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

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

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.

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)

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.

Christopher Brandmeier | Structural Option
https://www.engr.psu.edu/ae/thesis/portfolios/2015/aqb5205/index.html

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 procedures.

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
  o Produced by Stock & Associates Consulting Engineers Inc.
- SSM St. Clare Health Center Replacement Hospital Project Manual
  o CP-11 E/T Document Issuance
- IBC
  o 2003 Edition (as reference)
  o 2012 Edition (for further design studies)
- ASCE 7
  o ASCE 7-02 (as reference)
  o ASCE 7-10 (for further design studies and load calculations)
- ACI 318
  o 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

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Value (psf)</th>
<th>Code Minimum (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating Room</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>Offices</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Private Rooms</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Corridors (1st Floor)</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Corridors (other)</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>Stairs and Exits</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Equipment Rooms</td>
<td>125</td>
<td>125</td>
</tr>
</tbody>
</table>

Table 2: Typical Floor Dead Loads

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Original Design Values (psf)</th>
<th>Thesis Calculated Values (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hospital Floor</td>
<td>60</td>
<td>64</td>
</tr>
<tr>
<td>Hospital Roof</td>
<td>78</td>
<td>70</td>
</tr>
</tbody>
</table>

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

3 PLY SYSTEM
INSULATION
3.5" LW CONC.
3" 18G DECK
FRAMING BELOW

PENTHOUSE ROOF

3 PLY: 1 psf
5 9/16" Ins: 1.5(5.75) = 8.625
3.5" Conc: 48 psf
3" Deck:
FRAMING: 6 psf
MISC: 6 psf
\[ \frac{70}{\text{psf}} \]

LOW ROOF:

3 PLY: 1 psf
11" Ins: 1.5(11) = 16.5 psf
3.5" Conc: 48 psf
3" Deck:
FRAMING: 6 psf
MISC: 6 psf
\[ \frac{78}{\text{psf}} \]

ROOF LIVE:

20 PSF (REQUIRED BY ASCE 7-10)

NOTE: LESS THAN SNOW LOAD
FLOOR CONSTRUCTION DEAD:

DECK 1:

FINISHES
3½" LW CONC
17G 3" Metal Deck
FRAMING
Acoustical Ceiling or Gypsum (taken as misc)

FINISHES: 2 psf
DECK: 45 psf
FRAMING: 7 psf
CEILING: 2 psf
MISC/EQUIP: 5 psf

64 psf

DECK 2:

FINISHES
5" NW CONC
18G 3" Metal Deck
FRAMING
Acoustical Ceiling or Gypsum

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

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

\[ p_f = 0.7 \cdot c_e \cdot c_i \cdot i_3 \cdot p_g \]

Terrain Category: B

\[
\begin{align*}
    c_e & = 1.0 \\
    c_i & = 1.0 \\
    i_3 & = 1.2 \Rightarrow OC IV \\
    \text{SNOW LOAD} & = 20 \text{ psf}
\end{align*}
\]

\[ p_f = 0.7 \cdot (1.0)(1.0)(1.2)(20) \]

\[ = 16.8 \text{ psf} \]

\[ p_m = i_3 \cdot p_g \text{ for } p_g \leq 20 \text{ psf} \]

\[ = 24 \text{ psf} \]

- Drifts on Lower Roofs

Penthouse Roof:

\[ y = 0.13(20) \cdot 14 = 16.6 \text{ pcf} \]

\[ h_y = \frac{24}{16.6} = 1.445' \]

\[ h_d = 2' \]

\[ h_c = 16.75' \]

\[ W = 34.83' \times 50' \]

\[ l_u = 97' \times 100' \]

\[ h_d = 3' \]

\[ h_c = 16.75' \]

\[ W = 12 \]

N-S Elevation

\[ 24 \text{ psf} \]

\[ 4.5' \]

\[ 12' \]

\[ 72.8 \text{ psf} \]

\[ 18.75' \]

\[ 12' \]
GARDEN LEVEL ROOFS:

Area 1 = Area 2

\( h_u = 7.7' \approx 100' \)
\( h_d = 8' \)
\( h_c = 53' \)
\( \text{drift max} = \frac{24}{16.6} (16.6) + 3(16.6) \)
\( = 73.8 \text{ psf} \)

Area 3

\( h_u = 52' \approx 50' \)
\( h_d = 2' \)
\( h_c = 54' \)
\( \text{drift max} = \frac{24}{16.6} (16.6) + 2(16.6) \)
\( = 57.2 \text{ psf} \)

PARAPET DRIFT

1) \( h_u = 100' \)
\( h_d = 6.5' \)
\( h_c = -4.3' \)
\( \text{drift max} = 24 + 6.5(0.75)16.6 \)
\( = 104.93 \text{ psf} \)

2) \( h_u = 100' \)
\( h_d = 8' \)
\( h_c = -0.85' \)
\( \text{drift max} = 24 + 8(0.75)16.6 \)
\( = 61.35 \text{ psf} \)
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>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occupancy Category</td>
<td>-</td>
<td>IV</td>
</tr>
<tr>
<td>Basic Wind Speed</td>
<td>V</td>
<td>115 mph</td>
</tr>
<tr>
<td>Exposure Category</td>
<td></td>
<td>B</td>
</tr>
<tr>
<td>Wind Directionality Factor</td>
<td>Kd</td>
<td>0.85</td>
</tr>
<tr>
<td>Importance Factor</td>
<td>Ie</td>
<td>1.5</td>
</tr>
<tr>
<td>Topographical Factor</td>
<td>Kzt</td>
<td>1.0</td>
</tr>
<tr>
<td>Gust Effect Factor</td>
<td>G</td>
<td>0.8205</td>
</tr>
<tr>
<td>Enclosure Classification</td>
<td></td>
<td>Enclosed</td>
</tr>
</tbody>
</table>

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

\[ K_1 = 0.85 \]
\[ K_2 = 1.0 \]
\[ G = ? \]

APPROXIMATE NATURAL FREQUENCY:

\[ L_{eff} = 3.93 \]

1. \[ h = 90 < 300 \checkmark \]
2. \[ h = 90 < 4(3.93) = 157.2 \]

USE EQUATION 26.9-4

\[ n_a = \frac{75}{h} \]
\[ = \frac{75}{90} \]
\[ = 0.8333 \]

CALCULATE G:

\[ L_2 = 820 \left( \frac{54}{35} \right)^{0.385} = 377.09 \]

\[ Q = \sqrt{\frac{1}{1 + 0.03 \left( \frac{77.90}{377.09} \right)^{0.65}}} = 0.75214 \]

\[ V_2 = 0.45 \left( \frac{54}{35} \right)^{0.25} \left( \frac{88}{60} \right) \left( 115 \right) = 85.844 \]

\[ N_1 = \frac{0.833 \left( 87.9.90 \right)}{85.844} = 3.66 \]

\[ R_n = \frac{7.47 \left( 3.66 \right)}{\left( 1 + 10.3 \left( 3.66 \right)^{0.5} \right)} = 0.0417 \]

\[ R_L = \frac{1}{4.02} - \frac{1}{2(4.02)^2} \left( 1 - e^{-2(4.02)} \right) = 0.2179 \]

\[ R_0 = \frac{1}{4.02} - \frac{1}{2(4.02)^2} = 0.2484 \]

\[ R_L = \frac{1}{4.02} = 0.0554 \]
\[ R = \sqrt{\frac{1}{0.05} \left( 0.0612(0.2179)(0.2496)(0.53 + 0.47(0.0554)) \right)} \]
\[ = 0.1928 \]

\[ y_r = \sqrt{\frac{2 \ln(3600(0.8333)) + 0.597}{2 \ln(3600)(0.8333)}} \]
\[ = 4.1458 \]

\[ I_z = 0.8 \left( \frac{83}{54} \right)^{1/2} = 0.2764 \]

\[ (\varepsilon - \eta) \quad g_f = 0.925 \left( \frac{1 + 1.7(0.2764)}{1 + 1.7(3.4)(0.2764)} \right)^{3.4^2(0.832)^2 + (4.146)^2(0.1928)^2} \]
\[ = 0.925 \left( \frac{2.597}{2.597} \right) \]
\[ = 0.8591 \]
REPRESENTATIVE CALCULATION OF "P"

at $z = 90'$

$q_1h = 0.00256 (0.96)(1.0)(0.85)(115)^2$

$\approx 27.626$

at $z = 58'$

$q_2h = 0.00256 (0.85)(1.0)(0.85)(115)^2$

$\approx 24.46$

$p = \frac{24.46 (0.8)(0.859) -(27.626(0.18))}{21.78}$

Note that this result assumes a negative pressure on the interior of the building.

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.
WIND LOADS

VALUES IN PSF

N-S

19.5

7.3

18.2

17.1

15.8

14.2

11.9

SITE ELEV.

5.0

1st

2nd

3rd

4th

5th

6th

7th

OPEN TO OTHER STRUCTURE

E-W

18.3

11.5

17.2

16.1

14.9

13.4

11.2

5.0

2nd

3rd

4th

5th

6th

7th

OPEN TO OTHER STRUCTURE

GROUND
### Table 4: Wind Loads in the East-West Direction

<table>
<thead>
<tr>
<th>Location</th>
<th>Story Height (ft)</th>
<th>z (ft)</th>
<th>qzGCp (psf)</th>
<th>Tributary Width</th>
<th>External Pressure (kip)</th>
<th>Gpi</th>
<th>qhGCpi (psf)</th>
<th>Tributary Width</th>
<th>Internal Pressure (kip)</th>
<th>Total Pressure (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward</td>
<td>-16</td>
<td>16.0</td>
<td>11.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>16.0</td>
<td>11.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>14.0</td>
<td>11.2</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>14.0</td>
<td>13.4</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>44</td>
<td>14.0</td>
<td>14.9</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>14.0</td>
<td>16.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>72</td>
<td>18.8</td>
<td>17.2</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>90.75</td>
<td>0.0</td>
<td>18.3</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Leeward</td>
<td>90.75</td>
<td>90.8</td>
<td>-11.5</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Parapet WW</td>
<td>93</td>
<td>2.2</td>
<td>34.6</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Parapet LW</td>
<td>93</td>
<td>2.2</td>
<td>23.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Base Shear: 1085.2 kips

### Table 5: Wind Loads in the North-South Direction

<table>
<thead>
<tr>
<th>Location</th>
<th>Story Height (ft)</th>
<th>z (ft)</th>
<th>qzGCp (psf)</th>
<th>Tributary Width</th>
<th>External Pressure (kip)</th>
<th>Gpi</th>
<th>qhGCpi (psf)</th>
<th>Tributary Width</th>
<th>Internal Pressure (kip)</th>
<th>Total Pressure (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward</td>
<td>-16</td>
<td>16.0</td>
<td>11.7</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>16.0</td>
<td>11.7</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>14.0</td>
<td>11.9</td>
<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
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<td>0.0</td>
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<td>0.0</td>
<td>0.0</td>
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<td>0.0</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.18</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
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</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
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<tbody>
<tr>
<td>Occupancy Category</td>
<td>-</td>
<td>IV</td>
</tr>
<tr>
<td>Site Class</td>
<td></td>
<td>D</td>
</tr>
<tr>
<td>Seismic Design Category</td>
<td>-</td>
<td>D</td>
</tr>
<tr>
<td>Short Period Spectral Response Acceleration</td>
<td>$S_s$</td>
<td>0.414</td>
</tr>
<tr>
<td>One Second Spectral Response Acceleration</td>
<td>$S_1$</td>
<td>0.163</td>
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</table>

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

<table>
<thead>
<tr>
<th>Story</th>
<th>Dia.</th>
<th>Wx (kips)</th>
<th>Hx (ft)</th>
<th>k</th>
<th>Hxk</th>
<th>Wx*Hxk</th>
<th>Cvx</th>
<th>Fx</th>
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<tr>
<td>First Floor</td>
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<td>5076.90</td>
<td>16.00</td>
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<td>30.66</td>
<td>155682.65</td>
<td>0.05</td>
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<td>72.16</td>
<td>359902.54</td>
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<td>Roof</td>
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<td>3141.90</td>
<td>88.00</td>
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<td>0.28</td>
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<td>Penthouse Roof</td>
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<td>827.3</td>
<td>2867336.5</td>
<td>1</td>
<td>1434.6</td>
</tr>
</tbody>
</table>
SEISMIC MASS CALCULATION

FAÇADE AREAS

1, 2 FLOORS:

\[ P = (72.33') + (41') + (41.7') + (42') + (108.5') + (183.6') + (86.5') + (113.7') + (22.6') + (166') \]

\[ H = 16' \]

\[ P = 1183.2' \]

\[ A = 56167.38 \text{ ft}^2 \] (FROM PROGRAM)

3, 4, 5, 6 FLOORS:

\[ P = (41.7') + (214.7') + (158.33') + (113.7') + (11.33') + (86.5') + 77.33'(2) \]

\[ H = 14' \]

\[ P = 1078.19' \]

\[ A = 36158.84 \text{ ft}^2 \] (FROM PROGRAM)
7 FLOOR

\[ H = 18.802' \]

\[ P = (38 + 40)(z) + (113') + 67(z) + 2(31) \]
\[ = 522' \]

\[ A = 31(113) + 67(33) \]
\[ = 5714 \text{ sf} \]
- FACADE MASS:
  ASSUME 30% FENESTRATION @ 25 lbs/ft²

1) \[ m = 1123.2 \left(\frac{16}{2}\right) \left(0.3(25) + 0.7(51)\right) \]
   \[ = 817827.72 \text{ lbs} \]
   \[ = 818 \text{ kips} \]

2) \[ m = \left(\frac{818}{2}\right) \left(\frac{652}{2}\right) = 235 \text{ kips} \]

3,4,5) \[ m = 1078.1 \left(\frac{14}{2}\right) \left(0.3(25) + 0.7(51)\right) \]
   \[ = 652089.3 \text{ lbs} \]
   \[ = 652 \text{ kips} \]

6) \[ m = 522 \left(\frac{51}{10}\right) \left(\frac{18.09}{2}\right) + \left(\frac{652}{2}\right) = 566 \text{ kips} \]

7) \[ m = 522 \left(\frac{51}{10}\right) \left(\frac{18.09}{2}\right) = 240 \text{ kips} \]

- FLOOR MASS

1,2) \[ m = 56167.38 \left(\frac{64 + 10}{10}\right) \]
   \[ = 415653.41 \text{ lbs} \]
   \[ = 4156.5 \text{ kips} \]

3,4,5) \[ m = 36158.84 \left(\frac{64 + 10}{10}\right) \]
   \[ = 2675954.16 \text{ lbs} \]
   \[ = 2675.9 \text{ kips} \]

6) \[ m = 36158.84 \left(\frac{70}{10}\right) \]
   \[ = 253111.8 \text{ lbs} \]
   \[ = 2531.1 \text{ kips} \]

7) \[ m = 5714 \left(70\right) = 400 \text{ kips} \]

- SHEAR WALL MASS
  ASSUME 30% OPENING

1) \[ m = 150 \left(\frac{7}{12}\right) \left(32\right) \left(\frac{14}{2}\right) \]
   \[ = 102700 \text{ lbs} \]
   \[ = 102.7 \text{ kips} \]

2) \[ m = 150 \left(\frac{7}{12}\right) \left(32\right) \left(\frac{16.5+14}{2}\right) = 96 \text{ kips} \]

3,4,5) \[ m = 150 \left(\frac{7}{12}\right) \left(32\right) \left(\frac{14}{2}\right) \]
   \[ = 89.6 \text{ kips} \]

6) \[ m = \frac{89.6}{2} = 44.8 \text{ kips} \]

7) \[ m = 0 \]

- FRAMING MASS

1,2) \[ m = 56167.38 \left(\frac{10}{2}\right) \]
   \[ = 561.7 \text{ k} \]

3,4,5,6) \[ m = 36158.84 \left(\frac{10}{2}\right) \]
   \[ = 361.6 \text{ k} \]

7) \[ m = 5714 \left(10\right) \]
   \[ = 57.1 \text{ k} \]
SEISMIC MASS: \( M = M_{\text{facade}} + M_{\text{floor}} + M_{\text{shear wall}} \)

7
\[ m_7 = 240 + 400 = 640 \] (112)

6
\[ m_6 = 506.6 \text{ k} + 2531.1 \text{ k} + 44.8 \text{ k} = 3141.9 \text{ k} \] (2.69)

5
\[ m_5 = 3417.4 \text{ k} \] (2.93)

4
\[ m_4 = 3417.4 \text{ k} \] (2.93)

3
\[ m_3 = 652 + 2675.75 + 87.6 + 307 = 3417.4 \text{ k} \] (2.93)

2
\[ m_2 = 935 + 4156.5 + 96 + 521.8 = 5487.5 \text{ k} \] (2.75)

1
\[ m_1 = 818 + 4156.5 + 102.4 \text{ k} + 5076.9 \text{ k} = 5076.9 \text{ k} \] (2.80)
SEISMIC LOADS

\[ \begin{align*}
\text{101.4 k} & \rightarrow \text{FENT} \\
395.5 \text{ k} & \rightarrow 6\text{M} \\
319.9 \text{ k} & \rightarrow 5\text{M} \\
268.1 \text{ k} & \rightarrow 4\text{H} \\
173.2 \text{ k} & \rightarrow 3\text{D} \\
180.1 \text{ k} & \rightarrow 2\text{D} \\
79.89 \text{ k} & \rightarrow 1\text{F} \\
\text{GROUND} & 
\end{align*} \]

OVERTURNING

\[ M_0 = 77.89 \text{ (16)} + 180.1 \text{ (32)} + 193.2 \text{ (46)} + 268.1 \text{ (60)} \\
+ 319.9 \text{ (74)} + 395.5 \text{ (88)} + 101.4 \text{ (106)} \]

\[ = 101204.4 \text{ ft} \cdot \text{k} \]

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 assumptions 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 7: Example SCBF  
Figure 8: Example SRCSW  
Figure 9: Example SMF
### 4.2 Assumptions

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

<table>
<thead>
<tr>
<th>Lateral Component</th>
<th>Element Type</th>
<th>Assumptions</th>
<th>Modifiers</th>
</tr>
</thead>
</table>
| 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 6th 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)

<table>
<thead>
<tr>
<th>Element</th>
<th>Story</th>
<th>Disp.</th>
<th>Rel. K (k/in)</th>
<th>Dist X</th>
<th>Dist Y</th>
<th>Ri*Xi</th>
<th>Sum(Ri*Xi)</th>
<th>Sum(Ri)</th>
<th>COR</th>
<th>Model Values</th>
<th>% Error</th>
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<td>39518.98028</td>
<td>272</td>
<td>145.1722</td>
<td>138.02</td>
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<tr>
<td>1A-smf</td>
<td>4</td>
<td>0.012420</td>
<td>80.52</td>
<td>0.000</td>
<td>191.165</td>
<td>15391.70692</td>
<td></td>
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<td>4B-smf</td>
<td>4</td>
<td>0.020370</td>
<td>49.09</td>
<td>0.000</td>
<td>113.749</td>
<td>5584.128621</td>
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<td></td>
</tr>
<tr>
<td>4A-smf</td>
<td>4</td>
<td>0.012420</td>
<td>80.52</td>
<td>0.000</td>
<td>113.749</td>
<td>9158.510467</td>
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<td></td>
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<tr>
<td>8-smf</td>
<td>4</td>
<td>0.076880</td>
<td>13.01</td>
<td>0.000</td>
<td>0.000</td>
<td>0</td>
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<td></td>
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<tr>
<td>1B-smf</td>
<td>5</td>
<td>0.027780</td>
<td>36.00</td>
<td>0.000</td>
<td>191.165</td>
<td>6881.389489</td>
<td>29147.27257</td>
<td>201</td>
<td>145.1857</td>
<td>139.952</td>
<td>3.74</td>
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<td>1A-smf</td>
<td>5</td>
<td>0.016780</td>
<td>59.59</td>
<td>0.000</td>
<td>191.165</td>
<td>11392.43147</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4B-smf</td>
<td>5</td>
<td>0.027780</td>
<td>36.00</td>
<td>0.000</td>
<td>113.749</td>
<td>4094.62563</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4A-smf</td>
<td>5</td>
<td>0.016780</td>
<td>59.59</td>
<td>0.000</td>
<td>113.749</td>
<td>6778.825983</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8-smf</td>
<td>5</td>
<td>0.104440</td>
<td>9.57</td>
<td>0.000</td>
<td>0.000</td>
<td>0</td>
<td></td>
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<td></td>
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<tr>
<td>1B-smf</td>
<td>6</td>
<td>0.036170</td>
<td>27.65</td>
<td>0.000</td>
<td>191.165</td>
<td>5285.181089</td>
<td>22507.28806</td>
<td>155</td>
<td>145.2776</td>
<td>141.6316</td>
<td>2.57</td>
</tr>
<tr>
<td>1A-smf</td>
<td>6</td>
<td>0.021660</td>
<td>46.17</td>
<td>0.000</td>
<td>191.165</td>
<td>8825.715605</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4B-smf</td>
<td>6</td>
<td>0.036170</td>
<td>27.65</td>
<td>0.000</td>
<td>113.749</td>
<td>3144.835499</td>
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<td></td>
</tr>
<tr>
<td>4A-smf</td>
<td>6</td>
<td>0.021660</td>
<td>46.17</td>
<td>0.000</td>
<td>113.749</td>
<td>5251.555863</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8-smf</td>
<td>6</td>
<td>0.137070</td>
<td>7.30</td>
<td>0.000</td>
<td>0.000</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
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.
COM. ROUGH CHECK

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

\[
\begin{aligned}
x & = 205.4' \\
y & = (46.3') \rightarrow 191.165 - 46.3' = 144.9'
\end{aligned}
\]

MODEL VALUES
\[
\begin{aligned}
x & = 205.067' \\
y & = 143.076'
\end{aligned}
\]

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

<table>
<thead>
<tr>
<th></th>
<th>Fx (kip)</th>
<th>Fy (kip)</th>
<th>Fz (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model X</td>
<td>-453.585</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Model Y</td>
<td>0</td>
<td>-1190.67</td>
<td>0</td>
</tr>
<tr>
<td>Manual X</td>
<td>454.1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Manual Y</td>
<td>0</td>
<td>1085.2</td>
<td>0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th></th>
<th>F (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td>-1866.7</td>
</tr>
<tr>
<td>Manual</td>
<td>1434.6</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Element</th>
<th>Story</th>
<th>Rel. K (k/in)</th>
<th>COM</th>
<th>COR X</th>
<th>COR Y</th>
<th>e(acc.)</th>
<th>Da</th>
<th>j</th>
<th>Applied Load</th>
<th>Direct Shear Coefficient</th>
<th>Torsional Shear</th>
<th>Direct Shear</th>
<th>Total Shear</th>
<th>Model Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-srcsw</td>
<td>6</td>
<td>244.50</td>
<td>143.07</td>
<td>195.16</td>
<td>145.27</td>
<td>9.55</td>
<td>29.492</td>
<td>18653024.24</td>
<td>0</td>
<td>0.25265874</td>
<td>2.820727</td>
<td>-1.76446</td>
<td>0</td>
<td>2.820727</td>
</tr>
<tr>
<td>N-srcsw</td>
<td>6</td>
<td>284.90</td>
<td>143.07</td>
<td>195.16</td>
<td>145.27</td>
<td>9.55</td>
<td>0.4768</td>
<td>18653024.24</td>
<td>0</td>
<td>0.294048617</td>
<td>0.053139</td>
<td>-0.03247</td>
<td>0</td>
<td>0.053139</td>
</tr>
<tr>
<td>Aa1-scbf</td>
<td>6</td>
<td>101.42</td>
<td>143.07</td>
<td>195.16</td>
<td>145.27</td>
<td>9.55</td>
<td>0.10480469</td>
<td>18653024.24</td>
<td>0</td>
<td>0.11111551</td>
<td>8.208959</td>
<td>-5.13497</td>
<td>0</td>
<td>8.208959</td>
</tr>
<tr>
<td>Aa3-scbf</td>
<td>6</td>
<td>107.53</td>
<td>143.07</td>
<td>195.16</td>
<td>145.27</td>
<td>9.55</td>
<td>0.19516</td>
<td>18653024.24</td>
<td>0</td>
<td>0</td>
<td>18653024.24</td>
<td>0</td>
<td>7.4273</td>
<td>0</td>
</tr>
<tr>
<td>Ra-scbf</td>
<td>6</td>
<td>229.36</td>
<td>143.07</td>
<td>195.16</td>
<td>145.27</td>
<td>9.55</td>
<td>209.84</td>
<td>18653024.24</td>
<td>0</td>
<td>0.237012442</td>
<td>18.82704</td>
<td>-11.77694</td>
<td>0</td>
<td>18.82704</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Story</th>
<th>Pier</th>
<th>Load Case/Combo</th>
<th>Location</th>
<th>P (kip)</th>
<th>V2 (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Story1</td>
<td>smf-1B</td>
<td>WX Max</td>
<td>Bottom</td>
<td>0.372</td>
<td>71.045</td>
</tr>
<tr>
<td>Story1</td>
<td>smf-1A</td>
<td>WX Max</td>
<td>Bottom</td>
<td>7.003</td>
<td>105.246</td>
</tr>
<tr>
<td>Story1</td>
<td>smf-4B</td>
<td>WX Max</td>
<td>Bottom</td>
<td>0.961</td>
<td>71.396</td>
</tr>
<tr>
<td>Story1</td>
<td>smf-4A</td>
<td>WX Max</td>
<td>Bottom</td>
<td>7.469</td>
<td>105.451</td>
</tr>
<tr>
<td>Story1</td>
<td>smf-5A</td>
<td>WX Max</td>
<td>Bottom</td>
<td>0</td>
<td>26.93</td>
</tr>
<tr>
<td>Story1</td>
<td>smf-5B</td>
<td>WX Max</td>
<td>Bottom</td>
<td>0</td>
<td>43.802</td>
</tr>
<tr>
<td>Story1</td>
<td>smf-8B</td>
<td>WX Max</td>
<td>Bottom</td>
<td>0</td>
<td>16.621</td>
</tr>
<tr>
<td><strong>Total:</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>440.491</strong></td>
<td></td>
</tr>
</tbody>
</table>

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**

\[ h = 108.25' \]

\[ h(0.025) = \delta_x = \frac{C_d}{I_e} \delta_{xe} \]

\[ 108.25(12)(0.025) = \delta_x = \frac{5.5}{1.5} (3.06') \]

\[ 32.475'' = \delta_x = 11.22'' \checkmark \]

**WIND**

\[ \frac{n}{400} = \delta_{max} \]

\[ \frac{108.25(12)}{400} > 1.57'' \]

\[ 3.25 > 1.57'' \checkmark \]
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.
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 10x10 x 1/2
L = 19.8
K = 1.0
P_n = 186.7 kips (seismic load case)

CAPACITY:
C = \phi_c P_n = 549 @ KL = 20' \hspace{1cm} [AISC 530 TABLE 4-4]
T = \phi_t P_n = 561 k \hspace{1cm} [AISC 530 TABLE 5-5]

186.7 k < 549 k < 561 k \checkmark
COLUMN MEMBER CHECK

SECTION: W 41 x 233

\[ M_{xs} = 175 \text{ kft} \]
\[ M_{ys} = 110.4 \text{ kft} \]
\[ P_{xs} = 414.3 \text{ k} \]
\[ M_{xw} = 69.6 \text{ kft} \]
\[ M_{yw} = 44.1 \text{ kft} \]
\[ P_{xw} = 163 \text{ k} \]

\textbf{CASES:}
\[ 1.2D + 1.0(E_h + E_v) + 1.0L + 0.25 \]
\[ 1.2D + 1.0W + L + 0.5S \]

\[ T-AREA = \frac{(15.33 + 23.33)}{2} \]
\[ = 28.95 \text{ ft}^2 \]
\[ DL = 64 (6) (290) \]
\[ = 11,360 \text{ lbs} \]
\[ = 111.4 \text{ k} \]
\[ LL = 80 \left\{ \begin{array}{l}
0.5 \\
0.025 \times \frac{15}{290(s)}\end{array} \right\} \]
\[ = 80 \left\{ \begin{array}{l}
0.5 \\
0.61\end{array} \right\} \]
\[ = 51 \text{ psf} \]
\[ 51 (5) (290) \]
\[ = 73.9 \text{ k} \]
\[ SL = 24(1) (290) \]
\[ = 6.9 \text{ k} \]

\textbf{ASSUMED:} \[ E_h + E_v \]

\textbf{CASES:}
\[ S = 12(111.4) + 1.0(44.3) + (73.9 k) + 0.2(6.9 k) = 623.26 k \]
\[ W = 12(111.4) + 1.0(14.5) + (73.9 k) + 0.5(6.9 k) = 374.03 k \]

SEISMIC FORCES CONTROL.

INTERACTION EQ. USING ELM:

\[ KL = 1.0(6^2 - 24^2) = 14^2 \]
\[ p_{x} = 0.367 \times 10^{-3} \]
\[ p_{y} = 0.594 \times 10^{-3} \]
\[ b_{x} = 1.09 \times 10^{-3} \]
\[ b_{y} = 1.09 \times 10^{-3} \]

\[ p_{y} + 0.5 b_{x} M_{x} + b_{y} M_{y} \leq 1.0 \]
\[ 0.367 (623.26) + 0.594 (145) + 1.09 (110.6) \leq 1.0 \]
\[ 0.45 \leq 1.0 \checkmark \]
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 x 55

- \( P_n = 0 \)
- \( M_{x5} = 316.5 \)
- \( M_{x(d+L)} = ? \)

**LOAD CASE:**

\[
1.2D + 1.0E + 1.0L + 0.2S
\]

\[
= 1.2 \times 64 \times (23.33) + 1.0 \times 65.35 \times (23.33) / 2
\]

\[
= 16568 \text{ kips}\text{ft}
\]

\[
= 166 \text{ kips}\text{ft}
\]

**\( M_{x(d+L)} = \frac{(166 \times 30)^2}{12} \)**

\[
= 124.5 \text{ kips}\text{ft}
\]

**\( M_{\text{Total}} = M_{x5} + M_{x(d+L)} \)**

\[
= 316.5 + 124.5 \text{ kips}\text{ft}
\]

\[
= 441 \text{ kips}\text{ft}
\]

**TABLE 3-19 → \( \Phi M_p \) non-composite = 503 kips**

**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](image-url)
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)

<table>
<thead>
<tr>
<th>Location</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Hospitals</strong></td>
<td></td>
</tr>
<tr>
<td>Operating rooms, labs</td>
<td>60 **</td>
</tr>
<tr>
<td>Private rooms</td>
<td>40 **</td>
</tr>
<tr>
<td>Wards</td>
<td>40 **</td>
</tr>
<tr>
<td>Corridors (above 1st floor)</td>
<td>80 **</td>
</tr>
<tr>
<td>* Design for uniform load indicated or 1000# concentrated load over 2.5 feet square, whichever produces the greater load effect.</td>
<td></td>
</tr>
<tr>
<td><strong>Offices</strong></td>
<td></td>
</tr>
<tr>
<td>Offices</td>
<td>50 **</td>
</tr>
<tr>
<td>Lobbies &amp; 1st floor corridors</td>
<td>100 **</td>
</tr>
<tr>
<td>Corridors (above 1st floor)</td>
<td>80 **</td>
</tr>
<tr>
<td>** Design for uniform load indicated or 2000# concentrated load over 2.5 feet square whichever produces the greater load effect.</td>
<td></td>
</tr>
<tr>
<td><strong>Misc. Live Loads</strong></td>
<td></td>
</tr>
<tr>
<td>Corridors, except as otherwise indicated</td>
<td>100 **</td>
</tr>
<tr>
<td>Stairs and Exits</td>
<td>100 **</td>
</tr>
<tr>
<td>Dining Rooms and Restaurants</td>
<td>100 **</td>
</tr>
<tr>
<td>Retail Stores (first floor)</td>
<td>100 **</td>
</tr>
<tr>
<td>Mechanical rooms</td>
<td>125 ** (Includes allowance for equipment pads)</td>
</tr>
<tr>
<td>Storage – Light</td>
<td>125 **</td>
</tr>
<tr>
<td>** Design for uniform load indicated or 300# concentrated load over 4 inches square whichever produces the greater load effect</td>
<td></td>
</tr>
<tr>
<td>Partition loads</td>
<td>20 **</td>
</tr>
<tr>
<td>(Offices &amp; locations where partitions are subject to change)</td>
<td></td>
</tr>
</tbody>
</table>

### 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
- Roofing/Insulation/Ballast: 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: 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: 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: 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
- 115 PSF (Mass DL = 94 PSF + 10 PSF for Partition Mass)

Hospital Floor/Power Plant (Composite slab, 2 Hour)
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## WIND LOAD TABLES FOR AREAS A, B, F & G

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### TECHNICAL REPORT 4
### Wind Load Tables for Areas C & D

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#### Wind Load Tables for Areas C & D

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**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 "s" 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 inIBC 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 theIBC 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 inIBC Table 1609.6.2.1.(3).
5. Design wind loads are the basic wind loads, multiplied by the appropriate height and exposure coefficient fromTable 1609.6.2.1(4) and importance factor from Table 1604.5.
8 APPENDIX C: SEISMIC LOADS

USGS Design Maps Summary Report

User-Specified Input

(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

\[
\begin{align*}
S_a &= 0.414 \text{ g} & S_{max} &= 0.608 \text{ g} & S_{rms} &= 0.405 \text{ g} \\
S_r &= 0.163 \text{ g} & S_{max} &= 0.350 \text{ g} & S_{rms} &= 0.233 \text{ g}
\end{align*}
\]

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.

\[
\begin{align*}
\text{MCE} \text{ Response Spectrum} & \\
\text{Design Response Spectrum}
\end{align*}
\]

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.
### Design Parameters

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### Intermediate Calculations

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<th>Hx (ft)</th>
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<th>Hx^k</th>
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### Force Calculations

- \( F_w = 0.2Sdswpx \)
- \( F_d = 0.4Sdswpx \)
- \( F_{px} \) (diaph.)

\[
F_{px} = F_w + F_d
\]