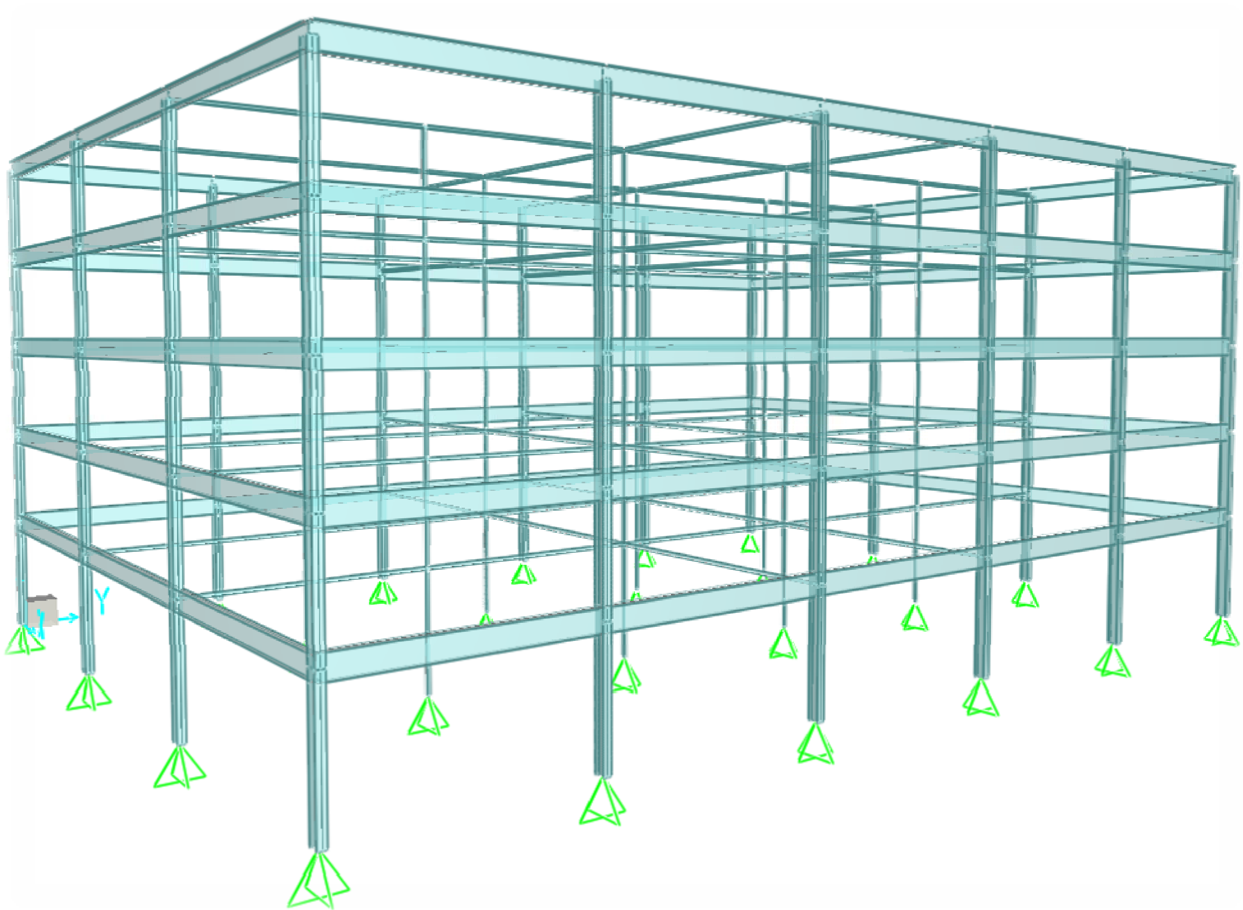
Seismic Evaluation & Design: Special Moment-Resisting Frame Structure



San Francisco State University, Canada College and NASA Sponsored Collaboration

By: Max DeAndreis, Peter Moala, Agustin Robles, Jose Valdovinos

NASA Mentor: Paul Grams

Advisor: Dr. Cheng Chen

Student Advisor: Qi Ming Zeng

Table of Contents

Abstract

This project focuses on designing a five-story steel moment-resisting frame in the earthquake-prone San Francisco Bay Area, California near the Hayward fault. The structural engineer's main priority is safety; buildings have to be designed with a strong infrastructure such that they will withstand severe earthquakes. The objective of this research is to understand how to implement today's seismic technologies into designing a cost-efficient and environmentally friendly building. Computer-aided programs SAP2000 (Structural Analysis Program) and MS Excel are used to design, simulate and analyze the structure. This research internship program allows for the development of project management, time management and teamwork skills, all of which help strengthen students' knowledge of seismic design in Civil Engineering and enhance preparation for academic and professional careers. The project intends to provide community college students research opportunities and make recommendations on improving the engineering curriculum at San Francisco State University and Cañada College.

1. **Introduction**

Earthquake civil engineering is concerned with the design and analysis of structures to withstand hazardous earthquakes at specific locations and steel seismic design is one of the main approaches to this mission. Starting in the late 1800's, steel began to replace cast iron and became readily available for applications in large scale engineering structures. This triggered a tide of tall buildings, including the Home Insurance Building in Chicago (1884), and the Manhattan Building in New York (1889) [1]. Steel frame buildings began to rise all across the nation without any major changes in their connections or design for nearly a century after the 1880's. But after the structural failures that occurred during the 1994 Northridge Earthquake, there was a fundamental rethinking in the design of seismic resistant steel moment connections. This led to FEMA funding the SAC Steel Project research, which redesigned seismic-resistant steel moment connections [2]. This project instituted strict building codes for all steel structures, such as the American Institute of Steel Construction (AISC), and the Los Angeles Region Uniform Code Program (LARUCP). These building codes and design specifications were strictly followed during the entire design process of our building during this project.

Our engineering team participated in the NASA Curriculum Improvements Partnership Award for the Integration of Research Summer Internship (CIPAIR) at San Francisco State University partnered with Canada College. This program strengthens students in science, technology, engineering and mathematics academic fields at minority intuitions, like Canada College. Our group was put into a situation involving us to complete an assignment that civil engineers face in their professional careers.

We were asked to design a five story special steel moment frame structurelocated at 3939 Bidwell Drive, Fremont, CA 94538. A moment-resisting frame is comprised of a rectangular system of rigidly jointed columns and beams that resist moment and shear forces developed during an earthquake by bending. The bending rigidity and strength of the frame members is the source of lateral stiffness and strength for the entire frame. This is going to be an office building designed with large open spaces in the center, and large windows to allow for the most natural light to enter these areas. As seen in Table 1, for the live loads (moving weight) on the building floors, we used the standard 50 psf (pounds per square foot) at each floor and 20 psf for the roof, as this floor is going to receive less traffic than the others. Dead loads (permanent weight), includes the weight of the building, were assigned as 95 psf on the roof, 92 psf on the second floor, and 90 psf on the third, fourth and fifth floors. The height of the first floor is 13 feet, and 11 feet for the second, third, fourth and fifth floors. As seen in Figure 1, the dimensions of the entire building are a width of 90, length of 125 and height of 57 feet. This building had to be designed according to AISC's code and ASCE's equilateral force procedures. Finally, we designed our structure on SAP2000 and modeled four selected earthquake ground motion models to see how the building reacted.

Building Specifications	Dead load (psf)	live load (psf)	Height (ft)
Roof	95	20	11
5	90	50	11
4	90	50	11
3	90	50	11
2	92	50	13

Table 1- Overview of the building specifications in each floor

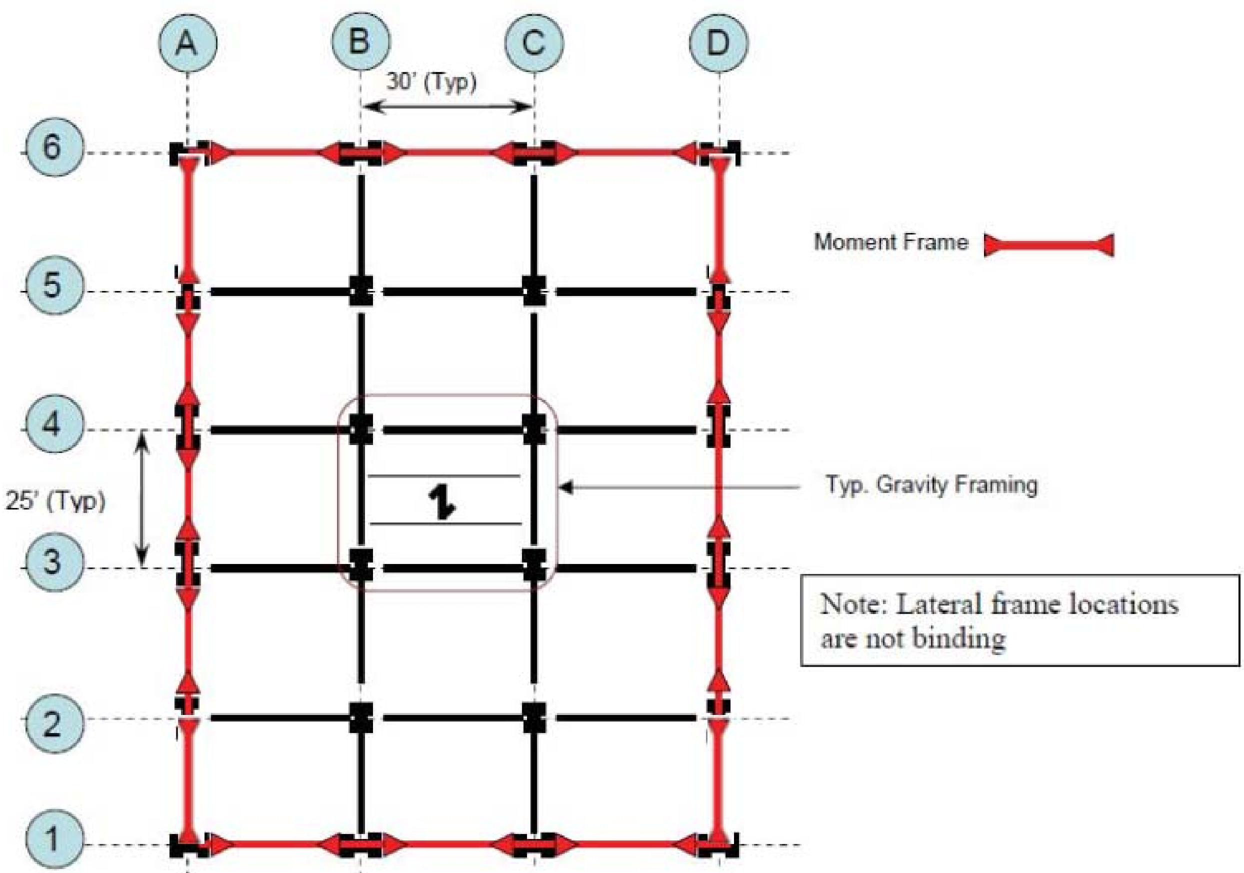


Figure 1- Top view (x-y dimensions) of building

The ASCE 7-05 equivalent lateral force procedure was used to complete our design and select the beam member sizes that will follow the AISC manual. The equivalent lateral force method involves the application of a set of representative or equivalent forces on each level of the structure that produce horizontal deflections that approximate the deflections caused by the ground motion.

2. Design Approach

1. Research Earthquake Probability in buildings location
2. Research the soil type and Shaking Amplification of our building's location.
3. Apply the ASCE 7-05 Equivalent Lateral Force Procedure to determine the base shear, the dead load and the period of the structure.
4. Calculate the beam members needed for each floor following the AISC codes using MS Excel.
5. Utilize SAP2000 for a step-by-step Time History Analysis of the building response when subjected to selected ground motions to evaluate how our building will resist different levels of earthquakes.

2.1 Earthquake Probability in building location

The USGS Geological Hazards Science Center provided us with the Earthquake Probability mapping of the occurrence of an earthquake with a magnitude of 6.6 or greater within the next 20 years around the Fremont area [4]. After observing these results in Figure 2, we knew that we had to design our building to withstand an inevitable large magnitude earthquake in the near future. This validates the need for researching and designing a moment-resisting frame structure for our building at this earthquake-prone location.

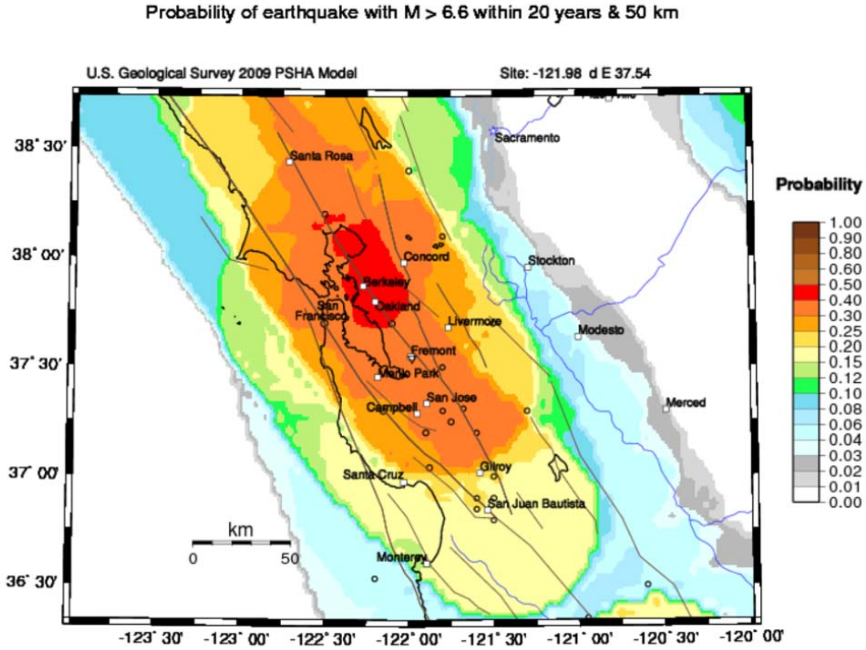


Figure 2- Probability of earthquakes near building location

Four historic earthquake grounds were selected from the Pacific Earthquake Engineering Research Center (PEER) Ground Motion Database based on the location, intensity, and the duration. Seismic activity data including duration and peak ground acceleration (g) from PEER were imported to SAP2000 to simulate the response of our designed frame to different earthquakes [5]. The graphs of these ground movements can be seen below in Figure 3, as well as the earthquake specifications can be seen in Table 2.

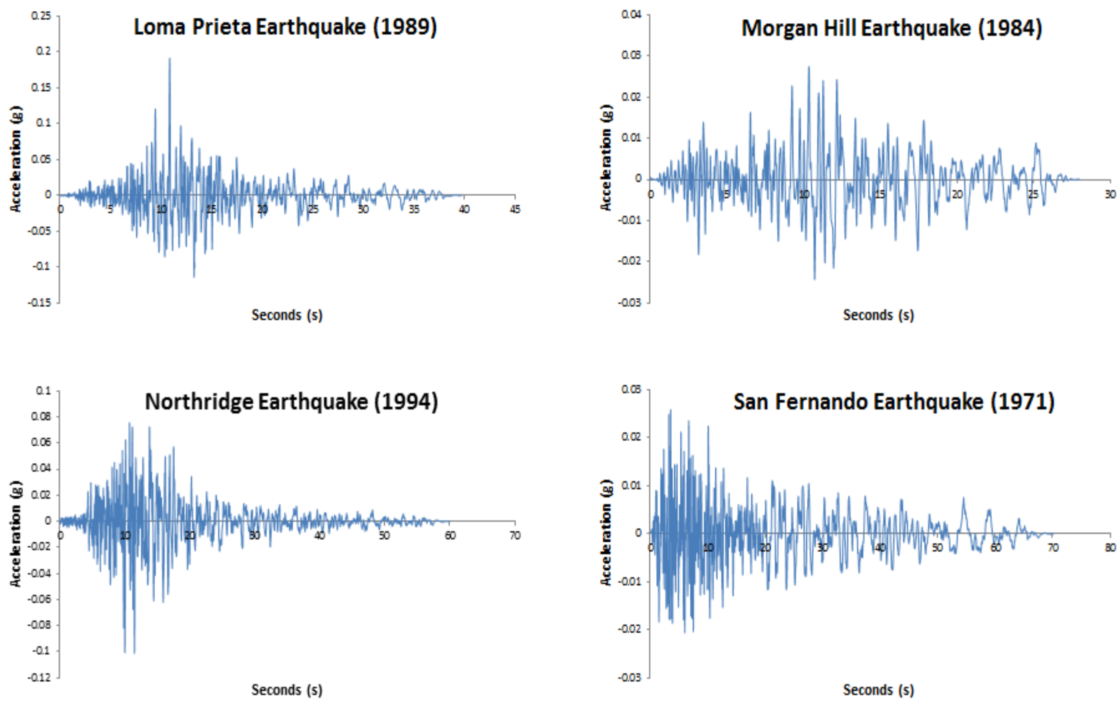


Figure 3- Depicts the ground motion of four historic earthquakes

Earthquake	Magnitude	Ground Acceleration (g)	Duration (s)	Cost	Loss of Life
Loma Prieta, 1989	6.90	0.191579	40	$8 Billion	63 killed, 3,757 injured
Morgan Hill 1984	6.19	0.027434	27	$7 Million	27 injured
Northridge 1994	6.70	0.101049	60	S20 Billion	57 killed, 8,700+ injured
San Fernando 1971	6.61	0.025879	70	$505 Million	65 killed, 2,000+ injured

Table 2- Specifications on the four selected earthquakes

2.2 Soil Type

Ground shaking is the primary cause of earthquake damage to man-made structures. Earthquake ground movements can damage buildings and infrastructures which can cause human injuries or deaths. Although many factors such as the earthquakes magnitude and the site’s location relative to a fault line vary for every earthquake, the soil of the location will always remain the same. The influence of the underlying soil on the local amplification of earthquake shaking is called the site effect.

The National Earthquake Hazards Reduction Program (NEHRP) has defined 5 soil types based on their influence on shaking amplification, as can be seen in Table 3. Soil types A and B are comprised of rock and bedrock, both of which have minimal shaking amplification. Soil type E on the other hand, includes water-saturated mud and produces the strongest amplification because soft soils amplify ground shaking [3].

Soil type A	Includes unweathered intrusive igneous rock. Occurs infrequently in the bay area. We consider it with type B (both A and B are represented by the color blue on the map). Soil types A and B do not contribute greatly to shaking amplification.
Soil type B	Includes volcanics, most Mesozoic bedrock, and some Franciscan bedrock. (Mesozoic rocks are between 245 and 64 million years old. The Franciscan Complex is a Mesozoic unit that is common in the Bay Area.)
Soil Type C	Includes some Quaternary (less than 1.8 million years old) sands, sandstones and mudstones, some Upper Tertiary (1.8 to 24 million years old) sandstones, mudstones and limestone, some Lower Tertiary (24 to 64 million years old) mudstones and sandstones, and Franciscan melange and serpentinite.
Soil Type D	Includes some Quaternary muds, sands, gravels, silts and mud. Significant amplification of shaking by these soils is generally expected.
Soil Type E	Includes water-saturated mud and artificial fill. The strongest amplification of shaking due is expected for this soil type.

Table 3- Depicts the five different soil types

The soil type map of the San Francisco Bay Area was sourced from the USGS Earthquake Hazards webpage. From Figure 4, we could see how there is soft soil all around the bay area right next to the water. These areas are of great concern for they will experience the strongest shaking in the event of a high magnitude earthquake.

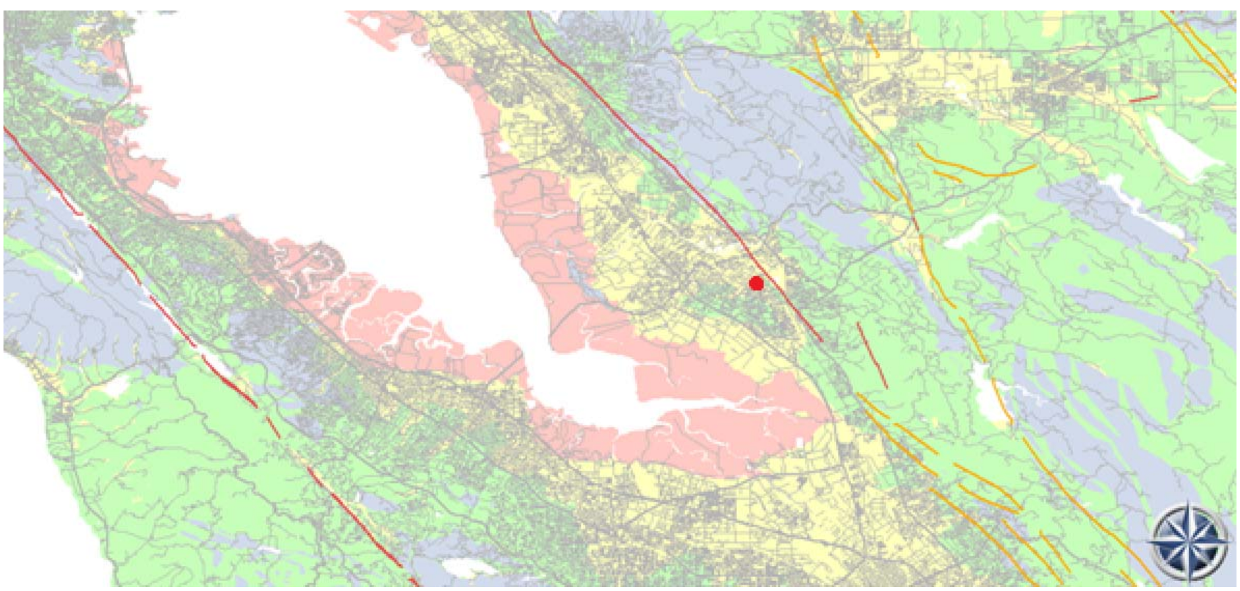


Figure 4- Map of the San Francisco Bay Area soil type

As seen in Figure 5, for the location of our structure at 3939 Bidwell Drive, Fremont, CA 94538, the soil was comprised of muds, sands, and silts, classifying it as soil type D. This location will experience significant amplification of shaking which means the damage would be increasingly devastating.

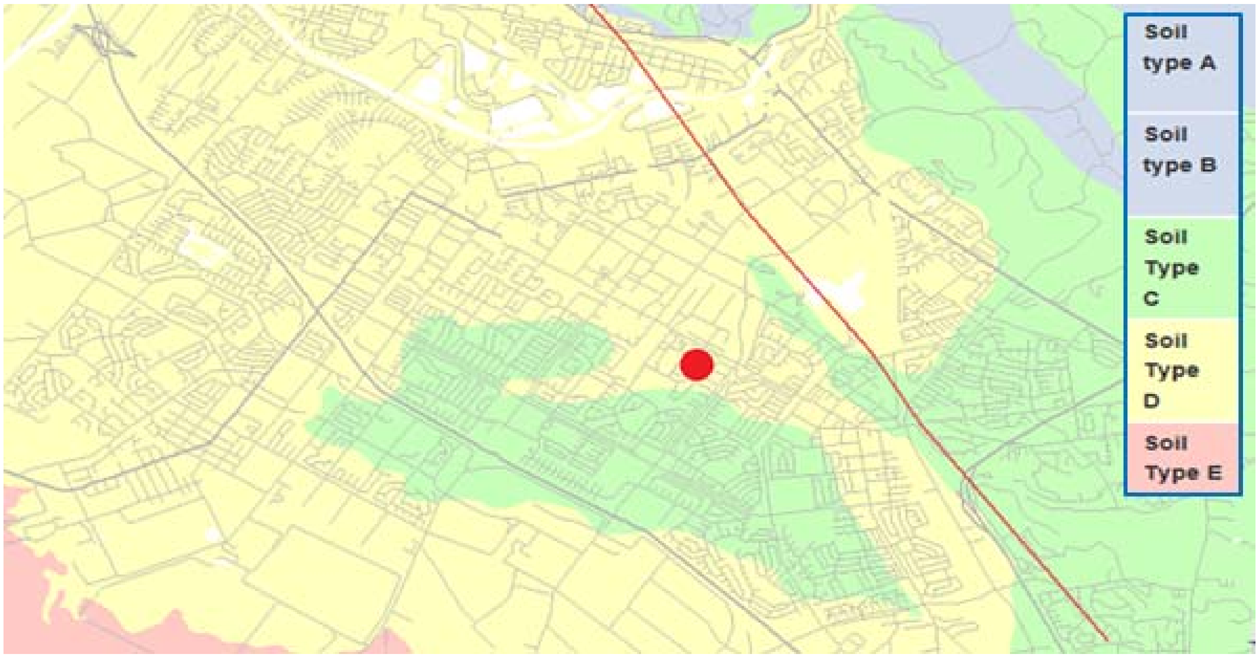


Figure 5- Map of soil type near building location

2.3 The ASCE 7-05 Equivalent Lateral Force Procedure

The ASCE 7-05 equivalent lateral force procedure was used to complete our design and select the beam member sizes that will follow the AISC manual. The equivalent lateral force method involves the application of a set of representative or equivalent forces on each level of the structure that produce horizontal deflections that approximate the deflections caused by the ground motion.

We used the US Seismic Design Maps Web Application to determine the Site Class for the location of our building. We input the site coordinates (37.54363°N, 121.97772°W), the design code reference document (ASCE 7-05), the site soil classification (Soil Type D), and the risk category (II). USGS provided us with a summary report of our location including six design spectral response acceleration parameters, as seen in Table 4 [6].

Site Class=	D
S_s	2.184
S_1	0.902
S_{DS}	1.456
S_{D1}	0.902
S_{MS}	2.184
S_{M1}	1.353

Table 4- Site Class Coefficients

The six design spectral response acceleration parameters were then used to determine the seismic design base shear for our structure. The Occupancy factor, which range from I to IV, is a classification that is used to decide structural requirements based on occupancy. The seismic importance factor, I_e, is used to increase the calculated load on a structure based on its occupancy and ranges from 1 to 1.5 [6]. Due to the nature of our structure not being used as a hospital, hazardous materials structure or any kind of structure that needs continued function during an earthquake it has an I_e of 1.The coefficients Ct and x are given in the ASCE 7-05 for steel moment frame structures [7]. All of which can be seen in Table 5.

Occupancy	II
Import. Factor	II
I_e	1
Special Steel Moment Frame	
R=	8
K=	1.105
x=	0.8
C_t=	0.028
$Ta=(C_t)(h_n)^x$=	0.710981

Table 5- Initial coefficients for the Equivalent Lateral Force Procedure

The horizontal components of seismic shaking can be converted into a parameter, V, the seismic base shear [7]. The most damaging movements to a buildings structure during seismic activity are the horizontal component of acceleration and the base shear which imposes lateral forces at the base. The coefficient for the upper limit of the calculated period, Cu is given by the value of S_{D1}.Due to the fact that the structure has to withstand earthquakes; the lateral force procedure helps place static loads on the structure with magnitude and direction that closely approximate the effects of an earthquake. The seismic base shear is the static load that is calculated to be placed in each floor and is used to analyze the structure reactions to an earthquake [7]. These calculations can be seen below in Table 6.

Design Base Shear	
C_u=	1.4
C_u*T_a=	0.995373
C_s=	0.182
V=	935.7075
.85 V=	795.3514

Table 6- Seismic Base Shear

The seismic base shear components of force located at each level, F_x, and the seismic design shear in each story, V_x. The effective seismic weight of the building assigned to each level, w_i.

All these values are needed to determine the strength of our building to withstand a certain earthquake's ground acceleration [7]. These forces have been calculated for each floor, as seen in Table 7.

Horizontal Distribution Forces and Accidental Torsions							
Floor	h_i	w_i	$w_i h_i^k$	$\frac{w_i h_i^k}{\Sigma(w_i h_i^k)}$	F_x	$.5F_x$	V_x
Units	ft	K			kips	kips	kips
Roof	57	1068.7500	93136.0560	0.3504	327.9068	163.9534	0
5	46	1012.5000	69621.3493	0.26120	245.1179	122.5590	327.9068
4	35	1012.5000	51474.2729	0.1937	181.2270	90.6135	573.0248
3	24	1012.5000	33925.6716	0.1277	119.4431	59.7216	754.2517
2	13	1035.0000	17613.5848	0.0663	62.0127	31.0063	873.6948
1	0	0.0000	0.0000	0.0000	0.0000	0.0000	935.7075
	sum	5141.2500	265770.9347	1.0000	935.7075	467.8538	935.7075

Table7- Horizontal Distribution Forces and Accidental Torsions

We were to check what size of the beam members, given by the AISC manual, would satisfy the moment value. The moment, M_u, is the required flexural strength using load combinations (live, dead, and seismic). Section modulus, Z_x, is a given cross-section used in the design of beams or flexural members. We were to check if the moment exceeds the required moment and if we have the proper section modulus for each member [8]. Below displays the beam membered sections used for our final design. Appendix A describes details of the beam membered sections used according to the AISC manual. The column sizes for our transverse beams can be seen in Table 8, and the longitudinal beams can be seen in Table 9.

Transverse 30 foot beams	Members	Wu(kip)	Mu($kip \cdot in$)	Calculated Zx (in^3)	Zx table (in^3)	Check
Roof	W21X83	9.479	710.925	189.58	196	OK
5	W24X76	9.9505	746.2875	199.01	200	OK
4	W24X76	9.9505	746.2875	199.01	200	OK
3	W24X76	9.9505	746.2875	199.01	200	OK
2	W21X93	10.1369	760.2675	202.738	221	OK

Table 8- Transverse Beams

Longitudinal 25 foot beam	Members	Wu(kip)	Mu($kip \cdot in$)	Calculated Zx(in^3)	Zx table(in^3)	Check
Roof	W21X68	11.3748	592.4375	157.9833	160	OK
5	W21X73	11.9406	621.9063	165.8417	172	OK
4	W21X73	11.9406	621.9063	165.8417	172	OK

3	W21X73	11.9406	621.9063	165.8417	172	OK
2	W24X68	12.16428	633.5563	168.9483	177	OK

Table 9- Longitudinal Beams

The member columns are checked by tests for effective slenderness and elastic buckling behavior. K, the effective length factor, is used for calculating the column slenderness, KL/r. Where L is the laterally unbraced length of the member and r is the governing radius of gyration.The flexural buckling, F_e, stress test and elastic buckling, F_{cr}, test is to confirm if the building can retain its shape after being hit by an earthquake. The nominal strength, Pn, checks for local stability for proper thickness of the column web and strong axis bending strength [7]. These calculations and tests can be seen in Table 10.

Columns		$\frac{KL}{r}$		Fe= $\frac{(\pi^2 \cdot E)}{(\frac{KL}{r})^2}$	F_{cr}	$P_n = F_{cr} \cdot A_g$	
	Members	Slenderness ratio	Check	elastic critical buckling (ksi)	Flexural buckling Stress (ksi)	nominal strength (kips)	Local Stability
roof	W12X40	68.0412	OK	61.8235	35.6418	417.0088	stable
5	W14X48	69.1099	OK	59.9262	35.2618	497.192	stable
4	W16X57	82.5	OK	42.0523	30.3977	510.6822	stable
3	W18X86	50.1901	OK	113.6217	41.5891	1052.2032	stable
2	W21X93	84.7826	OK	39.8184	29.5608	807.0101	stable

Table 10- Checks for the strength of the columns

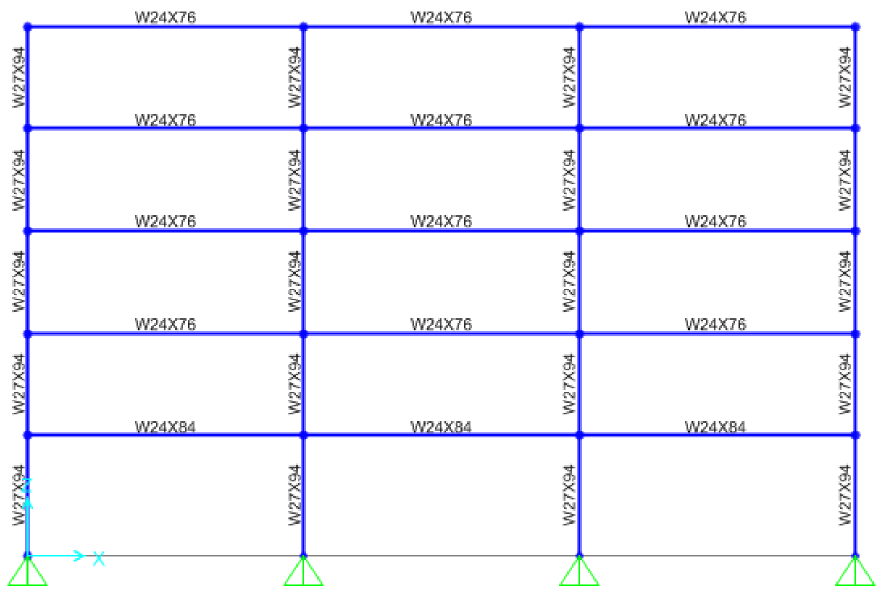
3. Results

After completing the Equivalent Lateral Force Procedure we created a model of our structure in SAP2000. Structural Analysis Program (SAP), created by Computers and Structures Inc. (CSI), is an intuitive user interface for engineers working on transportation, industrial, public works, sports, and other facilities. For our engineering project, SAP2000 was used to draw and design our building using SAPs 3D object based graphical modeling environment [10].

We assigned each beam to the appropriate width and length according to our calculated values. The beam members from our first attempt were less cost-efficient than we were hoping. All the columns from our first calculated values were identical at W27X94. This is inefficient as the columns should be heavier in the first floors, and decrease accordingly every floor heading upward since the base and the second floor columns experience the largest displacement. For our first attempt, the transverse beam members were W24X76 for the third, fourth and fifth floors and the beams on the second floor were heavier than the rest; yet for our final design we made the beams on the roof slimmer and smaller as the excess weight was unnecessary. For all the transverse/longitudinal beams, as well as the columns, we found that we could decrease the size of the beams on higher floors more than the lower floors and the infrastructure would still be able to safely support the building.

When we went over the ASCE 7-05 procedure, we corrected our calculations and found improvements that would satisfy our checks. The transverse beams were assigned as W24X76 for floors three to five and W21X83 for the roof and W21X93 for the second floor. The longitudinal beams were W21X73 for the third through fifth members with the second and the roof W24X68 and W21X68 respectfully. The beams significantly reduced in size compared to our first attempt which leads to a cost-efficient structure that satisfies the requirements for the ASCE building codes. Below are our SAP2000 design drawings viewed to show the transverse, longitudinal, and column assigned beam sizes of our first and final building designs. Below in Figure ^ and Figure 7, you can see the improvements that were made between our First and Final Beam selection for both the transverse and longitudinal directions.

First Design:



Final Design:

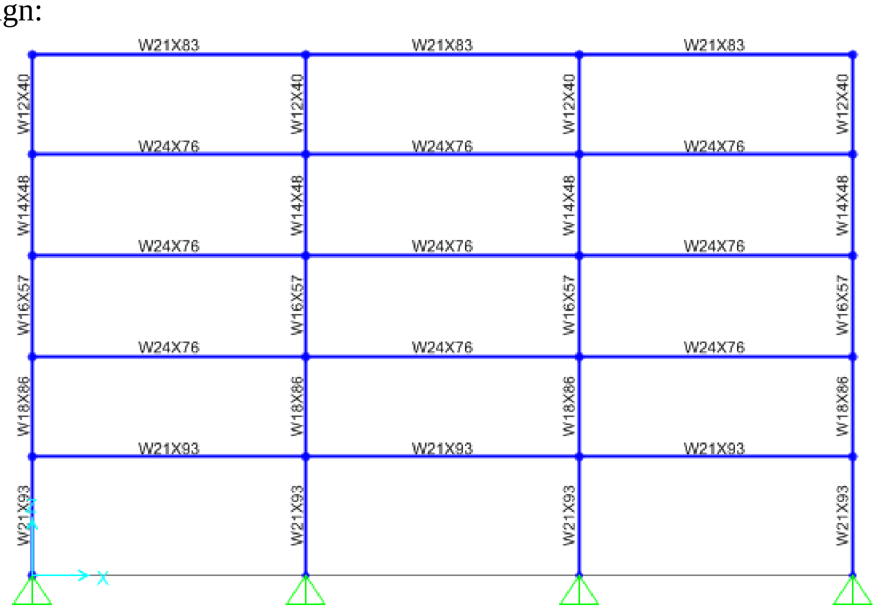
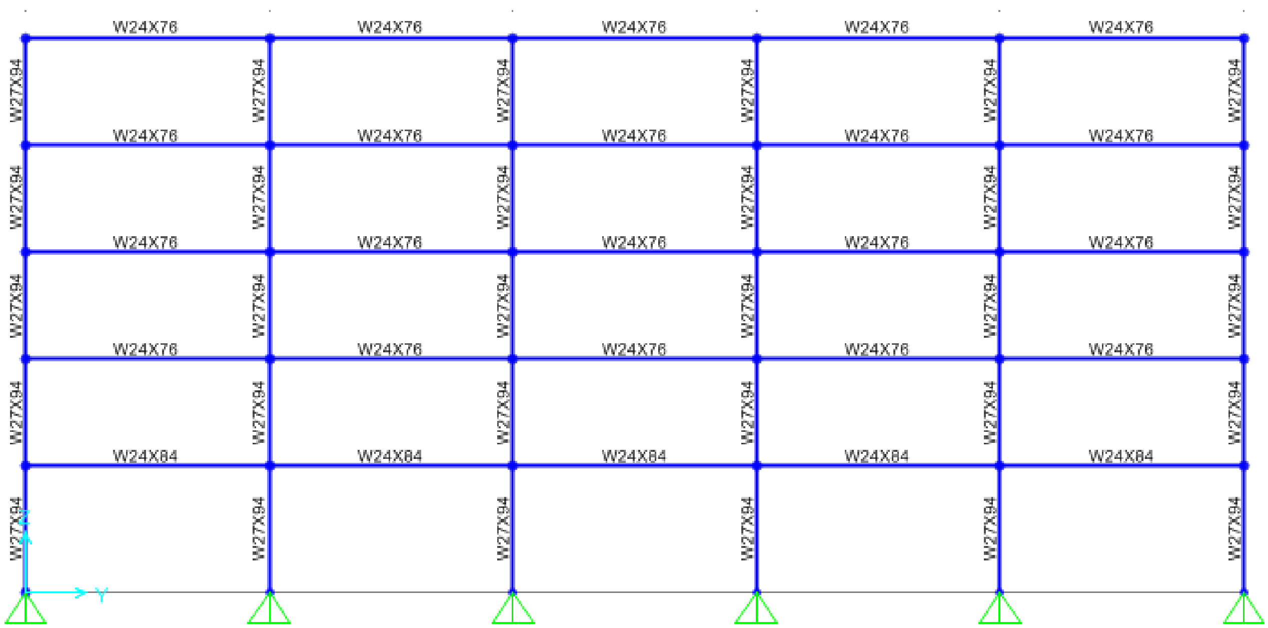


Figure 6- First and Final Beam selections for the transverse beam and column view

First Design:



Final Design:

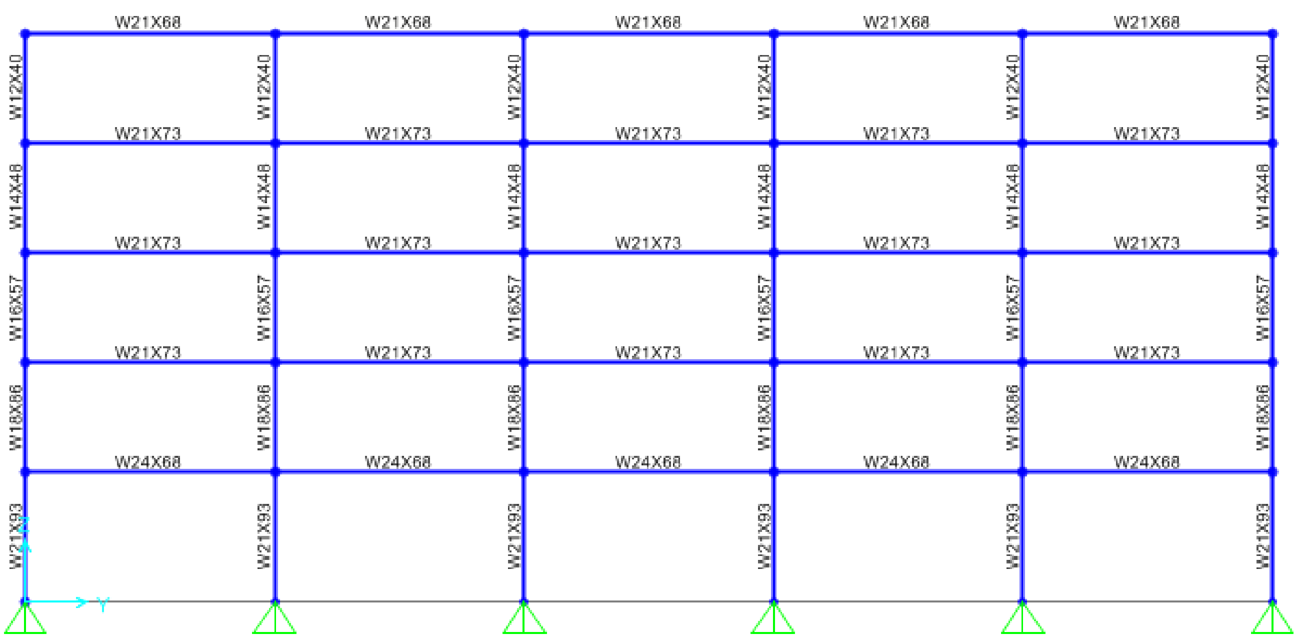


Figure 7- First and final beam selection for the longitudinal beam and column view

4. Analysis

Time History Analysis is incorporated into SAP2000 and to examine a structure's behavior over a specified duration due to an earthquake. Earthquake accelerograms, graphs that show ground acceleration over a period, which are supplied by the Pacific Earthquake Engineering Research Center Database are uploaded onto the SAP program [5]. The uploaded accelerograms act as a seismic load on a design and produce effects similar to that of the base shear. When checking for the Story Drift Displacement, a point at the center of the roof is utilized as the reference point and is used to confirm if the experimented displacement is within the limits of the Max Story Drift allowed [7].

Once completing our final design, we simulated the four earthquake accelerograms into the Time History Analysis on our design to investigate how it would perform. The building design responded differently to each selected earthquakes. As seen in Figure 8, the Loma Prieta earthquake showed the highest value for the base shear as it was expected since it had the highest magnitude of 6.9 on the Richer scale. Our design experienced the highest story drift displacement during the simulation of the Northridge earthquake. The duration of the Northridge earthquake's high intensity ground motion lasted for ten seconds compared to Loma Prieta which lasted less than 5 seconds. So the shear forces increased from the Northridge earthquake which caused more strain on the building resulting in greater story drift.

In the preliminary stages of our design, we observed that in all four earthquake simulations the beam members on the second floor experienced the largest deformation, thus we decided to increase the beam size in comparison to that of all the other floors. Also, we found the story drift could be minimized by making the columns in the first floors heavier than all the beams. Once we designed a structure that would withstand all the base shear forces, we redesigned the frame to decrease the weight of the beams while still satisfying the maximum allowable story drift. This reduction in the weight of the beam members greatly reduced the final cost of our design.

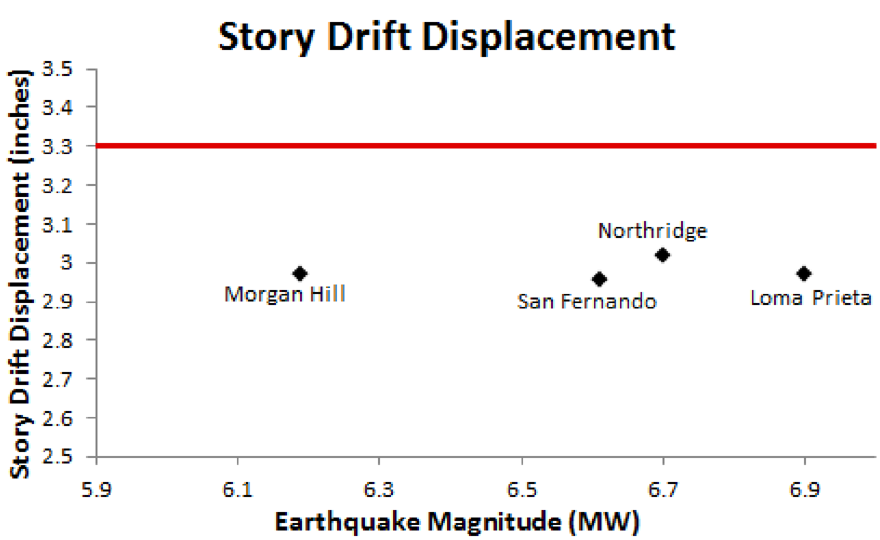


Figure 8- Story Drift Displacement

5. Conclusion

At the end of our research we expanded our knowledge in the field of civil engineering in protecting the society from earthquakes. We understood what a professional civil engineer does when hired to design and construct a building. We learned the basic principle in seismic design of steel frames and how the ASCE 7-05 Lateral Equivalent Force Procedure provides an exceptional tool for us student engineers to evaluate building codes that will provide a secure building that professionals will engineer in the field. We were exposed to SAP2000 which is utilized by civil engineering firms in over 160 countries for the design of major projects. This internship opportunity has given us an insight to graduate level course work and strengthened our fundamental engineering principles.

Our internship experience made our engineering group realize how trained civil engineers in the field will have to collaborate with other members on their team. Like us, trained civil engineers will need to make weekly meetings with their supervisor to discuss their progress on their design and provide feedback on what they can improve. They will need to make a detailed, tentative plan that they must follow until their deadline when the building must be constructed. Our research project could not have been completed by one engineer because it takes teamwork and collaboration on everyone’s part to get the project done.

5.1 Future work

To help propel NASA’s goal of human settlement in outer space, we analyzed special moment-resisting frames to the surface of the moon. We researched the landscape and studied the environment to gain a better understanding of the lunar conditions and determine if our structure would endure moon ground shaking. There are four different types of moonquakes, the technical term for seismic activity on the moon, which are deep moonquakes, meteorite impacts, thermal quakes, and shallow quakes. Shallow moonquakes is the most harmful type of moonquake as they are less intense (magnitude of 4 on the Richer scale) but last for a longer duration (up to 10 minutes) in comparison to earthquakes [11]. Shallow moonquakes due to the terrain of the moon being a large dry-rigid chunk of stone, seismic activity of the same magnitude/intensity on the moon would cause more damage than that on Earth where the water and soil dampen seismic vibrations. Low magnitude moonquakes will not cause serious damage to our structure but their extended duration causes issues such as low-cycle fatigue [12].

Acknowledgements

We would like to thank the NASA Office of Education for sponsoring this program through the Curriculum Improvement Partnership Award for the Integration of Research (CIPAIR). We would like to thank NASA for allowing us to be a part of their research team this summer at NASA Ames Research Center. We appreciate the authority of San Francisco State University (SFSU) for providing us with a good environment and facilities to complete this project. We wish to express our sincere gratitude to our mentor Paul Grams for giving us helpful guidance throughout the internship and providing us with interesting information on what NASA has to offer for us and the world. This research project would not have been possible without the tireless efforts of our advisors at SFSU, Dr. Cheng Chen and our graduate student mentor Qi Ming Zeng, who furthered our knowledge while still preserving our interest in our pursuit in civil engineering. We would like to thank Dr. Lenell Allen and Dr. Amelito Enriquez who provided us with this wonderful opportunity.

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Appendix

A) Table Gathered from the AISC DATABASE for the 30ft members per floor

AISC DATABASE	W	A	D	IX	ZX
W21X83	83	24.3	21.4	1830	196
W24X76	76	22.4	23.9	2100	200
W24X76	76	22.4	23.9	2100	200
W24X76	76	22.4	23.9	2100	200
W21X93	93	27.3	21.6	2070	221

B) Table Gathered from the AISC DATABASE for the 25ft members per floor

AISC DATABASE	W	A	D	IX	ZX
W21X68	68	20	21.1	1480	160
W21X73	73	21.5	21.2	1600	172
W21X73	73	21.5	21.2	1600	172
W21X73	73	21.5	21.2	1600	172
W24X68	68	20.1	23.7	1830	177

C) Distribution of Loads and AISCE Database of the Columns

AISC DATABASE	Members	W	A	D	IX	ZX	BF	TW	TF	KDES	H_TW	RY
Roof	W12X40	40	11.7	11.9	307	57	8.01	0.295	0.515	1.02	33.6	1.94
5	W14X48	48	14.1	13.8	484	78.4	8.03	0.34	0.595	1.19	33.6	1.91
4	W16X57	57	16.8	16.4	758	105	7.12	0.43	0.715	1.12	33	1.6
3	W18X86	86	25.3	18.4	1530	186	11.1	0.48	0.77	1.17	33.4	2.63
2	W21X93	93	27.3	21.6	2070	221	8.42	0.58	0.93	1.43	32.3	1.84