School Without Walls

Washington, D.C.

Tech Report # 3

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Structural Option

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12/2/09

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EXECUTIVE SUMMARY

The third technical report for the School Without Walls project contains an in depth analysis of the lateral system. The lateral system of the school consists of both braced frames and shear walls. To aid in the analysis, ETABS, a structural analysis computer program will be used. Within the model, shear walls were modeled as membranes and the floor system was modeled as a rigid diaphragm. Simplifications and assumptions made concerning the buildings geometry are consistent with those used in Technical Report 1 which allows the use of wind and seismic loads previously calculated.

Because of expansion joints, the School Without Walls in fact acts as three separate buildings with three different lateral systems. The four story addition utilizes a cross braced frame and eight shear walls which rise the extent of the building. The shear walls form two cores surrounding the stairwell and the elevator shaft. Because of the presence of these cores the shear walls can work together in order to resist lateral movement. Wind displacements were compared to the allowable drift of H/400 and seismic story drifts were compared to $.020h_{sx}$ and were found to be acceptable in both cases.

Each lateral resisting element was analyzed separate in order to determine the different stiffness's using ETABS. These were checked via hand calculations to ensure computer accuracy. The center of mass and center of rigidity were calculated using the ETABS program and was also check through hand calculations. It became apparent that this building is subject to torsional forces because of the eccentricity between the centers of mass and rigidities.

INTRODUCTION

The Grant School has stood in the heart of the George Washington University campus since 1882 and has housed the School Without Walls since 1977. The "School Without Walls" name comes from the faculties encouragement for students to use Washington D.C. as an active classroom, thus not restraining learning to the walls of the school.

The original 32,300 square foot, three story school was in dire need of modernization and expansion due to the increasing number of students and outdated mechanical and electrical equipment. The 68,000 square foot addition and renovation blends the 19th century school with a modern design. This is achieved by combining existing brick patterns with glass, steel and curtain walls. The School Without Walls project is expected to receive LEED Gold Certification.

The existing three story school is made up of four large classrooms per floor, one at each corner of the square building. The new addition of the school provides an additional two large classrooms on each floor, an open atrium space, a large student commons, roof terrace area and a library. The basement was also reengineered and redesigned to serve as scientific laboratories for the school.

This technical assignment investigates and analyzes the existing lateral resisting system for the School Without Walls. Computer programs including STAAD.pro and ETABS were utilized to assist in the analysis of the structure. The outputs from these programs were checked by hand calculation to ensure accuracy and reduce errors.

EXISTING STRUCTURAL SYSTEM

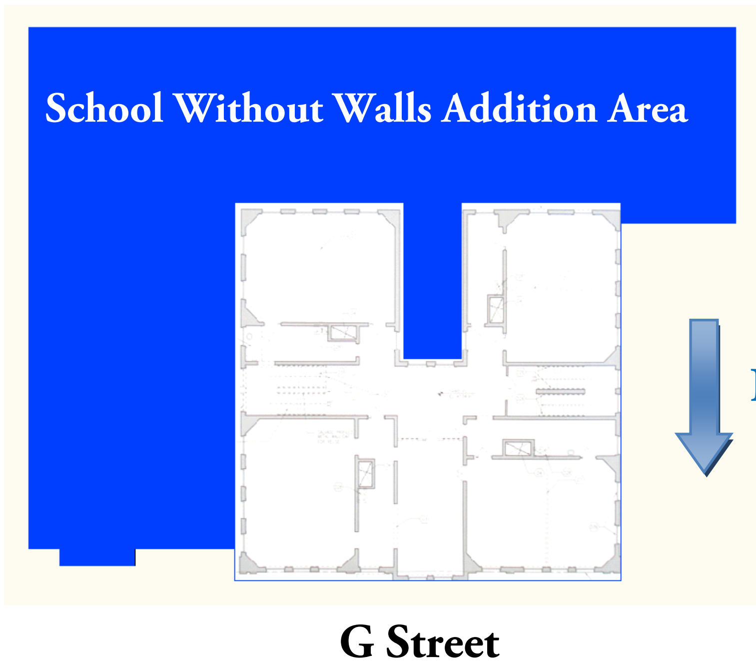


Figure 1: Floor Plan Showing Expansion

The 68,000 square foot addition to the School Without Walls project is located in blue in Figure 1. Due to expansion joints located at the interface of the addition and the existing building, the structural systems work independently. A detail of this expansion joint can be viewed in Figure 2. As stated in the drawing, along the expansion joint along the east side of the existing building is 4", and is 2" along the south side.

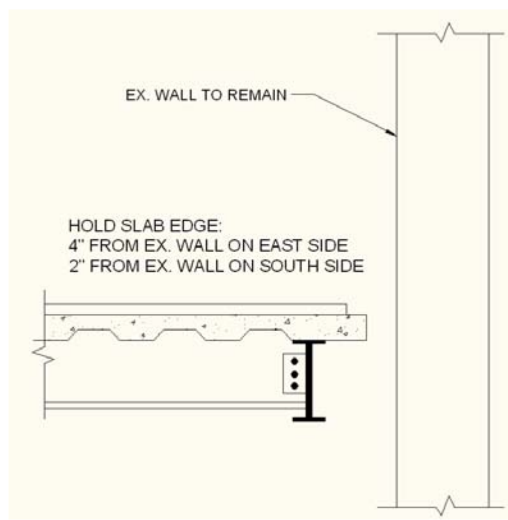


Figure 2: Expansion Joint

The new addition to the School Without Wall itself is divided by an expansion joint. This expansion therefore creates a total of three independently acting structural systems. This division of the new addition can be viewed in Figure 3. These areas will be referred to as "Area 1" and "Area 2" throughout this report, as located on the Figures 3 and 4.

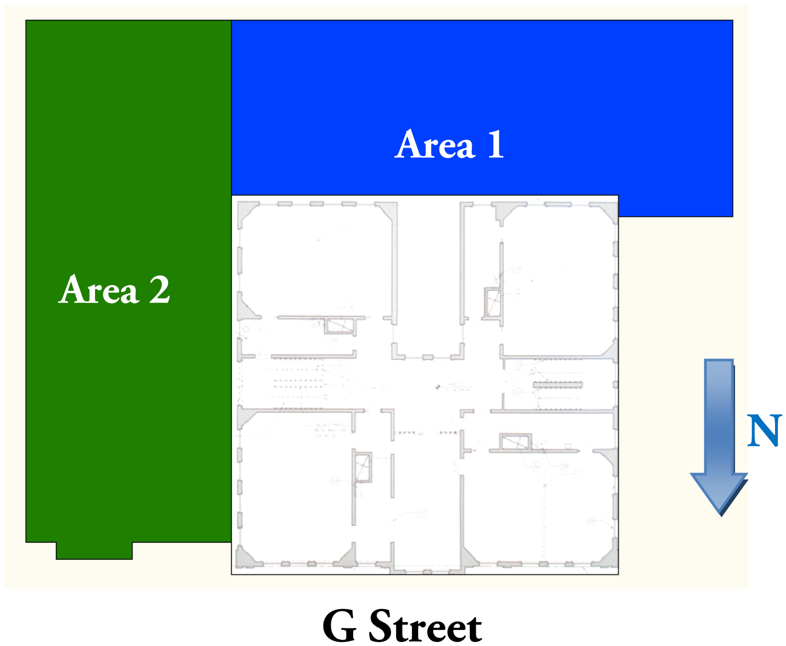


Figure 3: Floor Plan Showing Building Separation

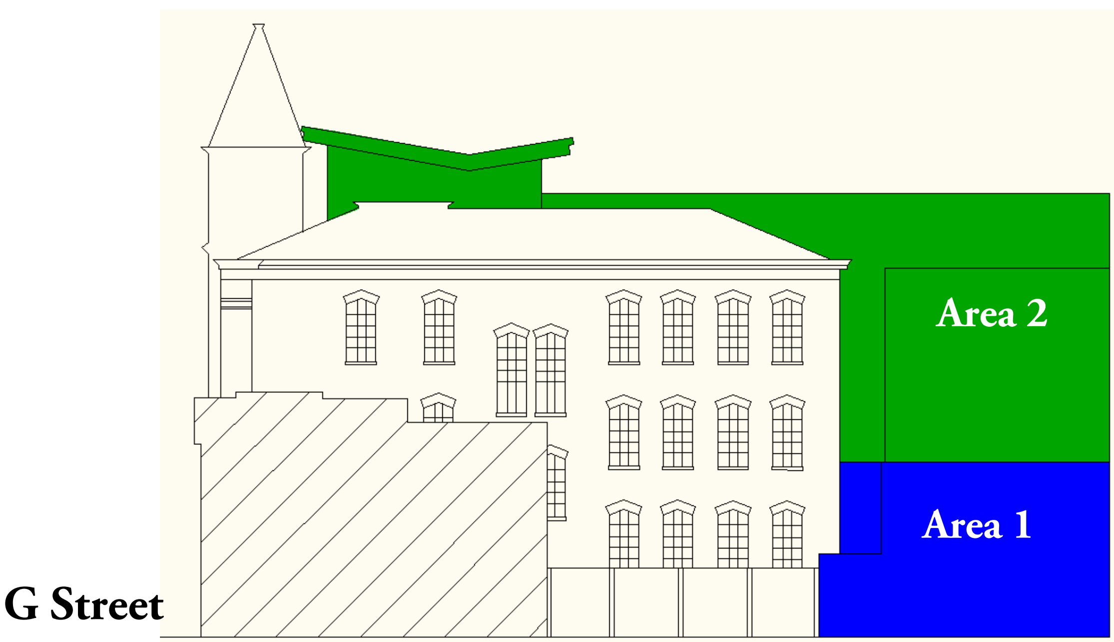


Figure 4: West Elevation

Foundation

The geotechnical engineering study was performed by Thomas L. Brown Associates, P.C. on January 28, 2007. After performing a series of in-situ tests, considering the lab test results, anticipated loads, and settlement analyses, a shallow foundation consisting of reinforced cast-in-place spread footings and grade beams was deemed appropriate. Based on the testing and analysis, the footings should be designed for an allowable bearing capacity of 3.0 ksf. The addition utilizes typical footings which are 2' 6" wide by 2'0" deep and rest on compacted earth 3'0" below the top of the slab-on-grade. Grade beams are also used in the foundation of the new addition. The beams measure 30"x30" along the east side and 30"x24" along the south side of the building.

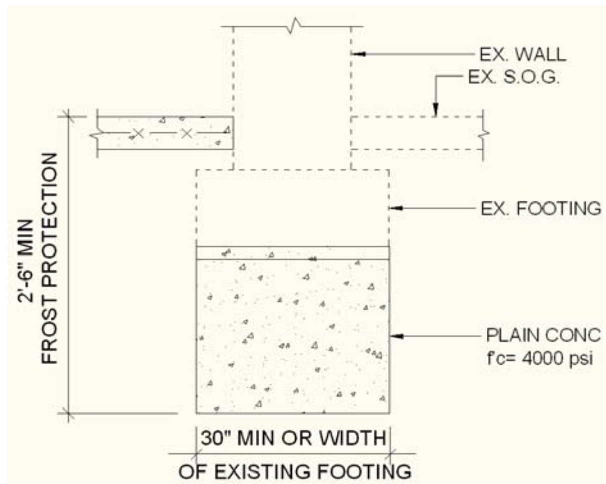


Figure 5: Underpinning Detail

Due to the increased load and the disruption of earth, underpinning the existing footings of the school became necessary. An underpinning detail is located in Figure 5. The underpinning sequence will be performed in sections no larger than 4 feet wide, approximately spaced 12-15 feet apart.

Floor System

The floor system of School Without Walls is a composite steel system. The floor slab of the new addition is 3 ¼” LWC topping over a 2” 20 GA LOK composite steel floor decking, bringing the total floor slab to 5 ¼” thick. Along the top flange of the beam, ¾”x4” long headed shear studs are used for composite action. A section of this floor system is shown above in Figure 7.

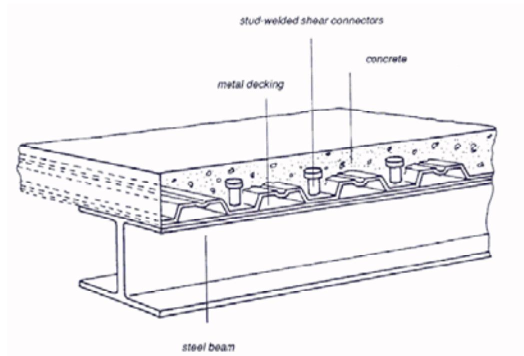


Figure 6: Typical Composite Steel Construction (www.epitech.com)

The columns which run along the perimeter of the existing building are set back from the structure, creating a cantilever. Moment connections are utilized at these columns in order to carry the load which is being cantilevered. A typical bay showing the cantilevered slab and moment connections is located below in Figure 7.

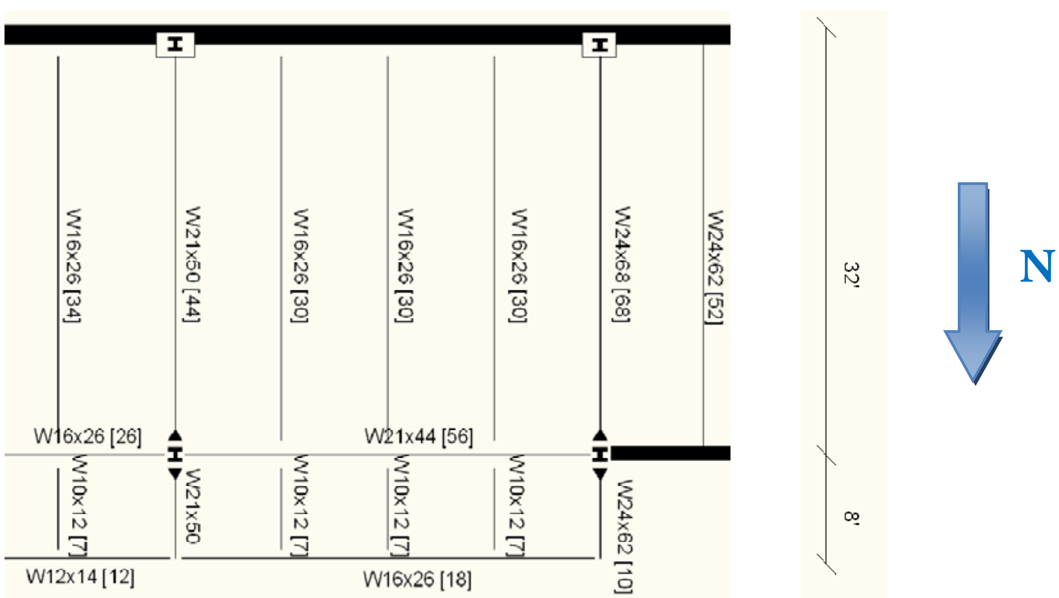


Figure 7: Typical Bay Showing Moment Connections and Cantilever

Lateral Loads

Wind Loads

A separate wind load analysis was conducted for Area1 and Area 2 due to the expansion joint separating them.

Area 1 has a total height h= 22.45', therefore, it is considered a low rise building. Method 1 as listed in Chapter 6 of ASCE 7-05 was used to carry out the wind analysis of Area 1.

Horizontal Pressures (psf)				Vertical Pressures (psf)			
A	B	C	D	E	F	G	H
12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8
Adjusted Pressures (psf)				Adjusted Pressures (psf)			
14.7	-7.7	9.8	-4.6	17.71	-10.1	-12.3	-7.8

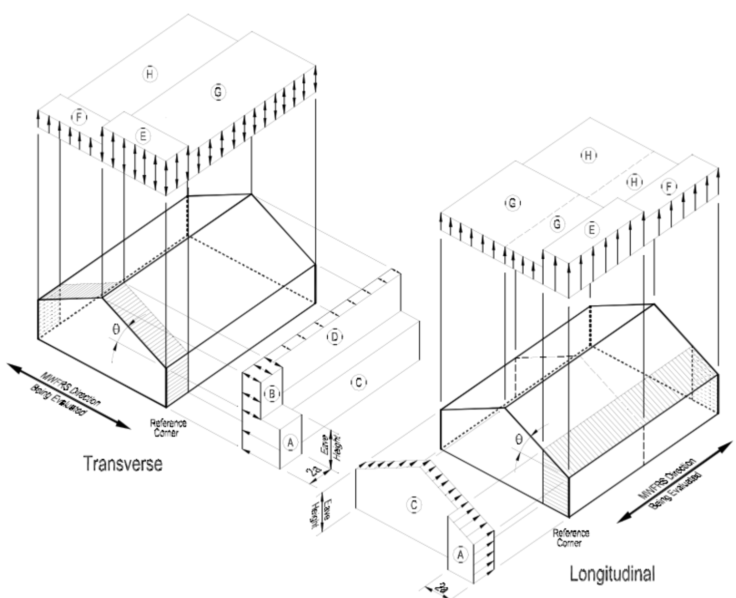


Figure 8: Wind Pressures for Area 1

Because the mean height of Area 2 is greater than 60’, wind loads for this portion of the building were analyzed using Method 2 in ASCE 7-05. It was assumed that the fourth floor covers the entire footprint of Area 2. The complex roof structure of the library was also ignored in the analysis due to its relatively small area and small influence it would have on the overall calculations. Details of these analyses and calculations can be found in Appendix D of this report.

Classification	Category
V, Basic Wind Speed (Fig. 6-1)	90 mph
K_d (Table 6-4)	0.85
I (Table 6-1)	1.15
Occupancy Category (Table 1-1)	III
Exposure Category	B
K_{zt} (Topographic Factor)	1

	Level	Actual Height(ft)	Estimate Height (ft)	k_z	q_z	Wind Pressures (psf)	
						N-S	E-W
Windward	T.O. Roof	63.61	64	0.87	17.63	11.99	11.67
	4	50.95	51	0.81	16.42	11.16	10.86
	3	35.7	36	0.74	15.00	10.20	9.92
	2	20.45	21	0.63	12.77	8.68	8.45
	1	5.25	6	0.57	11.55	7.86	7.64
Leeward	All	All	All	0.87	17.63	-3.90	-7.29

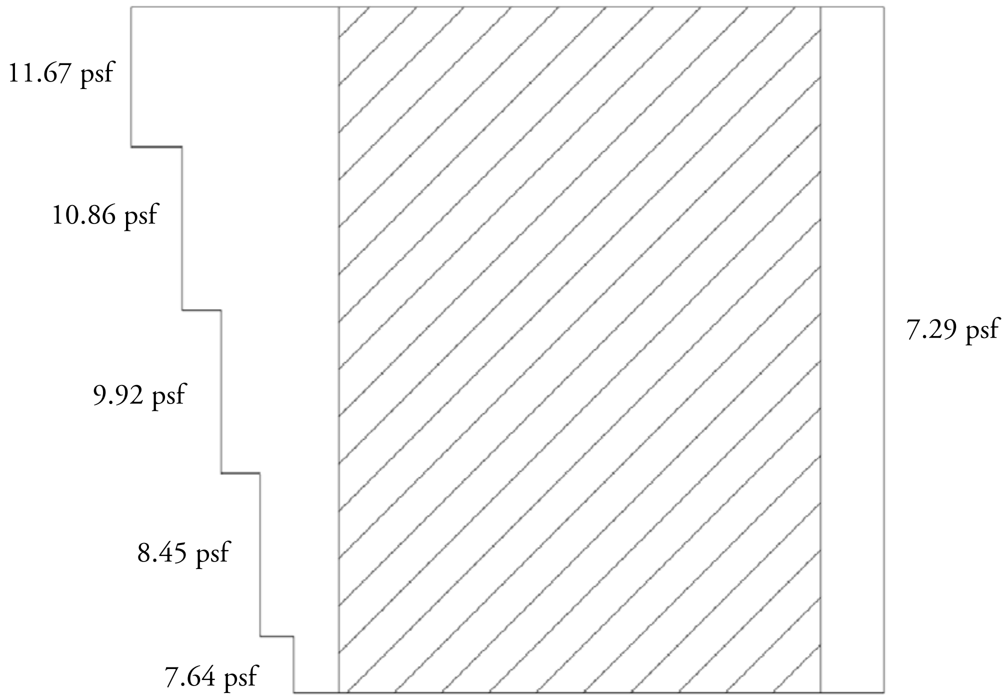


Figure 9: East-West Wind Pressure Diagram

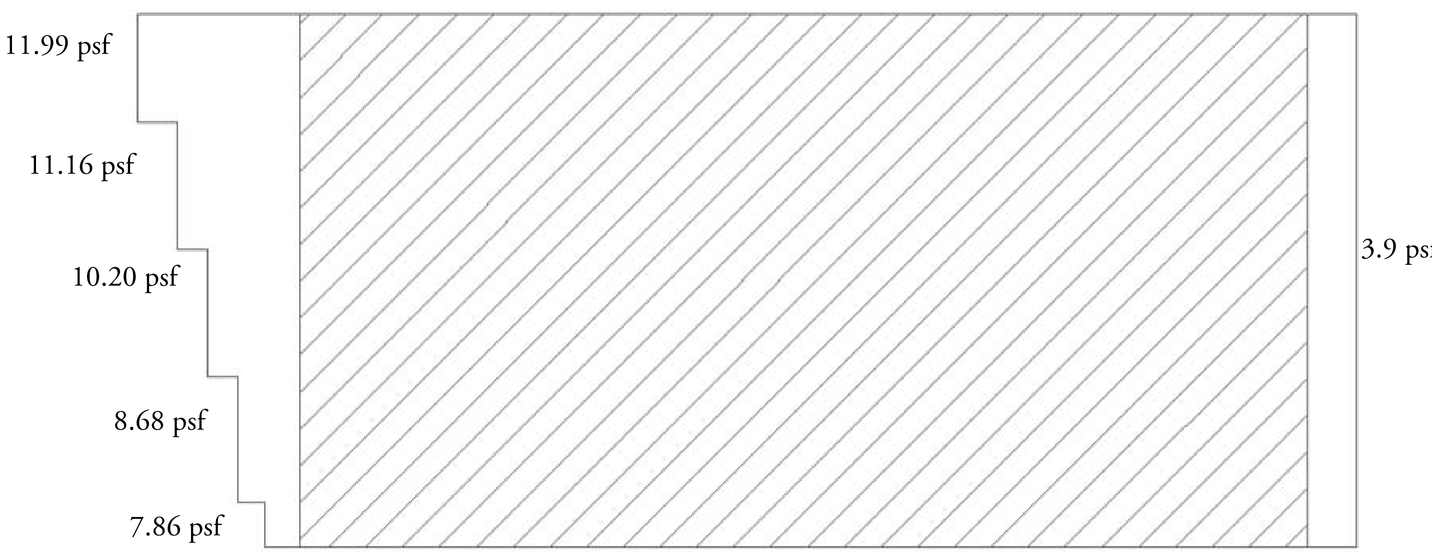


Figure 10: North-South Wind Pressure Diagram

Wind Forces									
				Load (kip)		Shear (kip)		Moment	
Level	Trib Height (ft)	Total Load N-S (psf)	Total Load E-W (psf)	N-S	E-W	N-S	E-W	N-S	E-W
T.O. Roof	6.33	15.89	18.96	4.62	15.47	0	0	294.35	984.66
4	14	15.07	18.15	9.70	32.78	4.62	15.47	494.31	1670.29
3	15.25	14.10	17.21	9.89	33.86	14.32	48.26	353.12	1208.92
2	15.25	12.58	15.74	8.82	30.96	24.22	82.12	180.52	633.16
1	10.25	11.76	14.93	5.54	19.74	33.04	113.08	29.10	103.66
						38.59	132.83	1351.42	4600.71

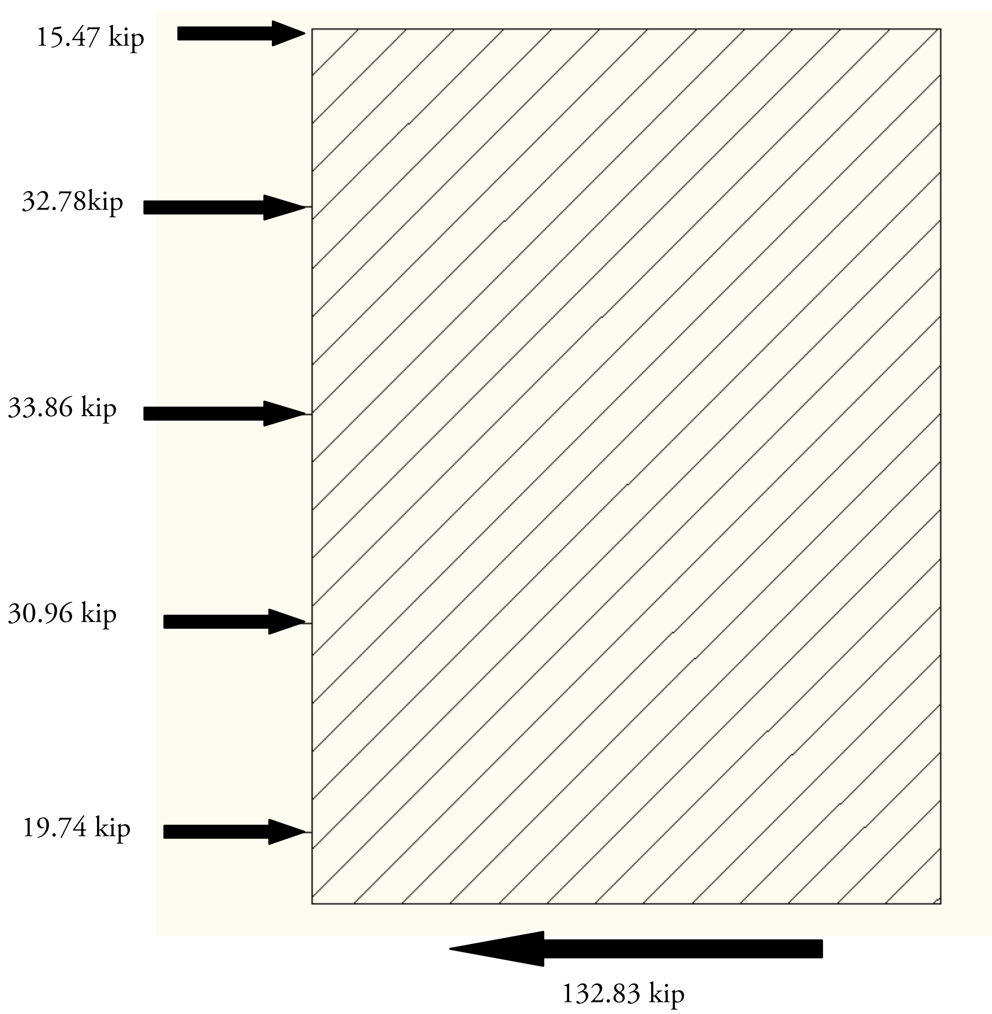


Figure 11: East-West Wind Force Diagram

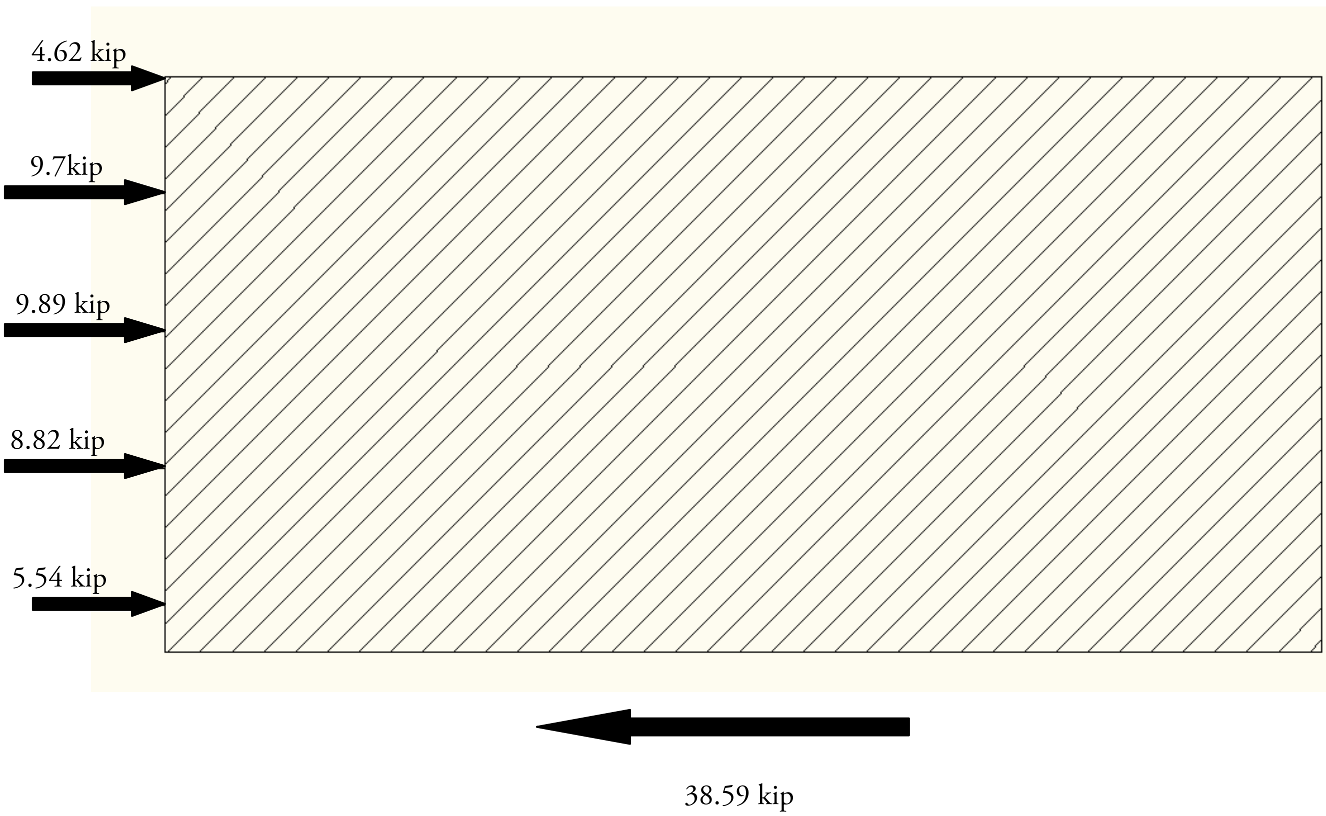


Figure 12: North-South Wind Force Diagram

As seen from the force diagrams located above, the wind forces that blow in the East-West direction create the largest loads on the building due to the fact that they are applied to a much larger area than the North-South winds.

Seismic Loads

The seismic loads for this tech report were calculated using Chapters 11 and 12 of ASCE 7-05. This seismic analysis includes dead loads from beams, slabs, columns, walls and M/E/P equipment. These calculations can be viewed in Appendix C of this report. All assumptions and calculations for the seismic analysis can also be found in Appendix C.

The seismic forces for the School Without Walls project are less than the lateral loads created by wind due to the fact that the building is located in an area with low seismic activity.

Floor	w_x (kip)	h_x	k	$w_x h_x^k$	$\sum w_i h_i^k$	F_x (kip)	Story Shear V_x (kip)	Moment (k-ft)
Roof	159.70	63.61	1.33	39996.05	224059.6	7.29	--	463.84
4	504.21	50.95	1.33	93997.37	224059.6	17.13	7.29	873.14
3	501.05	35.7	1.33	58201.43	224059.6	10.61	24.42	378.81
2	494.94	20.45	1.33	27402.01	224059.6	4.99	35.04	102.16
1	491.80	5.25	1.33	4462.74	224059.6	0.81	40.03	4.27
Total	2151.72	63.61	1.33	224059.62	224059.6	40.85	40.85	1822.24

PRELIMINARY LATERAL INVESTIGATION

The lateral system of School Without Walls works as three different systems due to expansion joints as stated before and show in Figures 3 and 4. Both braced frames and shear walls, located in blue and green respectively in Figure 13, are used to resist lateral loads that are applied to the building.

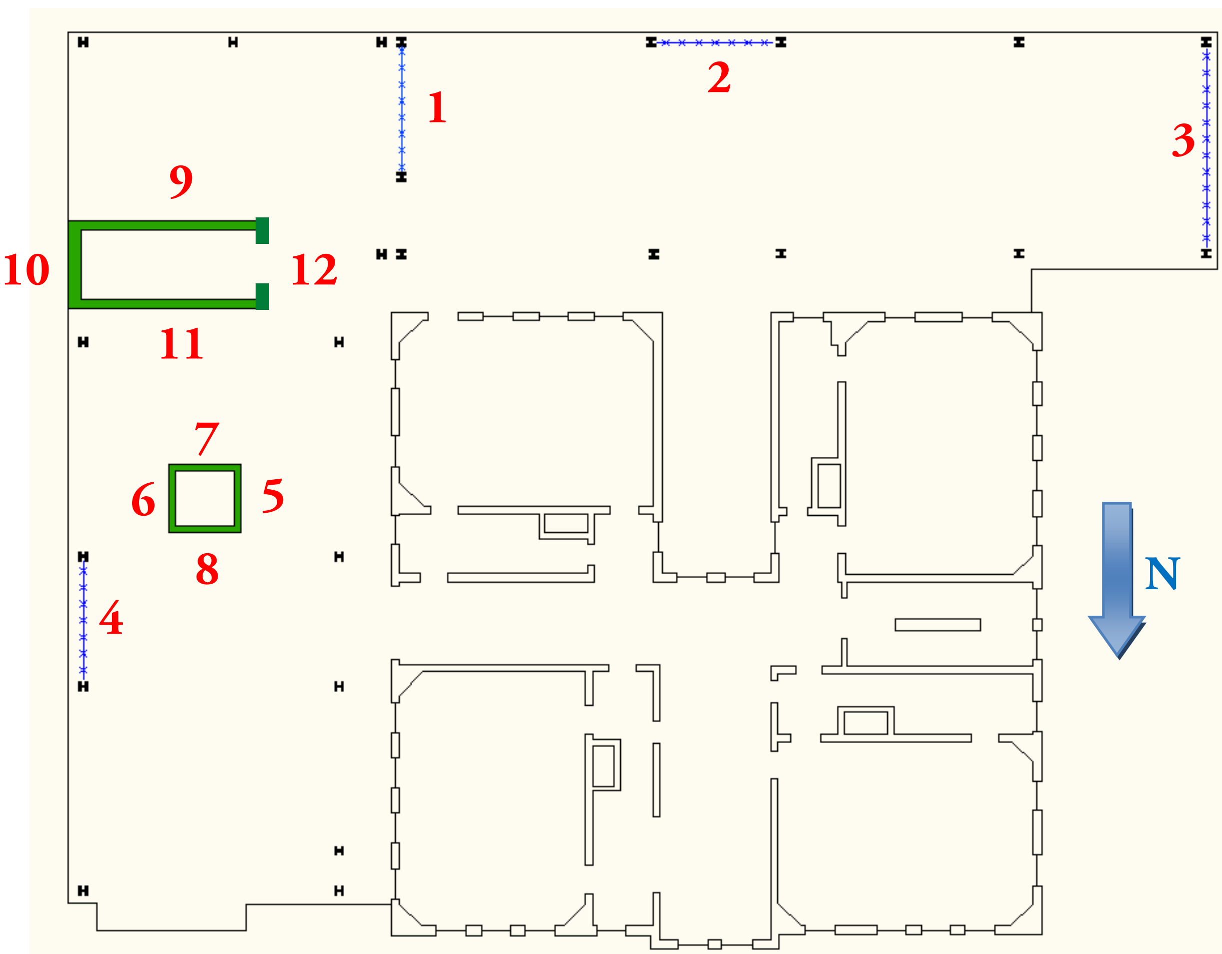


Figure 13: Summary of Lateral Systems

For this technical report, the lateral system for the two story addition (Area 1) is discussed and was modeled in ETABS to determine the center of mass and rigidity. The ETABS output was verified by hand calculations and the use of STAAD.Pro to determine the frame rigidities.

The lateral system for the four story addition (Area 2) was also modeled in the ETABS structural program. The shear walls were modeled as membrane elements meshed at a maximum size of 24”x24”. Each floor was modeled as a rigid diaphragm. The centers of mass and rigidity were verified via hand calculations to ensure accuracy of the model. The lateral load cases one and two and three, located in Figure 14 were incorporated into and investigated in the ETABS model.

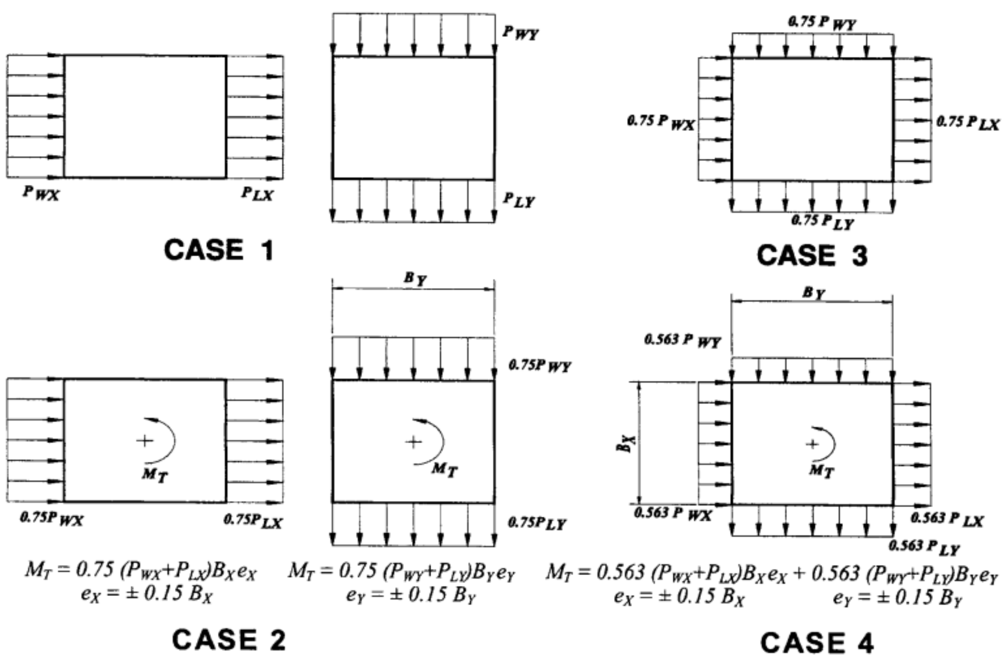


Figure 14: Load Cases Investigated

The following load combinations taken from ASCE 7-05 were considered and incorporated into the ETABS model:

1) 1.4D
2) 1.2D + 1.6L + 0.5L_r
3) 1.2D + 1.6L_r + (1.0L or 0.8W)
4) 1.2D + 1.6W + 1.0L + 0.5L_r
5) 1.2D + 1.0E + 1.0L
6) 0.9D + 1.6W
7) 0.9D + 1.0E

Area 1 Lateral System

The two story structure supporting the outside roof terrace (Area 1) utilizes only braced frames for lateral support. All of the braced frames located in this section of building are comprised of only HSS6x6x3/8 sections. Diagonal, cross, and chevron bracing are utilized in braced frames 1, 2 and 3 respectively as labeled in Figures 15 and 16. All of the braced frames extend the entirety of the two story section of building.

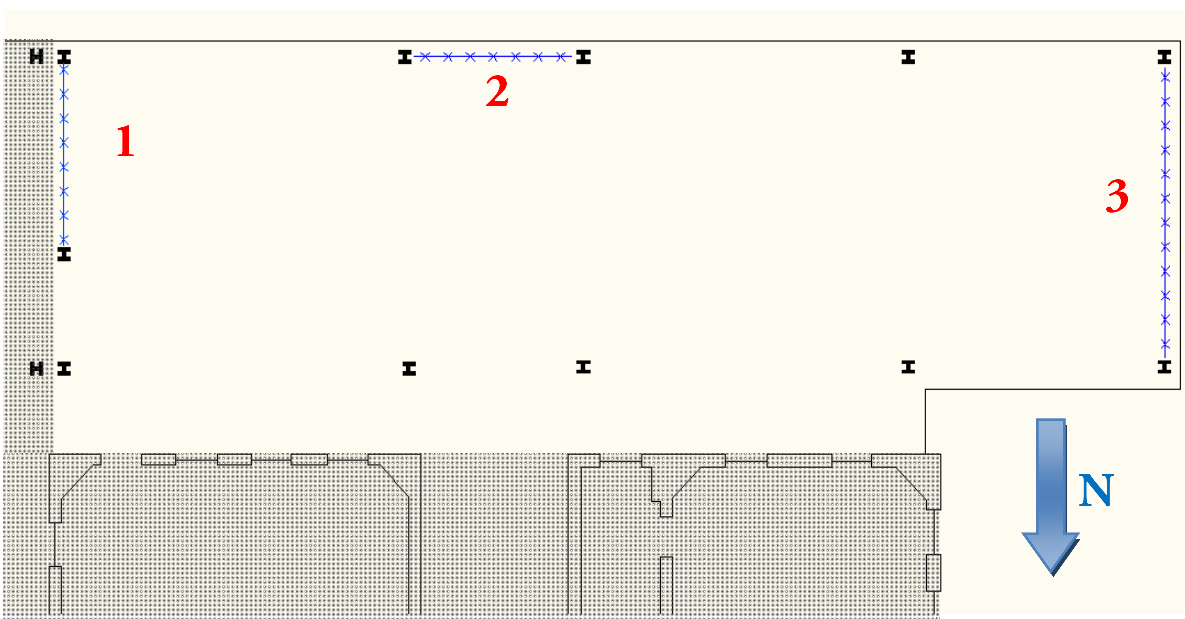


Figure 15: Plan View of Lateral Elements in Area 1

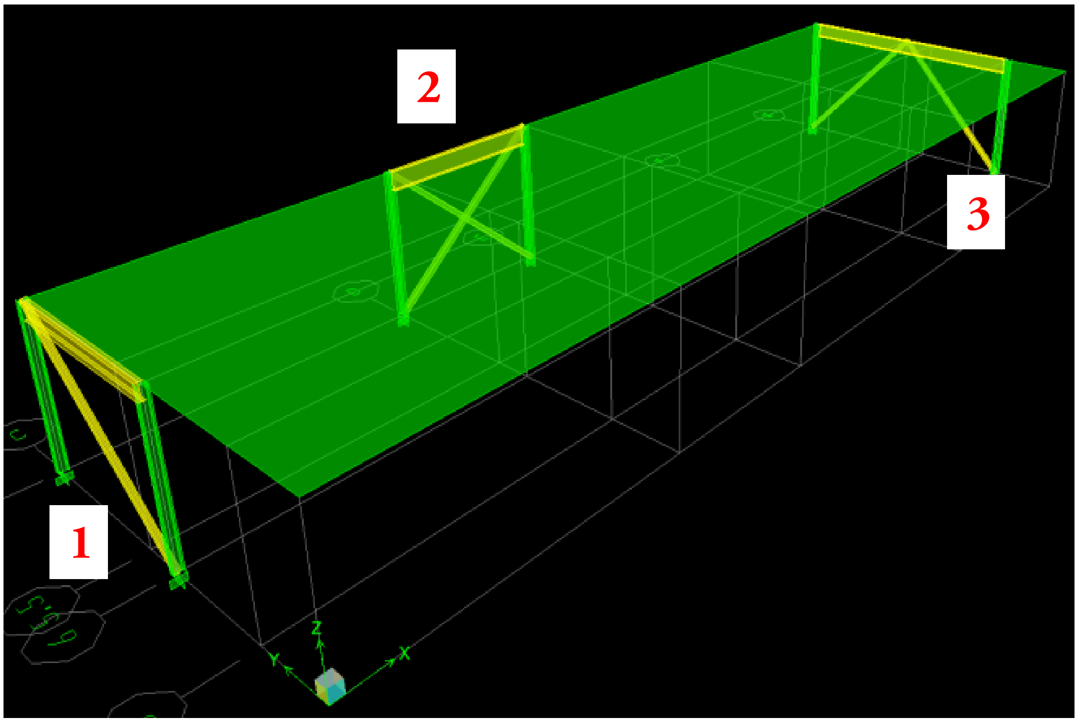


Figure 16: ETABS Model of Area 1

Relative Stiffness

To determine the relative stiffness of the lateral system, each frame was modeled and analyzed in STAAD.Pro. A 1kip load was placed at the highest node and the resulting deflection was calculated. To determine the stiffness of the braced framed elements, the following equation was used:

$$Ki = Pi / \Delta i$$

Where: Ki = Stiffness of lateral element i
Pi = Applied load
ΔI = Deflection of lateral element i

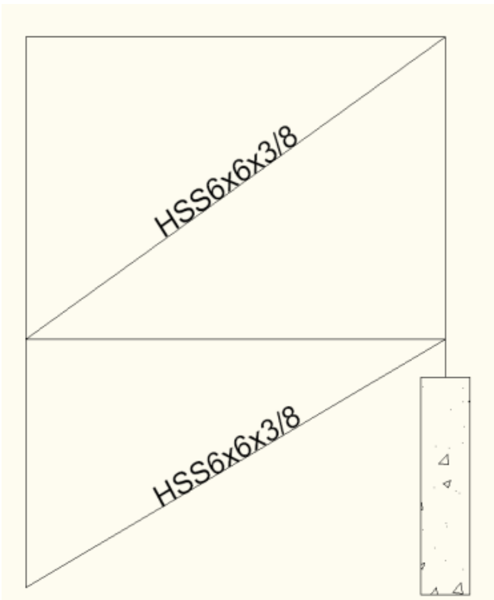


Figure 17: Braced Frame 1

Braced Frame 1 Analysis (Figure 17)

This is an example of a diagonal braced frame. A 1 kip load was applied to the top of the frame. After applying the load, the frame deflects .003 inches in the x direction. The wall stiffness was determined to be 333.33 k/in

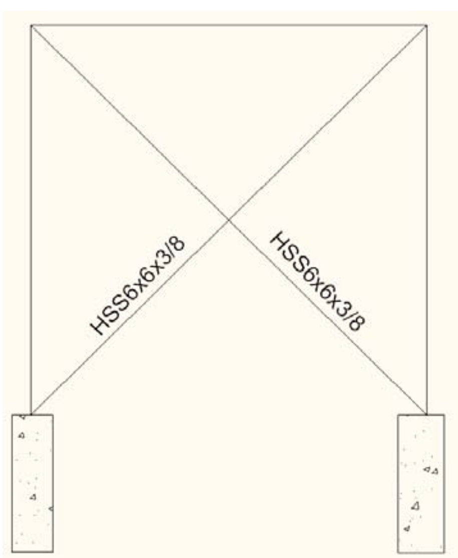


Figure 18: Braced Frame 2

Braced Frame 2 Analysis (Figure 18)

This is an example of a cross or X braced frame. A 1 kip load was applied to the top of the frame. After applying the load, the frame deflects .0012 inches in the x direction. The wall stiffness was determined to be 833.33 k/in

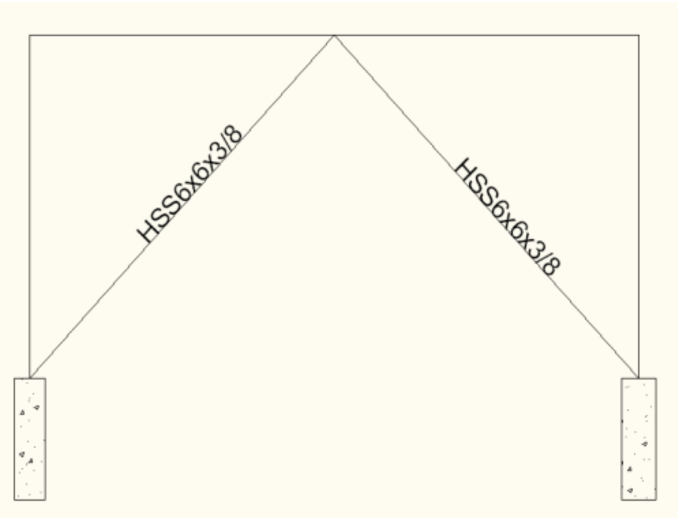


Figure 19: Braced Frame 3

Braced Frame 3 Analysis (Figure 19)

This is an example of a chevron braced frame. A 1 kip load was applied to the top of the frame. After applying the load, the frame deflects .00124 inches in the x direction. The wall stiffness was determined to be 806.45 k/in

Center of Mass and Rigidity

From the ETABS model, it was determined that the center of mass is located at coordinates X_m = 57.5' and Y_m = 19.25'. The center of rigidity is located at X_r =78' and Y_r = 38.5'. These outputs were confirmed by hand calculations and are located in Appendix B.

Area 2 Lateral System

The four story structure supporting the library, referred to as Area 2 in this technical report, uses a combination of a braced frame system and a shear wall system to resist lateral loads. The braced frame, comprised of HSS square sections reaches from the ground to the roof level. The shear walls are located around both the elevator core and the stair core. 12" concrete shear walls encompass the stair tower, and 8" shear walls surround the elevator core.

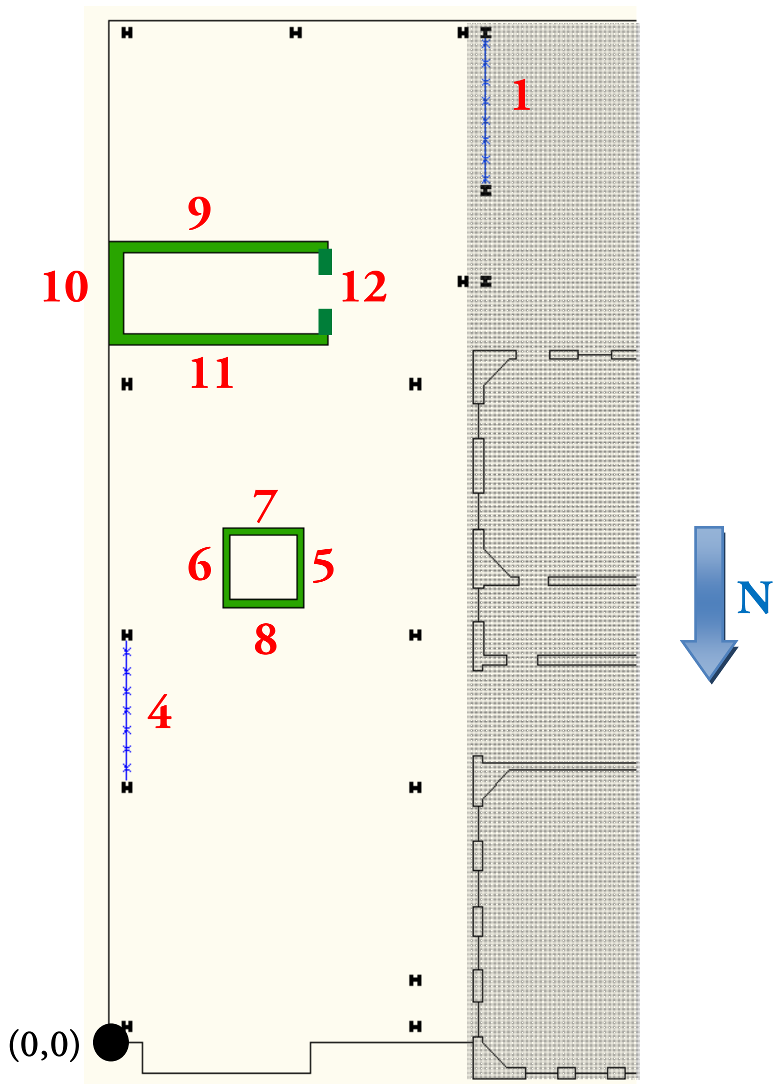


Figure 20: Area 2 Lateral Resisting System

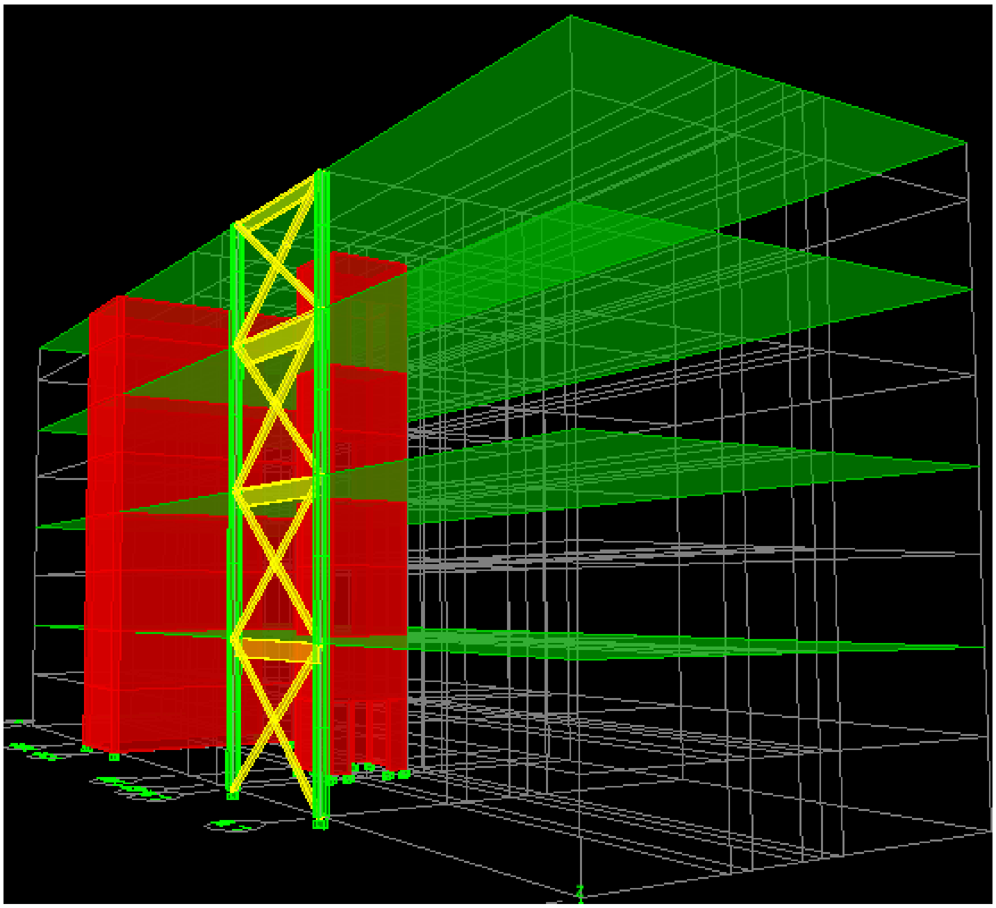


Figure 21: ETABS Model

Relative Stiffness

To determine the relative stiffness of the lateral system, the braced frame was modeled and analyzed in STAAD.Pro. The shear walls were modeled and analyzed individually in ETABS. A 1kip load was placed at the highest node and the resulting deflection was calculated. To determine the stiffness of the braced framed elements, the following equation was used:

$$Ki = Pi / \Delta i$$

Where: Ki = Stiffness of lateral element i
Pi = Applied load
ΔI = Deflection of lateral element i

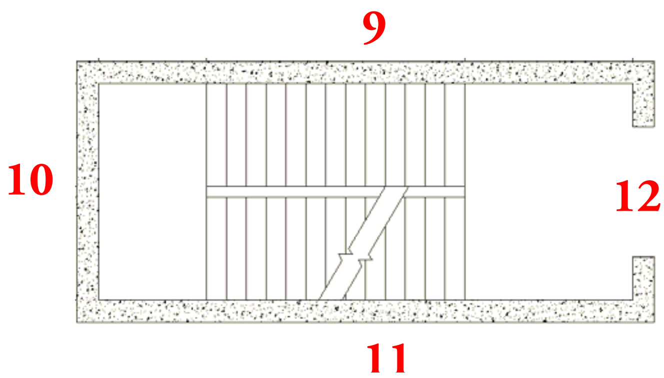


Figure 22: Stairwell Shear Walls

Stairwell Shear Walls

The shear walls surrounding the stair well are all 12" thick and are reinforced with #5 vertical bars spaced at 10" on center and #4 horizontal bars spaced at 12" on center at each face.

	Height (ft)	Thickness (ft)	I (ft^4)	E_c (k/ft^2)	Δ_p (in)	k (k/in)
Wall 9	64	1	1580.3	519119.5	.0014	714.28
Wall 10	64	1	144	519119.5	.01436	69.64
Wall 11	64	1	1580.3	519119.5	.0014	714.28

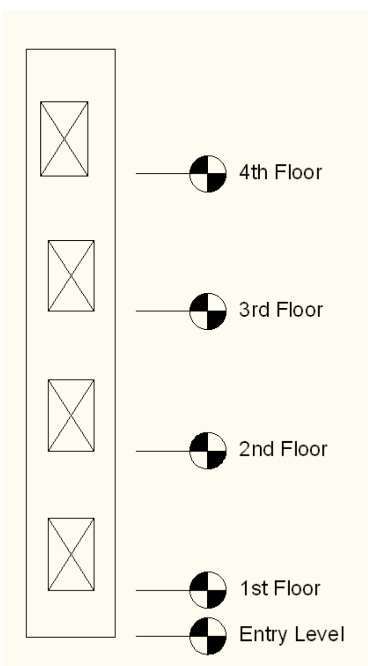


Figure 23: Braced Frame 12 Elevation

Shear Wall 12

This shear wall was modeled and analyzed in ETABS due to the openings the stair tower doors create. After applying a 1 kip load to the highest node it was found that the resulting deflection was .0548 inches. This defelection yields a wall stiffness of 18.24k/in.

Elevator Core Shear Walls

The shear walls surrounding the elevator core are all 8" thick and are reinforced with #5 vertical bars spaced at 10" on center and #4 horizontal bars spaced at 12" on center at each face. The elevator calls for openings in both shear walls 7 and 8. Figures 24 to 26 show the plan of the elevator shaft at each floor. Each shear wall will be modeled separately in order to obtain a more precise wall stiffness value. The elevations of shear walls 7 and 8 can be observed in Figures 27 and 28 respectively.

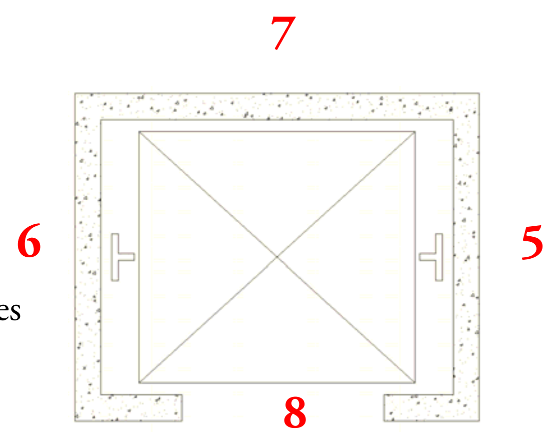


Figure 24: 4th Floor Elevator Plan

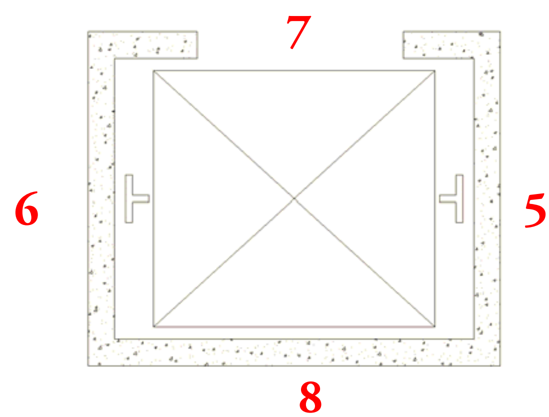


Figure 25: 2nd and 3rd Floor Elevator Plan

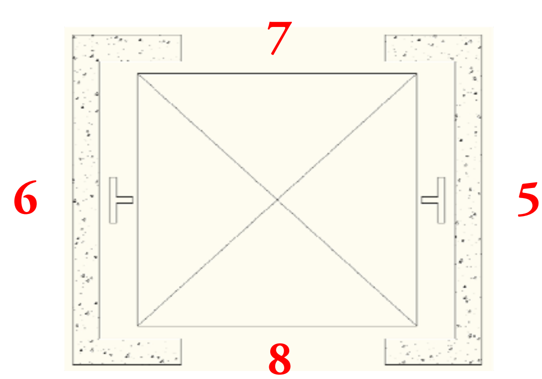


Figure 26: 1st Floor Elevator Plan

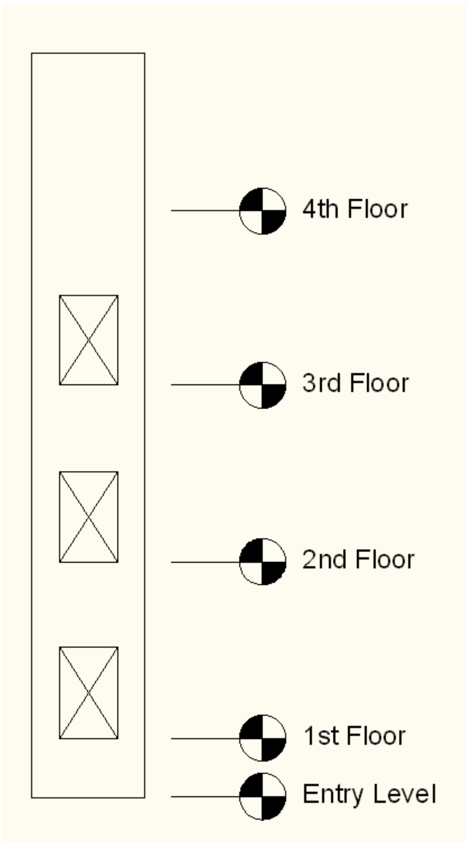


Figure 27: Shear Wall 7 Elevation

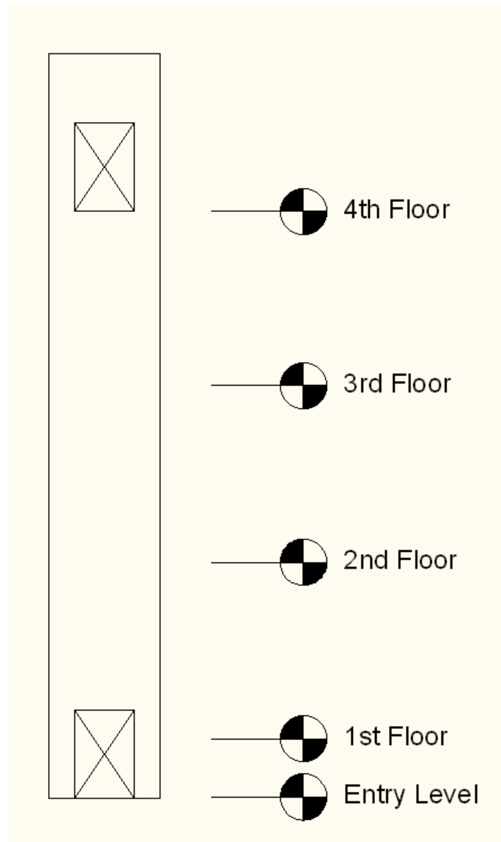


Figure 28: Shear Wall 8 Elevation

	Height (ft)	Thickness (ft)	E_c (k/ft^2)	Δ_p (in)	k (k/in)
Wall 5	64	.667	519119.5	.0421	23.75
Wall 6	64	.667	519119.5	.0421	23.75
Wall 7	64	.667	519119.5	.0508	19.68
Wall 8	64	.667	519119.5	.0467	21.41

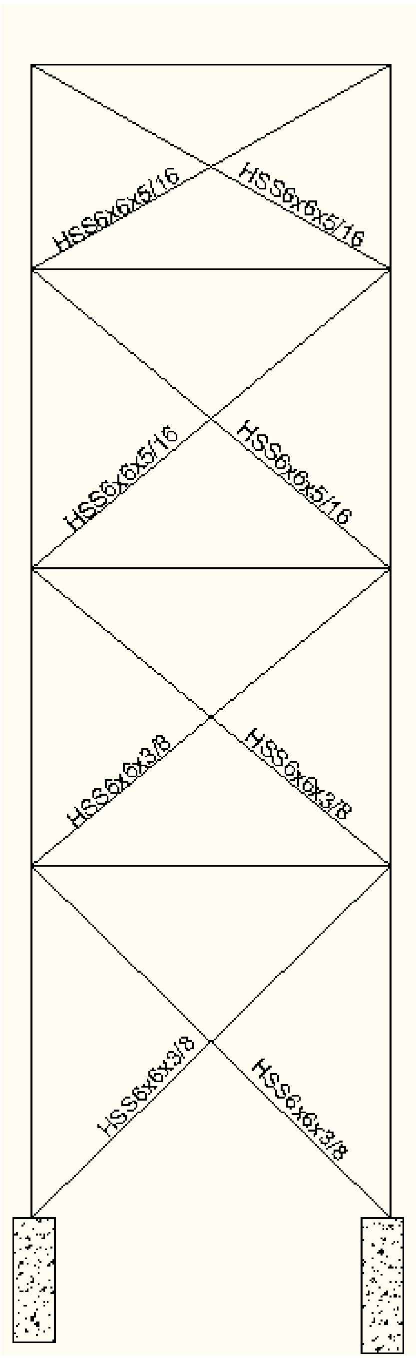


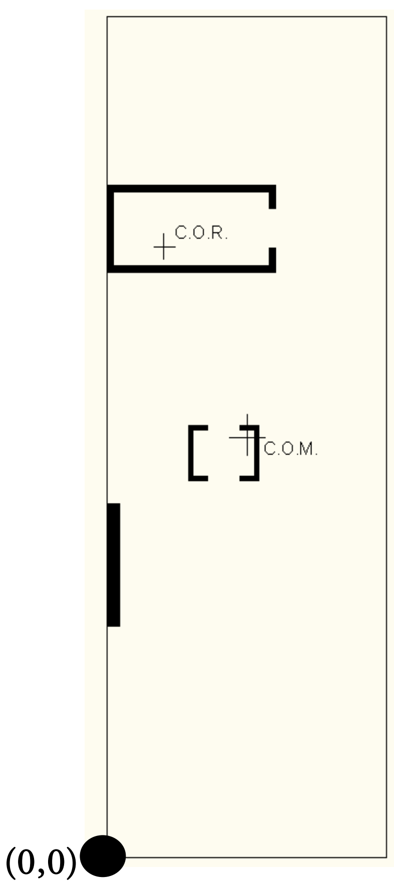
Figure 29: Braced Frame 4

Braced Frame 4 Analysis

Area 2 utilizes a cross braced frame which runs in the north-south direction along the east exterior wall to resist lateral loads. This fame is comprised of HSS6x6 sections for the entirety of its reach. The braced frame was modeled in STAAD.Pro and a 1 kip load was applied to its highest node. The output of this analysis provided a deflection of .011 inches due to this applied load. This deflection yields a frame stiffness of 90.9 k/in.

Center of Mass and Rigidity

From the ETABS model, it was determined that the center of mass of floor level 2 is located at coordinates X_m = 22.17' and Y_m = 63.38'. The center of rigidity is located at X_r =8.95' and Y_r = 92.23'. These outputs were confirmed by hand calculations and are located in Appendix B.



	ETABS OUTPUT			
	X_{cr} (ft)	Y_{cr} (ft)	X_{cm} (ft)	Y_{cm} (ft)
Roof	5.26	92.59	22.17	63.38
Level 4	5.62	92.19	22.17	63.38
Level 3	6.76	92.01	22.17	63.38
Level 2	8.95	92.23	22.17	63.38

Figure 30: Location of Centers of Mass and Rigidity

Torsion

Torsion is present in buildings when the center of mass and the center of rigidity are located at different points. This eccentricity between the two values therefore creates a moment.

To view the effects of torsion, torsional rigidity was first calculated using the equation for story three,

$$J=\sum R_i(d_i^2)$$

Where: R= wall rigidities
d= distance from center of rigidity to wall or frame

	Location		Center of Rigidity		Distance From C.R. to Element			
Lateral Element	X	Y	X	Y	X	Y	Rigidity	Torsional Rigidity
4.00	0.00	--	6.76	92.00	6.76	--	90.90	4153.91
5.00	23.31	--	6.76	92.00	16.55	--	23.75	6507.15
6.00	13.65	--	6.76	92.00	6.89	--	23.75	1127.46
7.00	--	65.14	6.76	92.00	--	26.86	19.68	14198.32
8.00	--	57.45	6.76	92.00	--	34.55	21.41	25557.17
9.00	--	100.30	6.76	92.00	--	8.30	714.28	49206.75
10.00	0.00	--	6.76	92.00	6.76	--	69.64	3182.38
11.00	--	89.30	6.76	92.00	--	2.70	714.28	5207.10
12.00	25.67	--	6.76	92.00	18.91	--	18.24	6522.41
								115662.66

The torsional rigidity for story three was determined to be 115,663 (k/in)ft^2. This value can then be applied to find the torsional shear in the different lateral elements.

Torsional shear was found in each of the lateral elements in Area 2 on story 3 using the equation:

$$V_i^t = V(e)(d_i)(R_i)/J$$

Where: J= torsional rigidity
d= distance from lateral element to the center of rigidity
R= wall rigidity
e= eccentricity

Case 1

	Distance From C.R. to Element			Eccentricity		Story Shear			V_i^t (k)
Lateral Element	**X**	**Y**	**Torsional Rigidity**	**X**	**Y**	**X**	**Y**	**Rigidity**	
4	6.76		115662.65	15.41			9.89	90.90	0.80
5	16.5525		115662.65	15.41			9.89	23.75	0.51
6	6.89		115662.65	15.41			9.89	23.75	0.21
7		26.86	115662.65		28.63	33.86		19.68	4.43
8		34.55	115662.65		28.63	33.86		21.41	6.19
9		8.3	115662.65		28.63	33.86		714.28	49.68
10	6.76		115662.65	15.41			9.89	69.64	0.62
11		2.7	115662.65		28.63	33.86		714.28	16.16
12	18.91		115662.65	15.41			9.89	18.24	0.45

Case 2

	Distance From C.R. to Element			Eccentricity		Story Shear			V_i^t (k)
Lateral Element	**X**	**Y**	**Torsional Rigidity**	**X**	**Y**	**X**	**Y**	**Rigidity**	
4	6.76		115662.66	22.05			7.41	90.90	0.87
5	16.55		115662.66	22.05			7.41	23.75	0.56
6	6.89		115662.66	22.05			7.41	23.75	0.23
7		26.86	115662.66		47.64	25.40		19.68	5.53
8		34.55	115662.66		47.64	25.40		21.41	7.74
9		8.30	115662.66		47.64	25.40		714.28	62.02
10	6.76		115662.66	22.05			7.41	69.64	0.67
11		2.70	115662.66		47.64	25.40		714.28	20.18
12	18.91		115662.66	22.05			7.41	18.24	0.49

Drift

When structurally designing a building, drift should be limited as much as possible. Wind and seismic drifts were computed using the ETABS model and were compared to the limitations located in ASCE 7-05. For wind criteria, it was assumed that the drift limit should not exceed 1/400[th] of the overall building height. This warrants a total building drift of 1.92". It was assumed that the drift due to seismic loads were .020h_{sx}, as seen in Figure 31. It is important to realize that the basement level will not be taken into account for drift calculation because it is restrained by lateral earth pressure.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
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	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 31: Allowable seismic drift

After applying the appropriate load cases to the ETABS model, story drifts were calculated. It appears that the deflections and drifts which occur were much smaller than anticipated. After further review of this system it becomes apparent that the shear walls surrounding the stair well and the elevator shaft are in fact working together to resist movement. Because of this shear wall design, hand calculation checks cannot be performed because it is beyond the scope of this technical report. Due to this, the ETAB outputs will be relied on to convey story drift and can be located in Appendix E.

Overturning Analysis

Overturning moment must be taken into account when designing the foundation to the school without walls system. This building is fairly rectangular and narrow thus creating critical overturning moments in the east-west direction. Due to the presence of grade beams in this direction it is apparent that the structural designer has taken into account the overturning moments present. Along with the grade beams, the mat footing bearing the shear walls are adequately sized and heavily reinforced to eliminate any overturning possibilities. Both mats supporting the shear walls are 2' thick with #7 rebar 12" O.C. at both the top and bottom and running in both directions.

Conclusion

This report has described, detailed and analyzed the lateral force resisting systems in both Area 1 and Area 2. After conducting calculations with the aid of ETABS, it is clear to see that the School Without Walls has a considerable amount of torsional force to the large difference between the center of mass and rigidity. To ensure accuracy of this model, hand calculations and checks were performed.

Because of the presence of the stairwell and elevator cores, the building does not warrant large story drifts which satisfies the drift criteria as listed in ASCE 7-05. Using ETABS, the openings located in these shear wall cores could be modeled in order to obtain a more accurate analysis of the lateral system.

Overturning appears to be a concern because of the large concrete mats and the grade beams present. A more detailed and in depth analysis will be necessary for this issue to determine the exact capacity of the foundation.

In future reports and proposals it would be wise to further investigate the large torsional force created and solutions to eliminate it. When investigating alternate systems, one must be cautious of the effect it can have on the foundation and thus the overturning moment.

Appendix A

Live Loads

Load Description	Load
Administrative Office	50 psf+20psf
Classrooms	40 psf+20psf
Corridors Above First Floor	80 psf
First Floor Corridors	100 psf
Student Commons	100 psf
Storage	125psf
Stack Room	150 psf
Roof Load	30 psf + add'l snow drift
Mechanical Room	150 psf
Roof Terrace	100 psf

Dead Loads

Load Description	Load
Metal Decking 20 Gage	3 psf
Normal Weight Concrete	150 pcf
Light Weight Concrete	110 pcf
Finishes	5 psf
M/E/P	10 psf

Snow Loads

Load Description	**Design Load and Factors**
Ground Snow Load	Pg= 25 psf
Snow Exposure Factor	Ce= 0.9
Snow Importance Factor	I= 1.1
Thermal Factor	Ct= 1.0
Flat Roof Snow Load	Pf= 17.3 psf

MATERIALS

Structural Steel:

Wide Flanges……………………………ASTM A-572 or A-992, Grade 50
Channels, Angles, Plates………………………………………… ASTM A-36
Hollow Structural Sections (HSS)………………………ASTM A-500, Grade B
Pipes……………………………………………ASTM A-53, Type E or S, Grade B

Metal Decking:

2" Composite Metal Deck………………………………………… ……..20 Gage

Bolts:

High Strength Steel Bolts……………………………ASTM A-325 or ASTM A-490
Anchor Bolts…………………………………………………ASTM F-1554, Grade 36

Concrete:

Over Composite Metal Deck…………………………………………f'c = 4,000 psi
Grout for CMU walls………………………………………………… f'c = 3,000 psi
All Concrete Components U.O.N…………………………………f'c = 4,000 psi

Reinforcing Steel:

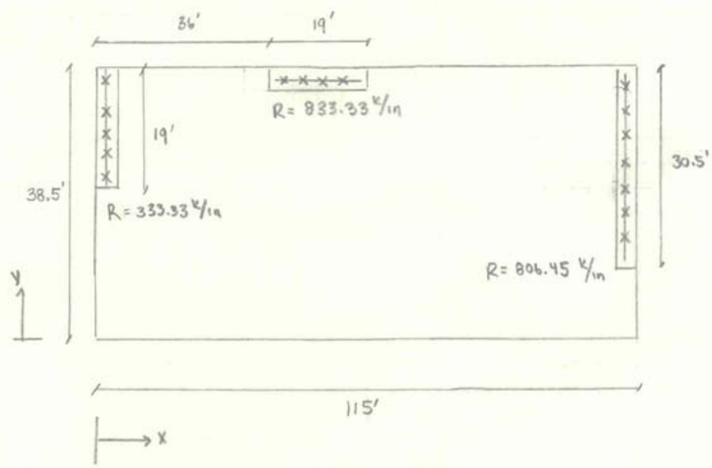
Reinforcing Bars……………………………………………ASTM A-615, Grade 60
Welded Reinforcing…………………………………………ASTM A-706, Grade 60

Wood:

All Wood U.O.N………………………………………… No. 2 Hem-Fir (North)

Appendix B

CENTER OF RIGIDITY – HAND CALCULATION



$$X_R = \frac{\Sigma x_i R_i}{\Sigma R_i} = \frac{806.45(115)}{806.45 + 333.33} = 81.5'$$

$$Y_R = \frac{\Sigma y_j R_j}{\Sigma R_j} = \frac{833.33(38.5)}{833.33} = 38.5'$$

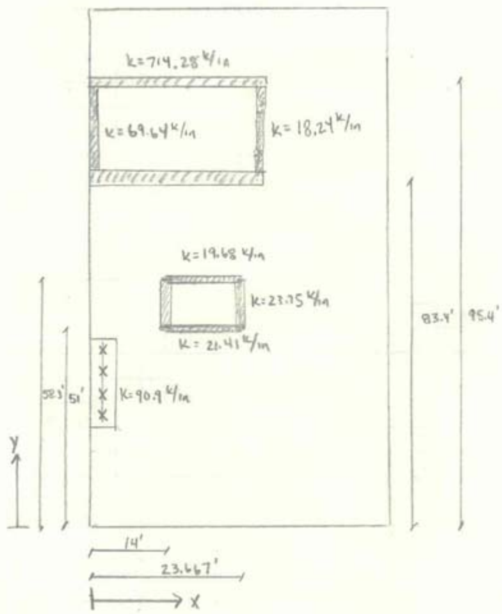
COMPARED TO ETABS OUTPUT OF

$X_R = 78'$ $Y_R = 38.4'$

$X_{CM} = 57.5'$ $Y_{CM} = 19.25'$

CENTER OF RIGIDITY

AREA 2

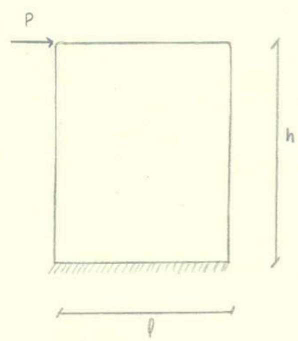


$$X_R = \frac{(23.75)(14) + (23.75)(23.667) + 25(18.25)}{69.64 + 90.9 + 23.75 + 23.75 + 18.25} = 8.27'$$

$$X_y = \frac{(21.41)(51) + (19.68)(58.3) + (714.28)(83.375) + (714.28)(95.375)}{21.4 + 19.68 + 714.28 + 714.28}$$

$$X_y = 88.4'$$

CONCRETE SHEAR WALL



$$\Delta_p = \frac{Ph^3}{3EI} + \frac{2.78Ph}{AE}$$

h = height of shear wall
t = thickness of shear wall
$A = l(t)$
$I = t(l^3)/12$

FOR STAIR TOWER SHEAR WALL RUNNING IN E-W DIRECTION

$h = 64'$
$t = 12'' = 1'$
$l = 26'\text{-}8'' = 26.667'$
$I = (1)(26.667)^3/12 = 1580.3\ ft^4$

ASSUME $f'_c = 4000psi$

$E_c = 57000\sqrt{f'_c} = 3604996.533\ psi \rightarrow 3600\ ksi$

$E_c = 519119.5\ k/ft^2$

$$\Delta_p = \frac{(1)(64^3)}{3(519119.5)(15803)} + \frac{2.78(1)(64)}{(26.667)(519119.5)}$$

$\Delta_p = 1.065E\text{-}4 + 1.285E\text{-}5 = 1.1936E\text{-}4\ ft$

$\Delta_p = .0014\ in$

FOR SOLID STAIR TOWER WALL RUNNING IN N-S DIRECTION

$h = 64'$
$t = 1'$
$l = 12'$
$I = (1)(12^3)/12 = 144\ ft^4$
$E_c = 519119.5\ k/ft^2$

$$\Delta_p = \frac{(1)(64^3)}{3(519119.5)(144)} + \frac{2.78(1)(64)}{12(519119.5)}$$

$\Delta_p = .01436$

FOR SOLID ELEVATOR TOWER WALL RUNNING IN N-S DIRECTION

$h = 64'$
$t = 8'' = .667'$
$l = 8'-4.25'' = 8.354'$
$I = .667(8.354^3)/12 = 32.4 ft^2$
$E_c = 519119.5\ k/ft^2$

$$\Delta_p = \frac{(1)(64^3)}{3(519119.5)(32.4)} + \frac{2.78(1)(64)}{5.572(519119.5)}$$

$\Delta_p = .0631 in$

Appendix C

SEISMIC CALCULATIONS (ASCE 7-05)

OCCUPANY CATEGORY III (TABLE 1-1)
IMPORTANCE FACTOR (I) 1.25 (TABLE 11.5-1)

USGS (http://earthquake.usgs.gov/research/hazmaps/design/)

SPECTRAL RESPONSE ACCEL. AT SHORT PERIOD (S_S): .154 g

SPECTRAL RESPONSE ACCEL. AT 1-SEC PERIOD (S_1): .05 g

SITE CLASS : C (VERY DENSE SOIL + SOFT ROCK)

SITE COEFFICIENTS

F_a (TABLE 11.4-1) : 1.2
F_v (TABLE 11.4-2) : 1.7

$S_{MS} = F_a S_S = 1.2(.154) = .185$ (EQN 11.4-1)

$S_{M1} = F_v S_1 = 1.7(.05) = .085$ (EQN 11.4-2)

$S_{DS} = 2S_{MS}/3 = 2(.185)/3 = .123$ (EQN 11.4-3)

$S_{D1} = 2S_{M1}/3 = 2(.085)/3 = .057$ (EQN 11.4-4)

DESIGN CATEGORY

$S_{DS} = .123$ $S_{DS} < .167$ $\therefore$ CATEGORY A

$S_{D1} = .057$ $S_{D1} < .067$ $\therefore$ CATEGORY A

$T_S = S_{D1}/S_{DS} = .463$

$T_a = C_t h_n^x$ (EQN 12.8-7) $C_t = .03$ (TABLE 12.8-2)
$x = .75$

$T_a = .03(63.6)^{.75} = .676$

$T = C_u T_a$ (SEC 12.8.2)

$C_u = 1.7$ (TABLE 12.8-1)

$T = 1.7(.676) = 1.15$

$T_L = 8$ (FIG 22-15)

$C_s = \dfrac{SD_1}{T\left(\frac{R}{I}\right)}$

$R = 3.25$ (TABLE 12.2-1)

ASSUME ORDINARY CONCENTRICALLY BRACED FRAME / ORDINARY REINFORCED CONCRETE SHEAR WALL

$C_s = \dfrac{.057}{(1.15)\left(\frac{3.25}{1.25}\right)}$

$\Omega = 2$ $\quad C_d = 3.25$

$C_s = .019 \therefore$ OK

$V = C_s W$

$W = 2021.487^k$

$V = .019(2021.5) = 38.4^k$

$k = .75 + .5T = .75 + .5(1.15) = 1.325$

Appendix D

WIND DESIGN

ASCE 7-05 METHOD 2

BASIC WIND SPEED $V = 90$ mph (FIGURE 6-1)

IMPORTANCE FACTOR $I = 1.15$ (TABLE 6-1)

OCCUPANCY CATEGORY III (TABLE 1-1)

EXPOSURE CATEGORY B (SECT 6.5.6.3)

VELOCITY EXPOSURE COEFFICIENT K_z
(TABLE 6-3)

FLOOR	ACTUAL HT. (ft)	EST. HT (ft)	K_z
1	5.25	6	.57
2	20.45	21	.63
3	35.7	36	.74
4	50.95	51	.81
ROOF	63.61	64	.87

INTERPOLATION

$$y = y_a + (x - x_a)\frac{(y_b - y_a)}{(x_b - x_a)}$$

FOR FLOOR 2 ($h = 21$)

$$K_{21} = .62 + (21 - 20)\frac{(.66 - .62)}{(25 - 20)} = .63$$

FOR FLOOR 3

$$K_{36} = .7 + (36 - 30)\frac{(.76 - .7)}{(40 - 30)} =$$

FOR FLOOR 4

$$K_{51} \approx .81$$

FOR ROOF

$$K_h = .85 + (64 - 60)\frac{(.89 - .85)}{(70 - 60)} = .87$$

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

WIND DIRECTIONALITY FACTOR $K_d = .85$

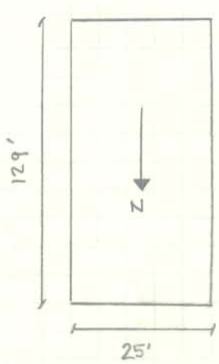
DETERMINE VELOCITY PRESSURE (q_z)

HEIGHT (ft)	K_z	q_z
6	.57	11.55
21	.63	12.77
36	.74	15
51	.81	16.42
64	.87	17.63

$$q_z = .00256\, K_z K_{zt} K_d V^2 I$$

ASSUME BUILDING IS RIGID $n_1 = {}^{100}/_{H} \geq 1$

$$g_Q = g_v = 3.4$$



	N-S DIRECTION	E-W DIRECTION
B (ft)	46'	129'
L (ft)	129'	46'
h	64.3	64.3

$$\bar{z} = .6h = 38.6\text{ ft} \geq z_{min}$$

$$z_{min} = 30\text{ ft}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .292$$

$$c = .3$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 337.16$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{B+h}{L_z}\right)^{.63}}}$$

N-S DIRECTION

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{46+64.3}{337.16}\right)^{.63}}}$$

$Q = .873$

E-W DIRECTION

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{129+64.3}{337.16}\right)^{.63}}}$$

$Q = .832$

GUST EFFECT FACTORS (G)

$$G_{N-S} = .925\left(\frac{1 + 1.7(3.4)(.292)(.873)}{1+1.7(3.4)(.292)}\right) = .851$$

$$G_{E-W} = .925\left(\frac{1 + 1.7(3.4)(.292)(.832)}{1 + 1.7(3.4)(.292)}\right) = .827$$

ENCLOSURE CLASSIFICATION: ENCLOSED (SECT 6.2)

INTERNAL PRESSURE COEFFICIENT $GC_{pi} = \pm .18$ (Fig 6-5)

EXTERNAL PRESSURE COEFFICIENTS, Cp

WINDWARD WALL = .8
SIDE WALL = -.7

LEEWARD WALL

$Cp_{N-S} = L/B = 129/46 = 2.8 \rightarrow -.26$

$Cp_{E-W} = L/B = 46/129 = .36 \rightarrow -.5$

$$Cp_{N-S} = -.3 + (2.8-2)\frac{(-.2-(-.3))}{(4-2)} = -.26$$

	Wind Direction	
	N-S	E-W
Stiffness	Rigid	Rigid
B (feet)	46	129
L (feet)	129	46
h (feet)	64.3	64.3
g_q	3.4	3.4
g_v	3.4	3.4
z(feet)	38.6	38.6
I_z	0.292	0.292
c	0.3	0.3
L_z	337.16	337.16
l (feet)	320	320
ϵ	1/3.0	1/3.0
Q	0.873	0.832
G	0.851	0.827

	N-S	E-W
Windward	0.8	0.8
Leeward	-0.26	-0.5
Sidewall	-0.7	-0.7

Appendix E

Case 1 Wind

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	Y1WIND	0	418.5	761.498	0.000019	
ROOF	Max Drift Y	Y1WIND	279.75	781.625	761.498		0.000031
STORY4	Max Drift X	Y1WIND	0	418.5	607.499	0.00002	
STORY4	Max Drift Y	Y1WIND	279.75	781.625	607.499		0.000033
STORY3	Max Drift X	Y1WIND	0	418.5	424.999	0.00002	
STORY3	Max Drift Y	Y1WIND	279.75	689.375	424.999		0.000034
STORY2	Max Drift X	Y1WIND	0	418.5	242.5	0.000031	
STORY2	Max Drift Y	Y1WIND	279.75	689.375	242.5		0.000031

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	X1WIND	0	418.5	761.498	0.00013	
ROOF	Max Drift Y	X1WIND	279.75	781.625	761.498		0.000036
STORY4	Max Drift X	X1WIND	0	418.5	607.499	0.00014	
STORY4	Max Drift Y	X1WIND	279.75	781.625	607.499		0.000038
STORY3	Max Drift X	X1WIND	0	418.5	424.999	0.000134	
STORY3	Max Drift Y	X1WIND	279.75	689.375	424.999		0.000043
STORY2	Max Drift X	X1WIND	0	418.5	242.5	0.000234	
STORY2	Max Drift Y	X1WIND	279.75	689.375	242.5		0.000058

Case 2 Wind

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	XWIND2	0	418.5	761.498	0.000146	
ROOF	Max Drift Y	XWIND2	279.75	781.625	761.498		0.000044
STORY4	Max Drift X	XWIND2	0	418.5	607.499	0.000158	
STORY4	Max Drift Y	XWIND2	279.75	781.625	607.499		0.000047
STORY3	Max Drift X	XWIND2	0	418.5	424.999	0.000154	
STORY3	Max Drift Y	XWIND2	279.75	689.375	424.999		0.000053
STORY2	Max Drift X	XWIND2	0	418.5	242.5	0.000276	
STORY2	Max Drift Y	XWIND2	279.75	689.375	242.5		0.000071

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	YWIND2	0	418.5	761.498	0.00002	
ROOF	Max Drift Y	YWIND2	279.75	781.625	761.498		0.000025
STORY4	Max Drift X	YWIND2	0	418.5	607.499	0.000021	
STORY4	Max Drift Y	YWIND2	279.75	781.625	607.499		0.000027
STORY3	Max Drift X	YWIND2	0	418.5	424.999	0.000021	
STORY3	Max Drift Y	YWIND2	279.75	689.375	424.999		0.000028
STORY2	Max Drift X	YWIND2	0	418.5	242.5	0.000034	
STORY2	Max Drift Y	YWIND2	279.75	689.375	242.5		0.000026

Case 3 Wind

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	WIND3	0	418.5	761.498	0.000112	
ROOF	Max Drift Y	WIND3	279.75	781.625	761.498		0.00005
STORY4	Max Drift X	WIND3	0	418.5	607.499	0.00012	
STORY4	Max Drift Y	WIND3	279.75	781.625	607.499		0.000053
STORY3	Max Drift X	WIND3	0	418.5	424.999	0.000115	
STORY3	Max Drift Y	WIND3	279.75	689.375	424.999		0.000058
STORY2	Max Drift X	WIND3	0	418.5	242.5	0.000199	
STORY2	Max Drift Y	WIND3	279.75	689.375	242.5		0.000067

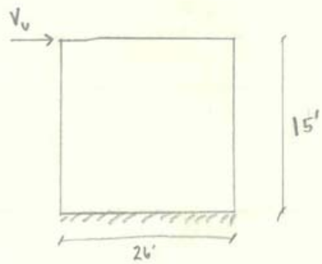
Seismic Drift

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	XQUAKE	0	418.5	761.498	0.00006	
ROOF	Max Drift Y	XQUAKE	279.75	781.625	761.498		0.000017
STORY4	Max Drift X	XQUAKE	0	418.5	607.499	0.000066	
STORY4	Max Drift Y	XQUAKE	279.75	781.625	607.499		0.000018
STORY3	Max Drift X	XQUAKE	0	418.5	424.999	0.000062	
STORY3	Max Drift Y	XQUAKE	279.75	689.375	424.999		0.000019
STORY2	Max Drift X	XQUAKE	0	418.5	242.5	0.000086	
STORY2	Max Drift Y	XQUAKE	279.75	689.375	242.5		0.000021

Story	Item	Load	X	Y	Z	Drift X	Drift Y
ROOF	Max Drift X	YQUAKE	0	418.5	761.498	0.00003	
ROOF	Max Drift Y	YQUAKE	279.75	781.625	761.498		0.000047
STORY4	Max Drift X	YQUAKE	0	418.5	607.499	0.000031	
STORY4	Max Drift Y	YQUAKE	279.75	781.625	607.499		0.000051
STORY3	Max Drift X	YQUAKE	0	418.5	424.999	0.000031	
STORY3	Max Drift Y	YQUAKE	279.75	689.375	424.999		0.000051
STORY2	Max Drift X	YQUAKE	0	418.5	242.5	0.00004	
STORY2	Max Drift Y	YQUAKE	279.75	689.375	242.5		0.000041

Appendix E

SW 11 SPOT CHECK



$F'_c = 4ksi$
$f_y = 60ksi$
$h = 12''$

1) CHECK PERMITTED SHEAR STRENGTH

$V_u \le (\phi V_n)_{max} = \phi 10\sqrt{f'_c}\; h\, d$

$d = .8(26)(12) = 249.6$

$\phi V_n = \frac{.75(10)\sqrt{4000}\,(12)(249.6)}{1000} = 1420.7 \therefore$ OK

2) SHEAR STRENGTH

$V_c = 2\sqrt{f'_c}\; h d = 2\sqrt{4000}\,(12)(249.6) = 380^k$

$V_c = 3.3\sqrt{f'_c}\,(12)(249.6) = 625^k \leftarrow$ GOVERNS

3) REQUIRED HORIZ REINF

$V_u \geq \frac{1}{2}\phi V_c$

$\frac{1}{2}(.75)(625) = 234.4 > V_u$

4) HORIZONTAL SHEAR REINFORCING

$.0025sh$

$s = \frac{l_w}{3} = \frac{26(12)}{3} = 104''$

$3h = 36''$

$18'' \leftarrow$ GOVERNS

$.0025(18)(12) = .54 \rightarrow (2)\#5$

$\rho_t = \frac{2(.31)}{12(18)} = .0029 > .0025$ ✓

5) VERTICAL SHEAR REINFORCEMENT

$$\rho = \frac{A_v}{sh}$$

$$A_v = .0025(18)(12) = .54 \rightarrow (2)\#5$$

AS COMPARED TO

(2) #4 HORIZONTAL @ 12"
(2) #5 VERTICAL @ 10"