

**STABILITY OF A FOUR STORY STEEL FRAME BUILDING UNDER
SEISMIC LOADING**

A Thesis

by

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Submitted to the Office of Graduate and Professional Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

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December 2016

Major Subject: Civil Engineering

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ABSTRACT

Progressive collapse and seismic resistance are separate topics that have many examples in the literature. However, because both of these events occur rarely, there are not many accurate research examples that have been conducted. Life threatening earthquakes occur once in every 50 years in high risk earthquake zones. Inelastic behavior of steel using a simple and reliable approach is an ongoing process. There are some conclusions about using lateral bracing and shear wall but this makes the design and the cost of the structure inaccurate for the contractor as well as limits the architectural designs.

In this project, a time history analysis of a four story moment resisting steel frame will be conducted. For the distribution of the energy released from the ground motion, a strong column-weak beam approach will be used. The structural system and every element in the system will be compact to resist flexure and lateral torsion that occur during the acceleration. Specific columns from the first floor will be removed and the structure will be accelerated under specific earthquake examples.

As a result of this project, an ideal four story steel frame resisting collapse under seismic loading will be obtained. Pros and cons of this method will be explained. This will influence further research development on the related topic. Postponing collapse events or limiting the local failure will save many lives and keep the economy stable.

DEDICATION

This Master's Thesis is dedicated my supportive mother, father and Pali. Without their love and support this degree would not have been possible.

ACKNOWLEDGEMENTS

I would like to start by thanking with my committee chair, Dr. Fry for giving me such an amazing project which fits my research interests. He has always been supportive, pushed me further to achieve my goals and has giving me a chance to join his research team. Dr. Reddy, whom always had an open door and helped me have a smooth transition with his guidance when I first started this degree here at Texas A&M University. Dr. Strouboulis, thank you for being in my committee and reviewing my project. Amanda Pelton, thank you reviewing my writings numerous times and always motivating me to work harder.

My research group, every single individual has helped me with different things and I would like to thank everyone individually for always being around for me. Sean, thank you for your guidance throughout this Master's Degree.

I would like to thank both the Turkish Fulbright Program and the American Fulbright Program for making it financially possible to study at Texas A&M University and to pursue the degree of my dreams.

My advisors in Celal Bayar University, thank you for being supportive throughout my bachelors, masters and during the Fulbright application process. It wouldn't be possible without your support.

Finally, thanks to my mother, father, sister and Pali for their encouragement, patience, love, guidance and endless support.

CONTRIBUTORS AND FUNDING SOURCES

This work was supported by Turkish Fulbright Program, the American Fulbright Program and University Transportation Center consisting of Dr. Gary Fry.

NOMENCLATURE

| | |
|-----------------|--|
| AISC | American Institute of Steel Construction |
| $A_{influence}$ | Influence area |
| A_g | Gross area |
| A_k | Load from extraordinary events |
| ASCE | American Society of Civil Engineering |
| ASD | Allowable Strength Design |
| ASTM | American Society for Testing and Materials |
| $A_{tributary}$ | Tributary area |
| B | Beam |
| C | Column |
| CBC Ratio | Beam to column connection ratio |
| C_d | Deflection Amplification Factor |
| CSI | Computers and Structures Inc. |
| D | Dead load |
| DCR | Demand Capacity Ratio |
| E_i | Earthquake load |
| E | Elasticity modulus |
| FEMA | Federal Emergency Management Agency |
| FNA | Fast Nonlinear Analysis |

| | |
|----------|--|
| FSL | Facility Security Level |
| ft | Feet |
| ft^2 | Square feet |
| F_v | Sum of forces in vertical, y direction |
| F_y | Yield strength |
| F_{yb} | Beam yield strength |
| F_{yc} | Column yield strength |
| g | Gravity |
| G | Girder |
| GSA | General Services Administration |
| I | Moment of inertia |
| IBC | International Building Code |
| IMF | Intermediate Moment Frames |
| Ksf | kips per square foot |
| L_f | Live load |
| L | Span length |
| L_r | Live roof load |
| LRFD | Load and Resistance Factor Design |
| M_o | Initial moment |

| | |
|-------------------------------|--|
| Ma | Moment at point A |
| Mb | Moment at point B |
| Mc | Moment at point C |
| $M_{plastic}$ | Plastic moment |
| M_{pb} | Beam plastic moment |
| M_{pc} | Column plastic moment |
| $M_{plastic-strainhardening}$ | Plastic moment with strain hardening |
| M_{yield} | Yield moment |
| M_{uv} | Additional moment due to shear amplification |
| M3 | Moment in Z direction at the removed column |
| NIST | National Institute of Standards and Technology |
| NSF | National Science Foundation |
| OMF | Ordinary Moment Frames |
| P | Axial stress at the removed column |
| P_{uc} | Required compressive strength |
| R | Response Modification Coefficient |
| R1 | First column removal |
| R2 | Second column removal |
| R3 | Third column removal |
| R4 | Fourth column removal |

| | |
|-----------|---|
| R5 | Fifth column removal |
| R6 | Sixth column removal |
| R7 | Seventh column removal |
| R_A | Support reaction at point A |
| R_B | Support reaction at point B |
| R_C | Support reaction at point C |
| R_y | Ratio of the expected yield stress to the specified min. yield stress |
| S | Snow load |
| SAC | Strategic Air Command |
| SDC | Seismic Design Category |
| SEI | Structural Engineering Institution |
| sec | Seconds |
| SMF | Special Moment Frames |
| S_x | Section modulus |
| v_B | Deflection at point B |
| $(v_B)_1$ | Deflection at point B after first loading |
| $(v_B)_2$ | Deflection at point B after second loading |
| $V(0)$ | Shear at the zero point |
| $V(L)$ | Shear at distance L |
| V_o | Initial shear |

| | |
|---------------|---|
| V_a | Shear at point A |
| V_b | Shear at point B |
| V_c | Shear at point C |
| V_d | Shear at point D |
| V_2 | Shear in y direction at the removed column |
| w | Uniform distributed load |
| W | Wide flange |
| x | x Direction |
| y | y Direction |
| Z_b | Beam plastic section modulus |
| Z_c | Column plastic section modulus |
| Z_x | Plastic section modulus |
| Z_{RBS} | Minimum plastic section modulus at the reduced beam section |
| Ω_o | Overstrength Factor |
| ρ | Redundancy Factor |
| δ_o | Initial deflection |
| θ_o | Initial rotation |
| $\delta(x)$ | Deflection at x distance |
| $\delta(L/2)$ | Deflection at $L/2$ distance |

$\delta(L)$ Deflection at L distance

$\delta(2L)$ Deflection at $2L$ distance

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1. INTRODUCTION AND BACKGROUND INFORMATION

1.1 Introduction

A load path mechanism is the redistribution of loads and forces of a vertical element when it is instantly removed. The idea of this method is to design a structure redundant enough to resist the lateral and gravity loads with absence of a critical vertical element. Alternative load path and redundancy are two major requirements for structural stability under blast loading (GSA, 2013). This is already a challenging process even when a structure without any loading must resist its dead load with a member missing. With today's technology, this process can be conducted using super computers and advanced finite element commercial software. In these softwares, along with the member removal some real life dynamic loading can be added and the structure could be analyzed under a dynamic load with some selected members missing.

Any loading that is not considered in the design of the structure is blast or abnormal loading (Breen, 1975). Blast loading can be manmade or a natural hazard. Manmade events include; sabotage, fire explosion, vehicle impact, lack of maintenance, and construction errors. Natural hazards can be flood, fire, tornado or earthquakes (Breen, 1975). All of these loading types require different consideration parameters. Blast loading has a very low probability of occurrence and leads to dynamic instability of the structure. For that reason, it is still not necessary to be included in every structure that has been built. It is usually taken into consideration for federal buildings, hospitals

and schools which need to operate right after a blast loading event, and for skyscrapers which have a very high cost and have greater significance to a community or region.

Resistance to earthquake loading became a very serious issue after the Northridge earthquake in 1994 (Yun et al.), which is also known as a historic earthquake. Even though the duration of the Northridge earthquake was 10-20 seconds, it led to 57 fatalities, 8700 injuries and \$13-\$40 billion worth property damage (Dreger, 1997). The ground motion was the highest recorded in North America, with a magnitude of 6.7 on the Richter scale, and could be felt about 220 miles from the epicenter. Federal Emergency Management Agency and Strategic Air Command (FEMA/SAC) started funding projects for earthquake hazard reduction programs after the Northridge Earthquake. Even though it has been titled as the prevention of structural collapse in literature, the priority is life safety in these structures, and then the economical losses are considered.

1.2 Background Information

Maintaining stability after blast loading has been a popular research topic since the 1960s when the Ronan building in London partially collapsed due to a fire explosion. Research increased steadily after the Murrah building attack in 1995 and the World Trade Center attack in 2001 (Nair, 2004). These are the three most recently discussed topics regarding collapse prevention. When the stability issues due to earthquake loads are considered, the research increased after the Northridge earthquake in 1994 in the United States (Yun et al. 2002). Much research has been conducted on connection, element and

eventually overall structural ductility and stability. Although these factors are important, the primary motivation of research is life safety. In specifications, collapse prevention of federal buildings has more regulations due to their importance to the nation. They are designed to be operational right after an earthquake or any abnormal loading.

There are four types of analysis methods that are used in the literature and allowed by the relevant codes. These methods are; linear static, nonlinear static, linear dynamic and nonlinear dynamic. Linear static analysis is the simplest and most conservative way to approach resistance. Previous General Services Administration Guidelines for collapse resistance (2003) recommended multiplying the load combination by two for consideration of dynamic factors. Because the dynamic amplification factor was too conservative (Hamburger, 2006), GSA (2013) removed this requirement in the latest revised publication. Nonlinear static analysis is a more accurate approach because material nonlinearities and geometric nonlinearities are included. This method has become increasingly used in the past decade due to its simplicity, compared to dynamic analysis and because less computation time is required now since the computers were not as accurate and as fast as they are today. This method is also called “pushover analysis” and is primarily used for seismic loading. The structure is laterally loaded using a triangular or linear load distribution, and the response of the structure to this loading is examined (Tavakoli and Alashti, 2013). The results were acceptably imperfect; however, because the earthquakes produce cyclic ground motions, the application of lateral forces did not exactly show the response under real seismic events.

Linear dynamic analysis has a level of accuracy between nonlinear static and nonlinear dynamic. Even though the dynamic loading has been included, material and geometric nonlinearities are not included. These nonlinearities can make a significant increase in forces. An example for this is the P-delta effect; which is when axial loading acting on the member with a lateral deflection produces extra moment. If the connections are not designed for this extra moment, it can lead to inelastic failure, which is not an acceptable failure for load distribution. GSA (2013) guidelines do not include this in their analysis methods any longer for these reasons. The most complex and most accurate analysis method is nonlinear dynamic analysis, commonly referred to as time-history analysis in seismic related areas. It includes both nonlinearities and dynamic motions during the analysis. The only limitations are that it is very sensitive to how the structure is modeled and can take more time and effort compared to other methods. However, with careful modelling and inputs, it produces a response very close to real life under dynamic loading. Therefore, it is very reliable and fast with the computer technology today.

Another modelling criteria is dimensional aspects. The analytical research literature shows two-dimensional and three-dimensional analysis are used for collapse resistance. Two-dimensional is preferred due to its simplicity and fast computation. However, it has been proven that three-dimensional model is more reliable not because it is a real life model but because the load distribution is more accurate in the third dimension. Song and Sezen (2013) proved that their experimental field results fits the best when three-dimensional nonlinear dynamic analysis was used. In addition,

compared to two-dimensional analysis another research has shown that three-dimensional analysis gives actual failure modes of the structure (Gerasimidis et al., 2015). With 3D analysis, the torsional effects will be considered and the results will not be conservative (Ficanha and Pravia, 2012).

Steel moment frames resist lateral loading by flexural and shear strength. There are three types of moment frames: Ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF). Each type consists of column to beam moment-resisting connections. OMFs do not have any special requirements. For this reason they are usually one story and limited with 65ft of structural height for Seismic Design Category, Figure 1, F and 35ft for SDC D and E (ASCE/SEI 7, 2013). Therefore they are permitted to be used in low seismic zones. IMF and SMF are more detailed versions of OMF and are systems resulting from the research conducted after the Northridge earthquake (AISC, 2012). The main difference between IMF and SMF is the allowable story drift angle. However, SMF is called “special” for satisfying specific details and expected high resistance under strong ground motion (Hamburger et al., 2009). Hamburger (2006) also recommended that having moment resisting frames at each floor level leads to a better load distribution by resisting blast loading or vertical member removal.

Some of the researchers were also able to conduct experimental analysis for collapse resistance. In these cases, a reinforced concrete building was evaluated before demolition. Sasani and Sagioglu (2008) measured the collapse resistance of Hotel San Diego. Their project only included experimental data and results. Sezen and

Giriunas (2009), were able to analyze a three story building located in Northbrook, Illinois. They conducted both analytical and experimental analysis. In both of the papers, the stress has been measured using strain obtained by strain gauge instrumentation. Even though both of the buildings analyzed were built with regulations from decades ago, they both satisfied current GSA (2003) deflection limitations. Song and Sezen (2013) analyzed a steel building using the same methodology. They analytically evaluated the structure with nonlinear static and nonlinear dynamic analysis. Both two-dimensional and three dimensional models are analyzed with both methods. These studies obtained very similar results when using three dimensional nonlinear analyses. Unfortunately, there are only few real life scale experiments conducted due to the cost associated with collapse analysis of an actual structure. Most of the structures are rather retrofitted without any demolition again for economic reasons. More information about structural stability, which is not in scope of this research project, and different frame type and behaviors could be found in Guide to Design Criteria for Metal Structures (Ziemian, 2010). It includes very detailed summary of the research on stability.

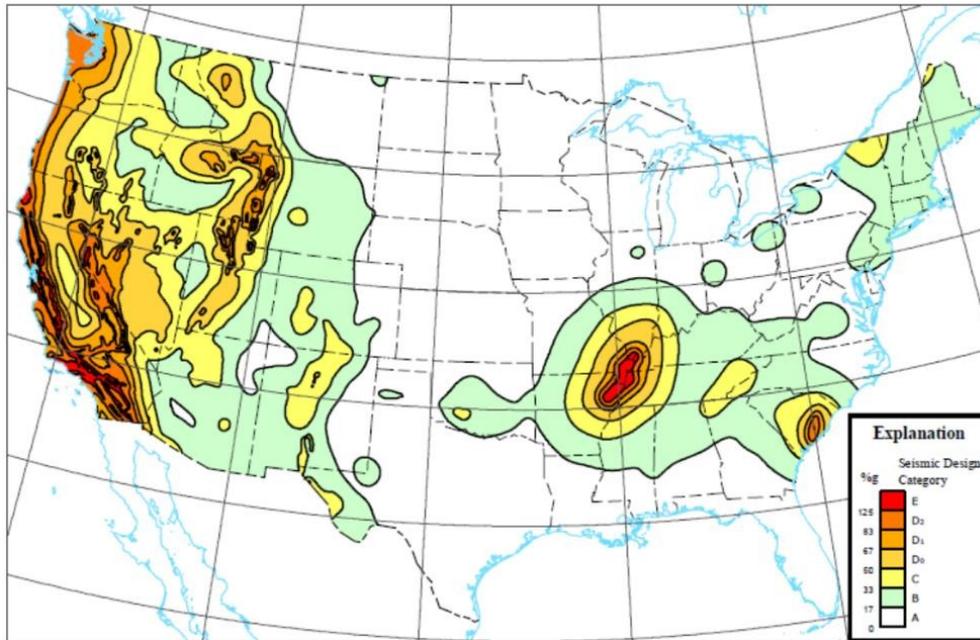


Figure 1 Seismic Design Category (IBC, 2012)

1.3 Objective

Although there has been plenty of research on seismic stability resistance of frame structures, more information is needed about the fundamentals of mechanics. For that reason, in this master's thesis a simple indeterminate beam is analyzed and the same analysis steps are applied to a four story steel SMF as an application example. For the reasons given in the background section, nonlinear dynamic analysis of a three-dimensional model will be used in this research project. The main goal of this research is to answer following questions:

1. The reason why vertical load members are important for load distribution.
2. How small details can change the overall stability and occupant safety.

3. Why is load redistribution path an important process to be considered in the design of any structure?
4. How to evaluate dynamic stability of a four story steel frame with a primary load member missing?
5. Is design for collapse resistance and seismic resistance good enough also to delay potential failure caused by fire in post-earthquake?

2. CODE REQUIREMENTS

2.1 General Services Administration Requirements

GSA has regulations for Alternative path analysis (2013). The main purpose of this document is to increase collapse resistance of any structure under any type of blast loading treatment. To reduce collapse potential, limitation of localized failure, bridging the lost member and providing proportioned and redundant design is required. The two main requirements are;

1. Load redistribution path – redistribution of loads when a load carrying vertical member is removed. In Figure 2, recommended column removal from (GSA,2013) has been shown.

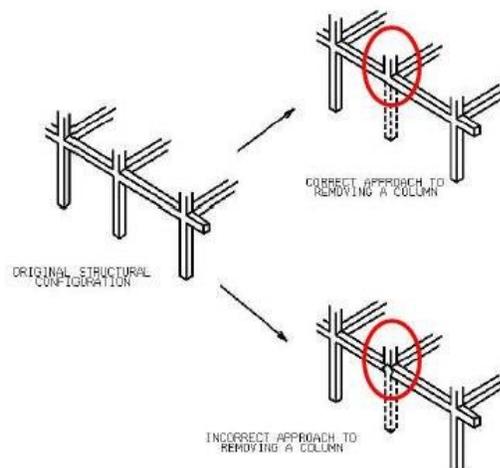


Figure 2 Column removal for alternative load path (GSA, 2013)

2. Redundancy – having a balanced design of member symmetry, and no column is removed for architectural reasons. This should be in good conjunction with load redistribution path.

The structure under consideration should be grouped under one of the five Facility Security Levels (FSL). Each of these levels has different requirements for this regulation to be applicable. In this project, the structure is assumed to be FSL 3&4, which by the guideline means all the required steps and analysis mentioned in this document must be satisfied for structures with four or more number of stories. It is allowed only for first story columns to be removed. Column removal of first story is chosen in this thesis for the reasons that will be explained in chapter 3. When a column is removed, beam-to-beam continuity is assumed to be maintained. Below is recommended column removal locations shown in Figure 3 for external column removal and Figure 4 for internal column removal.

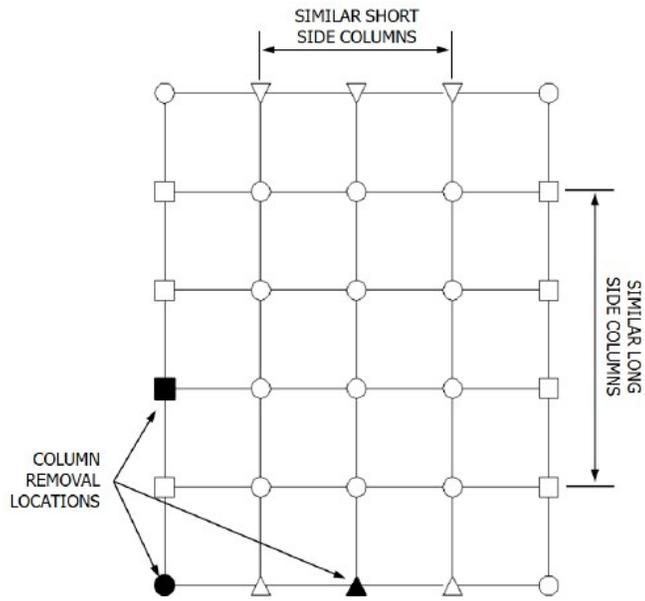


Figure 3 External column removal location in plan view GSA (2013)

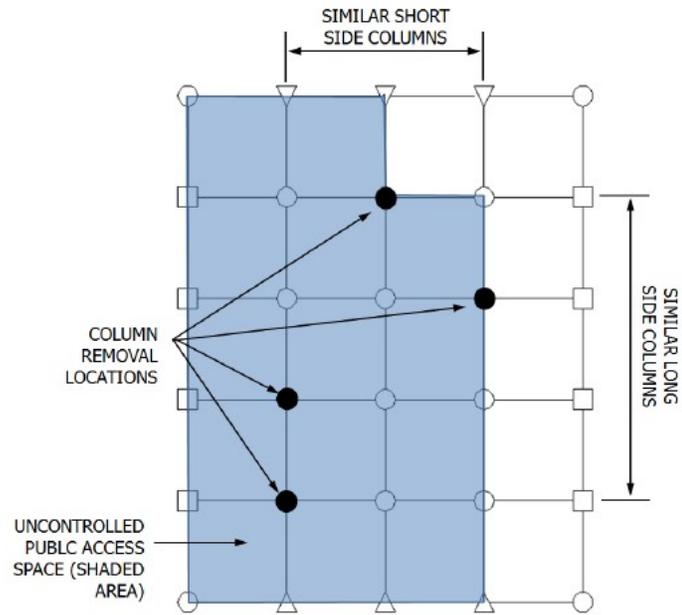


Figure 4 Internal column removal location in plan view GSA (2013)

For nonlinear dynamic analysis;

- Stiffness requirement in ASCE/SEI 41 (ASCE, 2014) should be satisfied. In this project, AISC Seismic Provisions (AISC, 2012) are used because they are for steel structures, specifically.
- There should be no geometric irregularities in the model used. However, this model irregular model is used to prove irregular buildings can be collapse resistance as well.
- Ductile connections must be used.
- Local stability and global stability in both vertical and horizontal direction should be satisfied.
- Lateral torsional buckling must be included. Due to limitations in finite element modelling in SAP 2000, this effect is not considered in this thesis.
- Load combination given in ASCE/SEI 7 for extraordinary events should be used.
- Column removal duration must be less than one tenth of the natural period of the structure. To avoid bad practice of this, the columns will be removed when the structure is statically stable.
- Design strengths, rotation capacities and beam-to-column connections should be determined from ASCE/SEI 07.

Most of the requirements mentioned by GSA (2013) and summarized above is included during modelling and analysis of the structure used in this thesis.

2.2 Minimum Design Loads for Buildings and Other Structures Requirements

Minimum Design Loads for Buildings and Other Structures Requirements, ASCE/SEI 7-10 (2013) is used in this research project for loads and load combinations, and some seismic requirements. This document recommends use of AISC 341 in conjunction to the requirements in this regulation to satisfy seismic requirements for steel buildings.

2.2.1 Load Combinations

Basic load combinations that include earthquake loading and blast loading from ASCE/SEI 07 (2013), section 2.3.2;

Load Combination 1;

$$1.4D$$

Load Combination 2;

$$1.2D+1.6L_f+0.5(L_r \text{ or } S \text{ or } R)$$

Load Combination 3;

$$1.2D+1.6(L_r \text{ or } S \text{ or } R)+(L_f \text{ or } 0.5W)$$

Load Combination 4;

$$1.2D+1.0E_l+L_f+0.2S$$

Load Combination 5;

$$0.9D+1.0E_l$$

Where D is the dead load, L_f is the live load, L_r is the live roof load, S is the snow load and E_f is the earthquake loading. Section 2.5.2.1 gives the following load combination for extraordinary events which are explained earlier as blast loading.

Below is the equation from Section 2.5.2, this equation is used as two different combinations;

$$(0.9 \text{ or } 1.2)D + A_k + 0.5L_f + 0.2S$$

Load Combination 6;

$$(0.9)D + A_k + 0.5L_f + 0.2S$$

Load Combination 7;

$$(1.2)D + A_k + 0.5L_f + 0.2S$$

Where, A_k , is the load from extraordinary events. It is a pressure loading and is not within the scope of this thesis for the modelling reasons. Snow load is taken as zero because most of the SDC E given in IBC (2012) is zero snow loading according to Figure 7-1 in ASCE/SEI 07 (2013).

3. TECHNICAL INFORMATION

3.1 Hand Calculation of a Simple Continuous Beam

In this subsection, effect of support removal of an indeterminate structure will be analyzed. The continuous simple beam is stable, robust and redundant and it has been optimally designed for case 1. The requirements mention in Design Guidelines for Progressive Collapse Resistance (GSA, 2013) are followed, and accordingly first the load path has been determined. When each support is removed, one at a time, the change in remaining support reactions, shear and moment diagrams are analyzed. The effect of this change on overall structure is explained using basic engineering judgement. Examining the effects when support at B and support at C is removed one at a time. Figure 5 shows case 1 for the beam.

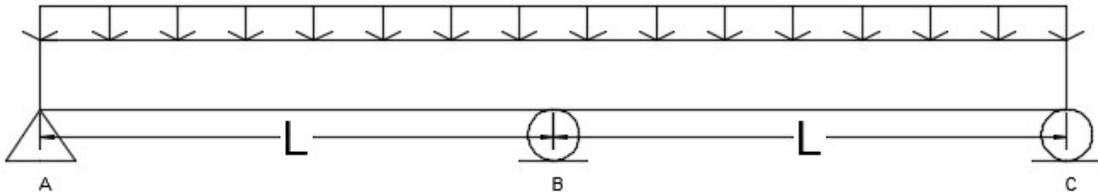


Figure 5 Simple indeterminate beam

Assumptions

- The beam is design to be elastic (optimal) for case 1.

- The beam is made Gr. 50 structural steel.
- The supports represent vertical load carrying members

3.1.1 Comparison of Case 1 and Case 2

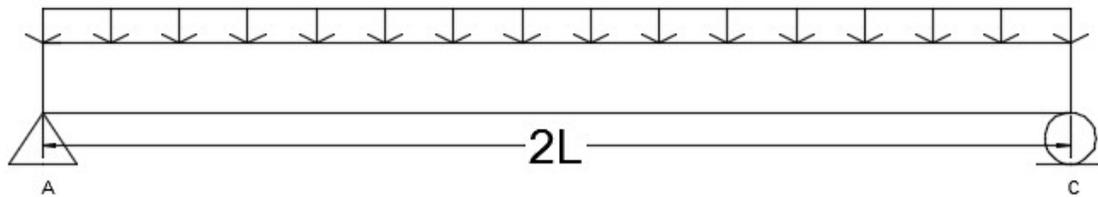


Figure 6 Simple beam with support B removed

In this case support at B is removed, as shown in Figure 6. The remaining support reactions are now 2.67 times what they carried in the 1st case. This also leads to increase at the initial shear value. The maximum moment carried by the 1st case increased 4 times and on the opposite direction (negative to positive). The maximum moment the beam has to carry is a lot greater in the 2nd case. The deflection at $L/2$ increased almost by 3.5 times and now there is a high mid-deflection which it was not considered in the design process. In this situation, the beam will no longer remain elastic since the beam was designed to have a zero deflection at the mid-span. Now has deflection caused by the beam and the loading. The calculation of support reactions, deflection, shear and moment and the diagrams can be found in Appendix A.

Table 1 Comparison of Case 1 and Case 2

| | Case 1 | Case 2 | Increase (%) |
|---------------|-------------------|-------------------|---------------------|
| R_A | $(3/8)wL$ | wL | 267 |
| R_B | $(5/4)wL$ | 0 | 0 |
| R_C | $(3/8)wL$ | wL | 267 |
| $V(0)$ | $(3/8)wL$ | wL | 267 |
| $V(L)$ | $(5/8)wL$ | 0 | 0 |
| $M(L)$ | $(-1/8)wL^2$ | $wL^2/2$ | -400 |
| δ_o | 0 | 0 | 0 |
| $\delta(L/2)$ | $(17/384)wL^4/EI$ | $(57/384)wL^4/EI$ | 335 |
| $\delta(L)$ | 0 | $(5/24)wL^4/EI$ | inf |
| $\delta(2L)$ | 0 | 0 | 0 |

3.1.2 Comparison of Case 1 and Case 3

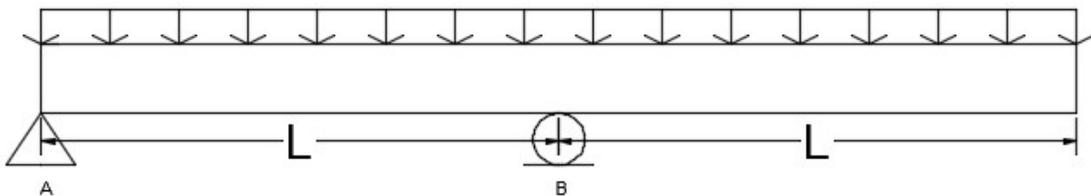


Figure 7 Simple beam with support C removed

In this case the support at right hand side is removed as shown in the Figure 7, above. Because the loading is symmetric, now only the support at B is carrying the loading and support at left hand side does not have any reaction force. The mid-span moment has increased 4 times on the same direction. Under this condition the point C

faces good amount of deflection. This again causes the beam to pass beyond the elastic range since it was designed for no deflection.

Table 2 Comparison of Case 1 and Case 3

| | Case 1 | Case 3 | Increase (%) |
|---------------|-------------------|-------------------|---------------------|
| R_A | $(3/8)wL$ | 0 | 0 |
| R_B | $(5/4)wL$ | $2wL$ | 160 |
| R_C | $(3/8)wL$ | 0 | 0 |
| $V(0)$ | $(3/8)wL$ | 0 | 0 |
| $V(L)$ | $(5/8)wL$ | wL | 160 |
| $M(L)$ | $(-1/8)wL^2$ | $(-wL^2/2)$ | 400 |
| δ_o | 0 | 0 | 0 |
| $\delta(L/2)$ | $(17/384)wL^4/EI$ | $(-7/384)wL^4/EI$ | -41 |
| $\delta(L)$ | 0 | 0 | 0 |
| $\delta(2L)$ | 0 | $(7/12)wL^4/EI$ | inf |

3.1.3 Result

After static analysis of the simple indeterminate beam it has been concluded that redundancy alone is not enough to maintain the stability after a vertical member has been removed.

$$M_{yield} = \frac{w(2L)^2}{8} = \frac{wL^2}{2} \quad (1)$$

$$\frac{M_{plastic}}{M_{yield}} = \frac{Z_x}{S_x} = 1.20 \quad (2)$$

$$M_{plastic} = 1.2 \left(\frac{wL^2}{2} \right) = \frac{3wL^2}{5} \quad (3)$$

If strain hardening is assumed, another 50% will be added to plastic moment which gives;

$$M_{plastic-strainhardening} = 1.5 \left(\frac{3wL^2}{5} \right) = \frac{9wL^2}{10} \quad (4)$$

For both support removal case;

- The moment is 2.2 times the strain hardening.
- The moment is 3.3 times the ultimate capacity.
- The beam collapsed in both cases because they were not designed such force increase.

3.2 Load Patterning

In this subsection effect of influence area and tributary area of the following floor plan to overall structure will be explained. Figure 8 shows the floor plan of the model used.

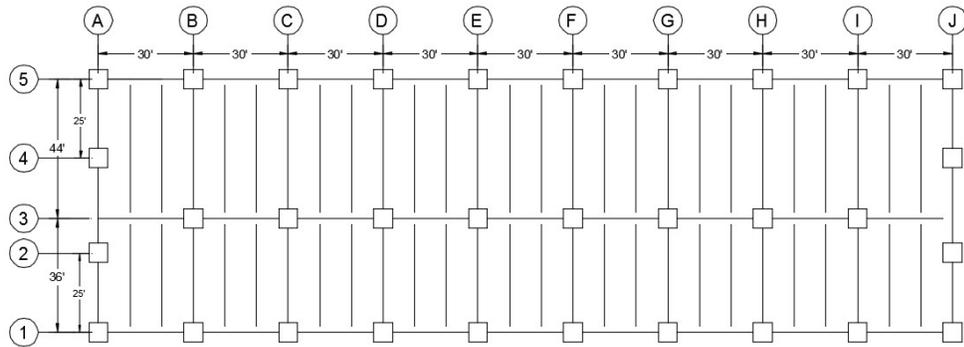


Figure 8 Floor plan

3.2.1 Floor Beam

Tributary area is the half of the area from one beam to the next beam from both sides, as illustrated in the Figure 9 below. Influence area is the area covered from the damaged beam to beams surrounding it. In the plan given below, if the blue floor beam is damaged, total of “L” times the length of the floor beam area will be affected from this when tributary area is the concern. However, “2L” times the length of the beam is the area affected because of the influence area.

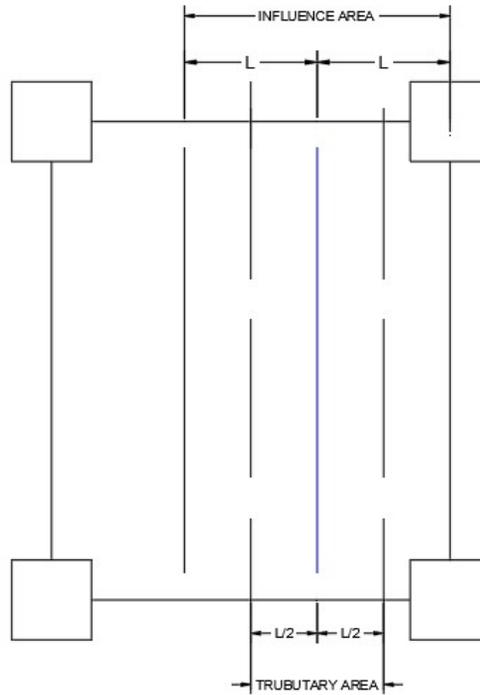


Figure 9 Influence area and tributary area of floor beams

For top beams the area is;

$$A_{tributary} = \left(\frac{10+10}{2} \right) \times 44 = 440 \text{ ft}^2$$

$$A_{influence} = (10+10) \times 44 = 880 \text{ ft}^2$$

For bottom beams the area is;

$$A_{tributary} = \left(\frac{10+10}{2} \right) \times 36 = 360 \text{ ft}^2$$

$$A_{influence} = (10+10) \times 36 = 720 \text{ ft}^2$$

3.2.2 Girder

Tributary area is the half of the area from one girder to the next girder from both sides, as illustrated in the Figure 10 below. Influence area of the area covered from the damaged girder to girders surrounding it.

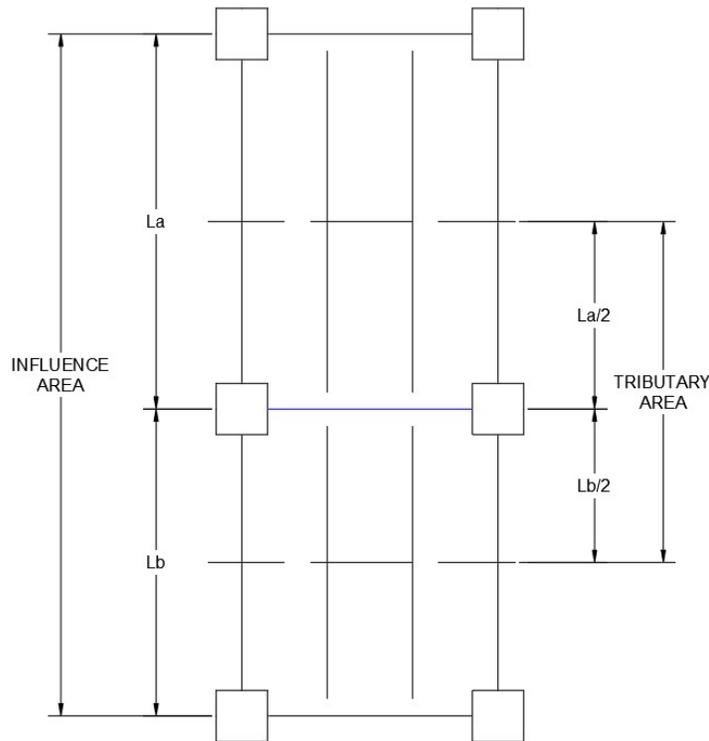


Figure 10 Influence area and tributary area of girders

In the plan given above, if the blue girder is damaged, total of “ $(L_a+L_b)/2$ ” times the length of the girder area will be affected from this when tributary area is the concern. However, “ (L_a+L_b) ” times the length of the girder is the area affected because of the influence area.

For middle girder the area is;

$$A_{tributary} = \left(\frac{44 + 36}{2} \right) \times (30) = 1200 \text{ ft}^2$$
$$A_{influence} = 80 \times 30 = 2400 \text{ ft}^2$$

3.2.3 Column

Tributary area is the half of the area from one column to the next column from both sides, as illustrated in the Figure 11 below. Influence area of the area covered from the damaged column to columns surrounding it.

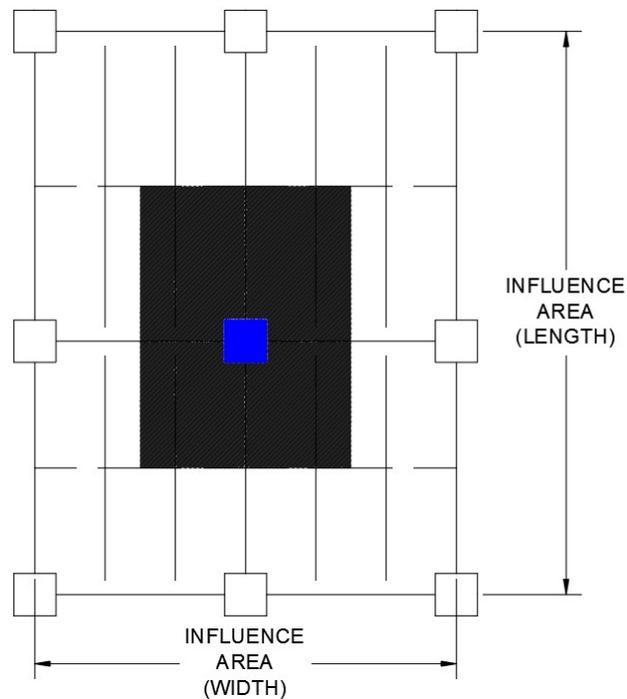


Figure 11 Influence area and tributary area of floors

In the plan given above, if the blue column is damaged, the shaded area will be affected from this when tributary area is the concern. However, the entire plan given is the area affected because of the influence area.

For middle column the area is;

$$A_{tributary} = \left(\frac{44 + 36}{2} \right) \times \left(\frac{60}{2} \right) = 1200 \text{ ft}^2$$
$$A_{influence} = 80 \times 60 = 4800 \text{ ft}^2$$

From the explanation above;

1. Most critical structural member in this case is a column. The area it effects is a lot larger than the other structural members
2. If most critical case is needed, a column should be removed from the first story. Same area calculated as in first story will be effected on higher floors.
3. According to the information above, removal of an internal column is more critical. For all possibilities to be observed, it is required in GSA (2013) both the external and internal columns at different locations will be removed.
4. Strong column-weak beam design recommended in all seismic regulations is because of the influence and tributary area.

3.3 Panel Zones

First requirement for inelastic response by AISC (2012) is ductile beam to column connections. In other words, the failure shouldn't start at the connection, which

is worse practice of ductile design. Panel zone is column web portion of element limited by the beam. It improves strength, stiffness and ductility of beam to column connections (Davila-Arbona, 2007). In panel zones moment forces are converted into shear forces and resisted. It also improves overall lateral flexibility of the entire structure (AISC, 2012). FEMA 355c (2000) has all the details required for welded moment resisting frames.

For its proven resistance to seismic resistance, panel zones are within scope of this project. As shown in Figure 2, the panel zones are kept in the system while vertical load carrying members are removed.

4. MODEL

4.1 Model Used

In this project, four story steel frame from Appendix E of GSA (2013) document is analyzed. This example is designed for a non-seismic region and is grouped in FSL 4. Alternative path and redundancy requirements are satisfied. It is a perimeter frame braced in transverse direction. All beam elements include a moment connection except for columns connecting to weak axis. The gravity connections are pinned connections. The structures soil connection is a pin connection as well. Roof is a metal deck and the floors are in composite action with metal deck and reinforced concrete. ASTM A992, Grade 50 Steel is used for all of the members. The roof and floor loading is obtained from IBC (2012) and the structure is mainly designed for wind. Member orientation and sizes can be found in the Appendix B. The member orientations and overall size of the frame is given below. Each structural member is given a specific label. The labels are given to member from bottom floor to the top and from left to right. Each member's specific label is given in a spread sheet given in the Appendix B. Below is an example of the floor plan and elevations. Figure 12 shows initial plan used, Figure 13, Figure 14 and Figure 15 shows elevations for different gridlines.

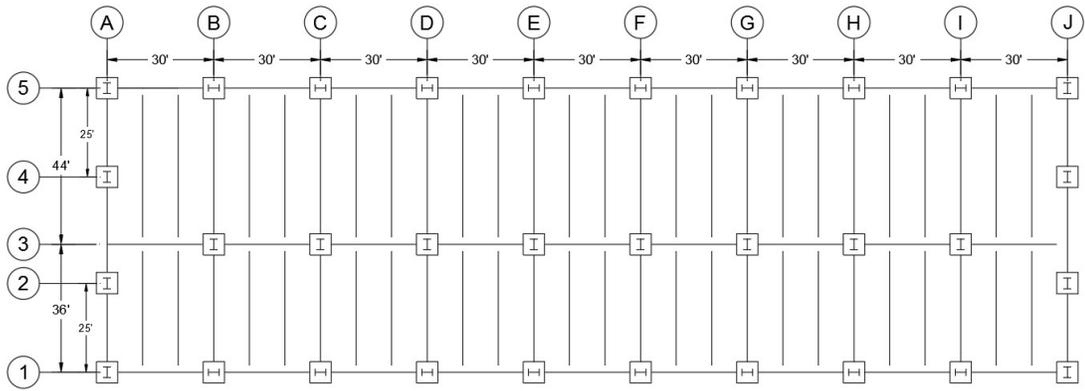


Figure 12 Initial floor plan of model used

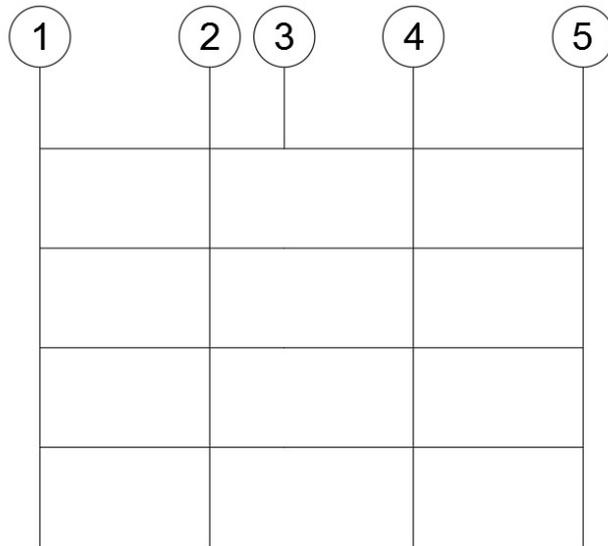


Figure 13 Gridline A and J elevations

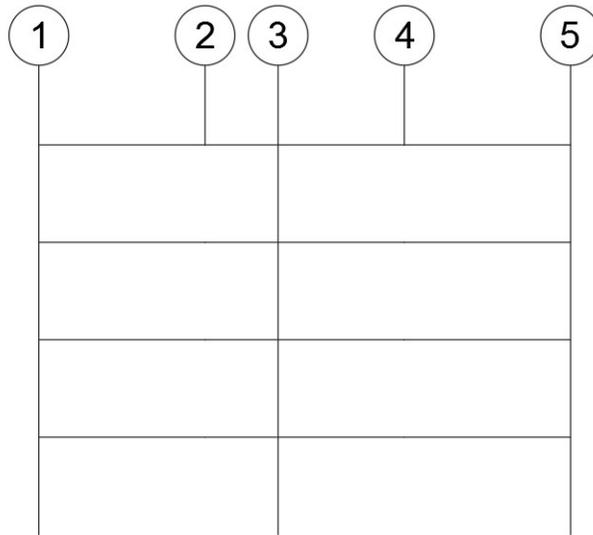


Figure 14 Gridlines B-I elevations

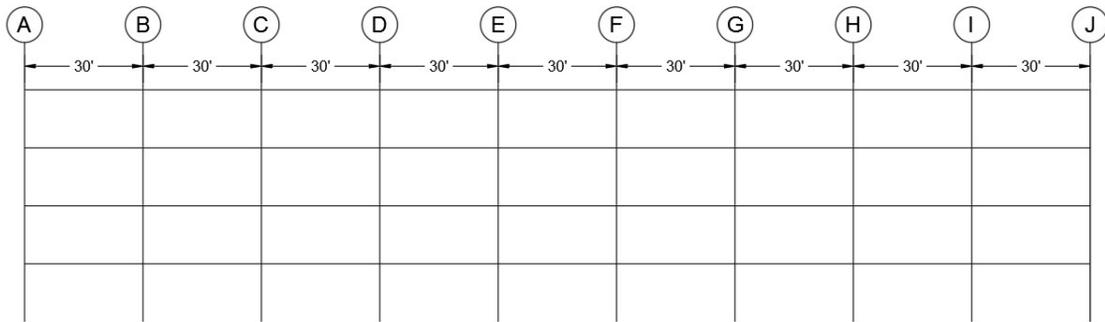


Figure 15 Gridline 1-5 elevations

4.2 Modifications In The Model

The main change made in this structure will be satisfying seismic regulations, mainly the strong column weak beam approach. For the first analysis the shapes given in GSA (2013) will be used. The only change made in the model is in the connections. The

floor beams are changed into pin connections to avoid torsion development. The main skeleton of the frame's beam-to column connections are changed into moment connections. As the seismic code requirements and limitations are applied, the member shapes will change. Another major change to the structure will be the change in slabs. Since the main objective of this project is to observe load path distribution between beams and columns, the slabs are not the major structural elements. However, the mass has effect to the inertia force and therefore it will affect the analysis under ground motion. For this reason, the slab will be assumed as an area and hand calculated loads and forces will be applied manually, the calculations can be found in Appendix C. All beam –to-column connections of the long side of the frame will be changed into moment connections to satisfy the special moment frame requirements. As explained in subchapter 3.3, panel zones will be included because of the improved resistance they add to the structure and for the non-conservative drift calculation reasons (AISC, 2012). Figure 16 shows screenshot of three dimensional model used.

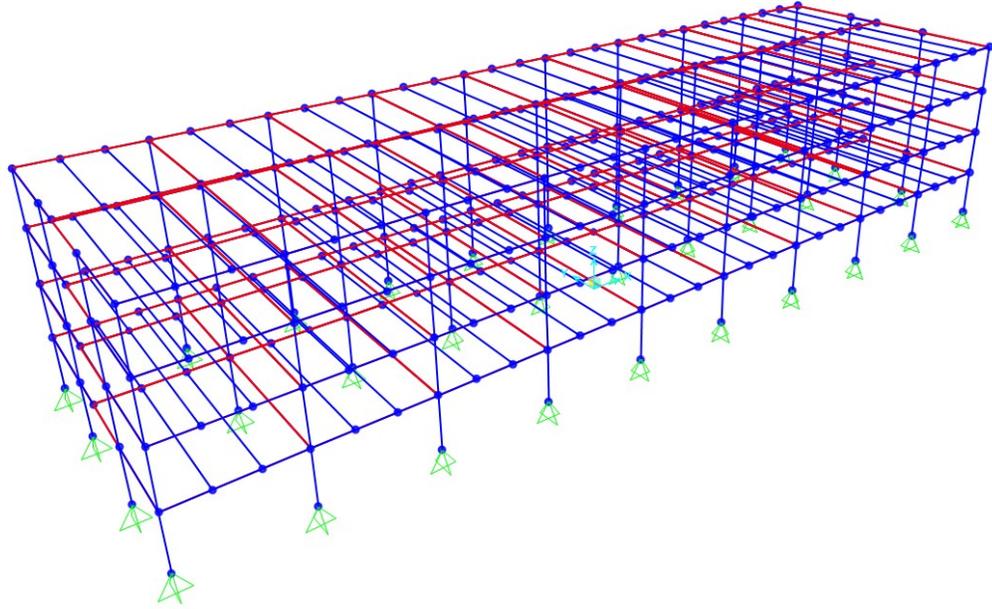


Figure 16 Three dimensional model

4.3 Seismic Region

This structure is assumed to be located in Seismic Risk Category I, and SDC E. The model is converted into a SMF and all the properties given in ASCE/SEI 7-10 are used for the model inputs. SMFs are not limited to any SDC. From Table 12.2-1, design coefficients for steel SMF are;

- Response Modification Coefficient, $R = 8$
- Overstrength Factor, $\Omega_o = 3$
- Deflection Amplification Factor, $C_d = 5.5$.

Requirements for Risk Category I, from ASCE/SEI 7-10 Section 12.3.4.2 are;

Redundancy Factor, $\rho = 1.3$

For this project, earthquake records from Northridge, El Centro and Izmit earthquakes are used. The ground motions are applied in both X and Y direction to capture the most critical response. Ground motion graphs are plotted and can be found in Appendix D. The reason past earthquakes are preferred is they are unique accelerations and there are so many examples in the literature. Unscaled earthquake records are imported in to SAP 2000 and are scaled by $g = 32.2 \frac{ft}{sec^2}$. The beam members' unbraced lengths are modified into zero because the members are laterally supported by the slab. The software package determines it differently before of the composite action modelling of the slab to the bare frame.

4.4 Modelling In SAP 2000

The structural members, columns and beams are oriented as it is given in Appendix B. The soil structure interaction is modelled as a pinned connection. For second order effects to be included, the time history analysis is nonlinear. Large P-delta effects are included, the elements are meshed manually into 10 pieces. The slabs are not in composite action with the frame.

Modelling steps are;

1. Grid definition
2. Material and section definition
3. Drawing structural elements (beams, column and slabs)
4. Assigning sections
5. Applying material properties

6. Defining restraints
7. Assigning loads (Uniform load to frames on area loading)
8. Importing ground motion data
9. Defining load patterns
10. Defining load combinations
11. Analyzing

Every time a member fails because of exceeding demand capacity ratio or beam/column moment ratio, step 4 will be repeated. Because the loading and load patterns will not change, only analysis has to be repeated.

4.5 Columns Removed

The floor plan below shows the columns being removed. Column removal locations satisfy the recommended locations (GSA, 2013). Figure 17 shows columns removed from the model used in this thesis.

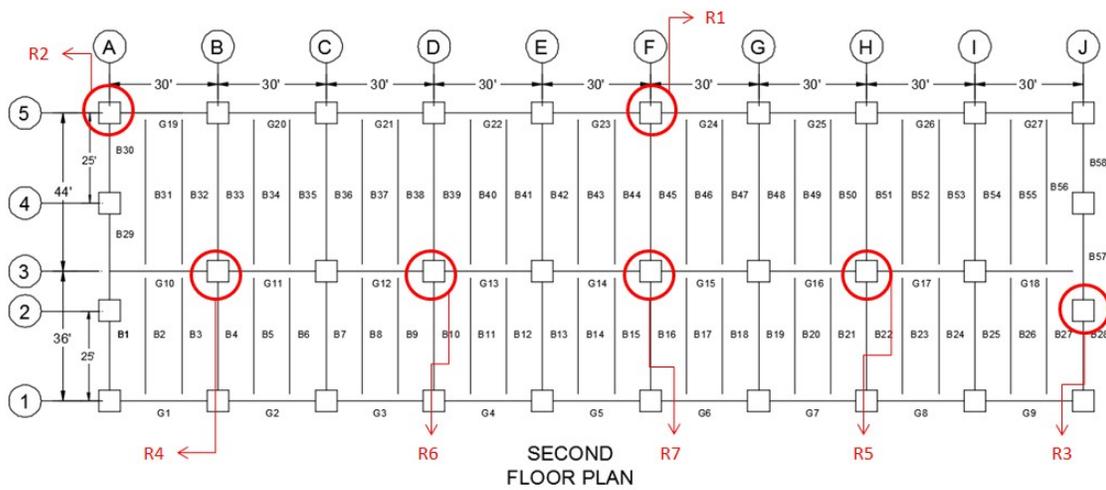


Figure 17 Column removals

Table below shows each column label for each removal case.

Table 3 Column Labels For Each Removal

| | |
|----|-------|
| R1 | C 67 |
| R2 | C 4 |
| R3 | C 114 |
| R4 | C 18 |
| R5 | C 90 |
| R6 | C 42 |
| R7 | C 66 |

5. ANALYSIS

5.1 Analysis

In this chapter analysis methods and steps are explained. Three dimensional nonlinear time history analysis is applied to each model. In SAP 2000 (CSI, 2016), it is named as Fast Nonlinear Analysis, FNA and load dependent Ritz vectors are used during nonlinear modal time analysis. The modal damping is 5%.

At each analysis step, the controlling property of each section is explained and supported with the code requirements. After each column removal and for each earthquake, failed members will be replaced with more compact sections for failure cases. The member properties on AISC Database v14.1 is used. The failures are determined by the Demand Capacity Ratio;

$$DCR = \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \quad (5)$$

As given in flow chart below, first the seismic requirements are applied. Main controlling factor at this step is strong column- weak beam approach. AISC 341-05 requires the plastic moment ration of columns to the beams should be equal to or more than 2. This requirement has to be satisfied for both strong axis bending and weak axis bending. Because $F_y = 50$, this could be easily obtained by plastic section modulus ratio.

$$\frac{M_{pc}}{M_{pb}} = \frac{F_y Z_c}{F_y Z_b} \geq 2.0 \quad (6)$$

Figure 18 contains steps of analysis. For each earthquake, the ground motions are applied in the X and Y direction to capture the most critical loading direction. Once the structure is stable under the ground motion given, the specific columns given in Chapter 3 are removed individually and the effect is recorded. The analysis will be carried in 9 steps and they are;

1. Analyzing the model by applying the earthquake in X direction and removing the Nth column
2. Fixing the failed members
3. Adding the removed column and analyzing the structure again
4. Fixing the model if any elements failed
5. Analyzing the model by applying the earthquake in Y direction, fixing if anything fails
6. Removing Nth column and analyzing the structure
7. Fixing failed members
8. Adding the removed column and analyzing the structure again
9. Fixing the model for the last time and using same model and repeating all the steps for (N+1)th column.

Where N represents column removal 1 to 7.

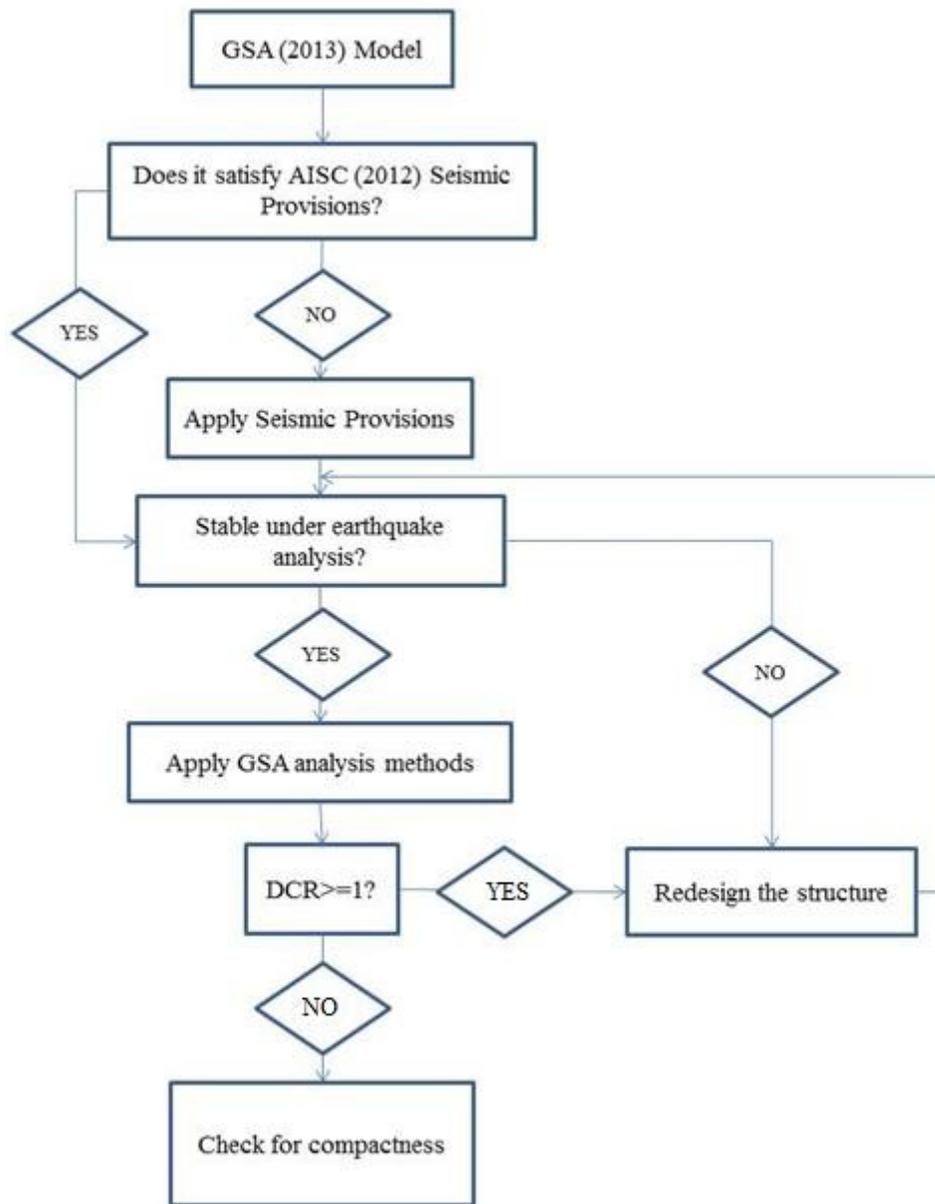


Figure 18 Analysis plan

5.2 Loads Applied

Below are the loads applied to the slab.

Table 4 Slab Area Loads

| | |
|--------|-----------|
| Droof | 0.068 ksf |
| Dfloor | 0.085 ksf |
| Lroof | 0.02 ksf |
| Lfloor | 0.1 ksf |
| Snow | 0 |

6. RESULTS

6.1 Results

In this chapter, results from analysis are represented. The shape changes are given in the tables below for each earthquake, separately for ground acceleration in X and Y direction, and for each column removal case. The earthquakes for analysis are El Centro, Northridge and Izmit earthquake. Because they all have similar high magnitudes, the failures are almost identical. In Appendix D, the screen shots of failures and their DCR (P-M) ratios are given.

Some members fail because they exceed the DCR and some interior columns fail because the beam/column moment ratio exceeds. This is controlled by the AISC 341 (2012). Below is the equation for LRFD, when beam sections are not reduced and it should be satisfied for beam-to-column connection.

$$\frac{\sum M_{pc}}{\sum M_{pb}} > 1.0$$
$$\frac{\sum Z_c (F_{yc} - P_{uc} / A_g)}{\sum (1.1 R_y F_{yb} Z_{RBS} + M_{uv})} > 1.0 \quad (7)$$

It can be seen from the Appendix D that after the column removal, when the surrounding failed beams are iterated, the failure in columns are due to beam/column ratio exceedance. When the above equation is solved, it has been found that the weak axis plastic section moduli of the interior columns are causing the ratio to exceed. Using

AISC 360 (AISC, 2010), it has been determined that the shape W14X730 has the largest plastic section modulus in weak axis. Once the columns failing due to beam to column ratio are replaced with the largest section, the error is still occurring. This cannot be fixed due to manufacturing limitation. It shows that in today's society, the buildings are optimally designed and they have just enough structural elements. When a column is removed however, to satisfy the redundancy and stability either the existing members will be replaced with stiffer sections, which adds extra load to the frame, or fails because of the beam to column ratio as shown in this project. Another solution can be adding extra columns, however, due to architectural reasons it is not a preferred option.

Even though the analysis show that some of the members failed, their failure will affect the members connecting to them. For example if a girder failed, the floor beams will fail because they no longer have a girder to support them.

Here are results from each removal case

6.1.1 Column Removal 1

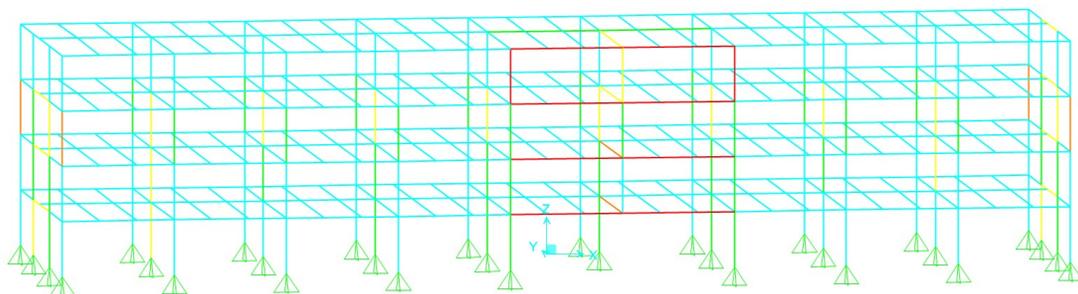


Figure 19 Failed 3D model after column removal 1

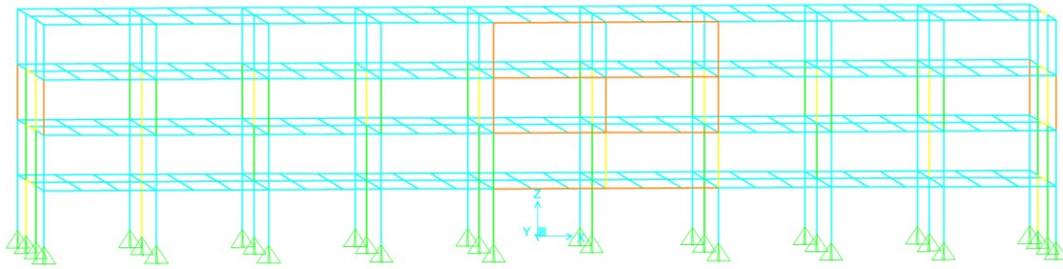


Figure 20 Fixed 3D model after column removal 1

In this removal case, 8 girders and 2 columns failed. This leads to failure of all the floor beams connecting to those failed girders. Eventually the frame column is removed will partially collapse. Figure 19 shows the failed structure and Figure 20 shows the fixed model.

6.1.2 Column Removal 2

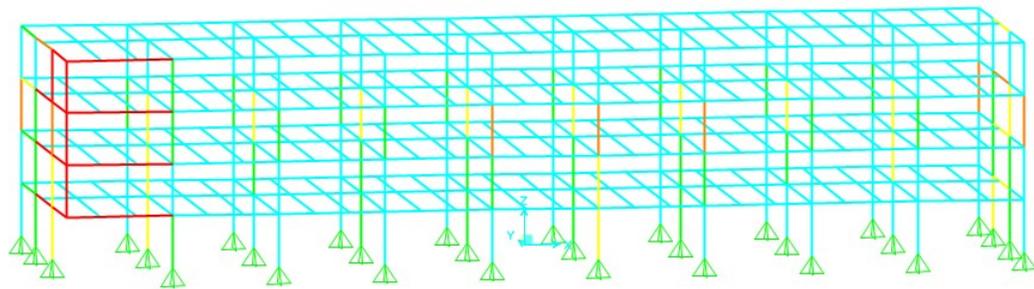


Figure 21 Failed 3D model after column removal 2

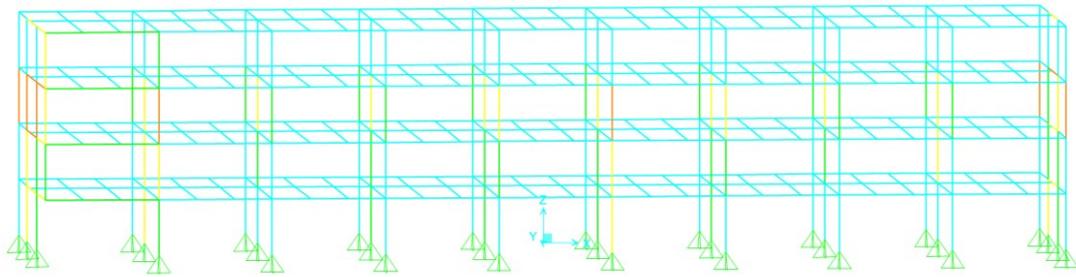


Figure 22 Fixed 3D model after column removal 2

In this removal case, 4 floor beams, 4 girders and 2 columns failed. This leads to failure of all the floor beams and girders connecting to those failed members. Eventually the frame column is removed will partially collapse. Figure 21 shows the failed structure and Figure 22 shows the fixed model.

6.1.3 Column Removal 3

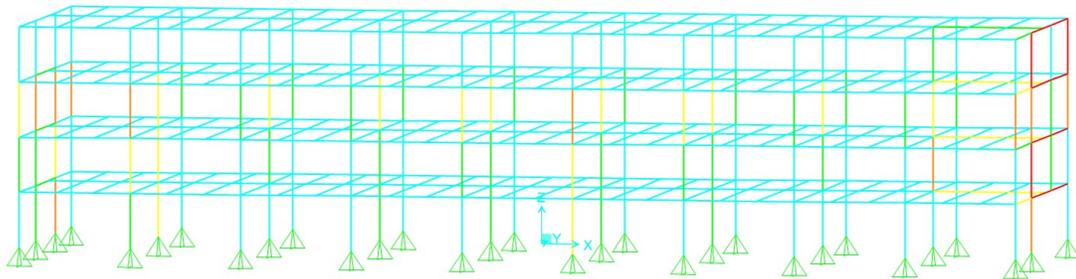


Figure 23 Failed 3D model after column removal 3

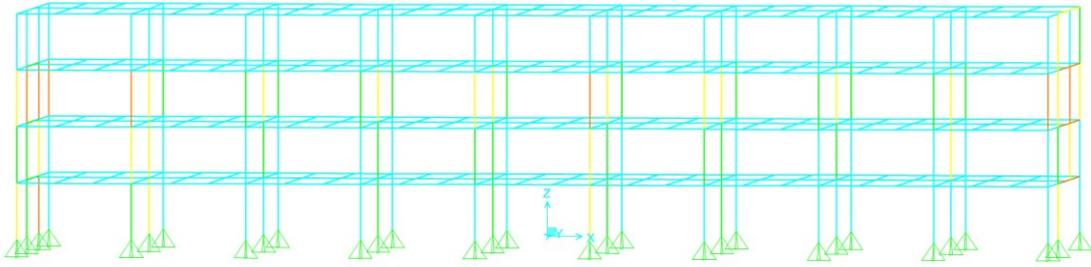


Figure 24 Fixed 3D model after column removal 3

In this removal case, 8 floor beams and 3 columns failed. This leads to failure of all the girders connecting to those failed floor beams. Eventually the frame column is removed will partially collapse. Figure 23 shows the failed structure and Figure 24 shows the fixed model.

6.1.4 Column Removal 4

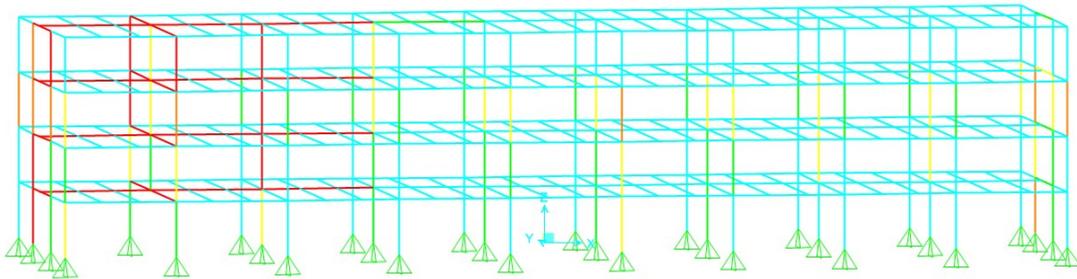


Figure 25 Failed 3D model after column removal 4

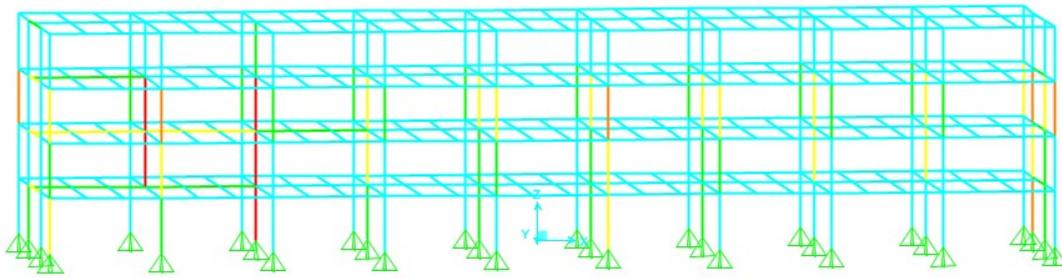


Figure 26 Fixed 3D model after column removal 4

In this removal case, 12 floor beams, 12 girders and 9 columns failed. This leads to failure of all the beams and girders connecting to those failed elements. Eventually the frame column is removed will partially collapse. The 5 columns in Figure 25 are shown as failed because they do not satisfy column to beam connection ratio and as explained in this chapter earlier, there are no available shapes to satisfy the ratio because 2 floor beams and 2 girders are connecting to a single column joint. Figure 26 shows the fixed model.

6.1.5 Column Removal 5

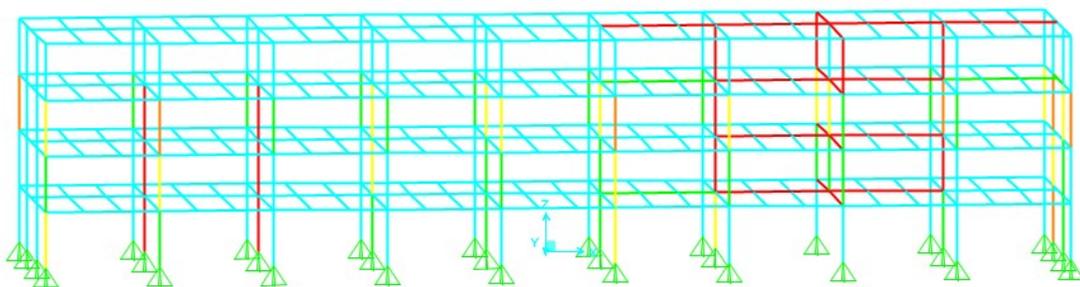


Figure 27 Failed 3D model after column removal 5

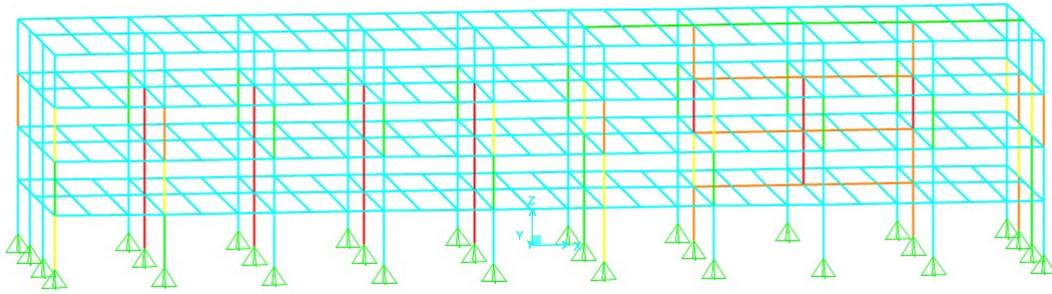


Figure 28 Fixed 3D model after column removal 5

In this removal case, 8 floor beams, 10 girders and 13 columns, 6 from previous case, failed. This leads to failure of all the beams and girders connecting to those failed elements. Eventually the frame column is removed will partially collapse. The 13 columns in Figure 27 are shown as failed because they do not satisfy column to beam connection ratio and as explained in this chapter earlier, there are no available shapes to satisfy the ratio because 2 floor beams and 2 girders are connecting to a single column joint. Figure 28 shows the fixed model.

6.1.6 Column Removal 6

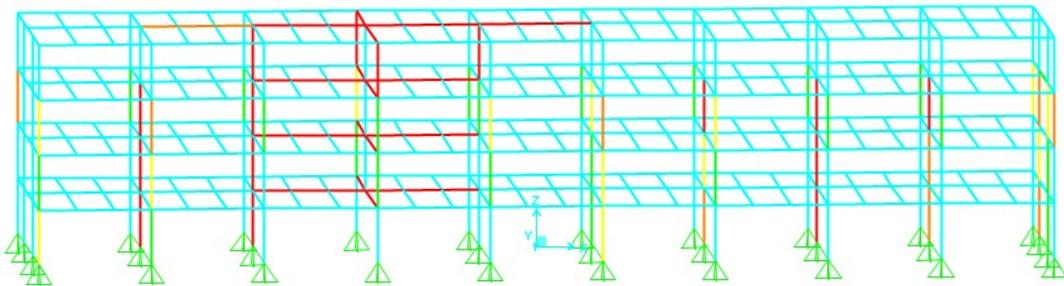


Figure 29 Failed 3D model after column removal 6

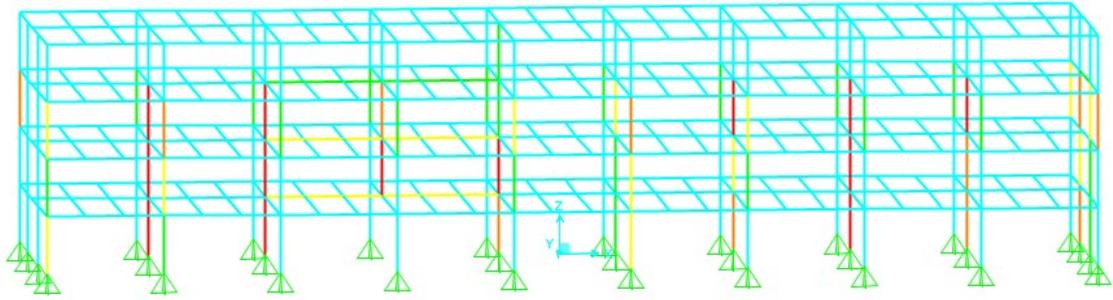


Figure 30 Fixed 3D model after column removal 6

In this removal case, 8 floor beams, 9 girders and 15 columns, 13 from previous case, failed. This leads to failure of all the beams and girders connecting to those failed elements. Eventually the frame column is removed will partially collapse. The 15 columns in Figure 29 are shown as failed because they do not satisfy column to beam connection ratio and as explained in this chapter earlier, there are no available shapes to satisfy the ratio because 2 floor beams and 2 girders are connecting to a single column joint. Figure 30 shows the fixed model.

6.1.7 Column Removal 7

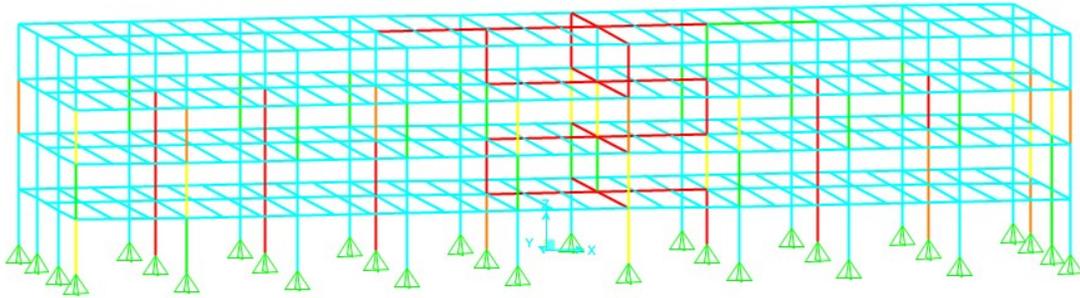


Figure 31 Failed 3D model after column removal 7

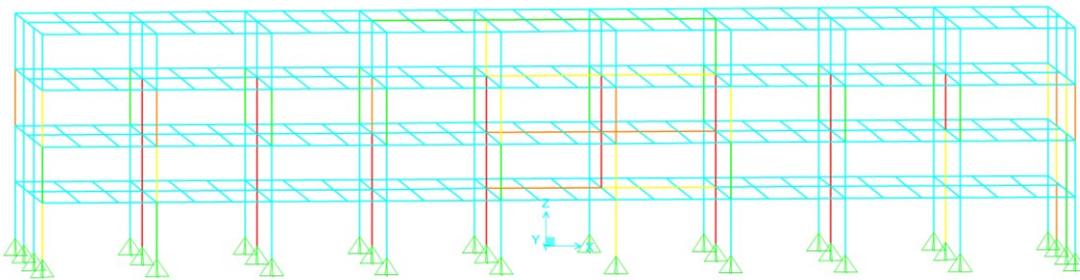


Figure 32 Fixed 3D model after column removal 7

In this removal case, 8 floor beams, 9 girders and 18 columns, 15 from previous case, failed. This leads to failure of all the beams and girders connecting to those failed elements. Eventually the frame column is removed will partially collapse. The 20 columns in Figure 31 are shown as failed because they do not satisfy column to beam connection ratio and as explained in this chapter earlier, there are no available shapes to satisfy the ratio because 2 floor beams and 2 girders are connecting to a single column joint. Figure 32 shows the fixed model.

7. CONCLUSIONS

7.1 Conclusion

In this project, load redistribution path for a steel frame under seismic loading is analyzed. It can be seen from the analysis and results that, once a structure is partially stable to resist ground motion, it also increases its resistance to vertical load bearing member loss. Even though the columns are removed from most critical locations, first story due to the influence area, mainly the members surrounding the removal failed under strong ground motion. All the girders connecting to removed column failed and their demand capacity ratio is almost 4 (in the simple beam problem in Chapter 3, the moments were almost 400% more). It is possible to fix the failures for exterior columns, however, when it comes to interior columns, some of the columns surrounding the removed column are failing because the beam/column ratio exceeds the capacity. With some hand calculations using the equation given in E3-1, AISC 341, it is the weak axis plastic section modulus controlling the beam/column ratio. Even though the section with the highest plastic section modulus in weak axis (AISC 360, shape W14x730), the error still occurs. Even though with final design, the structure is somewhat stable, it is inherently redundant. Due to manufacturing limitations, beam to column ratios are not satisfied because the beams needed very high weight shapes in order to transfer the high shear and moment caused in the system after the column removal.

It has been mentioned in Chapter 1, that blast loading can be fire explosions as well. One issue after earthquakes is post-earthquake fires. There are barely any

comments on that when designing for seismic resistance. The analysis steps in this project, with design iteration, partially add resistance to any post-earthquake effects. In a worst case scenario, because the load redistribution and redundancy is provided, the potential partial collapse will be delayed, leading to many life savings.

Although structural continuity and symmetry is very important both for seismic resistance and alternative load path, this research has shown that once all the required steps are followed, an irregular, asymmetric frame had a very decent response. Due to the distance in center of mass and center of rigidity, torsion effects were formed. However, this did not lead to any local torsional buckling of elements.

7.2 Future Research

As mentioned in AISC (2012), failure of connection is explicitly showing non-ductile design, the effect of beam-to-column connections will be examined. For that reason, similar analysis should be carried in a highly developed finite element software, where each element can be detailed. Even though it is recommended to keep the panel zones when a column is removed, in SAP 2000, the effect could not be examined.

Although GSA (2013) regulations only require one column removal at a time, examining effects of few column removal at a time can be beneficial and represent real life effects better. Removing a few columns from different stories at once can give interesting results.

Cost analysis and weight increase percentage and economical and seismic effects of these changes on the structure can be performed.

ASCE/SEI 7-10 has no load combination which includes both the extraordinary events and earthquake loading. By conducting analytical and experimental studies, a new load combination could be derived. That load combination will increase overall resistance of the structure and simplify the design process.

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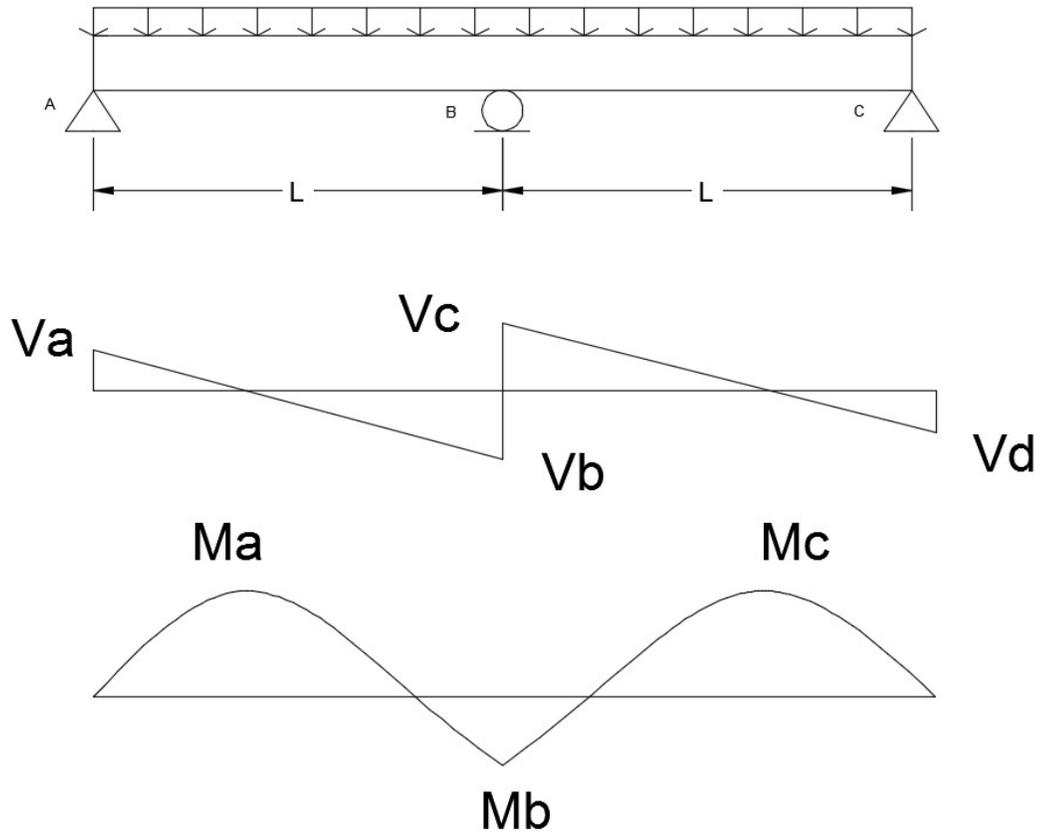
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APPENDIX A

CALCULATIONS FOR SIMPLE CONTINUOUS BEAM

Case 1



If R_B is redundant, $R_A = R_C = wL$.

Force-displacement relationship;

$$(v_B)_1 = \frac{5wL(2L)^4}{384EI} = \frac{5wL^4}{24EI}$$

$$(v_B)_2 = \frac{R_B(2L)^3}{48EI} = \frac{R_B L^3}{6EI}$$

$$v_B = (v_B)_1 - (v_B)_2 = 0$$

$$\frac{5wL^4}{24EI} = \frac{R_B L^3}{6EI} \Rightarrow R_B = \frac{5wL}{4}$$

Due to conservation of forces in vertical, y direction,

$$\sum F_v = 0$$

$$R_A = R_C = \frac{1}{2} \left(2wL - \frac{5wL}{4} \right) = \frac{3wL}{8}$$

$$V_a = -V_d = \frac{3wL}{8}$$

$$V_c = -V_b = \frac{5wL}{8}$$

$$M_a = M_c = \frac{3}{8} \times \frac{3}{8} \times \frac{wL^2}{2} = \frac{9wL^2}{128}$$

$$M_b = \frac{9wL^2}{128} - \frac{5}{8} \times \frac{5}{8} \times \frac{wL^2}{2} = -\frac{16wL^2}{128} = -\frac{wL^2}{8}$$

To find the maximum deflection, method of initial parameters is used;

$$\delta(x) = \delta_o + \theta_o x + \frac{M_o x^2}{2EI} - \frac{V_o x^3}{6EI} + \frac{wx^4}{24EI}$$

Initial conditions are:

$$\delta_o = M_o = 0$$

Therefore,

$$V_o = R_A = \frac{3wL}{8}$$

$$\delta(L) = \theta_o L - \frac{wL^4}{16EI} + \frac{wL^4}{24EI} = 0$$

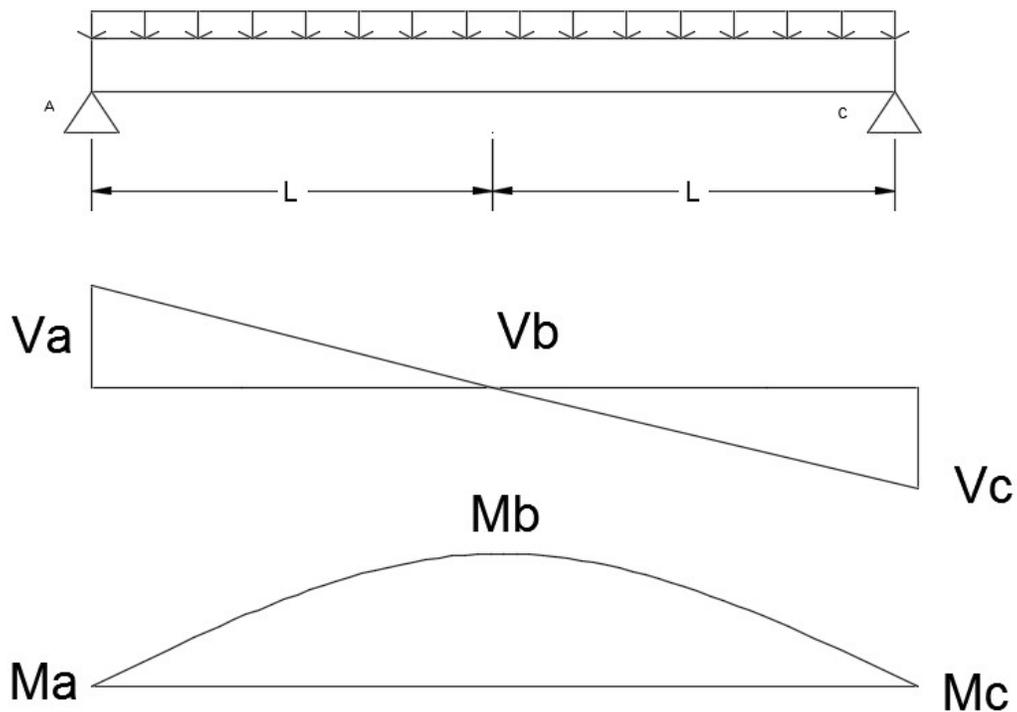
$$\theta_o L = \frac{wL^4}{16EI} - \frac{wL^4}{24EI}$$

$$\theta_o = \frac{wL^3}{48EI}$$

$$\delta(x) = \frac{wL^3 x}{48EI} - \frac{wLx^3}{16EI} + \frac{wL^4}{24EI}$$

$$\delta(L/2) = \frac{wL^3 L}{96EI} - \frac{wLL^3}{128EI} + \frac{wL^4}{24EI} = \frac{17wL^4}{384EI}$$

Case 2



$$R_A = R_C = wL$$

$$V_a = -V_c = wL$$

$$V_b = 0$$

$$M_a = M_c = 0$$

$$M_b = \frac{wL^2}{2}$$

To find the maximum deflection, method of initial parameters is used;

$$\delta(x) = \delta_o + \theta_o x + \frac{M_o x^2}{2EI} - \frac{V_o x^3}{6EI} + \frac{wx^4}{24EI}$$

Initial conditions are:

$$\delta_o = M_o = 0$$

Therefore,

$$V_o = R_A = wL$$

$$\delta(2L) = \theta_o 2L - \frac{8wL^4}{6EI} + \frac{16wL^4}{24EI} = 0$$

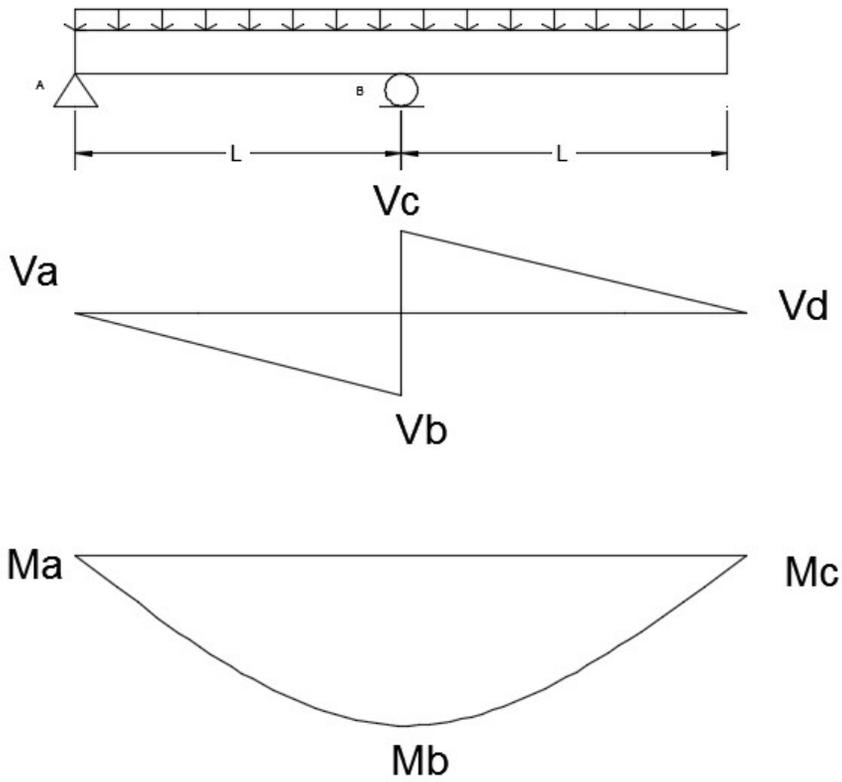
$$\theta_o = \frac{4wL^3}{6EI} - \frac{8wL^3}{24EI}$$

$$\theta_o = \frac{wL^3}{3EI}$$

$$\delta(L/2) = \frac{wL^3L}{6EI} - \frac{wLL^3}{48EI} + \frac{wL^4}{384EI} = \frac{57wL^4}{384EI}$$

$$\delta(L) = \frac{wL^3L}{3EI} - \frac{wLL^3}{6EI} + \frac{wL^4}{24EI} = \frac{5wL^4}{24EI}$$

Case 3



$$\sum M_a = 0$$

$$M_a = R_B L - \frac{w4L^2}{2} = 0$$

$$R_B = 2wL$$

$$V_c = -V_b = wL$$

$$V_a = V_d = 0$$

$$M_a = M_c = 0$$

$$M_b = \frac{wL^2}{2}$$

To find the maximum deflection, method of initial parameters is used;

$$\delta(x) = \delta_o + \theta_o x + \frac{M_o x^2}{2EI} - \frac{V_o x^3}{6EI} + \frac{wx^4}{24EI}$$

Initial conditions are:

$$\delta_o = M_o = 0$$

Therefore,

$$V_o = R_A = 0$$

$$\delta(L) = \theta_o L + \frac{wL^4}{24EI} = 0$$

$$\theta_o = -\frac{wL^3}{24EI}$$

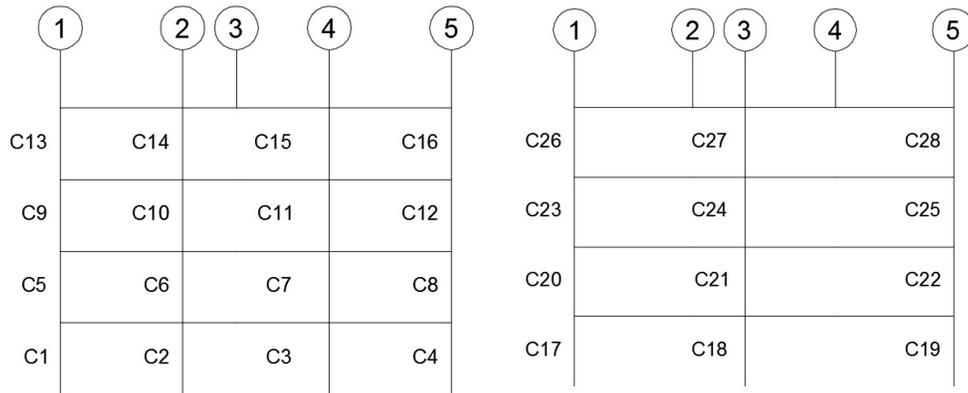
$$\delta(L/2) = -\frac{wLL^3}{48EI} + \frac{wL^4}{384EI} = -\frac{7wL^4}{384EI}$$

$$\delta(2L) = -\frac{2wLL^3}{24EI} + \frac{16wL^4}{24EI} = \frac{7wL^4}{12EI}$$

APPENDIX B

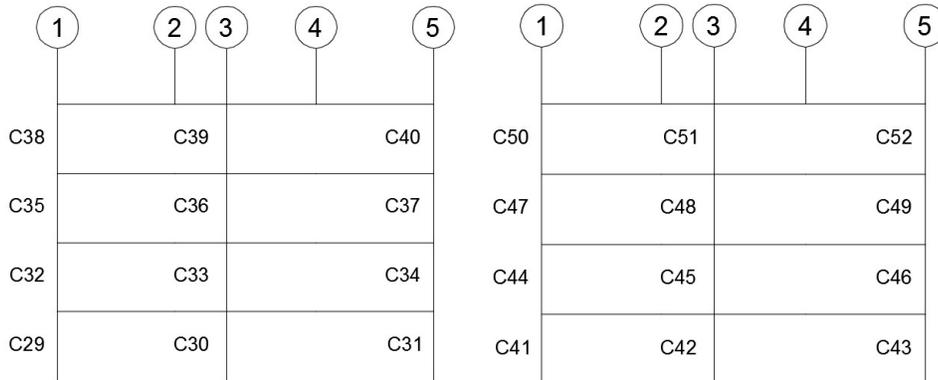
LABELS FOR EACH ELEMENT

Elevations of Gridline A-J



Gridline A

Gridline B



Gridline C

Gridline D

| | | | | | |
|-----|---|-----|---|---|-----|
| | ① | ② | ③ | ④ | ⑤ |
| C62 | | C63 | | | C64 |
| C59 | | C60 | | | C61 |
| C56 | | C57 | | | C58 |
| C53 | | C54 | | | C55 |

Gridline E

| | | | | | |
|-----|---|-----|---|---|-----|
| | ① | ② | ③ | ④ | ⑤ |
| C74 | | C75 | | | C76 |
| C71 | | C72 | | | C73 |
| C68 | | C69 | | | C70 |
| C65 | | C66 | | | C67 |

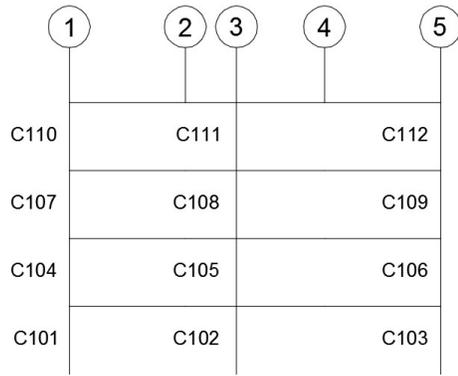
Gridline F

| | | | | | |
|-----|---|-----|---|---|-----|
| | ① | ② | ③ | ④ | ⑤ |
| C86 | | C87 | | | C88 |
| C83 | | C84 | | | C85 |
| C80 | | C81 | | | C82 |
| C77 | | C78 | | | C79 |

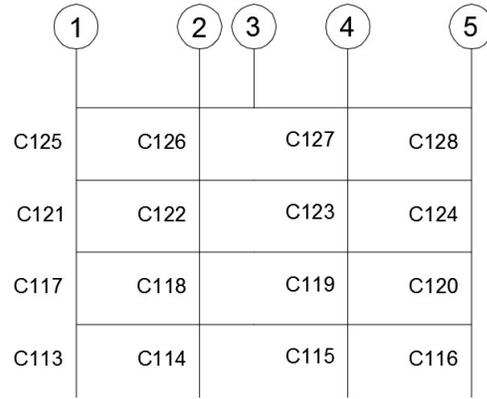
Gridline G

| | | | | | |
|-----|---|-----|---|---|------|
| | ① | ② | ③ | ④ | ⑤ |
| C98 | | C99 | | | C100 |
| C95 | | C96 | | | C97 |
| C92 | | C93 | | | C94 |
| C89 | | C90 | | | C91 |

Gridline H

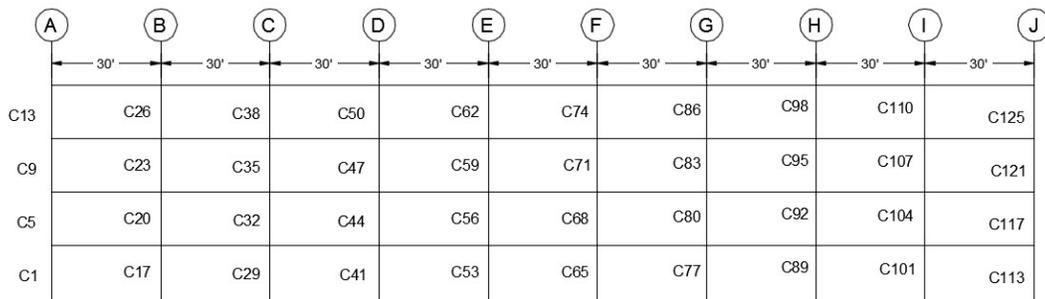


Gridline I

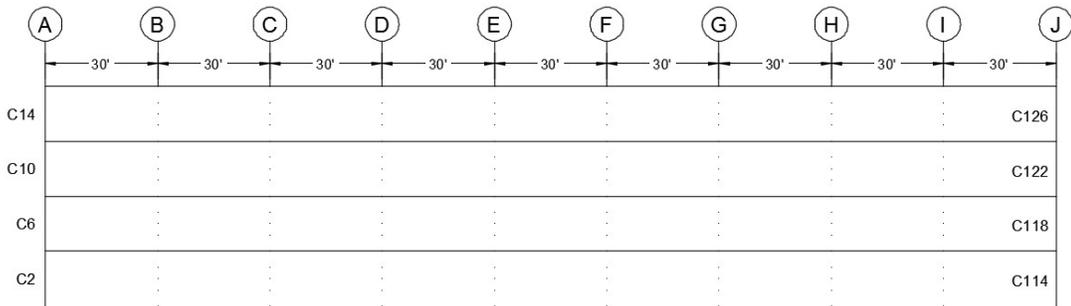


Gridline J

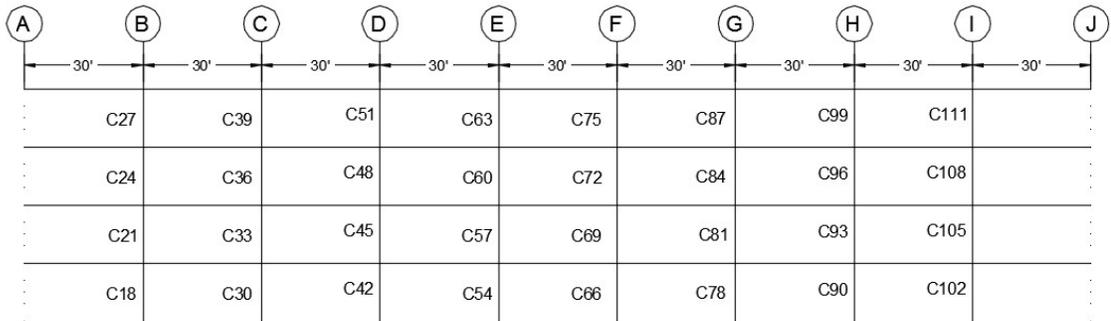
Elevations of Gridline 1-5



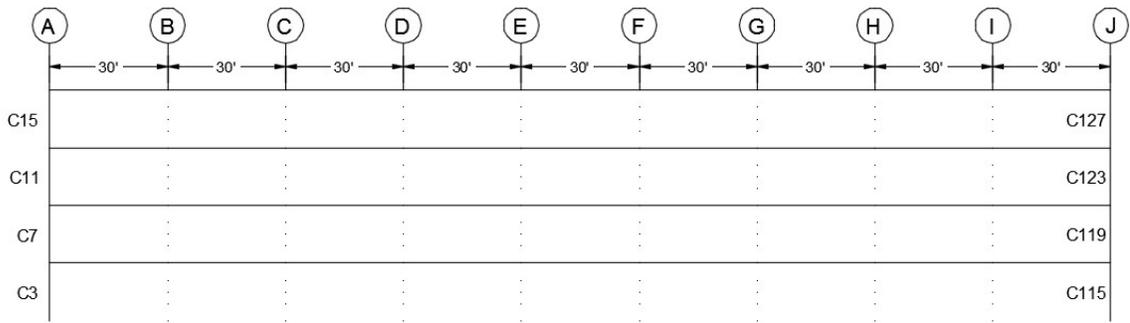
GRIDLINE 1



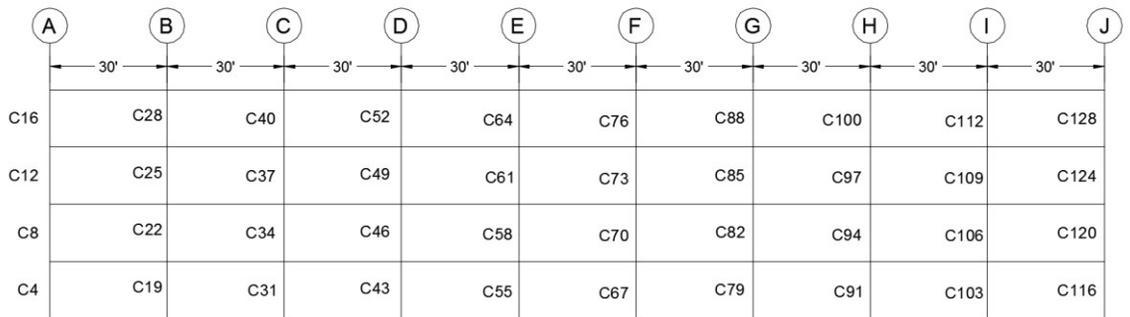
GRIDLINE 2



GRIDLINE 3



GRIDLINE 4

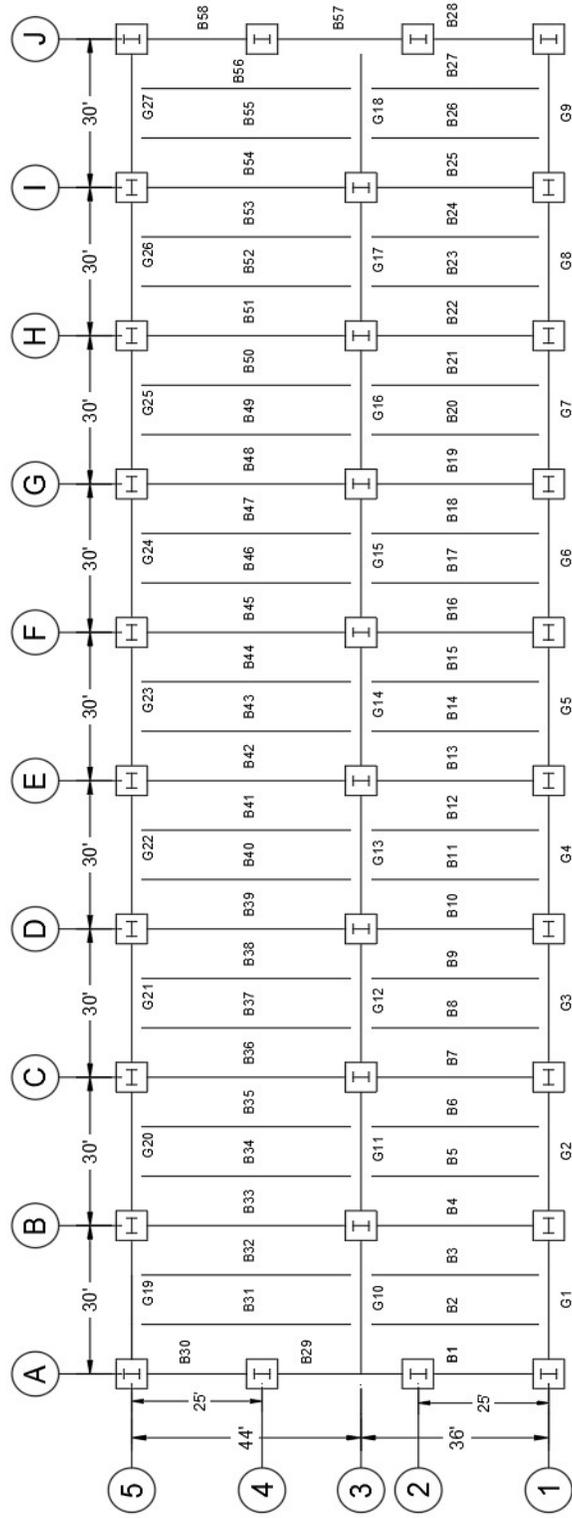


GRIDLINE 5

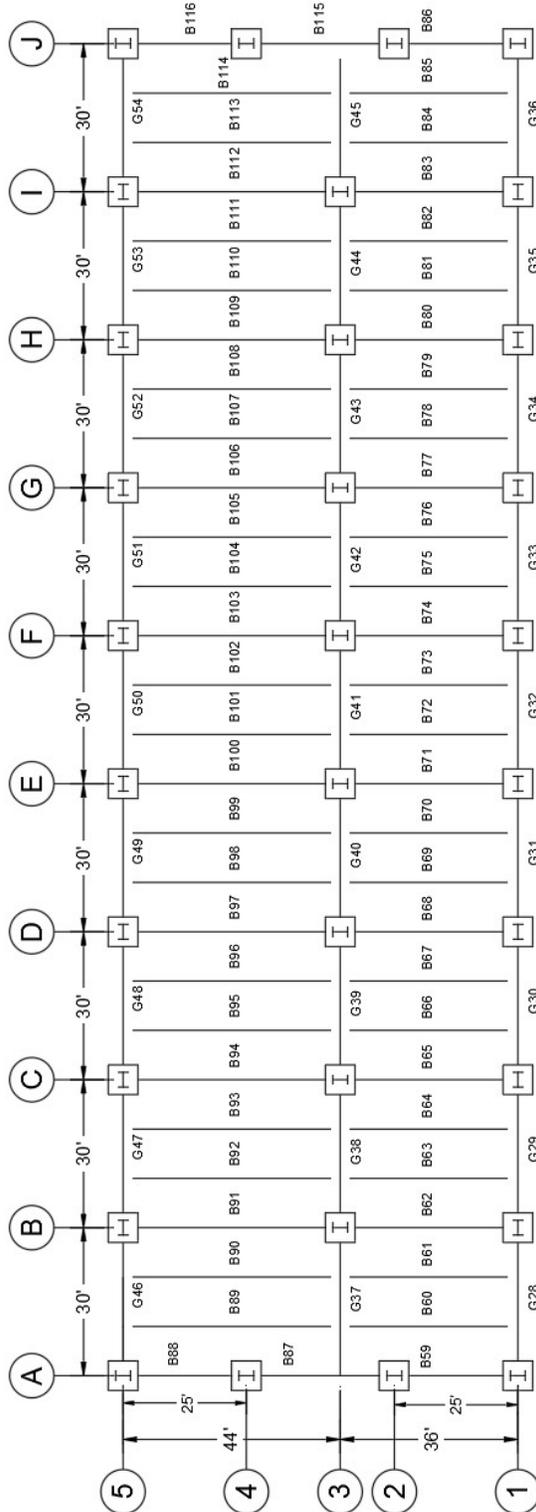
| | Section name | A | B | C | D | E | | | | | | |
|--------|--------------|-----|--------|-----|--------|---------|--------|---------|--------|---------|--------|---------|
| FIRST | 1 | C1 | W18x86 | C17 | W18x86 | C29 | W18x86 | C41 | W18x86 | C53 | W18x86 | |
| | 2 | C2 | W18x97 | 0 | C18 | W18x175 | C30 | W18x175 | C42 | W18x175 | C54 | W18x175 |
| | 3 | 0 | W18x97 | 0 | C19 | W18x86 | C31 | W18x86 | C43 | W18x86 | C55 | W18x86 |
| | 4 | C3 | W18x86 | 0 | C20 | W18x86 | C32 | W18x86 | C44 | W18x86 | C56 | W18x86 |
| SECOND | 1 | C4 | W18x86 | 0 | C21 | W18x130 | C33 | W18x130 | C45 | W18x130 | C57 | W18x130 |
| | 2 | C5 | W18x97 | 0 | C22 | W18x86 | C34 | W18x86 | C46 | W18x86 | C58 | W18x86 |
| | 3 | 0 | W18x97 | 0 | C23 | W18x55 | C35 | W18x55 | C47 | W18x55 | C59 | W18x55 |
| | 4 | C6 | W18x86 | 0 | C24 | W18x86 | C36 | W18x86 | C48 | W18x86 | C60 | W18x86 |
| | 5 | 0 | W18x60 | 0 | C25 | W18x55 | C37 | W18x55 | C49 | W18x55 | C61 | W18x55 |
| THIRD | 1 | C7 | W18x86 | 0 | C26 | W18x55 | C38 | W18x55 | C50 | W18x55 | C62 | W18x55 |
| | 2 | C8 | W18x40 | 0 | C27 | W18x55 | C39 | W18x55 | C51 | W18x55 | C63 | W18x55 |
| | 3 | 0 | W18x60 | 0 | C28 | W18x55 | C40 | W18x55 | C52 | W18x55 | C64 | W18x55 |
| | 4 | C9 | W18x40 | 0 | C29 | W18x86 | C41 | W18x86 | C53 | W18x86 | C65 | W18x86 |
| | 5 | 0 | W18x60 | 0 | C30 | W18x175 | C42 | W18x175 | C54 | W18x175 | C66 | W18x175 |
| FOURTH | 1 | C10 | W18x60 | 0 | C31 | W18x86 | C43 | W18x86 | C55 | W18x86 | C67 | W18x86 |
| | 2 | C11 | W18x40 | 0 | C32 | W18x86 | C44 | W18x86 | C56 | W18x86 | C68 | W18x86 |
| | 3 | 0 | W18x60 | 0 | C33 | W18x130 | C45 | W18x130 | C57 | W18x130 | C69 | W18x130 |
| | 4 | C12 | W18x86 | 0 | C34 | W18x86 | C46 | W18x86 | C58 | W18x86 | C70 | W18x86 |
| | 5 | 0 | W18x60 | 0 | C35 | W18x55 | C47 | W18x55 | C59 | W18x55 | C71 | W18x55 |
| | 6 | C13 | W18x40 | 0 | C36 | W18x86 | C48 | W18x86 | C60 | W18x86 | C72 | W18x86 |
| | 7 | 0 | W18x60 | 0 | C37 | W18x55 | C49 | W18x55 | C61 | W18x55 | C73 | W18x55 |
| | 8 | C14 | W18x40 | 0 | C38 | W18x55 | C50 | W18x55 | C62 | W18x55 | C74 | W18x55 |
| | 9 | 0 | W18x60 | 0 | C39 | W18x55 | C51 | W18x55 | C63 | W18x55 | C75 | W18x55 |
| | 10 | C15 | W18x86 | 0 | C40 | W18x55 | C52 | W18x55 | C64 | W18x55 | C76 | W18x55 |
| | 11 | 0 | W18x60 | 0 | C41 | W18x86 | C53 | W18x86 | C65 | W18x86 | C77 | W18x86 |
| | 12 | C16 | W18x40 | 0 | C42 | W18x175 | C54 | W18x175 | C66 | W18x175 | C78 | W18x175 |

| | Section name | F | G | H | I | J | | | | | |
|--------|--------------|-----|---------|-----|---------|------|---------|------|---------|------|--------|
| FIRST | 1 | C65 | W18x86 | C77 | W18x86 | C89 | W18x86 | C101 | W18x86 | C113 | W18x86 |
| | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C114 | W18x97 |
| | 3 | C66 | W18x175 | C78 | W18x175 | C90 | W18x175 | C102 | W18x175 | 0 | 0 |
| | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C115 | W18x97 |
| | 5 | C67 | W18x86 | C79 | W18x86 | C91 | W18x86 | C103 | W18x86 | C116 | W18x86 |
| SECOND | 1 | C68 | W18x86 | C80 | W18x86 | C92 | W18x86 | C104 | W18x86 | C117 | W18x86 |
| | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C118 | W18x97 |
| | 3 | C69 | W18x130 | C81 | W18x130 | C93 | W18x130 | C105 | W18x130 | 0 | 0 |
| | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C119 | W18x97 |
| | 5 | C70 | W18x86 | C82 | W18x86 | C94 | W18x86 | C106 | W18x86 | C120 | W18x86 |
| THIRD | 1 | C71 | W18x55 | C83 | W18x55 | C95 | W18x55 | C107 | W18x55 | C121 | W18x40 |
| | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C122 | W18x60 |
| | 3 | C72 | W18x86 | C84 | W18x86 | C96 | W18x86 | C108 | W18x86 | 0 | 0 |
| | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C123 | W18x60 |
| | 5 | C73 | W18x55 | C85 | W18x55 | C97 | W18x55 | C109 | W18x55 | C124 | W18x40 |
| FOURTH | 1 | C74 | W18x55 | C86 | W18x55 | C98 | W18x55 | C110 | W18x55 | C125 | W18x40 |
| | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C126 | W18x60 |
| | 3 | C75 | W18x55 | C87 | W18x55 | C99 | W18x55 | C111 | W18x55 | 0 | 0 |
| | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | C127 | W18x60 |
| | 5 | C76 | W18x55 | C88 | W18x55 | C100 | W18x55 | C112 | W18x55 | C128 | W18x40 |

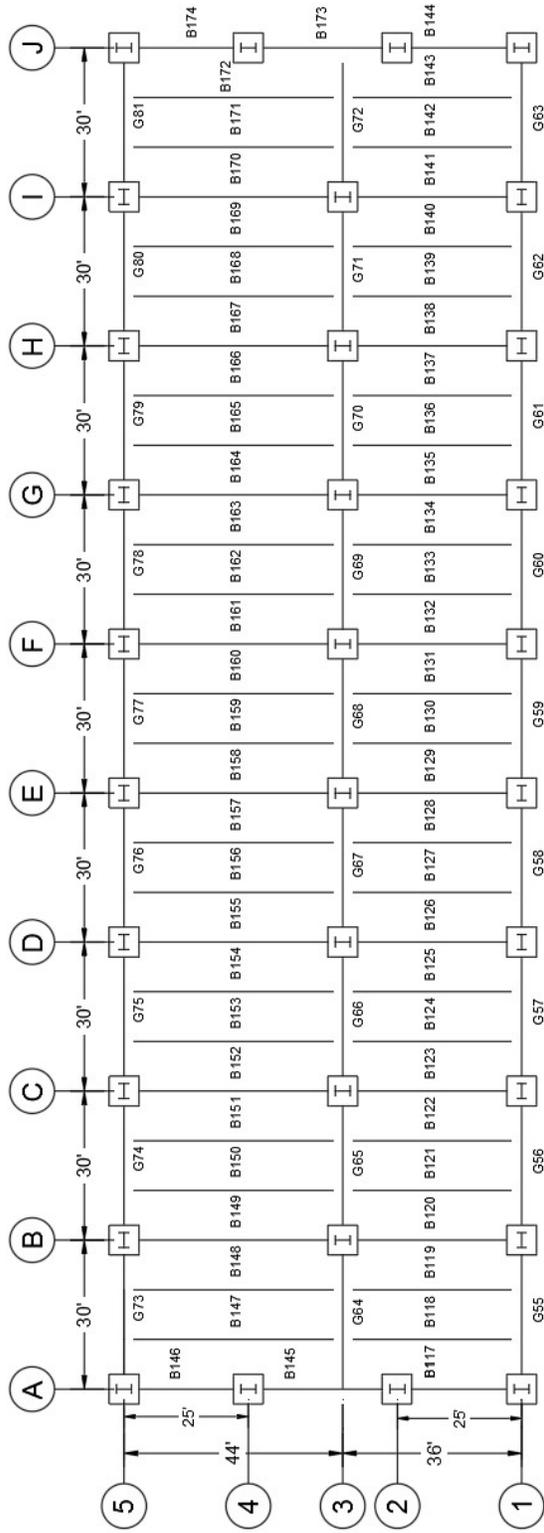
Floor plans of each story



SECOND FLOOR PLAN



THIRD FLOOR PLAN



FOURTH FLOOR PLAN

| SECOND FLOOR | B1 | W24x94 | B15 | W16x31 | B29 | W24x76 | B43 | W21x44 |
|--------------|-----|--------|-----|--------|-----|--------|-----|--------|
| | B2 | W16x31 | B16 | W16x31 | B30 | W24x94 | B44 | W21x44 |
| | B3 | W16x31 | B17 | W16x31 | B31 | W21x44 | B45 | W21x44 |
| | B4 | W16x31 | B18 | W16x31 | B32 | W21x44 | B46 | W21x44 |
| | B5 | W16x31 | B19 | W16x31 | B33 | W21x44 | B47 | W21x44 |
| | B6 | W16x31 | B20 | W16x31 | B34 | W21x44 | B48 | W21x44 |
| | B7 | W16x31 | B21 | W16x31 | B35 | W21x44 | B49 | W21x44 |
| | B8 | W16x31 | B22 | W16x31 | B36 | W21x44 | B50 | W21x44 |
| | B9 | W16x31 | B23 | W16x31 | B37 | W21x44 | B51 | W21x44 |
| | B10 | W16x31 | B24 | W16x31 | B38 | W21x44 | B52 | W21x44 |
| | B11 | W16x31 | B25 | W16x31 | B39 | W21x44 | B53 | W21x44 |
| | B12 | W16x31 | B26 | W16x31 | B40 | W21x44 | B54 | W21x44 |
| | B13 | W16x31 | B27 | W16x31 | B41 | W21x44 | B55 | W21x44 |
| | B14 | W16x31 | B28 | W24x94 | B42 | W21x44 | B56 | W21x44 |
| | | | | | | | B57 | W24x76 |
| | | | | | | | B58 | W24x94 |

| THIRD FLOOR | B59 | W24x62 | B73 | W16x31 | B87 | W24x68 | B101 | W21x44 |
|-------------|-----|--------|-----|--------|------|--------|------|--------|
| | B60 | W16x31 | B74 | W16x31 | B88 | W24x62 | B102 | W21x44 |
| | B61 | W16x31 | B75 | W16x31 | B89 | W21x44 | B103 | W21x44 |
| | B62 | W16x31 | B76 | W16x31 | B90 | W21x44 | B104 | W21x44 |
| | B63 | W16x31 | B77 | W16x31 | B91 | W21x44 | B105 | W21x44 |
| | B64 | W16x31 | B78 | W16x31 | B92 | W21x44 | B106 | W21x44 |
| | B65 | W16x31 | B79 | W16x31 | B93 | W21x44 | B107 | W21x44 |
| | B66 | W16x31 | B80 | W16x31 | B94 | W21x44 | B108 | W21x44 |
| | B67 | W16x31 | B81 | W16x31 | B95 | W21x44 | B109 | W21x44 |
| | B68 | W16x31 | B82 | W16x31 | B96 | W21x44 | B110 | W21x44 |
| | B69 | W16x31 | B83 | W16x31 | B97 | W21x44 | B111 | W21x44 |
| | B70 | W16x31 | B84 | W16x31 | B98 | W21x44 | B112 | W21x44 |
| | B71 | W16x31 | B85 | W16x31 | B99 | W21x44 | B113 | W21x44 |
| | B72 | W16x31 | B86 | W24x62 | B100 | W21x44 | B114 | W21x44 |
| | | | | | | | B115 | W24x68 |
| | | | | | | | B116 | W24x62 |

| FOURTH FLOOR | B117 | W24x62 | B131 | W16x31 | B145 | W24x68 | B159 | W21x44 |
|--------------|------|--------|------|--------|------|--------|------|--------|
| | B118 | W16x31 | B132 | W16x31 | B146 | W24x62 | B160 | W21x44 |
| | B119 | W16x31 | B133 | W16x31 | B147 | W21x44 | B161 | W21x44 |
| | B120 | W16x31 | B134 | W16x31 | B148 | W21x44 | B162 | W21x44 |
| | B121 | W16x31 | B135 | W16x31 | B149 | W21x44 | B163 | W21x44 |
| | B122 | W16x31 | B136 | W16x31 | B150 | W21x44 | B164 | W21x44 |
| | B123 | W16x31 | B137 | W16x31 | B151 | W21x44 | B165 | W21x44 |
| | B124 | W16x31 | B138 | W16x31 | B152 | W21x44 | B166 | W21x44 |
| | B125 | W16x31 | B139 | W16x31 | B153 | W21x44 | B167 | W21x44 |
| | B126 | W16x31 | B140 | W16x31 | B154 | W21x44 | B168 | W21x44 |
| | B127 | W16x31 | B141 | W16x31 | B155 | W21x44 | B169 | W21x44 |
| | B128 | W16x31 | B142 | W16x31 | B156 | W21x44 | B170 | W21x44 |
| | B129 | W16x31 | B143 | W16x31 | B157 | W21x44 | B171 | W21x44 |
| | B130 | W16x31 | B144 | W24x62 | B158 | W21x44 | B172 | W21x44 |
| | | | | | | | B173 | W24x68 |
| | | | | | | | B174 | W24x62 |

| ROOF FLOOR | B175 | W24x55 | B189 | W16x31 | B203 | W24x55 | B217 | W21x44 |
|------------|------|--------|------|--------|------|--------|------|--------|
| | B176 | W16x31 | B190 | W16x31 | B204 | W24x55 | B218 | W21x44 |
| | B177 | W16x31 | B191 | W16x31 | B205 | W21x44 | B219 | W21x44 |
| | B178 | W16x31 | B192 | W16x31 | B206 | W21x44 | B220 | W21x44 |
| | B179 | W16x31 | B193 | W16x31 | B207 | W21x44 | B221 | W21x44 |
| | B180 | W16x31 | B194 | W16x31 | B208 | W21x44 | B222 | W21x44 |
| | B181 | W16x31 | B195 | W16x31 | B209 | W21x44 | B223 | W21x44 |
| | B182 | W16x31 | B196 | W16x31 | B210 | W21x44 | B224 | W21x44 |
| | B183 | W16x31 | B197 | W16x31 | B211 | W21x44 | B225 | W21x44 |
| | B184 | W16x31 | B198 | W16x31 | B212 | W21x44 | B226 | W21x44 |
| | B185 | W16x31 | B199 | W16x31 | B213 | W21x44 | B227 | W21x44 |
| | B186 | W16x31 | B200 | W16x31 | B214 | W21x44 | B228 | W21x44 |
| | B187 | W16x31 | B201 | W16x31 | B215 | W21x44 | B229 | W21x44 |
| | B188 | W16x31 | B202 | W24x55 | B216 | W21x44 | B230 | W21x44 |
| | | | | | | | B231 | W24x55 |
| | | | | | | | B232 | W24x55 |

| SECOND FLOOR | G1 | W24x68 | G10 | W24x62 | G19 | W24x68 |
|--------------|----|--------|-----|--------|-----|--------|
| | G2 | W24x68 | G11 | W24x62 | G20 | W24x68 |
| | G3 | W24x68 | G12 | W24x62 | G21 | W24x68 |
| | G4 | W24x68 | G13 | W24x62 | G22 | W24x68 |
| | G5 | W24x68 | G14 | W24x62 | G23 | W24x68 |
| | G6 | W24x68 | G15 | W24x62 | G24 | W24x68 |
| | G7 | W24x68 | G16 | W24x62 | G25 | W24x68 |
| | G8 | W24x68 | G17 | W24x62 | G26 | W24x68 |
| | G9 | W24x68 | G18 | W24x62 | G27 | W24x68 |

| THIRD FLOOR | G28 | W24x68 | G37 | W24x62 | G46 | W24x68 |
|-------------|-----|--------|-----|--------|-----|--------|
| | G29 | W24x68 | G38 | W24x62 | G47 | W24x68 |
| | G30 | W24x68 | G39 | W24x62 | G48 | W24x68 |
| | G31 | W24x68 | G40 | W24x62 | G49 | W24x68 |
| | G32 | W24x68 | G41 | W24x62 | G50 | W24x68 |
| | G33 | W24x68 | G42 | W24x62 | G51 | W24x68 |
| | G34 | W24x68 | G43 | W24x62 | G52 | W24x68 |
| | G35 | W24x68 | G44 | W24x62 | G53 | W24x68 |
| | G36 | W24x68 | G45 | W24x62 | G54 | W24x68 |

| FOURTH FLOOR | G55 | W24x68 | G64 | W24x62 | G73 | W24x68 |
|--------------|-----|--------|-----|--------|-----|--------|
| | G56 | W24x68 | G65 | W24x62 | G74 | W24x68 |
| | G57 | W24x68 | G66 | W24x62 | G75 | W24x68 |
| | G58 | W24x68 | G67 | W24x62 | G76 | W24x68 |
| | G59 | W24x68 | G68 | W24x62 | G77 | W24x68 |
| | G60 | W24x68 | G69 | W24x62 | G78 | W24x68 |
| | G61 | W24x68 | G70 | W24x62 | G79 | W24x68 |
| | G62 | W24x68 | G71 | W24x62 | G80 | W24x68 |
| | G63 | W24x68 | G72 | W24x62 | G81 | W24x68 |

| | | | | | | |
|---------------|-----|--------|-----|--------|------|--------|
| ROOF FLOOR | G82 | W24x55 | G91 | W24x62 | G100 | W24x55 |
| | G83 | W24x55 | G92 | W24x62 | G101 | W24x55 |
| | G84 | W24x55 | G93 | W24x62 | G102 | W24x55 |
| | G85 | W24x55 | G94 | W24x62 | G103 | W24x55 |
| | G86 | W24x55 | G95 | W24x62 | G104 | W24x55 |
| | G87 | W24x55 | G96 | W24x62 | G105 | W24x55 |
| | G88 | W24x55 | G97 | W24x62 | G106 | W24x55 |
| | G89 | W24x55 | G98 | W24x62 | G107 | W24x55 |
| | G90 | W24x55 | G99 | W24x62 | G108 | W24x55 |

APPENDIX C
SLAB CALCULATION

To ignore the composite interaction and slab modelling, an area for slab has been defined. Mass and loads are calculated and applied manually to the slabs. SAP 2000 requires mass input as mass per unit area. Below is the mass used for per slab area.

For 4 inch metal deck, it is assumed to be structural steel.

$$m = \frac{w}{g}$$

$$g = 32.2 \text{ ft} / \text{sec}^2$$

$$d = \frac{w}{V}$$

$$d_{\text{steel}} = 488.2 \text{ lb} / \text{ft}^3$$

$$\frac{\text{mass}}{\text{unit area}} = \frac{488.2 * 0.25}{32.2 * 1000} \cong 0.004 \frac{\text{kips} * \text{s}^2}{\text{ft}^3}$$

For 4.5 inch reinforced concrete deck below is the mass per area.

$$m = \frac{w}{g}$$

$$g = 32.2 \text{ ft} / \text{sec}^2$$

$$d = \frac{w}{V}$$

$$d_{\text{RC}} = 150 \text{ lb} / \text{ft}^3$$

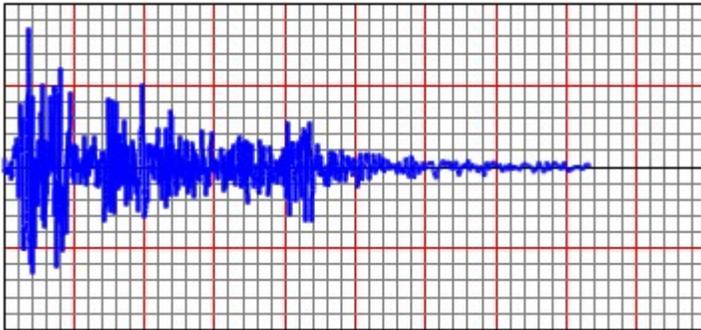
$$\frac{\text{mass}}{\text{unit area}} = \frac{150 * 0.375}{32.2 * 1000} \cong 0.0017 \frac{\text{kips} * \text{s}^2}{\text{ft}^3}$$

Total mass per area is;

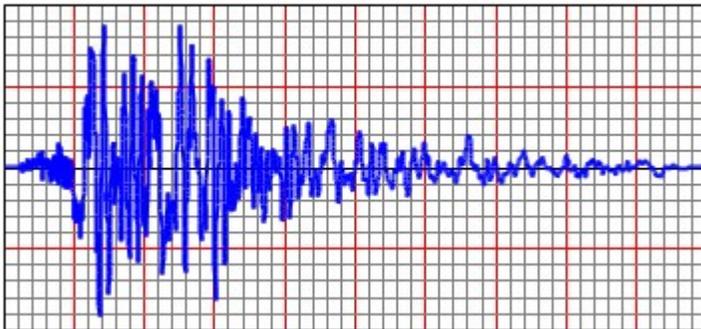
$$\sum \frac{mass}{area} = 0.0057 \frac{kips * s^2}{ft^3}$$

APPENDIX D
EARTHQUAKE PLOTS

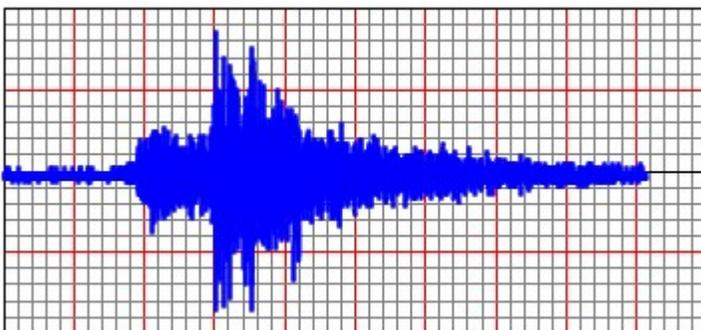
El Centro



Northridge



Izmit

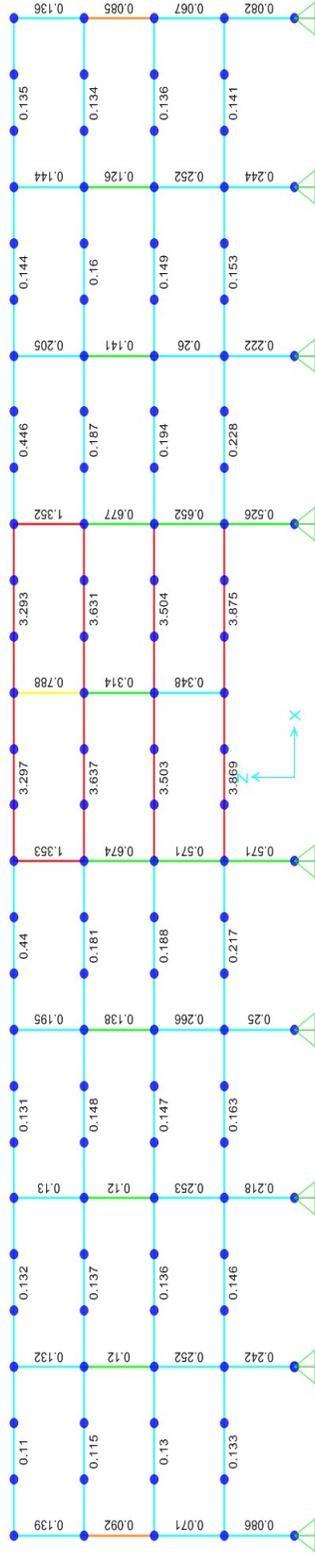
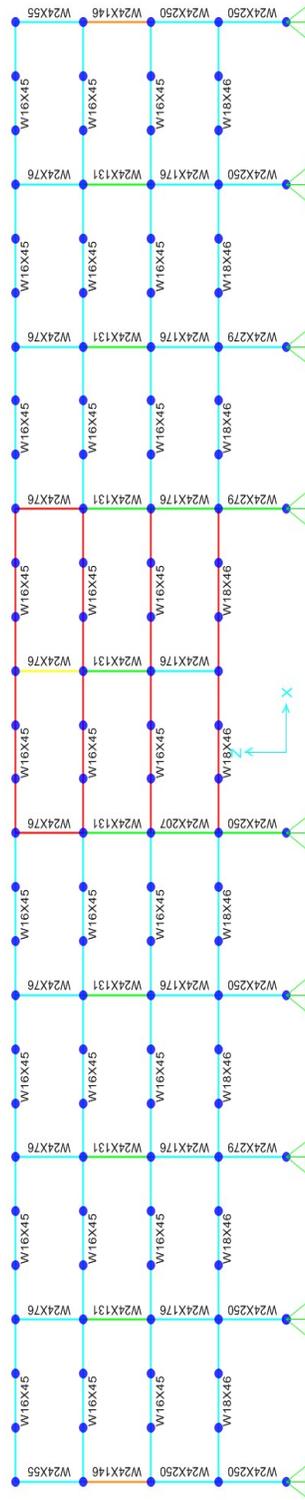


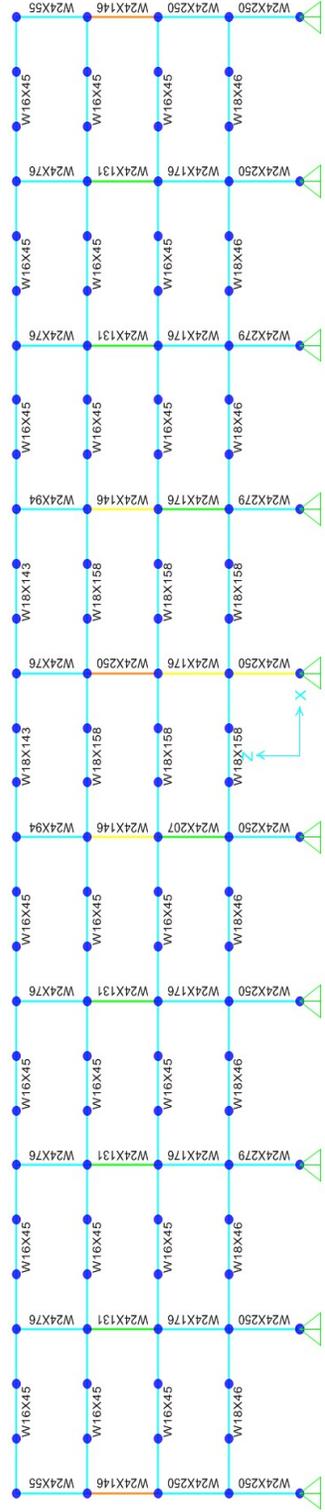
APPENDIX E

SCREENSHOTS OF THE MODEL AFTER THE COLUMN REMOVAL

REMOVAL 1

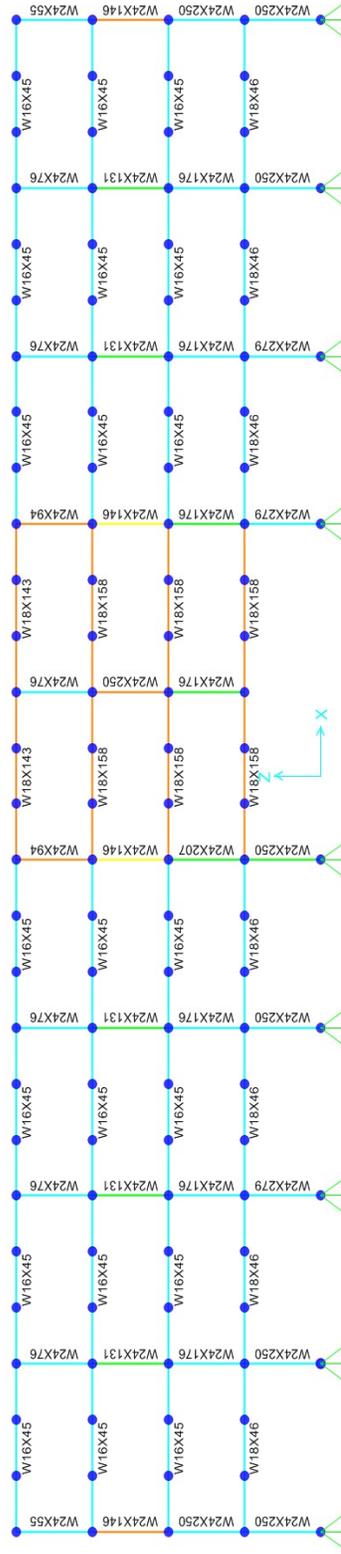
STEP 1



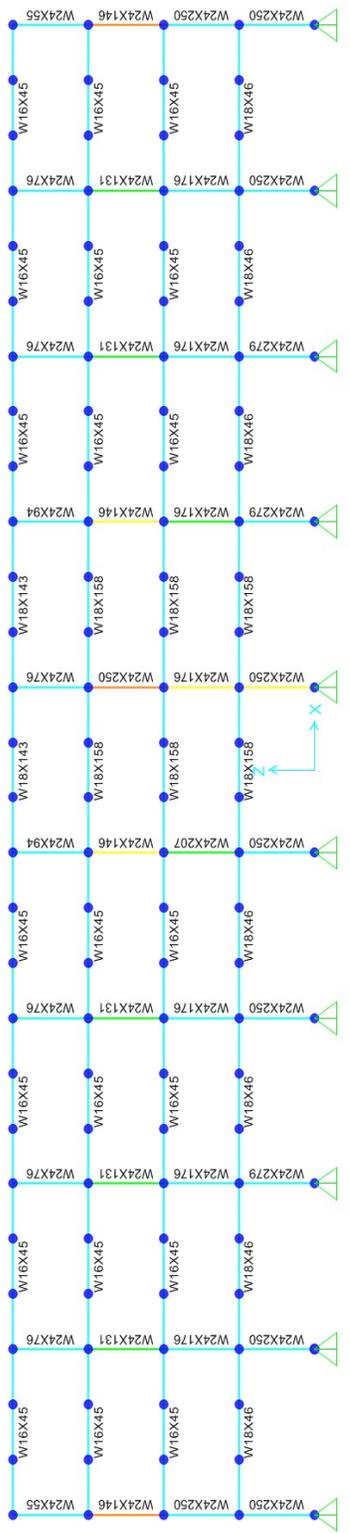


STEP 5

STEP 6



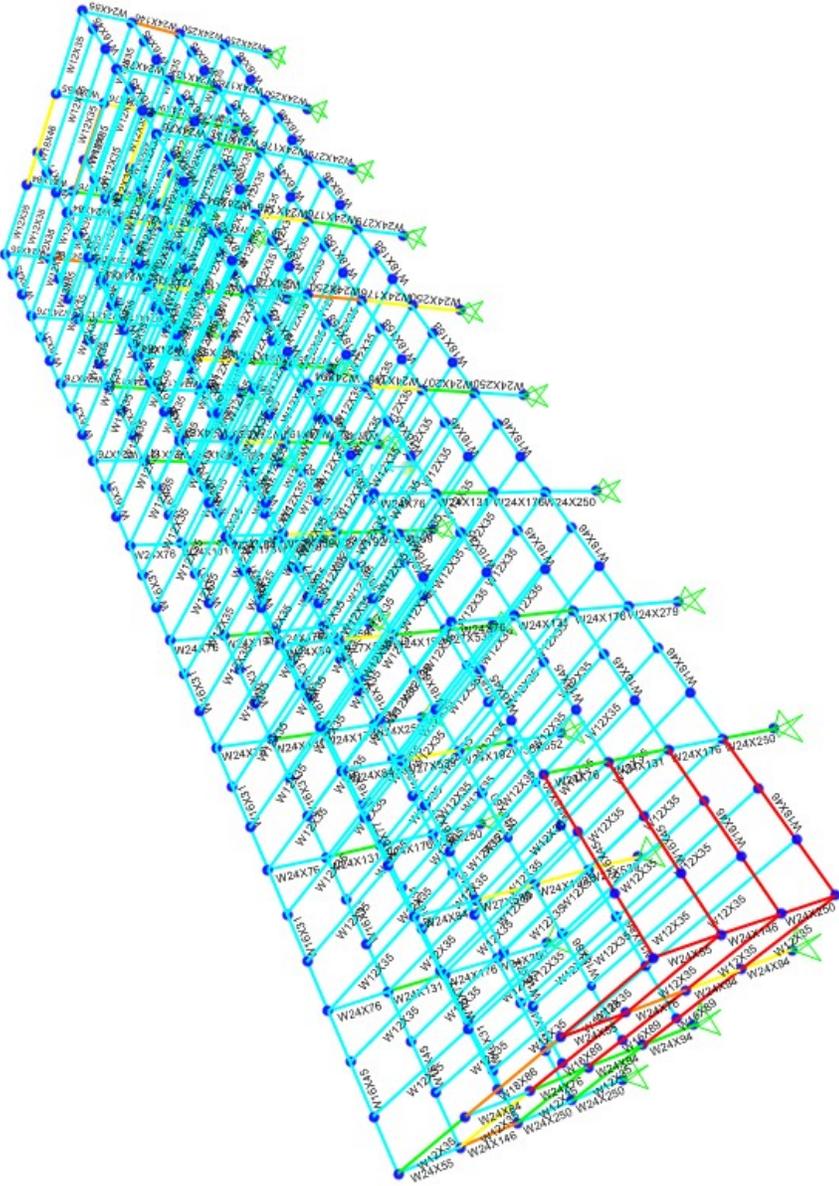
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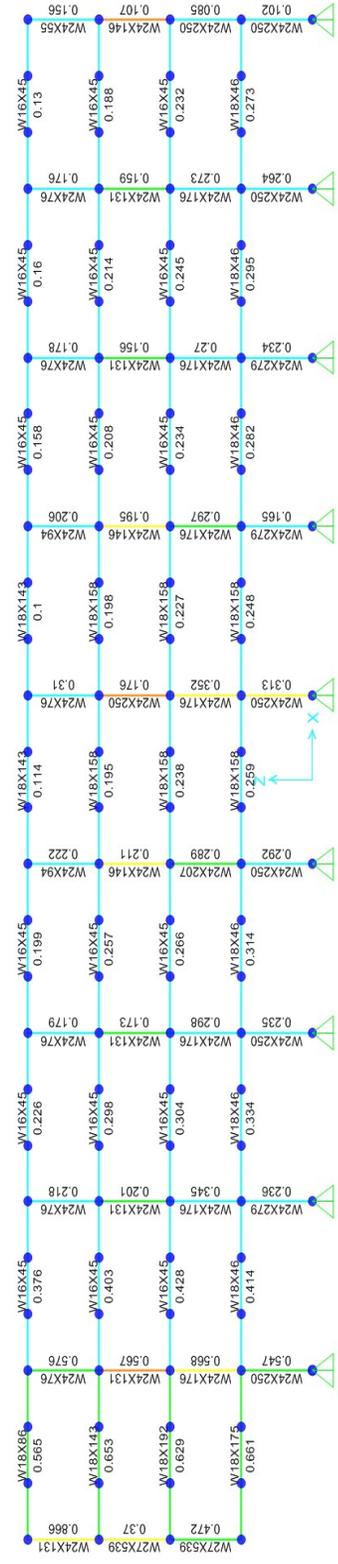
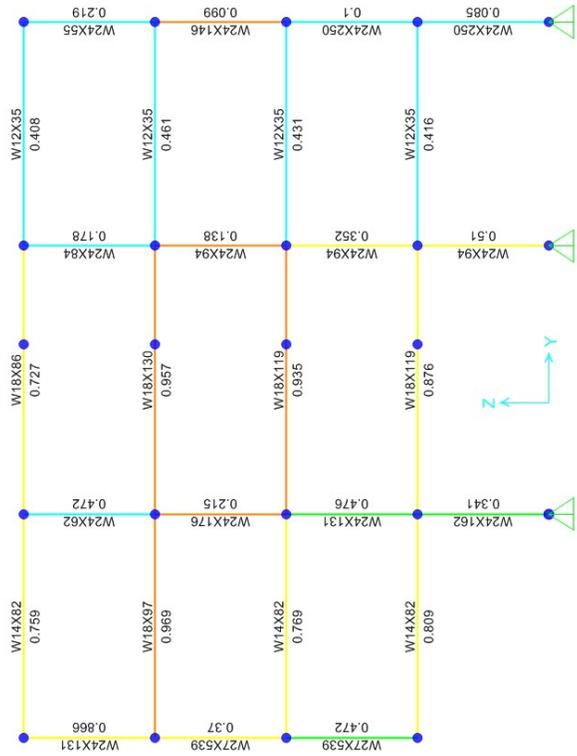
REMOVAL 2

THE FINAL MODEL GIVEN ABOVE IS USED

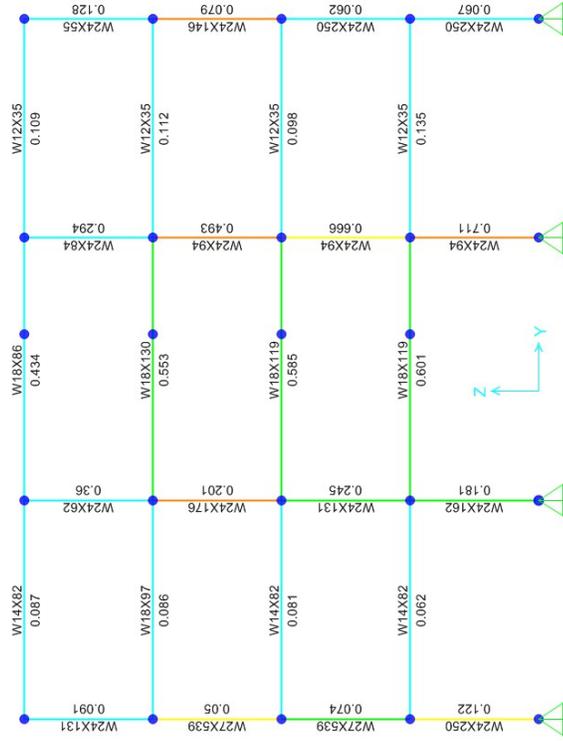
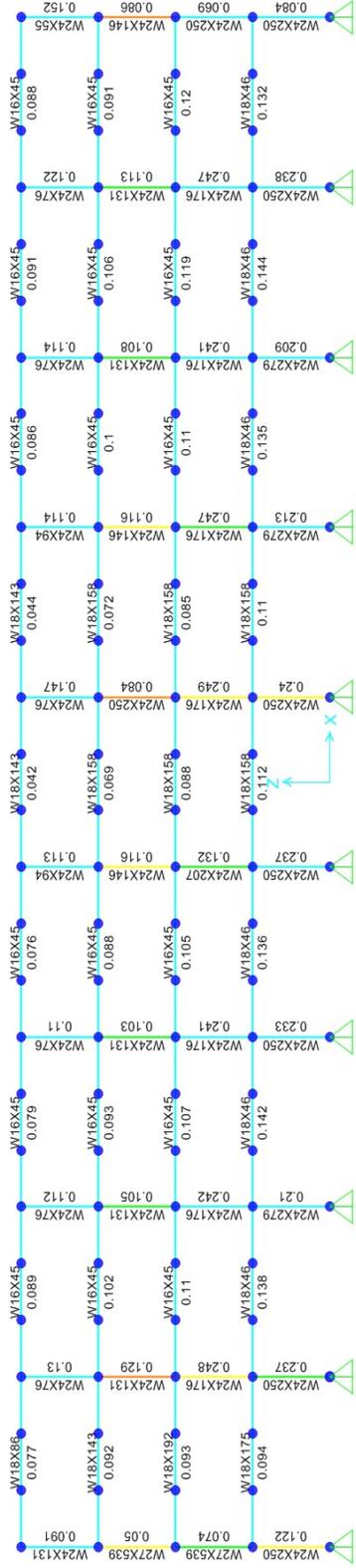
STEP 1



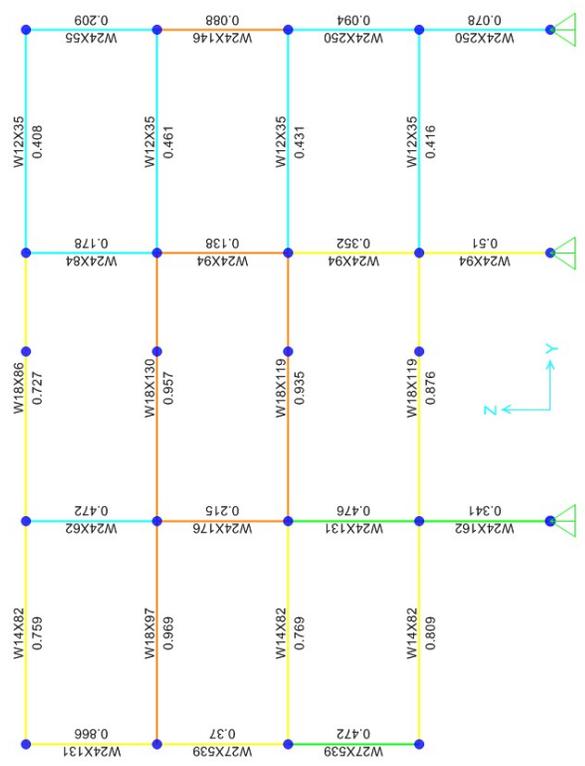
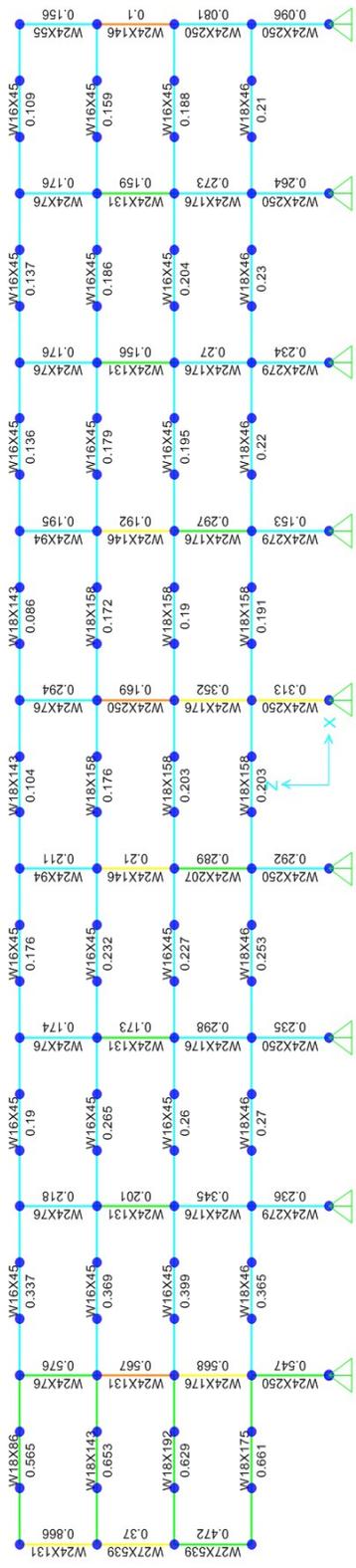
STEP 2



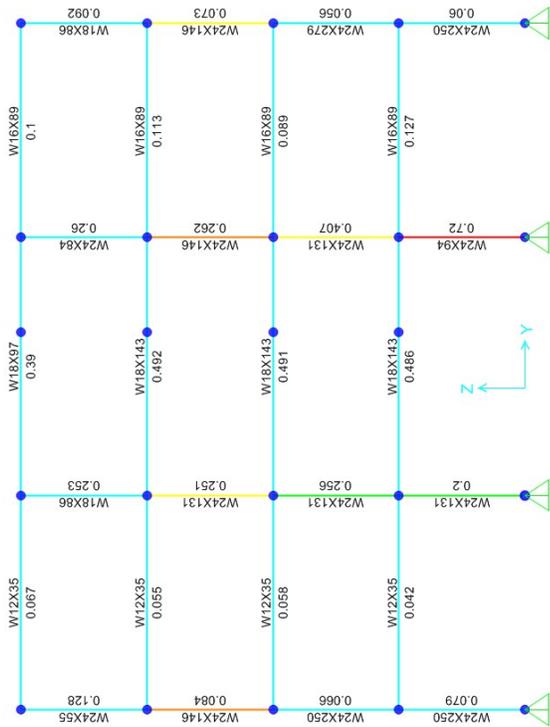
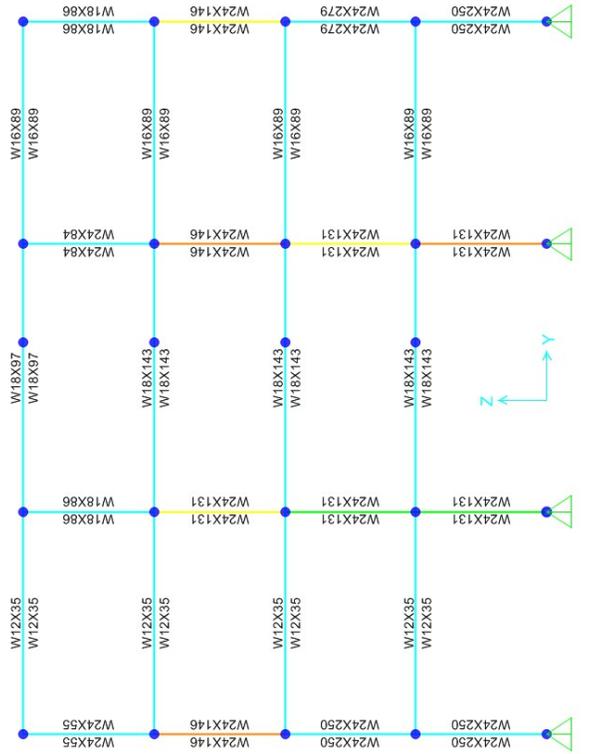
STEP 3



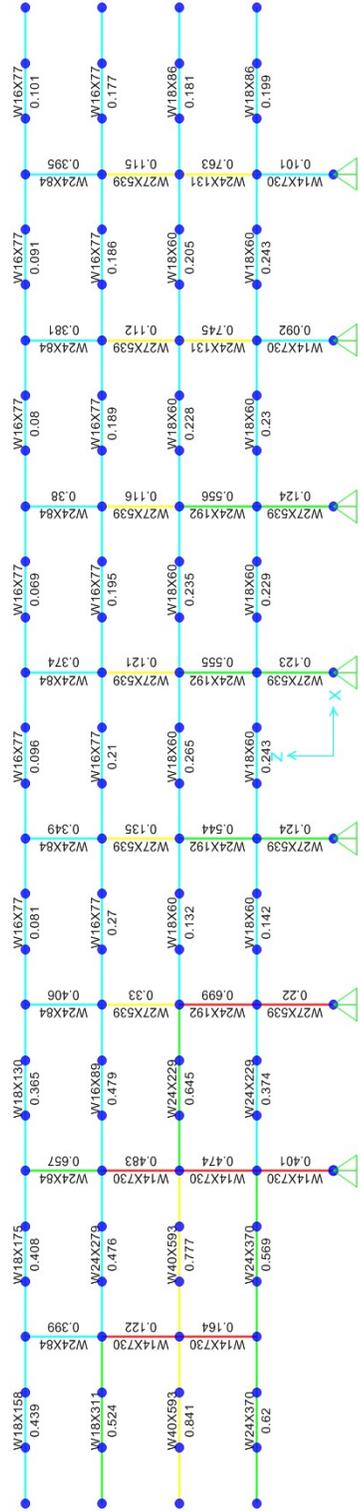
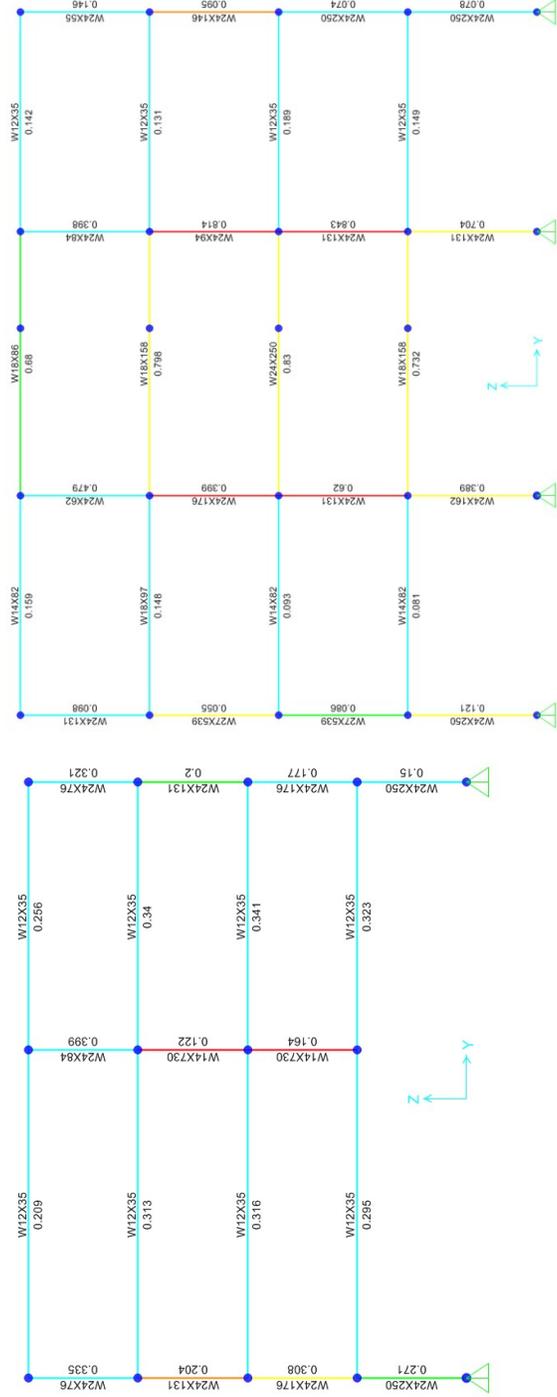
STEP 6



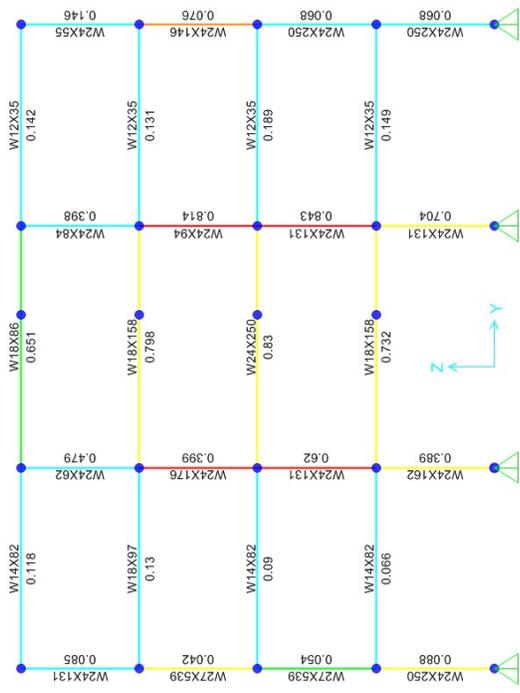
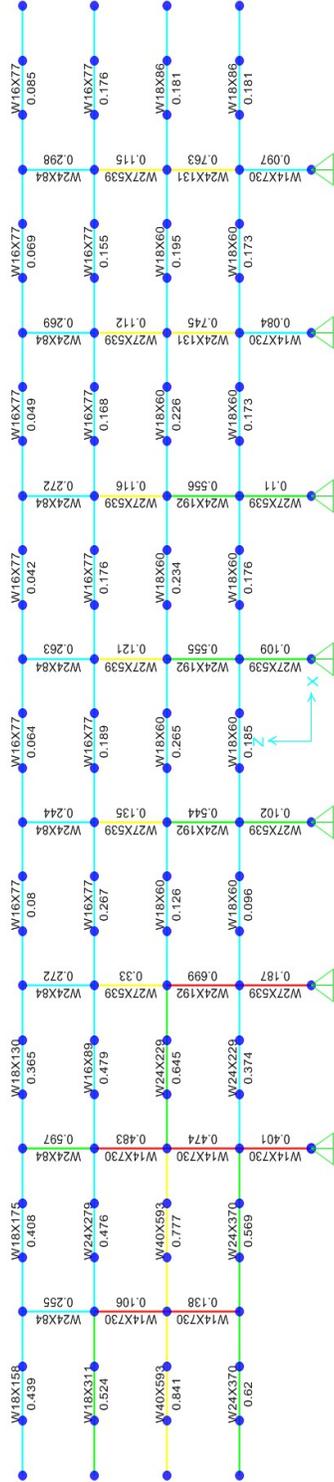
STEP 3

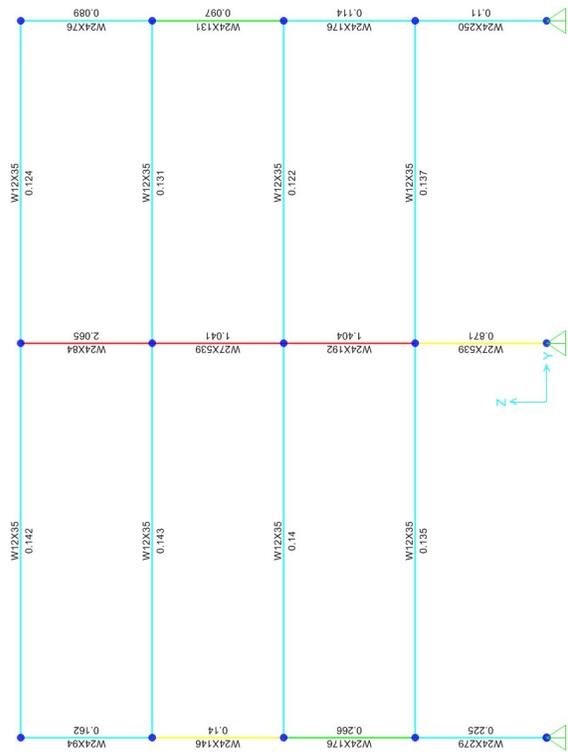


STEP 2

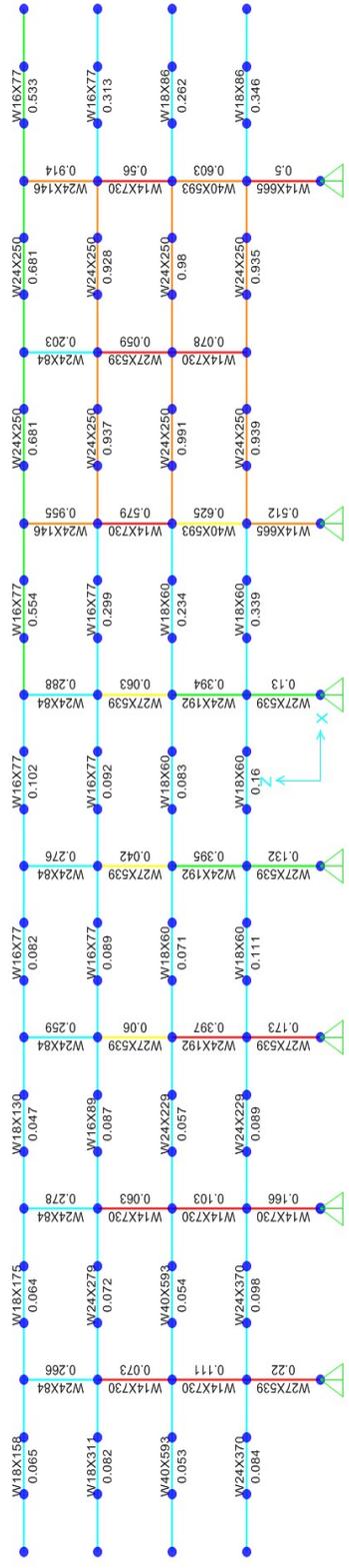


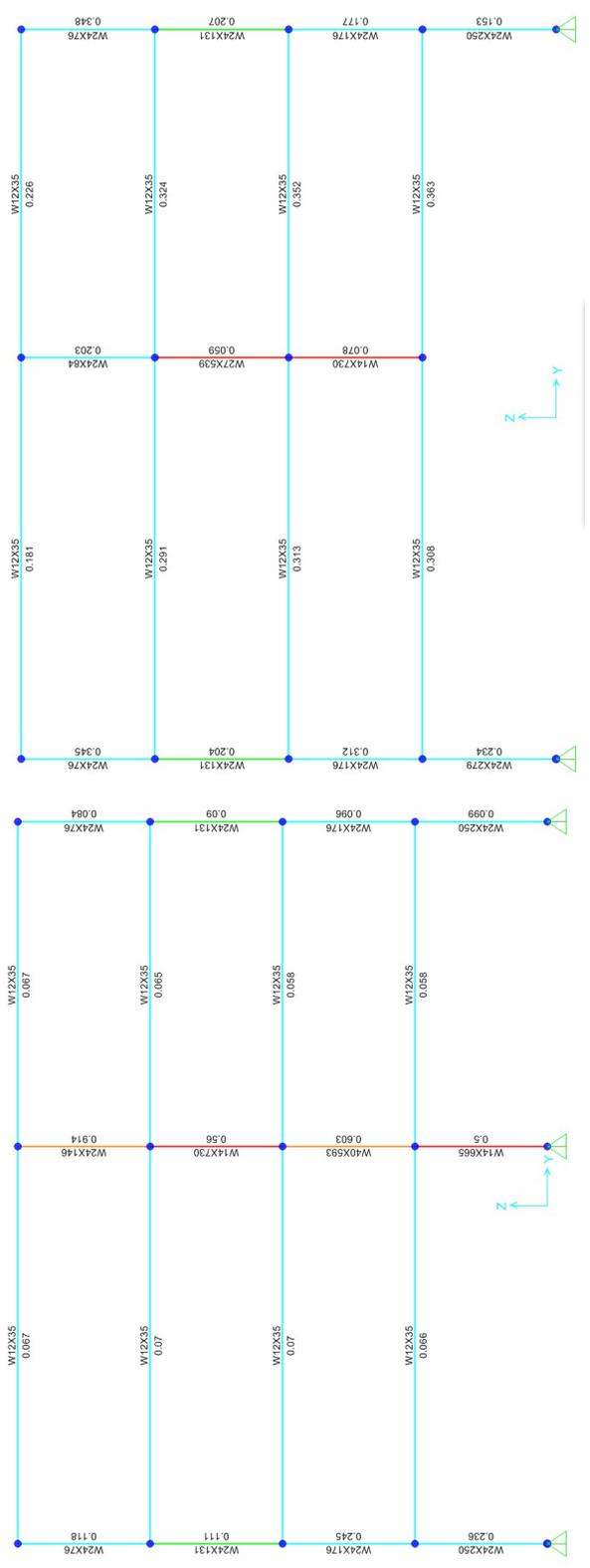
STEP 5

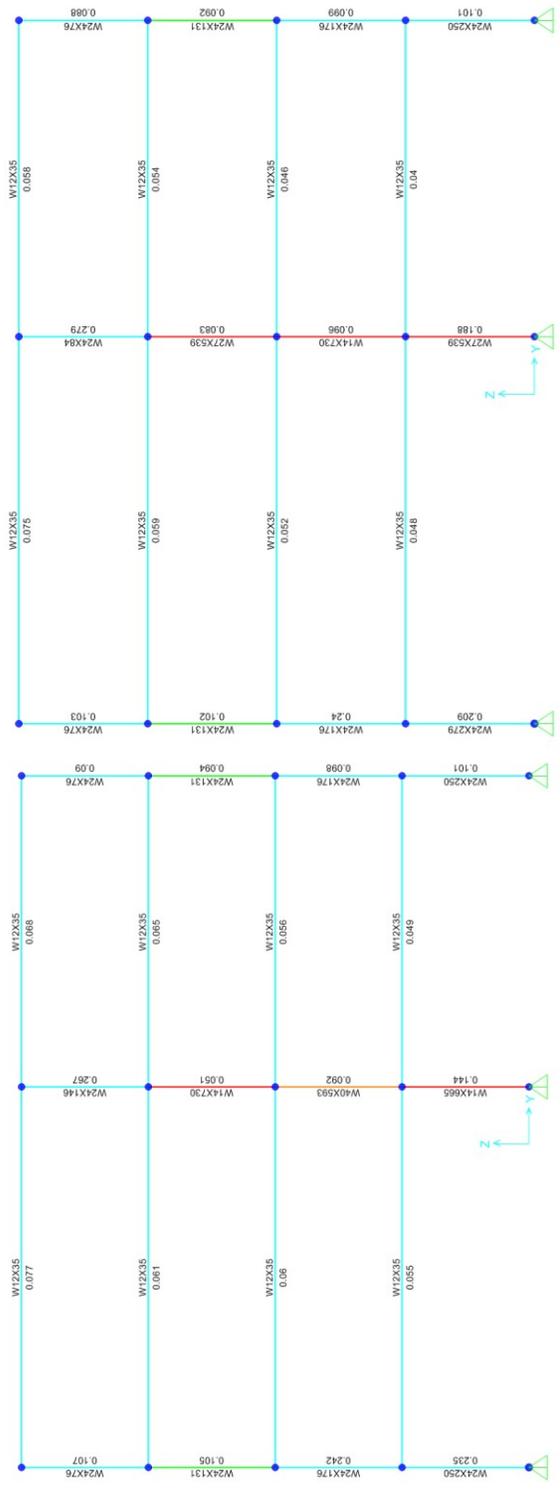


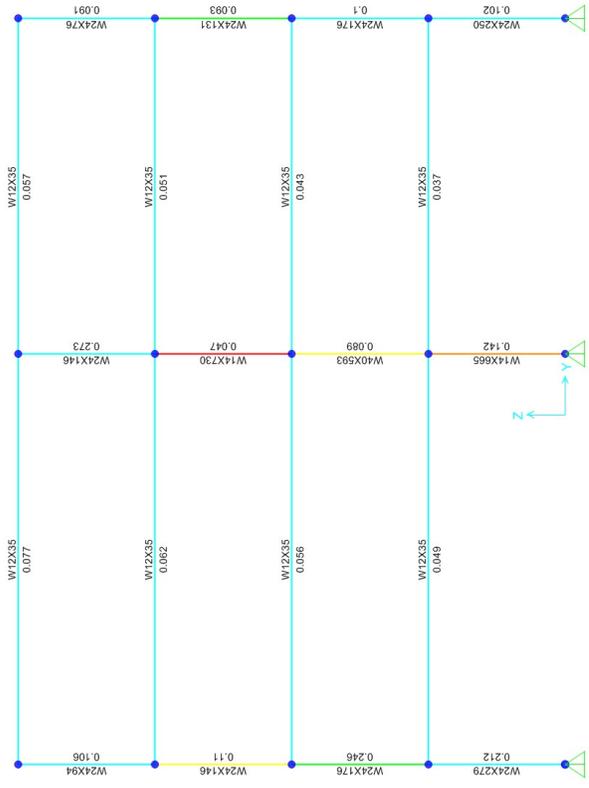


STEP 2









STEP 4

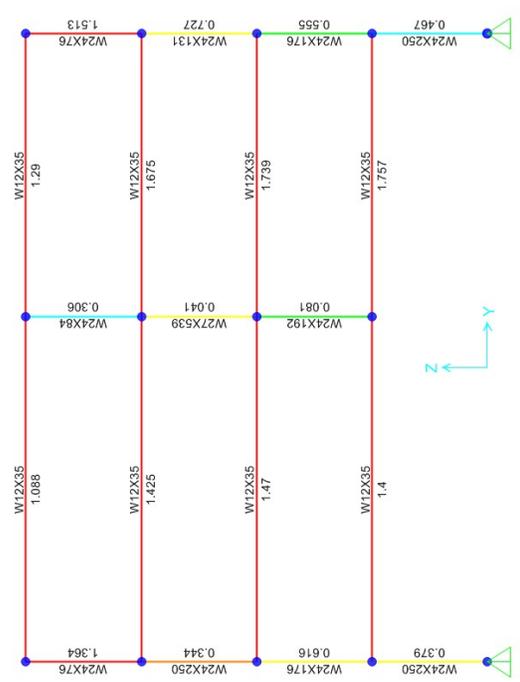
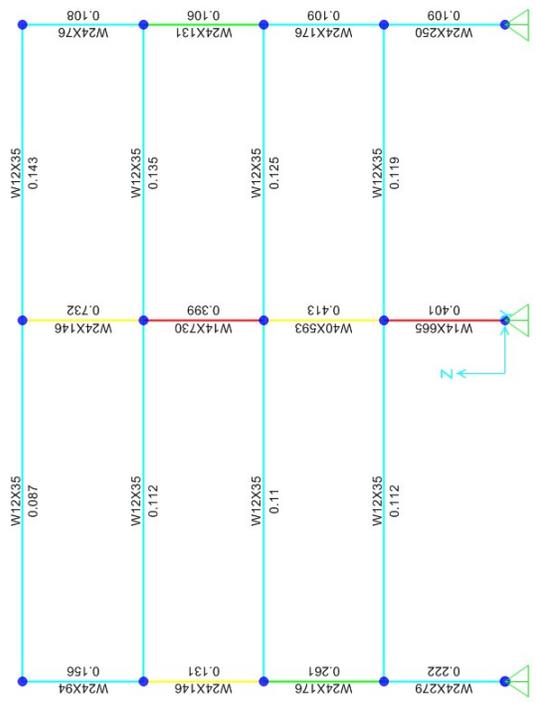
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STEP 5

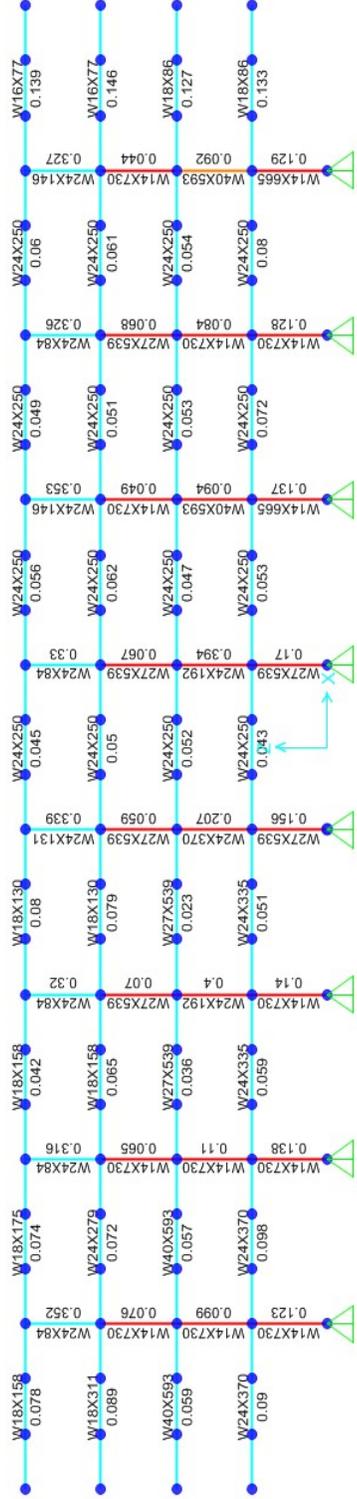
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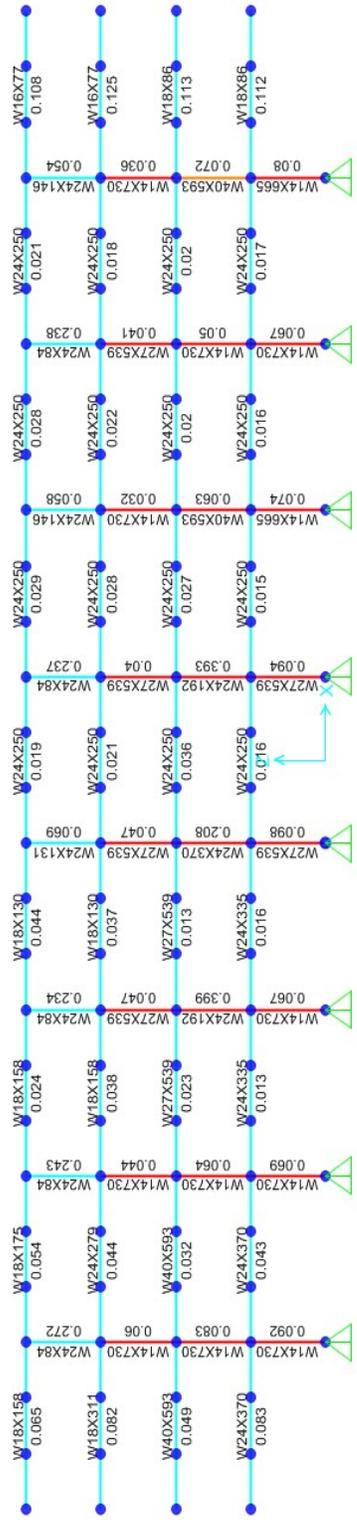
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STEP 3



STEP 5



STEP 6

