Mountain Hotel, Urban Virginia

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Structural Option

November 12, 2012

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Table of Contents

Title Page	1
Table of Contents	2
Executive Summary	3
Building Introduction	4
Structural Overview	5
Code Requirements	6
Gravity System Overview	7
Superstructure	7
Substructure	7
Design Loads	8
Lateral System	9
Modeled Lateral Systems Analysis	11
Wind Loads	12
Seismic Loads	15
Controlling Lateral Loads	16
Member Checks	17
Conclusion	18
Appendices	19
Appendix A – Design Loads	19
Appendix B – Wind Loads	20
Appendix C – Seismic Loads	23
Appendix E – Braced Wall Forces	26
Appendix D – Member Checks	27

Executive Summary

The objective of Technical Report III is a study to obtain an in depth understanding of how lateral forces due to wind and seismic are transmitted through the various load resisting elements of the Mountain Hotel. The lateral system for the Mountain Hotel is comprised of 18 light gauge braced shear walls, 12 specially reinforced masonry shear walls, and a concrete shear wall which encircles the perimeter on the lowest level.

The distribution of lateral forces was discussed and how the forces are transferred through the lateral system into the ground. A three dimensional model of the lateral force resisting elements was constructed using ETABS for determining the relative stiffness of each shear wall, and the centers of mass and rigidity, and torsion, and story drift. Torsion had a small impact on the structure in the E-W direction, and a 10% contribution to the N-S direction, which was to be expected due to the placement of the lateral elements.

Building drift requirements for seismic loads were determined. Deflections due to story drifts for seismic were checked against $0.02h_{xx}$. Overturning moments caused by seismic were evaluated in each direction for both the overall building and a shear wall. Overturning issues did not arise.

Member checks for a shear wall in each direction were performed in order to verify that the lateral members were adequate to resist the potential loads.

Building Introduction

Figure 4-1
An aerial view from bing.com maps with the building superimposed on. Hotel is in Red, Garage in Yellow.

The new hotel is to be located in a wealthy urban area of Virginia (Location shown in Figure 4-1). The site chosen for construction of the new hotel is a prominent location previously occupied by a chain of parking lots, which border the main street of the town.

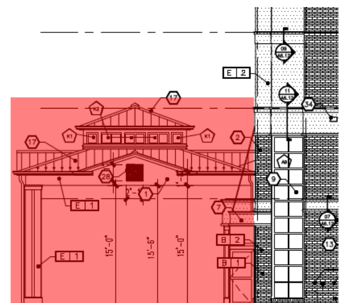


Figure 4-2
Porte Cochere attached to Hotel over main Entrance.

In order to match the new building into its surrounding architecture the first two floor facades are brick with large glazing panels, while the upper facade uses a palette of varying shades from brick red to white which enables it to match the brick and concrete of the surrounding buildings, including the adjacent concrete parking structure. However, in place of the brick or concrete, the upper stories of the hotel use a lighter more cost effective cladding, exterior insulation finishing system (EIFS) panels. The Porte Cochere on the west side, shown in Figure 4-2, will help funnel visitors into the main lobby where they can check-in and be directed to their rooms, other amenities, or sites of the town.

Guest rooms are located on the second through sixth floors totaling just over 40,000 square feet. Though the main function is to appease guests with a home away from home, it also contains meeting rooms for conferences, offices for hotel management, and a 40,000 square foot parking garage. Total building area is approximately 120,000 square feet.

Structural Overview

The hotel rests on reinforced concrete spread footings ranging from 12 to 42 inches in depth. Concrete piers transfer the load into the interior footings from the steel columns. The exterior concrete basement walls rest on strip footings, ranging from 12 to 24 inches, are load bearing and double as sheer walls for the lateral system. A500 Grade B hollow structural steel ranging from four to 16 inches, longer dimension, is used for the superstructure columns. Some of the floors are supported by wide flange beams, ranging from W8 to W21, while others are resting on steel stud bearing walls as shown in Figure 5-1. The lateral system employs a combination of reinforced concrete shear walls, specially reinforced masonry shear walls and light framed wall system with flat strap bracing extending from the ground floor to roof level in both the long and short directions. Floors ground through six are installed as a series of eight inch precast hollow-core planks ranging in length from 9' 2" to 25' 8". The roof is also built of four or eight inch hollow-core planks. Both the brick walls and EIFS system are attached to cold formed steel stud walls. The loading on the exterior facade is transferred through the wall framing to the floors and into the lateral system.

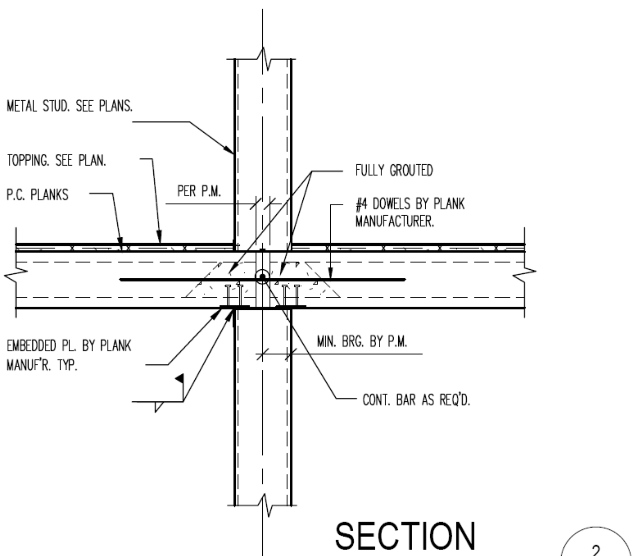


Figure 5-1
Hollow core plank ends spliced and resting on light gauge steel stud framed walls.

The garage is also supported on reinforced concrete spread footings 12 to 30 inches in depth, and strip footers 12 to 24 inches in depth. Piers transfer the load into the footings from the columns and the walls rest directly on the strip footings. Piers and beams are poured monolithic with the walls. Columns support two-way slabs and utilize drop panels, and edge beams.

Code Requirements

Standards and codes governing construction are as follows:

2009 ICC/ANSI A117.1

2009 International Building Code

2009 Virginia Uniform Statewide Building Code

2008 NEC – National Electric Code

2009 ICC – International Mechanical Code

2009 ICC – International Plumbing Code

2009 ICC – International Energy Conservation Code

- All concrete work shall be in accordance with ACI 301, ACI 318 and ACI 302 latest editions.
- All Masonry work shall be in accordance to: ACI 530/ASCE 5, "Building code requirements for Masonry structures"; ACI 530/ASCE 6, "specifications for masonry structures"
- Structural Steel Shall conform to the AISC "Specification for the design fabrication and erection of structural Steel for buildings", Latest edition, except chapter 4.2.1, code of standard practice
- All light gauge framing shall conform to "the specification for the design of cold-formed steel structural members", latest edition, by AISI
- All Wood framing shall conform to the "national design specification for wood construction" latest edition, published by the national forest products association,
- In addition to the requirements included in these structural notes, all construction and materials shall further conform to the applicable provisions of the following standards:
    1. American Society for Testing and Materials (ASTM)
    2. American Concrete Institute (ACI)
    3. National Concrete Masonry Association (NCMA)
    4. American Institute of Steel Construction (AISC)
    5. American Welding Society (AWS)
    6. American Iron and Steel Institute (AISI)
    7. Steel Structures Painting Council (SSPC)
    8. American Forest and Paper Association
    9. National Forest Products Association (NfoPA)

Governing the Parking Garage is all of the above with the inclusion of:

2006 International Building Code

2006 Virginia Uniform Statewide Building Code

Gravity System

Superstructure

This building uses several types of structural members to carry the various gravity induced loads to the earth. The hotel roof and all above grade floors utilize hollow core planks to support the dead loads of the structure as well as all the amenities people and other items. The planks typically rest on cold-formed steel stud shear walls which pass the load onto the floor below, and so on until it either reaches either a reinforced concrete shear wall or a wide flange beam which it can do so as high as the fourth floor, or as low as the first floor. W-shapes made to the ASTM standard A992 range in size from W6x15 to W33x130. ASTM A500 Hollow Structural Section (HSS), ranging from HSS 4x4x¼ HSS 12x12x½, columns hold the beams in place. Most of the HHS columns terminate in the lower floors; however there are several members that transfer load directly from the roof into the foundations. The Elevator and stair towers are an exception the typical framing types. They use specially reinforced masonry sheer walls to resist both gravity and lateral loads stretching from above the normal roof height and down into the foundation.

Substructure

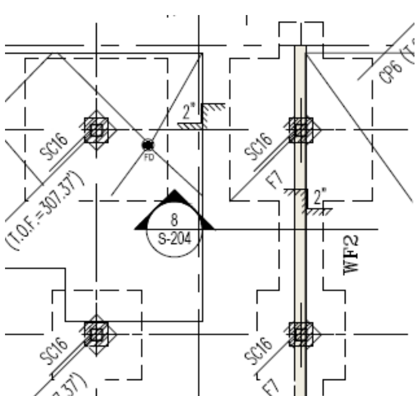


Figure 7-1
Shows a portion of the foundation plan using various structural elements.

The substructure uses a series of reinforced concrete shear walls to transfer the loads from the superstructure into the wall footings of the foundation (Figure 7-1). Under columns and column piers, there is a series of spread footings the largest of which is 16"x16"x42"deep. Footings maintain a minimum compressive strength of 3000psi. Other concrete members have an Fc of 5000psi. Footings rest upon soil which has a bearing pressure of 3000psf.

Design Loads

Load Combinations

Listed here are all the load combinations that are being considered. All load combinations are based on LRFD and come from ASCE 7-10.

- 1.4D
- 1.2D + 1.6L + 0.5(Lr or S or R)
- 1.2D + 1.6(Lr or S or R) + (L or 0.5W)
- 1.2D + 1.0W + L + 0.5(Lr or S or R)
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.0W
- 0.9D + 1.0E

Deflection Criteria

Typical Live Load Deflection Limit: L/480

Typical Total Load Deflection Limit: L/360

Drift Criteria

Allowable Building Drift limit: (L or H)/400

Inter-story Drift

Wind: (L or H)/400 to (L or H)/600

Seismic: .020 x h_{xx}

Lateral System

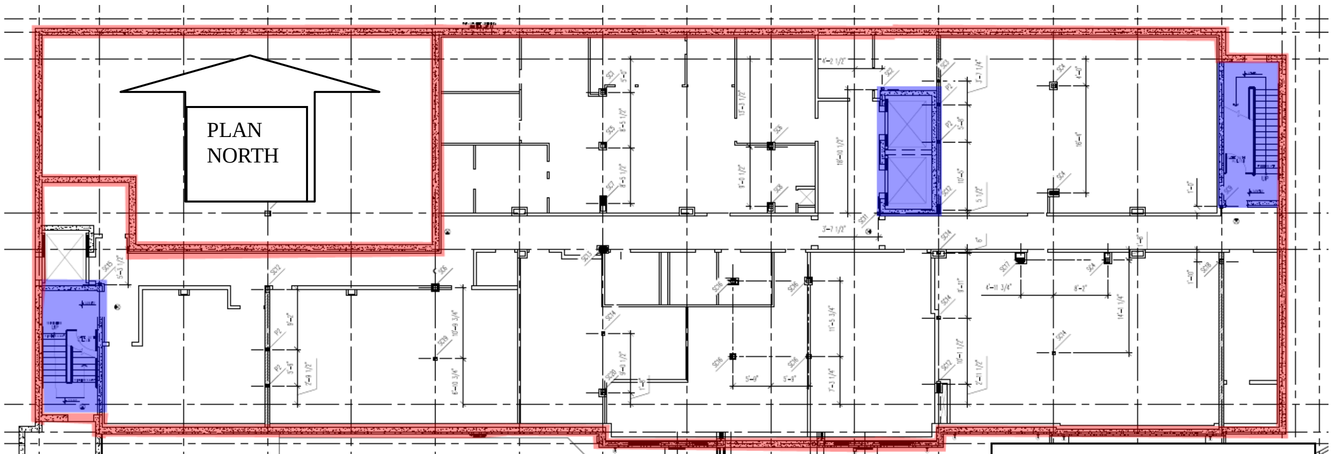


Figure 9-1
Locations of various Shear Walls

Lateral forces in the Mountain Hotel are resisted mainly by three different types of elements. Below grade, lateral forces are resisted through a system of reinforced concrete shear walls some of which are highlighted in red in Figure 9-1. The exterior walls are 14 inches in thickness while most of the interior walls are eight inches thick. A few of these walls extend up to the second story, but most of the superstructure employs cold-formed steel stud walls with flat strap bracing to resist wind and earthquake loadings. Braced walls are shown in Figure 9-2 and are highlighted in yellow. In the design of the light gauge elements the structural engineer specified locations, possible member sizes and what forces these elements were required to resist. However, it is expected that the

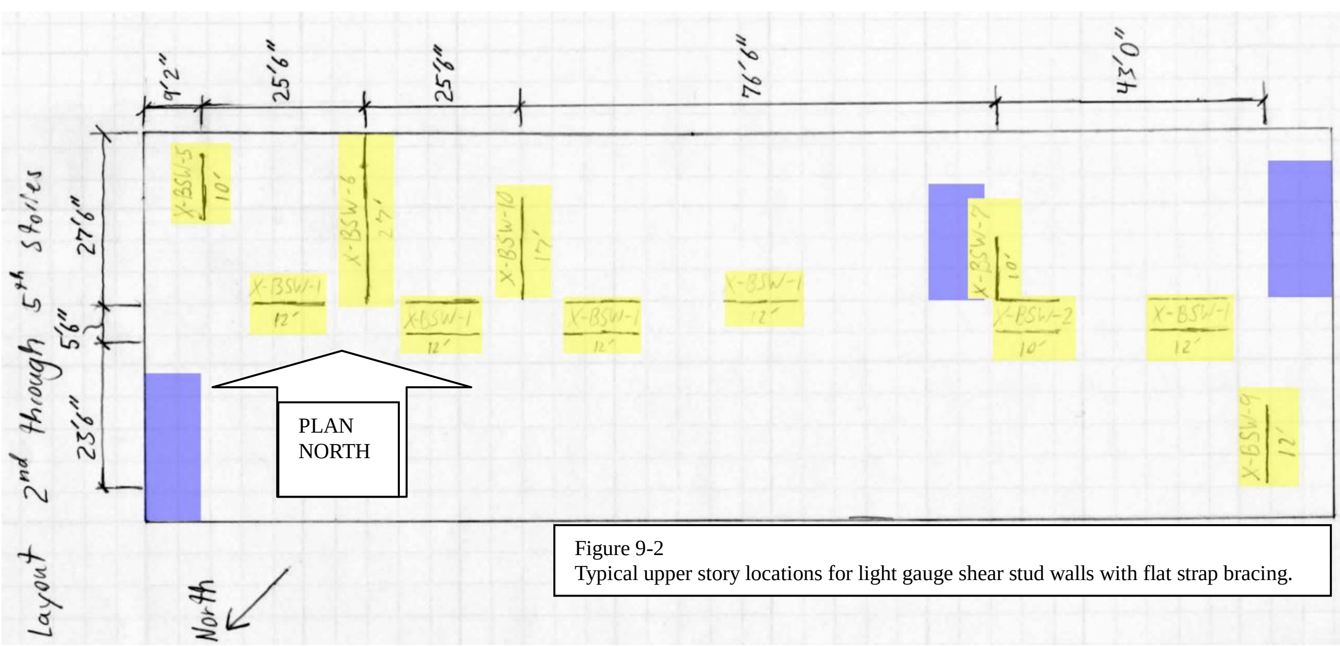


Figure 9-2
Typical upper story locations for light gauge shear stud walls with flat strap bracing.

light gauge provider size the individual members. In order to fully analyze the lateral system stiffness, straps were sized from the Marinoware cold formed steel framing system catalogue, to resist the forces specified in the drawings.

The elevator and two stair towers also contain specially reinforced masonry shear walls to resist forces in both the building's dimensions. Stair and Elevator tower locations are shaded in **blue**. A horizontal out of plane irregularity exists at the second floor under the E-W frames. A typical distribution of moment to the supporting beam is shown in Figure 10-1.

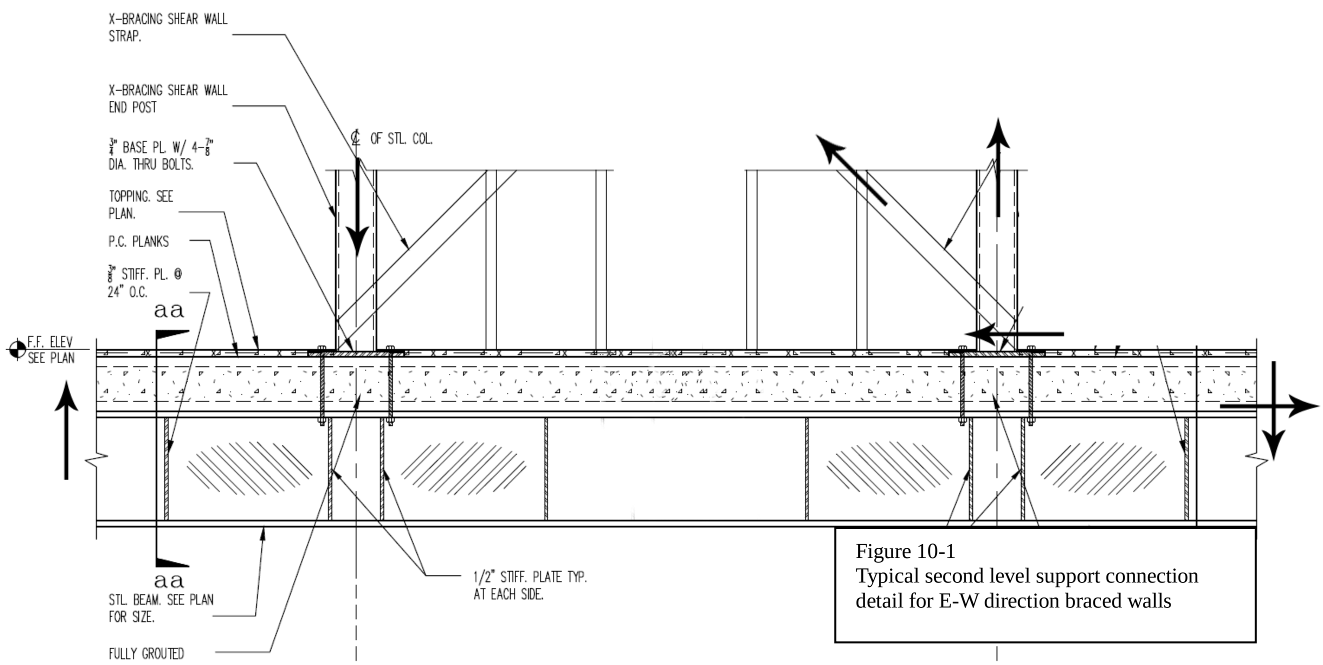


Figure 10-1
Typical second level support connection detail for E-W direction braced walls

Modeled Lateral System Analysis

A computer model was generated in ETABS for determining the relative stiffness of each shear wall and braced wall (shown in figure 11-1). Only the walls which are designed contribute to lateral resistance were considered. Because the floor planks are connected using both grout and rebar, it was assumed that the floors would act as rigid diaphragms, with the exception of the first level. It will most likely act as a semi rigid diaphragm resulting from the major stiffness decrease when moving from 14 inch concrete shear walls to light gauge braced walls.

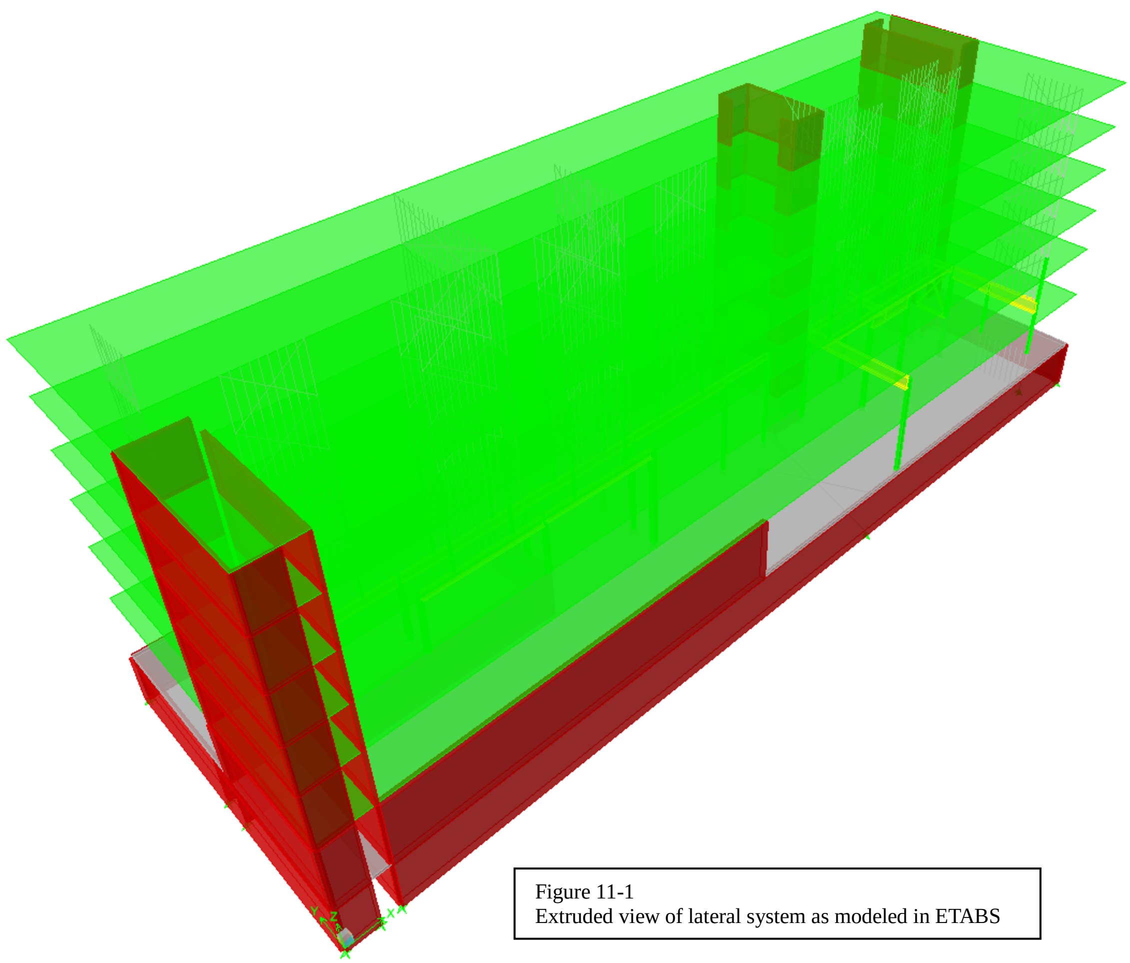


Figure 11-1
Extruded view of lateral system as modeled in ETABS

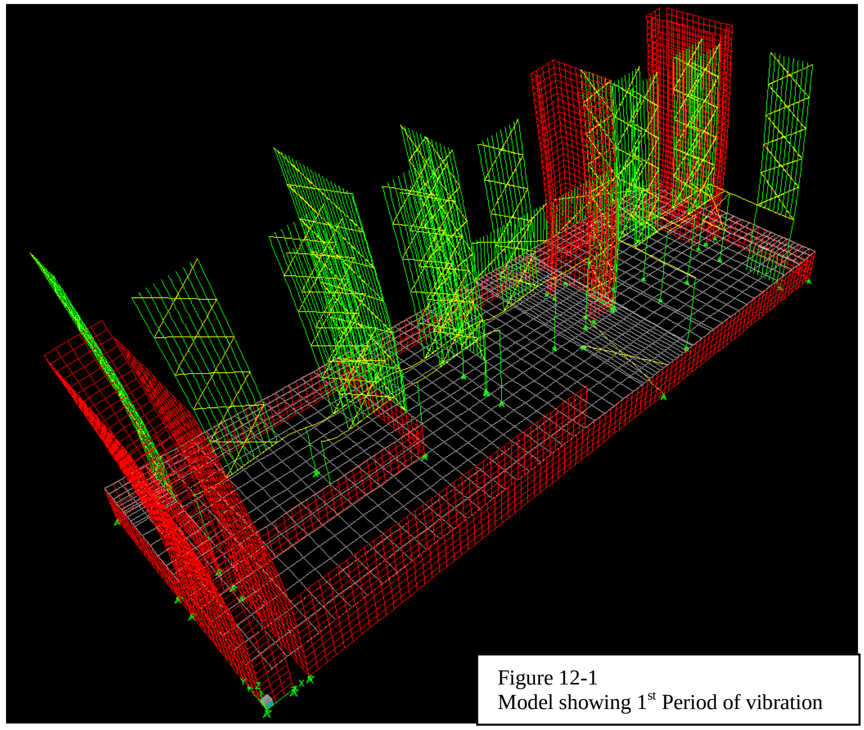


Figure 12-1
Model showing 1st Period of vibration

Because the concrete shear walls in the basement level are very stiff compared to the rest of the structure, lateral soil bearing pressures were neglected and the structure was modeled as if the basement sat at ground level. All elements were modeled using realistic weight but zero mass. Masses were calculated per story than lumped at the center of each diaphragm. The model was run and the periods of vibration were recorded in Table 12-1. Figure 12-1 shows the period in the x (E-W) direction.

Period of Vibration		
T1=1.111s (x)	T2=0.3943s (y)	T3=0.2419s (z)

Table 12-1

Centers of mass and rigidity were extracted from ETABS and recorded for the 2nd through roof diaphragms in Table 12-2 for further use in determining torsion issues.

Story	XCM	YCM	XCR	YCR
ROOF	1133	363	872.645	141.506
STORY6	1133	363	856.092	112.084
STORY5	1133	363	856.188	89.659
STORY4	1133	363	864.517	73.693
STORY3	1133	363	883.324	75.995
STORY2	1133	363	819.041	51.918

Table 12-2

Wind Loads

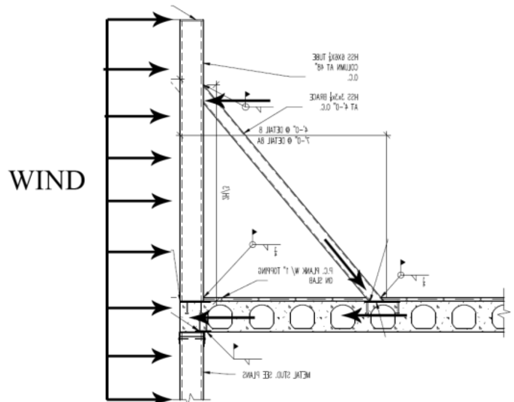


Figure 13-1
Transmission of wind force through parapet wall

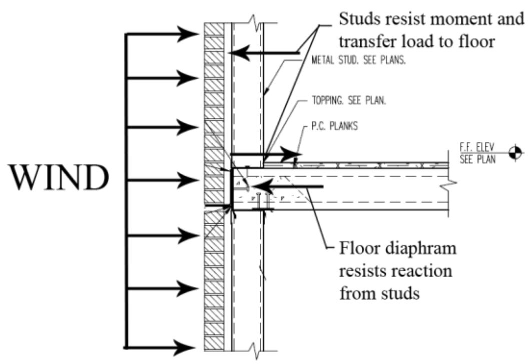


Figure 13-2
Load path for wind into the corresponding diaphragm

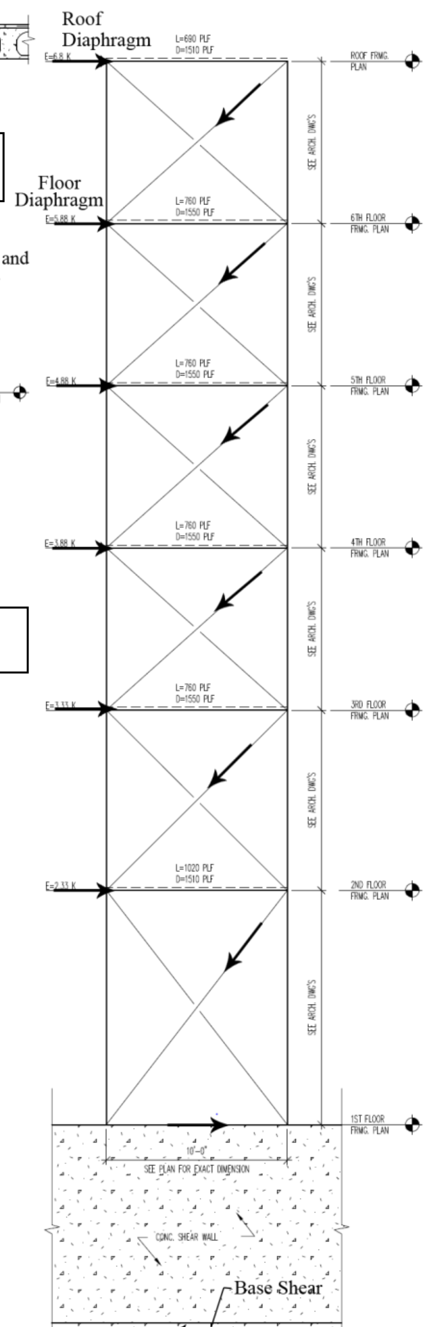


Figure 13-3
Shows the load path for later forces from the diaphragm contributions to the foundation

Wind loads on the building are collected by the exterior facade. Cladding, EIFS or brick, is mounted to the cold-formed stud walls. The studs transfer the wind loads into the floors diaphragms (shown in Figures 13-1 and 13-2) which are resisted by the flat strap braced (Shown in Figure 13-3) and masonry shear walls. Forces are transferred downward to each subsequent diaphragm utilizing light gauge flat strap bracing. Most of the braced walls transfer the load into single story concrete shear walls, which are joined to the foundation. This moment is finally resisted by a couple induced by the ground bearing pressures and the weight of the structure.

Basic wind loads for the hotel were calculated using the main wind force resisting system directional procedure outlined per chapter 27 of ASCE7-10. Several assumptions were made in order to use this procedure. First building was presumed to be rectangular with no depressions or extrusions in the facade. The parapet height was considered to be a constant nine foot height for the entire perimeter, and the shallow slope of the grade was considered to be negligible.

Wind pressures including windward, leeward, sidewall, and internal pressure were determined as shown in Figure 14-1 (N-S) and Figure 14-3 (E-W). Calculations spreadsheets and diagrams can be found in Appendix B. Excel was used to tabulate the story forces in each direction (Figure 14-2 (N-S) and Figure 14-4 (E-W)) and a total overturning moment was found for each direction (also in Figures 14-2 and 14-4).

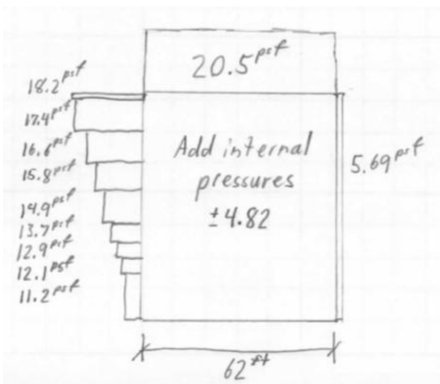


Figure 14-1
Wind pressures in the North – South direction

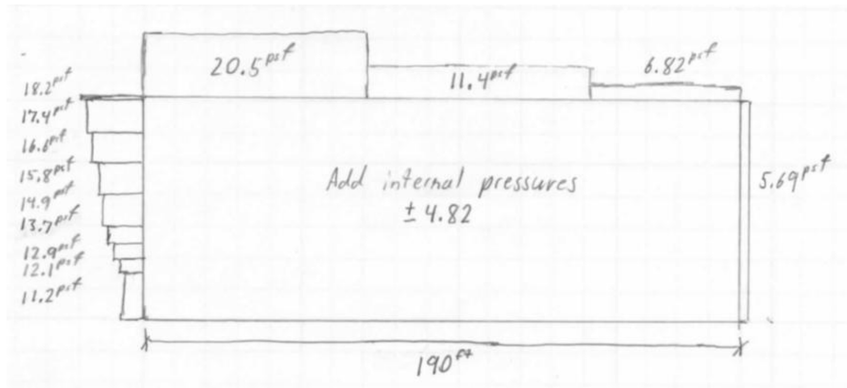


Figure 14-3
Wind pressures in the East – West direction

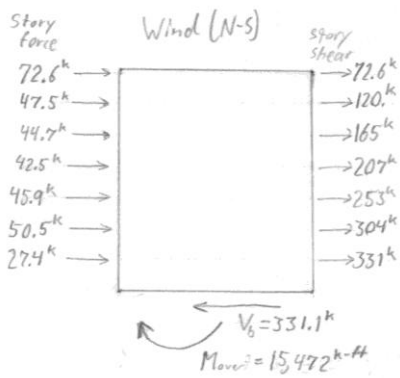


Figure 14-2
Story Forces in North – South direction
With base ear and overturning moment

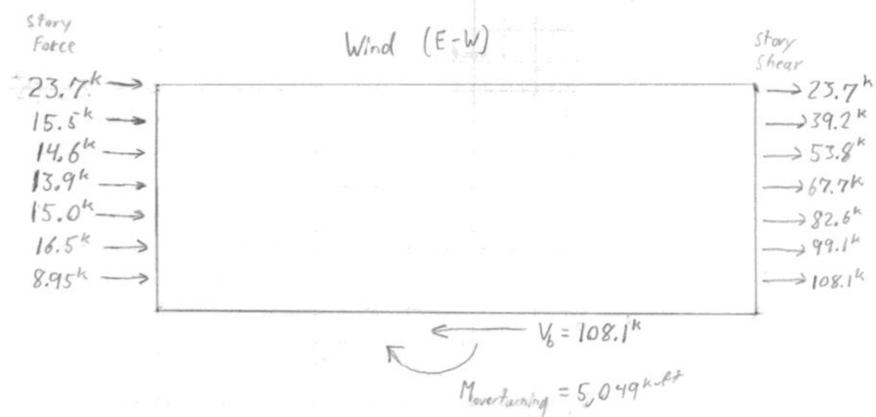


Figure 14-4
Story Forces in East – West direction
With base ear and overturning moment

Resultant story forces were applied to the center of each diaphragm as a load case in each direction. The lateral model was then used to determine drift. Drifts were compared to the drift limits specified in ASCE 7-10 to show that procedure yielded a logical result. Story drift due to wind is shown in Tables 15-1and 15-2.

Story	Height (in)	Load	DriftX (in)	DriftY (in)	Allowable Drift (in)
ROOF	112.00	WX	0.061936	0.00392	0.2800
STORY6	112.00	WX	0.060928	0.00392	0.2800
STORY5	112.00	WX	0.057568	0.003696	0.2800
STORY4	112.00	WX	0.05096	0.00336	0.2800
STORY3	134.00	WX	0.046096	0.002814	0.3350
STORY2	159.00	WX	0.014151	0.003657	0.3975
STORY1	120.00	WX	0.00096	0.00048	0.3000
TOTAL	861.00	WX	2.158527	0.155841	2.1525

Table 15-1

Story	Height (in)	Load	DriftX (in)	DriftY (in)	Allowable Drift (in)
ROOF	112.00	WY	0.010416	0.025424	0.2800
STORY6	112.00	WY	0.010192	0.024416	0.2800
STORY5	112.00	WY	0.00952	0.023856	0.2800
STORY4	112.00	WY	0.008288	0.022064	0.2800
STORY3	134.00	WY	0.00737	0.022512	0.3350
STORY2	159.00	WY	0.003657	0.020034	0.3975
STORY1	120.00	WY	0.00096	0.00288	0.3000
TOTAL	861.00	WY	0.369369	1.009953	2.1525

Table 15-2

Seismic Loads

Earthquake loads, are actually displacements induced as the mass of a structure attempts to regain equilibrium as the earth moves underneath it. These displacements are resisted by the main lateral force resisting system which sends the counter displacements, due to the momentum of the structure, back to the ground. In order to quantify the strength needed to resist these displacements the response force induced in the structure is determined using historical data.

Earthquake loads for the hotel were calculated using the equivalent lateral force procedure outlined per chapters 11 and 12 of ASCE7-10. The weight of each story was calculated to include the weight of the diaphragm plus the weight of half the wall above and below each story plus any other dead loads.

Corresponding masses was assigned to the center of each diaphragm. An overall building base shear was determined and used to find story forces and shear forces at each level (see Figures 16-1 and 16-2). These forces travel to the foundations via the same elements as wind. The load path of figure 13-3 applies. Calculations and spreadsheet can be found in Appendix C. Story and overall building drifts were tabulated and compared to the maximum allowable per ASCE 7-10.

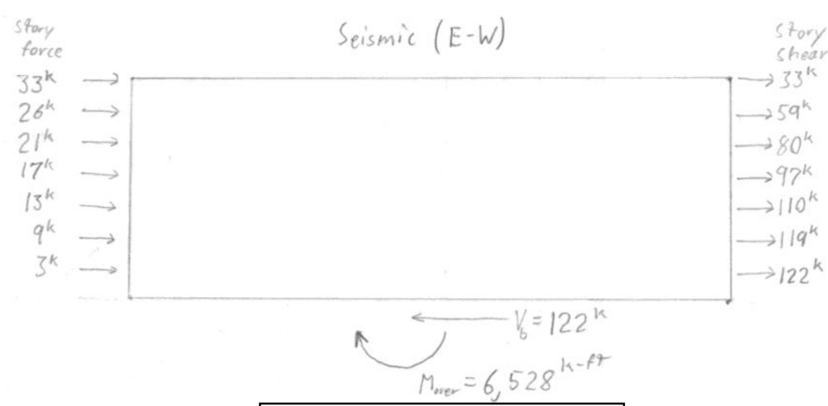


Figure 16-1
Seismic Story Forces and Story Shears in the East – West direction

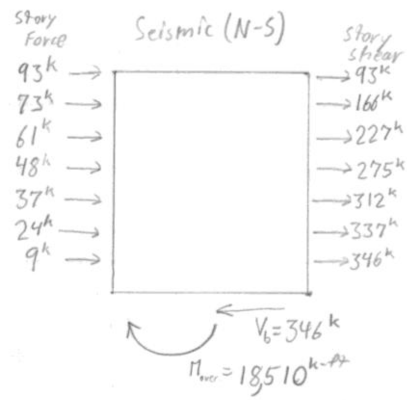


Figure 16-2
Seismic Story Forces and Story Shears in the North – South direction

Controlling Lateral Loads

After completing the wind and seismic load it can be concluded that seismic load clearly will control. The Structural Engineer did use the earthquake load to design the building with the critical combinations that have earthquake in them. From looking at the load combinations listed earlier in this report the load combination listed below seems to be the most critical and will be used for hand calculations while for modeling all combinations listed in the previous section will be considered.

1.2D + 1.0E + L + 0.2S

Four earthquake static load cases were added the model: one for story forces in each direction, and another for eccentricity (resultant moment) in each direction. Max drifts were extracted for each case. Comparison of sums of the drift in each direction to max allowable per ASCE 7-10 is shown in Tables 17-1 and 17-2.

Story	Height (in)	Load	DriftX (in/in)	DriftY (in/in)	Load	DriftX (in/in)	DriftY (in/in)	Max Drift X (in)	Max Drift Y (in)	Allowable Drift (in)
ROOF	112.00	EX	0.000787	0.000049	EXT	0.000002	0.000004	0.088368	0.005936	2.2400
STORY6	112.00	EX	0.000776	0.000049	EXT	0.000002	0.000004	0.087136	0.005936	2.2400
STORY5	112.00	EX	0.000733	0.000047	EXT	0.000002	0.000004	0.08232	0.005712	2.2400
STORY4	112.00	EX	0.000644	0.000042	EXT	0.000001	0.000003	0.07224	0.00504	2.2400
STORY3	134.00	EX	0.000478	0.000029	EXT	0.000001	0.000003	0.064186	0.004288	2.6800
STORY2	159.00	EX	0.000118	0.000031	EXT	0.000001	0.000002	0.018921	0.005247	3.1800
STORY1	120.00	EX	0.00001	0.000006	EXT	0	0	0.0012	0.00072	2.4000
TOTAL	861.00							0.414371	0.032879	2.1525

Table 17-1

Story	Height (in)	Load	DriftX (in/in)	DriftY (in/in)	Load	DriftX (in/in)	DriftY (in/in)	Max Drift X (in)	Max Drift Y (in)	Allowable Drift (in)
ROOF	112.00	EY	0.000121	0.000293	EYT	0.000014	0.000035	0.01512	0.036736	2.2400
STORY6	112.00	EY	0.000118	0.000284	EYT	0.000014	0.000033	0.014784	0.035504	2.2400
STORY5	112.00	EY	0.00011	0.000277	EYT	0.000014	0.000032	0.013888	0.034608	2.2400
STORY4	112.00	EY	0.000095	0.000255	EYT	0.000013	0.00003	0.012096	0.03192	2.2400
STORY3	134.00	EY	0.00007	0.000212	EYT	0.000012	0.000025	0.010988	0.031758	2.6800
STORY2	159.00	EY	0.000028	0.000149	EYT	0.000009	0.000017	0.005883	0.026394	3.1800
STORY1	120.00	EY	0.00001	0.000028	EYT	0.000001	0.000003	0.00132	0.00372	2.4000
TOTAL	861.00							0.074079	0.20064	2.1525

Table 17-2

Lateral Member Checks

Strength checks were performed on the lateral members to verify that they could carry the loads determined earlier in the report. Two walls were evaluated, a braced wall extending from the roof to the foundation in the N-S direction and the other a braced wall in the E-W direction which rests on a beam and the second floor level. It was determined that the strength capacity provided was way more than sufficient to carry the lateral loads. Overturning was calculated to be an issue, however due to the proximity of adjacent members and connected foundations it will most likely be resisted. Refer to Appendix E for the calculations and assumptions used for the checks.

Conclusion

After completing a lateral analysis of the Mountain Hotel it can be concluded that lateral loads are applied in the forms of wind and earthquake forces which cause shears in each story to be resisted by the various shear walls placed throughout the building. The floor diaphragms on all the above ground stories act rigidly such that the loads travel through the structure on the basis of relative stiffness. Using data from the accurate computer model it was determined that the building is adequate for resisting seismic forces. Story forces due to shear were less than those of seismic and therefore did not control. The overall building drift including torsion effects was .414" in the x-direction and .201" in the y-direction.

Torsion causes an increase in story drift which results in the necessity for a slightly higher design load. The centers of rigidity in the East – West direction are very close however improvements could be made in the North – South direction as they vary by about 5% between stories.

Due to the discontinuity of the braced walls in the E-W direction the structure exhibits out of plane discontinuity, creating large moments in the beams supporting the braced walls.

Overall it is felt that after completion of this technical report, a greater knowledge of lateral load distribution has been gained along with a better understanding of how resisting elements work together. A further investigation may need to be performed in the future depending on what changes are made to the structure during the spring semester

Appendix A – Design Loads

Roof Loads					
Snow Load					
	Ground Snow Load, Pg			30	PSF
	Flat Roof Snow Load, Pf = Ce*Pg			21	PSF
	Snow Exposure Factor, Ce			0.7	
	Snow Load Importance Factor, I			1.0	
	Snow Drift & Sliding Surcharge:				Per Code Requirements
Roof Load					
	Roof Live Load (Horizontal Projection)			37	PSF minimum
	Dead Load			81.5	PSF
Floor Loads (PSF)					+ Mechanical Unit Weight per MEP Drawings
		Dead Load	Live Load	Total Load	
Living Areas		81.5	40	121.5	
Common Areas		75.5/81.5	100	175.5/181.5	
Stairs		50	100	150	
Storage (Light)		81.5	125	206.5	
First Floor Only		106.5	100	206.5	
Wind Loads					
Basic Wind Speed (3- Second Gust)				90	MP
Occupancy Category				II	
Wind Importance Factor, 1				1.0	
Wind Exposure				B	
Internal Pressure Coefficient, Gcpi				±0.18	
Components and Cladding				21	PSF
Earthquake Loads					
Seismic Importance Factor, 1				1.0	
Mapped Spectral Response Accelerations				Ss = 15.5	% g
				S1 = 5.1	% g
Site Class				D	
Spectral Response Accelerations				Sds = 16.5	% g
				Sd1 = 8.2	% g
Seismic Design Category				B	
Design Base Shear				208	
Seismic Response Coefficients				0.028	
Response Modification Factors				4	
Foundation					
Footing Design Soil Bearing Pressure				3000	PSF
Back Fill Material Equivalent Fluid Pressure				55	PSF/FT
Deflection Limits					
			Live Load	Total Load	
Floor			SPAN/480	SPAN/360	
Floor Under Ceramic Tile			SPAN/720	SPAN/360	
Roof Trusses			SPAN/360	SPAN/240	
Roof Rafters			SPAN/240	SPAN/180	
Ceiling Joist			SPAN/360	SPAN/240	
Roof Ridge/Beam			SPAN/360	SPAN/120	

Appendix B – Wind Loads

Wind Calcs | Tech 1

Basic Wind Speed $V = 115^{mph}$ (ASCE7-10 Figure 26.5-1A)

Building Classification → Occpancy Category II → Importance Factor 1.0
(ASCE7-10 Table 1.5-2)

Exposure → B →

Velocity Pressure

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

MWFRS → K_z: 0-15ft → .57
15-20ft → .62
20-25ft → .66
25-30ft → .70
30-40ft → .76
40-50ft → .81
50-60ft → .85
60-70ft → .89
70-80ft → .93 = K_h

$K_{zt} \rightarrow 1.0$

$K_d = .85$

$q_z = .00256 K_z (1.0)(.85)(115^{mph})^2 = 28.7776 K_z$

Design Wind Pressures

$p = q G C_p - q_i (GC_{pi})$ (G = .85; GC_{pi} = ±.18 → Enclosed)

Windward

$p = 28.7776 K_z (.85)(.8) - 28.7776(.93)(\pm .18)$ (C_p → Figure 27.4-1)
$= 19.57 K_z \pm 4.82$

(All ±4.82 for internal Pressure)

$p_{15} = 19.57(.57) = 11.15^{psf}$
$p_{20} = 19.57(.62) = 12.13^{psf}$
$p_{25} = 19.57(.66) = 12.92^{psf}$
$p_{30} = 19.57(.70) = 13.70^{psf}$
$p_{40} = 19.57(.76) = 14.87^{psf}$
$p_{50} = 19.57(.81) = 15.85^{psf}$
$p_{60} = 19.57(.85) = 16.63^{psf}$
$p_{70} = 19.57(.89) = 17.42^{psf}$
$p_{80} = 19.57(.93) = 18.20^{psf}$

Wind Calcs | Tech 1

Sidewall: $C_p = -.7 \rightarrow p = 28.7776(.93)(.85)(.7) = 15.9^{psf}$

Leeward

Normal to 190′ wall $\rightarrow L/B = .326 \rightarrow C_p = -.5$

Normal to 62′ wall $\rightarrow L/B = 3.06 \rightarrow C_p = -.25$

$p = 28.7776(.93)(.85)\,C_p = 22.7\,C_p$

Normal to 190′ $\rightarrow p = 22.7(-.5) = -11.4^{psf}$

Normal to 62′ $\rightarrow p = 22.7(-.25) = -5.69^{psf}$

Roof

distance from windward edge: $0\text{-}h \rightarrow C_p = -.9,\ -.18$

$h\text{-}2h \rightarrow C_p = -.5,\ -.18$

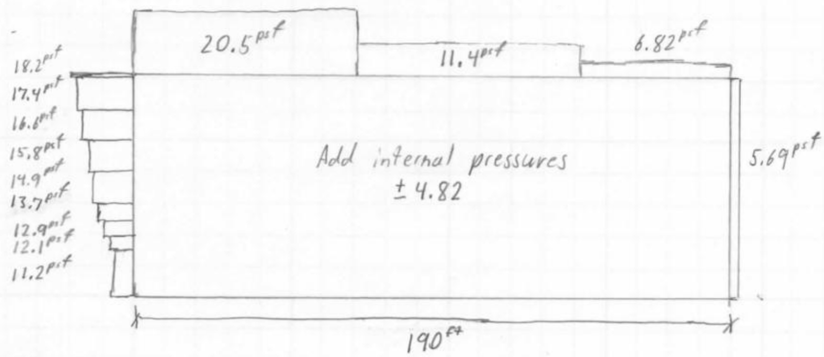
$>2h \rightarrow C_p = -.3,\ -.18$

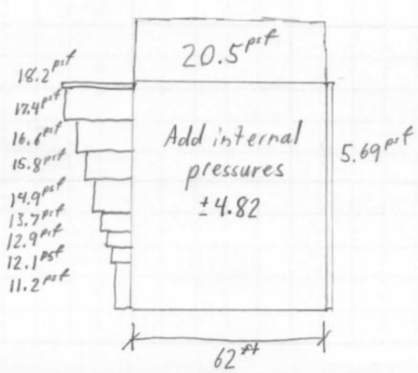
$0\text{-}h \rightarrow p = 22.7(-.9) = -20.5^{psf}$

$= 22.7(-.18) = -4.09^{psf}$

$h\text{-}2h \rightarrow p = 22.7(-.5) = -11.4^{psf}$

$>2h \rightarrow p = 22.7(-.3) = -6.82^{psf}$





Page 2 of 2

Normal to 62ft Wall			Width =	62
Elevation	Windward Pressure	Leeward Pressure	Internal Pressure	Total Pressure
71	18.2	5.69	4.82	28.71
70	17.4	5.69	4.82	27.91
60	16.6	5.69	4.82	27.11
50	15.8	5.69	4.82	26.31
40	14.9	5.69	4.82	25.41
30	13.7	5.69	4.82	24.21
25	12.9	5.69	4.82	23.41
20	12.1	5.69	4.82	22.61
15	11.2	5.69	4.82	21.71

Level	Story Height	Story Force
Roof	62	23706.475
6th	52.5	15496.59
5th	43	14572.635
4th	34	13884.435
3rd	24.5	14967.637
2nd	13.3	16488.745
Base Shear		99116.517

Total Moment 22587684

Normal to 190ft Wall			Width =	190
Elevation	Windward Pressure	Leeward Pressure	Internal Pressure	Total Pressure
71	18.2	5.69	4.82	28.71
70	17.4	5.69	4.82	27.91
60	16.6	5.69	4.82	27.11
50	15.8	5.69	4.82	26.31
40	14.9	5.69	4.82	25.41
30	13.7	5.69	4.82	24.21
25	12.9	5.69	4.82	23.41
20	12.1	5.69	4.82	22.61
15	11.2	5.69	4.82	21.71

Level	Story Height	Story Force
Roof	62	72648.875
6th	52.5	47489.55
5th	43	44658.075
4th	34	42549.075
3rd	24.5	45868.565
2nd	13.3	50530.025
Base Shear		303744.17

Total Moment 54886429

Appendix C – Seismic Loads

Seismic Calcs | Tech 3

Mapped max consideration spectral response acceleration at:

Figure 22-1 → $S_s = 13.5\%$

Figure 22-2 → $S_1 = 5.5\%$

Site Class D

Table 11.4-1 → $F_a = 1.6$

Table 11.4-2 → $F_v = 2.12$

eq 11.4-1 → $S_{ms} = F_a S_s = 1.6(.135) = 21.6\%$

eq 11.4-2 → $S_{m1} = F_v S_1 = 2.12(.055) = 11.7\%$

eq 11.4-3 → $S_{DS} = \frac{2}{3} S_{ms} = \frac{2}{3}(.216) = .144$

eq 11.4-4 → $S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3}(.117) = .076$

Seismic Design Category per Table 11.6-1 & 11.6-2 → A

Table 12.2-1 → R = 4 light-fram (cold-formed steel) wall systems using flat strap bracing

Table 12.8-2 → $C_t = .02$, $x = .75$

eq 12.8-7 → $T_a = C_t h_n^{\,x} = .02(71.75)^{.75} = .838^{sec}$ → $T = C_u T_a = 1.7(.838) = 1.4246$ (table 12.8-1)

Table 22-12 → $T_L = 8^{sec} > .44^{sec}$ $\qquad T_x = 1.111,\ T_y = .3943$

eq 12.8-2 & 12.8-3 → $C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \le \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$

$C_s = \frac{.144}{4} = .036$

$C_s \le \frac{.076}{1.43(4)} = .0133$ ← controls → $C_{sx} = \frac{.076}{1.111(4)} = .0171$, $C_{sy} = \frac{.076}{0.3943(4)} = .0482$

$C_s > .01$

Tech 3

Roof: $W_{rf} = 190^{ft} \times 60^{ft} \times 75.5^{psf} + 7500^{lb} + 2(190^{ft} + 60^{ft})\left(\frac{9\frac{1}{3}'}{2} + 9'\right)(30^{psf}) = 1{,}073{,}200$

Floors: $W_{f4\text{-}6} = 190^{ft} \times 60^{ft} \times 75.5^{psf} + 2(190^{ft} + 60^{ft})\left(\frac{9\frac{1}{3} + 9\frac{1}{3}}{2}\right)(30^{psf}) = 1{,}000{,}700^{lb}$

$W_{f3} = 190 \times 60 \times 75.5 + 2(190 + 60)\left(\frac{9\frac{1}{3} + 11\frac{1}{6}}{2}\right)(30^{psf}) = 1{,}014{,}450^{lb}$

$W_{f2} = 190 \times 60 \times 75.5 + 2(190 + 60)\left(\frac{11\frac{1}{6} + 13\frac{1}{3}}{2}\right)(30^{psf}) = 1{,}044{,}450^{lb}$

$W_{f1} = 190 \times 60 \times 75.5 + 2(190 + 60)\left(\frac{13\frac{1}{3} + 10}{2}\right)(30^{psf}) = 1{,}035{,}700^{lb}$

$W = \Sigma W_f = 938{,}200 + 3(1{,}000{,}700) + 1{,}014{,}450 + 1{,}044{,}450 + 1{,}035{,}700^{lb} = 7{,}169{,}900^{lb}$

eq 12.8-1 → $V = C_s W = .0133(7{,}169{,}900) =$

eq 12.8-11, 12.8-12 → $C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i^k}$, $F_x = C_{vx} V$

→ see attached spreadsheet

Page 2 of 3

X-Direction Loading

T=	0.838	s	
k=	1.169		
V_b=	122	kips	C_s= 0.017

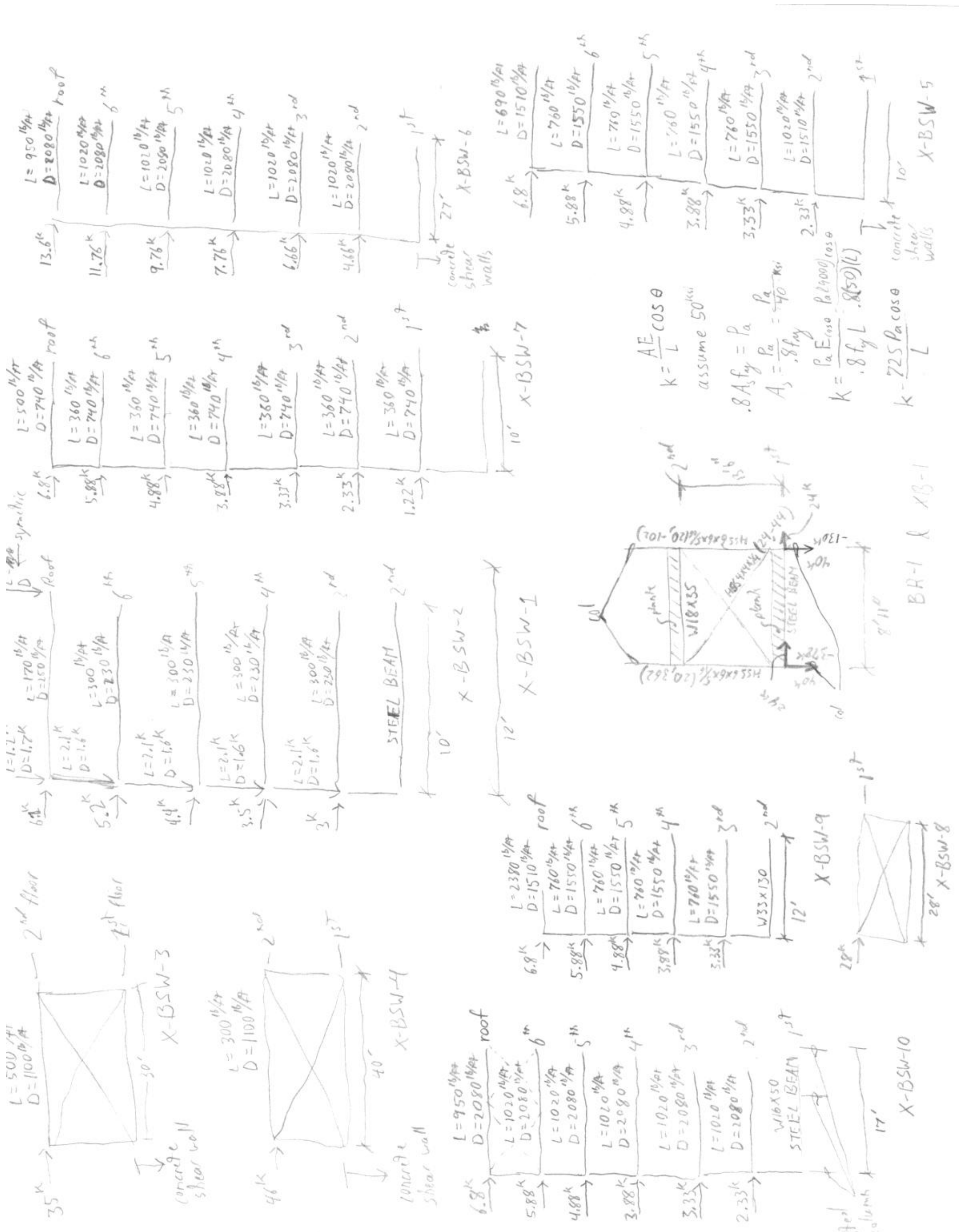
i	h_i	h	w	$w*h^k$	C_{VX}	f_i	V_i	By	5%By	Ax	M_z	M_{over}
	ft	ft	kips			kips	kips	ft	ft		k-ft	k-ft
7	9.33	71.75	1073.2	158605	0.268	33	33	60	3	1.0	98	2346
6	9.33	62.42	1000.7	125657	0.213	26	59	60	3	1.0	78	1617
5	9.33	53.08	1000.7	103980	0.176	21	80	60	3	1.0	64	1138
4	9.33	43.75	1000.7	82941	0.140	17	97	60	3	1.0	51	748
3	11.17	34.42	1014.45	63513	0.107	13	110	60	3	1.0	39	451
2	13.25	23.25	1044.45	41340	0.070	9	119	60	3	1.0	26	198
1	10	10	1035.7	15287	0.026	3	122	60	3	1.0	9	32
0	0	0	0	0	0.000	0	122	60	3	1.0	0	0
		Σ	7169.9	591323		122					366	6528

Y-Direction Loading

T=	0.838	s	
k=	1.169		
V_b=	346	kips	C_s= 0.0482

i	h_i	h	w	$w*h^k$	C_{VX}	f_i	V_i	Bx	5%Bx	Ax	M_z	M_{over}
	ft	ft	kips			kips	kips	ft	ft		k-ft	k-ft
7	9.33	71.75	1073.2	158605	0.268	93	93	190	10	1.0	881	6651
6	9.33	62.42	1000.7	125657	0.213	73	166	190	10	1.0	698	4584
5	9.33	53.08	1000.7	103980	0.176	61	227	190	10	1.0	577	3226
4	9.33	43.75	1000.7	82941	0.140	48	275	190	10	1.0	460	2121
3	11.17	34.42	1014.45	63513	0.107	37	312	190	10	1.0	353	1278
2	13.25	23.25	1044.45	41340	0.070	24	337	190	10	1.0	230	562
1	10	10	1035.7	15287	0.026	9	346	190	10	1.0	85	89
0	0	0	0	0	0.000	0	346	190	10	1.0	0	0
		Σ	7169.9	591323		346					3283	18510

Appendix D – Braced Wall Forces



Appendix E – Member Checks

Member Checks

X-BSW-7

from Marinoware tables

7.14k >> .3k

7.14k >> .27k

25k >> .49k

28.1k >> .67k

30k >> 1.5k

34.24k >> 1.67k

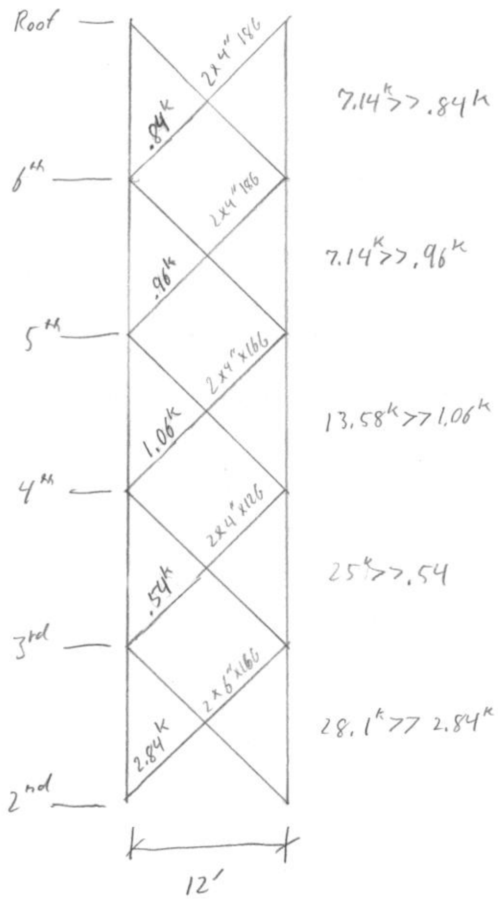
34.24k >> .56k

fram does not see forces anywhere close to design loads

error is most likly due to inaccurate member sizeing of flat straps.

it is possible that the forces specified on the frame are overdesigned to ensure for unknown member sizes

X-BSW-1



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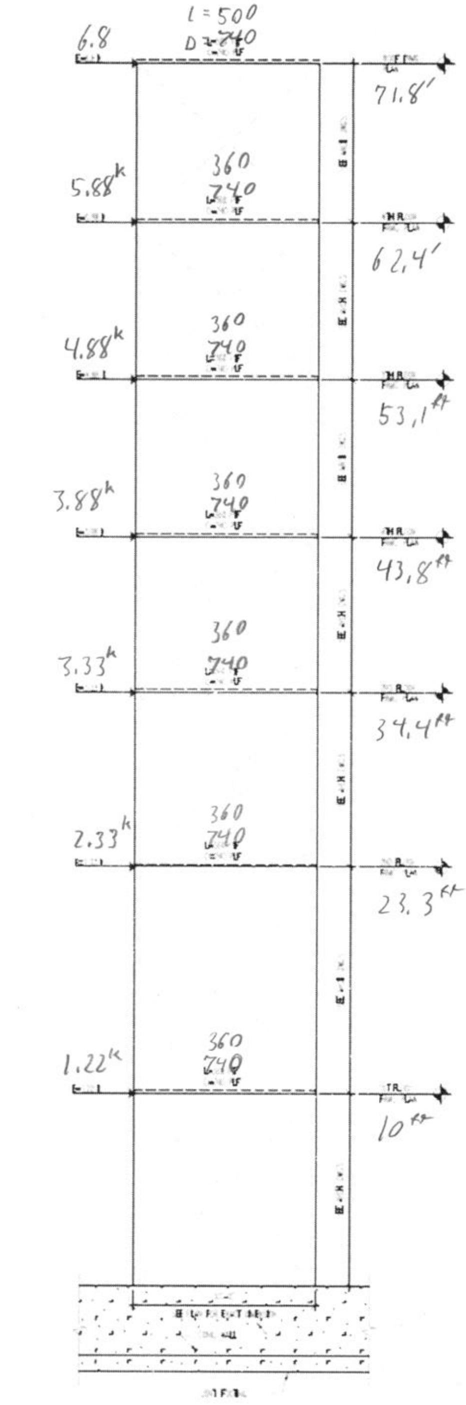


WALL BRACING @ X-BSW-7

$$\sum M_E = 6.8(71.8) + 5.88(62.4) + 4.88(53.1) + 3.88(43.8) + 3.33(34.4) + 2.33(23.3) + 1.22(10) = 1465^{k\text{-}ft}$$

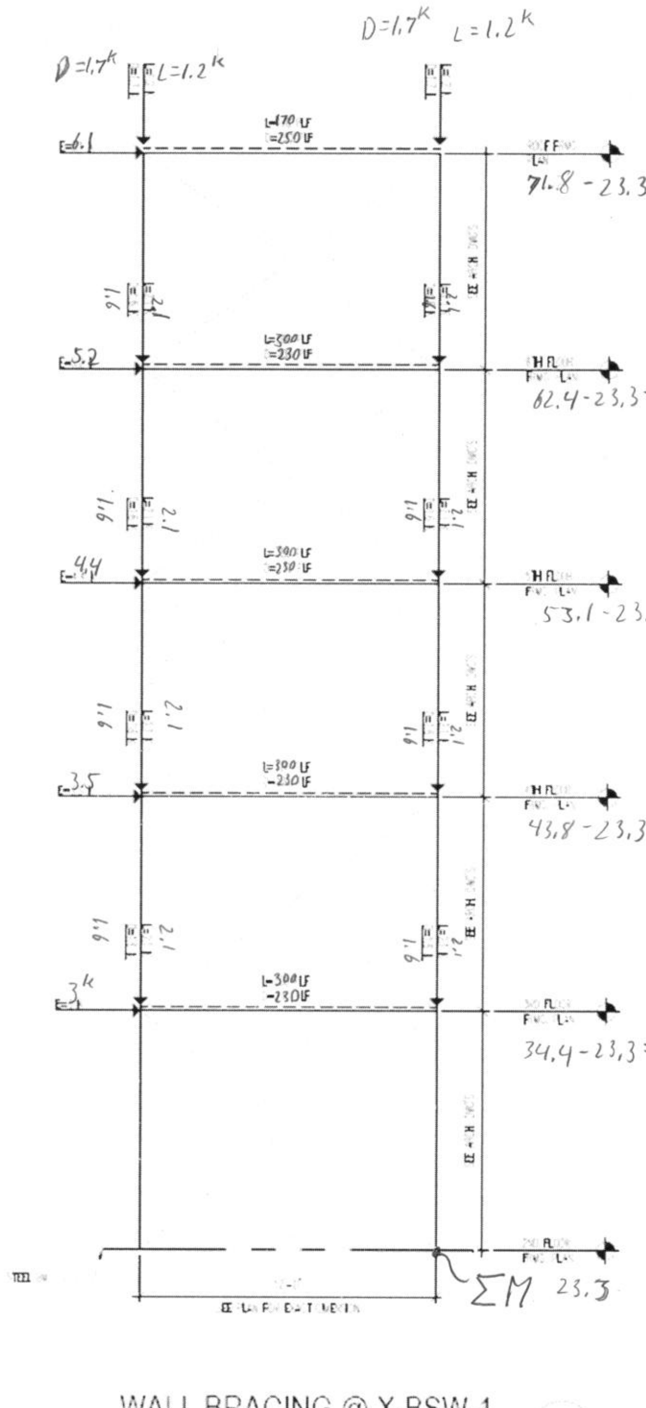
$$\sum M_{,9D} = 5^{ft}\big(6660(\underset{\#\text{stories}}{7})\big) = 233{,}100^{k\text{-}ft} \gg 1465^{k\text{-}ft}$$

→ overturning a problem

Soil Bearing →

$$q = \frac{P}{BL} \pm \frac{M6}{B^2L} = \frac{(7)1.2(740) + 1.6(6(360)+500) = 10.472^{K}\;(7)}{6^{ft}(23^{ft})} + \frac{1465^{k\text{-}ft}}{(36^{ft^2})(23^{ft})}$$

$$= 2301^{PSF} < 3000^{PSF}_{\text{allowed}}$$



WALL BRACING @ X-BSW-1

SCALE N.T.S S-310

$$\sum M_E = 6.1(48.5) + 5.2(31.9) + 4.4(29.8) + 3.5(20.5) + 3(11.1) = 697.9^{k\text{-}ft}$$

$$\sum M_{190} = 12(1.53 + 4(1.44)) + 6(2700 + 4(2484)) = 75{,}903^{k\text{-}ft} > 697.9^{k\text{-}ft}$$

moment imparted on diaphragm & beam which is transferred to other elements.

adjacent wall identical, and will combat overturning