TECHNICAL REPORT IV

Daniel Goff
Structural Option

Faculty Advisor
Linda M. Hanagan

Letter of Transmittal

Daniel Goff

Structural Option

November 17, 2014

Dr. Hanagan

Advisor

The Pennsylvania State University

Dear Dr. Hanagan,

The following technical report was prepared to meet requirements from AE 481W. The report contains a thorough structural analysis of the medical office buildings lateral force resisting system under wind and seismic loadings calculated in technical report II. The lateral systems loading results were compared to industry standards for strength and serviceability using an ETABS structural model. This models results were interpreted and then validated by hand.

Thank you for taking the time to review this report, I look forward to reviewing your feedback.

Sincerely,

Daniel E. Goff

Executive Summary

The Primary Health Networks Medical Office Building is located in Sharon, Pa in between Pitt and E Silver streets next to the Shenango River. It will be a 5 story structure rising 85 feet, having four elevated floors and a roof. The building offers 78,000 square feet of occupiable space and will cost approximately $10 million.

The site soil was found to have a bearing capacity of 2500psi allowing for concrete spread and mat footings to serve as a foundation for the building. The building is primarily a steel framed structure with steel columns supporting wide flange steel girders and steel bar joists. Typical sizes for floor joists and girders range from 10 inch to a maximum depth of 24 inches. The floor structure is concrete on metal deck for all four elevated floors, whereas the first floor is concrete slab on grade. Typical bay sizes range from 30'x26' to 33'-10"x30'.

The building's lateral force resisting system is comprised of three Ivany block shear walls. Ivany block is a concrete masonry unit with pre-determined locations for the rebar and having an f'm of 3000psi. The shearwalls are located around stairwells throughout the building.

Typical shear and moment connections are to be designed by the steel fabricator. Other connections typical to this building discussed in detail include joist to ivany block wall connections and concrete slab on metal deck to ivany block to wall connections.

The building was designed using the International Building code (IBC) edition 2009 which references the American Society of Civil Engineers (ASCE) document 7-05. The exception to this is the lateral loads on the building, which were determined with and designed to the IBC 2012 -edition which adopts ASCE 7-10.

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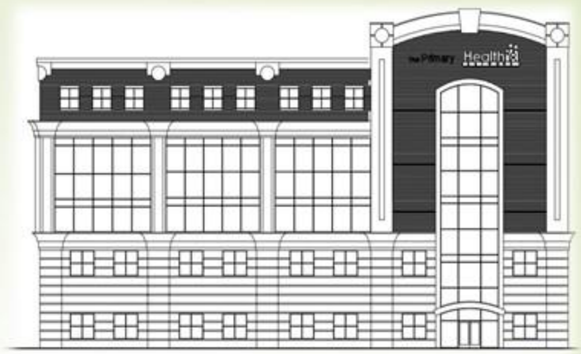
Building Abstract

The Primary Health Network's Medical Office Building
Sharon, PA



General Information

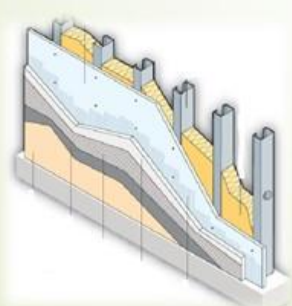
Height: 82ft
Size: 78,000 sq. ft.
Cost: $10 million
Construction: November 2014-January 2016
Project Delivery Method: Design-Build

Project Team

Owner: The Primary Health Network
Architect: John N Guitza Associates, Inc.
Structural Engineer: Taylor Structural Engineers
MEP Engineer: BDA Engineering
Construction Manager: Hudson Construction
Civil Engineer: Professional Service Industries, Inc.

Architecture

The primary architectural goal was to create a modern look with a strong focus on economy. This was accomplished by methods such as incorporating an exterior finish/insulation system (E.I.F.S. shown below).



Mechanical System

Variable Air Volume system comprised of (2) 65 ton units and (1) 30 ton unit

Lighting and Electrical Systems

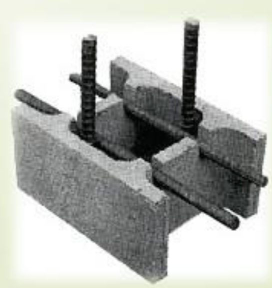
(5) 120/208V 3 Phase panel boards
(6) 480/277V 3 Phase panel boards

Low voltage dual technology occupancy sensors are used to increase efficiency

Structural System

Foundation: Concrete spread and Mat footings
Gravity: Steel columns and wide flange girders, steel bar joists, and normal weight concrete on metal deck floors

Lateral: 3 Ivany block shear walls
(Ivany Block Pictured below)



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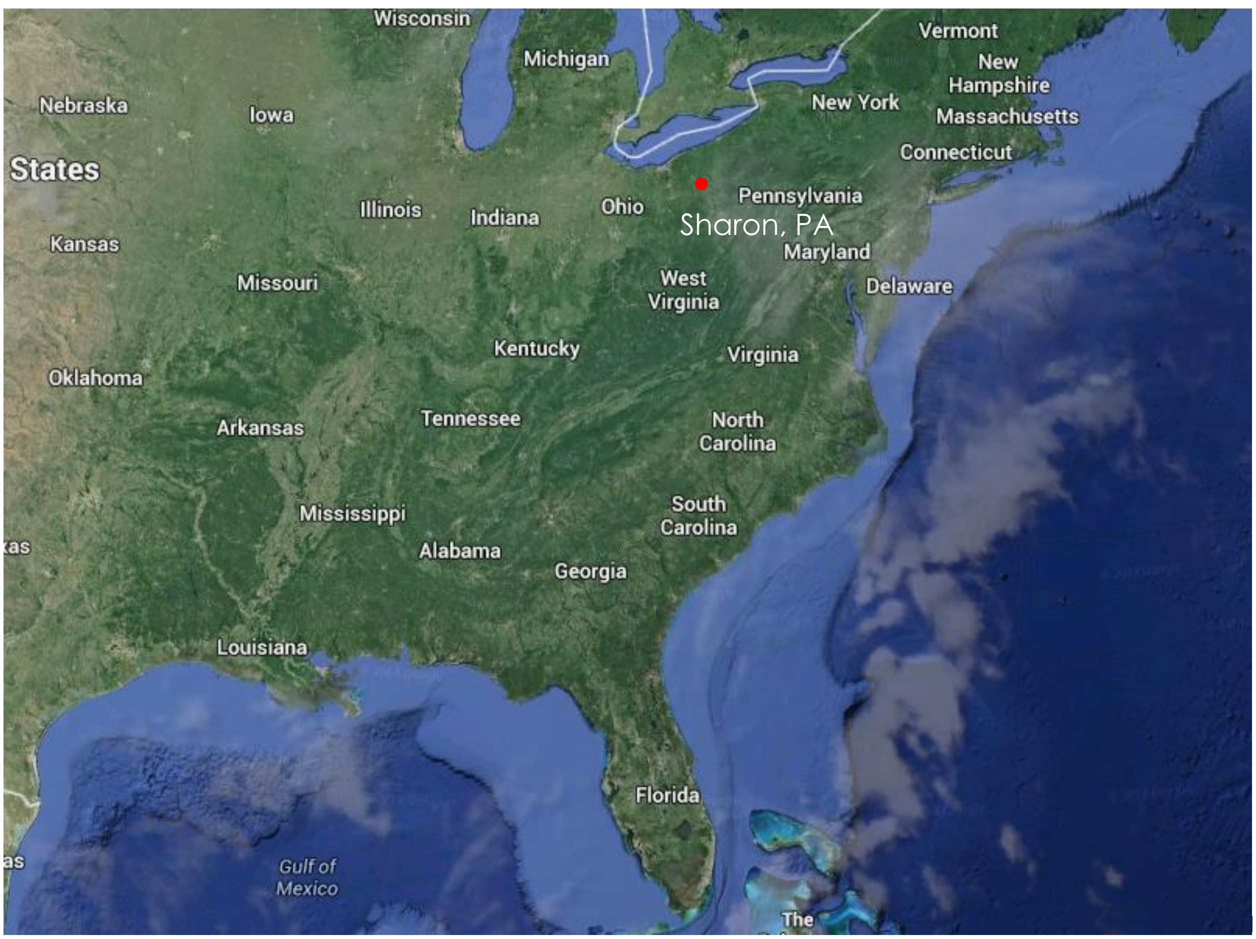
Site Plan

Location Plan



Preparatory Documents

Building Code:	2012 International Building Code (IBC)
Steel:	American Institute of Steel Construction (AISC)
Welding:	American Welding Society
Concrete:	American Concrete Institute (ACI)
Concrete Masonry:	American Concrete Institute (ACI)
	American Society of Civil Engineers (ASCE)
	ASCE 7-05
	ASCE 7-10 (for lateral loads only)

Load Summary and Revisions

All lateral loads were calculated in technical report II in accordance with ASCE 7-10. Wind loads were evaluated using the directional procedure subjected to a case 1 loading. This loading was selected to be the appropriate due to the buildings geometry and should result in the highest overall forces.

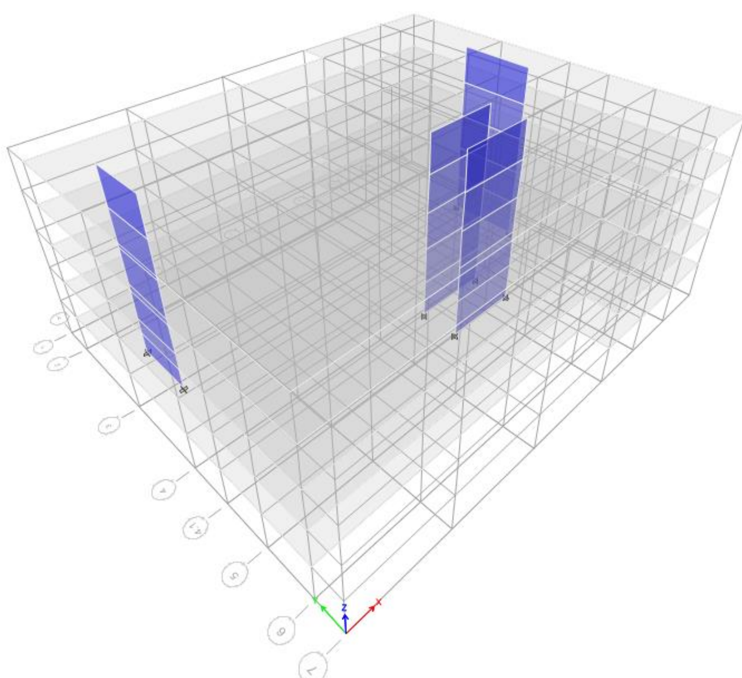
Story	Wind in the X	Wind in the Y	Seismic
Roof	64.33kip	53.69kip	29.84kip
Four	61.79kip	51.58kip	87.05kip
Three	59.13kip	49.35kip	64.66kip
Two	55.25kip	46.12kip	44.77kip
One	49.74kip	41.52kip	21.90kip

Above are the calculated story forces found in technical report II.

Modeling Process

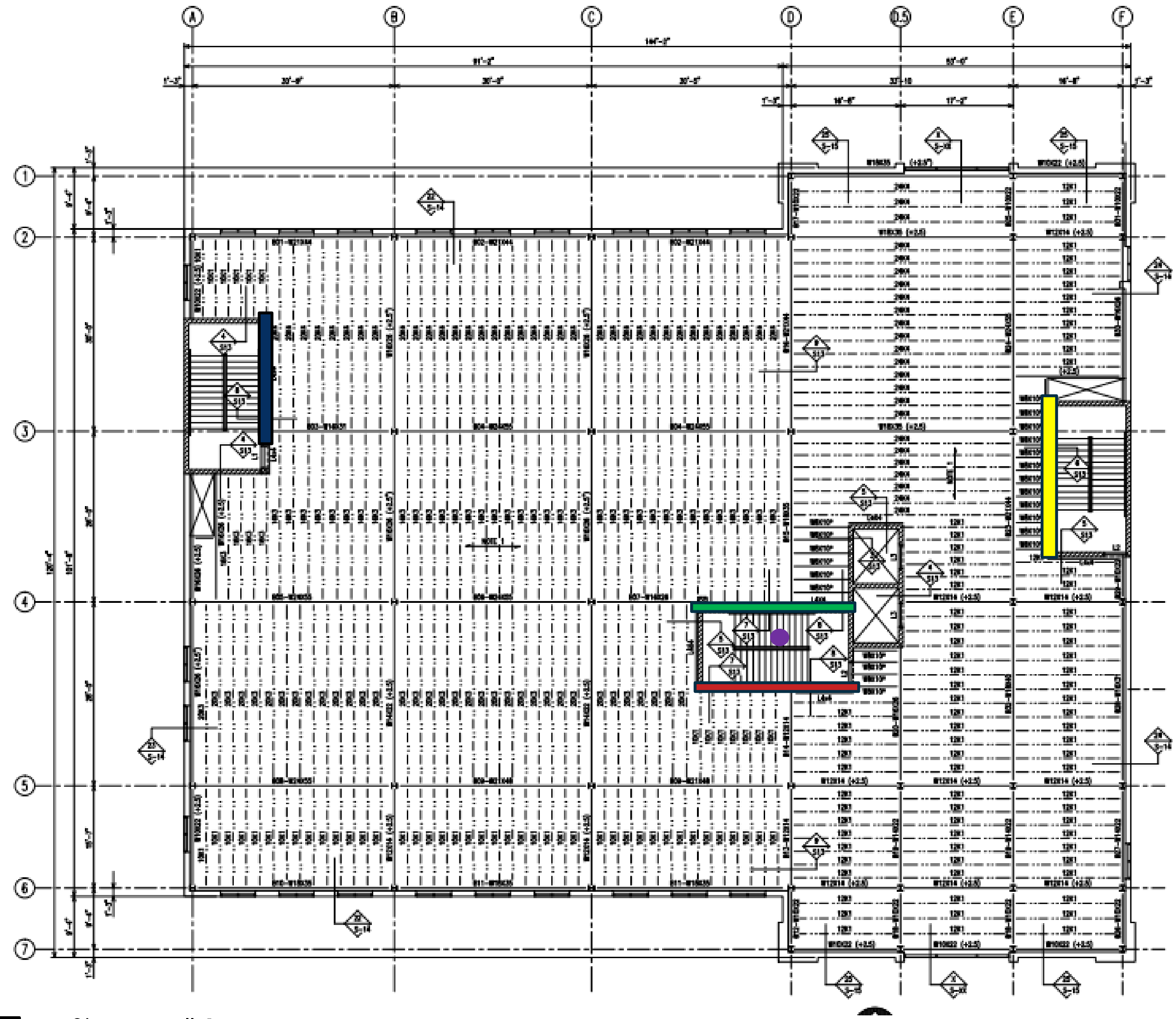
Ram Structural Systems was initially chosen as the modeling program due to familiarity. After modeling part of the gravity system it was determined that model was becoming unnecessarily over-complicated. To simplify the model only the lateral system was modeling using ETABS 2013. The four masonry shear walls were modeled using the properties associated with 3000psi f'm, 2700ksi E masonry. The cracked section modifier for concrete of 0.7 from ASCE7-10 was used for the full height of all walls. The fully grouted masonry walls will exhibit similar performance to 3000psi f'c concrete and therefore the concrete section of ASCE7-10 can be used. The walls were to the rigid floor diaphragm that was created at each level. The weight of the floor structure was included in all previously calculations and therefore the floor diaphragms were modeled as having no mass. The walls were modeled as fully fixed at the base level.

The wind loads were taken from technical report II and applied at the center of each diaphragm in its respective direction as a point load assuming the rigid diaphragm will distribute the load based on stiffness. The corrected seismic loads from technical report II were applied to the buildings center of mass as point loads at each floor level.



Building lateral system model from ETABS2013

Modeling checks



= Shearwall 1

= Shearwall 2

= Shearwall 3

= Shearwall 4

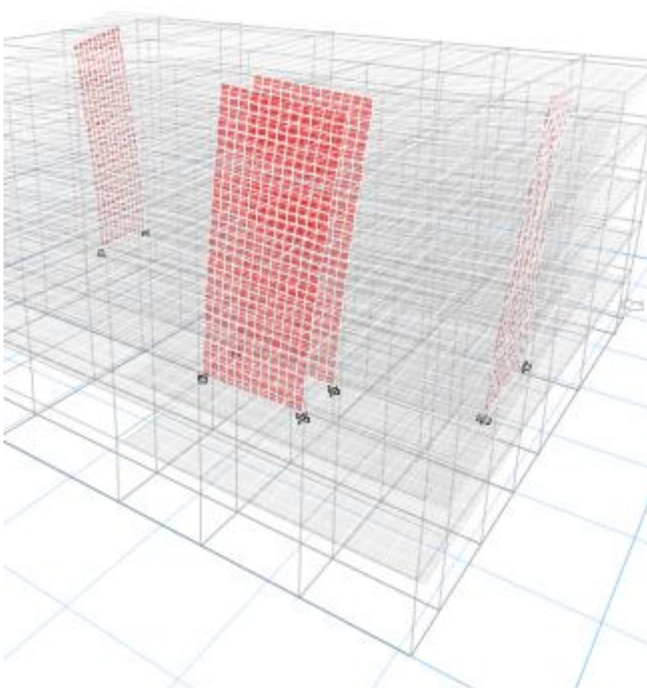
= Center of Rigidity (89.8ft,45.5 ft)

Center of Rigidity

Story	Diaphragm	Mass X lb-s²/ft	Mass Y lb-s²/ft	XCM ft	YCM ft	Cumulative X lb-s²/ft	Cumulative Y lb-s²/ft	XCCM ft	YCCM ft	XCR ft	YCR ft
Roof	D1	41.54	41.54	90.6919	64.2778	41.54	41.54	90.6919	64.2778	89.4399	45.5115
Story4	D1	166.16	166.16	82.6111	58.8232	207.7	207.7	84.2273	59.9141	88.9321	45.5194
Story3	D1	166.16	166.16	82.6111	58.8232	373.86	373.86	83.509	59.4293	87.9765	45.539
Story2	D1	166.16	166.16	82.6111	58.8232	540.02	540.02	83.2327	59.2428	85.9811	45.5744

The center of rigidity for the structure is highlighted in red above. The center of rigidity in the x direction moves to the right by a total distance of 3.46' over the height of the structure, this is a 2.4% difference and can be considered negligible. The reason for the change in XCR is due to the differing lengths of shear wall effective in this direction. Shear wall 1 (as seen in plan above) is only 19' in length whereas shear wall 4 is 24' in length. Rigidity is a factor of displacement, which is based heavily on wall length.

The center of rigidity was calculated by hand at the roof level in order to consider ultimate displacements and to consider the highest value eccentricity. The calculated value for the center of rigidity was found to be XCR=89.8ft and YCR=45.5ft. This gives an error value of 0.4% in the x direction, and a value of 0.03% error in the y direction.



The figure shows the shear walls deflected shapes, the walls deflected in a manner consistent with the loading as expected.

Wind load test results, x direction

Equilibrium check:

Loads:

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Roof	60	64.33	0
Story4	48	61.79	0
Story3	36	59.13	0
Story2	24	55.25	0
Story1	12	49.74	0
Base	0	0	0

Base Shear

Table 5.1 - Base Reactions

Load Case/Combo	FX kip	FY kip
Wind-x	-290.24	0

$\sum$ X-Direction = 290.24K, base shear = 290.24K

Story Drifts

Story	Load Case/Combo	Direction	Maximum in	Average in	Ratio
Roof	Wind-x	X	0.174266	0.090359	1.928599
Story4	Wind-x	X	0.130794	0.068267	1.915924
Story3	Wind-x	X	0.087663	0.046163	1.899001
Story2	Wind-x	X	0.048057	0.025668	1.872237
Story1	Wind-x	X	0.016696	0.009176	1.819578
Base	Wind-x	Y	0	0	

This gives an average story drift of L/1990. This is expected since shear walls are inherently the stiffest of standard lateral force resisting elements. The 12" thick shear walls are also longer than is typical for a building this size comprising 30% of the effective building length.

Wind load test results, y direction

Equilibrium check:

Loads:

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Roof	60	0	53.69
Story4	48	0	51.58
Story3	36	0	49.35
Story2	24	0	46.12
Story1	12	0	41.52
Base	0	0	0

Base Shear

Table 5.1 - Base Reactions

Load Case/Combo	FX kip	FY kip
Wind-y	-5.837E-06	-242.26

∑ Y-Direction = 242.26K, base shear = 242.26K

Story Drifts

Story	Load Case/Combo	Direction	Maximum in	Average in	Ratio
Roof	Wind-y	Y	0.547086	0.260333	2.101482
Story4	Wind-y	Y	0.40622	0.193288	2.101626
Story3	Wind-y	Y	0.26818	0.127566	2.102294
Story2	Wind-y	Y	0.143357	0.068139	2.103892
Story1	Wind-y	Y	0.04712	0.022342	2.109054
Base	Wind-y	Y	0	0	

This gives an average story drift of L/642. This is an acceptable value for story drifts and is expected for the same reasoning as state previously.

Seismic load Test results

Equilibrium check:

Loads:

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Roof	60	29.84	0
Story4	48	87.05	0
Story3	36	64.66	0
Story2	24	44.77	0
Story1	12	21.9	0
Base	0	0	0

Base Shear

Table 5.1 - Base Reactions

Load Case/Combo	FX kip	FY kip
Seismic	-248.22	0

∑ X-Direction = 248.22K, base shear = 248.22K

Story Drifts

Story	Load Case/Combo	Direction	Maximum in	Average in	Ratio
Roof	Seismic	X	0.196587	0.163496	1.202399
Story4	Seismic	X	0.148815	0.123984	1.20027
Story3	Seismic	X	0.100008	0.083494	1.197775
Story2	Seismic	X	0.054435	0.045579	1.194284
Story1	Seismic	X	0.018395	0.01549	1.187566
Base	Seismic	Y	0	0	

The total displacement is 0.517 in. giving a value of L/1856. This is consistent with previous results.

Hand Calculations

Determine Center of Rigidity

All shear walls will be treated as cantilevers

$G_{masonry} \approx 0.4E \qquad E_m = 900(3{,}000) = 2700 \text{ ksi}$

Determine Shear wall Stiffness

$$k = \frac{E}{4\left(\frac{h}{b}\right)^3 + 3\left(\frac{h}{b}\right)}$$

$$k_1 = \frac{2700}{4\left(\frac{960}{138}\right)^3 + 3\left(\frac{960}{138}\right)} = 8.67 \text{ k/in}$$

$$k_2 = \frac{2700}{4\left(\frac{960}{288}\right)^3 + 3\left(\frac{960}{288}\right)} = 17.07 \text{ k/in}$$

$$k_3 = k_4 = k_2 = 17.07 \text{ k/in}$$

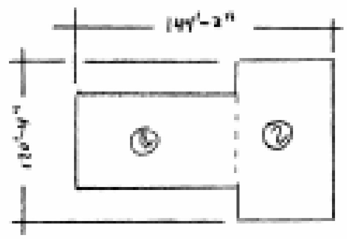
Center of Rigidity

Element	effective Direction	Dist from ref. Datum x	Dist from ref. Datum y	R_x	R_y	R_xY	R_yX
k_1	Y	11.5 ft	—	0	8.67	—	99.7
k_2	x	—	51.5 ft	17.07	0	879.1	—
k_3	x	—	39.5 ft	17.07	0	674.3	—
k_4	Y	129.5 ft	—	0	17.07	—	2210.6
Sum				34.15	25.74	1553.4	2310.3

$$\boxed{X_r = \frac{2310.3}{25.74} = 89.8 \text{ ft}} \qquad \boxed{Y_r = \frac{1553.4}{34.15} = 45.5 \text{ ft}}$$

Center of Mass

Assuming mass is uniformly distributed throughout the building, and assigning an arbitrary value of 1 psf to the structure the C.O.M. was calculated as follows



$$\bar{y} = \frac{120.167}{2} = 60.1'$$

$$\bar{x} = \frac{(9287)(45.75') + (6386)(118')}{(9287 + 6386)}$$

$$\bar{x} = 75.2'$$

Eccentricity

$$e = C.O.R - C.O.M$$

$$e_x = 89.8' - 75.2' = \underline{14.6'}$$

$$e_y = 45.5' - 60.1' = \underline{14.6'}$$

Torsional Rigidity (J)

$$J = \Sigma R_i d_i^2$$

$$J = (8.67)(78.3)^2 + (17.07)(6^2) + 17.07(6)^2 + 17.07(37.7)^2$$

$$= 81,288 \ \frac{k}{in} ft^2$$

Direct Shear on walls 1+4 (wind)

$V = 304^k$

$$V_1 = \frac{8.67}{8.67+17.07}(304) = 102.4^k$$

$$V_2 = \frac{17.07}{8.67+17.07}(304) = 201.6$$

Torsional Shear in walls 1+4 (wind)

$$V_1 = \frac{V(d_1 R_1)e}{J}$$

$$V_1 = \frac{304(78.3)(8.67)(14.6)}{81,288} = 37 \uparrow$$

$$V_2 = \frac{304(39.7)(17.07)(14.6)}{81,288} = 37 \downarrow$$

Total shear in walls 1+4 (wind)

$$V_1 = 102.4 + 37 = 139.4^k$$

$$V_2 = 201.6 - 37 = 164.6^k$$

Flexural Capacity of wall 1

Find M_{u_1} $\frac{8.67}{17.07+8.67} = 34\%$

Shear wall 1 recieves 34% of lateral loads

$(0.34)(53.9) = 18.3^k$
$(0.34)(51.6) = 17.5^k$
$(0.34)(49.4) = 16.8^k$
$(0.34)(46.1) = 15.7^k$
$(0.34)(41.5) = 14.1^k$

$M_u = (18.3)(80) + 17.5(64) + 16.8(48) + 15.7(32) + 14.1(16)$

$M_u = 4119$ ft-k

$$\emptyset M_n = 0.9(A_s f_y)(d - a/2)$$

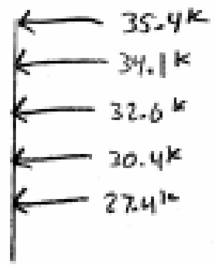
$$= 0.9(12(0.6)(60)((19-2)(12) - \frac{1.96(12)(0.6)}{2})$$

$$= 6,381^{k\text{-}ft}$$

$6381 > 4119$ ✓

Flexural Capacity of wall 4

Find M_{uy} Shear wall 4 recieves 66% of lateral loads



$$M_u = 35.4(80) + 34.1(64) + 32.6(48) + 30.4(32) + 27.4(16)$$

$$M_u = 7991^{\text{ft-k}}$$

$$\phi M_n = 0.9(A_s f_y)(d - a/2)$$

$$\phi M_n = 0.9\left(12(0.6)(60)\right)\left((24-2)12 - \frac{1.96(12)(0.6)}{2}\right)$$

$$= 8325^{\text{k-ft}}$$

$$8325 > 7991 \checkmark$$

Appendix A

Wind Loads

PENNSTATE Engineering

CLASS:
SHEET NO:
DESIGNED BY:
JOB NAME:
SECTION:
OF
DATE:

Wind Analysis

ASCE 7-10 Chapter 27

1.) risk category II — Table 1.4-1

2.) $V_u = 115$ mpH — Figure 26.5-1A

$I = 1.0$

3.) $K_d = 0.85$ — Table 26.6-1

exposure B — Section 26.7

$K_{zt} = 1.0$ — Table 26.8-1

Gust effect, $G = 0.85$ — Section 26.9

Enclosed Structure — Section 26.10

$GC_{pi} = \pm 0.18$ — Table 26.11-1

4.) $K(15') = 0.57$ — Table 27.3-1

$K(30') = 0.70$

$K(45') = 0.79$

$K(60') = 0.85$

$K(75') = 0.91$

$K(85') = 0.95$

Building natural frequency will be determined using eq 26.9-4

$n_a = \frac{75}{75'} = 1 \rightarrow$ Rigid

PENNSTATE Engineering

CLASS: SECTION:
SHEET NO: OF
DESIGNED BY: DATE:
JOB NAME:

5) Q_z or Q_h $\quad q_h = 0.00256(K_z)(K_{zt})(K_d)(V^2)$ $\quad$ EQ 27.3-1

$K_{zt} = 1.0$

$q_{15'} = 0.00256(0.85)(0.57)(115\,psf)^2 = 16.49\,psf$

$q_{30'} = 0.00256(0.85)(0.70)(115\,psf)^2 = 20.14\,psf$

$q_{45'} = 0.00256(0.85)(0.79)(115\,psf)^2 = 22.73\,psf$

$q_{60'} = 0.00256(0.85)(0.85)(115\,psf)^2 = 24.46\,psf$

$q_{75'} = 0.00256(0.85)(0.91)(115\,psf)^2 = 26.19\,psf$

$q_{85'} = 0.00256(0.85)(0.95)(115\,psf)^2 = 27.34\,psf$

6) $C_p = 0.8$ for windward walls $\quad$ Figure 27.4-1

$C_p = -0.5$ for leeward walls

$\hookrightarrow L/B = 144'/120' = 1.2 \quad C_p = -.46$ by interpolation.
use -0.5 for all directions

Roof

$q = .00256(0.95)(0.85)(115)^2 = 26.7\,psf$

$p = 26.7[-0.9-0.18] = 28.8\,psf$

net $= 0.7W - 0.6D = 20.16 - 4.8 = 15.36\,psf$

PENNSTATE Engineering

CLASS:
SHEET NO:
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JOB NAME:
SECTION:
OF
DATE:

7) $+GC_{pi}$ Case

@ H = 15' 27.4-1

$P_{windward} = (16.4\ psf)(0.85)(0.8) - (27.3)(0.18) = 6.24\ psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\ psf$

$\Sigma P = 22.76\ psf$

@ H = 30'

$P_{windward} = (20.1\ psf)(0.85)(0.8) - (27.3)(0.18) = 8.75\ psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\ psf$

$\Sigma P = 25.27\ psf$

@ H = 45'

$P_{windward} = (22.7\ psf)(0.85)(0.8) - (27.3)(0.18) = 10.52\ psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\ psf$

$\Sigma P = 27.04\ psf$

@ H = 60'

$P_{windward} = (24.5)(0.85)(0.8) - (27.3)(0.18) = 11.75\ psf$

$P_{leeward} = (27.3)(0.85)(-0.5) - (27.3)(0.18) = -16.52\ psf$

$\Sigma P = 28.27\ psf$

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@ H = 75'

$P_{windward} = (26.2\,psf)(0.85)(0.8) - (27.3)(0.18) = 12.90\,psf$

$P_{leeward} = (27.3\,psf)(0.85)(0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 29.42\,psf$

@ H = 85'

$P_{windward} = (27.3\,psf)(0.85)(0.8) - (27.3)(0.18) = 13.65\,psf$

$P_{leeward} = (27.3\,psf)(0.85)(-0.5) - (27.3)(0.18) = -16.52\,psf$

$\Sigma P = 30.17\,psf$

$-GC_{pi}$ Case

@ H = 15'

$P_{windward} = (16.4\,psf)(0.85)(0.8) + (27.3)(0.18) = 16.07\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69\,psf$

$\Sigma P = 22.75\,psf$

@ H = 30'

$P_{windward} = (20.1\,psf)(0.85)(0.8) + (27.3)(0.18) = 18.58\,psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69\,psf$

$\Sigma P = 25.27\,psf$

PENNSTATE Engineering

CLASS: SECTION:
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@ H=45'

$P_{windward} = (22.7psf)(0.85)(0.8) + (27.3)(0.18) = 20.35 psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 27.04 psf$

@ H=60'

$P_{windward} = (24.5psf)(0.85)(0.8) + (27.3)(0.18) = 21.57 psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 28.26 psf$

@ H=75'

$P_{windward} = (26.2psf)(0.85)(0.8) + (27.3)(0.18) = 22.73 psf$

$P_{leeward} = (27.3)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 29.42 psf$

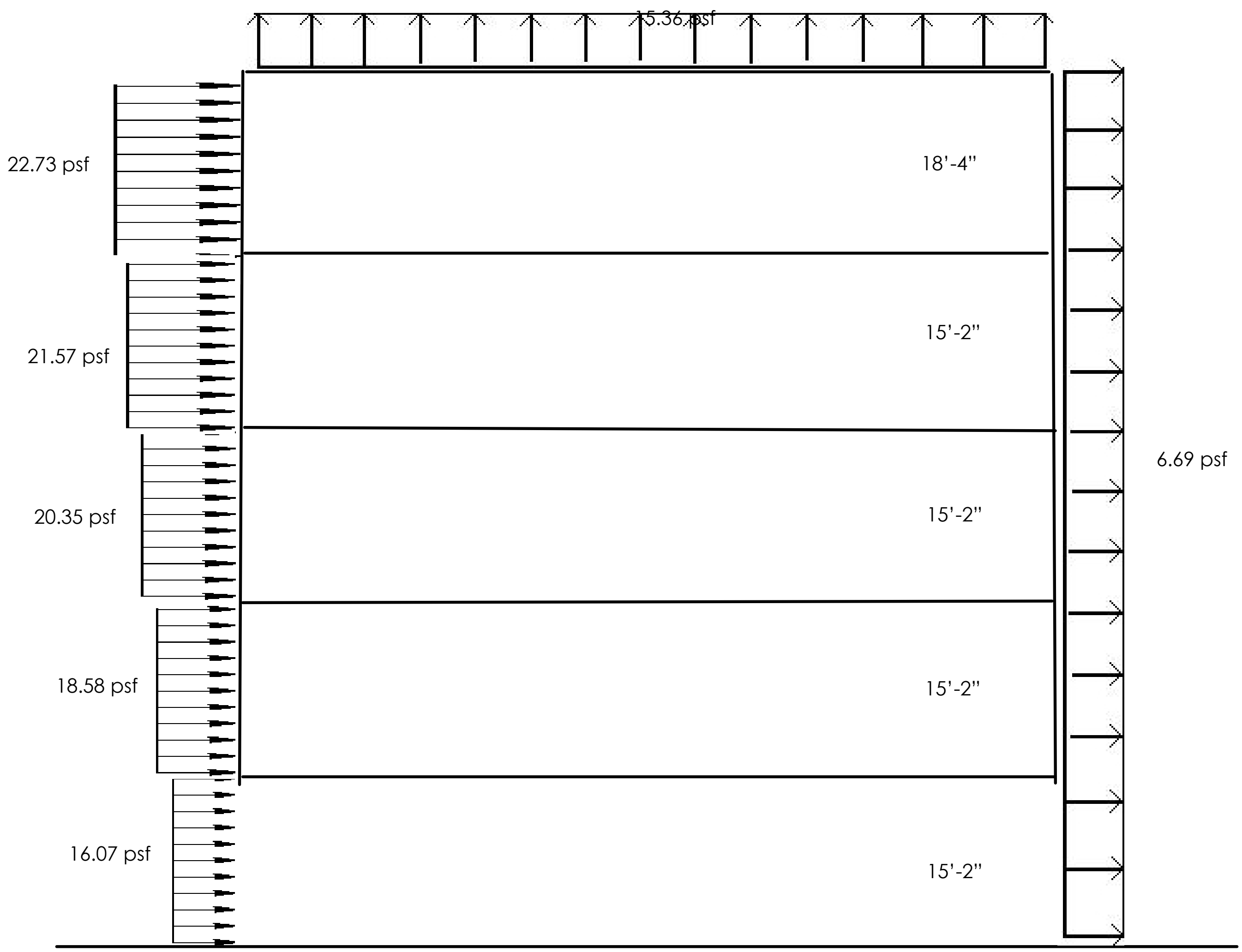
@ H=85'

$P_{windward} = (27.3psf)(0.85)(0.8) + (27.3)(0.18) = 23.48 psf$

$P_{leeward} = (27.3psf)(0.85)(-0.5) + (27.3)(0.18) = -6.69 psf$

$\Sigma P = 30.17 psf$

Wind Loading Diagram



Base Shear N-S direction

V=22.75psf(15.167'*144.167')+25.27 psf (15.167'*144.167')+27.04 psf (15.167'*144.167')+28.26 psf (15.167'*144.167')+29.42 psf (18.33'*144.167')

V=304kips

Base Shear E-W direction

V=22.75psf(15.167'*120.33')+25.27 psf (15.167'*120.33')+27.04 psf (15.167'*120.33')+28.26 psf (15.167'*120.33')+29.42 psf (18.33'*120.33')

V=254kips

Seismic Loads

PENNSTATE Engineering

CLASS: ____________ SECTION: ____________
SHEET NO: ____________ OF ____________
DESIGNED BY: ____________ DATE: ____________
JOB NAME: ____________

Seismic Analysis

Risk Category II Section 11.6

Seismic Importance Factor = 1.0

Seismic Site Class - assume C

$S_s = 0.170$ Figure 22-1

$S_1 = 0.055$ Figure 22-2

$F_a = 1.6$ Table 11.4-1

$F_v = 2.4$ Table 11.4-2

$S_{DS} = F_a S_s (2/3) = 0.181$ 11.4-3

$S_{D1} = F_v S_1 (2/3) = 0.088$ 11.4-4

Seismic Design Cotegory B Tables 11.6-1, 11.6-2

The building has Intermediate Reinforced Masonry Shear walls

$R = 4.0$ Table 12.2-1

$T_a = 0.02(75)^{0.75} = 0.51$ 12.8-7

$C_t = 0.02$ $x = 0.75$ Table 12.8-2

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$$C_s = \frac{0.18'}{\left(\frac{4.0}{1.0}\right)} = 0.045 \qquad 12.8\text{-}2$$

$$C_{s\,max} = \frac{0.088}{0.51'\left(\frac{4.0'}{1.0}\right)} = 0.043 \qquad 12.8\text{-}3 \quad \checkmark$$

$$C_{s\,min} = 0.044(0.181)(1.0) = 0.008 \qquad 12.8\text{-}5 \quad \checkmark$$

$$C_s = 0.045$$

Calculate Seismic Weight w

Roof

$$\text{Dead load} = \frac{20\,psf\,(120.33' \times 144.167')}{1000} = 347^k$$

$$\text{Live load} = \frac{30\,psf\,(120.33' \times 144.167')}{1000\,lb} = 520^k$$

$$\text{total} = 867^k$$

floor

$$\text{Dead load floors 2-5} = \frac{(58\,psf)(120.33')(144.167')(4)}{1000\,lb}$$

$$= 4025^k$$

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Floor

Live load floors 2-5 $= \frac{(80)(120.33)(144.167)(4)}{1000^{lb}}$

$= 5551^k$

total $= 9576^k$

Seismic weight $w = 10{,}443^k$

Base Shear

$V = C_s W$

$V = (0.045)(10{,}443) = 470^k$

Vertical Distribution

$F_x = C_{vx} = \left[\frac{w_x h_x^k}{\sum w_x h_x^k}\right] V$

$T_a = 0.51 \rightarrow$ use $k = 1$

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	w_i	h_x	$w_i h_i$	C_{vx}
floor 2	2394	15'-2	36,310	.085
floor 3	2394	30'-4	72,610	.169
floor 4	2394	45'-6	108,927	.254
floor 5	2394	60'-8"	145,236	.339
Roof	867	75'	65,025	.152
Sum			428 108	

Story forces

floor 2 = 39.6^K

floor 3 = 79.7^K

floor 4 = 119.6^K

floor 5 = 159.4^K

Roof = 71.4^K

Corrections to seismic Analysis

Seismic weight w

Roof = 347^k from tech II

floor Dead load = 55psf

flooring = 2psf
Slab-on-Deck = 35psf
Steel = 10psf
MEP = 8psf

$$= \frac{(55)(120.33)(147.167)(4)}{1000} = 3906^k$$

Exterior wall load

$$\frac{\overset{w}{(10psf)}\overset{h}{(79')}\overset{L}{(529')}}{1000} = 418^k$$

Shear wall loads

$$\frac{\overset{W}{(133psf)}\overset{H}{(79')}\overset{L}{(91')}}{1000} = 956^k$$

Total Seismic weight, w = 5527^k

Vertical Distribution

$$V = C_s W = (0.045)(5527^k) = 248.7^k$$

$$C_{vx} = \left[\frac{w_x h_x^k}{\sum w_x h_x^k}\right] V \qquad K = 1 \text{ from tech II}$$

	W_i	h_x	$w_i h_x$	C_{vx}
Floor 2	1295^k	15'-2"	19,640	0.088
Floor 3	1295^k	30'-4"	39,282	0.18
Floor 4	1295^k	45'-6"	58,923	0.26
Floor 5	1295^k	60'-8"	78,563	0.35
Roof	347^k	75'	26,025'	0.12
Sum			222,433	

Story Forces

Floor 2 = 21.9^k

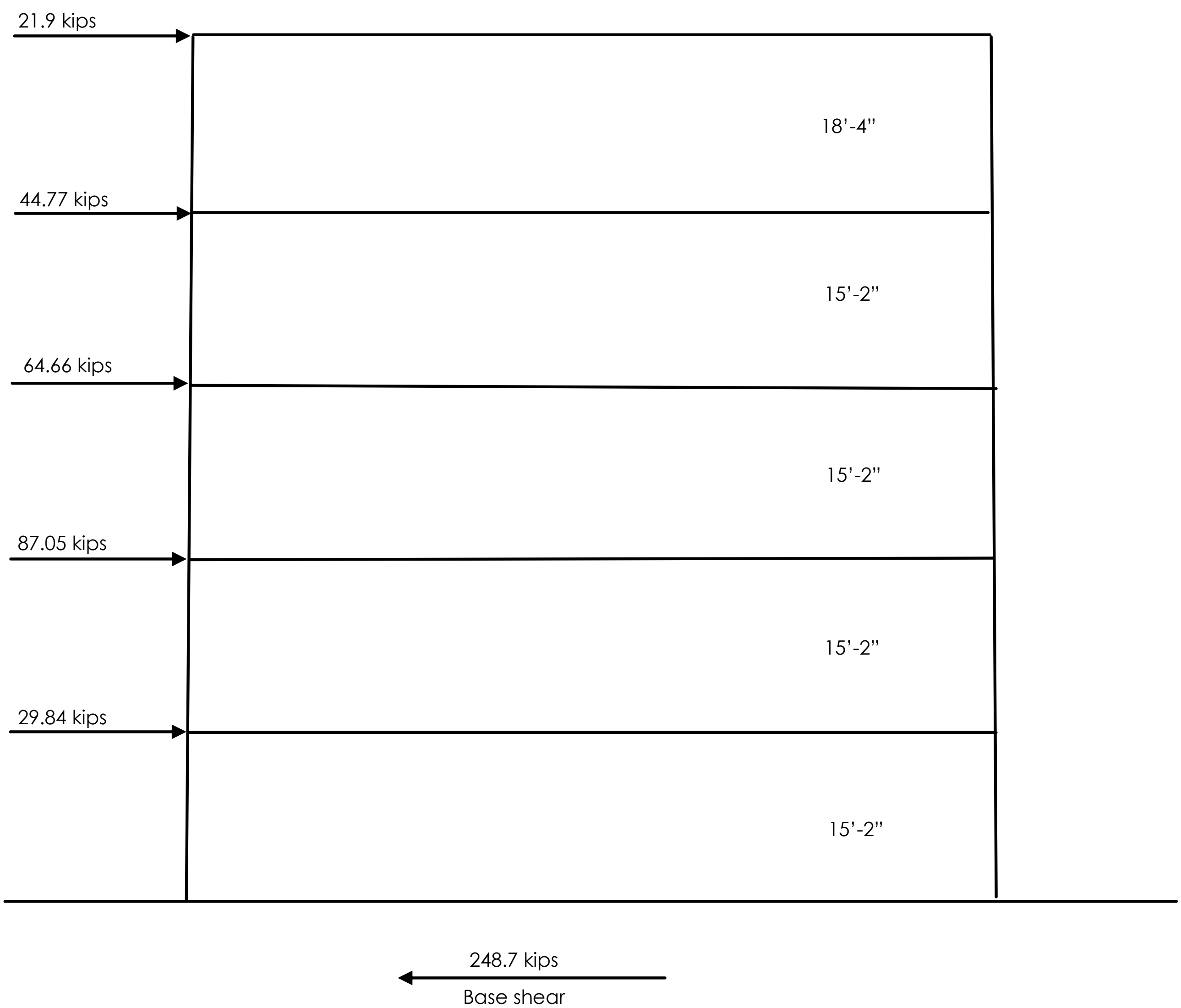
Floor 3 = 44.77^k

Floor 4 = 64.66^k

Floor 5 = 87.05^k

Roof = 29.84^k

Seismic Loading Diagram



Typical Floor Plan