Courtesy of HGA

Senior Thesis Final Report
Bed Tower Addition at Appleton Medical Center
Appleton, WI

Jessel Elliott – Structural
2012 Architectural Engineering
Senior Thesis Studio

Advisor: Dr. Richard Behr

Date: 04/04/2012

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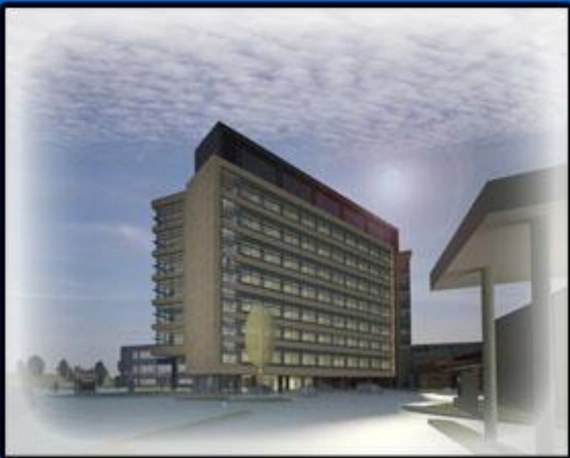
Abstract



Jessel Elliott
Structural Option

Bedtower Addition at Appleton Medical Center

1818 North Meade Street
Appleton, WI 54911

Architecture

- Triangular shape brings a unique form to the overall building layout
- Exterior made of limestone and cast stone bricks which help seperate space
- Large clear windows on the first floor allow natural daylight while tinted windows in the patient rooms make them cooler and dimmer

MEP

Mechanical:

- VAV Box with Reheat Control set to a 75° controlled temperature throughout the building
- Continuous AHU's serving all floors with inter-lock operation of suppy and return fans

Electrical:

- Main power by 480/277V 3 Phase 60 Hz 1200 A
- Other power by 120/208V 3 Phase 60 Hz 100 A
- Majority of equipment located in penthouse

General Information

Owner:	Appleton Medical Center
Occupancy:	Hospital
Size:	152,330 Sq. Ft
Height:	107' - 3"
Construction Date:	June 2008 - January 2011
Cost:	$59,100,000
Project Delivery:	Integrated Project Delivery

Project Team

Construction:	The Boldt Company
Civil:	McMahon Associates
Architect:	Hammel, Green and Abrahamson
Structural:	Hammel, Green and Abrahamson
Mechanical:	Tweet/Garot Mechanical
Electrical:	Excellence Electric
Fire Protection:	J. F. Ahern Co.

Structure

- 3'-6" mat slab foundation with piers
- 9 braced frames to assist in transferring lateral loads to the foundation
- Composite system throughout the entire building consisting of mostly normal weight concrete on metal decking

CPEP Website: http://www.engr.psu.edu/ae/thesis/portfolios/2012/JXE5007/index.html

Acknowledgements

For their kind support and guidance, I would like to thank these individuals and companies for their help during over course of the year:

- Pennsylvania State University
    - Dr. Richard Behr
    - Professor M. Kevin Parfitt
    - Professor Robert Holland
    - The rest of the AE faculty

- Hammel, Green, and Abrahamson (HGA)
    - Brian Genduso

- Appleton Medical Center

- Sheetz, Inc.

- My Architectural Engineering Peers

I would like to dedicate my thesis to family and friends. Without them, I would not be the person I am today. Their support has driven me to become as successful as I can be.

Executive Summary

The Bed Tower Addition at Appleton Medical Center is facility located at 1818 North Meade Street in Appleton, Wisconsin. It is an eight story building with a mechanical penthouse on top which houses 139 extra beds for patients. The existing structure uses a mat slab foundation, composite beam gravity system with slab on deck, and concentrically braced frames.

This report was to focus on two aspects of seismic design. The first phase was to move the rebuild the existing structure's footprint in San Francisco and modify the braced frames to withstand new loads. The second phase was to analyze a base isolation system combined with the existing structure and modified structure. Once the results were obtained, the last phase was to compare the differences between use of the existing structure and modified structure, but to focus on whether base isolation would be feasible alternative system for the new location.

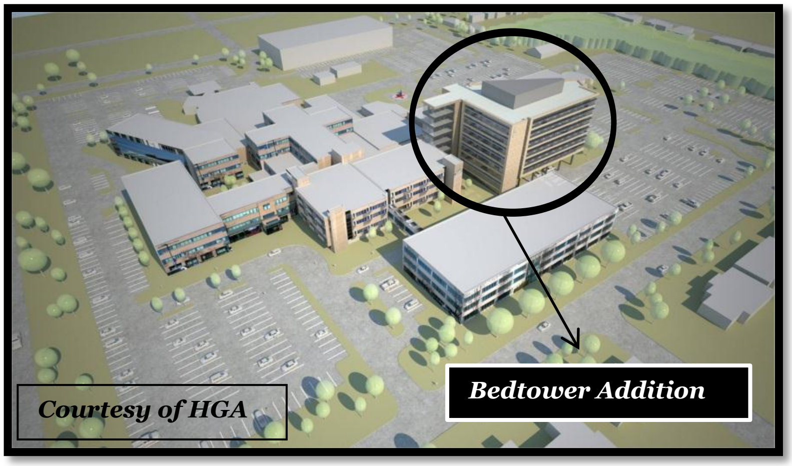
Using ASCE7-10, seismic design parameters were calculated to determine the vertical distribution of forces on the building with and without base isolation. It was then determined that the design base shear for the structure decreased by more than 211% with the use of base isolation. This resulted in a decrease in story forces and shears. Comparison of base isolation with both structures did warrant any conclusive results. The displacements and drifts were very similar to each other with minor differences. A better analysis comparing the various configurations of the base isolation system would have resulted in better comparison. However, the results showed that implementing the base isolation system reduced the drifts to a minimum and all calculations were less than the allowable drift limit.

From the report, it is plausible to say that the existing structure could be moved to a different location if base isolation is implemented. The ramifications of implementing a base isolation system would be costly though. The total project cost grew to $61, 300, 340 with the increase coming from installation costs and schedule changes. Use of base isolation would extend the schedule just 7 days and increased the total project cost by $2,200,340.

Architecture was also taken into consideration. A base isolation system needs a moat to allow for the entire building to displace without disruption. To cover up a moat, breakable lightweight greats were placed on top. The moat was also covered up by using a seismic retaining wall which extends three above the ground level for safety and is also disguised by hedges. Another difference in architecture during the move occurred in the façade. A new façade was designed to match the surroundings of San Francisco State University. Several buildings were used as inspiration and the new look should fit into SFSU.

Introduction

Bed Tower Addition at Appleton Medical Center, owned by ThedaCare is located in Appleton, Wisconsin approximately two hours (~106 miles) northeast from Madison, Wisconsin. The building was measured at a height of 107 ft and 3 in. above grade to the highest occupied floor, which entails 9 stories including a basement. The total size of the addition is 152,330 sq ft. This includes renovation done to the existing hospital plus the new addition itself.



Picture 1: *Bird's eye view of Appleton Medical Center*

The bedtower addition is to accommodate more patients for the hospital. Because of its size, it stands out amongst the rest of the complex. It has a unique triangular shape layout which is carried throughout all the floors of the building. The horizontal streaks of CMU along the exterior make the addition look sleek and long. Accommodating the long streaks are large areas of glass. Both materials work together to show floor separation and this gives the perception that the addition is taller than it actually is.

The first floor is the lobby area which consists of the registration and waiting area along with a mini coffee shop. Offices are located on the second floor area which is a very large space and has movable partitions. Third through eighth floors consist of patient rooms, waiting rooms, and floor manager offices. The second to fourth floors connect to the

Picture 2: *Perspective view of Bed Tower Addition entrance*

original hospital with the fourth floor extended into the original building, which is the emergency and surgery center.

On the exterior of the building, the façade consisted of two essential components which are a stone façade and large areas of glazing. Limestone and Cast Stone, architectural concrete building unit used to simulate natural cut stone, make up the entire exterior. Limestone makes up the crown running along the bottom of building. Cast stone is what is seen throughout the rest of the exterior which makes up the vertical façade.

Picture 3: *Bed Tower Addition*

Glazing makes up the other half of the exterior. There are three kinds of glazing. They are: 1) Clear Vision Glass; 2) Tinted Vision Glass; and 3) Spandrel Glass. The clear vision glass is used on the first floor where the lobby is located to allow the most daylight and energy. The tinted vision glass and spandrel glass work together to shade the patient rooms and stairwells and they don't transmit as much sunlight or energy as the clear vision glass.

Structurally, the addition is made up of a system of steel framing and composite deck. The foundation is a mat padding. On top of the roof, there is a large penthouse which holds the mechanical equipment. This is all supported by the steel framing of the building. For lateral loads, cross bracing is integrated within the frame.

Picture 4: *Construction of the addition*

Code

International Code

- 2006 International Building Code
  - Live load reduction used for typical floor loads and corridors above the first floor.

Design Codes

- ASTM International 2008
  - Concrete and testing of masonry
- ACI 318-08
  - Reinforced concrete design and construction
- AISC 360-05
  - Structural steel - Designed for "in place" loads
- SDI - Vulcraft Steel Deck 2008
  - Steel roof decking
  - Steel composite floor deck - Designed as unshored
- OSHA Safety Standards 2008
  - Steel erection
  - Steel joist erection
  - Metal decking erection
- ASCE 7-05
  - Wind loads

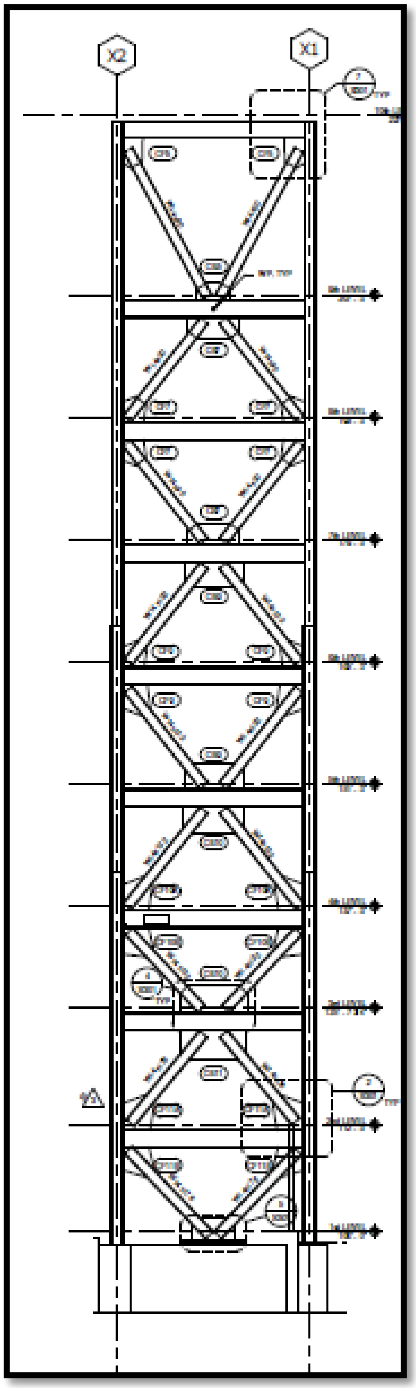


Figure 1: *Elevation of a braced frame system*

Courtesy of HGA

Structural System

The overall lateral system is a rigid frame with cross bracing. Rigid frames are commonly used when there is a need to provide unobstructed interior space with total adaptability. For the case of the Appleton Medical Center, a rigid frame was the best decision. It allowed the architects to create large spaces without being hindered by the structural system.

Bracing

Concentrically steel braced frames in each direction resist the lateral loads while the concrete slabs on metal deck act as the diaphragm which transfers the loads to the braced frames. There are 8 sections where the braced frames run vertically throughout the building. The typical frame runs from the top of the foundation to the top of the 9th level penthouse roof. Two others run to the top of the 9th level and the last one runs just between the 9th and 10th level. Shown on the previous page is a typical braced frame in Figure 1.

Connection to the mat foundation, explained later in the foundation section, helps transfer the lateral loads to the base. The braced beams are connected to the columns and floor beams by gusset plates for ease of construction and transfer of loads. Close-up of the braced frames are pictured on the left in Figure 2.

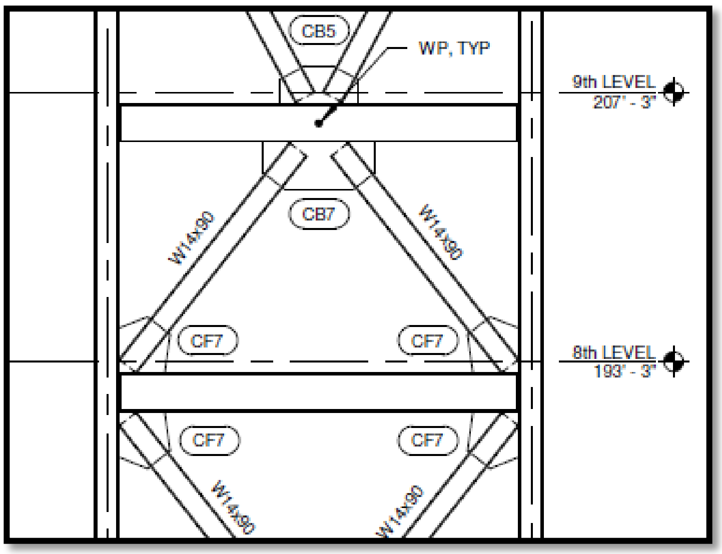


Figure 2: *Close-up of the braced frame system*

To the right are construction photos of the gusset plates used and connection to the foundation for the braced frames in Figures 3 and 4, respectively.

Figure 3: *Close-up of gusset plate construction for the braced frame*

All 3 Figures Courtesy of HGA

Figure 4: *Picture of a typical column connection to the foundation using a base plate*

Foundation

The geotechnical report was completed by (RVT) River Valley Testing Corporation. Originally, the foundation was designed with spread footings in mind, but after investigation by RVT they recommended three alternatives, which included the currently used mat foundation. Tests indicated that the natural soils on the site were able to hold bearing pressures ranging from 1,500 psf to more than 6,000 psf. The footings were then designed for a maximum soil bearing pressure of 3500 psf for just gravity loads and 4200 psf for gravity plus lateral loads. Spread footings range from 6 ft x 6 ft to 9 ft x 9 ft with depths being 1 to 2 ft. Maximum allowable interior column loads were to be 1,500 kips and the maximum allowable perimeter wall load was 3 kips per lineal foot.

Typical reinforcement for the mat slab includes the use of #7, #9, and #11 bars. The thickness of the mat slab is 3 ft 6 in. throughout the entire foundation under the triangular side of the addition. The area where the addition connects to the original part of the building has various thicknesses with 12 in. being the typical.

Most importantly, the braced frames are connected to the foundation to resist overturning moment. Typical thicknesses of these are 4 ft and run as long as the column spacing. Columns are connected to the bases by steel plates that are connected to the top of the concrete by 6 #6 hooks. The bases are reinforced by 5 #5 bars running horizontally and #5 bars running vertically spaced at 12 in. O.C. Pictured below in Figure 5 is a section and elevation of the braced frame to foundation connection with reinforcement.

Figure 5: *Detail of Typical Foundation Connection for the Braced Frames*

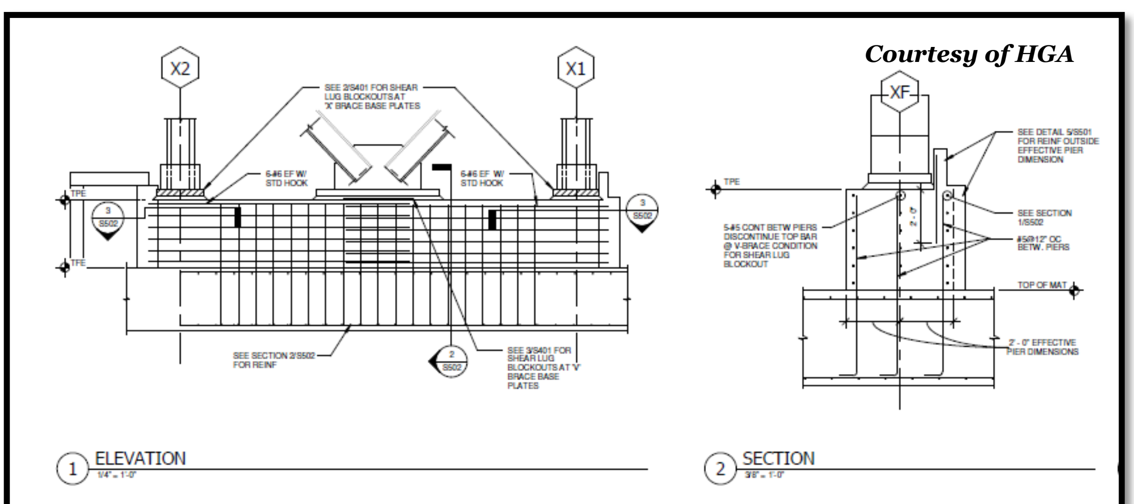


Figure 6 shows where the braced frames are connected at the foundation level in green. There is one more braced frame, but as stated earlier in the bracing section, this one is located on the top level.

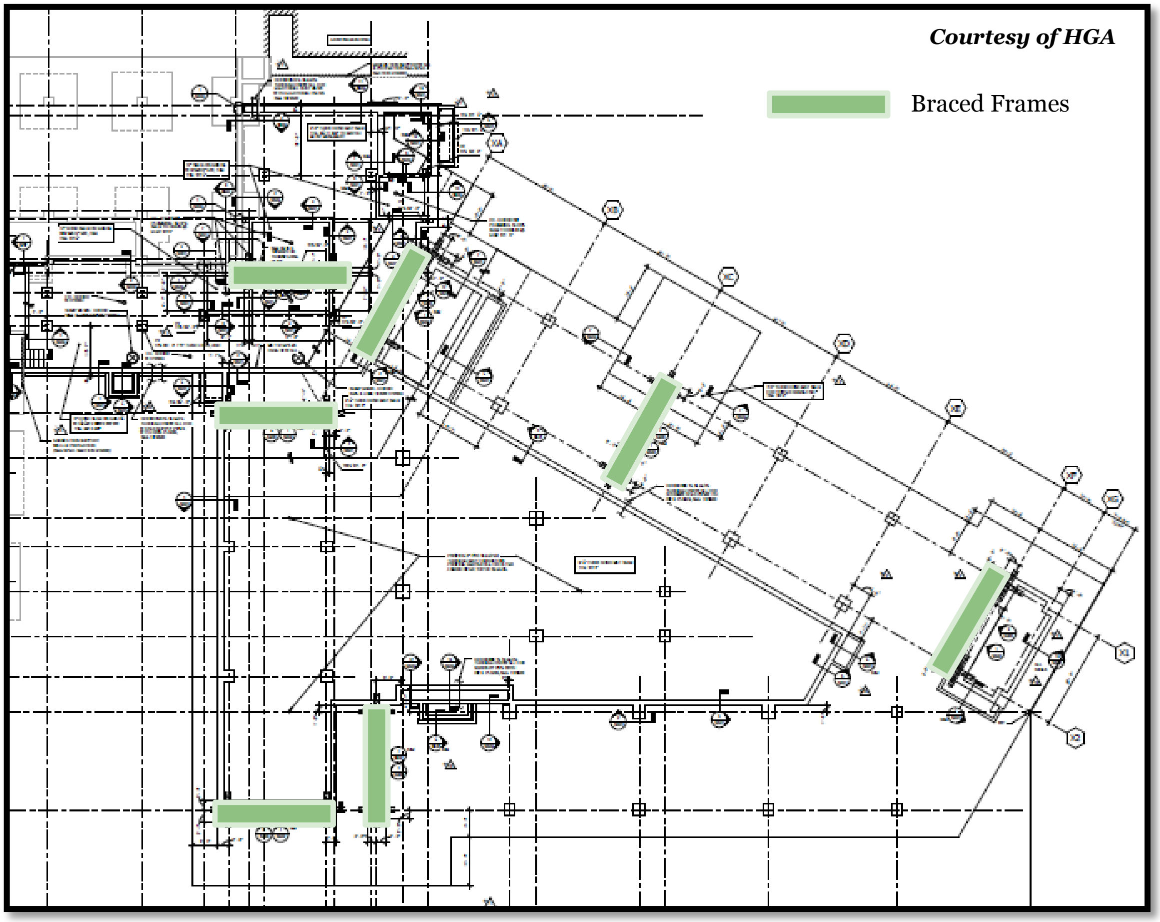


Figure 6: *Location of braced frames*

Floor Construction

Typical floor construction for the addition included the use of 4 types of "deck." Most floors were constructed of 3 in., 18 gage galvanized steel deck with a 4-½ in. normal weight concrete topping, making it a total thickness of 7-½ in. reinforced with 6x6 WWF. One floor was a combination of two decks. One "deck" was a 10 in. light weight concrete slab which was reinforced with #4 @ 18 in. O.C. running longitudinally. The other deck was a 2 in., 18 gage galvanized steel deck with a 3-½ in. light weight concrete topping making it a total thickness of 5-½ in. and reinforced with 6x6 WWF. Both the galvanized decks are composite and require a stud length of 5 in. for the

7-½ in. deck and 4 in. for the 5-½ in. deck. The roof deck was just a 1-½ in. 20 gage galvanized steel decking.

Bay sizes were typically set at 30 ft, especially on the outer spans of the building where the patient rooms are located. But, due to the irregular shape of the addition and use of the interior space, column lines were placed where columns were to not interfere with the working space of the interior. Bays of the interior ranged in various lengths. Decking typically spanned 10 ft and was supported by beams ranging from W14's to W21's with the typical being W16's. Lengths of the beams were typically 22 ft and were supported by girders ranging from W18's to W24's, but some exterior girders were W30's. Below in Figure 7 is a typical floor plan.

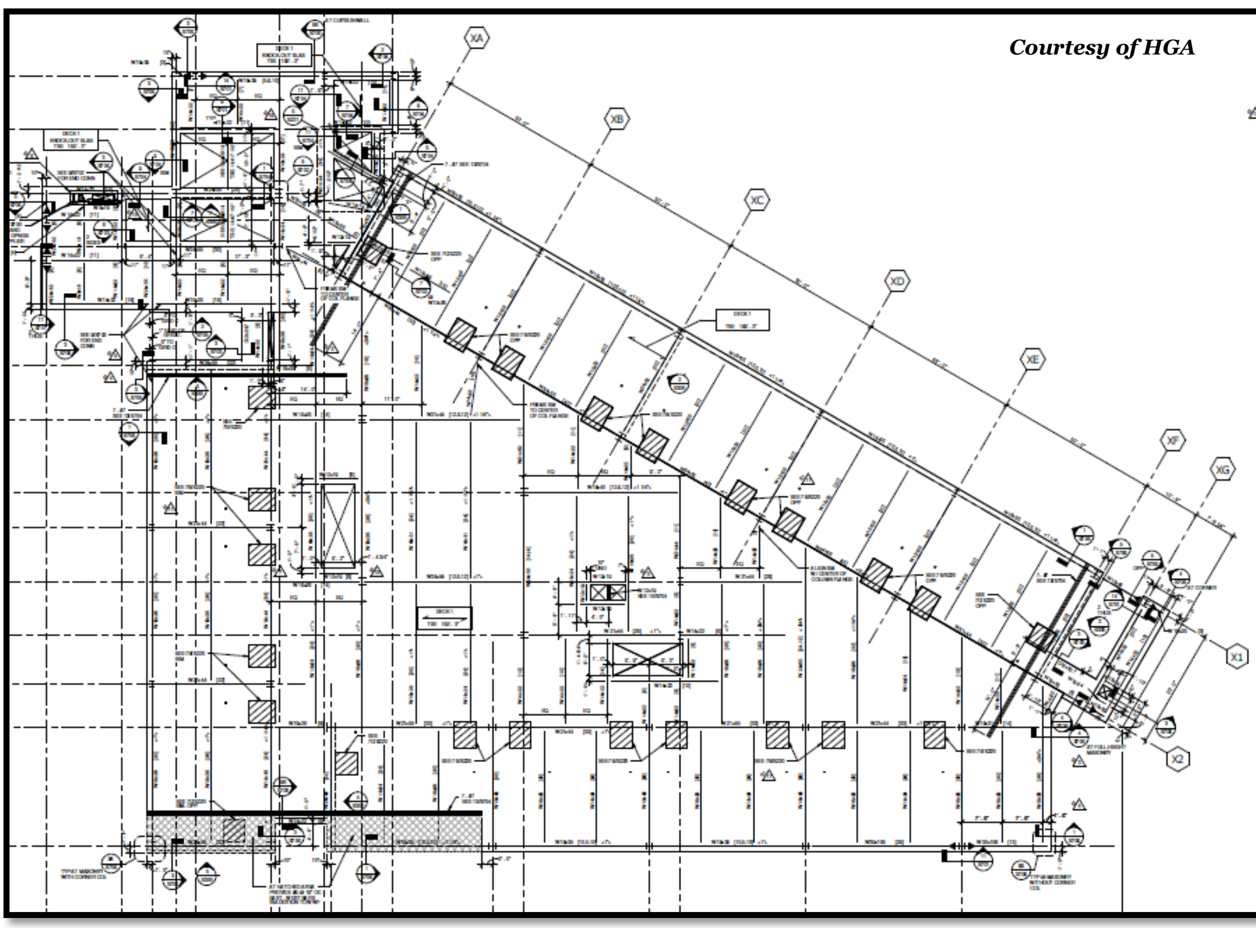


Figure 7: *Typical Floor Plan*

Important Information

Construction Materials and Building Loads

Materials used in construction were specified in the general structural notes on Sheet S001. More information on the materials was found on the floor plans and detailed sections and elevations as well.

Properties of Materials		
Material		*Strength*
Concrete	**Weight**	**f'c (psi)**
Composite Deck	145	3500
All other concrete	115	4000
Steel	**Grade**	**fy (ksi)**
Reinforcing Bars	A615	60
W Shapes	A992	50
Other Shapes	A36	36
Rectangular HSS	A500 - B	46
Round HSS	A500 - B	42
Bolts	A325/A490	60
Studs	A108	50

Figure 9: *Properties of Materials*

Dead Loads	
Material	Load (psf)
Superimposed	30
Composite Deck	
7.5" Thick 3" Steel	75
5.5" Thick 2" Steel	57
Roof	2.14
10" Slab	120 pcf

Figure 8: *Dead Loads*

Dead loads used for calculations were found in various ways. The composite deck and roof deck were found using the Vulcraft Roof and Steel Deck manual. The weight of the 10 in. light weight concrete slab was known and it was then assumed a superimposed dead load of 30 psf was used.

Live Loads		
Occupancy	Design (psf)	Thesis (psf)
Typ. Hosp. Floor	80	80
Corridors (Above 1st Floor)	80	80
Corridors (1st Floor)	100	100
Lobby Floor	100	100
Stair and Exits	100	100
Storage	125	125
Mechanical Room	125	125
Snow Load	34	34

Figure 10: *Live Loads*

Live loads were found using ASCE 7-05. However when doing research, typical hospital floors for patient rooms were found to be 40 psf, but it is believed that 80 psf was used because corridors (above 1st floor) with a load of 80 psf controlled. Because the patient rooms were found above the 1st floor, 80 psf was used for ease of calculations, although it is a conservative approach to this design.

Building Weight

In Technical Report 1, the total building weight was hand calculated. This process was very tedious and many human errors could have occurred. For this report, the total building weight was calculated with the assistance of a computer modeling program, to be explained later. The computer modeling program took into account self-weight of the steel beams, girders and columns as well as slab, deck, and superimposed dead load. Façade weight was added to these calculations and differences between the hand calculations and computer calculations were relatively close.

Floor Weights				
Level	Previous Weight (k)	Model Weight (k)	Façade (k)	Current Weight (k)
2	2275	1846	292	2137
3	2402	2253	277	2530
4	2192	1930	290	2220
5	2385	2229	317	2546
6	2373	2151	294	2445
7	2328	2132	294	2426
8	2323	2133	294	2427
9	2532	2077	363	2440
10	1840	314	198	512
Total	20651			19682

Figure 11: *Building Weight Comparison*

Relative Stiffness

One aspect focused on was the relative stiffness of the lateral braced frames within the structure. Relative stiffness is looking at the distribution of the forces within a diaphragm to the lateral systems. To further understand this concept, the stiffness of each frame was found with the help of RISA – 2D. Each braced frame was modeled with a 1 kip load applied to the top and columns modeled as pinned connections to the base. Figures 12 and 13 on the following page demonstrate this concept. Displacements were found for each braced frame and plugged into the equation:

$$K_f = \frac{P}{\Delta}$$

Once the stiffness of each frame was determined, the contribution of each frame to the overall system in its respective direction was found, also known as its relative stiffness. Hand calculations to find the relative stiffness can be found in Appendix B.

There are a few quick notes about the relative stiffness calculations. For a typical diaphragm there were three braced frames in the x-direction, one braced frame in the y-direction and three braced frames running diagonally. Each diagonal frame was broken up into its x and y components in order to calculate relative stiffness in each direction.

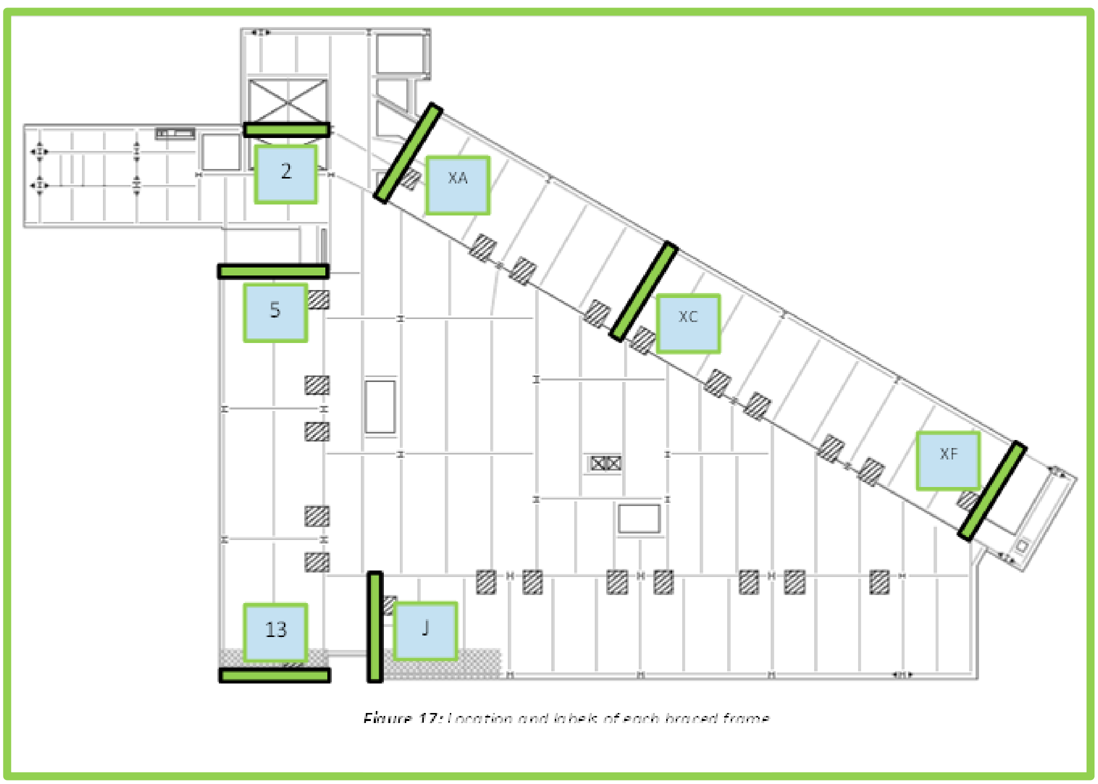
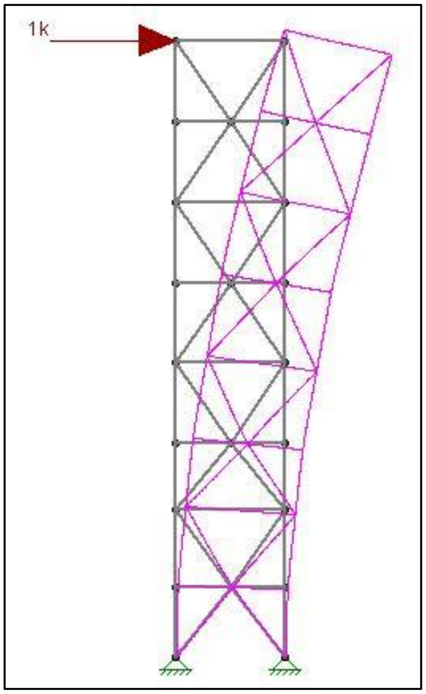
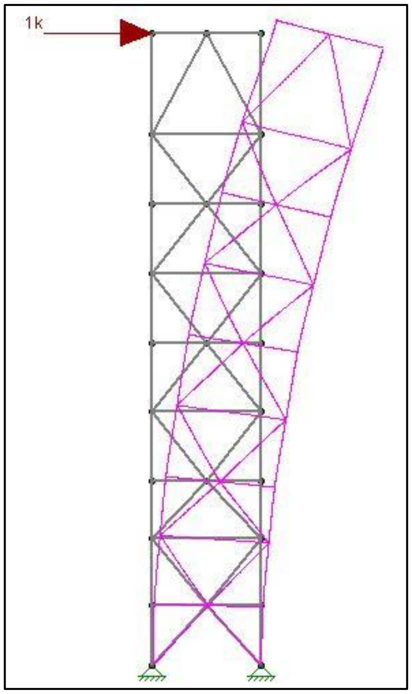


Figure 12: *Location and labels of each braced frame. Plan courtesy of HGA*

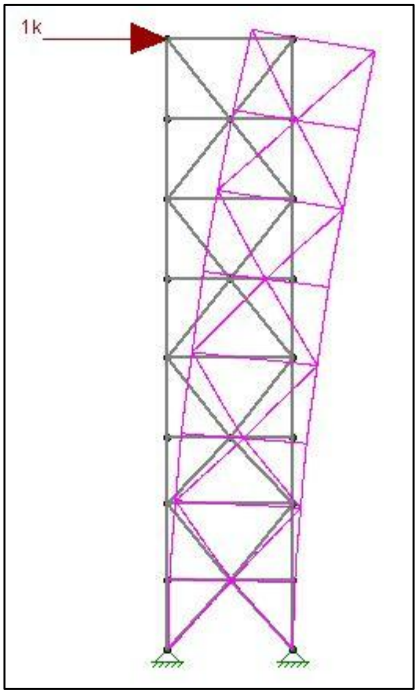
Frames running in the West/East direction



Frame 2



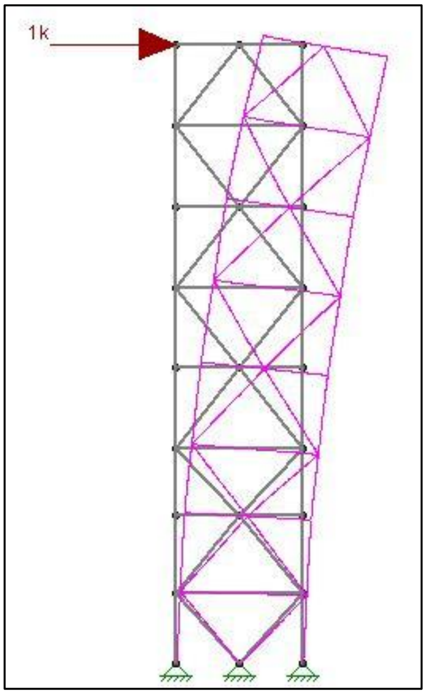
Frame 5



Frame 13

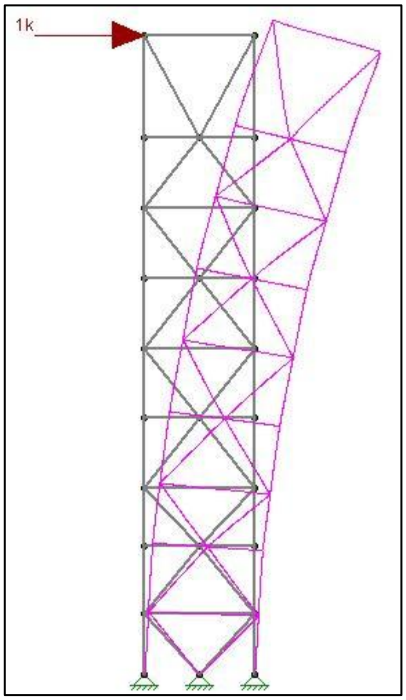
Figure 13: *Deflected shape of each braced frame*

Frame running in the South/North direction



Frame J

Typical Frame running diagonally



Frames XA, XC, XF

Frame Stiffness			
Frame	Force (k)	Δmax (in)	k (k/in)
XA	1	0.015	66.7
XC	1	0.015	66.7
XF	1	0.015	66.7
J	1	0.009	111.1
13	1	0.009	111.1
5	1	0.015	66.7
2	1	0.019	52.6

In the West/East Direction		
Frame	Stiffness (k/in)	Relative Stiffness
2	52.6	13.04%
5	66.7	16.51%
13	111.1	27.52%
XAy	57.8	14.31%
XCy	57.8	14.31%
XFy	57.8	14.31%
Total	403.7	100.00%

In the South/North Direction		
Frame	Stiffness (k/in)	Relative Stiffness
J	111.1	52.62%
XAy	33.4	15.79%
XCy	33.4	15.79%
XFy	33.4	15.79%
Total	211.2	100.00%

Figure 19: *Relative Stiffness Tables*

The figure above shows the relative stiffness of each braced frame in their respective directions. Because of their individual stiffness's, it can be seen which frames will take a majority of the loads. In the West/East direction, frame 13 will take 27.52% of the total load while frame J in the South/North direction will take 52.62% of the total load.

Now that the relative stiffness of each a typical diaphragm has been completed, to help understand where the exact location of each load will be applied, the center of mass and center of rigidity will be the next focus.

Center of Rigidity and Center of Mass

The center of rigidity and center of mass vary for each diaphragm. Loads applied to each diaphragm will be applied to the center of mass and if there happens to be an eccentricity between the center of mass and center of rigidity, torsion will occur.

Due to the irregular shape of the addition, it was assumed there would be some torsion. In order to confirm this, the RAM model was checked to see where the center of rigidity and center of mass was on each floor. RAM concluded that both points did not lie on top of each other meaning there was an eccentricity and torsion would occur. To double check that the RAM model was setup correctly, the center of rigidity was calculated by hand using the stiffness's found earlier. Hand calculations can be found in Appendix C. Important equations were:

$$\mathrm{X_r} = \frac{\sum R_i x_i}{\sum R_i} \qquad \mathrm{Y_r} = \frac{\sum R_i y_i}{\sum R_i}$$

Xr is the distance to the center of rigidity in the x-direction

Yr is the distance to the center of rigidity in the y-direction

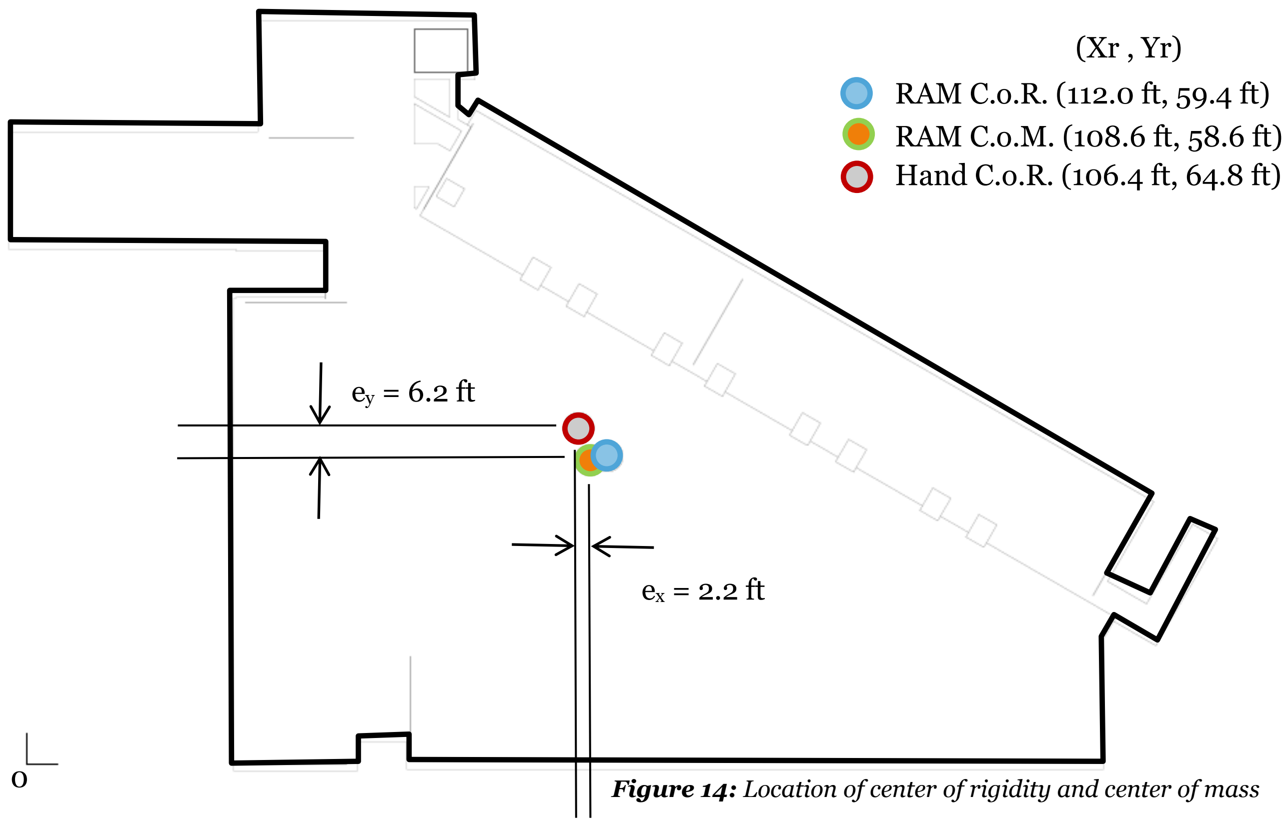


Figure 14: *Location of center of rigidity and center of mass*

Proposal Statement

After the conclusion of technical report 3, the bed tower addition was proven to be adequate in strength for lateral loads. From the analysis, it was determined that wind was the controlling force. However, the author this proposal wanted to learn more about seismic design. In order to do this, seismic forces need to control analysis. To accomplish this, a scenario had to be created where seismic design controlled.

The Scenario

A hospital in San Francisco, California wants to build a bed tower addition to comfortably accommodate more patients. They decide to build the addition similar to the one located in Wisconsin, because of its architectural similarities and unique triangular design which would fit perfectly on a plot of land already own.

With an idea in mind, design started, but problems quickly arose. The existing structural system was believed to not hold the adequate strength. Therefore, a new lateral structural system would have been designed.

Problem Solution

Two designs are being considered for the new bed tower addition. One design solution is to modify the existing braced frames to fit the seismic requirements of ASCE7-10. Additional braced frames might also be designed to further ensure adequate strength. If this were to occur, these new braced frames will be designed by hand and properly placed within the structural system so as to not disturb the existing architectural layout. Dampers are also being considered into the design of the new braced frames to see if vibration throughout the building could be reduced. This could be helpful for the hospital since less vibration would be less disturbing to the patients and staff inside. Once the above is completed, the new lateral system will then be compared to the existing one to check differences in strength and flexural capacities.

The other design being considered would be using base isolation. This is when the superstructure (the structural skeleton itself) is separated from the substructure (the foundation) by base isolators during an earthquake. During this process, the superstructure will move slower because of the base isolators absorbing a majority of the shock inflicted upon the building. For the purpose of this proposal, thorough research of this subject will have to be done. Such research includes, seeing if there are any hospitals in the San Francisco area which utilize base isolation and looking at various types of base isolation. A thorough cost analysis of the base isolation will also be considered.

Breadth Study I: Architectural Impact

Both design solutions can make an architectural impact on the building. In the event that additional braced frames are designed, they will need to be located so the entire building layout is not affected. This was also a problem during the design process of the actual structure. Braced frame locations were selected after the layout of the hospital was created. As a result, the structural engineers had difficulty in selecting the placement of the frames. Utilizing the base isolation method will also make a big impact. In order to design a base isolation system, a moat (cavity) space needs to surround the building to account for the large displacements created during an earthquake.

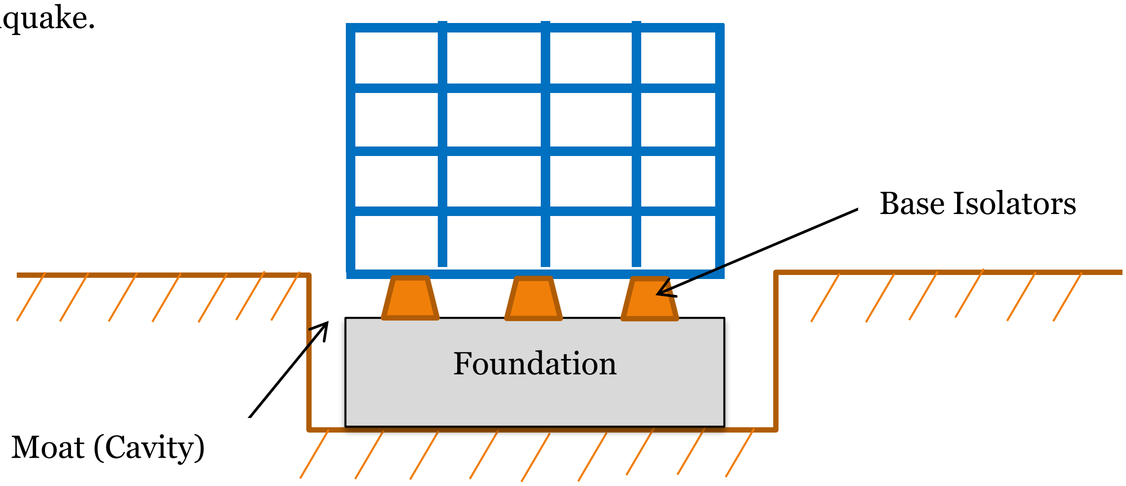


Figure 15: *Picture of a building with base isolation*

Breadth Study II: Construction and Cost Analysis

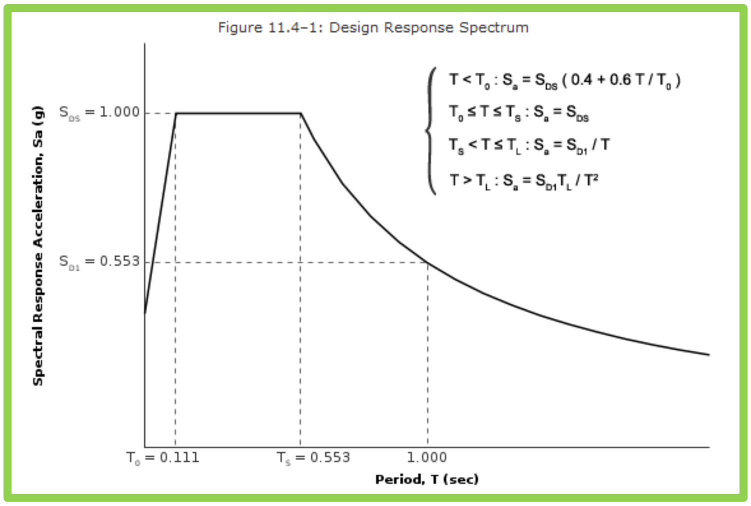
Redesign of the braced frames would impact construction and create additional costs. Construction impacts will include change in schedule to modify the braced frames. In addition, the length of the schedule could increase if more braced frames are designed. Implementing the base isolation system will also impact construction. Changes to the schedule and site layout will need to be reconsidered for the addition of the moat around the building and change in foundation design to support the base isolation system. Costs due to these changes will result in additional labor, formwork, and material costs.

Redesign of the Existing Structure

Due to the change in location from Appleton, Wisconsin to San Francisco, California the existing structure needed to be reevaluated. Because San Francisco is a seismic prone region, Chapter F of the seismic provisions of AISC 2010 was carefully taken into consideration. In addition, Chapter 17 of ASCE 7-10 (Seismic Design Requirements for Seismically Isolated Structures) was also reviewed.

It was determined that the new location of the hospital would be placed near San Francisco State University's campus. This location would be critical to acquire the values used for seismic design. It would also be used for the architectural breadth discussed later. The United States Geological Survey (USGS) provides detailed reports for seismic design depending on the exact location that is wanted. These detailed reports are called "DesignMaps." They will be used later in the report when going the through process of upgrading the structure to meet the new standards.

On the right are two graphs showing the Spectral Response Acceleration (g) for the DBE and MCE earthquakes. These were acquired from the website, but they could also be done by hand. Values found in the "DesignMaps" were checked by hand and they can be found in Appendix D.



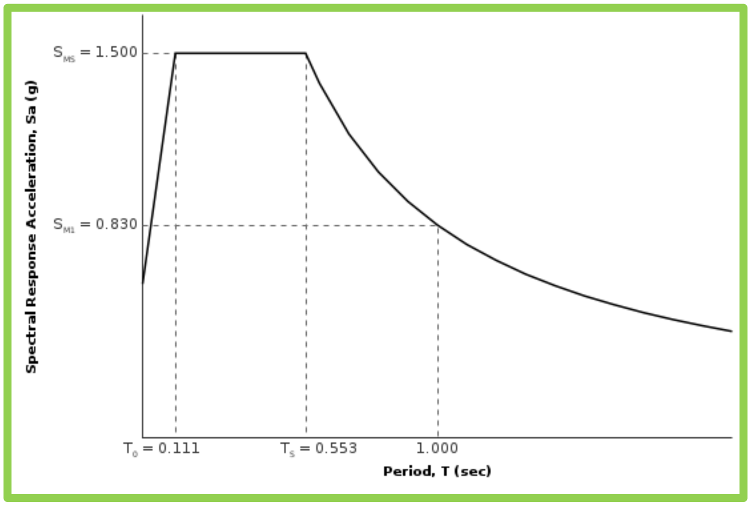


Figure 16: *(Top) Design Response Spectrum – DBE*
(Bottom) Design Response Spectrum – MCE
From "DesignMaps" of USGS

After verifying hand calculations with the USGS "DesignMaps" for San Francisco, the Seismic Design Category ended up being SDC – D. This is a big difference when comparing the structure to its Wisconsin location where it was SDC – A. Tables verifying the Seismic Design Category are pictured below but a more detailed report will be explained later.

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = IV and S_{DS} = 1.000, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = IV and S_{D1} = 0.553, Seismic Design Category = D

Figure 17: *Determination of Risk Category From "DesignMaps" of USGS*

From this point on, it is assumed that all calculations will be based on the new seismic design category. Analysis includes use of the two structural analysis programs. For the first part of the report, RAM Structural System was used to check again the seismic provisions. To analyze the base isolation, two models were made using SAP2000. One model used base isolation with the existing structure. The second model used base isolation with the modified structure determined from RAM. RAM was very useful for linear-elastic analysis while SAP2000 was helpful for nonlinear analysis.

Load Combinations

To determine the controlling load combination, the worst case scenario was taken into account. These load combinations from Chapter 2 of ASCE 7 -10 were taken into account:

1. 1.4D
2. 1.2D + 1.6L + 0.5S
3. 1.2D + 1.6S + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5S
5. 1.2D + 1.0E + L + 0.2S
6. 1.2D + 0.5L + 1.0E
7. 0.9D + 1.6W
8. 0.9D + 1.0E

RAM was very useful in determining the controlling load combination because there was a large number available due to the various numbers of wind and seismic load cases. However, after determining which wind and seismic load cases controlled, it was easy to eliminate many combinations. After RAM analyzed these applicable load combinations, it was determined that the controlling load combination in both directions turned out to be combination 6 (1.2D + 0.5L + 1.0E). This is not a surprise that controlling combination would include the earthquake load.

The controlling load combination will be very important when gravity members are checked because dead, live and earthquake loads have been included. This will determine which members to change for the new seismic criteria.

Lateral Loads - Wind Load Design

Chapter 26 of ASCE 7-010 was used to determine the wind load pressures. For simplicity of analysis, the addition was modeled as a rectangular box. Parameters for the box spanned between the furthest reaching corners of the building in both x and y directions. In Figure 12 below, is the rectangular box and dimensions used for the calculating wind load pressures.

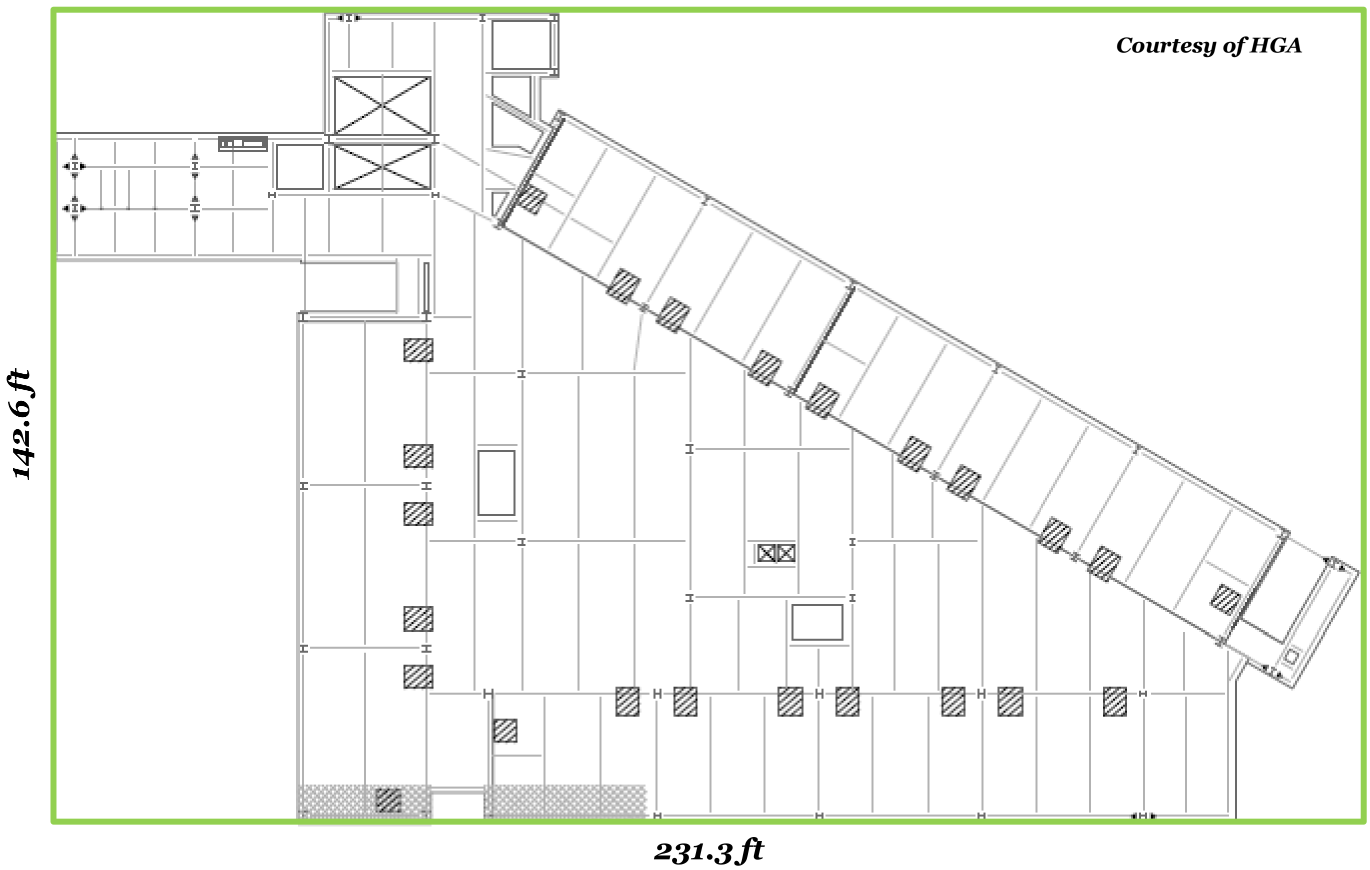
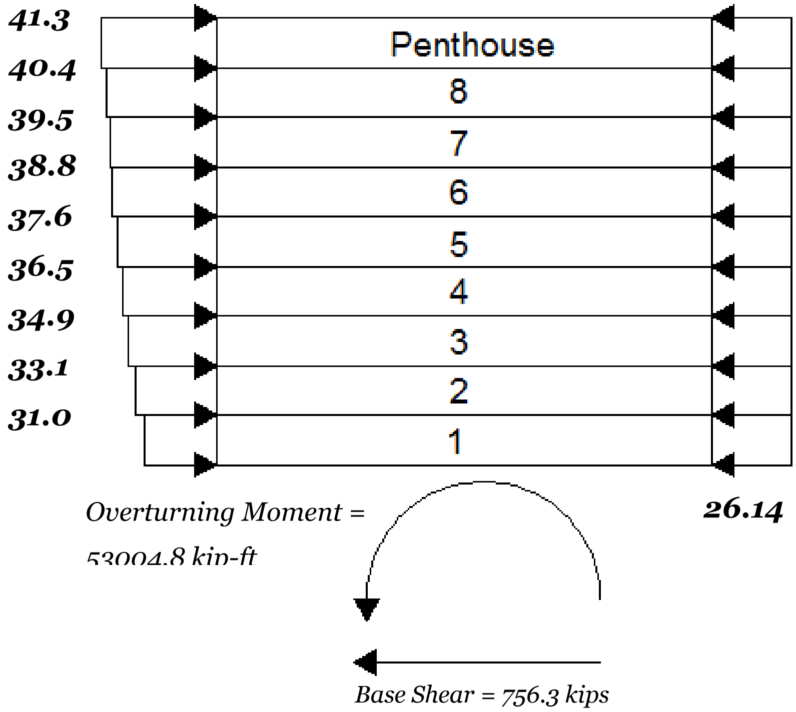


Figure 18: *Wind Load Parameters*

In Technical Report 1, the parameters were much smaller than the ones above. For this final report, wind load pressures were recalculated for the adjusted parameters and the results are listed on the following page. Figures 19 & 20 show the applied story pressures, forces, leeward pressure, total base shear and overturning moment for the East/West and South/North directions respectively.

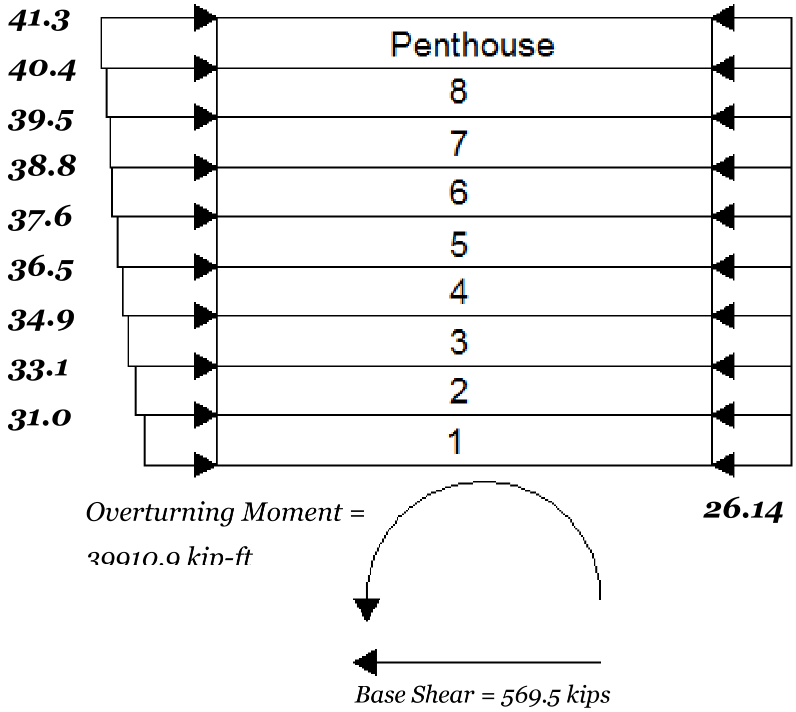
South to North					
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)	Leeward (psf)
1	0	0.00	0	0	0.00
2	12.25	30.96	66.55	815.29	-26.14
3	25.646	33.10	69.37	1778.97	-26.14
4	37.25	34.86	74.83	2787.31	-26.14
5	51.25	36.47	85.61	4387.30	-26.14
6	65.25	37.61	88.29	5760.71	-26.14
7	79.25	38.76	90.99	7211.28	-26.14
8	93.25	39.50	92.72	8646.53	-26.14
9	107.25	40.36	116.75	12521.14	-26.14
10	127.75	41.43	71.20	9096.28	-26.14
Base Shear		756.31	**Overturning M**	53004.8	

Figure 19: *West to East loading in pounds per square foot (PSF)*



West to East					
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)	Leeward (psf)
1	0	0.00	0	0	0.00
2	12.25	30.96	50.11	613.89	-23.64
3	25.646	33.10	52.23	1339.51	-23.64
4	37.25	34.86	56.34	2098.75	-23.64
5	51.25	36.47	64.46	3303.49	-23.64
6	65.25	37.61	66.48	4337.63	-23.64
7	79.25	38.76	68.52	5429.86	-23.64
8	93.25	39.50	69.82	6510.55	-23.64
9	107.25	40.36	87.91	9428.00	-23.64
10	127.75	41.43	53.61	6849.20	-23.64
Base Shear		569.48	**Overturning M**	39910.9	

Figure 20: *West to East loading in pounds per square foot (PSF)*



Lateral Loads - Seismic Design

Chapters 11 and 12 of ASCE 7-10 were used for seismic design. Last semester, when the building was in Appleton, Wisconsin the Seismic Design Category (SDC) was SDC – A .This was based on the seismic design criteria. Since then, the building has been moved to San Francisco, California. After checking seismic design criteria for the bay area, it was found to be SDC – D assuming Site Class C and Risk Category IV. The numbers calculated from ASCE7-10 can be found below with their respective equations and figures in which they were found. The figures correspond to the ones found in ASCE not the figures labeled in this report. The base shear ended up being 5113 kips which is close to seven times higher than the base shear found in the wind calculations. Because of this reason, it is easy to determine that seismic will in fact control design of all calculations going forward.

From Figure 22-1	S_s	1.500	g
From Figure 22-2	S_1	0.638	g
From Table 11.4-1	F_a	1.000	
From Table 11.4-2	F_v	1.300	
S_{MS} =	F_aS_s	1.500	g
S_{M1} =	F_vS_1	0.829	g
S_{DS} =	$2/3 * S_{MS}$	1.000	g
S_{D1} =	$2/3 * S_{M1}$	0.553	g
From Figure 22-12	T_L	12.000	s
From Figure 22-7	PGA	0.600	
From Table 11.8-1	F_{PGA}	1.000	
PGA_M =	$F_{PGA}PGA$	0.600	
From Figure 22-17	C_{RS}	1.042	
From Figure 22-18	C_{R1}	0.986	

Figure 21: *Seismic Parameters*

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_s \le 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1$	$S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_s

For Site Class = 2 and S_s = 1.500, F_a = 1.000

Figure 22: *Table 11.4-1 (USGS Design Maps)*

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = 2 and S_1 = 0.638, F_v = 1.300

Figure 23: *Table 11.4-2 (USGS Design Maps)*

Table 11.8-1: Site Coefficient F_{PGA}					
Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA ≥ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = 2 and PGA = 0.600, F_{PGA} = 1.000

Figure 24: *Table 11.8-1 (USGS Design Maps)*

From Table 12.2-1	Steel Ordinary Concentrically Braced Frames				
	R	Ω_o	C_d	I_e	
Values	3	2	3.25	1.5	

Figure 25: *ASCE7-10 Seismic Design Criteria*

Equivalent Lateral Force Procedure			
	Weight	19682	kips
$C_S = S_{DS}/(R/Ie)$		0.5	
From Table 12.8-2	C_t	0.02	
	x	0.75	
	h_n	127.8	ft
$T_a = C_t h_n^x$		0.76	
From Table 12.8-1	C_u	1.40	
$T = C_u T_a$		1.06	
C_s not to exceed $C_{s1} = S_{D1}/T(R/I_e)$		0.2598	
$V = C_{s1}W$		5112.765	kips

Figure 26: *Equivalent Lateral Force Calculations*

Modification of Braced Frames

The criteria and parameters found in the previous section were carried over to computer modeling to assist in computing deflections, drifts, and story shears. As part of the proposal, the existing structure was to be modified to withstand the seismic load of the new location using ASCE7-10. The vertical distribution of forces was drastically different from when the building was in Wisconsin. As stated before the SDC was A and because of this, vertical forces were taken to be $F_x = 0.01W_x$ as state by ASCE7-10. A comparison of the two cases are shown below with signifcant differences.

Vertical Distribution of Forces								
Level	w_x (kips)	h_x (ft)	k	h_x^k (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	M (k-ft)
10	512	127.75	1.28	496.8	254338	0.063	322.6	41206.8
9	2440	107.25	1.28	397.1	968943	0.240	1228.8	131792.9
8	2427	93.25	1.28	332.0	805787	0.200	1021.9	95294.0
7	2426	79.25	1.28	269.6	654048	0.162	829.5	65736.4
6	2445	65.25	1.28	210.2	513975	0.127	651.8	42532.3
5	2546	51.25	1.28	154.3	392886	0.097	498.3	25536.2
4	2220	37.25	1.28	102.6	227717	0.056	288.8	10757.6
3	2530	25.64583	1.28	63.6	160939	0.040	204.1	5234.5
2	2137	12.25	1.28	24.7	52798	0.013	67.0	820.3
					4031432		5112.765	418911.1
Base Shear		5112.765	Overturning Moment			418911.1		

Figure 27: *Vertical Distribution of Forces*

Seismic Load Calculations				
Level	Ht. (ft)	Weight (k)	Fx	M (k-ft)
1	0	0	0	0
2	12.25	2124.20	21.24	260.21
3	25.64583	2529.80	25.30	648.79
4	37.25	2207.10	22.07	822.14
5	51.25	2506.20	25.06	1284.43
6	65.25	2408.60	24.09	1571.61
7	79.25	2389.20	23.89	1893.44
8	93.25	2390.30	23.90	2228.95
9	107.25	2334.60	23.35	2503.86
10	127.75	550.10	5.50	702.75
Total		19440.10		11916.2
Base Shear		194.40	Overturning M	11916.2

Figure 28: *Vertical Distribution of Forces from Tech Report 1*

As stated before, the base shear was 5113 kips. The overturning moment was calculated to be 418,911 kip-ft. The resisting moment of the building was calculated as:

$$M_R = W \times (1/2)\ L_s$$

Where M_R = Resisting Moment

W = Weight of the building

L_s = Shortest Length of building

From earlier in the report, the shortest length was 142.06 ft and the weight was 19682 kips. The resisting moment was calculated to be 1,403,327 kip-ft. However, the factor of safety for overturning moment is $(2/3)\ M_R \geq M_o$. The resisting moment ended up being just over twice as much as the overturning moment so the building is safe as calculated below:

$$(2/3) * 1{,}403{,}327 \text{ kip-ft} = 935{,}551 \text{ kip-ft} > 418{,}911 \text{ kip-ft} \quad \text{OK!}$$

The computer program used to check for any modifications to the structure was RAM Structural System. RAM took into account all the parameters shown in the report. Analysis of the model showed that the existing structure had a few structural elements that needed to be upgraded. All first and second story columns on each lateral frame were found to be well under the minimum strength needed for compression members. Every member of the lateral system passed strength for tension. Several braced members did not pass slenderness requirements. The seismic provisions of AISC state that $KL/r \leq 4\sqrt{E/Fy}$. These members were upgraded in order to accommodate these needs. Pictures showing the changes in column sizes and braces are shown on the following pages.

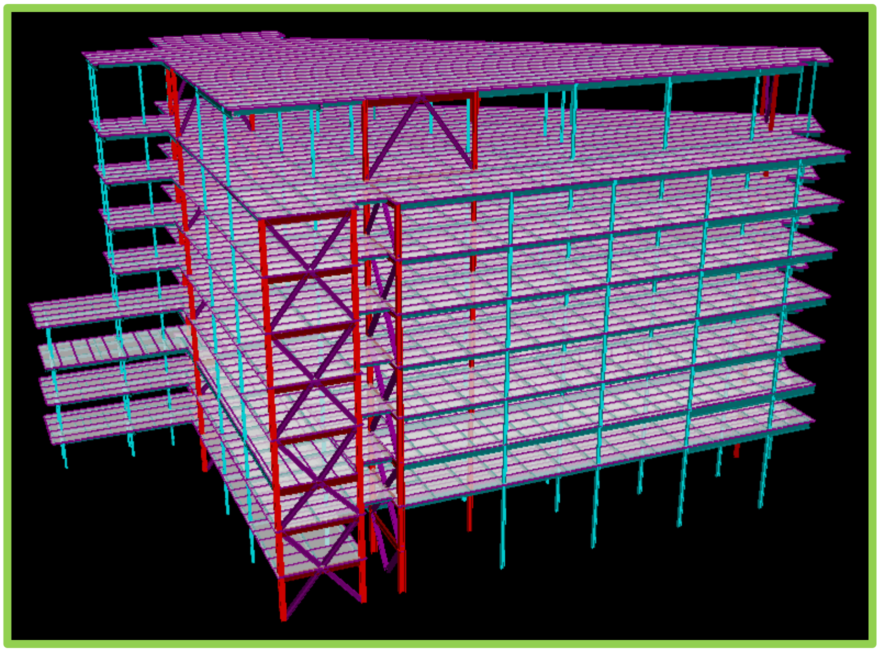


Figure 29: *3D Model from RAM Structural*

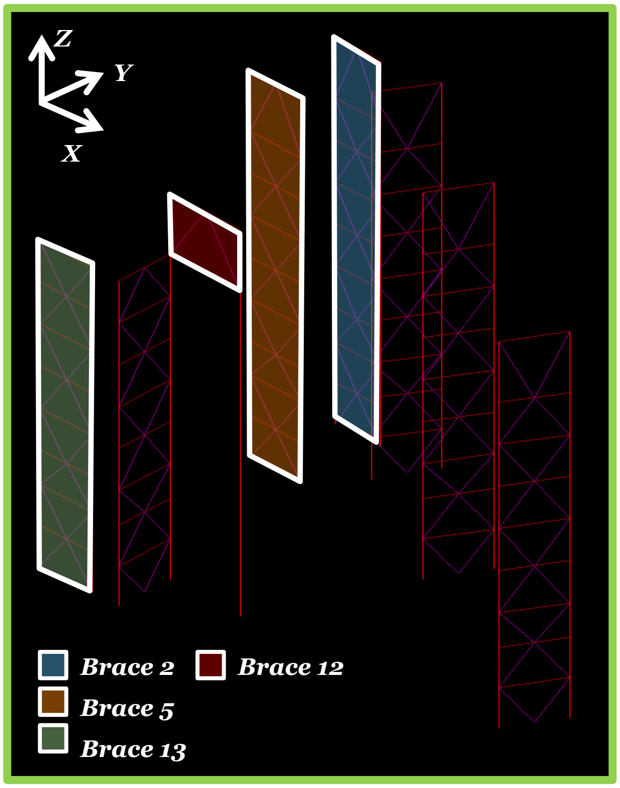


Figure 30: *Braced frames in the x-direction*

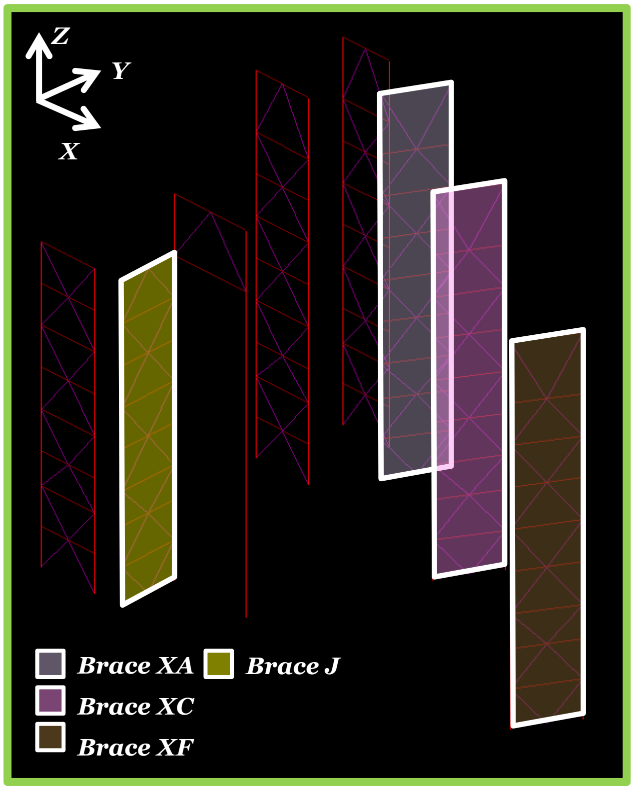


Figure 31: *Braced frames in the y-direction and diagonal*

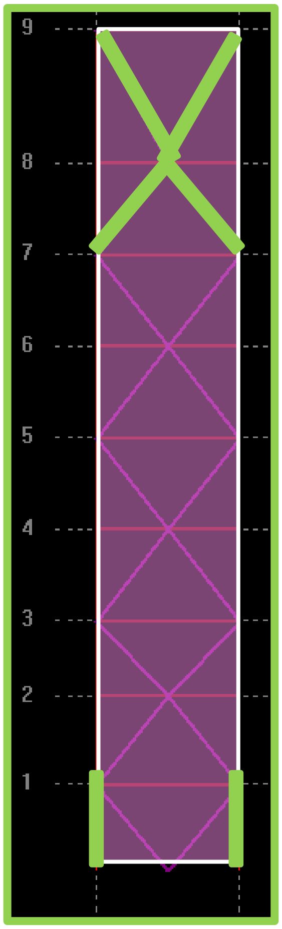


Figure 32: *Brace XA,XC*

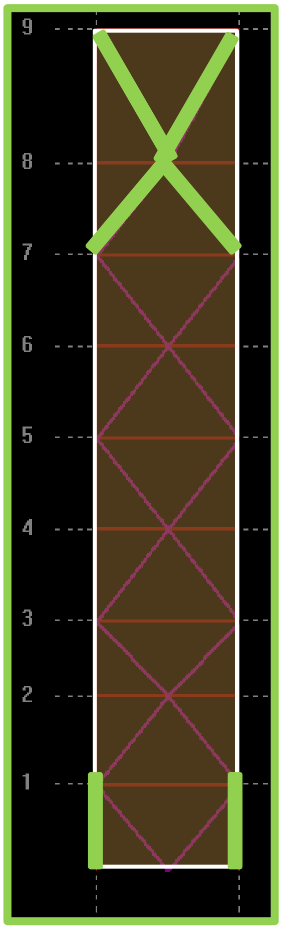


Figure 33: *Brace XF*

Brace XA, XC Modifications			
Level	(C)olumn (B)race	Original Member	New Member
1	C	14x398	14x500
7	B	14x90	14x82
8	B	14x90	14x82
9	B	14x90	14x82

Members modified

Brace XF Modifications			
Level	(C)olumn (B)race	Original Member	New Member
1	C	14x398	14x550
7	B	14x90	14x82
8	B	14x90	14x82
9	B	14x90	14x82

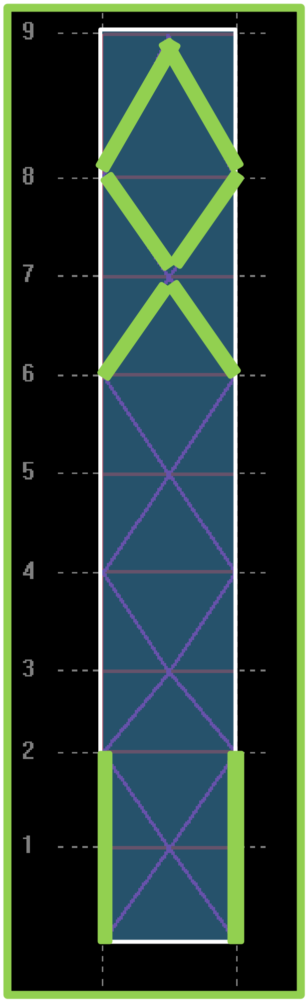


Figure 34: *Brace 2*

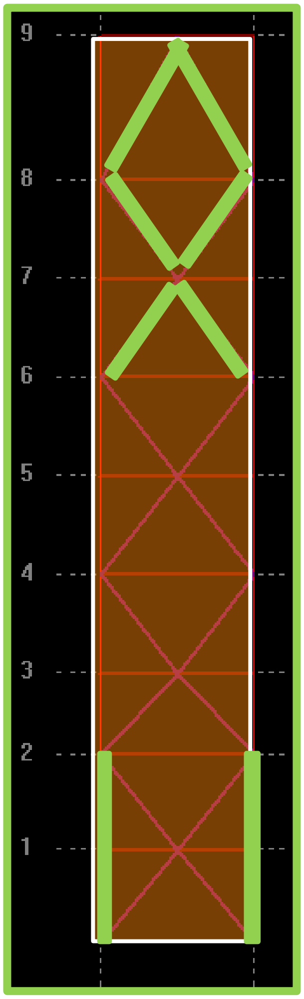


Figure 35: *Brace 5*

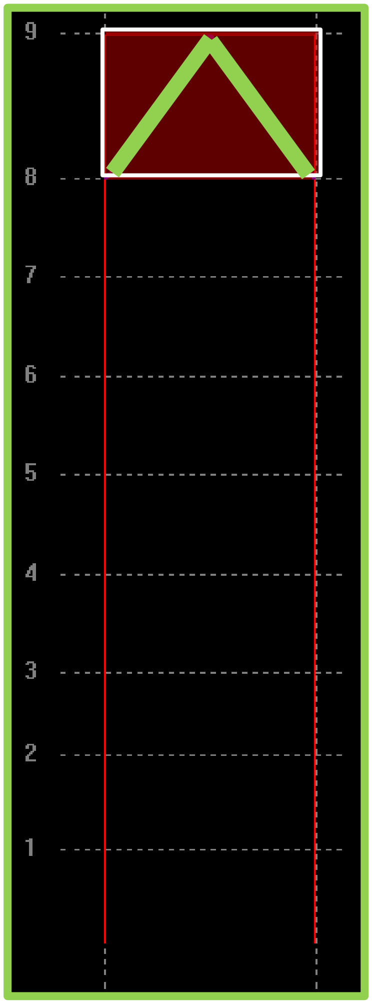


Figure 36: *Brace 12*

Brace 2 Modifications			
Level	(C)olumn (B)race	Original Member	New Member
1	C	14x398	14x550
2	C	14x398	14x550
7	B	14x90	14x82
8	B	14x90	14x82
9	B	14x90	14x82

Brace 5 Modifications			
Level	(C)olumn (B)race	Original Member	New Member
1	C	14x398	14x500
2	C	14x398	14x500
7	B	14x90	14x82
8	B	14x90	14x82
9	B	14x90	14x82

Members modified

Brace 12 Modifications			
Level	(C)olumn (B)race	Original Member	New Member
9	B	14x90	14x82

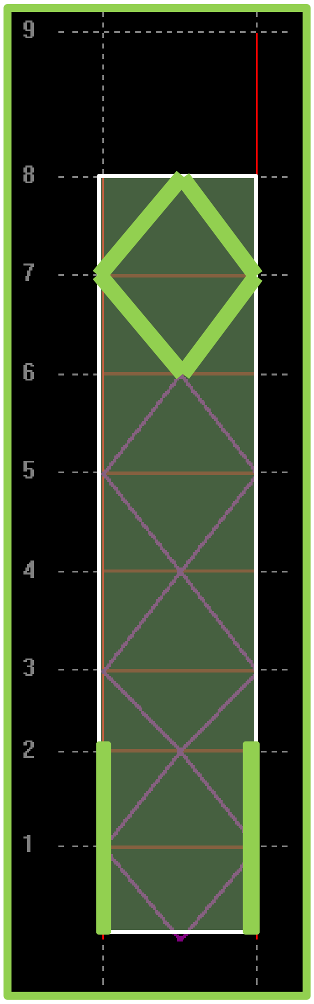


Figure 37: *Brace J*

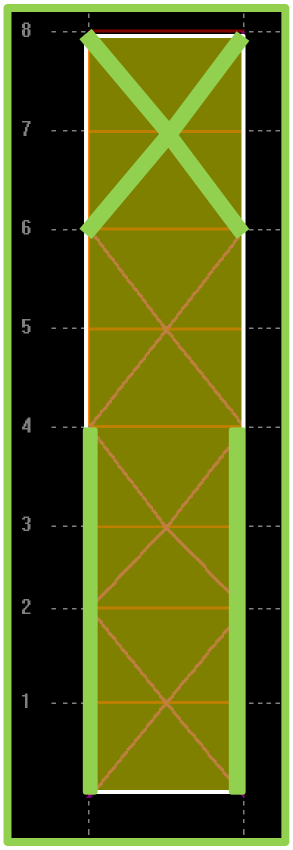


Figure 38: *Brace 13*

Brace J Modifications			
Level	(C)olumn (B)race	Original Member	New Member
1	C	14x398	14x730
2	C	14x398	14x500
3	C	14x342	14x500
7	B	14x90	14x82
8	B	14x90	14x82

Members modified

Brace 13 Modifications			
Level	(C)olumn (B)race	Original Member	New Member
1	C	14x398	14x550
2	C	14x398	14x550
3	C	14x342	14x370
4	C	14x342	14x370
7	B	14x90	14x82
8	B	14x90	14x82

To the right are pictures showing the deflected shape of the lateral system for two of the load cases. Because it was determined from the lateral load analysis that seismic would control, the four load cases used were as follows.

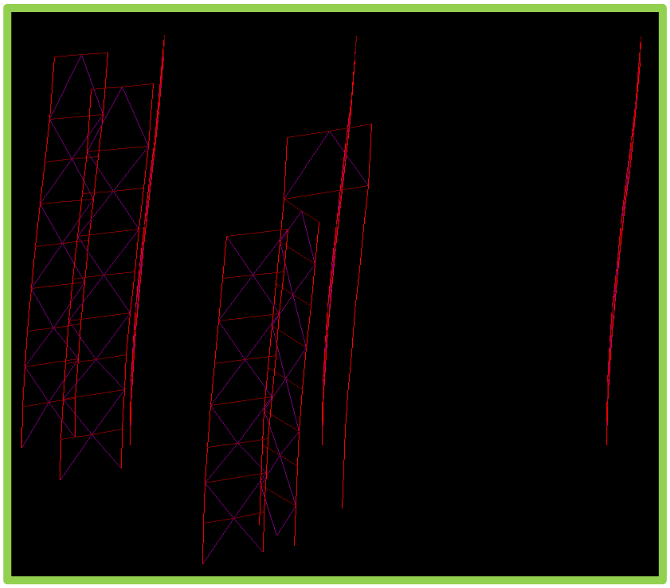


Figure 39: Deflected shape in the X-direction with positive eccentricity

Seismic

1. X-direction with positive eccentricity
2. X-direction with negative eccentricity
3. Y-direction with positive eccentricity
4. Y-direction with negative eccentricity

The eccentricity was assumed to be 5% and taken from the center of mass where seismic loads are assumed to be applied. It was then determined from the calculations from RAM that the seismic loads applied in the Y-direction with negative eccentricity. Displacements were fairly close between all the cases. A table showing the displacements at each floor for the different load cases can be found on the next page in Figure 41.

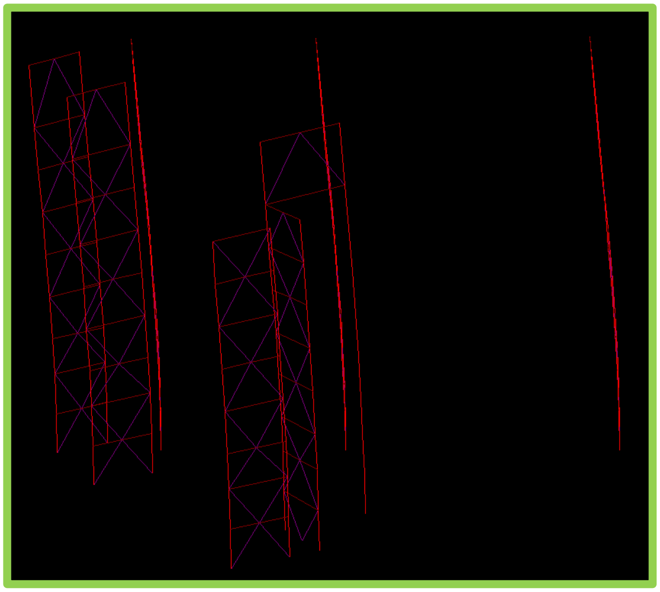


Figure 40: Deflected shape in the Y-direction with positive eccentricity

Level Displacements (in)										
		Load Cases								
Level	Height (ft)	E1	Drift	E2	Drift	E3	Drift	E4	Drift	Allow. Drift (in)
10	127.25	5.68	0.90	5.49	0.75	6.51	1.22	6.54	1.27	2.40
9	107.25	4.78	0.79	4.74	0.77	5.29	0.90	5.27	0.91	1.68
8	93.25	3.99	0.86	3.97	0.85	4.39	0.94	4.36	0.93	1.68
7	79.25	3.13	0.79	3.12	0.79	3.45	0.84	3.43	0.83	1.68
6	65.25	2.34	0.70	2.33	0.70	2.61	0.80	2.60	0.79	1.68
5	51.25	1.64	0.67	1.63	0.66	1.81	0.67	1.81	0.67	1.68
4	37.25	0.97	0.40	0.97	0.40	1.14	0.48	1.14	0.48	1.39
3	25.65	0.57	0.39	0.57	0.40	0.66	0.37	0.66	0.36	1.61
2	12.25	0.18	0.18	0.17	0.17	0.29	0.29	0.30	0.30	1.47
1	0.00									0
E1 - Seismic load in X-direction with pos. eccent.						E3 - Seismic load in Y-direction with pos. eccent.				
E2 - Seismic load in X-directino with neg. eccent.						E4 - Seismic load in Y-direction with neg. eccent.				

Figure 41: *Level Displacements of each load case in RAM*

All drifts passed max drifts allowed. It can also be seen that displacements were very close but the drifts for E4 totaled just slightly higher than the total drifts for E3 making E4 the most critical case. This is to be expected when doing base isolation as well. One reason why the critical case is in the Y-direction is because of the layout of the braced frames. As stated before with the help of the three diagonal frames, the X-direction has four frames while the Y-direction only has one. So the Y-direction should be less stiff and displacements should be higher than those found in the X-direction. It will be shown later in the report that during base isolation, the Y-direction also controls.

Base Isolation

Concept and History

The idea of base isolation is a very recent idea in the general timeline of structural engineering. According to NEHRP (National Earthquake Hazards Reduction Program), there are over 200 buildings utilizing base isolation. The concept of base isolation brings together damping elements to reduce lateral forces, accelerations and inter-story drifts on the building. In addition to protecting the building itself, base isolation protects the components within the building as well. There are three components to a base isolated building. They are the superstructure, base isolation system, and substructure.

The superstructure is the structural system of the building and is connected to the base isolation system. The base isolation system connects the superstructure to the substructure. It is here where the base isolators are present. While an earthquake occurs, the base isolators act as dampeners, dissipating energy through their design thus reducing the forces acting on the building. The base isolation system also needs a cavity where the base isolators will be placed. They will be pancaked between a slab and the foundation in this report. Lastly, the substructure is the foundation. During an earthquake, the base shear acting through the foundation is transferred to the base isolators.

Design of the Base Isolation System

The Seismic Design Handbook 2nd Edition by Farzad Naeim was the primary source used for design of the base isolation system. In the book, Naeim describes the intricacies of base isolation in addition to the design aspects of it. He makes a good point stating that base isolation is not used to reduce construction costs but reduce damage to the structures and its contents. This idea was the main focus going forward so it will be seen later that changes to the cost and schedule were affected.

Naiem then goes onto reference the guidelines of ASCE7-05 for seismically isolated structures. In ASCE7-10, the chapter for seismically isolated structures is Chapter 17. Base isolation design is dependent on several variables. The important variables are the minimum design displacement and the maximum design displacement. The minimum design displacement is the minimum amount of displacement the structure has to endure before the base isolators take effect. It is at that moment when the base isolators begin damping.

The following is a list of equations used for calculations. These equations were then used in Microsoft excel to determine the design displacements, which was a more efficient way of calculating than by hand. Screenshots of the excel spreadsheet used for this report can be found in Appendix E.

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D}$$

$$D_M = \left(\frac{g}{4\pi^2}\right)\frac{S_{M1}T_M}{B_M}$$

$$T_D = 2\pi\sqrt{\frac{W}{K_{D\,min}g}}$$

$$T_M = 2\pi\sqrt{\frac{W}{K_{M\,min}g}}$$

$$K_{Dmax} = (1+k)\frac{K_{Dmin}}{(1-k)}$$

$$K_{Mmax} = (1+k)\frac{K_{Mmin}}{(1-k)}$$

Where,
S_{D1} and S_{M1} = spectral coefficients
B_D and B_M = damping coefficients
T_D and T_M = isolated periods
g = gravitation acceleration (in/s^2)
W = weight of the building
K_{Dmin} and K_{Mmin} = minimum effective horizontal stiffness
k = ±% variation

$_D$ corresponds to DBE response
$_M$ corresponds to MCE response

Spreadsheets showing calculations for these equations can be found in Appendix XX.

A few assumptions were made for the calculation process. For this report, I wanted the isolation system have an effected period between T_D = 4.64 sec. and T_M = 5.46 sec. These periods were selected by multiplying the fixed base period of 1.09 sec. by 4.75 and 5 respectively. The effective damping ratio, $\beta_{D,}$ was assumed to be 15%. This value is commonly used in base isolation design according to Naiem. However, this value will be used when modeling the structure in SAP2000. There would also be a k of 10% variation in stiffness. Lastly, the damping coefficients for both B_D and B_M were assumed to be 1.35. This number can be found by linear interpolation in IBC-2006 (Table 1623.2.2.1).

Because T_D, S_{D1} g, and B_D are given, the minimum design displacement, using the equation listed above, was 18.61 in. Also, because T_M, S_{M1} g, and B_M are given, the maximum design displacement was 32.86 in. These values would be the range for the target displacement desired when modeling the structure later. In addition, because T_D, T_M, W, and g were given, K_{Dmin} and K_{Mmin} could be calculated. K_{Dmin} was 81 kips/in. and K_{Mmin} was 59 kips/in. K_{Dmax} and K_{Mmax} were 99 kips/in. and 72 kips/in. respectively, per equations used above.

After finding the above values, the design forces of both the superstructure and substructure would be determined. The equations are as follows:

$$V_b = K_{Dmax} D_D$$

$$V_s = \frac{K_{Dmax} D_D}{R_I}$$

Where,
V_b = minimum lateral seismic force on elements below the superstructure
V_s = minimum shear force on superstructure as if it were fixed
R_I = reduction factor related to R found in ASCE7-10 for seismically designed structures

$$1.0 \leq R_I = (3/8)R \leq 2.0$$

V_b and V_s were calculated to be 1845 kips and 1640 kips respectively. After determining the shear force on the superstructure, I did a vertical distribution of forces. Figure 42 shows this.

Vertical Distribution of Forces (V = 1640 kips)							
Level	w_x (kips)	h_x (ft)	Not used for Vertical Distribution of Isolated Structures	$w_x h_x$	C_{vx}	F_x (kips)	M (k-ft)
10	512	127.75		65408	0.054	88.7	11330.4
9	2440	107.25		261690	0.216	354.8	38057.4
8	2427	93.25		226318	0.187	306.9	28616.9
7	2426	79.25		192261	0.159	260.7	20660.7
6	2445	65.25		159536	0.132	216.3	14115.5
5	2546	51.25		130483	0.108	176.9	9067.8
4	2220	37.25		82695	0.068	112.1	4177.0
3	2530	25.64583		64884	0.054	88.0	2256.4
2	2137	12.25		26178	0.022	35.5	434.8
				1209452		1640	128716.9
Base Shear		1640	**Overturning Moment**		128716.9		

Figure 42: *Vertical distribution of design forces for base isolation*

If you compare the forces applied to the building before and after base isolation, it can be seen that there is an immense difference in seismic loading. Figure 43 shows the loads before base isolation. Figure 44 shows the design loads after base isolation. Each figure provides the force at each level in addition to the shear force seen on each level.

Story Shears			
Level	h_x (ft)	F_x (kips)	F_v (kips)
10	127.75	322.6	322.6
9	107.25	1228.8	1551.4
8	93.25	1021.9	2573.3
7	79.25	829.5	3402.8
6	65.25	651.8	4054.6
5	51.25	498.3	4552.9
4	37.25	288.8	4841.7
3	25.65	204.1	5045.8
2	12.25	67.0	5112.8

Figure 43: *Story forces and shears before base isolation*

Story Shears			
Level	h_x (ft)	F_x (kips)	F_v (kips)
10	127.75	88.7	88.7
9	107.25	354.8	443.5
8	93.25	306.9	750.4
7	79.25	260.7	1011.1
6	65.25	216.3	1227.5
5	51.25	176.9	1404.4
4	37.25	112.1	1516.5
3	25.65	88.0	1604.5
2	12.25	35.5	1640.0

Figure 44: *Story forces and shears after base isolation*

The tables show that the reduction in base shear on the structure is reduced by 67.9% when base isolation is implemented. Even though there is a reduction in base shear, the seismic load is still larger than the worst case wind load found above. Because of the smaller base shear, the structure could also be redesigned. However, due to time constraints, the structure was only modified and redesign was not taken into consideration.

Elements of a Base Isolation System

Before moving onto the analysis of the base isolation systems, I wanted to provide a quick look at the elements of a base isolation system.

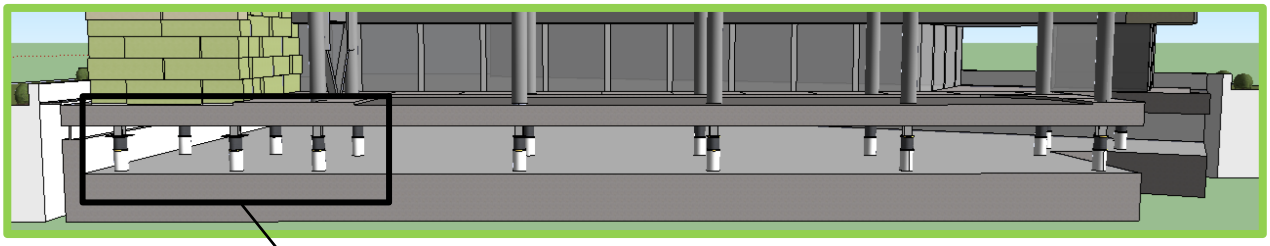


Figure 45: *General view of the base isolation system spanning the entire building*

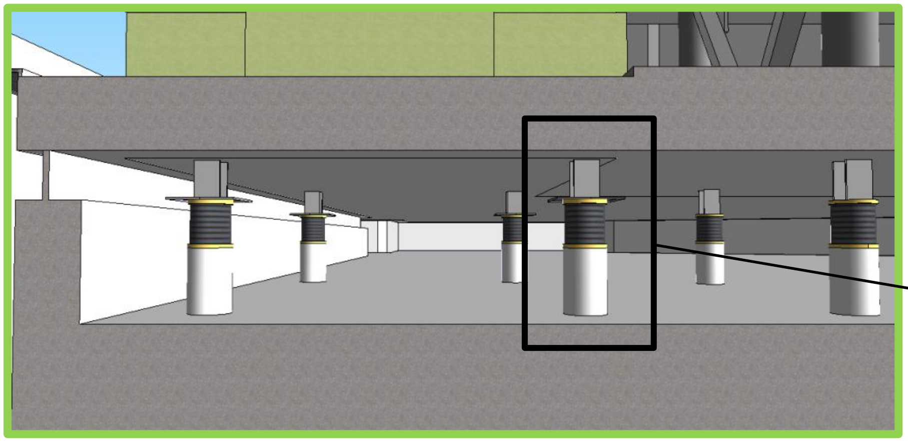


Figure 46: *Close-up of the base isolation system. Note the cavity between the top slab and the foundation. This is allows to the isolators*

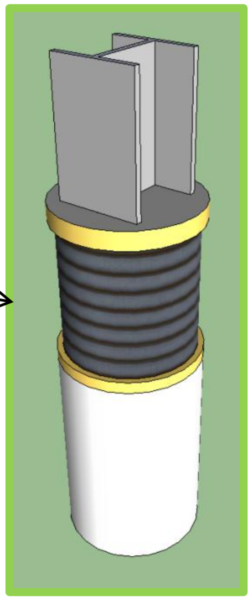


Figure 47: *(Right) Close=up of the base isolator itself. Pictured is a Sketchup Model of a lead rubber isolator*

In Figure 45, the elements of a base isolation can be seen. The isolators are located in the cavity between the superstructure (above the isolators) and the substructure (below the isolators).

On the following pages, the values found previously will be used in modeling the structure in SAP2000 as stated earlier. As proposed, the use of SAP2000 will help compare results of base isolation for two models. One model used the existing structure on a base isolation system. The second model used the modified structure. Comparisons will be made between displacements and drifts.

Analysis of Existing Structure with Base Isolation

Base isolation of the bed tower addition was modeled using SAP2000 because of its powerful nonlinear analysis application. According to ASCE7-10 there are two analysis procedures to choose from. They are the equivalent lateral force procedure and all others that do not fall under the requirements of of 17.4.1 shall be designed using a dynamic analysis procedure. The bed tower addition, because of its parameters, did not meet the requirements for using the equivalent lateral procedure. Instead, a time history analysis was performed.

For this report, an equivalent lateral force procedure was used set parameters for the desired design displacements while a time history analysis was used to check them. Since the building was moved to San Francisco, the time history of the 1989 Loma Prieta earthquake was chosen. It made significant destruction to the area and one of the most memorable earthquakes. The time history information was found on the Pacific Earthquake Engineering Research Center database. It provided the accelerations and response spectrum needed for analysis.

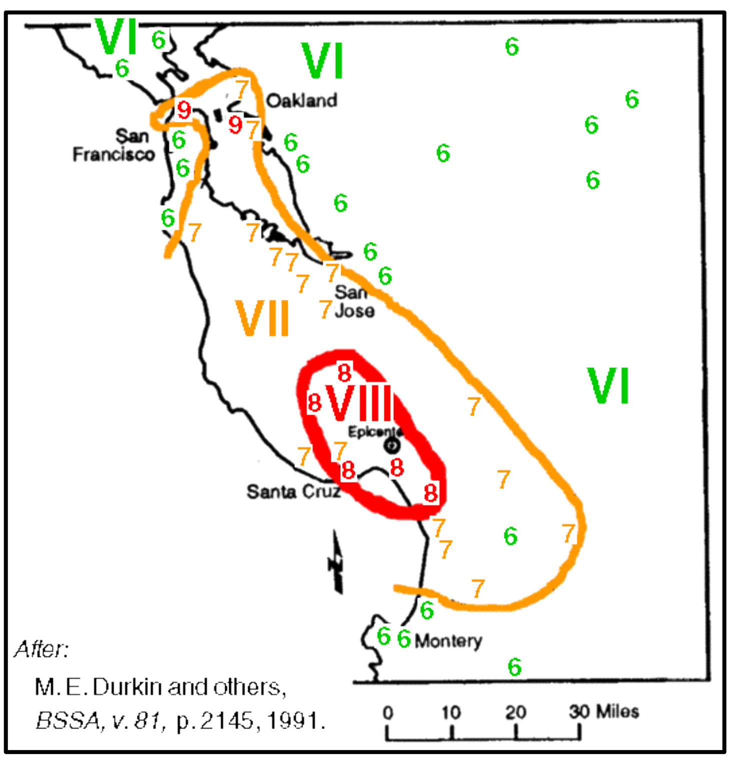


Figure 48: *Map showing location of epicenter of Loma Prieta relative to San Francisco (Nevada)*

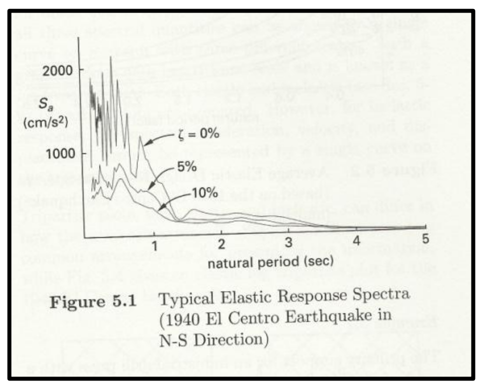


Figure 49: *Example response spectra*

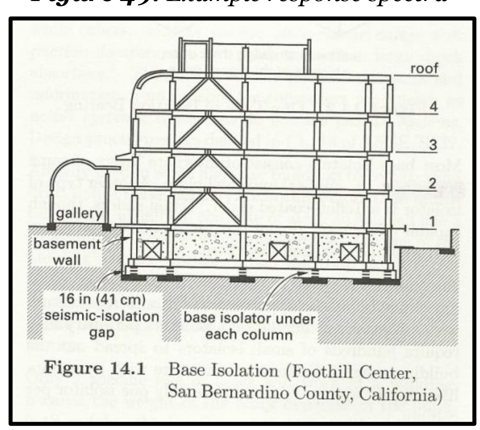


Figure 50: *Base Isolation example*

Loma Prieta's epicenter occurred 59 miles south-southeast of San Francisco, CA. In order to get the most accurate results, the time history found on the PEER Ground Motion Database website needed to be scaled (PEER stands for Pacficic Earthquake Engineering Research Center). The scaled time history was then put into SAP2000 to reenact what the structure would have felt as if it were present during the time of the earthquake.

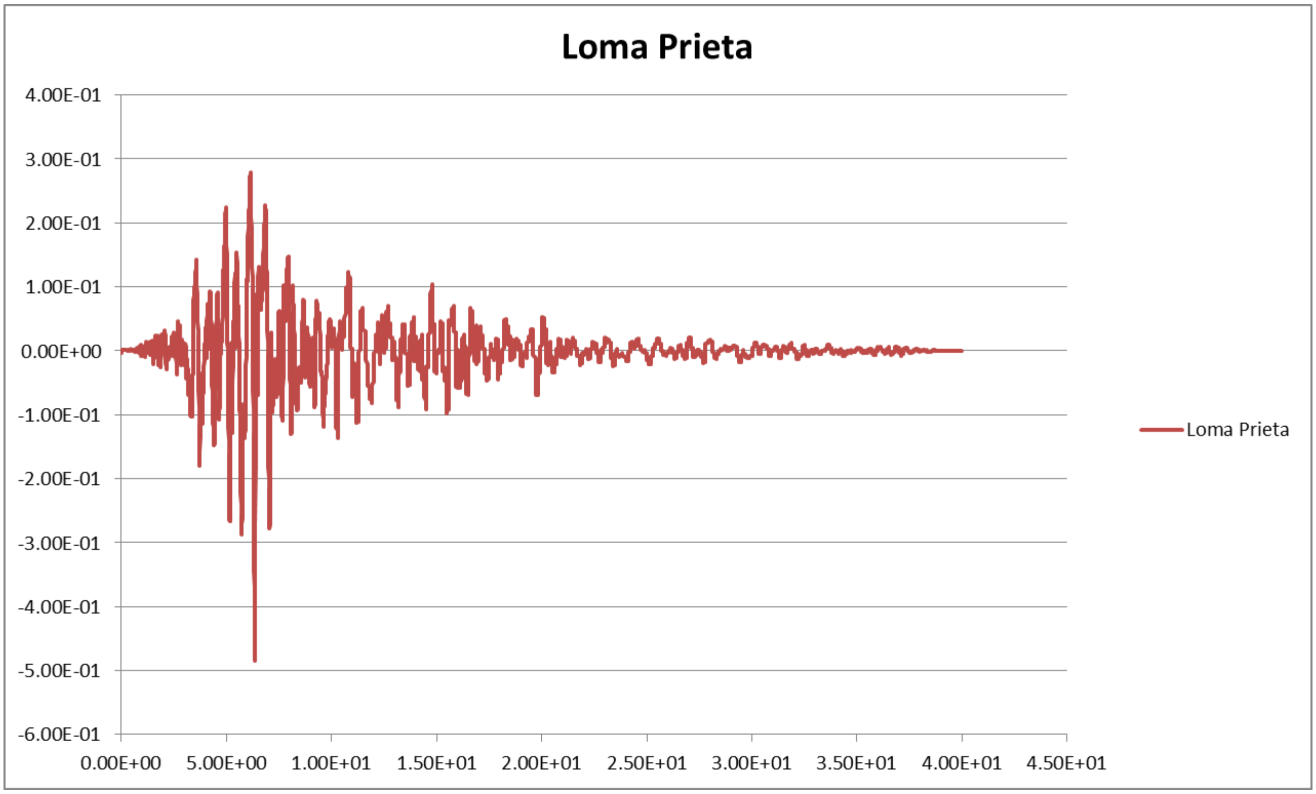


Figure 51: *Time history of 1989 Loma Prieta*

The graph above shows the time history of Loma Prieta. This was found to be the worst case scenario and so it was used for the nonlinear analysis time history in SAP2000. The earthquake is acting in the direction parallel to the fault.

For the purpose of this report, the only elements of the structure modeled in SAP2000 were the lateral frame system. As stated in the beginning of the report, there are eight concentrically braced frames throughout the building. Four frames run in the X-direction. Two are at full building height, one is at 8 stories high, and the fourth one is one story high. One frame runs in the Y-direction and that spans 8 stories high. Lastly, there are three frames running on a 60° angle, from the x-axis, all spanning full story height.

To the right is Figure 49 which is a 3D extruded view of the SAP2000 model used for analysis. The yellow elements are the concentrically braced frames modeled as line elements with frame sections. The red elements are the floors modeled as thin shell areas and the small green elements are the base isolators modeled as links.

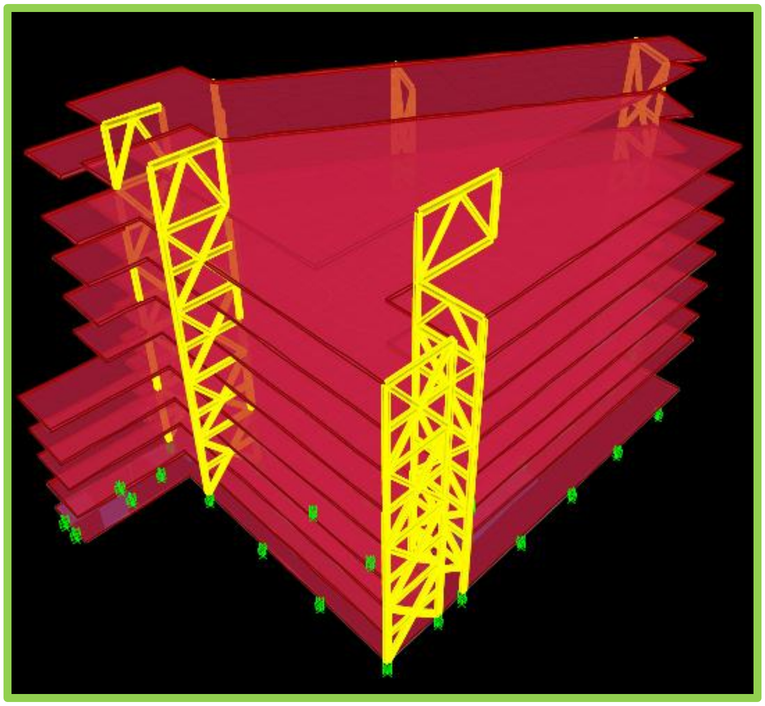


Figure 52: *3D Extruded SAP2000 Model of the Bed Tower Addition*

Each level was modeled with its own join constraint. Two joint constraints were used; diaphragm and body. With the use of the two, the area elements remained constrained to translation in the x and y axis and making it a rigid diaphragm. Rigid end offsets or insertion points were not used when modeling the braced frames. Panel zones were also not taken into account. The area elements were modeled as thin shell elements. Thin shell was used instead to ignore effects of shear deformations.

After modeling the structure, I had to define the load cases which needed to be used for analysis. Two nonlinear modal time history load cases were used. One load case was a Loma Prieta time history in the x-direction and the other being a Loma Prieta time history in the y-direction. Both directions were checked because the direction of earthquakes cannot be predicted. In each load case, the Loma Prieta time history given above was scaled by 2.643. This number was found using the PEER website under scaled earthquakes.

Once the analyses of the time histories were completed, several values were recorded. Displacements, drifts, and deflection were checked and would be used to compare base isolation with the modified structure.

BASE ISOLATION WITH EXISTING STRUCTURE

LOMA PRIETA X-DIRECTION

Brace 13 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	19.20	0.00	0.00
2	19.81	1.31	1.47
3	20.35	1.16	1.61
4	20.82	1.02	1.39
5	21.48	1.44	1.68
6	22.12	1.38	1.68
7	22.73	1.31	1.68
8	23.36	1.38	1.68
9	23.82	0.98	1.68
$\delta = S_9 - S_1$	4.62	$\Delta_{max} = 0.010hsx$	

Brace 5 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	18.45	0.00	0.00
2	19.00	1.17	1.47
3	19.46	1.00	1.61
4	19.86	0.87	1.39
5	20.41	1.20	1.68
6	20.90	1.06	1.68
7	21.43	1.15	1.68
8	22.01	1.24	1.68
9	22.43	0.92	1.68
10	22.97	1.17	2.40
$\delta = S_9 - S_1$	4.51	$\Delta max = 0.010hsx$	

Brace 2 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	18.19	0.00	0.00
2	18.70	1.12	1.47
3	19.14	0.95	1.61
4	19.52	0.82	1.39
5	20.04	1.13	1.68
6	20.51	1.00	1.68
7	20.97	1.00	1.68
8	21.52	1.20	1.68
9	21.93	0.89	1.68
10	22.44	1.10	2.40
$\delta = S_9 - S_1$	3.75	$\Delta max = 0.010hsx$	

Brace XA Displacements	
Level	Max Displacement (in)
1	18.15
10	22.37
$\delta = S_9 - S_1$	4.22

Brace XC Displacements	
Level	Max Displacement (in)
1	18.41
10	22.87
$\delta = S_9 - S_1$	4.47

Brace 12 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
9	23.47	0.00	1.68
10	24.08	1.32	2.40
$\delta = S_9 - S_1$	0.61	$\Delta max = 0.010hsx$	

Brace XF Displacements	
Level	Max Displacement (in)
1	18.78
10	23.61
$\delta = S_9 - S_1$	4.83

Figure 53: *Displacement and drift tables for braced frames in X-direction for base isolation system with existing structure*

BASE ISOLATION WITH EXISTING STRUCTURE
LOMA PRIETA Y-DIRECTION

Brace J Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	18.15	0.00	0.00
2	18.70	1.20	1.47
3	19.24	1.16	1.61
4	19.74	1.08	1.39
5	20.39	1.42	1.68
6	20.99	1.28	1.68
7	21.54	1.20	1.68
8	22.22	1.47	1.68
9	22.72	1.08	1.68
$\delta = S_9 - S_1$	4.57	$\Delta_{max} = 0.010hsx$	

Brace XA Displacements	
Level	Max Displacement (in)
1	18.26
10	23.84
$\delta = S_9 - S_1$	5.58

Brace XC Displacements	
Level	Max Displacement (in)
1	18.70
10	24.50
$\delta = S_9 - S_1$	5.80

Brace XF Displacements	
Level	Max Displacement (in)
1	19.35
10	25.47
$\delta = S_9 - S_1$	6.12

Figure 54: *Displacement and drift tables for braced frames in Y-direction for base isolation system with existing structure*

From the analysis, it can be seen that the displacements of each isolator ranged between 18.15 and 19.20 inches. It can be recalled that the minimum design displacements according to ASCE7-10 were to be between 18.61 and 32.86 in. Even though some of the isolators are below 18.61 inches, the analysis is still plausible given that the design displacements are a guide.

The max drifts were determined by finding the difference between story displacements. These drifts were then compared to the max allowable drift determined by the code: $\Delta_{max} = 0.010h_{sx}$. However the drifts from the analysis needed to be multiplied by a factor related to the parameters of the structure.

$$\Delta_x = \frac{C_d \Delta_{xe}}{I_e}$$

Where,
C_d = the deflection amplification
Δ_{xe} = the deflection at story x superstructure as if it were fixed
I_e = the importance factor

The resulting drifts, after multiplying the factors, can be seen in the green cells. Conditional formatting was also used in the spreadsheet. If the drifts were less than the max allowable drifts then the cells would turn green. It can be seen from the tables that all levels passed allowable drift limits.

It can also be recalled that the Y-direction was determined to be the most critical case from the RAM displacement outputs. The SAP2000 model also proves this. In the X-direction, the total displacement (the distance between absolute roof displacement and ground floor displacement) ranged between 3.75 in. and 4.83 in. without the exception of Brace 12 which only spans one floor. In the Y-direction, the total displacements range between 4.57 in. and 6.12 in. These displacements are lower than the ones found with a fixed base, but this makes sense because the base isolators dissipate energy from the base force resulting in lesser applied loads to the building.

Analysis of Modified Structure with Base Isolation

After analyzing the existing structure with base isolation, I wanted to compare the same results with the modified structure, found from RAM, on a base isolation system. The same assumptions were made as e existing structure so it was predicted that the displacements would be lower on the modified structure.

BASE ISOLATION WITH MODIFIED STRUCTURE
LOMA PRIETA X-DIRECTION

Brace 13 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	19.22	0.00	0.00
2	19.78	1.22	1.47
3	20.29	1.09	1.61
4	20.75	0.99	1.39
5	21.41	1.44	1.68
6	22.05	1.39	1.68
7	22.66	1.32	1.68
8	23.34	1.49	1.68
9	23.83	1.06	1.68
$\delta = S_9 - S_1$	4.61	$\Delta_{max} = 0.010h_{sx}$	

Brace 5 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	18.47	0.00	0.00
2	18.97	1.09	1.47
3	19.41	0.95	1.61
4	19.81	0.86	1.39
5	20.36	1.19	1.68
6	20.85	1.06	1.68
7	21.39	1.18	1.68
8	22.01	1.35	1.68
9	22.46	0.98	1.68
10	22.99	1.14	2.40
$\delta = S_9 - S_1$	4.52	$\Delta max = 0.010hsx$	

Brace 2 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	18.20	0.00	0.00
2	18.69	1.05	1.47
3	19.10	0.90	1.61
4	19.47	0.81	1.39
5	20.00	1.13	1.68
6	20.46	1.00	1.68
7	20.94	1.04	1.68
8	21.53	1.30	1.68
9	21.98	0.96	1.68
10	22.49	1.11	2.40
$\delta = S_9 - S_1$	3.77	$\Delta max = 0.010hsx$	

Brace XA Displacements	
Level	Max Displacement (in)
1	18.17
10	22.42
$\delta = S_9 - S_1$	4.25

Brace XC Displacements	
Level	Max Displacement (in)
1	18.42
10	22.90
$\delta = S_9 - S_1$	4.48

Brace 12 Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
9	23.49	0.00	1.68
10	24.05	1.21	2.40
$\delta = S_9 - S_1$	0.56	$\Delta max = 0.010hsx$	

Brace XF Displacements and Drifts	
Level	Max Displacement (in)
1	18.80
10	23.61
$\delta = S_9 - S_1$	4.81

Figure 55: *Displacement and drift tables for braced frames in X-direction for base isolation system with modified structure*

BASE ISOLATION WITH MODIFIED STRUCTURE

LOMA PRIETA Y-DIRECTION

Brace J Displacements and Drifts			
Level	Max Displace. (in)	Max Drift (in)	Allow. Drift (in)
1	18.14	0.00	0.00
2	18.67	1.15	1.47
3	19.18	1.11	1.61
4	19.67	1.05	1.39
5	20.31	1.39	1.68
6	20.89	1.26	1.68
7	21.46	1.22	1.68
8	22.18	1.58	1.68
9	22.72	1.16	1.68
$\delta = S_9 - S_1$	4.58	$\Delta_{max} = 0.010h_{sx}$	

Brace XA Displacements	
Level	Max Disp. (in)
1	18.25
10	23.88
$\delta = S_9 - S_1$	5.63

Brace XC Displacements	
Level	Max Disp. (in)
1	18.69
10	24.58
$\delta = S_9 - S_1$	5.88

Brace XF Displacements	
Level	Max Disp. (in)
1	19.36
10	25.61
$\delta = S_9 - S_1$	6.25

Figure 56: *Displacement and drift tables for braced frames in Y-direction for base isolation system with modified structure*

There was very little difference in displacements and drifts with the modified structure on top of base isolation. This proves that the base isolation system reduces the forces on the building significantly. With the modified structure on top, the Y-direction controlled deflections again just like the existing structure. To compare the numbers closely, the tables below will provide side by side comparisons, or lack thereof, between the displacements. However if you look at the drifts, they look very close but are slightly different. According the percent change in drift from one structure to the other, various levels have a difference. The differences in drifts are notable at the top and bottom of the structure. In the middle there is little change but there is nothing that significantly sticks out. The biggest change in displacement is 0.45% when the structure at level 5 of Brace J which decreases from 20.394 in. to 20.310 in. For drift, the largest change occurred at level 8 of Brace 13 which increases from 1.38 in. to 1.48 in. Other values can be found in figure 57 on the next page.

Comparison of Structures - Loma Prieta X-direction (all values in inches)

Brace 13

Level	Max Displacement: Existing Structure	Max Displacement: Modified Structure	% Change	Max Drift: Existing Structure	Max Drift: Modified Structure	% Change
1	19.2	19.2	-0.09%	0.00	0.00	
2	19.8	19.8	0.13%	1.31	1.22	6.92%
3	20.3	20.3	0.28%	1.16	1.09	6.00%
4	20.8	20.7	0.35%	1.02	0.99	3.18%
5	21.5	21.4	0.35%	1.44	1.44	0.59%
6	22.1	22.0	0.34%	1.38	1.39	-0.20%
7	22.7	22.7	0.32%	1.31	1.32	-0.31%
8	23.4	23.3	0.09%	1.38	1.49	-8.31%
9	23.8	23.8	-0.06%	0.98	1.06	-7.49%

Comparison of Structures - Loma Prieta X-direction (all values in inches)

Brace 5

Level	Max Displacement: Existing Structure	Max Displacement: Modified Structure	% Change	Max Drift: Existing Structure	Max Drift: Modified Structure	% Change
1	18.5	18.5	-0.09%	0.00	0.00	
2	19.0	19.0	0.11%	1.17	1.09	6.70%
3	19.5	19.4	0.24%	1.00	0.95	5.57%
4	19.9	19.8	0.27%	0.87	0.86	1.81%
5	20.4	20.4	0.27%	1.20	1.19	0.38%
6	20.9	20.8	0.27%	1.06	1.06	0.10%
7	21.4	21.4	0.20%	1.15	1.18	-2.42%
8	22.0	22.0	-0.02%	1.24	1.35	-8.19%
9	22.4	22.5	-0.16%	0.92	0.98	-7.38%
10	23.0	23.0	-0.10%	1.17	1.14	2.30%

Comparison of Structures - Loma Prieta X-direction (all values in inches)

Brace 2

Level	Max Displacement: Existing Structure	Max Displacement: Modified Structure	% Change	Max Drift: Existing Structure	Max Drift: Modified Structure	% Change
1	18.2	18.2	-0.09%	0.00	0.00	
2	18.7	18.7	0.10%	1.12	1.05	6.60%
3	19.1	19.1	0.22%	0.95	0.90	5.39%
4	19.5	19.5	0.24%	0.82	0.81	1.20%
5	20.0	20.0	0.24%	1.13	1.13	0.23%
6	20.5	20.5	0.24%	1.00	1.00	0.08%
7	21.0	20.9	0.16%	1.00	1.04	-3.41%
8	21.5	21.5	-0.06%	1.20	1.30	-8.14%
9	21.9	22.0	-0.19%	0.89	0.96	-7.34%
10	22.4	22.5	-0.21%	1.10	1.11	-0.80%

Comparison of Structures - Loma Prieta Y-direction (all values in inches)

Brace J

Level	Max Displacement: Existing Structure	Max Displacement: Modified Structure	% Change	Max Drift: Existing Structure	Max Drift: Modified Structure	% Change
1	18.1	18.1	0.06%	0.00	0.00	
2	18.7	18.7	0.18%	1.20	1.15	4.05%
3	19.2	19.2	0.30%	1.16	1.11	4.63%
4	19.7	19.7	0.37%	1.08	1.05	2.92%
5	20.394	20.310	0.42%	1.42	1.39	1.78%
6	21.0	20.9	0.45%	1.28	1.26	1.58%
7	21.5	21.5	0.39%	1.20	1.22	-1.64%
8	22.2	22.2	0.15%	1.47	1.58	-7.60%
9	22.7	22.7	0.00%	1.08	1.16	-6.69%

Comparison of Structures - Loma Prieta X-direction (all values in inches)

Brace 12

Level	Max Displacement: Existing Structure	Max Displacement: Modified Structure	% Change	Max Drift: Existing Structure	Max Drift: Modified Structure	% Change
9	23.5	23.5	-0.08%	0.00	0.00	
10	24.1	24.1	0.12%	1.32	1.21	7.77%

Figure 57: *Displacement and drift comparison tables for each braced frame*

Comparison of Structures - Loma Prieta X-direction (all values in inches)

Level	Brace XA Max Displacement: Existing Structure	Brace XA Max Displacement: Modified Structure	Brace XA % Change	Brace XC Max Displacement: Existing Structure	Brace XC Max Displacement: Modified Structure	Brace XC % Change	Brace XF Max Displacement: Existing Structure	Brace XF Max Displacement: Modified Structure	Brace XF % Change
1	18.2	18.2	-0.11%	18.4	18.4	-0.09%	18.8	18.8	-0.09%
10	22.4	22.4	-0.22%	22.9	22.9	-0.12%	23.6	23.6	0.02%

Comparison of Structures - Loma Prieta Y-direction (all values in inches)

Level	Brace XA Max Displacement: Existing Structure	Brace XA Max Displacement: Modified Structure	Brace XA % Change	Brace XC Max Displacement: Existing Structure	Brace XC Max Displacement: Modified Structure	Brace XC % Change	Brace XF Max Displacement: Existing Structure	Brace XF Max Displacement: Modified Structure	Brace XF % Change
1	18.3	18.2	0.07%	18.7	18.7	0.03%	19.4	19.4	-0.04%
10	23.8	23.9	-0.17%	24.5	24.6	-0.31%	25.5	25.6	-0.54%

Y-DIRECTION

Comments/Notes

After evaluating both options on base isolation it was evident that there were no significant differences to warrant one option over the other. There are many more logical approaches that would have made a greater impact. One approach could have change in location in addition to redesign of the building with a different medium such as concrete. A second approach could have been implementing various base isolation systems. Other than lead rubber isolators, there are elastomeric bearings, friction base isolators, and use of rockers. Combining the existing structure with these features could have produced different results because each one dissipates energy in its own unique way. Lastly, the third approach involves moving the base isolators into different configurations. This approach could have produced very different results. According to Naiem, there are four typical configurations for the base isolators.

(From Figure 59 going clockwise)

1. Location on grade below in basement
2. Location under bottom of first story columns
3. Location on top of columns in basement
4. Location on top of first story columns

In hindsight, I feel the latter approach would have been the most appropriate approach to learning base isolation. However, from the analysis done for this report, it has been proven that the use of base isolation greatly reduces forces applied on the building. The base isolators can dissipate energy, damping the structure, and keeping it stable in times of crisis.

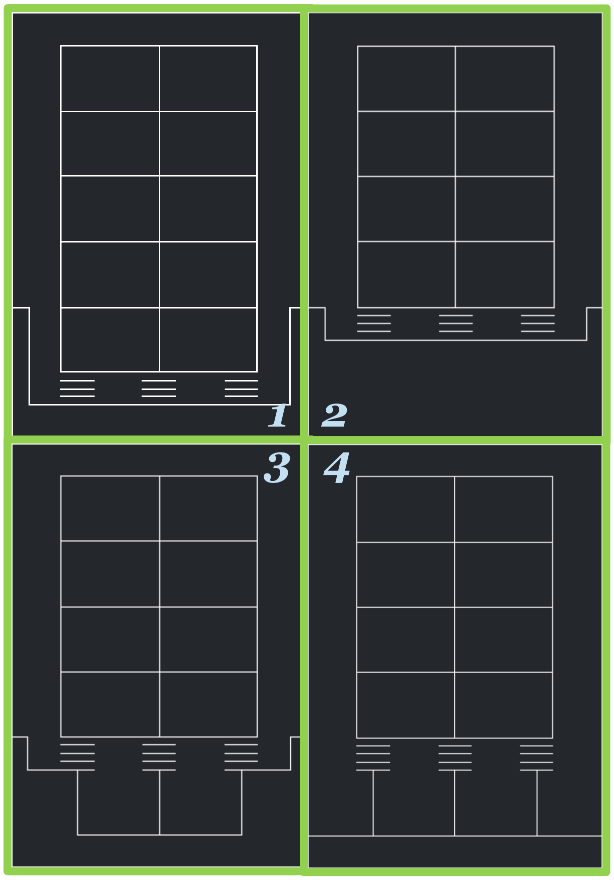


Figure 58: *Configurations of a base isolation system. Concepts from Naiem*

Architecture Breadth

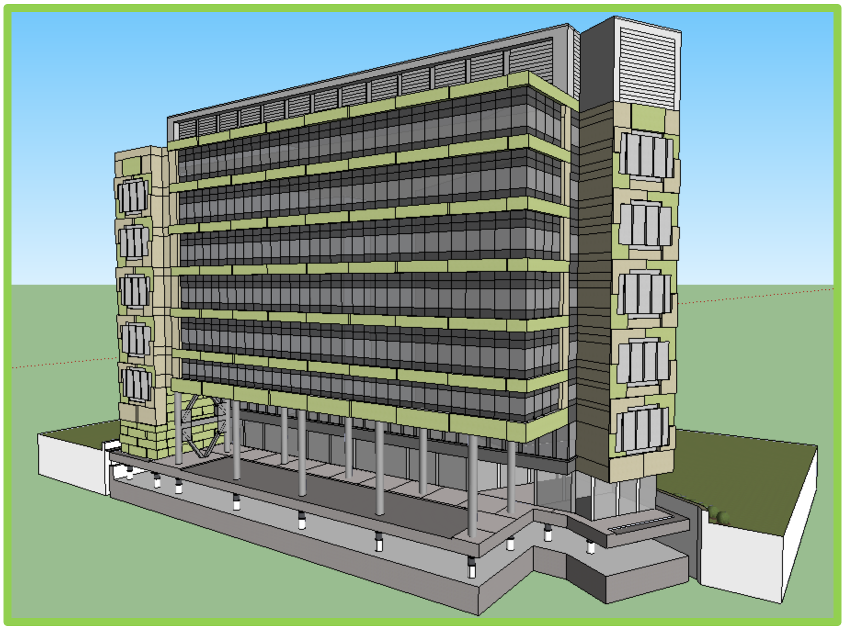


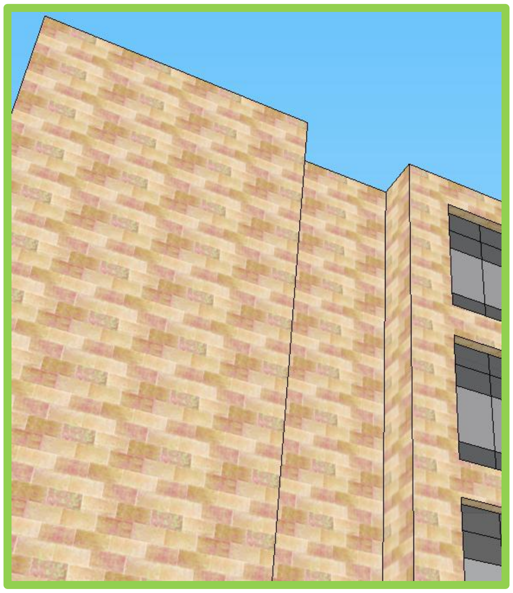
Figure 59: *(Top) Exterior view of the Bed Tower Addition in Wisconsin (Bottom) Exterior view of the Bed Tower Addition at SFSU*

The architecture of the bed tower addition was changed to match the façade of the new location. San Francisco State University (SFSU) was the new location chosen. In order to change the façade, small case studies were conducted on buildings located throughout the campus. One observation about the campus is that the façade of each building was made up of stucco or material look alike. The colors varied but the majority of the buildings were gray, tan, or white with colored accents. The second observation I noticed was that façade was made up of distinct panels. I thought this made the buildings unique and stand out. It also gave the buildings shape. Instead of seeing one color make up the façade, the panels broke up the color giving the building height and width.

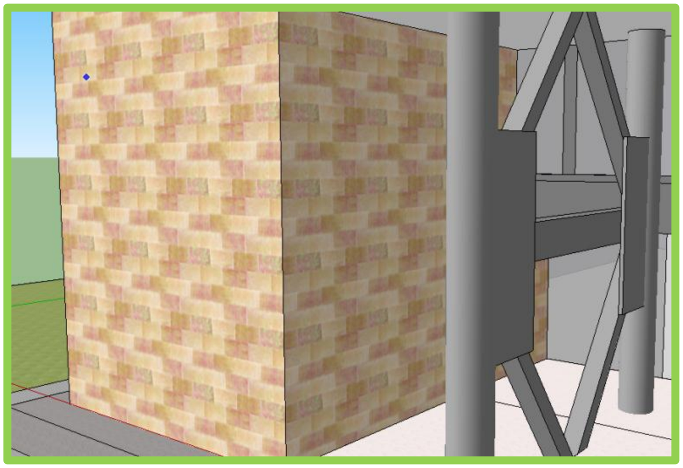
The following pages will show a few changes made to the façade. Some are very drastic while others were done to blend in with San Francisco State University's style. There are also four pictures of the buildings which influenced the change in architecture.

View showing the façade similar Hensil Hall

View showing the façade similar to the Admissions building



Wall near the entrance of the building



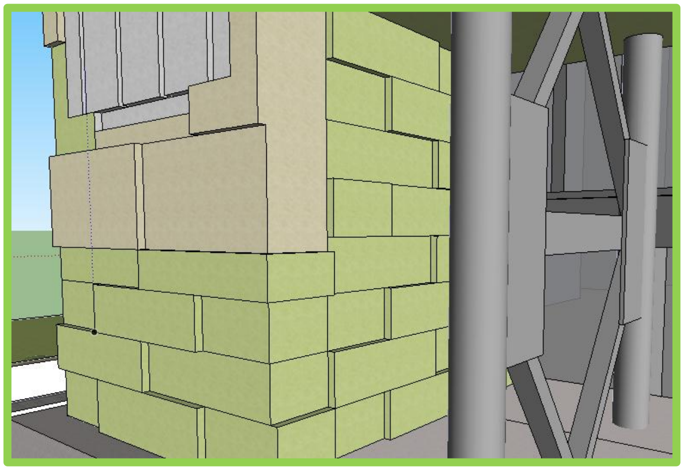


Figure 60: *Close-up views of the architectural changes to the façade of the building. Left views show the existing façade. Right views show the new façade. Case studies found on next page*

Figure 61: *Starting with the top left going clockwise are Student Services, Hensill Hall, Administration, and Humanities*

Another feature which needed to be addressed, was to find a way to cover the moat for the base isolation system. In Figure 60, is a picture of what the moat looks like. The moat is very important to the base isolation system. This cavity allows the building to "roll" back and forth without any hindrance. If the cavity was not designed for the maximum allowable displacement, the building could crash into the side of the moat. So it is important that the moat is large enough to allow the building to move efficiently.

According to research, most moats are covered up by an easily destructible element such as a light grate. Any material that is sacrificial to the building's expense when an earthquake occurs is the most desirable. For this building, I decided to use a light grate to cover up the moat. The grate will be useful for not allowing foreign objects into the moat such as leaves, large animals, or even people. Even though the grate is still considered light, it is still heavy and stiff enough to withhold weights such as people or animals but will break for an earthquake. A moat usually separates a retaining wall and the existing base isolation system. The retaining wall for the bed tower addition extended three feet above ground level as another safety precaution. In addition, shrubs would be planted along the retaining wall. With the use of the retaining wall and shrubs, this should help cover the moat and keep it hidden from the public.

The proposal did discuss that the architectural feature would be to find ways to cover the moat. However, during design, it was clear that the moat would be no bigger than three feet wide. During the proposal, I figured the moat would have been much larger and would have to incorporate a walking bridge. This was not the case and so the additional architectural feature mentioned above was used to expand on this architectural breadth.

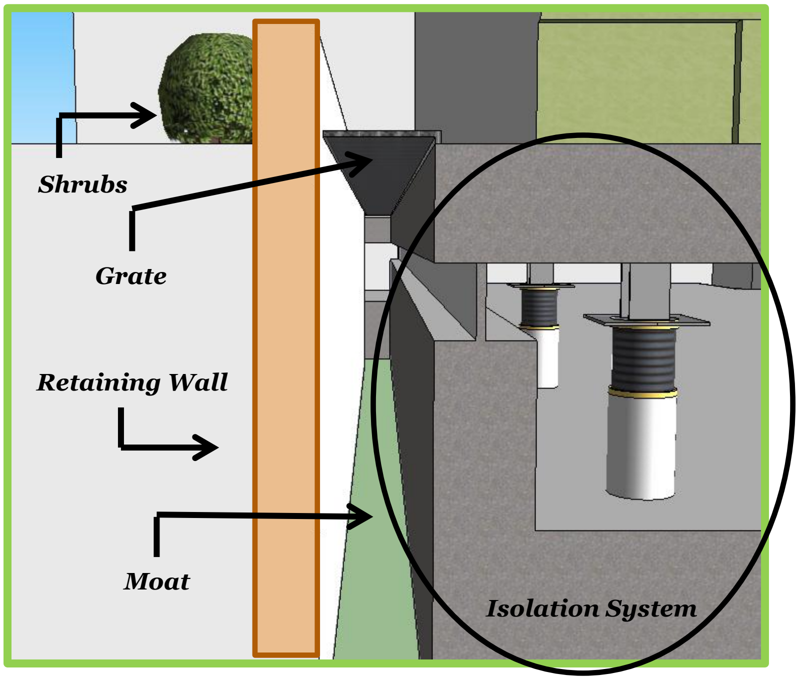


Figure 62: *Starting with the top left going clockwise are Student Services, Hensill Hall, Administration, and Humanities*

Construction Breadth

To construct the base isolation system, a few general features needed to be addressed. They are as follows:

1. Include a flat slab plate above the isolators
2. Create a moat for the displacement for the building
3. Create another floor below the flat slab for access to the isolators
4. Build the existing foundation below the isolators
5. Determine cost and installation time for the isolators

To include all the listed above, it would impact the cost and schedule of the building. Due to time constraints, only these five features would be checked. There are many other aspects of base isolation design which could have been addressed such as designing the seismic moat wall, flexible MEP connections, and testing/inspection costs.

Flat Slab Calculation

The flat slab was assumed to be the same thickness throughout the entire first floor of the building. To determine the thickness of the flat slab, the American Concrete Institute's ACI 318-08 building code book was used. Section 9.5 describes how to design flat slab's for deflection control. Table 9.5(c) provides the minimum thickness of slabs without interior beams (as shown on the right). To determine the thickest slab for simplicity of uniform construction, the longest clear span length found on the first floor was used as l_n. The longest clear span length was measured along one the of exterior 30 ft bays. It was assumed that the f_y of the concrete was 60 ksi and the that exterior panel did not use edge beams. So then the equation $l_n/30$ would be used to determine the minimum thickness of the flat slab, which was calculated to be: (30ft x 12 in/ft)/30 = 12 inches.

Table 9.5(c) - Minimum thickness of slabs without interior beams			
	Without Drop Panels		
	Exterior Panels		Interior Panels
f_y, psi	Without edge beams	With edge beams	
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
60,000	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$
75,000	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$

Figure 63: *ACI 318-08 Table*

Total area which the slab will cover is 18,955 ft^2 and when that is multiplied together, there is a total volume of 18,955 ft^3 of concrete.

Moat Calculation

The moat is a three foot wide gap between the base isolation system and the seismic retaining wall (Picture shown earlier in Architecture Breadth). The depth of the moat will be measured from the top of slab which is at ground level. It was assumed that there would be seven feet of "crawl" space for inspectors and engineers to access the isolators at any time. Also, the existing structure's mat foundation would be reused and this would go below the base isolators. With the one foot slab found above and the other values just listed, the total depth of the moat comes to 11.5 ft. Using AutoCAD, the area of the moat was found to be 2119 ft^2. After multiplying the depth and area together, the total volume of dirt needed to dig out for the moat construction was 24,370 ft^3.

Total Excavation for Base Isolation System

A floor to provide access to the isolators should be required for check and maintenance purposes. This creates a need for excavation of the new site. The excavation needs to be the height of the moat, which is 11.5 ft. Calculations include the new access floor and room for movement of the existing foundation to below the isolators. Excavation for the moat is not included because it was found above. The total area of excavation was 18,955 ft^2 and the total depth is 11.5 ft. The total volume needed for excavation was 217,982 ft^3.

Cost and Installation for the Base Isolators

Cost and Installation were hard values to assess because all isolators are custom made. According to a source from California, they said that isolators range between $8,000 and $20,000 per isolator. Installation costs vary widely but the lead time is about 6 weeks and 12-15 weeks for total project delivery time. The project delivery time includes installing all the isolators needed. It was difficult to find manufacturers to compare costs and installation time to. For the breadth it was assumed that the price per isolator would be the average of the range listed above at $14,000. The installation time was assumed to be 15 weeks given the worst case scenario.

Because the bed tower is an addition, there was a chance to fit the construction of the base isolators in between the construction of the link to the original hospital and the addition itself. If the base isolators were to be ordered May 21, 2009 one week after owner relocations of staff and files were started then the isolators would be delivered one day after the mat slab had been poured. Installation of the base isolation system would then take place during construction of the link saving more time than originally assumed. The installation of the base isolation system only hindered the schedule by one week. Appendix XX shows the original and new schedule. Circled in the appendix is where the base isolation system would be installed.

Overall though, the base isolation proved to be costly not making it a feasible approach but safety is much more important than cost. It could prevent not just equipment from being damaged but it could also save people's lives. For an extra 2 million dollars, people could live their lives with no worry.

	Total Project Time Delivery	Total Project Cost
Existing Structure	93 days	$ 59,100,000.00
Base Isolated Structure	100 days	$ 61,300,336.64
Time with respect to span of base isolation construction		
Cost with respect to the total project		

Conclusion

The focus of this report was to check if using a base isolation system would be a practical and feasible alternative to the structure. Implementing a base isolation system to the bed tower addition proved to be very effective in design but costly. Without base isolation, the structure would be designed to incur a base shear of 5112 kips. With base isolation, the design base shear decreased 211% to 1640 kips. This is a significant decrease given that the base shear in Wisconsin was 194 kips. Displacements were calculated between the minimum and maximum displacements required for base isolation. With the use of these displacements, it was practical to design the base isolation system for a desired period. The story to story drifts were also calculated to be less than the allowable drift limit. However, when comparing the two structures, there were no significant differences but because results concluded that both structures passed displacement and drift requirements, it is plausible that base isolation could be a practical solution.

Implementing base isolation would end up being costly though. It would have to take an extra $2,200, 340 dollars. In the overall aspect of the system, it is not too costly when equipment and lives are at stake, especially since it is a hospital. Therefore it can be concluded that if safety is a priority then going with base isolation would be a great practical and feasible solution.

Other alternatives could have explored but the learning experience was second to none. Seismic design of isolated structures was complicated and there is a reason why very few structures implement the idea. Use of base isolation will greatly reduce the load the building feels and therefore making it a safe which is exactly what the doctor ordered.

Appendices

Appendix A: AISC 2010 Seismic Provisions (Chapter F)

4b. K-Braced Frames

K-type braced frames are not permitted for OCBF.

5. Members

5a. Basic Requirements

Braces shall satisfy requirements of Section D1.1 for *moderately ductile members*.

5b. Slenderness

Braces in V or inverted-V configurations shall have KL/r ≤ $4\sqrt{E/Fy}$

6. Connections

6a. Diagonal Brace Connections

The required strength of diagonal brace connections is the load effect based upon the amplified seismic load.

Exception: The required strength of the brace connection need not exceed the following:

(1) In tension, the expected yield strength of the brace multiplied by 1.0 (LRFD) or divided by 1.5 (ASD), as appropriate. The expected yield strength shall be determined as RyFyAg.

(2) In compression, the expected brace strength in compression multiplied by 1.0 (LRFD) or divided by 1.5 (ASD), as appropriate. The expected brace strength in compression is permitted to be taken as the lesser of RyFyAg and 1.14FcreAg where Fcre is determined from Specification Chapter E using the equations for Fcr except that the expected yield stress RyFy is used in lieu of Fy. The brace length used for the determination of Fcre shall not exceed the distance from brace end to brace end.

(3) When oversized holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the load combinations stipulated by the applicable building code, not including the amplified seismic load

7. Ordinary Concentrically Braced Frames above Seismic Isolation Systems

7a. System Requirements

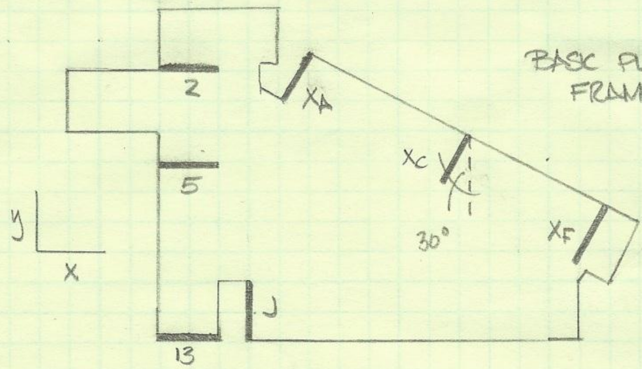
Beams in a V-type and inverted V-type braced frames shall be continuous between columns.

7b. Members

Braces shall have a slender ratio, KL/r ≤ $4\sqrt{E/Fy}$

Appendix B: Stiffness Calculations

RELATIVE STIFFNESS | TECH REPORT #3 | JESSEL ELLIOTT | 1/1



BASIC PLAN OF FRAME MEMBERS (TYPICAL)

FROM RISA, STIFFNESSES (k/in):

XA = 66.7
XC = 66.7
XF = 66.7
J = 111.1
13 = 111.1
5 = 66.7
2 = 52.6

ALL DUE TO A 1K LOAD

X/y COMPONENTS OF XA, XC, XF

$$\cos 30° = \frac{x}{K_{xc}} \Rightarrow x = 57.76$$

$$\sin 30° = \frac{y}{K_{xc}} \Rightarrow y = 33.35$$

IN THE X-DIRECTION

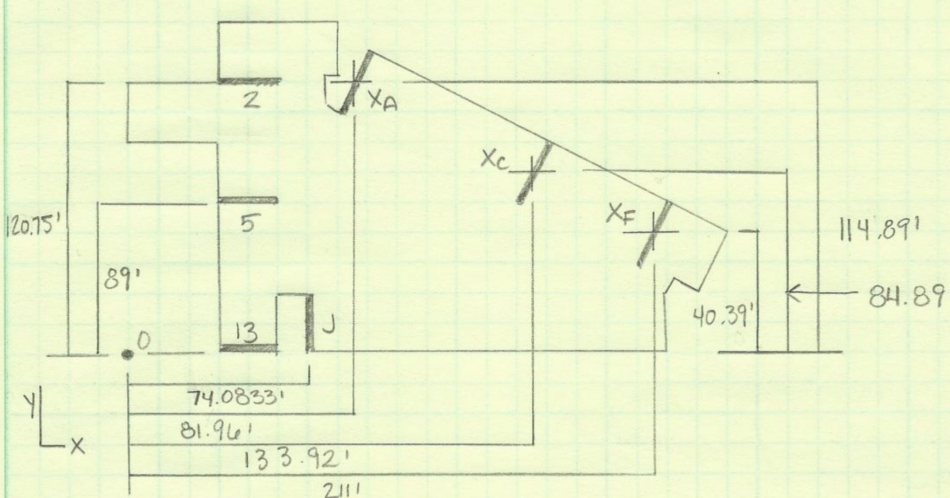
FRAME	STIFFNESS	RELATIVE STIFFNESS
2	52.6	13.03%
5	66.7	16.52%
13	111.1	27.52%
XA_x	57.76	14.31%
XC_x	57.76	14.31%
XF_x	57.76	14.31%
TOTAL:	403.68	100%

IN THE Y-DIRECTION

FRAME	STIFFNESS	RELATIVE STIFFNESS
J	111.1	52.62%
XA_y	33.35	15.79%
XC_y	33.35	15.79%
XF_y	33.35	15.79%
TOTAL:	211.15	100%

Appendix C: Center of Mass and Center of Rigidity Calculations

CENTER OF RIGIDITY | TECH REPORT #3 | JESSEL ELLIOTT | 1/1



FROM RELATIVE STIFFNESS CALCULATIONS:

STIFFNESSES:

2 = 52.6 k/in
5 = 66.7 k/in
13 = 111.1 k/in
J = 111.1 k/in

$X_{Ax} = 57.76$
$X_{Ay} = 33.35$
$X_{Cx} = 57.76$
$X_{Cy} = 33.35$
$X_{Fx} = 57.76$
$X_{Fy} = 33.35$

$$X_R = \frac{\Sigma R_i X_i}{\Sigma R_i} \qquad Y_R = \frac{\Sigma R_i Y_i}{\Sigma R_i}$$

$$X_R = \frac{(111.1)(74.0833) + (33.35)[81.96 + 133.92 + 211]}{111.1 + 3(33.35)}$$

$$= 106.4'$$

$$Y_R = \frac{(66.7)(89) + (52.6)(120.75) + (57.76)[40.39 + 84.89 + 114.89]}{111.1 + 66.7 + 52.6 + 57.76(3)}$$

$$= 64.8'$$

FROM RAM – $X_R = 111.97$, $Y_R = 59.38$

DUE TO THE DIFFICULTY OF THE ANGLED FRAMES X_A, X_C, X_F SEEMS PLAUSIBLE.

CENTER OF MASS FROM RAM: $X_M = 108.6$ $Y_M = 58.62$

ECCENTRICITY: $e_x = X_R - X_M = 106.4 - 108.6 = -2.2'$

$e_y = Y_R - Y_M = 64.8 - 58.62 = 6.18'$

Appendix D: Seismic Design Parameters

MULKEY
ENGINEERS & CONSULTANTS
P.O. Box 33127 • Raleigh, NC 27636-3127
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME SEISMIC LOADS
PROJECT NO. SHEET OF
SUBJECT
PREPARED BY J ELLIOTT DATE SP'12 CHECKED BY DATE

USING ASCE 7-10 (ASSUMED SITE CLASS C AND RISK CATEGORY IV)

FROM FIGURE 22-1, 22-2	$S_S = 1.500\,g$, $S_1 = 0.638\,g$	
TABLE 11.4-1, 11.4-2	$F_a = 1.0$, $F_v = 1.3$	
FIGURE 22-12	$T_L = 12$ SECONDS	
FIGURE 22-7	$PGA = 0.60$	
TABLE 11.8-1	$F_{PGA} = 1.0$	
FIGURE 22-17, 22-18	$C_{RS} = 1.042$, $C_{R1} = 0.986$	

EQUATION 11.4-1
$S_{MS} = F_a S_S = 1.0 \times 1.5 = 1.5g$
EQUATION 11.4-2
$S_{M1} = F_v S_1 = 1.3 \times 0.638 = 0.830g$

DESIGN SPECTRAL ACCEL PARAMETERS
EQUATION 11.4-3
$S_{DS} = 2/3\, S_{MS} = 1.000g$
EQUATION 11.4-4
$S_{D1} = 2/3\, S_{M1} = 0.553g$

EQUATION 11.8-1
$PGA_M = F_{PGA}\, PGA = 1.0 \times 0.6 = 0.6g$

$I_e = 1.50$

SECTION 11.6 – SEISMIC DESIGN CATEGORY

TABLE 11.6-1 SDC = D
TABLE 11.6-2 SDC = D

TABLE 12.2-1 DESIGN COEFFICIENTS & FACTORS FOR SEISMIC FORCE RESISTING SYSTEMS

$0.25\left(\frac{R}{1.5}\right) = 1.000$

3. STEEL ORDINARY CONCENTRICALLY BRACED FRAMES

SECTION	R	Ω_0	C_d	h_n
14.1	3 1/4	2	3 1/4	35'

EQUIVALENT LATERAL FORCE PROCEDURE

$V = C_s W$ $W = 19940\ K$ (FROM TECH 1 WEIGHT CALCS)

$C_S = \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \dfrac{1.000}{\left(\frac{3.25}{1.5}\right)}$ $C_S = 0.4615$

APPROX. FUND PERIOD (PG 90) TABLE 12.8-2

$T_a = C_t h_n^x = (.02)(127.8)^{(0.75)} = 0.76\ s$

TABLE 12.8-1

$S_{D1}\ 0.553 > .04$ $C_u = 1.4$

FUNDAMENTAL PERIOD T

$T = (0.76)(1.4) = \boxed{1.06\,s}$

FOR $T < T_L$ $C_s \le \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.553}{1.06\left(\frac{3.25}{1.5}\right)}$

$C_s \le \boxed{0.241} \le 0.46$

$V = (0.241)(19940) = 4805.54\ K$

INTERPOLATE K EXPONENT

$\dfrac{2.5-0.5}{2-1} = \dfrac{1.06-0.5}{K-1}$

$K = 1.28$

ME-02

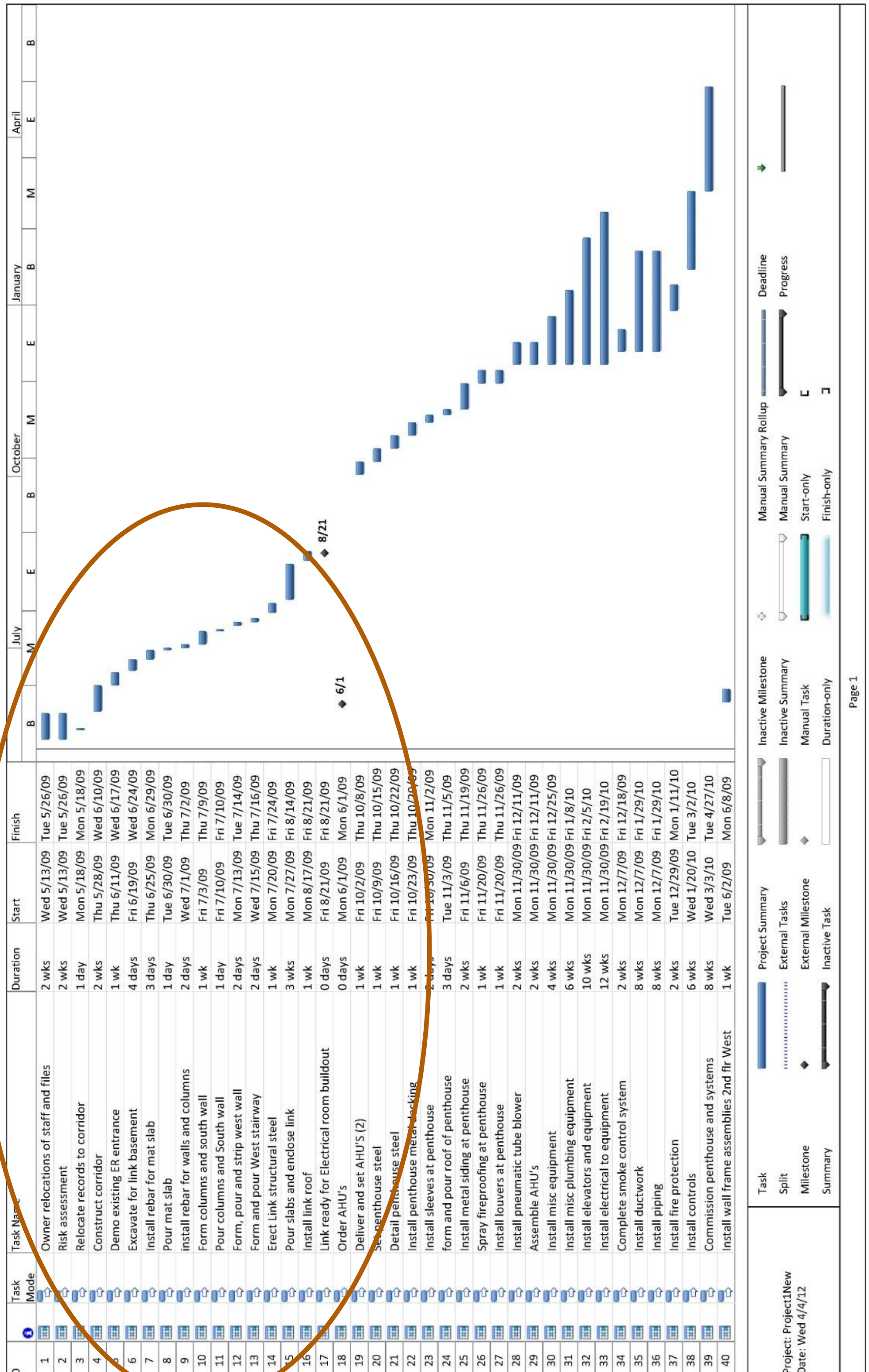
Appendix E: Isolation System Calculations

Minimum Design Displacements	(with 15% damping critical)		T	1.092	s
	10%	in variation	R	3.0	

(Displacement at the center of rigidity of the isolation system at the DBE)			(Displacement at the center of rigidity of the isolation system at the MCE)		
D_D	18.61	in	D_M	32.86	in
g	386.4	in/s^2	g	386.4	in/s^2
S_{D1}	0.553	g	S_{M1}	0.830	g
T_D	4.641	s	T_M	5.460	s
B_D	1.35		B_M	1.35	
β_D	15%		β_M	15%	
$K_{D\ min}$	81	kips/in	$K_{M\ min}$	59	kips/in
$K_{D\ max}$	99	kips/in	$K_{M\ max}$	72	kips/in
W	17099.29	kips	W	17099.29	kips
R_I	1.125				
Vb	1845				
Vs	1640				

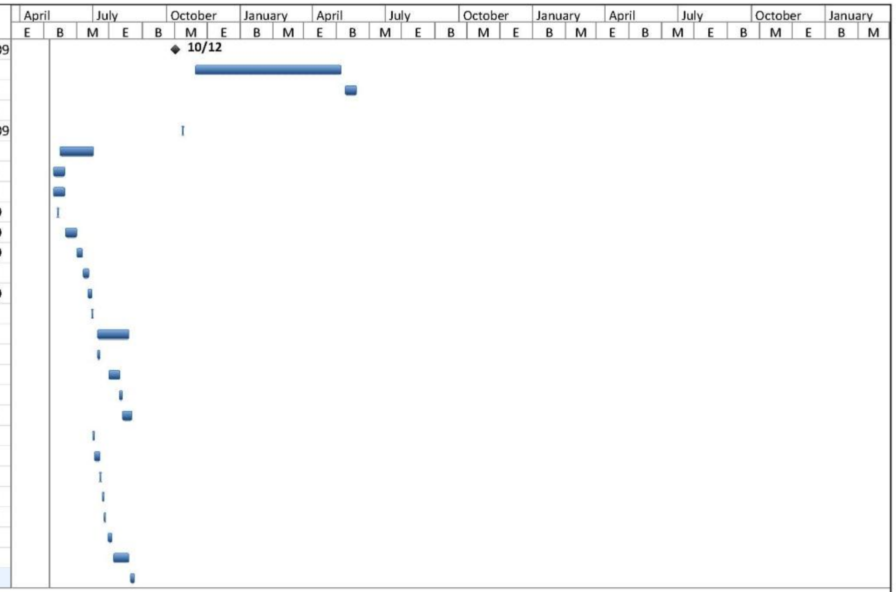
Isolator Design Procedure			(Lead-Rubber Isolators)			# of links	50	isolators
						$\Phi_{lead\ core}$	3.5	in
D_D	18.61	in	W_D	26467	k.in	$\Phi_{isolator}$	24	in
g	386.4	in/s^2	Q_d	356	kips	$A_{total\ pb\ new}$	241	in^2
S_{D1}	0.553	g	K_d	62	kips/in	$Q_{d\ new}$	361	kips
T_D	4.641	s	D_y	0.64	in	K_{pb}	19	kips/in
B_D	1.35		$A_{total\ pb}$	237	in^2	K_{rubber}	62	kips/in
β_D	15%					A_{rubber}	26903	in^2
K_H	81	kips/in	After finding $A_{total\ pb}$ calculate new area so it is ≥ old area			t_r	26.0	in
$F_{y\ pb}$	1.5	ksi				C_b	0.088	
G	0.06					C_s	0.078	

Appendix F: Construction Cost and Schedule Analysis



ID	Task Name	Duration	Start	Finish
1	Owner relocations of staff and files	2 wks	Wed 5/13/09	Tue 5/26/09
2	Risk assessment	2 wks	Wed 5/13/09	Tue 5/26/09
3	Relocate records to corridor	1 day	Mon 5/18/09	Mon 5/18/09
4	Construct corridor	2 wks	Thu 5/28/09	Wed 6/10/09
5	Demo existing ER entrance	1 wk	Thu 6/11/09	Wed 6/17/09
6	Excavate for link basement	4 days	Fri 6/19/09	Wed 6/24/09
7	Install rebar for mat slab	3 days	Thu 6/25/09	Mon 6/29/09
8	Pour mat slab	1 day	Tue 6/30/09	Tue 6/30/09
9	install rebar for walls and columns	2 days	Wed 7/1/09	Thu 7/2/09
10	Form columns and south wall	1 wk	Fri 7/3/09	Thu 7/9/09
11	Pour columns and South wall	1 day	Fri 7/10/09	Fri 7/10/09
12	Form, pour and strip west wall	2 days	Mon 7/13/09	Tue 7/14/09
13	Form and pour West stairway	2 days	Wed 7/15/09	Thu 7/16/09
14	Erect Link structural steel	1 wk	Mon 7/20/09	Fri 7/24/09
15	Pour slabs and enclose link	3 wks	Mon 7/27/09	Fri 8/14/09
16	Install link roof	1 wk	Mon 8/17/09	Fri 8/21/09
17	Link ready for Electrical room buildout	0 days	Fri 8/21/09	Fri 8/21/09
18	Order AHU's	0 days	Mon 6/1/09	Mon 6/1/09
19	Deliver and set AHU'S (2)	1 wk	Fri 10/2/09	Thu 10/8/09
20	Set penthouse steel	1 wk	Fri 10/9/09	Thu 10/15/09
21	Detail penthouse steel	1 wk	Fri 10/16/09	Thu 10/22/09
22	Install penthouse metal decking	1 wk	Fri 10/23/09	Thu 10/29/09
23	Install sleeves at penthouse	2 days	Fri 10/30/09	Mon 11/2/09
24	form and pour roof of of penthouse	3 days	Tue 11/3/09	Thu 11/5/09
25	Install metal siding at penthouse	2 wks	Fri 11/6/09	Thu 11/19/09
26	Spray fireproofing at penthouse	1 wk	Fri 11/20/09	Thu 11/26/09
27	Install louvers at penthouse	1 wk	Fri 11/20/09	Thu 11/26/09
28	Install pneumatic tube blower	2 wks	Mon 11/30/09	Fri 12/11/09
29	Assemble AHU's	2 wks	Mon 11/30/09	Fri 12/11/09
30	Install misc equipment	4 wks	Mon 11/30/09	Fri 12/25/09
31	Install misc plumbing equipment	6 wks	Mon 11/30/09	Fri 1/8/10
32	Install elevators and equipment	10 wks	Mon 11/30/09	Fri 2/5/10
33	Install electrical to equipment	12 wks	Mon 11/30/09	Fri 2/19/10
34	Complete smoke control system	2 wks	Mon 12/7/09	Fri 12/18/09
35	Install ductwork	8 wks	Mon 12/7/09	Fri 1/29/10
36	Install piping	8 wks	Mon 12/7/09	Fri 1/29/10
37	Install fire protection	2 wks	Tue 12/29/09	Mon 1/11/10
38	Install controls	6 wks	Wed 1/20/10	Tue 3/2/10
39	Commission penthouse and systems	8 wks	Wed 3/3/10	Tue 4/27/10
40	Install wall frame assemblies 2nd flr West	1 wk	Tue 6/2/09	Mon 6/8/09

ID	Task Mode	Task Name	Duration	Start	Finish
161		Building enclosure complete	0 days	Mon 10/12/09	Mon 10/12/09
162		1st Floor interior buildout	26 wks	Fri 11/6/09	Thu 5/6/10
163		Building commissioning and move-in	2 wks	Wed 5/12/10	Tue 5/25/10
164			0.2 wks	Thu 3/29/12	Thu 3/29/12
165			1 day?	Wed 10/21/09	Wed 10/21/09
166		Order base isolators	6 wks	Thu 5/21/09	Wed 7/1/09
167		Owner relocations of staff and files	2 wks	Wed 5/13/09	Tue 5/26/09
168		Risk Assessment	2 wks	Wed 5/13/09	Tue 5/26/09
169		Relocate records to corridor	1 day	Mon 5/18/09	Mon 5/18/09
170		Construct corridor	2 wks	Thu 5/28/09	Wed 6/10/09
171		Demo existing ER entrance	1 wk	Thu 6/11/09	Wed 6/17/09
172		Excavate land for base isolation system	1 wk	Fri 6/19/09	Thu 6/25/09
173		Install rebar for mat slab	3 days	Thu 6/25/09	Mon 6/29/09
174		Pour mat slab	1 day	Tue 6/30/09	Tue 6/30/09
175		Install base isolation system	6 wks	Tue 7/7/09	Fri 8/14/09
176		Excavate moat	3 days	Tue 7/7/09	Thu 7/9/09
177		Place formwork for flat slab	10 days	Tue 7/21/09	Mon 8/3/09
178		Pour flat slab	4 days	Mon 8/3/09	Thu 8/6/09
179		Finish and cure flat slab	8 days	Fri 8/7/09	Tue 8/18/09
180		Install rebar for walls and columns	2 days	Wed 7/1/09	Thu 7/2/09
181		Form columns and south wall	1 wk	Fri 7/3/09	Thu 7/9/09
182		Pour columns and South wall	1 day	Fri 7/10/09	Fri 7/10/09
183		Form, pour and strip west wall	2 days	Mon 7/13/09	Tue 7/14/09
184		Form and pour west stairway	2 days	Wed 7/15/09	Thu 7/16/09
185		Erect Link structural steel	1 wk	Mon 7/20/09	Fri 7/24/09
186		Pour slabs and enclose link	3 wks	Mon 7/27/09	Fri 8/14/09
187		install roof link	1 wk	Mon 8/17/09	Fri 8/21/09



Project: Project1New
Date: Wed 4/4/12

Task
Split
Milestone
Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
Deadline
Progress

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Construction Breadth Calculations									Crew C-2	48	LH	$2,046.80
			Depth						Crew C-20	64	LH	$3,182.40
Area Outside Moat	21075	sq. ft.	Top Slab	1	ft.	Perimeter	695	ft	Crew C-10	24	LH	$ 958.40
Area Inside Moat	18955	sq. ft.	Iso. Level	7	ft.	Total Wall SFCA	4865	sq. ft				
Total Moat Area	2119	sq. ft.	Found.	3.5	ft.	Total Wall Volume	14595	cubic ft.	B12A	16	LH	$1,623.60
			Total	11.5	ft.				B10B	12	LH	$1,755.80

Total Volume	Moat	24370	cubic ft.	902.5924	cubic yd	1	BCY	27	cubic ft.
	Slab	18955	cubic ft.	702.0526	cubic yd				
	Foundation/ Cavity	217987.3	cubic ft.	8073.605	cubic yd				

Line Number	Description	Unit	Crew	Daily Output	Labor Hours	Bare Material	Bare Labor	Bare Equipment	Bare Total	Total O&P
31113.35	Forms in Place, Elevated Slab									
	Flat plate, job built plywood to 15' high, 1 use	SF	C2	470.00	0.102	4.36	3.79		8.15	10.70
31113.85	Forms in Place, Walls									
	Wall, job built plywood to 8' high, 1 use	SFCA	C2	370.00	0.130	2.70	4.82		7.52	10.48
	Below grade, job built plywood, 1 use	SFCA	C2	225.00	0.213	2.79	7.90		11.69	16.52
33105.35	Normal Weight Structural Concrete									
	5000 psi	CY				109.00			109.00	120.00
33105.70	Placing Concrete									
	Slabs over 10" thick, pumped	CY	C20	180.00	0.356		11.55	4.32	15.87	22.50
	Walls, 12" thick, pumped	CY	C20	110.00	0.582		18.95	7.05	25.00	36.80
33529.30	Finishing Floors									
	Manual screed and bull float	SF	C10	4000.00	0.006		0.21		0.21	0.31
33913.50	Water Curing									
	With burlap, 4 uses assumed, 7.5 oz	CSF	2 Clab	55.00	0.291	7.20	8.80		16.00	21.65
312316.13	Excavating, Trench									
	6' to 10' deep, 1 CY excavator	BCY	B12A	400.00	0.040		1.42	1.62	3.04	0.96
312316.46	Excavating, Bulk, Dozer, Open Site									
	200 H.P., 50' haul, Common Earth	BCY	B10B	1230.00	0.010		0.36	0.87	1.23	1.20
51223.17	Columns, Structural									
	W Shape, A992 Steel, 2 tier, W14 x 82	LF	E2	984.00	0.057	89.50	2.38	1.59	93.47	104.36
	W14 x 370	LF	E2							
	W14 x 500	LF	E2							
	W14 x 550	LF	E2							
	W14 x 730	LF	E2							

	Moat Construction							Slab Construction							Land Excavation						
	Days	# of Crews Needed	Readjusted Days	Labor Hours/Crew	Cost	Labor Cost	Readjusted Cost	Days	# of Crews Needed	Readjusted Days	Labor Hours/Crew	Cost	Labor Cost	Readjusted Cost	Days	# of Crews Needed	Readjusted Days	Labor Hours	Cost	Labor Cost	Readjusted Cost
								40.3	4.0	10.1	483.4	$ 154,486.67	$ 82,445.65	$ 236,932.32							
								0.0				$ 76,523.73		$ 76,523.73							
								3.9	1.0	3.9	249.9	$ 386,751.82	$ 12,427.81	$ 399,179.63							
								4.7	1.0	4.7	113.7	$ 3,980.64	$ 23,001.60	$ 26,982.24							
								3.4	1.0	3.4	55.2	$ 3,032.87		$ 3,032.87							
	2.3	1.0	2.3	36.1	$ 74,084.78	$ 3,663.62	$ 77,748.41														
															6.6	1.0	6.6	80.7	$ 268,124.42	$ 11,813.03	$ 279,937.45
Total	2.3		2.3				$ 77,748.41	52.4		22.2				$ 742,650.79	6.6		6.6		$ 268,124.42		$ 279,937.45

Isolators	Cost	# of Isolators	Total Cost
	$ 22,000.00	50	$ 1,100,000.00
		Total project cost	$ 2,200,336.64