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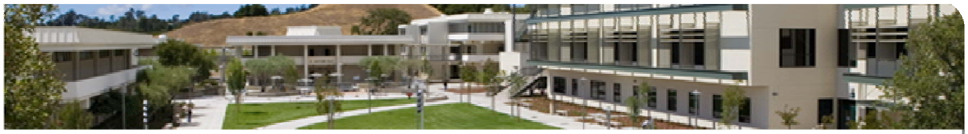

**Creating Opportunities for Minorities in Engineering, Technology, and Science (COMETS)**

# Seismic Evaluation & Design: Special Moment-Resisting Frame Structure

San Francisco State University, Canada College and NASA Sponsored Collaboration

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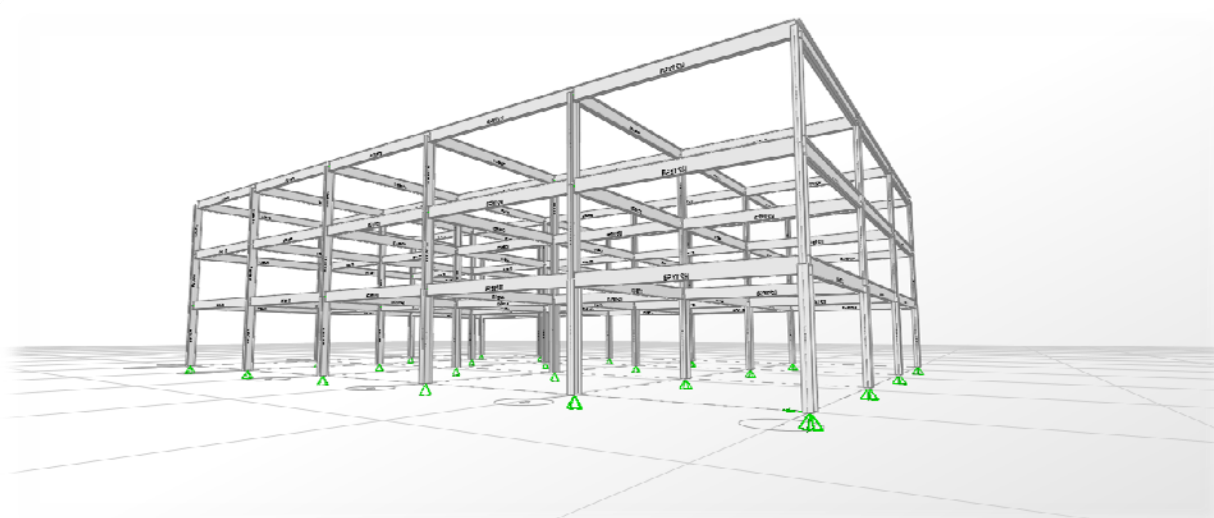

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## Introduction

Civil engineering is considered to be the classical field of engineering. But by the turn of the 20$^{th}$ century new materials and methods of connecting them had become readily available. And so mankind created a need for itself to push for taller, safer, stronger, and cheaper buildings.

For our project, we were tasked with designing a 3-story office building in an earthquake prone area. We incorporated ourselves into the style of conservative thinking that is reflected in the AISC Steel Manual and ASCE 7-05 Minimum Design Loads for Buildings. A requirement was that our structure should consist of special steel moment-resisting frames. And that would incorporate the most viewing area for the interior of the structure but also provide the strength to withstand earthquakes that have historically occurred in the Walnut Creek area.

Accomplishing the design and modeling phase would be a huge step forward for us as we have had no prior design experience. Our plan was to have every member design their own structure. But with everyone contributing their work to the overall report. In this approach, our hope was that for everyone to have gained the most understanding of the concepts. This sort of “do everything” approach stemmed from our initial apprehension of pouring through pages of two steel design textbooks, the ASCE 7-05 design procedure codes and AISC Steel manual design codes. Our thoughts were that if we were to just cover one section and not understand what was occurring with the other section, then we would miss a lot of connections. Until we all knew every part well, we would not be able to design a single building together.

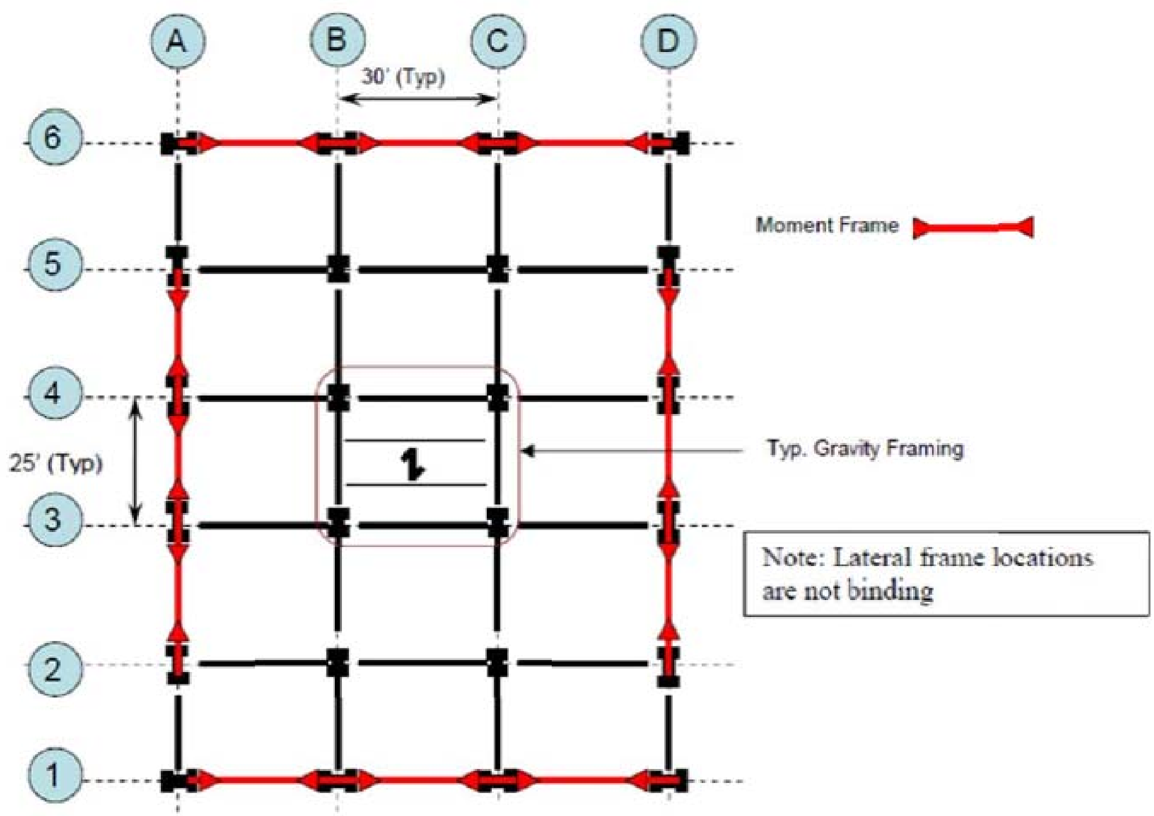
The layout of this report follows a chronological order. We begin by presenting the problem statement, our plan of approach and then a design phase. The design phase consisted of determining the beams and columns buckling requirements. Then actually testing for the load requirements for the beams and then columns. Each of course having different requirements. Lastly we finalize and improve upon our designs based on earthquake loads that we subject onto the structure. This analysis consists of two methods, an Equivalent Lateral Force and a performance based time history.

## Design Challenge

The procedure took throughout this internship has been to follow what Dr. Chen and our graduate student Qi Ming laid out for us. Part of the challenge was that resources were difficult to procure and as so we relied heavily on our graduate student's knowledge. But slowly we got used to the laborious 2100+ pages PDF online reference materials along with manual other PDF reference manuals that we procured.

### Problem Statement

Design a three story building in the Walnut Creek area. The building will be an office building in an earthquake prone area. We were to use the standard 50 [psf] or lbs per square ft as a live load for each floor. The roof, 3rd floor, and 2nd floor were designed to hold 95 [psf], 90 [psf], and 92 [psf] respectively. The square footage for each floor was to be 11250 sq ft. And the columns from the base to the top floor were 13ft, 11ft and 11ft. This building had to be designed according to AISC's code and ASCE's equilateral force procedures. The equilateral force procedure includes equivalent earthquake forces meant to imitate historical earthquake loads. And finally we designed and modeled the structure in SAP2000 repeating the design phase as necessary.



## General Procedure

1. Utilize the Equivalent Lateral Force Procedure in ASCE-07
2. Understand the Steel Design and the Structural Analysis book on beams and columns along with types of connections and block shear.
3. Followed the Equivalent Lateral Force Procedure analysis technique as laid out in the ASCE 7 manual.
4. Designed the beam members for each floor according to AISC codes from the LRFD Steel Design book, in excel.
5. Designed the column members for each floor according to AISC codes from the LRFD Steel Design book, in excel.
6. Modeled the beams and columns along with the base shear, live loads, and dead loads into SAP2000. Ran analysis.
7. Utilize SAP 2000's story deflection by elastic analysis to aid in the story drift determination under the ASCE-07 Equivalent Lateral Force Procedure.
8. Redesigned the building in accordance with ASCE-07 Equivalent Lateral Force Procedures 12.8.6 Story Drift Determination.
9. Asses and analyze four earthquakes to each building through a time history analysis.

## Execution

Being fresh new engineering students we took to any and all advice that our grad student, Qi Ming, instructed. We were recommended to follow the AISC Steel manual. At first we were daunted by how vast and confusing it initially was. But after spending much quality time with the PDF version of the book, we grew to rely on its every steps and requirements. The book aided in our understanding of why and how to analysis structures. These are the results from our work designing each member section. Also an appendix is attached that includes the calculations and constants for each result.

We began with the local buckling check for our beams and columns; this requirement ensures that the section members we pick, from the AISC database, will conform resist certain buckling requirements.

## Local Buckling

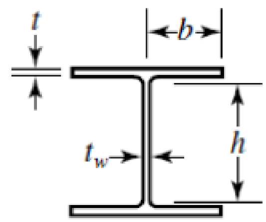
All beams and column members have to pass a web and flange thickness ratio test. This is also referred to as Local Buckling referencing to section B4 Chapter B in the AISC Steel Manual. Classification of sections for buckling is necessary to prevent local buckling.

Chapter B section B4. requires that we check the beams and columns of the members for compact, non-compact, and slenderness. For our purposes we required that the beams and columns be compact and non-slender. This was especially important for our column sections as elements that are too slenderness will cause buckling.

The classification for each section breaks down into two basic elements, stiffened and unstiffened elements. For an I-Beam or W section, the beam contains a central web sandwiched between a top flange and bottom flange. Generally we want compact sections for our beams, this is because compact beams tend to buckle less and so it is a desired trait in beams. But for our columns, because they hold vertical loads, we desire a non-compact shape. Non-compact shapes allow for plastic and elastic buckling behavior and are desired for earthquake resistant frames since the columns will be allowed to sway.

**Figure 4.9** illustrates a typically W section for column sections. We are required to check that the ratio between the flange element's thickness and width conform to the AISC code. It is also necessary for to check the web for the height and web thickness. The web is considered the stiffened element and the unstiffened elements are the top and bottom flanges.

**FIGURE 4.9**




$$b/t \leq 0.56\sqrt{E/F_y}$$
$$h/t_w \leq 1.49\sqrt{E/F_y}$$

## Local Buckling Results

### Columns Flange Check

| Columns | Design Step 1: AISC B4. Classification of Sections for Local Buckling, Flange | Upper limit | Width-Thickness Ratios of Members, Flange | Local Stability Check for Unstiffened Elements, Flange |
|---|---|---|---|---|
| Members | AISC 13th Ed. LRFD Formula | $\lambda_r = 0.56*(E/F_y)^{1/2}$ | $\lambda = b_f/(2*t_f)$ | $\lambda_r = 0.56*(E/F_y)^{1/2} > \lambda = b_f/(2*t_f)$ |
| W18X65 | Roof | 13.49 | 5.06 | Okay |
| W18X71 | 3rd | 13.49 | 4.71 | Okay |
| W18X97 | 2nd | 13.49 | 6.41 | Okay |

### Columns Web Check

| Columns | Design Step 1: AISC B4. Classification of Sections for Local Buckling, Web | Upper limit | Width-Thickness Ratios of Members, Web | Local Stability Check for Stiffened Elements, Web |
|---|---|---|---|---|
| Members | AISC 13th Ed. LRFD Formula | $\lambda_r = 1.49*(E/F_y)^{1/2}$ | $\lambda = h/(t_w)$ | $\lambda_r = 1.49*(E/F_y)^{1/2} > \lambda = h/(t_w)$ |
| W18X65 | Roof | 35.88 | 35.70 | Okay |
| W18X71 | 3rd | 35.88 | 32.40 | Okay |
| W18X97 | 2nd | 35.88 | 30.00 | Okay |

### Beam Flange Check

| Trans Member Check: | Flange Check | Member Properties | Compact Checker | Non Compact Checker | Slender Checker |
|---|---|---|---|---|---|
| Check Flange Overall | Formula | $\lambda = b_f/(2*t_f)$ | $\lambda \leq \lambda_p$ | $\lambda_p < \lambda \leq \lambda_r$ | $\lambda > \lambda_r$ |
| W21X68 | Roof | 6.04 | Yes, Compact | No, Compact | Not Slender |
| W21X68 | 3rd | 6.04 | Yes, Compact | No, Compact | Not Slender |
| W21X68 | 2nd | 6.04 | Yes, Compact | No, Compact | Not Slender |

| Long Member Check: | Flange Check | Member Properties | Compact Checker | Non Compact Checker | Slender Checker |
|---|---|---|---|---|---|
| Check Flange Overall | Formula | $\lambda = b_f/(2*t_f)$ | $\lambda \leq \lambda_p$ | $\lambda_p < \lambda \leq \lambda_r$ | $\lambda > \lambda_r$ |
| W21X55 | Roof | 7.87 | Yes, Compact | No, Compact | Not Slender |
| W21X55 | 3rd | 7.87 | Yes, Compact | No, Compact | Not Slender |

| | | | | | |
|---|---|---|---|---|---|
| W21X55 | 2nd | 7.87 | Yes, Compact | No, Compact | Not Slender |

### Beam Web Check

| Trans Member Check: | Web Check | Member Properties | Compact Checker | Non Compact Checker | Slender Checker |
|---|---|---|---|---|---|
| Check Web Overall | Formula | $\lambda = h/(t_w)$ | $\lambda \leq \lambda_p$ | $\lambda_p < \lambda \leq \lambda_r$ | $\lambda > \lambda_r$ |
| W21X68 | Roof | 43.60 | Yes, Compact | No, Compact | Not Slender |
| W21X68 | 3rd | 43.60 | Yes, Compact | No, Compact | Not Slender |
| W21X68 | 2nd | 43.60 | Yes, Compact | No, Compact | Not Slender |

| Long Member Check: | Web Check | Member Properties | Compact Checker | Non Compact Checker | Slender Checker |
|---|---|---|---|---|---|
| Check Web Overall | Formula | $\lambda = h/(t_w)$ | $\lambda \leq \lambda_p$ | $\lambda_p < \lambda \leq \lambda_r$ | $\lambda > \lambda_r$ |
| W21X55 | Roof | 50.00 | Yes, Compact | No, Compact | Not Slender |
| W21X55 | 3rd | 50.00 | Yes, Compact | No, Compact | Not Slender |
| W21X55 | 2nd | 50.00 | Yes, Compact | No, Compact | Not Slender |

## Local Buckling Conclusion

From the database tables above, we were able to pick each beam and column and they reflect the requirements that were stated above. For our columns, we required them to be less than the upper limit $\lambda_r$ meaning that our column members were non-compact. This is good news as we will see later that non-compact columns tend to allow for elastic and plastic buckling behavior. If they were totally inelastic they would never pass in an earthquake as the columns would be unable to reform their original shape.

For beams we were able to achieve compactness in the flange and web sections. This allows for minimal buckling of our beams. This is good news as for beams; they require that the floors have minimal buckling.  If they were not then, for example someone brings in heavy equipment or there is a large gathering of people, the floor would noticeable buckle!  So a compact beam is the most favorable section for our beam selections.

## Design of Beams and Requirements

For the beam's design, we had two types per floor.  One type is a 30 ft long in the transverse direction of the building and the next is a 25 ft long beam in the longitudinal direction.  There are a total of 18 beams in the transverse direction and 20 beams in the longitudinal direction.  And for each floor there was a different dead load requirement along with the live load and the earthquake load, story force, distributed to each floor.

We first started out with AISC Chapter F, F1 and F2.  From section F2.1 we were able to assume that the nominal moment would be equal to the plastic moment.  This assumption is okay because our beams passed this check; $Z_x$ / $S_x$ < 1.5.  This aided us greatly in minimizing our calculation and more importantly, minimized the need for learning newer and more difficult codes in our determination of the nominal moments.

### Beam Nominal Moment Check

The moment required were calculated and included the members' weight, dead loads, live loads, and horizontal earthquake loads.  As you can observe from the tables below, the moments of each member greatly exceeds the required moment.  This is because of story drift factors that are described further along in the text.  But generally the moment is not the biggest concern for beams, the biggest concern is deflection.

| TRANS MEMBER CHECK: | Step 1 & 2, Check for Design Strength | Moment Check [kip-inch] | Check Against [kip-inch] | Check Okay |
|---|---|---|---|---|
| Floor | AISC Steel Design Requirements | $M_{u\ beam}$ | $M_{u\ required}$ | $M_{u\ beam}$ vs $M_{u\ required}$ |
| Roof | W21X68 | 7200 | 2094.741 | Okay |
| 3rd | W21X68 | 7200 | 2032.553 | Okay |
| 2nd | W21X68 | 7200 | 2057.428 | Okay |

| LONG MEMBER | Step 1 & 2, Check for Design Strength | Moment Check [kip-inch] | Check Against [kip-inch] | Check Okay |
|---|---|---|---|---|

| CHECK: | | | | |
|---|---|---|---|---|
| Floor | AISC Steel Design Requirements | $M_{u\ beam}$ | $M_{u\ required}$ | $M_{u\ beam}$ vs $M_{u\ required}$ |
| Roof | W21X55 | 5670 | 1669.090 | Okay |
| 3rd | W21X55 | 5670 | 1617.266 | Okay |
| 2nd | W21X55 | 5670 | 1637.996 | Okay |

## Beam Deflection Check

A beam will deflect no matter what load you place on it. Even the beams own weight adds to the deflections. But this is the most important and determining factor for our decisions in deciding which beam section to select.

| TRANS MEMBER CHECK: | Maximum Permissible Live Load Deflection | Material Property | Formula Constant [inch] | Check |
|---|---|---|---|---|
| Floor | AISC Steel Design Requirements | Δ = ( 5/384)*((wL*L4)/(E*Ix)) | L/360 | $\Delta = (5/384)*((w_L*L^4)/(E*I_x)) < L/360$ |
| Roof | W21X68 | 0.988 | 1 | Okay |
| 3rd | W21X68 | 0.959 | 1 | Okay |
| 2nd | W21X68 | 0.971 | 1 | Okay |

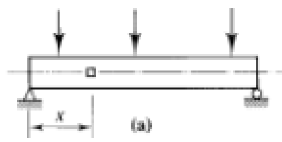
| LONG MEMBER CHECK: | Maximum Permissible Live Load Deflection | Material Property [inch] | Formula Constant [inch] | Check |
|---|---|---|---|---|
| Floor | AISC Steel Design Requirements | Δ = ( 5/384)*((wL*L4)/(E*Ix)) | L/360 | $\Delta = (5/384)*((w_L*L^4)/(E*I_x)) < L/360$ |
| Roof | W21X55 | 0.710 | 0.833 | Okay |
| 3rd | W21X55 | 0.688 | 0.833 | Okay |

| | | | | |
|---|---|---|---|---|
| 2nd | W21X55 | 0.697 | 0.833 | Okay |

## Shear Strength of Beam

The web section of the beam generally has to handle the shear forces from the loads above it. Usually however the shear strength of the beam is much greater than the required shear strength from the provided loads. Below are our results from the shear strength check.

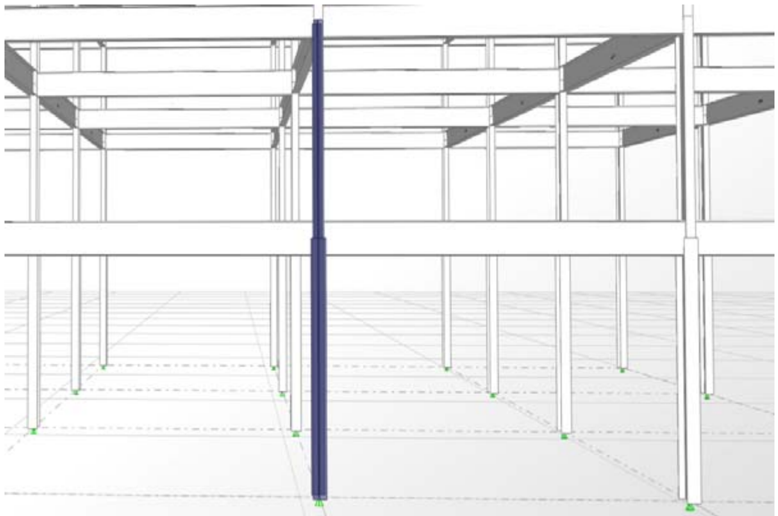
An example of shear is provided:




| TRANS MEMBER CHECK: | Step 3, Check Shear Strength | Maximum Shear | $V_{u\ required}$ Transverse Shear per Member [kips-ft] | Check Shear |
|---|---|---|---|---|
| Floor | AISC Steel Design Requirements | $\Phi_v*V_n$, [kips] | $V_{u\ required} = (1/2)*w_u*L$ | $\Phi_v*V_n > V_u$ |
| Roof | W21X68 | 244.971 | 34.912 | Okay |
| 3rd | W21X68 | 244.971 | 33.876 | Okay |
| 2nd | W21X68 | 244.971 | 34.290 | Okay |

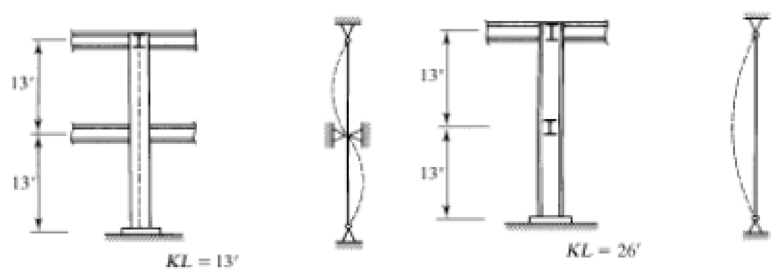
| LONG MEMBER CHECK: | Step 3, Check Shear Strength | Maximum Shear | $V_{u\ required}$ Longitudinal Shear per Member [kips] | Check Shear |
|---|---|---|---|---|
| Floor | AISC Steel Design Requirements | $\Phi_v*V_n$, [kips] | $V_{u\ required} = (1/2)*w_u*L$ | $\Phi_v*V_n > V_u$ |
| Roof | W21X55 | 210.6 | 33.382 | Okay |
| 3rd | W21X55 | 210.6 | 32.345 | Okay |
| 2nd | W21X55 | 210.6 | 32.760 | Okay |

## Design of Column Requirements



While the design of the beams is important for each floor, it is the columns that have to support the weight of the entire building. So it is the columns that we have to pay the most attention to. After checking the local stability above, we now have columns that are able to handle plastic and elastic buckling conditions. Under AISC Chapter E, Design for Compression Members, it states that we must check the effective length and slenderness ratios of our column members. This is important as there is a limit that our column must not exceed in terms of its effective length. Once it exceeds this limit our columns would be greatly susceptible to buckling. Buckling is bad, but we also want to take in consideration high ceilings. Generally clients or the owners would want to include high ceilings and thus high columns as they allow for a more appealing aesthetic feel to the environment.

### Effective Length and Slenderness Ratio Limitations




(a) Minor Axis Buckling (b) Major Axis Buckling

There is an effective length that each column provides. If the effective length is low then there will be some minor axis buckling, as shown in Fig (a) above. But if the effective length is high then there will be some major axis buckling, as shown in Fig (b) above. So it is important to check for this limitation. We also checked our members' slenderness ratio as this pertains to effective lengths. As buckling is a major and real concern all of our chosen members passed the tests.

| Columns | AISC 13th Ed. LRFD Formula | Slenderness Ratio, Chapter E2. | AISC Chapter E Section E2. | Check Status |
|---|---|---|---|---|
| Floors | Chapter E2. Slenderness Limitations and Effective Length | $(KL/r_y)$ | Do not exceed 200 | $(KL/r_y) <$ 200 |
| Roof | W18X65 | 78.11 | 200 | Okay |
| 3rd | W18X71 | 77.65 | 200 | Okay |
| 2nd | W18X97 | 58.87 | 200 | Okay |

## Compressive Strength for Flexural Buckling

We now want to check our column members for their compressive strengths. Since we now know that our columns are non-compact sections, we know that there compressive strengths are governed by inelastic buckling. So we check the $F_{cr}$ according to the code, and we notice that it tells us that we are indeed correct; our members use the inelastic $F_{cr}$.

| Columns | AISC 13th Ed. LRFD Formula | Elastic Columns w/initial crookedness, $F_{cr}$ | Inelastic Columns, $F_{cr}$ | Check $F_{cr}$ |
|---|---|---|---|---|
| Members | Design Step 1, Flexural Buckling Stress , $F_{cr}$ conditions | $F_{cr} = 0.877*F_e$ | $F_{cr} =$ $(0.658\text{^}(F_y/F_e))*F_y$ | $(F_e \geq 0.44*F_y)$ or $(F_e < 0.44*F_y)$ |
| W18X65 | Roof | 41.15 | 32.01 | Use Inelastic, Fcr |
| W18X71 | 3rd | 41.63 | 32.18 | Use Inelastic, Fcr |
| W18X97 | 2nd | 72.43 | 38.81 | Use Inelastic, Fcr |

Once our $F_{cr}$ has been picked, we can now factor that into our nominal compressive strength values, as shown below.

| Columns | Design Step 1 AISC Requirement, $F_{cr\ value}$ | (User picks similar column type if stated below) | $F_{cr}$, Same for similar columns only! | Nominal Compressive Strength |
|---|---|---|---|---|
| Members | AISC 13th Ed. LRFD Formula | Columns Similar | Fcr = (0.658^(Fy/Fe))*Fy | $P_n = A_g*F_{cr}$ |

| | | | | |
|---|---|---|---|---|
| W18X65 | Roof | Inelastic Columns, Fcr | 32.01 | 611.34 |
| W18X71 | 3rd | Inelastic Columns, Fcr | 32.18 | 669.24 |
| W18X97 | 2nd | Inelastic Columns, Fcr | 38.81 | 1106.04 |

| Columns | Design Step 1: AISC E1. General Provisions | Sum of Factored Loads | Design Compressive Strength | Relationship between Load and Strength |
|---|---|---|---|---|
| Members | AISC 13th Ed. LRFD Formula | $P_{u\ required}$ = 1.2*D + 1.6*L | $\Phi_c$*$P_n$ | $P_u \leq \Phi_c$*$P_n$ |
| W18X65 | Roof | 95.04 | 550.20 | Okay |
| W18X71 | 3rd | 188.12 | 602.32 | Okay |
| W18X97 | 2nd | 282.40 | 995.44 | Okay |

And finally we check that our column meets the compressive strength required by our factored loads. Notice that we exceed the required loads by a large factor. This is because the columns determining factor is in the story drift calculation. The story drift includes the story forces and earthquake loads into consideration, and is a much stricter code.

## Analysis Technique

The job of a civil engineer is to ensure that the buildings we create are built to withstand the tests of time and nature. And because of such, it has been proven necessary to perform a number of analysis techniques in our building designs. Such two techniques are time history analysis, a performance based analysis technique, and ASCE 7-05's equivalent lateral force procedure. The latter is a procedure that is designed to mimic real loads caused by earthquakes, while the former is meant to test the building performance against an actual earthquake. Our three-story will be designed according to both methods. Our goal is to determine which method will produce the best results with the most minimal design specifications.

## Structural Design Methods

Safety and Usability are the major things Engineers take into consideration when designing a structure. There are several approaches to designing structures that must withstand seismic, wind, snow, and other loads. One approach is to create models that are good approximates of the actual structure and observe how these models respond to the different loads applied to them. The other approach is the detailed analysis of the structure. Detailed analysis can also have various approaches, such as the Equivalent Lateral Force Procedure and Time History Analysis.

## Equivalent Lateral Force Procedure

The Equivalent Lateral Force Procedure involves applying static forces on the structure and analyzing how it reacts to these forces. Also, the forces are usually applied at the joints of the members, which makes the cross-sectional members act like two-force members. The most important force in this procedure is the base shear, or the sum of all the lateral forces affecting the structure. The strength or capacity of the members must be able to withstand the base shear. To find out if the appropriate members are selected, engineers perform the story drift check. Other factors, such as the seismic response coefficient, response modification factor and important factor must be taken into account when following this procedure. ASCE 7 section 12.8 carefully enumerates the equations and conditions that must be satisfied when following the Equivalent Lateral Force Procedure.

### *Seismic Response Coefficient*

The Seismic Response Coefficient, Cs, is used to determine the base shear of the structure. According to ASCE 7, the base shear can be obtained by multiplying the Response Coefficient by the structure's effective weight. The effective weight of the building includes the dead load and other loads, such as live and snow loads.

### *Response Modification Factor*

When engineers design a structure, they expect the building to sustain permanent damage. Even the best designed buildings are susceptible to inelastic deformation. With this is mind, the goal of the engineer is to design a building that will not collapse. The Response Modification Factor, R, accounts for the ability of the structure to absorb energy without collapsing or its energy dissipation capacity. This modification factor depends on the type of structure being examined. The more ductile the structure is, the higher its modification factor. A ductile structure means it has the ability to change shape under stress before it breaks.

### *Importance Factor*

The Importance Factor, I, depends on the use of the building. It can be determined by referring to ASCE 7 table 11.5-1, which includes different Occupancy Categories for buildings. Facilities such as hospitals and schools have a high Importance Factor. Facilities like storage buildings are assigned with lower a Importance Factor.

### Time History Analysis

It is very important to understand how buildings move before, during, and after an earthquake. Time History graphs allow engineers to study the structure's behavior over a specified amount of time. The main difference between Equivalent Lateral Force Procedure (ELFP) and Time History Analysis (THA) is the type of load used to simulate an earthquake. In ELFP, the base shear is the main load, and the analysis is static. In THA, simulations are done by incorporating real earthquakes recorded in the past.

The first step in performing a Time History Analysis is to decide what accelerogram to use. Accelerograms are graphs that show ground acceleration over a period of time. The Pacific Earthquake Engineering Research conducts the recording of these accelerograms and allows the public to download earthquake data from their website.

These accelerograms are then uploaded onto structural analysis programs, such as SAP2000. The uploaded earthquake must be amplified to mimic the effects of the base shear. Designers apply the earthquake data as a load combination and run the simulation. Checking the story drift is different from the story drift check in static analysis. The highest joint is usually chosen to be examined because the total deflection can be seen there. Most programs have features that let the designer analyze the displacement history of that joint. The largest displacement must not exceed the allowable displacement determined by the building code.

## Analysis Implementation

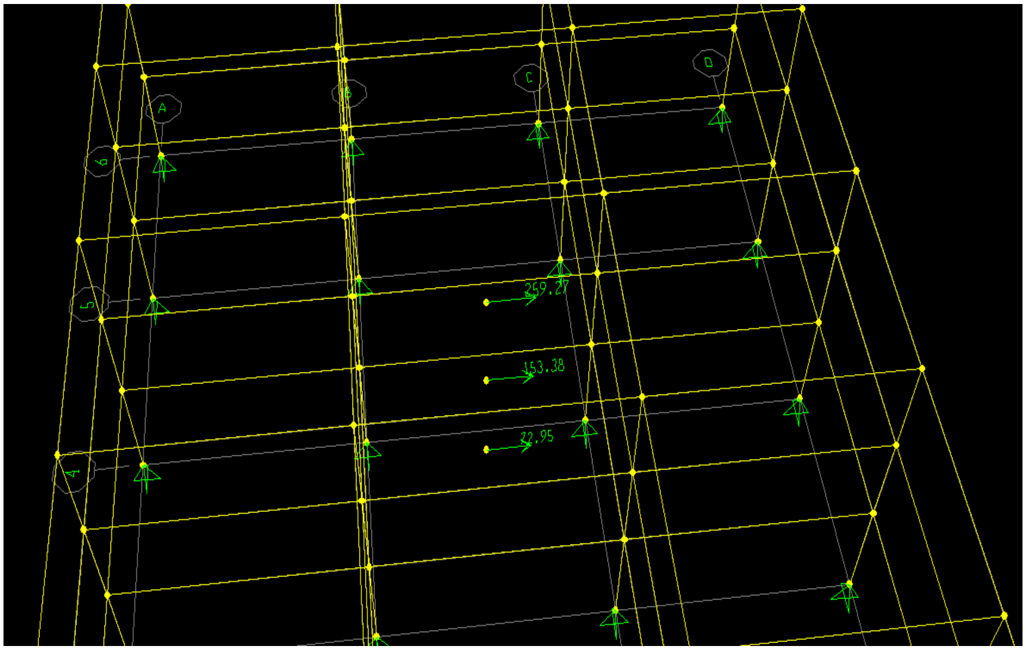
### ASCE 7-05 12.8.1 Equivalent Lateral Force Procedure

The Equivalent Lateral Force Procedure was the last step in our design process. This includes calculating the story forces for each individual level, assigning it in SAP2000 and running the simulation to get our deflection by elasticity test result. Once this result has been obtained, we are able to test if our building will pass ASCE 7-05's requirement for max allowable story drift. Below is a tabulation of our results after it has been run through SAP2000's analysis program.

| Floors | Deflection by Elastic Test U1 ,$\delta_{xe}$ | Deflection of Level x ,$\delta_x$ | Max Allowable Drift $\Delta_a$ | Drift Check, $\delta_2$ - $\delta_1$ | [units] | Check |
|---|---|---|---|---|---|---|
| Roof | 1.940 | 10.669 | 3.3 | 1.576 | [inch] | Okay |
| 3rd | 1.653 | 9.093 | 3.3 | 2.823 | [inch] | Okay |
| 2nd | 1.140 | 6.270 | 3.9 | 1.140 | [inch] | Okay |

From the table above, it is clear that our building has passed the drift check. At the most extreme end, the max allowable drift for the 3rd floor is 3.3 inches. Our building managed a reasonable 2.82 inch drift.

So at our most extreme end we were able to allow up to 85% of allowable drift. This tells me that we did not exceed the requirement and did not over perform the requirement. Thus saving in total weight of our building material and of course costs.




*Above displays the story forces applied at the center of each level for our 3D model in SAP2000.*

## Time History Analysis:

Our goal is to analyze the performance of the ASCE 7-05 procedure with versus time history analysis and model the performance in a 3-story structure. The frame will be composed entirely of special moment-resisting frames. We will first apply the equivalent lateral force procedure, and then pick our beams and columns, run an analysis test, and finally determine if the beams we chose would pass the story drift check; repeat as necessary until the story drift conditions satisfy. The procedure remains the true for the time history analysis. The difference between the two methods is that the ELFP relies on a maximum computed base shear to distribute the lateral forces for all stories. That is ELFP will test the building for the maximum predicted earthquake that ASCE 7 determined as allowable. This process for determining the maximum base shear for our building was developed through research into earthquake code requirements pertaining to certain earthquake prone areas. While the time history analysis method will pit our design element in a performance based test based on actual earthquake data.

How the building drifts per floor is the beam and column restrictive factor for in our designs. Buildings prior to these codes did not drift uniformly and as such, structures of the past would crumble during an earthquake. But a building that has some allowable drift is better able to dissipate the energy from an earthquake throughout to the rest of the structure.

## Results:

In our previous findings, our results have indicated that special moment-resisting frames utilize heavier building materials, but they offer superior expansive views and more flexible esthetic designs. Still they are much more affordable and simpler to implement than base isolated dampening systems.

In our procedures we tested frame system after frame system, each time analyzing the drift between each floor levels, taking note of the changes each time. Our general findings indicated that the 3rd floor beams incurred the largest drifts. Further testing indicated that if we increased the beam size for the 3rd floor beams, we incurred less drift. The 3rd floor system was the determining factor in controlling the amount of drift for our building system. This was true for both lateral and longitudinal directional earthquake forces. If we were not to consider for the ease of construction, it would be possible to just beef up the 3rd floor beams in order to lighten up the columns for the rest of the building. As it were, the columns more than exceeded the required dead and live loads and so our design could benefit from a lighter design.

On one occasion when designing our structure according to ASCE 7-05 we were able to reach less than 1% of the allowable drift. This developed some interesting performance achievements. Notably, our average difference until reaching the max limit was 26.37% for all 4 performance induced earthquakes. Performance wise, this is good news as it is well within ASCE 7 design requirements. And since 3 out of 4 of the earthquakes occurred in California, our designs would save lives. However taking into account the 1995 Kobe earthquake that struck Japan, causing the most damage in terms of lives in this project, we observed rather close story drifts developing. The closest being 6.95% to the max allowable drift. Despite this finding, it is still within the allowable ranges.

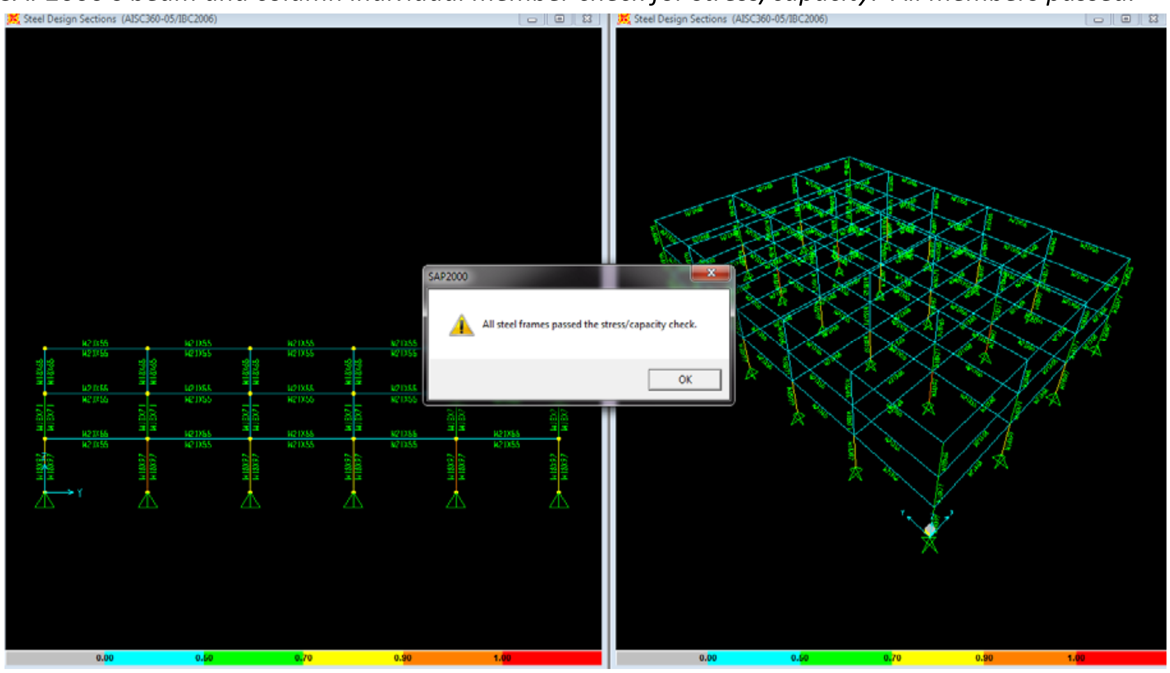
After much calculation and learning of the codes and how to utilize it in our SAP2000 student edition software, we were able to present a 3D model of our design! The beams that we have picked are represented in the tables below. Below them are results from SAP2000's beam and column analysis checks.

| Beam Selection | **Trans Beam Selection** | | **Long Beam Selection** | |
|---|---|---|---|---|
| Floor Levels | Transverse Beam | Status Check | Longitudinal Beam | Status Check |
| Roof | W21X68 | Okay | W21X55 | Okay |
| 3rd | W21X68 | Okay | W21X55 | Okay |

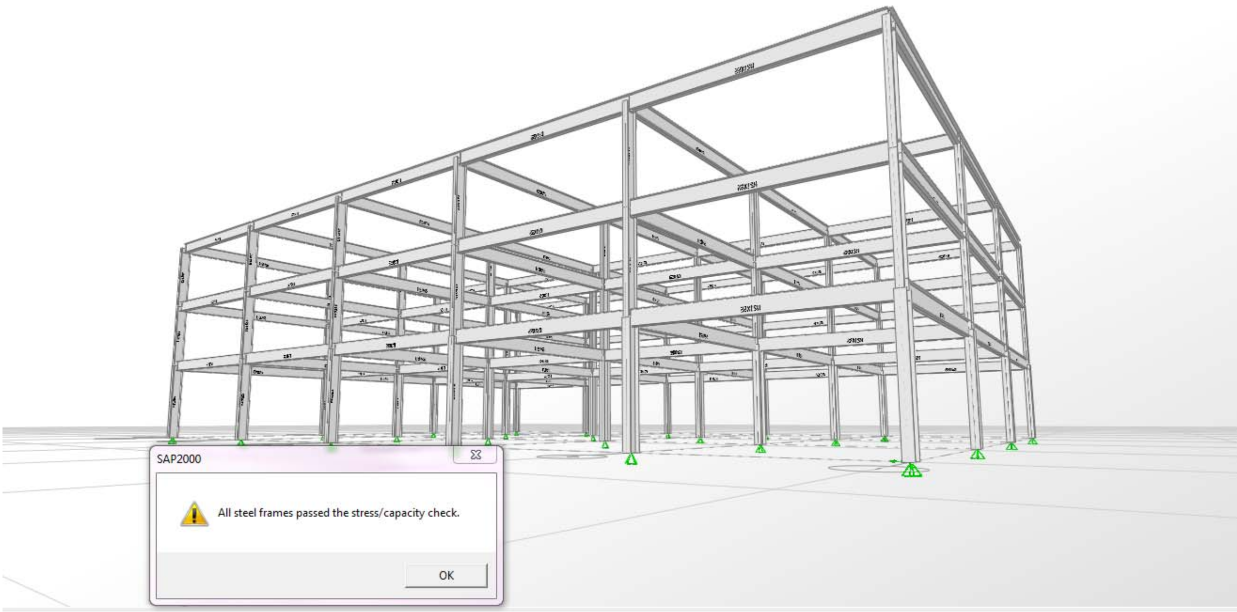
| | | | | |
|---|---|---|---|---|
| 2nd | W21X68 | Okay | W21X55 | Okay |

| Columns | **Column Selection** | |
|---|---|---|
| Floor Levels | Columns | Overall Status Check |
| Roof | W18X65 | Okay |
| 3rd | W18X71 | Okay |
| 2nd | W18X97 | Okay |

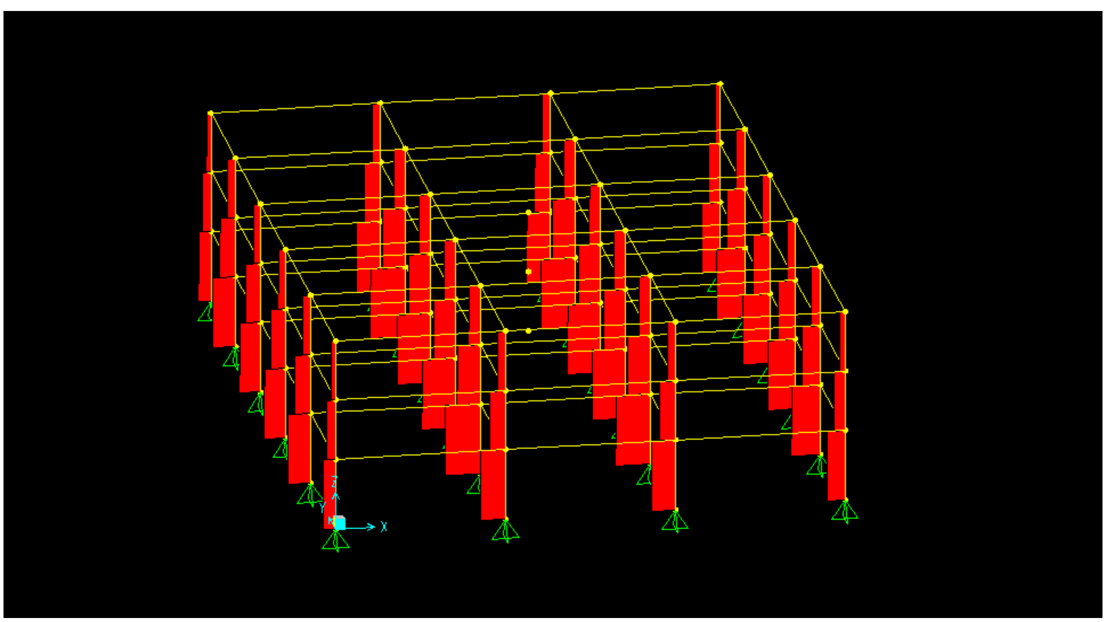
*SAP2000's beam and column individual member check for stress/capacity. All members passed.*



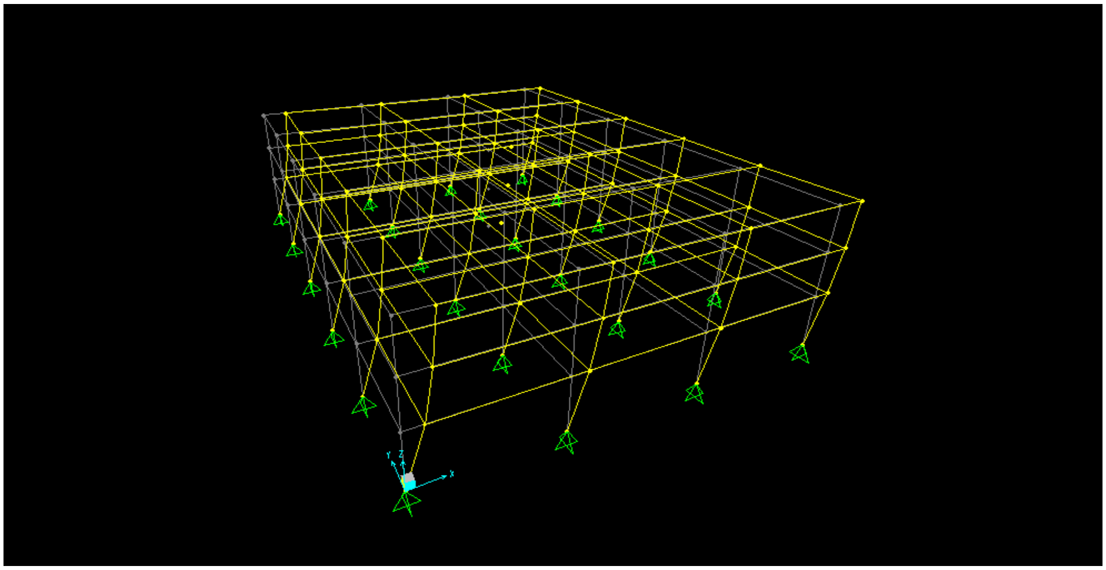
*A higher resolution visual quality check of the building and design.*



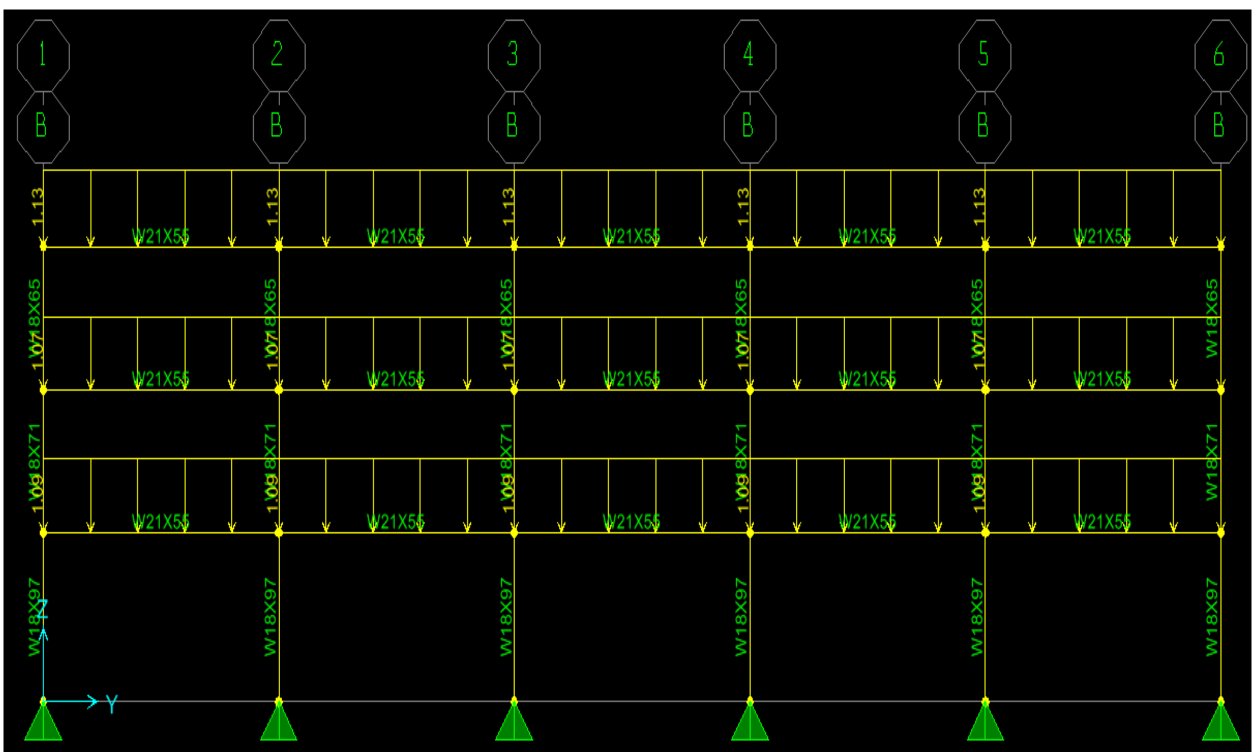

*Below is a Compression analysis on each column member.  The columns on the right of the building are experiencing more compression because the earthquake force was directed from left to right.  And because of that the columns on the left of the building are experiencing less compressive force.  This is good because the columns experiencing the most compressive forces are able to still pass the minimum design checks.*



*Below is a the story drift simulation in SAP2000. We inputed each force on each level to achieve a uniform drift and thus a simulated earthquake, abiet in one direction only for now.*



*This picture displays how we applied our dead loads.  Similarly the live loads and other loads were applied in this fashion aswell.  We used distributed loads as they best simulated real world loads.*



## Seismic System Research

Choosing the appropriate frame for a building is crucial in providing a safe and stable environment for the building. Buildings are susceptible to collapse if the wrong type of frame is chosen. It is very important to acknowledge and understand what the building will be used for, who will occupy the building, when the building is expected to be completed, what kind of seismic activity is present in the area, and how much funding will the project receive. It is also important to understand what types of seismic frames or systems are available in the market.

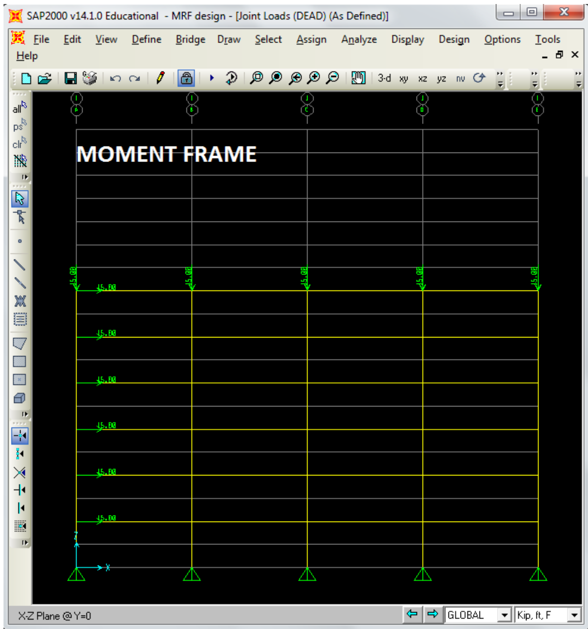
Two of the most commonly used frames in the engineering industry are Moment Resisting Frames (MRF) and Braced Frames. Moment Resisting Frames are usually made of steel and they can resist loads in the lateral direction such as winds or earthquake loads. They are intended to remain elastic and exhibit ductile behavior, meaning they stretch before breaking apart, during a major earthquake (propertyrisk.com). Eccentrically Braced Frames (EBF) are a type of braced frame that has high elastic stiffness and superior inelastic performance characteristics (tufts.edu). Much like a truss, EBFs work in tension and compression, unlike MRFs, where bending moment affects the members.

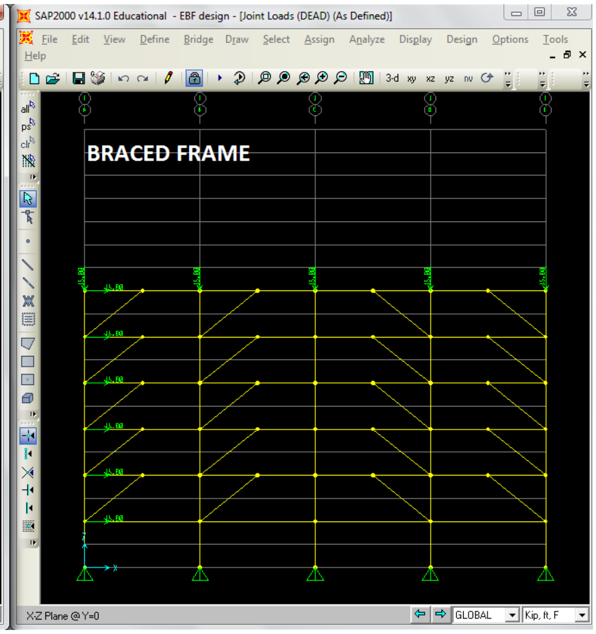
To improve the performance of the seismic frame, engineers utilize various seismic or energy dissipation systems. One such system is the Damping system. As its name suggests, this system “dampens” the seismic energy absorbed by the frames. This system has a chamber containing incompressible fluid that transfers between the chamber, thus converting kinetic energy into heat energy. This heat energy is safely dissipated into the environment (akira-wada.com). Another seismic system that reduces the damage done by earthquakes is base isolation. Base-isolated structures absorb less shear forces across their isolation surface than structures that are not isolated from the base. Although the main structure is isolated from the base, it does not mean that the building is earthquake proof (Berkeley.edu).

To determine which Seismic Frame performs better under various types of loads, we have created models on SAP2000 to examine and compare the behavior of MRFs and EBFs.

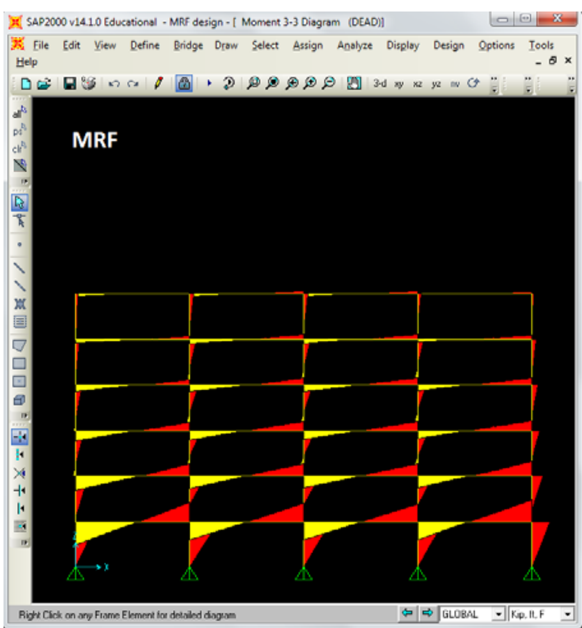
## Moment Frame Vs Eccentrically Braced Frame

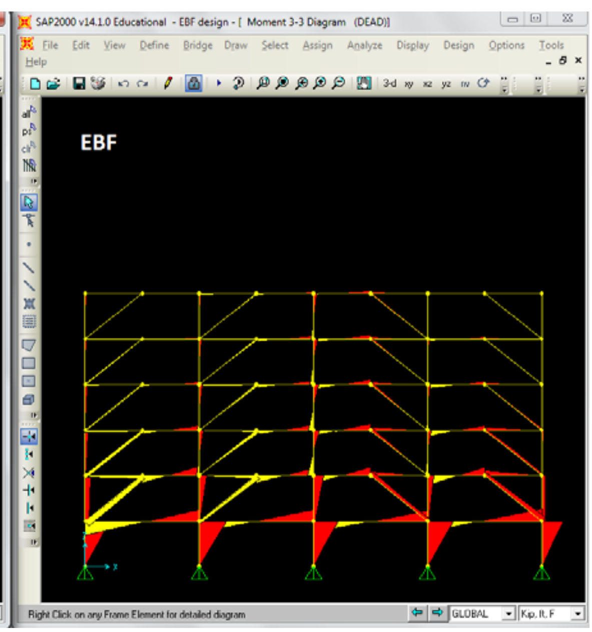
We have created 2D models of moment and braced frames on SAP2000. They are of equal length and height. They also use the same type of cross-section beams. We then applied dead and earthquake loads onto the frames and ran a simulation on SAP.




After the simulation, we discovered that the Moment Resisting Frame (MRF) had a larger deflection than the Eccentrically Braced Frame (EBF). Also, there are less shear, axial force, and bending moment acting on the EBF than the MRF.



### Conclusions on Types of Frames

Eccentrically Braced Frames work better than Moment Resisting Frames, but they are harder to build and cost more. The Damping System can be incorporated into either frame to improve the frame's performance. Another way of improving the performance of a frame is to incorporate base isolation. If the owner of the building wants to use an economic and reliable frame, an MRF with isolated base is his choice. But if budget is not an issue and he wants a very strong frame, an EBF with damping systems is his choice.

## Conclusions

The conclusion we made from these results tell us that for most US based structures and seismic activity, the ASCE 7-05 Equivalent Lateral Force Procedure performs within the acceptable limits. Our building designs could have been lowered to save weight in material costs.

In the ten weeks leading up to the finalization of our project, we would like to thank all of the San Francisco State structural engineering graduates students for lending us their knowledge and allowing us to partake in their study space. Of note, we are especially great full and indebted to our graduate student Qi Ming Zeng. Without Qi Ming Zeng's tireless commitments and off hours help, we would have had a great disconnect in regards to retain passion and knowledge in the field of civil engineering. And of course without the help of our faculty advisors Dr. Cheng Chen and Dr. Amelito Enriquez there would be nothing! Thank you.

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A Beginner's Guide To ASCE 7-05, www.bgstructuralengineering.com

# Appendix

## Local Buckling Constants

| **Global Flange Member Properties** | Check Against [Lower Limit] | Check Against [Upper Limit] |
|---|---|---|
| Formula | $\lambda_p = 0.38*(E/F_y)^\wedge(1/2)$ | $\lambda_r = 1.0*(E/F_y)^\wedge(1/2)$ |
| Member Properties | 9.15 | 24.08 |

| **Global Web Member Properties** | Check Against [Lower Limit] | Check Against [Upper Limit] |
|---|---|---|
| Formula | $\lambda_p = 3.76*(E/F_y)^\wedge(1/2)$ | $\lambda_r = 5.70*(E/F_y)^\wedge(1/2)$ |
| Member Properties | 90.55 | 137.27 |

## Beam Constants

| TRANS MEMBER PROPERTIES | Reference Box A: $W_u$ Reference Calculations, for Trans Member | live load per transverse 30 ft, L | | dead load per transverse 30 ft, D |
|---|---|---|---|---|
| Floor Levels | # number ft per floor | # total rate [kips/ft] | # number ft per floor | # total rate [kips/ft] |
| Roof | 14.8 | 0.493 | 28.1 | 0.938 |
| 3rd | 14.8 | 0.493 | 26.6 | 0.888 |
| 2nd | 14.8 | 0.493 | 27.2 | 0.908 |

| TRANS MEMBER PROPERTIES | Step 1 & 2 , Design Strength | Factored Uniform Load, $W_u$ Transverse [12.4.2.3] | $M_{u\ max}$ = Transverse [kips-ft] | $M_{u\ max}$ Transverse [kips-inch] |
|---|---|---|---|---|
| Floor | AISC Steel Design | $W_u = (1.2 + 0.2*S_{DS})*D + \rho*Q_E +$ | $M_{u\ max} =$ | $M_{u\ max} =$ |

| | Requirements | $1*L + 0.2*S$ | $(1/12)*W_u*L^2$ | $(1/12)*W_u*L^2$ |
|---|---|---|---|---|
| Roof | W21X68 | 2.327 | 174.562 | 2094.741 |
| 3rd | W21X68 | 2.258 | 169.379 | 2032.553 |
| 2nd | W21X68 | 2.286 | 171.452 | 2057.428 |

| TRANS MEMBER PROPERTIES | Step 3, Shear Strength | Factored Uniform Load, $W_u$ Transverse [12.4.2.3] | Maximum Nominal Shear |
|---|---|---|---|
| Floor | AISC Steel Design Requirements | $W_u = (1.2 + 0.2*S_{DS})*D + \rho*Q_E + 1*L + 0.2*S$ | $V_{n\ max} = 0.6*F_y*A_w$ , [kips] |
| Roof | W21X68 | 2.327 | 272.19 |
| 3rd | W21X68 | 2.258 | 272.19 |
| 2nd | W21X68 | 2.286 | 272.19 |

## Column Calculations

| Columns | AISC 13th Ed. LRFD Formula | Sum of Factored Loads | Elastic Critical Buckling Stress [ksi] | Slenderness Parameter, $\lambda_c$ |
|---|---|---|---|---|
| Floors | AISC Constants | $P_{u\ required} = 1.2*D + 1.6*L$ | $F_e = (\pi^2*E)/(K*L/r_y)^2$ | $\lambda_c = (K*L/r*\pi)*(F_y/E)^{1/2}$ |
| Roof | W18X65 | 95.04 | 46.92 | 1.03 |
| 3rd | W18X71 | 188.12 | 47.47 | 1.03 |
| 2nd | W18X97 | 282.40 | 82.59 | 0.78 |

## Equivalent Lateral Force Procedure Work

Reference Box 3

| Vertical Distribution Factor | | | | | | |
|---|---|---|---|---|---|---|
| Level | $w_x$ [kips] | $h_x$ [ft] | $w_x h_x$ [k-ft] | $\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ | $F_x$ [kips] | Sum V [kips] |
| | | | | | | |
| Roof | 1068.75 | 35 | 90338.54 | 0.534 | 259.27 | 0.00 |
| 3rd | 1012.50 | 24 | 53443.96 | 0.316 | 153.38 | 259.27 |
| 2nd | 1035.00 | 13 | 25418.02 | 0.150 | 72.95 | 412.65 |
| Total | 3116.25 | | 169200.52 | 1.000 | 485.60 | 485.60 |