LETTER OF TRANSMITTAL

Date: November 17, 2014

To: Dr. Thomas Boothby
TEBARC@engr.psu.edu

From: Nick Dastalfo
njd5133@psu.edu

Dear Dr. Boothby,

The enclosed documents include my Structural Technical Report 4 for AE481W – Senior Thesis. Technical Report 4 includes a structural analysis of the lateral system of 8621 Georgia Avenue in Silver Springs, Maryland.

This report includes a complete lateral analysis of the building. The analysis includes results obtained via 3D modeling software as well as various spot checks of the performed analysis. Information for the strength, drift, and story drift due to wind and seismic loading is presented. An overturning and foundation investigation was also performed under these loading conditions.

Thank you for taking the time to read and review my report. I am eagerly looking forward to discussing the project with you in the future.

Sincerely,

Nick Dastalfo

Technical Report 4

8621 Georgia Avenue
Silver Spring, Maryland

Nick Dastalfo | Structural

Advisor: Dr. Thomas Boothby

November 17, 2014

Executive Summary

The building at 8621 Georgia Avenue is proposed to be built on an existing 0.69 acre parking lot located in the downtown business district of Silver Spring, Maryland. The 17 story, 347,000 ft^2 project will create more downtown multi-family housing and parking for the booming region. The project has recently finished the permit phase of development and is nearly the start of construction.

The building will be the tallest of the surrounding buildings and will be clearly visible along specific urban view corridors and pedestrian heavy areas. Therefore, detailed focus was cast on the architectural impact of the form of the glass curtain wall clad building in these locations. Being the tallest building in the area came along with the challenges of remaining under the zoning height restriction of the area. Efforts were made to decrease the floor to floor height by using post tensioning in order to squeeze the most amount of floors into the building. The height and exposure of the building will both be a factor in the applied wind load it experiences.

The first four stories used for parking, retail, and café have flat plate concrete slab floors with minimal use of concrete drop panels and beams when necessary. The 5th through 17th floor utilize post-tensioned concrete flat plates with spans varying from 15'-10" to 24'-0" throughout these 12 floors of apartments. The variation in column locations and the use of transfer girders were eliminated due to strategic placing of columns in a regular grid that was appropriate for both the parking garage and the apartments. The primary lateral system consists of a configuration of 14 shear walls and occasional drop beams.

The building was designed considering live loads, gravity loads, snow loads, wind loads, seismic loads, and lateral loads. The lateral force resisting system in the building is primarily made up of shear walls around the two stair/elevator towers of the structure. The lateral system will be analyzed in greater detail using computer modeling software.

The design for this building was governed by the International Building Code 2012 as well as the 'Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). These codes reference other standards that were integral in the design process and include ACI318-11 and parts 1-5 of the ACI Manual of Standard Practice, PTI's "Post Tensioning Manual, 6th Edition, the "Manual of Standard Practice" from CRSI, and AISC's Steel Construction Manual, 14th Edition.

This report will cover all of these features and many more, in greater detail.

Table of Contents

8621 Georgia Avenue
Silver Spring, Maryland

General Building Data:

Building Height: 161 feet
Number of Stories: 17 floors
Size: 347,009 ft^2
Cost: $51 million
Occupancy: Mixed Use
-Residential, Parking Garage, Retail
Construction Team:
Owner – FP Wilco, LLC
Architect – BBG-BBGM
Developer/Contractor – Foulger-Pratt, LLC
Structural Eng. – Holbert Apple Associates

Architecture:

The façade of the building brings a refreshing modern addition to the skyline of the developing city of Silver Spring. The position of the building takes advantage of two major view corridors in the urban fabric and has an inviting present on the busy Georgia Avenue.

Structural Systems:

This concrete building utilizes mild reinforced cast-in-place two way flat slabs with full drop panels for the parking garage on floors 1-4 and a post-tensioned cast-in-place two way flat slab for the remainder of the apartment level floors. The lateral system is comprised of 14 concrete shear walls located around stair and elevator cores. The column grid is relatively square vary from 16-24' in length.

Construction:

Construction is scheduled to be 24-28 months and will begin in early 2015. Important factors will be coordinating work with the surrounding existing buildings on all sides and impact of the high water table on the foundation construction.

MEP:

Floors 1-4 (parking garage) will be open and designed as an open structure. Each apartment will be conditioned by a conventional split system heat pump with back-up electric heat. Outdoor air is provided by an exterior louver.

Lighting / Electrical:

The building will have 277/480V as the primary power with 480-120/208V transformers. Branch lighting/power panels will be placed in the cellar and every 4^{th} apartment level. These panels serve the local receptacles, lighting, and HVAC units.

Project Sponsor: Holbert Apple Associates

Site and Location Plan

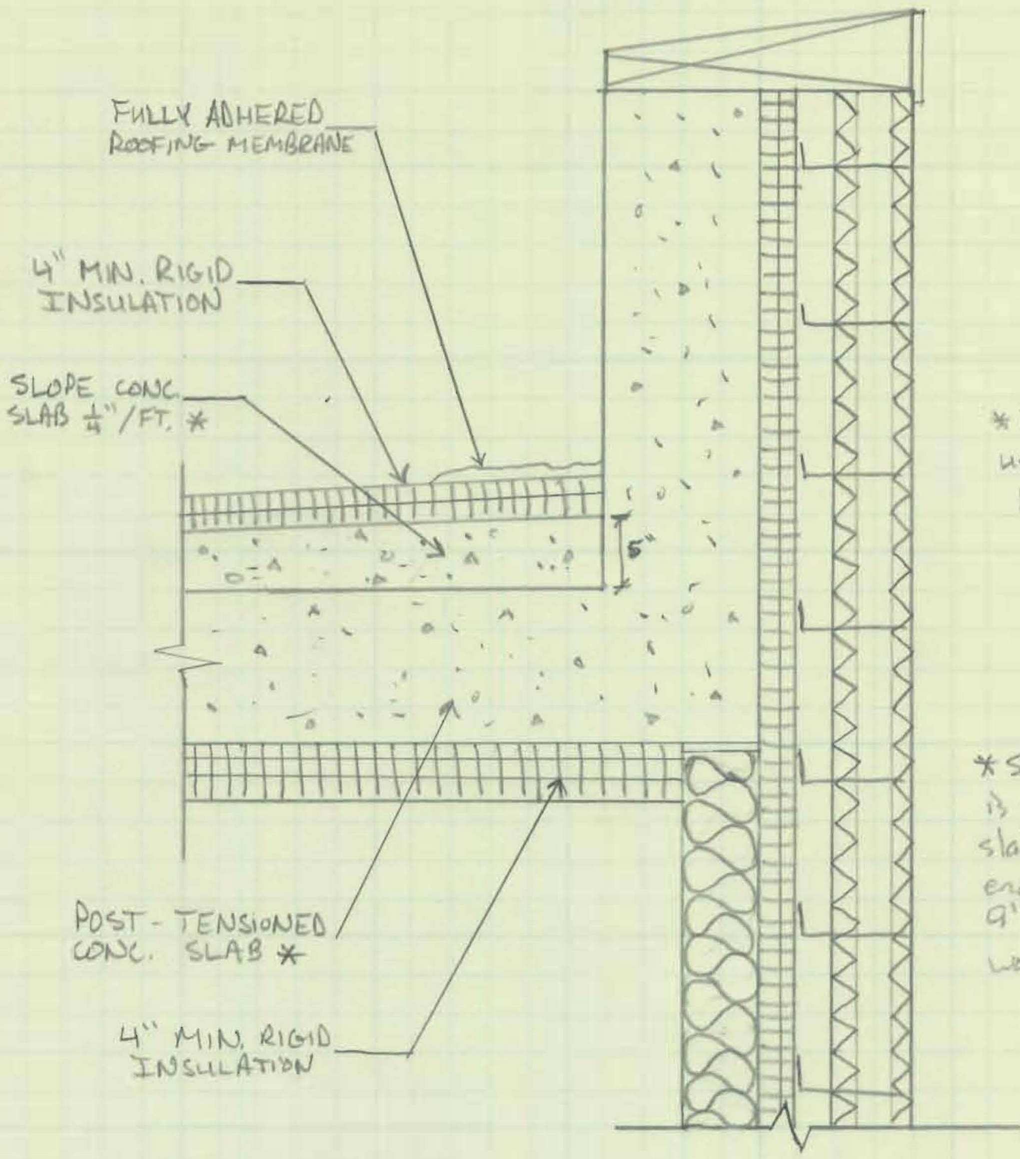
Documents Referenced for Report

Shown below is a list of the design codes, standards or other references that were used in the structural analysis of 8621 Georgia Avenue for Technical Report 4.

- American Society of Civil Engineers
  - ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
- Montgomery County Building Codes and Standards
- American Concrete Institute
  - ACI 318-08:Building Code Requirements for Structural Concrete
- International Building Code 2012
- 8621 Georgia Avenue Silver Spring, MD
  - Construction Drawings
  - Specifications
  - Correspondence with Project Engineers

Typical Roof Dead Load on 17th Floor

Detail Cross-Section at Parapet



* Lightweight Concrete used for roof sloping LWC = 120 PCF

* Slab on south wing is 9", whereas the slab on the north end is 7½". Use 9" to determine worst case loading

Uniformly Distributed Dead Loads

- Rigid Insulation (2×4 = 8") = 12 psf
- 5" LW Concrete = 50 psf
- 9" NW Concrete = 112.5 psf
- Roofing Membrane = 2 psf
- Collateral = 3.5 psf

Total = 180 psf

Typical Roof Live Loads

ASCE 07-10, Table 4-1: Minimum Uniformly Distributed Live Loads

Ordinary Flat Roof .... 20 psf

* See Snow loads, for controlling roof live load, where applicable.

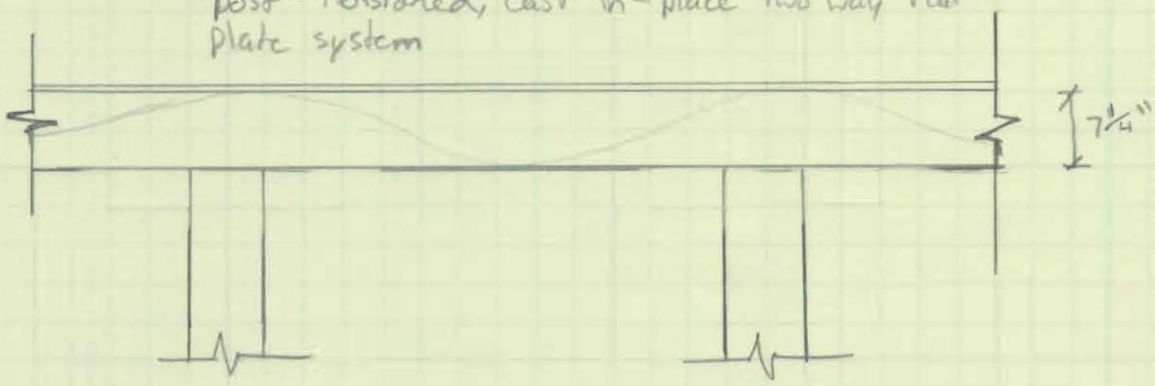
* The Engineer also added an additional 30 psf superimposed dead load. This design decision may have been made for a number of foreseeable factors such as: snow accumulation, ponding, roof maintenance, etc.

* The Engineer also chose to increase the minimum live load, provided in ASCE 07-10, to 30 psf.

Floor Dead Loads

Apartment Levels

- post-tensioned, cast-in-place two way flat plate system



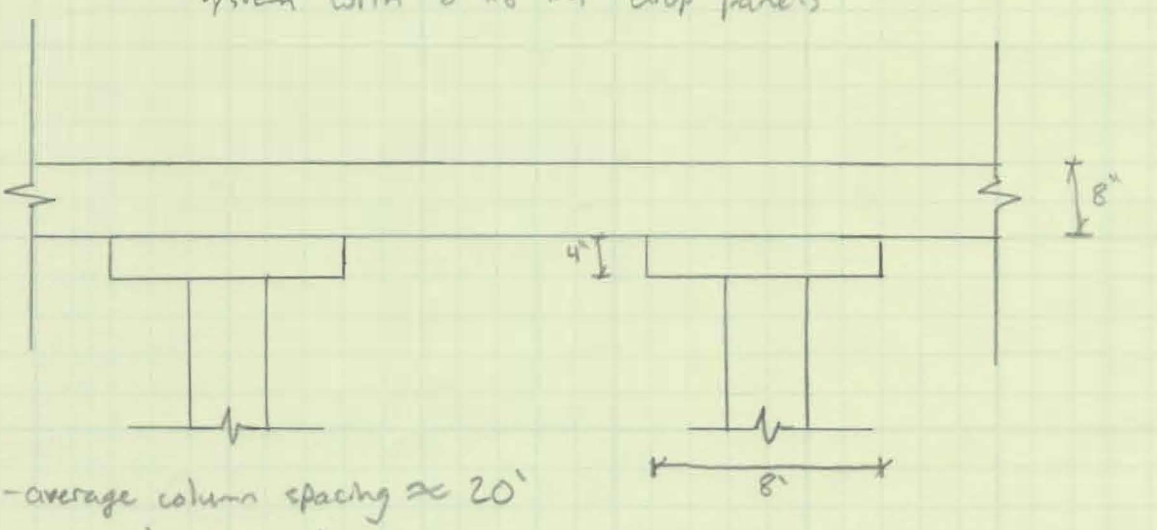
Uniformly Distributed Dead Loads

- 7¼ Concrete = 90.6 psf
- Floor Finish = 2 psf
- Collateral = 5 psf

Total = 97.6 psf

Parking Garage

- 8" mild-reinforced cast-in-place two way flat slab system with 8'×8'×4" drop panels



- average column spacing ≈ 20'

$\frac{8'}{20'}(12") + \frac{12'}{20'}(8") = 9.6" \rightarrow 10"$ avg. thickness

Uniformly Distributed Loads

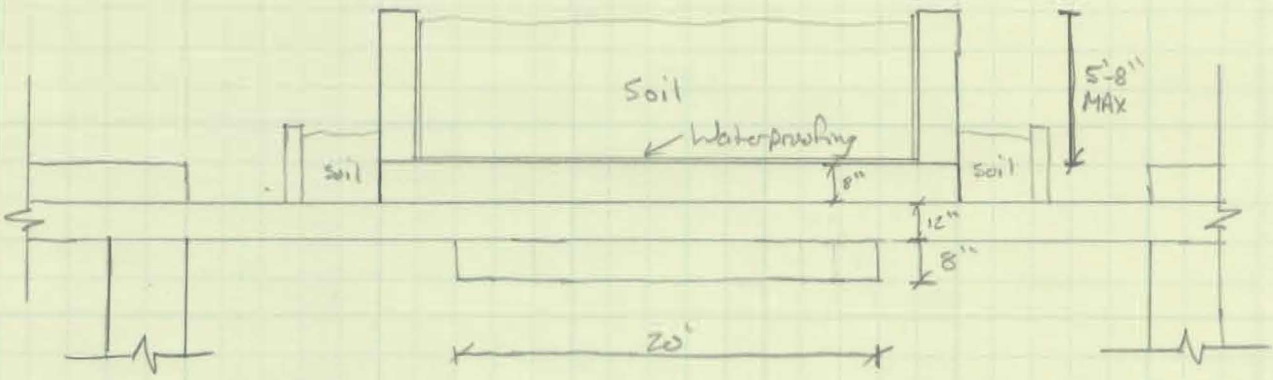
- 10" Concrete = 125 psf
- Collateral = 5 psf

Total = 130 psf

Floor Dead Loads Continued...

Under Bioretension Area:

- The bio-retension area is located on the 5th floor set back and is supported by mild reinforced concrete with a continuous drop panel.



- For worst case, assume planter is saturated $\gamma = 62.4$ pcf

Uniformly Distributed Load

- Avg 20" Concrete = 250 psf
- 5'-8" Water = 353.8 psf
- Water-proofing = 2 psf
- Collateral = 5 psf

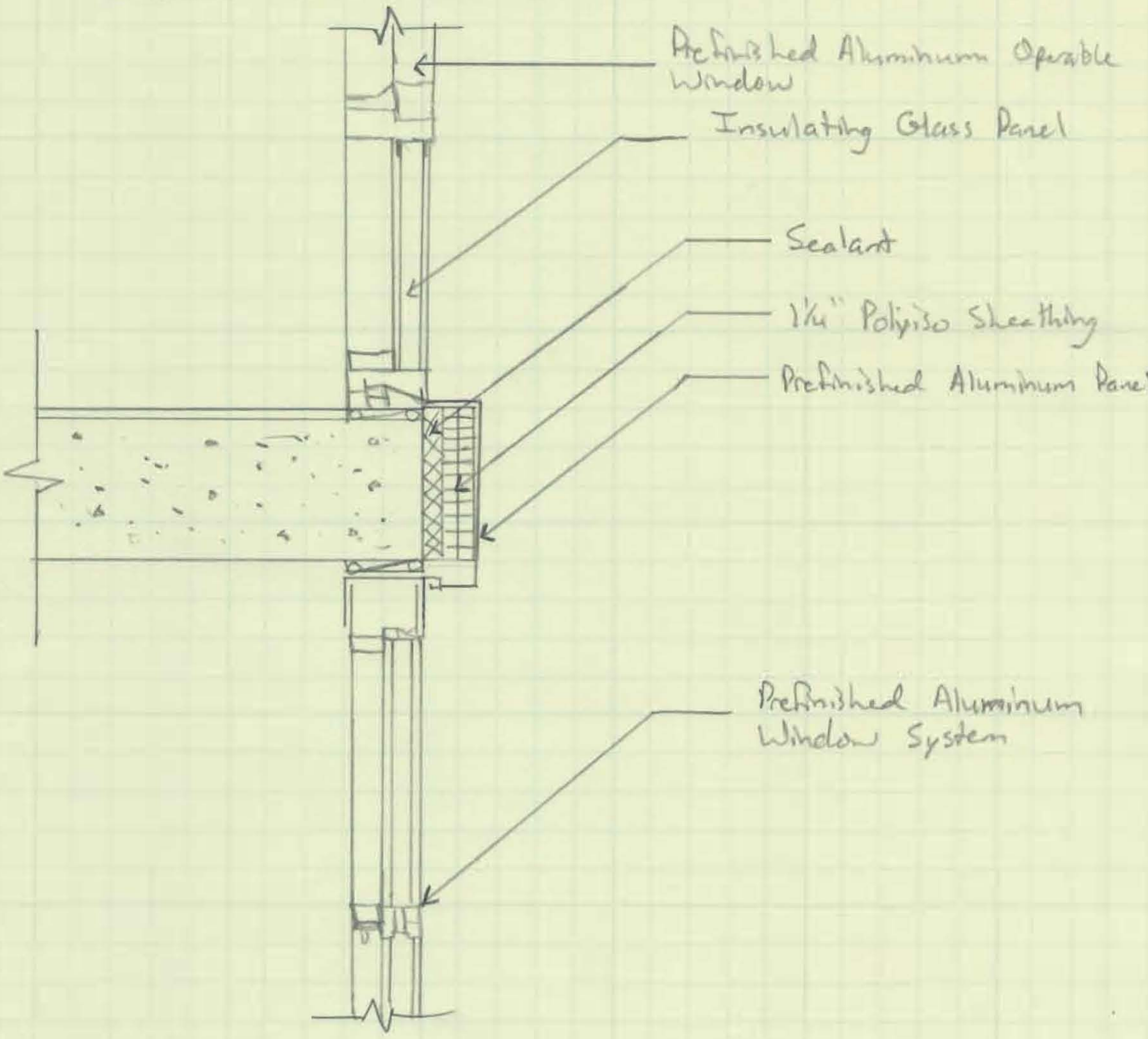
Total = 610.8 psf

Floor Live Loads

Occupancy	Design Value	ASCE 7-10 Code Minimum
Residential	40 + 10 (partition)	40 psf + 10 (partitions)
Parking Garage	50 psf	40 psf
First Floor Retail	100 psf	100 psf
Public Space	100 psf	100 psf
Fitness Gym	100 psf	100 psf

Wall Loads

Typ. Curtain Wall Slab - Operable Window

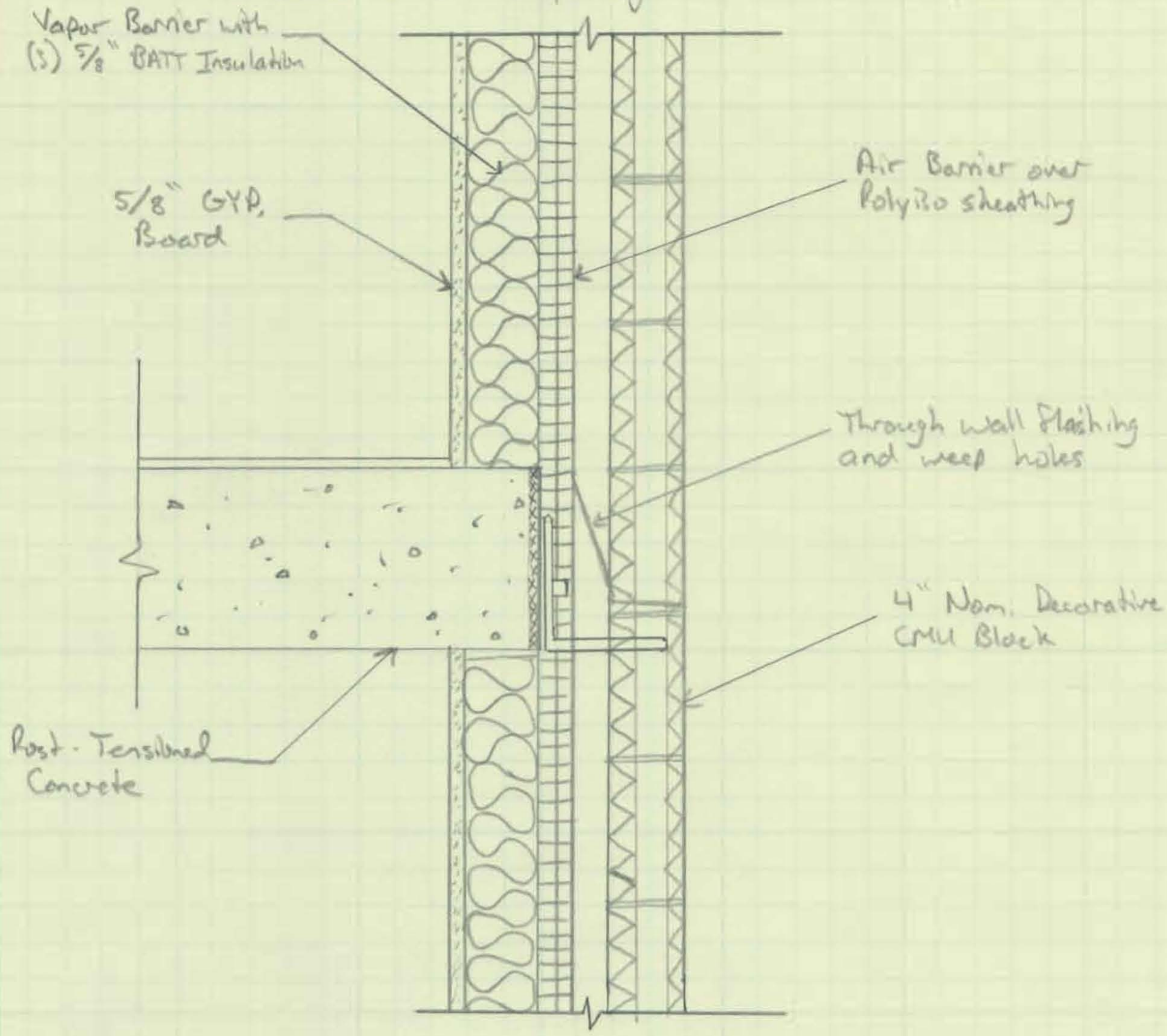


Window System ...... 8psf × 10' = 80 plf
↳ AISC Manual
Fasteners .... 5 plf

Total = 85 plf

Typical Exterior Masonry Wall

- The facade is covered in windows but the greatest wall load will result from a fully masonry section.
- assume a 10' story height



Gypsum Board - $\frac{5}{8}$" × 4 psf/in × 10' = 25 plf
Batt Insulation - $3(\frac{5}{8}) = 1\frac{7}{8}$" × 1 psf/in × 10' = 18.75 plf
Poly Iso - 1 pcf × $\frac{1}{12}$' × 10' ≈ 1 plf
4" Nom. Concrete, 30 psf × 10' = 300 plf

Total = 345 plf

Snow Loads

$$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$$

Exposure Factor, C_e - Table 7-2

$C_e = 1.0$

Thermal Factor, C_t - Table 7-3

$C_t = 1.0$

Importance Factor, I_s - Table 1.5-2

$I_s = 1.0$

Ground Snow Load, p_g - Figure 7-1

$p_g = 25$ psf → listed in ASCE

* Local code specifies a use of $p_g = 30$ psf

$$p_f = 0.7(1.0)(1.0)(1.0)(30\text{ psf}) = 21\text{ psf}$$

$p_f = 21.0$ psf

Snow Drift

Is calculating snow drifts required?

If $\frac{h_c}{h_b} < 0.2$, snow drift loads are not applicable

$$h_b = \frac{p_s}{\gamma} = \frac{30}{0.13 p_g + 14} = \frac{30}{.13(30)+14} = 1.68$$

γ = snow density

$\gamma = 0.13 p_g + 14$, Equation 7.7-1

for parapet, $h_c = 4.33'$

$$\frac{h_c}{h_b} = \frac{4.33}{1.68} = 2.58 > 0.2 \quad \therefore \text{Consider Snow Drift}$$

- All roofs are flat or close to flat. The snow drifting will occur at the parapet or at the 16th/17th floor height difference, by the 16th floor pool.

Parapet Snow Drift

$l_u = 102'-2"$ $\quad p_g = 30$ psf

Leeward:

$h_d = 3.3'$ (Figure 7-9)

$h_d < h_c$, therefore $w = 4h_d = 13.2'$

Windward

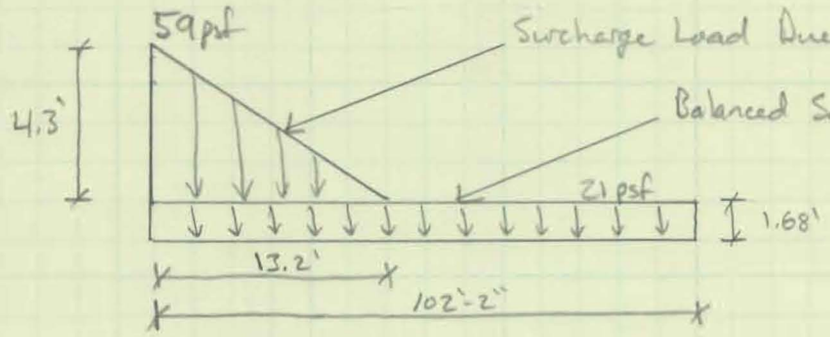
$h_d = \frac{3}{4}(3.3) = 2.48'$

$3.3' > 2.48'$ $\quad \therefore$ Use $h_d = 3.3'$ in design

Snow Drift Continued

$p_d = h_d \cdot \gamma = 3.3\left[0.13(30)+14\right]$

$p_d = 59$ psf



Snow Drift on 16th / 17th Floors

$h_b = 1.68'$, $h_c = 12.32'$

$\frac{12.32'}{1.68'} = 7.33 > 0.2$ $\therefore$ Consider Snow Drift

Leeward:

$l_u = 33'$ $p_g = 30$ psf

$h_d = 1.5$ (Figure 7-9)

$h_c > h_d$, therefore $w = 4\frac{h_d^2}{h_c} = 4\frac{(1.5)^2}{1.68}$

$w = 5.36'$

Windward:

$l_u = 152'$ ← worst case distance, along CL 6 & 7

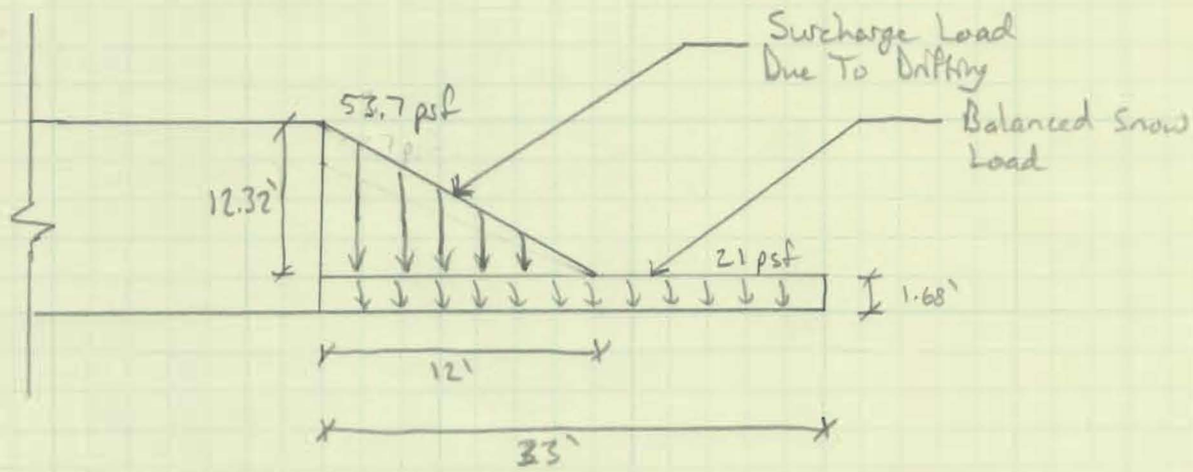
$h_d = 4.0\left(\frac{3}{4}\right) = 3.0$ $3.0 > 1.5$, $\therefore$ Use 3.0 in design

$h_d > h_c$, therefore $w = 4h_d = 12'$

* windward snow drift produces worst case w

Snow Drift Continued...

$$p_d = 3.0\left(0.13(30)+14\right) = 53.7 \text{ psf} \qquad p_d = 53.7 \text{ psf}$$



Wind Load Calculations

- ASCE 7-10 Chapters 26-30
  - Main Wind Force Resisting System (MWFRS) (Directional Procedure)

① Risk Category (Table 1.5-1)

Category II
- All buildings and other structures except those in Risk Categories I, III, and IV.

② Basic Wind Speed, V (Figure 26.5-1A)

$V = 115$ mph

③ Wind Load Parameters

A.) Wind Directionality Factor, K_d (Table 26.6-1)

$K_d = 0.85$
- For buildings with a MWFRS and Components and cladding.

B.) Exposure Category (Section 26.7)

Exposure Category C
- All cases where categories B and D don't apply

C.) Topographic Factor, K_{zt} (Table 26.8-1)

$K_{zt} = 1.0$
- Building is not located on a ridge, escarpment, or hill.

D.) Gust Effect Factor, G, (Section 26.9)

i.) Frequency Determination (Section 26.9.3)

① Building height < 300 ft.
② Building height < 4 L_{eff}

Meets requirements for alternative analysis in Section 26.9.3

i.) Frequency Determination Continued...

$$n_a = \frac{385(C_w)^{0.5}}{h}$$ -for concrete shear wall buildings

where,

$$C_w = \frac{100}{A_B}\sum_{i=1}^{n}\left(\frac{h}{h_i}\right)^2 \cdot \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

Apply Summation to each Shear Wall...

Shear Wall 1 (at CL C, 3-4) E-W

floors 1-4

$h = 161'$

$h_i = 39'-10''$

$A_i = \frac{14''}{12} \times 18'-4'' = 21.39\ ft^2$

$D_i = 18'-4''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{21.39}{1+.83\left(\frac{39.83}{18.33}\right)^2} = 71.05$$

floors 5-17

$h = 161'$

$h_i = 118'-4''$

$A_i = \frac{14''}{12} \times 28'-4'' = 33.06\ ft^2$

$D_i = 28'-4''$

$$\left(\frac{161}{118.33}\right)^2 \cdot \frac{33.06}{1+.83\left(\frac{118.33}{28.33}\right)^2} = 61.2$$

Shear Wall 2 (at CL C, 5-6) E-W

$h = 161'$

$h_i = 158'-2''$

$A_i = \frac{14''}{12} \times 18'-4'' = 21.39\ ft^2$

$D_i = 18'-4''$

$$\left(\frac{161}{158.17}\right)^2 \cdot \frac{21.39}{1+0.83\left(\frac{158.17}{18.33}\right)^2} = 0.35$$

Shear Wall #3 (at CL 3, C-D) N-S

$h = 161'$
$h_i = 129'\text{-}4''$
$A_i = \frac{12''}{12} \times 9'\text{-}4'' = 9.33\ ft^2$
$D_i = 9'\text{-}4''$

$$\left(\frac{161'}{129.33}\right)^2 \cdot \frac{9.33}{1+.83\left(\frac{129.33}{9.33}\right)^2} = 0.09$$

Shear Wall #4 (at CL 4, C-D) N-S

floors 1-4
$h = 161'$
$h_i = 28'\text{-}10''$
$A_i = \frac{12''}{12} \times 21'\text{-}6'' = 21.5\ ft^2$
$D_i = 21'\text{-}6''$

$$\left(\frac{161}{28.83}\right)^2 \cdot \frac{21.5}{1+0.83\left(\frac{28.83}{21.5}\right)^2} = 269.02$$

floors 5-17
$h = 161'$
$h_i = 129'\text{-}4''$
$A_i = \frac{12''}{12} \times 9'\text{-}4'' = 9.33\ ft^2$
$D_i = 9'\text{-}4''$

$$\left(\frac{161'}{129.33}\right)^2 \cdot \frac{9.33}{1+0.83\left(\frac{129.33}{9.33}\right)^2} = 0.09$$

Shear Wall #5 (at CL G, 5-6) E-W

$h = 161'$
$h_i = 169'\text{-}8''$
$A_i = \frac{12''}{12} \times 19'\text{-}6'' = 19.5\ ft.$
$D_i = 19'\text{-}6''$

$$\left(\frac{161.0}{169.67}\right)^2 \cdot \frac{19.5}{1+0.83\left(\frac{169.67}{19.5}\right)^2} = 0.28$$

Shear Wall #6 (at CL J, 4-6) E-W

$h = 161'$
$h_i = 169'-8''$
$A_i = \frac{12''}{12} \times 42'-4'' = 42.33 \text{ ft.}^2$
$D_i = 42'-4''$

$$\left(\frac{161}{169.67}\right)^2 \cdot \frac{42.33}{1 + 0.83\left(\frac{169.67}{42.33}\right)^2} = 2.66$$

Shear Wall #7 (at CL 5.5, G-J) N-S

$h = 161'$
$h_i = 177'-8''$
$A_i = \frac{12''}{12} \times 32'-8'' = 32.67 \text{ ft.}^2$
$D_i = 32'-8''$

$$\left(\frac{161}{177.67}\right)^2 \cdot \frac{32.67}{1 + 0.83\left(\frac{177.67}{32.67}\right)^2} = 4.87$$

Shear Wall #8 (at CL 6, G-J) N-S

$h = 161'$
$h_i = 177'-8''$
$A_i = \frac{12''}{12} \times 32'-6'' = 32.5 \text{ ft.}^2$
$D_i = 32'-6''$

$$\left(\frac{161}{177.67}\right)^2 \cdot \frac{32.5}{1 + 0.83\left(\frac{177.67}{32.5}\right)^2} = 1.11$$

Shear Wall #9 (at CL 5, C-D) N-S

$h = 161'$
$h_i = 39'-10''$
$A_i = \frac{12''}{12} \times 21'-6'' = 21.5 \text{ ft}^2$
$D_i = 21'-6''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{21.5}{1 + 0.83\left(\frac{39.83}{21.5}\right)^2} = 91.28$$

Shear Wall #10 (at CL. 6, C-D) N-S

$h = 161'$
$h_i = 39'\text{-}10''$
$A_i = \frac{12''}{12} \times 19'\text{-}10'' = 19.83'$
$D_i = 19'\text{-}10''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{19.83}{1+0.83\left(\frac{39.83}{19.83}\right)^2} = 74.51$$

Shear Wall #11 (at CL C.5, 5-6) E-W

$h = 161'$
$h_i = 39'\text{-}10''$
$A_i = \frac{12''}{12} \times 18'\text{-}4'' = 18.33 \text{ ft.}^2$
$D_i = 18'\text{-}4''$

$$\left(\frac{161'}{39.83'}\right)^2 \cdot \frac{18.33}{1+.83\left(\frac{39.83}{18.33}\right)^2} = 60.89$$

Shear Wall #12 (at CL D, 5-6) E-W

$h = 161'$
$h_i = 39'\text{-}10''$
$A_i = \frac{12''}{12} \times 18'\text{-}4'' = 18.33 \text{ ft}^2$
$D_i = 18'\text{-}4''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{18.33}{1+0.83\left(\frac{39.83}{18.33}\right)^2} = 60.89$$

Shear Wall #13 (at CL. F.3, 7-8) E-W

$h = 161'$
$h_i = 44'\text{-}10''$
$A_i = \frac{12''}{12} \times 16' = 16 \text{ ft.}^2$
$D_i = 16'$

$$\left(\frac{161}{44.83}\right)^2 \cdot \frac{16}{1+.83\left(\frac{44.83}{16}\right)^2} = 27.46$$

Shear Wall #14 (at CL. H, 4-5) E-W

$h = 161'$

$h_i = 51'\text{-}4''$

$A_i = \frac{12''}{12} \times 22'\text{-}9'' = 22.75 \text{ ft.}^2$

$D_i = 22'\text{-}9''$

$$\left(\frac{161}{51.33}\right)^2 \cdot \frac{22.75}{1 + .83\left(\frac{51.33}{22.75}\right)^2} = 42.83$$

• Find C_w

North - South

$A_B = \left[(31'\text{-}7'' \times 118') + (134'\text{-}4'' \times 100'\text{-}1'') + (44' \times 118'\text{-}6'') + (16'\text{-}3'' \times 99'\text{-}4'')\right.$

$= 23,998.7 \text{ ft.}^2$

$$\sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{1 + 0.83\left(\frac{h_i}{D_i}\right)^2} = 0.09 + 269.02 + 0.09 + 4.87 + 1.11 + 91.28 + 74.51 = 440.97$$

$$C_w = \frac{100}{23,998.7}(440.97) = 1.837$$

East - West

$A_B = 23,998.7 \text{ ft.}^2$

$$\sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{1 + 0.83\left(\frac{h_i}{D_i}\right)^2} = 71.05 + 61.2 + 0.35 + 0.28 + 2.66 + 60.89 + 60.89 + 27.46 + 42.83 = 327.61$$

$$C_w = \frac{100}{23,998.7}(327.61) = 1.365$$

• Frequency

$$n_{a,N\text{-}S} = \frac{385\, C_w^{0.5}}{h} = \frac{385(1.837)^{0.5}}{161} = 3.24 \text{ Hz.}$$

$$n_{a,E\text{-}W} = \frac{385\, C_w^{0.5}}{h} = \frac{385(1.365)^{0.5}}{161} = 2.79 \text{ Hz.}$$

$n_a > 1$ Hz therefore the structure can be considered rigid.

D). Gust Effect Factor Continued...

- For a rigid structure G can be taken as 0.85 or by the following formula (Section 26.9.4)

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

$g_Q = g_v = 3.4$ (Section 26.9.4)

For exposure Category C: (Table 26.9-1)
$c = 0.2$
$\ell = 500$ ft
$\bar{\epsilon} = 0.2$
$z_{min} = 15$ ft.

$$\bar{z} = 0.6\,h = 0.6(161) = 96.6\text{ ft} > z_{min} = 15\text{ ft.}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{96.6}\right)^{\frac{1}{6}} = 0.167$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{134.33+161}{619.82}\right)^{0.63}}}$$

$$= 0.847$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{96.6}{33}\right)^{0.2} = 619.82$$

$$G = 0.925\left(\frac{1+1.7(3.4)(0.167)(.847)}{1+1.7(3.4)(0.167)}\right) = 0.894$$

- The smallest B will result in the largest Gust Effect factor; Therefore the B from the North-South Direction was used to find the controlling Gust Effect Factor. This is greater than the alternate of 0.85 given in Section 26.9.4, use the greater of the two, which will produce a greater wind load.

$$G = 0.894$$

E). Enclosure Classification (Section 26.10)

Enclosed (Section 26-2).

F.) Internal Pressure Coefficient (Table 26.11-1)

$GC_{pi} = +/- 0.18$ (Enclosed)

④ Determine Velocity Pressure Exposure Coefficient, K_z (Table 27.3-1)

At floor 2 where $z < 15$ ft, $K_z = 2.01\left(\frac{15}{z_g}\right)^{2/\alpha}$, otherwise
$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$

For Exposure Category C, $\alpha = 9.5$ $z_g = 900$ ft (Table 26.9-1)

$K_z\ (10\text{ft}) = 2.01\left(\frac{15}{900}\right)^{\frac{2}{9.5}} = 0.8489$

$K_z\ (19.5\text{ft}) = 2.01\left(\frac{19.5}{900}\right)^{\frac{2}{9.5}} = 0.8971$

$K_z\ (29\text{ft}) = 2.01\left(\frac{29}{900}\right)^{\frac{2}{9.5}} = 0.9753$

$K_z\ (40\text{ft}) = 2.01\left(\frac{40}{900}\right)^{2/9.5} = 1.0436$

$K_z\ (49\text{ft}) = 2.01\left(\frac{49}{900}\right)^{2/9.5} = 1.0891$

$K_z\ (59\text{ft}) = 2.01\left(\frac{59}{900}\right)^{2/9.5} = 1.1326$

$K_z\ (68\text{ft}) = 2.01\left(\frac{68}{900}\right)^{\frac{2}{9.5}} = 1.1669$

$K_z\ (77\text{ft}) = 2.01\left(\frac{77}{900}\right)^{2/9.5} = 1.1979$

$K_z\ (87\text{ft.}) = 2.01\left(\frac{87}{900}\right)^{2/9.5} = 1.2291$

$K_z\ (96\text{ft}) = 2.01\left(\frac{96}{900}\right)^{2/9.5} = 1.2548$

$K_z\ (105\text{ft}) = 2.01\left(\frac{105}{900}\right)^{2/9.5} = 1.2787$

$K_z\ (115\text{ft}) = 2.01\left(\frac{115}{900}\right)^{2/9.5} = 1.3034$

$K_z\ (124\text{ft}) = 2.01\left(\frac{124}{900}\right)^{2/9.5} = 1.3243$

$K_z\ (133\text{ft}) = 2.01\left(\frac{133}{900}\right)^{2/9.5} = 1.3439$

K_z Determination Continued...

$$K_z(146\text{ ft}) = 2.01\left(\frac{146}{900}\right)^{2/9.5} = 1.3706$$

$$K_z(158\text{ ft}) = 2.01\left(\frac{158}{900}\right)^{2/9.5} = 1.3936$$

$$K_z(161\text{ ft}) = 2.01\left(\frac{161}{900}\right)^{2/9.5} = 1.3991$$

⑤ Determine Velocity Pressure Exposure, q_z

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

K_z = varies by height
K_{zt} = 1.0
K_d = 0.85
V = 115 mph

$$q_z = 0.00256(K_z)(1.0)(.85)(115)^2 = 28.78\ K_z$$

q_z (10 ft) = 24.4 psf
q_z (19.5 ft) = 25.8 psf
q_z (29 ft) = 28.1 psf
q_z (40 ft.) = 30.0 psf
q_z (49 ft.) = 31.3 psf
q_z (59 ft.) = 32.6 psf
q_z (68 ft.) = 33.6 psf
q_z (77 ft.) = 34.5 psf
q_z (87 ft.) = 35.4 psf
q_z (96 ft.) = 36.1 psf
q_z (105 ft.) = 36.8 psf
q_z (115 ft.) = 37.5 psf
q_z (124 ft.) = 38.1 psf
q_z (133 ft.) = 38.7 psf
q_z (146 ft.) = 39.4 psf
q_z (158 ft.) = 40.1 psf
q_z (161 ft.) = 40.3 psf

*Also see attached spreadsheet for values of K_z and q_z *

⑥ Determine External Pressure Coefficient, C_p

North - South: $\frac{L}{B} = \frac{194'}{134'} = 1.45$

East - West: $\frac{L}{B} = \frac{134'}{194} = 0.69$

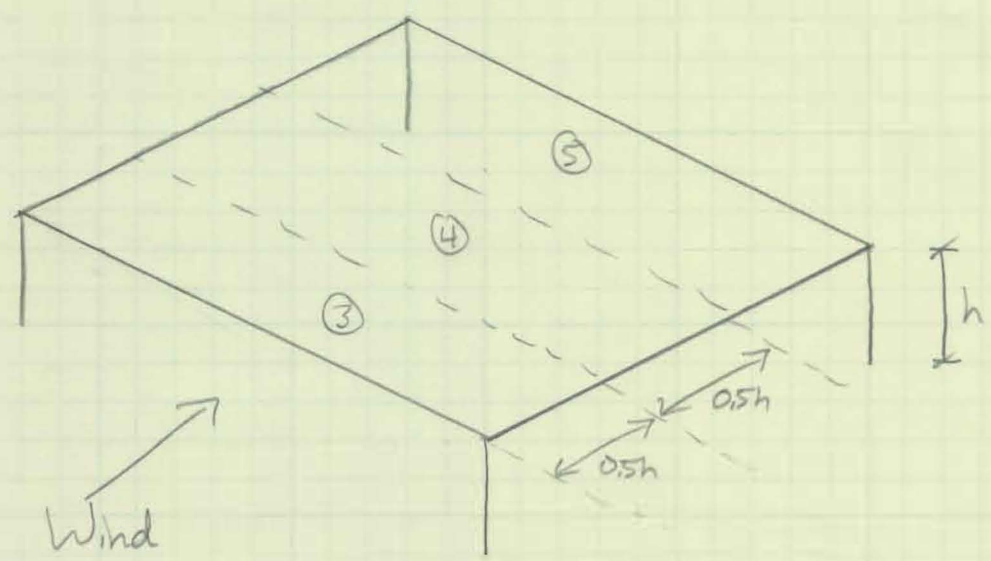
Walls:
- Windward => $C_p = 0.8$
- Leeward => $C_p = -0.3$ in N/S, $C_p = -0.5$ in E/W
- Side Wall => $C_p = -0.7$

Roofs:
- Flat roof, $\theta = 0$

From Table 27.6-2 for h=160ft., V=115mph

Zone #	Pressure
1	0
2	0
3	-42.6 psf
4	-38.0 psf
5	-31.2 psf

For Flat Roofs, Table 27.6-2

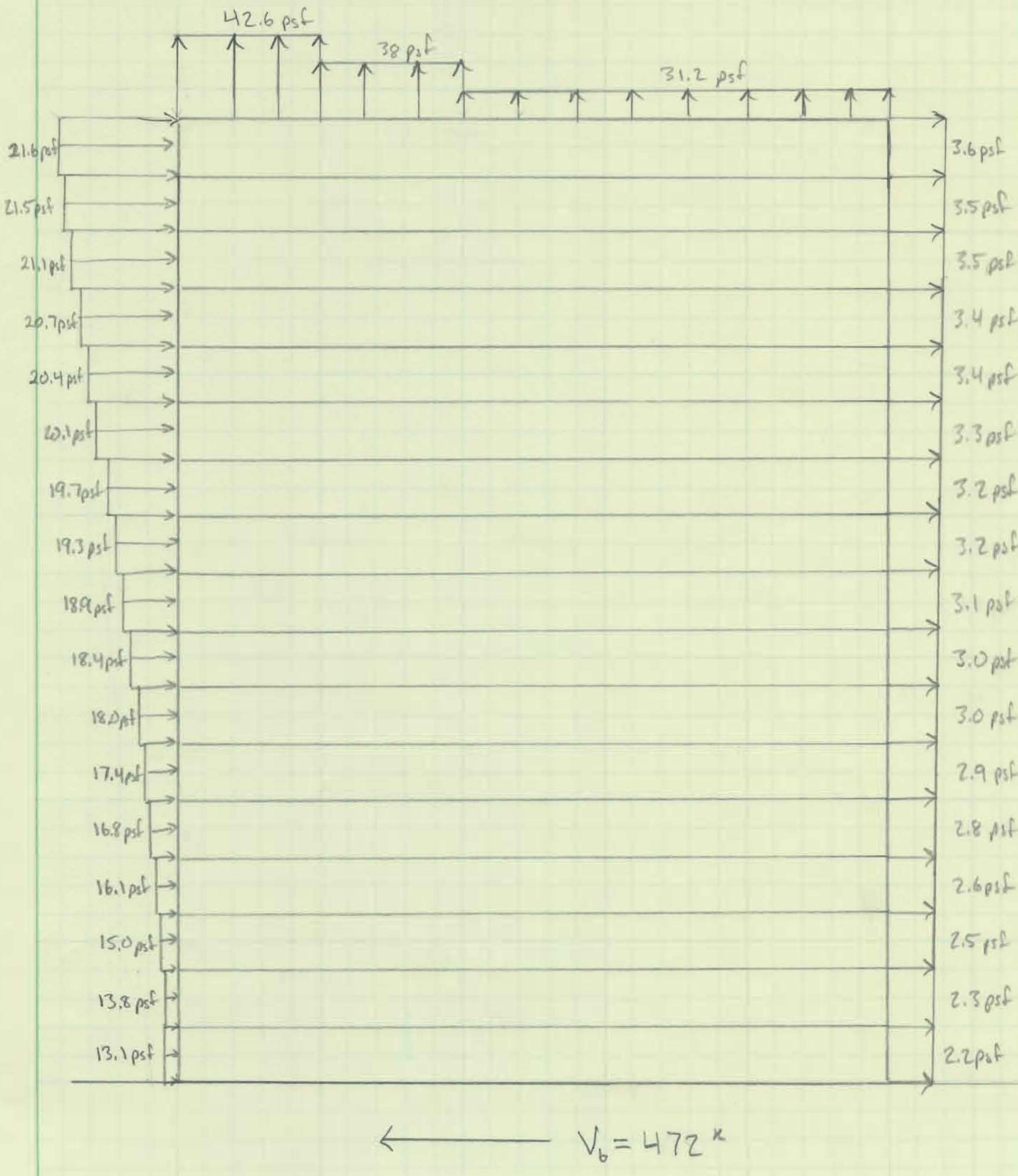


$$p = q\,G C_p - q_i\,(G C_{pi})$$

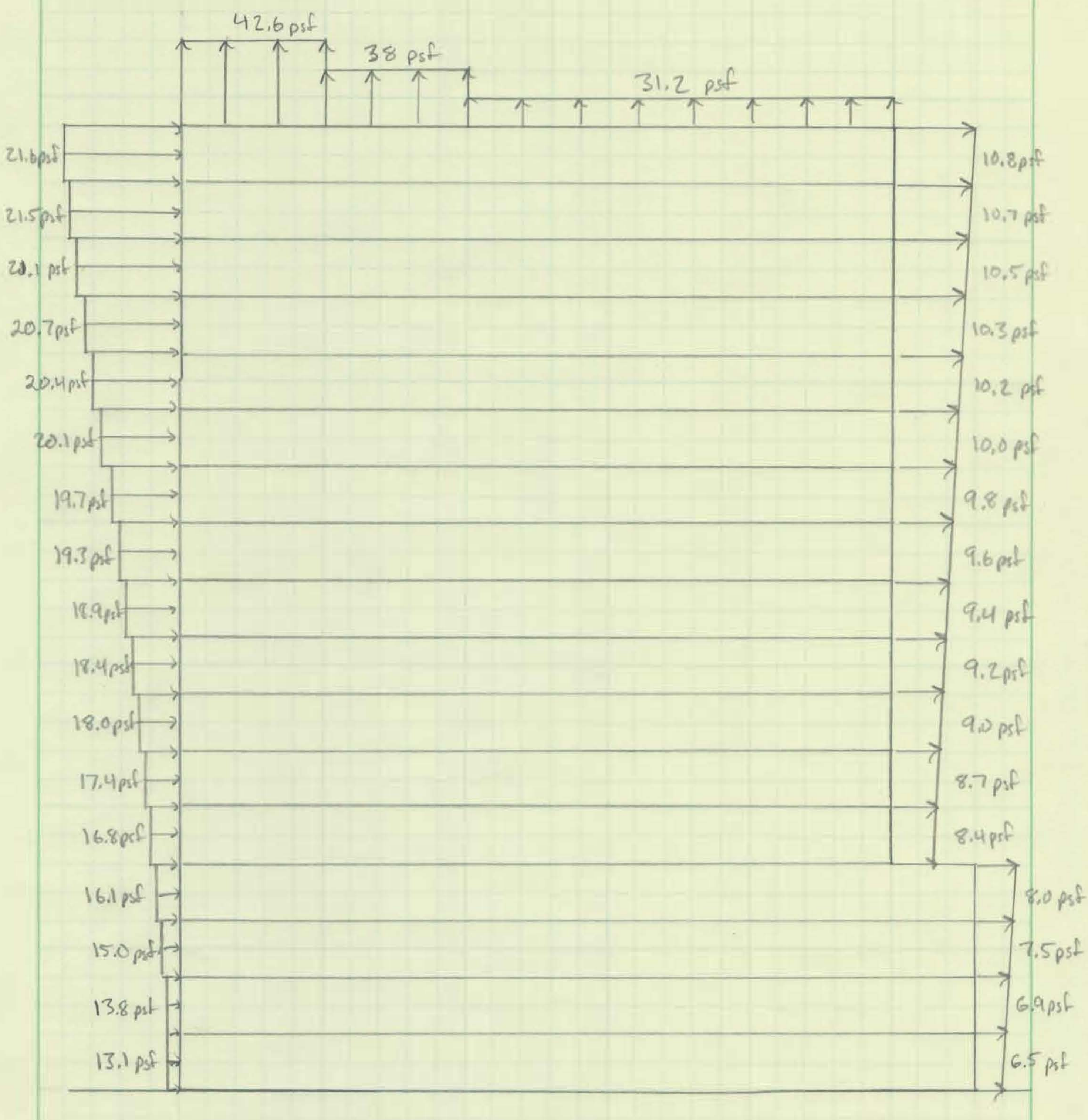
$q = q_z$ for winward walls
$\quad = q_z(161\text{ ft})$ for other walls
$G = 0.894$
$GC_{pi} = \pm 0.18$
$C_p = 0.8$ (windward)
$\quad -0.3$ (N/S leeward)
$\quad -0.5$ (E/W leeward)
$\quad -0.7$ (side wall)

* See Excel Sheets for Pressure on Roof and Walls

Wind Pressure Elevation - N/S, in psf



Wind Pressure Elevation - E/W, in psf



① Find Mapped Acceleration Parameters (Fig. 22-1-6)

$S_s = 0.125$
$S_1 = 0.055 > 0.04 \rightarrow$ Not assumed to be Seismic Design Category A

② Site Classification (Chapter 20)

Site Classification : C

③ Maximum Considered Earthquake Spectral Response Acceleration Parameters

$$\left.\begin{array}{l} F_a = 1.2 \\ F_v = 1.7 \end{array}\right\} \text{Table 11.4-1-2}$$

$S_{MS} = F_a S_s = 1.2(0.125) = 0.15$

$S_{M1} = F_v S_1 = 1.7(0.055) = 0.094$

④ Design Spectral Parameters, Section 11.4.4

$S_{DS} = \frac{2}{3} S_{MS} = 0.10$

$S_{D1} = \frac{2}{3} S_{M1} = 0.062$

⑤ Importance Factor, Table 1.5-2

$I_e = 1.0$

⑥ Risk Category

Risk Category II $\rightarrow$ No special provisions (Section 11.6)

⑦ Seismic Design Category

Table 11.6-1 $\rightarrow$ SDC = A

Table 11.6-2 $\rightarrow$ SDC = A

∴ Use Seismic Design Category A

⑧ Exemption, Section 11.7

- Buildings with Seismic Design Category A are exempt from Seismic Design Criteria and must only comply with Section 1.4

Consider Section 1.4: General Structural Integrity...

Per Section 1.4.1-2

- A continuous load path has been provided for the building. The load path for the gravity system consists of mild reinforced and post-tensioned concrete slabs distributing the load to concrete columns and shear walls which transfer the loads down into the foundation. This load path is discussed in more detail in Technical Report #1.

The load path in the lateral system consists of a precast concrete or window glazing facade that transfers the lateral wind (controls over seismic) pressure the the concrete diaphragms at each floor. The lateral forces are distributed to 14 shear walls. The lateral load path is described in greater detail in Technical Report #1.

This section also requires adequate strength for the following load cases:

- $1.2D + 1.0N + L + 0.2S$
- $0.9D + 1.0N$
- $D + 0.7N$
- $D + 0.75(.7N) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
- $0.6D + 0.7N$

- These cases and further analysis will be performed in the following Technical reports.

Per Section 1.4.3 - Lateral Forces

The story forces are given by

$$F_x = 0.01\, W_x$$

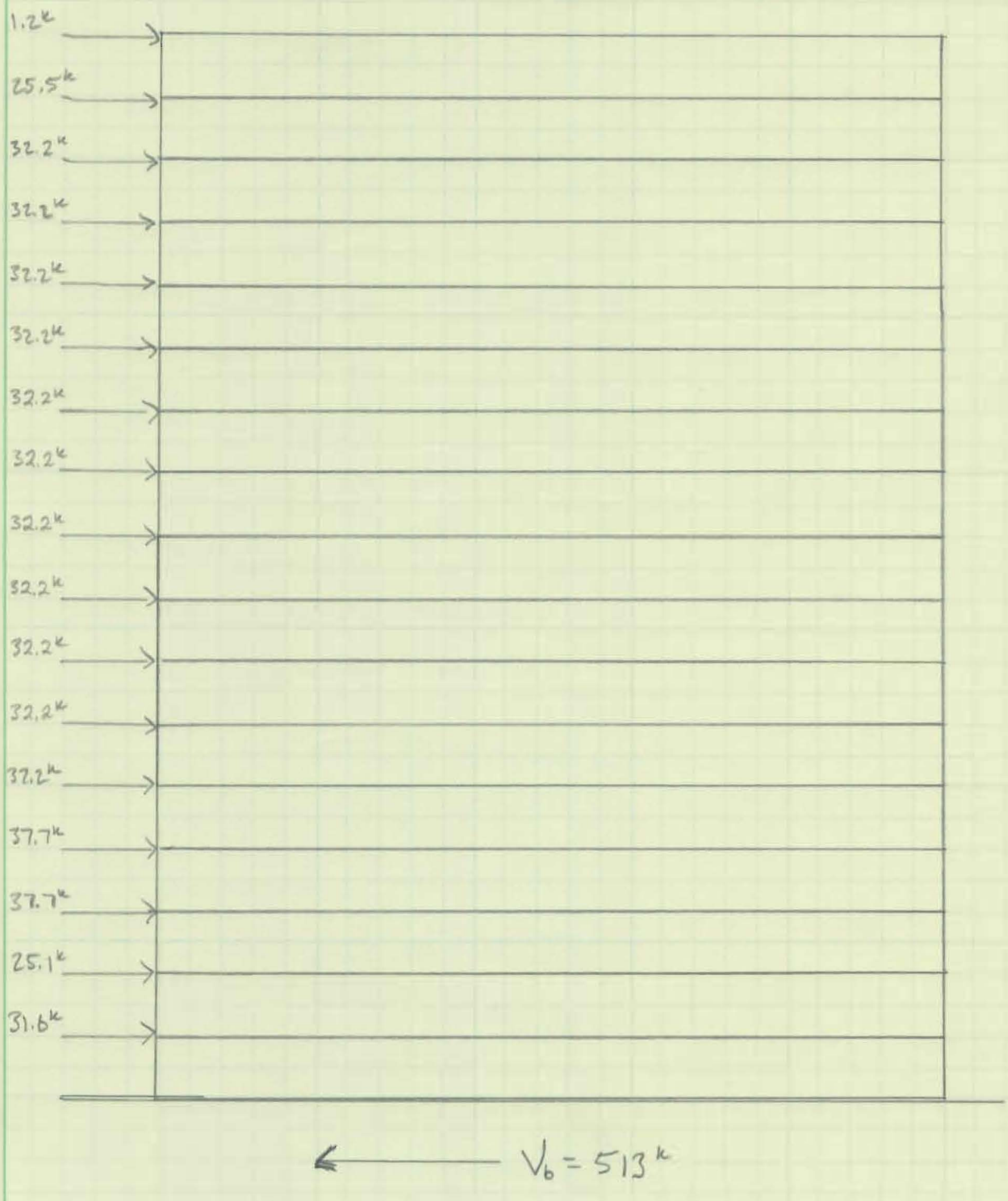
W_x = total dead load per story

- See attached spreadsheet for story forces *

Seismic Story Forces - N/S, in kips

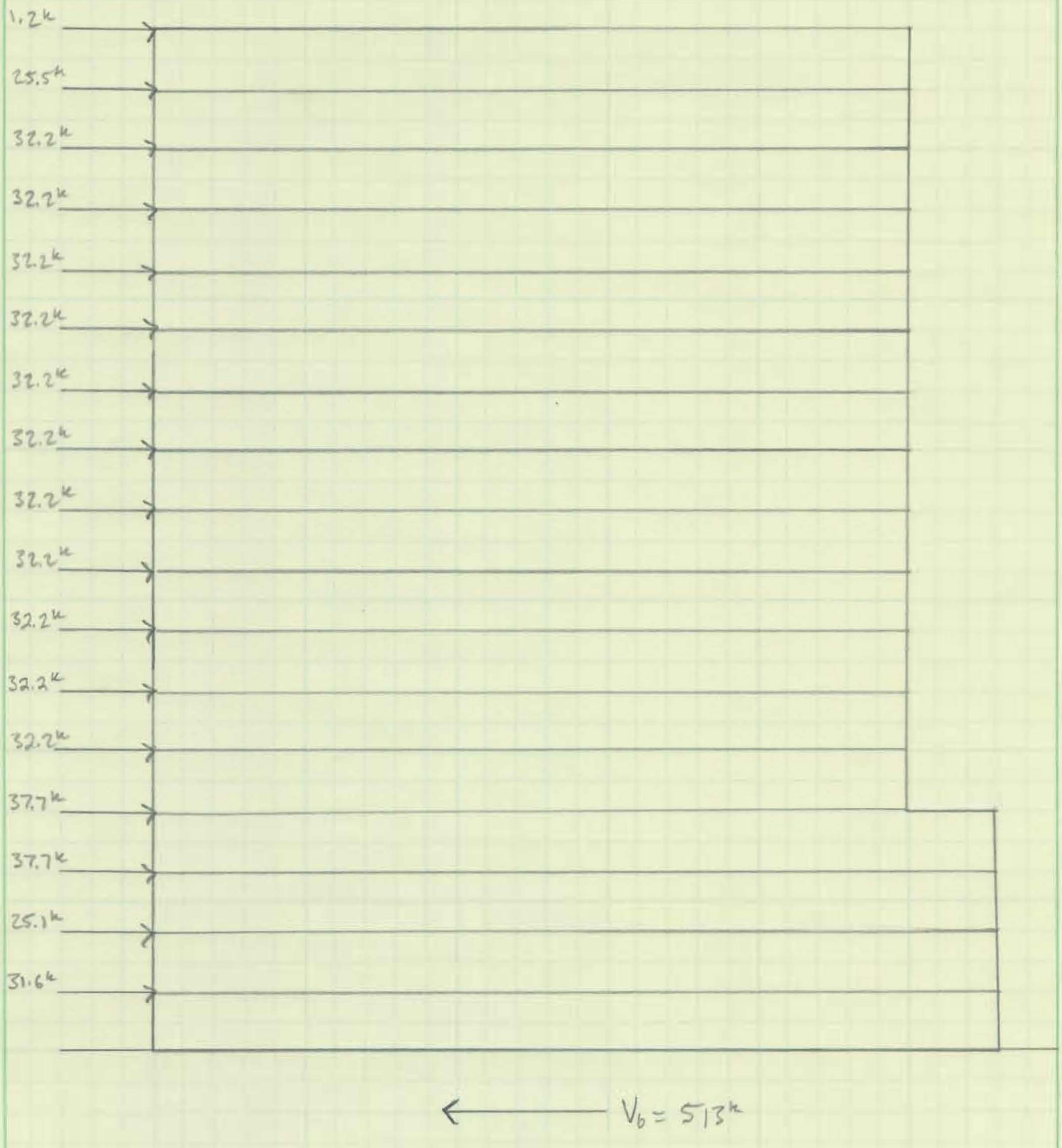
Assumption:

- to estimate story weight, assume an average slab thickness of 10" concrete → 125 psf
- Add allowance for wall weights, columns, etc. + 25 psf.
- Therefore, assume W_x = 150 psf × Area



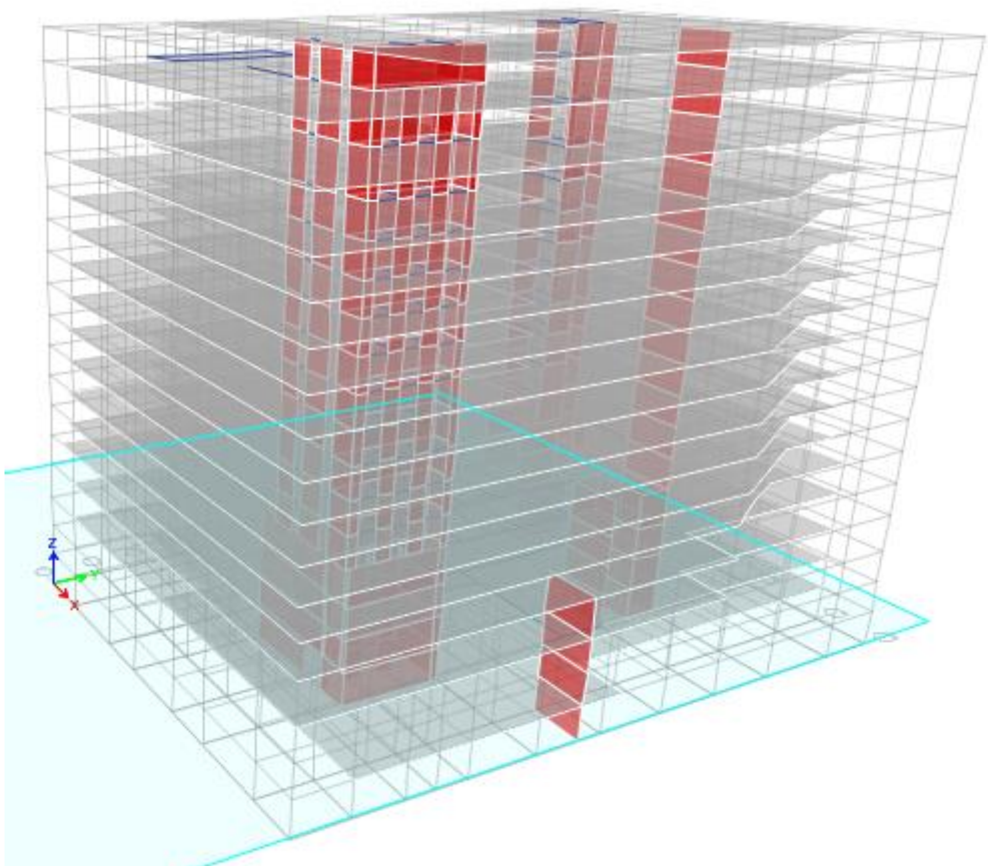
Seismic Story Forces - E/W, in Kips

- With Simplified Method for buildings in a Seismic Design Category A, the seismic story forces are the same in both directions.

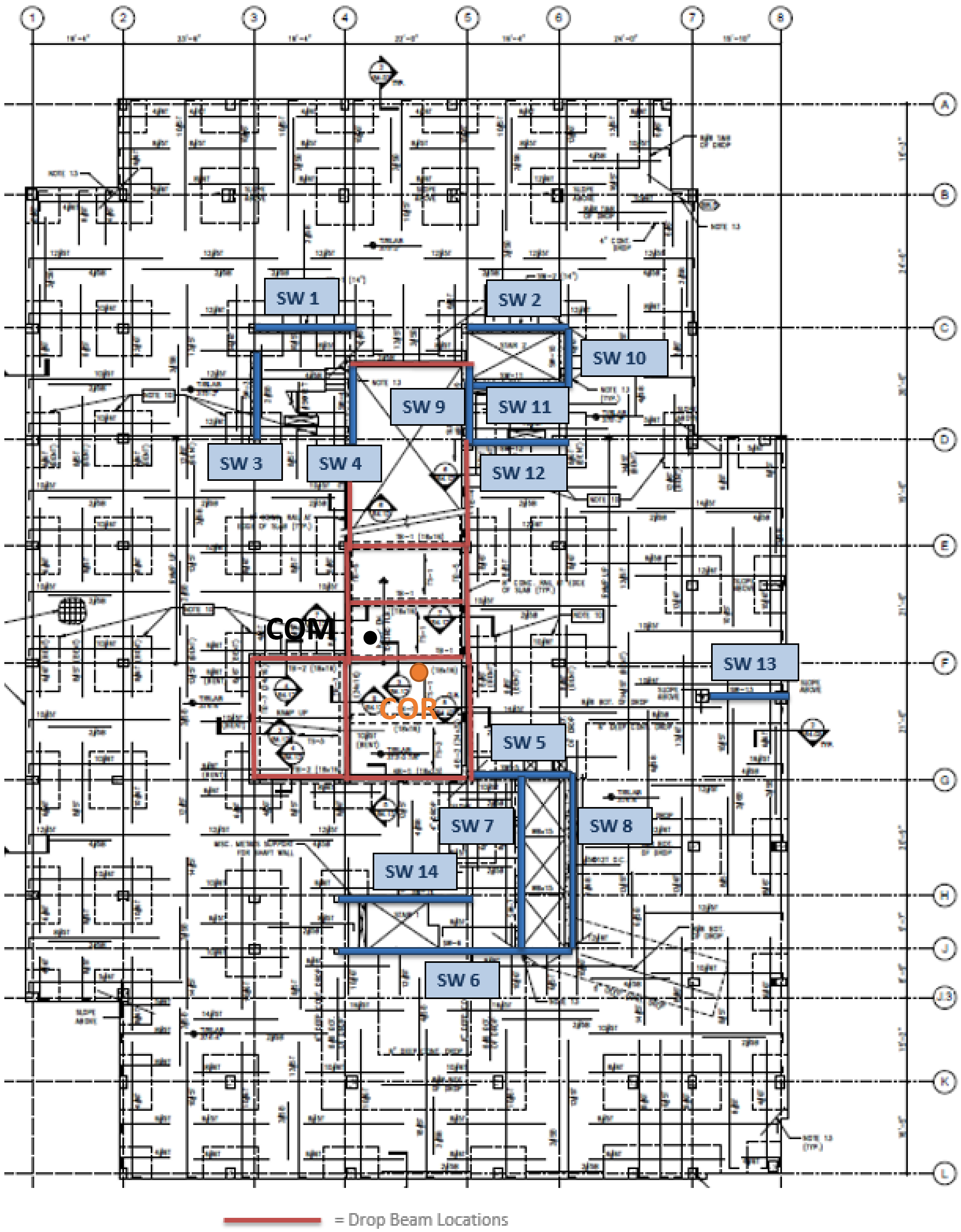


Lateral Analysis

The scope of the analysis for technical report 4 includes an in depth lateral analysis of Georgia 8621. A 3D model of the building was created in ETABS to model the lateral force resisting elements and to distribute the story forces. The results of the analysis were used to obtain the actual forces resisted by each lateral force resisting element as well as the story drifts/displacements of each floor.



The lateral force resisting system elements that were modeled were the shear walls and drop beams of the building. The building utilizes 14 different shear walls as well as occasional drop beams in high stress areas to create the lateral system of the building. All of the shear walls are 12” thick except for shear wall #1 and #2, which are 14” thick. A diagram of the provided shear walls is given on the following page.



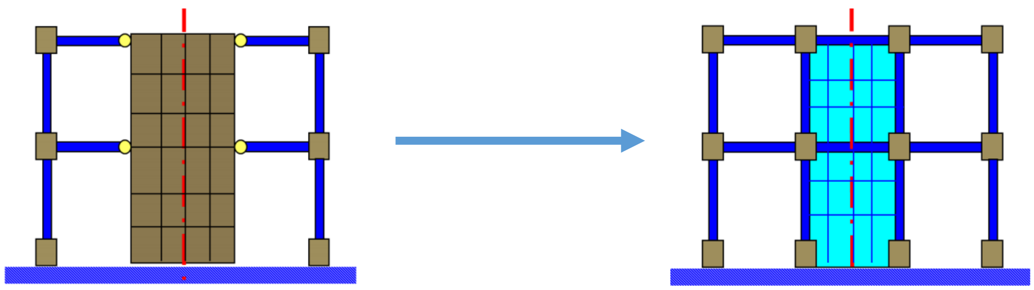
The above floor plan is for the third floor parking level.

Modeling Decisions

The structure considered for this analysis is a 17 story concrete building with shear walls as its primary lateral resisting members. There are some drop beams on the lower 4 levels to accommodate the parking garage. Although all concrete frames transfer some moment and lateral force, only the shear walls, drop beams, and columns directly supporting them will be included in the model. This decision is made both to simply the model but also to conservatively determine the loads on these elements.

The 14 shear walls in the building were all modeled as membrane elements. Membranes do not account for out-of-plane shear forces because they have no out-of-plane stiffness. This is ideal because in our theoretical lateral analysis we assume that shear walls can only resist in-plane loads.

In modeling the shear walls as membranes, extra effort had to be taken to assure the proper shear and moment continuity where beams framed into the shear walls. Additional “fake” beams and columns (the same thickness as the shear wall) had to be added in these circumstances. This was especially the case on some of the coupled shear walls to adequately model the coupling beams.



The diaphragms on every floor were modeled as being rigid. This allowed the lateral forces to transfer and be distributed to the lateral force resisting elements. The forces transferred from the rigid diaphragm are distributed based on the location of the lateral force resisting elements.

The openings in the floor diaphragms were not modeled. Large opening in the shear walls for doors were included but all other smaller openings were not modeled. This was done as a means to avoid unnecessary complexity within the model. The decision to disregard these openings will have negligible results on the model.

Building Properties

When buildings are exposed to lateral loads the act through different point of the building depending on the nature of the load. Wind and seismic forces interact with the building differently because wind is a pressure force whereas seismic force is a function of mass. The tables below will located the point at which these forces act through.

Center of Mass:

The center of mass represents the mean position of the mass located in a building or on a floor. The center of mass is the location in which external loads and moments on a building act through. The seismic forces on a building act through the center of mass.

Center of Mass by Floor						
	ETABS		Calculated by Hand		Error	
Floor	X Direction	Y Direction	X Direction	Y Direction	X	Y
17	70.02	107.89	78.15	90.32	11.60%	16.29%
16	67.67	88.36	78.15	90.32	15.49%	2.21%
15	71.25	90.95	78.15	90.32	9.67%	0.66%
14	71.23	90.95	78.15	90.32	9.67%	0.66%
13	71.23	90.95	78.15	90.32	9.67%	0.66%
12	71.23	90.95	78.15	90.32	9.67%	0.66%
11	71.23	90.95	78.15	90.32	9.67%	0.66%
10	71.23	90.95	78.15	90.32	9.67%	0.66%
9	71.23	90.95	78.15	90.32	9.67%	0.66%
8	71.23	90.95	78.15	90.32	9.67%	0.66%
7	71.23	90.95	78.15	90.32	9.67%	0.66%
6	71.23	90.95	78.15	90.32	9.67%	0.66%
5	65.92	94.71	78.15	90.32	18.55%	4.64%
4	66.79	93.74	84.90	96.24	27.11%	2.67%
3	66.85	94.18	79.46	96.24	14.38%	2.19%
2	57.68	120.03	79.46	96.24	37.76%	19.83%
1	64.71	103.18	79.46	96.24	22.79%	6.73%

The detailed spreadsheet containing the calculated values are provided in the appendix. One discrepancy in the results is that ETABS included the slab in the COM calculation whereas the hand spot checks just included the shear walls. The footprint of the floor plan changes on the bottom 4 floors and the top two floors while the shear wall configurations do not change. Therefore larger error is expected on those floors due to that. The slab in the X direction steps back a bay above floor 4 which accounts for some of the variability in that direction. The Y direction mass distribution is fairly consistent throughout the building height, which is reflected by the low margin of error for those calculations.

Center of Rigidity:

The center of rigidity is the centroid of the stiffness for a building or individual floor. The stiffness elements considered for the center of rigidity are the shear walls and drop beams previously mentioned in this report. Forces that act through any point other than the COR cause an incidental torsion on the building because the load is applied eccentrically to the centroid of stiffness. Because 8621 Georgia Avenue is a rectangular building with relatively well distributed lateral force resisting elements, it is expected that the COR and COM points will not differ greatly. Therefore, the accidental torsion on the building should be minimal.

Center of Rigidity by Floor						
	ETABS		Calculated by Hand		Error	
Floor	**X Direction**	**Y Direction**	**X Direction**	**Y Direction**	**X**	**Y**
17	90.867	98.442	82.368	93.04	9.35%	5.49%
16	90.192	98.960	83.369	92.481	7.56%	6.55%
15	89.562	99.562	83.271	92.543	7.02%	7.05%
14	88.907	100.04	82.368	93.04	7.35%	6.99%
13	88.344	100.331	82.368	93.04	6.76%	7.27%
12	87.095	100.590	82.368	93.04	5.43%	7.51%
11	87.742	100.525	82.368	93.04	6.12%	7.45%
10	86.410	100.47	82.368	93.04	4.68%	7.40%
9	85.706	100.072	82.368	93.04	3.89%	7.03%
8	85.030	99.239	82.368	93.04	3.13%	6.25%
7	84.486	97.701	82.368	93.04	2.51%	4.77%
6	84.320	95.004	82.368	93.04	2.31%	2.07%
5	85.046	90.579	82.368	93.04	3.15%	2.72%
4	85.864	86.273	78.807	85.959	8.22%	0.36%
3	85.385	86.316	78.504	86.468	8.06%	0.18%
2	84.938	90.165	78.504	86.468	7.57%	4.10%
1	90.227	89.499	78.654	86.222	12.83%	3.66%

The detailed spreadsheet and calculations associated with this table is located in the appendix.

Center of Pressure:

The lateral wind forces applied to a building are pressure loads on the façade that we simplify to story forces based on the exposed surface area the pressure is acting on. Because the wind force is dependent on the geometric exposure of the building, the resultant force acts through the centroid of that area. Therefore the wind forces will act through these points, which are called the Center of Pressure.

Center of Pressure by Floor		
Floor	**X Direction**	**Y Direction**
17	67.16	108.35
16	67.16	91.90
15	67.16	91.90
14	67.16	91.90
13	67.16	91.90
12	67.16	91.90
11	67.16	91.90
10	67.16	91.90
9	67.16	91.90
8	67.16	91.90
7	67.16	91.90
6	67.16	91.90
5	67.16	95.96
4	67.16	95.96
3	67.16	95.96
2	59.23	116.12
1	59.23	104.17

Wind Forces

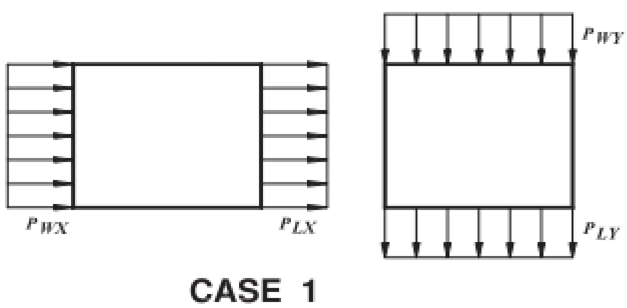
The wind analysis of the building was conducted in accordance with the Main Wind Force Resisting System directional procedure for determining wind loads. This procedure outlines 4 wind load cases to be considered. The various cases consider wind from each of the 4 major faces of the building and incorporate torsional moment of the building due to the wind.

Case 1:

The first case of the wind analysis is simply applying the full load orthogonal to the building in each of the two primary axis. The east/west direction is the long direction of the building, which has a greater surface area for the wind pressure to act over. The base shear values in each direction are also given.

Case 1 N/S Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	Story Force (k)
1	10.167	134.33	13.10	-4.39	1365.73	23.89
2	9.333	134.33	13.80	-4.64	1253.70	23.12
3	9.333	134.33	15.00	-5.06	1253.70	25.15
4	11	134.33	16.10	-5.40	1477.63	31.77
5	9.333	134.33	16.80	-5.63	1253.70	28.13
6	9.333	134.33	17.40	-5.87	1253.70	29.17
7	9.333	134.33	18.00	-6.05	1253.70	30.15
8	9.333	134.33	18.40	-6.21	1253.70	30.85
9	9.333	134.33	18.90	-6.37	1253.70	31.68
10	9.333	134.33	19.30	-6.50	1253.70	32.34
11	9.333	134.33	19.70	-6.62	1253.70	33.00
12	9.333	134.33	20.10	-6.75	1253.70	33.66
13	9.333	134.33	20.40	-6.86	1253.70	34.17
14	9.333	134.33	20.70	-6.97	1253.70	34.68
15	12.333	134.33	21.10	-7.09	1656.69	46.71
16	12.667	134.33	21.50	-7.22	1701.56	48.87
17	9.333	134.33	21.60	-7.25	1253.70	36.17
					Base Shear =	553.52

Case 1 E/W Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	Story Force (k)
1	10.167	175.5	13.1	-4.392	1784.31	31.21
2	9.333	192	13.8	-4.644	1791.94	33.05
3	9.333	192	15	-5.058	1791.94	35.94
4	11	192	16.1	-5.4	2112.00	45.41
5	9.333	192	16.8	-5.634	1791.94	40.20
6	9.333	192	17.4	-5.868	1791.94	41.69
7	9.333	192	18	-6.048	1791.94	43.09
8	9.333	192	18.4	-6.21	1791.94	44.10
9	9.333	192	18.9	-6.372	1791.94	45.29
10	9.333	192	19.3	-6.498	1791.94	46.23
11	9.333	192	19.7	-6.624	1791.94	47.17
12	9.333	192	20.1	-6.75	1791.94	48.11
13	9.333	192	20.4	-6.858	1791.94	48.84
14	9.333	192	20.7	-6.966	1791.94	49.58
15	12.333	192	21.1	-7.092	2367.94	66.76
16	12.667	192	21.5	-7.218	2432.06	69.84
17	9.333	160.5	21.6	-7.254	1497.95	43.22
					Base Shear =	779.74

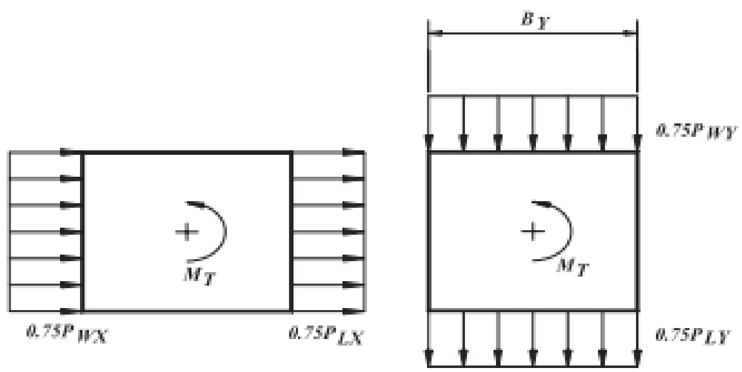


Case 2:

The second case addresses the effects of potential quartering wind conditions and their effects. Three quarters of the design wind pressures are considered in addition to a torsional moment about a vertical axis of the building with an eccentricity equal to 15% of the windward face.

Case 2 N/S Wind Forces									
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.75 * Story Force (k)	B (ft.)	e (ft.)	M (ft.*k)
1	10.167	134.33	13.10	-4.39	1365.73	17.92	134.33	20.15	361.02
2	9.333	134.33	13.80	-4.64	1253.70	17.34	134.33	20.15	349.44
3	9.333	134.33	15.00	-5.06	1253.70	18.86	134.33	20.15	380.02
4	11	134.33	16.10	-5.40	1477.63	23.83	134.33	20.15	480.10
5	9.333	134.33	16.80	-5.63	1253.70	21.09	134.33	20.15	425.04
6	9.333	134.33	17.40	-5.87	1253.70	21.88	134.33	20.15	440.84
7	9.333	134.33	18.00	-6.05	1253.70	22.61	134.33	20.15	455.62
8	9.333	134.33	18.40	-6.21	1253.70	23.14	134.33	20.15	466.26
9	9.333	134.33	18.90	-6.37	1253.70	23.76	134.33	20.15	478.81
10	9.333	134.33	19.30	-6.50	1253.70	24.26	134.33	20.15	488.77
11	9.333	134.33	19.70	-6.62	1253.70	24.75	134.33	20.15	498.74
12	9.333	134.33	20.10	-6.75	1253.70	25.25	134.33	20.15	508.70
13	9.333	134.33	20.40	-6.86	1253.70	25.63	134.33	20.15	516.43
14	9.333	134.33	20.70	-6.97	1253.70	26.01	134.33	20.15	524.16
15	12.333	134.33	21.10	-7.09	1656.69	35.03	134.33	20.15	705.82
16	12.667	134.33	21.50	-7.22	1701.56	36.65	134.33	20.15	738.46
17	9.333	134.33	21.60	-7.25	1253.70	27.13	134.33	20.15	546.67
					Base Shear=	415.14			

Case 2 E/W Wind Forces									
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.75 * Story Force (k)	B (ft.)	e (ft.)	M (ft.*k)
1	10.167	134.33	13.10	-4.39	1365.73	23.41	175.50	26.33	616.22
2	9.333	134.33	13.80	-4.64	1253.70	24.79	192.00	28.80	713.89
3	9.333	134.33	15.00	-5.06	1253.70	26.96	192.00	28.80	776.36
4	11	134.33	16.10	-5.40	1477.63	34.06	192.00	28.80	980.81
5	9.333	134.33	16.80	-5.63	1253.70	30.15	192.00	28.80	868.33
6	9.333	134.33	17.40	-5.87	1253.70	31.27	192.00	28.80	900.61
7	9.333	134.33	18.00	-6.05	1253.70	32.32	192.00	28.80	930.80
8	9.333	134.33	18.40	-6.21	1253.70	33.07	192.00	28.80	952.55
9	9.333	134.33	18.90	-6.37	1253.70	33.96	192.00	28.80	978.17
10	9.333	134.33	19.30	-6.50	1253.70	34.67	192.00	28.80	998.53
11	9.333	134.33	19.70	-6.62	1253.70	35.38	192.00	28.80	1018.89
12	9.333	134.33	20.10	-6.75	1253.70	36.09	192.00	28.80	1039.25
13	9.333	134.33	20.40	-6.86	1253.70	36.63	192.00	28.80	1055.04
14	9.333	134.33	20.70	-6.97	1253.70	37.18	192.00	28.80	1070.84
15	12.333	134.33	21.10	-7.09	1656.69	50.07	192.00	28.80	1441.95
16	12.667	134.33	21.50	-7.22	1701.56	52.38	192.00	28.80	1508.63
17	9.333	134.33	21.60	-7.25	1253.70	32.42	160.50	24.08	780.42
					Base Shear=	584.81			

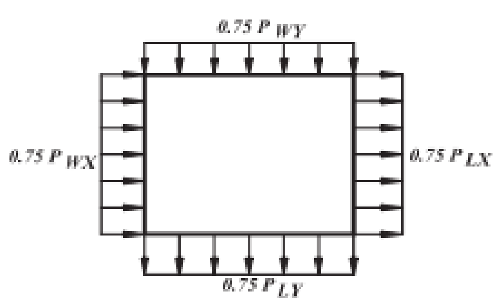


Case 3:

This case is the same described in case 1 but with three quarters of the design wind pressure being applied simultaneously to each side. The forces given in the following tables would be applied concurrently to the building as oppose to individually like in the first two cases.

Case 3 N/S Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.75 * Story Force (k)
1	10.167	134.33	13.10	-4.39	1365.73	17.92
2	9.333	134.33	13.80	-4.64	1253.70	17.34
3	9.333	134.33	15.00	-5.06	1253.70	18.86
4	11	134.33	16.10	-5.40	1477.63	23.83
5	9.333	134.33	16.80	-5.63	1253.70	21.09
6	9.333	134.33	17.40	-5.87	1253.70	21.88
7	9.333	134.33	18.00	-6.05	1253.70	22.61
8	9.333	134.33	18.40	-6.21	1253.70	23.14
9	9.333	134.33	18.90	-6.37	1253.70	23.76
10	9.333	134.33	19.30	-6.50	1253.70	24.26
11	9.333	134.33	19.70	-6.62	1253.70	24.75
12	9.333	134.33	20.10	-6.75	1253.70	25.25
13	9.333	134.33	20.40	-6.86	1253.70	25.63
14	9.333	134.33	20.70	-6.97	1253.70	26.01
15	12.333	134.33	21.10	-7.09	1656.69	35.03
16	12.667	134.33	21.50	-7.22	1701.56	36.65
17	9.333	134.33	21.60	-7.25	1253.70	27.13
					Base Shear =	415.14

Case 3 E/W Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.75 * Story Force (k)
1	10.167	175.5	13.1	-4.392	1784.31	23.41
2	9.333	192	13.8	-4.644	1791.94	24.79
3	9.333	192	15	-5.058	1791.94	26.96
4	11	192	16.1	-5.4	2112.00	34.06
5	9.333	192	16.8	-5.634	1791.94	30.15
6	9.333	192	17.4	-5.868	1791.94	31.27
7	9.333	192	18	-6.048	1791.94	32.32
8	9.333	192	18.4	-6.21	1791.94	33.07
9	9.333	192	18.9	-6.372	1791.94	33.96
10	9.333	192	19.3	-6.498	1791.94	34.67
11	9.333	192	19.7	-6.624	1791.94	35.38
12	9.333	192	20.1	-6.75	1791.94	36.09
13	9.333	192	20.4	-6.858	1791.94	36.63
14	9.333	192	20.7	-6.966	1791.94	37.18
15	12.333	192	21.1	-7.092	2367.94	50.07
16	12.667	192	21.5	-7.218	2432.06	52.38
17	9.333	160.5	21.6	-7.254	1497.95	32.42
					Base Shear =	584.81



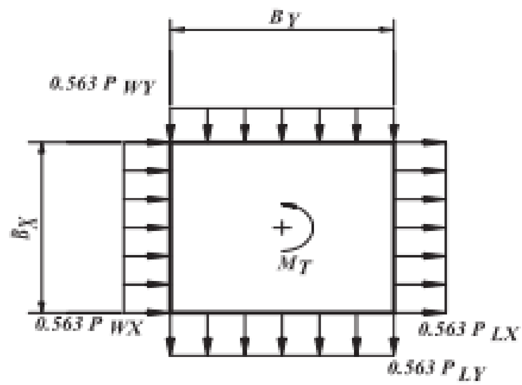
CASE 3

Case 4:

This case is the same described in case 3 but with 56.3% of the full design wind pressure being applied simultaneously to each side.

Case 4 N/S Wind Forces									
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.563 * Story Force (k)	B (ft.)	e (ft.)	M (ft.*k)
1	10.167	134.33	13.10	-4.39	1365.73	13.45	134.33	20.15	361.02
2	9.333	134.33	13.80	-4.64	1253.70	13.02	134.33	20.15	349.44
3	9.333	134.33	15.00	-5.06	1253.70	14.16	134.33	20.15	380.02
4	11	134.33	16.10	-5.40	1477.63	17.89	134.33	20.15	480.10
5	9.333	134.33	16.80	-5.63	1253.70	15.83	134.33	20.15	425.04
6	9.333	134.33	17.40	-5.87	1253.70	16.42	134.33	20.15	440.84
7	9.333	134.33	18.00	-6.05	1253.70	16.97	134.33	20.15	455.62
8	9.333	134.33	18.40	-6.21	1253.70	17.37	134.33	20.15	466.26
9	9.333	134.33	18.90	-6.37	1253.70	17.84	134.33	20.15	478.81
10	9.333	134.33	19.30	-6.50	1253.70	18.21	134.33	20.15	488.77
11	9.333	134.33	19.70	-6.62	1253.70	18.58	134.33	20.15	498.74
12	9.333	134.33	20.10	-6.75	1253.70	18.95	134.33	20.15	508.70
13	9.333	134.33	20.40	-6.86	1253.70	19.24	134.33	20.15	516.43
14	9.333	134.33	20.70	-6.97	1253.70	19.53	134.33	20.15	524.16
15	12.333	134.33	21.10	-7.09	1656.69	26.30	134.33	20.15	705.82
16	12.667	134.33	21.50	-7.22	1701.56	27.51	134.33	20.15	738.46
17	9.333	134.33	21.60	-7.25	1253.70	20.37	134.33	20.15	546.67
					Base Shear=	311.63			

Case 4 E/W Wind Forces									
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.563 * Story Force (k)	B (ft.)	e (ft.)	M (ft.*k)
1	10.167	134.33	13.10	-4.39	1365.73	17.57	175.50	26.33	616.22
2	9.333	134.33	13.80	-4.64	1253.70	18.61	192.00	28.80	713.89
3	9.333	134.33	15.00	-5.06	1253.70	20.24	192.00	28.80	776.36
4	11	134.33	16.10	-5.40	1477.63	25.56	192.00	28.80	980.81
5	9.333	134.33	16.80	-5.63	1253.70	22.63	192.00	28.80	868.33
6	9.333	134.33	17.40	-5.87	1253.70	23.47	192.00	28.80	900.61
7	9.333	134.33	18.00	-6.05	1253.70	24.26	192.00	28.80	930.80
8	9.333	134.33	18.40	-6.21	1253.70	24.83	192.00	28.80	952.55
9	9.333	134.33	18.90	-6.37	1253.70	25.50	192.00	28.80	978.17
10	9.333	134.33	19.30	-6.50	1253.70	26.03	192.00	28.80	998.53
11	9.333	134.33	19.70	-6.62	1253.70	26.56	192.00	28.80	1018.89
12	9.333	134.33	20.10	-6.75	1253.70	27.09	192.00	28.80	1039.25
13	9.333	134.33	20.40	-6.86	1253.70	27.50	192.00	28.80	1055.04
14	9.333	134.33	20.70	-6.97	1253.70	27.91	192.00	28.80	1070.84
15	12.333	134.33	21.10	-7.09	1656.69	37.58	192.00	28.80	1441.95
16	12.667	134.33	21.50	-7.22	1701.56	39.32	192.00	28.80	1508.63
17	9.333	134.33	21.60	-7.25	1253.70	24.33	160.50	24.08	780.42
					Base Shear=	438.99			



Wind Drift Checks:

The worst case drift conditions for each wind load case were determined and listed below. The maximum drifts experienced were compared to the accepted industry standard limit of H/400 for drift. All cases pass the allowable drift limits under wind loads. For each case, the maximum drift shown was measure at the 17th level of the building.

Drift due to Wind Load Cases			
Load Case	**Maximum Drift (in)**	**Allowable Drift (in)**	**Pass/Fail**
Wind Case 1 – X Direction	4.16	5.025	PASS
Wind Case 1 – Y Direction	4.52	5.025	PASS
Wind Case 2 – X Direction (+M)	2.71	5.025	PASS
Wind Case 2 – X Direction (-M)	4.07	5.025	PASS
Wind Case 2 – Y Direction (+M)	2.80	5.025	PASS
Wind Case 2 – Y Direction (-M)	4.76	5.025	PASS
Wind Case 3	3.00	5.025	PASS
Wind Case 4 (Additive +Moments)	3.78	5.025	PASS
Wind Case 4 (Additive –Moments)	3.59	5.025	PASS
Wind Case 4 (+M's in Opposite Directions)	3.69	5.025	PASS
Wind Case 4 (-M's in Opposite Directions)	4.28	5.025	PASS

Discussion of Wind Loads:

The 4 wind cases prescribed by ASCE 7-10 were analyzed and presented in the previous tables. The story forces and story drifts were determined for the given load cases. The story forces were exerted at the center of pressure for each floor and resulted in the previously noted drifts which passed the allowable threshold for lateral displacement. This drift limit is in place entirely for serviceability and not strength. Although, a strength check on two of the shear walls will be performed later in this report.

The figure shown in the bottom left of the page shows a stress contour for max shear stresses over all 14 shear walls in the building. As expected, the shear values increase as building height decreases. This fact is demonstrated by the transition from green to yellow/orange contours on the figure.

All of the shear walls experienced similar displacement values in magnitude because they are all roughly the same distant from the center of mass. The tight groupings of the shear walls also contributed to this condition.

The figure in the bottom right of the page shows an elevation of the contours due to the moments induced on the shear walls from cases 2 and 4. Shear walls 1 and 2 are shown. These are the stiffest shear walls, as they are the only ones that are 14” thick.

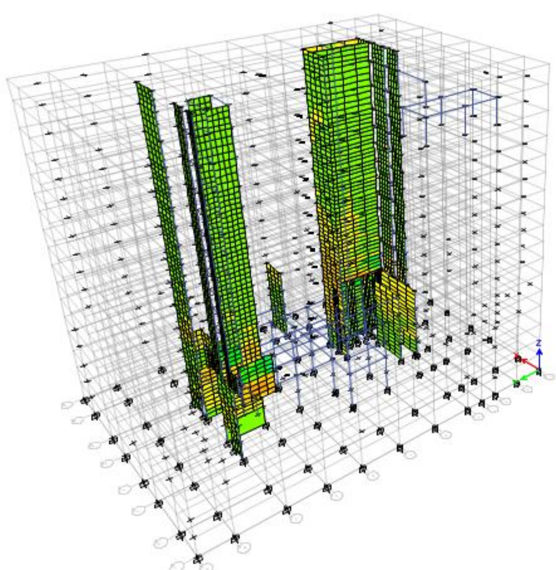


Figure 1: 3D Stress contour of shear walls under wind loading.

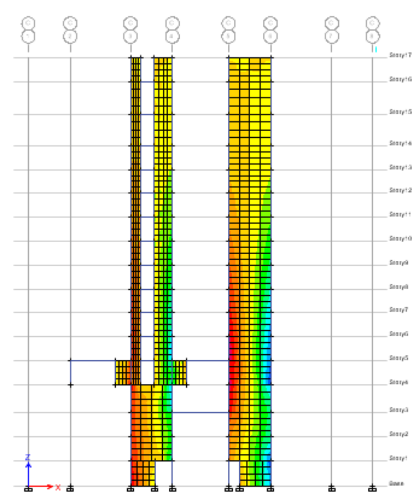


Figure 2: Elevation view of shear walls 1 and 2 under wind loading.

Shear Wall Strength Checks

① Find Controlling Load Case (ASCE 7-10, Section 2.3.2)

② $1.2D + 1.6L + 0.5L_r$

④ $1.2D + 1.0W + L + 0.5S$

⑥ $0.9D + 1.0W$

⑦ $0.9D + 1.0E$

→ Because Wind controls over seismic and creates greater story forces. Therefore case 4 and 6 will produce the critical horizontal load on a shear wall. Between the two of them, case 4 will create the greatest vertical force. Case 6 is primarily there to check against uplift. Due to the weight and nature of the structure I do not think uplift is a concern.

That being said, Case 4 will be used to check the strength of the shear walls.

The Shear Walls to be evaluated will be SW2 and SW8. Both walls will be analyzed at the base.

Shear Wall #2

Location = C5-C6
Height = 9'-4"
Length = 12'-0"
thickness = 14"

Shear Wall #8

Location = G6-J6
Height = 9'-4"
Length = 24'-0"
thickness = 12"

Shear Wall #2

Determine Dead Load

Level 17 = $[(98psf)(299.75\ ft^2) + (12)(14/12)(9.33)(150)]/1000 = 48.97^k$
Level 16 =
Level 15 =
Level 14 =
Level 13 = Levels 5-16 are the same as 17 $= 48.97^k$
Level 12 =
Level 11 =
Level 10 =
Level 9 =
Level 8 =
Level 7 =
Level 6 =
Level 5 = $[(130psf)(299.75\ ft^2) + (12)(14/12)(9.33)(150)]/1000 = 58.56^k$
Level 4 = $[(130psf)(228.62\ ft^2) + (12)(14/12)(9.33)(150)]/1000 = 49.31^k$
Level 3 = $[(130psf)(190.75\ ft^2) + (12)(14/12)(9.33)(150)]/1000 = 44.39^k$
Level 2 = Level 2 is the same as Level 3 $= 44.39^k$

Total $P = 12(48.97) + 58.56 + 49.31 + 2(44.39)$

$P = 784.29^k$

Determine Wind Load

- Wind Load found in Case 1 of Wind Loading Analysis

$W = 779.74^k$

Determine Live Load

Level 5-17 = $(50psf)(299.75\ ft^2)/1000 = 14.99^k$
Level 4 = $(50psf)(228.62\ ft^2)/1000 = 11.43^k$
Level 2-3 = $(50psf)(190.75\ ft^2)/1000 = 9.58^k$

$L = 13(14.99^k) + 11.43 + 2(9.58)$

$L = 225.46^k$

Determine Snow Load

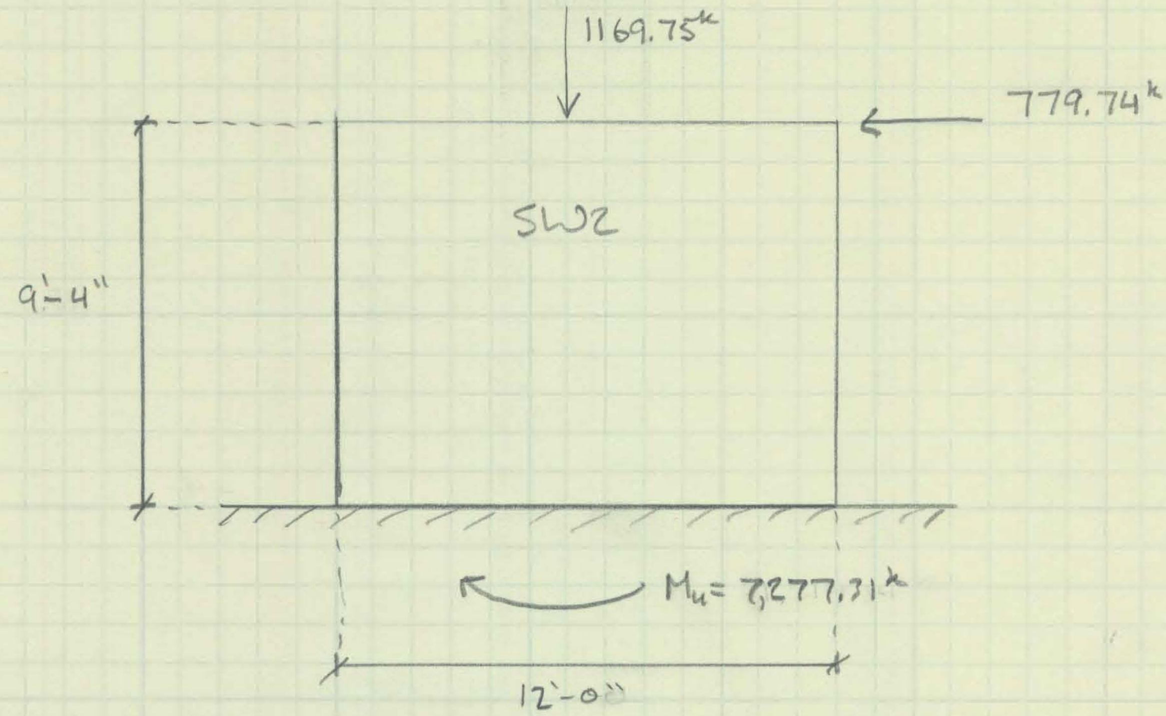
$S = (21psf)(299.75\ ft^2)/1000 = 6.29^k$

$S = 6.29^k$

Loading

$P_u = 1.2(784.29) + 225.46 + 0.5(6.29) = 1169.75^k$

$V_u = 1.0(779.74) = 779.74^k$



Check Strength of SW2

$\phi V_n \geq V_u = 779.74^k$

Determine V_c

Properties:

$\lambda = 1.0$

$f'_c = 5000$ psi

$h = 14''$

$d = 0.8\,l_w = 0.8(12) = 9.6' \times 12 = 115.2''$

$N_u = 7277.31^k$

$$V_c = 2\sqrt{5000}\,(14)(115.2)\left(\tfrac{1}{1000}\right)$$
$$= 228.1^k$$

$$V_c = \left(3.3\sqrt{5000}\,(14)(115.2) + \frac{7277.31(115.2)}{4 \cdot 144}\right)\left(\frac{1}{1000}\right)$$
$$= 337.8^k$$

$$V_c = \left[0.6\sqrt{5000} + \frac{144\left(1.25\sqrt{5000} + \frac{0.2(7277.31)}{14\,(144)}\right)}{\frac{12 \times 9.33\,V_u}{V_u} - \frac{144}{2}}\right]\frac{(14)(115.2)}{1000}$$

$$= \left[42.43 + \frac{12831.9}{39.96}\right] \times 1.613 = 586.4$$

$$V_c = \min\begin{cases} 337.8 \\ 586.4 \end{cases} \rightarrow V_c = 337.8^k > 228.1$$

$\therefore$ Use $V_c = 337.8^k$

$\phi V_c = 0.75(337.8) = 253.4^k < 779.74^k$

$\therefore$ Wall w/o reinforcement is no good... find V_s.

Determine V_s

$$V_s = \frac{A_v f_y d}{s}$$

Reinforcement: #5 @ 12" each way, each face
$f_y = 60^{ksi}$
$d = 115.2"$
$s = 12"$

$A_v = \underbrace{2(2)}_{\text{2 rows, each face}}(.31) = 1.24 in^2 / 12 in$

$$V_s = \frac{1.24 in^2 (60 ksi)(115.2)}{12"}$$
$= 714.24^k$

$V_n = V_c + V_s = 337.8 + 714.24 = 1052.0^k$

$\phi V_n = 0.75(1052) = 789.0^k > V_u = 779.74^k$ ✓

∴ SW2 is adequate for Strength

Shear Wall #8

Determine Dead Load

Level 17 $= [(130\text{ psf})(936\text{ ft}^2) + (28)(12/12)(9.33)(150)] / 1000 = 160.87^k$
Level 15-16 $= [(98\text{ psf})(576\text{ ft}^2) + (28)(12/12)(12.5)(150)] / 1000 = 108.95^k$
Level 5-14 $= [(98\text{ psf})(576\text{ ft}^2) + (28)(12/12)(9.33)(150)] / 1000 = 95.63^k$
Level 4 $= [(98\text{ psf})(576\text{ ft}^2) + (28)(12/12)(11)(150)] / 1000 = 102.65^k$
Level 3 $= [(98\text{ psf})(576\text{ ft}^2) + (28)(12/12)(9.33)(150)] / 1000 = 95.63^k$
Level 2 $= [(28)(12/12)(9.33)(150)] / 1000 = 39.2^k$

Total $P = 160.87 + 2(108.95) + 10(95.63) + 102.65 + 95.63 + 39.2$

$P = 1572.55^k$

Determine Wind Load

- Wind load found in Case 1 of Wind loading analysis

$W = 553.52^k$

Determine Live Load

Level 17 $= (50\text{ psf})(936\text{ ft}^2)/1000 = 46.8^k$
Level 3-16 $= (50\text{ psf})(576\text{ ft}^2)/1000 = 28.8^k$

$L = 46.8^k + 15(28.8)$

$L = 478.8^k$

Determine Snow Load

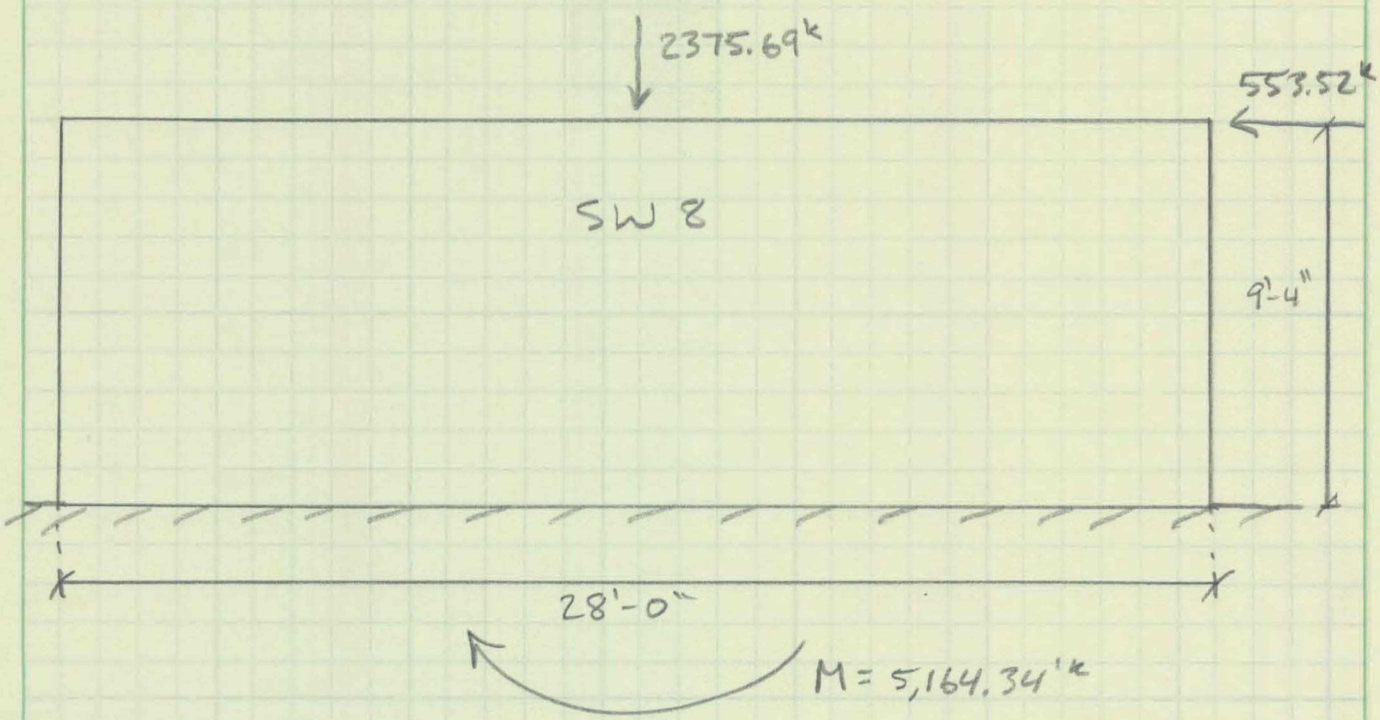
$S = (21\text{ psf})(936\text{ ft}^2)/1000 = 19.66^k$

$S = 19.66^k$

Loading

$P_u = 1.2(1572.55) + 478.8 + 0.5(19.66) = 2375.69^k$

$V_u = 1.0(553.52) = 553.52^k$



Check Strength of SW8

$$\phi V_n \geq V_u = 553.52^k$$

Determine V_c

Properties:

$\lambda = 1.0$

$f'_c = 5000$ psi

$h = 12"$

$d = 0.8 l_w = 0.8(28)(12) = 268.8"$

$N_u = 5{,}164.34'^k$

$$V_c = 2\sqrt{5000}(12)(268.8)\left(\frac{1}{1000}\right)$$

$$= 456.17^k$$

$$V_c = \left[3.3\sqrt{5000}(12)(268.8) + \frac{5164.34(268.8)}{4(12)(28)}\right]\left(\frac{1}{1000}\right)$$

$$= 753.7^k$$

$$\phi V_c = \phi V_n = 0.75(753.7) = 565.28^k > V_u = 553.52^k$$

∴ SW8 is adequate for strength. The strength check shows that the shear wall is adequate w/o reinforcing. But reinforcing is most likely required for flexure. A shear wall spot check for flexure is beyond the scope of this report but would be needed to determine if the wall really didn't need reinforcing.

Seismic Forces

As discussed in Technical Report 2, 8621 Georgia Avenue falls into a Seismic Design Category A. Due to this, the building is exempt from the more detailed analysis for seismic loading found in ASCE Ch. 11. The seismic loading for this building is governed by the provisions in Section 1.4 for the general structural integrity of the building.

Therefore, the seismic story forces are given by taking 1/100th of the story weight. A rough approximation of the story weights was performed in Technical Report 2. The following table includes a more detailed summation of the total dead load structural mass on each floor. Because the simplified method for determining seismic story forces is entirely dependent on mass, the story forces are the same in both the X and Y direction.

Tables 12.3-1, 2 were investigated for horizontal and vertical building irregularities. None of the irregularities are applicable for Seismic Design Category A so no additional requirements are necessary. The building maintains a relatively geometric profile throughout its perimeter and height so this is a reasonable conclusion.

		Columns						Shear Walls				Floors					
		16"x24"			18"x24"			12		14		8		7.25			
	Floor ht.	#	Area (ft2)	Force (k)	#	Area (ft2)	Force (k)	Length (ft.)	Force (k)	Length (ft.)	Force (k)	Area (ft2)	Force (k)	Area (ft2)	Force (k)	Total Story Weight (k)	Lateral Seismic Story Force (k)
Floor 17	9.33	48	2.67	179.20	0	3	0	262.39	367.34	46.66	76.21	0	0	784.00	71.05	693.81	6.94
Floor 16	12.67	48	2.67	243.20	0	3	0	262.39	498.54	46.66	103.43	0	0	17008.00	1541.35	2386.52	23.87
Floor 15	12.33	61	2.67	300.93	0	3	0	262.39	485.42	46.66	100.71	0	0	21479.00	1946.53	2833.59	28.34
Floor 14	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 13	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 12	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 11	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 10	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 9	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 8	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 7	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 6	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 5	9.33	60	2.67	224.00	0	3	0	262.39	367.34	46.66	76.21	21479.00	2147.90	0	0	2815.45	28.15
Floor 4	11.00	0	2.67	0	74	3	366.30	274.57	453.04	36.66	70.57	25136.00	2513.60	0	0	3403.51	34.04
Floor 3	9.33	0	2.67	0	52	3	218.40	274.57	384.40	36.66	59.88	25136.00	2513.60	0	0	3176.27	31.76
Floor 2	9.33	0	2.67	0	62	3	260.40	274.57	384.40	36.66	59.88	16746.00	1674.60	0	0	2379.27	23.79
Floor 1	10.17	0	2.67	0	66	3	301.95	274.57	418.72	36.66	65.22	21076.00	2107.60	0	0	2893.50	28.93
																Base Shear	441.42

Seismic Drift Checks

After a seismic analysis of the building was performed using ETABS. The results below document the story displacement and story drift. The allowable drift limit under seismic load was determined using Table 12.12-1 in ASCE 7-10 for allowable Seismic Story Drift. For a building of risk category I, the allowable story drift is 2%. The maximum drift values occurred at the 17th floor and all passed the allowable drift limit.

Displacements due to Seismic Loading								
	X Direction				**Y Direction**			
Floor	**Story Displacement (in.)**	**Story Drift (%)**	**Allowable Drift (%)**	**Pass/Fail**	**Story Displacement (in.)**	**Story Drift (%)**	**Allowable Drift (%)**	**Pass/Fail**
17	2.85	0.144	2%	PASS	1.52	0.076	2%	PASS
16	2.74	0.142	2%	PASS	1.45	0.076	2%	PASS
15	2.41	0.138	2%	PASS	1.28	0.073	2%	PASS
14	2.00	0.125	2%	PASS	1.07	0.067	2%	PASS
13	1.78	0.120	2%	PASS	0.95	0.064	2%	PASS
12	1.55	0.113	2%	PASS	0.83	0.060	2%	PASS
11	1.33	0.105	2%	PASS	0.71	0.056	2%	PASS
10	1.11	0.097	2%	PASS	0.60	0.052	2%	PASS
9	.90	0.087	2%	PASS	0.49	0.047	2%	PASS
8	.71	0.077	2%	PASS	0.38	0.041	2%	PASS
7	.52	0.064	2%	PASS	0.28	0.035	2%	PASS
6	.35	0.050	2%	PASS	0.20	0.028	2%	PASS
5	.20	0.034	2%	PASS	0.12	0.020	2%	PASS
4	.07	0.015	2%	PASS	0.07	0.014	2%	PASS
3	.03	0.008	2%	PASS	0.04	0.012	2%	PASS
2	.04	0.002	2%	PASS	0.03	0.011	2%	PASS

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

Structure	Risk Category: I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$\boxed{0.020h_{sx}}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.

Discussion:

The effective seismic forces on the building were analyzed for both the X and Y directions of the building. All floors passed the allowable drift limit under seismic loading in both directions. As previously mentioned, the seismic forces on the building are attributed to the weight of the building. As simplifications for the model were utilized, some elements bearing mass were not modeled. Although most of the significant elements effecting the seismic weight were modelled, this will have a small effect on the drifts and story forces.

The story drift of the building was much larger (nearly twice as much) in the X direction as compared to the Y direction. This result was expected due to the geometry of the building. The stiffness as well as the length of the building in the Y direction is greater than it is in the X direction. The higher stiffness in that direction resulted in a smaller displacement. This same condition was also present in the displacements due to wind loading.

Overturning and Foundation Impact

The overturning and foundation impacts due to wind and seismic loading were considered. The table below shows the base shear and overturning moment applied due to each load case. The controlling overturning moments, in both direction, were caused by case 1 of the wind load cases. The applied moments were compared to the resisting moment due to the building weight. The safety factor between the resisting and applied moments was calculated. Code dictates that the safety factor is greater than 1.5 but standard industry practice uses a factor between 2 and 3. The factors resulting from this analysis are both in excess of 56. Therefore, the building is more than adequate to handle the overturning moment. This result is not surprising because overturning is typically not a problem for concrete buildings due to their density and weight.

Overturning Moments

Load Cases	Base Shear X Direction (k)	Base Shear Y Direction (k)	Overturning X Direction (‘ k)	Overturning Y Direction (‘ k)
Wind Case 1 – X Direction	779.74	-	**52,242.58**	-
Wind Case 1 – Y Direction	-	553.52	-	**53,137.92**
Wind Case 2 – X Direction (+M)	584.81	-	39,182.27	-
Wind Case 2 – X Direction (-M)	584.81	-	39,182.27	-
Wind Case 2 – Y Direction (+M)	-	415.14	-	
Wind Case 2 – Y Direction (-M)	-	415.14	-	39,853.44
Wind Case 3	584.81	415.14	39,182.27	39,853.44
Wind Case 4 (Additive +Moments)	438.99	311.63	29,412.33	39,853.44
Wind Case 4 (Additive –Moments)	438.99	311.63	29,412.33	29,916.48
Wind Case 4 (+M’s in Opposite Directions)	438.99	311.63	29,412.33	29,916.48
Wind Case 4 (-M’s in Opposite Directions)	438.99	311.63	29,412.33	29,916.48
Seismic X	441.42	-	29,575.14	-
Seismic Y	-	441.42	-	42,376.32

Resisting Moment:

X Direction:

$M_{resisting}$ = 44,142.34^k x 67 ft. = 2,957,536.78 ‘ k

$$\frac{2{,}957{,}536.78}{52{,}242.58} = 56.6 > 1.5$$

Y Direction:

$M_{resisting}$ = 44,142.34^k x 96 ft. = 4,237,664.64 ‘ k

$$\frac{4{,}237{,}664.64}{53{,}137.92} = 79.7 > 1.5$$

Conclusion

Technical Report 4 consisted of a complete analysis of the lateral system of 8621 Georgia Avenue. The analysis was comprised of 3D computer modeling in order to accurately distribute lateral forces due to wind and seismic onto the building. The overturning and foundation effects due to these forces was also investigated. Strength and drift checks were performed for both wind and seismically loaded members.

ETABS was the computer software that was utilized to create an accurate and simple model of the buildings lateral system. The shear walls, drop beams and supporting columns were modelled as the individual parts of the lateral system. The induced forces, moments, and displacements of these members was recorded and analyzed.

The lateral system was exposed to wind and seismic loading. The load cases set forth in ASCE 7-10 were used to determine the controlling combinations on the structure. The wind and seismic cases considered both direct loading as well as torsion induced moments due to a center of mass and center of rigidity differential. The strength checks were performed on two of the shear walls. Both passed for shear capacity. The story drifts of the floors was also calculated for wind and seismic loading. Both conditions passed the allowable drift limits.

The overturning and foundation impacts these loads would have on the building was considered. The cumulative moments at the base of the structure were calculated and compared to the resisting moment of the structure. This comparison was carried out in both the X and Y direction of the building. In both cases, the building was determined to be adequate for overturning moment while causing minimal foundation issues.

After the completion of this lateral system analysis, it has been determined that the lateral system of 8621 Georgia Avenue is sufficiently designed to resist the lateral loading conditions of wind and seismic.

In addition to the results obtained in Technical Report 3, it has been concluded that 8621 Georgia Avenue is adequately designed for both gravity and lateral loads.

Appendix A

Center of Mass

Floor	height
1	10.167
2	9.333
3	9.333
4	11
5	9.333
6	9.333
7	9.333
8	9.333
9	9.333
10	9.333
11	9.333
12	9.333
13	9.333
14	9.333
15	12.333
16	12.667
17	9.333
Concrete =	150
Height =	10.167

Shear Wall	Length	Thickness	Weight	Position from (0,0)	
				X	Y
1	18.33	14	32613.194	86.32	40.25
	28.33	14	50405.444	86.32	40.25
2	18.33	14	32613.194	56.16	40.25
3	9.33	12	14228.717	94.49	50.25
4	21.5	12	32788.575	78.16	51.25
	9.33	12	14228.717	78.16	51.25
5	19.5	12	29738.475	56.16	121.25
6	42.33	12	64555.367	58.99	151.58
7	32.67	12	49823.383	48	136.42
8	32.67	12	49823.383	40	136.42
9	21.5	12	32788.575	56.16	52.25
10	19.83	12	30241.742	39.83	45.25
11	18.33	12	27954.167	56.16	50.25
12	18.33	12	27954.167	56.16	60.25
13	16	12	24400.8	8.17	107.25
14	22.75	12	34694.888	67.16	142

	Center of Mass	
Floor	X	Y
1	79.46	96.24
2	79.46	96.24
3	79.46	96.24
4	84.90	110.00
5	78.15	90.32
6	78.15	90.32
7	78.15	90.32
8	78.15	90.32
9	78.15	90.32
10	78.15	90.32
11	78.15	90.32
12	78.15	90.32
13	78.15	90.32
14	78.15	90.32
15	78.15	90.32
16	78.15	90.32
17	78.15	90.32

-Taken from point L-1

$$x_{cm} = \frac{\sum_{i=1}^{N} m_i x_i}{M}$$

Center of Rigidity

E =	4069
G =	1627.6

Floor	height
1	10.167
2	9.333
3	9.333
4	11
5	9.333
6	9.333
7	9.333
8	9.333
9	9.333
10	9.333
11	9.333
12	9.333
13	9.333
14	9.333
15	12.333
16	12.667
17	9.333
h =	9.333

Shear Wall	Direction	b	b^3	h/b	Distance from (0,0)		Rx	Ry	Rx y	Ry x
1	X	18.33	6158.677	0.509165	47.68	151.75	2451.9	-	372083	-
	X	28.33	22737.34	0.329439	47.68	151.75	3973.4	-	602957	-
2	X	18.33	6158.677	0.509165	77.84	151.75	2451.9	-	372083	-
3	Y	9.33	812.1662	1.000322	39.51	141.75	-	1016.8	-	40172
4	Y	21.5	9938.375	0.434093	55.84	140.75	-	2939.9	-	164162
	Y	9.33	812.1662	1.000322	55.84	140.75	-	1016.8	-	56776
5	X	19.5	7414.875	0.478615	77.84	70.75	2632.8	-	186273	-
6	X	42.33	75848.12	0.220482	75.01	40.42	6053.6	-	244686	-
7	Y	32.67	34869.64	0.285675	86	55.58	-	4622.1	-	397499
8	Y	32.67	34869.64	0.285675	94	55.58	-	4622.1	-	434476
9	Y	21.5	9938.375	0.434093	77.84	139.75	-	2939.9	-	228839
10	Y	19.83	7797.729	0.470651	94.17	146.75	-	2683.7	-	252721
11	X	18.33	6158.677	0.509165	77.84	141.75	2451.9	-	347564	-
12	X	18.33	6158.677	0.509165	77.84	131.75	2451.9	-	323044	-
13	X	16	4096	0.583313	125.83	84.75	2088.4	-	176989	-
14	X	22.75	11774.55	0.410242	66.84	50	3130.6	-	156528	-

Center of Rigidity	
X	Y
82.368	93.040

COR		
Floor	X	Y
1	78.654	86.222
2	78.504	86.468
3	78.504	86.468
4	78.807	85.959
5	82.368	93.04
6	82.368	93.04
7	82.368	93.04
8	82.368	93.04
9	82.368	93.04
10	82.368	93.04
11	82.368	93.04
12	82.368	93.04
13	82.368	93.04
14	82.368	93.04
15	83.271	92.543
16	83.369	92.481
17	82.368	93.04

$$X_R = \frac{\sum R_y x}{\sum R_y}$$

$$Y_R = \frac{\sum R_x\, y}{\sum R_x}$$