SSM – St. Clare Health Center: Fenton, Missouri

Technical Report 4

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11-17-2014

Executive Summary

SSM St. Clare Health Center is a 420,000 square foot hospital located in a residential area of Fenton, Missouri. The building and parking areas sit on a 54 acre site, which was previously a 9-hole golf course with gently varying topography, large stands of trees, and a 3 acre pond. The hospital program contains a wide variety of medical use spaces, including 158 emergency supported inpatient beds, diagnostic and surgical services, administrative offices, dietary facilities, and pharmaceutical dispensaries. Budgeted at $226.8 million, the hospital was constructed with an Integrated Project Delivery method and came in well under budget at $223.5 million.

Structurally, the hospital is a composite steel frame building resting on massive concrete drilled piers which are connected by grade beams. The structure is broken up into three buildings (bed tower, surgery tower, and interventional care unit) isolated by expansion joints. These individual buildings each contain their own lateral force resisting systems which include special moment frames (SMF), special concentrically braced frames (SCBF), special reinforced concrete shear walls (SRCSW), and ordinary concentrically braced frames (OCBF).

HGA Architects and Engineers served as the primary architects and structural engineers on the project. They worked closely with the MEP engineers, KJWW, and the construction manager, Alberici Construction, through an integrated “Lean” project delivery contract that focused on improving coordination and quality by sharing project risks. The project began construction in September of 2006 and reached completion in March of 2009.

SSM St. Clare Health Center was designed in 2004 and uses the 2003 Edition of the International Building Code and ASCE 7-02 as a reference standard. Design loads were determined based on these codes, additional St. Louis County Codes and Ordinances, and practical engineering judgments. This report uses ASCE 7-10 as the reference for calculating wind and seismic lateral forces.

SSM St. Clare Health Center

Fenton, Missouri: St. Louis County

Rendering of full hospital complex

General Information

Full Height:	90 feet
Number of Stories:	6
Size:	427,000 gross square feet
Cost:	$223.5 million
Date of Construction:	Sept. 2006 – March 2009
Project Delivery Method:	Integrated "Lean" Project Delivery

Project Team

Owner:	SSM Health Care, St. Louis
Owner's Program Manager:	Hammes Company
Architect of Record:	HGA Architects and Engineers
Associate Architect:	Mackey Mitchel Associates
Structural Engineers:	HGA Architects and Engineers
MEP Engineers:	KJWW Engineering
Construction Manager:	Alberici Construction
Elevator Consultants:	Lerch, Bates & Associates Inc.

Bed tower façade

Architecture

- 158 emergency supported inpatient beds
- Diagnostic and surgical services
- Dietary facilities and pharmaceutical dispensaries
- Floor plans developed using Lean process principles classically used in manufacturing facilities.

Typical patient room

Structural Systems

- Framing
  - Steel framing, composite deck and lightweight concrete
  - Composite wide flange members
- Foundations
  - Slab on grade
  - Drilled concrete piers connected by grade beams
- Lateral System
  - special moment frames (SMF)
  - special concentrically braced frames (SCBF)
  - special reinforced concrete shear walls (SRCSW)
  - ordinary concentrically braced frames (OCBF)

Ground floor atrium

Mechanical Systems

- Fan coil units in each patient room fed by central boiler and chiller system
- VAV dedicated outside air for ventilation.

Lighting and Electrical Systems

- Back up generators designed to power the entire hospital for >90 minutes
- Ultrasonic ceiling sensors and infrared wall switch sensors for energy savings.

Construction

- Special noise control procedures implemented to minimize disturbance to local residential neighborhoods.

Photos compliments of HGA Architects and Engineers

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1 General Information

1.1 Purpose

This report details the process and results of a lateral system analysis conducted on SSM St. Clare Health Center. The document is intended to show a mastery of modeling techniques and model validation proceedures.

1.2 Scope

The major sections of this document discuss controlling load cases and the lateral systems that resist them. The building's location and relevant resource documents used in its design are also presented. The appendices to this document contain the original load calculations from HGA Architects and Engineers. The analysis focuses on the bed tower, labelled sections "A" and "B" on the record drawings.

1.3 Site Location and Plan

SSM St. Clare Health Center is located in Fenton, Missouri (St. Louis County) in a relatively open residential area. The site was previously a golf course, which provided open space and gently sloping terrain. Figure 1 shows the relative placement of the site in Missouri, while Figures 2 through 5 show the building's location on the site as dictated by zoning codes and city ordinances as well as its relative proximity to the New Madrid fault line, which has a great effect on the site's seismic characteristics and is of particular relevance to this report.

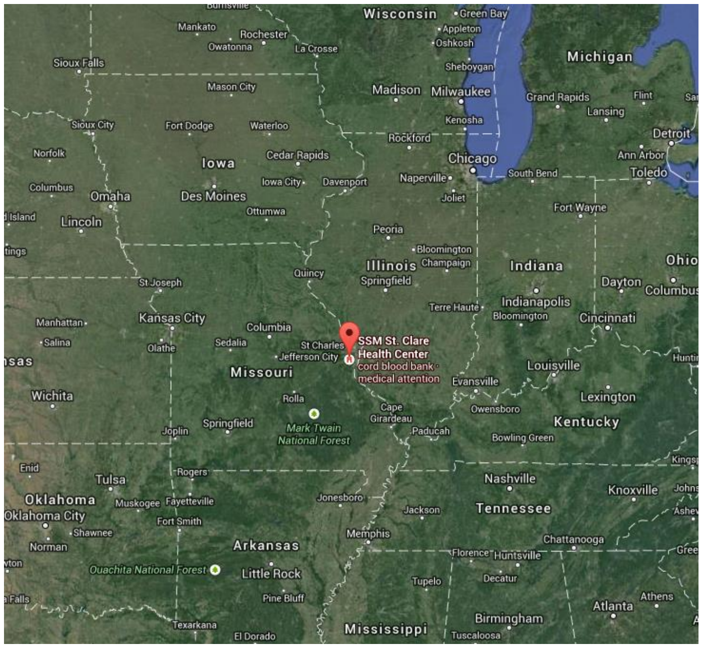


Figure 1: Building Location

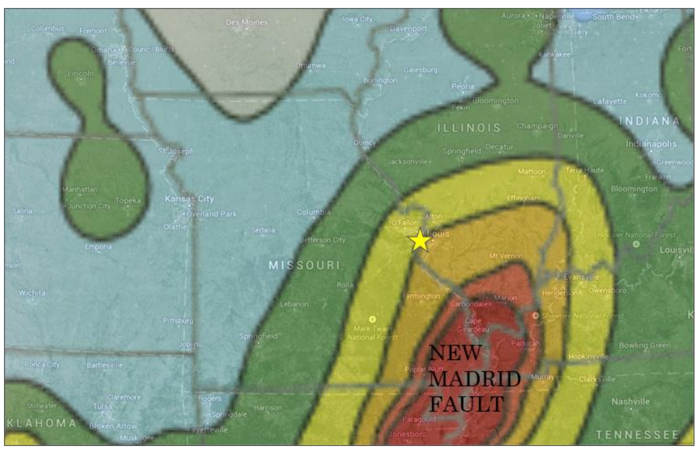


Figure 2: Location relative to New Madrid Fault Line

Figure 3: Rendering of SSM Health Center Complex

Figure 4: Original Site, Golf Course

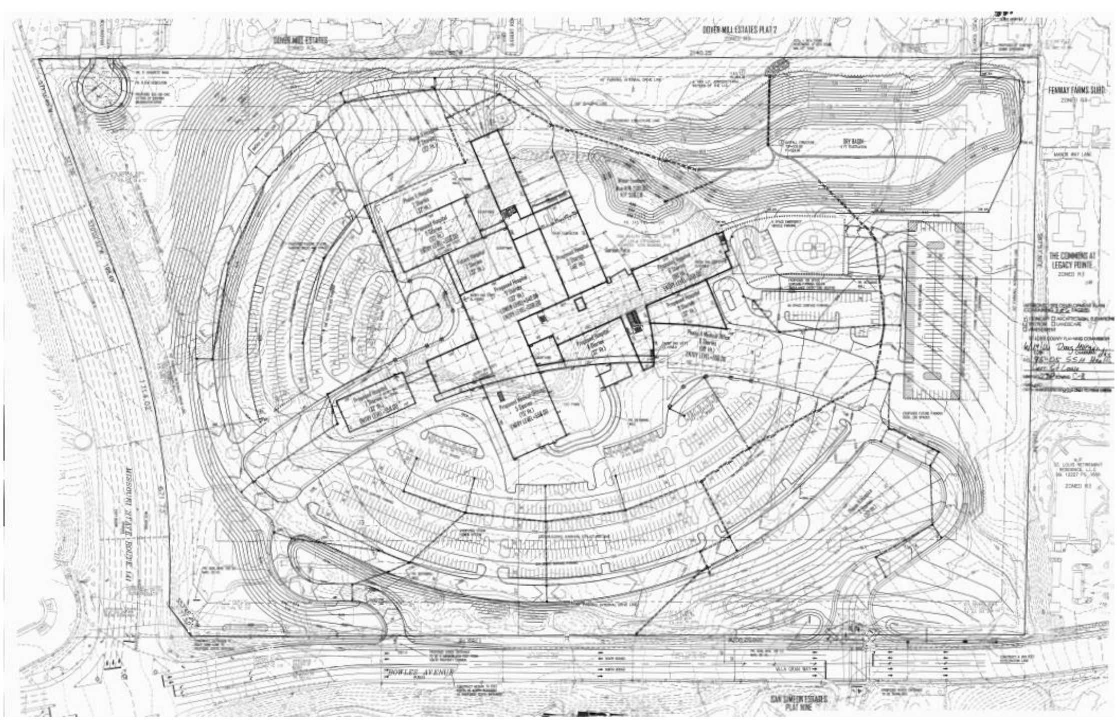


Figure 5: Building Orientation on Site

1.4 List of Preparatory Documents

- SSM St. Clare Health Center Site Development Plan
    - Produced by Stock & Associates Consulting Engineers Inc.
- SSM St. Clare Health Center Replacement Hospital Project Manual
    - CP-11 E/T Document Issuance
- IBC
    - 2003 Edition (as reference)
    - 2012 Edition (for further design studies)
- ASCE 7
    - ASCE 7-02 (as reference)
    - ASCE 7-10 (for further design studies and load calculations)
- ACI 318
    - ACI 318-11 (for modeling modifiers)
- Vulcraft Steel Deck Catalogue, 2008 Edition
- AISC Steel Manual 14th Edition

2 Gravity Loads

This section examines the dead, live, and snow loads used to design the building's gravity system. The original design calculations for gravity loads can be found in Appendix A. Dead loads are determined based on standard material weights, manufacturer data, and engineering experience. Future analyses of the building will focus on the bed tower. The majority of these loads are not present in the bed tower, but are listed here for comparison to the calculated loads, and as a reference.

2.1 Dead and Live Loads

Table 1: Typical Live Loads

Live Load	Value (psf)	Code Minimum (psf)
Operating Room	60	60
Offices	50	50
Private Rooms	40	40
Corridors (1st Floor)	100	100
Corridors (other)	80	80
Stairs and Exits	100	100
Equipment Rooms	125	125

Table 2: Typical Floor Dead Loads

Dead Load	Original Design Values (psf)	Thesis Calculated Values (psf)
Hospital Floor	60	64
Hospital Roof	78	70

Note in Figure 6 on the next page that the bed tower's floor plan is congested with corridors. This means that conservatively, a live load of 80 psf can be assumed for the entire floor area unless a higher load occurs. The highest load to occur in the hospital outside of a corridor is an operating room with movable partitions; however, 60 psf + 20 psf returns the load conservatively to 80psf. The entire floor slab is the same 64 psf "Hospital Floor" assembly.

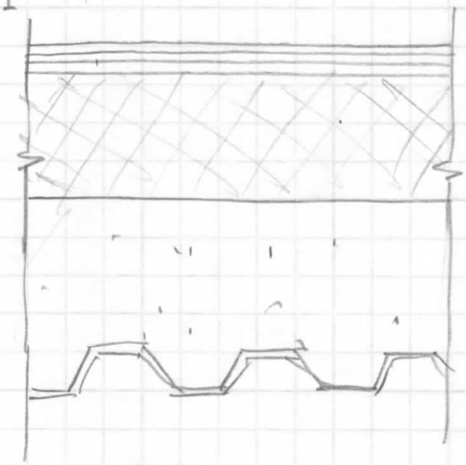
The presence of the 20 psf movable partition load becomes relevant when calculating the seismic mass of the building. ASCE 7-10 requires an additional 10 psf mass load to be applied to all diaphragms where a movable partition load is used. In the case of this report, the 60 psf + 20 psf case is assumed to control for its effect on the lateral loads (particularly seismic).

Gravity Loads

ROOF CONSTRUCTION DEAD:

SEE A431

MBIT-1



PENTHOUSE ROOF

3PLY: 1 psf
5 3/4" Ins: 1.5(5.75) = 8.625
3.5" Conc: } 48 psf
3" Deck }
FRAMING: 6 psf
MISC: 6 psf
70 psf

BEAMS	26 × 4 × 30' =	3120
GIRDERS	76 × 1 × 30' =	2280
AREA		900 sf
		= 6 psf

LOW ROOF

3PLY: 1 psf
11" Ins: 1.5(11) = 16.5 psf
3.5" Conc: 48 psf
3" Deck:
FRAMING: 6 psf
MISC: 6 psf
78 psf

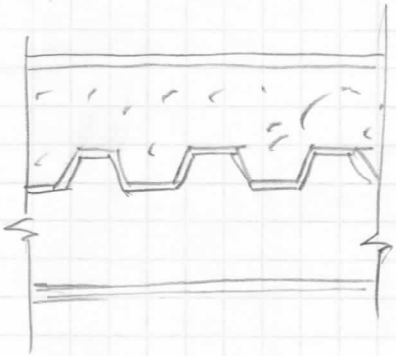
ROOF LIVE:

20 PSF (REQUIRED BY ASCE 7-10)

NOTE: LESS THAN SNOW LOAD

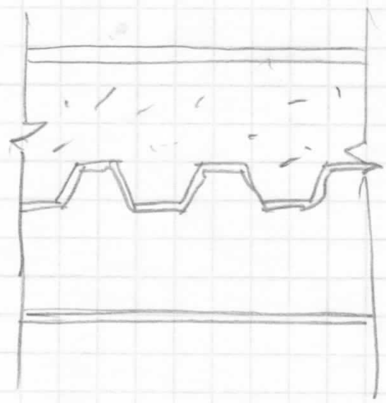
FLOOR CONSTRUCTION DEAD:

DECK 1



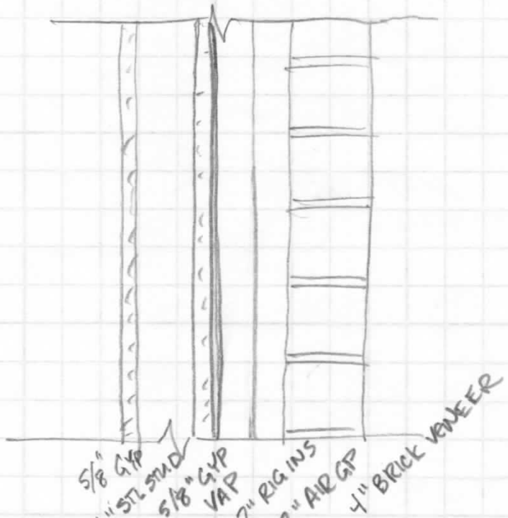
FINISHES:	2	psf
DECK:	48	psf
FRAMING:	7	psf
CEILING:	2	psf
MISC/EQUIP	5	psf
	64	psf

DECK 2:



FINISHES:	2	psf
DECK:	80	psf
FRAMING:	6	psf
CEILING:	2	psf
MISC/EQUIP:	5	psf
	95	psf

EXTERIOR ENCLOSURE



5/8" Gyp	:	2.75	psf
6" STL STUD	:	3	psf
5/8" GYP	:	2.75	psf
VAP	:	0.5	psf
2" RGD INS	:	3.0	psf
AIR	:	0	psf
BRICK	:	39	psf
		51	psf

Wall is supported at each floor by a steel angle. Vertical loads are transferred through the steel structure into the foundations.

Figure 6: Architectural Plan of Bed Tower (example typical 30'x30' bays in red)

2.2 Snow Loads

The following section contains example calculations of snow loads and snow drift loads on SSM St. Clare Health Center.

Snow Loads

- **FLAT ROOF SNOW LOADS**

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

Terrain Category : B
C_e : 1.0
C_t : 1.0
I_s : 1.2 → OC IV
SNOW LOAD : 20 psf

$$p_f = 0.7(1.0)(1.0)(1.2)(20)$$
$$= 16.8 \text{ psf}$$

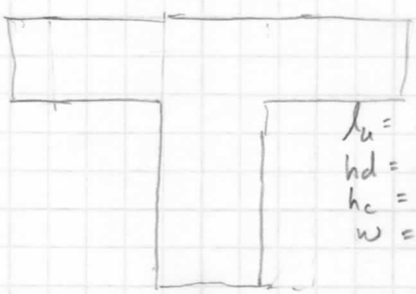
NOTE: RAIN ON SNOW SURCHARGE OF 5 PSF APPLIES.

$$p_m = I_s p_g \quad \text{for } p_g \leq 20 \text{ psf}$$
$$= \boxed{24 \text{ psf}}$$

- **DRIFTS ON LOWER ROOFS**

PENTHOUSE ROOF:

$\gamma = 0.13(20) + 14 = 16.6$ pcf
$h_b = 24/16.6 = 1.4457'$

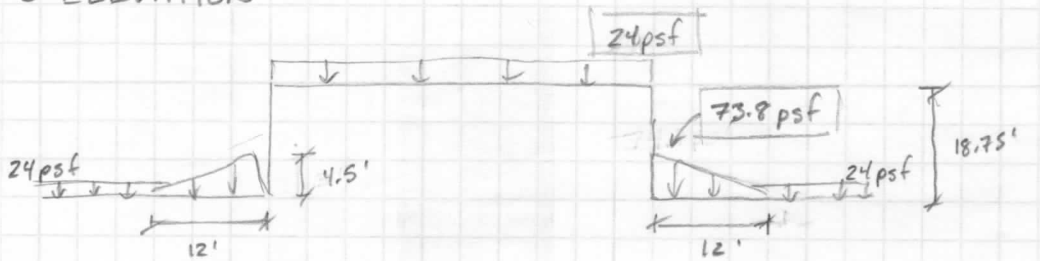


$l_u = 34.33' \sim 50'$
$hd = 2'$
$h_c = 16.75'$
$w = 8$

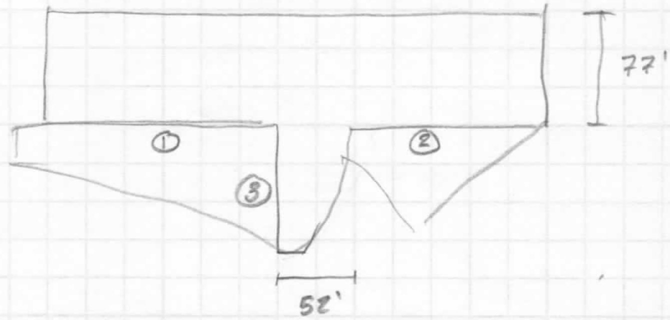
NOTE: N-S direction drifts truncated at 4'

$l_u = 97' \sim 100'$
$hd = 3'$
$h_c = 16.75'$
$w = 12$

N-S ELEVATION



GARDEN LEVEL ROOFS:



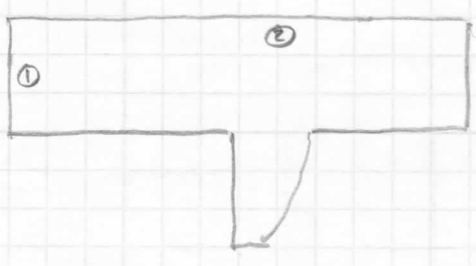
Area ① = Area ②

$l_u = 77' \sim 100'$
$h_d = 3'$
$h_c = 53'$
drift max $= \frac{24}{16.6}(16.6) + 3(16.6)$
$= \boxed{73.8 \text{ psf}}$

Area ③

$l_u = 52' \sim 50'$
$h_d = 2'$
$h_c = 54'$
drift max $= \frac{24}{16.6}(16.6) + 2(16.6)$
$= \boxed{57.2 \text{ psf}}$

PARAPET DRIFT



1) $l_u = 400$
$h_d = 6.5'$
$h_c = -4.3'$
drift max $= 24 + 6.5(0.75)16.6$
$= 104.93$ psf

2) $l_u = 100$
$h_d = 3'$
$h_c = -0.83'$
drift max $= 24 + 3(0.75)16.6$
$= 61.35$ psf

3 Lateral Loads

This section begins the discussion of lateral loads on SSM St. Clare Health Center. The primary loads reviewed were the wind load patterns and seismic load patterns. Appendix C contains the original design calculation values for design. The loads were determined the basic wind procedure. This report uses a newer version of the code and different calculation method as discussed below.

3.1 Wind Loads

The original structural design team used wind loads calculated by ASCE 7-02 methods; however, for simplification with software and comparison to current codes, the wind loads calculated in this report reference ASCE 7-10.

The building is located in Fenton, Missouri on an open site that was previously a golf course. The surrounding landscape consists mainly of trees and residential neighborhoods, making the site exposure category B. The risk category is IV for a hospital and the importance factor is 1.5. Based on wind maps from ASCE 7-10, the basic wind speed for the area is 115 mph. Important design parameters used in calculations can be seen in Table 3 below.

Table 3: Wind Design Parameters

Parameter	Symbol	Value
Occupancy Category	-	IV
Basic Wind Speed	V	115 mph
Exposure Category	-	B
Wind Directionality Factor	K_d	0.85
Importance Factor	I_e	1.5
Topographical Factor	K_{zt}	1.0
Gust Effect Factor	G	0.8205
Enclosure Classification	-	Enclosed

The building's reentrant corner geometries made calculation of wind loads challenging. To approximate length to width ratios, the main bed tower and the arm of the tower were separated into individual sections and calculated independently. The total forces were then added back together to achieve a story shear value. Tables 4 and 5 contain the raw wind load data for the two building portions. The graphics that follow show the data as applied to the building elevation in the East-West and North-South directions.

WIND LOADS

WIND DESIGN CRITERIA

RISK CATEGORY : IV
BASIC WIND SPEED : 115 mph (originally 90 in ASCE 7-02)
EXPOSURE CATEGORY : B
IMPORTANCE FACTOR : 1.15

$K_d = 0.85$
$K_{zt} = 1.0$
$G = ?$

APPROXIMATE NATURAL FREQUENCY:

$L_{eff} = 393$

① $h = 90 < 300$ ✓
② $h = 90 < 4(393) = 1572$ ✓

NOTE: THE PENTHOUSE ROOF IS ASSUMED TO BE THE MEAN ROOF HEIGHT

USE EQUATION 26.9-4

$$n_a = 75/h = 75/90 = 0.8333$$

CALCULATE G:

$$L_{\bar{z}} = 320\left(\frac{54}{33}\right)^{0.333} = 377.09$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{77+90}{377.09}\right)^{0.63}}} = 0.85214$$

$$V_{\bar{z}} = 0.45\left(\frac{54}{33}\right)^{0.25}\left(\frac{88}{60}\right)(115) = 85.844$$

$$N_1 = \frac{0.833(377.09)}{85.844} = 3.66$$

$$R_n = \frac{7.47(3.66)}{(1+10.3(3.66))^{5/3}} = 0.0617$$

$$R_h = \frac{1}{4.02} - \frac{1}{2(4.02)^2}\left(1-e^{-2(4.02)}\right) = 0.2179$$

$$R_B = \quad = 0.2486$$

$$R_L = \quad = 0.0554$$

$$R = \sqrt{\frac{1}{0.05}(0.0617)(0.2179)(0.2486)(0.53+0.47(0.0554))}$$

$$= 0.1928$$

$$g_r = \sqrt{2\ln(3600(0.8333))} + \frac{0.577}{\sqrt{2\ln((3600)(0.8333))}}$$

$$= 4.1458$$

$$I_z = 0.3\left(\frac{33}{54}\right)^{1/6} = 0.2764$$

(E-W) $$G_f = 0.925\left(\frac{1+1.7(0.2764)\sqrt{3.4^2(0.852)^2+(4.146)^2(0.1928)^2}}{1+1.7(3.4)(0.2764)}\right)$$

$$= 0.925\left(\frac{2.412}{2.597}\right)$$

$$= 0.8591$$

REPRESENTATIVE CALCULATION OF "P"

at $z = 90'$

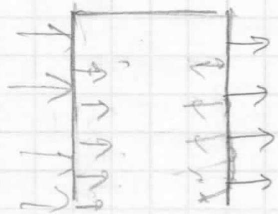
$$q_h = 0.00256(0.96)(1.0)(0.85)(115)^2$$
$$= 27.626$$

at $z = 58'$

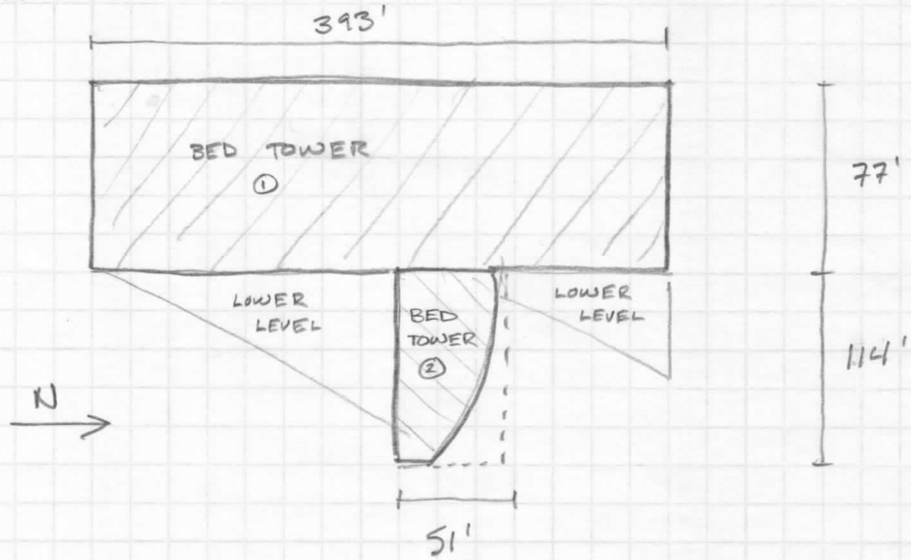
$$q_z = 0.00256(0.85)(1.0)(0.85)(115)^2$$
$$= 24.46$$

$$p = 24.46(0.8)(0.859) - (-27.626(0.18))$$
$$= 21.78$$

NOTE THAT THIS RESULT ASSUMES A NEGATIVE PRESSURE ON THE INTERIOR OF THE BUILDING



PLAN



Assumptions:

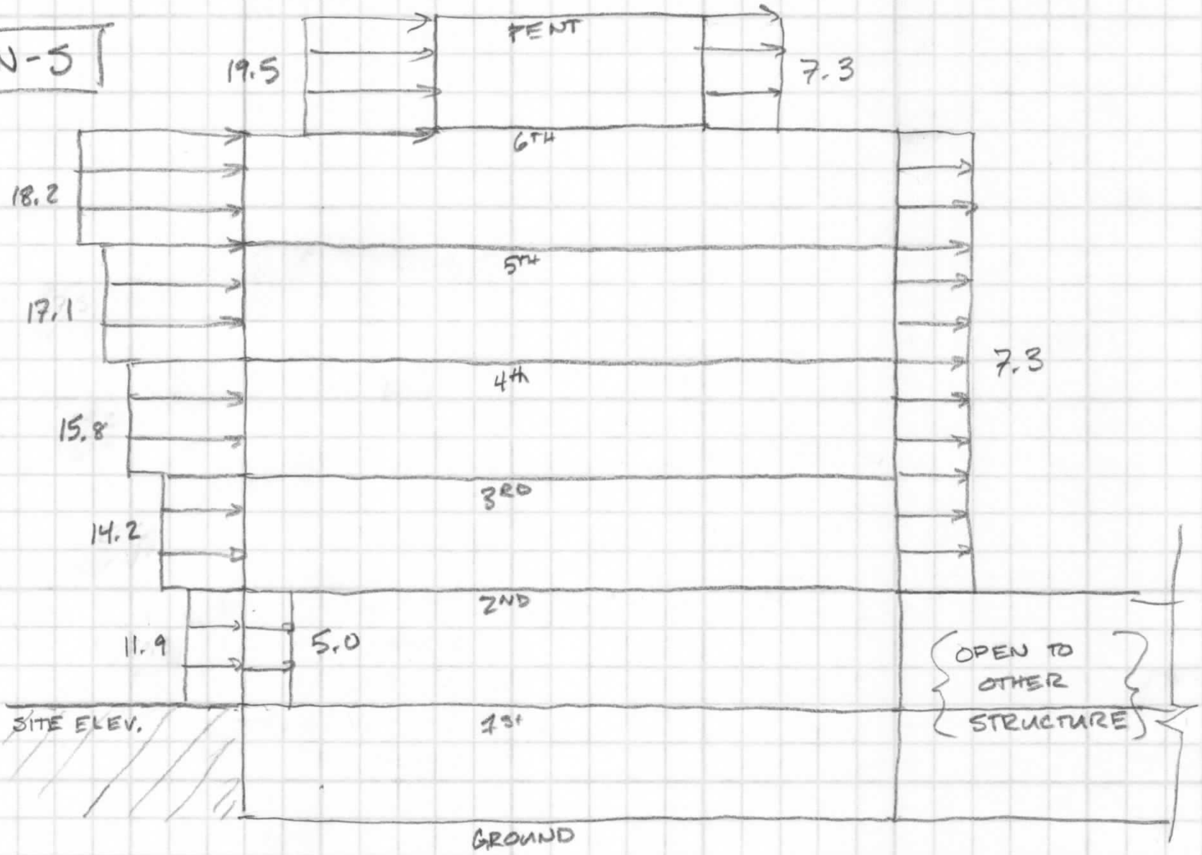
- lower level dimensions have minimal effect on tower loads
- E-W winds on Bed Tower section 2 create only "side" loads.
- Building has negative internal pressure.

REVISED

WIND LOADS

(VALUES IN PSF)

N-S



E-W

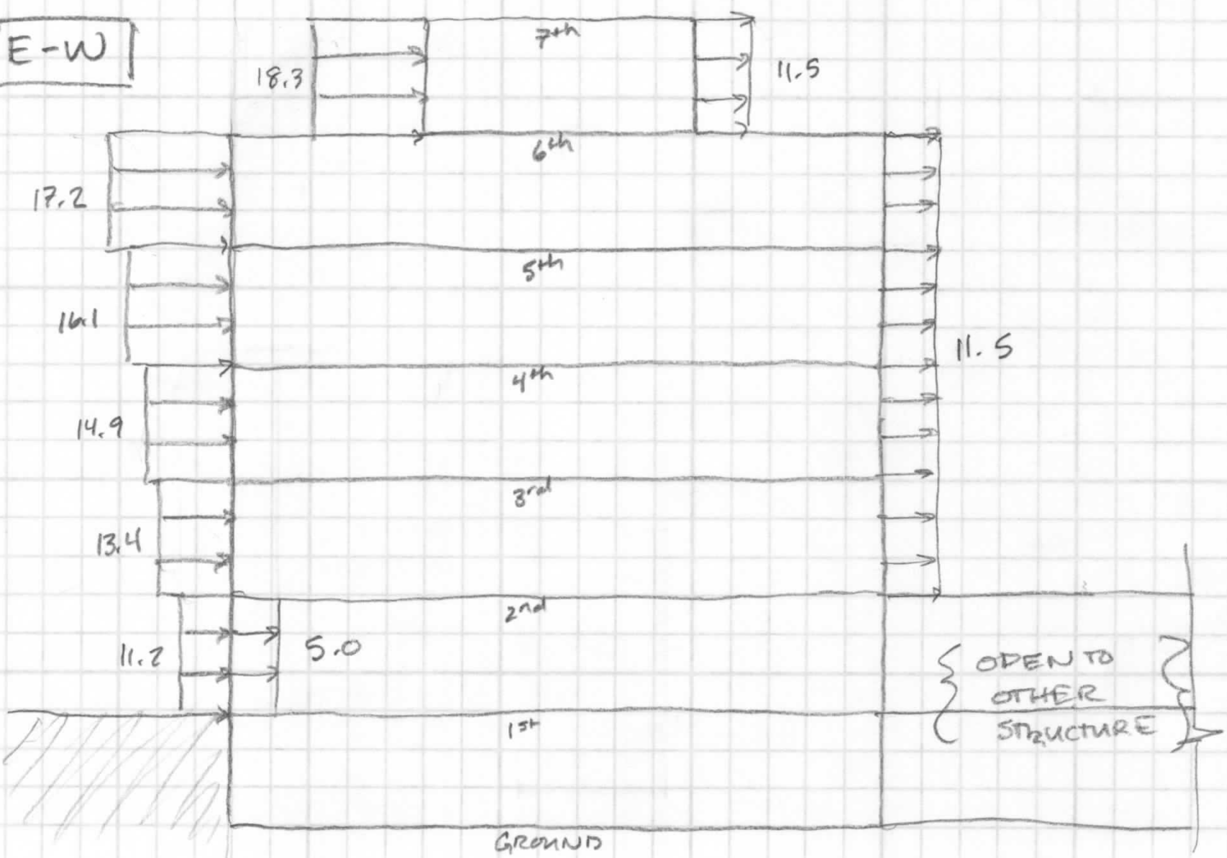


Table 4: Wind Loads in the East-West Direction

East-West Direction											
ARM			External Pressure			Internal Pressure					
Location	z (ft)	Story Height (ft)	qzGCp (psf)	Tributary Width	External Pressure (kip)	GCpi	qhGCpi (psf)	Tributary Width	Internal Pressure (kip)	Total Pressure (kip)	
Windward	-16	16.0	11.0	0.0	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	0	16.0	11.0	42.3	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	16	14.0	11.2	42.3	**7.1**	0.18	5.0	0.0	**0.0**	**7.1**	
	30	14.0	13.4	42.3	**7.9**	0.18	5.0	0.0	**0.0**	**7.9**	
	44	14.0	14.9	42.3	**8.8**	0.18	5.0	0.0	**0.0**	**8.8**	
	58	14.0	16.1	42.3	**9.5**	0.18	5.0	0.0	**0.0**	**9.5**	
	72	18.8	17.2	42.3	**11.9**	0.18	5.0	0.0	**0.0**	**11.9**	
	90.75	0.0	18.3	42.3	**7.3**	0.18	5.0	0.0	**0.0**	**7.3**	
Leeward	90.75	90.8	-11.5	42.3	**43.9**	0.18	5.0	0.0	**0.0**	**43.9**	
Parapet WW	93	2.2	34.6	42.3	**3.2**	1.5	41.5	0.0	**0.0**	**3.2**	
Parapet LW	93	2.2	23.1	42.3	**2.1**	-1	-27.7	0.0	**0.0**	**2.1**	
TOWER			External Pressure			Internal Pressure					
Windward	-16	16.0	11.4	0.0	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	0	16.0	11.4	224.8	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	16	14.0	11.6	224.8	**39.0**	0.18	5.0	150.0	**11.2**	**50.2**	
	30	14.0	13.9	374.8	**72.7**	0.18	5.0	0.0	**0.0**	**72.7**	
	44	14.0	15.5	374.8	**81.1**	0.18	5.0	0.0	**0.0**	**81.1**	
	58	14.0	16.7	374.8	**87.7**	0.18	5.0	0.0	**0.0**	**87.7**	
	72	18.8	17.8	374.8	**109.1**	0.18	5.0	0.0	**0.0**	**109.1**	
	90.75	0.0	19.0	71.0	**12.6**	0.18	5.0	0.0	**0.0**	**12.6**	
Leeward	90.75	90.8	-4.8	374.8	**161.6**	0.18	5.0	0.0	**0.0**	**161.6**	
Parapet WW	93	2.2	41.5	374.8	**33.7**	1.5	41.5	0.0	**0.0**	**33.7**	
Parapet LW	93	2.2	27.7	374.8	**22.5**	-1	-27.7	0.0	**0.0**	**374.8**	
									Base Shear:	1085.2	kips

Table 5: Wind Loads in the North-South Direction

North-South Direction											
ARM			External Pressure			Internal Pressure					
Location	z (ft)	Story Height (ft)	qzGCp (psf)	Tributary Width	External Pressure (kip)	GCpi	qhGCpi (psf)	Tributary Width	Internal Pressure (kip)	Total Pressure (kip)	
Windward	-16	16.0	11.7	0.0	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	0	16.0	11.7	113.8	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	16	14.0	11.9	113.8	**20.3**	0.18	5.0	0.0	**0.0**	**20.3**	
	30	14.0	14.2	113.8	**22.6**	0.18	5.0	0.0	**0.0**	**22.6**	
	44	14.0	15.8	113.8	**25.2**	0.18	5.0	0.0	**0.0**	**25.2**	
	58	14.0	17.1	113.8	**27.3**	0.18	5.0	0.0	**0.0**	**27.3**	
	72	18.8	18.2	113.8	**34.0**	0.18	5.0	0.0	**0.0**	**34.0**	
	90.75	0.0	19.5	67.0	**12.2**	0.18	5.0	0.0	**0.0**	**12.2**	
Leeward	90.75	90.8	-7.3	113.8	**75.4**	0.18	5.0	0.0	**0.0**	**75.4**	
Parapet WW	93	2.2	0.0	113.8	**0.0**	1.5	41.5	0.0	**0.0**	**0.0**	
Parapet LW	93	2.2	0.0	113.8	**0.0**	-1	-27.7	0.0	**0.0**	**0.0**	
TOWER			External Pressure			Internal Pressure					
Windward	-16	16.0	10.5	0.0	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	0	16.0	10.5	0.0	**0.0**	0.18	5.0	0.0	**0.0**	**0.0**	
	16	14.0	10.7	0.0	**0.0**	0.18	5.0	77.3	**5.8**	**5.8**	
	30	14.0	12.7	77.3	**13.8**	0.18	5.0	0.0	**0.0**	**13.8**	
	44	14.0	14.2	77.3	**15.4**	0.18	5.0	0.0	**0.0**	**15.4**	
	58	14.0	15.4	77.3	**16.7**	0.18	5.0	0.0	**0.0**	**16.7**	
	72	18.8	16.4	77.3	**20.7**	0.18	5.0	0.0	**0.0**	**20.7**	
	90.75	0.0	17.5	31.0	**5.1**	0.18	5.0	0.0	**0.0**	**5.1**	
Leeward	90.75	90.8	-10.9	77.3	**76.7**	0.18	5.0	0.0	**0.0**	**76.7**	
Parapet WW	93	2.2	33.0	77.3	**5.5**	1.5	41.5	0.0	**0.0**	**5.5**	
Parapet LW	93	2.2	22.0	77.3	**3.7**	-1	-27.7	0.0	**0.0**	**77.33**	
									Base Shear:	454.1	kips

3.2 Seismic Loads

Seismic design loads were also originally calculated using ASCE 7-02; however, this report uses ASCE 7-10 load criteria.

The site is located within a New Madrid Fault affected area, and the site soil conditions are relatively poor. This combination of factors places the structure in a seismic design category D. Other relevant seismic design parameters are shown below in Table 6.

Table 6: Seismic Design Parameters

Parameter	Symbol	Value
Occupancy Category	-	IV
Site Class	-	D
Seismic Design Category	-	D
Short Period Spectral Response Acceleration	S_S	0.414
One Second Spectral Response Acceleration	S_1	0.163

Seismic loads have been calculated based on building mass. For simplification, the building mass consists of 4 main components: diaphragms and framing, building enclosure, concrete shear walls, and superimposed movable partitions. The combined mass of these elements has been calculated at each story for a given story height and diaphragm area. The forces are then given an eccentricity of 5% to either direction to account for accidental torsions. The forces applied at each story, calculated manually, are shown below.

The building has several types of structural irregularities that should be noted for future design considerations. According to ASCE 7-10 Tables 12.3-1 and 12.3-2, the building has reentrant corner irregularities and out-of-plane offset irregularities (at the penthouse floor) in the plan dimension, and a weight irregularity and in-plane discontinuity irregularity (brace frame at grid Ra) in the vertical dimension. These irregularities influence the amplification of design loads for the design of connections, collectors, and other special elements. For the sake of this report, design forces are assessed from the 3D computer model after validation, and overstrength is not applied. The C_d factors for drift are applied where appropriate. For more information on assumptions related to seismic factors, see the "Computer Modeling" section.

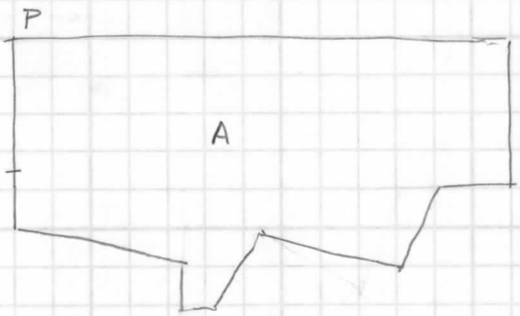
Table 7: Seismic Design Loads

Story	Dia.	Wx (kips)	Hx (ft)	k	Hxk	Wx*Hxk	Cvx	Fx
First Floor	1	5076.90	16.00	1.23	30.66	155682.65	0.05	77.89
Second Floor	2	4987.50	32.00	1.23	72.16	359902.54	0.13	180.07
Third Floor	3	3417.40	46.00	1.23	112.95	385998.01	0.13	193.12
Fourth Floor	4	3417.40	60.00	1.23	156.80	535862.22	0.19	268.10
Fifth Floor	5	3147.40	74.00	1.23	203.15	639381.21	0.22	319.90
Roof	6	3141.90	88.00	1.23	251.60	790509.88	0.28	395.51
Penthouse Roof	7	640.00	106.00	1.23	316.59	202619.66	0.07	101.37
		23828.5	422	7.4	827.3	2867336.5	1	1434.6

SEISMIC MASS CALCULATION

FACADE AREAS

1, 2 FLOORS:



PLAN VIEW
NTS

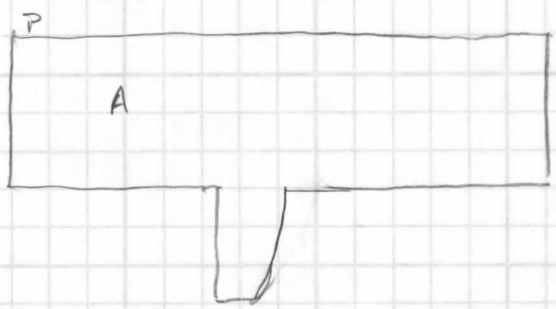
$H = 16'$

$$P = (77.333')(2) + (41') + (417') + (42') + (108.5') + (133.6') + (86.5') + (11.33') + (22.6') + (166')$$

$$= 1183.2'$$

$A = 56169.38 \text{ ft}^2$ (FROM PROGRAM)

3, 4, 5, 6 FLOORS



PLAN VIEW
NTS

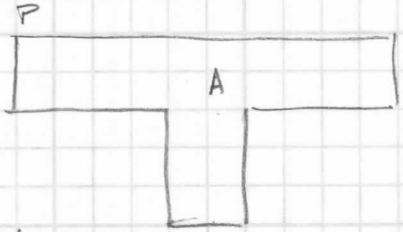
$H = 14'$

$$P = (417') + (214.7') + (158.33') + (113.7') + (11.33) + (86.5') + 77.33(2)$$

$$= 1078.19'$$

$A = 36158.84 \text{ ft}^2$ (FROM PROGRAM)

7 FLOOR



$H = 18.802'$

$P = (33') + (90')(2) + (113') + 67(2) + 2(31)$
$= 522'$

$A = 31(113) + 67(33)$
$= 5714 \text{ sf}$

- FACADE MASS:

ASSUME 30% FENESTRATION @ 25 lbs/sf

1.) $m = 1183.2(16)(0.3(25) + 0.7(51))$
$= 817827.84$ lbs
$= 818$ kips

2) $m = (818/2) + (652/2) = 735$ kips

3,4,5,) $m = 1078.19(14)(0.3(25) + 0.7(51))$
$= 652089.3$ lbs
$= 652$ kips

6) $m = 522(51)\left(\frac{18.08}{2}\right) + (652/2) = 566$ kips

7) $m = 522(51)\left(\frac{18.08}{2}\right) = 240$ kips

- FLOOR MASS

1,2) $m = 56169.38(64 + 10)$ USED FOR PARTITION LOADS PER ASCE 7.10
$= 4156534.12$
$= 4156.5$ kips

3,4,5) $m = 36158.84(64 + 10)$
$= 2675754.16$ lbs
$= 2675.75$ kips

6) $m = 36158.84(70 \text{ psf})$
$= 2531118.8$ lbs
$= 2531.118$ kips

7) $m = 5714(70) = 400$ kips

- SHEAR WALL MASS

ASSUME 30% OPENING

1) $m = 150\left(\frac{16}{12}\right)(32)(16)$
$= 102400$ lbs
$= 102.4$ kips

2) $m = 150\left(\frac{16}{12}\right)(32)\left(\frac{16+14}{2}\right) = 96$ kips

3,4,5) $m = 150\left(\frac{16}{12}\right)(32)(14)$
$= 89.6$ kips

6) $m = \frac{89.6}{2} = 44.8$ kips

7) $m = 0$

- FRAMING MASS

~~1,2) $m = 56169.38(10)$~~
~~$= 561.7$ k~~

~~3,4,5,6) $m = 36158.84(10)$~~
~~$= 361.6$ k~~

~~7) $M = 5714(10)$~~
~~$= 57.1$ k~~

SEISMIC MASS: $M = m_{facade} + m_{floor} + m_{shear\ wall}$

7 $m_7 = 240 + 400$ (112)
$= 640$

6 $m_6 = 566\ k + 2531.118\ k + 44.8\ k$ (2.69)
$= 3141.9\ k$

5 $m_5 = 3417.4\ k$ (2.93)

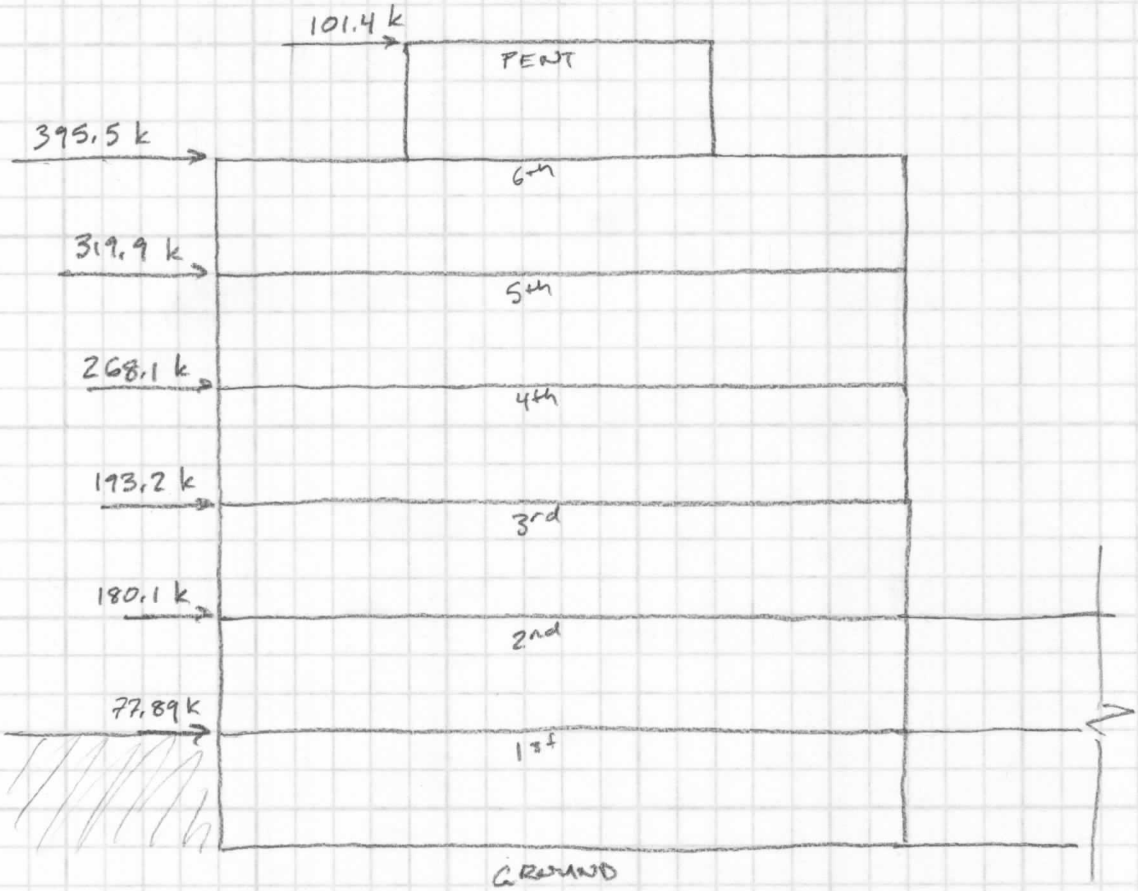
4 $m_4 = 3417.4\ k$ (2.93)

3 $m_3 = 652 + 2675.75 + 89.6$ (2.93)
$= 3417.4\ k$

2 $m_2 = 735 + 4156.5 + 96$ (2.75)
$= 4987.5\ k$

1 $m_1 = 818 + 4156.5 + 102.4\ k$ (2.80)
$= 5076.9\ k$

SEISMIC LOADS



OVERTURNING

$$M_o = 77.89(16) + 180.1(32) + 193.2(46) + 268.1(60) + 319.9(74) + 395.5(88) + 101.4(106)$$

$$= 101204.4 \text{ ft k}$$

MOMENT CAUSES COMPRESSION AND POTENTIALLY UPLIFT. DUE TO THE SIZE OF THE DRILLED PIERS, UPLIFT WILL NOT BE CONSIDERED IN THIS REPORT.

ETABS INCLUDES OVERTURNING FORCES IN ITS CALC. OF AXIAL FORCE IN COLUMNS. COLUMN CHECK WILL THUS ACCOUNT FOR OVERTURNING.

4 COMPUTER MODELING

This section details the process and results of creating a lateral system model of SSM St. Clare Health Center's patient bed tower. The three dimensional model was constructed in Etabs, while two dimensional rigidity checks were conducted in SAP 2000.

4.1 MODEL DEVELOPMENT

Each lateral element was first constructed in two dimensions in SAP 2000 to verify that the modeling technique and modeling assumtions were effective. Then, the same technique was used to model the elements in Etabs along a three dimensional grid. Rigid diaphragms were created to represent the 3 inch steel deck with 3.5 inches of concrete topping.

Loads were applied to the model by automatic generation. These automatically generated loads were then compared to manual calculations to verify the software's method and assumptions.

Several models were generated with different assumptions for base fixity; from fixed to pinned columns and pins at the base versus diaphragm constraints at the first level (ground level). It was determined that the foundation walls did not have sufficient connection to the first floor diaphragm to warrant a pinned connection at the first level. The final model exhibited the most realistic behavior and most similar loading to hand calculations. Figures 7 through 10 are depictions of the final Etabs model.

Further refinement can be conducted on seismic loading. Also, a modal analysis would be interesting for studying the effects of mass participation in torsional modes.

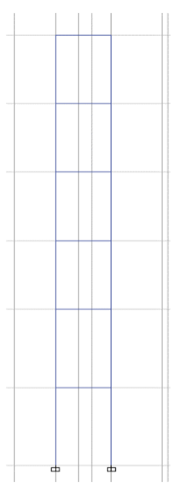


Figure 9: Example SMF

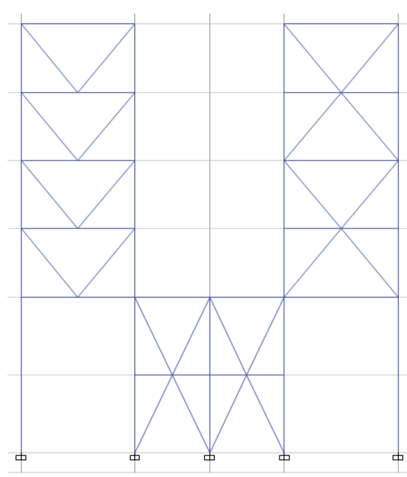


Figure 7: Example SCBF

Figure 8: Example SRCSW

4.2 ASSUMPTIONS

Table provides a list of major assumptions associated with each component of the lateral system.

Lateral Component	Element Type	Assumptions	Modifiers
Special Moment Frames	Beam	• Beams are fixed-fixed • Composite action is negligible • Full moment fixity at base (pinned with fixed moment in-plane)	• Self-weight set to zero • Insertion point 8 (top center) with 6.5 in. vertical offset for beams • Nodes at similar levels constrained to diaphragm
Special Concentrically Braced Frames	Beam	• Beams are pinned-pinned • Braces are pinned-pinned • Columns are fixed-fixed • Composite action is negligible • Full moment fixity at base (pinned with fixed moment in-plane) • Braces automatically adjust to insertion point to remain concentric	• Self-weight set to zero • Insertion point 8 (top center) with 6.5 in. vertical offset for beams • Moment releases at both ends of beams and braces • Nodes at similar levels constrained to diaphragm
Special Reinforced Concrete Shear Walls	Thin-Shell	• 16 in. thick shell element • Shear wall extends below base to foundations where it is fixed at the sub-base level. • Shell method is more accurate than frame method. • No out-of-plane rigidity • All floors are cracked (designed as “special” reinforced for ductility)	• Self-weight set to zero • Moment and shear modifiers out-of-plane set to zero. • Moment in-plane set to 0.7 per ACI 318-11
Diaphragms	N/A	• Rigid diaphragm • Continuous over entire level. • Center of diaphragm mass is center of story mass • Penthouse loads applied at 6^{th} story COM. • Mass distributed uniformly	• Self-weight set to zero • Superimposed mass equal to total of floor assembly, facades, shear walls, and misc. applied uniformly

4.3 Model Validation

Validation of the lateral Etabs model included COR/COM checks, wind load comparisons, seismic load comparisons, and torsional behavior comparisons.

4.3.1 Center of Rigidity and Center of Mass Checks

The center of rigidity was checked using stiffness values from two dimensional SAP 2000 models of each of the lateral components. Single kip loads were applied to each story of the lateral resisting elements and total deflection was measured at that story. Relative stiffness for each story was calculated, and from these relative stiffnesses the centers of rigidity of each story were evaluated as shown in Tables 8 and 9. The COR values are compared to the model generated values and mostly agree to within 5%, indicating that the model is using correct stiffness values.

Table 8: Center of Rigidity in Model Global Y (N-S axis)

Element	Story	Disp.	Rel. K (k/in)	Dist X	Dist Y	Ri*Xi	Sum(Ri*Xi)	Sum(Ri)	COR	Model Values	% Error
1B-smf	1	0.001680	595.24	0.000	191.165	113788.6905	487622.2108	3518	138.6012	137.71	0.65
1A-smf	1	0.001130	884.96	0.000	191.165	169172.5664					
4B-smf	1	0.001680	595.24	0.000	113.749	67707.55952					
4A-smf	1	0.001130	884.96	0.000	113.749	100662.5664					
8-smf	1	0.009190	108.81	0.000	0.000	0					
5B-smf	1	0.003560	280.90	0.000	80.832	22705.61798					
5A-smf	1	0.005950	168.07	0.000	80.832	13585.21008					
1B-smf	2	0.007110	140.65	0.000	191.165	26886.77918	118917.7762	870	136.6731	134.6118	1.53
1A-smf	2	0.004540	220.26	0.000	191.165	42106.82819					
4B-smf	2	0.007110	140.65	0.000	113.749	15998.41069					
4A-smf	2	0.004540	220.26	0.000	113.749	25054.77974					
8-smf	2	0.025960	38.52	0.000	0.000	0					
5B-smf	2	0.014040	71.23	0.000	80.832	5757.264957					
5A-smf	2	0.025960	38.52	0.000	80.832	3113.713405					
1B-smf	3	0.013430	74.46	0.000	191.165	14234.17722	59264.31973	408	145.3599	135.763	7.07
1A-smf	3	0.008340	119.90	0.000	191.165	22921.46283					
4B-smf	3	0.013430	74.46	0.000	113.749	8469.746835					
4A-smf	3	0.008340	119.90	0.000	113.749	13638.93285					
8-smf	3	0.052690	18.98	0.000	0.000	0					
1B-smf	4	0.020370	49.09	0.000	191.165	9384.634266	39518.98028	272	145.1722	138.02	5.18
1A-smf	4	0.012420	80.52	0.000	191.165	15391.70692					
4B-smf	4	0.020370	49.09	0.000	113.749	5584.128621					
4A-smf	4	0.012420	80.52	0.000	113.749	9158.510467					
8-smf	4	0.076880	13.01	0.000	0.000	0					
1B-smf	5	0.027780	36.00	0.000	191.165	6881.389489	29147.27257	201	145.1857	139.952	3.74
1A-smf	5	0.016780	59.59	0.000	191.165	11392.43147					
4B-smf	5	0.027780	36.00	0.000	113.749	4094.62563					
4A-smf	5	0.016780	59.59	0.000	113.749	6778.825983					
8-smf	5	0.104440	9.57	0.000	0.000	0					
1B-smf	6	0.036170	27.65	0.000	191.165	5285.181089	22507.28806	155	145.2776	141.6316	2.57
1A-smf	6	0.021660	46.17	0.000	191.165	8825.715605					
4B-smf	6	0.036170	27.65	0.000	113.749	3144.835499					
4A-smf	6	0.021660	46.17	0.000	113.749	5251.555863					
8-smf	6	0.137070	7.30	0.000	0.000	0					

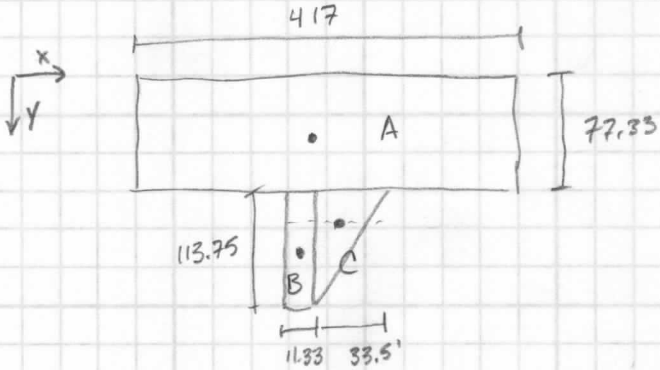
Table 9: Center of Rigidity in Model Global X (E-W axis)

Element	Story	Disp.	Rel. K (k/i	Dist X	Dist Y	Ri*Xi	Sum(Ri*Xi	Sum(Ri)	COR	Model Values	% Error
S-srcsw	1	0.000244	4092	165.668	0.000	677856	2437450	14145	172.3206	172.1045	0.13
N-srcsw	1	0.000217	4608	194.683	0.000	897157.6					
Aa1-scbf	1	0.000628	1592	0.000	0.000	0					
Aa3-scbf	1	0.000580	1724	0.000	0.000	0					
Ra-scbf	1	0.000470	2129	405.000	0.000	862436.1					
S-srcsw	2	0.000559	1789	165.668	0.000	296364.9	1092755	6252	174.7738	176.896	1.20
N-srcsw	2	0.000509	1964	194.683	0.000	382406.6					
Aa1-scbf	2	0.001270	787	0.000	0.000	0					
Aa3-scbf	2	0.001450	690	0.000	0.000	0					
Ra-scbf	2	0.000978	1022	405.000	0.000	413983.4					
S-srcsw	3	0.001010	990	165.668	0.000	164027.7	692366.4	3594	192.6556	186.5426	3.28
N-srcsw	3	0.000908	1101	194.683	0.000	214385.2					
Aa1-scbf	3	0.002770	361	0.000	0.000	0					
Aa3-scbf	3	0.002730	366	0.000	0.000	0					
Ra-scbf	3	0.001290	775	405.000	0.000	313953.5					
S-srcsw	4	0.001240	806	165.668	0.000	133603.2	475433.9	2487	191.139	190.4143	0.38
N-srcsw	4	0.001500	667	194.683	0.000	129788.8					
Aa1-scbf	4	0.004210	238	0.000	0.000	0					
Aa3-scbf	4	0.003950	253	0.000	0.000	0					
Ra-scbf	4	0.001910	524	405.000	0.000	212041.9					
S-srcsw	5	0.002690	372	165.668	0.000	61586.62	283959.8	1447	196.2195	191.9442	2.23
N-srcsw	5	0.002340	427	194.683	0.000	83197.95					
Aa1-scbf	5	0.006570	152	0.000	0.000	0					
Aa3-scbf	5	0.006570	152	0.000	0.000	0					
Ra-scbf	5	0.002910	344	405.000	0.000	139175.3					
S-srcsw	6	0.004090	244	165.668	0.000	40505.62	188860.8	968	195.1639	192.6691	1.29
N-srcsw	6	0.003510	285	194.683	0.000	55465.3					
Aa1-scbf	6	0.009860	101	0.000	0.000	0					
Aa3-scbf	6	0.009300	108	0.000	0.000	0					
Ra-scbf	6	0.004360	229	405.000	0.000	92889.91					

The center of mass was approximated using geometric forms as shown on the following page. Checking one floor was sufficient to verify the model’s accuracy, as the values were nearly identical.

C.O.M. ROUGH CHECK

ASSUMPTIONS: MASS IS EVENLY DISTRIBUTED ACROSS FLOOR
CURVED SECTION CAN BE APPROX. AS TRIANGLE



SHAPE	AREA (ft^2)	CENT. X	CENT. Y	$A_i X_i$	$A_i Y_i$
A	32248	208.5'	38.7'	6723708	1247997
B	1289	164'	134.2'	211396	172983.8
C	1905	180.8	115.2'	344424	219456
TOTALS:	35442			7279528	1640436.8

$x = 205.4'$
$y = (46.3) \rightarrow 191.165 - 46.3 = 144.9'$

MODEL VALUES

$x = 205.067'$
$y = 143.076'$

✓ VALUES ARE APPROXIMATELY EQUAL. CHECK OF 1 FLOOR IS SUFFICIENT TO VALIDATE SOFTWARE METHOD.

4.3.2 Wind Load Comparison

The method used to apply wind loads to the three dimensional model was auto-generation on the diaphragm edges. Etabs simplified the edge geometry to reflect the dimensions of a rectangle, and calculated the forces in the X (N-S) and Y (E-W) directions based on the projected areas on each face. Despite the difference in calculation method, the manual calculation and Etabs calculation are remarkably similar as shown in Table 10. The similarity is likely coincidence, as the two values both underestimate different aspects of the wind load. The manual calculation has an increased leeward pressure with a lower windward pressure due to the accurate building dimensions and the open stories respectively. Conversely, the Etabs calculation has a small leeward pressure, but a larger windward pressure because the program cannot account for an open story. The two effects happen to nearly cancel, and the results are highly similar for the two methods. This validates the applied wind loads.

Table 10: Wind Load Base Reactions

	Fx (kip)	Fy (kip)	Fz (kip)
Model X	-453.585	0	0
Model Y	0	-1190.67	0
Manual X	454.1	0	0
Manual Y	0	1085.2	0

4.3.3 Seismic Load Comparison

Applied seismic loads are more varied than the wind loads. The Etabs auto-generated loads are significantly greater than the manually calculated loads. This is probably due to the effects of the building period. By varying the "user defined" building period, the forces were brought much closer; however, given the accuracy building stiffness calculations and the fact that the Etabs loads are more conservative, the software-generated loads were used to compute story drifts and member forces. A comparison of seismic drift case loads can be seen in Table 11.

Table 11: Seismic Load Base Reactions

	F (kips)
Model	-1866.7
Manual	1434.6

4.3.4 Torsional Shear Check

A check was conducted to verify that the proper story shears were being distributed based on the center of mass and center of rigidity. The 6th story was used with a seismic load to remove the effects of the other stories on torsional forces.

Table 12: Story 6 Pier Shear values accounting for Torsion

Element	Story	Rel. K (k/in)	COM	COR X	COR Y	e(acc.)	Da	J	Applied Load	Direct Shear Coefficient	Torsional Shear	Torsional Shear	Direct Shear	Total Shear	Model Values
S-srcsw	6	244.50	143.07	195.16	145.27	9.55	29.492	18653024.24	0	0.25265874	2.820727	-1.76446	0	2.820727	2.715
N-srcsw	6	284.90	143.07	195.16	145.27	9.55	0.4768	18653024.24	0	0.294408617	0.053139	-0.03324	0	0.053139	2.714
Aa1-scbf	6	101.42	143.07	195.16	145.27	9.55	195.16	18653024.24	0	0.10480469	7.74273	-4.84332	0	7.74273	5.423
Aa3-scbf	6	107.53	143.07	195.16	145.27	9.55	195.16	18653024.24	0	0.11111551	8.208959	-5.13497	0	8.208959	18.465
Ra-scbf	6	229.36	143.07	195.16	145.27	9.55	209.84	18653024.24	0	0.237012442	18.82704	-11.7769	0	18.82704	17.179
1B-smf	6	27.65	143.07	195.16	145.27	9.55	-45.895	18653024.24	621	0.178454266	-0.49636	0.310489	110.8201	111.1306	109.546
1A-smf	6	46.17	143.07	195.16	145.27	9.55	-45.895	18653024.24	621	0.298000499	-0.82887	0.518485	185.0583	185.5768	158.677
4B-smf	6	27.65	143.07	195.16	145.27	9.55	31.5213	18653024.24	621	0.178454266	0.340907	-0.21325	110.8201	111.161	91.074
4A-smf	6	46.17	143.07	195.16	145.27	9.55	31.5213	18653024.24	621	0.298000499	0.56928	-0.3561	185.0583	185.6276	179.658
8-smf	6	7.30	143.07	195.16	145.27	9.55	145.27	18653024.24	621	0.047090471	0.414585	-0.25934	29.24318	29.65777	19.005

4.3.5 Equilibrium Check

An equilibrium check was conducted for wind at the base story (model story 1). Compare the 440.5 kip total value from Table 13 with the previous 453.6 kip total base shear from Table 10. The difference of 13 kips can be explained by the residual stiffnesses of the shear walls and braced frames in the out-of-plane direction due to fixed connections at the bases. This source of error is minimal; however, a more detailed refinement could be undertaken to eliminate out-of-plane shears by adjusting the base fixities.

Table 13: Story 1 Wind Forces in the X (N-S) Direction

Story	Pier	Load Case/Combo	Location	P	V2
				kip	kip
Story1	smf-1B	WX Max	Bottom	0.372	71.045
Story1	smf-1A	WX Max	Bottom	7.003	105.246
Story1	smf-4B	WX Max	Bottom	0.961	71.396
Story1	smf-4A	WX Max	Bottom	7.469	105.451
Story1	smf-5A	WX Max	Bottom	0	26.93
Story1	smf-5B	WX Max	Bottom	0	43.802
Story1	smf-8B	WX Max	Bottom	0	16.621
				Total:	440.491

5 Code and Member Checks

5.1 Drift Checks

The seismic drift for the building is plotted below in Figure 10. Red represents the Y (E-W) axis and blue, the X (N-S). It is clear that the Y direction is stiffer than the X direction under the same loading, which makes intuitive sense when comparing the stiffness of concrete shearwalls and braced frames to that of special moment frames. The largest applicable C_d for the building is 5.5 for special moment frames. C_d/I_e yields the amplification factor for allowable drift, which in this case is 3.67. The full calculation can be found on the following pages.

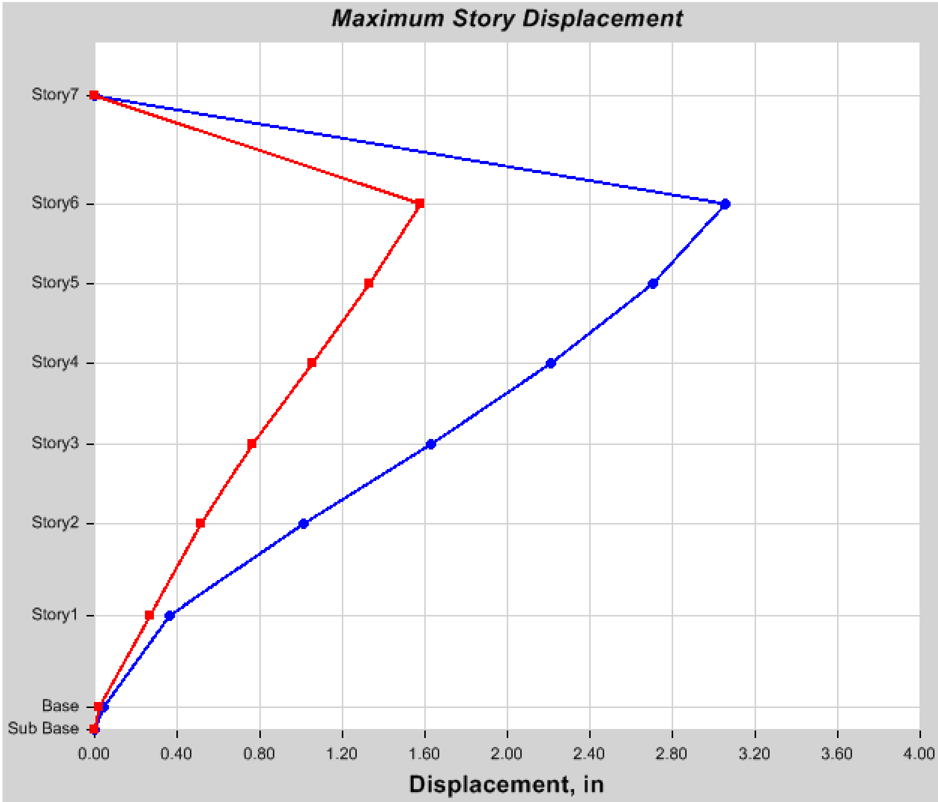


Figure 10: Seismic Drifts

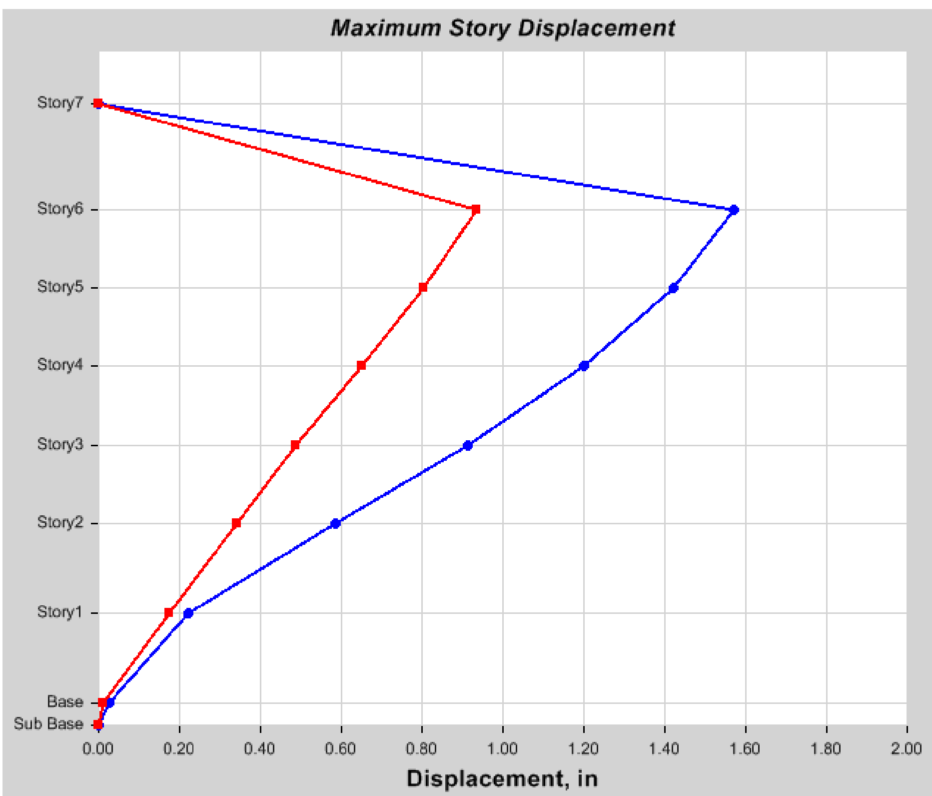


Figure 11: Wind Drifts

DRIFT LIMIT CHECK

SEISMIC

$h = 108.25'$

$$h(0.025) \geq \delta_x = \frac{C_d}{I_e} \delta_{xe}$$

$$108.25(12)(0.025) \geq \delta_x = \frac{5.5}{1.5}(3.06")$$

$$32.475" \geq \delta_x = 11.22" \checkmark$$

DRIFT REQUIREMENTS MET.

WIND

$$\frac{h}{400} > \delta_{max}$$

$$\frac{108.25(12)}{400} > 1.57"$$

$$3.25 > 1.57" \checkmark$$

5.2 MEMBER CHECKS

The checks in this section will evaluate one critical member for each element and material type.

- Steel Brace
- Steel Beam
- Steel Column
- Concrete Shear Wall Section

The steel members are each part of brace frame "Aa3," from grid line Aa on the Southernmost end of the building. The steel brace and column members and their forces under seismic loading (not seismic drift loading) are shown in Figures 13 through 16. The steel beam on grid line 4 is shown in figures 17 through 20. The concrete shear wall section is taken from "SWS," the Southernmost shear wall.

Figure 12 below shows the locations of the three lateral components on the building plan, circled in red.

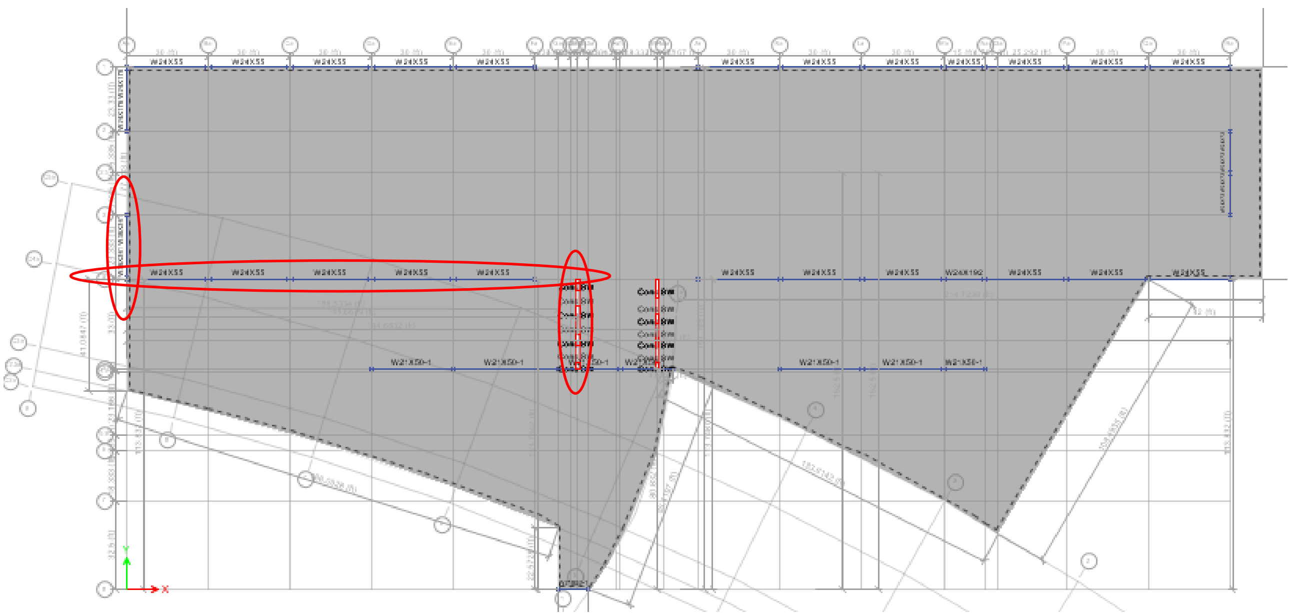


Figure 12: Lateral Component Locations

5.2.1 Brace and Column Member Check

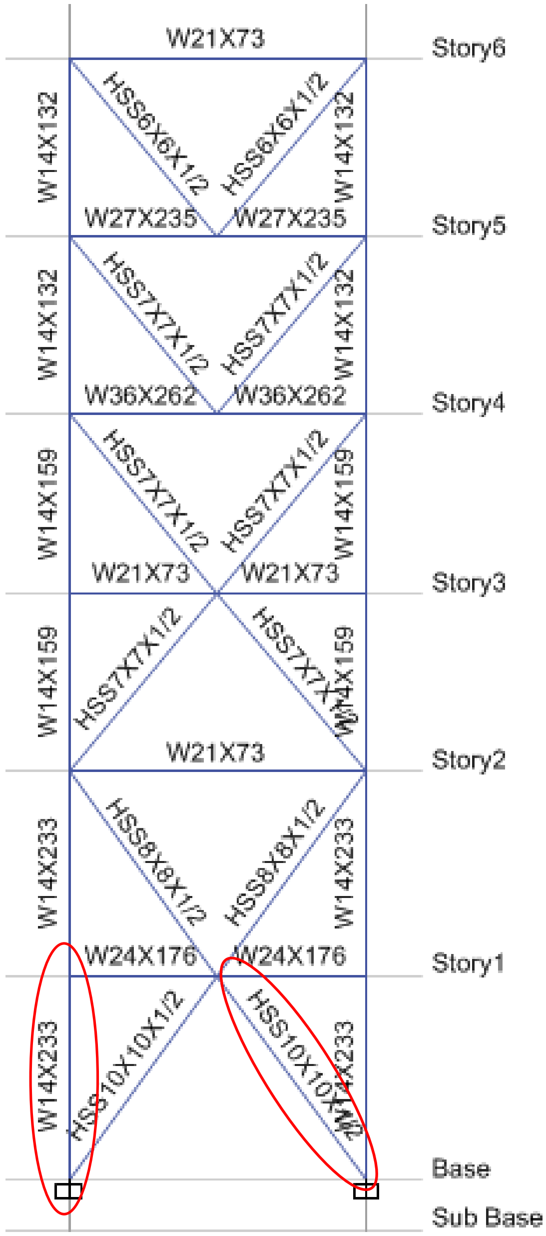


Figure 13: Brace Frame Section Assignments

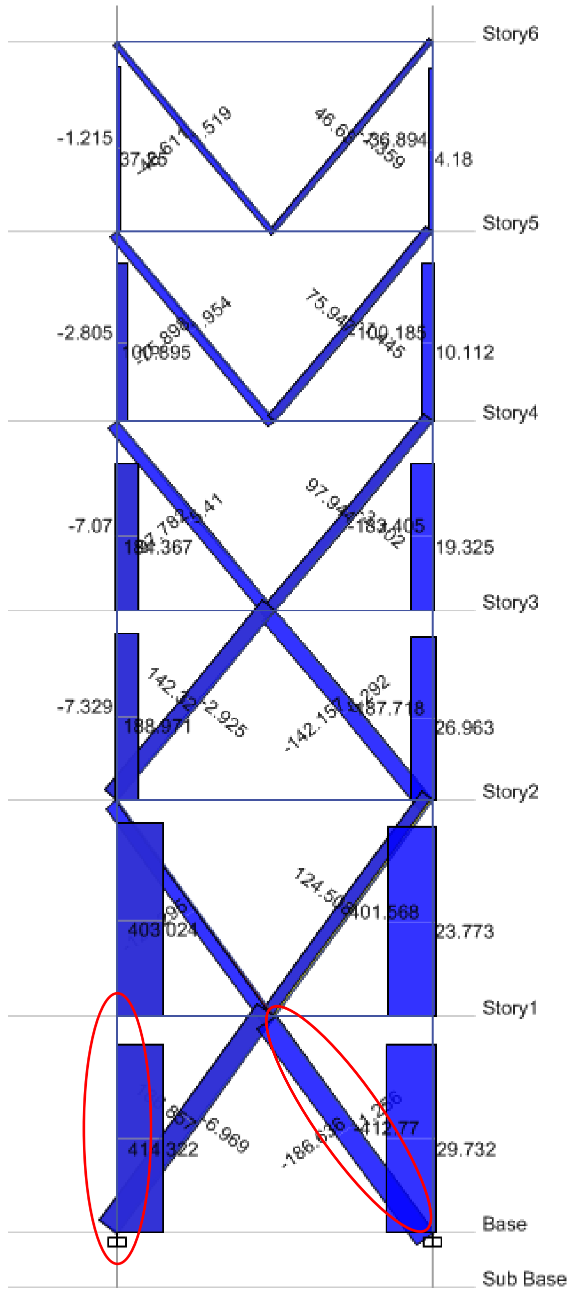


Figure 14: Brace Frame Axial Forces

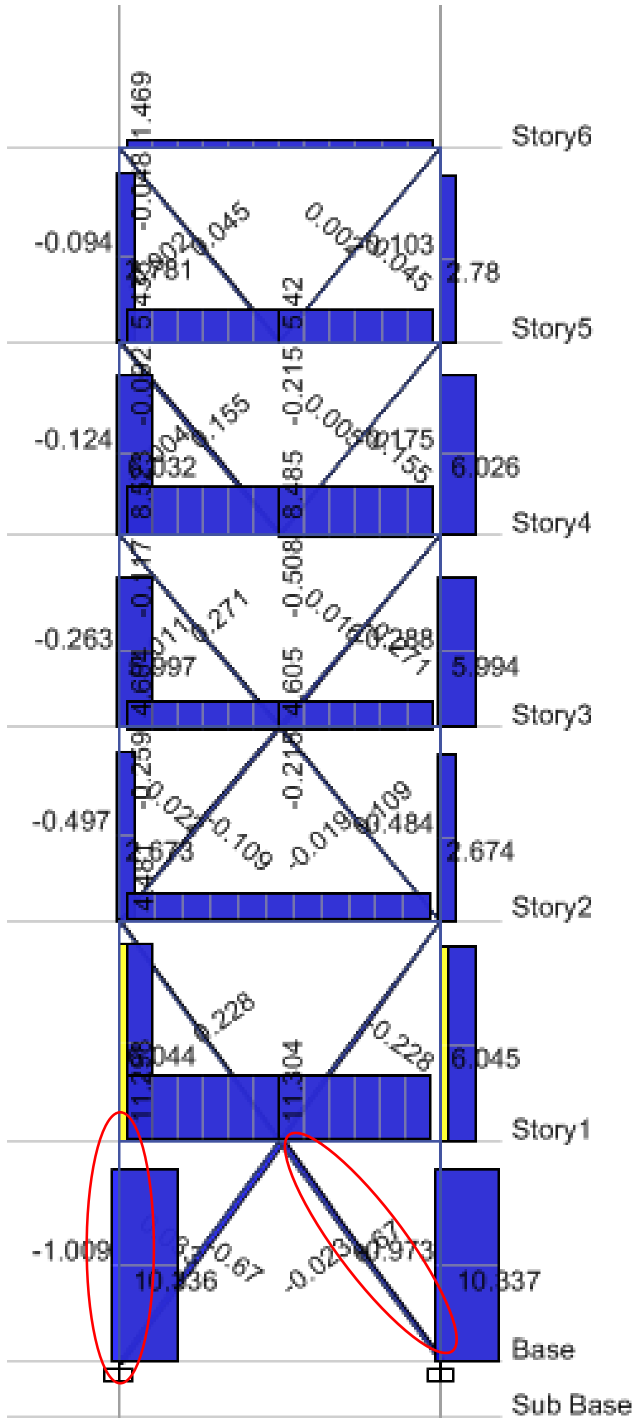


Figure 15: Brace Frame Shears

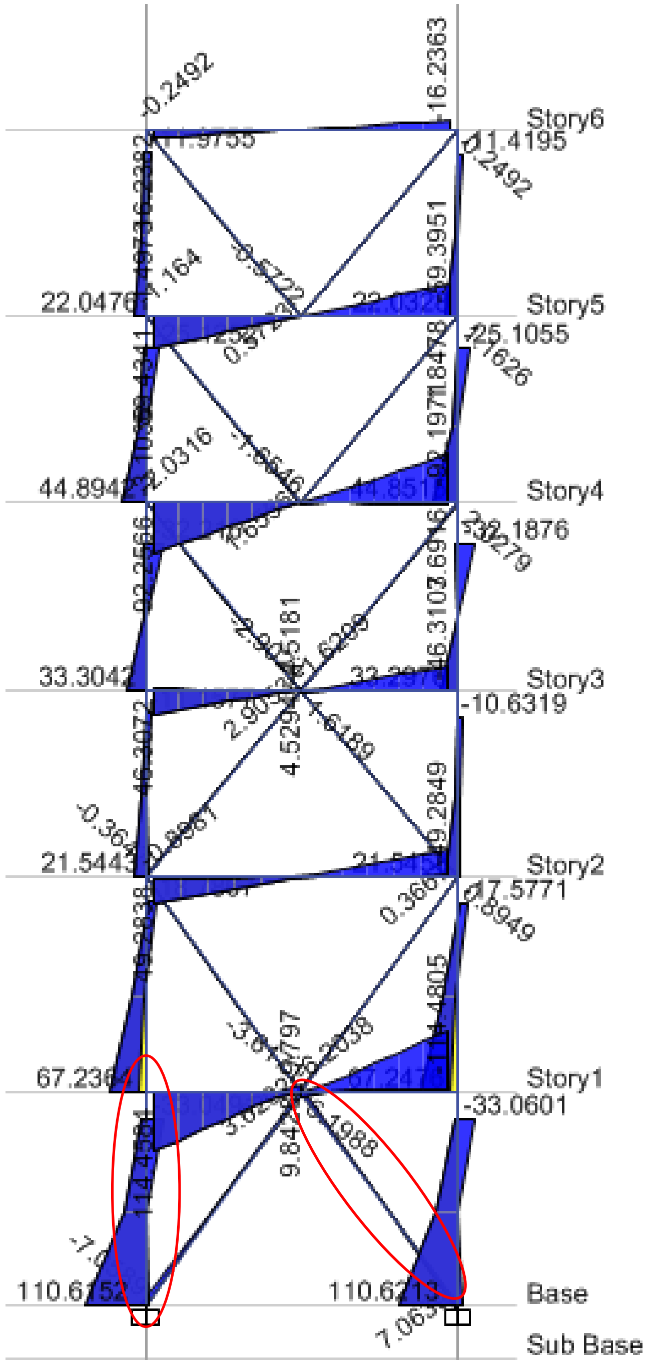


Figure 16: Brace Frame Moments

BRACE MEMBER CHECK

SECTION: HSS 10x10x1/2

$L = 19.8$

$K = 1.0$

$P_u = 186.7$ kips (seismic load case)

CAPACITY:

$C = \phi_c P_n = 549$ @ $KL = 20'$ [AISC 530 TABLE 4-4]

$T = \phi_T P_n = 561k$ [AISC 530 TABLE 5-5]

$186.7\,k < 549\,k < 561\,k$ ✓

COLUMN MEMBER CHECK

SECTION: W14 ×233

$M_{rxs} = 195$ kft

$M_{rys} = 110.6$ k ft ← NOTE! THIS VALUE INDICATES AN ERROR IN MODELING, AS SHEAR IS INCLUDED IN THE OUT-OF-PLANE DIRECTION.

$P_{us} = 414.3$ k

$M_{rxw} = 69.6$ k ft

$M_{ryw} = 44.1$ k ft

$P_{uw} = 163$ k

CASES: $1.2D + 1.0(E_h + E_v) + 1.0L + 0.2S$

$1.2D + 1.0W + L + 0.5S$

$$\text{T-AREA} = \left(\frac{15.33' + 23.33'}{2}\right)(15')$$

$$= 289.95 \approx 290 ft^2$$

$$DL = 64(6)(290)$$
$$= 111360 \text{ lbs}$$
$$= 111.4 \text{ k}$$

$$LL = 80 \begin{Bmatrix} 0.5 \\ 0.25 + \frac{15}{290(5)} \end{Bmatrix}$$

$$= 80 \begin{Bmatrix} 0.5 \\ 0.64 \end{Bmatrix}$$

$$= 51 \text{ psf}$$
$$51(5)(290)$$
$$= 73.9 \text{ k}$$

$$SL = 24 \text{ psf}$$
$$24(1)(290)$$
$$= 6.9 \text{ k}$$

ASSUMED = $E_h + E_v$

CASES: $S = 1.2(111.4) + 1.0(414.3) + (73.9\text{ k}) + 0.2(6.9\text{ k}) = \boxed{623.26\text{ k}}$

$W = 1.2(111.4) + 1.0(163) + (73.9\text{ k}) + 0.5(6.9\text{ k}) = 374.03\text{ k}$

SEISMIC FORCES CONTROL.

INTERACTION EQ. USING ELM:

$$KL = 1.0\left(16' - \frac{24}{12}\right) = 14'$$

$p = 0.367 \times 10^{-3}$

$b_x = 0.544 \times 10^{-3}$

$b_y = 1.07 \times 10^{-3}$

$$pP_r + b_x M_x + b_y M_y \leq 1.0$$

$$10^{-3}\left[0.367(623.26) + 0.544(195) + 1.07(110.6)\right] \leq 1.0$$

$$\boxed{0.45 \leq 1.0 \checkmark}$$

5.2.2 Beam Check

Figure 17: Special Moment Frame Section Assignments

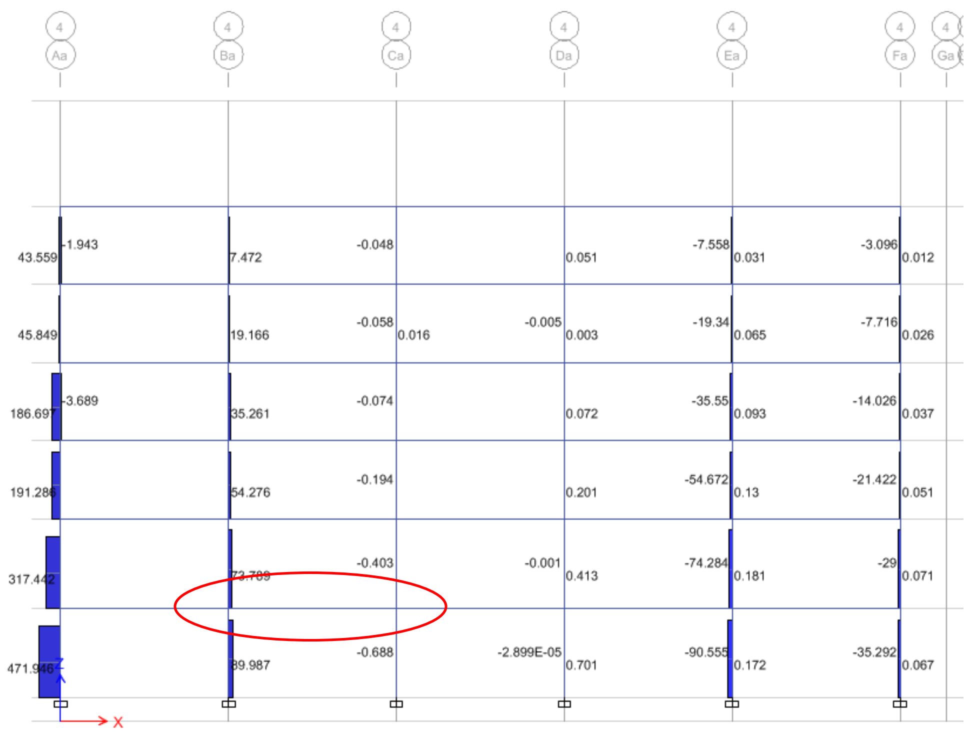


Figure 18: Special Moment Frame Axial Forces

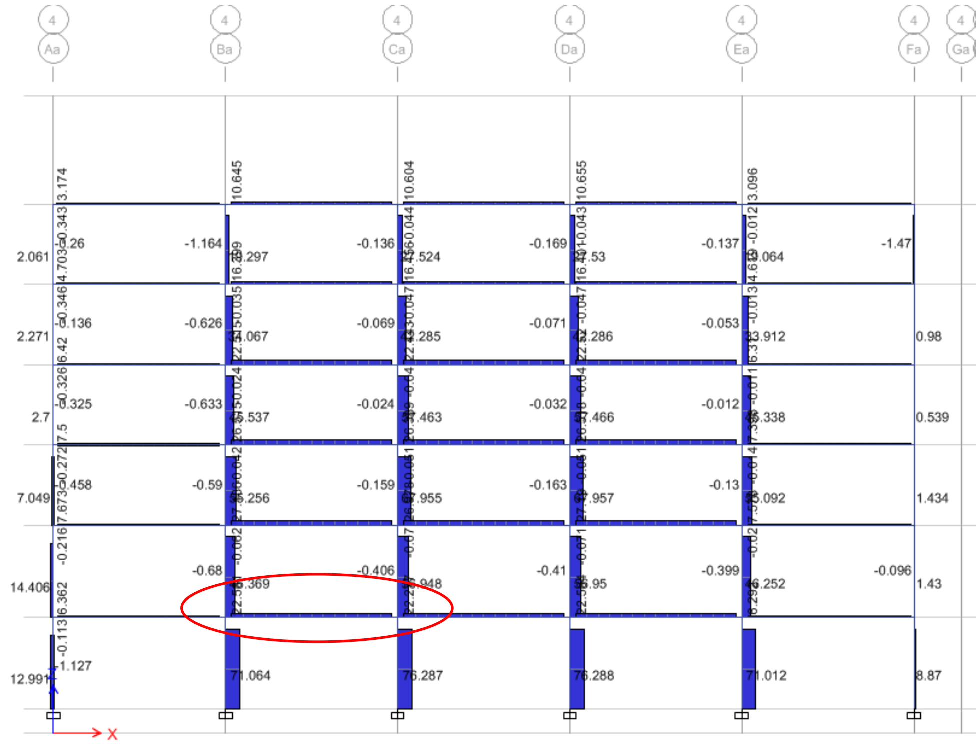


Figure 19: Special Moment Frame Shears

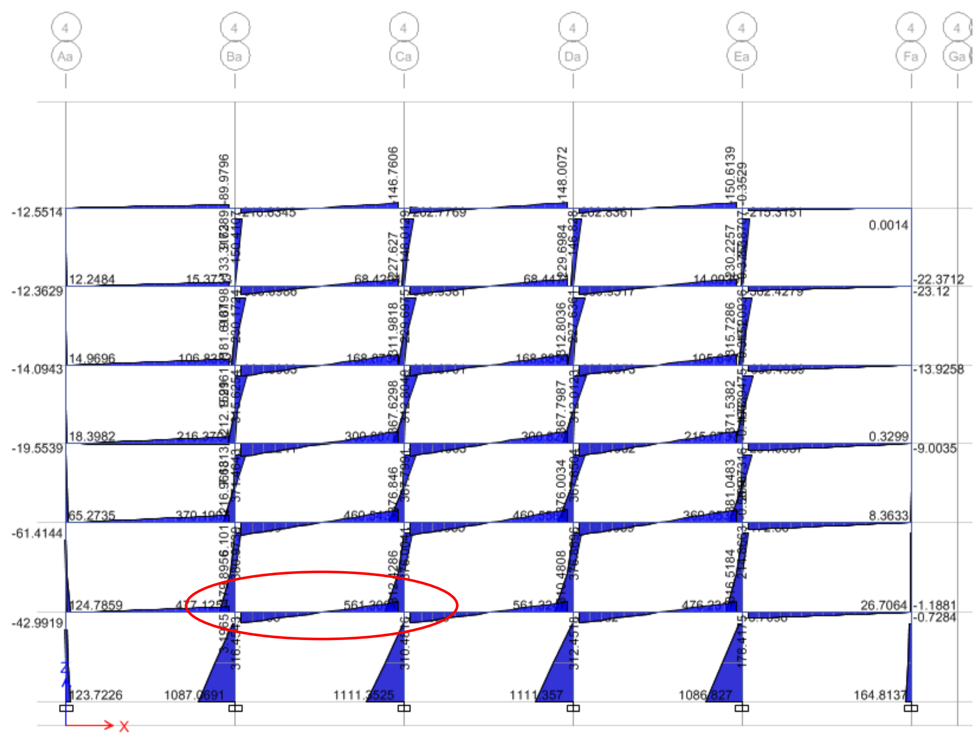


Figure 20: Special Moment Frame Moments

BEAM MEMBER CHECK

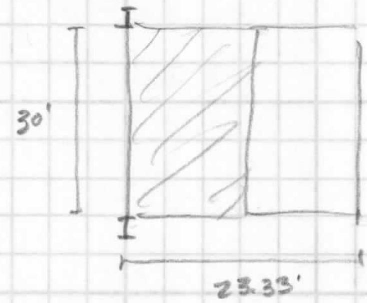
SEISMIC LOAD CASES CONTROL AS DEMONSTRATED IN COLUMN CHECK

SECTION: W24 × 55

$P_u = 0$

$M_{rxS} = 316.5$

$M_{rx(D+L)} = ?$



$$L = 80\left\{\begin{array}{l}0.5\\0.25 + \frac{15}{\sqrt{30(23.33)}}\end{array}\right\}$$

$$= 80\{0.817\}$$

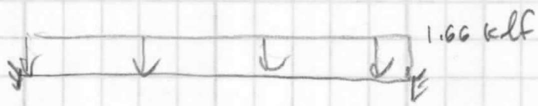
$$= 65.35$$

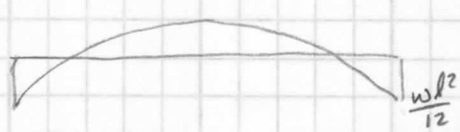
LOAD CASE: $1.2D + 1.0E + 1.0L + 0.2S$

$$1.2(64)\left(\frac{23.33}{2}\right) + 1.0(65.35)\left(\frac{23.33}{2}\right)$$

$= 1658 \text{ plf}$

$= 1.66 \text{ klf}$





$$M_{rx(D+L)} = \frac{(1.66)(30)^2}{12}$$

$= 124.5 \text{ kft}$

$M_{TOTAL} = M_{rxS} + M_{rx(D+L)}$

$= 316.5 + 124.5 \text{ kft}$

$= 441 \text{ kft}$

TABLE 3-19 → ϕM_p non-composite = 503 kft ✓

MEETS STRENGTH REQUIREMENTS

5.2.3 Concrete Shear Wall Member Check

Rather than a numerical check for the specially reinforced concrete shear wall, this section provides a qualitative analysis of the resulting shell stresses. Figure 21 shows the F_{22} vertical stresses in the wall for the seismic load case in the Y (E-W) direction. Appropriately, the stresses are positive on one side of the wall and negative on the other, with the extremes at the farthest ends and toward the bottom. This distribution is consistent with axial forces due to seismic overturning.

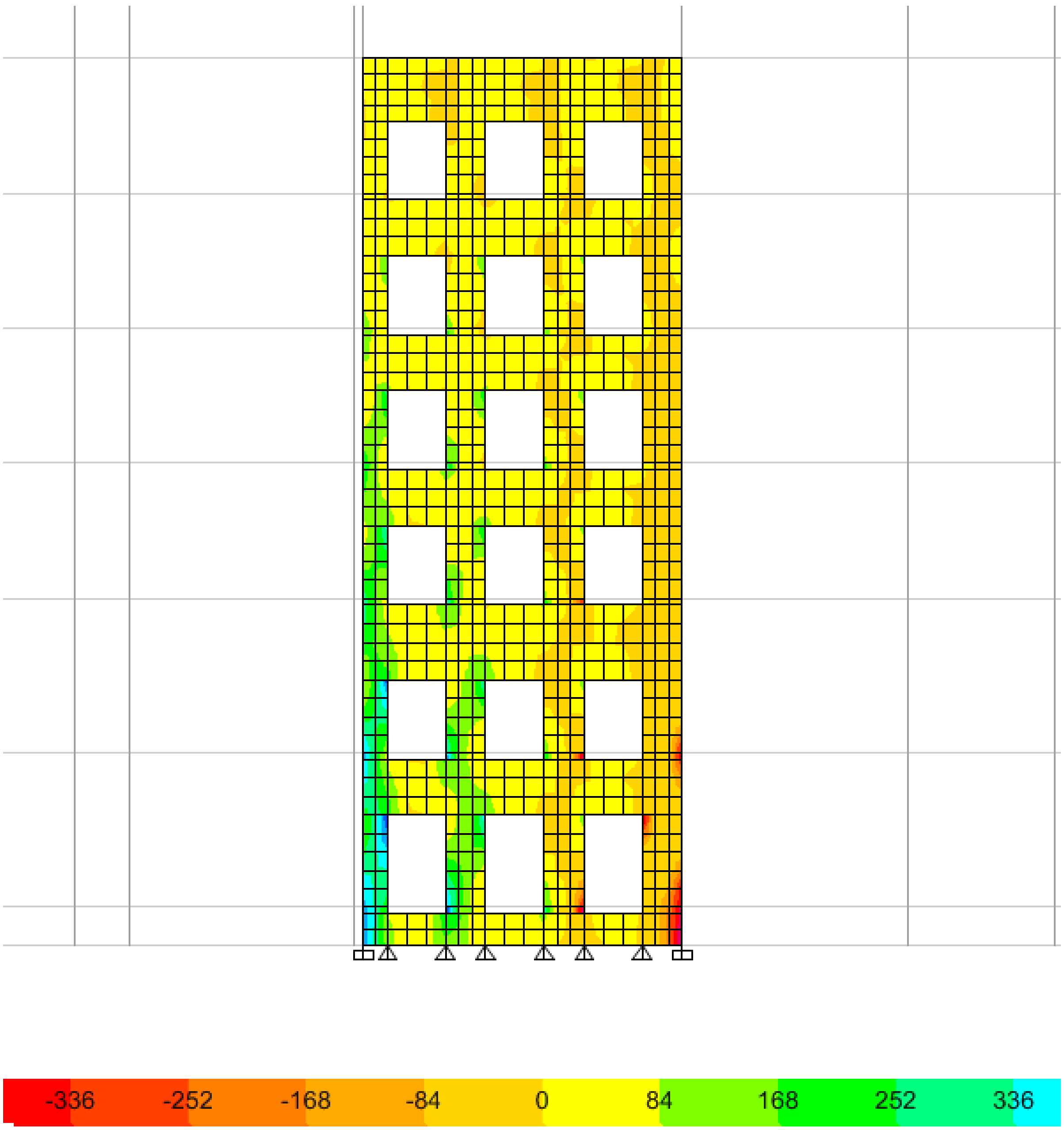


Figure 21: Shear Wall Vertical Forces

The horizontal shear stress distribution is more uniform, but shows signs of a conventional shear distribution including “X” banding across the coupling beams and stress concentrations at corners of the wall openings. The magnitudes of the shear stresses decrease as the wall height increases, note the strong corner stresses at the bottom and light stresses at the top. This is also consistent with seismic loads accumulating in the shear wall from top to bottom.

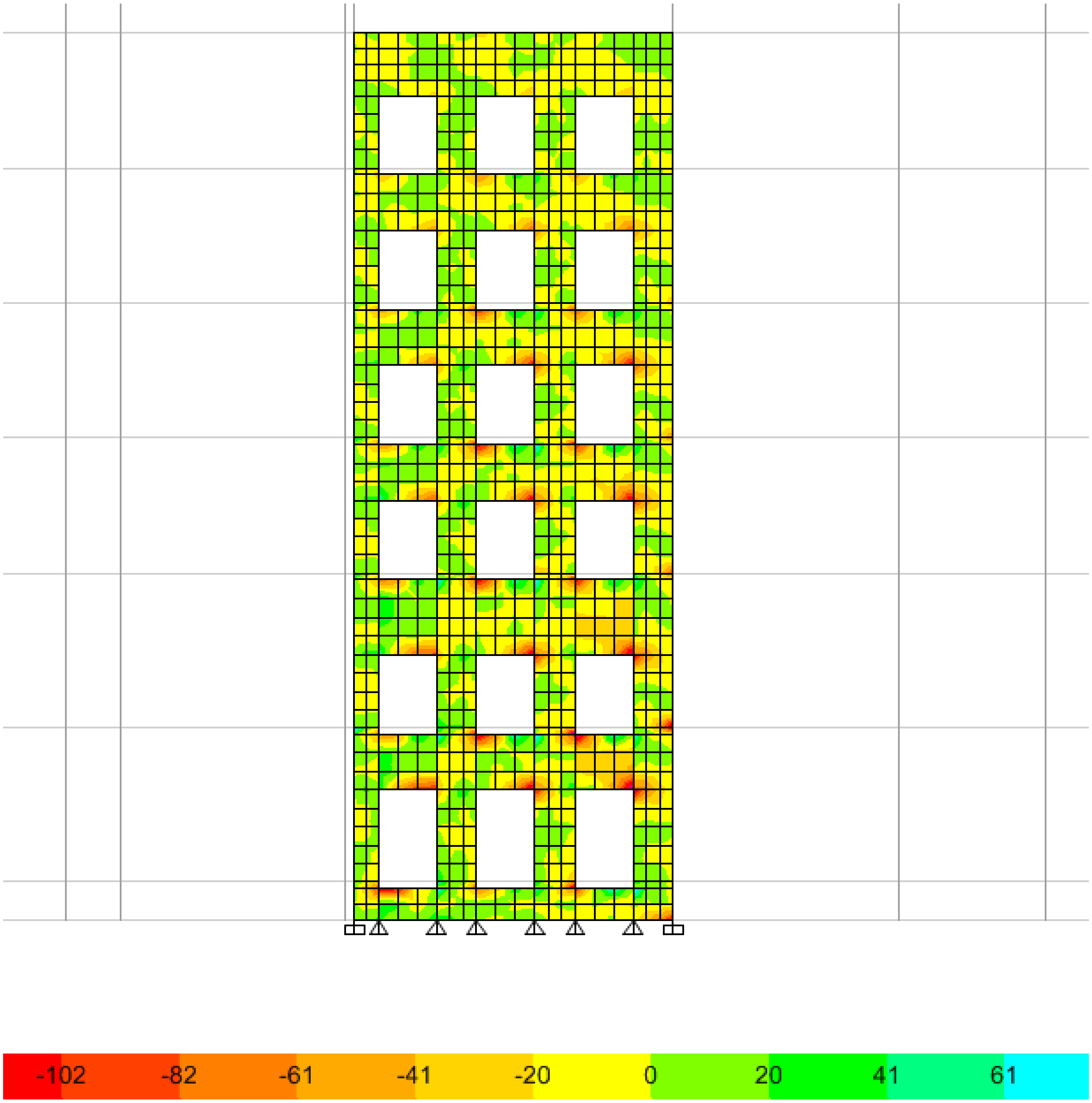


Figure 22: Shear Wall Shears

6 APPENDIX A: GRAVITY LOADS

Design Criteria (Live Loads)

Hospitals

Operating rooms, labs	60 PSF *
Private rooms	40 PSF *
Wards	40 PSF *
Corridors (above 1st floor)	80 PSF *

* Design for uniform load indicated or 1000# concentrated load over 2.5 feet square, whichever produces the greater load effect.

Offices

Offices	50 PSF **
Lobbies & 1st floor corridors	100 PSF **
Corridors (above 1st floor)	80 PSF **

** Design for uniform load indicated or 2000# concentrated load over 2.5 feet square whichever produces the greater load effect.

Misc. Live Loads

Corridors, except as otherwise indicated	100 PSF
Stairs and Exits	100 PSF ***
Dining Rooms and Restaurants	100 PSF
Retail Stores (first floor)	100 PSF
Mechanical rooms	125 PSF (Includes allowance for equipment pads)
Storage – Light	125 PSF

*** Design for uniform load indicated or 300# concentrated load over 4 inches square whichever produces the greater load effect

Partition loads 20 PSF
(Offices & locations where partitions are subject to change)

Design Floor Live Loads (Typical unless noted otherwise in calculations)
Typical floors: 80 PSF (60 PSF + 20 PSF Partitions) or (80 PSF Corridors)
First floor (typical): 100 PSF (60 PSF + 20 PSF Partitions) or (100 PSF Corridors)
First floor (equip): 120 PSF (60 PSF + 20 PSF Partitions + 40 PSF Equipment)
Mechanical Rooms: 125 PSF
Elevator Machine Rooms: 500 PSF
Interstitial Level: 25 PSF
Roof Top Mechanical Unit Support: 50 PSF (Live Load + Snow Load)

Other Live Loads

Handrails and guards	50 PLF or 200# concentrated load @ top rail
Components	50# over 1 foot square
Grab bars, shower seats, dressing rm. seats	250# load in any direction at any point

Impact Loads
Elevator loads shall be increased by 100 percent for impact
Machinery weight shall be increased to allow for impact
Elevator machinery: 100 percent
Light machinery, shaft or motor driven: 20 percent
Reciprocating machinery or power driven units: 50 percent
Hangers for floors or balconies: 33 percent

Live Load Reduction
Live loads to columns will be reduced in accordance with IBC Section 1607.9.1. Live loads that exceed 100 PSF and roof live loads will not be reduced.

Distribution of Floor Loads
Uniform floor live loads shall be patterned to produce the greatest effect on continuous framing.

Roof Loads
Uniform roof live loads shall be patterned to produce the greatest effect on continuous framing.
Minimum roof load will be less than snow load
See section 1607.11 for other roof loads (roof gardens, landscaped roofs, canopies)

Interior Walls and Partitions

Interior Partitions	5 PSF horizontal pressure
Medical Equipment	
MRI Equipment (four pt loads)	29000 lb/4 = 7250 lb
MRI Equip minus equip allowance	7250 lb – (40 PSF)*(25 ft2) = 6250 lb

Design Criteria (Dead Loads)

Hospital Floor (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Ceiling/Mechanical/Misc	12 PSF
	60 PSF (Mass DL = 69 PSF + 10 PSF for Partition Mass)
Hospital Roof (Future Floor) (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Ceiling/Mechanical/Misc	12 PSF
Roofing/Insulation/Ballast	18 PSF
	78 PSF (Mass DL = 87 PSF)
Hospital Roof (No future floors) (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume 9 PSF)
Ceiling/Mechanical/Misc	12 PSF
Roofing/Insulation/Ballast	18 PSF
	78 PSF (Mass DL = 87 PSF)
Power Plant Roof (No future floors) (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume 9 PSF)
Ceiling/Misc	7 PSF
Mechanical Piping	60 PSF
Roofing/Insulation/Ballast	18 PSF
	133 PSF (Mass DL = 142 PSF)
Penthouse Floor (Composite slab, 2 Hour)	
3" Deck + 3 ½" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Mechanical/Misc	12 PSF
	60 PSF (Mass DL = 69 PSF + 10 PSF for Partition Mass)
Penthouse Roof (Steel Roof Deck)	
Steel Deck	3 PSF
Beams/Girders/Columns	Self Wt (Assume = 7 PSF)
Mechanical/Misc	7 PSF
Roofing/Insulation/Ballast	18 PSF
	28 PSF (Mass DL = 35 PSF)
Roof Top Mechanical Unit Support	
Beams/Girders/Columns	Self Wt (Assume = 7 PSF)
Mechanical Unit	60 PSF
Miscellaneous Pipes & Ducts 15 PSF	
	75 PSF (Mass DL = 82 PSF)
Hospital Floor – Piping Zone (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Mechanical Piping	60 PSF
Ceiling/Misc	7 PSF
	115PSF (Mass DL = 94 PSF + 10 PSF for Partition Mass)
Hospital Floor/Power Plant (Composite slab, 2 Hour)	

3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
Mechanical Piping	60 PSF
Ceiling/Misc	7 PSF
	115PSF (Mass DL = 94 PSF + 10 PSF for Partition Mass)
Hospital Floor – MRI Zone (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
2" Concrete Topping	18 PSF
Mass for Permanent Equip	(15 PSF Mass DL)
Ceiling/Mechanical/Misc	12 PSF
	78 PSF (Mass DL = 102 PSF + 10 PSF for Partition Mass)
Hospital Floor – Piping Zone plus MRI Zone (Composite slab, 2 Hour)	
3" Deck + 3 1/2" LW Conc	48 PSF
Beams/Girders/Columns	Self Wt (Assume = 9 PSF)
2" Concrete Topping	18 PSF
Mass for Permanent Equip	(15 PSF Mass DL)
Mechanical	30 PSF
Ceiling/Misc	7 PSF
	103 PSF (Mass DL = 127 PSF + 10 PSF for Partition Mass)
MOB Floor (Non-Composite slab, 0 Hour)	
1 ½" Deck + 2" LW Conc	29 PSF
Beams/Girders/Columns	Self Wt (Assume 9 PSF)
Ceiling/Mechanical/Misc	7 PSF
	36 PSF (Mass DL = 45 PSF + 10 PSF for Partition M ass)

7 APPENDIX B: WIND LOADS

WIND LOAD TABLES FOR AREAS A,B,F & G

Zone (Note 1)	Area (sq ft) (Note 2)	Design Wind Loads (psf) 0' to 30' (Note 3)		Design Wind Loads (psf) 30' to 40' (Note 3)		Design Wind Loads (psf) 40' to 50' (Note 3)		Design Wind Loads (psf) 50' to 60' (Note 3)		Design Wind Loads (psf) 60' to 70' (Note 3)		Design Wind Loads (psf) 70' to 80' (Note 3)		Design Wind Loads (psf) 80' to 90' (Note 3)		Design Wind Loads (psf) 90' to 100' (Note 3)		Design Wind Loads (psf) 100' to 110' (Note 3)		Design Wind Loads (psf) 110' to 120' (Note 3)	
1 ROOF	10	10	-23.6	10	-25.3	10	-26.7	10	-27.8	10	-29.0	10.0	-30.1	10.0	-31.0	10.0	-31.8	10.0	-32.5	10.0	-33.2
	20	10	-22.9	10	-24.5	10	-25.9	10	-27.0	10	-28.1	10.0	-29.2	10.0	-30.0	10.0	-30.8	10.0	-31.5	10.0	-32.2
	50	10	-20.7	10	-22.2	10	-23.4	10	-24.4	10	-25.4	10.0	-26.3	10.0	-27.1	10.0	-27.8	10.0	-28.4	10.0	-29.0
	100	10	-20.0	10	-21.4	10	-22.6	10	-23.5	10	-24.5	10.0	-25.4	10.0	-26.1	10.0	-26.8	10.0	-27.4	10.0	-28.0
2 ROOF	10	10	-36.4	10	-39.2	10	-41.5	10	-43.3	10	-45.2	10.0	-47.1	10.0	-48.5	10.0	-49.9	10.0	-51.0	10.0	-52.2
	20	10	-34.9	10	-37.6	10	-39.8	10	-41.6	10	-43.4	10.0	-45.2	10.0	-46.5	10.0	-47.9	10.0	-49.0	10.0	-50.1
	50	10	-32.1	10	-34.5	10	-36.6	10	-38.2	10	-39.8	10.0	-41.4	10.0	-42.6	10.0	-43.9	10.0	-44.9	10.0	-45.9
	100	10	-30.7	10	-33.0	10	-34.9	10	-36.5	10	-38.0	10.0	-39.5	10.0	-40.7	10.0	-41.8	10.0	-42.8	10.0	-43.8
3 ROOF	10	10	-49.1	10	-53.0	10	-56.3	10	-58.9	10	-61.4	10.0	-64.0	10.0	-66.0	10.0	-67.9	10.0	-69.6	10.0	-71.2
	20	10	-47.0	10	-50.7	10	-53.8	10	-56.3	10	-58.7	10.0	-61.2	10.0	-63.1	10.0	-64.9	10.0	-66.5	10.0	-68.0
	50	10	-44.2	10	-47.6	10	-50.5	10	-52.8	10	-55.1	10.0	-57.4	10.0	-59.2	10.0	-60.9	10.0	-62.4	10.0	-63.8
	100	10	-42.0	10	-45.3	10	-48.1	10	-50.2	10	-52.4	10.0	-54.6	10.0	-56.3	10.0	-57.9	10.0	-59.3	10.0	-60.6
4 WALL	10	16.5	-22.3	17.6	-22.3	18.5	-22.3	19.2	-22.3	20.0	-22.3	20.7	-22.3	21.2	-22.3	21.8	-22.3	22.2	-22.3	22.7	-22.3
	20	16.5	-22.3	17.6	-22.3	18.5	-22.3	19.2	-22.3	20.0	-22.3	20.7	-22.3	21.2	-22.3	21.8	-22.3	22.2	-22.3	22.7	-22.3
	50	15.1	-21.3	16.0	-21.3	16.9	-21.3	17.5	-21.3	18.2	-21.3	18.8	-21.3	19.3	-21.3	19.8	-21.3	20.2	-21.3	20.6	-21.3
	100	14.4	-20.3	15.3	-20.3	16.0	-20.3	16.6	-20.3	17.3	-20.3	17.9	-20.3	18.3	-20.3	18.8	-20.3	19.2	-20.3	19.5	-20.3
5 WALL	10	16.5	-40.9	17.6	-40.9	18.5	-40.9	19.2	-40.9	20.0	-40.9	20.7	-40.9	21.2	-40.9	21.8	-40.9	22.2	-40.9	22.7	-40.9
	20	16.5	-40.9	17.6	-40.9	18.5	-40.9	19.2	-40.9	20.0	-40.9	20.7	-40.9	21.2	-40.9	21.8	-40.9	22.2	-40.9	22.7	-40.9
	50	15.1	-36.8	16.0	-36.8	16.9	-36.8	17.5	-36.8	18.2	-36.8	18.8	-36.8	19.3	-36.8	19.8	-36.8	20.2	-36.8	20.6	-36.8
	100	14.4	-32.7	15.3	-32.7	16.0	-32.7	16.6	-32.7	17.3	-32.7	17.9	-32.7	18.3	-32.7	18.8	-32.7	19.2	-32.7	19.5	-32.7

WIND LOAD TABLES FOR AREAS A,B,F & G

Zone (Note 1)	Area (sq ft) (Note 2)	Design Wind Loads (psf) 0' to 30' (Note 3)	Design Wind Loads (psf) 30' to 40' (Note 3)	Design Wind Loads (psf) 40' to 50' (Note 3)	Design Wind Loads (psf) 50' to 60' (Note 3)	Design Wind Loads (psf) 60' to 70' (Note 3)	Design Wind Loads (psf) 70' to 80' (Note 3)	Design Wind Loads (psf) 80' to 90' (Note 3)	Design Wind Loads (psf) 90' to 100' (Note 3)	Design Wind Loads (psf) 100' to 110' (Note 3)	Design Wind Loads (psf) 110' to 120' (Note 3)
2 ROOF OVERHANG	10	-27.8	-29.9	-31.6	-33.0	-34.4	-35.8	-36.8	-37.8	-38.7	-39.6
	20	-27.4	-29.4	-31.1	-32.5	-33.8	-35.2	-36.2	-37.2	-38.1	-38.9
	50	-26.8	-28.8	-30.5	-31.8	-33.1	-34.4	-35.4	-36.4	-37.3	-38.1
	100	-26.4	-28.4	-30.0	-31.3	-32.6	-33.9	-34.9	-35.8	-36.6	-37.4
3 ROOF OVERHANG	10	-43.4	-46.9	-49.7	-52.0	-54.2	-56.5	-58.2	-59.9	-61.3	-62.7
	20	-34.9	-37.6	-39.8	-41.6	-43.4	-45.2	-46.5	-47.9	-49.0	-50.1
	50	-23.6	-25.3	-26.7	-27.8	-29.0	-30.1	-31.0	-31.8	-32.5	-33.2
	100	-15.1	-16.0	-16.9	-17.5	-18.2	-18.8	-19.3	-19.8	-20.2	-20.6

WIND LOAD TABLES FOR AREAS C & D

Zone (Note 1)	Area (sq ft) (Note 2)	Design Wind Loads (psf) 0' to 30' (Note 3)		Design Wind Loads (psf) 30' to 40' (Note 3)		Design Wind Loads (psf) 40' to 50' (Note 3)		Design Wind Loads (psf) 50' to 60' (Note 3)	
1 ROOF	10	10	-22.8	10	-24.5	10	-25.9	10	-27.1
	20	10	-22.1	10	-23.8	10	-25.1	10	-26.2
	50	10	-20.0	10	-21.4	10	-22.7	10	-23.6
	100	10	-19.3	10	-20.7	10	-21.8	10	-22.8
2 ROOF	10	10	-35.6	10	-38.4	10	-40.7	10	-42.6
	20	10	-34.2	10	-36.8	10	-39.1	10	-40.9
	50	10	-31.3	10	-33.8	10	-35.8	10	-37.4
	100	10	-29.9	10	-32.2	10	-34.1	10	-35.7
3 ROOF	10	10	-48.4	10	-52.3	10	-55.5	10	-58.1
	20	10	-46.2	10	-49.9	10	-53.0	10	-55.5
	50	10	-43.4	10	-46.9	10	-49.7	10	-52.1
	100	10	-41.3	10	-44.5	10	-47.3	10	-49.5
4 WALL	10	15.7	-17.7	16.8	-17.7	17.7	-17.7	18.5	-17.7
	20	15.7	-17.7	16.8	-17.7	17.7	-17.7	18.5	-17.7
	50	14.3	-16.9	15.3	-16.9	16.1	-16.9	16.7	-16.9
	100	13.6	-16.1	14.5	-16.1	15.3	-16.1	15.9	-16.1
5 WALL	10	15.7	-32.5	16.8	-32.5	17.7	-32.5	18.5	-32.5
	20	15.7	-32.5	16.8	-32.5	17.7	-32.5	18.5	-32.5
	50	14.3	-29.2	15.3	-29.2	16.1	-29.2	16.7	-29.2
	100	13.6	-25.9	14.5	-25.9	15.3	-25.9	15.9	-25.9

NOTES:

1. ZONES ARE PORTIONS OF THE WALLS OR ROOFS WHERE WIND LOADS ARE APPLIED, AS SHOWN ON THE "COMPONENT AND CLADDING LOAD DIAGRAM". THE WIDTH OF THE EDGE STRIPS "a" SHALL BE 10 PERCENT OF THE LEAST HORIZONTAL DIMENSION OR 40 PERCENT OF THE EAVE HEIGHT "h", WHICHEVER IS LESS, BUT NOT LESS THAN EITHER 4 PERCENT OF THE LEAST HORIZONTAL DIMENSION OR 3 FEET.
2. AREA IS THE "EFFECTIVE WIND AREA" ON ELEMENTS OF THE COMPONENTS AND CLADDING, AND CLADDING FASTENERS, AS DEFINED IN IBC 1609.2.
3. BASIC WIND LOAD IS THE WIND LOAD ON COMPONENTS AND CLADDING FOR A BUILDING WITH A MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B, AS SHOWN IN THE IBC TABLE 1609.6.2.1(2).
4. BASIC WIND LOAD IS THE WIND LOAD ON ROOF OVERHANG COMPONENTS AND CLADDING FOR A BUILDING WITH A MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B, AS SHOWN IN IBC TABLE 1609.6.2.1(3).
5. DESIGN WIND LOADS ARE THE BASIC WIND LOADS, MULTIPLIED BY THE APPROPRIATE HEIGHT AND EXPOSURE COEFFICIENT FROM TABLE 1609.6.2.1(4) AND IMPORTANCE FACTOR FROM TABLE 1604.5.

WIND LOAD TABLES FOR AREAS C & D

Zone (Note 1)	Area (sq ft) (Note 2)	Design Wind Loads (psf) 0' to 30' (Note 3)	Design Wind Loads (psf) 30' to 40' (Note 3)	Design Wind Loads (psf) 40' to 50' (Note 3)	Design Wind Loads (psf) 50' to 60' (Note 3)
2 ROOF OVERHANG	10	-35.6	-38.4	-40.7	-42.6
	20	-34.2	-36.8	-39.1	-40.9
	50	-31.3	-33.8	-35.8	-37.4
	100	-29.9	-32.2	-34.1	-35.7
3 ROOF OVERHANG	10	-48.4	-52.3	-55.5	-58.1
	20	-46.2	-49.9	-53.0	-55.5
	50	-43.4	-46.9	-49.7	-52.1
	100	-41.3	-44.5	-47.3	-49.5

8 APPENDIX C: SEISMIC LOADS

USGS Design Maps Summary Report

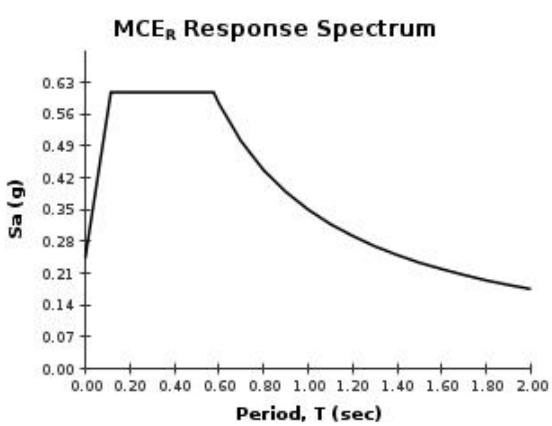
User–Specified Input

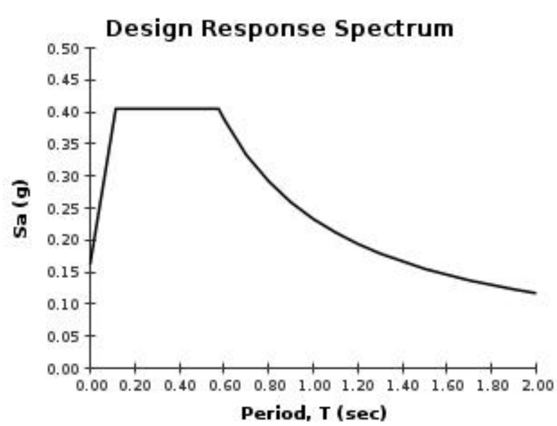
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	38.52197°N, 90.4727°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	IV (e.g. essential facilities)

USGS–Provided Output

$S_S =$	0.414 g	$S_{MS} =$	0.608 g	$S_{DS} =$	0.405 g
$S_1 =$	0.163 g	$S_{M1} =$	0.350 g	$S_{D1} =$	0.233 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





Design Parameters

Categories	Parameter	Value	Units	Description	Reference
Site Code Factors	Occ. Category	IV		Occupant Category	Table 1-1
	Site Class	D		Site Class (A, B, C, D, E, or F)	Chapter 20
	SDC	D		Seismic Design Category	11.6-11.7
	I_E	1.50		Seismic Importance Factor	Table 11.5-1
Seismic Response	S_S	0.414		Short Period MCE Spectral Response Acceleration (%g)	Figure 22-1
	S_1	0.163		One Second MCE Spectral Response Acceleration (%g)	Figure 22-1
	F_a	1.468		Site Coefficient at Short Periods	Table 11.4-1
	F_v	2.148		Site Coefficient at 1 Second Period	Table 11.4-2
Period	T_L	12.00	s	Long-period Transition Period	Figure 22-15
	T_b	999.00	s	Building Period determined from Modal Analysis	
	C_t	0.02		Building Period Coefficient	12.8.1.1
	x	0.75		Building Period Coefficient	
	h_n	106.00	ft	Height of building	
	C_u	1.47			
	N	0.00	#	Number of Stories (leave blank unless apprx Ta desired)	
SFRS Coefficients	R	6.00		Response Modification Coefficient	Table 12.2-1
	Ω	2.50		Overstrength Factor	Table 12.2-1
	C_d	5.00		Deflection Amplication Factor	Table 12.2-1
Shear Wall Data	Concrete/masonry shear walls?	NO			
	Direction	X		X or Y?	
	Ab	1200	sqft	Area of base of Structure	

Intermediate Calculations

Categories	Calculated Values	Value	Units	Description	Reference
Seismic Response	S_{MS}	0.608		Short Period MCE Spectral Response Acc., site adjusted	Eq. 11.4-1
	S_{M1}	0.350		One Second MCE Spectral Response Acc., site adjusted	Eq. 11.4-2
	S_{DS}	0.405		5% Damped Design Spectral Response Acc. at Short Periods	Eq. 11.4-3
	S_{D1}	0.233		5% Damped Design Spectral Response Acc. at 1 Second Period	Eq. 11.4-4
	S_a	2.981		Design Spectral Response Acceleration	11.4.5
Periods	T_a	0.66	s	Approximate Fundamental Period	12.8.2
	T_0	0.12	s		
	T_s	0.58	s		
	T	0.97	s	Period of the Structure	
Coefficients	C_w	0.00		Shear Wall Coefficient	12.8-10
	Cs	0.060		T<=TL	
	Cs	0.101		T>TL	
	Cs	0.101		S1>0.6g	
	$C_{s\ final}$	0.060		Seismic Response Coefficient	12.8.1.1
Base Shear	V	1434.59		Base Shear	

Story Data and Forces											
Story	Dia.	Wx (kips)	Hx (ft)	k	Hx^k	$Wx*Hx^k$	Cvx	Fx	0.2SdsIwpx	0.4SdsIwpx	Fpx (diaph.)
First Floor	1	5076.90	16.00	1.23	30.66	155682.65	0.05	77.89	617.10	1234.20	617.10
Second Floor	2	4987.50	32.00	1.23	72.16	359902.54	0.13	180.07	606.23	1212.47	606.23
Third Floor	3	3417.40	46.00	1.23	112.95	385998.01	0.13	193.12	415.39	830.77	415.39
Fourth Floor	4	3417.40	60.00	1.23	156.80	535862.22	0.19	268.10	415.39	830.77	415.39
Fifth Floor	5	3147.40	74.00	1.23	203.15	639381.21	0.22	319.90	382.57	765.14	382.57
Roof	6	3141.90	88.00	1.23	251.60	790509.88	0.28	395.51	381.90	763.80	395.51
Penthouse Roof	7	640.00	106.00	1.23	316.59	202619.66	0.07	101.37	77.79	155.58	101.37
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
				1.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		23828.50	422.00	7.41	827.33	2867336.51	1.00	1434.59			2933.56