the bottom 6 levels, were not reduced because of the significant steel amount.

- Post-tensioned beams in model B had their modulus of elasticity increased to four times their actually value. This was done to account for the extreme compressive forces used to minimize possible tension stresses in the members.
- Beam elements included a 0.5 rigid end offset multiplier that assume each end to be 50% rigid for bending and shear deformation.
- Seismic loads were applied to the center-of-mass of each floor diaphragm.
- Wind Loads were applied at the center-of-pressure.
- Coupling beams act between the shear wall returns as specified by the design engineer.
    - Coupling beams are sized to be the thickness of the slab, width of the shear wall and material properties of the slab.

Lateral Load Consideration

Load Combinations

AISC 7-05 section 2.3, strength design load combinations were considered for factoring gravity and lateral loads in analysis. When only gravity load cases are considered, load case 2 usually governs. However, when lateral loads are involved in analysis, load cases 4, 5, 6 or 7 may govern depending on lateral load magnitudes and whether overturning is addressed. The load combinations considered for thesis analysis are listed below.

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

For the thesis building being analyzed, these combinations were entered into the ETABS models. It was determined, by looking at shears at the first level walls, the load case including 1.6W were larger in the north-south direction and the east-west direction, which can be seen in table 8 below. This is primarily due to the large surface areas of the façade, which produce larger wind pressures, and therefore larger story forces on the structure. The wind loads in east-west directions had a much more significant increase compared to the north-south direction. Wind case 1 was used for comparison of direct shear forces at the first level because it was found that case 1 controlled over the other three wind cases, per AISC 7-05 section 6.4 figure 6-9. Case 2 was very close in the regards to the lateral displacement of the roof level, which was used for determining the controlling case.

However, when looking at displacements, which are a serviceability concern and no load combination multipliers are used, variations were noticed. The model with only the shear walls had seismic loading as the greater cause of displacement in the east-west and north-south direction. When the entire structure was modeled for lateral resistance, seismic had larger displacements in both directions as well, as seen in table 9. This may be due to the increased mass of the concrete members' participation in seismic drift. Drift is an issue that should not be overlooked, but because the differences among wind compared to seismic is not overly large, spot checks will be performed with load combinations including wind loads. It is believed that since strength issues are part of the code and drift is only a serviceability concern, combinations that controlled in strength design would be more critical and therefore were analyzed.

Section Cut at level 1		
Load Combination	Direct Shear Force (k)	
	Shear Walls Only	Entire Structure
Combo 4 X-Dir (N/S)	**494.00**	**319.67**
Combo 5 X-Dir (N/S)	307.82	220.86
Combo 6 X-Dir (N/S)	**494.00**	**319.67**
Combo 7 X-Dir (N/S)	307.82	220.86
Combo 4 Y-Dir (E/W)	**418.21**	**438.96**
Combo 5 Y-Dir (E/W)	123.84	289.64
Combo 6 Y-Dir (E/W)	**418.21**	**438.94**
Combo 7 Y-Dir (E/W)	123.84	289.62
Case 1 of wind used to determine direct shear difference		

Table 8: Load Combination Shear Forces

		Shear Walls Only		Entire Structure	
Story	Load	UX	UY	UX	UY
MAIN ROOF	WIND X-Dir (N/S)	3.10	0.02	1.96	-0.04
MAIN ROOF	WIND Y-Dir (E/W)	-0.01	0.67	-0.06	0.26
MAIN ROOF	SEISMIC X-Dir (N/S)	**4.37**	-0.01	**2.74**	-0.06
MAIN ROOF	SEISMIC Y-Dir (E/W)	-0.03	**0.93**	-0.06	**0.31**

Table 9: Wind vs. Seismic Lateral Displacement

Load Path and Distribution

Loads travel throughout a building until they reach the ground. The path which loads are distributed is based on member relative stiffness. The members with a higher relative stiffness have larger forces induced into them. Given that the floor slabs were treated as rigid diaphragms, the lateral loads of model were transferred directly to the central core shear walls and distributed accordingly in each direction. SAP was used in conjunction with hand calculations to determine the relative stiffness of each wall in the core individually. A 1 kip load was applied at the top of the wall to determine the displacement and then the stiffness (K) on each wall was calculated as 1/ ΔP.

Model B included columns and beams that would transfer portions of the load through them to either the supporting columns or the shear walls. Shear walls were assumed to not take any out-of-plane forces, but in reality the walls orthogonal to the applied loads would participate by acting similar to the flanges of a steel W-shape. The lateral system of model A, with only shear walls as seen in figure 16 below, was used for relative stiffness calculations of table 10.

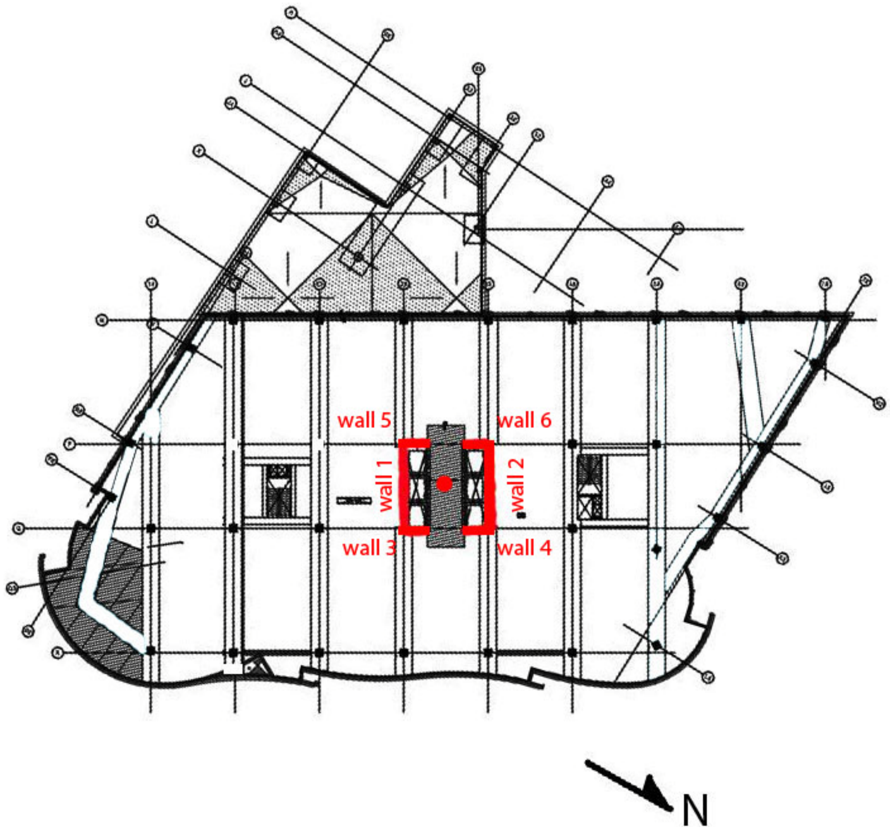


Figure 16: Lateral System (shear walls) with Labels

	P (#)	L (in)	t (in)	h (in)	A (in^2)	I (in^4)	E (psi)
Wall 1	1000	381.96	12	1620	4583.52	55725459	4415201
Wall 2	1000	381.96	12	1620	4583.52	55725459	4415201
Wall 3	1000	114.96	12	1620	1379.52	1519289	4415201
Wall 4	1000	114.96	12	1620	1379.52	1519289	4415201
Wall 5	1000	114.96	12	1620	1379.52	1519289	4415201
Wall 6	1000	114.96	12	1620	1379.52	1519289	4415201

	Δ Flex (in)	Δ Shear (in)	Δ P (in)	K	SAP Δ (in)	SAP K	**Rel K (%)**
Wall 1	0.006	0.0002	0.0060	167154.1003	0.0073	136986.3	**50**
Wall 2	0.006	0.0002	0.0060	167154.1003	0.0073	136986.3	**50**
Wall 3	0.211	0.0007	0.2120	4716.823759	0.2607	3835.827	**25**
Wall 4	0.211	0.0007	0.2120	4716.823759	0.2607	3835.827	**25**
Wall 5	0.211	0.0007	0.2120	4716.823759	0.2607	3835.827	**25**
Wall 6	0.211	0.0007	0.2120	4716.823759	0.2607	3835.827	**25**

Table 10: Relative Stiffness Calculation

Torsion

Eccentricities resulting from lateral loads not being applied at the center-of-rigidity (COR) cause torsion on the building. Wind loads are applied at the center-of-pressure (COP), while seismic forces are applied at the center-of-mass (COM). In the case of 800 North Glebe, neither of these two centers coincides with the center of rigidity. Refer to table 11a figure 17 to view the difference of the COM to the COR of model A.

	Shearwalls Only									
	Center of Rigidity				Center of Mass					
Floor	ETABS Output		Hand Calculations		ETABS Output		Hand Calculations			
	X	Y	X	Y	X	Y	X	Y	e_x	e_y
Main Roof	1260	710	1260	708.5	1121.84	691.88	NA	NA	-138.16	-16.62
10	1260	710	1260	708.5	1121.84	691.88	NA	NA	-138.16	-16.62
9	1260	710	1260	708.5	1121.84	691.88	NA	NA	-138.16	-16.62
8	1260	710	1260	708.5	1121.84	691.88	NA	NA	-138.16	-16.62
7	1260	710	1260	708.5	1121.84	691.88	NA	NA	-138.16	-16.62
6	1260	710	1260	708.5	1085.62	862.13	NA	NA	-174.38	153.63
5	1260	710	1260	708.5	1085.62	862.13	NA	NA	-174.38	153.63
4	1260	710	1260	708.5	1082.53	889.30	NA	NA	-177.47	180.80
3	1260	710	1260	708.5	1082.53	889.30	NA	NA	-177.47	180.80
2	1260	710	1260	708.5	1082.53	889.30	NA	NA	-177.47	180.80
Center of rigidity hand calculated with origin at bottom left corner of the arbitrary load box drawn above. ETABS placed origin at intersection of column lines 1.4 and R.										

	Entire Structure					
	Center of Rigidity		Center of Mass			
Floor	ETABS Output		ETABS Output			
	X	Y	X	Y	e_x	e_y
Main Roof	1175.2	419.3	1128.1	690.3	-47.07	270.98
10	1184.6	446.0	1128.1	690.3	-56.46	244.25
9	1193.9	471.8	1128.1	690.3	-65.82	218.50
8	1202.8	497.5	1128.1	690.3	-74.69	192.76
7	1211.2	525.8	1128.1	690.3	-83.06	164.45
6	1219.2	560.9	1120.6	724.5	-98.56	163.64
5	1226.2	606.2	1115.5	747.7	-110.74	141.52
4	1232.9	651.8	1111.6	767.4	-121.28	115.59
3	1238.2	702.2	1108.7	782.2	-129.52	79.98
2	1241.9	762.8	1106.5	793.7	-135.39	30.85

Table 11 a & b: COR vs. COM

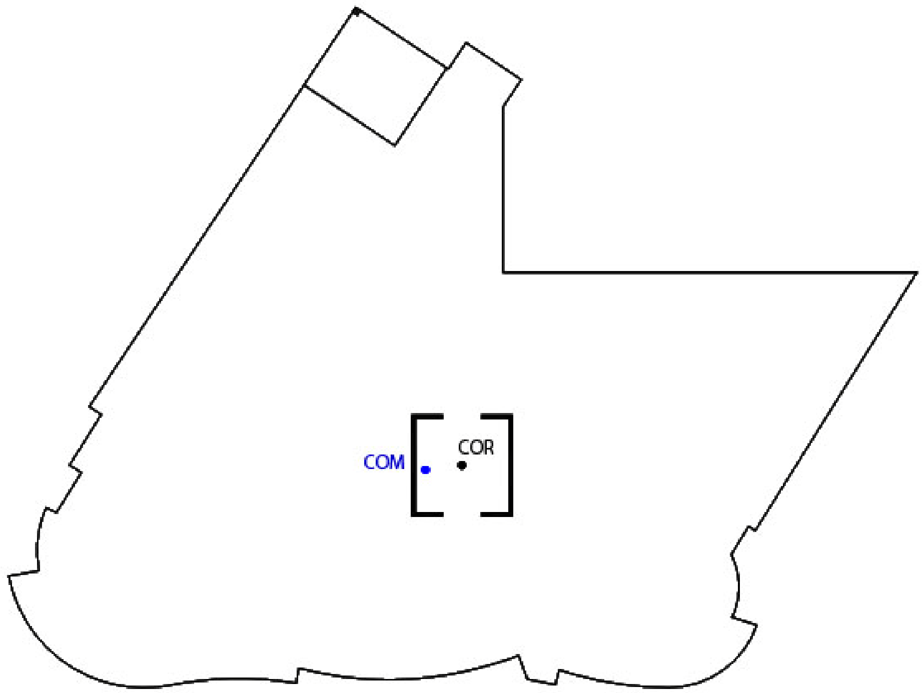


Figure 17: Shearwalls Only COM vs. COR Levels 2-4

The eccentricity of the COM to the COR causes a torsional moment on the building. AISC 7-05 section 12.8.4 was used to determine this total moment produced by inherent torsion and accidental torsion. Inherent torsion is, as stated by section 12.8.4.1, "For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the center-of-mass and the center of rigidity." Accidental torsion is, as specified by section 12.8.4.2, "The accidental torsional moments, M_{ta}, (kip) caused by assumed displacement of the center-of-mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces." To obtain the overall building moment, M_{ta} was added to M_t, creating the largest torsional moment, shown in table 12.

	North-South Direction (Short Wall Resisting)				East-West Direction (Long Wall Resisting)			
Floor	Floor Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	M_t total (ft-k)	Floor Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	M_t total (ft-k)
PH Roof	21.01	-349.23	225.89	575.11	21.01	-2903.1	262.66	3165.76
Main Roof	128.67	-2138.51	1383.21	3521.72	128.67	-17777.2	1608.39	19385.55
10	105.52	-1753.75	1134.35	2888.10	105.52	-14578.7	1319.01	15897.73
9	85.45	-1420.16	918.57	2338.73	85.45	-11805.6	1068.11	12873.71
8	72.50	-1204.95	779.38	1984.33	72.50	-10016.6	906.25	10922.89
7	58.77	-976.69	631.73	1608.42	58.77	-8119.08	734.57	8853.65
6	48.84	7502.87	525.00	-6977.87	48.84	-8516.24	610.47	9126.71
5	34.96	5371.45	375.86	-4995.59	34.96	-6096.95	437.04	6533.99
4	23.26	4205.91	250.07	-3955.83	23.26	-4128.44	290.78	4419.23
3	12.86	2325.58	138.27	-2187.31	12.86	-2282.75	160.78	2443.54
2	5.95	1076.63	64.01	-1012.62	5.95	-1056.8	74.44	1131.24
			Total	-6212.81			Total	**94753.99**

Table 12: Model A Torsional Moment Analysis

It was found that the torsional moment in the east-west direction was larger. This is primarily due to the fact that the building shape does not step back on the perpendicular face, and therefore, the eccentricity stays the same the entire height of the building. The torsional moment in the other direction changes signs on the sixth floor, where the major building set back occurs, switching the eccentricity from negative to positive.

Shear

A building experiences a direct shear and possibly a torsional shear when a lateral load is applied. Direct shear is the force acting on the floor diaphragms applied directly to the lateral resisting members. To determine the direct shear, multiple the story shear by the relative stiffness of each participating member.

Torsional shear is the force cause by eccentricity. The torsional shear is similar to torsional moment, as it takes into account the difference in distance from the COM to the COR. The following equation was used to determine the torsional shear, and an example is shown at level 4, supporting level 5 diaphragm.

$$V_i = \frac{V_{tot} e d_i k_i}{J}$$

V_i = torsional shear of element i
V_{tot} = story shear
e = distance from COM to COR
d_i = distance from element I to COR
k_i = relative stiffness of element i
$J = \Sigma k_i \times d_i^2$

Torsional Shear in Shear Wall 1 at Level 4

		Factored Story Shear P (k)	Stiffness K_i	Distance from COM to COR e (in)	Distance from Wall i to COR di (in)	$(K_i)*(d_i)^2$	Torsional Shear (k)
Wall 1	E/W	703.22	167154	-177.47	-180	5415789600	346.67
Wall 2	E/W	703.22	167154	-177.47	180	5415789600	346.67
Wall 3	N/S	580.83	4717	180.8	-184.4	160463441.9	8.44
Wall 4	N/S	580.83	4717	180.8	184.4	160463441.9	8.44
Wall 5	N/S	580.83	4717	180.8	-184.4	160463441.9	8.44
Wall 6	N/S	580.83	4717	180.8	184.4	160463441.9	8.44
						10831579200.0	

Table 13: Torsional Shear Analysis

Shear Strength

The lateral shear forces on the wall were calculated for each wall participating in resisting the load. However, a strength check must be performed to verify that each wall is capable of transferring both direct and torsional shear. ACI 381-08 section 21.9.4.1, *Special Structural Walls and Coupling Beams Shear Strength* states:

$$V_n = A_{cv}\left[\left(\alpha_c \lambda \sqrt{f'_c}\right) + \left(\rho_t f_y\right)\right]$$

This equation recognizes the higher shear strength of walls with high shear-to-moment ratios. Where chord reinforcement is provided near wall edges in concentrated amounts for resisting bending moments, reinforcement should not be include in calculating ρ_t. However, the extra steel provided in the short shear walls is included for resisting shear forces and therefore shall be accounted for in thesis calculations. For comparison purposes, V_n is greater than the applied V_u in either case. Table 14 shows the shear strength of all six walls at level 4 supporting level 5. A diagram of the shear wall reinforcement is seen in figure 18, which details the reinforcement. Refer to the appendix for detailing information table.

	Shear Wall Strength Check at Level 4										
	Direct Shear (k)	Torsional Shear (k)	Total Shear V_u (k)	Length (in)	Thickness (in)	Vertical Reinforcing	A_{cv} (in^2)	α_c	ρ_t	ΦV_n (k)	
Wall 1 (E/W)	351.608	346.666	698.3	381.96	12	#5@12	4583.52	2	0.0043	1420.61	OK
Wall 2 (E/W)	351.608	346.666	698.3	381.96	12	#5@12	4583.52	2	0.0043	1420.61	OK
Wall 3 (N/S)	145.208	8.435	153.6	114.96	12	#7@12 & (16)#8 w/in 3' of each end	1379.52	2	0.0669	4310.34	OK
Wall 4 (N/S)	145.208	8.435	153.6	114.96	12	#6@12 & (8)#7 w/in 3' of each end	1379.52	2	0.0283	1919.17	OK
Wall 5 (N/S)	145.208	8.435	153.6	114.96	12	#6@12 & (12)#7 w/in 3' of each end	1379.52	2	0.0394	2608.93	OK
Wall 6 (N/S)	145.208	8.435	153.6	114.96	12	#7@12 & (12)#8 w/in 3' of each end	1379.52	2.0	0.0522	3402.16	OK

Table 14: Shear Strength Check

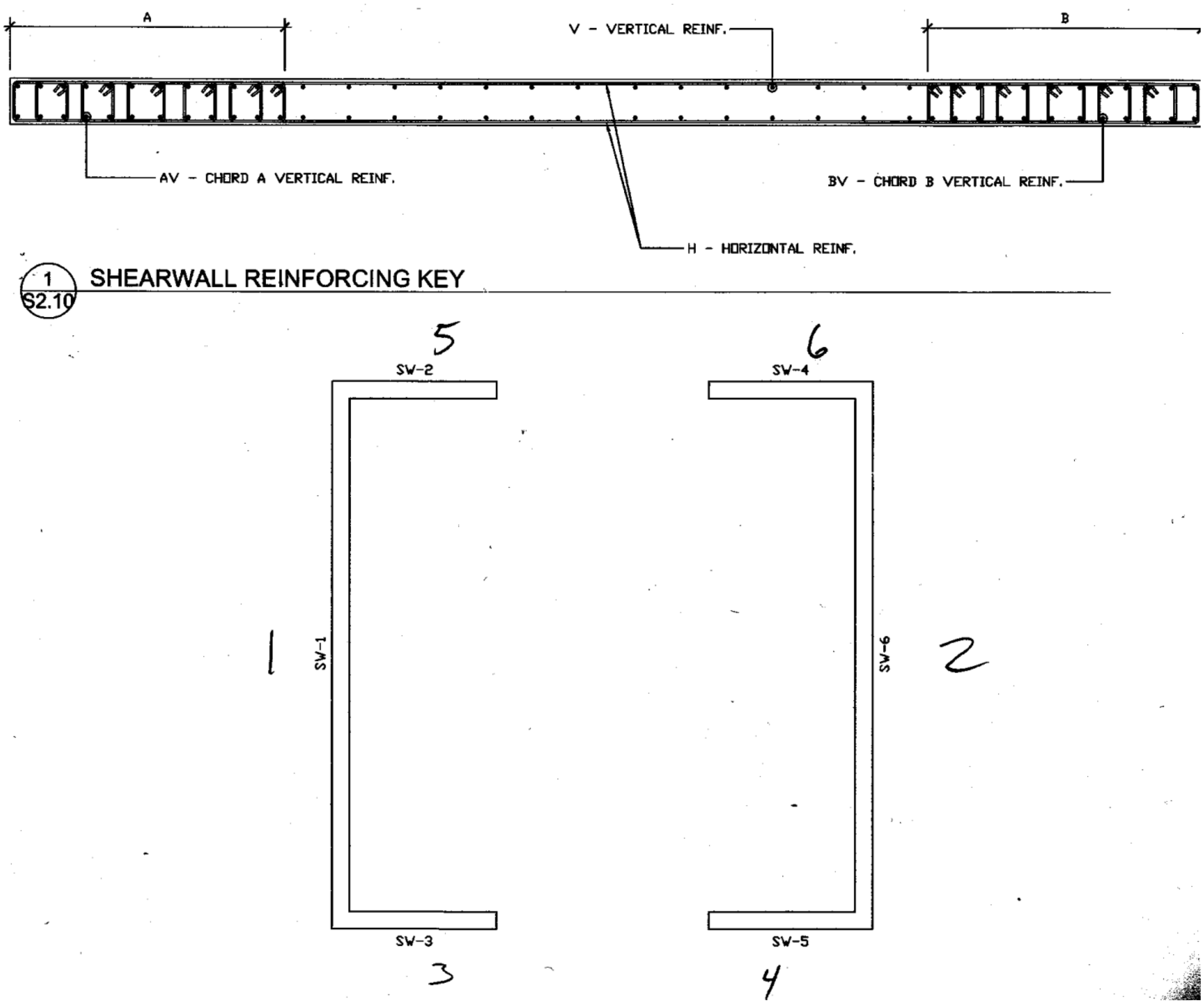


Figure 18: Shear Wall Reinforcement Layout

Drift and Displacement

Story drift and lateral displacements are not considered strength design concerns but are regarded as serviceability issues. Seismic drift is addressed in AISC 7-05 while wind drift is not addressed in the code. Seismic is limited based on building occupancy category and wind is normally limited to L/400, based on standard engineering practice over the years. In the case of 800 North Glebe:

Wind: Δ_{max} = (153.75’ x 12”/1’) / 400 = 4.61”

The wind displacement in the east-west direction, long shearwalls resisting, calculated by hand was found to be 0.4663” and the displacement from ETABS was calculated to be 0.6691”. Hand calculations were performed on individual shear walls to obtain approximate values using the following equation:

$$\Delta_{tot} = \Delta_{flexure} + \Delta_{shear} = \frac{Ph^3}{3E_c I} + \frac{2.78Ph}{E_r A}$$

Both of the calculated displacements at the main roof level are well below the allowable wind displacement of 4.61”. When looking at the north-south direction, short walls resisting, the displacement at the main roof level was found to be 3.10” from ETABS. However, as stated earlier, the seismic loading conditions had greater building displacements at the main roof level. Lateral displacement in the north-south direction was found to be 4.37” and 0.931” in the east-west direction. Both of these values are below the allowable drift limits.

Interstory drift was calculated by ETABS for both load cases and can be found in the drift tables 15 and 16 below. The limits for interstory drifts at typical floors (12’-9”) are 0.375” for wind and 2.00” for seismic. Interstory displacements from both ETABS models are significantly less than the allowable limits for both cases. The values from floor to floor do not deviate from one another by any significant value, with the exception of the 2^{nd} level.

Wind Interstory Drift (shearwalls only)			
Story	Allowable Drift (in)	Actual Drift X (in)	Actual Drift Y (in)
8th	0.375	0.002407	0.000947
7th	0.375	0.002375	0.000912
6th	0.375	0.002356	0.000863
5th	0.375	0.002188	0.000789
4th	0.375	0.002158	0.000688
3rd	0.375	0.001843	0.00056
2nd	0.375	0.000708	0.000174

Wind Interstory Drift (entire structure)			
Story	Allowable Drift (in)	Actual Drift X (in)	Actual Drift Y (in)
8th	0.375	0.001526	0.000232
7th	0.375	0.001577	0.000229
6th	0.375	0.001692	0.000219
5th	0.375	0.001621	0.000203
4th	0.375	0.001489	0.000177
3rd	0.375	0.001208	0.000141
2nd	0.375	0.000577	0.0001

Table 15: Wind Interstory Drift

Seismic Interstory Drift			
Story	Allowable Drift (in)	Actual Drift X (in)	Actual Drift Y (in)
8th	2.0	0.003494	0.002561
7th	2.0	0.003413	0.002454
6th	2.0	0.00322	0.002324
5th	2.0	0.002915	0.002109
4th	2.0	0.002516	0.001814
3rd	2.0	0.001943	0.001439
2nd	2.0	0.000875	0.000232

Seismic Interstory Drift (entire structure)			
Story	Allowable Drift (in)	Actual Drift X (in)	Actual Drift Y (in)
8th	2.0	0.002203	0.000261
7th	2.0	0.002245	0.000272
6th	2.0	0.002352	0.000277
5th	2.0	0.002192	0.000272
4th	2.0	0.001935	0.000255
3rd	2.0	0.001498	0.00023
2nd	2.0	0.000665	0.000163

Table 16: Seismic Interstory Drift

Overturning

Overturning moments are an important effect to consider because they affect various parts of the building, primarily the foundations. 800 North Glebe includes three levels of below grade parking supported by 30”x30” concrete columns tied to primarily 12’x12’x56” square concrete foundations. The outer columns along the east face of the building are tied into 72” diameter concrete caissons. The shear walls are supported by a 6’-0” thick concrete mat foundation 58’-6” wide by 45’-4” long. The moments create reactions at the base of the shear walls that are transferred to the foundations. Using load case 6, the maximum upward reactions on the returns was 400k, while the maximum reactions on the long shear walls was 170 k.
The size of the supporting foundations, in addition to the sheer mass of the building, will have the overturning effects of lateral loads creating a minimal effect.

Overturning moments can be calculated by multiplying the story forces by the height each level. The overturning moments for the lateral loads applied can be found in table 16. The overturning moment, similar to the other calculations, were preformed with wind loads being controlled by case 1. It was found that the east-west wind was greater than the north-south wind, as it was seen in the shear calculations as well. Again it is believed this is because of the large uniform surface area of the façades.

		North/South Wind		East/West Wind		Seismic	
Floor	Building Height (ft)	Story Force (k)	Overturning Moment (ft-k)	Story Force (k)	Overturning Moment (ft-k)	Story Force (k)	Overturning Moment (ft-k)
PH Roof	153.75	62.69	9639.20	75.73	11643.80	21.01	3230.68
PH	135.25	108.40	14661.52	130.99	17716.30	128.67	17402.73
10	121.5	90.46	10990.35	109.40	13291.76	105.52	12820.75
9	107.75	85.33	9194.83	103.29	11129.91	85.45	9207.10
8	95	80.18	7617.27	97.15	9229.53	72.50	6887.53
7	82.25	78.07	6421.63	94.71	7789.70	58.77	4833.48
6	69.5	75.69	5260.67	91.94	6390.11	48.84	3394.19
5	56.75	72.93	4138.82	88.74	5035.91	34.96	1984.18
4	44	69.59	3062.13	84.87	3734.12	23.26	1023.56
3	31.25	65.25	2039.14	79.83	2494.64	12.86	401.96
2	18.5	31.38	580.48	38.47	711.60	5.95	110.16
1	0	0.00	0.00	0.00	0.00	0.00	0.00
Σ Total Overturning Moment (ft-k)=			73606		**89167**		61296

Table 17: Overturning Moment Analysis Based on Factored Story Forces

Conclusion

Upon completion of the lateral load analysis of 800 North Glebe, a broader understanding of the lateral system was obtained. It was found that when looking at strength design, wind was greater, and when looking at serviceability issues, seismic was greater. Because of the differences among which loading condition was greater for strength and serviceability, no single load case would control the entire building design. However, thesis calculations were performed with the assumption that wind loading would play a greater role in lateral system design because of the significant surface area of the façade. This led to the load case of 0.9D + 1.6W being used.

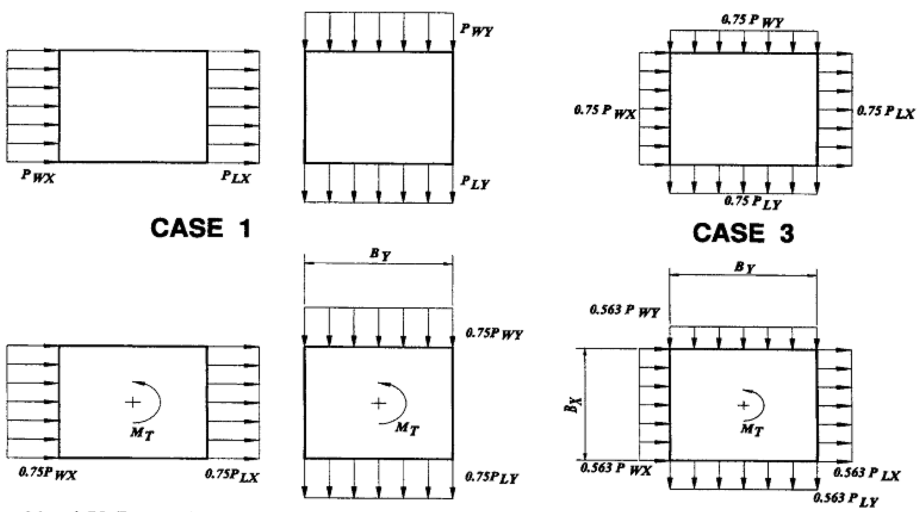
Lateral loads, either from wind or seismic, were designed to be resisted by the central core shearwalls. In reality the entire structure would help resist lateral loads through the post-tensioned concrete beams and normally reinforced concrete columns. Because it was believed the entire structure would participate, two separate models were created to compare drift, displacement, shear, torsion and period of vibrations. It was determined that the model with the entire structure had a lower building displacement, smaller shear and torsion forces on the shear walls and lower structural period of vibrations, resulting in a stiffer structure. If the building would actually being resisting lateral loads with the central core shear walls, the increased torsion would cause the exterior portions to "flap in the wind" and cause major damage to the façade and its connections.

A reason for only the shear walls being designed to support lateral loads may be because when only the shear walls are assumed to take the load, they are designed to carry much larger forces than they actually experience. Another reason for the only the shear walls being included is that special reinforcing shop drawings must be included for concrete moment frames. More steel must be detailed and inspected, which is time consuming and expensive.

In general, it was found that the lateral system for 800 North Glebe met strength and serviceability requirements. Further investigation will need to be conducted to determine if individual columns along the exterior of the building are adequately sized to support increased loading related to all forces acting upon them.

Appendix A: Lateral Loads

Main Wind Force Resisting System–Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$ $\quad e_X = \pm\,0.15\,B_X$

$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_Y = \pm\,0.15\,B_Y$

CASE 2

$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_X = \pm\,0.15\,B_X$ $\quad e_Y = \pm\,0.15\,B_Y$

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

52 ASCE 7-05

WIND

METHOD 2 ANALYTICAL PROCEDURE OF CHAP 6

SAMPLE CALC @ LEVEL 9 IN N/S DIRECTION

$K_z = 2.01 \cdot (z/z_g)^{2/\alpha}$

$K_{z_9} = 2.01 \left(\frac{107.75}{1200}\right)^{2/7} = 1.01$

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

$= (0.00256)(1.01)(1.00)(0.85)(90)^2(1.0) = 17.80$

$q_h = 0.00256\, K_h K_{zt} K_d V^2 I$

$= (0.00256)(1.12)(1.0)(0.85)(90)^2(1.0) = 19.7$

C17

$n_1 = 100/H = 100/153.75 = .65 \leq 1\,Hz$ ∴ Flexible

$g_Q = g_v = 3.4$

$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$

$= \sqrt{2\ln(2341.5)} + \frac{0.577}{\sqrt{2\ln(2341.5)}} = 4.086$

$\bar{z} = 0.6h = (0.6)(153.75) = 92.25 \geq z_{min}$ ✓ OK

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = (0.3)\left(\frac{33}{92.25}\right)^{1/6} = 0.253$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = (320)\left(\frac{92.25}{33}\right)^{1/3} = 450.78$

1

$$Q = \sqrt{\frac{1}{\left(1 + 0.63\left(\frac{B+H}{L_{\bar{z}}}\right)^{.63}\right)}}$$

$$= \sqrt{\frac{1}{1 + 0.63\left(\frac{202+153.75}{450.78}\right)^{.63}}} = 0.780$$

$V = 90$ mph from Fig 6-1

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = (0.45)\left(\frac{92.25}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 76.81$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.65)(450.78)}{76.81} = 3.82$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = \frac{(7.47)(3.82)}{(1+10.3(3.82))^{5/3}} = 0.0602$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{where } \eta = 4.6 n_1 h / \bar{V}_{\bar{z}}$$

$R_h = 0.153$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{where } \eta = 4.6 n_1 B / \bar{V}_{\bar{z}}$$

$R_B = 0.119$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{where } \eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$$

$R_L = 0.0358$

2

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)} = 0.17307$$

.2 for concrete

$$G_f = 0.925 \left[\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z} \right] = \boxed{0.81912}$$

PRESSURE COEFF: C_p (FIGURE 6-6)

→ WINDWARD $C_p = .8$

→ LEEWARD (N/S) $C_p = -.45$ w/ $L/B = 1.16$

(E/W) $C_p = -.5$ w/ $L/B = .86$

VELOCITY PRESSURE EXPOSURE COEFF @ HT Z : K_z (Table 6-3)

$$K_z = [2.01]\left[\frac{h_z}{1200}\right]^{2/7} = (.26511)(h_z)^{2/7}$$

$$\text{@ level 9: } K_z = (.62511)(107.75)^{2/7} = \boxed{1.01}$$

VELOCITY PRESSURE @ HT Z: q_z (Eq 6-15)

$$q_z = (.00256)(K_z)(K_{zt})(K_d) V^2 I$$

$$= (.00256)(K_z)(1.0)(.85)(90)^2(1.0) = 17.6256(K_z)$$

$$\text{@ level 9: } q_z = (17.6256)(1.01) = \boxed{17.7936 \ \#/ft^2}$$

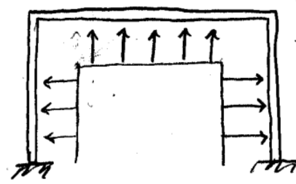
$$\text{@ MEAN ROOF HT: } q_h = (17.6256)(1.11747) = \boxed{19.6961 \ \#/ft^2}$$

3

PRESSURES: p_z & p_h (Eq 6-19)

$p_z = q_z G_f C_p - \cancel{q_h (GC_{pi})}$ WINDWARD

$p_h = q_h G_f C_p - \cancel{q_h (GC_{pi})}$ LEEWARD



* INTERNAL PRESSURES CANCEL

N/S: $W_w \Rightarrow p_z = q_z(.8191)(.8) = \boxed{.6553 q_z}$

$L_w \Rightarrow p_h = (19.7)(.8191)(-.45) = \boxed{-7.26 \text{ psf}}$

E/W: $W_w \Rightarrow p_z = q_z(.8175)(.8) = \boxed{.654 q_z}$

$L_w \Rightarrow p_h = (19.7)(.8175)(-.5) = \boxed{-8.05 \text{ psf}}$

FORCES OF WINDWARD PRESSURE (kips):

@ level 9 (N/S): $\frac{215}{1000}\left[(11.66 \text{ psf})\left(\frac{13.75'}{2}\right) + (11.25)\left(\frac{12.75'}{2}\right)\right]$

$\Rightarrow (.215)[80.16 + 71.72] = \boxed{32.65^k}$

STORY SHEAR (kips):

@ level 9 (N/S): sum level forces above

$\Rightarrow (24.75) + (42.58) + (35.07) = \boxed{102.4^k}$

4

Floor Perimeter (ft)	Floor Area	Floor to Floor Height	Superimposed Dead Load (#)	Floor Weight (#)	Column Weight Above (#)	Column Weight Below (#)	Beam Weight (#)	Curtain Wall Weight (#/ft)	Curtain Wall Weight (#)
250	3208.0	0.0	132550.0	363476.3	0.0	155592.7	0.0	185	46250
717	25572.5	18.5	639313.2	3118897.1	155592.7	188187.9	687975.0	322.5	231232.5
755	26640.5	13.8	666012.9	3036057.4	188187.9	188187.9	714150.0	275	207625
755	26559.5	13.8	663987.9	3028407.4	188187.9	175312.9	714150.0	265	200075
760	26607.5	12.8	665187.9	3033132.4	175312.9	182868.7	972400.0	255	193800
766	27806.8	12.8	695169.1	3167335.5	182868.7	182868.7	972400.0	255	195330
892	32687.8	12.8	817194.1	3803996.8	182868.7	230150.0	714150.0	255	227460
898	32545.3	12.8	813633.0	3787743.5	230150.0	230150.0	729112.5	255	228990
898	33394.3	12.8	834858.0	3901671.0	230150.0	238782.8	717637.5	255	228990
869	32750.3	12.8	818757.3	3784200.1	238782.8	238782.8	747000.0	255	221595
880	35847.3	12.8	896182.3	4248281.4	238782.8	389173.3	747000.0	312.5	275000
		18.5	0.0	0.0	389173.3	0.0	0.0	185	0
			7642845.7	35273198.8	2400057.6	2400057.6	7715975.0	3075.0	2256347.5

Shear Wall Weight (#)	Total Building Weight (#)	Total Building Weight (k)	
0	698054.0	698	
282994.50	5304515.4	5305	
210333.75	5210829.8	5211	
195036.75	5165422.8	5165	
195036.75	5417993.6	5418	
201537.98	5597765.0	5598	
201537.98	6177612.5	6178	
201537.98	6221571.9	6222	
201537.98	6353882.2	6354	
201537.98	6250911.0	6251	
201537.98	6996270.2	6996	
0	389358.3	389	
2092629.6	59784186.7	=	59784

SEISMIC | ASCE 7-05 | 1

SEISMIC GROUP II
SITE CLASS D
IMPORTANCE 1.0

S_S & S_1 used from structural notes based on discussion w/ engineer & provided info

$S_S = 0.179$ (@ .2sec)

$S_1 = 0.063$ (@ 1.0sec)

SITE COEFF:

$F_A = 1.6$ (Table 11.4-1)

$F_V = 2.4$ (Table 11.4-2)

SITE MODIFICATION ACCELERATION

$S_{MS} = F_A S_S = (1.6)(.179) = .2864$
$S_{M1} = F_V S_1 = (2.4)(.063) = .1512$
∴ Design Category B (Table 11.6-2)

$x = .75$ & $C_t = 0.02$ b/c bldg supported laterally by concrete shear walls

$T_a = .1N = (.1)(11 \text{ levels}) = \boxed{1.1}$ (Eq 12.8-8)

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{.101}{.191} = .529 \Rightarrow (.8)T_s = .423$

$T = T_a < C_u T_a = (1.7)(1.1) = \boxed{1.868}$

1

$C_s = \min \begin{cases} S_{DS}/(R/I) = (.191)/(5.5/1.0) = .347 \\ S_{D1}/T(R/I) = \dfrac{(.101)}{(1.868)(5.5/1.0)} = .0098 \end{cases}$

BUT NOT LESS THAN .010 * USE

$V = C_s W_{tot}$ FROM Bldg Wt in EXCEL

$= (.01)(59780) \doteq \boxed{597.8^k}$

SHOWN IN EXCEL

@ level 9: $h_x^k = (107.75)^{1.684} = 2644.77$

$w_x h_x^k = (5165.42)(2644.77) = 13661341$

$C_{vx} = \dfrac{w_x h_x^k}{\Sigma w_i h_i^k} = \dfrac{(13661341)}{95574826} = 0.143$

$F_x = C_{vx} V = (.143)(597.8) = \boxed{85.45^k}$

Structural Seismic Data from RAM Model

Seismic ASCE 7-02 / IBC 2003 Equivalent Lateral Force

Site Class: D	Importance Factor: 1.00	Ss: 0.179 g	S1: 0.063 g
Fa: 1.600	Fv: 2.400	SDs: 0.191 g	SD1: 0.101 g

Seismic Use Group: II Seismic Design Category: B
Provisions for: Force
Ground Level: Base

Dir	Eccent	R	Ta Equation	Building Period-T
X	+ And -	5.5	Alternate Eq	Calculated
Y	+ And -	5.5	Alternate Eq	Calculated

Dir	Ta	Cu	T	T-used	Eq95521-1	Eq95521-2	Eq95521-3	k
X	1.100	1.698	5.897	1.868	0.035	0.010	0.0084	1.684
Y	1.100	1.698	2.198	1.868	0.035	0.010	0.0084	1.684

Total Building Weight (kips) = 73181.57

Appendix B: Torsion

	North-South Direction (Short Wall Resisting)				East-West Direction (Long Wall Resisting)			
Floor	Floor Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	M_t total (ft-k)	Floor Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	M_t total (ft-k)
PH Roof	21.01	-349.23	225.89	575.11	21.01	-2903.1	262.66	3165.76
Main Roof	128.67	-2138.51	1383.21	3521.72	128.67	-17777.2	1608.39	19385.55
10	105.52	-1753.75	1134.35	2888.10	105.52	-14578.7	1319.01	15897.73
9	85.45	-1420.16	918.57	2338.73	85.45	-11805.6	1068.11	12873.71
8	72.50	-1204.95	779.38	1984.33	72.50	-10016.6	906.25	10922.89
7	58.77	-976.69	631.73	1608.42	58.77	-8119.08	734.57	8853.65
6	48.84	7502.87	525.00	-6977.87	48.84	-8516.24	610.47	9126.71
5	34.96	5371.45	375.86	-4995.59	34.96	-6096.95	437.04	6533.99
4	23.26	4205.91	250.07	-3955.83	23.26	-4128.44	290.78	4419.23
3	12.86	2325.58	138.27	-2187.31	12.86	-2282.75	160.78	2443.54
2	5.95	1076.63	64.01	-1012.62	5.95	-1056.8	74.44	1131.24
			Total	-6212.81			Total	94753.99

Appendix C: Shear

SHEAR 1

CONTROLLING LOAD CASES:

N/S : 0.9D + 1.6W + 1.6H

E/W : 0.9D + 1.6W + 1.6H

DIRECT SHEAR = (factored story force) * (Rel Stiffness %)/100

TORSIONAL SHEAR = $\dfrac{k_i d_i e P}{\Sigma k_j d_j^2}$

k_i : Rel stiffness of elem i
d_i : dist to COR from elem
e : dist from COM to COR
P : story shear load

WALL 1 @ level 4 (E/W)

Story Shear P = 703.22^k

COR = 1260"

COM = 1082.53"

$e = 1082.53 - 1260" = -177.47"$

$d_i = 1260 - 1080 = 180"$

$\Sigma k_j d_j^2 = 66418.1$ (from excel)

$$F_{it} = \frac{(167154)(-180)(-177.47)(703.22^k)}{1.083 \times 10^{10}} = 346.7^k$$

$$F_{id} = \frac{k_{iy}}{\Sigma k_{jy}} P = \frac{(167154)(703.22)}{(167154+167154)} = 351.6^k$$

* COM & COR based on ETABS output because COM was not hand-calced due to the irregular shape.

SHEAR STRENGTH (ACI 318-08) section 21.9.4

$$\phi V_n = \phi A_{cv}\left[\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y\right]$$

$\phi = .75$
A_{cv} = conc gross area
$\alpha_c = 2.0$ for $h/\ell \geq 2.0$
$\rho_t = \dfrac{A_v}{s\,h}$ — h: wall thickness, s: reinf spacing

ϕV_n of wall 1 @ level 4

$\phi = .75$

$A_{cv} = (31.83' * 12'')\cdot(12'') = 4583.52\ in^2$

$\alpha_c = 2.0$

$\rho_t = \dfrac{(2)(.31)}{(12'')(12'')} = .004306$

$$\phi V_n = (.75)(4583.52)\left[(2.0)\frac{\sqrt{6000}}{1000} + (.004306)(60)\right]$$

$$\boxed{\phi V_n = 1420.71^k}$$

Appendix D: Drift and Displacement

DISPLACEMENT (Wall 1) 1

$$\Delta = \Delta_{flex} + \Delta_{shear} = \frac{Ph^3}{3EI} + \frac{2.78\,Ph}{E_r A}$$

$E_c = 57000\sqrt{f'_c} = 4.42 \times 10^6$ psi

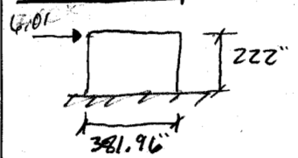
$E_r = .4 E_c = 1.77 \times 10^6$ psi

$A = lt = (381.96)(12) = 4583.12$

$I = \frac{(12)(381.96)^3}{12} = 5.57 \times 10^7 \text{ in}^4$

- Ex for wall 1 for first five levels. The remaining can be found in excel.
- Use of wind lateral forces for drift & displacement
- Wall 1 recieves 1/2 of lateral force based on relative stiffness between wall 1 & 2.

Level 2 disp

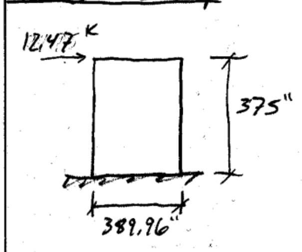


$$\Delta_{flex} = \frac{(6.01)(222)^3}{3(4.42\times10^3)(5.57\times10^7)} = .000089''$$

$$\Delta_{shear} = \frac{(2.78)(12.02)(222)}{(1.77\times10^3)(4583.12)} = .000458''$$

$$\Delta = .00055''$$

level 3 disp



$$\Delta_{flex} = \frac{(24.95)(375)^3}{(3)(4.42\times10^3)(5.57\times10^7)} = .000891''$$

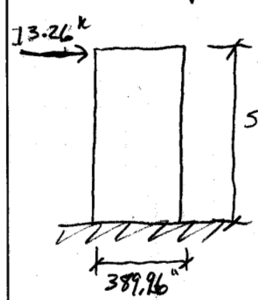
$$\Delta_{shear} = \frac{(2.78)(24.95^k)(375)}{(1.77\times10^3)(4583.12)} = .001606''$$

$$\Delta = .00250''$$

1

2

level 4 disp

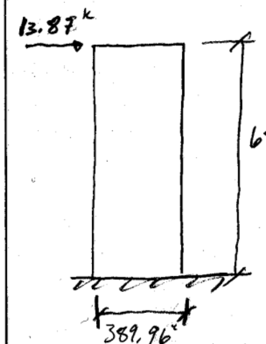


$$\Delta_{flex} = \frac{(26.52)(528)^3}{3(4.42 \times 10^3)(5.57 \times 10^7)} = .00264''$$

$$\Delta_{shear} = \frac{(2.78)(26.52)(528)}{(1.77 \times 10^3)(4583.12)} = .002405''$$

$$\Delta = .00505''$$

level 5 disp



$$\Delta_{flex} = \frac{(27.73)(681)^3}{3(4.42 \times 10^3)(5.57 \times 10^7)} = .00593''$$

$$\Delta_{shear} = \frac{(2.78)(27.73)(681)}{(1.77 \times 10^3)(4583.12)} = .00324''$$

$$\Delta = .00918''$$

$$\Delta_{tot\ bldg} = .46628''$$

$$\Delta_{allow} = L/400 = \frac{(1845'')}{400} = 4.6125'' \geq .46628''$$

✓ OK

ETABS wind drift in the "Y" direction was found to be .6691"

2

Wall 1 Displacement Calculation

Floor	Story Height (ft)	Height Above Ground h_x (in)	Floor Lateral Force (k)	Ec (ksi)	Er (ksi)	I (in^4)	A (in^2)	Length (in)	Δ Flex (in)	Δ Shear (in)	Lateral Displacement (in)
PH Roof	0	1845	11.83	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.10068	0.00750	0.10818
Main Roof	222	1623	20.47	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.11855	0.01141	0.12995
10	165	1458	17.09	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.07178	0.00856	0.08033
9	165	1293	16.14	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.04727	0.00717	0.05443
8	153	1140	15.18	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.03047	0.00594	0.03641
7	153	987	14.80	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.01928	0.00502	0.02429
6	153	834	14.37	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.01129	0.00411	0.01541
5	153	681	13.87	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.00593	0.00324	0.00918
4	153	528	13.26	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.00264	0.00240	0.00505
3	153	375	12.47	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.00089	0.00161	0.00250
2	153	222	6.01	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.00009	0.00046	0.00055
1	222	0	0.00	4.42E+3	1.77E+3	5.57E+7	4583.5	381.96	0.000	0.000	0.000
								Σ Displacement (in)=			**0.46628**

ETABS Displacement and Story Drift

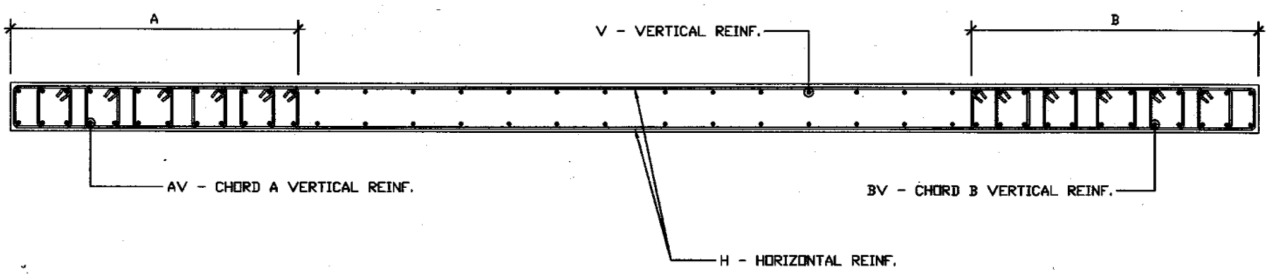
		Shear Walls Only				Entire Structure			
Story	**Load**	**UX**	**UY**	**DriftX**	**DriftY**	**UX**	**UY**	**DriftX**	**DriftY**
MAIN ROOF	LATERALX	4.3742	-0.0064	0.003307		2.7431	-0.0589	0.001882	
					0.000126				0.000273
	LATERALY	-0.0278	0.9307	0.000951		-0.0583	0.3094	0.000061	
					0.0026				0.000205
	WINDX	3.1012	0.0153	0.002279		1.9576	-0.0396	0.001294	
					0.000077				0.000184
	WINDY	-0.0068	0.6691	0.000228		-0.0646	0.2646	0.000075	
					0.00099				0.000207
10TH	LATERALX	3.8772	-0.0048	0.003411		2.4685	-0.0495	0.002012	
					0.000125				0.000292
	LATERALY	-0.0245	0.8137	0.000965		-0.0499	0.2804	0.000055	
					0.002611				0.000221
	WINDX	2.758	0.0163	0.00234		1.7685	-0.0329	0.00138	
					0.000063				0.000196
	WINDY	-0.006	0.585	0.000229		-0.056	0.24	0.000076	
					0.000971				0.000229
9TH	LATERALX	3.3641	-0.0032	0.003482		2.1755	-0.0407	0.00212	
					0.000123				0.000302
	LATERALY	-0.0212	0.6989	0.000967		-0.0416	0.2466	0.00006	
					0.00261				0.000244
	WINDX	2.4045	0.0171	0.002389		1.5672	-0.0267	0.001457	
					0.000054				0.000203
	WINDY	-0.0052	0.504	0.000229		-0.0473	0.212	0.000073	
					0.000967				0.000232
8TH	LATERALX	2.8402	-0.0017	0.003494		1.8668	-0.0324	0.002203	
					0.000117				0.000308
	LATERALY	-0.0179	0.5849	0.000951		-0.0336	0.2115	0.000063	
					0.002561				0.000261
	WINDX	2.0429	0.0177	0.002407		1.3547	-0.0208	0.001526	
					0.000042				0.000208
	WINDY	-0.0044	0.4236	0.000226		-0.0388	0.183	0.000069	
					0.000947				0.000232

Story	Load	UX	UY	DriftX	DriftY	UX	UY	DriftX	DriftY
7TH	LATERALX	2.3141	-0.0002	0.003413		1.546	-0.0247	0.002245	
					0.000113				0.000323
	LATERALY	-0.0147	0.4739	0.000914		-0.0262	0.1759	0.000064	
					0.002454				0.000272
	WINDX	1.6776	0.0183	0.002375		1.132	-0.0153	0.001577	
					0.000039				0.000224
	WINDY	-0.0036	0.3455	0.000219		-0.0307	0.1534	0.000064	
					0.000912				0.000229
6TH	LATERALX	1.8014	0.0015	0.00322		1.2362	-0.0144	0.002352	
					0.00005				0.000326
	LATERALY	0.0967	0.3914	0.001662		-0.0131	0.1418	0.000065	
					0.002324				0.000277
	WINDX	1.3401	0.0237	0.002356		0.9184	-0.007	0.001692	
					0.000093				0.000245
	WINDY	0.0242	0.2764	0.0004		-0.0239	0.1236	0.000072	
					0.000863				0.000219
5TH	LATERALX	1.313	0.0023	0.002915		0.9122	-0.0097	0.002192	
					0.000006				0.000304
	LATERALY	0.0716	0.2868	0.000794		-0.0081	0.1072	0.000063	
					0.002109				0.000272
	WINDX	0.9913	0.0222	0.002188		0.6867	-0.0038	0.001621	
					0.000201				0.000247
	WINDY	0.0181	0.2048	0.000194		-0.0165	0.0946	0.000059	
					0.000789				0.000203
4TH	LATERALX	0.8678	0.0024	0.002516		0.6103	-0.0058	0.001935	
					0.00003				0.000257
	LATERALY	0.0575	0.194	0.00148		-0.0037	0.0748	0.000061	
					0.001814				0.000255
	WINDX	0.6696	0.0194	0.002158		0.4667	-0.0014	0.001489	
					0.000318				0.000236
	WINDY	0.0148	0.1401	0.000369		-0.0102	0.0671	0.000045	
					0.000688				0.000177

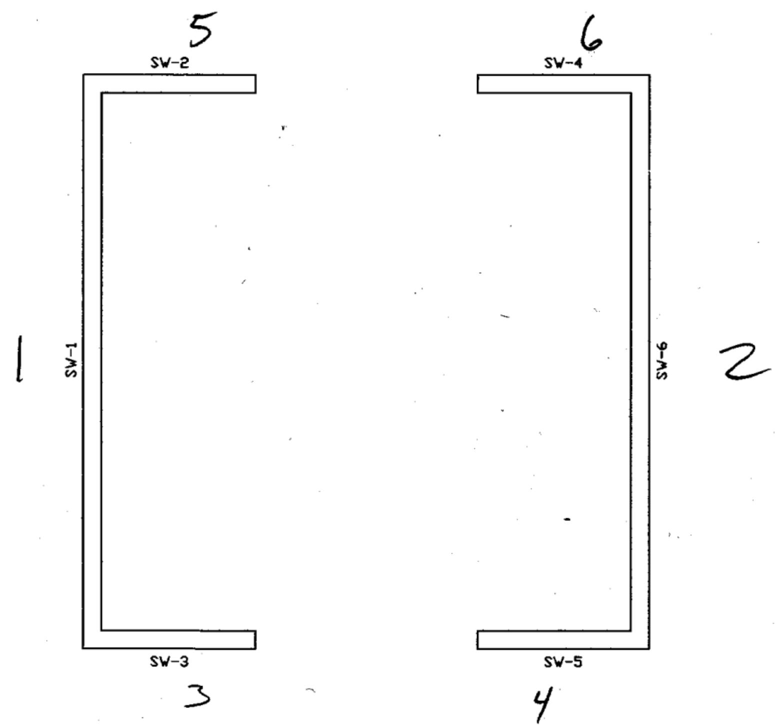
Story	Load	UX	UY	DriftX	DriftY	UX	UY	DriftX	DriftY
3RD	LATERALX	0.487	0.0019	0.001943		0.346	-0.003	0.001498	
					0.00005				0.000193
	LATERALY	0.0342	0.1141	0.001184		-0.0011	0.0461	0.00007	
					0.001439				0.00023
	WINDX	0.3842	0.0143	0.001843		0.2691	0	0.001208	
					0.000406				0.000207
	WINDY	0.009	0.084	0.000306		-0.0051	0.0421	0.000025	
					0.00056				0.000141

Story	Load	UX	UY	DriftX	DriftY	UX	UY	DriftX	DriftY
2ND	LATERALX	0.1968	0.0011	0.000875		0.1408	-0.001	0.000665	
					0.000005				0.00005
	LATERALY	0.0156	0.0519	0.000069		0.0001	0.0226	0.00007	
					0.000232				0.000163
	WINDX	0.1594	0.0078	0.000708		0.1113	0.0006	0.000577	
					0.000035				0.000092
	WINDY	0.0042	0.0392	0.000019		-0.0017	0.0209	0.000012	
					0.000174				0.0001

Appendix E: Shearwall Reinforcing Details



1 S2.10 SHEARWALL REINFORCING KEY



CONCRETE SHEAR WALL SCHEDULE

F'c	SW1						SW2						SW3						SW4						SW5						SW6					
	A (ft.)	AV	B (ft.)	BV	H	V	A (ft.)	AV	B (ft.)	BV	H	V	A (ft.)	AV	B (ft.)	BV	H	V	A (ft.)	AV	B (ft.)	BV	H	V	A (ft.)	AV	B (ft.)	BV	H	V	A (ft.)	AV	B (ft.)	BV	H	V
6 ksi	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#7	3'-0"	(12)#7	#4@12	#6@12	3'-0"	(16)#8	3'-0"	(16)#8	#4@10	#7@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	2'-0"	(8)#7	2'-0"	(8)#7	#4@12	#6@10	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#7	3'-0"	(12)#7	#4@10	#6@12	3'-0"	(16)#8	3'-0"	(16)#8	#4@10	#7@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	2'-0"	(8)#7	2'-0"	(8)#7	#4@12	#6@10	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#7	3'-0"	(12)#7	#4@10	#6@12	3'-0"	(16)#8	3'-0"	(16)#8	#4@10	#7@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	2'-0"	(8)#7	2'-0"	(8)#7	#4@12	#6@10	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#7	3'-0"	(12)#7	#4@10	#6@12	3'-0"	(16)#8	3'-0"	(16)#8	#4@10	#7@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	2'-0"	(8)#7	2'-0"	(8)#7	#4@12	#6@10	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#7	3'-0"	(12)#7	#4@10	#6@12	3'-0"	(16)#8	3'-0"	(16)#8	#4@10	#7@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	2'-0"	(8)#7	2'-0"	(8)#7	#4@12	#6@10	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	3'-6"	(24)#8	3'-6"	(24)#8	#4@10	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	3'-6"	(24)#8	3'-6"	(24)#8	#4@10	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	3'-6"	(24)#8	3'-6"	(24)#8	#4@10	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	--	--	--	--	#4@12	#5@12
6 ksi	--	--	--	--	#4@12	#5@12	3'-0"	(12)#8	3'-0"	(12)#8	#4@12	#7@12	3'-6"	(24)#8	3'-6"	(24)#8	#4@10	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	3'-6"	(16)#8	3'-6"	(16)#8	#4@12	#7@12	--	--	--	--	#4@12	#5@12