An Analysis of Viscous Dampers in Outrigger Systems Subjected to Wind Loads and Seismic Excitations

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Acknowledgements

This research was made possible by the US Department of Education through the Minority Science and Engineering Improvement Program (MSEIP, Award No. P120A150014) for students who reside near the San Mateo Community College School District. A limited quantity of students were selected and given the opportunity to conduct the following research at San Francisco State University.

We would also like to acknowledge the involvement of graduate mentor, Karlel Cornejo and faculty advisor and well regarded professor, Dr. Cheng Chen for their involvement in this research. Their notability in giving the students direction and advice was integral to even starting this research.

Abstract

The use of powerful computational methods is essential when conducting engineering research. There is a necessity for the most reliable, advanced, and the most intelligible simulations for Civil Engineering and Engineering in its generality.

In this project, the behavior of a 44-story building was analyzed with an outrigger system that uses Buckling Restrained Braces (BRBs), dampers and other bracing methods to reduce structural vibration under earthquake, wind, dead, and live loads.

A heavy amount of this research utilized the software known as SAP2000. SAP2000 was used to assemble the building structure that included modeled BRB's, dampers, composite and steel columns, and other members that were assembled all according to the original prototype building blueprints that were initially given in a project report. With this, a thorough structural analysis was conducted for the remainder of the research period on modeling viscous dampers and further optimizing the prototype structure in order to improve the overall structure, and therefore

improving upon the initial blueprints while also observing structures that prove to be adaptable against earthquakes.

1.0 Introduction

Earthquakes are known to take place in the region classified as "The Ring Of Fire." They are made up of different types of Seismic waves and are caused by a sudden shift below the earth's surface. A minor upheaval inside the earth generates tectonic plate movement and therefore creating from one of the slightest shocks to a high-scale catastrophic temblor. According to the USGS (U.S. Geological Survey) and NEIC (National Earthquake Information Center) around 12,000-14,000 earthquakes are activated, detected, and registered to occur around the world annually. As technology advances, there is an incessant search for analytical tools that offer the most accurate calculations in real world scenarios. Tools like MATLab and SAP2000 offer explicit libraries that help predict a building's structural integrity before it is actually built, while also offering a direct gateway to rendering in AutoCAD.

The quality of a high rise building is determined by its lateral strength and resistance to movement. By testing different types of structural supports on the prototype model of a 44 story building (4 basement floors, 40 floors above ground), an observation of the effect of seismic excitations on the structure can be made. The structure can then be optimized to improve its general resilience against seismic activity as well as other load cases that it may encounter.

2.0 Background

The goal of this project was to design a building with the ability to outperform earthquakes. As skyscrapers continue to grow in height, new systems are needed to sustain seismic impacts. The structure of a building must be optimized in order for it to be more adaptable to not only seismic waves, but to wind loads, dead loads, and live loads. This research project focuses on how a 40-story outrigger system with Buckling Restrained Frames (BRBFs) behaves under earthquakes. Viscous dampers are further explored between outrigger trusses and columns to reduce the structural vibration. Using the structural software SAP2000, students are first instructed to model a 40-story steel structure from the blueprints of an actual design and to analyze the structural performance under selected ground motions. They are then expected to incorporate dampers between outrigger trusses and columns as well as evaluate their efficiency and effectiveness for seismic vibration control. The next goal would be to analyze how various dampers will affect the response of the structure.

2.1 Brief Overview of Seismic Wave Analysis

When the tectonic plates shift, they create seismic waves that affect buildings. In order to understand how seismic waves can affect buildings, it is integral to understand the nature of seismic waves. Earthquakes operate in a similar fashion to that of ripples in water. Their performance is divided into two entities. The initial wave is ordinarily the strongest, where there are weaker but potentially destructive aftershock waves. In every seismic wave, there is a p-wave and s-wave. A p-wave is commonly known as the primary wave, which is the initial wave that permeates from the epicenter of seismic activity. The s-wave is commonly known as a secondary wave or an aftershock. In *Equation 1* and *Equation 2*, there are simplified versions of the p-wave and s-wave equations available in the Notable Equations section. This is how SAP2000 can interpret seismic activity with respect to the ground where structures were created. It is these simulations where frames, joints, and columns can observed and optimized for structures to adapt to earthquakes, wind loads, dead loads, and live loads.

3.0 Methods

The bulk of this research was done by using SAP2000 to build the actual framework of the building. This would include the material types, specifications of the actual frames, outrigger systems, viscous dampers, buckling restrained braces (BRB's), added dead loads, live loads, composite columns, and the input of complex earthquake functions. With all of this information, it is easy to see how most of this work was done with SAP2000.

MATLab was used in an attempt to run multiple simulations at once to change damper capacities and locations to optimize the building, and to have multiple building data displayed on one graph in order to further emphasize the differences between the displacement, frequency, and any reaction forces that each improvement has on the base structure.

Much of the information conveyed from the initial blueprints of 44-story structure, along with many of the calculations of thousands of frames were done using Excel, which was instrumental in organizing large groups of numbers, graphs, and sets of input data that were used in the models made in SAP2000.

AutoCAD was also used to increase the span of visualization on the 44-story structure. Some parts of the structure were hard to look at in a web of frames, numbers, and frame information scattered by SAP2000. For this reason, an AutoCAD model was made for a more aesthetically pleasing visage while also allowing for a higher detailed model.

3.1 Structural Frames

The 44-story building structure is made up of two fundamental frames; the Gravitational Resistive Frame (GRF) and the Lateral Resistive Frame (LRF), see *Figure 1* in the Figures section. The main purpose of the GRF is to provide standing stability of the structure as a whole, which resist loads and prevents deflection along the vertical axis. This frame was tested against dead and live loads. The main purpose of the LRF is to resist loads and prevents deflection in the horizontal direction.This frame was tested against nonlinear modal history earthquakes and linear static wind loads.

3.1.1 Outrigger Systems

An outrigger system is an exterior structure that connects to the outside walls of a high rise structure. Outriggers support the balance of the building against lateral load cases. Along with the system comes the outrigger truss. These truss assemblies are placed outward of the structure. These trusses form a shape of a cantilever in the edifice. *Figure 2* in the Figures section illustrates the cantilever shape mentioned previously.

Outrigger systems are primarily used to resist wind loads. Despite this fact, outriggers in this project were engineered to withstand major seismic loading case.

Although SAP2000 does not cause the building to fall to the ground in the program; however, this software did not run the analysis when the connections from the inner core were effectively transferred from the building to the outrigger system.

3.1.2 Buckling Restrained Brace (BRBs)

In order for a frame to refrain from deflecting a destructive amount, Buckling Restrained Braces can be used to combat compressive forces. These are composite members normally made out of a steel member enclosed in a cement surrounding with an exterior shell of steel. This utilizes the steel's ability to resist tensile forces and attain the ability to resist buckling under compressive axial forces with the given support by the outer shells. The properties of the BRB's can be described by the formulas shown in *Equation 3* located in the Notable Equations section. Conversely, with these two properties, the inner diagonal member is prevented from buckling in the event of being set under compression or tension. BRBs are classified by their carrying capacity and are found in the Lateral Resisting Frame of the building as seen in *Figure 2* located in the Figures section.

3.1.3 Composite Columns

Another member that is used to stabilize and prevent deflection in a given structure is a Composite column. These columns, as illustrated in the Figures section, Figure 3 are made up of two materials; an interior center of cement and an exterior shell of steel. These were assigned in the Lateral Resisting Frame and varied from floor to floor. These member-columns were classified by their given size that was modeled in SAP2000.

3.1.4 Viscous Dampers

In *Figure 4*, the entire construction of a viscous damper can be examined. Compressible hydraulic fluid is kept inside the cylinder, which is usually a silicon oil. When the applied structure reaches some threshold velocity which in turn yields a displacement, the viscous damper counteracts this movement and dampens that velocity, effectively "damping" the movement of the building. With this, the piston of the damper has liquid moving from a larger area to a smaller area, which is the source of the dissipation of energy.

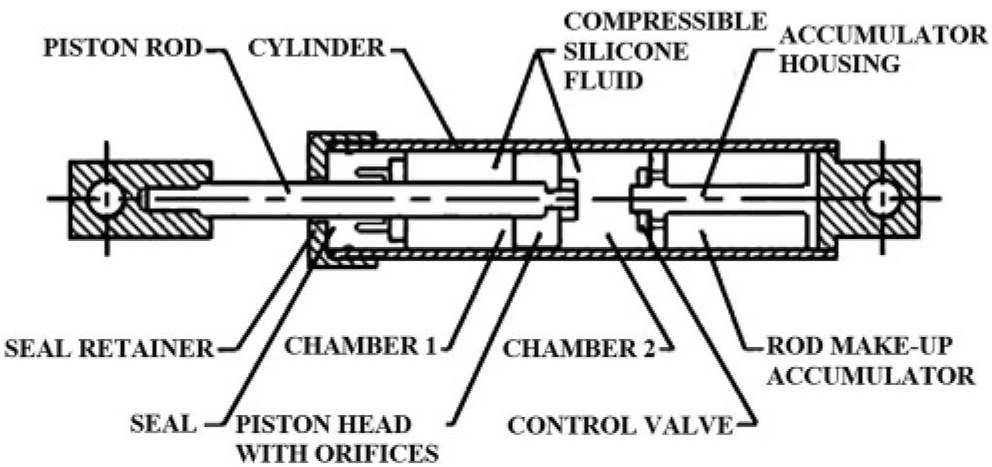


Figure 4 - Model of a Viscous Damper

Even though viscous dampers are usually purposed for resisting shear, here they are used in this project to resist wind loads and the effects of seismic excitation.

3.1.5 Building Standards

In order to see the amount that a differing Gravitational Resisting Frame would affect the building structure's performance under lateral loads, two building structures were assembled having followed different standards. The two standards that were used in the Gravitational Resistance Frame designs were the Allowable Stress Design (ASD) and the Live & Resistive Factor Design (LRFD), set by the American Society of Civil Engineers (ASCE). These are two different philosophies that take in different considerations and variables that decide the weight, size and material of a given member. As discussed in *Section 3.3*, all the horizontal members throughout the structure were steel W16*100 wide flange beams. The two designs ultimately differed in the vertical columns, varying in floor level, in the gravitational resistive frame. In the case of this structure, ASD tended to use slightly heavier and larger wide-flange columns whereas LRFD tended to use lower amount of material.

After modeling the the two structures under the ASD and LRFD standards, they were then subjected to the Loma Prieta earthquake: their maximum deflections were attained through linear analysis. It was found that the standard followed for the gravitational resistance frame did not affect the structure's performance under lateral loads. This was as expected, as the Lateral Resistance Frame was identical in the two structure-designs. The difference between the ASD structure and LRFD structure in maximum-deflection was found to be less than five-hundredths of a foot. In continuing the research, one of the two standards-followed was chosen as the primary structure design; the LRFD standard. This standard was chosen since it is currently the common practice in the field; it also came out to be a more cost-effective design since it uses less material and lighter members.

The structure with applied the LRFD standards was then tested against area wind loads as well as wind loads applied at the center of diaphragms as well as scaled versions of the Loma Prieta earthquake. There were multiple configurations of the 44-story building, which can be found in Table 1.

Table 1 - Building Models

Building Models	
Model	**Description of Model**
ASD Structure	Followed the Allowable Stress Design standards set by the ASCE; used heavier and larger wide-flange columns in the gravity resisting frame.
LRFD Structure	Followed the Live & Resistive Factor Design standards set by the ASCE; used lighter and smaller wide-flange columns in the gravity resisting frame.
LRFD - Hinge BRBs	BRBs throughout the structure were modeled as hinges, used for pushover analysis.
LRFD - Link BRBs	BRBs throughout the structure were modeled as links, a multi

	linear plastic, used for nonlinear analysis.
LRFD - No BRB’s	Followed the Live & Resistive Factor Design standards set by the ASCE; contained no BRBs in the structure.
Phase 2 Alpha	Dampers were oriented by mirroring all of the BRBs on the outrigger trusses about the horizontal except for the bottom most outrigger truss, where a cross pattern was utilized instead.
Phase 2 Beta	Dampers were oriented by mirroring all of the BRBs on the outrigger trusses about the horizontal.

3.2 Simple Two-Story Model

At the beginning of the research period, students were introduced to the Structural Analysis Program known as SAP2000. In the SAP2000 website, there are many exercises that could be taken and completed in order for the user to become more familiar with the software in terms of materials, loads, structures, frames, and full scale high rise buildings. Some of the two story exercises that were made to simplify the goal of what this research is trying to achieve. In *Figure 7*, note that this simple 2D structure is made up of individual frames without any material specification. It is only two stories, and also has an xy viewing plane to analyze the building floor by floor. Exercises like this increased in difficulty and complexity during the first week of research.

3.3 Construction of the 44-Story Structure in SAP2000

After attempting a simple two-story model, a 44-story building with 4 basements and 40 floors on top was constructed. In SAP2000, this can be done by choosing a 3D frame, and specifying the number of stories, and the number of bays in the x and y directions. The building follows the floor plan shown in *Figure 5*, and has the dimension of 170ft in width and 107 feet in length. On the 20th, 30th, and 40th floors, there are outrigger trusses connecting the inner core and outer gravity resisting frames as shown in *Figure 6* in the Figures section below. For the vertical dimensions, the four basement floors are 12 feet high. The ground floor or the 1st floor is 18 feet high. From the 2nd to 40th floors, the height is 13.5 feet. Assigning the exact dimensions of the structure can be achieved by defining the width and lengths of the bays in SAP2000.

Moreover, the building was designed with complex specifications of frame sections on different columns. For all the horizontal beams, the sections W16x100 were used. For the columns, the LRFD standard was used because it used less material when compared to the ASD format. The LRFD structural format is made up of varying sections ranging from W14x283 to W14x730.

3.4 Applied Loads

Both dead and live loads were added to the building in accordance with LRFD standards given by the American Society of Civil Engineers (ASCE) . The specific dead and live load distribution per floor can be seen in *Table 2*.

There were also wind loads added to the resultant structures as well. Multiple variants of wind loads being had been introduced from different angles, directions, and even with different magnitudes. The most probable wind loads were kept running while the improbable wind loads were switched not to run in SAP2000. More details on each load will be covered in the next series of sections.

Table 2 - Dead and Live Loads Dispersion

Floor	Dead Load (psf)	Live Load (psf)
Roof	79.5	25
33-39	79.5	40
28-32	79.5	40
24-27	79.5	40
20-23	79.5	40
15-19	79.5	40
10-14	79.5	55
5-9	79.5	40
1st	79.5	40

Ground (Core)	220	100
Ground (Plaza)	480	100
Basement	54.5	40

3.4.1 Dead Loads

Dead loads are any loads that are not expected to move. For instance, walls and floors. As indicated in *Table 2*, the dead loads on the ground floor are the greatest due to the heavy planters. On the other hand, the dead loads on the basements are the smallest because the building is already embedded in the ground that it does not need heavy materials to keep the structures in place.

3.4.2 Live Loads

Live loads are the loads that can be moved, such as people, chairs, and tables. Similar to dead loads, live loads are the heaviest on the ground floor as it is expected to have more people and furniture in the lobby. Both dead and live loads are assigned corresponding loads following in the standards set by ASCE as shown in Table *2*. References to the specifics of ASCE Dead and Live Load distribution can be seen in the Works Cited section.

3.4.3 Wind Loads

The lateral resistance of the structures were tested by applying wind loads. According to the American Society of Civil Engineers (ASCE 7-16), the ordinances last modified in 2017, there are different parameters to take into consideration when observing the behavior of a tall structure against high rise air currents. Pictured in *Figure 8* there is user interface cue for the wind load case modifiers used by the ASCE 7-16.

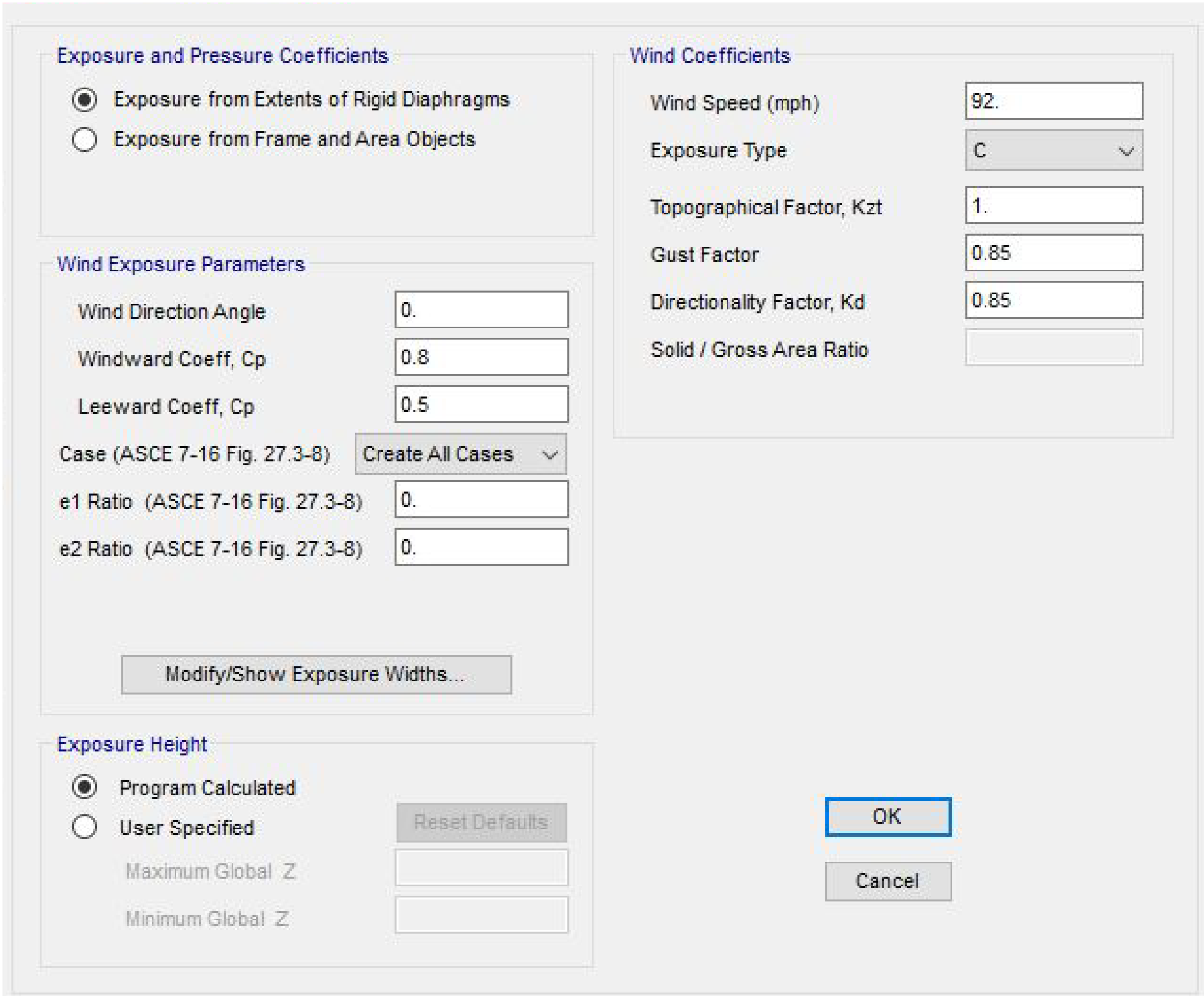
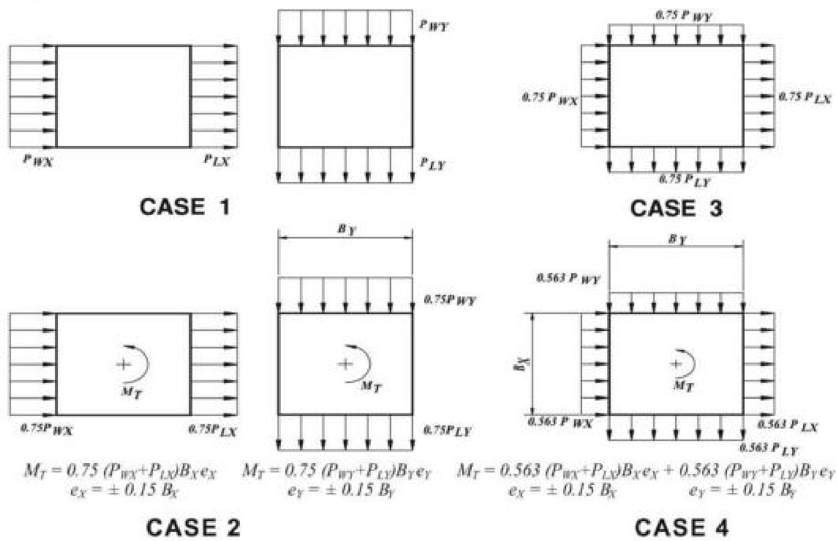


Figure 8 - Image taken from the interface of SAP2000 where a list of the wind coefficients are prompted in the top right corner.

In accordance to the wind coefficients, the user also has the option to set the wind exposure parameters individually. After entering the wind coefficients, SAP2000 creates 12 different load cases that affect the distinct directions of the wind patterns. These 12 wind patterns are described by the *Figure 9* in four diverse cases.

Diagrams



Notation

P_{WX}, P_{WY} = Windward face design pressure acting in the *x*, *y* principal axis, respectively.
P_{LX}, P_{LY} = Leeward face design pressure acting in the *x*, *y* principal axis, respectively.
$e(e_X, e_Y)$ = Eccentricity for the *x*, *y* principal axis of the structure, respectively.
M_T = Torsional moment per unit height acting about a vertical axis of the building.

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.
Case 2. Three-quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.
Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of Sections 27.3.1 and 27.3.2 as applicable for buildings of all heights.
2. Diagrams show plan views of buildings.

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

Figure 9: Diagrams of wind directionality given the cases and the e-ratios from the ASCE 7-16 Standards.

Structures are also labeled in accordance to the risk factors an edifice is able to present in the case of its failure. Wind risk categories are determined by the ASCE 7-16 standards and are due to the operations of the structure. These risk categories are available to be viewed in Table 3a and Table 4.

Table 3: Simplified Risk Categories

<u>Building Risk Category:</u>

I. Negligible Risk, ie. where people are not expected to be
II. All buildings and structures which do not belong elsewhere in the categories
III. Buildings with:
    A. A large amount of occupancy
    B. Limited escape abilities
    C. Utilities
    D. Hazardous substances
    E. Threat of mass disruption to everyday civilian life

Table 4: Risk Category Section in ASCE 7-16

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure.	I
All buildings and other structures except those listed in Risk Categories I, III, and IV.	II
Buildings and other structures, the failure of which could pose a substantial risk to human life.	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.[a]	
Buildings and other structures designated as essential facilities.	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community.	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

When conducting simulations, wind speeds are required to be higher than the average wind speeds experienced in the future area, address, or location of the desired structure. On *Figure 10*, the risk categories are listed with their recommended wind speeds. The original 44-story model has a risk category of II. The gust-effect factor is a ratio of the maximum to the mean response of a structure, and is always permitted to be of the scalar value of 0.85 if the structure is a rigid element according to the latest ASCE standards.

Figure 10: Recommended wind speeds with according to the ATC Hazards by Location

The exposure type of a building consists of its type of location and the building's physical structure. There are distinct exposures ranging from exposure type B to exposure type D. Height is a very important factor as well when dealing with high winds. Table 5 gives information on exposure types and their respective requirements.

Table 5: Exposure types and their descriptions

Exposure Type B	• Buildings with mean roof height < 30 ft • Surrounded by buildings with mean roof height < 30 ft
Exposure Type C	• Exposure B and D do not apply
Exposure Type D	• Flat areas • Unobstructed areas • Water Surfaces All the above should be within 600 ft of radius or the product of the scalar 20 and structure height.

Topographic effects are commonly known as large areas where there is a sudden relief or stress in the normal direction. The topographic factor is authorized to be a value of 1 for any building, large object, or structure not containing speed-up wind reducing or inducing areas.

Distinct building designs have been created in the past decades. These buildings merge with natural events in an alternative approach. The building's shapes play an essential role to how the wind will infract the structure. There are dome shaped structures and freely shaped modern buildings far from the norm in a city's framework. In these cases, wind directions and speeds have to be viewed differently. The directionality factor, Kd, could be best found by using the structure's topography as seen in Table 6.

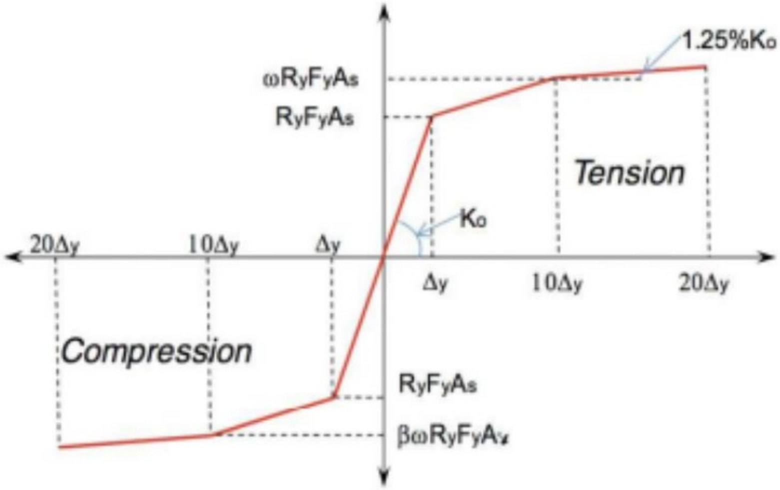
Table 6 Wind Directionality Factor from ASCE 7-16

Structure Type	Directionality Factor K_d
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Circular Domes	1.0[a]
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Octagonal	1.0[a]
Round	1.0[a]
Solid Freestanding Walls, Roof Top Equipment, and Solid Freestanding and Attached Signs	0.85
Open Signs and Single-Plane Open Frames	0.85
Trussed Towers	
Triangular, square, or rectangular	0.85
All other cross sections	0.95

[a]Directionality factor $K_d = 0.95$ shall be permitted for round or octagonal structures with nonaxisymmetric structural systems.

3.4.3.1 Wind Loads using Hinges

SAP2000 and its perception of Buckling Restrained Braces previously discussed in Section 3.1.2, is described by an extensive array of approaches when conducting simulations of the models under any lateral displacement or load. Because of the behavior of SAP2000, it was possible to take two different means when developing these models for the BRBs on the building. In these tests, there was no use of any dampers in any of the modeled structures. The method of using hinges to describe the behavior of BRBs was used in one of the first models. To model these hinges, the scaling force for the diagonal members had to be changed in SAP2000 in order to obtain a backbone curve that is similar to that shown in *Figure 11*. This scaling force is also known as the product of the "Fy" and the "As" values depicted in the same *Figure 11*.



General backbone curve for the nonlinear BRB element. The vertical axis represents force and the horizontal axis represents deformation. A_s **= area of yielding steel core,** K_o **=** A_sE/L**,** E **= 29,000ksi,** F_y**=38ksi,** R_y **= 1.1,** ω **= 1.25,** β **= 1.1, and** L **=70% of the brace length (using center-line to center-line geometry). Image courtesy of Dutta and Hamburger [2010].**

Figure 11 - Image of backbone curve of Buckling Restrained Braces with a brief description of its values taken from the original blueprints of the building.

The model using hinges was set to react in the U2 direction to better test BRB frames. The displacements of the model using BRBs as hinges is shown in *Table 7*.

3.4.3.2 Wind Loads using Links

BRBs modeled by links were used to compare those modeled by the use of hinges, both under multiple cases of linear static wind loads. These links were modeled on SAP2000 as a support type: Multi Linear Plastic. Its directional properties were constrained about the “U1 - direction”, which is the longitudinal axis and were set to non linear.

12 load patterns and cases were run in SAP2000 with differing wind exposure parameters and wind coefficients. The exposure parameters determined how the wind would interact and affect the building whereas the wind coefficients determined the characteristics of the wind. For all of the cases, the windward coefficients were set to 0.8, the leeward coefficients were set to 0.5 and the wind speed was set to 92 miles per hour. Wind loads were modeled by assigning an individual diaphragm to each floor and applying the wind loads to each diaphragm. The maximum and minimum deflection in the U2-direction (the y-direction), was recorded for each of the 12 load cases applied to the three structures with No BRBs, Link-BRBs, and Hinge-BRBs.

As the directional properties for hinges were set to linear and the load case type for wind was linear static, a decrease in deflection was expected for the structure with the Hinge BRBs compared to that of the structure with no BRBs.

As shown in *Table 7*, the max deflections of the structure with link-BRBs came out to have the same amount of displacement as the one with no BRBs. This could be due to SAP2000 having a possible inability to apply a linear static force to a nonlinear element. The structure with the hinge-BRBs had a significant decrease in deflections compared to that of the structure with no BRBs. The structure with no BRBs had a maximum deflection of about 79.5 inches in the positive U2 direction whereas the structure with hinge-BRBs had a max deflection of about 12.9 inches in the positive U2 direction. It is important to restate that these structures were designed to resist deflection in the y-direction. The cases tested varied in wind direction angle and torsion on the structure caused by the wind. The most effective way to model BRBs against wind loads is proven to be the method using hinges.

Table 7 - Load Case Wind Deflections for 3 Structures

	NO BRBs		**LINK-BRBs**		**HINGE-BRBs**	
LOAD CASE:	**U2 max: (in)**	**U2 min: (in)**	**U2 max: (in)**	**U2 min: (in)**	**U2 max: (in)**	**U2 min: (in)**
Wind	9.42E-07	-0.001018	9.42E-07	-0.001018	4E-10	-3E-10
Wind-2	79.482002	-0.136359	79.48195	-0.136359	12.872815	0
Wind-3	0.054393	-0.055278	0.054393	-0.055278	0.827168	-0.825784
Wind-4	0.053751	-0.055874	0.053751	-0.055874	0.825784	-0.827168
Wind-5	59.750672	-0.102709	59.750633	-0.102709	11.742579	0
Wind-6	59.743974	-0.102698	59.743934	-0.102698	11.727164	0
Wind-7	0.10227	-59.612242	0.10227	-59.612203	0	-9.654611
Wind-8	59.610761	-0.102268	59.610722	-0.102268	9.654611	0
Wind-9	0.077223	-44.889305	0.077223	-44.889275	0	-9.42308
Wind-10	0.077231	-44.894781	0.077231	-44.894751	0	-9.43569
Wind-11	44.893669	-0.07723	44.89364	-0.07723	9.43569	0
Wind-12	44.888159	-0.077222	44.888129	-0.077222	9.42308	0

3.4.4 Loma Prieta Ground Motion

The durability of the 40-story building was tested using the acceleration vs. time history data of Loma Prieta ground motion retrieved from *Center for Engineering Strong Motion* database.

Loma Prieta hit the Bay Area back in 1989 with a magnitude of 6.9 in the Richter scale , and is used as a standard major earthquake for both linear and non-linear analysis throughout the project. The Loma Prieta ground motion was used to simulate seismic activity taking place in the San Francisco Bay Area. The structure was engineered to adapt to magnitudes that match that of the Loma Prieta Earthquake.

4.0 Testing Processes

For the sake of thorough research, the effectiveness of the outriggers, buckling restrained braces, and damper placement were tested in the structure. The improvements to the structure was the difference of the maximum displacement yielded with the building subjected to the Loma Prieta Earthquake, which was plotted using MATLab as seen in *Figure 12*.

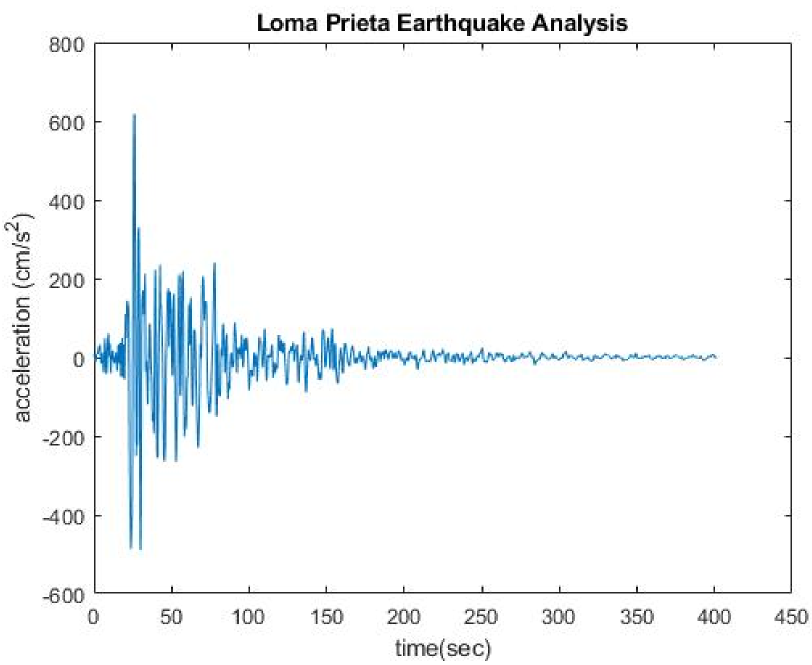


Figure 12 - Loma Prieta Earthquake interpreted by MATLab

After general construction of the building was completed, optimization using dampers was divided into four different subphases. Phase 1 involved creating several buildings all with the same loads and materials with the only differentiating factor being the damper configuration within the structure. Whichever damper structure proved to be the most effective would move on to Phase 2. This Phase 2 involved changing the coefficients of the prevailing structure in order to select the best possible structure subjected to the same ground motion.

Phase 2 was divided into two components Phase 2 Alpha, and Phase 2 Beta. Both of these configurations were two different structures that proved to be the most effective in terms of passing the tests of Phase 1. Phase 2 Alpha proved to be the most effective overall (as seen in *Figure 13*). It was decided to include the results of Phase 2 Beta (as seen in *Figure 14*) because it was almost as effective as Phase 2 Alpha but used less dampers, which would lower the cost of construction while still upholding a high degree of structural integrity. In the following sections, specific details of the testing processes and structures will be explained.

4.1 Phase 1

A total of 7 different damper configurations were tested during the process of Phase 1. The main basis of comparison between these structures would be their maximum displacement in inches when subjected to the Loma Prieta Earthquake. Originally, the main goal of Phase 1 was to select the single best damper configuration. The results shown in *Figure 15* shows that C2 was the best structure overall. C2 was later referred to as Phase 2 Alpha. Looking at the graph, the results of structure C1 were not very different in value from structure C2. The structure C1 used less material to make while yielding a high enough degree of structural integrity. This structure was included in experimentation Phase 2.

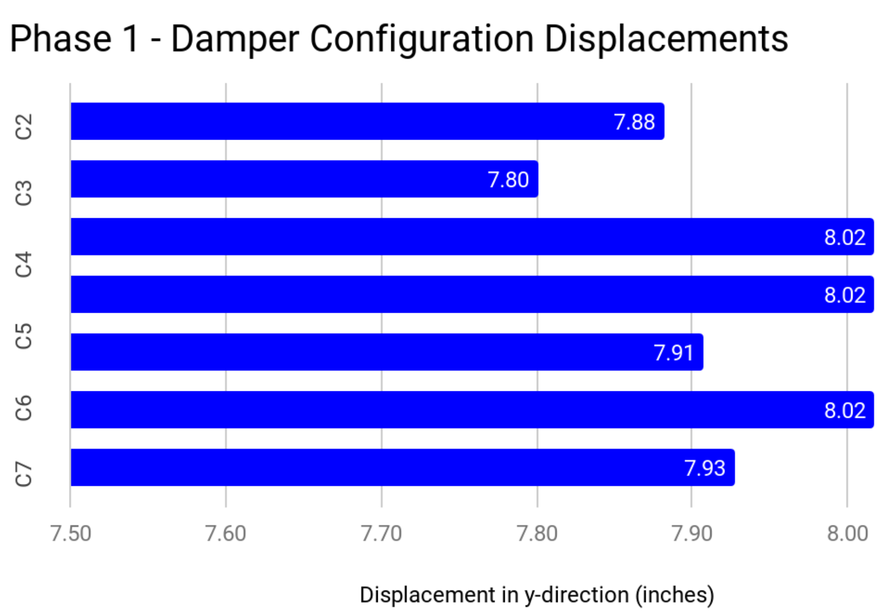


Figure 15 - Phase 1 Summary of Displacement of Differing Damper Configuration where C stands for configuration.

4.2.1 Phase 2 Alpha

This building configuration is made of of the general LRFD structural format, but uses 40 dampers throughout the entire structure. The total deflection for Phase 2 Alpha is about 7.57 inches in the y-direction when subjected to the Loma Prieta Earthquake ground motion. There was a focus on the deflection in the y-direction would be to directly test the effectiveness of both the outrigger trusses and dampers, as they were built adjacent with this direction in SAP2000. The original displacement of the constructed building without dampers was 8.02. Given this, "Building Alpha" improved the displacement of the structure by about 5.6% when compared to the structure without dampers.

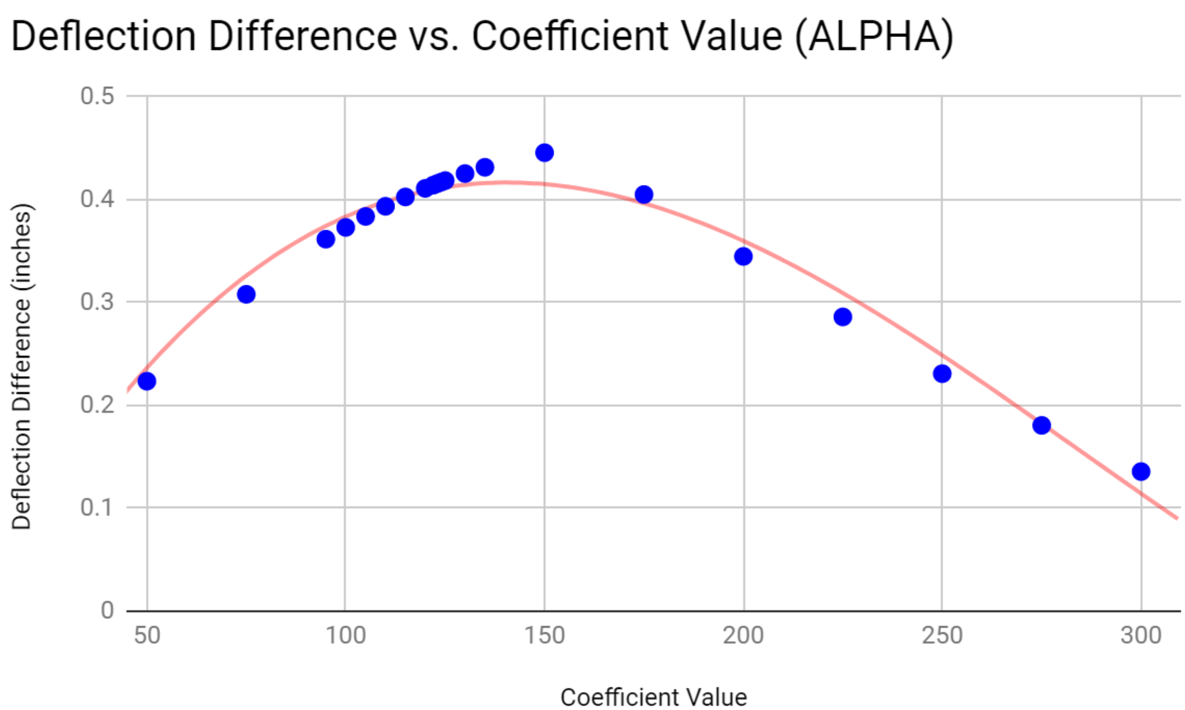


Figure 13 - Phase 2 Alpha Coefficient Analysis with respect to Displacement in inches

4.2.2 Phase 2 Beta

This building configuration also used the general LRFD structural format, as the entire frame used less material while still being structurally effective. There were only 32 dampers in use for the entirety of Phase 2 Beta but deflected about 0.17 inches more than Phase 2 Alpha. The exact total deflection for Phase 2 Beta was about 7.74 inches in the y-direction when subjected to the Loma Prieta Earthquake ground motion. In a real engineering application, it would be more ideal to use a model that is similar to Phase 2 Beta since the difference is minimal when compared to Phase 2 Alpha, despite being a small margin more effective. Keeping the original displacement without dampers in mind, the marginal percentage that Phase 2 Beta has improved upon the structure without dampers would be about 3.4% improved when compared to the structure without dampers.

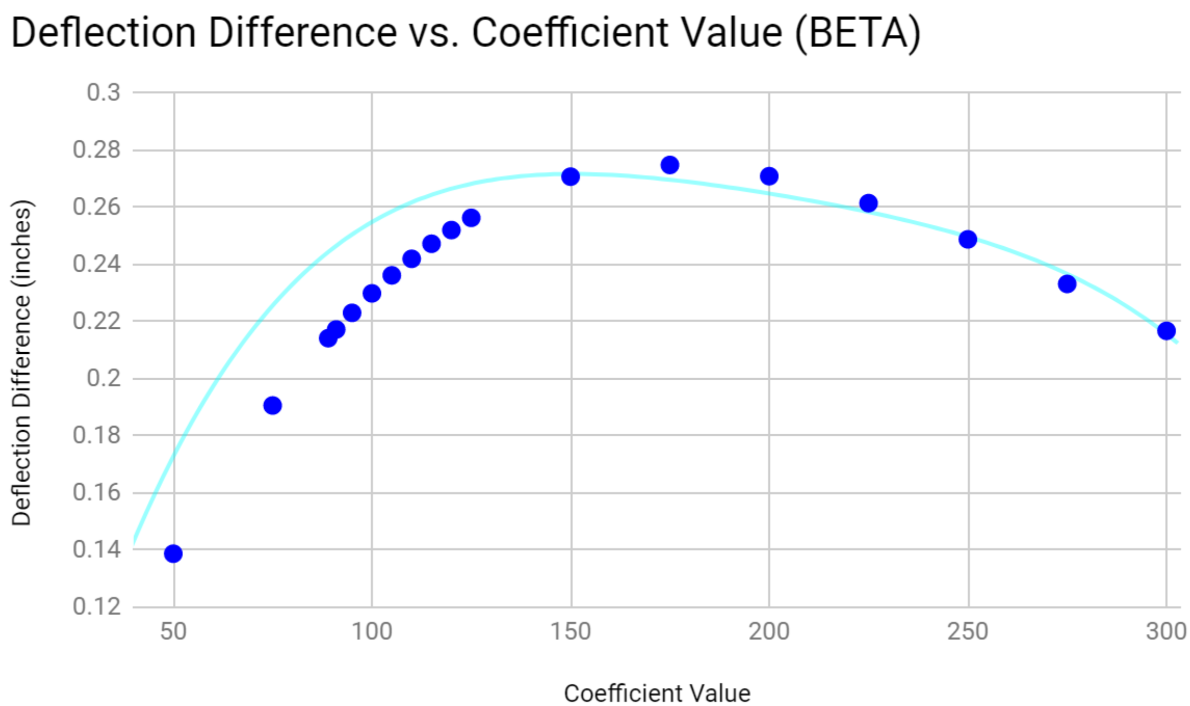


Figure 14 - Phase 2 Beta Coefficient Analysis with respect to Displacement in inches

4.3 Analysis of Testing Processes

Having two buildings to compare was optional with the project as the main goal was to fully optimize one structure. There were two optimized structures with Phase 2 Alpha being the most effective. Phase 2 Beta is almost as effective as Alpha, yet uses significantly less material. A relationship can be observed between the coefficient of a damper in a building and the overall stiffness in the structural material makeup. The more stiffness in a structure, the less of a coefficient the dampers would have as the stiffness of the base material would absorb more shock. The building stiffness was then set to 2000 psi for a moderate value. Both structures would be slightly ductile in terms of building standards, and more stress would be dissipated in the outriggers, dampers, and buckling restrained braces that were scattered across both resultant models at strategic points. Having a higher degree of stiffness would make the building stronger, but would subsequently increase the chances of crack propagation in areas where cyclic loading is high, especially when repetitively subjected to dead loads, live loads, wind loads, and all degrees of seismic activity. For the moderate applied stiffness and high damper coefficients, there was an optimization of the deflection in the original prototype structure when subjected to earthquakes. For Phase 2 Alpha, the deflection decreased by 28.52%, making increasing productivity by over a quarter percentile. As for Phase 2 Beta, the deflection decreased by about

26.87% given the decrease in material use but just as nearly as effective as the aforementioned configuration.

4.4 Hysteresis Analysis Graphing

The purpose of having the hysteresis analysis to analyze the two resultant structures would be to observe the energy dissipation with respect to the deformation when subjected to a seismic load seen in the nonlinear time-history load cases that describe the Loma Prieta Earthquake in a function format. A hysteresis analysis was done on both Phase 2 Alpha and Beta, both subjected to the Loma Prieta ground motion, analyzing the same damper and buckling restrained braces in both models. To see how the force would change with respect to displacement, the Loma Prieta Earthquake was scaled up by a factor of three in both Phase 2 Alpha and Beta structural models. The results of such experimentation can be seen in the displayed in *Figures 16-19*.

4.4.2 Hysteresis Analysis on Phase 2 Alpha

Focusing on *Figure 16*, the first test was to see the displacement of Link 1097 (BRB of focus in Phase 2 Alpha). The force going through this particular frame did not yield a large amount of vibration, which makes sense when the stiffness, stress capacity, and restraining nature of a BRB is taken into account. There is no indication of failure based on the graph when buildings are subjected to the Loma Prieta Earthquake. There is only elastic deformation in this frame so it is not necessarily being affected by the Loma Prieta Earthquake alone. As for Link 1092 (Damper in focus in Phase 2 Alpha), there is plenty of vibration when analyzing the force with respect to displacement, but this really just shows that force is being absorbed by the dampers and effectively dissipated, which is expected for a viscous damper.

Looking at *Figure 17*, when the Loma Prieta Earthquake was scaled up by a factor of three, Link 1097 reached it's tensile strength and entered three cycles of elastic deformation and plastic deformation. The main reason why it was decided to increase the Loma Prieta Earthquake by a scale factor of three was because it was necessary to use these results as a basis of comparison with the regular Loma Prieta Earthquake and to better observe the structural behavior of the buckling restrained braces. Because a BRB has reached a level of plastic deformation, it is quite possible that crack propagation or even failure is possible to occur within this simulation. Because Link 1097 is a buckling restrained brace, it won't be inclined to deform as much as the dampers, which has a significant increase in deformation when compared to the Phase 2 Alpha damper case that was previously explained. When observing Link 1092, it behaves like any damper would under a high magnitude of seismic activity, absorbing a high amount of force and deforming heavily under plastic deformation. With the Loma Prieta Earthquake being multiplied by 3 in SAP2000, the dampers act as expected, absorbing a high amount of force. But the buckling restrained braces have clearly reached there limit here. An overestimate in engineering is needed and done so effectively.

For Phase 2 Alpha under the regular Loma Prieta Earthquake, the BRB's doesn't experience plastic deformation until when the earthquake is increased by a factor of 3. Again, it is very possible that cracks could propagate when a concrete steel composite BRB endures plastic deformation, which may easily lead to failure. This did not occur during the regular Loma Prieta Earthquake, since such activity is expected for a damper. The damper dissipates kinetic energy in both seismic cases and are designed to be in the presence of such movement.

4.4.3 Hysteresis Analysis on Phase 2 Beta

Focusing on *Figure 18*, the initial test focused on observing the displacement of Link 1079 (BRB of Focus in Phase 2 Beta). Like the test results as seen in Phase 2 Alpha, the buckling restrained brace only experienced elastic deformation. There is a low chance that cracks would propagate within the structure. As for Link 1108 (Damper pf Focus in Phase 2 Beta), there was less vibration coursing through the dampers when compared to Phase 2 Alpha. This can be explained by the actual amount of kinetic energy being absorbed by the dampers. More kinetic energy was being absorbed in Phase 2 Alpha, yielding less displacement in the structure overall. A larger margin of displacement could be seen in Phase 2 Beta because there is less force being absorbed with respect to the displacement in Link 1108.

When the Loma Prieta was scaled up by a factor of three, the results of both Link 1108 and Link 1079 in Phase 2 Beta became strikingly similar to that of the results of Phase 2 Alpha when the seismic activity was scaled up. These similarities can be seen when comparing *Figure 19* with *Figure 19*. Differences in the force with respect to displacement seemed to decrease when the seismic activity was intensified, and the differences in damper structure did not seem to matter. Differences in the plastic deformation and the possible crack deformation that came along with the buckling restrained braces were minimal. The increased damper vibration graphs in both Phase 2 Alpha and Beta seem to act in a similar fashion, reacting the same way to the vibrations sourcing from seismic activity.

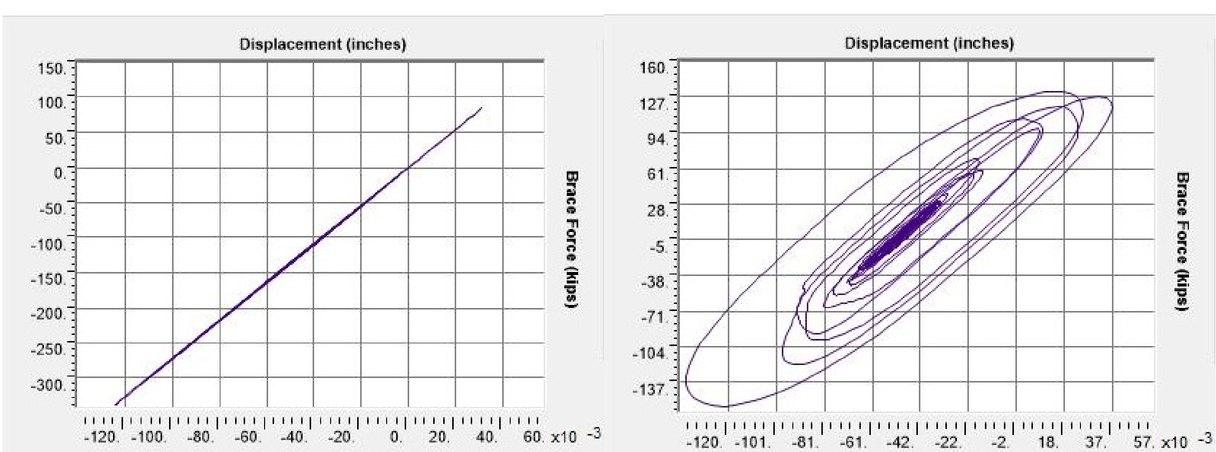


Figure 16 - Phase 2 Alpha Hysteresis Results subjected to Loma Prieta Earthquake

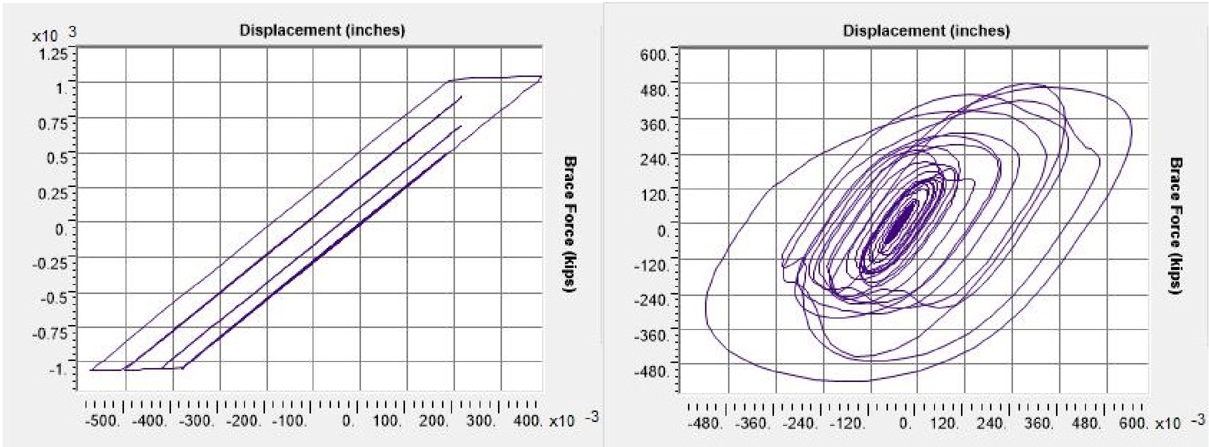


Figure 17 - Phase 2 Alpha Hysteresis with Loma Prieta scaled by factor of 3

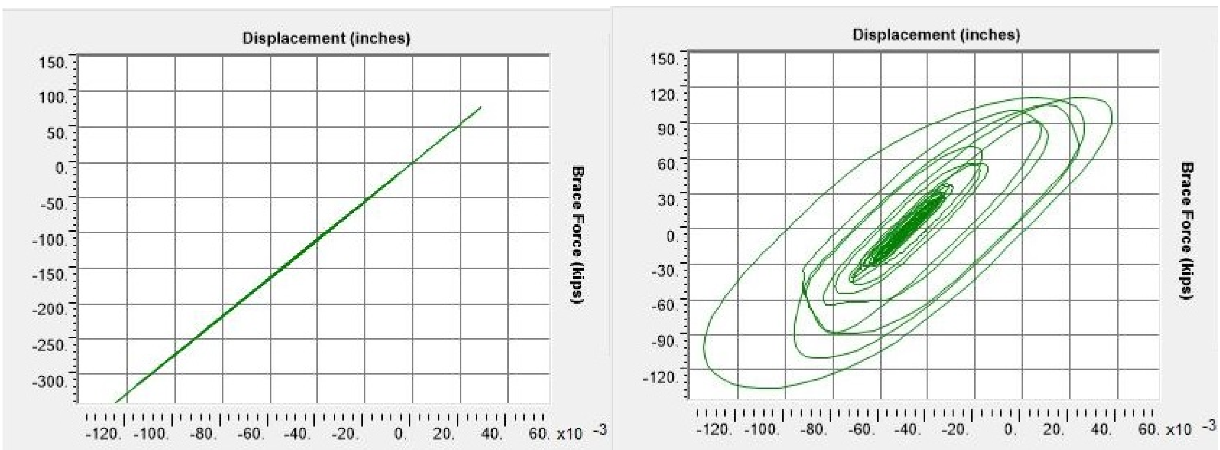


Figure 18 - Phase 2 Beta Hysteresis Results subjected to Loma Prieta Earthquake

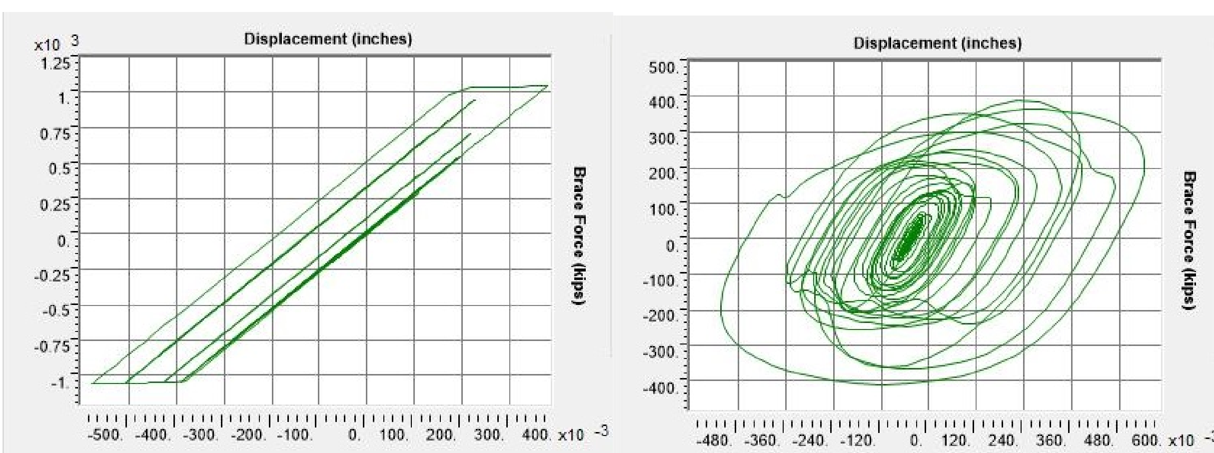


Figure 19 - Phase 2 Beta Hysteresis with Loma Prieta scaled by factor of 3

5.0 Project Summary / Conclusion

At the beginning of research, there was the prototype skeleton model that included the specific frames, wide flange sections, and composite column sections. Two different structural formats were handled when trying to see which structural configuration was the most outstanding when building the prototype skeleton: ASD and LRFD. In terms of effectiveness, LRFD and ASD formats did not differ so much in displacement, but in terms of material consumption, the LRFD format proved to be the most effective. With the LRFD skeleton specifications alone, there was a deflection of about 10.59 inches in the y-direction.

By optimizing the structure of the building with the addition of outriggers and buckling restrained braces, the deflection was reduced to about 8.02 inches. With this, Phase 1 and Phase 2 of structural optimization came into play, where researchers used seven different computers to run seven different damper configurations, and then proceeded to optimize the stiffness coefficients of the dampers.

Phase 2 was then divided into Phase 2 Alpha and Phase 2 Beta, in which Alpha was more effective whereas Beta was almost as effective but used less materials than Alpha. Phase 2 Alpha had a displacement of 7.57 inches whereas Phase 2 Beta had a displacement of 7.74 inches. The building skeleton, building without dampers, and both configurations with dampers were all subjected to the Loma Prieta Earthquake ground motion acceleration.

After experimentation and optimization, Phase 2 Alpha and Beta were subjected to wind load tests and hysteresis tests. The wind load tests were not done with the dampers applied to both buildings, as they were interfering with the wind during simulations and would therefore not run. During the hysteresis tests, there was a direct correlation with the stiffness coefficient and the amount of force that dampers would be able to take with respect to displacement.

With this observation being said, further possible improvements to this research would be to look into damper structures with lower or higher coefficients and to test the effectiveness of each structure with respect to those already made in this project.

Figures

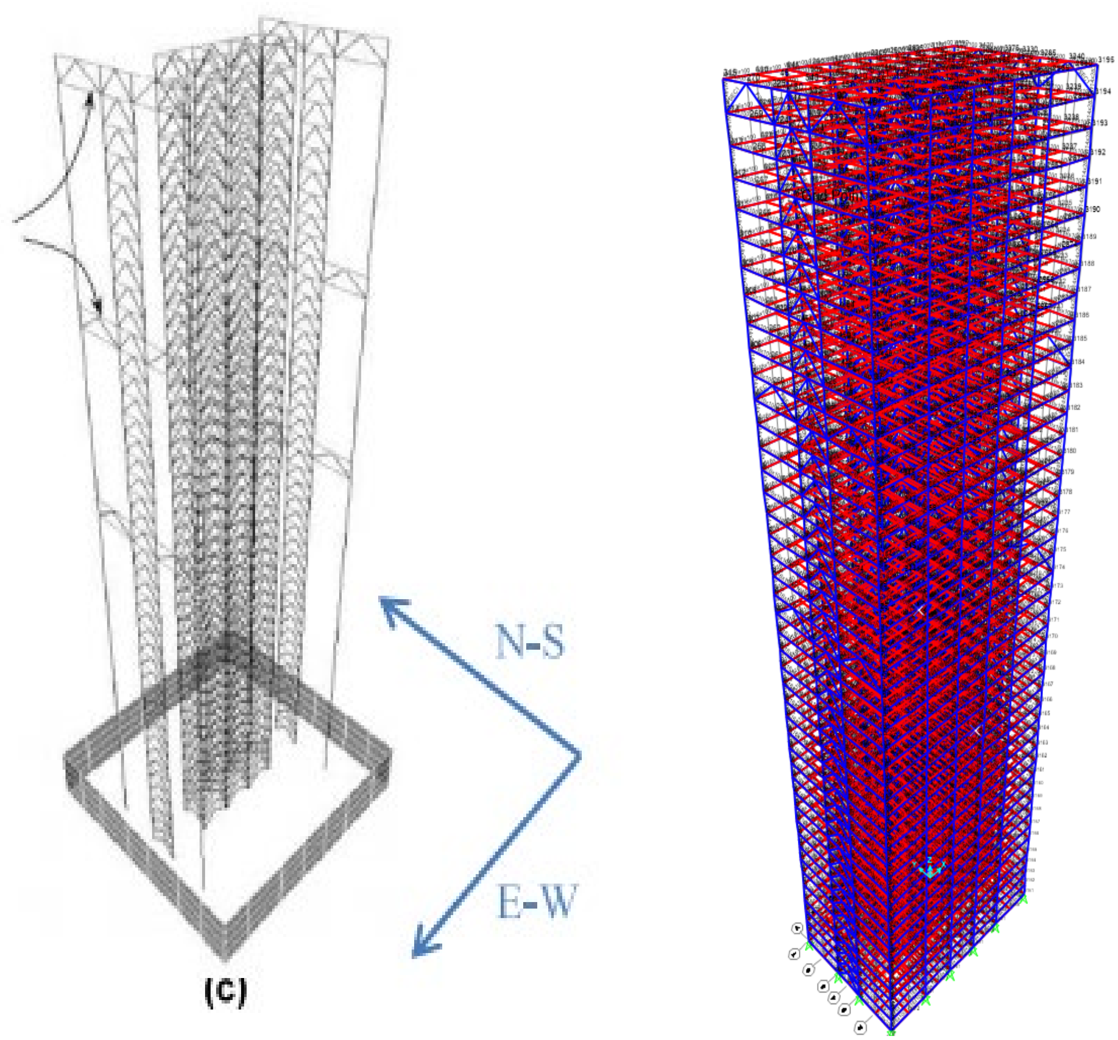


Figure 1 - Comparison of Skeleton Prototype Frame to SAP2000 Structure

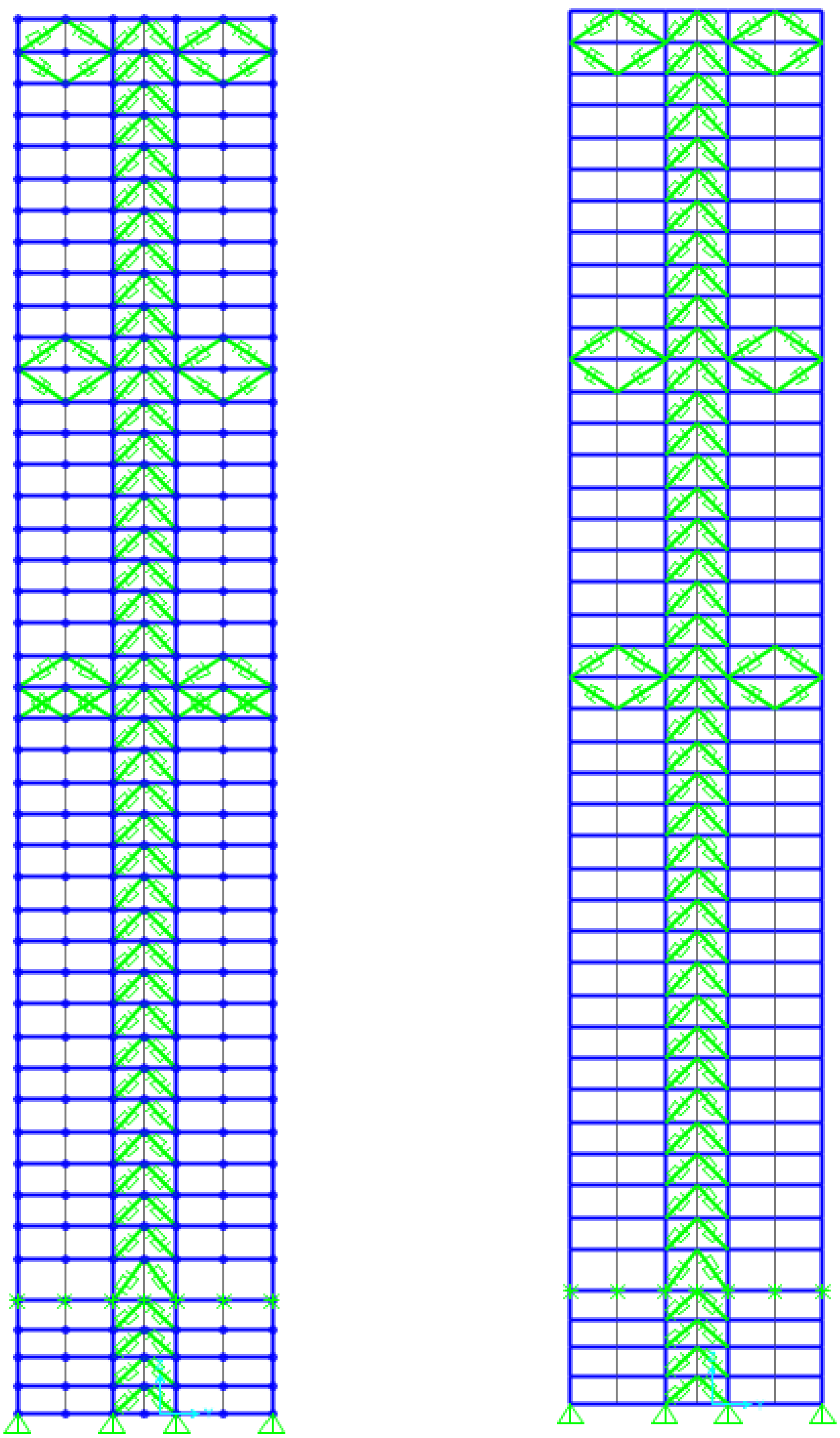


Figure 2 - Comparison between Phase 2 Alpha and Phase 2 Beta (Respectively)

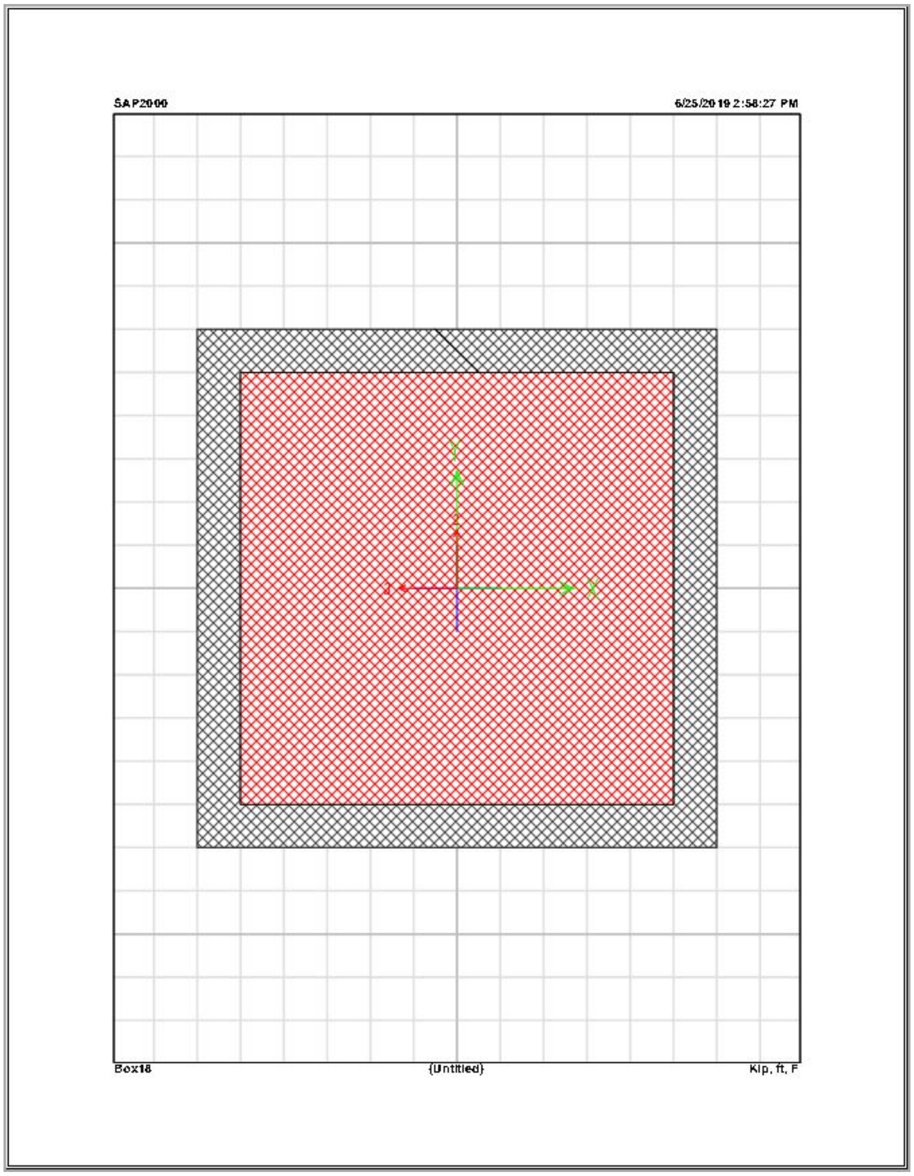


Figure 3 - Composite Column Section View / Box 18

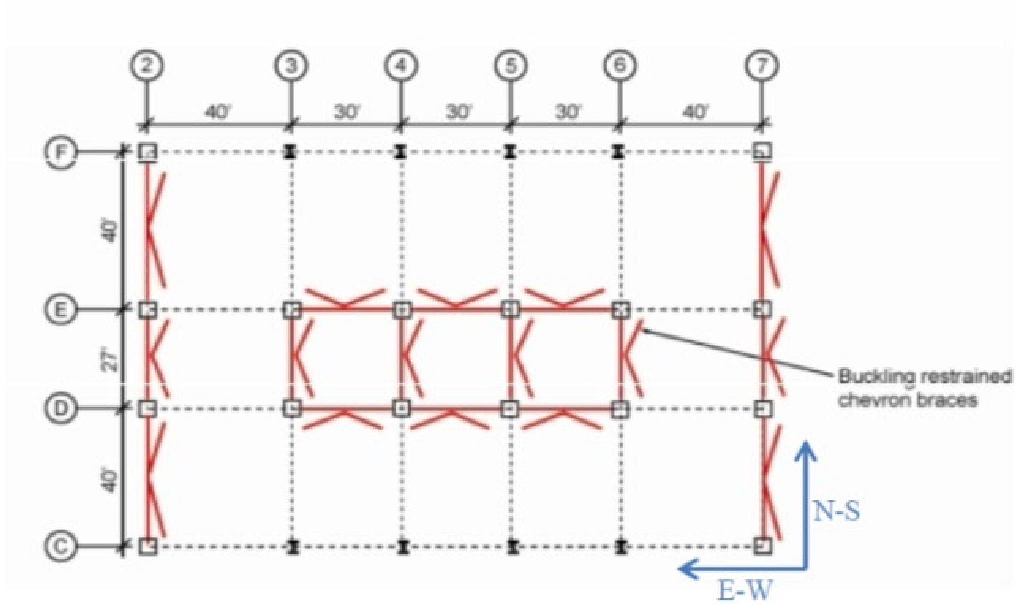


Figure 5 - Floor Plan for 20th, 30th, and 40th Floors

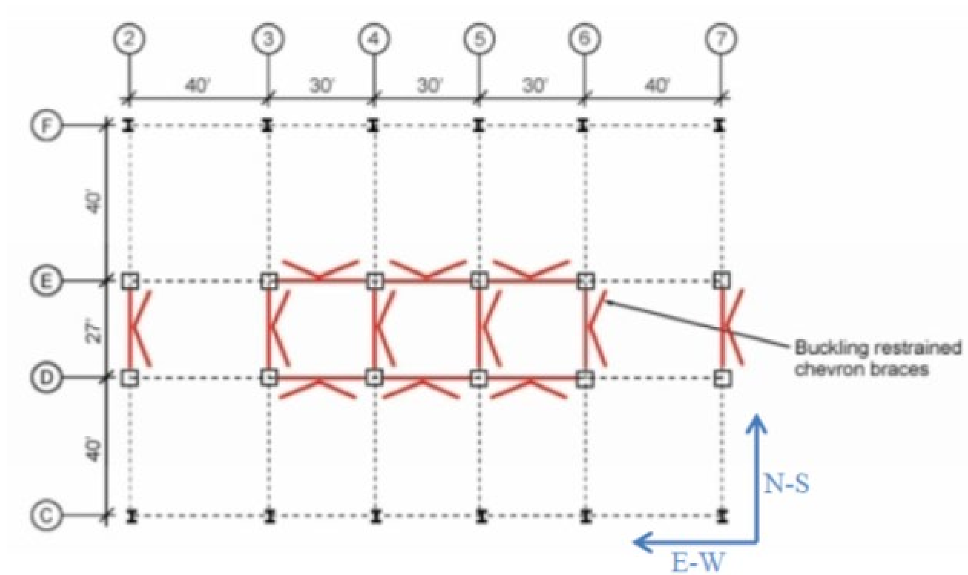


Figure 6 - Floor Plan for All Other Floors

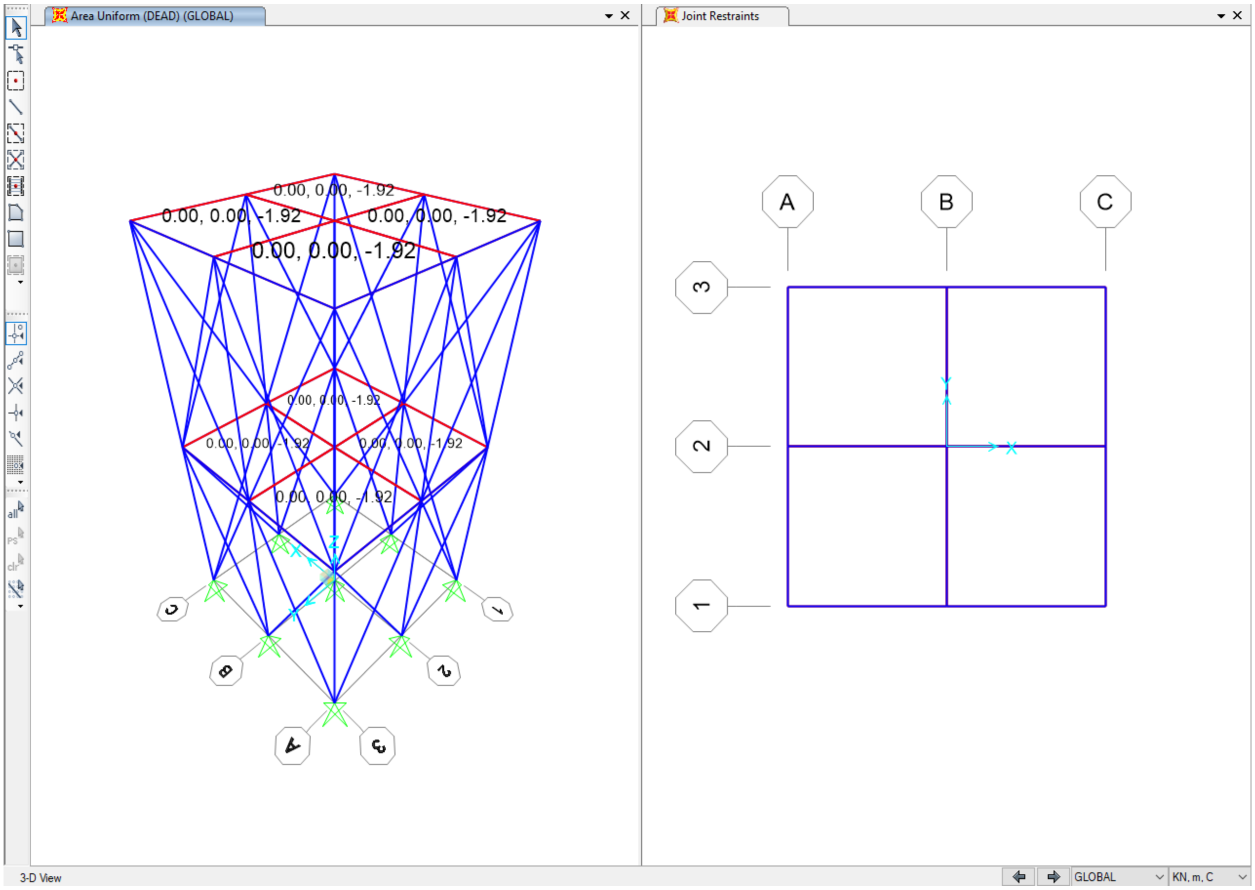


Figure 7 - Preliminary Research Example / Two Story 3D SAP2000 Model

Notable Equations

Equation 1 - Equation for the Primary Wave of an Earthquake

$$V_p = \sqrt{\frac{K + \frac{4}{3}\mu}{\rho}}$$

Equation 2 - Equation for the Secondary Wave of an Earthquake

$$V_s = \sqrt{\frac{\mu}{\rho}}$$

Equation 3 - Backbone Curve for BRB equations (Links)

$$\Delta y = \frac{L\sigma}{E}$$
$$k_0 = \frac{A_s E}{L}$$
$$h = 10\Delta y * tan(\frac{1.25}{100} * k_0)$$
$$D = C + \frac{h}{F_y A_s}$$

Where As = area of yielding steel core, E = 29,000ksi, Fy =38ksi, Ry = 1.1, ω = 1.25, β = 1.1, and L =70% of the brace length (using center-line to center-line geometry).

Equation 4 - General Equation of Motion in Structures

$$\sum F_x = m\ddot{x} + c\dot{x} + kx = 0$$

c = damping coefficient
k = stiffness
m = mass

Glossary of Important Terminology

- Buckling Restrained Brace:
  supports that help against the instability that leads to structural failure

- ASD Structural Format:
  known as Allowable Stress Design, and uses more large wide flange columns, yielding a heavier structure overall. Because this uses more material, it is a little more expensive

- LRFD Structural Format:
  known as Live and Resistive Factor Design, which focuses on a balance of low material usage and structural effectivity. With this, costs are minimized while still upholding the quality of construction.

- Dead Load:
  any load within a building that isn't expected to move

- Live Load:
  any load within the building that can be moved

- Topography:
  physical arrangement, shape, or form of artificial or natural areas

- Viscous Damper:
  object with a silicon oil compressive fluid inside a piston shell that has a depressing, subduing, or inhibiting effect against exterior loads, effectively dampening their kinetic energy

- Outrigger:
  a beam, spar, or framework projecting from or over the side of a structure, effectively yielding a higher degree of stability and structural integrity

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