ANALYSIS AND DESIGN OF STEEL FRAME FOR SHAKE TABLE VERIFICATION

A Thesis

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By

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ABSTRACT

Analysis and Design of Steel Frame for Shake Table Verification

By

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This thesis concerns the analysis and design of a steel frame to be used on a new shake table being installed at California State University, Los Angeles. The steel frame has two moment frames in one direction and two cable braced frames in the other. A preliminary analysis was done with mathematical models to obtain member sizes based on desired period. A SAP 2000 analysis was conducted to obtain theoretical predictions and demands values for design. The SAP 2000 model was subjected to two recorded earthquake ground motions each applied in different directions. The design was done according to the AISC Steel Construction Manual. In future work, the analysis results will be compared to the experimental values. The model and predictions will be used to characterize and calibrate the shake table.

ACKNOWLEDGMENTS

I would like to thank God for giving me the opportunity to pursue an education that will better my future. I would like to thank the Ahl al-Bayt and Imam Ahmad Al Hassan (a.s) for their teachings. I would like to thank my parents for helping me throughout my college career. I would like to thank Ahmad Samadi for giving me knowledge and motivation for a better future. I would like to thank Faeza Jawed. I would like to express my sincere gratitude to Professor Rodriguez-Nikl for his knowledge, patience, and goodwill throughout the research project. I want to also thank Freddy Cerezo for providing the steel structure drawings. I would like to thank my friends who helped me understand different concepts of civil engineering. I would like to thank Professor Tufenkjian and Purasinghe for providing recommendations for the research project. I want to gratefully acknowledge support from the Nabih Yousef Graduate Fellowship.

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CHAPTER 1

Introduction

Section 1: Goals and Overview

An analysis and design of a steel frame was done in order to verify and calibrate the shake table. This was done by modeling a 3D frame and subjecting it to ground motions. This gives the user values that will be compared to future experimental results. The frame will also be designed to remain elastic. In the remainder of this introduction Section 2.1 will introduce the shake table and the constraints it places on the design. Section 2.2 will discuss the different lateral systems used in the study, i.e. moment and braced frames. Section 2.3 will introduce the theoretical force-deflection relation that will be used for preliminary design. Section 2.4 will discuss relevant foundations of structural dynamics. Section 2.5 will discuss the planned range of tests for future of shake table verification and will mention the different testing that would be conducted.

Section 2: Background

Section 2.1: Shake Table

The shake table has different properties that will limit the analysis and design. This shake table was funded by the National Science Foundation and manufactured by Shore Western Manufacturing. The shake table is 5 ft. by 5 ft. and has a bolt hole arrangement of 1 ft. on center. The table can shake independently in two directions and is capable of 2g acceleration with a one-ton payload. The bolt hole arrangement results in a structure that is 4 ft. by 4 ft. in plan and the biaxial loading requires lateral resistance in both directions. Figure 1 displays the shake table and bolt hole arrangements:

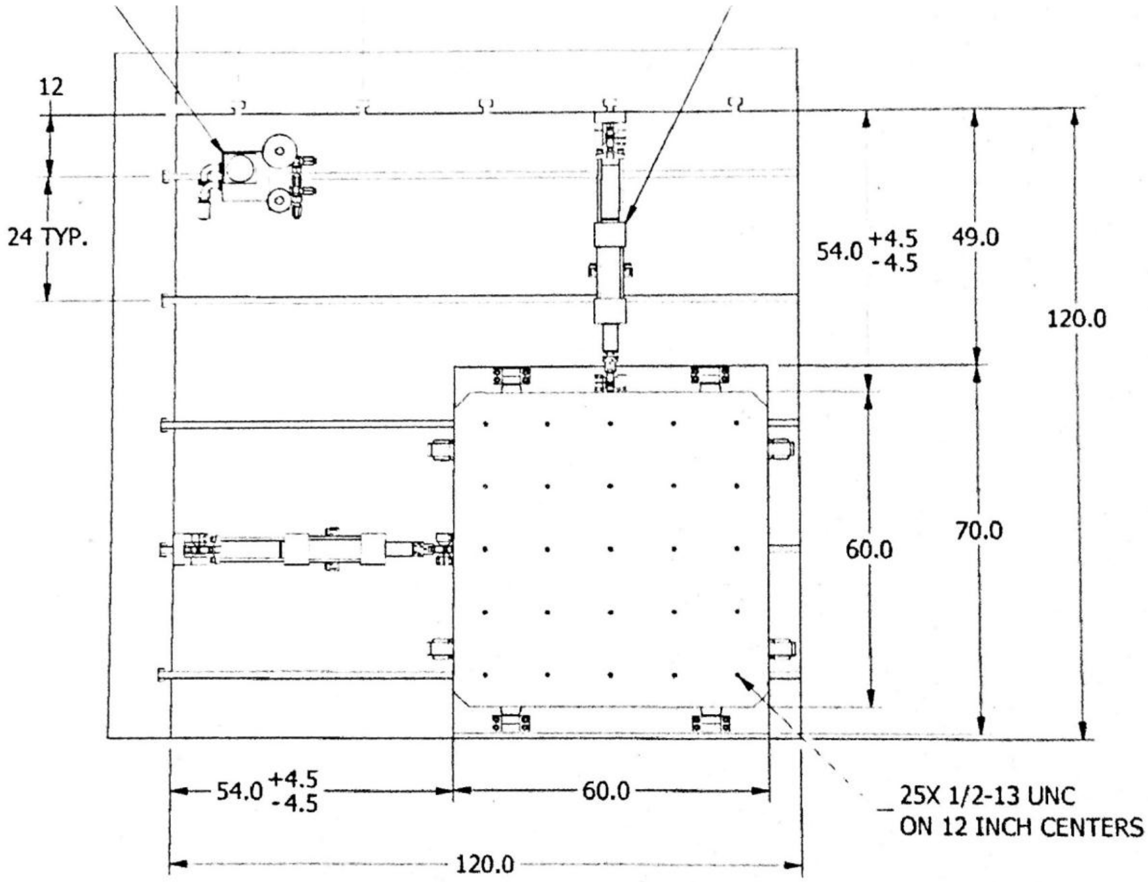

Figure 1. Plan view of Shake Table

Section 2.2: Lateral Systems

The part of a structure that resists lateral loads is called the lateral system. This project uses both braced and moment frames. A braced frame has a pinned connection between beams and columns. Lateral resistance is provided by a lateral component of the diagonal brace. Braced frames experience lower deflection and high acceleration values compared to moment frames. Brace arrangements that are used in practice include x-braces, eccentric braces, and concentric braces. The x-braced frame makes an x shape, connecting at beam-column joints. The eccentrically braced frames have braces that make a v shape but do not meet at one point of the structure. The concentrically braced frames have braces that do meet up at a point in the structure. The diagonal members can be slender or non-slender. The braced frame used in this design uses slender cables in an x-configuration. As will be discussed in chapter 4, cables present several difficulties but are still used for ease of erection.

Moment frames are formed with columns and beams connected by rigid connections that transfer a moment. Moment frames are classified by their ductility capacity as ordinary, special, and intermediate moment frames in increasing order of ductility capacity. The type of moment frame that will be used in the project is an ordinary moment frame because the moment frame will be kept elastic. Figure 2 illustrates a moment frame similar to the one that will be designed.

Figure 2. Moment frame. Photograph taken in the breezeway of Music building at California State University, Los Angeles

Section 2.3: Analytical Force-deflection Relation

For preliminary member sizing a theoretical equation was derived using the stiffness method see e.g., (Hibbeler, 2008). Figure 3 displays the stick model for the moment frame. The degrees of freedom (DOF) are shown in Figure 4.

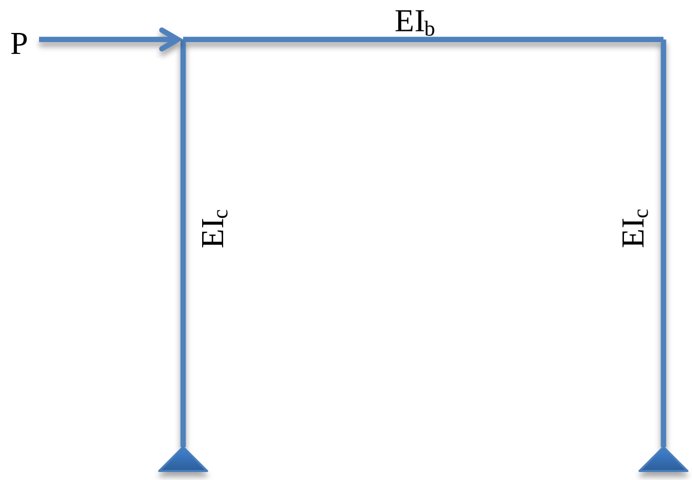

Figure 3. Moment frame Model

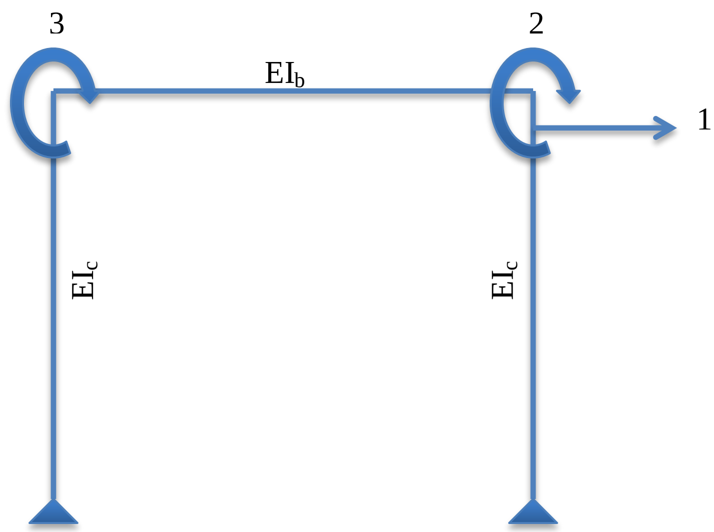

Figure 4. The Degrees of Freedoms for Moment Frame

The stiffness matrix can be shown to be:

$$K = \begin{bmatrix} \frac{6EI_c}{h^3} & \frac{3EI_c}{h^2} & \frac{3EI_c}{h^2} \\ \frac{3EI_c}{h^2} & \frac{3EI_c}{h} + \frac{4EI_b}{L} & \frac{2EI_b}{L} \\ \frac{3EI_c}{h^2} & \frac{2EI_b}{L} & \frac{3EI_c}{h} + \frac{4EI_b}{L} \end{bmatrix}$$

Where EI_c and h are the flexural, stiffness and height of the column respectively and EI_b and L are the flexural and stiffness and length of the beam respectively. Using a load vector

$$P = \begin{bmatrix} 1 \\ 0 \\ 0 \end{bmatrix}$$

and the stiffness equation

$$U = K^{-1} * P$$

the following equation was found for the first element of the deflection U

$$U_1 = \frac{2EI_b h^3 + EI_c L h^2}{12 EI_b EI_c}$$

This is the lateral deflection that results from a unit lateral load. The lateral stiffness of the frame is the inverse of U_1

$$K_{lat} = \frac{1}{U_1}$$

$$K_{lat} = \frac{12EI_b EI_c}{2EI_b h^3 + EI_c L h^2} \quad (1)$$

This will be used to select member sizes for the beam and column after selecting a target period as is explained in the following section.

The same was done for the braced frame system, which is represented by Figure 5:

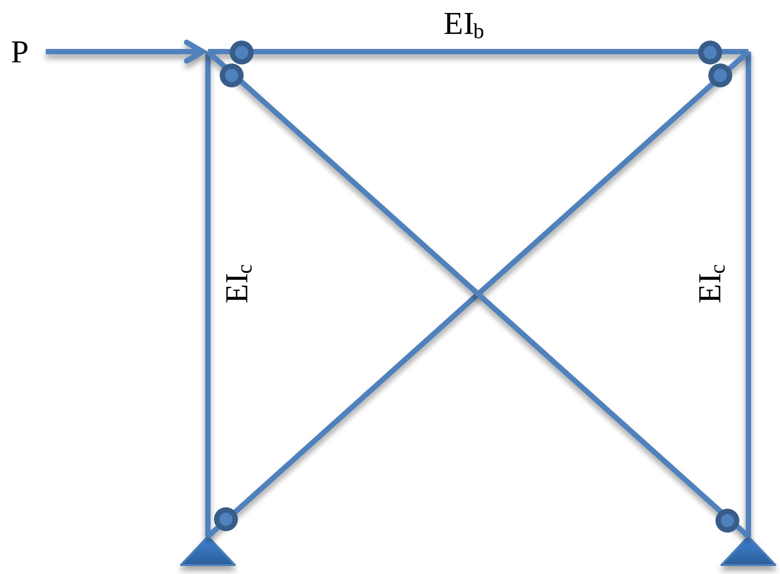

Figure 5. Braced frame Model

This is a single degree of freedom; statically determinate structure since the compression braces carries no load. The lateral stiffness can be found directly as

$$K_{lat} = K = \frac{EA}{L} cos(\theta)^2 \quad (2)$$

E is the modulus of elasticity taken as 29000 ksi, A is the area of the cable, L is the length of the cable, and θ is the angle at which the cable is oriented. This will be used to select a preliminary cable size.

Section 2.4: Structural Dynamics

Section 2.4-1: Single Degree of Freedom. A single degree of freedom system is a system whose motion is defined just by a single independent function. Figure 6 displays a single degree of freedom system:

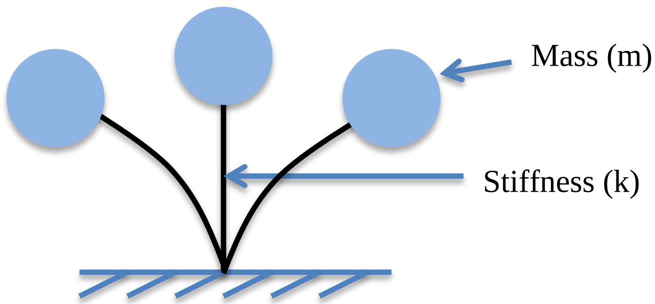

Figure 6. Components of a Single Degree of Freedom System

The equation that results in this generation is the following (Chopra 2012)

$$m\ddot{u}(t) + c\dot{u} + ku(t) = p(t) \quad (3)$$

where m represents mass, c is the damping constant, k is the stiffness, p is an external force, u is displacement, $\dot{u}$ is velocity, $\ddot{u}$ is the acceleration, and t is time. A structure is said to go through free vibration when $p = 0$. Free vibrations are studied to understand properties of vibrating systems. In free vibration with no damping, equation 3 becomes

$$m\ddot{u} + ku = 0 \quad (4)$$

The solution is of the form

$$u = A\cos(w_n t) + B\sin(w_n t)$$

When A and B are constants that are not needed here and w_n is the circular frequency, which will be found shortly. Differentiating this equation twice yields

$$\ddot{u} = -w_n{}^2 u \quad (5)$$

Combining equations 4 and 5 yields

$$({w_n}^2 m + k)u = 0$$

from which the circular frequency is found as

$${w_n}^2 = \frac{k}{m}.$$

The circular frequency in radians over time, can be rewritten in terms of period T_n as

$$T_n = \frac{2\pi}{w_n} = 2\pi \sqrt{\frac{m}{k}}. \quad (6)$$

The natural period of the system is the time for an undamped system to complete one cycle of free vibration is the natural period of the system. The unit for the natural period is in seconds. This equation will be used with the previously derived stiffness to find member sizes and mass values that result in a desired target value.

Section 2.4-2: Multiple Degrees of Freedom. A multiple degree of freedom has several independent stiffness and mass degrees of freedom. In order to derive a MDOF systems are presented by the following equation (Chopra 2012).

$$\mathbf{m}\ddot{\boldsymbol{u}}+\mathbf{c}\dot{\boldsymbol{u}}+\mathbf{ku}=\mathbf{p} \quad (7)$$

In this equation **m** is the mass matrix, **c** is the damping matrix, **k** is the stiffness matrix, **p** is an external force vector,$\ddot{\boldsymbol{u}}$, $\dot{\boldsymbol{u}}$, **u** are the acceleration, velocity, and displacement vectors respectively. In undamped free vibration

$$\mathbf{m}\ddot{\boldsymbol{u}}+\mathbf{ku}=\mathbf{0} \quad (8)$$

The displacement with separate space and time components is written as the following:

$$\boldsymbol{u}(t) = \sum_{n=1}^{N} \phi_n q_n(t) \equiv \sum_{n=1}^{N} \mathbf{u}_n(t) \quad (9)$$

In this equation ϕ_n is the n^th^ mode shape, $q_n(t)$is the n^th^ modal coordinate. The vibration can be shown to be harmonic, i.e.,

$$q_n(t) = A_n \cos(\omega_n t + \varphi_n) \quad (10)$$

In this equation A_n is a constant, ω_n is the natural frequency, φ_n is the phase shift. Frome this it can be shown that $\ddot{q_n} = -w_n{}^2 U_n$ and then from (8) and (9)

$$[(\boldsymbol{k} - w_n{}^2\boldsymbol{m})\phi_n]q_n = \boldsymbol{0}$$

This equation is used to determine the different mode shapes and frequencies of the structure, by solving the eigenvalue problem.

Inserting (9) into (7) and premultiply by ${\phi^t}_r$ yields

$${\phi^t}_r\left(\boldsymbol{m}\sum_{n=1}^{N}\phi_n\ddot{q_n} + \boldsymbol{c}\sum_{n=1}^{N}\phi_n\dot{q_n} + \boldsymbol{k}\sum_{n=1}^{N}\phi_n q_n = \boldsymbol{p}\right)$$

Using the fact that ${\phi^t}_r\boldsymbol{m}\phi_n$ = ${\phi^t}_r\boldsymbol{k}\phi_n$=**0** for r $\neq n$ and assuming the same for c yields

$$\underbrace{{\phi^t}_r\boldsymbol{m}\phi_n}_{M_n}\ddot{q_n} + \underbrace{{\phi^t}_r\boldsymbol{c}\phi_n}_{C_n}\dot{q_n} + \underbrace{{\phi^t}_r\boldsymbol{k}\phi_n}_{K_n}q_n = \underbrace{{\phi^t}_r\boldsymbol{p}}_{P_n}$$

The natural frequency can be represented by

$$\omega_n = \sqrt{\frac{K_n}{M_n}}$$

Section 2.4-3: Rayleigh Damping. Because the cable required use of a nonlinear link in SAP 2000, the analysis required direct integration which in turn required use of Rayleigh damping. The Rayleigh damping process creates a damping matrix that is proportional to the mass and stiffness matrix as follows (Chopra 2012).

$$\boldsymbol{c} = a_0\boldsymbol{m} + a_1\boldsymbol{k}$$

For the equation above **c** is the damping matrix, **m** is the mass matrix, and **k** is the stiffness matrix. The constants a_0 and a_1 are selected based on damping ratios in two modes i and j, If $\zeta_i = \zeta_j = \zeta$

$$a_0 = \zeta \frac{2\omega_i \omega_j}{\omega_i + \omega_j} \quad a_1 = \zeta \frac{2}{\omega_i + \omega_j}$$

and damping in any other mode is

$$\zeta_n = \frac{a_0}{2\omega_n} + \frac{a_1 \omega_n}{2}$$

In this thesis a 2% value was used for damping in modes 1 and 3.

Section 2.5: Stages for Shake Table Verification

A progression of tests will be directed to characterize table behavior. Parameters of interest include equivalent damping, transfer function and performance characteristics such as signal fidelity. The test series will be modeled on previous analyses for the following shake tables: Rice University (Conte et al., 2000), University of Nevada, Reno (Thoen et al., 2004), UCSD-NEES (Luco et al., 2010), (Ozcelik 2008), (Ozcelik et al. 2008a), (Ozcelik et al. 2008b), and EUCENTRE TREESLab (Airouche et al., 2008).

The testing will be conducted in three stages. Stage I will be conducted on a bare table. The table will be subjected to white noise, harmonic, triangular, and seismic signals.

Stage II will be conducted with the structure designed in this thesis. The same tests as mentioned in stage I will be used in this phase. Test information will be utilized to create a numerical model of the Cal State LA shake table.

Stage III is used to run structural system verification tests on the frame designed in this thesis. This stage will compose of shake table specimen testing for structural identification and performance. The tests will compose of white noise, harmonic, triangular, sine sweep, free vibration, and seismic signals.

Section 3: Organization of Thesis

Chapter 2 will present the details of the analysis process and the results. The results will include the demand values and the theoretical acceleration data. Chapter 3 will present the design calculations and resulting drawings. Chapter 4 will summarize the thesis, discuss possible sources of error, and recommend future work.

CHAPTER 2

Analysis

Section 4: Requirements and Constraints

The objective for the preliminary analysis was to achieve a target period by selecting member sizes and magnitude of seismic mass. The period of 0.1 seconds, is typical for a one-story structure. The shake table dimensions limited the size of the structure to 4 ft. by 4 ft. in plan. The structure has two perimeter moment frames in one direction and perimeter cable braced frame in the other. To allow the mass to be loaded and unloaded from the structure sandbags will be attached to a steel basket attached to the top of the structure.

Section 5: Approximate Analyses for Trial Member Sizes

A preliminary analysis was conducted in order to get trial member sizes. The goal of the analysis is to find member sizes and mass that will produce a period of 0.1 seconds. Using equations 1 and 6, many combinations of beam, column, and mass will satisfy this condition. The member sizes selected were among the smallest available. The member sizes picked were WT 3x6 for beam and W 6x15 for the column. Using equations 1 and 5 yields

$$K_{lat} = 3.81 \text{ kip/in.}$$

$$m = 0.00097 \, \frac{kip * sec^2}{in.} = 744 \, lbs.$$

The SAP 2000 model uses a lumped mass at the four corners that includes self-weight of the frame and basket and the additional mass. The self-weight of the structure must be subtracted from total to determine the actual mass value to be added in an experiment.

The braced frame direction will also be designed to achieve period as close possible to 0.1 seconds using equations 2 and 6 and the mass selected above. The cable area, length and angle orientation were needed. The area of the cable cross-section was calculated as follows. A diameter of 0.25 in. was selected as the smallest practical size.

$$\text{Diameter d}= \frac{1}{4}\text{ in.} = 0.25\text{ in.}$$

$$\text{Area A}= \frac{\pi(0.25)^2}{4}= 0.049 in.^2$$

The length of the cable was found as follows:

$$L = \sqrt{BxH}$$

$$B=4\text{ ft. } H= 4\text{ ft.}$$

$$L = \sqrt{48\ in. * 48 in.}$$

$$L=67.88\text{ in.}$$

The lateral stiffness of the braced frame is (for E = 29000 ksi, and $\theta = 45°$)

$$\text{Stiffness K}= \frac{EA}{L}\cos^2\theta,\ \theta = 45°$$

$$K= \frac{(29{,}000\ ksi)(0.049\ in.^2)}{67.88\ in.}(\cos^2 45°)= 10.484\ \frac{kip}{in.}$$

Using this stiffness value, the period is found to be 0.06 sec. as follows.

$$T = 2\pi\sqrt{\frac{m}{2k_{br}}} = 0.060\ sec\ \text{(Actual Period)}$$

$$T = 2\pi\sqrt{\frac{m}{4k_{br}}} = 0.043\ sec\ \text{(SAP Period)}$$

The area value could have been reduced to get the target period but it would require a very small cable diameter. The cable stiffness $\frac{EA}{L}$ is 20.971 kip/in.

Section 6: SAP 2000 Model

The finite element analysis program that was used in order to model and analyze the structure was SAP 2000 version 18 (CSI 2015). The SAP 2000 model is not required for obtaining the acceleration and displacement values. The SAP model is ultimately used to obtain member forces faster and easier. The model will also be convenient for future work that includes eccentric masses and other irregularities that is not modeled easily. In the global axis system, the y-axis is in the direction of the moment frame, the x-axis is the direction of the braced frame, and the z-axis is in the vertical direction. The brace's local has the x-axis along the length of the hook and the y and z-axis perpendicular to the brace. Figure 7 displays the global and brace element axes. The hook property was selected for the brace because it tension only properties characteristic of cables. The model undergoes a time history analysis, with two separate ground motion. Since the shake table can move in two directions, ground motion will be applied in the global x and y-direction. The ground motions used in the analysis were Northridge and El Centro earthquakes, both scaled to 1.0 PGA. Figures 8 and 9 displays the ground motions.

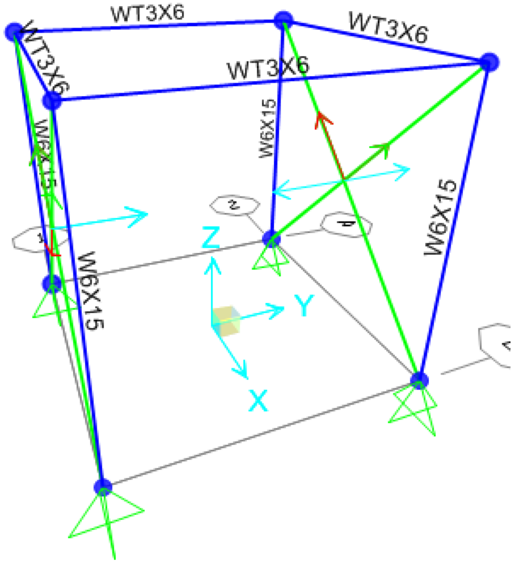

Figure 7. Global and Brace Element Axes

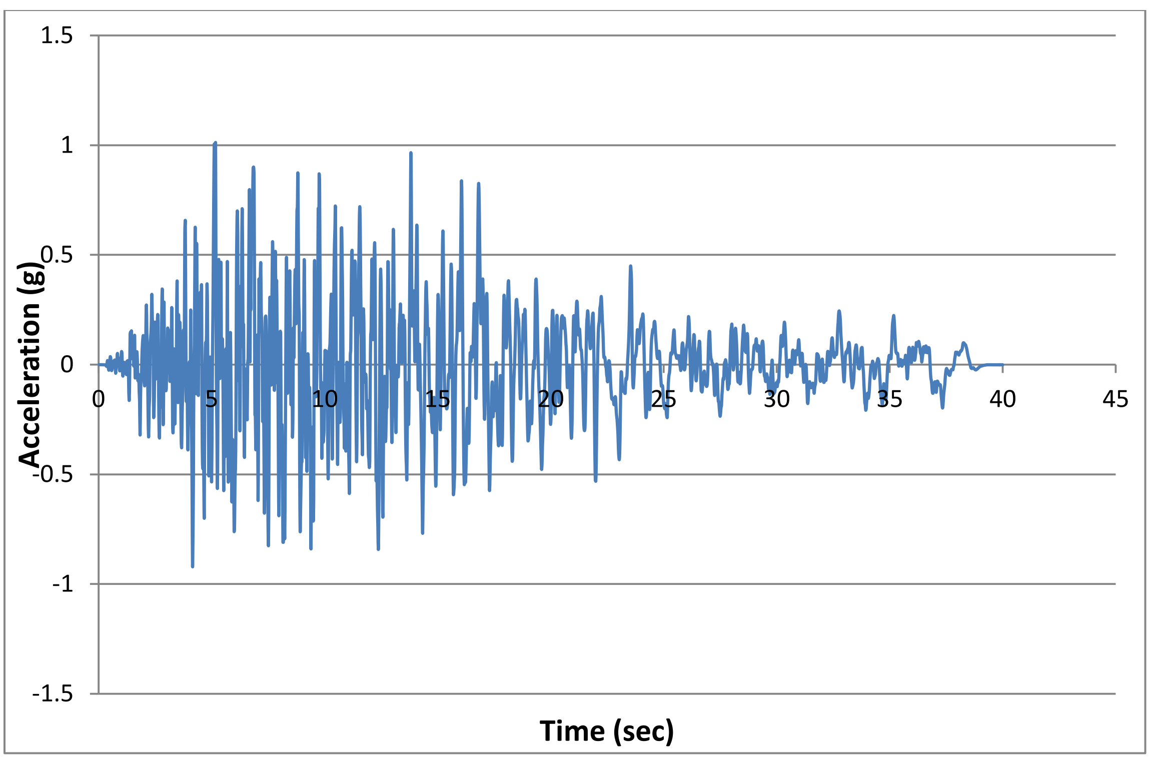

Figure 8. Northridge Earthquake Time History (Scaled to 1.0g)

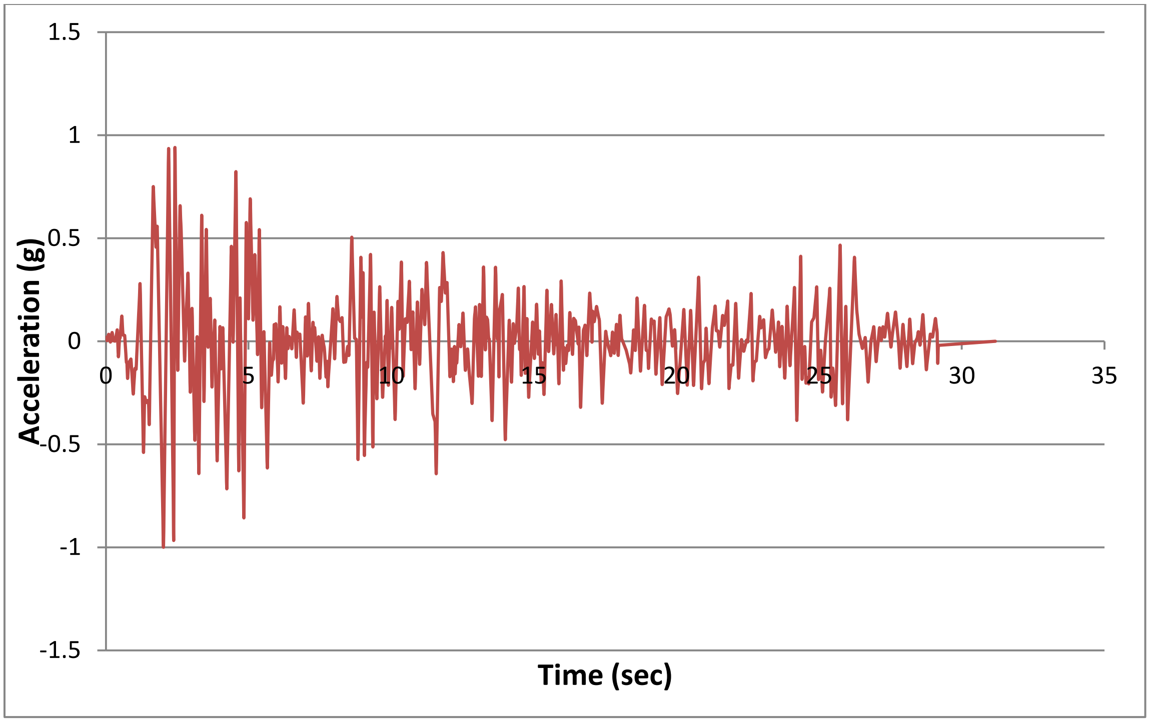

Figure 9. El Centro Earthquake Time History (Scaled to 1.0g)

The structure was loaded with Dead Load (self weight) and seismic load. A time history analysis was conducted. Loads were combined using ASCE 7-10 (American Society of Civil Engineers 2013) LRFD (Load and Resistance Factor Design) load combinations. The load combinations used in the analysis were 1.2D±1.0E, 0.9D±1.0E, and an Envelope load case. The D represents the Dead load while the E represents the lateral Earthquake load.

The analysis was nonlinear because of the nonlinear hook element. Direct integration was required because of a discrepancy in the braced frame period that will be discussed shortly. The numbers of output steps for Northridge earthquake is 5000 and the step size is 0.01 sec. For El Centro earthquake the number of output steps is 3000 and step size is 0.01 sec. The members selected in preliminary analysis were used for the SAP 2000 model. Beams and columns cross sections were chosen from built-in database. The cable stiffness was defined in the local x direction using the stiffness found above 20.93 kip/in. Column bases were modeled as pins. The loads and masses were lumped evenly to the four corners of the structure in both x- and y- direction. The self-weight of the cable was neglected.

Figures 10 to17 shows the member forces and mode shapes. Maximum shear and moment for the beam were 1006.88 lbs. and 23.9 kip (Figure10). Maximum shear and moment for the column were 498 lbs. and 23.9 kip.-in (Figure 11). Maximum axial force in the beam was 504.7 lbs. (Figure 12). Maximum axial force in the column was 1313 lbs. respectively (Figure 13). The maximum axial in the cable was 1325 lbs. respectively (Figure 14). Figures 15 through 17 show the different modes of the structure. The first mode (Figure 15) consists of deformation in the moment frame direction. The second mode (Figure 16) is the torsional mode. The third mode (Figure 17) consists of deformation in the braced frame direction. For the third mode, SAP outputs 0.04 sec for period, while the hand calculation was calculated to be 0.060 sec as seen in the following equations.

$$T = 2\pi\sqrt{\frac{m}{2k_{br}}} = 0.060\ sec \text{ (Actual Period)}$$

$$T = 2\pi\sqrt{\frac{m}{4k_{br}}} = 0.043\ sec \text{ (SAP Period)}$$

This is because SAP accounts for the stiffness of both braces in each braced frame, instead of accounting for the fact that one brace must always be unloaded. This is the main reason that modal analysis cannot be used. Acceleration and displacement histories are reported in figures 19-26 for node 8. All four roof nodes (Figure 18) reported the same values to rigidity provided by the diaphragm cables. Table 1 summarizes maximum displacements and accelerations for each ground motion.

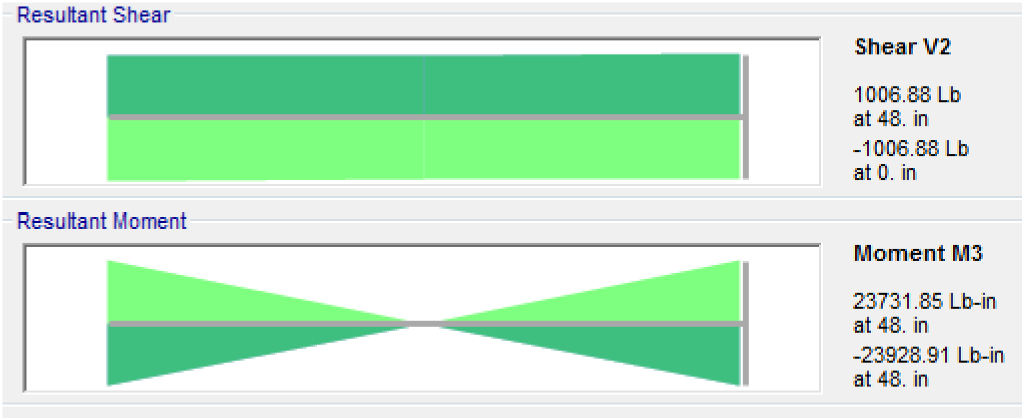

Figure 10. Moment Frame Beam Shear and Moment Envelopes

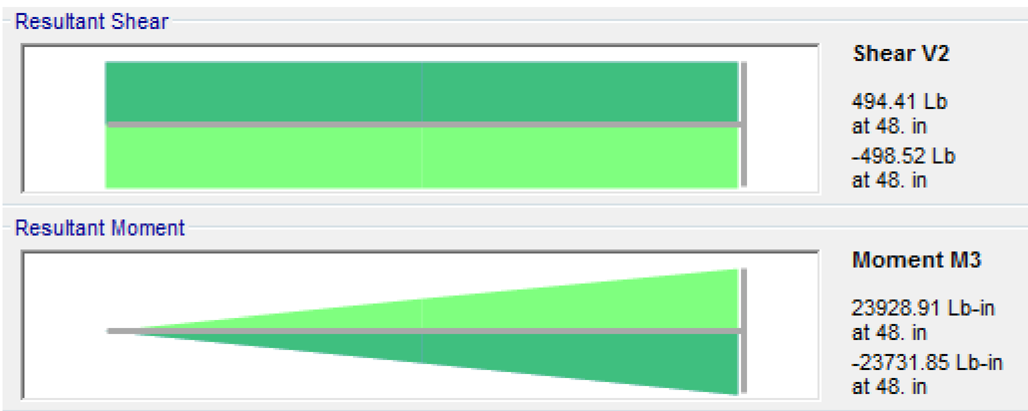

Figure 11. Column Shear and Moment Envelopes

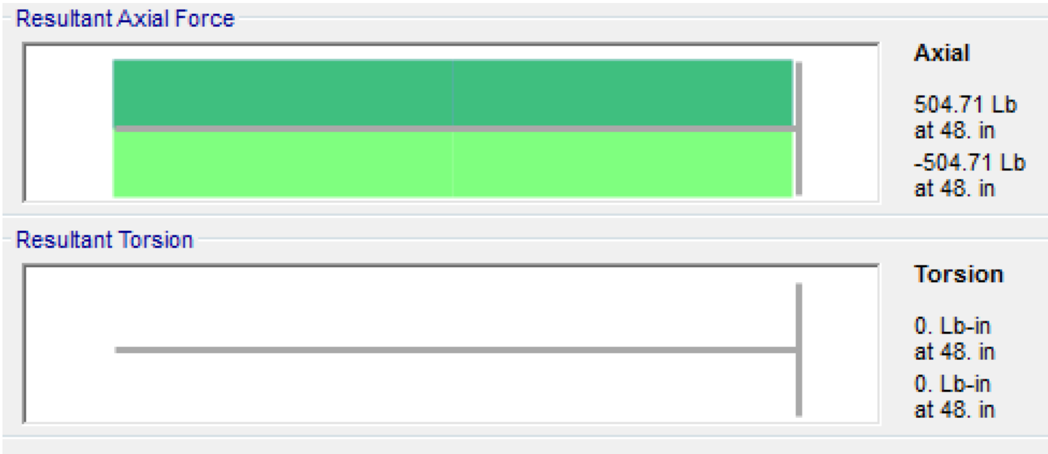

Figure 12. Beam Axial Force

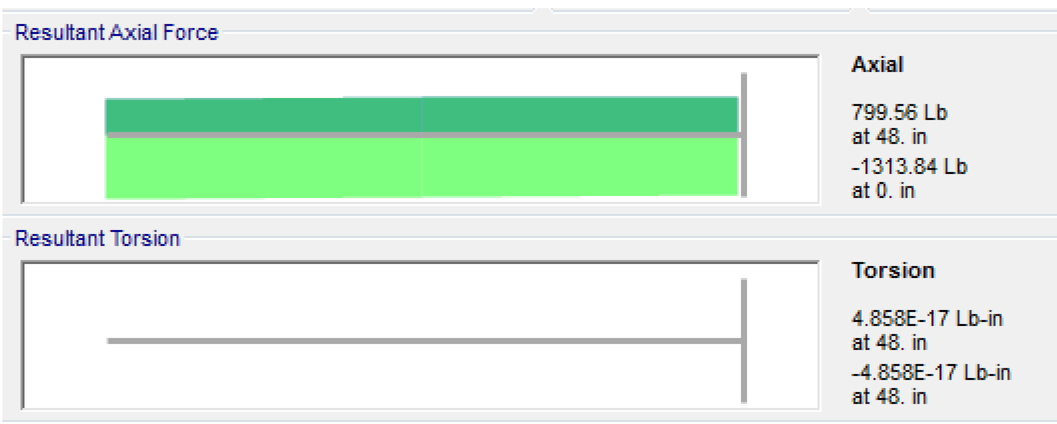

Figure 13. Column Axial Force

Link Text	LinkElem Text	Station Text	OutputCase	CaseType Text	StepType Text	P Lb
6	6	I-End	North-x	NonDirHist	Max	1325.01
6	6	J-End	North-x	NonDirHist	Max	1325.01

Figure 14. Hook Axial Force

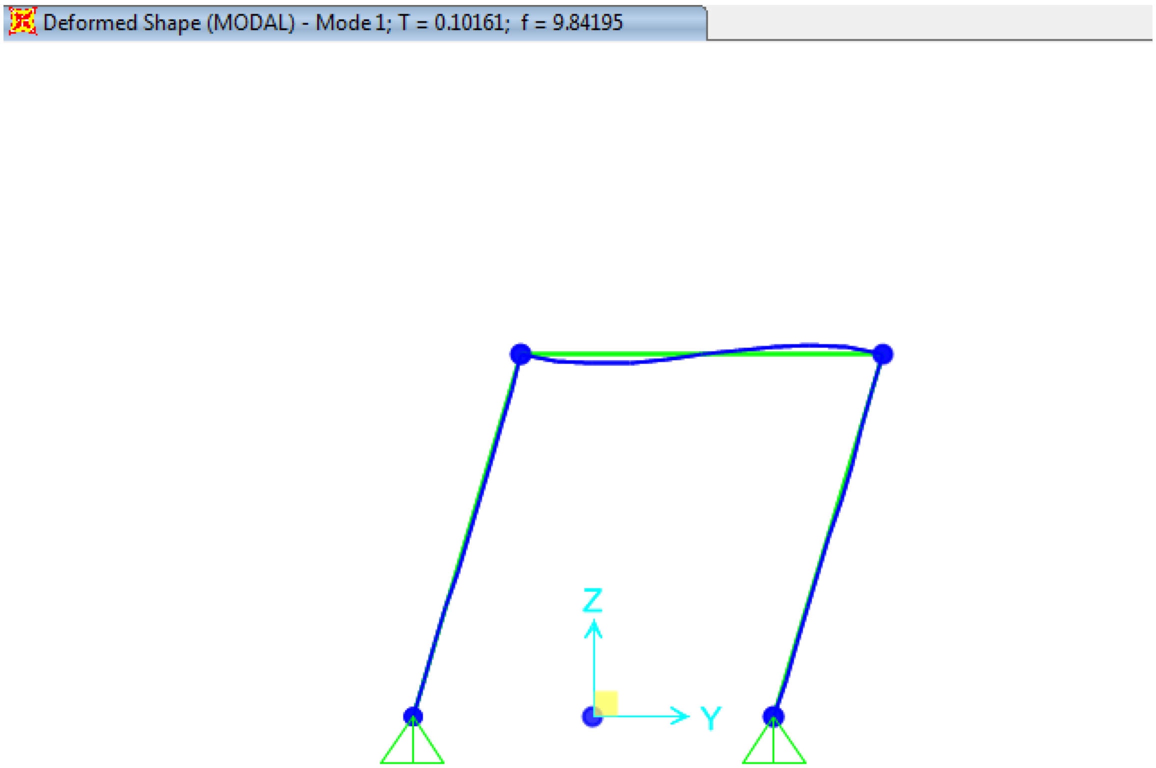

Figure 15. Moment Frame Drift and Deformed Shape (Mode Shape 1)

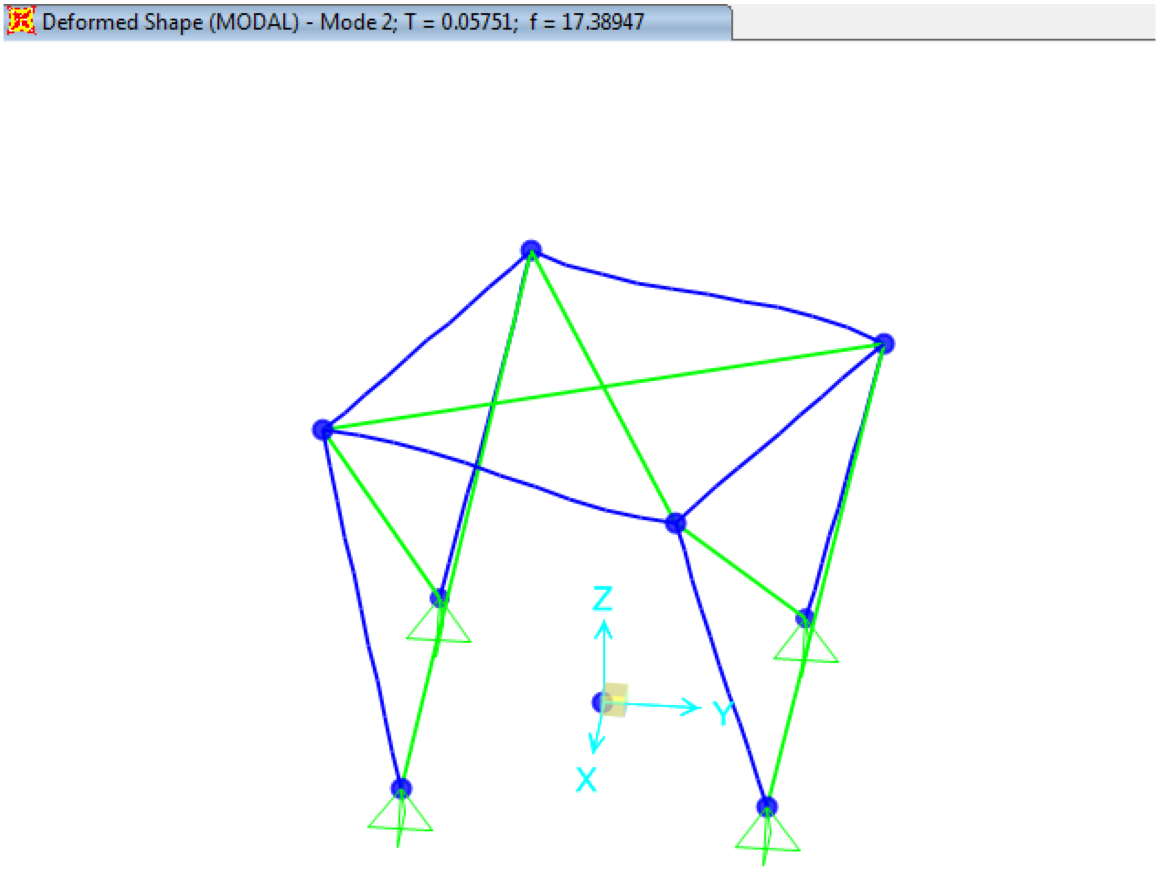

Figure 16. Torsional Mode (Mode Shape 2)

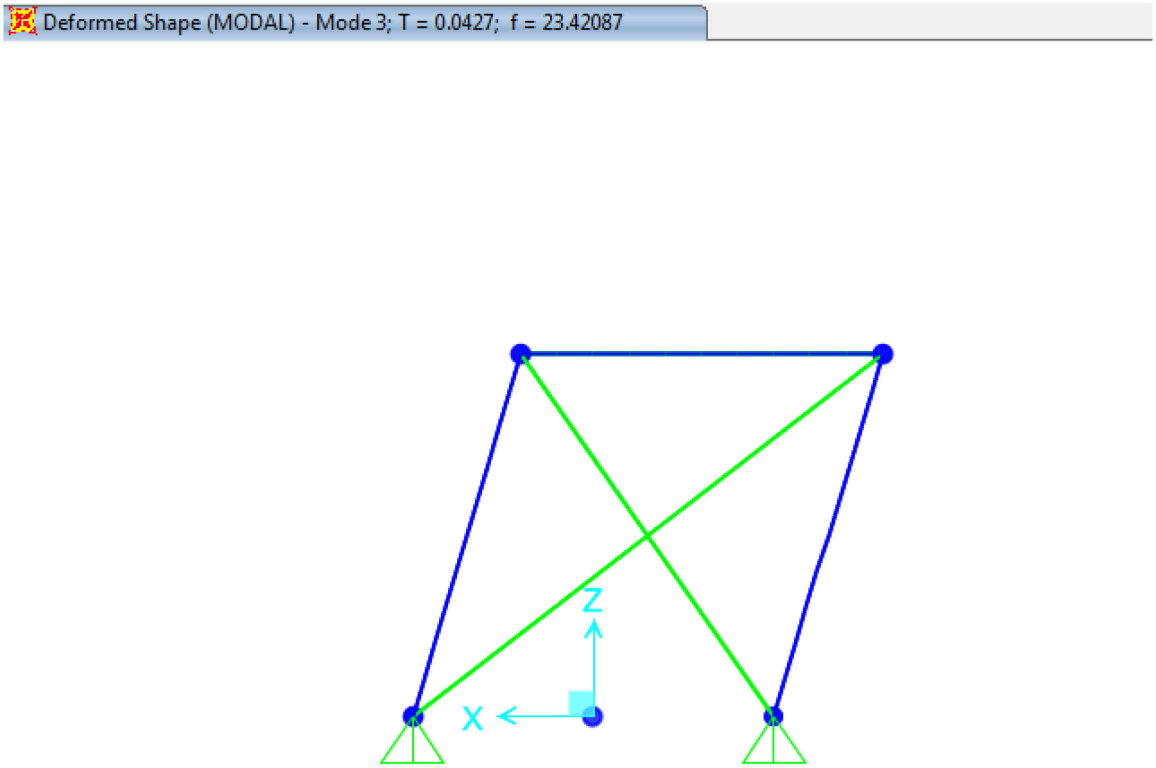

Figure 17. Braced frame Drift and Deformed Shape (Mode Shape 3)

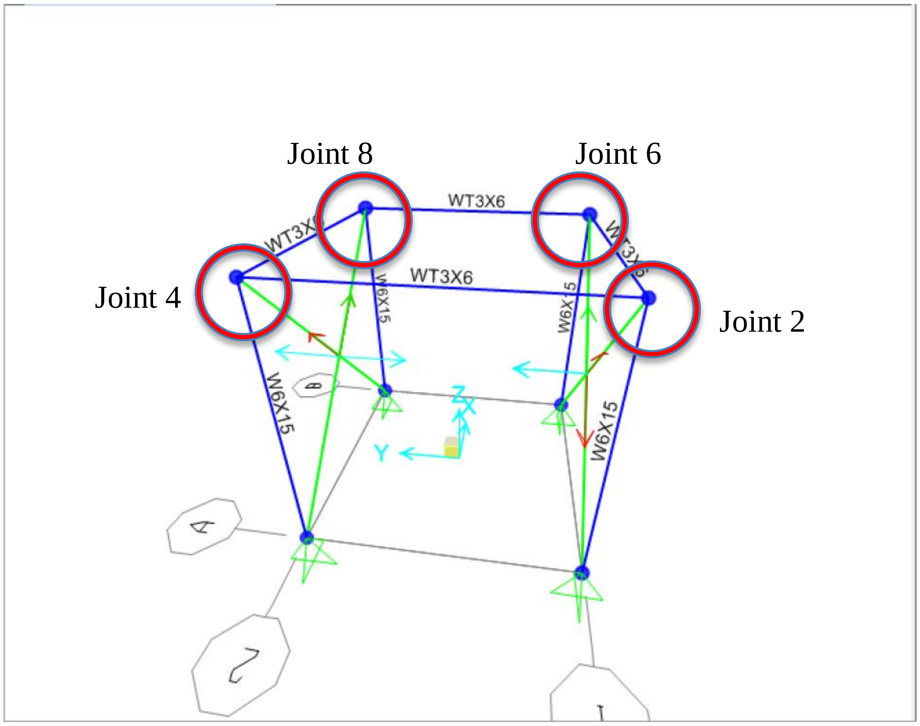

Figure 18. Joint Locations

Table 1

Joint 8 Max Displacement and Acceleration Values

	El Centro Braced	Northridge Braced	El Centro Moment	Northridge Moment
Max Acceleration (g)	1.61	2.69	2.42	2.65
Max Displacement (in.)	0.057	0.09	0.24	0.27

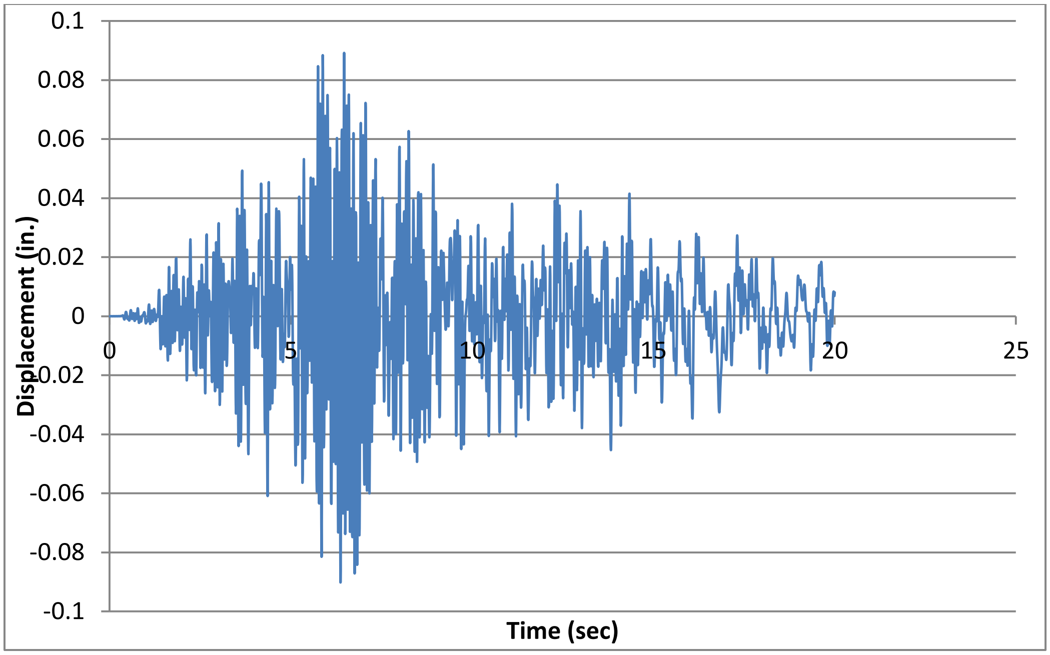

Figure 19. Joint 8 Roof Displacement Plot (Northridge Braced Frame (x))

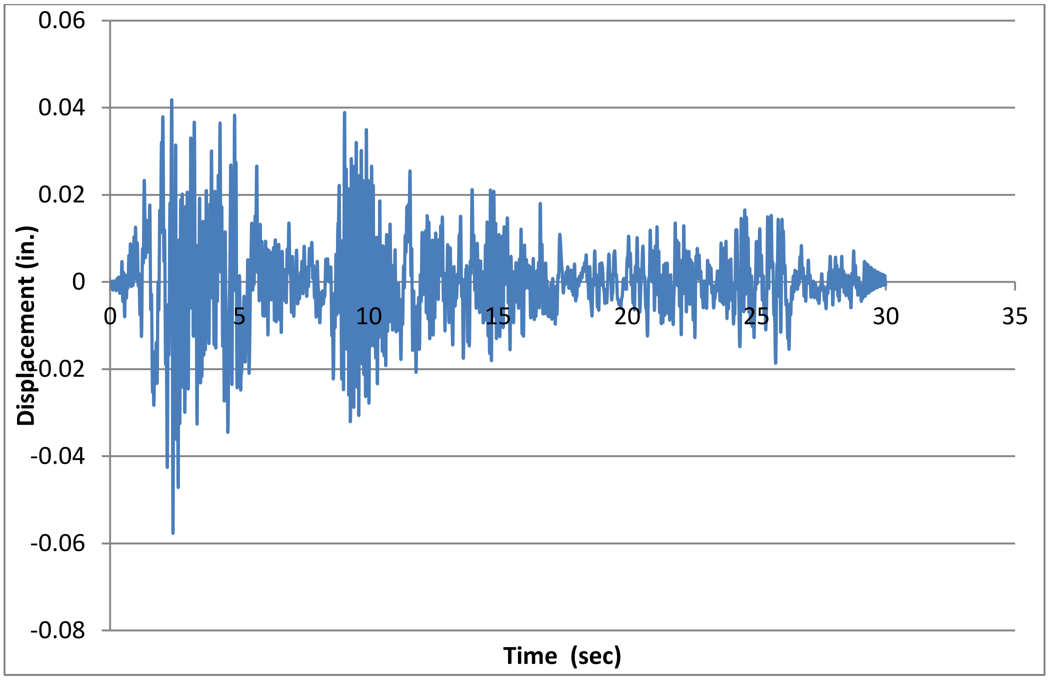

Figure 20. Joint 8 Roof Displacement Plot (El Centro Braced Frame (x))

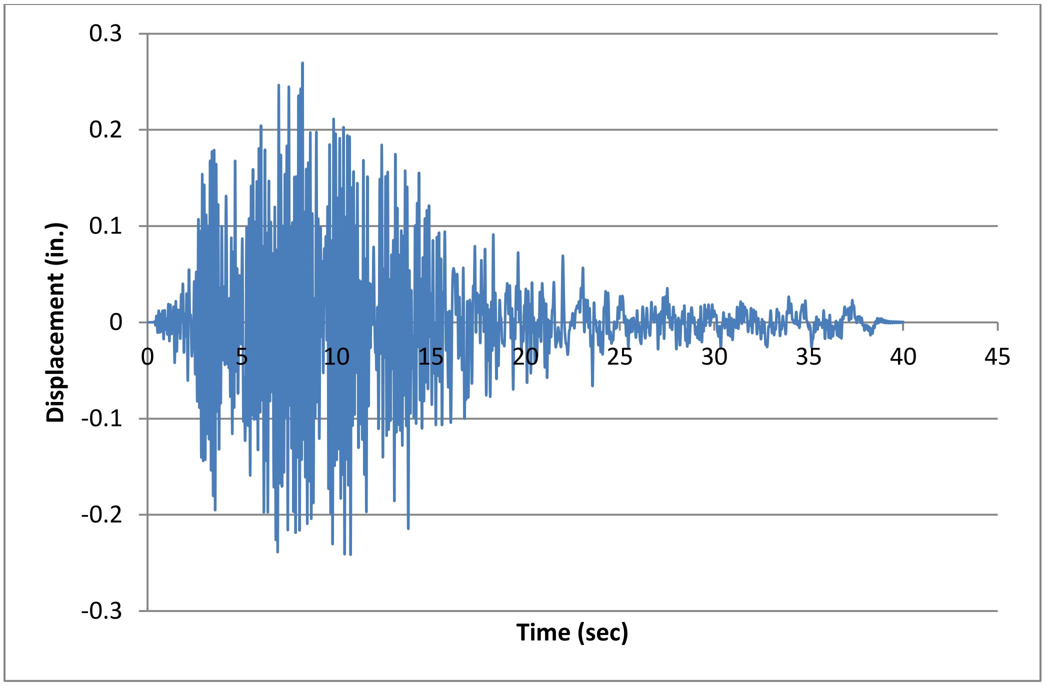

Figure 21. Joint 8 Roof Displacement Plot (Northridge Moment Frame (y))

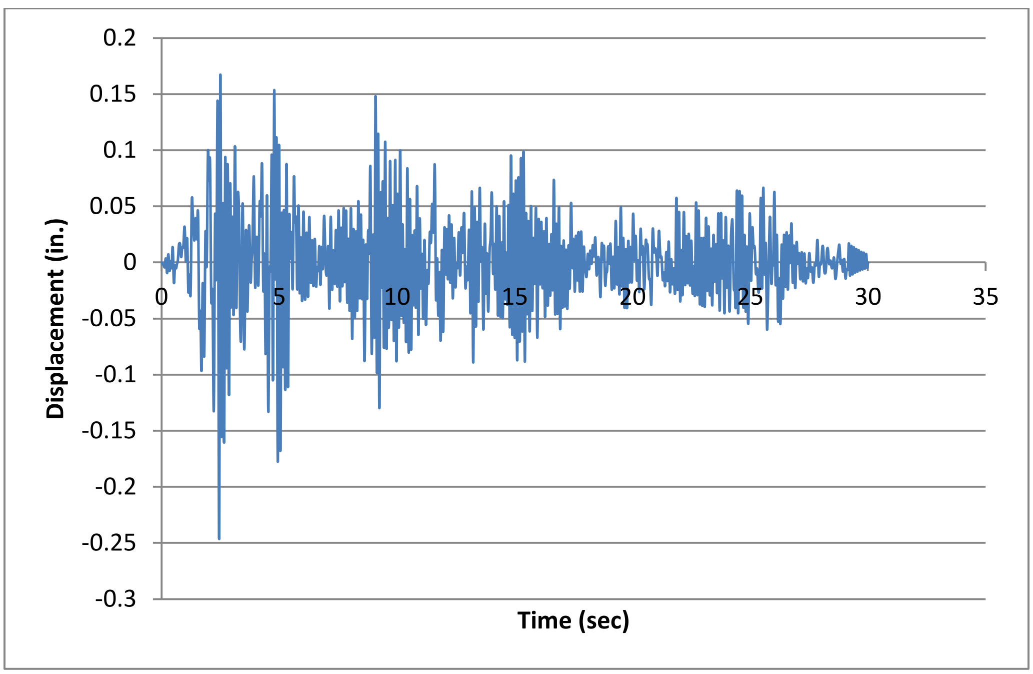

Figure 22. Joint 8 Roof Displacement Plot (El Centro Moment Frame (y))

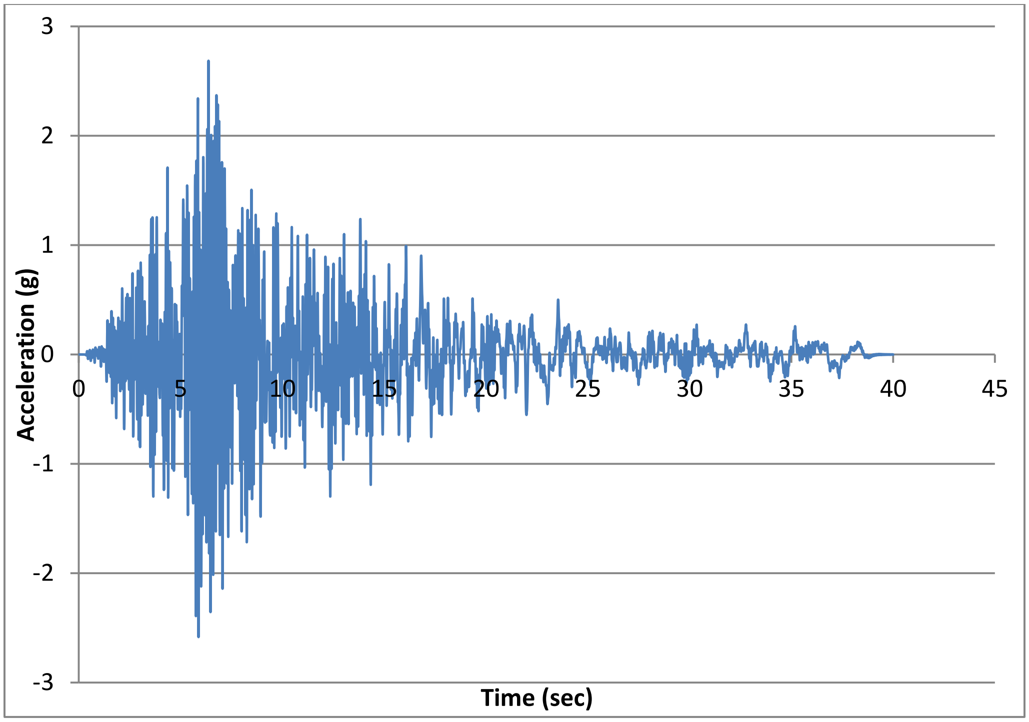

Figure 23. Joint 8 Roof Acceleration Plot (Northridge Braced Frame (x))

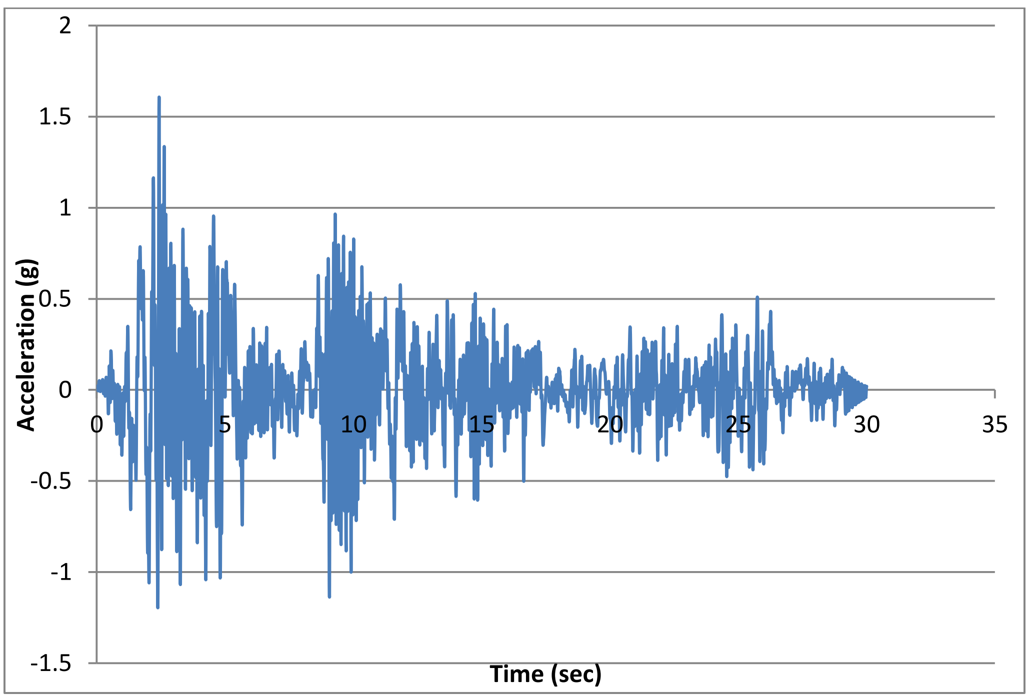

Figure 24. Joint 8 Roof Acceleration Plot (El Centro Braced Frame (x))

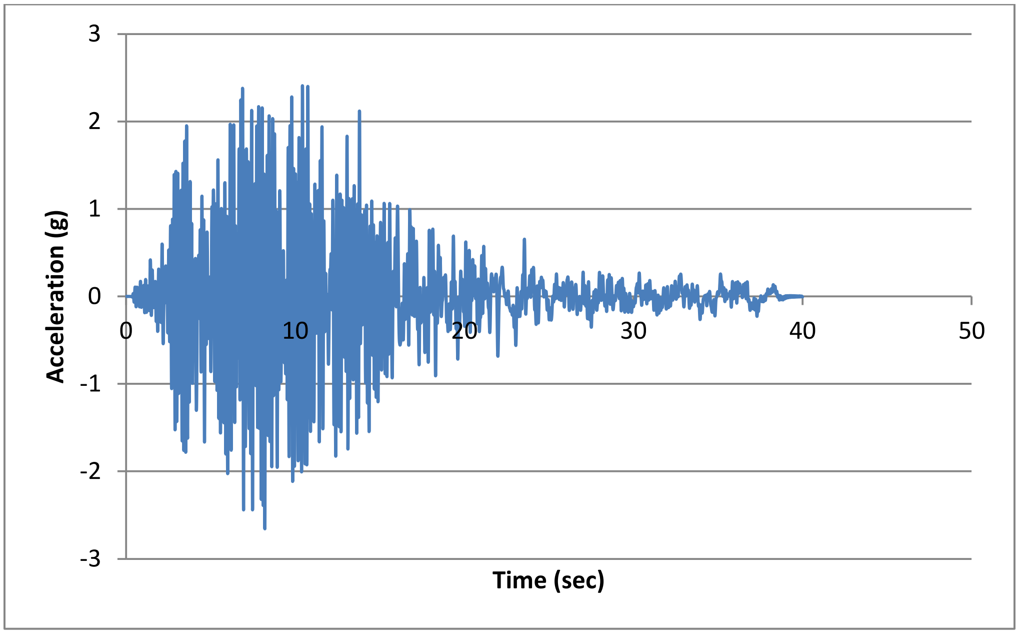

Figure 25. Joint 8 Roof Acceleration Plot (Northridge Moment Frame (y))

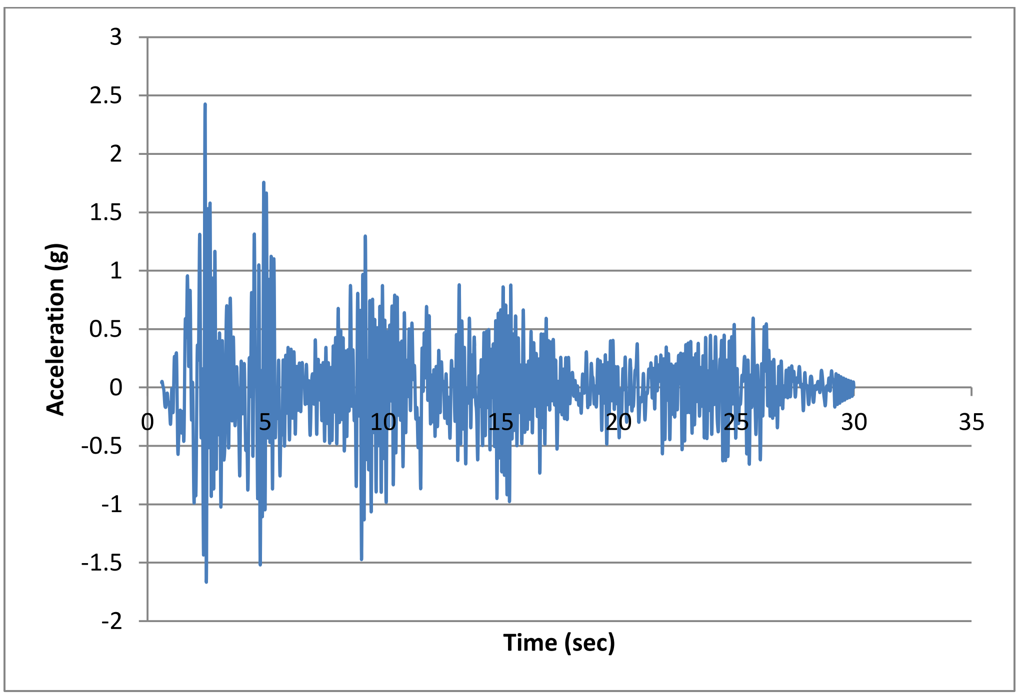

Figure 26. Joint 8 Roof Acceleration Plot (El Centro Moment Frame (y))

CHAPTER 3

Design

The member forces from SAP 2000 were used to check the capacities according to the AISC Steel Construction Manual (AISC 2011). Figures 27 to 34 display the structural drawings of the test specimen. Figure 27 shows the moment and braced frame of the structure. The braced frame also shows the connections for the cross-cables (connections 2 and 3) but the cables are not shown. Figure 28 is the plan view of the structure, Figure 29 displays the beam that is used for the braced frame and Figure 30 is the beam that is used for the moment frame. This figure shows the angled gusset plates that will be used to connect the diaphragm cross-cables. Figure 31 displays the braced frame beam-to-column connection. Figure 32 displays the connection at the column base with a base plate to connect to the shake table and gusset plate for the cable. Figure 33 is the moment frame connection with the complete joint penetration (CJP) welding and continuity plates. Figure 34 is the loading basket of the structure that will hold the superimposed weight.

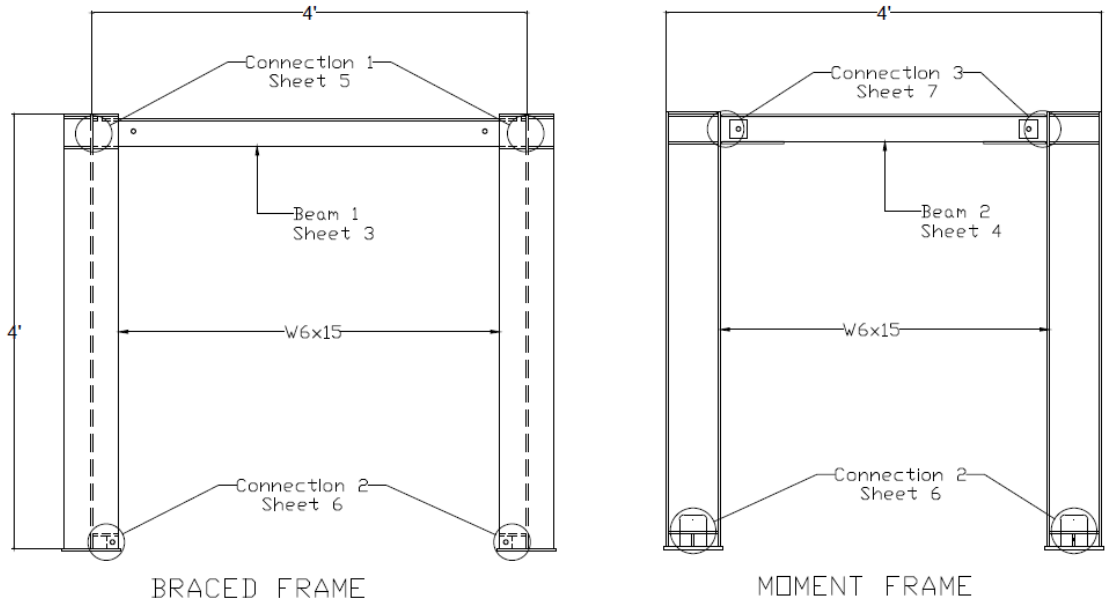

Figure 27. Braced and Moment Frames (Drawn by Freddy Cerezo)

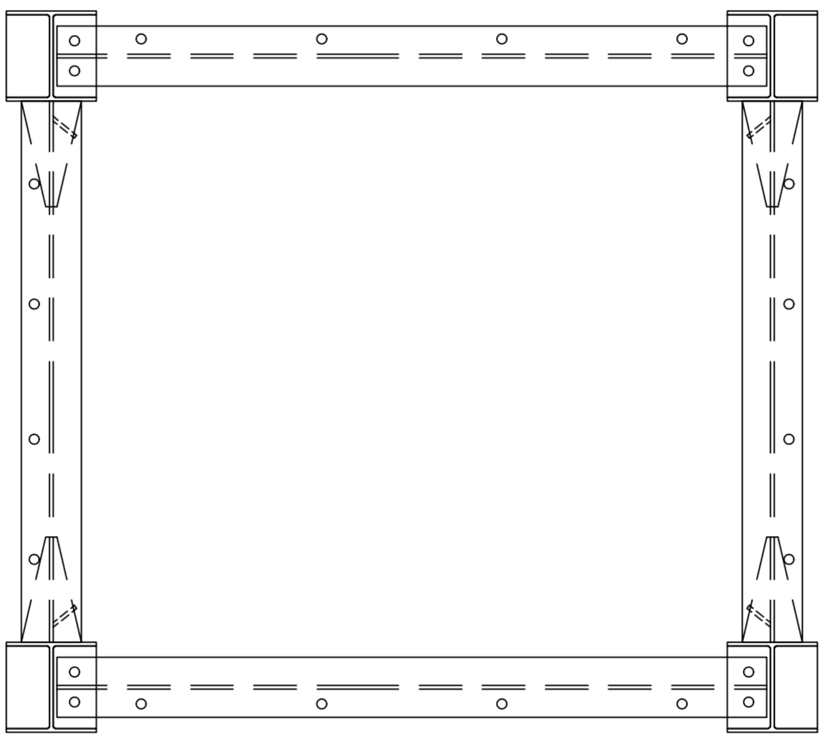

Figure 28. Plan View (Drawn by Freddy Cerezo)

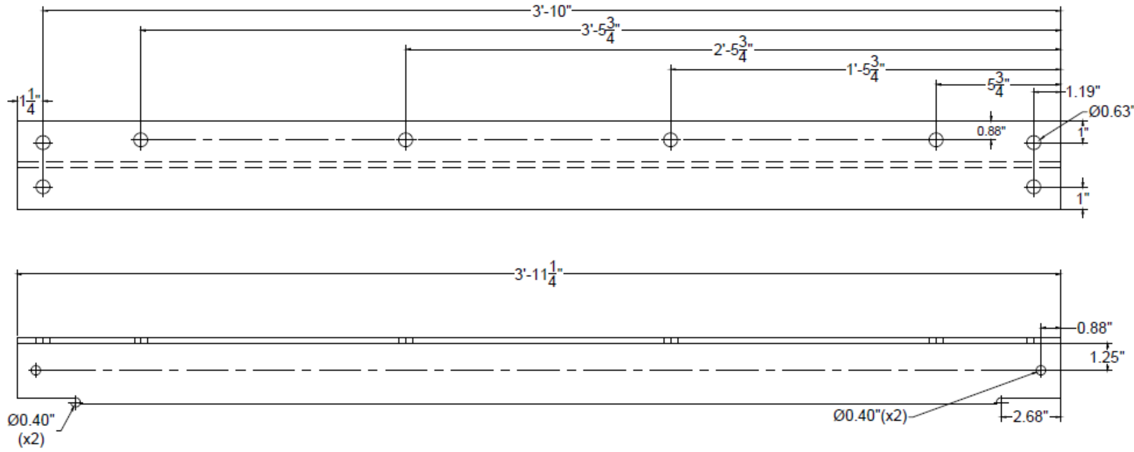

Figure 29. WT 3X6 Braced Frame Beam (Drawn by Freddy Cerezo)

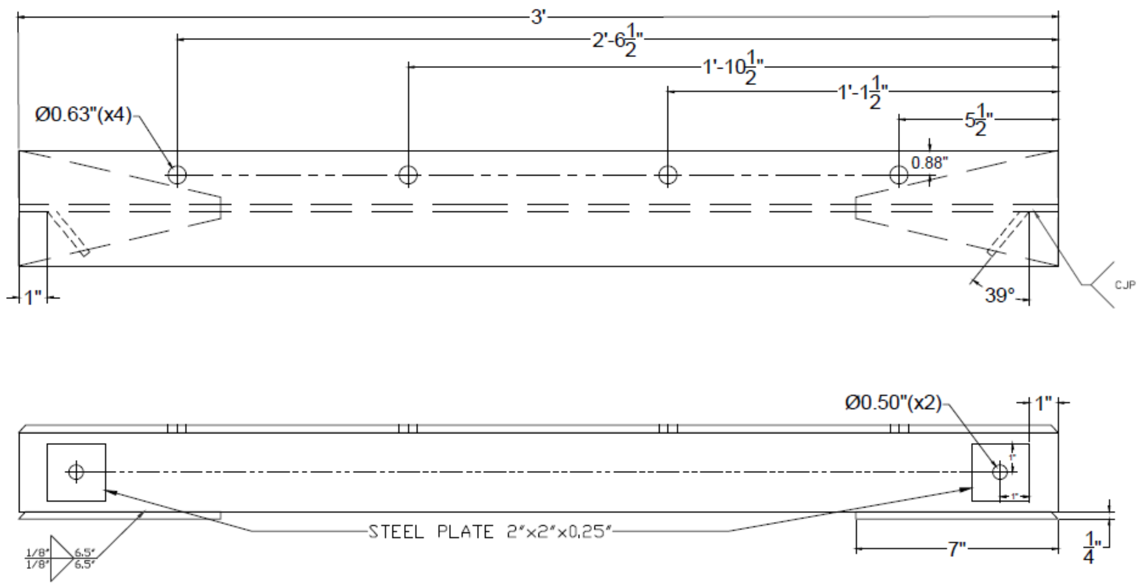

Figure 30. WT 3X6 Moment Frame Beam (Drawn by Freddy Cerezo)

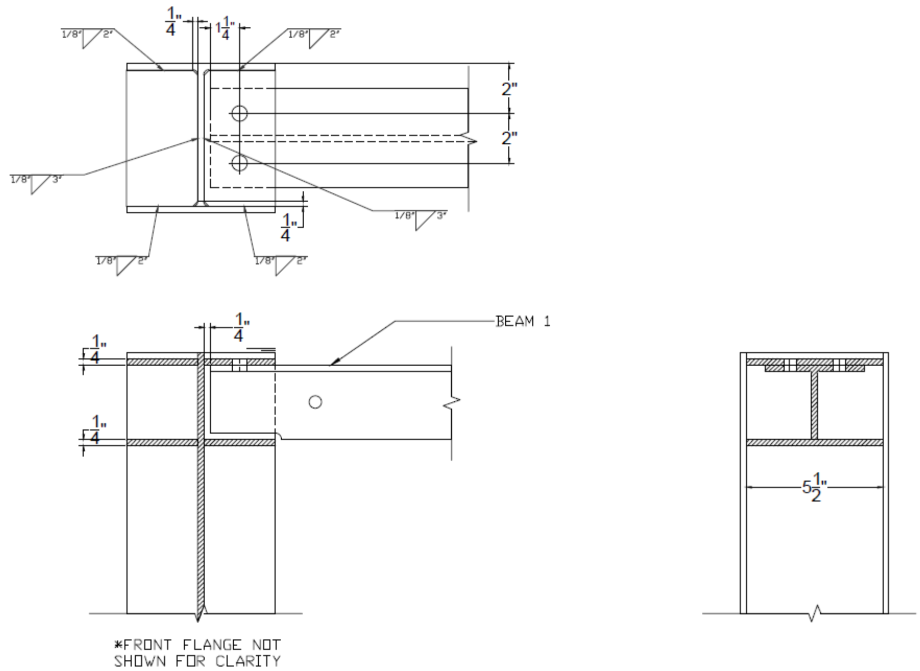

Figure 31. Column Top Connection Showing Bolted Connection for Braced Frame Beam and Continuity Plates for Moment Prame (Drawn by Freddy Cerezo)

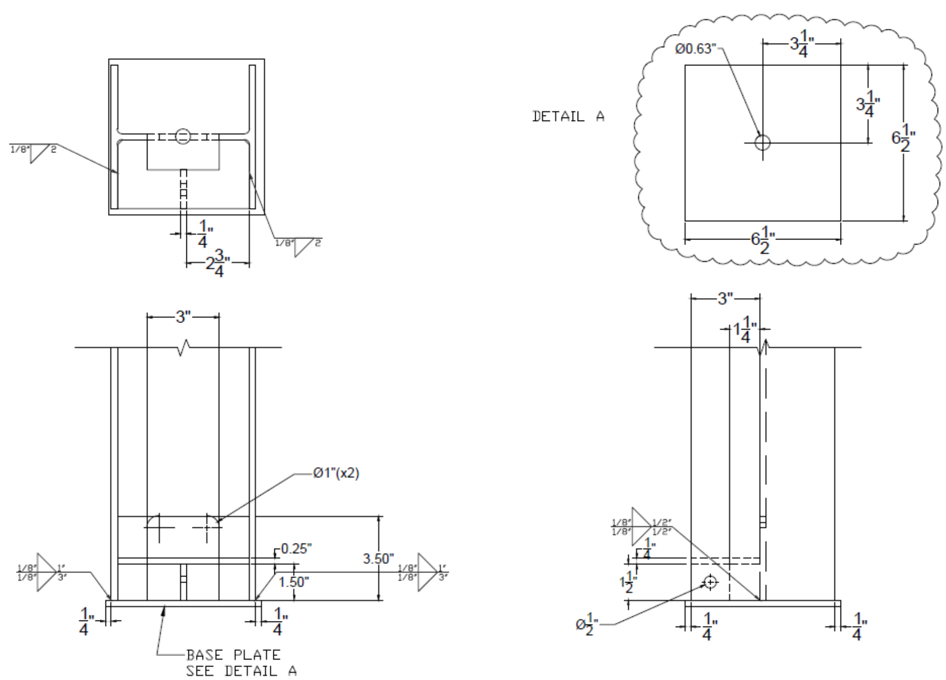

Figure 32. Column Base Plate, Gusset Plate and Stiffener Plate (Drawn by Freddy Cerezo)

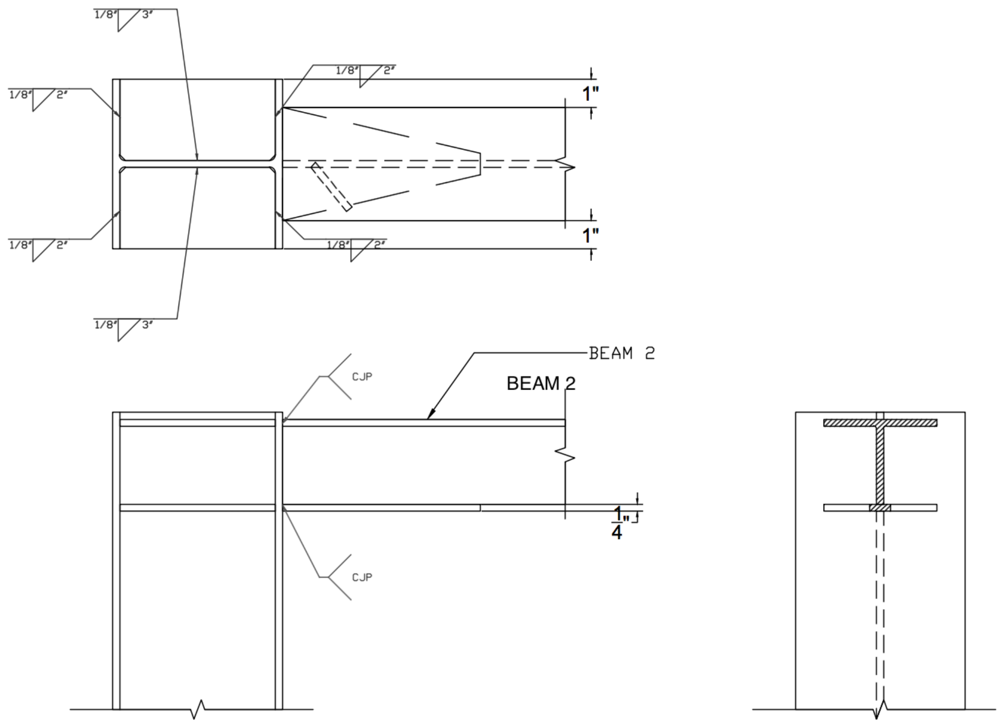

Figure 33. Moment Frame Connection (Drawn by Freddy Cerezo)

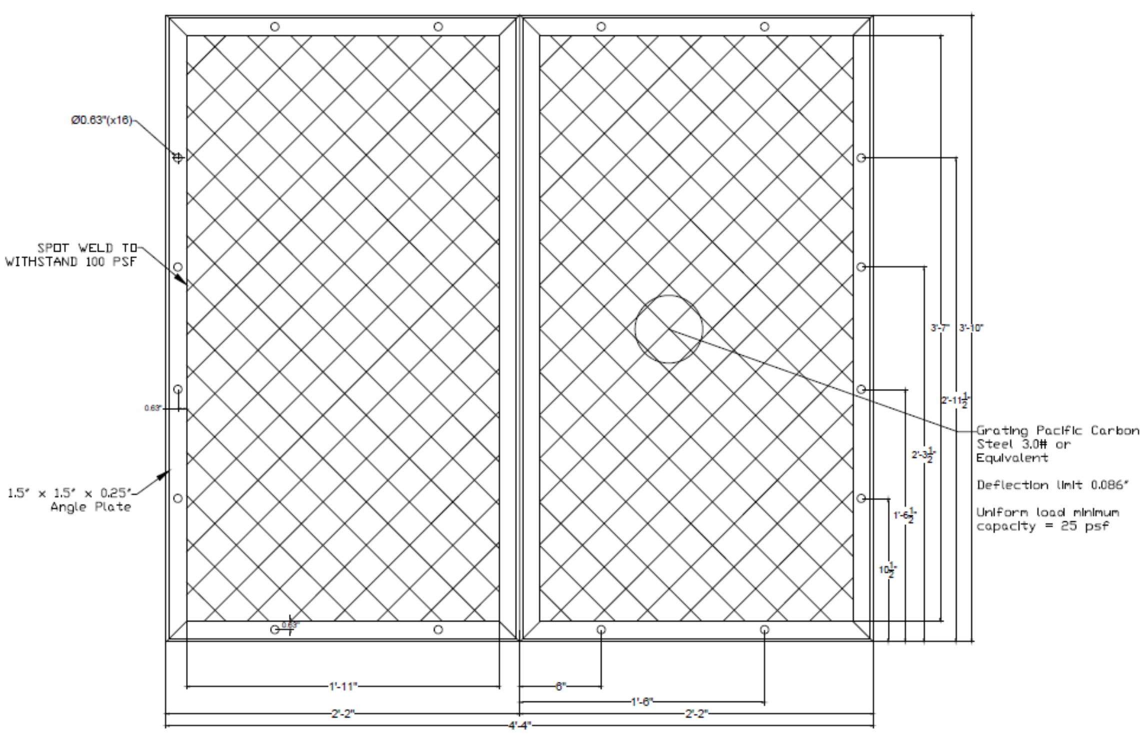

Figure 34. Loading Basket (Drawn by Freddy Cerezo)

Section 7: Moment Frame

Section 7.1 Beam (WT 3X6)

The moment frame beam was checked for shear, moment, and lateral torsional buckling.

Section 7.1-1: Shear. The maximum shear value that the structure will see is $\boldsymbol{V_u}$ =1.006 kips The following formula was used in order to design the shear of the beam:

$$V_n = (0.6)F_y A_w C_v \quad \text{(G2-1)}$$

$$\phi = 1.0$$

$$\phi V_n = (1.0)(0.6)(50ksi)(0.693\ in.^2)(1)$$

$$\phi V_n = 20.79\ kips > 1.006\ kips$$

Section 7.1-2: Moment. The maximum moment that the beam experiences is $\mathbf{M_u = 23.9\ kips * in}$. The end of the moment beam has a built up with a higher moment capacity than the WT section. The moment formula used in order to find the beam's capacity was:

$$\emptyset M_n = R_{pc} F_y S_{xc} \quad \text{(F4-1)}$$

$$R_{pc} = 1.0 \text{ (Conservatively taken as 1 for simplicity)}$$

To use the equation above, the section modulus was found for the built up section (Figure 33). The flange thickness of the WT 3X6 beam is 0.28 in. The plate thickness that is attached to the bottom of the beam has a thickness of 0.25 in. The depth of the web between the flanges is 2.74 in. Using this information the centroid of the built up section was found.

$$\bar{Y} = \frac{\bar{y}_{wt} \cdot A_{wt} + \bar{y}_{pl} \cdot A_{pl}}{A_{wt} \cdot A_{pl}}$$

$$\bar{Y} = \frac{2.343\ in.\cdot 1.78\ in.^2 + 0.125 \cdot 1\ in.^2}{1.78\ in.^2 + 1\ in.^2}$$

$$\bar{Y} = 1.545 \text{ in.}$$

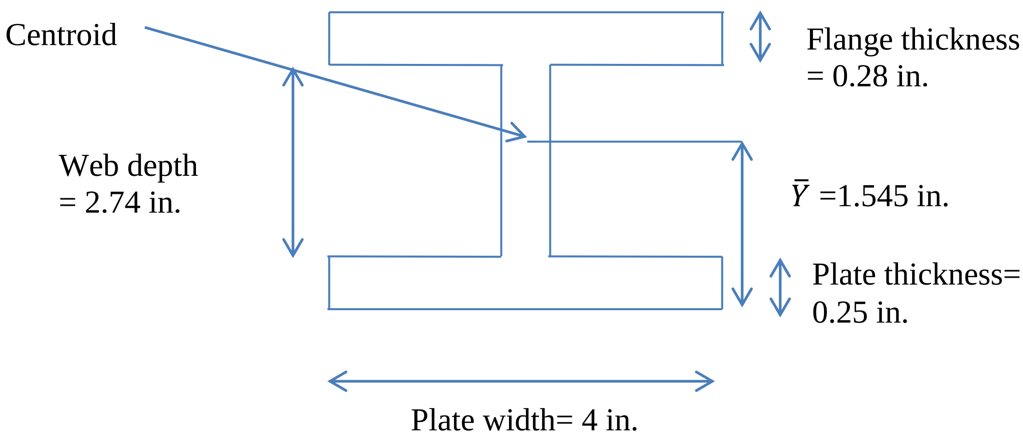

Figure 35. Built up section Dimensions

Next, the moment of inertia was found using the parallel axis theorem. Using the bottom of the section as the reference axis.

$$I_{x(beam)} = I_x + A{d_1}^2$$

$$I_{x(beam)} = (1.32 in.^4 + (1.78\ in.^2)(2.593 \text{ in.} - 1.545 \text{ in.})^2$$

$$I_{x(plate)} = {}^{1}/_{12}\, bh^3 + A{d_2}^2$$

$$I_{x(plate)} = {}^{1}/_{12}\,(4 in.)(0.25\ in.)^3 + (4 \text{ in.})(0.25 \text{ in.})(1.545\ in. - 0.125\ in.)^2$$

$$I_x = I_{x(beam)} + I_{x(plate)}$$

$$= 4.61\ in.^4$$

Next, the section modulus was calculated as follows.

$$S_{xc} = \frac{4.61\ in.^4}{1.725\ in.}$$

$$= 2.67$$

Finally, the moment capacity was calculated as follows.

$$\emptyset M_n = ((1.0)(50ksi)(2.67\ in.^3\ )$$

$$\emptyset M_n = 133.6\ kip * in. > \ M_{end} = 23.9\ kip * in.\ \ \underline{\text{OK}}$$

A moment check was also done on the beam where the end plate terminates. The demand moment value was found by the following calculations, which scales the end moment by similar triangles. This calculation uses the section modulus of the WT section.

$$M_u = M_{end}\frac{L_1}{L/2}$$

L=36 in. $L_1 = L/2 - 7\ in.$ (7 in. plate at bottom of beam)

$$M_u = 14.6\ kip * in.$$

$$M_n = M_p$$

$$M_p = \emptyset F_y S_x \text{ (F4-1)}$$

$$M_p = (0.9)(50ksi)(0.564\ in.^3\ )$$

$$\emptyset M_p = 25.38\ kip * in. > \ 14.6\ kip * in.\ \ \underline{\text{OK}}$$

Section 7.1-3: Lateral Torsional Buckling. The following equation was used in order to check the lateral torsional buckling for the beam:

$$M_n = M_{cr} = \frac{\pi\sqrt{(E)(I_y)(G)(J)}}{L_b} \times (B + \sqrt{1 + B^2}) \quad \text{(F9-4)}$$

$$B = 2.3\left(\frac{d}{L_b}\right) \times \left(\sqrt{\frac{I_y}{J}}\right)$$

d= 3.15 in., $I_y = 1.50\ in.^4$, $J = 0.111in.^4$, $L_b = 36\ \ in.$, $E = 29{,}000\ ksi$, $G = 11{,}200\ ksi$

$$B = 2.3\left(\frac{3.14\ in.}{36\ in.}\right) \times \left(\sqrt{\frac{2.21in.^4}{0.111in.^4}}\right) = 0.89$$

$$M_n = (0.9)\frac{\pi\sqrt{(29{,}000ksi)(1.50in.^4)(11{,}200\ ksi)(0.111in.^4)}}{36\ in.} \times (0.89 + \sqrt{1 + 0.89^2})$$

$$M_n = 1285.9\ kip * in. \text{ (for LTB)}$$

M_n for LTB exceeds the flexural capacity, so flexure governs. Flexure was already checked above, so the section is adequate.

Section 7.2: Columns (W 6X15)

The columns were designed for shear and combined axial force and moment. The connection design considered base plate, column-to-plate welds, plate-to-table bolts and local stiffeners.

Section 7.2-1: Shear. The following calculations describe the design check for shear:

$$\emptyset V_n = \phi 0.6 F_y A_w C_v \quad \text{(G2-1)}$$

$$C_v = 1.0$$

$$\emptyset V_n = (1.0)(0.6)(50\text{ ksi})(1.8452\ in.^2)(1.0)$$

$$\emptyset V_n = 55.35\text{ kips}$$

This value was then compared to the demand found in the analysis.

$$\emptyset V_n = 55.35\text{ kips} > V_u = 498\text{ lbs.} \quad \underline{\text{OK}}$$

Section 7.2-2: Column Combined Loading. The maximum demand for the axial force was 1313 lbs. The following calculations were done to determine the column compression capacity:

$$\emptyset P_n = \emptyset F_{cr} A_g \quad \text{(E3-1)}$$

In order to find the effective length factor (k value), the alignment chart was used. Figure 35 displays the labeling at the column ends:

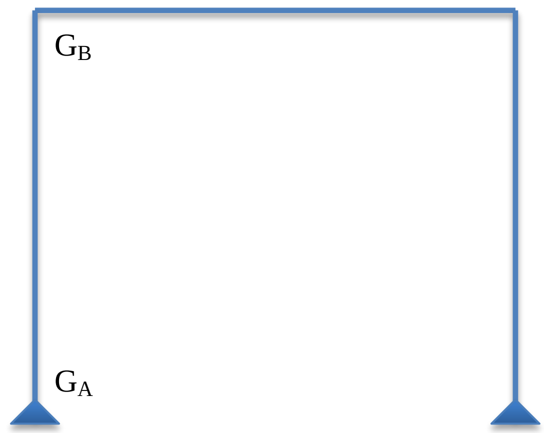

Figure 36. Label of Column ends to find K factor

$$G_B = \frac{\Sigma\left(\frac{EI}{L}\right)_C}{\Sigma\left(\frac{EI}{L}\right)_G} \quad \text{(C-A-7-3)}$$

Where represents C= Column and G= Girder.

$$G_B = \frac{\left(\frac{(29000\ ksi)(29.1 in.^4)}{48\ in.}\right)_C}{\left(\frac{(29000\ ksi)(1.32 in.^4)}{48\ in.}\right)_G}$$

$$= 22.045$$

The pin connection at the base uses $G_A = 10$ as suggested by (Segui 2012). The alignment chart results in k = 3.5. The following equations describe the axial capacity calculation.

$$K = 3.5$$

$$When\ \frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$$

$$\frac{3.5(48\ in.)}{2.56\ in.} \leq 4.71\sqrt{\frac{29{,}000\ ksi}{50\ ksi}}$$

$$65.625 \leq 234.08$$

$$\text{Then } F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$F_e = 66.45\ ksi$$

$$F_{cr} = (0.658^{\frac{50}{66.45}})(50\ \text{ksi})$$

$$F_{cr} = 36.5\ ksi$$

$$\emptyset P_n = \emptyset F_{cr} A_g$$

$$\emptyset P_n = (0.9)(36.5\ ksi)(4.43 in.^2)$$

$$\emptyset P_n = 145.5\ kips$$

This value will be used in combined loading equations. The next check was the moment capacity.

$$M_n = F_y S_y \quad \text{(F6-1)}$$

$$\emptyset M_n = (0.9)(50\ ksi)(3.11\ in.^3)$$

$$\emptyset M_n = 140\ kip\ in.$$

The calculated axial force and the bending moment capacities were then checked in combined loading.

$$\frac{P_u}{2P_n} + \frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} < 0.2 \quad \text{(C-H1-5b)}$$

$$\frac{P_u}{P_n} = \frac{1.313\ \text{kips}}{145.5\ \text{kips}}$$

$$=0.009$$

$$\frac{1.313}{2(145.5)} + \frac{23.9}{140}$$

$$=0.175 < 1 \quad \underline{\text{OK}}$$

Section 7.3: Beam-Column Connection

The beam-to-column connection will be connected with a complete joint penetration weld. A bottom plate will be added to the T-beam to allow a standard I-beam moment connection. The fillet weld connecting the plate to the WT must develop the strength of this plate. The thickness of the plate will be 0.25 in. in order to stay consistent with the other plate thicknesses. The width of the plate is also 4 in. The following calculation was done in order to design the weld length for the bottom plate.

$$\emptyset F_u \cdot t \cdot b \leq \emptyset R_n = \emptyset\,[(0.707)\,(w)\,(0.6)\,(F_{exx})]\,(L) \quad (8\text{-}1)$$

$$w = 0.125 \text{ in. (weld size)}$$

$$(58 \text{ ksi})\,(0.25 \text{ in.})\,(4.03 \text{ in.}) = \emptyset\,[(0.707)\,(0.125 \text{ in.})\,(0.6)\,(70 \text{ ksi})]\,(L)$$

$$\frac{(0.75)(58 \text{ ksi})\,(0.25 \text{ in.})\,(4.03 \text{ in.})}{[(0.9)(0.707)\,(0.125 \text{ in.})\,(0.6)\,(70 \text{ ksi})]} = L$$

Divide by 2 for both sides

$$L = \frac{13.11\ in.}{2} = 6.5 \text{ in.}$$

The weld length gives the minimum length needed for the bottom plate. The length chosen in the design was 7 in. A continuation plate will be provided in the joint region.

Section 7.4: Column Base Connection

Section 7.4-1: Base Plate. The next parameter that was designed is the base plate, which is welded to the column of the structure. The following equation was used (Segui 2012):

$$t_{min} = L\sqrt{\frac{2\ P_u}{(0.9)(F_y)(B)(N)}} \quad \text{(14-7a)}$$

B=N

$$t_{min} = (value\ in\ between\ web\ and\ flange\ thickness) \approx 0.25\ in.$$

L represents one of the dimensions of the base plate as follows:

$$\text{L}= m = \frac{N-0.95d}{2} \quad (14-2) or\ n = \frac{B-0.8b_f}{2} \quad (14-3)$$

$$d = 6\ in.$$

N and B are equal because a square base plate was chosen. This will then be substituted for L in the initial equation:

$$t_{min} = \left(\frac{B-0.8b_f}{2}\right)\sqrt{\frac{2\ P_u}{(0.9)(F_y)(B^2)}}$$

After manipulating and moving all of the unknowns to one side the following equation were found:

$$\frac{B^2}{\left(\frac{B-0.8b_f}{2}\right)^2} = \frac{2P_u}{(0.9)(F_y)(t^2)}$$

$$P_u = 1.313\ kips$$

$$P = 1.313\ kips$$

$$\frac{B^2(0.9)(F_y)(t^2)}{2\left(\frac{B-0.8b_f}{2}\right)^2} = 1.313\ kips$$

$$B = 1.74 \text{ in.}$$

The equation gave a very small-required base plate dimension. Any size that is larger than this value will satisfy the design. The final dimensions were based on weld edge distance. The minimum edge distance is 0.125 in. for weld size 0.125 in. The flange width is 6 in. and a 6.5 in. by 6.5 in. base plate was used to satisfy the minimum edge distance.

Section 7.4-2: Bolt Connection. The base plate is connected by a single A325 bolt at the column centerline and was designed for using the following equation for shear.

$$F_{nv} = 54 \text{ ksi}$$

$$R_n = F_{nv} (A_b) \quad \text{(J3-1)}$$

$$498 \text{ lbs.} = (54\text{ksi}) \left(\frac{\pi d^2}{4}\right)$$

$$d = 0.108 \text{ in.}$$

$$\text{Use } d = 0.50 \text{ in.}$$

(Minimum edge distance = 0.75 in.)

Since the bolt diameter required is much smaller than the minimum bolt diameter that is found in the AISC manual, the minimum value was used. There is no uplift that must be resisted by the bolt.

Section 7.4-3: Reduced Section at Base. Since the bolt is on the column centerline, there must be a cutout in the column web. Figure 37 shows one half of the reduced cross section.

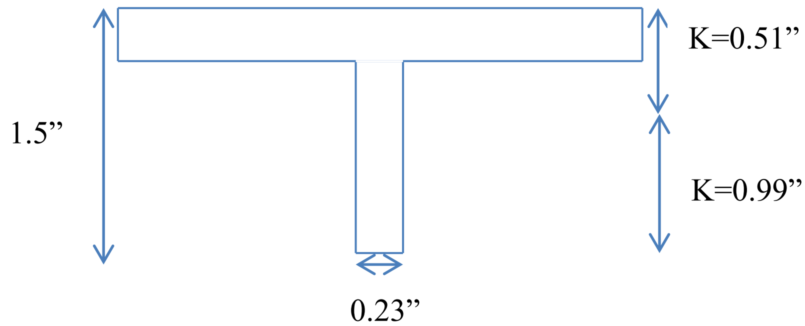

Figure 37. Base of Cutout Column Dimensions

The following check shows that transverse stiffeners would be are not required for the column.

$$\frac{h}{t_w} = \frac{0.99\ in.}{0.23\ in.} = 4.30$$

$$\frac{h}{t_w} < 2.46\sqrt{\frac{E}{F_y}} = 69.82$$

The shear capacity check for the reduced web follows:

$$\emptyset V_n = 0.6 F_y A_w C_v \quad \text{(G2-1)}$$

$$= (0.6)(50\ ksi)(1.5\ in. \cdot t_w)(1.0)(2)$$

$$= 10.35\ \text{kips} \cdot 2$$

$$\emptyset V_n = (0.9)\ (10.35 \cdot 2)$$

$$= 18.6\ \text{kips} > 498\ \text{lbs.} \quad \underline{\text{OK}}$$

The stiffener in the Figure 32 is not required for shear but is included to stabilize the gusset plate and help transfer forces. In addition the cutout in the column will be rounded at the corners in order to decrease the built up stress. The dimensions of the opening are 3 in. wide and 3.5 in. high to provide space to fit a socket wrench.

Section 8: Braced Frame

Section 8.1: Beam and Column

The beam in the braced frame only has nominal forces. For the column, the moment frame forces govern the design.

Section 8.2: Brace/Cable

The braced frame was also designed and the first step for the design is to check the correct cable diameter. It is not yet known what kind of tension element will be used. This check should be considered preliminary. It uses a 0.25 inch diameter A36 member.

$$\emptyset AF_y = (0.9)\,(0.049 in.^2)(36\text{ksi}) = 1.587 \text{ kips}$$

$$1.587 > 1.325 \text{ kips} \quad \underline{\text{OK}}$$

During the selection of the actual cross section at the time of fabrication the assumptions on effective area and modulus need to be checked with the actual cross section chosen. A wire strand cable will have reduced effective modulus and a specified safe working load. Both the SAP 2000 analysis and the design check must be updated.

Section 8.3: Gusset Plate and holes in beam

The following calculations check welds and block shear for all gusset plates and block shear for the holes in the web of the T-beam. The following calculations were done for the fillet weld design:

$$\emptyset R_n = \emptyset[(0.707)(w)(0.6)(Fe_{xx})](L) \quad (8\text{-}1)$$

$$[(0.707)(w)(0.6)(70ksi)](2 \text{ in.}) > 1.325 \text{ kips}$$

$$w > 0.0223 \text{ in.}$$

$$\text{Minimum Weld} = \frac{1}{8} \text{ in.} = 0.125 \text{ in.}$$

$$\text{Minimum Weld Edge Distance} = 0.125 \text{ in.}$$

$$\text{Use } w = 0.125 \text{ in.}$$

The thickness and length of the weld found was 0.0233 in. and 2 in. A minimum weld thickness of 0.125 in. will be used. Since the capacity of the 2 in. weld with a thickness of 0.125 in. is much higher than the forces the frame will see, any length with the same weld thickness is adequate. A 3 in. by 3 in. gusset plate with a thickness of 0.25 in will be used to stay consistent with other structural components. To connect the cable to the beam, a cutout in the beam will be made. A block shear check will not be done on the beam because it has a higher capacity than the gusset plate. The following calculations were done to check the block shear:

$$R_n = (0.6)(F_u)(A_{nv}) + (U_{bs})(F_y)(A_{nt}) \leq (0.6)(F_y)(A_{gv}) + (U_{bs})(F_u)(A_{nt}) \quad (J4\text{-}5)$$

$$T = \text{thickness of plate} = 0.25 \text{ in.}$$

$$A_{nv} = (3 \text{ in.})(0.25 \text{ in.}) = 0.75\ in.^2 \quad A_{nt} = (3 \text{ in.})(0.25 \text{ in.}) - (0.375 \text{ in.})(0.25 \text{in.}) = 0.65\ in.^2$$

$$A_{gv} = 0.65\ in.^2$$

$$R_n=(0.6)(58\text{ksi})(0.75\ in.^2) + (1)(36\text{ksi})(0.65\ in.^2) \leq (0.6)(36\text{ksi})(0.65\ in.^2)$$

$$+(1)(58\text{ksi})(0.65\ in.^2)$$

$$R_n= 49.5\ kips \ \leq 51.74\ kips$$

Section 8.4: Beam-to-Column connection (Braced Frame)

The beams are suspended from the column continuity plates using two bolts. The following calculations were done in order to find the plate thickness based on bearing behind the bolt. The beam carries no vertical (transverse) loads except the weight of the beam. This is because the basket carrying superimposed weight rests only on the moment frame. The connection will be designed to carry the full tension in the cable.

$$\phi R_n = \phi(A_g)(F_y) \text{ (J4-1)}$$

F_y= 36 ksi, A_g=(Plate Thickness)(Diameter of Bolt)

1.325 lbs.= (0.375 in.)(Plate Thickness)(36 ksi)

t= 0.098 in.

Use Plate Thickness of 0.25 in.

The plate thickness that was found was 0.098 in. A practical plate thickness that will be used is 0.25 in. The minimum weld thickness that was used is 0.125 in and a 7 in. weld to connect the plate to the beam.

The bolt size that will be used to connect the plate to the beam will also be designed to meet the demand value of the structure. The following calculations were done in order to find the correct bolt diameters:

Use A 325 Bolts

F_{nt}=90 ksi

F_{nv} =54 ksi

Tension

$R_n = (F_{nt})(A_b)$ (J3-1)

$$1.325 \text{ lbs.} = (90 \text{ ksi})\left(\frac{\pi(d^2)}{4}\right)$$

d= 0.137 in.

Shear

$R_n = (F_{nv})(A_b)$ (J3-1)

$$1.325 \text{ lbs.} = (54 \text{ ksi})\left(\frac{\pi(d^2)}{4}\right)$$

d= 0.177 in.

Use d= 0.50 in. for diameter

Minimum spacing = 3(d) = 3(0.50 in.) =1.5 in.

Minimum edge distance = 0.75 in.

After doing the calculations for the bolt design the minimum bolt diameter that needed to be used for the design is 0.177 in. A 0.5 in. diameter bolt is selected.

Section 9: Diaphragm

Section 9.1: Cross Cables

Diaphragm rigidity is provided by cross-bracing with cables connected to the moment beams. The maximum force that the top cables see is 1.24 lbs., which are orders of magnitude less than the design force for the bracing cables. For simplicity, the same cables are used. The cables will be attached with a 2 in. x 2 in. x 0.25 in. angled gusset plate.

Section 9.2: Loading Basket

The basket will hold the weight added to the structure. This element will be attached to the top of the structure and is only attached to the moment frame. The basket frame will be built of angles and a wire mesh will support the weight. Two such baskets that will be bolted together side by side. Figure 34 displays the basket design.

The wire mesh will be spot welded to the frame to withstand a minimum of 100 psf. The fabricator will design the spot welds. The next part of the basket that needs to be designed is the frame built up of angle members. An allowable stress approach was used to design the frame. The angle carries load according to its tributary width. This load is divided by the member length to obtain a uniform load for design.

$$w = \frac{\left(\frac{744\ lbs.}{2}\right)}{4\ ft.} = 93\ {}^{lbs.}/_{ft.}$$

The flexure equation is used to select a cross section with the appropriate C and I.

$$M = \frac{wl^2}{8} = \frac{(93\frac{lb}{ft})(4ft)(\frac{12\ in.}{ft.})}{8} = 0.558 \text{ kip-in}$$

$$\sigma_{demand} = \frac{M\ C}{I}$$

For an angle measuring 1.5” x 1.5” x 0.25”, the following centroid and moment of inertia are found. A plate will be bent to form the angles; A36 steel is assumed.

Area	y-bar	Area*y-bar
0.375 in.^{2}	0.75 in.	0.28125 in.^{3}
0.3125 in.^{2}	0.325 in.	0.0390625 in.^{3}

Y-bar = 0.4659

C =1.5-0.4659

C = 0.8125 in.

$$I = 2[\frac{1}{12}(0.25 \text{ in.})(1.5 \text{ in.})^3 + (0.375 \text{ in}^2)(0.2841 \text{ in.})^2 + \frac{1}{12}(1.25 \text{ in.})(0.25 \text{ in.})^3 +$$

$$(0.3125 \text{ in.}^2)(0.3409 \text{ in.})^2] = 0.276 \text{ in.}^4$$

The moment of inertia is doubles because it is a double angle plate. The stress is shown to be adequate.

$$\sigma_{demand} = \frac{(0.558\ kip{-}in)(0.8125\ in)}{(0.138\ in^4)}$$

$$\sigma_{demand} = 3.29 \text{ ksi} > 36 \text{ ksi}$$

The bolts connecting the basket to the moment frame will also be based on a lateral acceleration at the roof of 2g. This is an arbitrary value chosen as a design specification. Each individual basket portion will take half of the load. The following calculations were used in order to find the bolt diameter and number of bolt needed for the connection. Mass is the weight divided by g; the design force is mass multiplied by 2g.

$$w = \frac{0.744\ kips}{2} = 0.372\ kips$$

$$\text{Force} = \frac{w}{\cancel{g}}(2\ \cancel{g})$$

$$P = \text{Force} = (0.372 \text{ kips})(2) = 0.744 \text{ kips}$$

The following is the capacity of a single bolt in shear.

$$F_{nv} = 54 \text{ ksi}$$

$$R_n = F_{nv}(A_b) \text{ (J3-1)}$$

$$0.744 \text{ kips} = (54\text{ksi})\left(\frac{\pi d^2}{4}\right)$$

$$d = 0.132 \text{ in.}$$

Use d= 0.50 in. for diameter

Minimum spacing = 3(d)= 3(0.50 in.) =1.5 in.

The total capacity of the bolt group is provided by eight bolts.

$$\emptyset N \cdot R_n = (0.9)(8)(54\text{ksi})\left(\frac{\pi(0.5)^2}{4}\right)$$

$$= 76.32\ kips$$

Comparing this capacity to the design weight of 0.744 kips shows that the bolts can support an acceleration at the basket far in excess pf 2g.

CHAPTER 4

Conclusion

Summary

A structure was designed for use in characterizing the behavior of a new shake table. The structure was designed to achieve a period as close as practically possible to 0.1 seconds. A preliminary analysis was performed to get trial member sizes. An analysis model was then created using SAP 2000. The structure was subjected to two earthquake records: El Centro and Northridge earthquake. The results of the analysis gave the demand values for each element and the theoretical roof acceleration values. The demand values were used to design all members and connections for the frame.

Possible Sources of Error

Some aspects of the analysis and design details may produce discrepancies between predictions and experimental results. Additional weight will be added with sandbags and held down using tie downs. There may also be play between the basket and the frame. The finite rigidity of this arrangement may induce additional dynamic effects.

The assumed value of damping (2%) may be incorrect, and the restrictions imposed by Rayleigh damping may further bias results. The effect of the connection flexibility on the stiffness will also become a potential error

Recommended Future Work

A possible area for future work concerns the cable cross-braces. The actual cross section of the tension brace has not yet been designed. It could be a solid bar or a cable. The chosen cross section will modify assumed modulus and strength values. The analysis and design will have to be updated. Filiatrault and Tremblay (1998) show that when slender braces move through an elastic buckling state, there is a moment where there is minimal amount of load before moving to elastic tensile loading, and eventually tensile yielding. The tensile force during this progression generates a damaging amount of impact loading on the connections of the structure. This, tensile only braces are not allowed in seismic applications. This structure may provide a platform to understand this bracing system further (see e.g., the work by Mousavi et al., 2014)

REFERENCES

AISC (American Institute of Steel Construction). (2011). Steel Construction Manual 14th Ed. Reston, Virginia

Airouche, A. H., Casarotti, C., Thoen, B. K., Dacarro, F., Pavese, A. (2008). “Numerical Modeling and Experimental Identification of the Eucentre Trees Lab Shake Table” 14th World Conference on Earthquake Engineering

ASCE (American Society of Civil Engineers). (2013) Minimum Design Loads for Buildings and Other Structures 3rd Ed. Reston, Virginia

Chopra, A. (2012). Dynamics of Structures 4th. Ed. Upper Saddle River, New Jersey

Conte, J. P., Trombetti, T. L. (2000). “Linear dynamic modeling of a uni-axial servo-hydraulic shaking table system” Earthquake Engineering and Structural Dynamics., 29, 1375-1404. http://dx.doi.org/10.1002/1096-9845(200009)29:9<1375::AID-EQE975>3.0.CO;2-3

CSI (2015) Analysis Reference Manual For SAP2000, ETABS, SAFE and CSiBridge Version 17. Computers & Structures, Inc, Berkeley

Filiatrault, A., Tremblay, R. (1998) “Design of Tension-Only Concentrically Braced Steel Frames for seismic induced impact loading.” Design of Tension-Only Concentrically Braced Steel Frames., 12(20), 1087-1096. https://doi.org/10.1016/S0141-0296(97)00205-8

Hibbler, R. (2012). Structural Analysis 8th. Ed. Upper Saddle River, New Jersey

Luco, J. E., Ozcelik, O., Conte, J. P. (2009). “Acceleration Tracking Performance of the UCSD-NEES Shake Table” Journal of Structural Engineering., 136(5), 481-490. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000137

Mousavi, S. A., Zahrai, S. M., Saatcioglu, M. (2014). “Toward buckling free tension-only braces using slack free connections” Journal of Constructional Steel Research., 115, 329-345. http://dx.doi.org/10.1016/j.jcsr.2015.08.048

Ozcelik, O. (2008). “A Mechanics-based Virtual Model of NEES-UCSD Shake Table: Theoretical Development and Experimental Validation,” PhD thesis University of San Diego, CA

Ozcelik, O., Luco, J. E., Conte, J. P. (2008a). “Identification of the Mechanical Subsystem of the NEES-UCSD Shake Table by a Least-Squares Approach”Journal of Engineering Mechanics., 1(23), 23-34. https://doi.org/10.1061/(ASCE)0733-9399(2008)134:1(23)

Ozcelik, O., Luco, J. E., Conte, J. P., Trombetti, T. L., Restrepo, J. I. (2008b). “Dynamics Experimental characterization, modeling and identification of the NEES-UCSD shake table mechanical system” Earthquake Engineering and Structural., 37, 243-264. https://doi.org/10.1002/eqe.754

Segui, W. (2012), Steel Design 5th Ed. Stamford, Connecticut

Thoen, B. K., Laplace, P. N. (2004). “Offline Tuning of Shaking Tables” 13th World Conference on Earthquake Engineering

Tremblay, R., Filiatrault, A. (1996) “Seismic Impact Loading In Inelastic Tension-Only Concentrically Braced Steel Frames: Myth or Reality?” Earthquake Engineering and Structural Dynamics., 25, 1373-1389. https://doi.org/10.1002/(SICI)1096-9845(199612)25:12<1373::AID-EQE615>3.0.CO;2-Y