Impact of Vertical Ground Motion on Seismic Response of Steel Frame Structures

by

Hongyang Wu

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Approved by

Justin D. Marshall, Chair, Associate Professor of Civil Engineering
Mary L. Hughes, Lecturer of Civil Engineering
James S. Davidson, Professor of Civil Engineering

Abstract

It is a well-known fact that all earthquakes have three orthogonal components of acceleration including two horizontal and one vertical acceleration. Current design practice for design of structures in United States only focuses on the impact of the horizontal component of earthquakes. However, according to previous research, vertical peak ground acceleration (PGA) can be higher than the horizontal peak acceleration in the same earthquake, which may contribute to structural collapse. Further research is needed to investigate the impact of vertical ground motion on seismic response of structures. In this paper, four three-story and four six-story steel frame structures are chosen to investigate and represent this problem. For each structure height a special moment frame with reduced beam section (RBS) connections and buckling-restrained brace (BRB) frames are designed by the equivalent lateral force (ELF) method based on ASCE 7-10 and analyzed by using nonlinear dynamic analysis. A suite of 40 strong ground motion records are selected including horizontal and vertical ground motions in this study. The range of the ratio of vertical to horizontal acceleration in this study is from 0.5 to 1.2. All the structural models are analyzed under two different loading cases: 1) Horizontal Only and 2) Horizontal plus Vertical. There is a significant impact of vertical ground motion on the column axial force, vertical acceleration and beam midspan vertical deflection. The demand on rotation of reduced beam sections in upper stories also experience a significant influence from the vertical ground motions.

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Chapter 1 Introduction

1.1 Motivation for the Research

According to recent earthquake records, the vertical acceleration component in some earthquakes is found to have higher value than the horizontal component. However, traditional design methods assume the magnitude of vertical acceleration to be 1/2 to 2/3 of the horizontal acceleration. Seismic requirements in these codes design a structure to resist strong ground motion based on the ductile and inelastic behavior of the structural system. Seismic design methods in current codes do not typically directly consider the impact of the vertical component. This may result in a significant collapse risk. In the past couple years, field evidence has been found that shows damage from the vertical component of strong ground motions.

The impact of the vertical component of strong ground motion has been investigated only recently, therefore, the research on this topic is limited. Iyengar and Shinozuka (1972) did some investigation on the vertical ground motion by using a cantilever beam. Anderson and Bertero (1973) did research on a ten story building which only had an unbraced, single bay frame. In this study four special moment frames with reduced beam section and four buckling-restrained frames will be used to investigate the impact of vertical ground motion on the seismic response of steel frame structures.

1.2 Scope of work

The purpose of this thesis is to investigate the impact of vertical ground motion on the seismic response of steel frame structures. In order to understand overall seismic response of steel frame structures due to vertical ground motion, eight different kinds of steel frames are used in this paper. These eight models include four special moment frame models which are designed in this paper based on the information given by Sabelli (2001) and four buckling restrained braced frame models which are modified according to the SAC building models of Sabelli (2001). All the models in this study are designed to be located in Los Angeles. The design is completed by equivalent lateral force (ELF) method according to ASCE 7-10 along with SAP2000 (CSI, 2011) software. Finite element analysis modeling and nonlinear dynamic analysis are completed by using Perform 3D (CSI, 2011). The finite element modeling procedure for the mesh method of beam, mass, beam, reduced beam section (RBS), column, panel zone, floor, brace and base will be discussed in this paper. BRB modeling parameters were provided by Xie (2015). Forty amplitude-scaled strong ground motions are selected to complete the nonlinear dynamic analysis on different models in this study. All the structural models are analyzed under two different loading cases: 1) Horizontal Only and 2) Horizontal plus Vertical.

1.3 Organization of thesis

Chapter 1 is the introduction to define the problem in this thesis. The motivation of the research and scope of the work are also discussed in this chapter.

Chapter 2 provides the literature review on vertical ground motion and the impact of vertical ground motion on different kinds of structures.

Chapter 3 provides the overview on the selected structures in this paper. The detailed design procedure for the three-story and six-story special moment frame with reduced beam sections are presented. The finite element modeling procedure for different kinds of elements in the moment frames and braced frames in SAP 2000 and Perform 3D are also discussed. Forty scaled ground motion records are also introduced in this chapter.

Chapter 4 presents the detailed results of the three-story and the six-story Perform 3D nonlinear dynamic analysis. Overall statistical data and graphs of the impact of vertical ground motions on the selected models are provided in this chapter. Some results of the individual case are also provided in this chapter to help to understand the difference of structure behavior under two different loading cases.

Chapter 5 is the conclusion of this thesis. This chapter will summarize the whole research paper and provide recommendations for further research.

Chapter 2 Literature Review

2.1 Introduction of vertical ground motion

It is a well-known fact that structures are subjected to three different dimensional earthquake ground motions. But earthquake design (ELF Method) for buildings usually only considers the horizontal component of ground motion while the vertical component of ground motion is generally neglected. Many building codes including NBC 105, IS 1893, UBC 97 and many other codes worldwide assume the vertical component of the ground motion to be 2/3 of the horizontal component which is originally provided by Newmark et al (1973). Figure 2-1 shows the ground motion acceleration records of E1-centro 1940 (Shrestha, 2009).

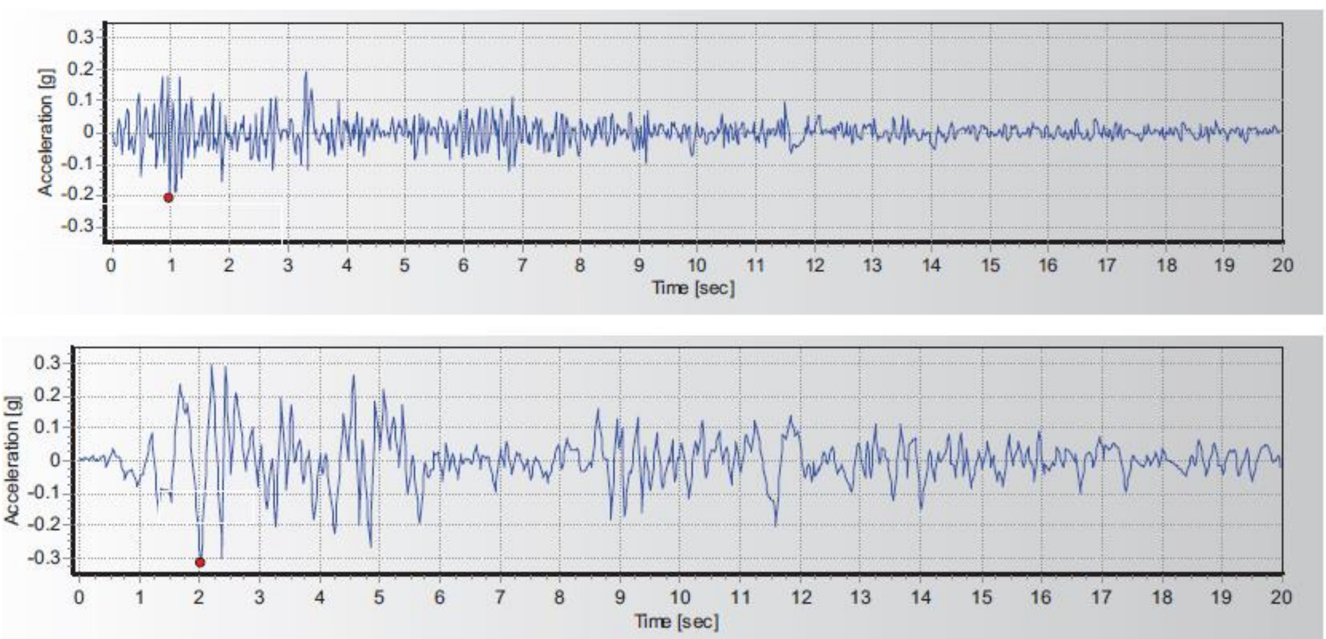

Figure 2-1: Ground Motion Time History of El-Centro 1940 Vertical (0.21g) and Horizontal (0.32g) Respectively from Top (SHRESTHA, 2009)

However, in recent years, many authors have started to think about the impact of vertical accelerations and have done significant research about the influence of vertical component of earthquakes on structures. The effect of vertical component of an earthquake on overall seismic response of structures has gained considerable interest to the profession. Many studies and reports' data show that vertical peak ground acceleration (PGA) may be even higher than the horizontal component value in the same earthquake which may cause significant damage on some critical parts of structures that can increase chances of collapse. Table 2-1 provides a list of earthquakes with significant V/H ratio. The V/H ratio was recommended to be more than 1 within a 5 km radius of the earthquake source, more than 2/3 within 25 km radius and has relationship with earthquake magnitude according to the studies by Collier and Elnashai (2001).

Table 2-1: Basic Ground Motion Information (Collier & Elnashai, 2001)

Event	Station(Mw)	Hor1(g)	Hor2(g)	Ver(g)	V/H
Gazli, Uzbeksitan 1976	Karakyr(6.8)	0.71	0.63	1.34	1.89
Imperial valley, USA 1979	El cenro array 6 (6.5)	0.41	0.44	1.66	3.77
Nahhani, Canada 1985	Site1(6.8)	0.98	1.10	2.09	1.90
Morgan hill, USA 1984	Gilroy array#7(6.2)	0.11	0.19	0.43	2.25
Loma-prieta, USA 1989	LGPC(6.9)	0.56	0.61	0.89	1.47
Northridge, USA 1994	Arleta fire station(6.7)	0.34	0.31	0.55	1.61
Kobe, Japan 1995	Port Island (6.9)	0.31	0.28	0.56	1.79
Chi Chi, Taiwan 1999	TCU 076 (6.3)	0.11	0.12	0.26	2.07

2.2 Near-fault influence of vertical ground motion

According to recent research on earthquakes such as the 1989 Loma Prieta, 1994 Northridge, 1995 Kobe and 1999 Chi-Chi, the vertical component of the ground motion was found to exceed the horizontal component of ground motion, which directly refutes the current code provision that assumes the value of the vertical-to-horizontal (V/H) spectral ratio of strong ground motion is around 2/3. Niazi and Bozorgnia (1989; 1990; 1991; 1992) analyzed more than 700 horizontal and vertical response spectra from 12 different strong ground motions recorded by the SMART-1 strong motion array in Taiwan. In the most recent study, they (Niazi & Bozorgnia, 1993) tested 159 horizontal and vertical response spectra from the 1989 Loma Prieta, California, earthquake and Bozorgnia et al (1995) analyzed 123 vertical and horizontal response spectra of 41 different kinds of soil sites in Northridge, California. Ansary and Yamazaki (1998) analyzed 2166 horizontal and vertical response spectra from 387 earthquakes recorded by 76 Japan Meteorological Agency (JMA) sites in Japan. All of these studies have the very similar conclusion: that the vertical-to-horizontal ratio (V/H) of strong ground motion is highly related to the period. The V/H ratio is higher when the period is shorter. They also figured out that the V/H ratios are relatively weakly correlated with magnitude, especially beyond the immediate vicinity of the fault. Significant research focuses on the near-fault recordings. They found that the V/H ratios of peak ground acceleration and response spectra also have strong relationships with source-to-site distance and could get close to or exceed 1.0 at the short period. Figure 2-2 shows an example study for predicting the V/H acceleration ratio and the influence of earthquake magnitude, distance to the rupture and fault mechanism on V/H ratio (Ambrasseey & Douglas, 2003).

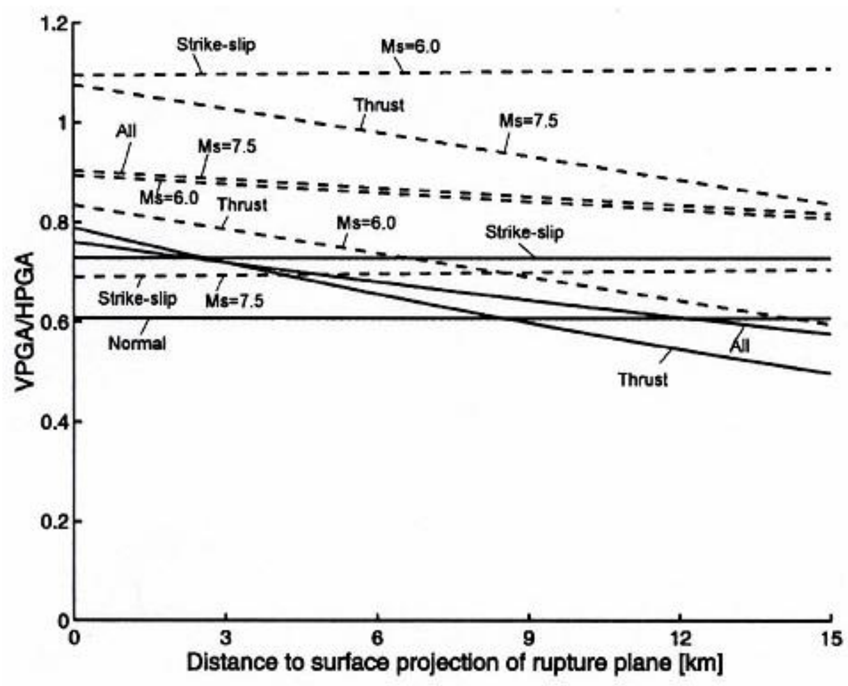

Figure 2-2: Effect of Magnitude, Distance and Fault Mechanism on the Ratio of Horizontal to Vertical Peak Ground Acceleration. (Ambrasseey & Douglas, 2003)

As Figure 2-3 (Yilmaz, 2005) shows, the ratio of vertical and horizontal peak ground accelerations is approximately linearly increasing with increasing magnitude. Similarly, Figure 2-4 and Figure 2-5 (Yilmaz, 2005) implies the changing trend of the ratio of vertical to horizontal peak ground motion velocity and displacement related to increasing magnitude. The opposite occurs with the ratio of peak acceleration, a linearly decreasing behavior with respect to increasing magnitude is observed in these two figures.

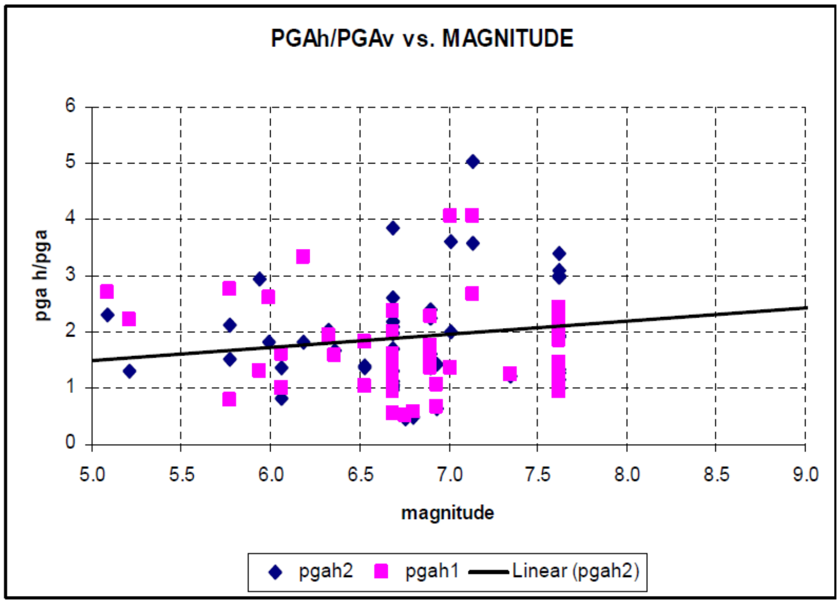

Figure 2-3: Relationship between Ratio of Horizontal to Vertical Peak Ground Accelerations and Magnitude of Earthquake (Yilmaz, 2005)

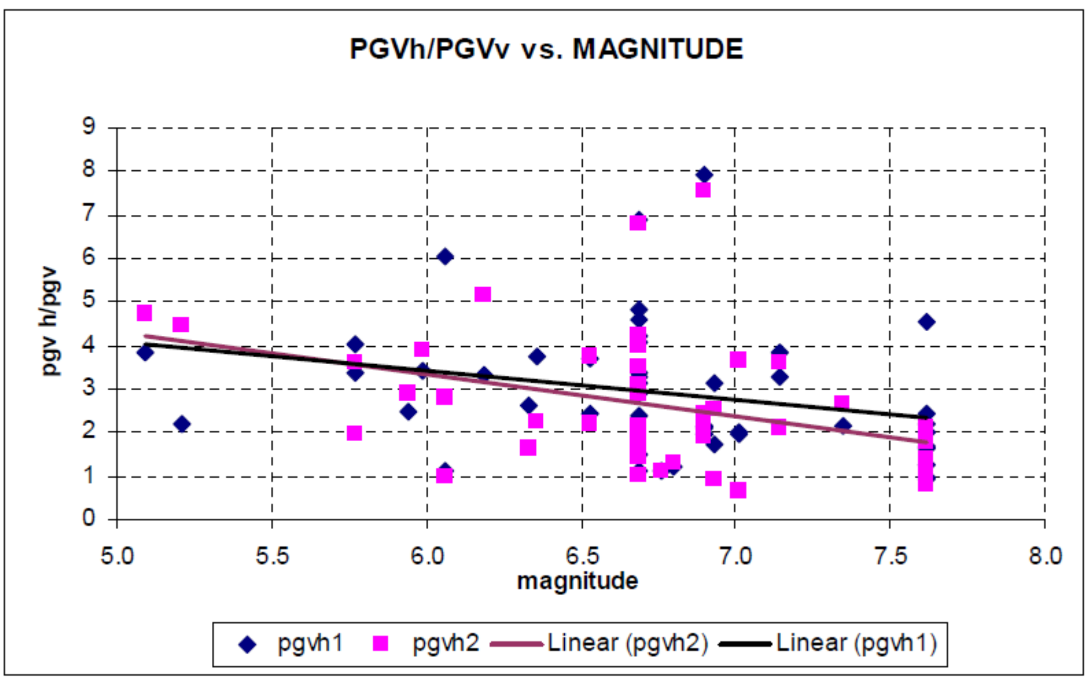

Figure 2-4: Relationship between Ratio of Horizontal to Vertical Peak Ground Velocity and Magnitude of Earthquake. (Yilmaz, 2005)

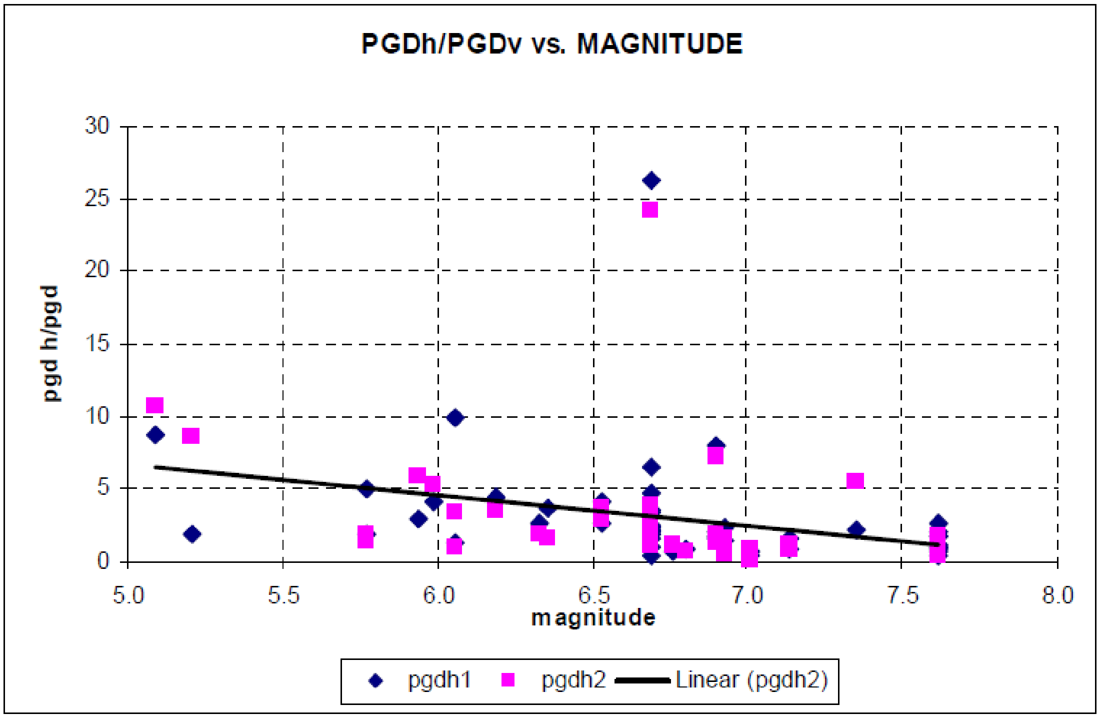

Figure 2-5: Relationship between Ratio of Horizontal to Vertical Peak Ground Displacement and Magnitude of Earthquake. (Yilmaz, 2005)

Bureau (1981) and Campbell (1982) recognized that the value of V/H in the near-fault region of large earthquakes was significantly different from that predicted at small magnitudes and large distances. Based on these studies, Campbell (1985) recommended that the standard engineering rule-of-thumb of assuming V/H =2/3 when estimating vertical ground motion component for design should be re-evaluated. Many of the papers mentioned previously have very similar conclusions.

Several researchers have given a lot of seismological explanations to prove that the V/H value has a strong relationship with distance from the epicenter and local site conditions. Silva (1997) pointed out that for short distances at soil sites the large contrast in shear-wave (S-wave) velocity at the rock/soil interface causes incident inclined SV-waves to be converted to P-waves (compressional waves) (S-to-P conversion) as they propagate through this boundary. These converted P-waves are subsequently amplified and refracted into a more vertical angle of incidence by a shallow P-wave velocity gradient. Because earthquake sources emit much larger S-wave amplitudes than P-wave amplitudes (by about a factor of 5), this has the significant effect of increasing the amplitude of the vertical component of ground motion over that caused by direct P-waves only. Silva explains that this effect is diminished at near-source rock sites because of the small S-wave velocity and the small S-wave and P-wave velocity gradients, which result in less S-to-P conversion which causes smaller value of V/H. According to Kawase and Aki (1987) and Silva (1997), at larger distances the SV-wave is beyond its critical angle of incidence and does not propagate to the surface very effectively. Therefore, at large distances, the lower-amplitude direct P-waves will dominate the vertical component of ground motion causing relatively smaller values of V/H.

Amirbekian and Bolt (1998) examined the differences between the spectral characteristics of near-fault vertical and horizontal ground motions from a seismological point of view. They finally reached the conclusion that the high-amplitude, high-frequency vertical accelerations that are observed on near-fault place recordings are most likely generated by the S-to-P conversion within the transition zone between the underlying bedrock and the overlying softer sedimentary layers, consistent with Silva's hypothesis.

According to an analysis of five significant earthquakes in California, Beresnev et al (2002) found that SV-waves dominate vertical ground motions at periods longer than about 0.1 second; and at shorter periods, P-waves may contribute a lot to these ground motions.

According to Figure 2-6, although the value of V/H is not very sensitive to magnitude and the distance from the rupture, we still can find that the value of V/H is decreasing along the distance from the rupture. For period T=0.1 second, as shown in Figure 2-7, especially for the firm soil at short distance, V/H increases with magnitude and decreases rapidly with distance.

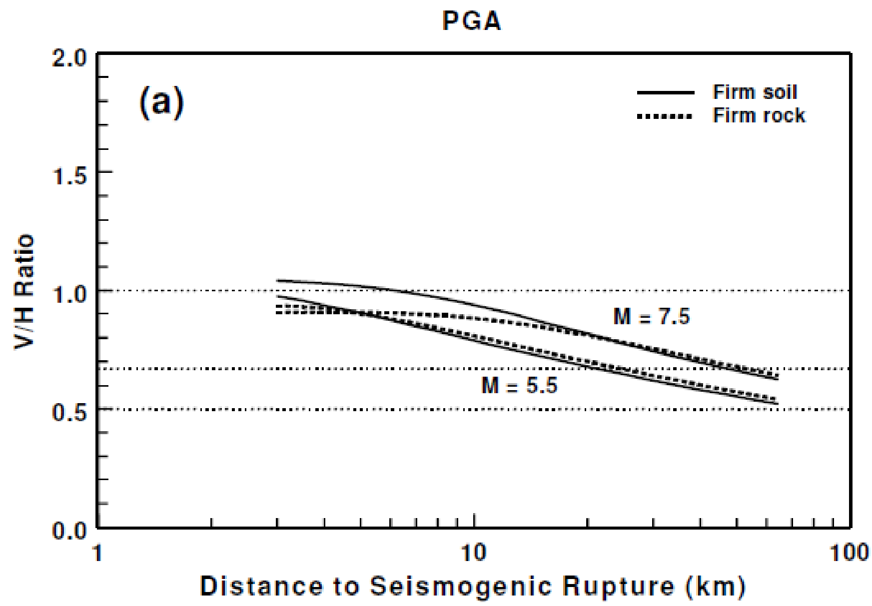

Figure 2-6: V/H Value of PGA along the Distance from the Rupture (Bozorgnia & Campbell, 2004)

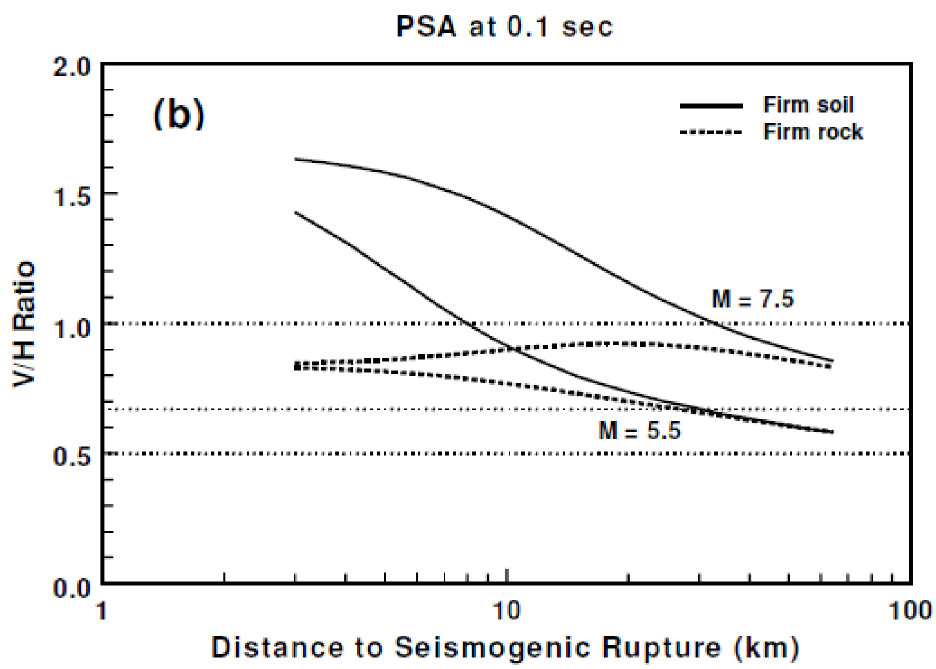

Figure 2-7: V/H Value of PSA=0.1 sec along the Distance from the Rupture (Bozorgnia & Campbell, 2004)

2.3 Frequency and period influence of vertical ground motion

A couple years ago, the Commission of the European Communities (1993) allowed V/H to vary with period in the European Building Code (EC8), but mainly as a means of reducing V/H from 2/3 at short periods to 1/2 at long periods. The 1997 Uniform Building Code (UBC-97) recognized the fact that V/H is dependent on source-to-site distance at relatively short distances and recommended using site special vertical response spectra for sites located close to active faults. However, neither the UBC-97 nor the 2000 International Building Code (IBC-2000) gives guidance on how to develop a general vertical design spectrum, especially in the near-fault region. The American Petroleum Institute, in Recommended Practice for Planning; Designing and Constructing Fixed Offshore Platforms (RP 2A-WSD), recommends using V/H = 1/2, but the main focus is on long period structures.

Figure 2-8 shows the horizontal and vertical response spectrum in El-centro earthquake in 1940, we can easily figure out that the vertical response acceleration is much higher than the horizontal component in the short period and decreases rapidly when it comes to relatively long period. Figure 2-9 shows that the high vertical acceleration component in the very short period causes much higher ratio of vertical to horizontal spectral acceleration compared with the code provision value 2/3. Although this value should be lower than 2/3 in most situations, the vertical component has tendency to concentrate all its energy in narrow high frequency band which can cause damage to structures.

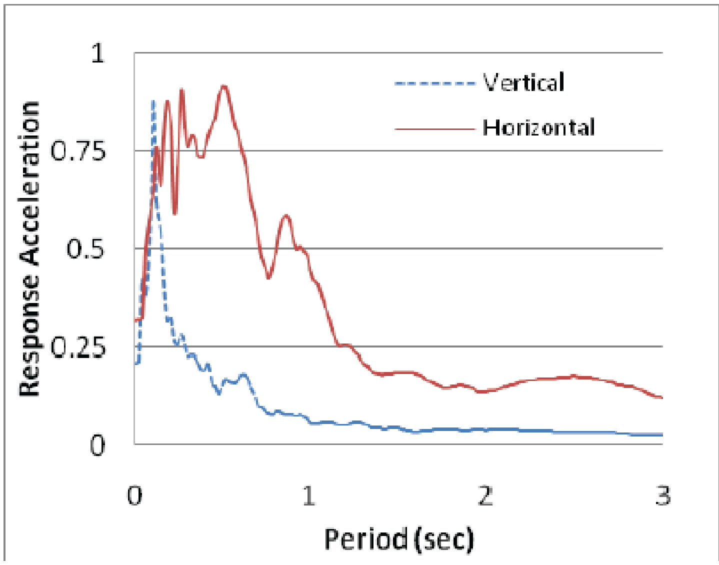

Figure 2-8: Response Spectrum for 1940 El-centro earthquake (Shrestha, 2009)

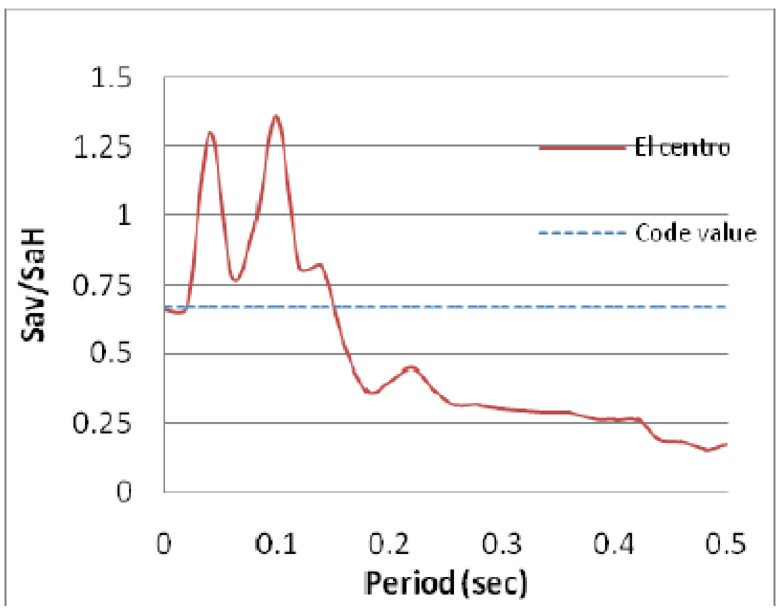

Figure 2-9: Comparison of Spectral Ratio at Short Period for El-Centro with Code Value (Shrestha, 2009)

There are three western North America (WNA) attenuation relations which are used to give the estimates of horizontal and vertical components of ground motion in engineering practice. They can also be used to predict V/H spectra. These relations are Sadigh et al (1993), Abrahamson and Silva (1997), and Campbell (1997; 2000; 2001). The Sadigh et al (1993) relation is only valid for rock site, while the other relations address other site conditions as well. All of them define the faulting mechanism as either strike slip or reverse slip. Sadigh et al. defined rock as a geologic unit with no more than a meter of soil overlying bedrock, but the other researchers have found that deeper soil units were also apparently used (Stewart, Liu, & Choi, 2003). Abrahamson and Silva classified sites as either generic soil or generic rock, where generic soil is a geologic unit with at least 20 meters of soil overlying bedrock and generic rock is a geologic unit with less than 20 meters of soil overlying bedrock. Campbell (Campbell, 1997) classified sites as either alluvium (firm soil with at least 10 meters of soil overlying bedrock), soft rock, or hard rock. Campbell (2003a; 2003b; 2003c) gives a more detailed summary and comparison of all four attenuation relations. The results are shown in Figure 2-10 and 2-11.

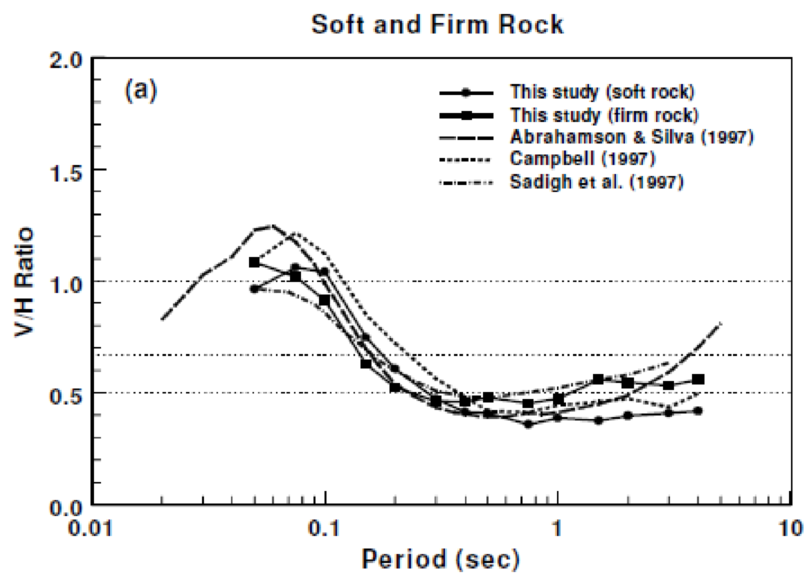

Figure 2-10: Relationship between the V/H Ratios of PSA and Period for Soft and Firm Rock according to Four Different Researchers (Campbell & Bozorgnia, 2003)

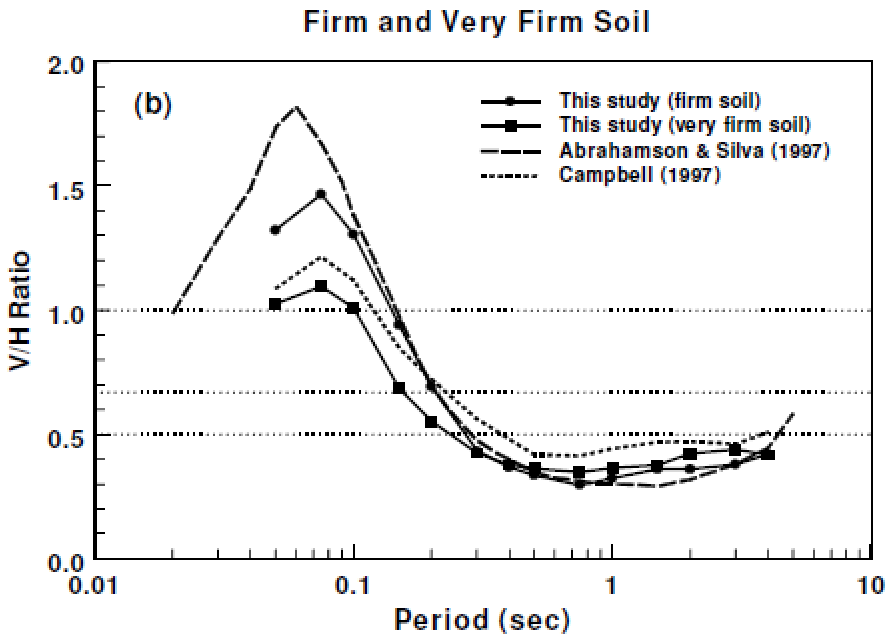

Figure 2-11: Relationship between the V/H Ratios of PSA and Period for Firm and Very Firm Soil according to Four Different Researchers (Campbell & Bozorgnia, 2003)

2.4 Field evidence of the damaging effect of vertical ground motion

It is commonly argued that the vertical strong motion component is insignificant for the effect of damage because of its low energy content. It is obvious that the energy content of the vertical component is significantly less than the energy of the corresponding horizontal component. However, such an explanation for neglecting the vertical component is not enough, since the horizontal energy content is dominated by long period pulses which are non-existent in vertical strong-motion records. What is of significance in earthquake response is the relationship between structural and excitation periods. Hence, the strong-motion energy stored in which frequency range is the absolute criterion to decide on the importance of the components of earthquake (Papazoglou, 1995; Elnashai & Papazoglou, 1995).

Papazoglou and Elnashai (1996) did research about the Kalamata, Greece, earthquake in 12 September, 1986 which had a magnitude of 5.7. This event had an epicenter located less than 9 km from the town center and a focal depth of 7 km (Elnashai & Pilakoutas, 1986). Figure 2-12 shows that some cracks occurred in the mid-height of the reinforcement concrete pedestals.

Figure 2-12: A Bench which has Displaced Horizontally without Friction at the Interface (Papazoglou & Elnashai, 1996)

Some researchers doubt that the reason for these cracks is because of the flexure tensile force due to the horizontal component in the earthquake. However, according to the damage inflicted on reinforced concrete buildings, Elnashai et a1 (Elnashai & Pilakoutas, 1986; Elnashai, Pilakoutas, & Ambraseys, 1988; Elnashai, Pilakoutas, Ambraseys, & I. D. Lefas, 1987) report an unusually high number of symmetric compression and shear-compression failures in columns and shear walls, as opposed to bending failures. Such failures are even observed in buildings with a soft ground story where bending failure is always expected. Although some of these kind of failures can be attributed to bad detailing design and poor construction errors, it is still reasonable to claim that the vertical component in this strong ground motion also acted in a significant role. It is obvious that the variation of vertical forces inevitably gave a reduction in shear strength due to loss or reduction of the concrete contribution. These could have easily led to the observed shear failures in walls and columns. Moreover, the increase in axial compressive forces due to vertical component of ground motion combined with poor details also led to a large number of compression and compression-shear failures in walls and columns, in cases where the transverse shear capacity was not exceeded by demand. These were often proved by symmetric buckling of the reinforcement or by X-type shear failure at middle height of columns or walls. An example is shown in Figure 2-13 where compressive failure of a first story reinforcement concrete column took place despite the presence of an incomplete infill panel which would preferentially induce a shear failure in the short column (Papazoglou & Elnashai, 1996).

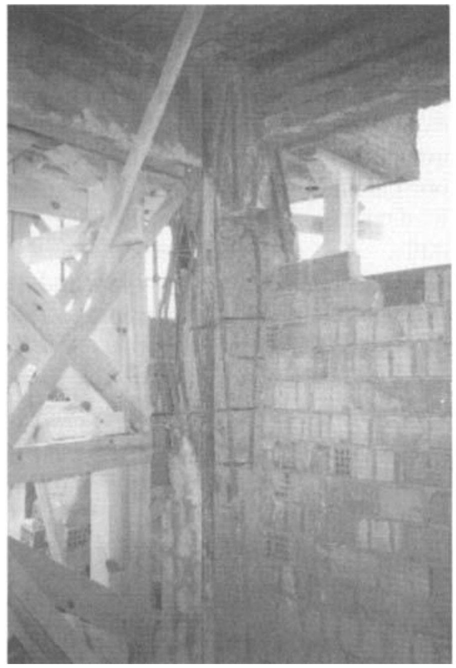

Figure 2-13: Compressive Column Failure in a Residential Building in Kalamata. (Papazoglou & Elnashai, 1996).

There are some other photos shown in Figure 2-14 through 2-16 as the field evidence given by Papazoglou and Elnashai which can be explained by the effect of vertical ground motion on the damage of buildings.

Figure 2-14: Shear-Bond Splitting Failure in 3rd story of Holiday Inn Hotel in Van Nuys. (Papazoglou & Elnashai, 1996).

Figure 2-15: Second Story collapse of Kaiser Perrnanente Building in Balboa Boulevard at Northridge. (Papazoglou & Elnashai, 1996).

Figure 2-16: Collapsed CSUN Parking Structure. The Perimeter Frame does not Show Signs of Distress where Gravity System has not Failed. (Papazoglou & Elnashai, 1996).

2.5 Analytical research on the damage effect of vertical motion based on cantilever model

Iyengar and Shinozuka (1972) did research on the effect of vertical acceleration and distribution mass on tall buildings which could be idealized as cantilevers which can be seen in Figure 2-17. The effect of the vertical acceleration is considered only to change the weight of the structure. The extensional motion and the effect of second horizontal component are not included in this analysis.

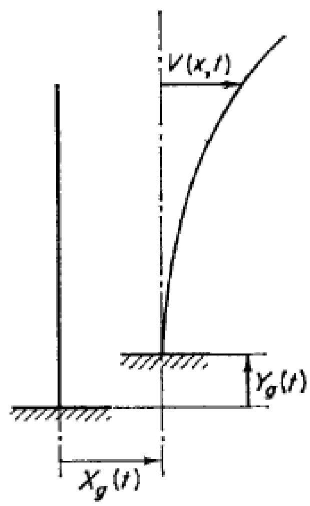

Figure 2-17: Cantilever Model (Iyengar & Shinozuka, 1972)

The most significant responses of the structure under their study would be the relative deflection at top, bending moment and shear force at the base (R. M. S response). These three responses have been studied in some detail for three different kinds of cantilevers which have different fundamental frequencies and heights. Table 2-2 shows the properties for these three different kinds of cantilever models (Iyengar & Shinozuka, 1972).

Table 2-2: The Properties of Three Different Cantilever Models (IYENGAR & SHINOZUKA, 1972)

Structure	h (ft)	m (lb ft^{-2} sec^2)	EI (psf)	f_1^* (cps)
I	50	10	$3{\cdot}6 \times 10^8$	1·343
II	100	10	$3{\cdot}6 \times 10^8$	0·336
III	200	10	$25{\cdot}2 \times 10^8$	0·222

Figure 2-18 shows the relationship between the estimates of the R. M. S. responses and the time history. These figures clearly indicate the effect of the self-weight and the vertical acceleration. As is expected, the difference is more significant with the taller structures (II, III) than with the shorter structure (I). For structural design the highest absolute peak responses are more significant and hence these are also obtained in all the samples during the first five seconds of earthquake.

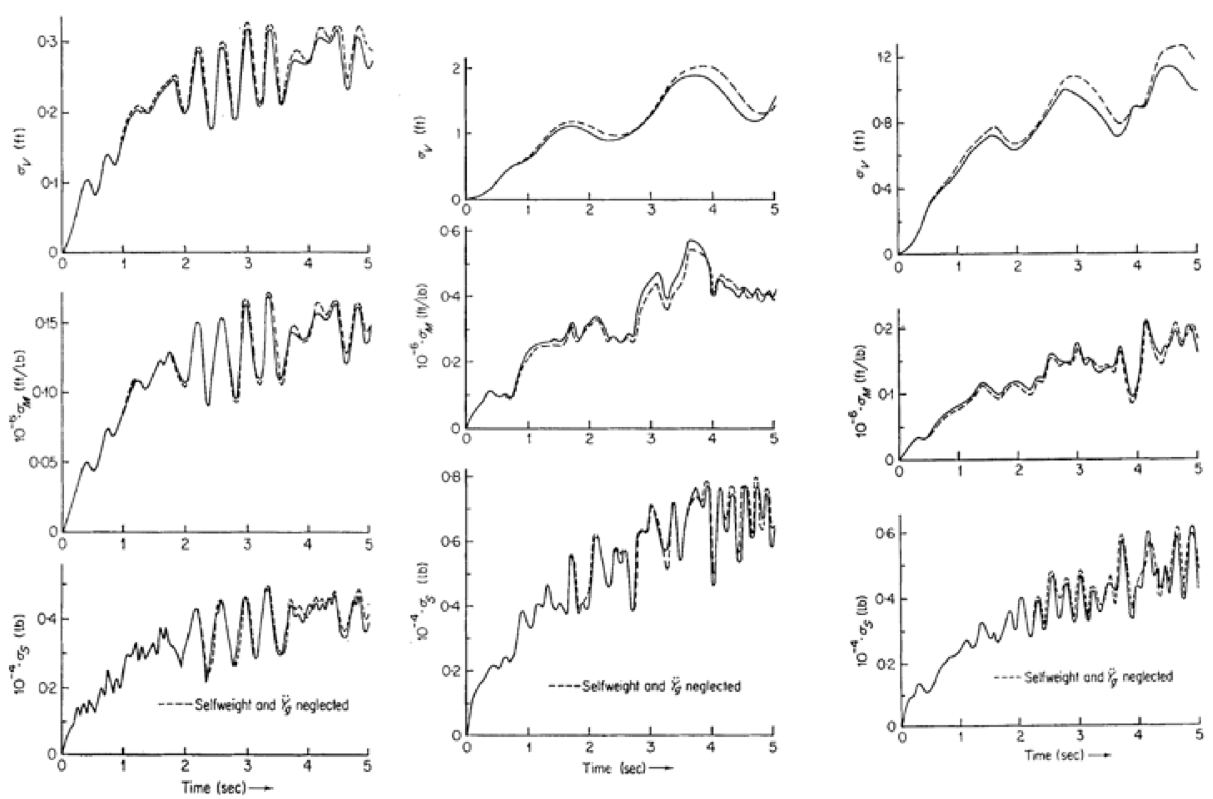

Figure 2-18: R. M. S Response of Structure I, II, III (Iyengar & Shinozuka, 1972)

2.6 Analytical research on the damaging effect of vertical motion

It is quite obvious that the ratio of vertical component to horizontal component in strong ground motion could easily exceed the value which is recommended by most code in the world at initial period. Then the question in mind will be, does the higher spectra ratio at short period of the response spectra have significant effect on the behavior of structures in the earthquake?

Anderson and Bertero (1973) did research on a ten story building which only had an unbraced, single bay frame. Following seismic design practice, this frame was designed for seismic loads according to the Uniform Building Code (1970) and the members were designed according to the allowable stress design(ASD) procedures described in the AISC Specification (1970) although more efficient seismic design methods have been provided by Anderson and Gupta (1972) and Bertero (1972). Figure 2-19 shows the analytical model in their study. They tested their model with the response spectrum of Pacoima and Taft earthquake. There are three different kinds of load cases for them, horizontal load only (H), horizontal load plus gravity load (HG) and horizontal load plus vertical load plus gravity load (HGV).

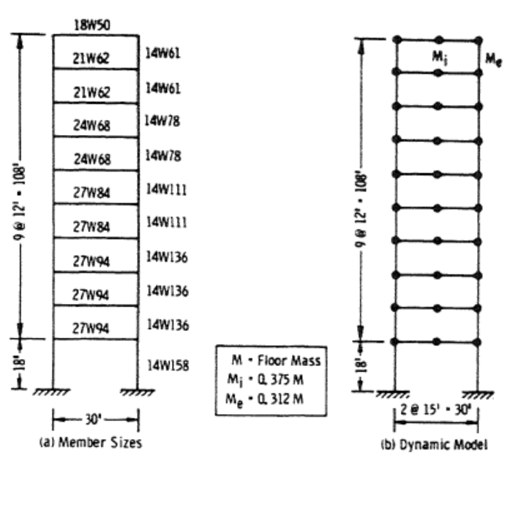

Figure 2-19: Analytical Model (Anderson & Bertero, 1973)

They concluded that the inclusion of gravity load results in a significant increase in the ductility requirement of the upper story girders and lower story columns. Inclusion of the vertical component of ground motion can result in a further increase in the ductility requirements of these elements and can also increase significantly the ductility requirement of the upper story columns.

Figure 2-20 shows the maximum vertical acceleration at each node for the HGV load case. We can find that the vertical response of the frame to the two different ground motion is quite similar. Figure 2-21 shows the maximum girder ductility at face of column while Figure 2-22 shows the maximum girder ductility at mid-span. Figure 2-23 shows the maximum column ductility.

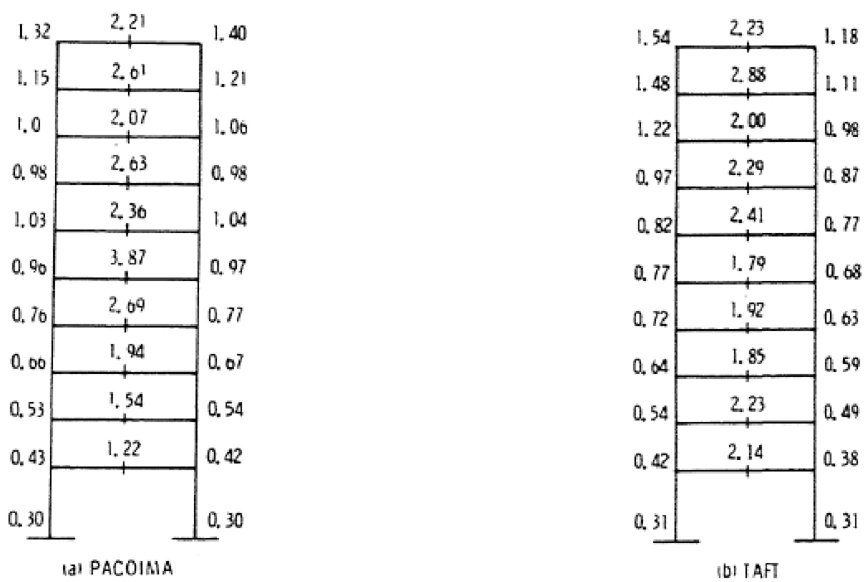

Figure 2-20: Maximum Vertical Accelerations (Anderson & Bertero, 1973)

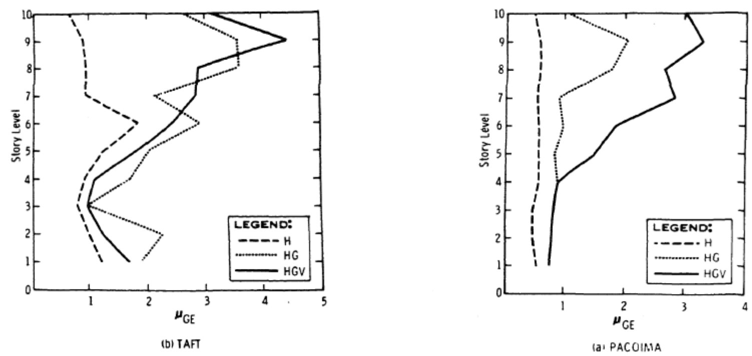

Figure 2-21: Maximum Girder Ductility at Face of Column (Anderson & Bertero, 1973)

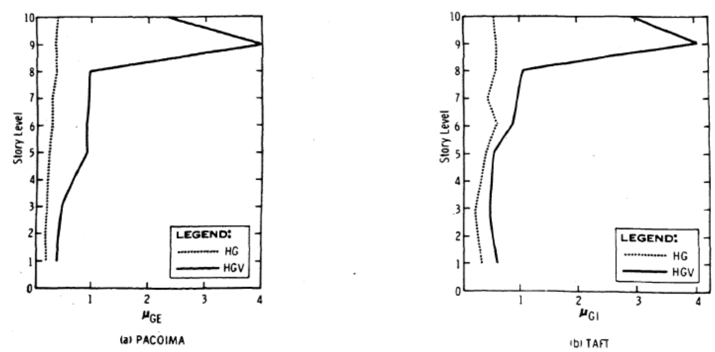

Figure 2-22: Maximum Girder Ductility at Midspan (Anderson & Bertero, 1973)

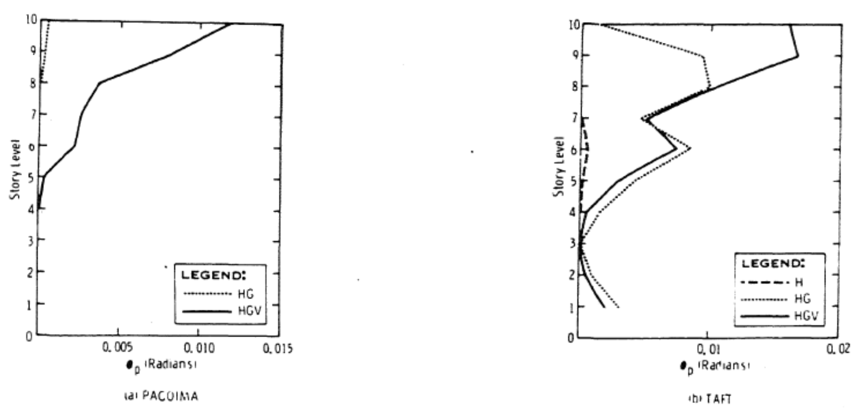

Figure 2-23: Maximum Column Ductility (Anderson & Bertero, 1973)

Buildings seem to be much stiffer in the vertical direction than the horizontal direction. Papadopoulou (1989) found that for reinforced concrete moment resisting frames, the ratio of vertical to horizontal fundamental period can vary from 6.67 to 2.5 for a range of stories from 8 to 1. Table 2-3 shows the vertical and horizontal periods for a reinforced concrete moment resisting frame building from first story to eighth story.

Table 2-3: Relationship between First Mode Horizontal and Vertical Periods for RC Buildings.
(Papadopoulou, 1989)

Number of floors	Horizontal period (s)	Vertical period (s)	Ratio
1	0·1	0·040	2·50
2	0·2	0·064	3·13
3	0·3	0·082	3·66
4	0·4	0·091	4·40
5	0·5	0·099	5·05
6	0·6	0·106	5·66
7	0·7	0·114	6·14
8	0·8	0·120	6·67

Steel structures also have a similar result. Papaleontiou and Roesset (1993) have done a lot of research on four 3-bay steel moment resisting frames which have spans of 4.5-8.4 m. These structures are taken from several other studies and the design strategies for these buildings are not consistent. Particularly, the 4-story and the 10-story frames are relatively much more flexible in the horizontal direction than the other two so that this kind of difference will significantly affects the ratio of vertical-to-horizontal periods. However, these examples can still be used to prove a very broad trend of this important ratio, as applied to steel moment resisting frames. Table 2-4 shows the relationship between vertical component and horizontal component in strong motion for the steel structure buildings with different number of floors.

Table 2-4: Relationship between First mode Horizontal and Vertical Periods for Steel Buildings. (Papaleontiou & Roesset, 1993)

Number of floors	Horizontal period (s)	Vertical period (s)	Ratio
4	1·0	0·16	6·25
10	2·22	0·20	11·10
16	1·54	0·19	8·11
20	2·27	0·25	9·08

A couple of lumped parameter MDOF structural models have been analyzed by Papadopoulou (1989) to indicate that the strong vertical ground motion can lead to column tension. Figure 2-24 shows that for the buildings and records examined, column tension in upper stories always occurs for peak ground acceleration exceeding 0.43g and for buildings which have more than two stories, even though horizontal motion is not considered in the analysis (Papadopoulou, 1989).

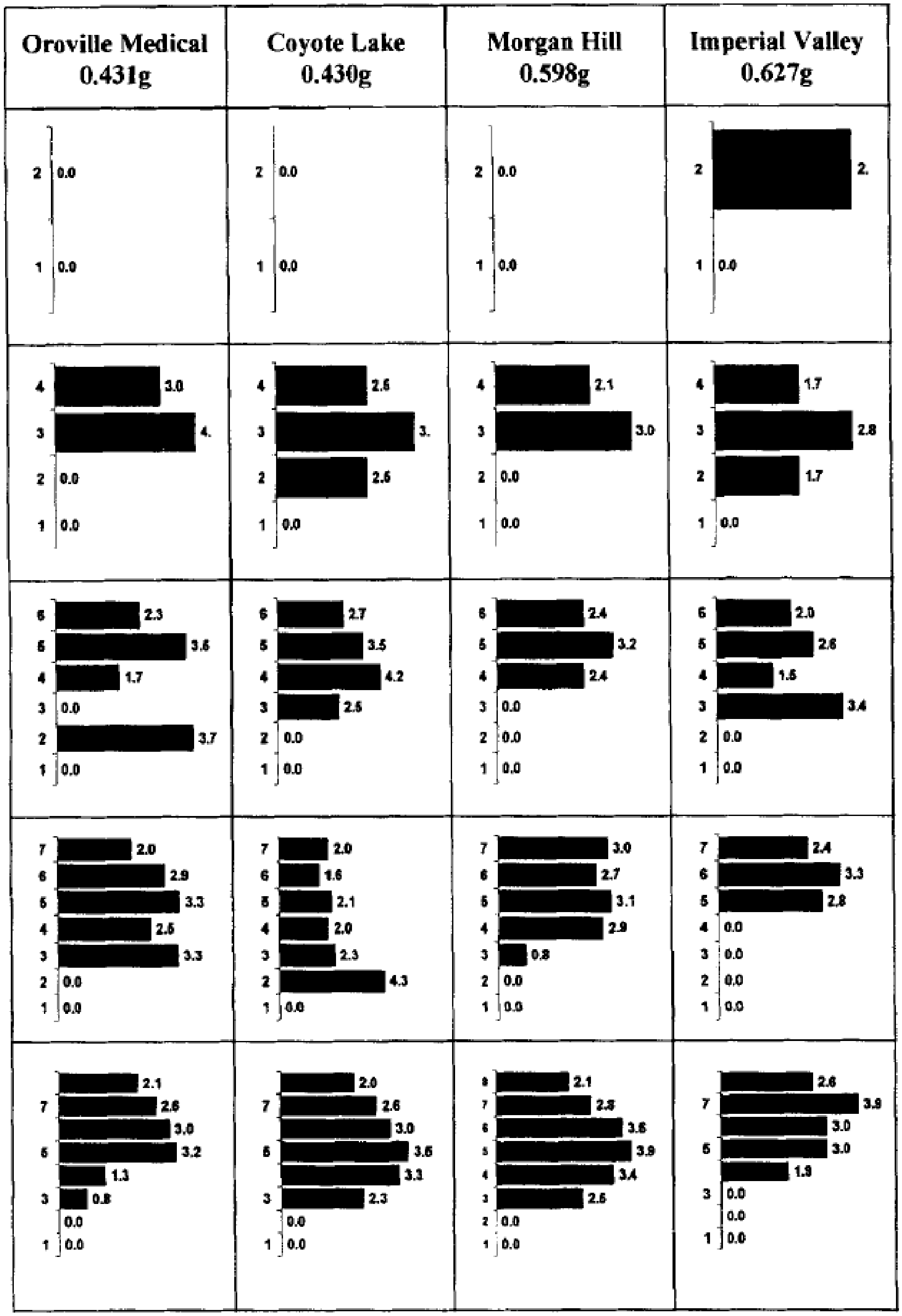

Figure 2-24: Pattern of Maximum Tensile Displacements along the Height of Multi-Story RC Buildings subjected to Various Vertical Earthquake Motions. (Papadopoulou, 1989)

Papaleontiou and Roesset applied a linear time-history analysis to steel buildings using the 1989 Loma Prieta records from Capitola (a_v= 0.52g; a_h = 0.47g). They didn't apply the gravity load to these steel structure models. Table 2-5 shows the maximum compressive and tensile axial forces for exterior columns, where the effect of overturning moments is larger (Papaleontiou & Roesset, 1993).

Table 2-5: Effect of Vertical Motion on Axial Force Response of Steel Frames. (Papaleontiou & Roesset, 1993)

	Axial forces (kN)				Contribution of vertical motion to total axialforce (%)	
No. of storeys	Roof H	Roof $H+V$	Ground H	Ground $H+V$	Roof	Ground
4	– 22/ + 40	± 110	± 200	± 450	72	56
10	± 22	± 150	± 490	± 850	85	42
16	– 58/ + 80	± 290	± 5400	± 7100	76	24
20	± 49	± 135	± 3300	± 4000	64	21

What's more, Koukleri (1992) did the nonlinear dynamic analysis for an 8 story, 3 bay moment resisting reinforced concrete frame building which is designed according to UBC. The objective of this analysis is to confirm the occurrence of net tensile forces and displacements, thus dispelling the question of high frequency excitation often used to support the insignificance of the vertical component. Table 2-6 shows the effect of vertical motion on tensile column forces and displacements for a reinforced concrete frame.

Table 2-6: Effect of Vertical Motion on Tensile Column Forces and Displacements for a RC Frame. No Tension occurs under only Horizontal Loading. (Koukleri, 1992)

Column	Tensile force (kN) $(H + V)$	Tensile displ. $(H + V)$ (mm)
1st storey exterior	475	1·90
4th storey exterior	350	1·30
8th storey exterior	150	0·60
1st storey interior	500	1·95
4th storey interior	750	3·75
8th storey interior	210	0·70

Table 2-7 shows that the vertical component of strong ground motion significantly contributes to the axial column force in reinforced concrete frame according to the Koukleri's analysis.

Table 2-7: Effect of Vertical Motion on Compressive Column Forces for a RC Frame. (Koukleri, 1992)

Column	Compressive force (kN) H	Compressive force (kN) $H + V$	Contribution of vertical motion to total axial force (%)
1st storey exterior	1500	1750	14
4th storey exterior	750	1250	40
8th storey exterior	125	350	64
1st storey interior	1450	2500	42
4th storey interior	800	2525	68
8th storey interior	215	1200	82

Georgantzis (1995) applied a non-linear analysis on a coupled shear wall 8-storey 3-bay RC frames which are designed by Fardis (1994) according to EC-2/EC-8 for an acceleration of 0.15 g. He found that the vertical component of strong ground motion can contribute a lot to the upper-story failure. This result is totally consistent with the higher variation of axial forces in upper stories and the field evidence. What's more, it is clear to find that column shear failure becomes the controlling factor for ultimate response when the vertical component is included in the analysis. Figure 2-25 shows the critical behavior of Priestley's building model (Priestley, et al, 1993) for the response to the 1971 San Fernando record at Castsaic Old Ridge.

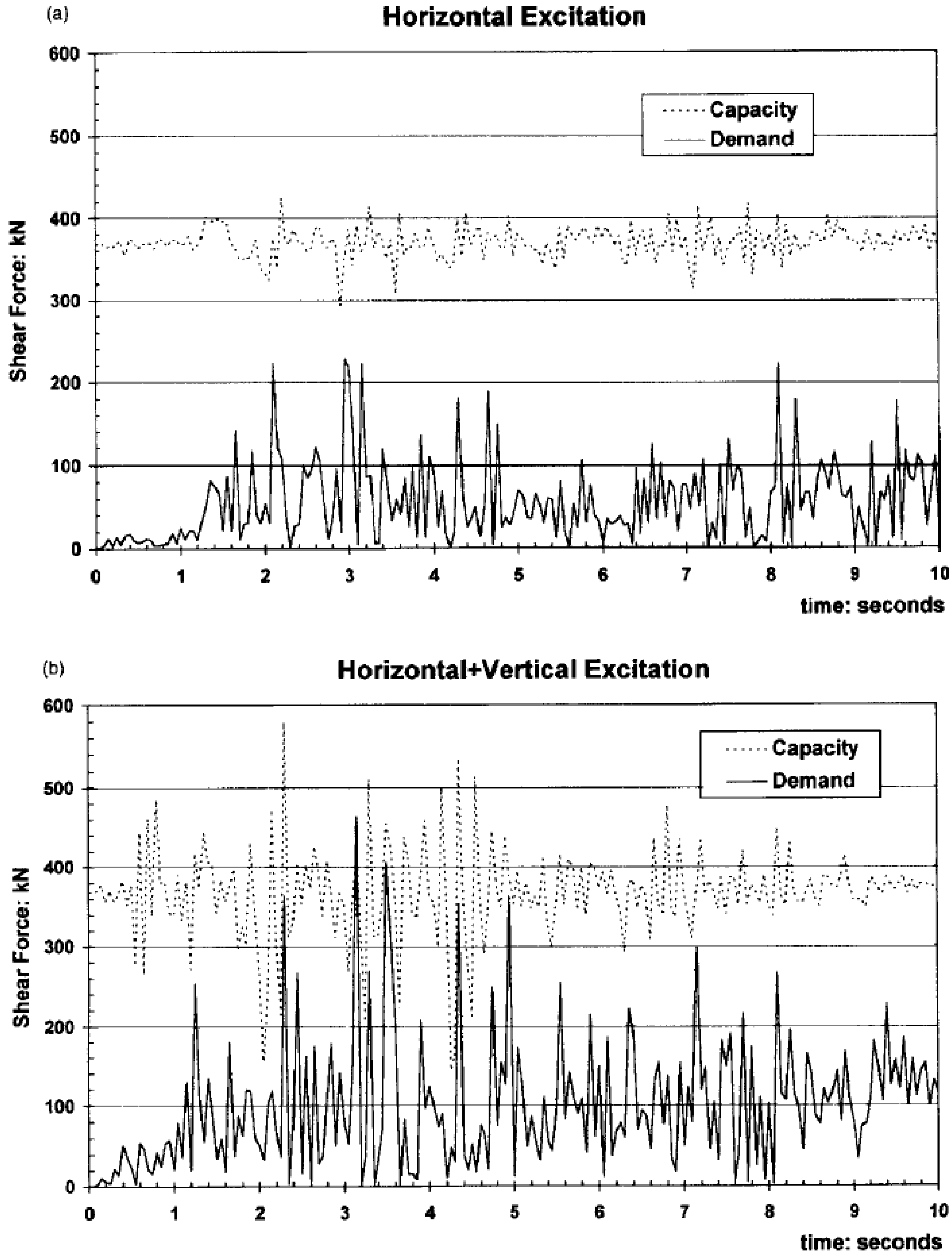

Figure 2-25: Effect of Vertical Earthquake Motion on Shear Response of RC Columns. Response to the 1971 San Fernando Record at Castaic Old Ridge. (Georgantzis, 1995)

2.7 Research on modeling mass

The accuracy of 2D dynamic analysis of a structure obviously depends upon the accuracy of the modeled dynamic properties. Whalen, Archer and Bhatia (Whalen, Archer, & Bhatia, 2004) have shown that improper vertical mass modeling, brought about by a simple but inappropriate assumption about mass lumping, can cause significantly changes in estimated seismic performance of a structure.

Figure 2-26 shows the floor plan in the north–south direction for three different LA SAC buildings (ATC, 2000). All of them have only two exterior moment frames to resist the horizontal motions so that each of them can typically take half the total horizontal mass of each floor. Due to the description above, each moment frame can take half the total vertical mass of each floor as well. This assumption and models were eventually adopted into many different dynamic analyses of the SAC buildings. Unfortunately, this will lead to an over prediction of the mass in the vertical direction. Although the horizontal inertial forces can be assumed to be equally distributed to the two exterior frames, the vertical inertial forces are not only resisted by the columns in the moment frame but also resisted by the gravity columns. Figure 2-26 shows the tributary areas of the vertical mass for the moment frame are much smaller than the tributary areas of the horizontal mass.

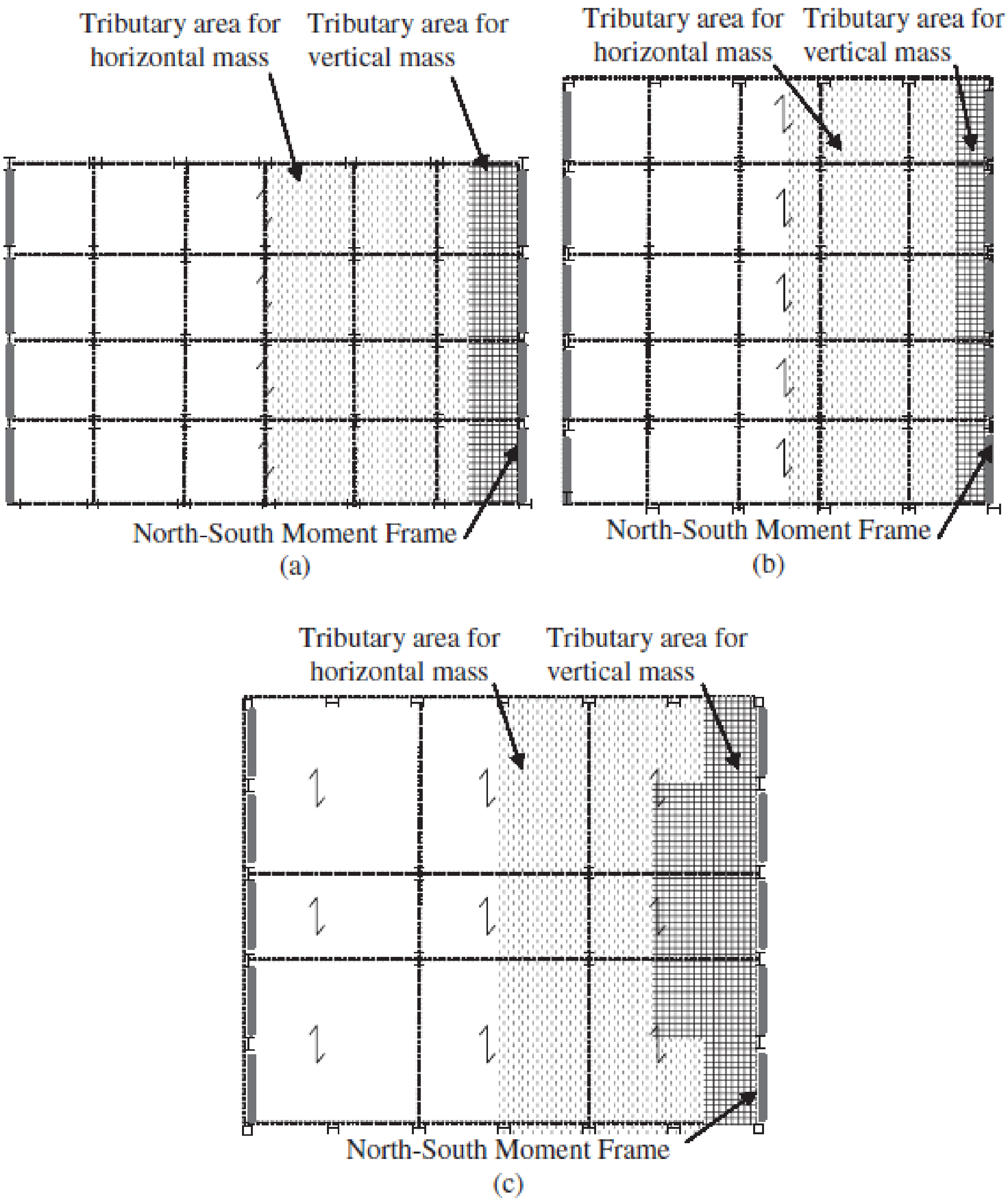

Figure 2-26: Tributary Areas for Horizontal and Vertical Mass: (a) LA 3-storey SAC building, (b) LA 9-storey SAC building, (c) LA 20-storey SAC building (Whalen, Archer, & Bhatia, 2004)

2.6 Summary

In this chapter, we can easily find that the observed and predicted V/H spectra have strong relationship with the fundamental period, source-to-site distance and local site condition while they have a relatively weak relationship with earthquake magnitude and faulting mechanism. Most of the codes we are using underestimate the effect of vertical component of strong motion in the short period while overestimate it in the long period. A couple of field observations and a lot of analytical results indicate that certain failure situations are mostly caused by the effect of vertical earthquake force. It is obvious to see that the vertical component of strong motions have a significant impact on the building behavior during the earthquake, especially the near-fault earthquake. The method for how to correctly model the mass both horizontally and vertically in 2D frames is also discussed in this chapter.

Chapter 3 Modeling of Special Moment Frame (SMF) and BRB Frames

3.1 Introduction

Structure design based on current seismic code is expected to make structures deform in the inelastic range under the design level ground motions. Steel structural elements can dissipate earthquake energy through inelastic deformation. This chapter concentrates on the element inelastic behavior for various elements such as beams, columns, panel zones and so on. Perform 3D is the software platform which will do the nonlinear dynamic analysis. Figure 3-1 shows the generalized force-deformation relation for steel elements or components.

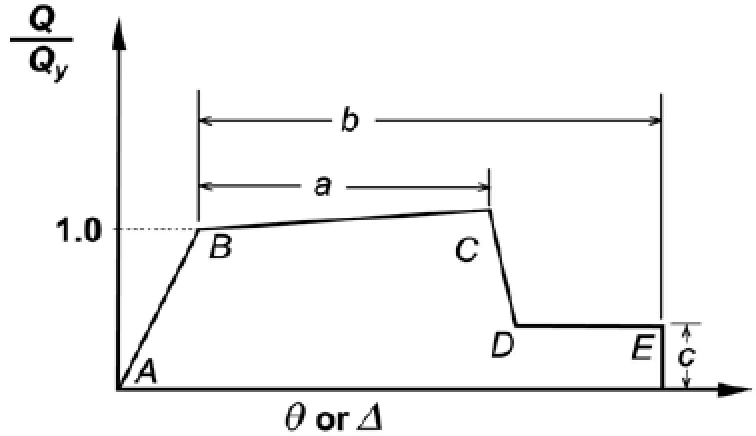

Figure 3-1: Generalized Force-Deformation Relation for Steel Elements or Components (FEMA, 2000)

3.2 Basic information about the building

3.2.1 Three-story Moment Frame Structure

The three story and six story moment frame and brace frame models are modified according to the SAC building models of Sabelli (2001). Figure 3-2 shows the plan view of the three story moment frame structure. The building is built as an office building and the story height for each floor is 13 ft. The plan view for the floor system shows that the dimension of north-south direction is 124 ft while the dimension of east-west direction is 184 ft. The structure has four bays in the north-south direction and six bays in the east-west direction. The bay size is 30 ft by 30 ft and there is a 12 ft tall penthouse on the roof of the building which is represented as a dash line rectangle. The dimensions of the penthouse are 30 ft by 60 ft. In the east-west direction, each bay has two secondary beams running from north to south. The distance between them is 10 ft. Four exterior frames are responsible for the seismic resistance.

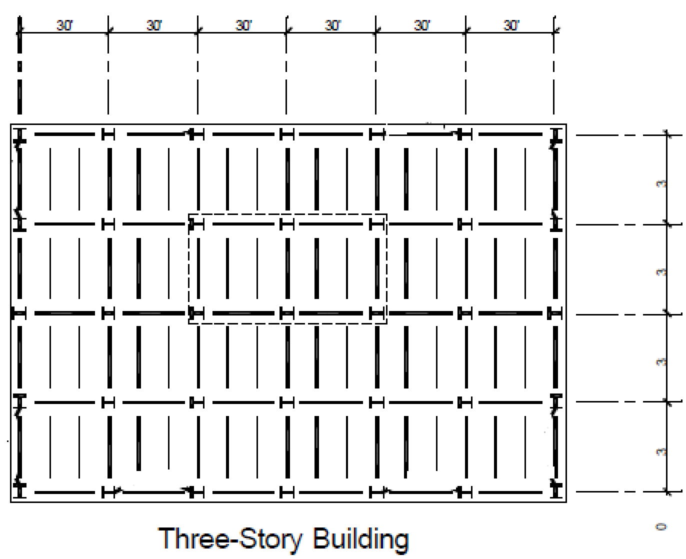

Figure 3-2: Plan View of Three-Story Moment Frame Structure (Sabelli, 2001)

3.2.2 Six-story Moment Frame

The six-story moment frame has a similar plan view to the three-story moment frame. The height of first floor is 18 ft and the height of the remaining floors is 13 ft. The dimensions of the building plan are 154 ft by 154 ft. The corner columns only have moment connections on the strong axis side. Wherever a beam connects to a column that is oriented in the weak-direction, a moment release is applied at the beam-to-column interface. There are 5 bays in each direction and the bay size is the same as the three-story moment frame structure. Secondary beams are also set up the same way with the three-story moment frame structure. A penthouse is put on the roof using the same area but different location. The exterior four bays are responsible for the seismic resistance. Figure 3-3 shows the plan view of the six-story moment frame structure.

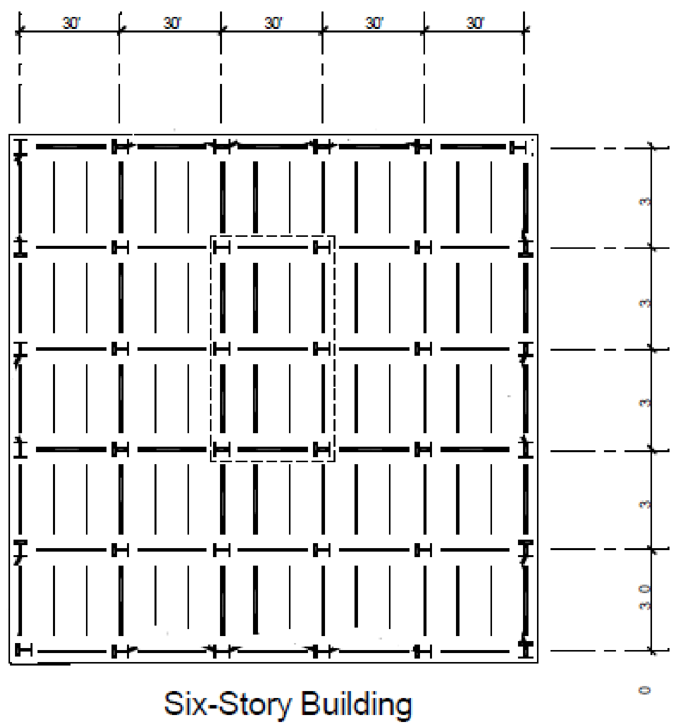

Figure 3-3: The plan view of six-story moment frame structure (Sabelli, 2001)

3.2.3 Three-story BRB Frame

The three-story braced frame structure is designed based on the three-story moment frame geometry. The BRBs (north-south direction) are put in the first bay and the fourth bay and they are designed in a chevron configuration and a single diagonal configuration. Figure 3-4 shows the plan view of the three-story BRB brace frame structure.

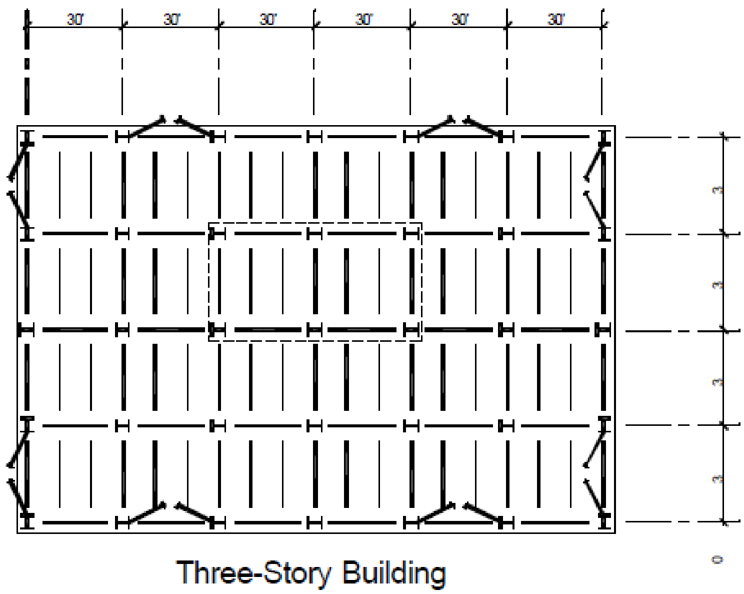

Figure 3-4: Plan View of the Three-Story BRB Braced Frame Structure (Sabelli, 2001)

3.2.4 Six-story BRB Frame

The six-story brace frame structure is designed based on the geometry of the six-story moment frame structure. The BRBs (north-south direction) are put in the first bay, third bay and fifth bay. They are designed in both a chevron and a single diagonal configuration. Figure 3-5 shows the plan view of the six-story BRB brace frame structure.

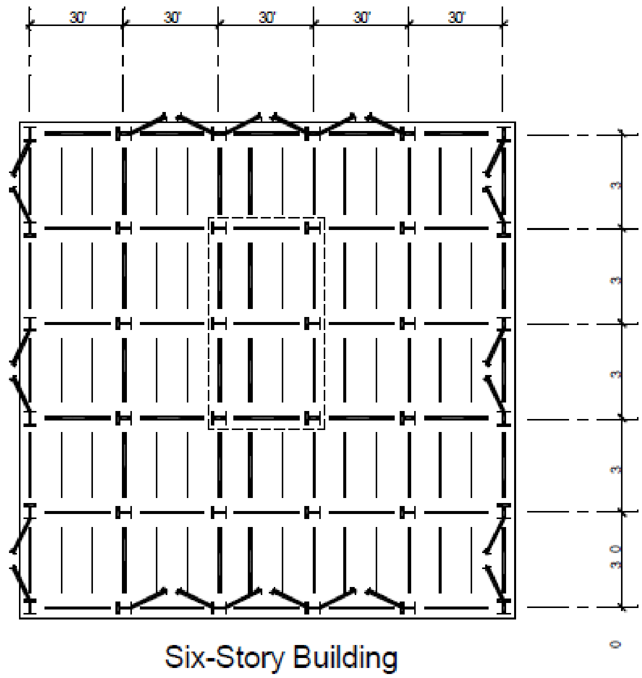

Figure 3-5: Plan View of the Six-Story BRB Braced Frame Structure (Sabelli, 2001)

3.3 Basic Seismic Design

3.3.1 Basic Seismic Information

According to the Sabelli report (2001), all these structures are located in Los Angeles, CA. The latitude and longitude of Los Angeles are 34°05′N and 118°242′W, respectively. The importance factor for all the structures is 1. Both site class and seismic design category is D. The maximum considered earthquake spectral response for short period, SS, is 2.09g while it is 0.77g for the one second period, S1. Fa is equal to 1.0 while Fv is equal to 1.5 which is given by the report. The design spectral response acceleration is 1.393g for short period, Sds, and 0.77g for long period, Sd1.

3.3.2 Equivalent Lateral Force (ELF) Procedure for Structure Design

Seismic loads were determined according to ASCE 7-10 (2010) Chapters 11 and 12. Accelerations, factors, and period limits were obtained from the Sabelli report. 3-D models are created in SAP2000 V15 (CSI, 2011) to design the three-story and six-story special moment frame structures. The BRB braced frames were designed by Xie (2015). Seismic weights for each floor were determined by using the assumed dead weights for the floor, columns, cladding, Mechanical/HVAC/Plumbing, etc. Table 3-1 shows the detailed information about the gravity load. The original design loads were based on calculated approximate periods. The fundamental period of the structures, which came from the SAP2000 model, were used to recalculate the equivalent lateral force and to check the drift for each floor which control the seismic moment frame design. The loads and the load distribution were calculated according to ASCE section 12.8.1 and 12.8.3. Detailed seismic load calculations can be found in Appendix A.

Table 3-1: Basic Information for Seismic Design (Sabelli, 2001)

Steel Weight	As Designed
Ceilings/ Flooring Weight	3" Metal Deck with 2.5" Normal-Weight Concrete
Roofing	7 psf
Ceilings/ Flooring	3 psf
Mechanical/ Electrical	7 psf
Mechanical/ Electrical at Penthouse	47 psf
Exterior Wall	25 psf
Metal Decking Weight	42 psf
Partition(Gravity Design)	20 psf
Partition(Seismic Design and Analysis)	10 psf
Live Load	50 psf

The three-story and six-story moment frame lateral resisting system design was accomplished with the assistance of SAP 2000 v.15 (CSI, 2011). Cross sections and properties of columns and beams in the moment frame structure are listed in the Tables 3-2, 3-3, 3-4 and 3-5. Figures 3-6, 3-7, 3-8 and 3-9 show the elevation view of the three-story moment frame as beam which means north-south direction moment frame, three-story moment frame as girder which means east-west direction moment frame, six-story moment frame as beam and six-story moment frame as girder, respectively. The information about beams, columns and braces for the three-story and six-story brace frame are listed in the Tables 3-6, 3-7, 3-8 and 3-9. The 3D model story drifts and allowable story drifts are shown in Table 3-10 and 3-11. LA3NCH and LA6NCH represent the three-story and six-story braced frame with normal yield length BRBs in chevron configuration at LA while LA3NSD and LA6NSD represent the three-story and six-story braced frame with normal yield length BRBs in single diagonal configuration at LA (Xie, 2015). The earthquake model story drift was checked against the allowable story drift according to ASCE 7-10 in Table 12.12-1.

Table 3-2: Sizes of Beam and Column for Three-Story Moment Frame as Beam Model

Member	Location	Size
Column	Exterior	W14X193
	Interior	W14X211
Beam	1st Floor	W27X94
	2nd Floor	
	3rd Floor	W18X40

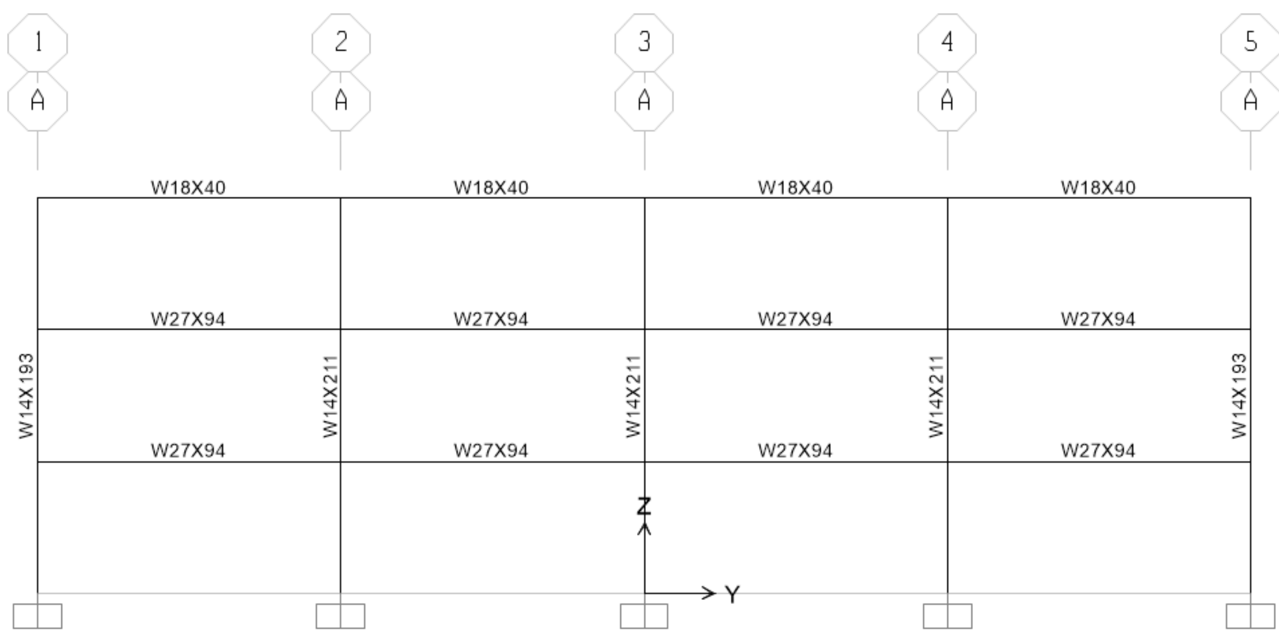

Figure 3-6: Elevation View of Three-Story Moment Frame as Beam

Table 3-3: Sizes of Beam and Column for Three-Story Moment Frame as Girder Model

Member	Location	Size
Column	Exterior	W14X193
	Interior	W14X159
Beam	1st Floor	W24X55
	2nd Floor	
	3rd Floor	W18X35

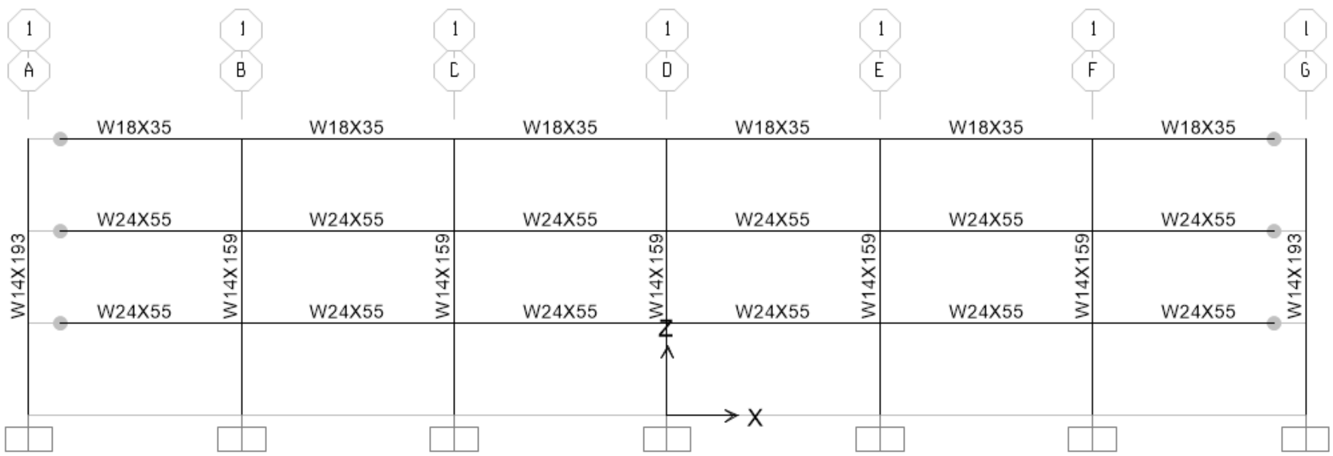

Figure 3-7: Elevation View of Three-Story Moment Frame as Girder

Table 3-4: Sizes of Beam and Column for Six-Story Moment Frame as Beam Model

Member	Location	Size
Column	Exterior Low	W14X257
	Exterior High	W14X176
	Interior Low	W14X257
	Interior High	W14X159
Beam	1st Floor	W30X108
	2nd Floor	
	3rd Floor	W27X94
	4th Floor	W24X76
	5th Floor	W24X62
	6th Floor	W18X40

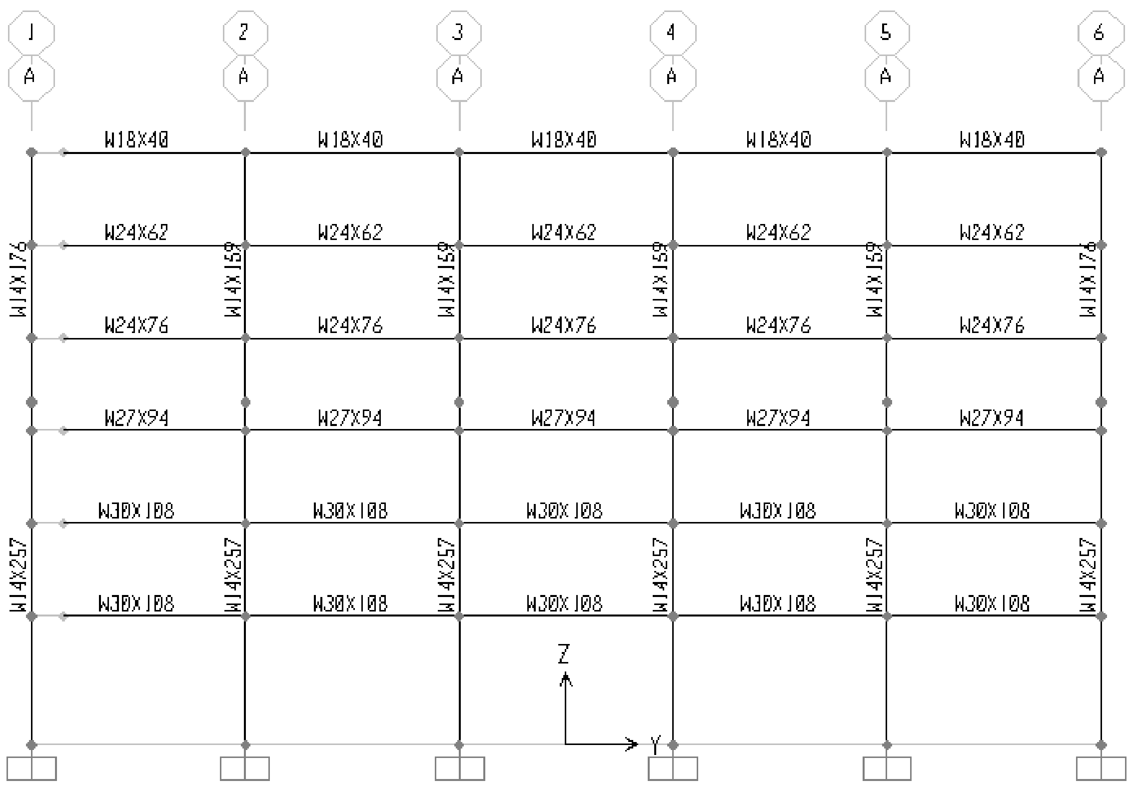

Figure 3-8: Elevation View of Six-Story Moment Frame as Beam

Table 3-5: Sizes of Beam and Column for Six-Story Moment Frame as Girder Model

Member	Location	Size
Column	Exterior Low	W14X257
	Exterior High	W14X176
	Interior Low	W14X311
	Interior High	W14X211
Beam	1st Floor	W30X108
	2nd Floor	
	3rd Floor	W27X94
	4th Floor	
	5th Floor	W24X76
	6th Floor	W18X40

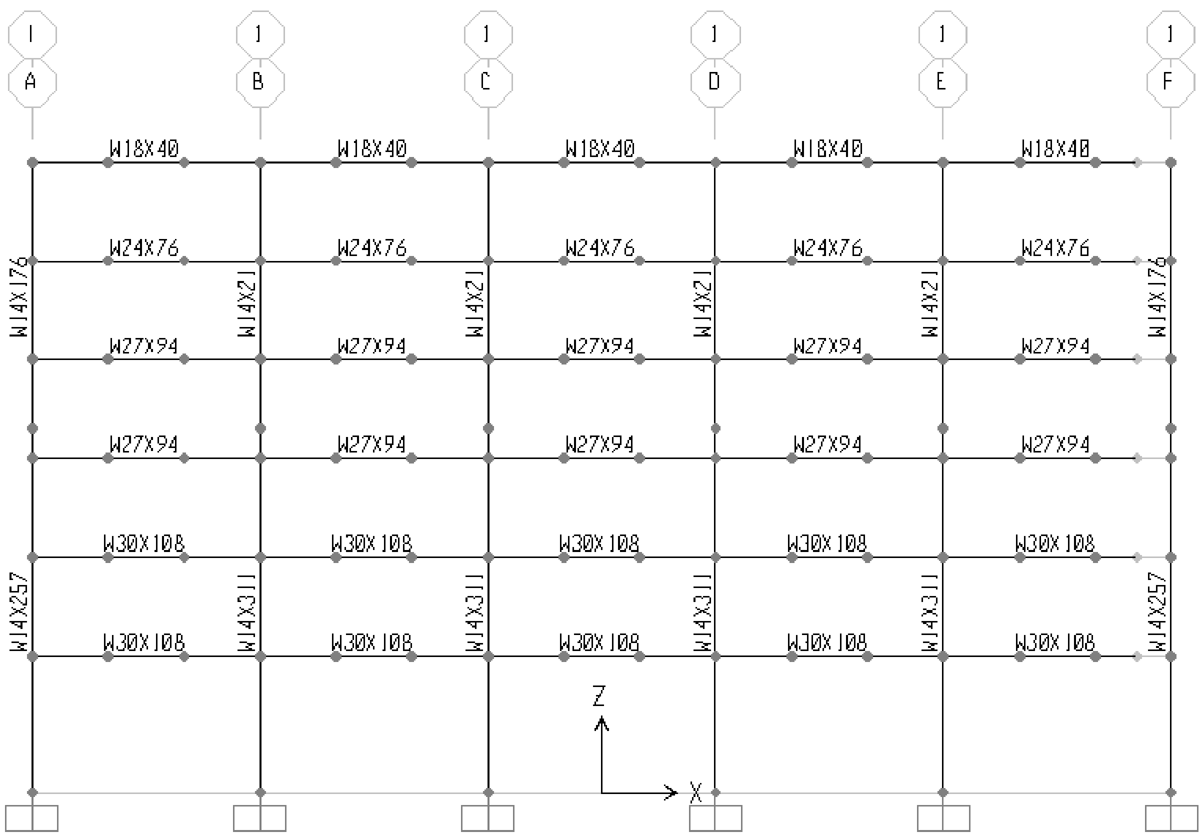

Figure 3-9: Elevation View of Six-Story Moment Frame as Girder

Table 3-6: Column and Beam Sections of Three-Story Braced Frame (Xie, 2015)

Story	Braced Frame Column	Braced Frame Beam	Non-brace frame columns (minor axis)					
			Side	Interior	Mechanical	Perpendicular to Brace Frame	I_y (in^4)	Z_y (in^3)
3	W12x96	W14x48	W14x48	W14x61	W14x74	W12x96	2015	559
2								
1								

Table 3-7: Column and Beam Sections of Six-Story Braced Frame (Xie, 2015)

Story	Braced Frame Column	Braced Frame Beam	Non-Brace Frame Columns (minor axis)				
			Interior	Mechanical	Perpendicular to Brace Frame	I_y (in^4)	Z_y (in^3)
6	W14x132	W14x48	W14x43	W14x53	W14x132	2591	605
5							
4							
3	W14x211	W14x48	W14x90	W14x99	W14x211	7136	1421
2							
1							

Table 3-8: BRB Properties of Three-Story Braced Frame (Xie, 2015)

Model	Story Level	Yield Force (kips)	Yielding Length (in)	Model	Story Level	Yield Force (kips)	Yielding Length (in)
LA3NCH	3rd story	161.5	152.9	LA3NSD	3rd story	266.0	270.1
	2nd story	247.0	138.5		2nd story	418.0	258.3
	1st story	304.0	131.9		1st story	513.0	264.0

Table 3-9: BRB Properties of Six-Story Braced Frame (Xie, 2015)

Model	Story Level	Yield Force (kips)	Yielding Length (in)	Model	Story Level	Yield Force (kips)	Yielding Length (in)
LA6NCH	6th story	57.0	145.9	LA6NSD	6th story	76.0	292.0
	5th story	76.0	161.7		5th story	114.0	290.1
	4th story	104.5	146.5		4th story	152.0	285.0
	3rd story	114.0	161.4		3rd story	180.5	283.2
	2nd story	123.5	161.4		2nd story	190.0	282.7
	1st story	133.0	199.7		1st story	199.5	327.8

Table 3-10: Drift Check Table for Three-Story Moment Frame

Story	Moment Frame Displacement-NS (ft)	Moment Frame Displacement-EW (ft)	Story Height	Moment Frame Story Drift-NS	Moment Frame Story Drift-EW	2.5% Story Height	OK?
1	0.038	0.039	13	0.231	0.238	0.325	YES
2	0.085	0.087	13	0.283	0.29	0.325	YES
3	0.128	0.127	13	0.259	0.239	0.325	YES

Table 3-11: Drift check table for six-story moment frame

Story	Moment Frame Displacement-NS (ft)	Moment Frame Displacement-EW (ft)	Story Height	Moment Frame Story Drift-NS	Moment Frame Story Drift-EW	2.5% Story Height	OK?
1	0.067	0.068	18	0.405	0.413	0.45	YES
2	0.115	0.12	13	0.292	0.313	0.325	YES
3	0.163	0.171	13	0.291	0.31	0.325	YES
4	0.215	0.223	13	0.314	0.309	0.325	YES
5	0.265	0.269	13	0.299	0.281	0.325	YES
6	0.303	0.308	13	0.231	0.238	0.325	YES

Fundamental period of vibration for buildings is very important in seismic engineering which is mainly decided by mass and stiffness of structures. Table 3-12 is a summary of all the model periods.

Table 3-12: Summary of the Models' Periods

Location		Model Period (s)		
		1st	2nd	3rd
3 story	LA-Brace Frame-chevron	0.519	0.197	0.129
	LA-Brace Frame-Single Diagonal	0.576	0.228	0.151
	LA Moment Frame with MF Beams as Beams	1.255	0.392	0.205
	LA Moment Frame with MF Beams as Girders	1.392	0.419	0.330
6 story	LA-Brace Frame-chevron	1.267	0.460	0.264
	LA-Brace Frame-Single Diagonal	1.097	0.395	0.244
	LA Moment Frame with MF Beams as Beams	2.106	0.876	0.470
	LA Moment Frame with MF Beams as Girders	1.935	0.782	0.430

3.4 Modeling Procedure

3.4.1 Basic Modeling Conception

The moment frame structures are modeled as 3D models for design purpose while modeled as 2D models for analysis. The reason to use 2D models instead of 3D models is to shorten the analysis time on the computer since the 3D model have much more nodes and elements to be analyzed than the 2D model. The 2D models are created in SAP 2000 first. The exterior frame of three-story and six-story structures, which is a moment frame or a brace frame, is used to represent half of the structure for each model. Figure 3-10 shows the three-story moment frame model as an example. Story height is 13 ft for each floor and bay dimension is 30 ft. Columns are all fixed at the base of the moment frame and pinned at the base of the brace frame. There is a ghost column for each model to represent the remaining gravity columns tributary to the modeled frame which are not included in the 2D model. The ghost column is not included in the lateral load resistant system. They will carry the gravity load with their tributary areas which has a p-delta effect on the structure. The ghost column is set up 30 ft away from the right exterior column which is pinned at the base for both the moment frame and brace frame. In SAP 2000, the ghost column is assigned as “other frame section property type” and “general section” by moment of inertia and cross section area which means only the geometry properties of gravity columns are used. The material of the ghost column is assigned as A992Fy50 which is same as the frame element in the lateral load resisting system. The ghost column has the same joint elevation with the other columns in the 2D model. Ghost column joints are constrained by the body constraint. In the braced frame, beams are all pinned in the ends to the columns. However, in the moment frame, most of beams are fully connected to the columns while the beams which are connected to the columns in the weak axis will be pinned to the weak axis of the columns.

Each beam has three segments which have same length (10 ft). All the nodes along the beams are restrained in the out of plane direction. The gravity loads including dead loads and live loads with the tributary area of the seismic resisting systems are distributed on the beams. The remaining gravity load which should be assigned on the gravity beams and columns is transferred to become the point load which is assigned on the joints of ghost column to represent the gravity load for each floor. Mass is very important in the dynamic analysis model. The mass lumped at the nodes along the beams will be detailed later. There is no mass assigned to the nodes of the ghost column.

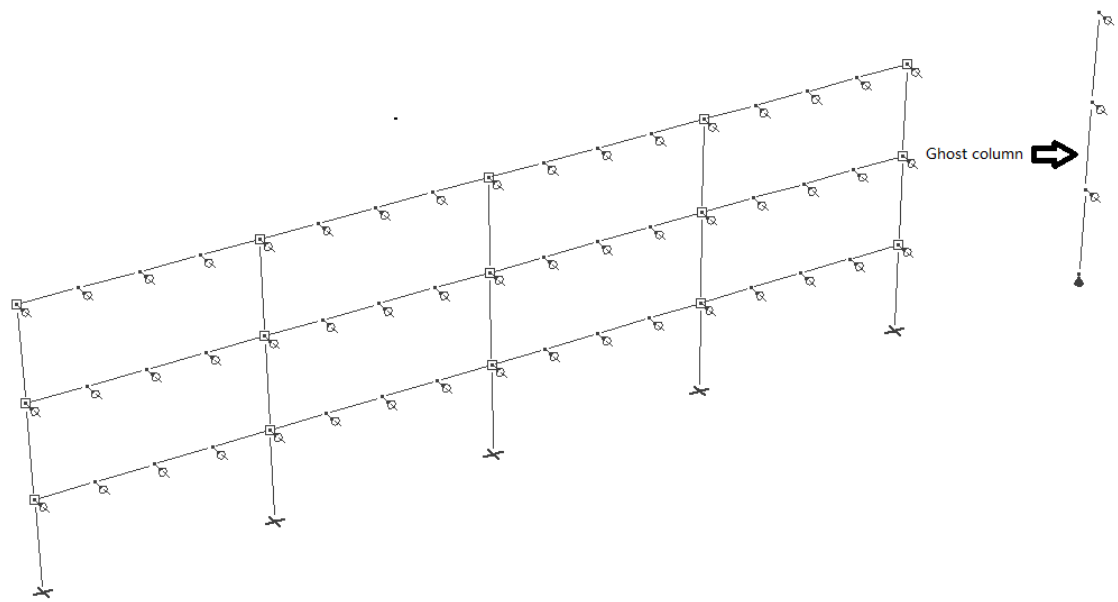

Figure 3-10: Three-story Moment Frame as Beam Analysis Model

3.4.2 Modeling procedure in Perform 3D

Analysis models are built in SAP 2000 with basic location information and properties of frame first. Then the SAP 2000 file can be exported as a Perform3D structures file and be transferred to Preform 3D V.5 (CSI, 2011), a nonlinear structural analysis program distributed by CSI to perform the nonlinear dynamic procedure. The transferred models in Perform 3D have the same geometry information which is developed in SAP 2000.

3.4.2.1 Study of mesh requirement for the girder and beam element

When doing finite element analysis of buildings including the vertical component of earthquakes, it is necessary to put nodes along the beams and girders. The assigned horizontal and vertical mass on these nodes is a very important dynamic property. A 3D model has been built by Ju et al. (2000) to investigate how to separate the girders and beams which creates less error and saves time in the analysis. They divided the main girder between two columns into one, two and three two-node beam elements named Mesh-0. Mesh-1, and Mesh-2, respectively. Figure 3-11 shows the three different types of mesh models.

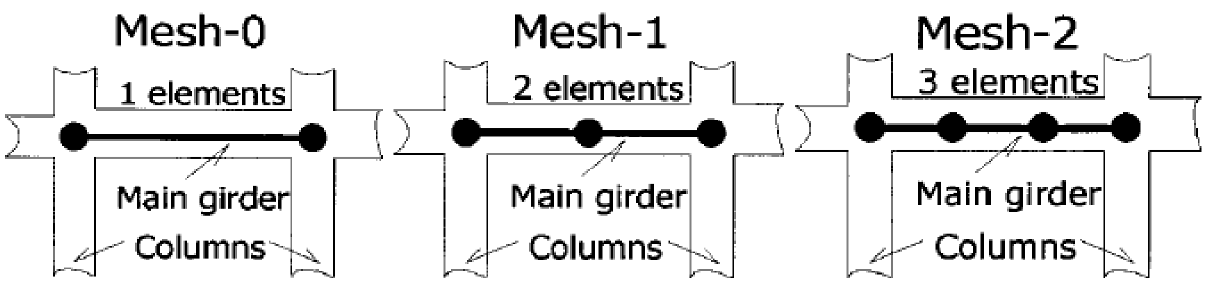

Figure 3-11: Three Different Types of Mesh Model *(Ju, Liu, & Wu, 2000)*

One hundred and eighty time history analyses including the condition of two plans, five building heights, two mass schemes, three mesh types and three vertical earthquakes were applied to find the accuracy due to mesh type. Table 3-13 shows the percentage of averaged error by using different kinds of models.

Table 3-13: Percentage of Averaged Error by Using Different Kinds of Models *(Ju, Liu, & Wu, 2000)*

Building stories (1)	Lumped Mass Mesh-0 (2)	Lumped Mass Mesh-1 (3)	Lumped Mass Mesh-2 (4)	Consistent Mass Mesh-0 (5)	Consistent Mass Mesh-1 (6)
(a) Rectangular Building					
5	22.70	5.44	1.53	16.88	1.29
10	12.73	1.24	0.60	11.72	1.07
15	7.09	0.67	0.31	10.46	0.63
20	5.02	0.56	0.19	5.95	0.42
25	4.24	0.30	0.08	5.05	0.34
(b) L-Shaped Building					
5	40.79	7.97	2.02	23.01	1.70
10	13.87	3.15	0.80	9.67	1.15
15	10.37	1.27	0.52	9.56	0.80
20	5.82	0.87	0.36	3.11	0.29
25	4.41	0.49	0.14	3.61	0.27

According to their research, lower buildings create more error while the mesh-2 method will give relatively accurate results. For this case, the mesh-2 method was chosen to avoid the unnecessary error when the models are analyzed by computer.

3.4.2.2 Mass

The lumped mass method is commonly used in dynamic structural analysis. However, the lumped mass assumptions are not always consistent with the underlying physical behavior and can therefore modify the dynamic properties of the modeled structure. There are significant errors in mass modeling, especially for vertical degrees of freedom on dynamic behavior and seismic response. Overestimation or underestimation of the lumped masses associated with vertical displacements in 2D frame models of these structures will lead to inaccurate modal periods and associated modal participation factors for dynamic response (Whalen, Archer, & Bhatia, 2004).

The mass modeling methods for the 2D models are shown in Figure 3-12 and 3-13 to represent how much horizontal and vertical mass will be put into the seismic resistance frames. For the north-south direction, exterior moment frames resist the horizontal motion. One of the frames can typically take half the mass of each floor as total horizontal mass while the tributary area of the vertical mass is much smaller than the tributary area of the horizontal mass. Figures 3-14, 3-15, 3-16, 3-17 show how to distribute the vertical and horizontal mass to the nodes along the beam and joints in the moment frame as beam models and moment frame as girder models respectively. The brace frame models are same as the moment frame as beam model. The three-story building is used as an example.

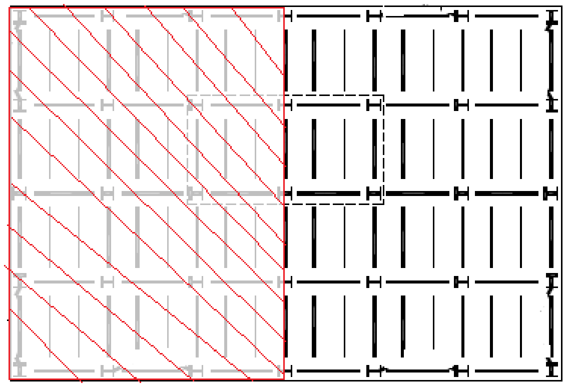

Figure 3-12: The Tributary Area for Horizontal Mass

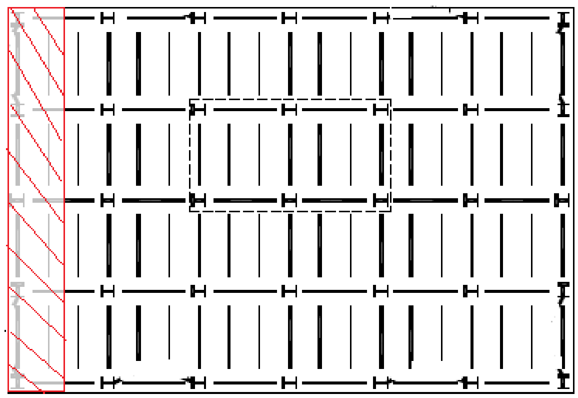

Figure 3-13: The Tributary Area for Vertical Mass (MF as Beam)

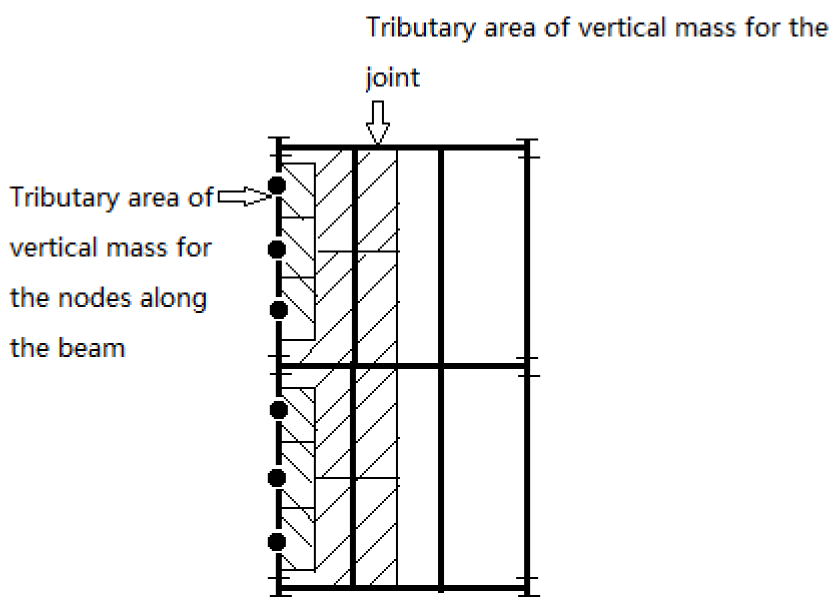

Figure 3-14: Detail of Tributary Area of Vertical Mass in the Moment Frame as Beam Model (two bays)

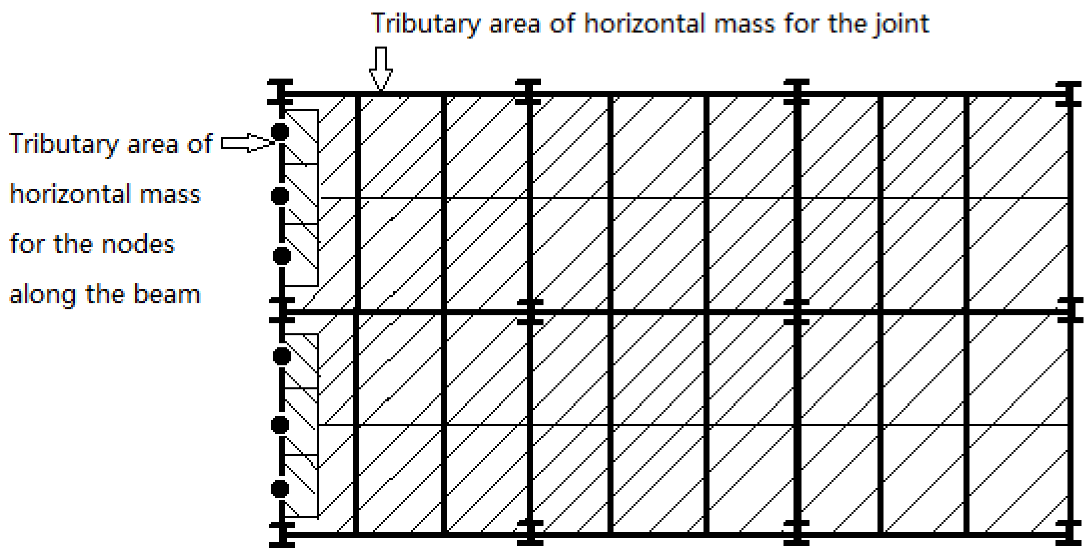

Figure 3-15: Detail of Tributary Area of Horizontal Mass in the Moment Frame as Beam Model (two bays)

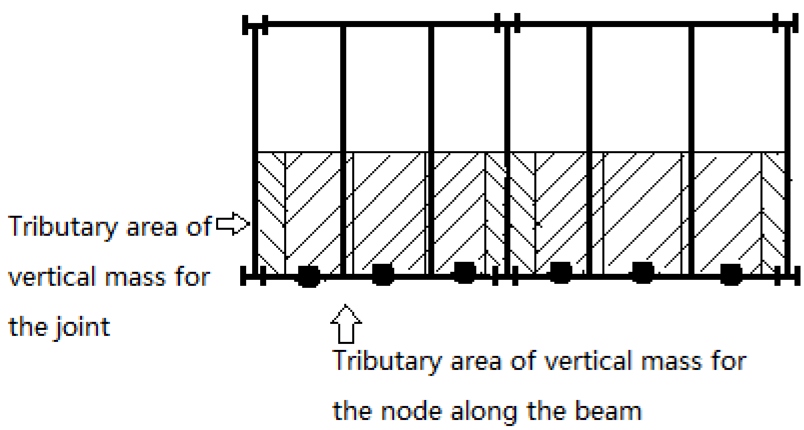

Figure 3-16: Detail of Tributary Area of Vertical Mass in the Moment Frame as Girder model (two bays)

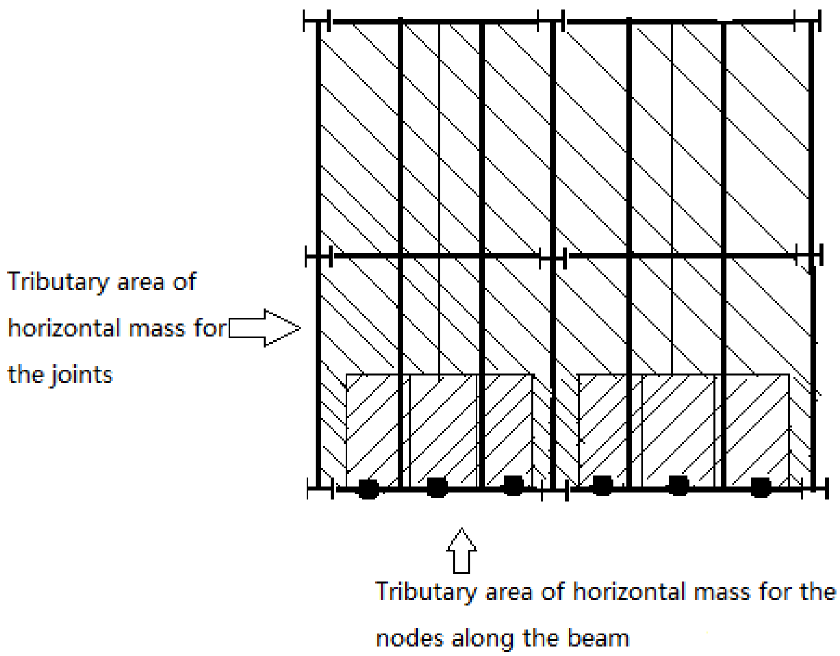

Figure 3-17: Detail of Tributary Area of Horizontal Mass in the Moment Frame as Girder Model (two bays)

3.4.2.3 Beam and Reduced Beam Section (RBS)

Beams are very important parts in the special moment resistance frames (SMRFs). The effect of gravity load is much smaller than the lateral load on the SMRF beams. Most engineers assume that the plastic hinges will occur near the face of column when the buildings are under the effect of earthquakes. However, after the Northridge earthquake, reports such as FEMA 267 (ATC, 1995) recommended moving the plastic hinges in the beam away from the column face to protect the columns and connections. There are two major methods to move the plastic hinge away from the column face. One is to increase the capacity of beam at the column face by putting cover plates on the beam flanges; the other one is to reduce the strength of beam at a distance away from column face by reducing the beam flange. In this case, the reduced beam section method was chosen. Figure 3-18 shows the sketch of the radius cut RBS geometry detail. Table 3-14 and 3-15 show the geometrical characteristics and strength properties of reduced beam section for all the moment frame models.

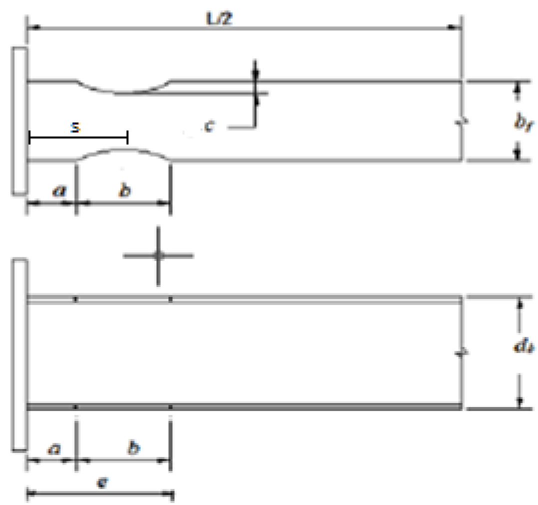

Figure 3-18: Radius Cut RBS Geometry Detail (Ajay & Gaurang, 2013)

Table 3-14: Geometrical Characteristics of Reduced Beam Section

Member	a	b	c	e	s
	in	In	in	in	in
W30X108	6.30	22.35	2.10	28.65	17.48
W27X94	6.00	20.70	2.00	26.70	16.35
W24X76	5.39	17.93	1.80	23.32	14.36
W24X62	4.22	17.78	1.41	22.00	13.11
W24X55	4.21	17.70	1.40	21.91	13.06
W18X40	3.61	13.43	1.20	17.04	10.32
W18X35	3.60	13.28	1.20	16.88	10.24

Table 3-15: Strength Properties of Reduced Beam Section

Member	Z	z	z/Z	M_z
	in	in	in	kip-in
W30X108	346.0	252.6	0.73	12630
W27X94	278.0	203.0	0.73	10150
W24X76	200.0	141.1	0.71	7055
W24X62	153.0	112.1	0.73	5605
W24X55	134.0	99.4	0.74	4970
W18X40	78.4	55.3	0.71	2765
W18X35	66.5	49.4	0.74	2470

Z= Plastic section modulus of original beam in its strong axis

z= Plastic section modulus of beam in its strong axis at minimum part of RBS

Mz= Reduced beam section capacity

The strength properties and location of the RBS is according to the simplified model for the reduced beam section. In the analytical model, the single spring model is used to represent the RBS for the beams in Perform 3D. Figure 3-19 shows the simplified model for RBS.

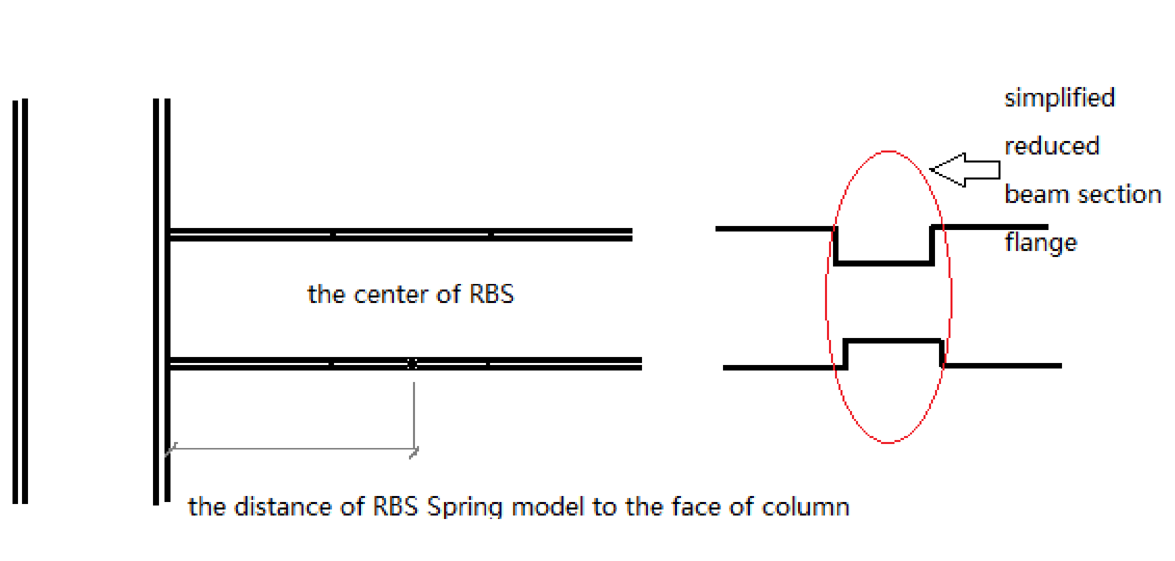

Figure 3-19: Simplified Model for RBS

The modeling part and analysis part are the two major parts in Perform 3D. In the modeling part, the cross sections do not need to be defined for the beams and girders since they are already defined in the SAP 2000 model. However, an inelastic rotation hinge still needs to be created as a single spring to represent the reduced beam section. A sketch of the beam is shown in the Figure 3-20.

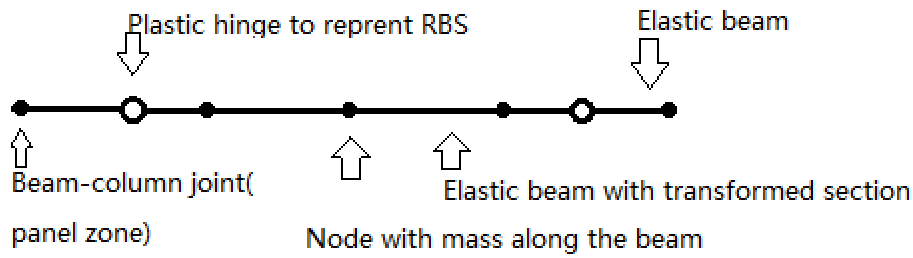

Figure 3-20: Sketch of Beam in the Moment Frame

There are eight elements in one beam compound including four elastic beam elements and two plastic hinge elements. The elastic beam is modeled as "Beam, Steel Type, Nonstandard Section" in Perform 3D with the dimensions and stiffness transferred from the SAP 2000 model. The inelastic strength and elastic strength are set up according to the steel manual (AISC, 2012). The plastic hinge (RBS) can be modeled as single-spring system. This single-spring system is defined by a hysteretic model confined within a force-displacement boundary according to FEMA 440a (ATC, 2009a). Figure 3-21 shows the generic force-displacement capacity boundary used for the single-spring system models. Table 3-16 shows the detail of the force-displacement capacity boundary for eight different single-spring models.

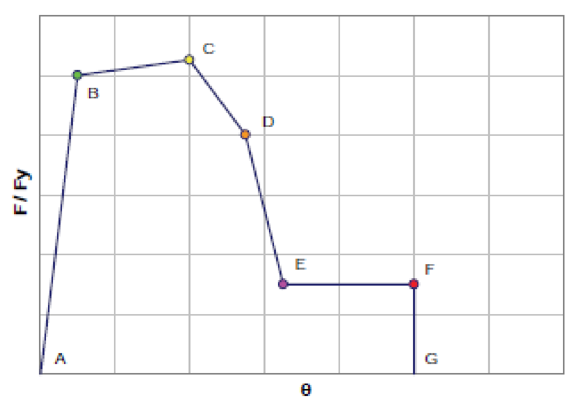

Figure 3-16: Generic Force-Displacement Capacity Boundary (ATC, 2009a)

Table 3-16: Force-Displacement Capacity Boundary Control Points for Single-Spring System (ATC, 2009a)

Prototype	Type	Quantity	Points of the force-deformation capacity boundary						
			A	B	C	D	E	F	G
Typical gravity frame	1a	F/Fy	0	0.25	1	0.55	0.55	0.55	0
		θ	0	0.005	0.025	0.04	0.07	0.07	0.07
	1b	F/Fy	0	0.25	1	0.55	0.55	0.55	0
		θ	0	0.005	0.025	0.04	0.12	0.12	0.12
Non-ductile moment frame	2a	F/Fy	0	1	0.15	0.15	0.15	0.15	0
		θ	0	0.01	0.03	0.05	0.06	0.06	0.06
	2b	F/Fy	0	1	0.15	0.15	0.15	0.15	0
		θ	0	0.01	0.05	0.055	0.06	0.06	0.06
Ductile moment frame	3a	F/Fy	0	1	1.05	0.45	0.45	0.45	0
		θ	0	0.01	0.04	0.06	0.08	0.08	0.08
	3b	F/Fy	0	1	1.05	0.8	0.8	0.8	0
		θ	0	0.01	0.04	0.06	0.08	0.08	0.08
Stiff non-ductile system	4a	F/Fy	0	1	0.3	0.3	0.3	0.3	0
		θ	0	0.004	0.02	0.06	0.08	0.08	0.08
	4b	F/Fy	0	1	0.5	0.5	0.5	0.5	0
		θ	0	0.004	0.04	0.06	0.08	0.08	0.08
Stiff, highly pinched non-ductile system	5a	F/Fy	0	0.67	1	0.6	0.067	0.067	0
		θ	0	0.002	0.005	0.028	0.04	0.06	0.06
	5b	F/Fy	0	0.67	1	0.6	0.067	0.067	0
		θ	0	0.002	0.005	0.042	0.06	0.06	0.06
Elastic-perfectly-plastic	6a	F/Fy	0	1	1	1	1	1	0
		θ	0	0.01	0.02	0.03	0.07	0.07	0.07
	6b	F/Fy	0	1	1	1	1	1	0
		θ	0	0.01	0.02	0.03	0.12	0.12	0.12
Limited-ductile moment frame	7a	F/Fy	0	1	1	0.2	0.2	0.2	0
		θ	0	0.01	0.02	0.025	0.04	0.04	0.04
	7b	F/Fy	0	1	1	0.2	0.2	0.2	0
		θ	0	0.01	0.02	0.04	0.06	0.06	0.06
Non-ductile gravity frame	8a	F/Fy	0	1	1	0	0	0	0
		θ	0	0.025	0.025	0.025	0.025	0.025	0.025
	8b	F/Fy	0	1	1	0.55	0.55	0.55	0
		θ	0	0.025	0.025	0.03	0.04	0.04	0.04

In this case, the special moment frame is a ductile moment frame so the median value is chosen between the upper limit (3a) and the lower limit (3b) to constitute the basic F-D relationship and strength loss in Perform 3D. Table 3-17 shows the value used for the plastic hinge in the model.

Table 3-17: Force-Displacement Capacity Boundary Control Points for the Model

Prototype	Quantity	Points of the force-deformation capacity boundary						
		A	B	C	D	E	F	G
Ductile Moment Frame	F/Fy	0	1	1.05	0.6	0.6	0.6	0
	ϴ	0	0.01	0.04	0.06	0.08	0.08	0.08

The plastic hinge is modeled as "Moment Hinge, Rotation Type" in Perform 3D. The force-displacement relationship and strength loss information is set based on the FEMA440a report (ATC, 2009a). Figure 3-22 shows the relationship between moment and rotation for the plastic moment hinge. The moment hinge will be active when the moment reaches the plastic moment due to the reduced beam section. For stiffness degradation, all the energy factors for all the points in the model are 0.7 which means a large amount of degradation. The beams in the lower stories of the moment frame are mainly influenced by the p-delta effect while the beams in the higher stories are mostly affected by stiffness degradation. The reason the stiffness degradation is set to a large number is to see how stiffness degradation influences the plastic hinge demands in the higher beams based on the RBS hysteretic loops.

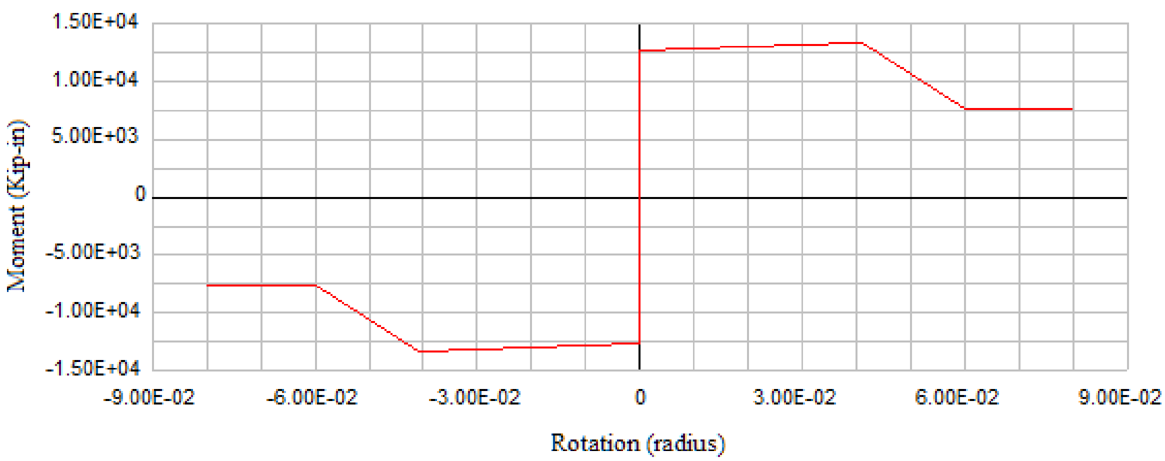

Figure 3-22: Relationship between Moment and Rotation for the Plastic Moment Hinge

After the information for the rotation type moment hinges are set up, they are placed on the beams for the SMRFs. The location of moment hinge was decided based on the location of center point of reduced beam section. Tables 3-18, 3-19, 3-20 and 3-21 show the location of moment hinges for the models. Figure 3-23 illustrates the data presented in Tables 3-18, 3-19, 3-20 and 3-21 mean.

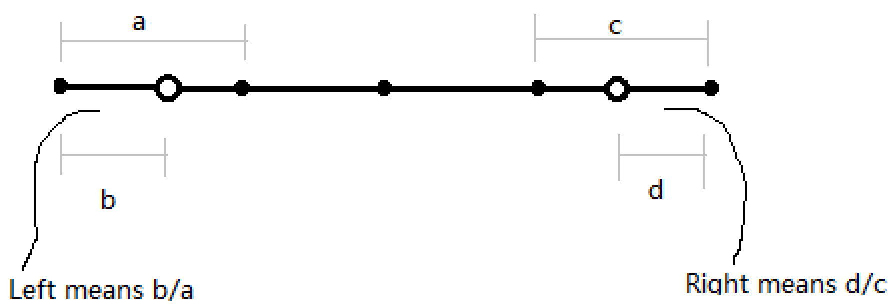

Figure 3-23: Sketch to show the Location of Plastic Hinge in the Beam

Table 3-18: Beam Plastic Hinge Location for Three-Story MF as Beam Model

story		Exterior Beam	Interior Beam
1	Left	0.268	0.268
	Right	0.268	0.268
2	Left	0.268	0.268
	Right	0.268	0.268
3	Left	0.200	0.200
	Right	0.200	0.200

Table 3-19: Beam Plastic Hinge Location for Three-Story MF as Girder Model

story		Left Exterior Beam	Interior Beam	Right Exterior Beam
1	Left	-	0.264	0.264
	Right	0.264	0.264	-
2	Left	-	0.264	0.264
	Right	0.264	0.264	-
3	Left	-	0.200	0.200
	Right	0.200	0.200	-

Table 3-20: Beam Plastic Hinge Location for Six-Story MF as Beam Model

story		Left Exterior Beam	Interior Beam	Right Exterior Beam
1	Left	0.284	0.284	0.284
	Right	0.284	0.284	-
2	Left	0.284	0.284	0.284
	Right	0.284	0.284	-
3	Left	0.272	0.272	0.272
	Right	0.272	0.272	-
4	Left	0.244	0.244	0.244
	Right	0.244	0.244	-
5	Left	0.232	0.232	0.232
	Right	0.232	0.232	-
6	Left	0.200	0.200	0.200
	Right	0.200	0.200	-

Table 3-21: Beam Plastic Hinge Location for Six-Story MF as Girder Model

story		Left Exterior Beam	Interior Beam	Right Exterior Beam
1	Left	0.288	0.288	0.288
	Right	0.288	0.288	-
2	Left	0.288	0.288	0.288
	Right	0.288	0.288	-
3	Left	0.276	0.276	0.276
	Right	0.276	0.276	-
4	Left	0.268	0.268	0.268
	Right	0.268	0.268	-
5	Left	0.244	0.244	0.244
	Right	0.244	0.244	-
6	Left	0.200	0.200	0.200
	Right	0.200	0.200	-

Each beam in the braced frame has four elastic beam elements. All of them are assigned as “Beam, Steel Type, Nonstandard Section” and the two joints connected to the column have the moment releases. Figure 3-24 shows the sketch for the beam in the brace frame.

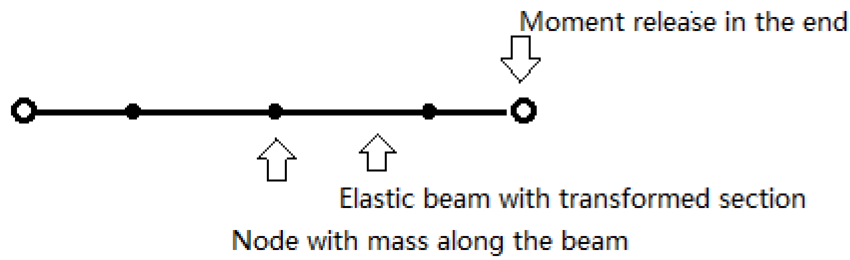

Figure 3-24: Sketch of the Beam in the Braced Frame

3.4.2.4 Columns in the lateral load resisting system

The modeling of columns in the lateral load resisting system is very similar to the beams. Figure 3-25 shows a sketch of a column in the moment frame. There are five elements in one column compound including three elastic beam elements and two plastic hinge elements. The elastic column is modeled as "Column, Steel Type, Nonstandard Section" in Perform 3D. The dimensions and stiffness are transferred from SAP 2000 model. The inelastic strength and elastic strength are set up according to the steel manual (AISC, 2012).

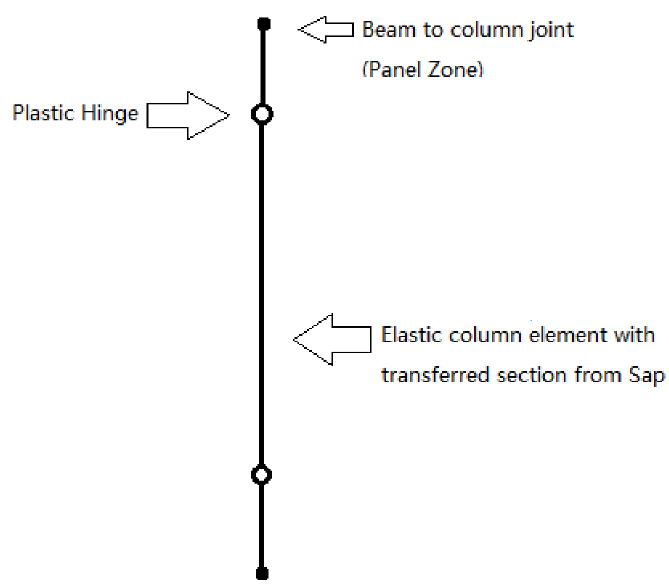

Figure 3-25: Sketch of Column in the Moment Frame

Inelasticity in the column is modeled in case it occurs. Due to the "weak beam strong column" requirement and panel zone doubler plate design, significant yielding is not expected. Once a hinge forms, collapse of the structure will be imminent. The column hinges are modeled as "P-M2-M3 Hinge, Steel Rotation Type". The basic relationship between moment and rotation is described as an elastic perfectly plastic model. The P-M2-M3 Hinge will be active when the demand in the column touches the interaction diagram (yield surface). In addition, the P-M2-M3 Hinge in Perform 3D requires the information to define a yield surface. In this part, the default value is used to set up the yield surface directly. Figure 3-26 shows the E-P-P relationship for the P-M2-M3 Hinge in the column. The blue one (higher one) is for the column oriented in the strong axis while the red one (lower one) is for the column oriented in the weak axis. Figure 3-27 shows the yield surface for the P-M2-M3 Hinge in the column.

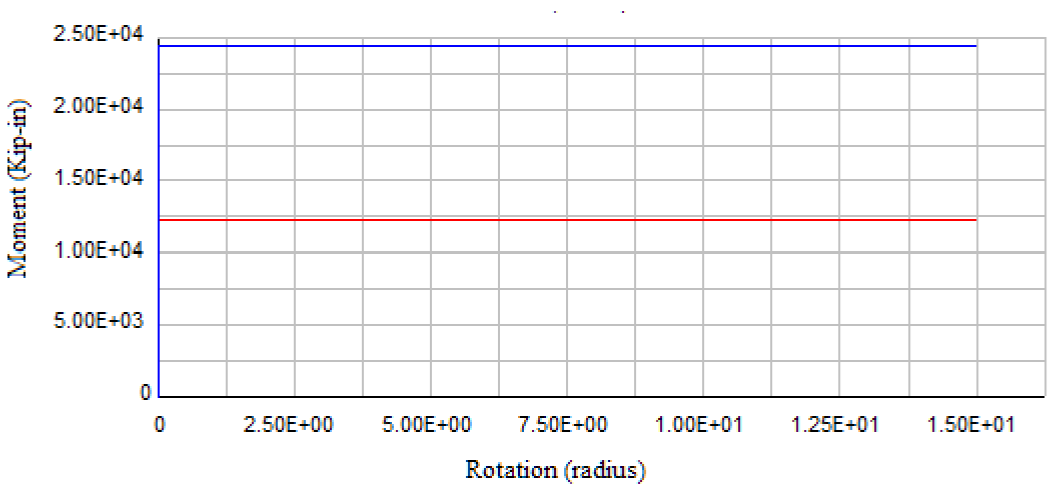

Figure 3-26: E-P-P relationship for the P-M2-M3 Hinge in the column

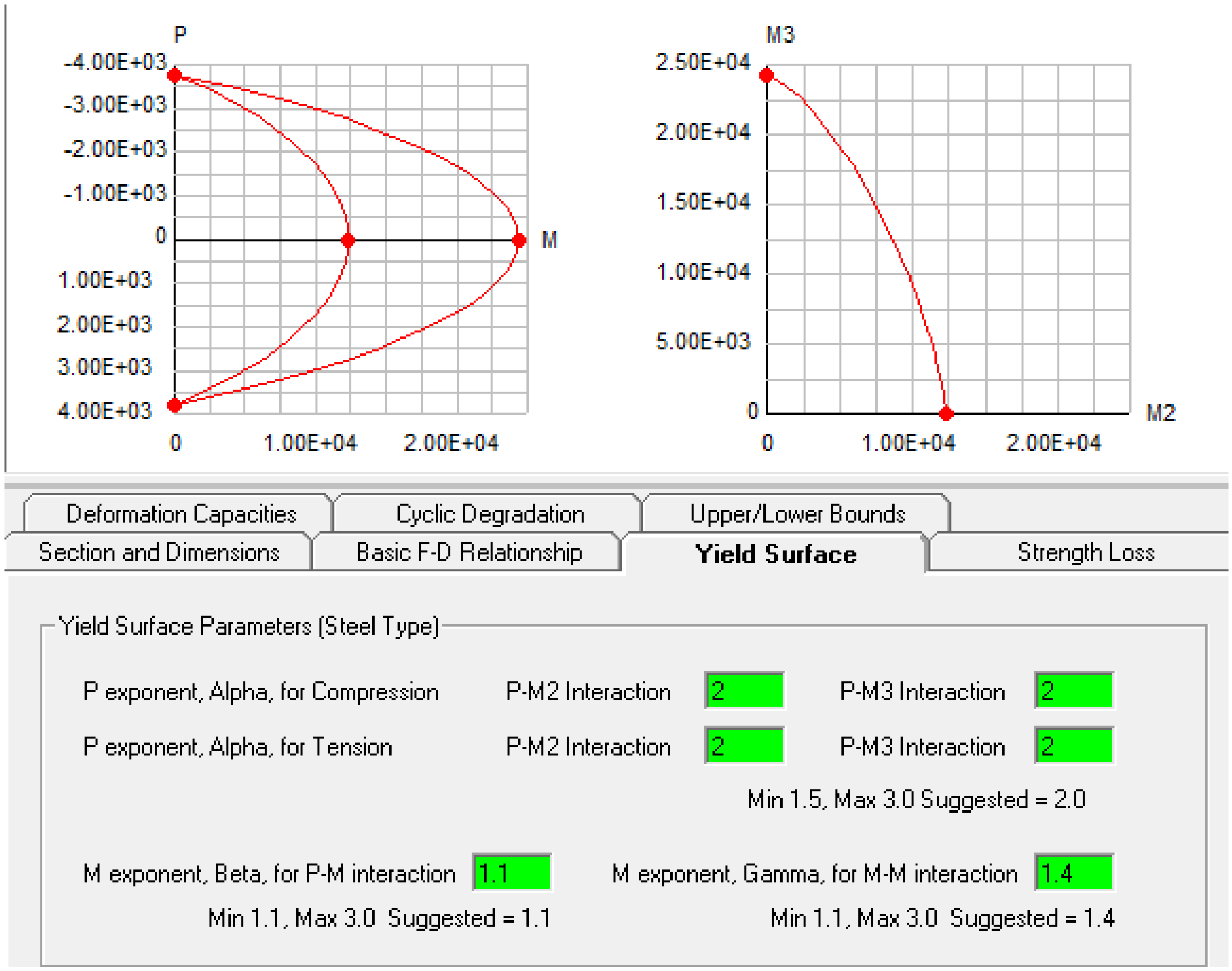

Figure 3-27: Yield Surface for the P-M2-M3 Hinge in the Column

The column hinges are assumed to be at the base and beam flange. Tables 3-22, 3-23, 3-24 and 3-25 show the location of moment hinges for the models. Figure 3-28 illustrates the data presented in Tables 3-22, 3-23, 3-24 and 3-25.

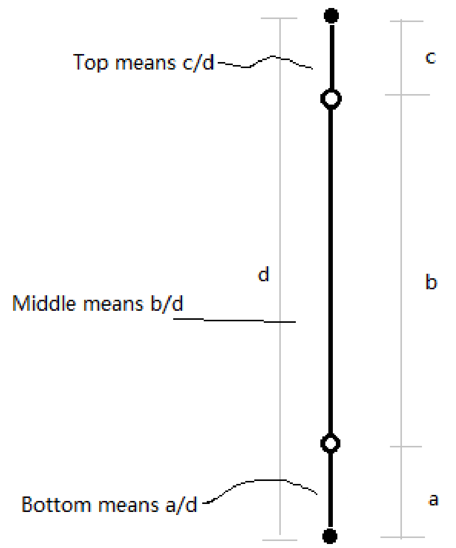

Figure 3-28: Sketch to show the Location of Plastic Hinge in the Column

Table 3-22: Column Plastic Hinge Location for Three-Story MF as Beam Model

Story		Corner Column	Edge Column
1	Top	0.088	0.088
	Middle	0.862	0.862
	Bottom	0.050	0.050
2	Top	0.088	0.088
	Middle	0.823	0.823
	Bottom	0.088	0.088
3	Top	0.057	0.057
	Middle	0.854	0.854
	Bottom	0.088	0.088

Table 3-23: Column Plastic Hinge Location for Three-Story MF as Girder Model

Story		Corner Column	Edge Column
1	Top	0.076	0.076
	Middle	0.875	0.876
	Bottom	0.050	0.048
2	Top	0.076	0.076
	Middle	0.849	0.849
	Bottom	0.076	0.076
3	Top	0.057	0.057
	Middle	0.868	0.868
	Bottom	0.076	0.076

Table 3-24: Column Plastic Hinge Location for Six-Story MF as Beam Model

Story		Corner Column	Edge Column
1	Top	0.069	0.069
	Middle	0.893	0.893
	Bottom	0.038	0.038
2	Top	0.096	0.096
	Middle	0.809	0.809
	Bottom	0.096	0.096
3	Top	0.088	0.088
	Middle	0.816	0.816
	Bottom	0.096	0.096
4	Top	0.111	0.111
	Middle 1	0.889	0.889
	Middle 2	0.713	0.713
	Bottom	0.288	0.288
5	Top	0.076	0.076
	Middle	0.847	0.847
	Bottom	0.077	0.077
6	Top	0.057	0.057
	Middle	0.867	0.867
	Bottom	0.076	0.076

Table 3-25: Column Plastic Hinge Location for Six-Story MF as Girder Model

Story		Corner Column	Edge Column
1	Top	0.069	0.069
	Middle	0.893	0.891
	Bottom	0.038	0.040
2	Top	0.096	0.096
	Middle	0.809	0.809
	Bottom	0.096	0.096
3	Top	0.088	0.088
	Middle	0.816	0.816
	Bottom	0.096	0.096
4	Top	0.128	0.128
	Middle 1	0.872	0.872
	Middle 2	0.713	0.713
	Bottom	0.288	0.288
5	Top	0.077	0.077
	Middle	0.835	0.835
	Bottom	0.088	0.088
6	Top	0.057	0.057
	Middle	0.866	0.866
	Bottom	0.077	0.077

Each column in the braced frame has only one elastic column element. All columns are assigned as “Column, Steel Type, Nonstandard Section”. The columns are connected continuously with each other and pinned at the base.

3.4.2.5 Strong Column-Weak Beam

When designing SMRFs, P-delta instability must be controlled. It is better to have a relatively uniform distribution of lateral drift over the structure’s height. It is very important to avoid plastic hinging at the tops and bottoms of columns within a single story. When these plastic hinges happen in a single story, more and more inelastic deformation will happen in the columns which will cause significant drift and a very large P-delta effects. Most building codes require designs to create hinges in the beam rather than the columns. These requirements are the reason why the “strong-column weak-beam” design is required. AISC 341 (2012), chapter 9 gives a “strong column weak beam” design procedure where the ratio of the sum of column flexural strength divided by the sum of beam flexure strength at each joint should be larger than 1.0. When calculating the available flexure strength in columns, it’s very important to consider the effect of the axial force in the column. When the ratio for “strong column weak-beam” is 2 or greater, AISC 341, chapter 9 permits an assumption that columns will remain elastic. However, according to recent research, it is found that the “strong column weak-beam” provision may not be enough to avoid the plastic hinge to occur in the column under some strong ground motion including horizontal and vertical component (Gupta & Krawinler, 1999) . Table 3-26 shows the ratio for “strong-column weak-beam” design at each joint and the design detail is attached in appendix A. Joints 1, 2 and 3 represent the joints along the corner column while joints 4, 5 and 6 represent the joints along the edge column from first story to third story in the three-story structure. Similarly, joints 1, 2, 3, 4, 5 and 6 represent the joints along the corner column while joints 7, 8, 9, 10, 11 and 12 represent the joints along the edge column from first story to sixth story in six-story structure.

Table 3-26: "Strong-Column Weak-Beam" Ratio at each Joint

Ratio	3 Story NS	3 Story EW	6 Story NS	6 Story EW
Joint 1	1.5	-	1.5	1.6
Joint 2	1.5	-	1.6	1.7
Joint 3	2.7	-	2.0	2.1
Joint 4	1.5	2.1	1.9	1.3
Joint 5	1.5	2.2	2.4	1.9
Joint 6	2.7	2.2	2.4	2.4
Joint 7	-	-	1.3	1.7
Joint 8	-	-	1.4	1.8
Joint 9	-	-	1.8	2.3
Joint 10	-	-	1.5	1.5
Joint 11	-	-	2.0	2.1
Joint 12	-	-	2.0	2.8

3.4.2.6 Panel Zone

The effect of panel zone deformations on the flexibility of steel moment resisting frames is important for both elastic and inelastic response. Panel zones experience large shear force because of the transfer of moment from beam flanges. The panel zone will start to yield when the shear force increases. Then the yielding will be transferred to the corner point. Figure 3-29 shows the behavior of the panel zone. The doubler plates in the panel zone are designed in the model. Table 3-27 shows the thickness of the doubler plates in the panel zone, where required, at each joint.

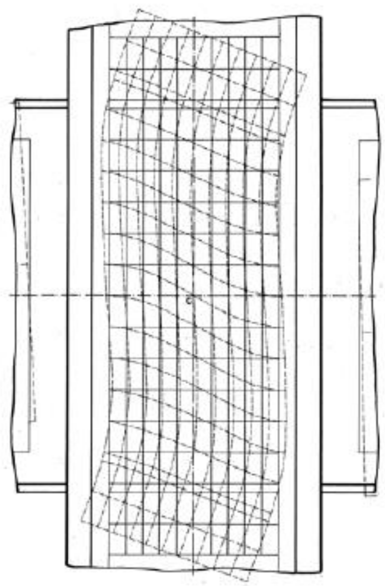

Figure 3-29: Panel zone behavior

Table 3-27: Thickness of Doubler Plates for Panel Zone at each Joint

Column Web Doubler Plate Thickness(in)	3 story NS	3 story EW	6 story NS	6 story EW
Joint 1	-	-	-	-
Joint 2	-	-	-	-
Joint 3	-	-	-	-
Joint 4	0.75	0.25	-	-
Joint 5	0.75	0.25	-	-
Joint 6	-	-	-	-
Joint 7	-	-	0.75	0.25
Joint 8	-	-	0.5	0.25
Joint 9	-	-	0.25	-
Joint 10	-	-	0.75	0.75
Joint 11	-	-	0.5	0.5
Joint 12	-	-	-	-

There are two models that can be used to model panel zone behavior. One is the Krawinkler model (Krawinkler, 1978) and the other one is the scissors model. The Krawinkler model was chosen to represent the panel zones at each joint. Figure 3-30 shows a sketch of the Krawinkler model for the panel zone. The model has four rigid links which are connected by four rotational springs at the corners. The lower left and upper right springs have no stiffness in them which means they just act like a real hinges. The upper left spring is used to represent the shear resistance in the panel zone while the lower right spring is used to represent the column flange bending resistance. Figure 3-31 shows the relationship between the moment and the shear strain in the panel zone.

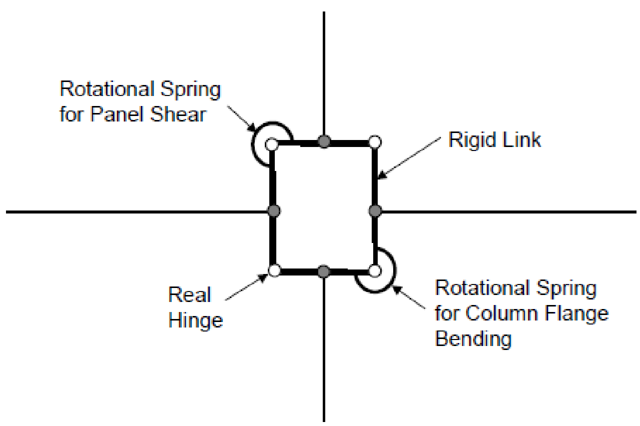

Figure 3-30: The Krawinkler model (Charney & Downs, 2004)

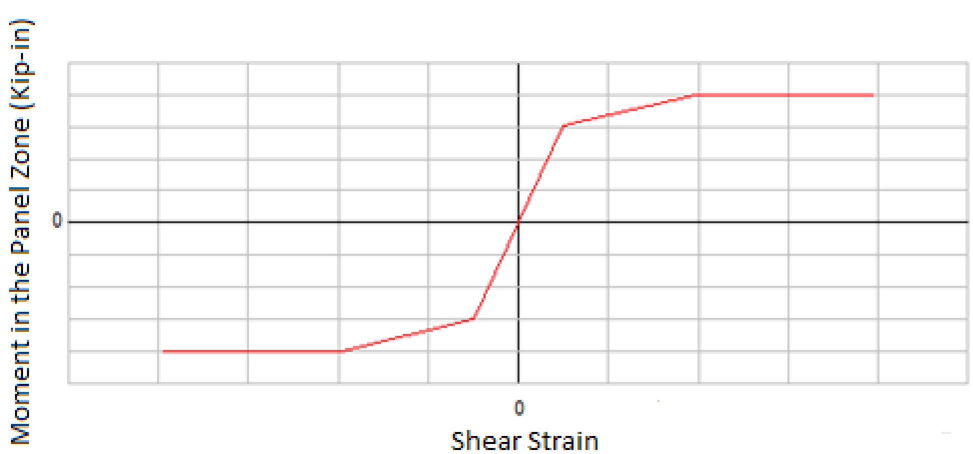

Figure 3-31: Relationship between Moment and Shear Strain in the Panel Zone

3.4.2.7 Floor

Slabs are usually assumed to be rigid in-plane. However, according to Ju et al. (2000), the influence of floor stiffness is not negligible when structures experience vertical earthquakes. This is especially true for low rise buildings with thick slabs. Figure 3-32 shows the error of building analysis caused from variation of floor thickness.

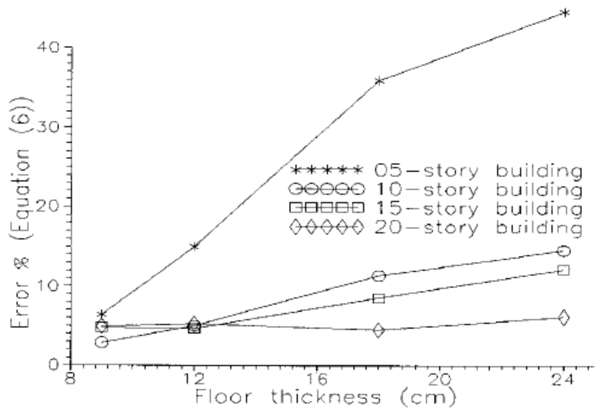

Figure 3-32: Building Analysis Error Caused by Variation of Floor Thickness

Because a 2D model is used for analysis, the slab is not modeled directly. The steel beam and the concrete slab can be assumed to be composite along the length of the elastic portion of the beam. The transformed method is used to represent the composite section. The concrete slab part is transferred into steel beam to increase the width of the steel beam's top flange to replace the original two beam elements in the middle. Table 3-28 shows the dimension of W shape composite beam in the model. Figure 3-33 illustrates the data presented in Table 3-28.

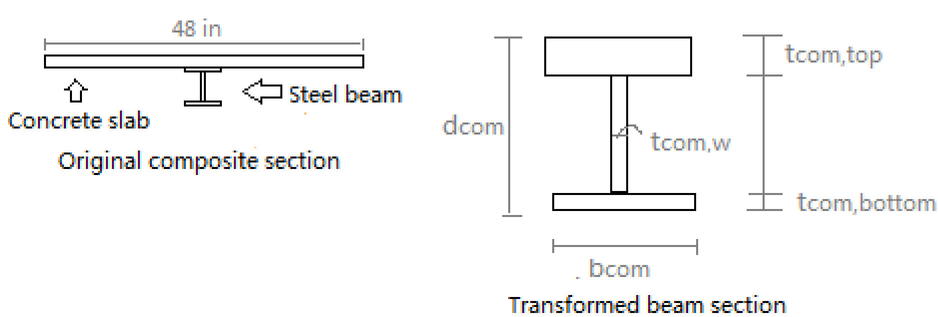

Figure 3-33: Sketch of Original Composite Section and Transformed Beam Section

Table 3-28: The Dimensions of W-Shape Composite Beam

Section	n	L_{flo}	t_{flo}	$E_{concrete}$	E_{steel}	d	tw	bf	tf	$y_{com,fl}$	$t_{com,top}$	$t_{com,bot}$	b_{com}	$t_{com,w}$	d_{com}
		in	in	ksi	ksi	in	in	in	in	in	in	in	in	in	in
W27X94	8.04	48	3	3605	29000	27.6	0.49	10	0.745	0.950	2.54	0.745	10	0.49	29.82
W18X40	8.04	48	3	3605	29000	17.9	0.315	6.02	0.525	1.236	3.50	0.525	6.02	0.315	20.88
W24X55	8.04	48	3	3605	29000	23.6	0.395	7.01	0.505	1.211	3.06	0.505	7.01	0.395	26.34
W18X35	8.04	48	3	3605	29000	17.7	0.3	6	0.425	1.286	3.41	0.425	6	0.3	20.69
W24X76	8.04	48	3	3605	29000	23.9	0.44	8.99	0.68	1.032	2.67	0.68	8.99	0.44	26.27
W24X62	8.04	48	3	3605	29000	23.7	0.43	7.04	0.59	1.162	3.13	0.59	7.04	0.43	26.43
W30X108	8.04	48	3	3605	29000	29.8	0.545	10.5	0.76	0.920	2.46	0.76	10.5	0.545	31.95

3.4.2.8 Base Fixity

Base restraint has a significant effect on steel moment frame behavior under earthquake loads. The assumption that the column bases are pinned will overestimate the column flexibility while the assumption that the column bases are fixed will underestimate the column flexibility. This can have a large effect on the first story drift. It is reasonable to assume all the columns in the 2-D moment frame models are fixed at base, because all of the columns in the lateral load resisting system are designed as fixed at base. What's more, the remaining gravity columns in the half of building which are designed as pinned at base provide some restraint to the 2-D moment model since most column bases and foundations will provide some restraint against the rotation. All the columns in the 2-D braced frame are assumed to be pinned at the base which is conservative.

3.4.2.9 BRBs

All the BRBs in the models are the same as those modeled by Xie (2015). There are two basic components, one "Buckling Restrained Brace" component and one "Elastic Bar" component in the "BRB Compound component". The transition part of BRBs is represented by "Elastic Bar" component while the steel yielding in the BRBs is represented by "Buckling Restrained Brace".

3.4.2.10 Load

Dead load, live load and roof live load are assigned in the Load Patterns section according to the calculation based on the tributary area of the frame. All the loads on the frame are distributed loads which are assigned as element loads in Perform 3D. The dead load, live load and roof live load in the ghost column are point loads which are assigned as nodal loads in Perform 3D. A limit state of 10% maximum story drift is applied in the "limit states" section under the modeling phase to make sure the structure won't deflect too much under the earthquake loads.

Load cases are defined in the "set up load cases" section under the analysis phase. The gravity load case is defined as Dead Load + 0.5 Live Load +0.5 Roof Live Load. Dynamic earthquake load cases including the horizontal earthquake and vertical earthquake are defined according to the 20 horizontal earthquake records and 16 vertical earthquake records which are amplified by the different acceleration scale factors. These earthquake records are selected based on magnitude, vertical to horizontal ratio (V/H), site class, and distance from source. The V/H value of the ground motions in the study is between 0.512 and 1.254 which means most of them are higher than 2/3. The V/H value of group one earthquakes is between 0.5 and 0.6. The V/H value of group two earthquakes is between 0.7 and 0.8. The V/H value of group three earthquakes is between 0.9 and 1. The V/H value of group four earthquakes is greater than 1. These selected earthquake records also include the situation of far away from the epicenter (Far Field) and near the epicenter (Near Field). There are two scale factors for each horizontal earthquake record. One is used by the three-story structures and the other one is used by six-story structures. The scale factors for vertical earthquake records are the same as the related horizontal earthquake records. All the earthquake records are scaled to approach the design hazard curve with 2% chance of exceedance in 50 years. These earthquake records are applied to the models in Perform 3D to do the nonlinear dynamic history analysis. Table 3-29 shows the basic information on the selected earthquake records (ATC, 2009b) and the scale factors for the three-story and six-story structures. Figure 3-34 shows the scaled FF01 load case in Perform 3D for the six-story SMRF model.

Table 3-29: Basic Information and Scale Factors of the Selected Earthquake Records

Number	Name	Earthquake	Station	Group	V-H Ratio	Time step(s)	LA3 scale factor	LA6 scale factor
1	FF01-1	Northridge	Beverly Hills	2	0.773	0.01	2.56	3.1
2	FF13-1	Loma Prieta	Capitola	3	0.92	0.005	2.65	2.8
3	FF14-1	Loma Prieta	Gilroy Array #3	1	0.538	0.005	2.8	2.92
4	FF14-2	Loma Prieta	Gilroy Array #3	2	0.791	0.005	2.79	4.81
5	FF15-2	Manjil	Iran Transverse Comp	3	0.944	0.02	2.46	2.79
6	FF19-1	Chi-Chi	CHY101	1	0.576	0.005	1.94	2.3
7	FF21-2	San Fernando	USGS Station 135	3	0.936	0.01	4.36	5.8
8	FF22-1	Friuli	Tolmezzo	1	0.585	0.005	4	5
9	FF22-2	Friuli	Tolmezzo	2	0.724	0.005	4.56	5.2
10	NF02-2	Imperial Valley	USGS Station 5028	4	1.254	0.005	1.45	1.53
11	NF05-1	Loma Prieta	CDMG Station58065	4	1.151	0.005	3.21	2.99
12	NF05-2	Loma Prieta	CDMG Station58065	4	1.174	0.005	3.27	3.45
13	NF16-1	Imperial Valley	USGS Station 5054	2	0.712	0.005	2.6	3.28
14	NF16-2	Imperial Valley	USGS Station 5054	1	0.512	0.005	2.01	2.6
15	NF17-2	Imperial Valley	UNAM/UCSD Station 6621	2	0.752	0.01	4.64	4.8
16	NF21-2	Loma Prieta	CDMG Station 57007	3	0.978	0.005	3.15	4
17	NF22-1	Cape Mendocino	CDMG Station 89005	1	0.537	0.02	1.37	1.8
18	NF25-2	Kocaeli	Yarimca	4	1.147	0.005	1.98	2.17
19	NF27-2	Chi-Chi	TCU084	3	0.898	0.005	2.22	2.66
20	NF28-1	Denali	PS10	4	1.232	0.005	2.29	2.65

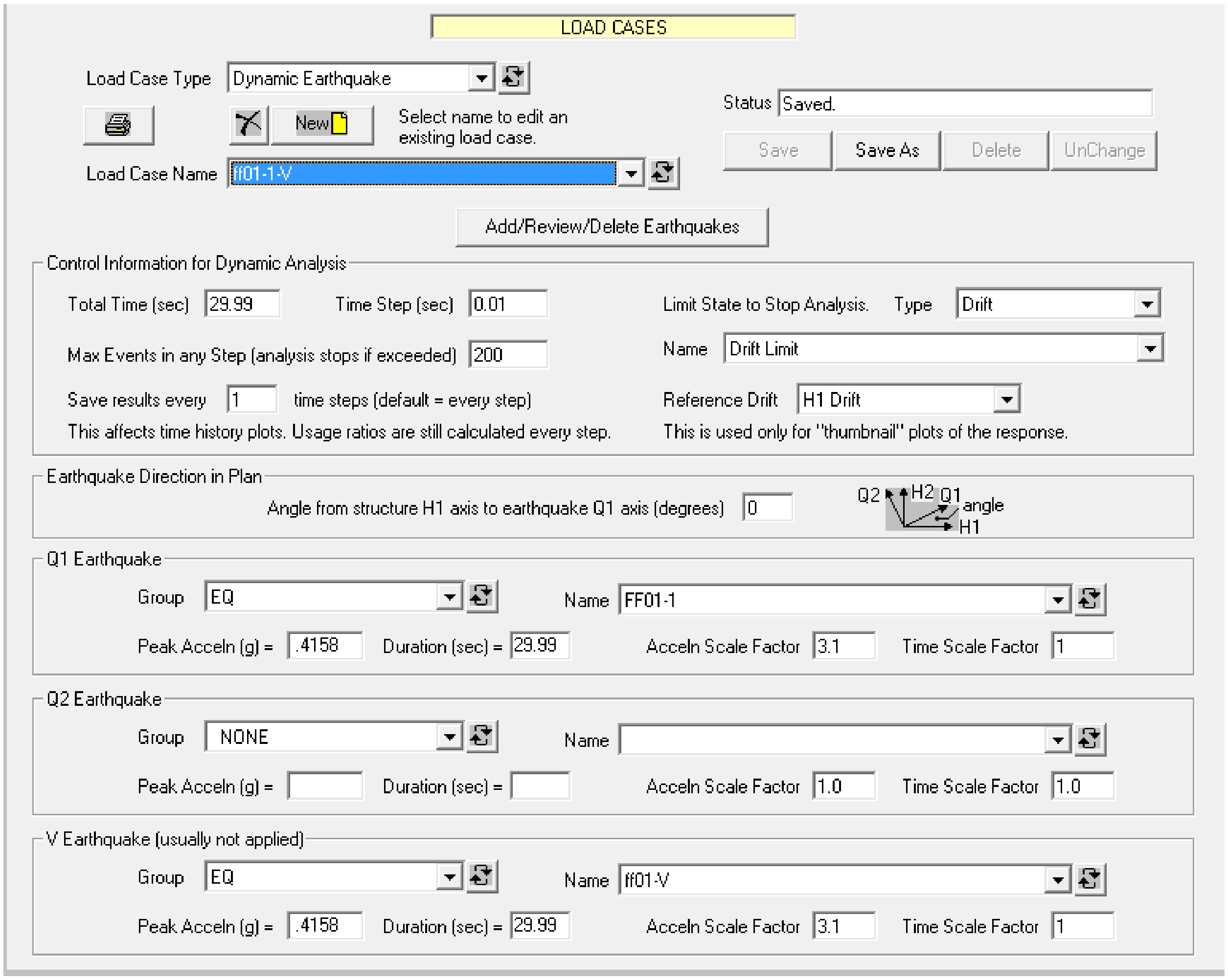

Figure 3-34: Scaled FF01 Load Case in Perform 3D for Six-Story SMRF Model

Analysis series are set up in the "Run Analysis" section. Two series were created for each model including gravity load + horizontal dynamic earthquake load (G+H) and gravity load + horizontal dynamic earthquake load +vertical dynamic earthquake load (G+H+V). P-delta effects are included for each analysis. Modal damping is set for all models at 2%. Rayleigh damping is modeled as small as possible to make sure the inelastic deformation of RBS and BRBs in the model dissipate most of the energy in the earthquakes. Figure 3-33 shows the detail of Rayleigh damping in the model in the Perform 3D. In the analysis list, each analysis has a gravity load case and a dynamic earthquake load case.

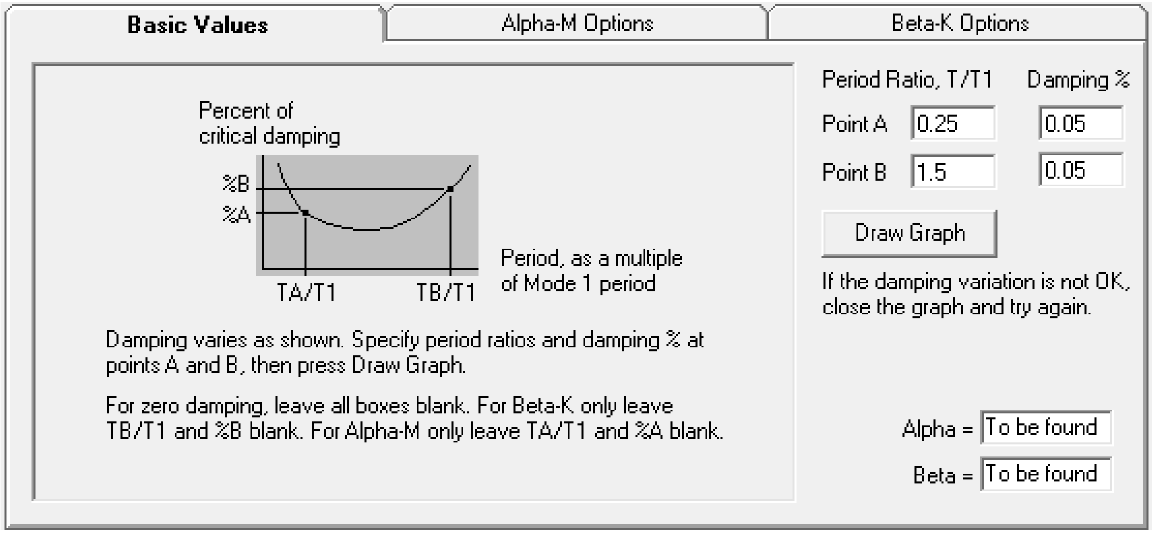

Figure 3-35: Detail of Rayleigh Damping for All the Models

3.5 Summary

In this chapter, the modeling of the special moment frame and the BRB frames is presented. The basic information for the three-story and the six-story buildings are provided first. Then a seismic design procedure for three-story and six-story moment frame by ELF method is discussed in detail in this chapter. The design procedure is completed by using the AISC Seismic Provisions for Structural Steel Buildings (AISC, 2005) and ASCE Standard 7-10 (ASCE, 2010) along with the computer program SAP2000 (CSI, 2011). In addition, this chapter discussed the finite element modeling procedure for the mesh method of beam, mass, beam, RBS, column, panel zone, floor, brace and base by using Perform 3D (CSI, 2011). The gravity load case and dynamic earthquake load case are also discussed in this chapter.

Chapter 4 Results and Discussion

4.1 Introduction

Structural elements of buildings experience inelastic behavior under severe ground motions. Four three-story structures and four six-story structures were discussed in chapter 3 and analyzed in Perform 3D. The results from Perform 3D models can be exported in text files. Story drift, story residual drift, column axial force, rotation in the RBS, BRB axial deformation, energy absorbed by RBS and BRB, midspan beam deflection, midspan beam vertical acceleration, roof acceleration and total energy absorbed by the structures were collected. An overall comparison between two different loading cases will be completed in this chapter according to the average value, median value and difference of the data exported from Perform 3D. Special cases will also be presented in this chapter to help understand the impact of vertical ground motion on the seismic response of steel frame structures.

4.2 Modal Analysis

In earthquake engineering, modal analysis uses mass and stiffness to find the natural frequency of structures. The natural frequency is very important in seismic analysis. If the natural frequency of structure matches an earthquake's frequency, the structure may continue to resonate and experience structural damage. A structural model with “n” degrees of freedom has “n” natural frequencies of free vibration and “n” mode shapes associated with the natural frequencies. This suite of mode shapes is the basis of the displacement vector in that any displaced shape of structure can be made up of a combination of these linearly independent mode shapes. Most of the time, structural engineers can only concern themselves with a small number of these modes because more than 99% of horizontal effective mass will be included in the first few modes when only the impact of horizontal ground motion is considered. However, in this study, vertical ground motions were included in the analysis, so the vertical modes which include effective vertical mass are very important. Figures 4-1 and 4-2 show the first horizontal mode shape which includes 83% effective horizontal mass and the first vertical mode shape which includes 25% effective vertical mass of the six-story moment frame as beam model, respectively.

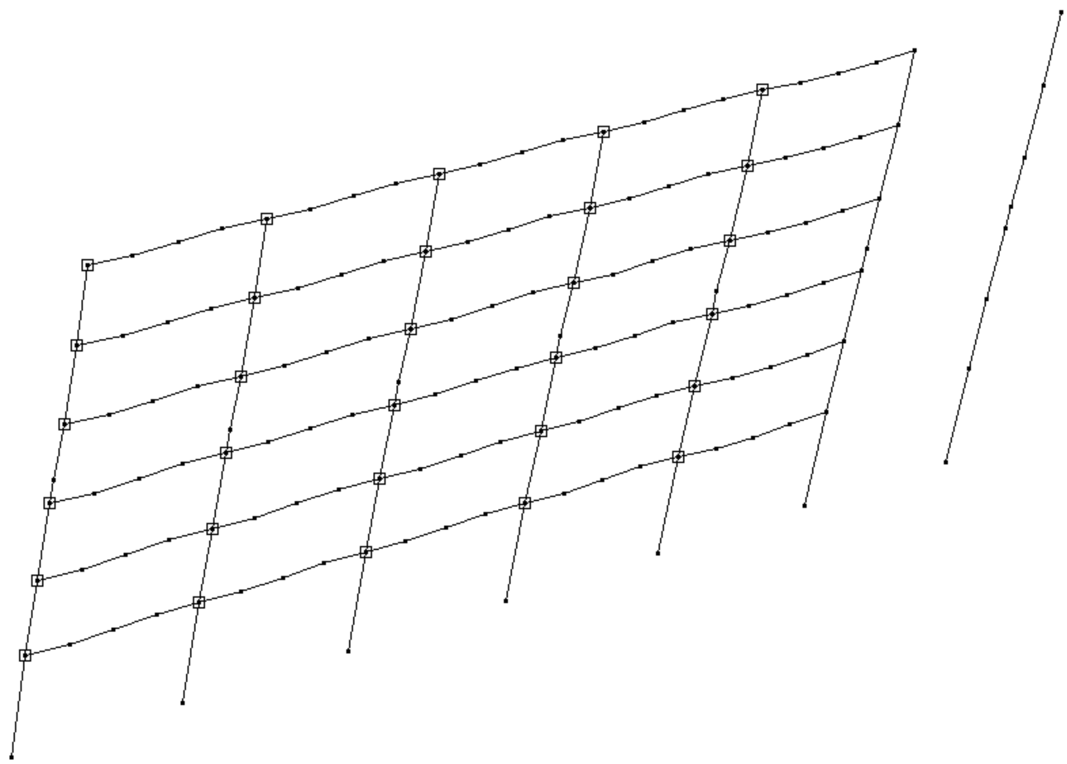

Figure 4-1: First Horizontal Mode Shape of Six-Story Moment Frame as Beam Model

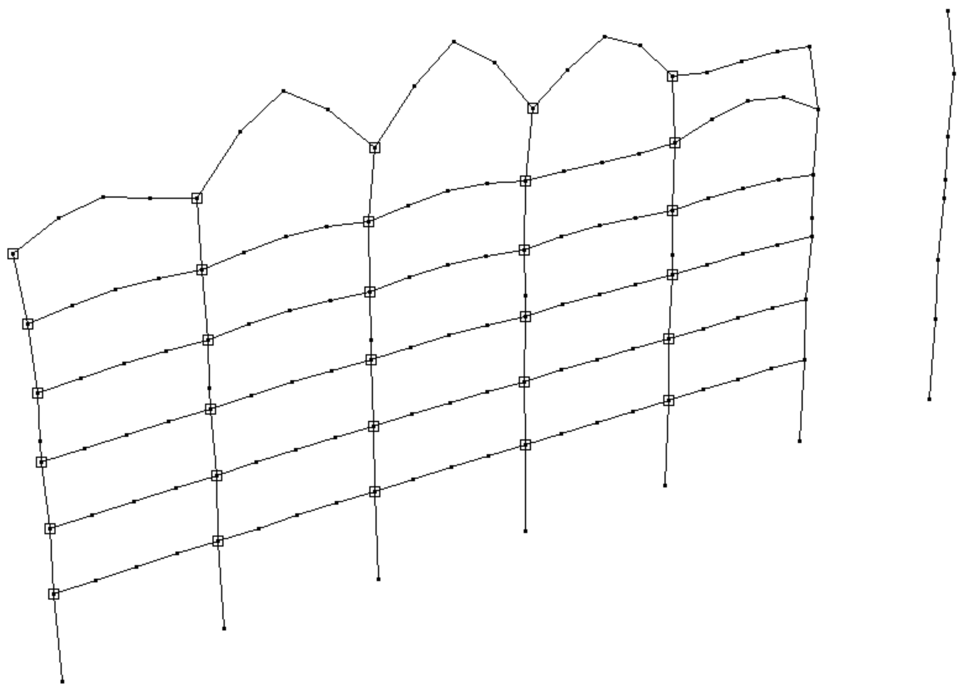

Figure 4-2: First Vertical Mode Shape of Six-Story Moment Frame as Beam Model

The modal analysis method is only applicable to the linear elastic structures. In this study, nonlinear dynamic analysis was used to do the seismic analysis. However, the horizontal and vertical mode shape can show the trend of impact of horizontal and vertical ground motion on the steel frame structures.

4.2 Story Drift and Story Residual Drift

Lateral displacement in seismic analysis is the movement of a structure under strong ground motion. Story drift is defined as the difference in lateral displacement between two adjacent stories divided by the story height. During a strong ground motion, large lateral forces and lateral drifts can be imposed on structures. What's more, if the drift of the structure becomes too large, P-delta effects can cause instability of the structure and potentially collapse.

The data of max drift for all the stories during the earthquake is calculated according to the nodal displacements for each floor. Residual drift measures the remaining story drift for all the stories after the earthquake. Tables 4-1 through 4-4 show the average value of maximum story drift for each story under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration. Average absolute difference in this study is the average of the absolute value of the difference for each earthquake based on the Horizontal Only case as basis between two different loading cases.

Table 4-1: Summary of Story Drift for the Three-Story Moment Frame as Beam Model

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.91%	2.89%	1.86%	1.23%	1.21%	13.8%
2	3.64%	3.63%	2.42%	1.96%	1.96%	13.0%
3	4.72%	4.70%	1.26%	2.06%	2.04%	15.3%

Table 4-2: Summary of Story Drift for the Three-Story Moment Frame as Girder Model

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	3.22%	3.25%	3.97%	1.63%	1.84%	74.2%
2	4.07%	4.12%	3.51%	2.67%	2.95%	39.6%
3	4.82%	4.88%	2.95%	2.84%	3.11%	38.8%

Table 4-3: Summary of Story Drift for the Three-Story Braced Frame with Chevron Configuration

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.38%	2.35%	1.92%	0.60%	0.58%	6.08%
2	1.89%	1.89%	1.28%	0.69%	0.67%	18.5%
3	1.58%	1.58%	3.57%	0.63%	0.61%	66.9%

Table 4-4: Summary of Story Drift for the Three-Story Braced Frame with Single Diagonal Configuration

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.60%	2.61%	1.31%	0.86%	0.87%	2.72%
2	2.11%	2.10%	1.16%	1.02%	1.01%	5.51%
3	1.96%	1.95%	1.52%	0.88%	0.87%	10.5%

Figures 4-3 through 4-6 show box plots of maximum story drift for each story under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration.

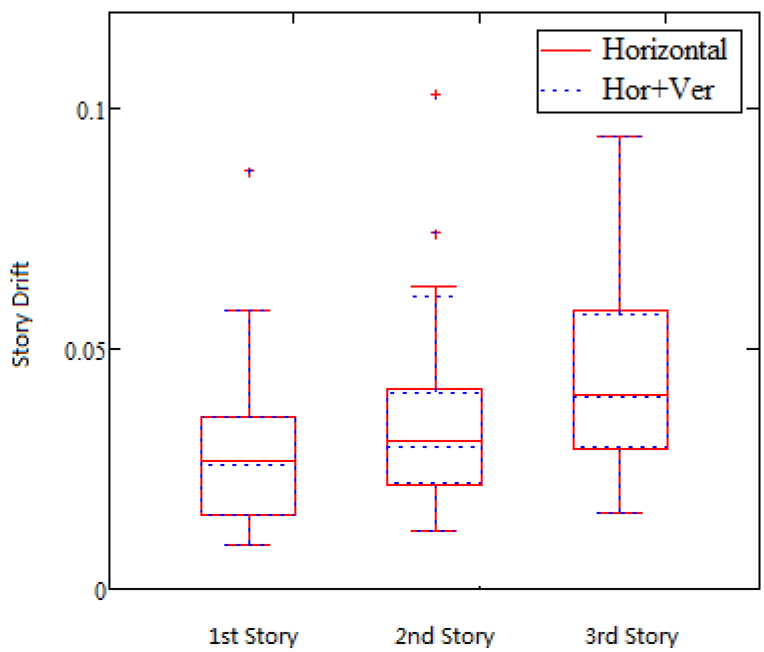

Figure 4-3: Box Plot of Max Story Drift for Three-story Moment Frame as Beam Model

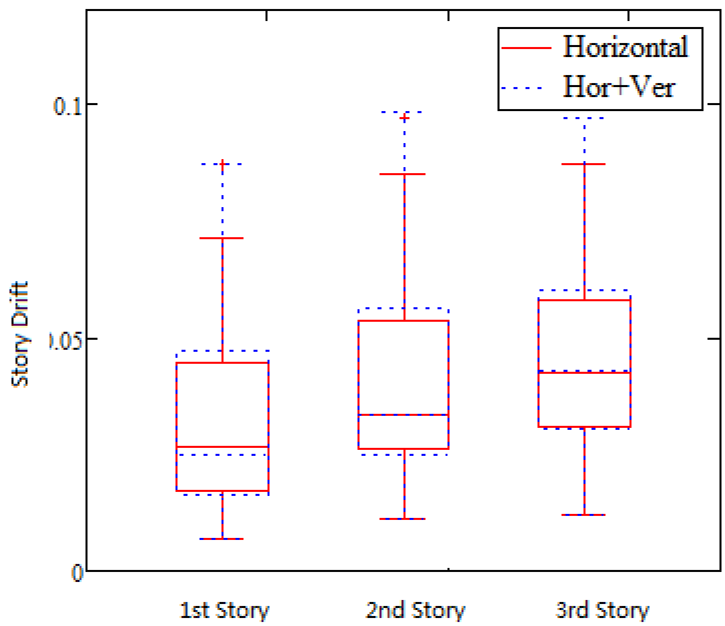

Figure 4-4: Box Plot of Max Story Drift for Three-story Moment Frame as Girder Model

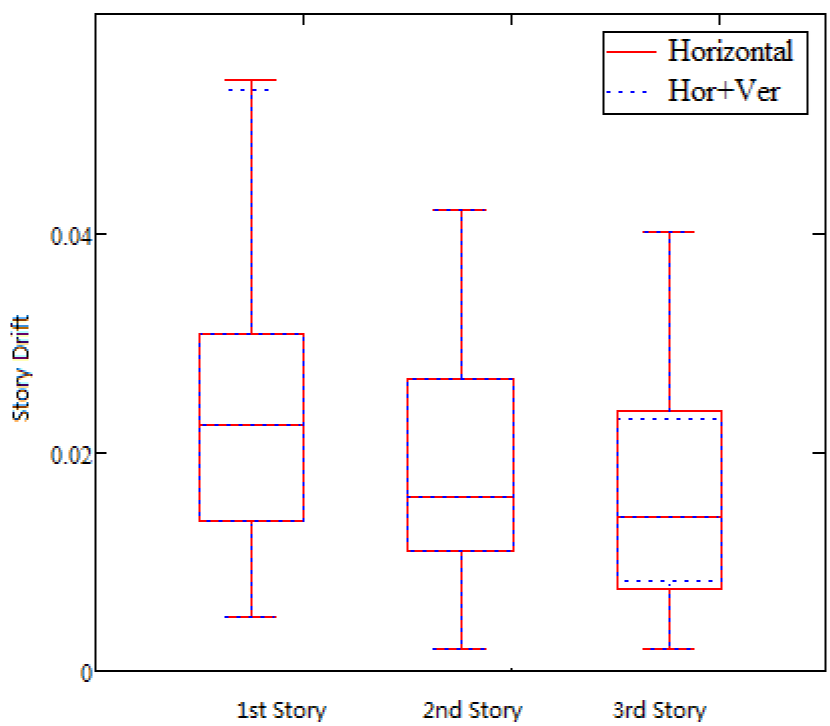

Figure 4-5: Box Plot of Max Story Drift for Three-story Chevron Braced Frame Model

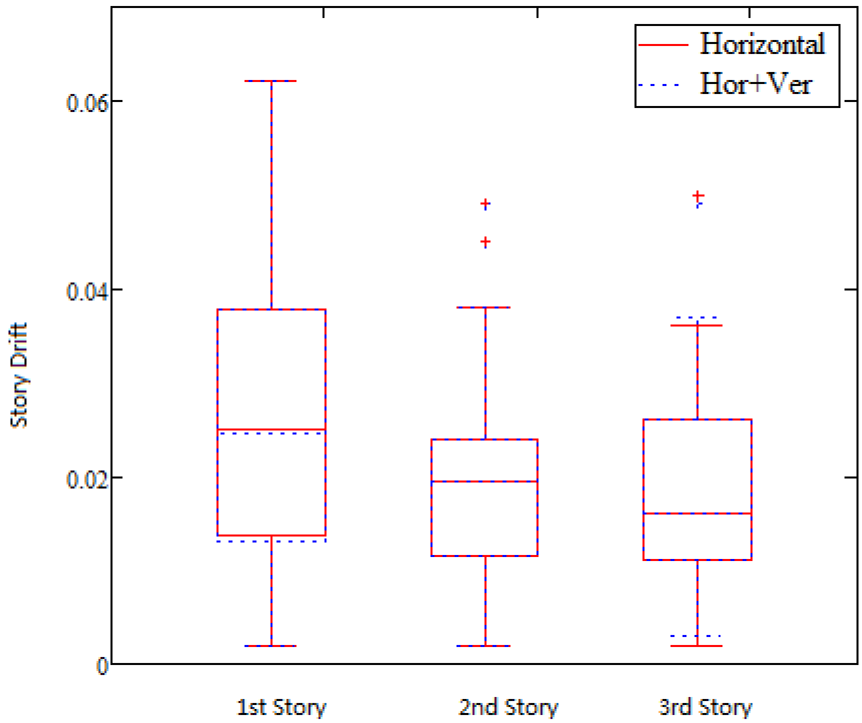

Figure 4-6: Box Plot of Max Story Drift for Three-story Single Diagonal Braced Frame Model

Tables 4-5 through 4-8 show the average value of maximum story drift for each story under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-5: Summary of Story Drift for the Six-Story Moment Frame as Beam Model

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	3.43%	3.23%	12.2%	2.79%	2.22%	65.0%
2	3.66%	3.41%	5.37%	3.14%	2.54%	43.9%
3	3.84%	3.61%	4.78%	3.28%	2.73%	42.5%
4	4.26%	4.17%	5.43%	3.50%	2.98%	20.1%
5	4.89%	4.90%	7.47%	3.91%	3.41%	41.9%
6	5.56%	5.61%	10.2%	3.98%	3.50%	32.3%

Table 4-6: Summary of Story Drift for the Six-Story Moment Frame as Girder Model

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	3.30%	3.20%	4.21%	2.38%	2.25%	18.2%
2	3.63%	3.57%	3.62%	2.60%	2.47%	12.4%
3	3.66%	3.64%	4.17%	2.60%	2.47%	14.0%
4	3.68%	3.65%	3.62%	2.53%	2.40%	32.0%
5	3.99%	3.92%	5.69%	2.41%	2.32%	32.2%
6	4.74%	4.67%	6.29%	2.46%	2.37%	30.2%

Table 4-7: Summary of Story Drift for the Six-Story Braced Frame with Chevron Configuration

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	3.25%	3.24%	1.97%	0.48%	0.48%	21.3%
2	2.63%	2.62%	2.49%	0.52%	0.52%	418%
3	2.15%	2.15%	2.99%	0.55%	0.53%	36.0%
4	1.84%	1.84%	4.74%	0.57%	0.53%	18.5%
5	1.63%	1.61%	4.95%	0.56%	0.52%	20.2%
6	1.54%	1.53%	4.92%	0.54%	0.50%	21.9%

Table 4-8: Summary of Story Drift for the Six-Story Braced Frame with Single Diagonal Configuration

Story	Max Story Drift			Residual Story Drift		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	3.18%	3.18%	1.52%	1.05%	1.05%	11.8%
2	2.73%	2.73%	1.39%	1.05%	1.06%	8.11%
3	2.39%	2.37%	2.10%	1.01%	1.02%	7.59%
4	2.12%	2.11%	1.83%	0.96%	0.97%	7.40%
5	2.01%	2.00%	1.70%	0.94%	0.95%	8.55%
6	2.19%	2.18%	1.52%	0.94%	0.95%	9.70%

Figures 4-7 through 4-10 show the box plot of maximum story drift for each story under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

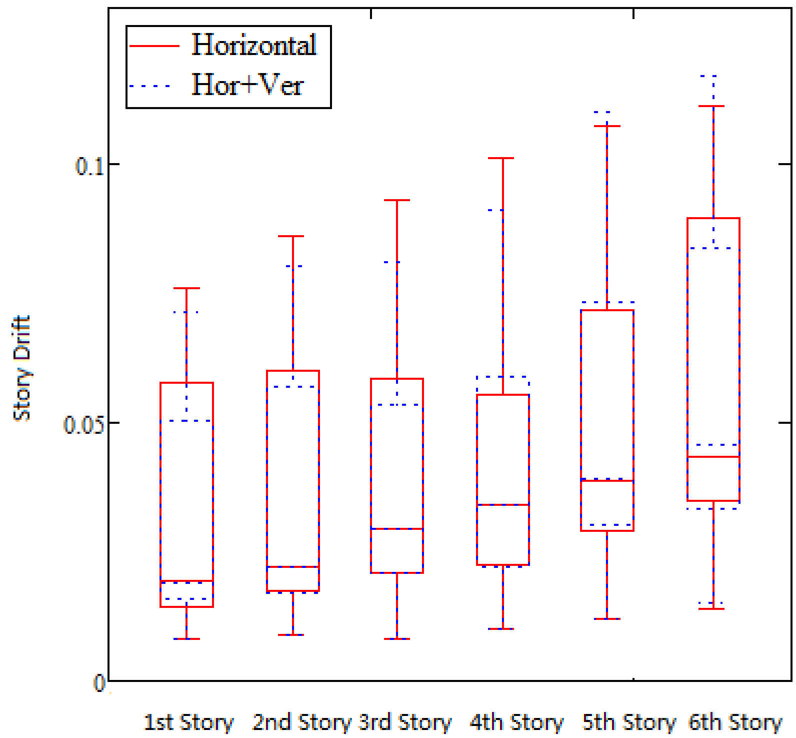

Figure 4-7: Box Plot of Max Story Drift for Six-Story Moment Frame as Beam Model

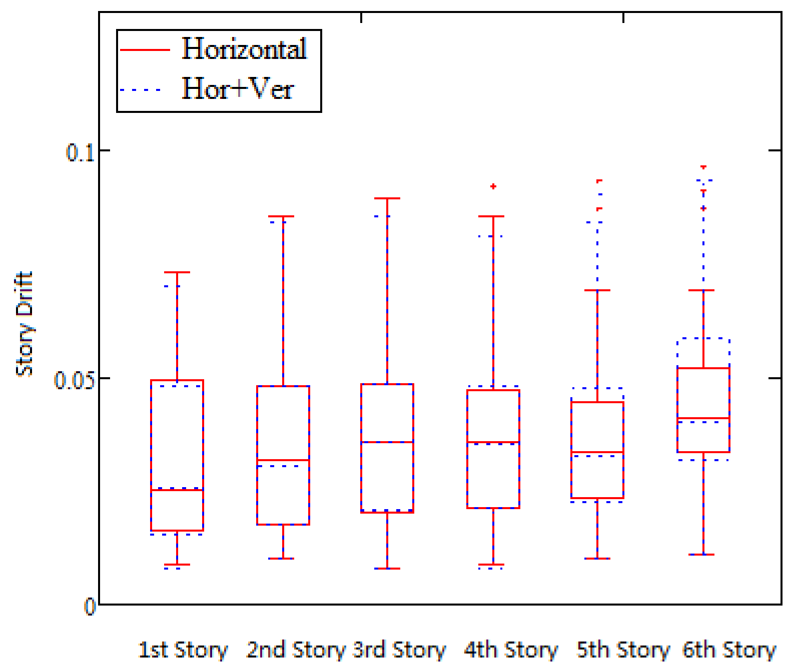

Figure 4-8: Box Plot of Max Story Drift for Six-story Moment Frame as Girder Model

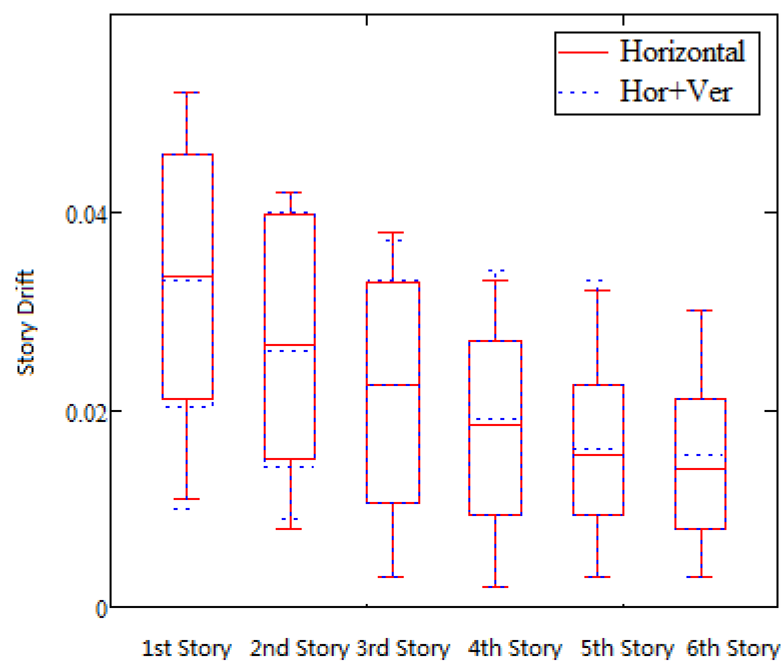

Figure 4-9: Box Plot of Max Story Drift for Six-story Chevron Braced Frame Model

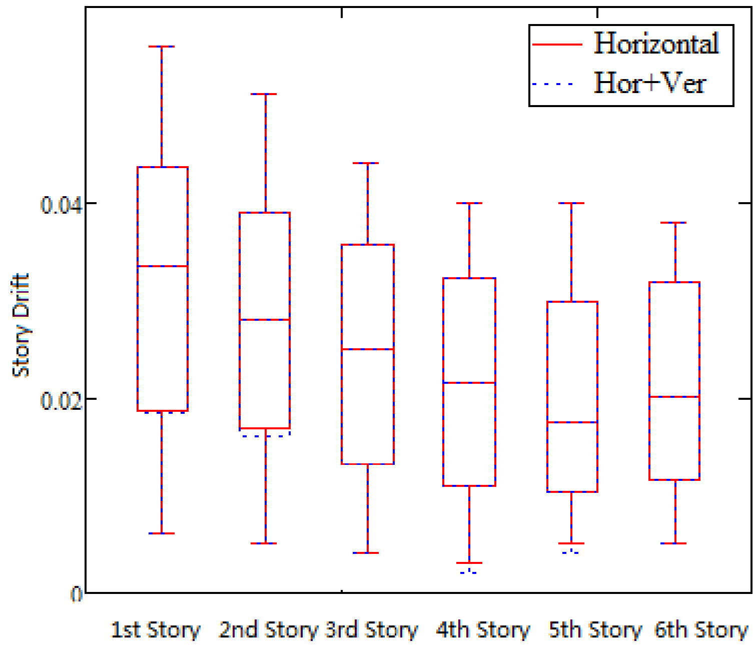

Figure 4-10: Box Plot of Max Story Drift for Six-story Single Diagonal Braced Frame Model

Figures 4-11 and 4-12 show the average absolute difference of maximum story drift for each story under forty selected earthquakes including horizontal and vertical ground motions between two different loading conditions (Horizontal Only and Horizontal + Vertical) in the three-story models and six-story models, respectively.

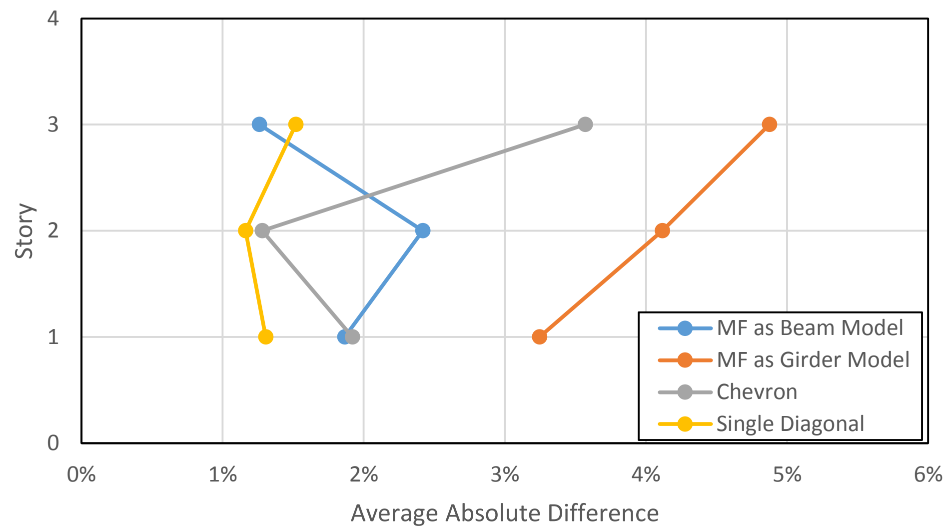

Figure 4-11: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition in Three-Story Models

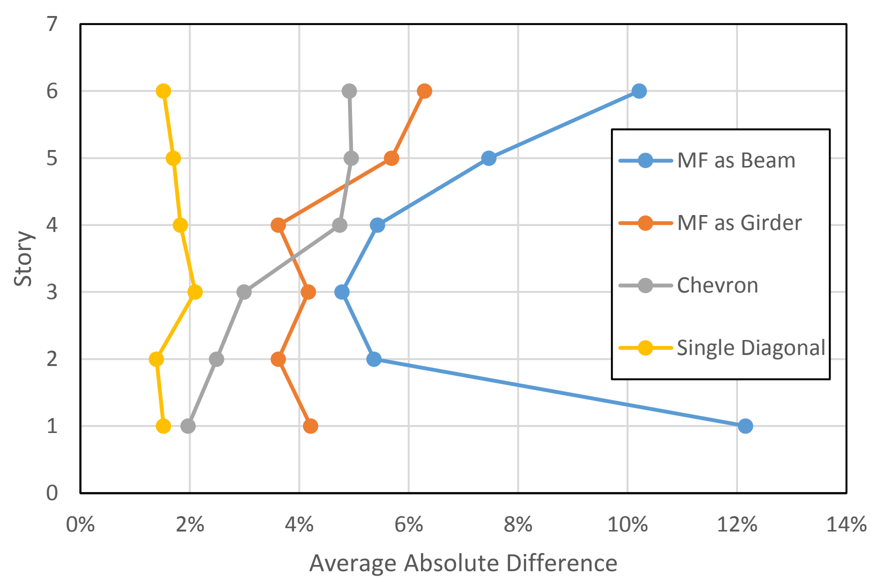

Figure 4-12: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition in Six-Story Models

The average and median value of maximum story drift for each story under forty selected earthquakes including horizontal and vertical ground motions for all the models in the study are very close to each other between the two different loading conditions (Horizontal Only and Horizontal + Vertical). What's more, the average absolute difference values of three-story moment frame, three-story braced frame and six-story braced frame are less than 5% which are very small. This means the impact of vertical ground motion on these models is not significant. However, there is some impact of vertical ground motion on the two six-story moment frame models. This is especially true for the first story which has greater height and top stories. The maximum drift for each story may increase or decrease when vertical earthquake accelerations are applied to the model. This is the reason why the average and median value are very close to each other between two different loading cases while the average absolute difference value is relatively large in the six-story moment frame models. Figure 4-13 shows the maximum drift for each story in the moment frame as beam model under FF13-1 (Horizontal Only) and FF13-1(Horizontal + Vertical). The impact of vertical ground motion is significant in this case. The remaining data about the drift of all the models in this study is provided in Appendix B.

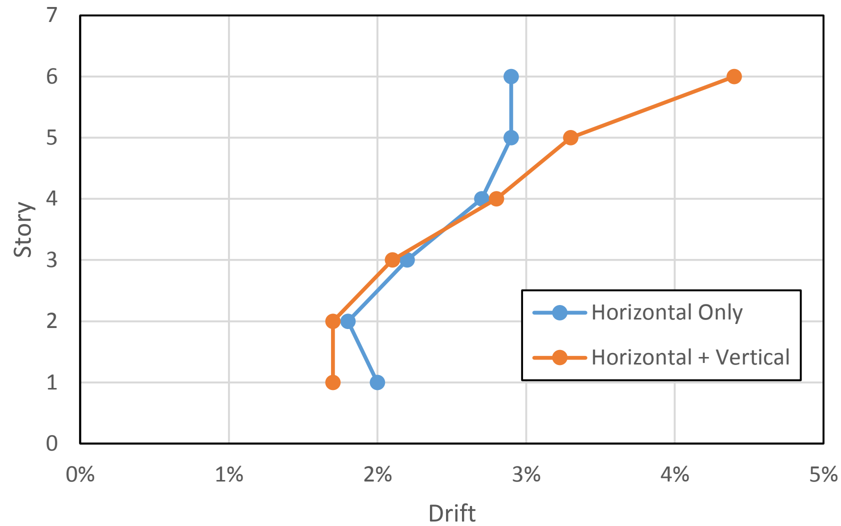

Figure 4-13: Maximum Drift for Each Story in the Moment Frame as Beam Model under FF13-1

Tables 4-9 through 4-12 show the average and average absolute difference value of maximum story drift for each story by earthquake group under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-9: Summary of Story Drift by Earthquake Group for the Three-Story Moment Frame as Beam Model

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	1.90%	1.88%	1.18%	5.08%	5.06%	0.43%
2	2.60%	2.60%	1.79%	6.28%	6.24%	0.63%
3	3.62%	3.58%	0.82%	8.02%	8.02%	0.00%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.66%	2.58%	2.62%	2.00%	2.05%	3.24%
2	3.04%	2.94%	3.15%	2.64%	2.72%	4.11%
3	3.98%	3.92%	2.40%	3.24%	3.26%	1.82%

Table 4-10: Summary of Story Drift by Earthquake Group for the Three-Story Moment Frame as Girder Model

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.12%	2.08%	1.38%	4.90%	4.84%	1.98%
2	3.10%	3.02%	2.55%	5.58%	5.56%	1.69%
3	3.86%	3.84%	1.41%	7.00%	6.92%	1.32%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	1.82%	1.70%	6.16%	4.02%	4.36%	6.37%
2	2.68%	2.60%	4.01%	4.90%	5.28%	5.80%
3	3.30%	3.22%	2.82%	5.10%	5.52%	6.27%

Table 4-11: Summary of Story Drift by Earthquake Group for the Three-Story Braced Frame with Chevron Configuration

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.12%	2.06%	2.82%	3.04%	3.04%	0.00%
2	1.56%	1.54%	1.25%	2.66%	2.66%	0.00%
3	1.31%	1.31%	0.19%	2.34%	2.28%	2.59%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.08%	2.06%	2.08%	2.27%	2.24%	2.80%
2	1.67%	1.68%	2.73%	1.67%	1.67%	1.15%
3	1.51%	1.50%	3.10%	1.17%	1.23%	8.40%

Table 4-12: Summary of Story Drift by Earthquake Group for the Three-Story Braced Frame with Single Diagonal Configuration

Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.34%	2.34%	2.49%	3.40%	3.38%	0.74%
2	1.65%	1.65%	0.61%	2.96%	2.94%	0.95%
3	1.62%	1.62%	0.00%	2.80%	2.76%	3.81%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.15%	2.16%	0.29%	2.51%	2.55%	1.71%
2	1.86%	1.84%	2.48%	1.96%	1.96%	0.62%
3	1.99%	1.98%	1.15%	1.42%	1.44%	1.11%

Figures 4-14 through 4-17 show the average absolute difference of maximum story drift by earthquake group for each story under forty selected earthquakes including horizontal and vertical ground motions between two different loading condition (Horizontal Only and Horizontal + Vertical) in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration, respectively.

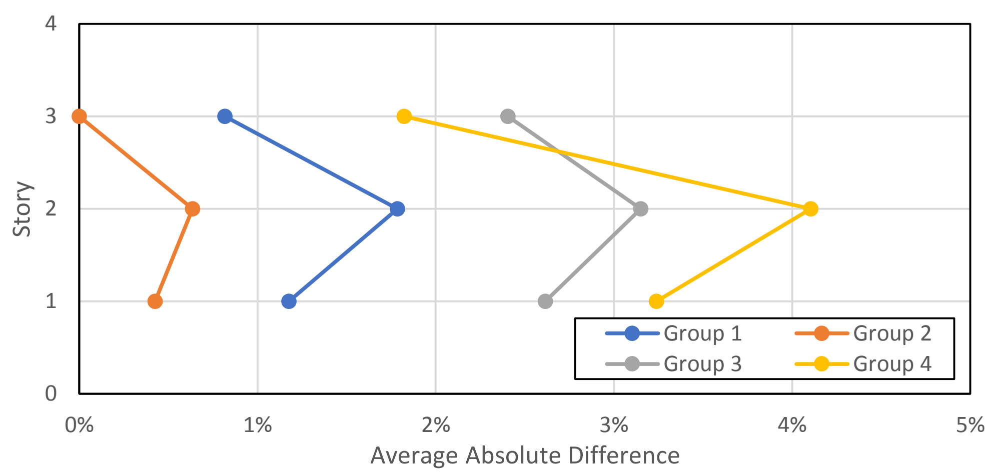

Figure 4-14: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in Three-Story MF as Beam Model

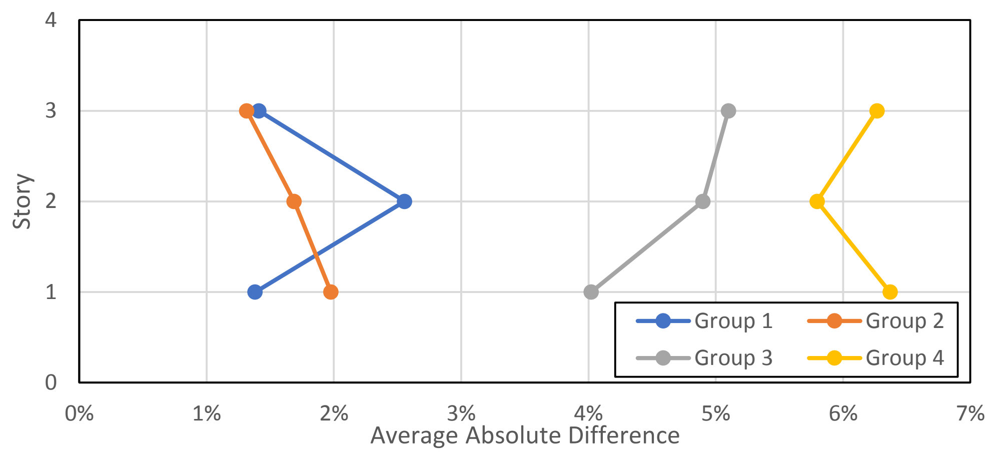

Figure 4-15: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake in Three-Story MF as Girder Model

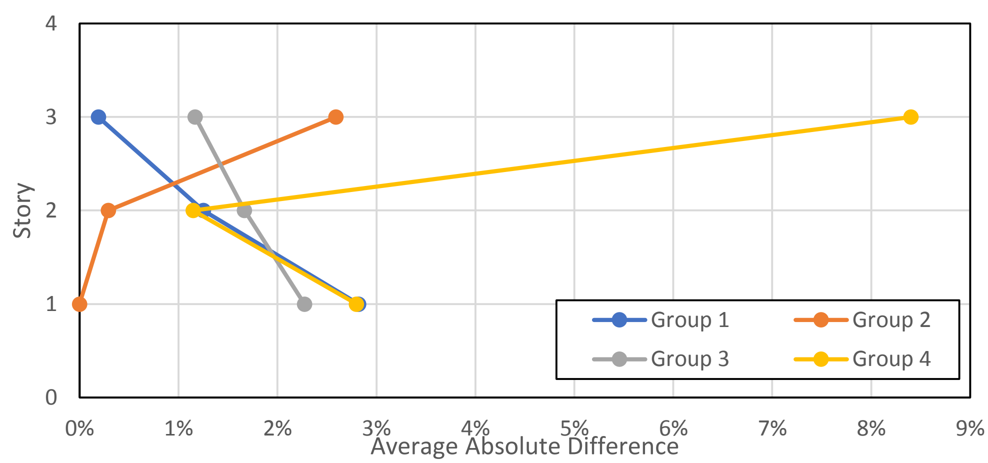

Figure 4-16: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in Three-Story Braced frame with Chevron Configuration

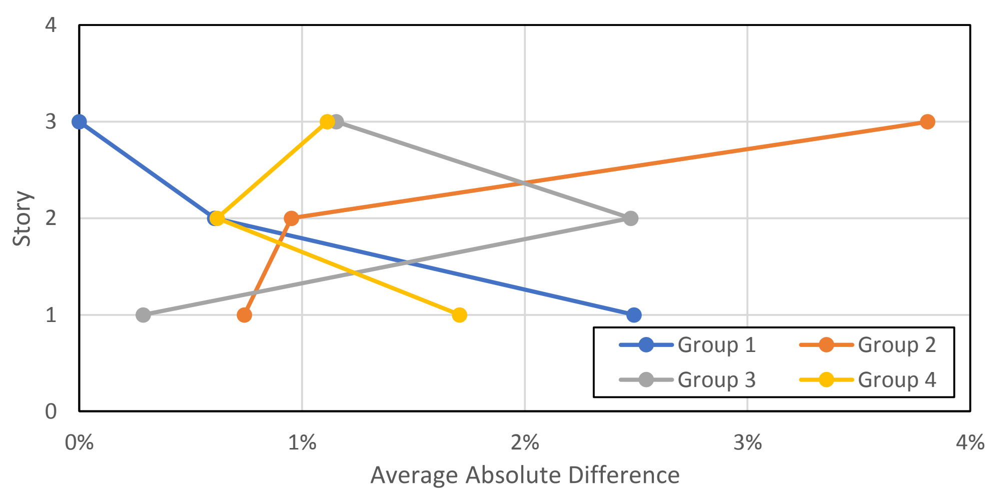

Figure 4-17: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in Three-Story Braced frame with Single Diagonal Configuration

Tables 4-13 through 4-16 show the average and average absolute difference value of maximum story drift for each story by earthquake group under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-13: Summary of Story Drift by Earthquake Group for the Six-Story Moment Frame as Beam Model

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.86%	2.80%	2.69%	2.95%	3.14%	26.3%
2	3.02%	2.98%	1.68%	2.98%	2.96%	2.54%
3	3.08%	3.06%	3.59%	3.06%	2.92%	4.65%
4	3.24%	3.16%	4.90%	3.80%	3.62%	4.69%
5	3.64%	3.66%	4.74%	5.08%	4.82%	6.92%
6	4.32%	4.34%	8.47%	6.66%	6.24%	7.38%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.72%	2.16%	9.94%	5.17%	4.81%	9.75%
2	3.07%	2.51%	8.20%	5.55%	5.18%	9.06%
3	3.58%	3.16%	5.21%	5.62%	5.30%	5.66%
4	4.20%	4.06%	2.36%	5.81%	5.83%	9.76%
5	4.72%	4.86%	6.21%	6.12%	6.26%	12.0%
6	4.90%	5.32%	11.43%	6.34%	6.54%	13.6%

Table 4-14: Summary of Story Drift by Earthquake Group for the Six-Story Moment Frame as Girder Model

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.66%	2.64%	2.16%	2.81%	2.79%	4.24%
2	2.74%	2.76%	2.51%	3.02%	2.98%	3.27%
3	2.80%	2.78%	2.59%	3.04%	3.10%	6.69%
4	2.72%	2.66%	2.52%	3.46%	3.48%	8.42%
5	3.10%	3.04%	1.97%	4.46%	4.16%	8.90%
6	4.06%	4.00%	4.33%	5.64%	5.24%	7.53%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.88%	2.74%	3.80%	4.84%	4.64%	6.62%
2	3.30%	3.24%	4.10%	5.46%	5.30%	4.60%
3	3.32%	3.36%	5.28%	5.48%	5.33%	2.11%
4	3.18%	3.22%	1.30%	5.35%	5.23%	2.26%
5	3.24%	3.34%	7.16%	5.16%	5.16%	4.74%
6	3.92%	4.08%	8.32%	5.34%	5.36%	4.97%

Table 4-15: Summary of Story Drift by Earthquake Group for the Six-Story Braced Frame with Chevron Configuration

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.84%	2.78%	3.60%	3.30%	3.34%	2.13%
2	2.20%	2.17%	2.66%	2.72%	2.75%	4.72%
3	1.80%	1.78%	2.24%	2.30%	2.31%	6.32%
4	1.50%	1.47%	2.79%	2.06%	2.05%	8.96%
5	1.38%	1.29%	6.42%	1.90%	1.91%	6.79%
6	1.32%	1.25%	4.83%	1.86%	1.88%	7.76%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.96%	2.92%	1.61%	3.88%	3.90%	0.56%
2	2.30%	2.28%	2.59%	3.28%	3.28%	0.00%
3	1.77%	1.77%	1.68%	2.71%	2.76%	1.74%
4	1.44%	1.42%	5.01%	2.36%	2.41%	2.21%
5	1.18%	1.16%	4.66%	2.06%	2.08%	1.94%
6	1.01%	1.03%	6.12%	1.97%	1.95%	0.98%

Table 4-16: Summary of Story Drift by Earthquake Group for the Six-Story Braced Frame with Single Diagonal Configuration

Max Story Drift						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.93%	2.91%	0.99%	3.10%	3.10%	2.31%
2	2.45%	2.45%	0.01%	2.61%	2.60%	4.30%
3	2.07%	2.07%	0.04%	2.30%	2.25%	7.42%
4	1.84%	1.82%	1.37%	2.06%	2.02%	5.91%
5	1.82%	1.82%	0.00%	2.08%	2.04%	6.71%
6	1.92%	1.92%	0.00%	2.41%	2.37%	5.19%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	2.90%	2.90%	2.36%	3.80%	3.82%	0.41%
2	2.41%	2.39%	1.26%	3.46%	3.46%	0.00%
3	2.05%	2.03%	0.93%	3.13%	3.13%	0.00%
4	1.79%	1.79%	0.03%	2.81%	2.81%	0.03%
5	1.54%	1.54%	0.05%	2.61%	2.61%	0.02%
6	1.61%	1.62%	0.81%	2.80%	2.80%	0.07%

Figures 4-18 through 4-21 show the average absolute difference of maximum story drift by earthquake group for each story under forty selected earthquakes including horizontal and vertical ground motions between two different loading conditions (Horizontal Only and Horizontal + Vertical) in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration, respectively.

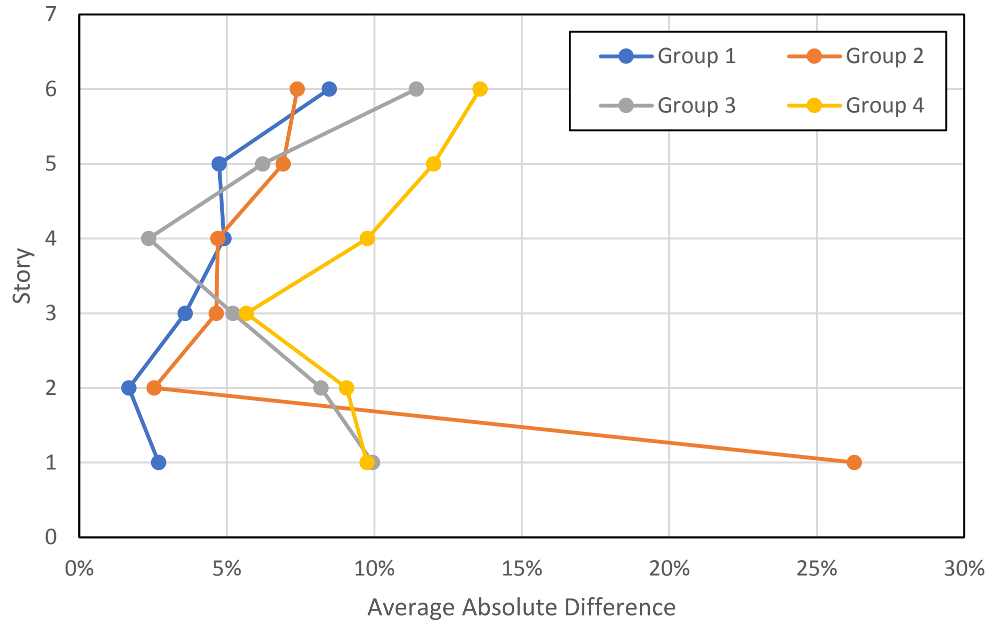

Figure 4-18: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in Six-Story MF as Beam Model

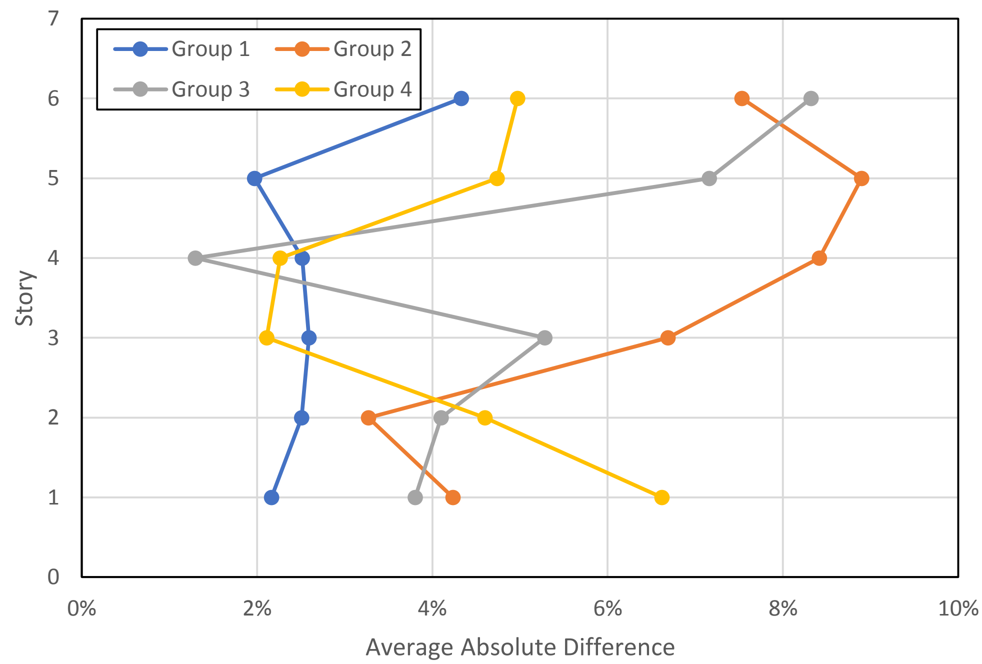

Figure 4-19: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in Six-Story MF as Girder Model

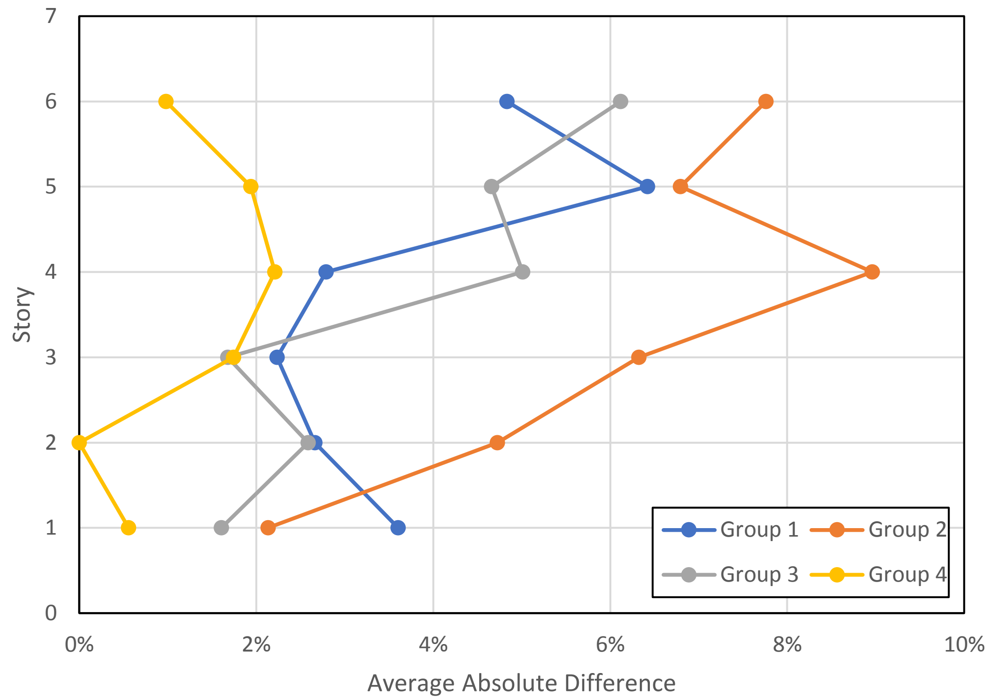

Figure 4-20: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in six-Story Braced frame with Chevron Configuration

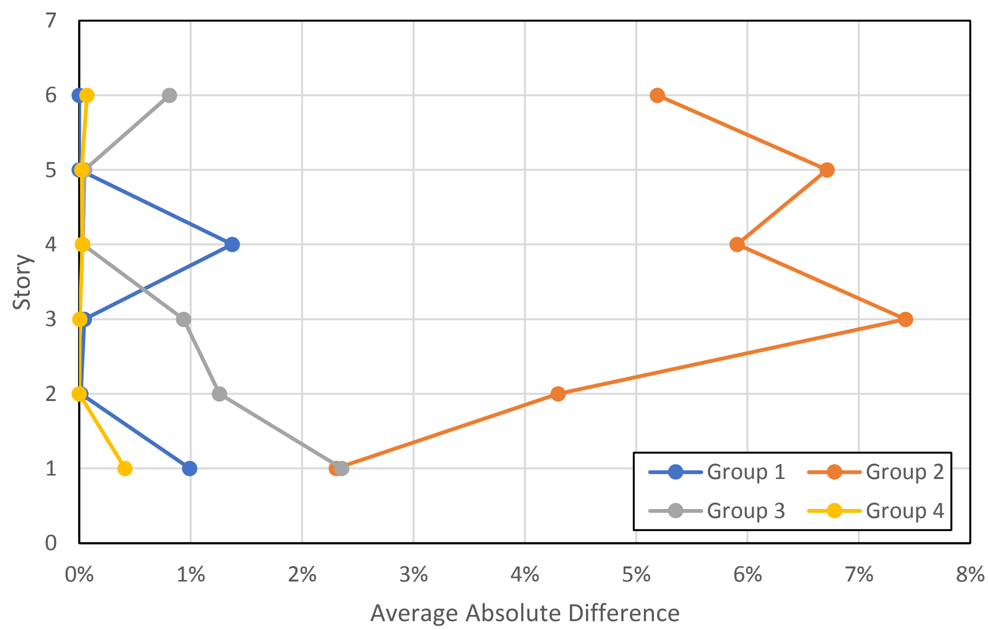

Figure 4-21: Average Absolute Difference of Maximum Story Drift between Two Different Loading Condition by Earthquake Group in Six-Story Braced frame with Single Diagonal Configuration

There are twenty earthquake suites in this study which include the horizontal and vertical components of strong ground motion. They are classified as four different earthquake groups in this study based on the vertical to horizontal ratio (V/H) of earthquake. Most of the V/H values in this study are larger than 2/3 which is commonly used by current codes. According to the data analysis by earthquake groups, there is no clear evidence showing that the higher or lower V/H value will cause more or less impact on the maximum drift in the steel frame structures. That means the impact of vertical ground motion on maximum drift of steel frame structures in each earthquake suite is insignificant and limited.

4.3 Column Axial Force

Columns are the most common type of compression member in building structures. In the moment frame, these elements are also subjected to bending which will become beam-column elements. In seismic design, columns are not allowed to fail first, which could cause the structure to collapse. What's more, column axial force has a direct relationship with the P-delta effect and buckling in nonlinear analysis. It is necessary for engineers to pay attention to the column axial force.

The data on maximum axial force for all the stories during the earthquake is calculated according to the element axial force at node i of the columns for each floor. Exterior column axial force and interior column axial force are collected separately. Tables 4-17 through 4-20 show the average value of maximum column axial force for each story under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration. All the column forces in this study are normalized column axial force which is the axial force from Perform 3D divided by the column yield force (Fy*Ag). The contribution of vertical motion to total axial force is calculated based on the Horizontal + Vertical case as the basis. If the Horizontal Only case is used as basis, the difference between two different loading cases for most models will be more than 100%.

Table 4-17: Summary of Column Axial Force for the Three-Story Moment Frame as Beam Model

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1st	0.087	0.107	19.2%	0.066	0.125	47.2%
2nd	0.054	0.069	22.5%	0.045	0.087	48.1%
3rd	0.019	0.027	29.2%	0.024	0.043	45.4%

Table 4-18: Summary of Column Axial Force for the Three-Story Moment Frame as Girder Model

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.048	0.067	28.6%	0.107	0.178	39.7%
2	0.035	0.045	21.3%	0.086	0.165	48.0%
3	0.020	0.023	14.7%	0.061	0.108	43.9%

Table 4-19: Summary of Column Axial Force for the Three-Story Braced Frame with Chevron Configuration

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.330	0.357	7.69%	0.386	0.470	17.9%
2	0.153	0.179	14.4%	0.194	0.277	29.7%
3	0.038	0.060	36.7%	0.063	0.122	48.8%

Table 4-20: Summary of Column Axial Force for the Three-Story Braced Frame with Single Diagonal Configuration

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.326	0.356	8.40%	0.587	0.700	16.2%
2	0.304	0.325	6.43%	0.216	0.341	36.7%
3	0.039	0.062	36.4%	0.163	0.228	28.5%

Figures 4-22 through 4-25 show the histogram of average value of column maximum normalized axial force for each story under two different loading cases for each selected earthquake in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration.

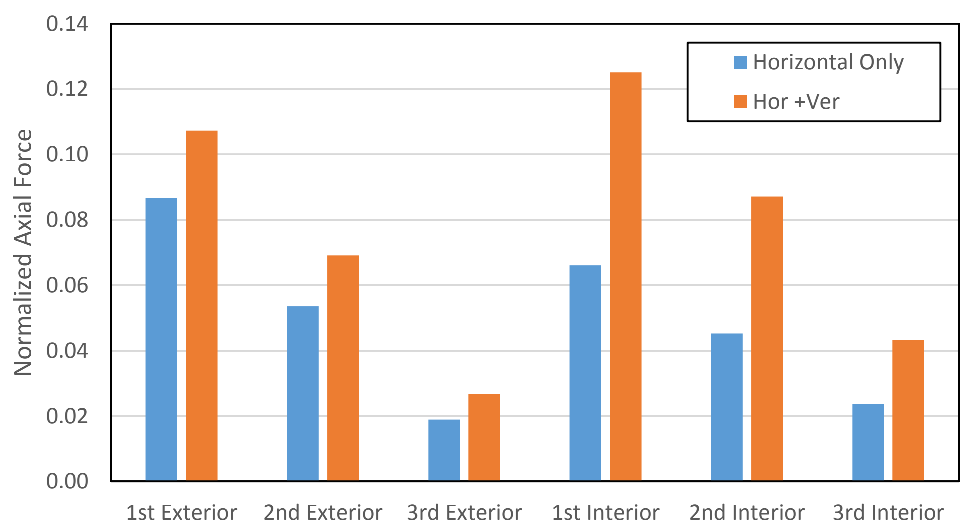

Figure 4-22: Max Normalized Story Axial Forcefor Three-Story Moment Frame as Beam Model

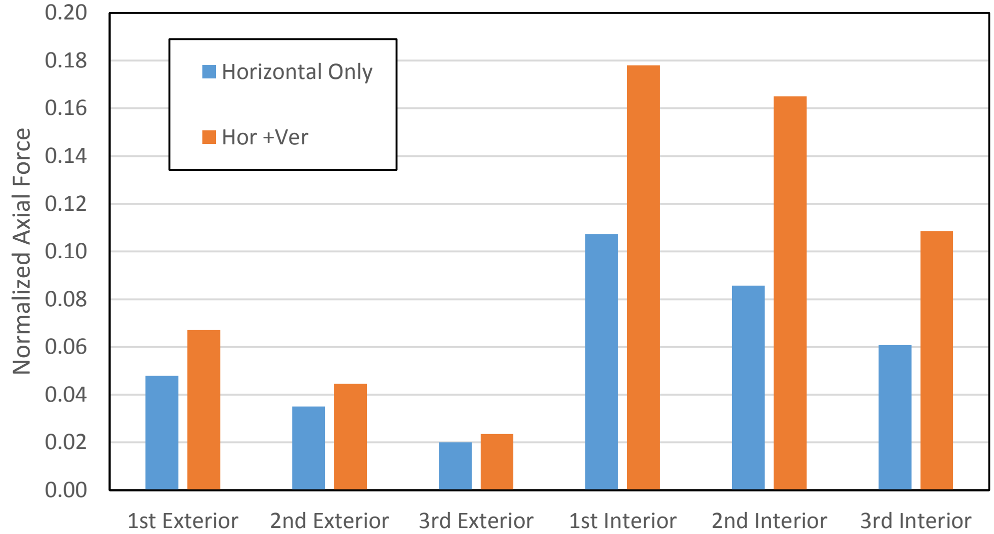

Figure 4-23: Max Normalized Story Axial Force for Three-Story Moment Frame as Girder Model

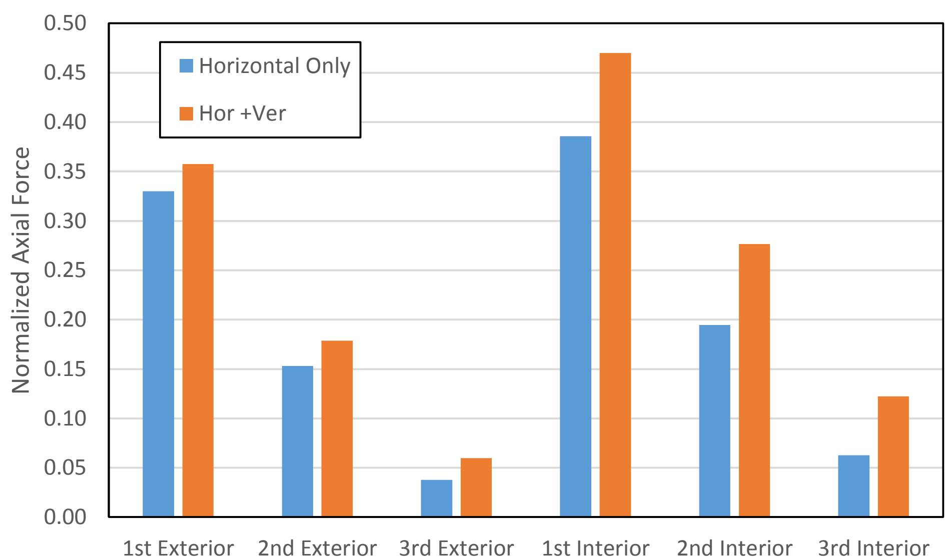

Figure 4-24: Max Normalized Story Axial Force for Three-Story Braced Frame with Chevron Configuration

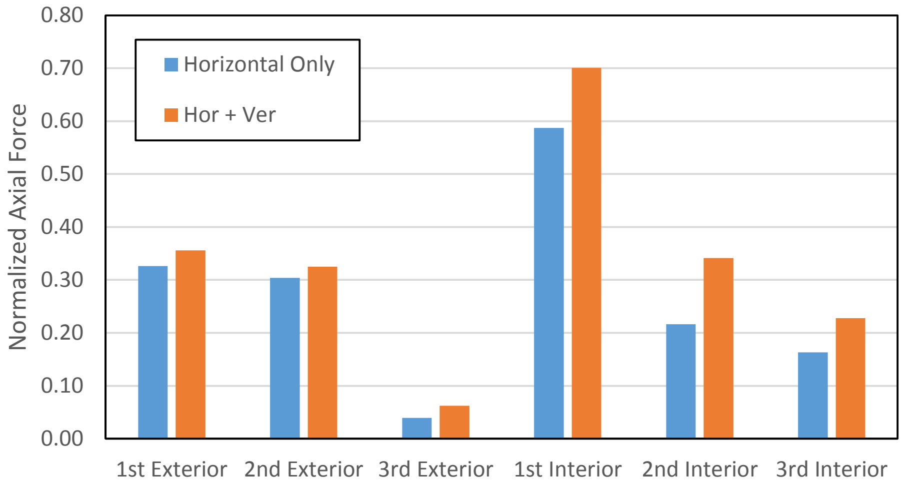

Figure 4-25: Max Normalized Story Axial Force for Three-Story Braced Frame with Single Diagonal Configuration

Tables 4-21 through 4-24 show the average value of maximum normalized column axial force for each story under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration respectively.

Table 4-21: Summary of Column Axial Force for the Six-Story Moment Frame as Beam Model

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.111	0.143	22.1%	0.142	0.283	50.0%
2	0.086	0.115	25.2%	0.118	0.250	52.8%
3	0.063	0.088	28.8%	0.093	0.209	55.5%
4	0.061	0.102	40.4%	0.102	0.263	61.1%
5	0.037	0.065	43.1%	0.068	0.179	62.2%
6	0.016	0.031	48.6%	0.033	0.086	61.6%

Table 4-22: Summary of Column Axial Force for the Six-Story Moment Frame as Girder Model

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.134	0.169	20.8%	0.127	0.226	44.0%
2	0.108	0.141	23.3%	0.104	0.192	45.5%
3	0.081	0.111	26.9%	0.082	0.158	48.3%
4	0.084	0.113	26.0%	0.090	0.169	46.8%
5	0.048	0.071	31.3%	0.059	0.122	51.3%
6	0.022	0.032	31.0%	0.030	0.054	44.2%

Table 4-23: Summary of Column Axial Force for the Three-Story Braced Frame with Chevron Configuration

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.187	0.190	1.69%	0.229	0.241	4.84%
2	0.141	0.143	1.03%	0.177	0.186	5.17%
3	0.099	0.101	2.05%	0.129	0.138	5.97%
4	0.099	0.101	2.83%	0.135	0.147	8.01%
5	0.052	0.055	5.53%	0.077	0.088	12.2%
6	0.014	0.017	16.6%	0.027	0.039	29.9%

Table 4-24: Summary of Column Axial Force for the Three-Story Braced Frame with Single Diagonal Configuration

Story	Normalized Column Axial Force					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force	Horizontal Only	Horizontal + Vertical	Contribution of Vertical motion to total axial Force
1	0.215	0.220	2.17%	0.211	0.231	8.46%
2	0.126	0.132	4.16%	0.196	0.211	7.12%
3	0.119	0.122	3.05%	0.119	0.134	11.1%
4	0.089	0.098	8.42%	0.165	0.181	8.84%
5	0.077	0.082	5.33%	0.072	0.090	20.4%
6	0.014	0.021	33.0%	0.047	0.059	19.8%

Figures 4-26 through 4-29 show the histogram of average value of column maximum normalized axial force for each story under two different loading cases for each selected earthquake in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

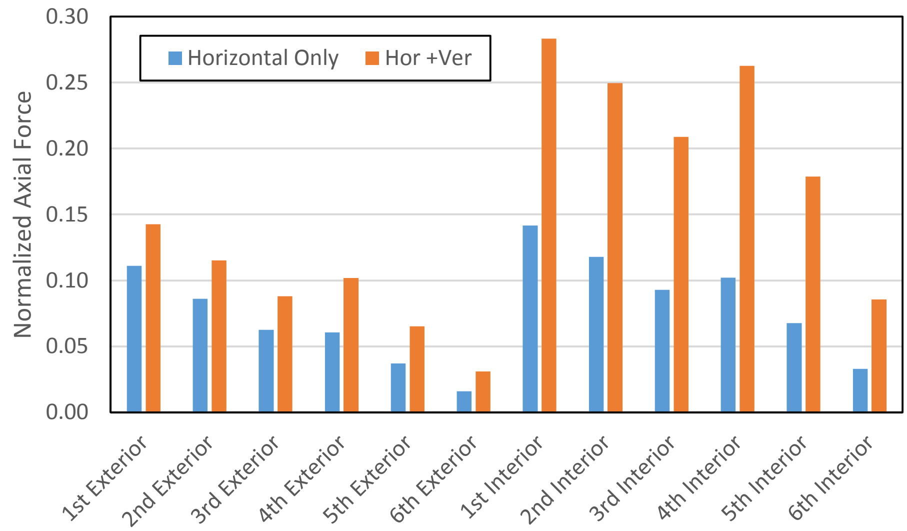

Figure 4-26: Max Normalized Story Axial Force for Six-Story Moment Frame as Beam Model

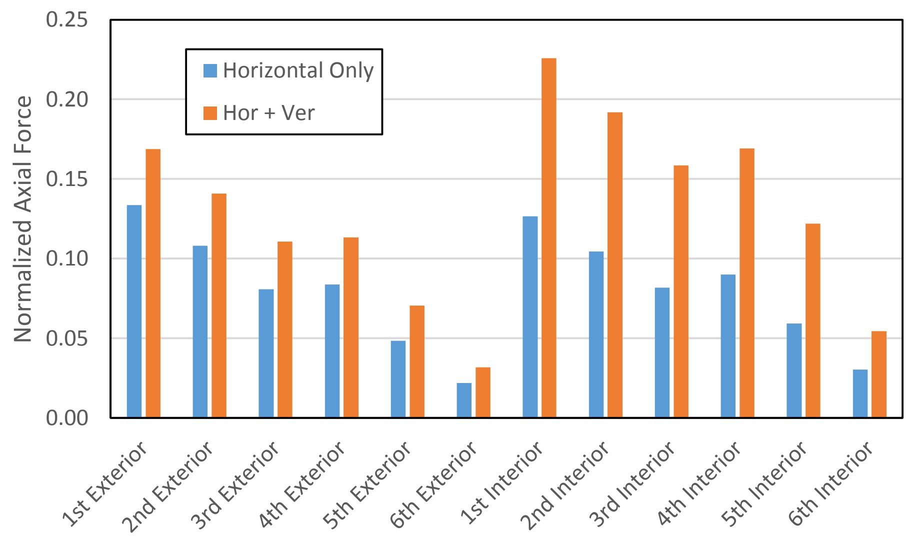

Figure 4-27: Max Normalized Story Axial Force for Six-Story Moment Frame as Girder Model

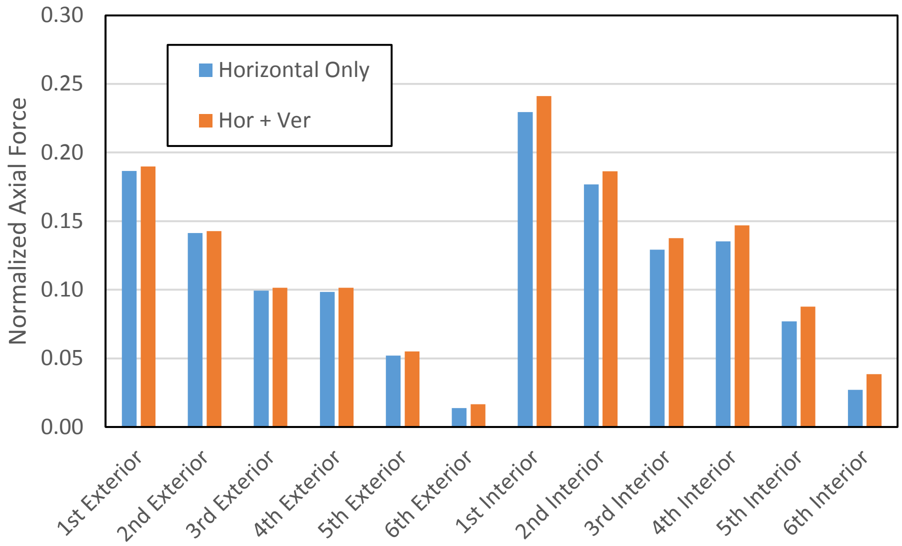

Figure 4-28: Max Normalized Story Axial Force for Six-Story Braced Frame with Chevron Configuration

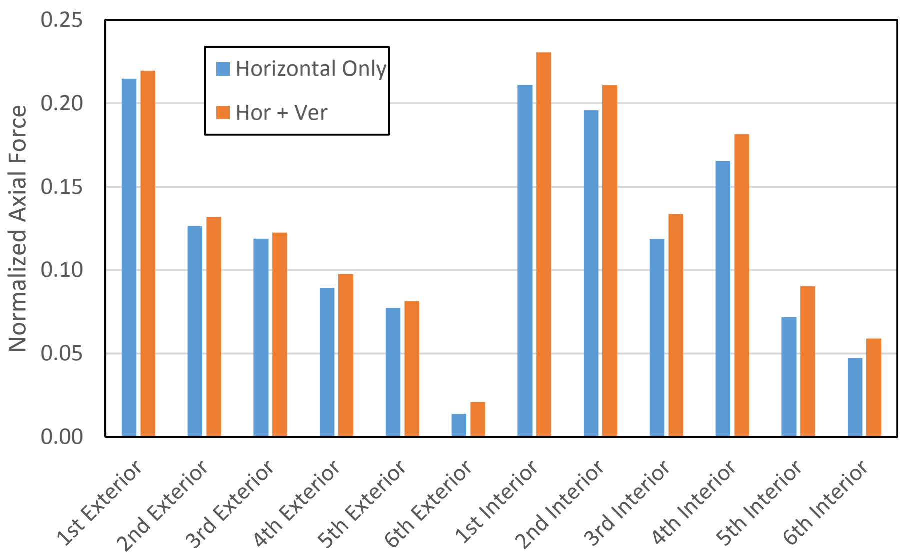

Figure 4-29: Max Normalized Story Axial Force for Six-Story Braced Frame with Single Diagonal Configuration

The average value of column maximum normalized axial force for each story increases for all the models in the study when the vertical ground motion is added. It is clear that the impact of vertical ground motion on the interior columns is much larger than that on the exterior columns which is obvious because the vertical joint mass in the interior columns is larger than that in the exterior columns. In addition, the moment frames get more influence from vertical ground motions compared with braced frame in this study. For the moment frame, the impact of vertical acceleration is significant for each floor similarly. However, in the braced frame, the normalized axial force increases significantly in the top story while the effect of vertical earthquake in the remaining stories is relatively not significant compared with the top story. The reason why the top story has larger impact is because the vertical component in the braces is transferred to the columns. Therefore, the increase of axial force due to vertical ground motion is a smaller percentage overall.

4.4 Acceleration

4.4.1 Roof Horizontal Acceleration

The seismic body and surface waves create inertial forces within the building. The acceleration, or the rate of change of the velocity of the waves setting the building in motion, determines the percentage of the building mass or weight that must be dealt with as a horizontal force. All the acceleration in this study is measured in terms of the acceleration due to gravity or g. The maximum roof horizontal acceleration during the earthquake is calculated according to the nodal absolute acceleration. Figure 4-30 shows the node chosen to represent the roof in the six-story moment frame as beam model. The nodes in the remaining models have the same location with this example.

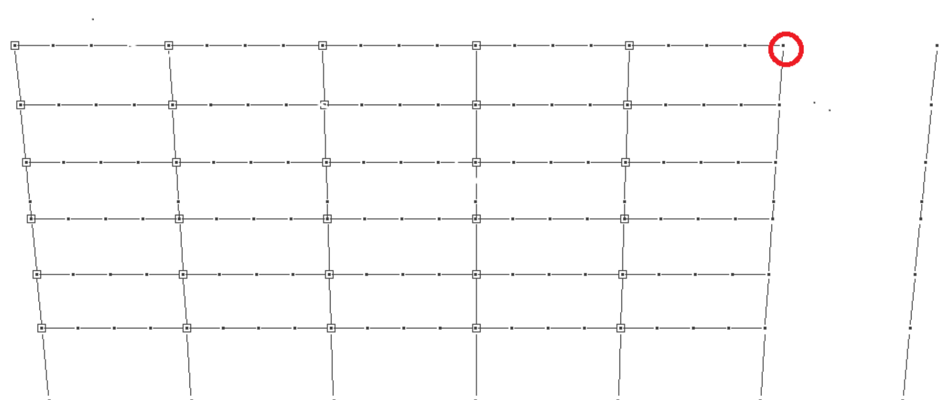

Figure 4-30: Node Chosen to Represent Roof Horizontal Acceleration

Tables 4-25 and 4-26 show the average value of maximum roof horizontal acceleration under forty selected earthquakes including horizontal and vertical ground motions in the three-story models and six-story models respectively.

Table 4-25: Summary of Roof Horizontal Acceleration for the Three-Story Models

Model	Roof Horizontal Acceleration		
	Horizontal Only	Horizontal + Vertical	Difference
3-Story MF as Beam	0.866	0.939	8.39%
3-Story MF as Girder	0.826	0.848	2.63%
3-Story Chevron	1.100	1.125	2.22%
3-Story Single Diagonal	0.977	1.014	3.82%

Table 4-26: Summary of Roof Horizontal Acceleration for the Six-Story Models

Model	Roof Horizontal Acceleration		
	Horizontal Only	Horizontal + Vertical	Difference
6-Story MF as Beam	0.762	1.209	58.6%
6-Story MF as Girder	0.929	1.003	8.00%
6-Story Chevron	0.429	0.429	-0.08%
6-Story Single Diagonal	0.583	0.583	-0.02%

According to the tables above, the average value of maximum roof acceleration for most models in the study are very close to each other between the two different loading conditions except the six-story moment frame as beam model. In the six-story moment frame as beam model, the average value of roof maximum horizontal acceleration increased by 58.63% when the vertical and horizontal ground motions are added to the structure at same time. Figures 4-31 and 4-35 show the time history response of roof absolute horizontal acceleration in the six-story moment frame as beam model as an example under FF-14-1 and NF-28-1. Figures 4-32 and 4-36 show the time history response of roof absolute horizontal acceleration in the six-story moment frame as girder model as an example under FF-14-1 and NF-28-1. The result of the remaining models is very similar to the six-story moment as girder model.

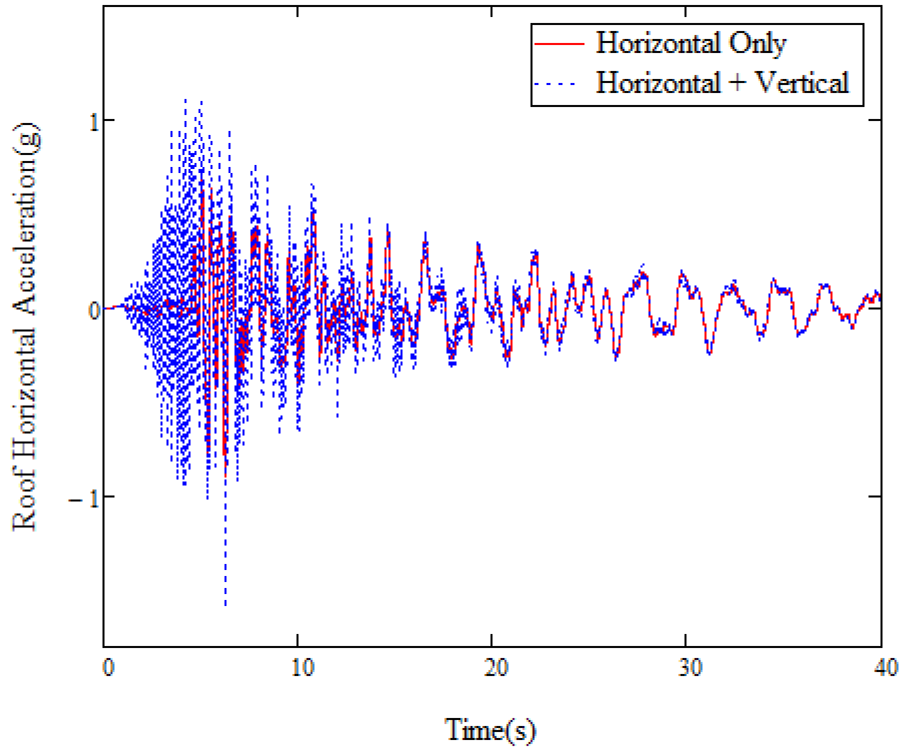

Figure 4-31: Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Beam Model under FF-14-1

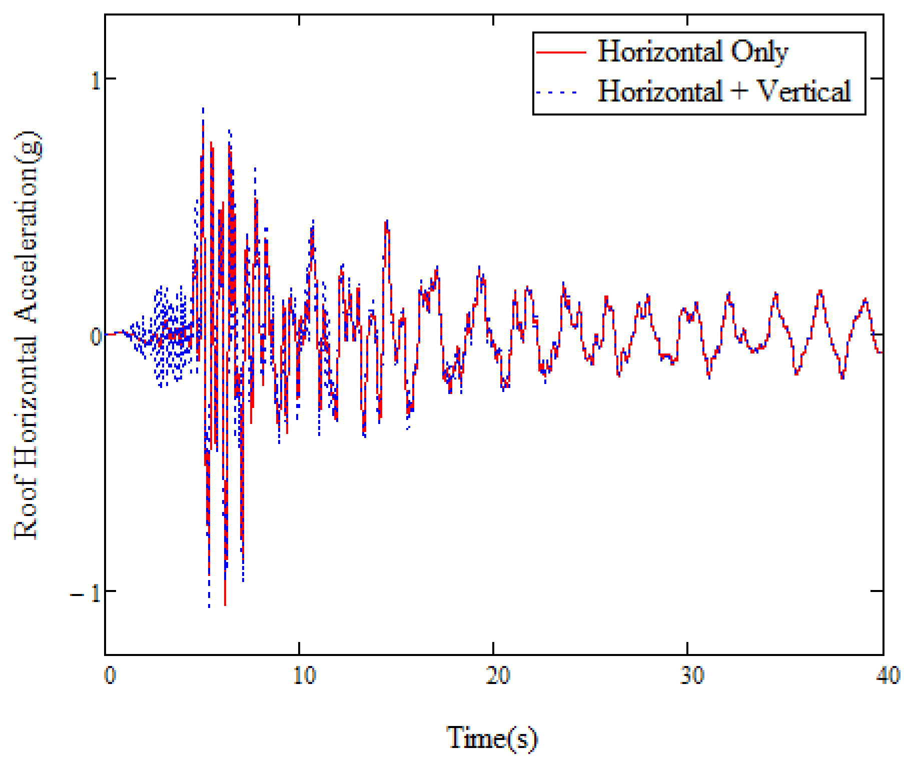

Figure 4-32:Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Girder Model under NF-14-1

Figure 4-33 and Figure 4-34 shows the detail of time history response of roof absolute horizontal acceleration from zero seconds to ten seconds in the six-story moment frame as beam model and the six-story moment frame as girder model under NF-14-1, respectively.

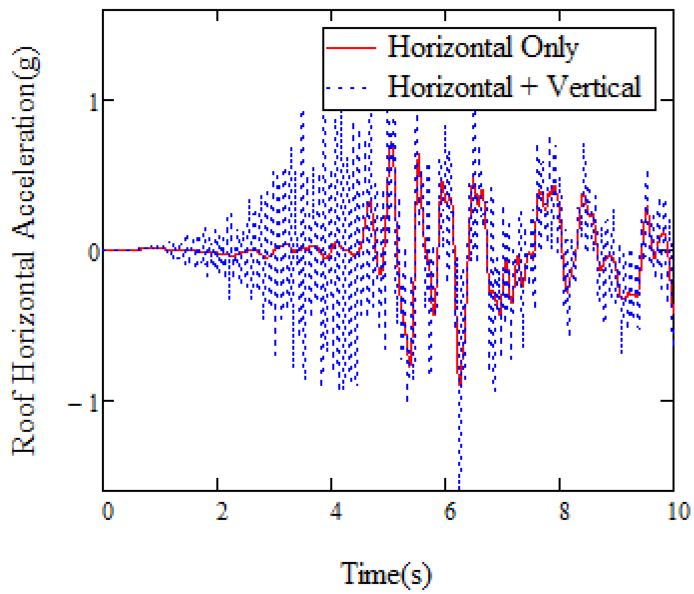

Figure 4-33: Detail of Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Beam Model under NF-14-1

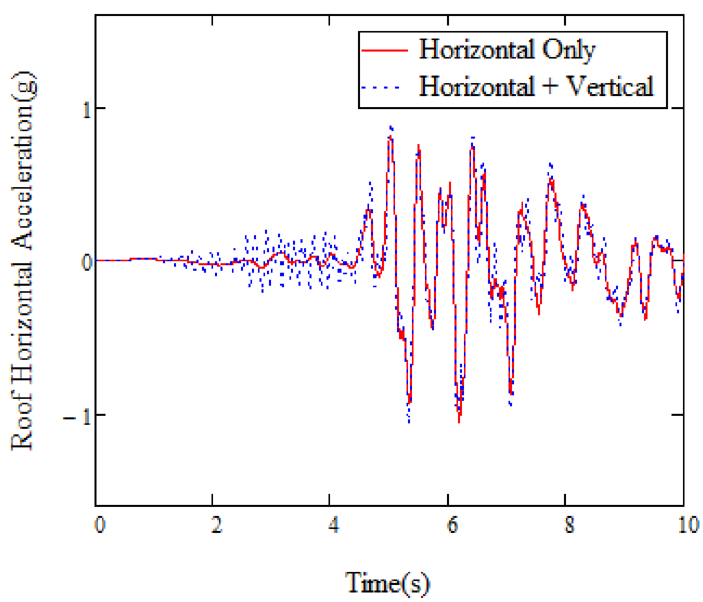

Figure 4-34: Detail of Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Girder Model under NF-14-1

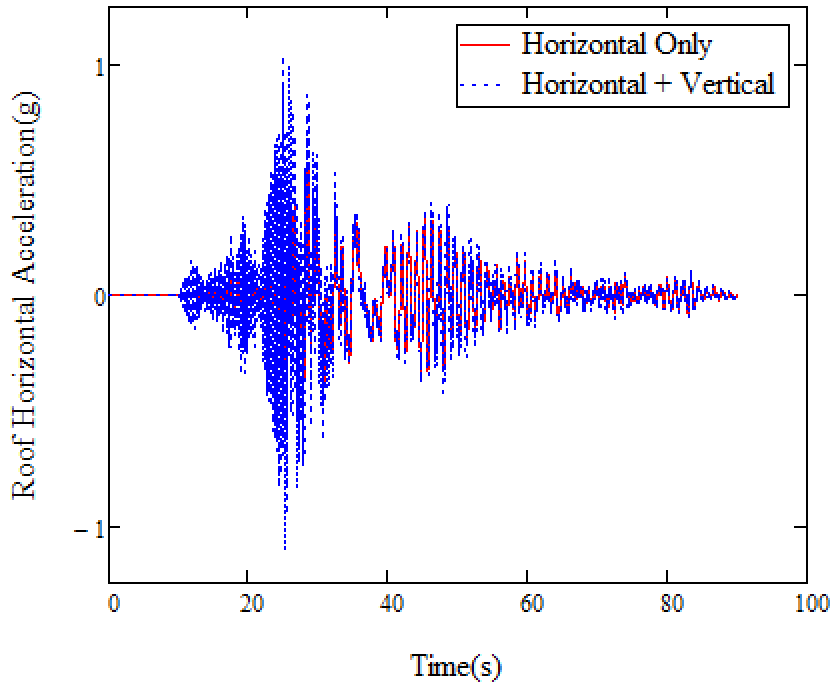

Figure 4-35: Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Beam Model under NF-28-1

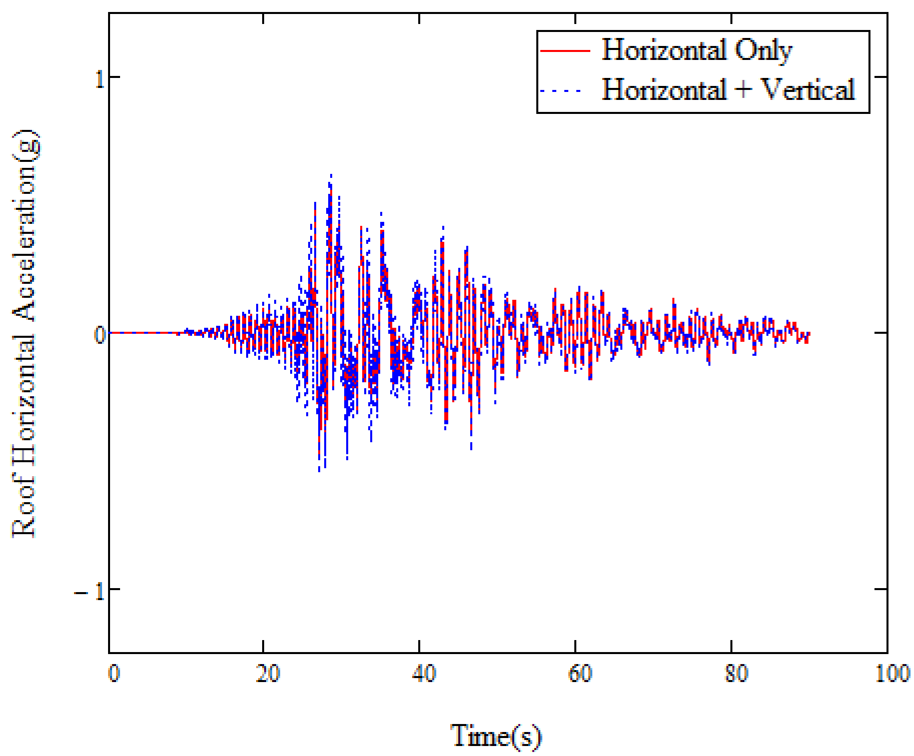

Figure 4-36: Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Girder Model under NF-28-1

Figure 4-37 and Figure 4-38 shows the detail of time history response of roof absolute horizontal acceleration from 15s to 35s in the six-story moment frame as beam model and the six-story moment frame as girder model under NF-28 respectively.

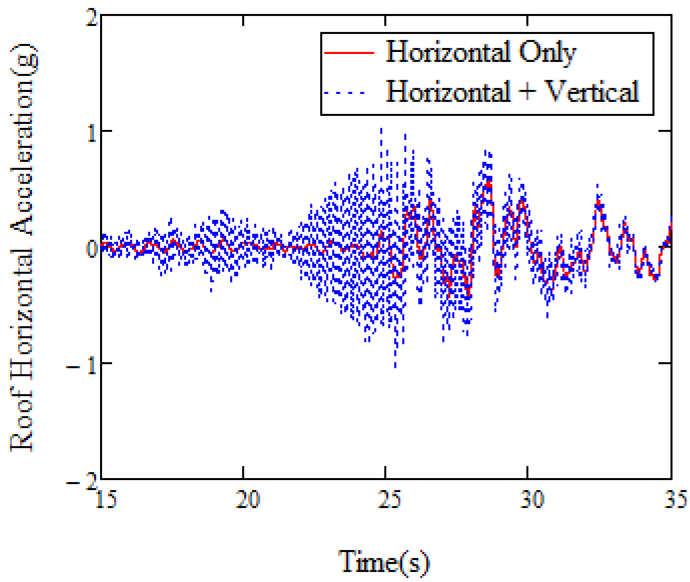

Figure 4-37: Detail of Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Beam Model under NF-28-1

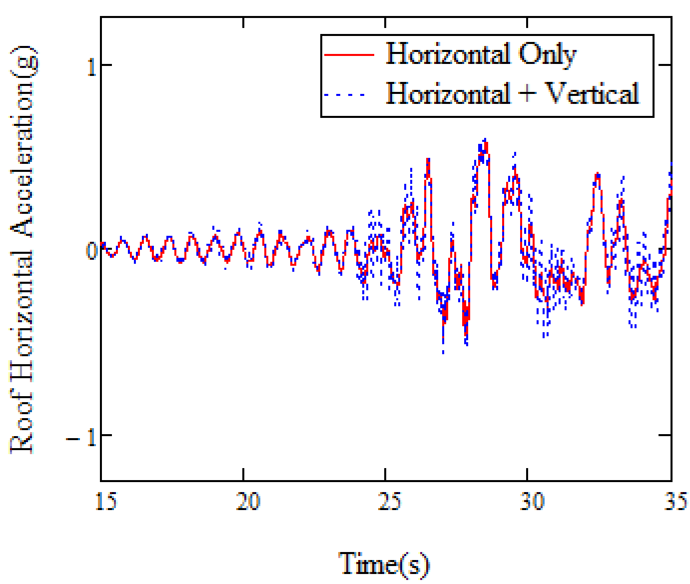

Figure 4-38: Detail of Time History Response of Roof Absolute Horizontal Acceleration in the Six-Story Moment Frame as Girder Model under NF-28-1

The impact of vertical ground motion on the six-story moment frame as beam model is much more significant than that on the six-story moment frame as girder model. However, the effect of vertical ground motion is similar to each other in these two models. After adding the vertical acceleration to the model, the roof horizontal acceleration starts to oscillate based on the roof acceleration pattern when the structure is only subjected to the horizontal ground motion. That means the global frequency of roof horizontal acceleration won't change because of the vertical ground motion. The difference between these two results is the amplitude of oscillation in six-story moment frame as beam model is much larger than that in the six-story moment frame as girder model. The reason the amplitude of oscillation is larger in the six-story moment frame as beam model is because the first vertical mode period which is 0.1016s is close to the first peak value in the response spectrum of earthquake NF-28-1 including horizontal and vertical acceleration. Figure 4-39 and Figure 4-40 show the response spectrums with 2% damping ratio of NF-28-1 (Horizontal Only) and NF-28-1 (Horizontal + Vertical) for the six-story moment frame as beam model, respectively. The X axis represents the period of structure in seconds and the Y axis represents the roof horizontal acceleration in units of g.

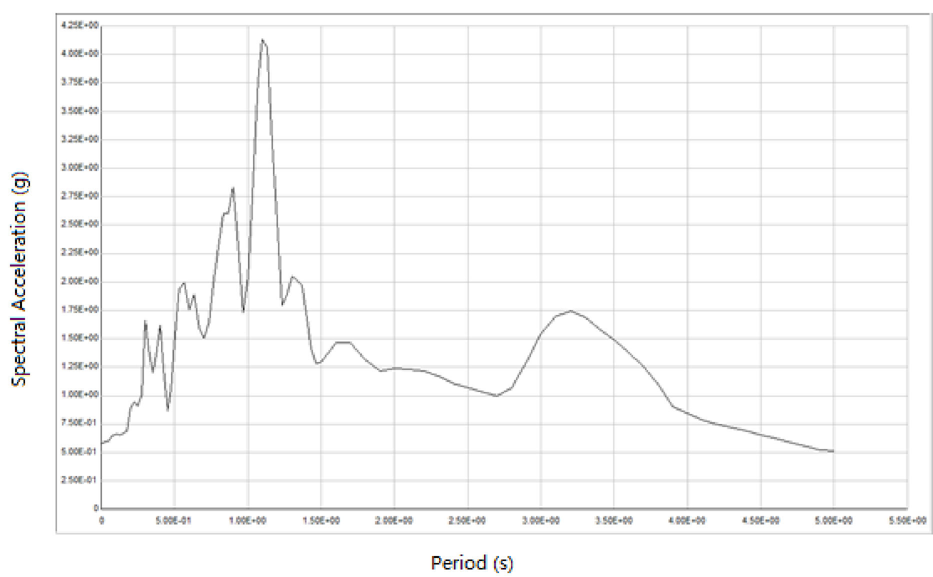

Figure 4-39: Response Spectrums with 2% Damping Ratio under NF-28-1 (Horizontal Only) in the Six-Story Moment Frame as Beam Model

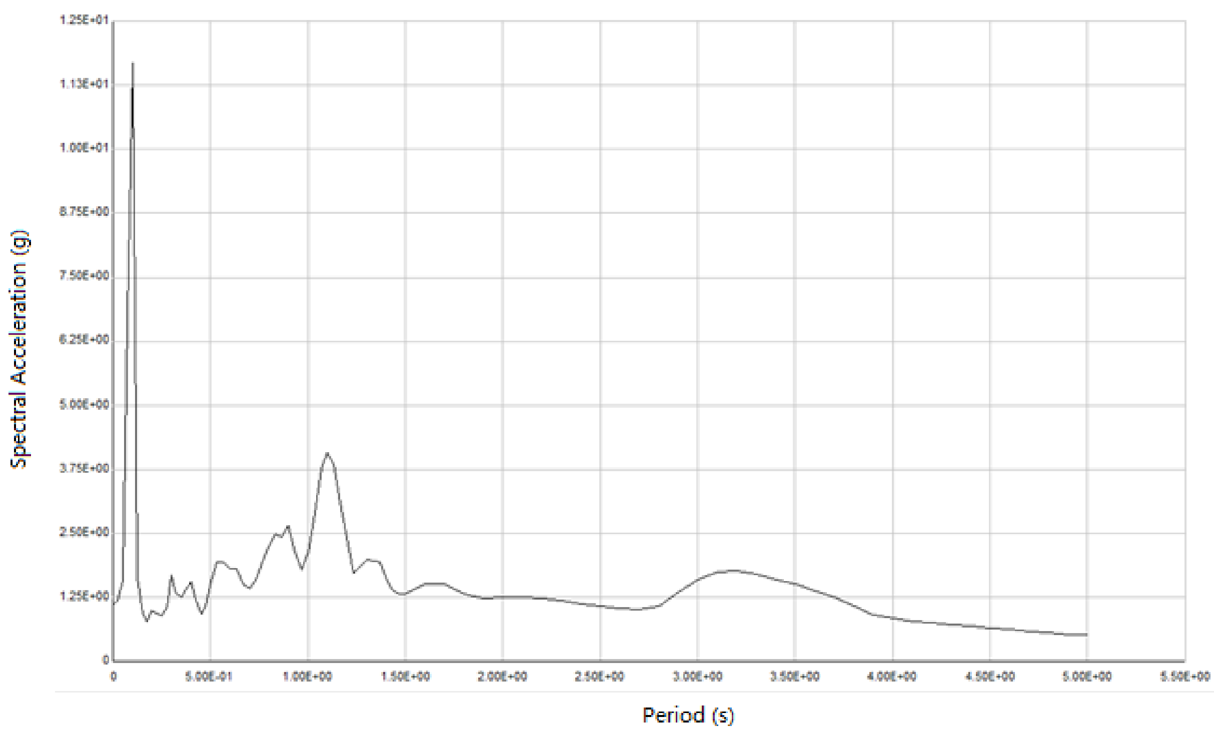

Figure 4-40: Response Spectrums with 2% Damping Ratio under NF-28-1 (Horizontal + Vertical) in the Six-Story Moment Frame as Beam Model

Since maximum roof acceleration in the six-story moment frame as beam model is very different from the remaining models, a double check has been done in this study. The six-story story moment frame as girder model was chosen to become the basis which would be modified to become six-story moment frame as beam model because they are very close to each other. Figure 4-41 shows the response spectrum with 2% damping ratio under NF28-1 (Horizontal + Vertical) in the six-story moment frame as girder model. Then the beams and columns were changed to be same with those in the six-story moment frame as beam model. Figure 4-42 shows the response spectrum with 2% damping ratio under NF28-1 (Horizontal + Vertical) in the new model after changing the beam and column size. Then the reduced beam section and plastic hinge in the column were changed to be same with the six-story moment frame as beam model. Figure 4-43 shows the response spectrum with 2% damping ration under NF28-1 (Horizontal +

Vertical) in the new model after changing the reduced beam section and plastic hinge in the columns.

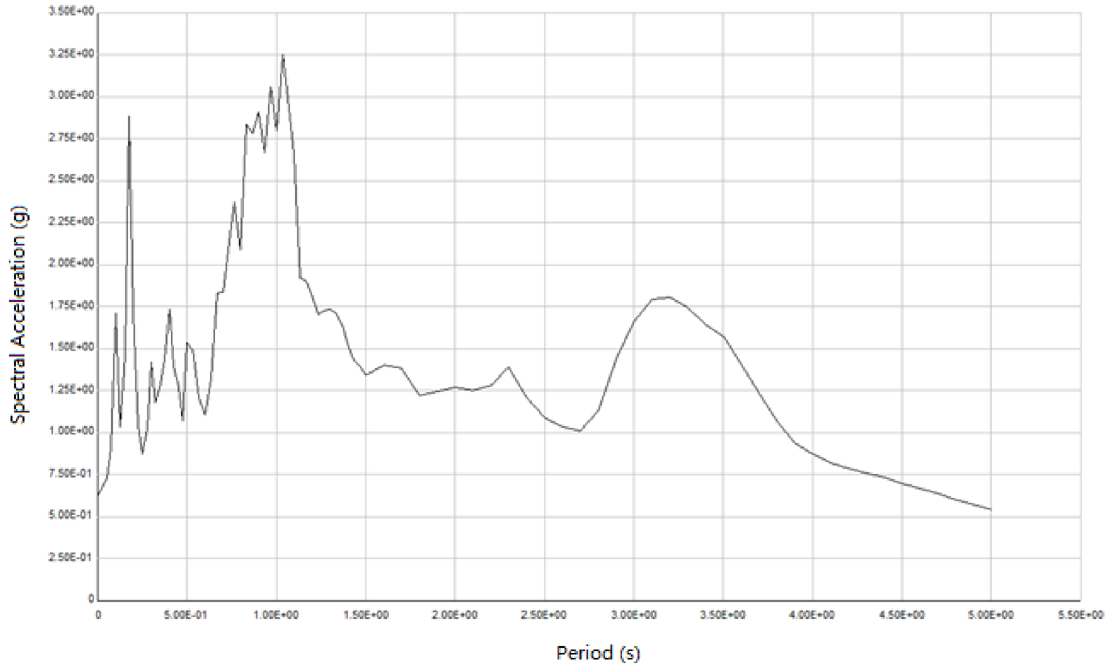

Figure 4-41: Response Spectrums with 2% Damping Ratio under NF-28-1 (Horizontal + Vertical) in the Six-Story Moment Frame as Girder Model

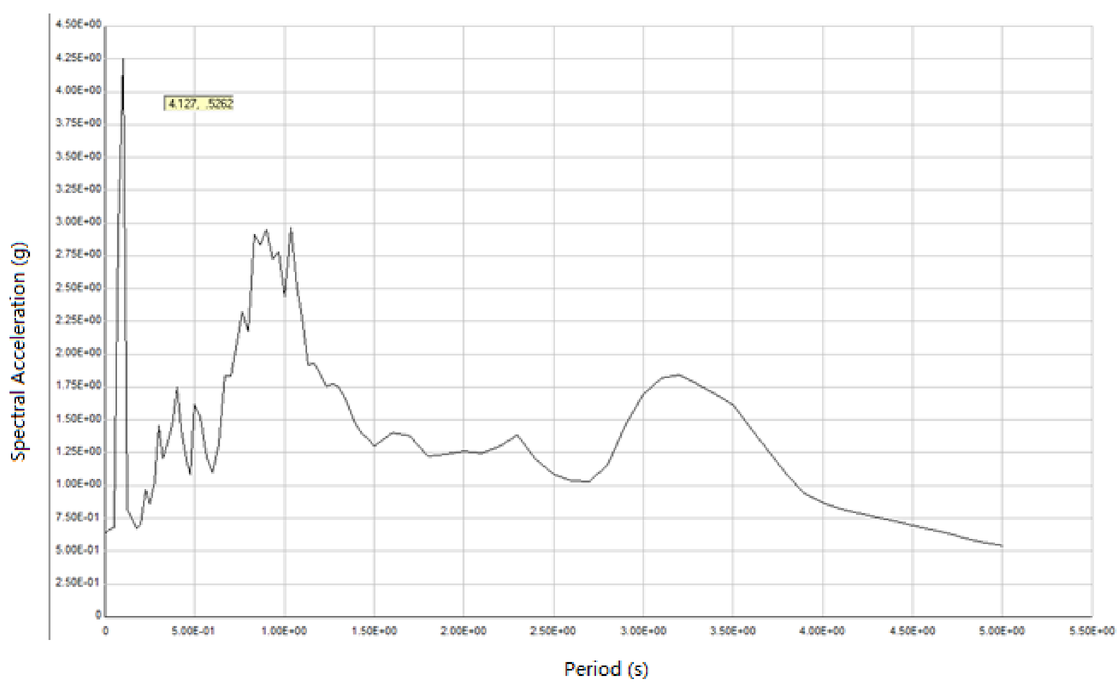

Figure 4-42: Response Spectrums with 2% Damping Ratio under NF-28-1 (Horizontal + Vertical) in the New Model after Changing the Beam and Column Size

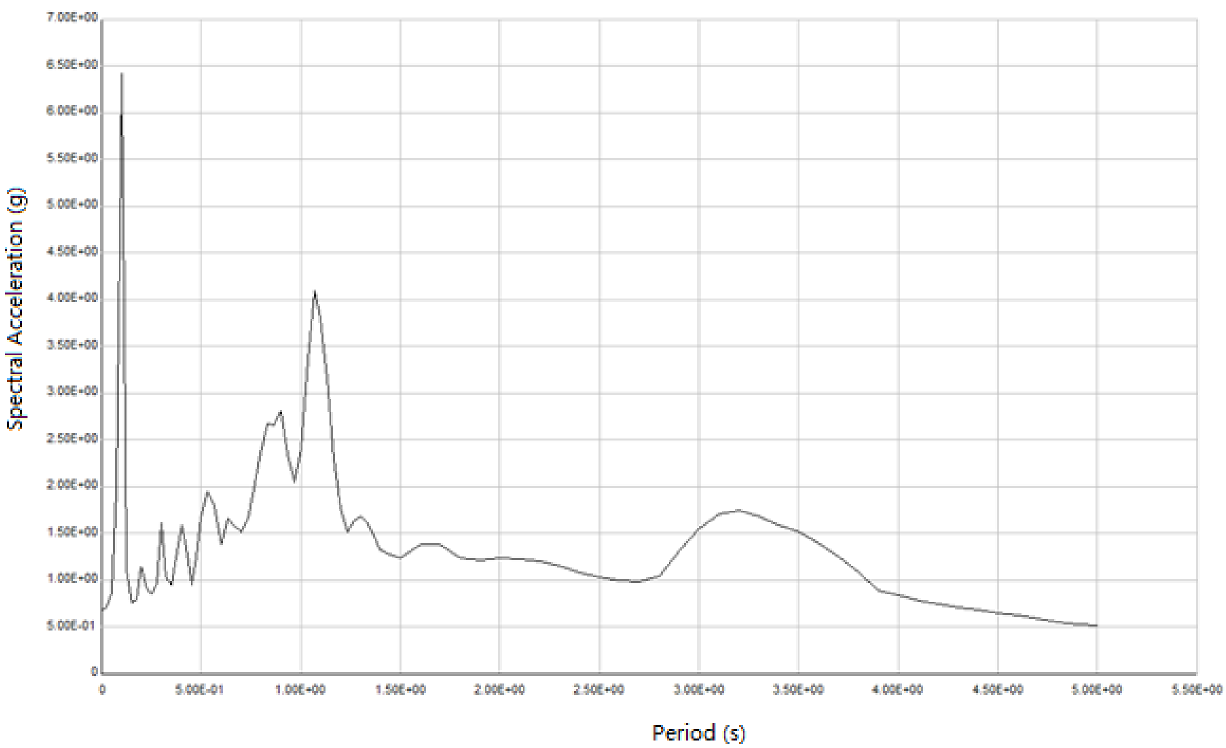

Figure 4-43: Response Spectrums with 2% Damping Ratio under NF-28-1 (Horizontal + Vertical) in the New Model after Changing the RBS and Plastic Hinge in the Column

According to the results above, the response spectrum with 2% damping ratio under NF-28-1 (Horizontal + Vertical) in the six-story moment frame as girder model is changing to become similar with that in the six-story moment frame as beam model when changing the elements to become the six-story moment frame as beam model. That means the six-story moment frame as beam model is correct. It also highlights the potential for vertical ground motions to affect the horizontal acceleration significantly in the case where fundamental frequencies are similar.

4.4.2 Vertical Acceleration

The data of maximum vertical acceleration for all the stories during the earthquake is calculated according to the data of nodal vertical absolute acceleration at the beam midspan for each floor. The vertical accelerations for the exterior span and interior span are collected separately. Figure 4-44 shows the nodes which are chosen to represent the exterior spans and interior spans in the six-story moment frame as beam model and the nodes in the remaining models have the same location with this example model. Tables 4-27 through 4-30 show the average value of maximum vertical acceleration for each story under the forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration, respectively.

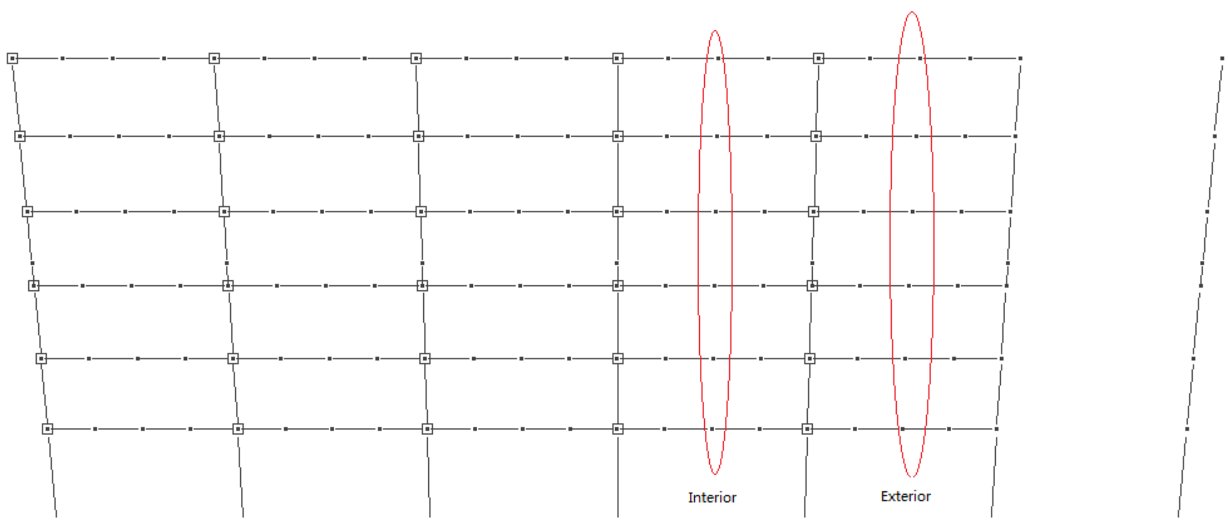

Figure 4-44: The Nodes Chosen to Represent Exterior Span and Interior Span to Measure Vertical Acceleration

Table 4-27: Summary of Vertical Acceleration for the Three-Story Moment Frame as Beam Model

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	0.827	3.607	77.1%	0.862	4.174	79.3%
2	0.962	3.932	75.5%	0.870	5.479	84.1%
3	1.104	3.727	70.4%	1.113	4.179	73.4%

Table 4-28: Summary of Vertical Acceleration for the Three-Story Moment Frame as Girder Model

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	0.768	2.969	74.1%	0.565	3.472	83.7%
2	0.777	3.136	75.2%	0.592	3.499	83.1%
3	1.942	3.123	37.8%	0.816	3.717	78.0%

Table 4-29: Summary of Vertical Acceleration for the Three-Story Braced Frame with Chevron Configuration

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	3.717	5.819	36.1%	0.277	2.613	89.4%
2	3.956	5.477	27.8%	0.467	3.052	84.7%
3	3.584	6.179	42.0%	0.540	3.378	84.0%

Table 4-30: Summary of Vertical Acceleration for the Three-Story Braced Frame with Single Diagonal Configuration

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	1.145	3.344	65.8%	0.379	2.653	85.7%
2	1.967	4.539	56.7%	0.650	3.218	79.8%
3	1.967	4.295	54.2%	0.937	3.643	74.3%

Tables 4-31 through 4-34 show the average value of maximum vertical acceleration for each story under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-31: Summary of Vertical Acceleration for the Six-Story Moment Frame as Beam Model

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	0.725	3.953	81.7%	0.942	3.857	75.6%
2	0.657	4.729	86.1%	0.754	4.699	84.0%
3	0.652	5.448	88.0%	0.668	6.736	90.1%
4	0.698	9.175	92.4%	0.704	8.665	91.9%
5	0.974	8.192	88.1%	0.780	10.906	92.9%
6	1.167	5.722	79.6%	1.085	10.399	89.6%

Table 4-32: Summary of Vertical Acceleration for the Six-Story Moment Frame as Girder Model

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	0.600	2.405	75.1%	0.505	3.322	84.8%
2	0.628	2.835	77.9%	0.528	3.318	84.1%
3	0.548	2.974	81.6%	0.469	3.700	87.3%
4	0.554	3.360	83.5%	0.524	4.165	87.4%
5	0.766	3.591	78.7%	0.564	5.698	90.1%
6	1.638	3.325	50.7%	0.864	4.567	81.1%

Table 4-33: Summary of Vertical Acceleration for the Six-Story Braced Frame with Chevron Configuration

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	2.889	5.485	47.3%	0.185	3.200	94.2%
2	1.790	3.597	50.3%	0.229	3.234	92.9%
3	1.675	4.030	58.4%	0.305	3.377	91.0%
4	1.813	4.161	56.4%	0.380	3.525	89.2%
5	1.550	3.834	59.6%	0.446	3.659	87.8%
6	1.470	4.125	64.4%	0.431	3.751	88.5%

Table 4-34: Summary of Vertical Acceleration for the Six-Story Braced Frame with Single Diagonal Configuration

Story	Vertical Acceleration					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	0.187	3.440	94.6%	0.208	4.575	95.5%
2	0.222	3.139	92.9%	0.252	3.762	93.3%
3	0.191	3.015	93.7%	0.213	3.545	94.0%
4	0.149	2.898	94.9%	0.165	3.282	95.0%
5	0.125	2.826	95.6%	0.123	3.043	96.0%
6	0.101	2.769	96.4%	0.082	2.897	97.2%

The average values of vertical acceleration for each story increase significantly for all the models in the study when the vertical ground motion is included. The impact of vertical ground motion on both the exterior and interior span is direct and significant which means vertical ground motions in this study dominate the vertical acceleration in the beam midspan. Figure 4-45 shows the time history response of vertical acceleration on the exterior span of second floor in the six-story moment frame as beam model under NF-28. Figure 4-46 and 4-47 show the result separately to see them clearly.

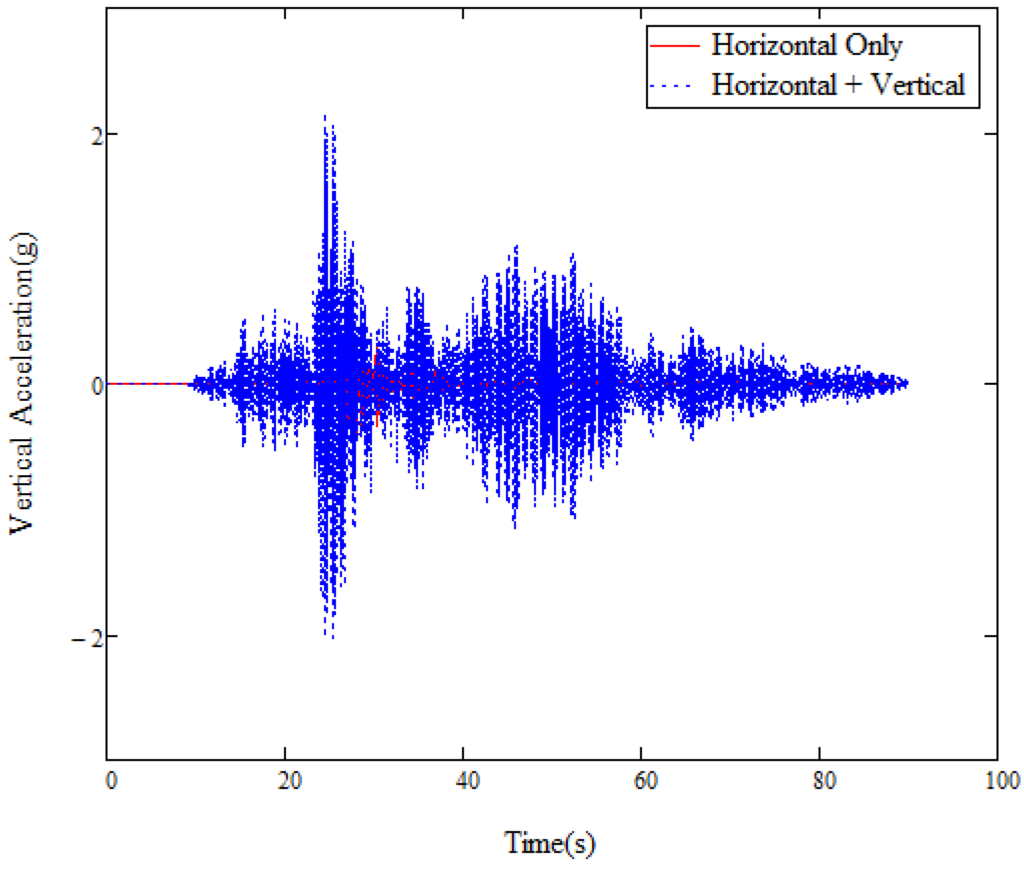

Figure 4-45: Time History Response of Vertical Acceleration in the Six-Story Moment Frame as Beam Model under NF-28

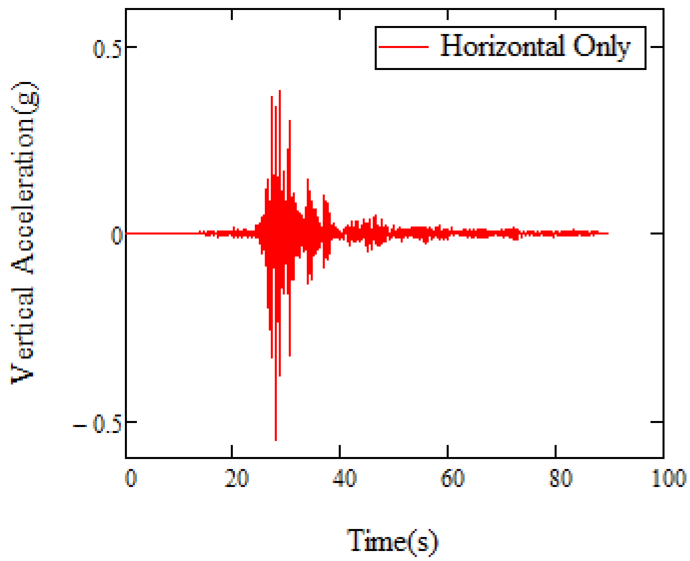

Figure 4-46: Time History Response of Vertical Acceleration in the Six-Story Moment Frame as Beam Model under NF-28 (Horizontal Only)

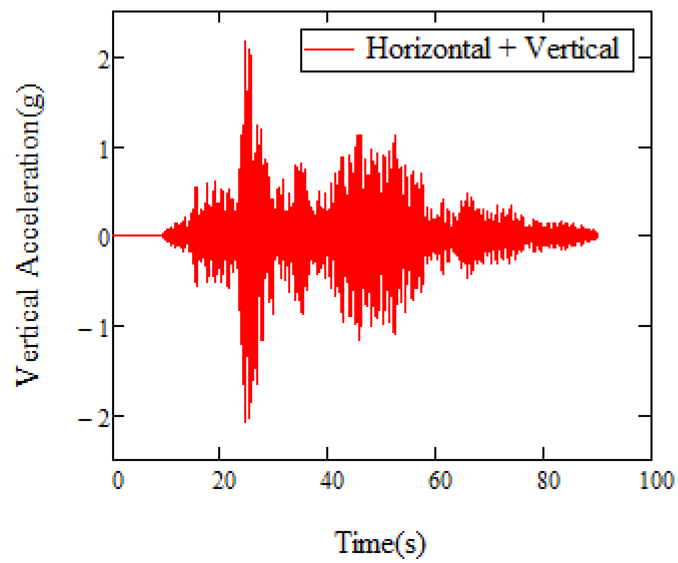

Figure 4-47: Time History Response of Vertical Acceleration in the Six-Story Moment Frame as Beam Model under NF-28 (Horizontal + Vertical)

The maximum vertical acceleration of the beam midspan comes earlier in the response when the vertical ground motions are added to the structures because the speed of vertical wave in the earthquake is faster than that of horizontal wave. The peak vertical ground motion will hit the structure earlier than the horizontal ground motion.

4.5 Vertical Deflection in the Beam

In the earthquake engineering, beam midspan deflection is not typically a concern. However, in the structural analysis and design and nonstructural view, it is necessary to predict the deflection of the beam midspan because many applications have limitations on the amount of deflection that can be tolerated. Additionally, damage to non-structural components such as ceiling tiles, sprinkler systems and piping is connected to the midspan deformation. The vertical deflection for the exterior span and interior span are collected separately. The nodes chosen to represent the exterior spans and interior spans are the same as those chosen to get the vertical acceleration. Tables 4-25 through 4-38 show the average value of maximum vertical deflection for each story under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration respectively. All vertical deflections in this study are normalized by the beam span.

Table 4-35: Summary of Vertical Deflection for the Three-Story Moment Frame as Beam Model

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	0.732	0.891	21.8%	0.644	0.861	33.7%
2	0.791	0.958	21.1%	0.687	0.976	42.1%
3	2.840	3.514	23.7%	2.896	3.512	21.3%

Table 4-36: Summary of Vertical Deflection for the Three-Story Moment Frame as Girder Model

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	3.818	4.535	18.8%	1.340	2.059	53.7%
2	4.168	4.849	16.4%	1.350	2.164	60.3%
3	7.580	8.379	10.5%	3.575	5.193	45.2%

Table 4-37: Summary of Vertical Deflection for the Three-Story Braced Frame with Chevron Configuration

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	3.639	3.667	0.78%	6.562	11.890	81.2%
2	4.422	4.391	-0.69%	6.859	12.128	76.8%
3	5.417	5.553	2.51%	7.157	12.808	79.0%

Table 4-38: Summary of Vertical Deflection for the Three-Story Braced Frame with Single Diagonal Configuration

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	6.403	13.123	105.0%	6.663	11.673	75.2%
2	6.376	13.318	108.9%	6.893	11.928	73.0%
3	6.604	15.976	141.9%	7.382	12.650	71.4%

Figures 4-48 through 4-51 show the line chart of average value of maximum normalized vertical deflection for each story under two different loading case for each selected earthquake in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration, respectively.

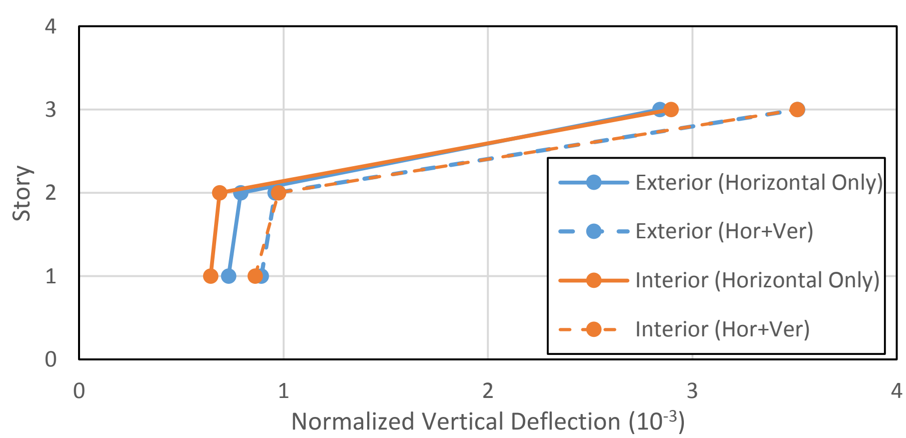

Figure 4-48: Max Normalized Vertical Deflection for Three-Story Moment Frame as Beam Model

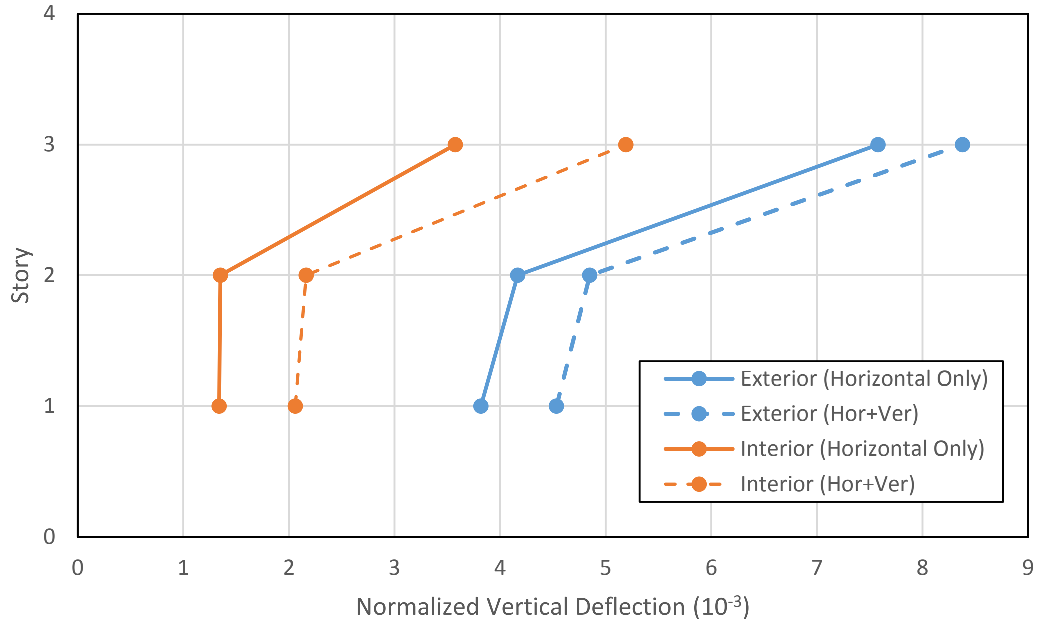

Figure 4-49: Max Normalized Vertical Deflection for Three-Story Moment Frame as Girder Model

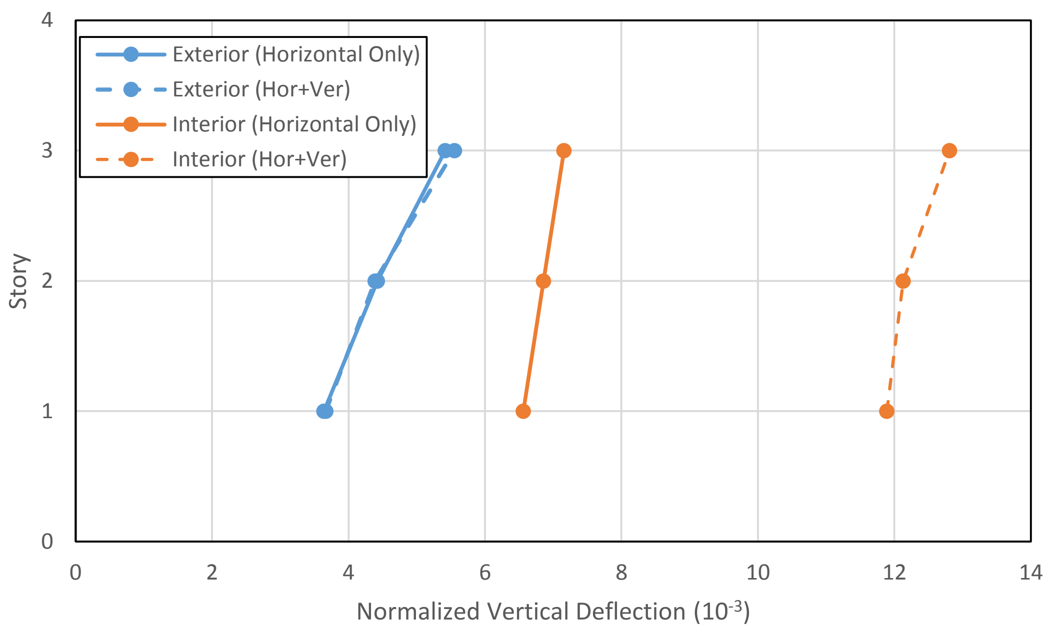

Figure 4-50: Max Normalized Vertical Deflection for Three-Story Braced Frame with Chevron Configuration

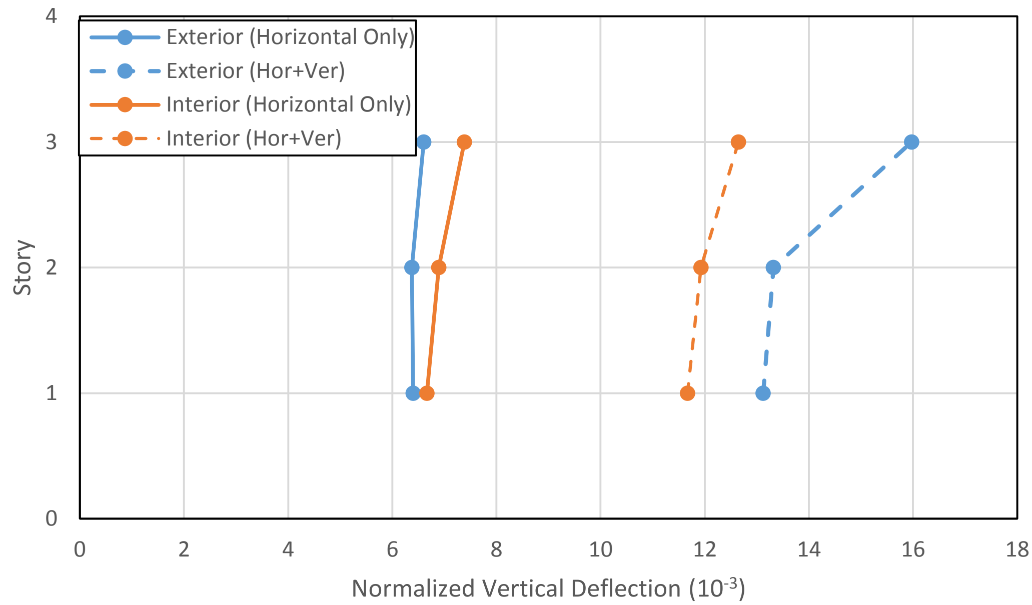

Figure 4-51: Max Normalized Vertical Deflection for Three-Story Braced Frame with Single Diagonal Configuration

Tables 4-39 through 4-42 show the average value of maximum normalized vertical deflection for each story in the exterior and interior span under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration respectively.

Table 4-39: Summary of Vertical Deflection for the Six-Story Moment Frame as Beam Model

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	3.101	3.133	1.02%	0.760	0.886	16.5%
2	3.143	3.180	1.17%	0.784	1.038	32.5%
3	3.421	3.546	3.63%	0.921	1.441	56.5%
4	3.847	4.473	16.3%	1.271	2.188	72.1%
5	4.234	4.952	17.0%	1.535	3.000	95.5%
6	5.887	6.608	12.3%	3.106	5.391	73.5%

Table 4-40: Summary of Vertical Deflection for the Six-Story Moment Frame as Girder Model

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	3.152	3.484	10.5%	1.069	1.405	31.4%
2	3.236	3.550	9.68%	1.130	1.452	28.5%
3	3.357	3.996	19.0%	1.593	1.924	20.8%
4	3.345	3.969	18.6%	1.746	2.059	17.9%
5	3.977	5.162	29.8%	2.806	3.147	12.1%
6	7.249	8.191	13.0%	5.505	5.908	7.33%

Table 4-41: Summary of Vertical Deflection for the Six-Story Braced Frame with Chevron Configuration

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	5.111	5.723	12.0%	5.869	10.850	84.9%
2	5.207	5.603	7.61%	5.840	10.719	83.6%
3	4.978	5.341	7.31%	5.894	10.797	83.2%
4	5.419	5.649	4.25%	5.940	10.863	82.9%
5	5.660	5.913	4.46%	5.978	10.903	82.4%
6	6.035	6.648	10.2%	6.269	12.944	107%

Table 4-42: Summary of Vertical Deflection for the Six-Story Braced Frame with Single Diagonal Configuration

Story	Normalized Vertical Deflection (10^{-3})					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Difference	Horizontal Only	Horizontal + Vertical	Difference
1	6.237	16.073	157.7%	6.275	12.318	96.3%
2	5.955	11.559	94.1%	5.977	10.384	73.7%
3	5.931	11.530	94.4%	5.939	10.349	74.3%
4	5.876	11.479	95.4%	5.890	10.291	74.7%
5	5.854	11.416	95.0%	5.843	10.228	75.1%
6	5.921	11.835	99.9%	5.869	10.380	76.9%

Figures 4-52 through 4-55 show the line chart of average value of maximum normalized vertical deflection for each story under two different loading case for each selected earthquake in the six-story moment frame as beam model, six-story moment as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration respectively.

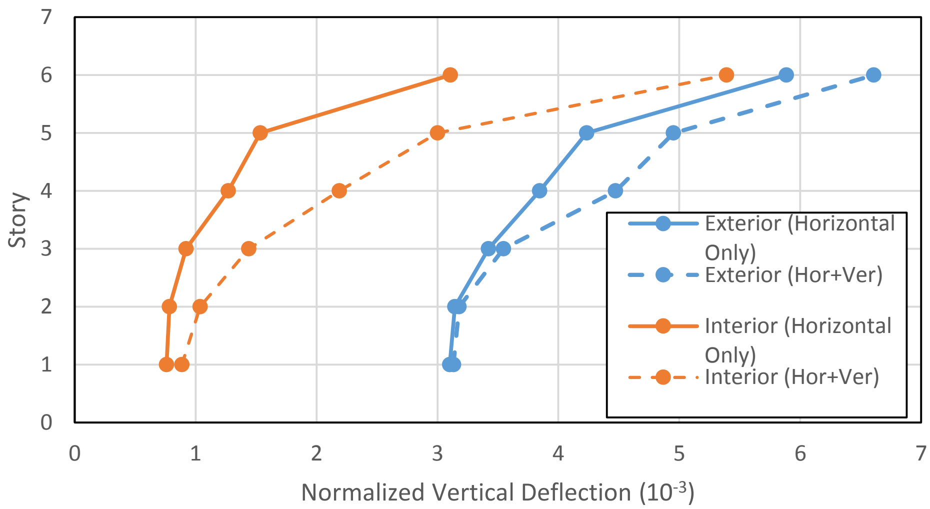

Figure 4-52: Max Normalized Vertical Deflection for Six-Story Moment Frame as Beam Model

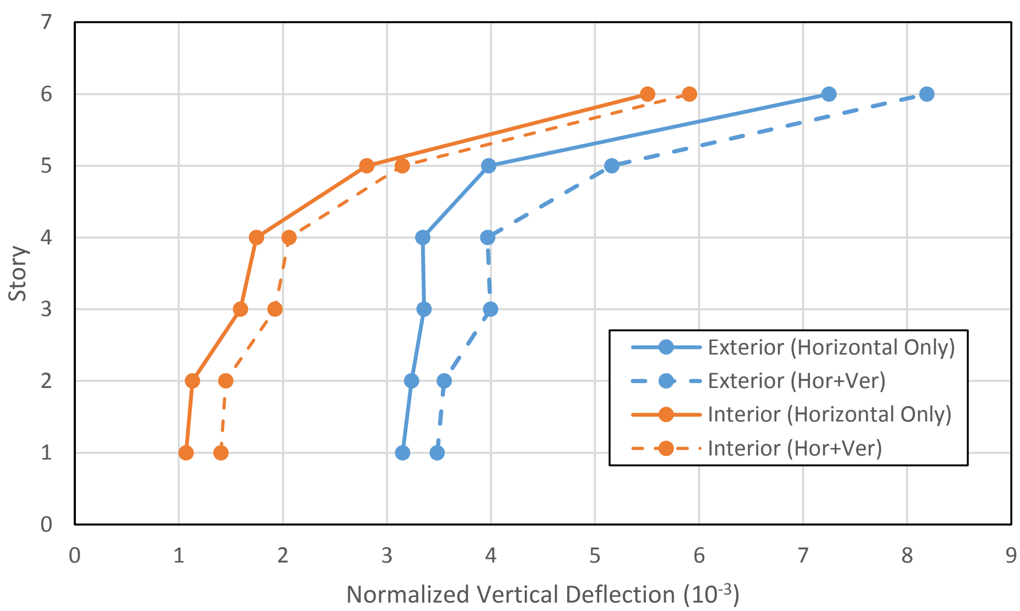

Figure 4-53: Max Normalized Vertical Deflection for Six-Story Moment Frame as Girder Model

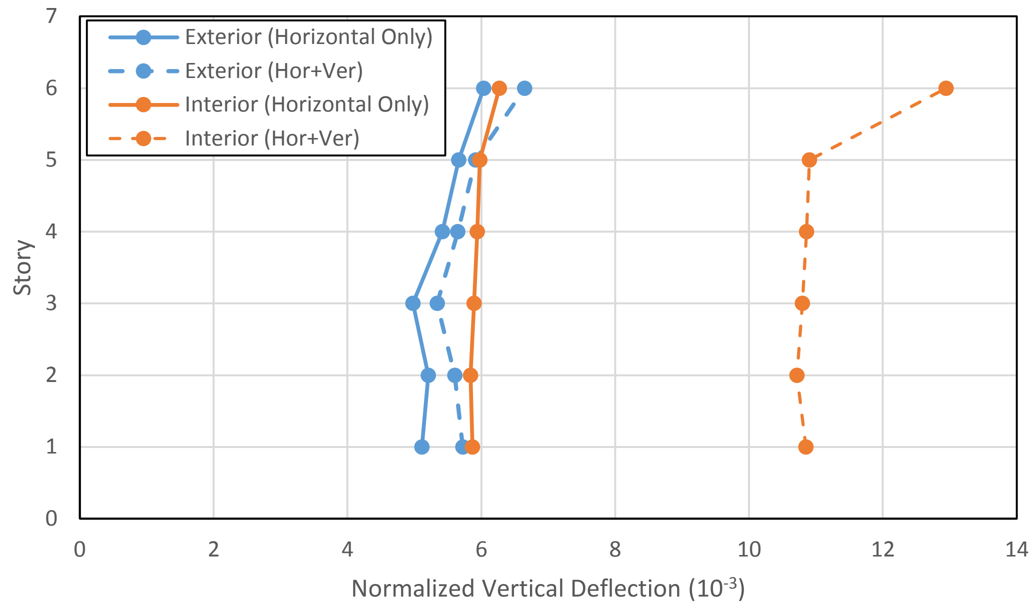

Figure 4-54: Max Normalized Vertical Deflection for Six-Story Braced Frame with Chevron Configuration

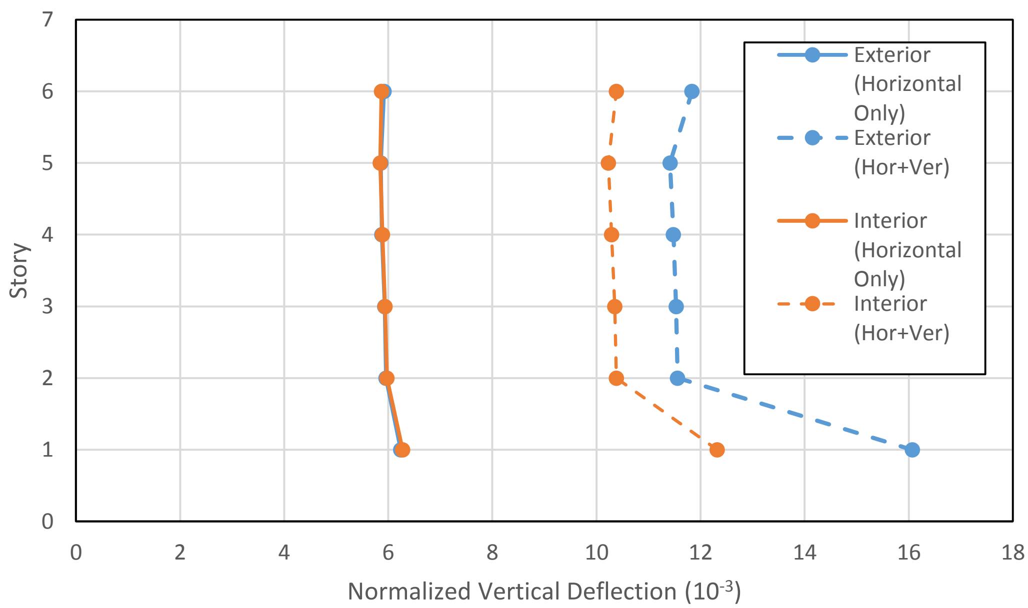

Figure 4-55: Max Normalized Vertical Deflection for Six-Story Braced Frame with Single Diagonal Configuration

The average values of vertical deflection for each story increase significantly when vertical ground motion is included for either exterior span or interior span in all the models except the exterior span in the three-story and six-story braced frame with chevron configuration in this study. This result is very similar with the result of vertical acceleration when the vertical ground motion is added to the structures. The reason the impact of vertical ground motion on the exterior span in the braced frames is not significant because the nodes chosen are connected with the braces directly, which prevents vertical deflection of the beam midspan. What's more, the difference between the two loading conditions in the exterior span of the six-story braced frame with chevron configuration is larger than that in the three-story braced frame with chevron configuration. That means the impact of vertical ground motion on beam deflection is increased in the six-story braced frames compared with three-story braced frame. Again, it's clear that the impact of vertical ground motion on the either exterior span or interior span is direct and significant which means vertical ground motions in this study dominate the vertical deflection in the beam midspan in both the exterior and interior span. The impact of vertical ground motion on the beam deflection of all the stories in the braced frames is larger because all these beams are controlled by gravity load according to the current design procedure. Figure 4-56 and Figure 4-57 show the time history response of vertical deflection on the interior span of sixth floor in the six-story moment frame as beam model and six-story braced frame with single diagonal configuration under NF-28-1, respectively.

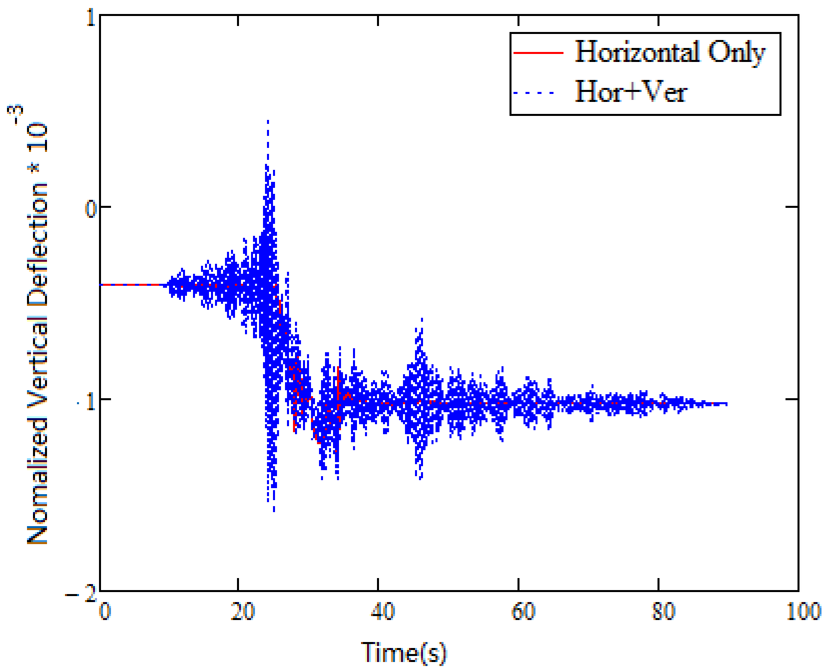

Figure 4-56: Time History Response of Vertical Deflection of the Sixth Floor in the Six-Story Moment Frame as Beam Model under NF-28-1

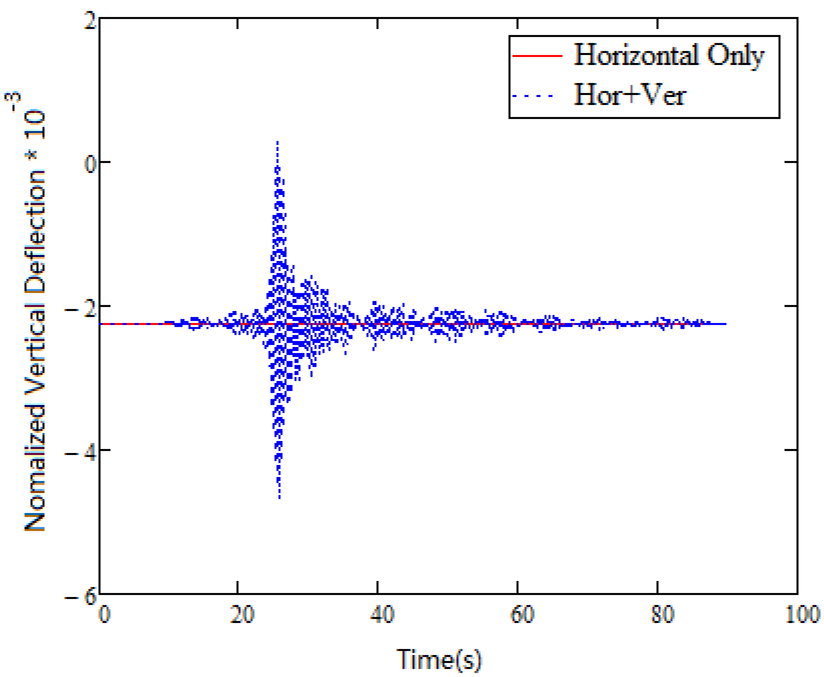

Figure 4-57: Time History Response of Vertical Deflection of the Sixth Floor in the Six-Story Braced Frame with Single Diagonal Configuration Model under NF-28-1

4.6 Reduced Beam Section (RBS) Rotation and BRB Deformation

After the Northridge earthquake in 1994 and the Kobe earthquake in 1995, a lot of research and testing efforts have been put into finding better methods to design and construct seismic resistant steel frames. The reduced beam section connection is one of the most popular and most economical type amongst post-Northridge and -Kobe connections. The effectiveness of reduced beam section (RBS) connections was widely investigated following the previously mentioned earthquakes. In addition, for the braced frames, the BRBF system has become more and more popular recently due to a number of structural performance advantages over conventional braced frames. In this study, the reduced beam section is one of the major inelastic elements which will absorb the energy from earthquake through the inelastic deformation in the moment frames while a buckling restrained brace is the major element which can absorb energy through inelastic deformation in the braced frames.

The inelastic deformation of the exterior span and interior span are collected separately for the reduced beam sections and buckling restrained braces. Figures 4-58 through 4-60 show the elements which are chosen to represent the exterior spans and interior spans in the six-story moment frame as beam model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration, respectively. The elements chosen in the remaining models are the same as shown in the figures. Tables 4-45 through 4-48 show the average value of maximum deformation for each story under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration respectively. All the axial deformations of buckling restrained braces in this study are normalized by the yield deformations of BRBs which are provided by Corebrace LLC in the design information.

Essentially this shows the ductility demand for the braces. Tables 4-43 and 4-44 show the yield strengths of BRBs for the three-story braced frames and six-story braced frames, respectively.

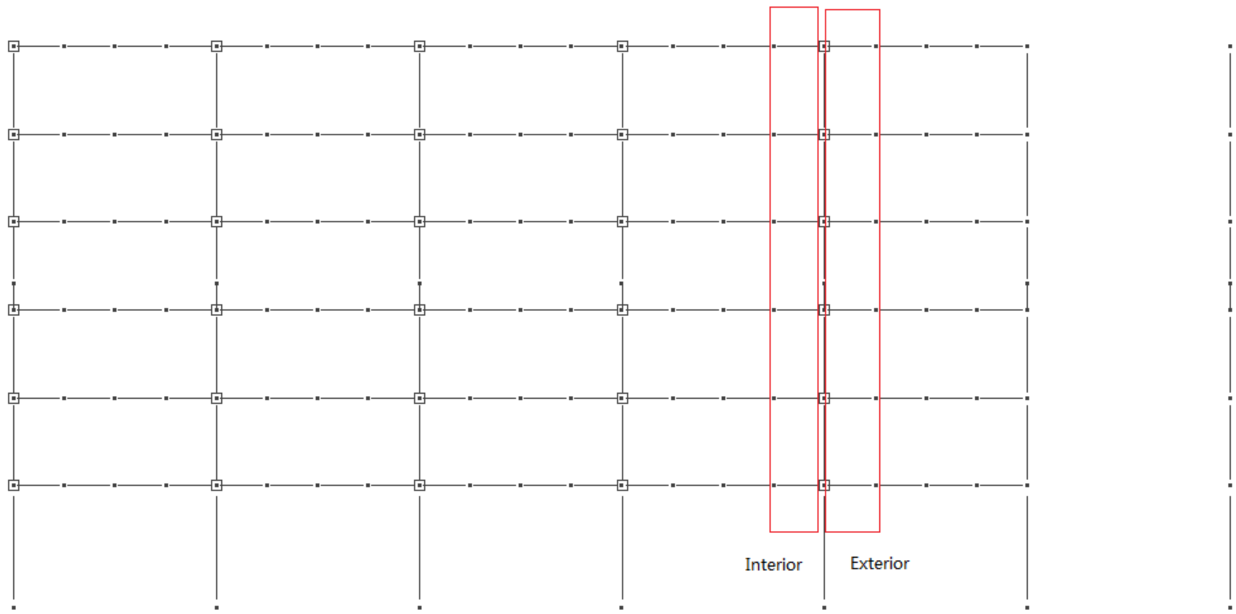

Figure 4-58: Elements Chosen to Represent Exterior Span and Interior Span to Measure RBS Deformation

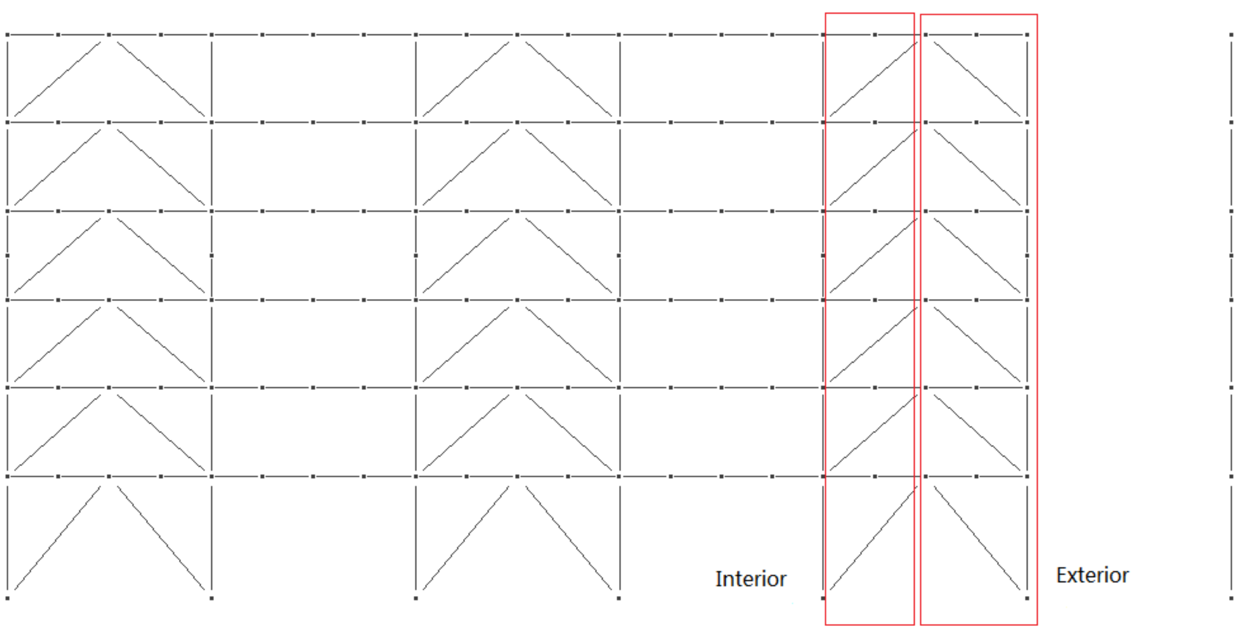

Figure 4-59: Elements Chosen to Represent Exterior Span and Interior Span to Measure BRB Deformation in Braced Frame with Chevron Configuration model

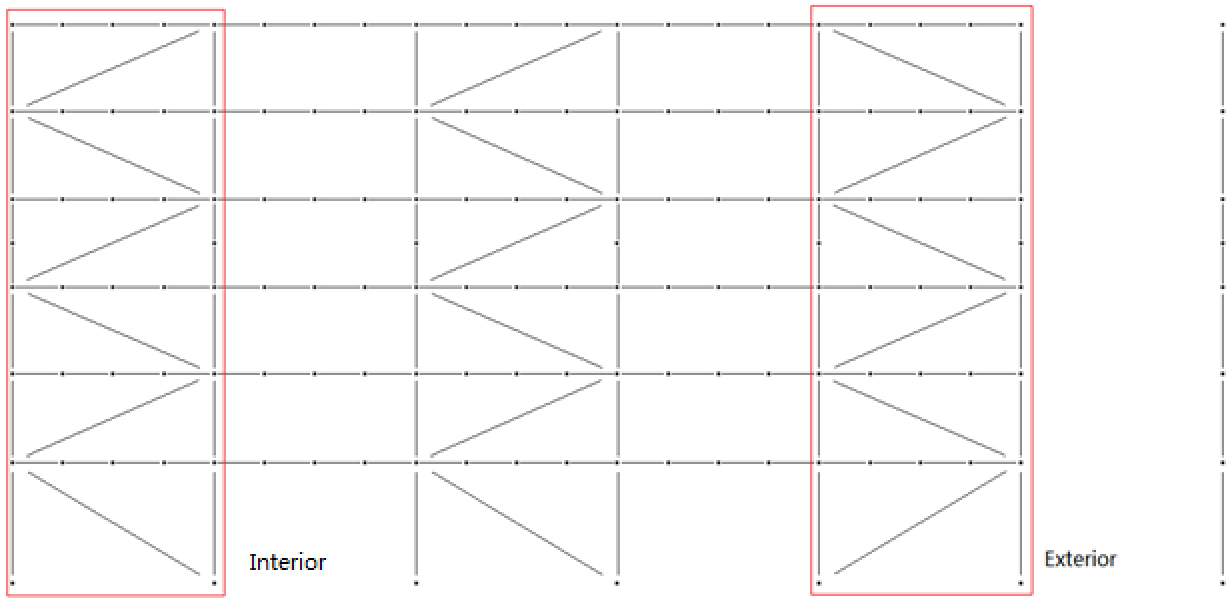

Figure 4-60: Elements Chosen to Represent Exterior Span and Interior Span to Measure BRB Deformation in Braced Frame with Single Diagonal Configuration model

Table 4-43: Yield Deformation of Brace in the Three-Story Braced Frame

Story	Yield Deformation(in)			
	Chevron		Single Diagonal	
	Tension	Compression	Tension	Compression
1	0.19	0.19	0.36	0.36
2	0.20	0.20	0.35	0.35
3	0.21	0.21	0.36	0.36

Table 4-44: Yield Deformation of Brace in the Six-Story Braced Frame

Story	Yield Deformation(in)			
	Chevron		Single Diagonal	
	Tension	Compression	Tension	Compression
1	0.27	0.27	0.42	0.43
2	0.22	0.22	0.37	0.37
3	0.22	0.22	0.37	0.37
4	0.21	0.21	0.37	0.37
5	0.22	0.22	0.38	0.38
6	0.20	0.20	0.38	0.38

Table 4-45: Summary of RBS Deformation for the Three-Story Moment Frame as Beam Model

Story	RBS Rotation (Radians)					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.032	0.031	2.87%	0.032	0.031	3.08%
2	0.039	0.038	3.05%	0.038	0.038	2.70%
3	0.054	0.054	4.25%	0.056	0.056	4.10%

Table 4-46: Summary of RBS Deformation for the Three-Story Moment Frame as Girder Model

Story	RBS Rotation (Radians)					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.035	0.039	16.0%	0.044	0.047	13.3%
2	0.044	0.046	17.0%	0.052	0.054	13.3%
3	0.055	0.057	10.9%	0.063	0.066	13.4%

Table 4-47: Summary of BRB Deformation for the Three-Story Braced Frame with Chevron Configuration Model

Story	BRB Normalized Deformation					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	17.7	17.7	3.81%	15.5	15.4	3.17%
2	14.4	14.2	3.73%	12.5	12.4	3.41%
3	12.9	13.0	5.19%	11.6	11.5	3.15%

Table 4-48: Summary of BRB Deformation for the Three-Story Braced Frame with Single Diagonal Configuration Model

Story	BRB Normalized Deformation					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	11.4	11.4	0.85%	11.4	11.4	0.93%
2	9.6	9.6	1.22%	9.7	9.7	1.36%
3	8.3	8.3	1.37%	8.4	8.4	1.20%

Figures 4-61 through 4-64 show the box plot of maximum RBS/BRB deformation for each story under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration. The first three box plots represent the exterior spans while the remaining represent the interior spans.

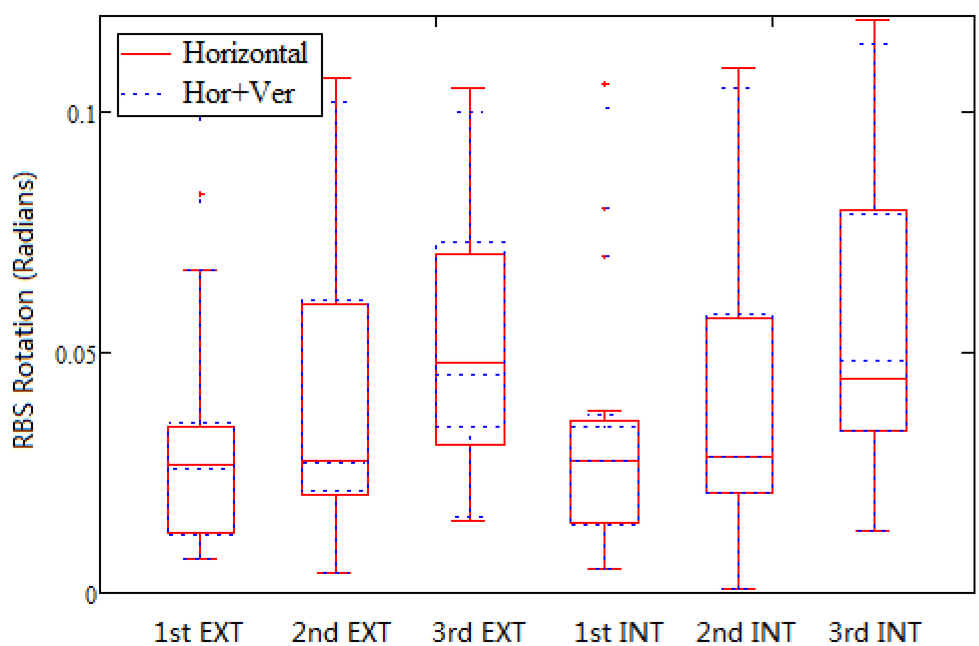

Figure 4-61: Box Plot of Max Story RBS Rotation for Three-story Moment Frame as Beam Model

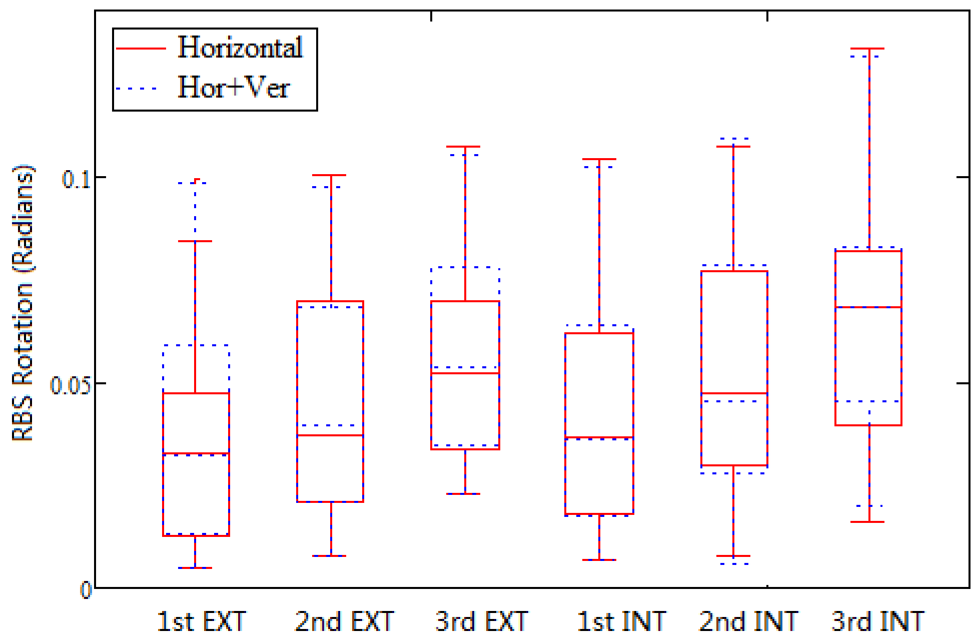

Figure 4-62: Box Plot of Max Story RBS Rotation for Three-story Moment Frame as Girder Model

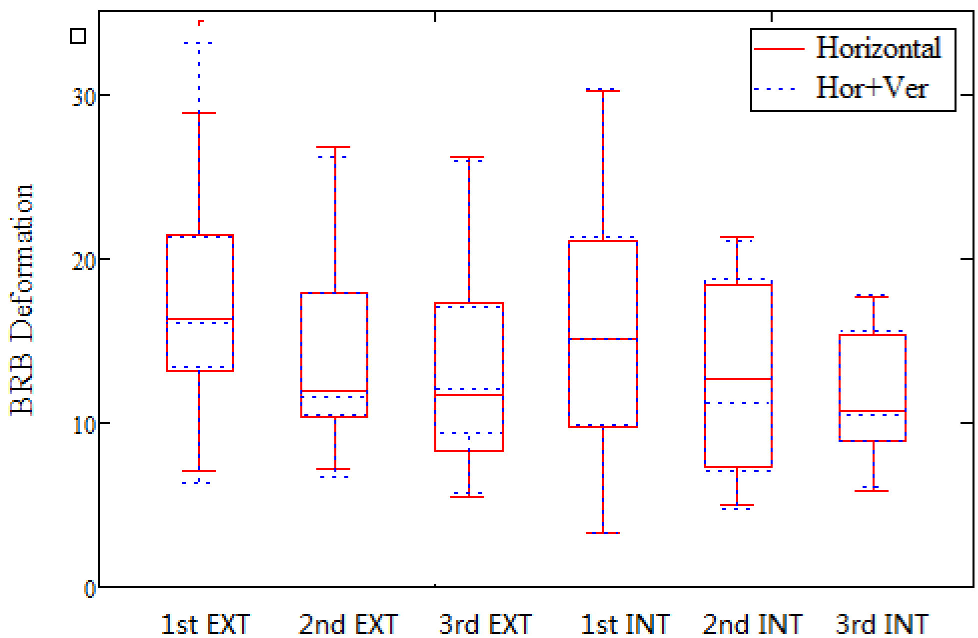

Figure 4-63: Box Plot of Max Story BRB Deformation for Three-story Braced Frame with Chevron Configuration Model

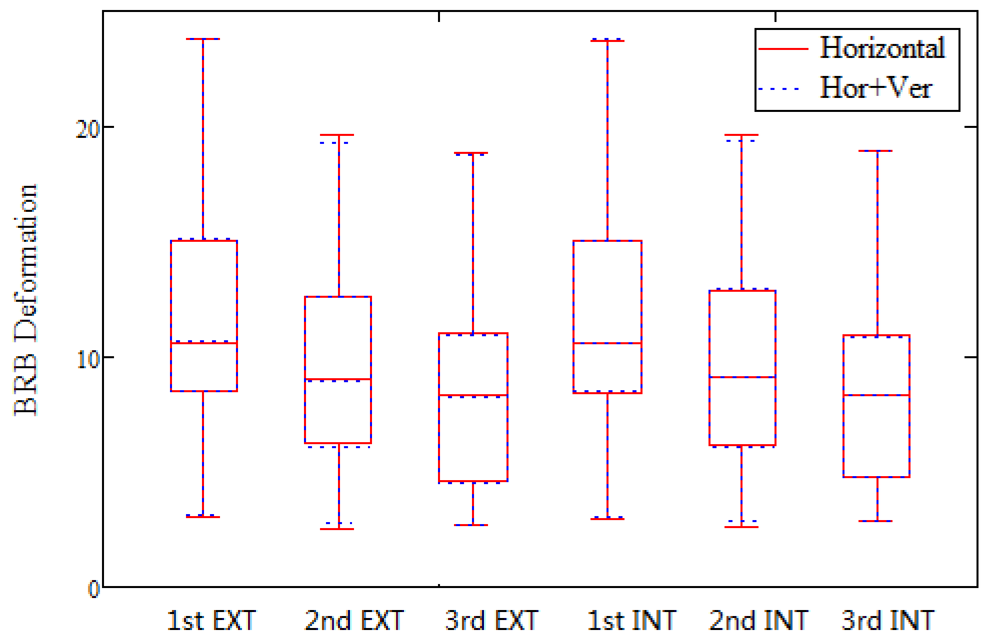

Figure 4-64: Box Plot of Max Story BRB Deformation for Three-story Braced Frame with Single Diagonal Configuration Model

Tables 4-49 through 4-52 show the average value of maximum RBS/BRB deformation for each story under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-49: Summary of RBS Deformation for the Six-Story Moment Frame as Beam Model

Story	RBS Rotation (Radians)					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.038	0.033	13.6%	0.046	0.040	11.4%
2	0.035	0.030	19.2%	0.042	0.037	9.75%
3	0.037	0.035	16.9%	0.046	0.043	11.6%
4	0.047	0.047	14.9%	0.055	0.055	8.76%
5	0.056	0.057	15.9%	0.066	0.065	10.7%
6	0.060	0.061	18.4%	0.073	0.074	15.0%

Table 4-50: Summary of RBS Deformation for the Six-Story Moment Frame as Girder Model

Story	RBS Rotation (Radians)					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.034	0.033	14.3%	0.041	0.040	4.76%
2	0.034	0.033	7.60%	0.042	0.041	7.46%
3	0.032	0.033	16.1%	0.040	0.040	7.01%
4	0.030	0.030	13.2%	0.038	0.037	10.5%
5	0.037	0.040	19.1%	0.045	0.047	13.5%
6	0.043	0.047	20.0%	0.055	0.059	13.8%

Table 4-51: Summary of BRB Deformation for the Six-Story Braced Frame with Chevron Configuration Model

Story	BRB Normalized Deformation					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	22.1	22.0	1.59%	20.4	20.5	2.98%
2	19.5	19.4	2.44%	17.9	17.9	2.39%
3	16.7	16.6	3.52%	15.1	15.1	3.57%
4	15.5	15.6	3.36%	13.7	13.3	4.96%
5	13.7	13.6	3.14%	12.5	12.4	3.10%
6	15.1	15.1	3.27%	13.6	13.7	4.07%

Table 4-52: Summary of BRB Deformation for the Six-Story Braced Frame with Single Diagonal Configuration Model

Story	BRB Normalized Deformation					
	Exterior			Interior		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	17.3	17.3	0.73%	17.4	17.4	0.81%
2	13.5	13.5	1.11%	13.5	13.5	1.11%
3	11.9	12.0	1.34%	12.0	12.1	1.15%
4	10.6	10.6	1.55%	10.7	10.7	1.57%
5	9.9	9.9	1.00%	9.9	9.9	0.87%
6	10.8	10.8	1.07%	11.1	11.1	1.00%

Figures 4-65 through 4-68 show the box plot of maximum RBS/BRB deformation for each story under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration. The first six box plots represent the exterior spans while the remaining represent the interior spans.

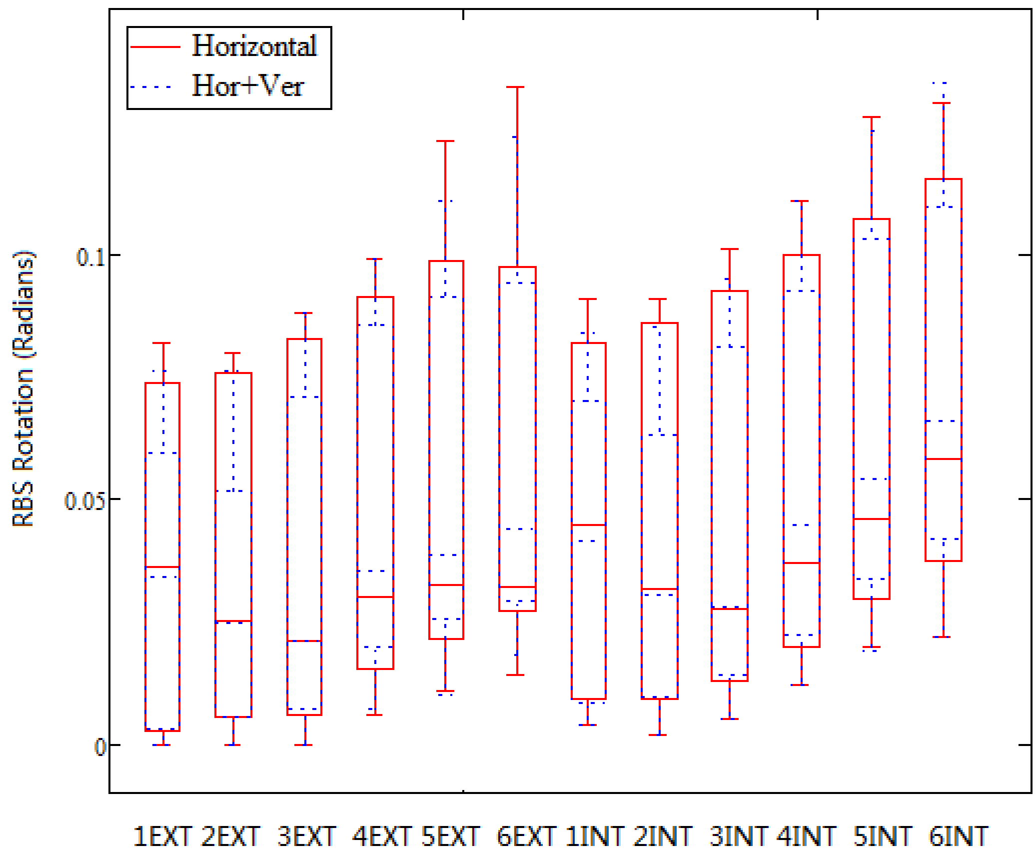

Figure 4-65: Box Plot of Max Story RBS Deformation for Six-story Moment Frame as Beam Model

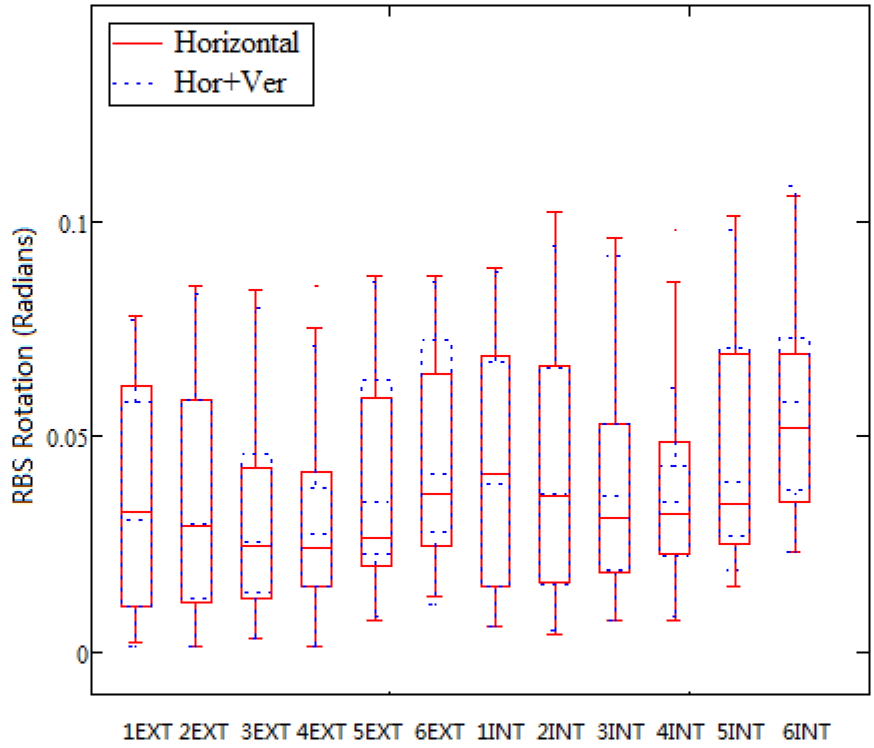

Figure 4-66: Box Plot of Max Story RBS Deformation for Six-story Moment Frame as Girder Model

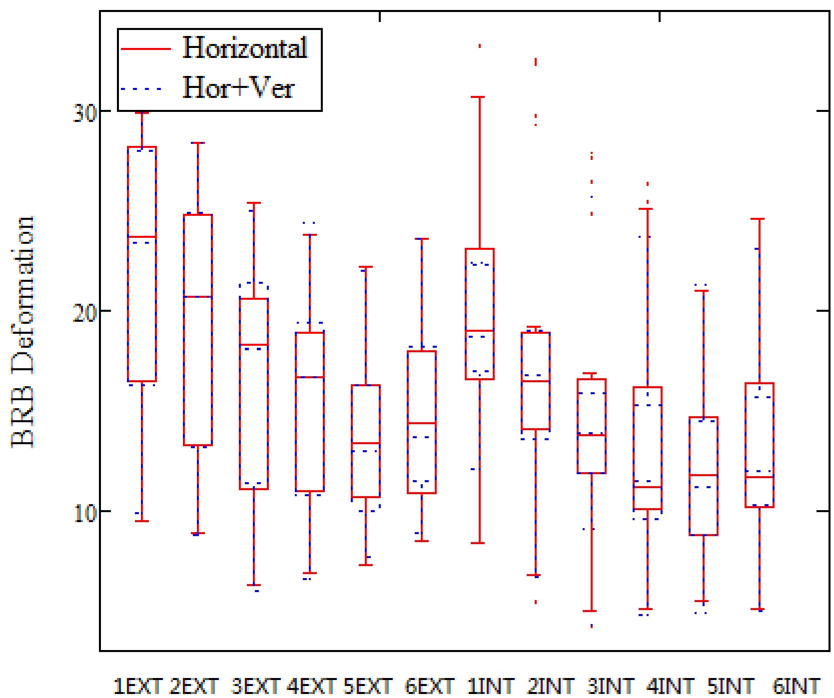

Figure 4-67: Box Plot of Max Story BRB Deformation for Six-story Braced Frame with Chevron Configuration Model

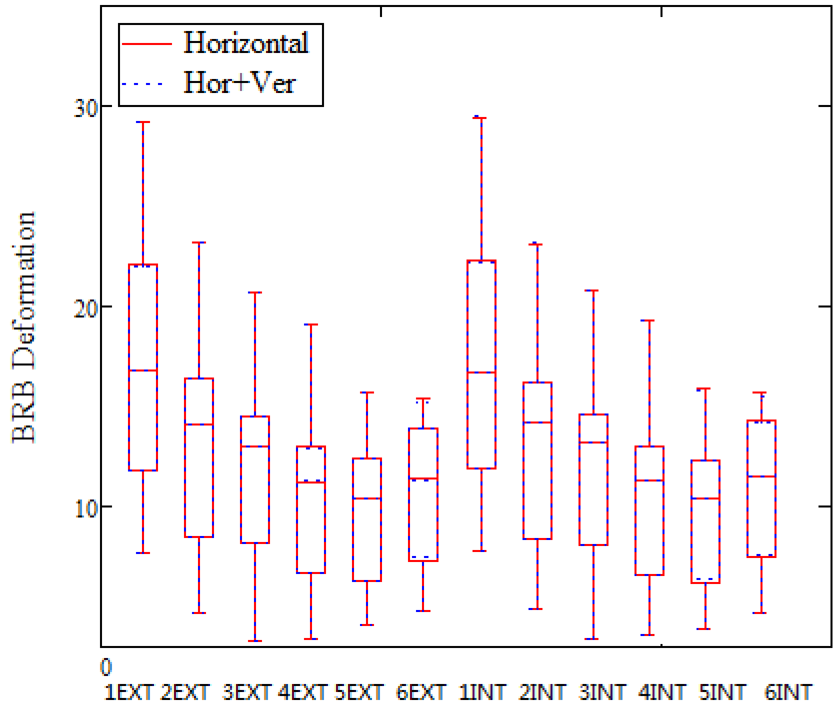

Figure 4-68: Box Plot of Max BRB Deformation of each Story for Six-story Braced Frame with Single Diagonal Configuration Model

Figures 4-69 through 4-72 show the average absolute difference of maximum RBS/BRB deformation in the exterior spans and interior spans for each story under forty selected earthquakes including horizontal and vertical ground motions between two different loading condition (Horizontal Only and Horizontal + Vertical) in the three-story models and six-story models respectively.

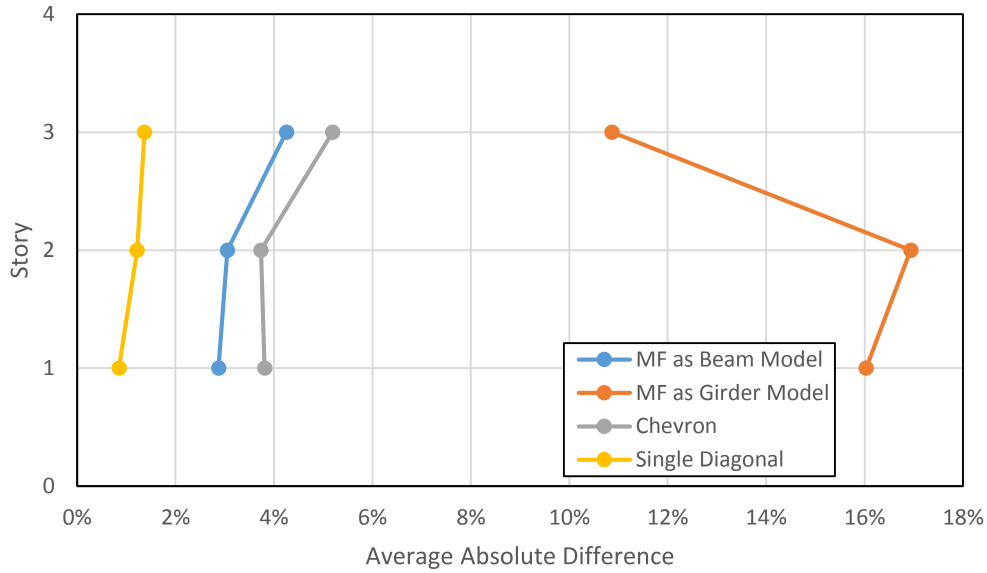

Figure 4-69: Average Absolute Difference of Maximum RBS/BRB deformation in the exterior span between Two Different Loading Conditions in Three-Story Models

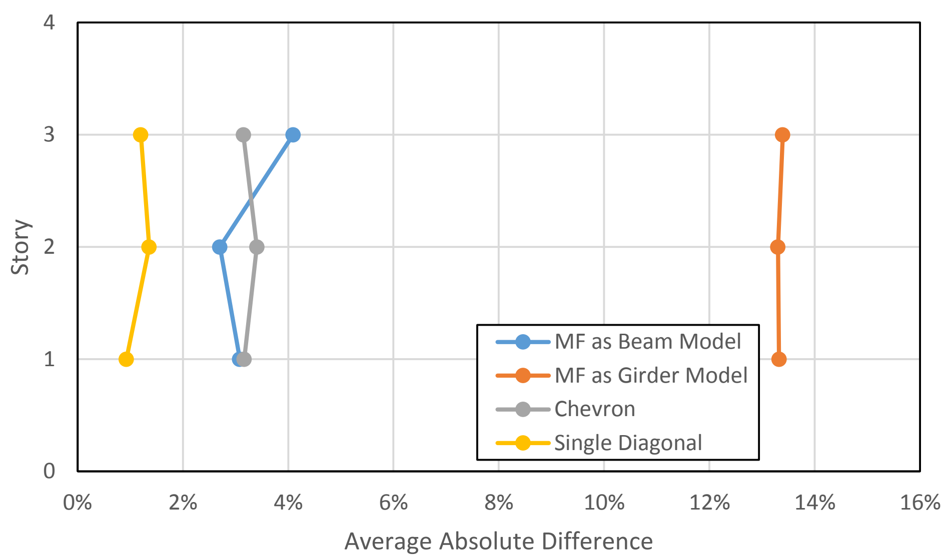

Figure 4-70: Average Absolute Difference of Maximum RBS/BRB deformation in the interior span between Two Different Loading Conditions in Three-Story Models

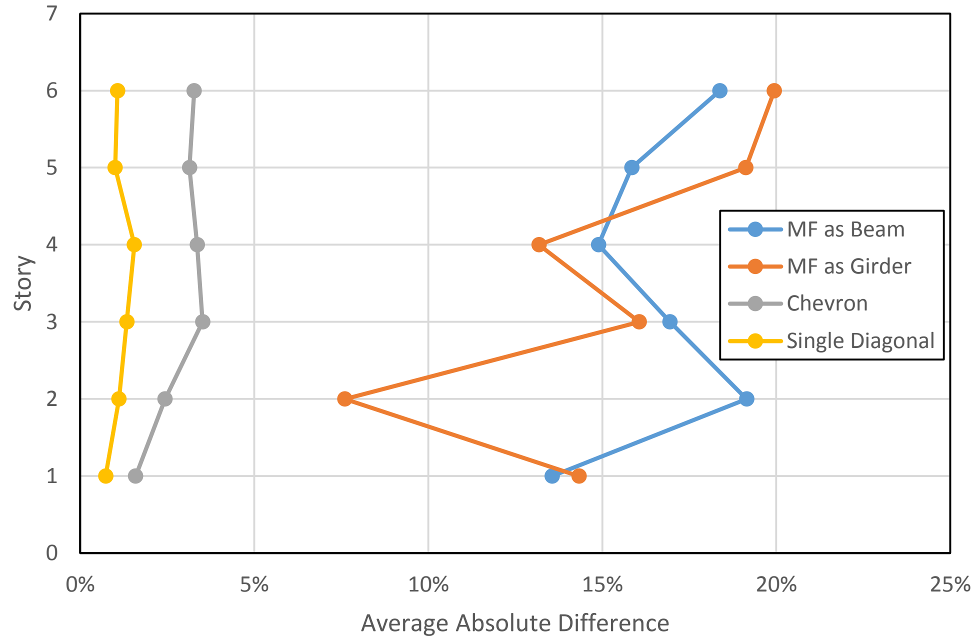

Figure 4-71: Average Absolute Difference of Maximum RBS/BRB deformation in the exterior span between Two Different Loading Conditions in Six-Story Models

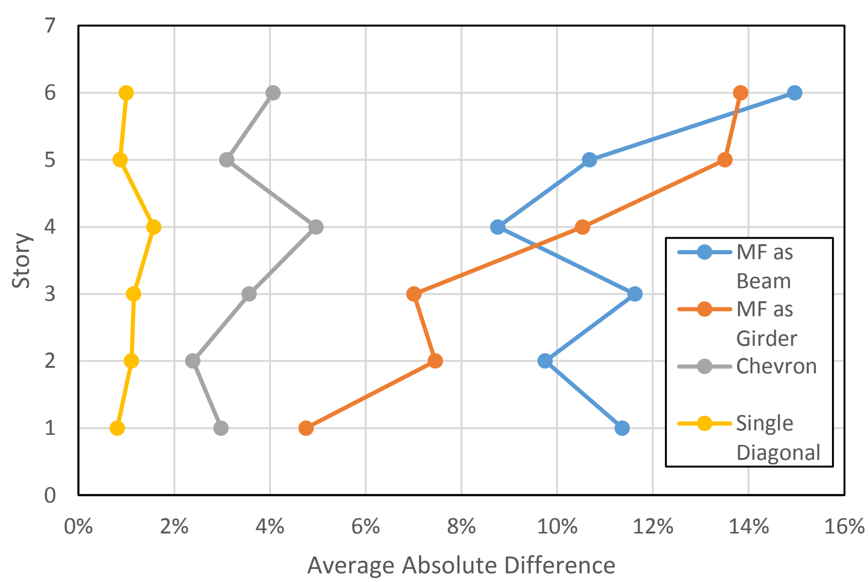

Figure 4-72: Average Absolute Difference of Maximum RBS/BRB deformation in the interior span between Two Different Loading Conditions in Six-Story Models

The average value of maximum RBS/BRB deformation for each story under forty selected earthquakes including horizontal and vertical ground motions for all the models in the study are very close to each other between the two different loading conditions (Horizontal Only and Horizontal + Vertical). The median value of maximum RBS/BRB deformation is also close to each other in the three-story moment frames, three-story braced frames and six-story braced frames while the median value in the six-story moment frames increases, especially in the upper three stories, when the vertical ground motions are added to the structures. What's more, the average absolute difference values of the three-story moment frame, three-story braced frame and six-story braced frame are relatively small compared with that in the six-story moment frame models. This means the impact of vertical ground motion on these models is not significant. However, there is some impact of vertical ground motion on the two six-story moment frame models, especially for the higher stories according to the average absolute difference. In addition, the impact of vertical ground motions is very subtle. The maximum deformation for each story may increase or decrease when you add vertical earthquake excitation. This is the reason why the average values are very close to each other between two different loading cases while the average absolute difference value is relatively large in the six-story moment frame models. Figure 4-73 shows the maximum RBS deformation for each story in the exterior span of the six-story moment frame as beam model under NF16-1 (Horizontal Only) and NF16-1(Horizontal + Vertical). Figure 4-74 shows the hysteresis loop of RBS in the sixth story exterior span of the six-story moment frame as beam model under NF16-1 (Horizontal Only) and NF16-1(Horizontal + Vertical). The impact of vertical ground motion is significant in this case which is not negligible. The remaining data about the deformation of all the models in this study is in appendix B.

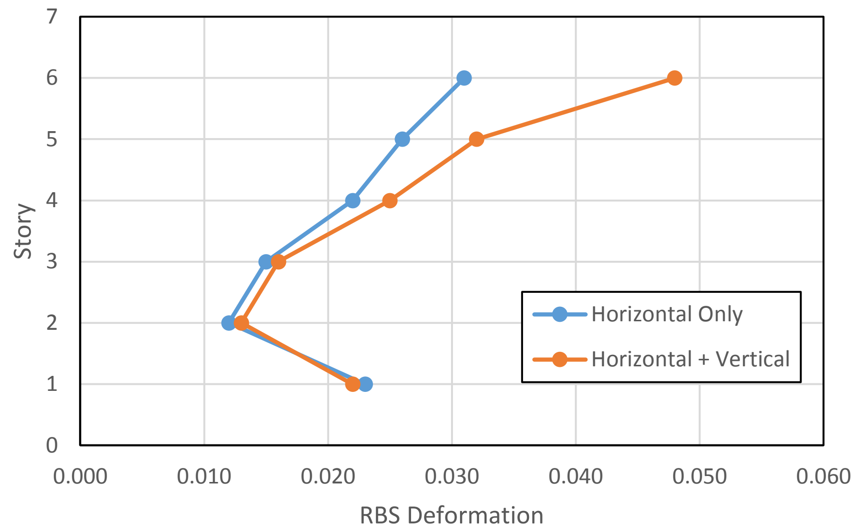

Figure 4-73: Maximum RBS Deformation for Each Story in the Moment Frame as Beam Model under NF16-1

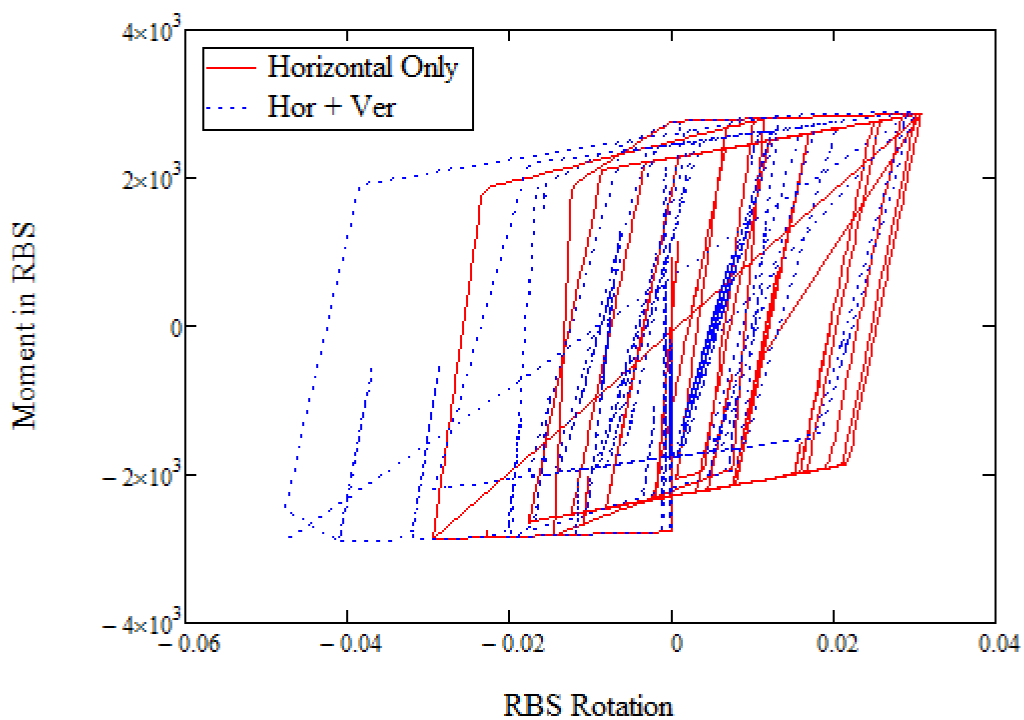

Figure 4-74: Hysteresis Loop of RBS in the Sixth Story Exterior Span of the Six-story Moment Frame as Beam model under NF16-1

Tables 4-53 through 4-56 show the average and average absolute difference value of maximum RBS rotation or BRB deformation for each story by earthquake group in the exterior spans under forty selected earthquakes including horizontal and vertical ground motions in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-53: Summary of RBS Rotation by Earthquake Group for the Three-Story Moment Frame as Beam Model

RBS Rotation in the Exterior Span (Radians)						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.014	0.014	3.40%	0.049	0.048	1.55%
2	0.018	0.018	5.14%	0.063	0.062	2.31%
3	0.028	0.029	9.30%	0.083	0.082	2.57%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.022	0.021	3.09%	0.043	0.042	3.44%
2	0.031	0.032	2.39%	0.042	0.042	2.38%
3	0.045	0.046	2.90%	0.059	0.058	2.24%

Table 4-54: Summary of RBS Rotation by Earthquake Group for the Three-Story Moment Frame as Girder Model

RBS Rotation in the Exterior Span (Radians)						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.017	0.018	9.08%	0.038	0.038	1.39%
2	0.023	0.025	15.9%	0.046	0.046	4.59%
3	0.032	0.033	11.6%	0.061	0.061	4.43%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.025	0.032	41.0%	0.061	0.066	12.7%
2	0.041	0.044	33.6%	0.065	0.070	13.7%
3	0.062	0.064	17.2%	0.066	0.070	10.3%

Table 4-55: Summary of BRB Deformation by Earthquake Group for the Three-Story Braced Frame with Chevron Configuration

BRB Normalized Deformation						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	15.4	15.0	3.34%	20.0	20.3	5.59%
2	12.0	11.7	2.53%	18.3	17.6	5.72%
3	11.1	10.9	2.23%	16.7	16.7	4.17%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	16.6	16.8	4.04%	19.0	18.7	2.27%
2	13.4	13.4	4.79%	14.1	14.1	1.89%
3	13.0	13.3	4.36%	10.7	11.2	10.0%

Table 4-56: Summary of BRB Deformation by Earthquake Group for the Three-Story Braced Frame with Single Diagonal Configuration

BRB Normalized Deformation						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	9.2	9.2	0.83%	12.8	12.8	0.58%
2	6.7	6.7	0.83%	11.8	11.7	0.85%
3	5.9	5.9	0.98%	11.6	11.5	1.22%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	10.3	10.4	1.62%	13.3	13.3	0.38%
2	9.9	9.9	2.17%	10.2	10.0	1.02%
3	8.6	8.6	1.04%	7.2	7.4	2.24%

Figures 4-75 through 4-78 show the average absolute difference of maximum RBS rotation or BRB axial deformation in the exterior spans by earthquake group for each story under forty selected earthquakes including horizontal and vertical ground motions between two different loading condition (Horizontal Only and Horizontal + Vertical) in the three-story moment frame as beam model, three-story moment frame as girder model, three-story braced frame with chevron configuration and three-story braced frame with single diagonal configuration, respectively.

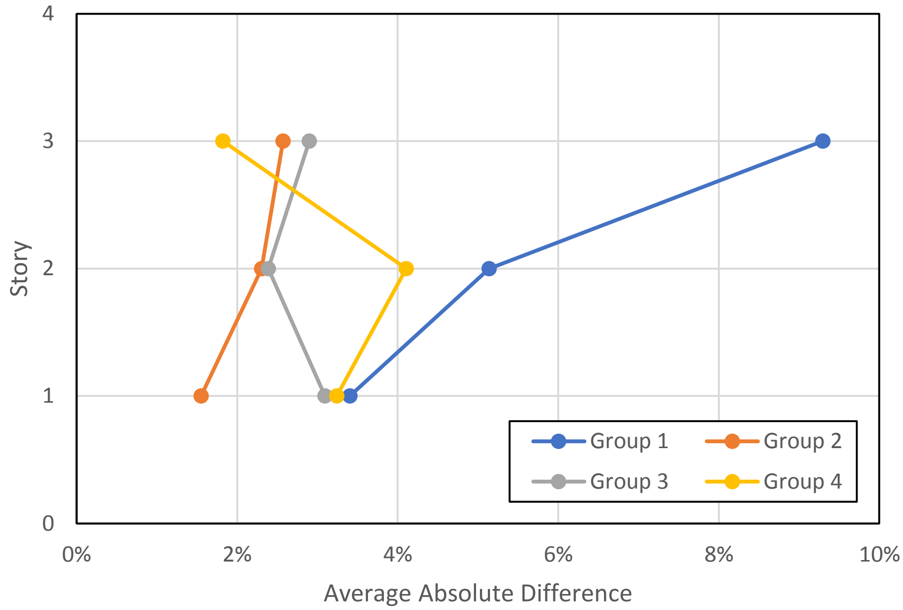

Figure 4-75: Average Absolute Difference of Maximum RBS Rotation between Two Different Loading Conditions by Earthquake Group in Three-Story MF as Beam Model

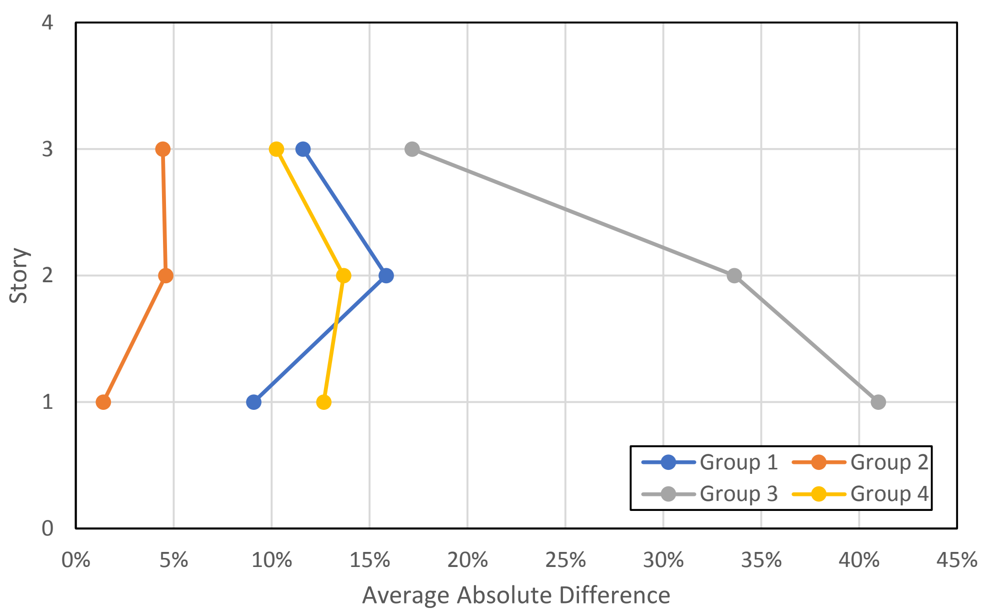

Figure 4-76: Average Absolute Difference of Maximum RBS Rotation between Two Different Loading Conditions by Earthquake Group in Three-Story MF as Girder Model

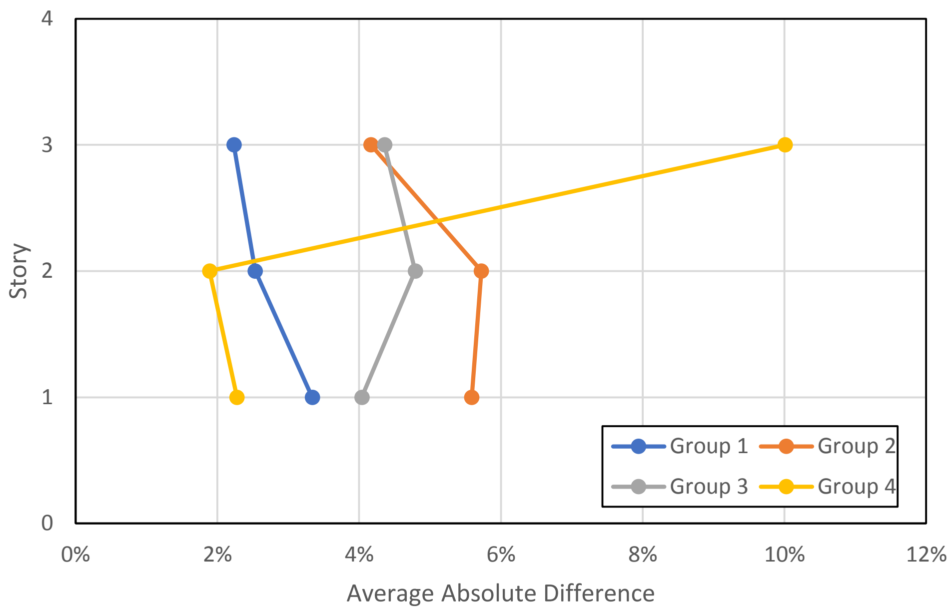

Figure 4-77: Average Absolute Difference of Maximum BRB Deformation between Two Different Loading Conditions by Earthquake Group in Three-Story Braced Frame with Chevron Configuration

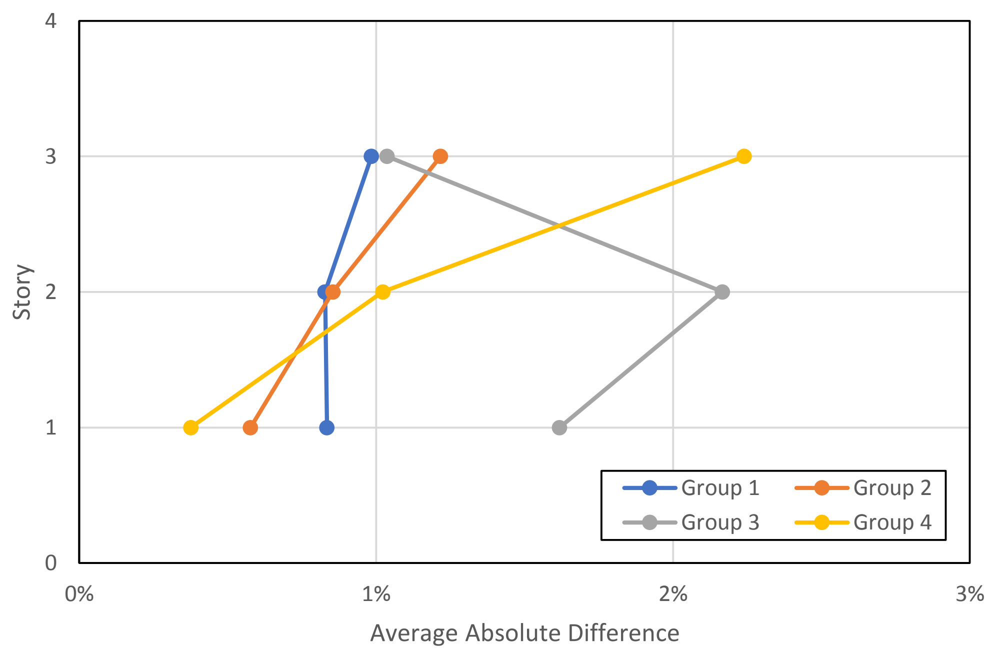

Figure 4-78: Average Absolute Difference of Maximum BRB Deformation between Two Different Loading Conditions by Earthquake Group in Three-Story Braced Frame with Single Diagonal Configuration

Tables 4-57 through 4-60 show the average and average absolute difference value of maximum RBS rotation or BRB axial deformation for each story by earthquake group under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration.

Table 4-57: Summary of RBS Rotation by Earthquake Group for the Six-Story Moment Frame as Beam Model

RBS Rotation in the Exterior Span (Radians)						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.015	0.015	14.5%	0.037	0.030	12.0%
2	0.013	0.013	6.78%	0.031	0.022	14.8%
3	0.013	0.013	29.4%	0.035	0.029	12.4%
4	0.018	0.020	17.5%	0.051	0.049	17.3%
5	0.023	0.025	16.8%	0.068	0.066	13.0%
6	0.027	0.029	8.65%	0.075	0.075	19.0%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.032	0.024	17.8%	0.068	0.062	10.0%
2	0.033	0.025	47.8%	0.063	0.060	7.15%
3	0.039	0.034	9.00%	0.062	0.062	17.0%
4	0.051	0.050	5.24%	0.067	0.068	19.5%
5	0.065	0.065	8.75%	0.070	0.073	24.8%
6	0.069	0.069	4.79%	0.068	0.071	41.1%

Table 4-58: Summary of RBS Rotation by Earthquake Group for the Six-Story Moment Frame as Girder Model

RBS Rotation in the Exterior Span (Radians)						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.021	0.022	21.9%	0.029	0.028	15.6%
2	0.021	0.021	12.3%	0.031	0.030	6.96%
3	0.019	0.020	36.7%	0.032	0.033	9.95%
4	0.017	0.016	12.0%	0.031	0.033	26.1%
5	0.019	0.020	15.8%	0.045	0.049	23.8%
6	0.026	0.028	19.9%	0.052	0.054	15.2%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	0.024	0.022	16.8%	0.063	0.061	2.99%
2	0.025	0.024	8.74%	0.060	0.059	2.39%
3	0.025	0.026	12.7%	0.052	0.052	4.95%
4	0.027	0.026	7.68%	0.046	0.046	6.82%
5	0.033	0.040	28.8%	0.049	0.050	8.04%
6	0.043	0.054	32.6%	0.050	0.051	12.2%

Table 4-59: Summary of BRB Deformation by Earthquake Group for the Six-Story Braced Frame with Chevron Configuration

BRB Normalized Deformation						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	19.0	19.2	2.02%	22.1	21.6	2.57%
2	15.9	15.6	2.22%	20.3	19.7	3.02%
3	13.4	12.9	6.68%	17.9	17.6	2.86%
4	12.5	12.5	2.84%	17.3	17.1	2.61%
5	12.0	11.7	3.29%	15.3	15.2	3.67%
6	13.9	13.8	2.43%	17.3	16.9	3.54%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	20.5	19.8	4.31%	26.9	26.9	0.42%
2	17.5	17.5	4.86%	24.3	24.5	1.57%
3	14.5	14.7	5.23%	21.0	21.3	1.55%
4	12.8	13.4	8.25%	19.2	19.5	1.82%
5	11.2	11.4	9.22%	16.4	16.7	2.09%
6	11.9	12.6	11.5%	17.6	17.7	1.60%

Table 4-60: Summary of BRB Deformation by Earthquake Group for the Six-Story Braced Frame with Single Diagonal Configuration

BRB Normalized Deformation						
Story	Group 1			Group 2		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	13.4	13.3	0.47%	17.2	17.4	2.04%
2	9.9	9.8	0.47%	13.4	13.7	3.60%
3	8.2	8.2	0.72%	12.3	12.5	4.17%
4	7.4	7.4	0.69%	10.9	11.1	4.75%
5	7.6	7.6	0.57%	10.3	10.5	2.57%
6	8.7	8.7	0.59%	12.2	12.3	2.90%
Story	Group 3			Group 4		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	17.1	17.1	0.30%	21.6	21.6	0.10%
2	13.1	13.1	0.27%	17.5	17.5	0.10%
3	11.5	11.5	0.42%	15.7	15.7	0.05%
4	10.1	10.0	0.72%	14.1	14.1	0.06%
5	9.2	9.1	0.76%	12.5	12.5	0.10%
6	9.5	9.4	0.71%	13.0	13.0	0.07%

Figures 4-79 through 4-82 show the average absolute difference of maximum RBS rotation or BRB axial deformation in the exterior spans by earthquake group for each story under forty selected earthquakes including horizontal and vertical ground motions between two different loading condition (Horizontal Only and Horizontal + Vertical) in the six-story moment frame as beam model, six-story moment frame as girder model, six-story braced frame with chevron configuration and six-story braced frame with single diagonal configuration, respectively.

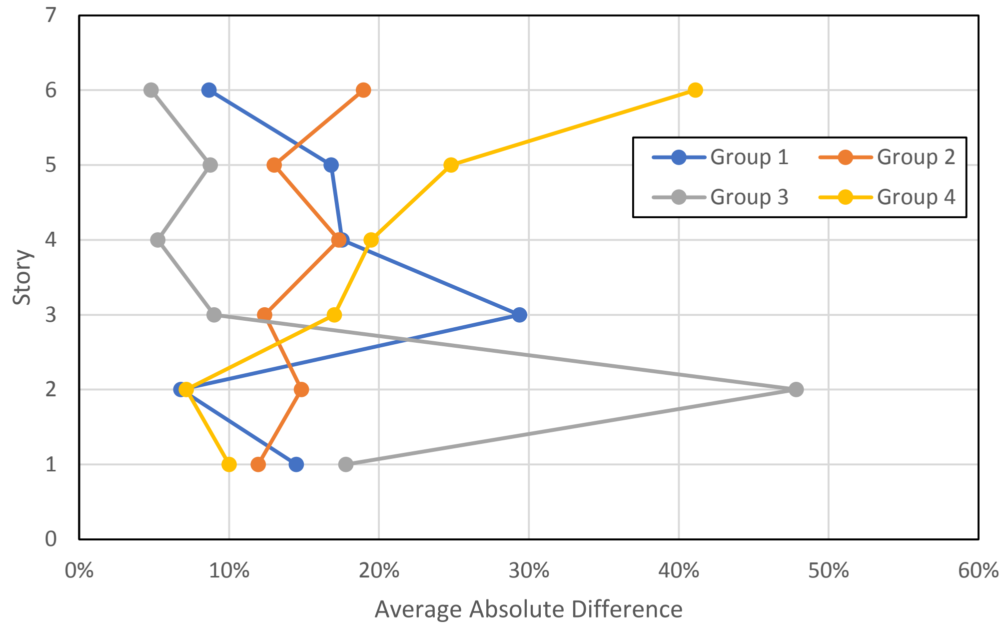

Figure 4-79: Average Absolute Difference of Maximum RBS Rotation between Two Different Loading Conditions by Earthquake Group in Six-Story MF as Beam Model

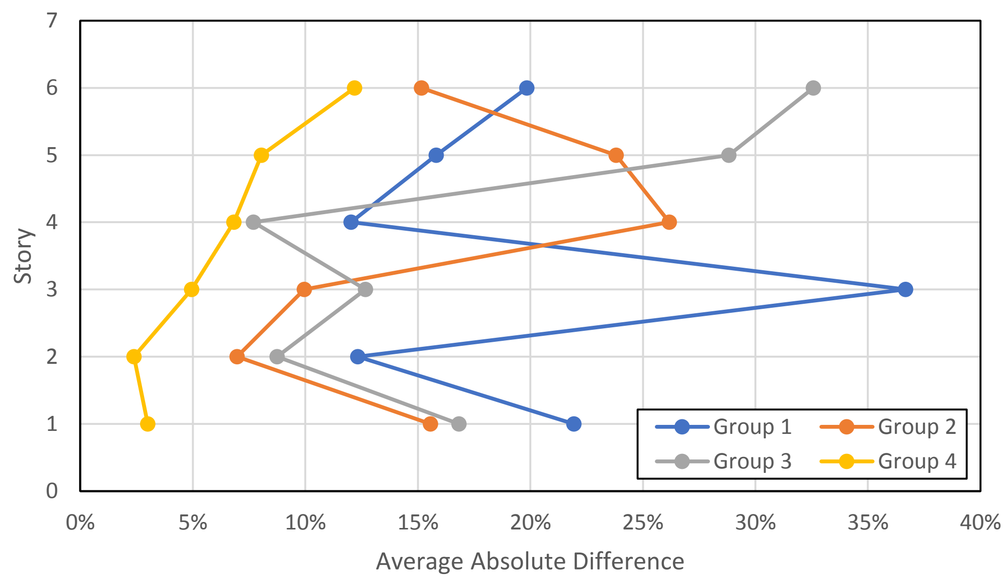

Figure 4-80: Average Absolute Difference of Maximum RBS Rotation between Two Different Loading Conditions by Earthquake Group in Six-Story MF as Girder Model

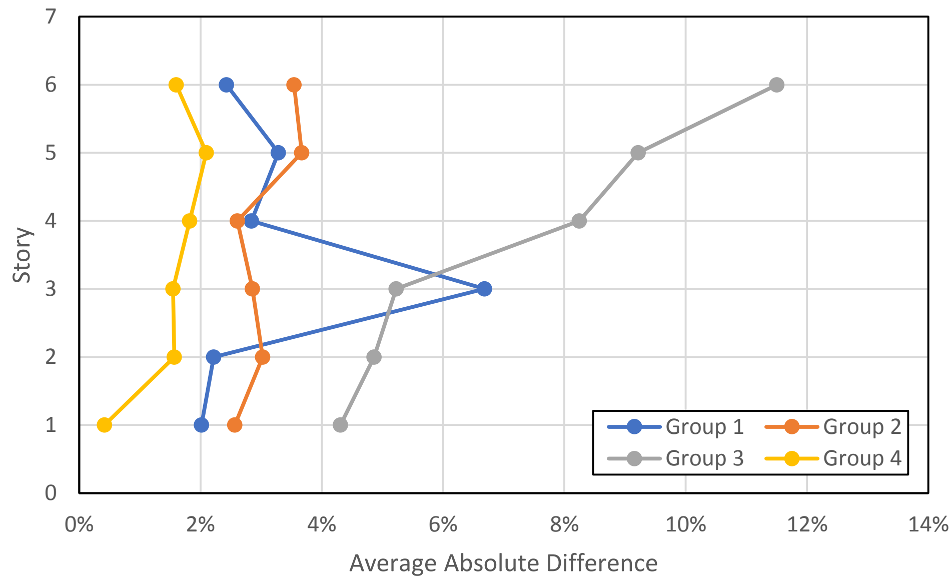

Figure 4-81: Average Absolute Difference of Maximum BRB Deformation between Two Different Loading Conditions by Earthquake Group in Six-Story Braced Frame with Chevron Configuration

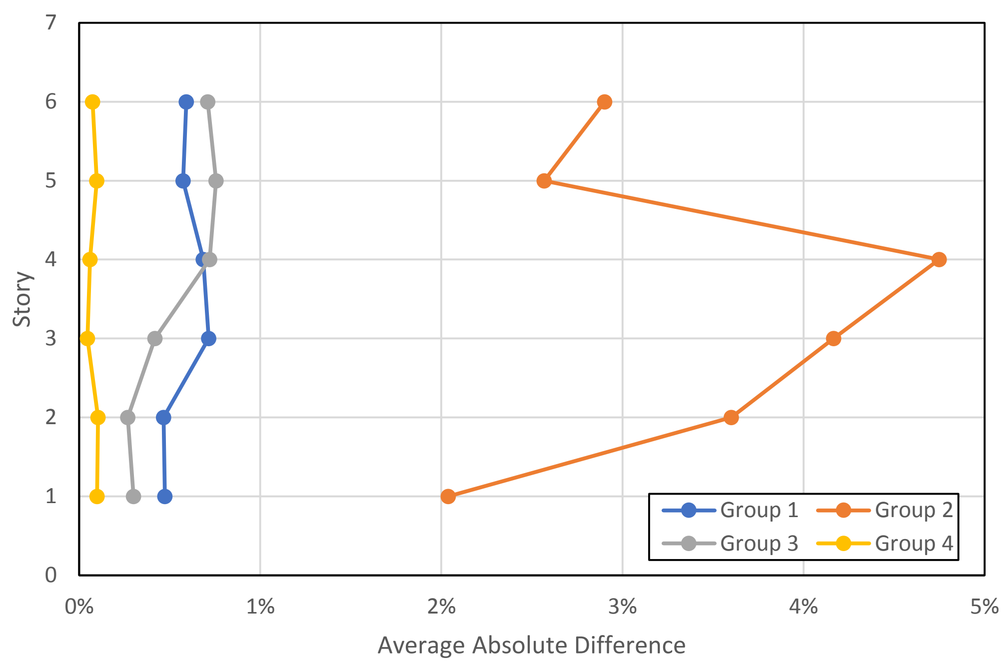

Figure 4-82: Average Absolute Difference of Maximum BRB Deformation between Two Different Loading Conditions by Earthquake Group in Six-Story Braced Frame with Single Diagonal Configuration

According to the data analysis by earthquake groups, there is no clear evidence and trend showing that the higher or lower V/H value will cause more or less impact on the maximum RBS rotation or BRB axial deformation in the steel frame structures. The impact of vertical ground motion on maximum RBS rotation or BRB deformation of three-story steel frame structures and braced frame structures in each earthquake suite is insignificant and limited. However, there is a impact of vertical ground motion on the six-story moment frame structures.

4.7 Energy

Usually the tremendous energy released in an earthquake is mostly worn off by diastrophism, tectonic movements, earth's surface cracks, etc. Even though a very small portion of the energy impacts structures, it could cause severe damage and collapse to the structures and result in gigantic catastrophe. Figure 4-83 shows the energy dissipation in the six-story moment frame as beam model under NF 16-1(Horizontal + Vertical). The values on the X axis represent time in seconds, while values on the Y axis represent the percent of energy dissipation. Tables 4-61 and 4-62 show the energy dissipation for each earthquake including two different loading situations in six-story moment frame as beam model and six-story braced frame with chevron configuration model, respectively. The remaining energy dissipation information for the other models is in Appendix B.

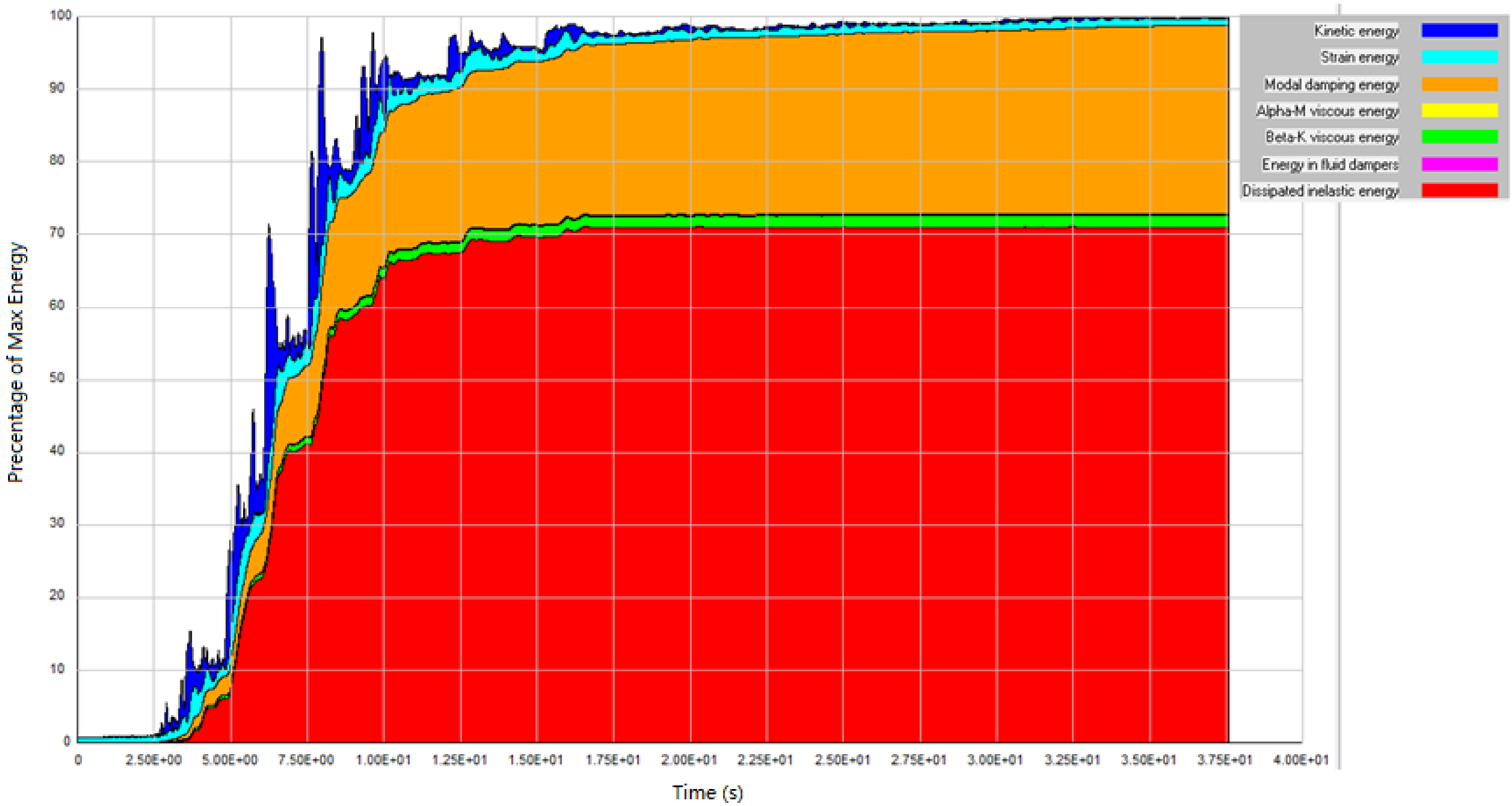

Figure 4-83: Energy Dissipation in the Six-Story Moment Frame as Beam Model under NF 16-1(Horizontal + Vertical)

Table 4-61: Energy Dissipation for Each Earthquake in the Six-Story Moment Frame as Beam Model

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Difference	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	144100	146100	106500	108200	1.39%	1.60%
FF13-1	98920	116000	69410	83680	17.27%	20.56%
FF14-1	31980	32840	17090	17730	2.69%	3.74%
FF14-2	136700	144400	101300	107800	5.63%	6.42%
FF15-2	150100	161500	114400	122300	7.59%	6.91%
FF19-1	212500	217300	169900	173900	2.26%	2.35%
FF21-2	51710	54480	34680	37120	5.36%	7.04%
FF22-1	36820	39070	22010	24060	6.11%	9.31%
FF22-2	65210	69460	43690	47720	6.52%	9.22%
NF02-2	80780	80770	66760	66560	-0.01%	-0.30%
NF05-1	85240	88290	63580	66900	3.58%	5.22%
NF05-2	113100	114200	88190	90310	0.97%	2.40%
NF16-1	90400	93880	63540	66740	3.85%	5.04%
NF16-2	79500	84240	52140	56620	5.96%	8.59%
NF17-2	145700	149700	111800	114800	2.75%	2.68%
NF21-2	30080	31400	17490	18800	4.39%	7.49%
NF22-1	57880	59580	34440	36430	2.94%	5.78%
NF25-2	146800	144200	122400	119700	-1.77%	-2.21%
NF 27-2	169200	167300	124900	123600	-1.12%	-1.04%
NF28-1	140500	140600	116700	116700	0.07%	0.00%

Table 4-62: Energy Dissipation for Each Earthquake in the Six-Story Braced Frame with Chevron Configuration Model

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Difference	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	156300	159000	128700	129900	1.73%	0.93%
FF13-1	130500	132200	108600	109700	1.30%	1.01%
FF14-1	48360	49410	38520	38780	2.17%	0.67%
FF14-2	180100	182900	149100	149900	1.55%	0.54%
FF15-2	151100	154300	127500	128300	2.12%	0.63%
FF19-1	151500	151800	131100	131300	0.20%	0.15%
FF21-2	46630	47110	38290	38560	1.03%	0.71%
FF22-1	41660	42630	32160	32450	2.33%	0.90%
FF22-2	77500	78090	61750	61700	0.76%	-0.08%
NF02-2	80010	80210	68590	68630	0.25%	0.06%
NF05-1	87120	88340	72630	73440	1.40%	1.12%
NF05-2	86660	99430	72500	75060	14.74%	3.53%
NF16-1	125100	126600	102200	103200	1.20%	0.98%
NF16-2	120200	121000	98950	99520	0.67%	0.58%
NF17-2	169700	170400	145200	145300	0.41%	0.07%
NF21-2	151300	154000	123400	124000	1.78%	0.49%
NF22-1	68820	68470	31480	31380	-0.51%	-0.32%
NF25-2	113000	113600	97160	97290	0.53%	0.13%
NF 27-2	222600	225100	189500	190200	1.12%	0.37%
NF28-1	123300	123400	104100	104000	0.08%	-0.10%

The total energy dissipated by the structure and energy dissipated by the element inelastic deformation under forty selected earthquakes including horizontal and vertical ground motions for the braced frame models in the study are very similar when comparing the two different loading conditions (Horizontal Only and Horizontal + Vertical). Slight increases in response occur when the vertical ground motions are included in the moment frame models. That means the impact of vertical ground motions on energy dissipation is insignificant on the structures.

4.8 Summary

In this chapter, detailed results of the three-story models and the six-story models are analyzed and presented. Statistical data and graphs showing the impact of vertical ground motions on the column axial force, vertical acceleration and vertical deflection are presented for both exterior spans and interior spans in all the models. The impact of vertical earthquake motion on story drift, roof horizontal acceleration and energy are also discussed in this chapter. Mean value and boxplot which can help to find median value and difference value are calculated in this chapter to find the difference between the two different loading cases. The impact of vertical ground motion on the story drift, roof acceleration and energy in all the steel frame structures is insignificant and limited while the impact on the column axial force, vertical acceleration and vertical deflection for all the models in this study is significant. What's more, the demand in the upper stories of the six-story moment frame structures show more significant differences in response due to the addition of vertical ground motions. Besides the overall results, some results of the individual case are provided in this chapter to help to understand the structure behavior.

Chapter 5 Conclusion and Recommendation

5.1 Summary

The goal of the study is an investigation into the impact of vertical ground motion on seismic response of steel frame structures by using nonlinear response history analysis.

In order to get a full understanding of impact of vertical ground motion, four three-story buildings and four six-story buildings were designed and modeled. These eight models which included special moment frames and buckling restrained braced frames in this research are as follows: three-story moment frame as beam, three-story moment frame as girder, three-story braced frame with chevron configuration, three-story braced frame with single diagonal configuration, six-story moment frame as beam, six-story moment frame as girder, six-story braced frame with chevron configuration, six-story braced frame with single diagonal configuration. All the basic information including gravity load information and seismic information in this paper was provided by Sabelli's report (2001). The seismic load for design was calculated by using the equivalent lateral force (ELF) method. The whole seismic design procedure for three-story and six-story special moment frames was completed by using the AISC Seismic Provisions for Structural Steel Building (AISC, 2005) and ASCE Standard 7-10 (ASCE, 2010) along with the computer program SAP 2000 (CSI, 2012). Reduced beam section was chosen and designed as one of the major energy-absorbing elements in the structures. "Weak Beam Strong Column" check as well as panel zone design were included in this study to make sure the columns would not fail first during earthquakes. All the braces in this study were designed by Xie (2015). The finite element modeling procedure was completed by another computer program Perform 3D (CSI, 2011).

Detailed results of all the models in this study by nonlinear dynamic analysis were extracted from the Perform 3D and analyzed. Results include maximum story drift for each story during earthquake, residual story drift after strong ground motion, maximum normalized axial force in the exterior and interior columns, roof horizontal acceleration, vertical acceleration and vertical normalized deflection in the midspan of beams, rotation of reduced beam sections, normalized axial deformation in the buckling restrained braces and the energy absorbed by the whole structure and inelastic elements. Behaviors of the eight structures in this study were investigated and compared between two different loading cases.

5.2 Conclusion

The detailed results and conclusions have been presented in tabular and graphical form in Chapter 4. The structure behavior is measured based on the response of buildings during the earthquakes. This was investigated analytically by looking at maximum drift, residual drift, maximum axial force in the column, maximum rotation for the reduced beam sections, maximum axial deformation for the buckling restrained braces and vertical deflection in the beams. Two additional aspects are the roof horizontal acceleration and the vertical absolute acceleration in the beam were investigated to show the direct impact of vertical ground motions on buildings. What’s more, sensitive equipment and some non-structural components can be affected by large acceleration. Total energy and energy dissipated by inelastic element deformation are also investigated to give an overall impact of vertical ground motion on the structures.

In general, the vertical ground motion has minor effect on maximum drift and residual drift for each story in the building. For the three-story and six-story braced frame models, the average absolute difference values are less than 5%. However, there is a reasonable effect of vertical ground motion on the upper stories of the six-story moment frame. From an individual view for each earthquake in this research paper, some individual earthquakes have a significant impact on the steel moment frames.

Axial forces in both the exterior and interior column increase significantly after vertical ground motions are added to the structure. The impact of vertical ground motions on the interior columns is larger than that on the exterior columns because the vertical mass on the joints along the interior columns is larger compared with that in the exterior columns. In addition, the moment frames get more impact from vertical ground motions compared with braced frames.

Roof horizontal acceleration is also presented in this paper. The impact of vertical ground motion is insignificant on the roof horizontal acceleration except for the six-story moment frame as beam model. The reason the roof horizontal acceleration increases significantly is because the first vertical period of the structure is very close to peak value of the response spectrum. The increase in vertical acceleration is a direct effect of the vertical ground motion on the structure. The vertical ground motions dominate the vertical acceleration in all the models in this study.

Similar with the impact of vertical ground motions on the vertical acceleration in the structures, the vertical ground motions dominate the vertical deflection in the midspan of the beam. The deflection at the beam midspan increases significantly when the vertical ground motions are added to the structure, especially for the braced frames. However, the deflection of beam in the exterior spans of braced frames with chevron configuration is not as large as the remaining models since the braces are connected with the midspan of beams in the exterior spans which resist the beams from deflecting.

The impact of vertical ground motions on axial deformation of buckling restrained braces in the braced frame is insignificant according to the average value, median value and the average absolute difference value. However, there is a decent impact of vertical ground motions on rotation of reduced beam sections, especially the reduced beam sections in the upper stories. From an individual view for each earthquake in this research paper, some records have a significant impact on the reduced beam sections steel moment frames.

The impact of vertical ground motions on the energy absorbed by steel frame structures is negligible.

5.3 Recommendations

This thesis provides a study on the impact of vertical ground motion on seismic response of steel frame structures. However, according to previous research, there will be a larger effect of vertical ground motion on tall buildings, buildings with longer spans, buildings with vertical irregularities (cantilever sections and transfer girders) and longer span bridges. The heights of steel models in this paper are limited to three stories and six stories. More effort should be put on investigating the impact of vertical ground motion on the higher buildings and longer span bridges. In addition, all the steel frame structures in this thesis are regular and standard. More effort should be put on the irregular structures such as the buildings which have very long span beams, overhangs and so on.

According to the results in this paper, the impact of vertical ground motion on the axial force in the columns is significant. It can cause a significant influence on the shear capacity of concrete columns. More effort should be put on investigating the impact of vertical ground motion on reinforced concrete structures.

Some kinds of buildings can have much larger mass on the top of building than that in the remaining stories for some special usages, such as a garden on the top of building. This kind of building can have more impacts on it. More effort should be put on investigating the impact of vertical ground motion on the structures which have large mass on the upper stories.

According to the result of roof horizontal acceleration, the vertical ground motions and horizontal ground motions can have some interactional impacts on the structure. More effort should be put on investigation of the interactional impact of horizontal ground motions and vertical ground motion on the structures.

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Appendix A

$ORIGIN \equiv 1$

3 story building Seismic Design

Site:Los Angeles,LA

Site Class D

Occupancy Category. II

$S_s := 2.09$ $F_a := 1.0$ $S_{ms} := F_a \cdot S_s = 2.09$ $S_{ds} := \frac{2}{3} \cdot S_{ms} = 1.393$

$S_1 := 0.77$ $F_v := 1.5$ $S_{m1} := F_v \cdot S_1 = 1.155$ $S_{d1} := \frac{2}{3} \cdot S_{m1} = 0.77$

Seismic Design Category: D

Column Height

$CH_1 := 13$ ft $CH_2 := 13$ ft $CH_3 := 13$ ft

Building Dimension

$N - S$ $w_{NS} := 120$ ft

$W - E$ $w_{EW} := 180$ ft

Building Dimensions including cladding

$w_{NS\_Clad} := w_{NS} + 4 = 124$ ft

$w_{EW\_Clad} := w_{EW} + 4 = 184$ ft

Material weights subject to change. Basic weight assumptions listed here:

Column Weight $w_{col} := 0.15$

Floor Weight $w_{floor} := 0.067$

Roof Weight $w_{.roof} := 0.066$

Cladding Weight $w_{clad} := 0.025$

HVAC/Mech/Plumbing $w_{mech} := 0.007$

Partition Weight $w_{par} := 0.01$

Penthouse Mechnical/Electronic equipment $w_{Penthouse} := 0.047$

42in. tall parapet at the roof

Parapet Height $H_{par} := 3.5$

Cladding Weight

$$w_{clad\_1} := w_{clad} \cdot \frac{\left(CH_1 + CH_2\right)}{2} \cdot \left(w_{EW\_Clad} + w_{NS\_Clad}\right) \cdot 2 = 200.2$$

$$w_{clad\_2} := w_{clad} \cdot \frac{\left(CH_2 + CH_3\right)}{2} \cdot \left(w_{EW\_Clad} + w_{NS\_Clad}\right) \cdot 2 = 200.2$$

$$w_{clad\_r} := w_{clad} \cdot \left(\frac{CH_3}{2} + H_{par}\right) \cdot \left(w_{EW\_Clad} + w_{NS\_Clad}\right) \cdot 2 = 154$$

Column Weight

$$w_{col\_1} := w_{col} \cdot \frac{(CH_1 + CH_2)}{2} \cdot 24 = 46.8$$

$$w_{col\_2} := w_{col} \cdot \frac{(CH_2 + CH_3)}{2} \cdot 24 = 46.8$$

$$w_{col\_r} := w_{col} \cdot \frac{(CH_3)}{2} \cdot 20 = 19.5$$

Seismic Weight

$$W_1 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW})\right] + w_{clad\_1} + w_{col\_1} = 2.061 \times 10^3$$

$$W_2 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW})\right] + w_{clad\_2} + w_{col\_2} = 2.061 \times 10^3$$

$$W_{withoutpenthouse} := \left[(w_{floor} + w_{mech}) \cdot (w_{NS} \cdot w_{EW})\right] + w_{clad\_r} + w_{col\_r} = 1.772 \times 10^3$$

$$W_{withpenthouse} := \left[(w_{floor} + w_{Penthouse} - w_{mech}) \cdot (w_{NS} \cdot w_{EW})\right] + w_{clad\_r} + w_{col\_r} = 2.485 \times 10^3$$

$$W_r := \frac{11}{12} \cdot W_{withoutpenthouse} + \frac{1}{12} \cdot W_{withpenthouse} = 1.831 \times 10^3$$

$$W := \begin{pmatrix} W_1 \\ W_2 \\ W_r \end{pmatrix} = \begin{pmatrix} 2.061 \times 10^3 \\ 2.061 \times 10^3 \\ 1.831 \times 10^3 \end{pmatrix} \qquad W_{total} := W_1 + W_2 + W_r = 5.954 \times 10^3 \quad \text{kips}$$

Moment Frame Direction

$CH_1 := 13 \qquad CH_2 := 13 \qquad CH_3 := 13$

$$H_{total} := \sum_{i=1}^{3} CH_i = 39 \qquad I_e := 1 \qquad C_{s\_min} := 0.044 S_{ds} \cdot I_e = 0.061$$

$R_m := 8 \qquad C_t := 0.028 \qquad x := 0.8 \qquad C_{dm} := 5.5 \qquad \Omega_{om} := 3$

$$T_{am} := C_t \cdot H_{total}{}^x = 0.525 \qquad C_u := 1.4 \qquad T_m := C_u \cdot T_{am} = 0.735$$

$$C_{s1} := \frac{S_{ds}}{\left(\frac{R_m}{I_e}\right)} = 0.174 \qquad C_{s2} := \frac{S_{d1}}{T_m \cdot \left(\frac{R_m}{I_e}\right)} = 0.131$$

$$C_{sm} := C_{s2}$$

$$H := \begin{pmatrix} 13 \\ 26 \\ 39 \end{pmatrix}$$

$$V_m := C_{sm} \cdot W_{total} = 779.972 \qquad k := 1 + \frac{(T_{am} - 0.5)}{2} = 1.012$$

$$CVT := \sum_{j=1}^{3} \left[W_j \cdot (H_j)^k \right] = 1.582 \times 10^5$$

$$i := 1..3$$

$$CV_i := \frac{W_i \cdot (H_i)^k}{CVT} \qquad CV = \begin{pmatrix} 0.175 \\ 0.353 \\ 0.472 \end{pmatrix} \qquad F_m := CV \cdot V_m = \begin{pmatrix} 136.383 \\ 275.122 \\ 368.467 \end{pmatrix}$$ storyforce(k)

$$w_{mEW} := \frac{F_m}{w_{EW}} = \begin{pmatrix} 0.758 \\ 1.528 \\ 2.047 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmEW} := F_m \cdot 0.05 w_{EW} = \begin{pmatrix} 1.227 \times 10^3 \\ 2.476 \times 10^3 \\ 3.316 \times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$w_{mNS} := \frac{F_m}{w_{NS}} = \begin{pmatrix} 1.137 \\ 2.293 \\ 3.071 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmNS} := F_m \cdot 0.05 w_{NS} = \begin{pmatrix} 818.297 \\ 1.651 \times 10^3 \\ 2.211 \times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$SS := \begin{pmatrix} F_{m_3} + F_{m_2} + F_{m_1} \\ F_{m_2} + F_{m_3} \\ F_{m_3} \end{pmatrix} = \begin{pmatrix} 779.972 \\ 643.589 \\ 368.467 \end{pmatrix}$$ Story Shears(k)

Joint Masses for Dynamic Model

$$W = \begin{pmatrix} 2.061\times 10^{3} \\ 2.061\times 10^{3} \\ 1.831\times 10^{3} \end{pmatrix} \quad \text{(kip)} \qquad W_{penthouse} := \begin{pmatrix} W_1 \\ W_2 \\ W_{withpenthouse} \end{pmatrix} = \begin{pmatrix} 2.061\times 10^{3} \\ 2.061\times 10^{3} \\ 2.485\times 10^{3} \end{pmatrix} \quad \text{kips}$$

$$W_{outpenthouse} := \begin{pmatrix} W_1 \\ W_2 \\ W_{withoutpenthouse} \end{pmatrix} = \begin{pmatrix} 2.061\times 10^{3} \\ 2.061\times 10^{3} \\ 1.772\times 10^{3} \end{pmatrix} \quad \text{kips}$$

$$M_{penthouse} := \frac{W_{penthouse}}{32.2} = \begin{pmatrix} 64.019 \\ 64.019 \\ 77.165 \end{pmatrix} \qquad M_{outpenthouse} := \frac{W_{outpenthouse}}{32.2} = \begin{pmatrix} 64.019 \\ 64.019 \\ 55.028 \end{pmatrix}$$

$$m_{penthouse} := \frac{M_{penthouse}}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 2.806\times 10^{-3} \\ 2.806\times 10^{-3} \\ 3.382\times 10^{-3} \end{pmatrix}$$

$$m_{outpenthouse} := \frac{M_{outpenthouse}}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 2.806\times 10^{-3} \\ 2.806\times 10^{-3} \\ 2.412\times 10^{-3} \end{pmatrix}$$

Determine joint masses based tributary area

$$A_c := \left(\frac{30}{2} + 2\right)\cdot\left(\frac{30}{2} + 2\right) = 289 \qquad A_{e\_NS} := \left(\frac{30}{2} + 2\right)\cdot 30 = 510$$

Note : NS means the walls running from north to south (East and West Faces)

$$A_{e\_EW} := \left(\frac{30}{2} + 2\right)\cdot 30 = 510 \qquad A_i := 30\cdot 30 = 900$$

$$m_c := m_{outpenthouse} \cdot A_c = \begin{pmatrix} 0.811 \\ 0.811 \\ 0.697 \end{pmatrix}$$

$$m_{e\_NS} := m_{outpenthouse} \cdot A_{e\_NS} = \begin{pmatrix} 1.431 \\ 1.431 \\ 1.23 \end{pmatrix}$$

$$m_{e\_EW} := m_{outpenthouse} \cdot A_{e\_EW} = \begin{pmatrix} 1.431 \\ 1.431 \\ 1.23 \end{pmatrix}$$

$$m_i := A_i \cdot m_{outpenthouse} = \begin{pmatrix} 2.525 \\ 2.525 \\ 2.171 \end{pmatrix}$$

$$m_{ipc} := A_i \cdot m_{outpenthouse} \cdot \frac{3}{4} + A_i \cdot m_{penthouse} \cdot \frac{1}{4} = \begin{pmatrix} 2.525 \\ 2.525 \\ 2.389 \end{pmatrix}$$

$$m_{ipe} := A_i \cdot m_{outpenthouse} \cdot \frac{2}{4} + A_i \cdot m_{penthouse} \cdot \frac{2}{4} = \begin{pmatrix} 2.525 \\ 2.525 \\ 2.607 \end{pmatrix}$$

Moment Frame Direction

$CH_1 := 13$ $\quad CH_2 := 13$ $\quad CH_3 := 13$

$H_{total} := \sum_{i=1}^{3} CH_i = 39$ $\quad I_e := 1$ $\quad C_{s\_min} := 0.044 S_{ds} \cdot I_e = 0.061$

$R_m := 8$ $\quad C_t := 0.028$ $\quad x := 0.8$ $\quad C_{dm} := 5.5$ $\quad \Omega_{om} := 3$

$T_{am} := C_t \cdot H_{total}^{\ x} = 0.525$ $\quad C_u := 1.4$ $\quad T_m := C_u \cdot T_{am} = 0.735$

I get the period T from the model in SAP2000.

$T_{mEW} := 1.255$

$$C_{s1} := \frac{S_{ds}}{\left(\frac{R_m}{I_e}\right)} = 0.174 \qquad C_{s2} := \frac{S_{d1}}{T_{mEW}\left(\frac{R_m}{I_e}\right)} = 0.077$$

$C_{sm} := C_{s2}$

$$H := \begin{pmatrix} 13 \\ 26 \\ 39 \end{pmatrix}$$

$V_m := C_{sm} \cdot W_{total} = 456.639$ $\qquad k := 1 + \frac{(T_{mEW} - 0.5)}{2} = 1.377$

$$CVT := \sum_{j=1}^{3} \left[W_j \cdot (H_j)^k \right] = 5.387 \times 10^5$$

$i := 1..3$

$$CV_i := \frac{W_i \cdot (H_i)^k}{CVT} \qquad CV = \begin{pmatrix} 0.131 \\ 0.34 \\ 0.529 \end{pmatrix} \qquad F_m := CV \cdot V_m = \begin{pmatrix} 59.824 \\ 155.433 \\ 241.382 \end{pmatrix}$$

storyforce(k)

$$w_{mEW} := \frac{F_m}{w_{EW}} = \begin{pmatrix} 0.332 \\ 0.864 \\ 1.341 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmEW} := F_m \cdot 0.05 w_{EW} = \begin{pmatrix} 538.414 \\ 1.399 \times 10^3 \\ 2.172 \times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$SS := \begin{pmatrix} F_{m_3} + F_{m_2} + F_{m_1} \\ F_{m_2} + F_{m_3} \\ F_{m_3} \end{pmatrix} = \begin{pmatrix} 456.639 \\ 396.815 \\ 241.382 \end{pmatrix}$$ Story Shears(k)

$$T_{mNS} := 1.392$$

$$C_{s1} := \frac{S_{ds}}{\left(\frac{R_m}{I_e}\right)} = 0.174 \qquad C_{s2} := \frac{S_{d1}}{T_{mNS} \cdot \left(\frac{R_m}{I_e}\right)} = 0.069$$

$$C_{sm} := C_{s2} \qquad H := \begin{pmatrix} 13 \\ 26 \\ 39 \end{pmatrix}$$

$$V_m := C_{sm} \cdot W_{total} = 411.697 \qquad k := 1 + \frac{(T_{mNS} - 0.5)}{2} = 1.446$$

$$CVT := \sum_{j=1}^{3} \left[W_j \cdot (H_j)^k \right] = 6.793 \times 10^5$$

$$i := 1..3$$

$$CV_i := \frac{W_i \cdot (H_i)^k}{CVT} \qquad CV = \begin{pmatrix} 0.124 \\ 0.337 \\ 0.539 \end{pmatrix} \qquad F_m := CV \cdot V_m = \begin{pmatrix} 50.986 \\ 138.911 \\ 221.8 \end{pmatrix}$$ storyforce(k)

$$w_{mNS} := \frac{F_m}{w_{NS}} = \begin{pmatrix} 0.425 \\ 1.158 \\ 1.848 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmNS} := F_m \cdot 0.05 w_{NS} = \begin{pmatrix} 305.914 \\ 833.466 \\ 1.331 \times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$SS := \begin{pmatrix} F_{m_3} + F_{m_2} + F_{m_1} \\ F_{m_2} + F_{m_3} \\ F_{m_3} \end{pmatrix} = \begin{pmatrix} 411.697 \\ 360.711 \\ 221.8 \end{pmatrix}$$ Story Shears(k)

Design information

Governing Code: 1994 Uniform Building Code

1997 NEHRP Recommended Provisions for Seismic

Regulations for New Buildings and Other Structures

Occupancy: Office-type occupancy

Location: Downtown Los Angeles

Site Characterization: Deep stiff soil

Site Class D per NEHRP Provisions

Floor-to-Floor Heights: Three-Story Building 13'-0" (3.96 m)

Six-Story Building

Penthouse 12'-0" (3.66 m)

First Story 18'-0" (5.49 m)

All Other Stories 13'-0" (3.96 m)

Building Weights: Steel Framing As Designed

Floors and Roof 3" (7.6 cm) Metal Deck with 2½"

(6.4 cm) Normal-Weight Concrete

Roofing 7 psf (34.2 kg/m2)

Ceilings/Flooring 3 psf (14.6 kg/m2)

Mechanical/Electrical 7 psf (34.2 kg/m2)

47 psf (229 kg/m2)at Penthouse

Partitions 20 psf (97.6 kg/m2)

(Gravity Design)

10 psf (48.8 kg/m2)

(Seismic Design and Analysis)

Exterior Wall 25 psf (122 kg/m2)

Dimensions Bay Size 30'-0" x 30'-0" (9.14 m by 9.14 m)

2'-0" (61.0 cm) Perimeter Wall Offset from Gridline

Three-Story Building 124'-0"x184'-0" (37.8 m by 56.1 m)

Six-Story Building 154'-0"x154'-0" (46.9 m by 46.9 m)

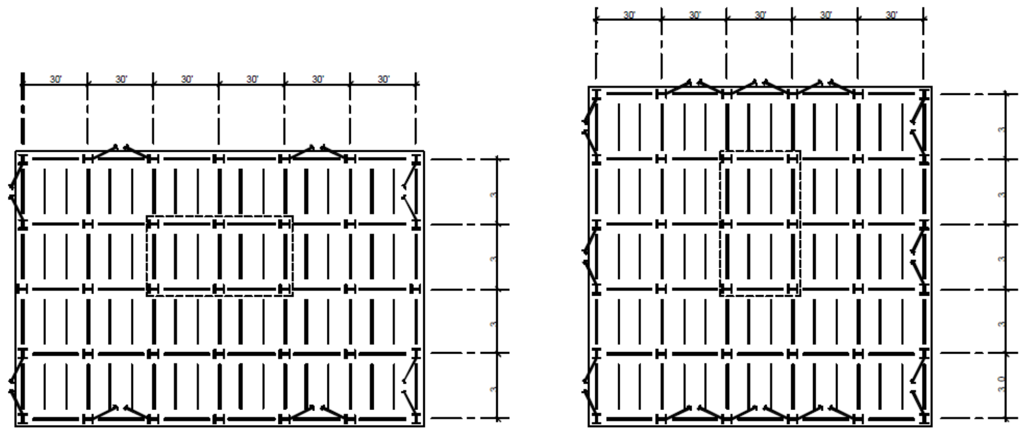
42" (107 cm) Parapets

Foundation Design Three-Story Building Spread Footings

Six-Story Building Pile Foundations

Live Loadings: 50 psf (2.39 MPa)

Wind Loadings: Exposure B per the Uniform Building Code.



Three-Story Building Six-Story Building

Figure 1
Plans of Three-Story and Six-Story Buildings

Preliminary Sizing Calculations

1) Column

Column Height

$CH_1 := 13 \ \text{ft} \qquad CH_2 := 13 \ \text{ft} \qquad CH_3 := 13 \ \text{ft}$

Building Dimension

$N - S \qquad w_{NS} := 120 \ \text{ft}$

$W - E \qquad w_{EW} := 180 \ \text{ft}$

Building Dimensions including cladding

$w_{NS\_Clad} := w_{NS} + 4 = 124 \ \text{ft}$

$w_{EW\_Clad} := w_{EW} + 4 = 184 \ \text{ft}$

Tributary areas

$$A_c := \left(\frac{30}{2} + 2\right)\cdot\left(\frac{30}{2} + 2\right) = 289 \qquad A_{e\_NS} := \left(\frac{30}{2} + 2\right)\cdot 30 = 510$$

$$A_{e\_EW} := \left(\frac{30}{2} + 2\right)\cdot 30 = 510 \qquad A_i := 30 \cdot 30 = 900$$

Dead Load

Material weights subject to change. Basic weight assumptions listed here:

Column Weight $w_{col} := 0.15$

HVAC/Mech/Plumbing $w_{mech} := 0.007$

Floor Weight $w_{floor} := 0.067$

Partition Weight $w_{par} := 0.02$

Roof Weight $w_{.roof} := 0.066$

Penthouse Mechnical/Electronic equipment $w_{Penthouse} := 0.047$

Cladding Weight $w_{clad} := 0.025$

42in. tall parapet at the roof

Parapet Height $H_{par} := 3.5$

Column Weight

$i := 1..3$

$$COL := CH \cdot w_{col} \cdot 24 = \begin{pmatrix} 46.8 \\ 46.8 \\ 46.8 \end{pmatrix} \quad \text{kips}$$

Cladding Weight

$$CLAD := w_{clad} \cdot CH \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = \begin{pmatrix} 200.2 \\ 200.2 \\ 200.2 \end{pmatrix} \quad \text{kips}$$

$$CLAD_3 := CLAD_3 + w_{clad} \cdot H_{par} \cdot \left[(w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 \right] = 254.1 \quad \text{kips}$$

$$CLAD = \begin{pmatrix} 200.2 \\ 200.2 \\ 254.1 \end{pmatrix} \quad \text{kips}$$

Dead Load

$$DD := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW}) \right] + CLAD + COL = \begin{pmatrix} 2.277 \times 10^3 \\ 2.277 \times 10^3 \\ 2.331 \times 10^3 \end{pmatrix} \quad \text{kips}$$

$$DL := \frac{DD}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 0.1 \\ 0.1 \\ 0.102 \end{pmatrix} \quad \text{ksf}$$

without the consideration of penthouse load

$$DD_3 := DD_3 + (w_{Penthouse} - w_{mech})\cdot(w_{NS}\cdot w_{EW}) = 3.195\times 10^3$$ kips

$$DD = \begin{pmatrix} 2.277\times 10^3 \\ 2.277\times 10^3 \\ 3.195\times 10^3 \end{pmatrix}$$ kips

$$DLP := \frac{DD}{w_{NS\_Clad}\cdot w_{EW\_Clad}} = \begin{pmatrix} 0.1 \\ 0.1 \\ 0.14 \end{pmatrix}$$ ksf

Live Load

$$LL := \begin{pmatrix} 0.05 \\ 0.05 \\ 0.05 \end{pmatrix}$$ ksf

Roof Live Load

Effective Length factor K

We assume the factor K is 1.5 for unbraced column.

$$K_{unbraced} := 1.5$$

Effective Length

Corner Column, E-W edge Column, Internal Column,N-S edge Column

$$L_{unbraced} := K_{unbraced}\cdot CH = \begin{pmatrix} 19.5 \\ 19.5 \\ 19.5 \end{pmatrix}$$ ft

Select the column size based on load combination 2

Interior columns out of Penthouse range

$$P_{int} := \sum_{i=1}^{2} \left[\left(1.2DL_i + 1.6LL_i\right)\cdot A_i\right] + \left(1.2DL_3 + 0.5LL_3\right)\cdot A_i = 492.455$$ kips

Interior columns within Penthouse range

$$P_{intpc} := \sum_{i=1}^{2} \left[\left(1.2DL_i + 1.6LL_i\right)\cdot A_i\right] + \left(1.2DL_3 + 0.5LL_3\right)\cdot 0.75A_i + \left(1.2DLP_3 + 0.5LL_3\right)\cdot 0.25A_i$$

$P_{intpc} = 502.679$ kips Penthouse Conner

$$P_{intpe} := \sum_{i=1}^{2} \left[\left(1.2DL_i + 1.6LL_i\right)\cdot A_i\right] + \left(1.2DL_3 + 0.5LL_3\right)\cdot 0.5A_i + \left(1.2DLP_3 + 0.5LL_3\right)\cdot 0.5A_i$$

$P_{intpe} = 512.904$ kips Penthouse Edge

$P_{e\_NS} := \frac{A_{e\_NS}}{A_i} \cdot P_{int} = 279.058$ kips $P_{e\_EW} := \frac{A_{e\_EW}}{A_i} \cdot P_{int} = 279.058$ kips

$P_c := \frac{A_c}{A_i} \cdot P_{int} = 158.133$ kips

Internal Column W14X74

Internal Penthouse Conner W14X82

Internal Penthouse Edge W14X82

E-W edge Column W14X61

Check External Column Size Under Seismic Combination

N-S edge Column W14X61

Check Seismic Combination $F_y := 50$

Corner Column W14X48

Interior columns out of Penthouse range

$$P_{ints} := \sum_{i=1}^{2} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i \right] + \left(1.2 DL_3 + 0.5 LL_3\right) \cdot A_i = 438.455 \text{ kips}$$

$$A_{intsr} := \frac{P_{ints}}{0.4 F_y} = 21.923 \text{ in}^2$$

Interior columns within Penthouse range

$$P_{intpcs} := \sum_{i=1}^{2} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i \right] + \left(1.2 DL_3 + 0.5 LL_3\right) \cdot 0.75 A_i + \left(1.2 DLP_3 + 0.5 LL_3\right) \cdot 0.25 A_i$$

$P_{intpcs} = 448.679$ kips Penthouse Conner

$$A_{intpcsr} := \frac{P_{intpcs}}{0.4 F_y} = 22.434 \text{ in}^2$$

$$P_{intpes} := \sum_{i=1}^{2} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i \right] + \left(1.2 DL_3 + 0.5 LL_3\right) \cdot 0.5 A_i + \left(1.2 DLP_3 + 0.5 LL_3\right) \cdot 0.5 A_i$$

$P_{intpes} = 458.904$ kips Penthouse Edge

$$A_{intpesr} := \frac{P_{intpes}}{0.4 F_y} = 22.945 \text{ in}^2$$

$P_{e\_NSs} := \frac{A_{e\_NS}}{A_i} \cdot P_{ints} = 248.458$ kips $P_{e\_EWs} := \frac{A_{e\_EW}}{A_i} \cdot P_{ints} = 248.458$ kips

$A_{e\_NSsr} := \frac{P_{e\_NSs}}{0.4 F_y} = 12.423$ in^2 $A_{e\_EWsr} := \frac{P_{e\_EWs}}{0.4 F_y} = 12.423$ in^2

$P_{cs} := \frac{A_c}{A_i} \cdot P_{ints} = 140.793$ kips $A_{csr} := \frac{P_{cs}}{0.4 F_y} = 7.04$ in^2

Internal Column W14X74 A=21.8 in^2

OK!

Internal Penthouse Conner W14X82 A=24 in^2

Internal Penthouse Edge W14X82 A=24 in^2

E-W edge Column W14X61 A=17.9 in^2

N-S edge Column W14X61 A=17.9 in^2

Corner Column W14X48 A=14.1 in^2

2) Girders and Beams

Tributary area

$A_{GNSedge} := (30 + 2)\cdot 30 = 960$

$A_{GEWedge} := (30)\cdot\left(\frac{30}{2} + 2\right) = 510$

$A_{Gcenter} := 30\cdot 30 = 900$

$A_{Gcorner} := \left(\frac{30}{2} + 2\right)\cdot(30 + 2) = 544$

3 beams per girder. Spaced at 10' on center.

$A_{BNSedge} := (30)\cdot(5 + 2) = 210$

$A_{BEWedge} := (30 + 2)\cdot 10 = 320$

$A_{Bcenter} := 10\cdot 30 = 300$

$A_{Bcorner} := (5 + 2)\cdot(30 + 2) = 224$

Select the girder size based on load combination 2

We assume Dead Load, Live Load and Roof Live Load are uniform load in the girders.

Girders

$$DL_{GNSedge} := \frac{DL\cdot A_{GNSedge}}{30} = \begin{pmatrix} 3.194 \\ 3.194 \\ 3.27 \end{pmatrix} \quad klf$$

$$DL_{GEWedge} := \frac{DL\cdot A_{GEWedge}}{30} = \begin{pmatrix} 1.697 \\ 1.697 \\ 1.737 \end{pmatrix} \quad klf$$

$$DL_{GCenter} := \frac{DL\cdot A_{Gcenter}}{30} = \begin{pmatrix} 2.994 \\ 2.994 \\ 3.065 \end{pmatrix} \quad klf$$

$$DL_{GCorner} := \frac{DL\cdot A_{Gcorner}}{30} = \begin{pmatrix} 1.81 \\ 1.81 \\ 1.853 \end{pmatrix} \quad klf$$

$$DL_{GCenterp} := \frac{DLP\cdot 0.5A_{Gcenter} + DL\cdot 0.5A_{Gcenter}}{30}$$

$$DL_{GCenterp} = \begin{pmatrix} 2.994 \\ 2.994 \\ 3.633 \end{pmatrix} \quad klf$$

$$LL_{GNSedge} := \frac{LL\cdot A_{GNSedge}}{30} = \begin{pmatrix} 1.6 \\ 1.6 \\ 1.6 \end{pmatrix} \quad klf$$

$$LL_{GEWedge} := \frac{LL\cdot A_{GEWedge}}{30} = \begin{pmatrix} 0.85 \\ 0.85 \\ 0.85 \end{pmatrix} \quad klf$$

$LL_{GCenter} := \frac{LL \cdot A_{Gcenter}}{30} = \begin{pmatrix} 1.5 \\ 1.5 \\ 1.5 \end{pmatrix}$ klf $\quad LL_{Gcorner} := \frac{LL \cdot A_{Gcorner}}{30} = \begin{pmatrix} 0.907 \\ 0.907 \\ 0.907 \end{pmatrix}$ klf

$uniformload_{GNSedge} := 1.2 \cdot DL_{GNSedge} + 1.6 \cdot LL_{GNSedge}$

$uniformload_{GNSedge_3} := 1.2 DL_{GNSedge_3} + 0.5 LL_{GNSedge_3}$

$uniformload_{GEWedge} := 1.2 \cdot DL_{GEWedge} + 1.6 \cdot LL_{GEWedge}$

$uniformload_{GEWedge_3} := 1.2 DL_{GEWedge_3} + 0.5 LL_{GEWedge_3}$

$uniformload_{GCenter} := 1.2 \cdot DL_{GCenter} + 1.6 \cdot LL_{GCenter}$

$uniformload_{GCenter_3} := 1.2 DL_{GCenter_3} + 0.5 LL_{GCenter_3}$

$uniformload_{GCenterp} := 1.2 \cdot DL_{GCenterp} + 1.6 \cdot LL_{GCenter}$

$uniformload_{GCenterp_3} := 1.2 DL_{GCenterp_3} + 0.5 LL_{GCenter_3}$

$uniformload_{GCorner} := 1.2 \cdot DL_{GCorner} + 1.6 \cdot LL_{Gcorner}$

$uniformload_{GCorner_3} := 1.2 DL_{GCorner_3} + 0.5 LL_{Gcorner_3}$

$uniformload_{GNSedge} = \begin{pmatrix} 6.393 \\ 6.393 \\ 4.724 \end{pmatrix} \quad uniformload_{GEWedge} = \begin{pmatrix} 3.396 \\ 3.396 \\ 2.509 \end{pmatrix}$

$uniformload_{GCenter} = \begin{pmatrix} 5.993 \\ 5.993 \\ 4.428 \end{pmatrix} \quad uniformload_{GCorner} = \begin{pmatrix} 3.623 \\ 3.623 \\ 2.677 \end{pmatrix}$

$uniformload_{GCenterp} = \begin{pmatrix} 5.993 \\ 5.993 \\ 5.11 \end{pmatrix}$

Assume the girders are simple supported, the Max Moment is 1/8XqXL^2X2/3 in the midspan for moment frame while /8XqXL^2 for other girders.

$L_{span} := 30$

$$M_{uGNS} := \frac{1}{8} \cdot uniformload_{GNSedge} \cdot L_{span}^{2} = \begin{pmatrix} 719.205 \\ 719.205 \\ 531.41 \end{pmatrix} \qquad \phi M_{nGNS} := \begin{pmatrix} 750 \\ 750 \\ 540 \end{pmatrix} \cdot kip \cdot ft$$

W24x7
W24x7
W21x6

$$M_{uGEW} := \frac{2}{3} \cdot \frac{1}{8} \cdot uniformload_{GEWedge} \cdot L_{span}^{2} = \begin{pmatrix} 254.718 \\ 254.718 \\ 188.208 \end{pmatrix} \qquad \phi M_{nGEW} := \begin{pmatrix} 294 \\ 294 \\ 203 \end{pmatrix} \cdot kip \cdot ft$$

W18x4
W18x4
W16x3

$$M_{uGCenter} := \frac{1}{8} \cdot uniformload_{GCenter} \cdot L_{span}^{2} = \begin{pmatrix} 674.254 \\ 674.254 \\ 498.197 \end{pmatrix} \qquad \phi M_{nGCen} := \begin{pmatrix} 750 \\ 750 \\ 540 \end{pmatrix} \cdot kip \cdot ft$$

W24x7
W24x7
W21x6

$$M_{uGCorner} := \frac{2}{3} \cdot \frac{1}{8} \cdot uniformload_{GCorner} \cdot L_{span}^{2} = \begin{pmatrix} 271.7 \\ 271.7 \\ 200.755 \end{pmatrix} \qquad \phi M_{nGCor} := \begin{pmatrix} 294 \\ 294 \\ 203 \end{pmatrix} \cdot kip \cdot ft$$

W18x4
W18X4
W16x3

$$M_{uGCenterp} := \frac{1}{8} \cdot uniformload_{GCenterp} \cdot L_{span}^{2} = \begin{pmatrix} 674.254 \\ 674.254 \\ 574.88 \end{pmatrix} \qquad \phi M_{nGCenp} := \begin{pmatrix} 750 \\ 750 \\ 600 \end{pmatrix} \cdot kip \cdot ft$$

W24x7
W24x7
W21x6

Choose appropriate Girder sizes from AISC Steel Construction Manual

Design Tables: 3-2
W24x76 for all NS-edge Girders, except roof. Use W21x62 for roof NS-edge Girders

W18x40 for all EW-edge Girders, except roof. Use W16x31 for roof EW-edge Girders

W24x76 for all Center Girders, except roof. Use W21x62 for roof Center Girders

W24x76 for all Center Penthouse Girders, except roof. Use W21x68 for roof Center Girders

W18x40 for all Corner Girders, except roof. Use W16x31 for roof Corner Girders

Beams

$$DL_{BNSedge} := \frac{DL \cdot A_{BNSedge}}{30} = \begin{pmatrix} 0.699 \\ 0.699 \\ 0.715 \end{pmatrix} \quad klf \qquad DL_{BEWedge} := \frac{DL \cdot A_{BEWedge}}{30} = \begin{pmatrix} 1.065 \\ 1.065 \\ 1.09 \end{pmatrix} \quad klf$$

$$DL_{BCenter} := \frac{DL \cdot A_{Bcenter}}{30} = \begin{pmatrix} 0.998 \\ 0.998 \\ 1.022 \end{pmatrix} \quad klf \qquad DL_{BCorner} := \frac{DL \cdot A_{Bcorner}}{30} = \begin{pmatrix} 0.745 \\ 0.745 \\ 0.763 \end{pmatrix} \quad klf$$

$$DL_{BCenterpe} := \frac{DL \cdot 0.5A_{Bcenter} + DLP \cdot 0.5A_{Bcenter}}{30} = \begin{pmatrix} 0.998 \\ 0.998 \\ 1.211 \end{pmatrix} \quad klf$$

$$DL_{BCenterpc} := \frac{DLP \cdot A_{Bcenter}}{30} = \begin{pmatrix} 0.998 \\ 0.998 \\ 1.4 \end{pmatrix} \quad klf$$

$$LL_{BNSedge} := \frac{LL \cdot A_{BNSedge}}{30} = \begin{pmatrix} 0.35 \\ 0.35 \\ 0.35 \end{pmatrix} \quad klf \qquad LL_{BEWedge} := \frac{LL \cdot A_{BEWedge}}{30} = \begin{pmatrix} 0.533 \\ 0.533 \\ 0.533 \end{pmatrix} \quad klf$$

$$LL_{BCenter} := \frac{LL \cdot A_{Bcenter}}{30} = \begin{pmatrix} 0.5 \\ 0.5 \\ 0.5 \end{pmatrix} \quad klf \qquad LL_{Bcorner} := \frac{LL \cdot A_{Bcorner}}{30} = \begin{pmatrix} 0.373 \\ 0.373 \\ 0.373 \end{pmatrix} \quad klf$$

$$uniformload_{BNSedge} := 1.2 \cdot DL_{BNSedge} + 1.6 \cdot LL_{BNSedge}$$

$$uniformload_{BNSedge_3} := 1.2 DL_{BNSedge_3} + 0.5 LL_{BNSedge_3}$$

$$uniformload_{BEWedge} := 1.2 \cdot DL_{BEWedge} + 1.6 \cdot LL_{BEWedge}$$

$$uniformload_{BEWedge_3} := 1.2 DL_{BEWedge_3} + 0.5 LL_{BEWedge_3}$$

$$uniformload_{BCenter} := 1.2 \cdot DL_{BCenter} + 1.6 \cdot LL_{BCenter}$$

$$uniformload_{BCenter_3} := 1.2 DL_{BCenter_3} + 0.5 LL_{BCenter_3}$$

$$uniformload_{BCorner} := 1.2 \cdot DL_{BCorner} + 1.6 \cdot LL_{Bcorner}$$

$$uniformload_{BCorner_3} := 1.2 DL_{BCorner_3} + 0.5 LL_{Bcorner_3}$$

$$uniformload_{BCenterpc} := 1.2 \cdot DL_{BCenterpc} + 1.6 \cdot LL_{BCenter}$$

$uniformload_{BCenterpc_3} := 1.2DL_{BCenterpc_3} + 0.5LL_{BCenter_3}$

$uniformload_{BCenterpe} := 1.2 \cdot DL_{BCenterpe} + 1.6 \cdot LL_{BCenter}$

$uniformload_{BCenterpe_3} := 1.2DL_{BCenterpe_3} + 0.5LL_{BCenter_3}$

$uniformload_{BNSedge} = \begin{pmatrix} 1.398 \\ 1.398 \\ 1.033 \end{pmatrix}$ $uniformload_{BEWedge} = \begin{pmatrix} 2.131 \\ 2.131 \\ 1.575 \end{pmatrix}$

$uniformload_{BCenter} = \begin{pmatrix} 1.998 \\ 1.998 \\ 1.476 \end{pmatrix}$ $uniformload_{BCorner} = \begin{pmatrix} 1.492 \\ 1.492 \\ 1.102 \end{pmatrix}$

$uniformload_{BCenterpc} = \begin{pmatrix} 1.998 \\ 1.998 \\ 1.931 \end{pmatrix}$ $uniformload_{BCenterpe} = \begin{pmatrix} 1.998 \\ 1.998 \\ 1.703 \end{pmatrix}$

Assume the girders are simple supported, the Max Moment is 1/8XqXL^2 in the midspan.

$L_{span} := 3($

$M_{uBNS} := \frac{1}{8} \cdot uniformload_{BNSedge} \cdot L_{span}^{\ 2} = \begin{pmatrix} 157.326 \\ 157.326 \\ 116.246 \end{pmatrix}$ $\phi M_{nGNS} := \begin{pmatrix} 162 \\ 162 \\ 117 \end{pmatrix} \cdot kip \cdot ft$ W12x3(W10x2(

$M_{uBEW} := \frac{1}{8} \cdot uniformload_{BEWedge} \cdot L_{span}^{\ 2} = \begin{pmatrix} 239.735 \\ 239.735 \\ 177.137 \end{pmatrix}$ $\phi M_{nGEW} := \begin{pmatrix} 249 \\ 249 \\ 177 \end{pmatrix} \cdot kip \cdot ft$ W18x3: W14x3(

$M_{uBCen} := \frac{1}{8} \cdot uniformload_{BCenter} \cdot L_{span}^{\ 2} = \begin{pmatrix} 224.751 \\ 224.751 \\ 166.066 \end{pmatrix}$ $\phi M_{nGCen} := \begin{pmatrix} 249 \\ 249 \\ 177 \end{pmatrix} \cdot kip \cdot ft$ W18x3: W14x3(

$M_{uBCor} := \frac{1}{8} \cdot uniformload_{BCorner} \cdot L_{span}^{\ 2} = \begin{pmatrix} 167.814 \\ 167.814 \\ 123.996 \end{pmatrix}$ $\phi M_{nGCor} := \begin{pmatrix} 177 \\ 177 \\ 125 \end{pmatrix} \cdot kip \cdot ft$ W14x3(W14x2:

$$M_{uBCenpc} := \frac{1}{8} \cdot uniformload_{BCenterpc} \cdot L_{span}^{2} = \begin{pmatrix} 224.751 \\ 224.751 \\ 217.188 \end{pmatrix} \qquad \phi M_{nGCenpc} := \begin{pmatrix} 249 \\ 249 \\ 231 \end{pmatrix} \cdot kip \cdot ft$$

W18x3

W14x3

$$M_{uBCenpe} := \frac{1}{8} \cdot uniformload_{BCenterpe} \cdot L_{span}^{2} = \begin{pmatrix} 224.751 \\ 224.751 \\ 191.627 \end{pmatrix} \qquad \phi M_{nGCenpe} := \begin{pmatrix} 249 \\ 249 \\ 203 \end{pmatrix} \cdot kip \cdot ft$$

W18x3

W16x3

*Reminder - All beams run from North to South

W12x30 for all NS-edge beams, except roof. Use W10x26 for roof NS-edge beams.

W18x35 for all EW-edge beams, except roof. Use W14x30 for roof EW-edge beams.

W18x35 for all Center beams, except roof. Use W14x30 for roof Center beams

W18x35 for all Center Penthouse Beams, except roof. Use W14x38 for roof Penthouse Beams

W18x35 for all Edge Penthouse Beams, except roof. Use W16x31 for roof Penthouse Beams

W14x30 for all Corner beams, except roof. Use W14x22 for roof Corner beams

$ORIGIN \equiv 1$

6 story building

Site:Los Angeles,LA

Site Class D

Occupancy Category. II

$S_s := 2.09$ $F_a := 1.0$ $S_{ms} := F_a \cdot S_s = 2.09$ $S_{ds} := \frac{2}{3} \cdot S_{ms} = 1.393$

$S_1 := 0.77$ $F_v := 1.5$ $S_{m1} := F_v \cdot S_1 = 1.155$ $S_{d1} := \frac{2}{3} \cdot S_{m1} = 0.77$

Seismic Design Category: D

Column Height

$CH_1 := 18$ $CH_2 := 13$ $CH_3 := 13$ $CH_4 := 13$ $CH_5 := 13$ $CH_6 := 13$

Building Dimension

$N - S$ $w_{NS} := 150$

$W - E$ $w_{EW} := 150$

Building Dimensions including cladding

$w_{NS\_Clad} := w_{NS} + 4 = 154$

$w_{EW\_Clad} := w_{EW} + 4 = 154$

Material weights subject to change. Basic weight assumptions listed here:

Column Weight $w_{col} := 0.15$

Floor Weight $w_{floor} := 0.067$

Roof Weight $w_{.roof} := 0.066$

Cladding Weight $w_{clad} := 0.025$

HVAC/Mech/Plumbing $w_{mech} := 0.007$

Partition Weight $w_{par} := 0.01$

Penthouse Mechnical/Electronic equipment $w_{Penthouse} := 0.047$

42in. tall parapet at the roof

Parapet Height $H_{par} := 3.5$

Cladding Weight

$$w_{clad\_1} := w_{clad} \cdot \frac{(CH_1 + CH_2)}{2} \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = 238.7$$

$$w_{clad\_2} := w_{clad} \cdot \frac{(CH_2 + CH_3)}{2} \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = 200.2$$

$$w_{clad\_3} := w_{clad} \cdot \frac{(CH_3 + CH_4)}{2} \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = 200.2$$

$$w_{clad\_4} := w_{clad} \cdot \frac{(CH_4 + CH_5)}{2} \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = 200.2$$

$$w_{clad\_5} := w_{clad} \cdot \frac{(CH_5 + CH_6)}{2} \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = 200.2$$

$$w_{clad\_r} := w_{clad} \cdot \left(\frac{CH_6}{2} + H_{par} \right) \cdot (w_{EW\_Clad} + w_{NS\_Clad}) \cdot 2 = 154$$

Column Weight

$$w_{col\_1} := w_{col} \cdot \frac{(CH_1 + CH_2)}{2} \cdot 36 = 83.7$$

$$w_{col\_2} := w_{col} \cdot \frac{(CH_2 + CH_3)}{2} \cdot 36 = 70.2$$

$$w_{col\_3} := w_{col} \cdot \frac{(CH_3 + CH_4)}{2} \cdot 36 = 70.2$$

$$w_{col\_4} := w_{col} \cdot \frac{(CH_4 + CH_5)}{2} \cdot 36 = 70.2$$

$$w_{col\_5} := w_{col} \cdot \frac{(CH_5 + CH_6)}{2} \cdot 36 = 70.2$$

$$w_{col\_r} := w_{col} \cdot \frac{(CH_6)}{2} \cdot 36 = 35.1$$

Seismic Weight

$$W_1 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_1} + w_{col\_1} = 2.212 \times 10^3$$

$$W_2 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_2} + w_{col\_2} = 2.16 \times 10^3$$

$$W_3 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_3} + w_{col\_3} = 2.16 \times 10^3$$

$$W_4 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_4} + w_{col\_4} = 2.16 \times 10^3$$

$$W_5 := \left[(w_{floor} + w_{mech} + w_{par}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_5} + w_{col\_5} = 2.16 \times 10^3$$

$$W_{withoutpenthouse} := \left[(w_{floor} + w_{mech}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_r} + w_{col\_r} = 1.854 \times 10^3$$

$$W_{withpenthouse} := \left[(w_{floor} + w_{Penthouse} - w_{mech}) \cdot (w_{NS} \cdot w_{EW}) \right] + w_{clad\_r} + w_{col\_r} = 2.597 \times 10^3$$

$$W_r := \frac{11}{12} \cdot W_{withoutpenthouse} + \frac{1}{12} \cdot W_{withpenthouse} = 1.916 \times 10^3$$

$$W := \begin{pmatrix} W_1 \\ W_2 \\ W_3 \\ W_4 \\ W_5 \\ W_r \end{pmatrix} = \begin{pmatrix} 2.212 \times 10^3 \\ 2.16 \times 10^3 \\ 2.16 \times 10^3 \\ 2.16 \times 10^3 \\ 2.16 \times 10^3 \\ 1.916 \times 10^3 \end{pmatrix} \qquad W_{total} := W_1 + W_2 + W_3 + W_4 + W_5 + W_r = 1.277 \times 10^4$$

Moment Frame Direction

$CH_1 := 18 \quad CH_2 := 13 \quad CH_3 := 13 \quad CH_4 := 13 \quad CH_5 := 13 \quad CH_6 := 13$

$$H_{total} := \sum_{i=1}^{6} CH_i = 83 \qquad I_e := 1 \qquad C_{s\_min} := 0.044 S_{ds} \cdot I_e = 0.061$$

$R_m := 8 \quad C_t := 0.028 \quad x := 0.8 \quad C_{dm} := 5.5 \quad \Omega_{om} := 3$

$$T_{am} := C_t \cdot H_{total}{}^x = 0.96 \qquad C_u := 1.4 \qquad T_m := C_u \cdot T_{am} = 1.344$$

$$C_{s1} := \frac{S_{ds}}{\left(\frac{R_m}{I_e}\right)} = 0.174 \qquad C_{s2} := \frac{S_{d1}}{T_m \cdot \left(\frac{R_m}{I_e}\right)} = 0.072 \qquad H := \begin{pmatrix} 18 \\ 31 \\ 44 \\ 57 \\ 70 \\ 83 \end{pmatrix}$$

$C_{sm} := C_{s2}$

$$V_m := C_{sm} \cdot W_{total} = 914.202 \qquad k := 1 + \frac{(T_{am} - 0.5)}{2} = 1.23$$

$$CVT := \sum_{j=1}^{6} \left[W_j \cdot (H_j)^k \right] = 1.606 \times 10^6$$

$i := 1..6$

$$CV_i := \frac{W_i \cdot (H_i)^k}{CVT} \qquad CV = \begin{pmatrix} 0.048 \\ 0.092 \\ 0.141 \\ 0.194 \\ 0.25 \\ 0.274 \end{pmatrix} \qquad F_m := CV \cdot V_m = \begin{pmatrix} 44.084 \\ 84.019 \\ 129.263 \\ 177.735 \\ 228.84 \\ 250.262 \end{pmatrix}$$

storyforce(k)

$$w_{mEW} := \frac{F_m}{w_{EW}} = \begin{pmatrix} 0.294 \\ 0.56 \\ 0.862 \\ 1.185 \\ 1.526 \\ 1.668 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmEW} := F_m \cdot 0.05 w_{EW} = \begin{pmatrix} 330.628 \\ 630.142 \\ 969.473 \\ 1.333\times 10^3 \\ 1.716\times 10^3 \\ 1.877\times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$w_{mNS} := \frac{F_m}{w_{NS}} = \begin{pmatrix} 0.294 \\ 0.56 \\ 0.862 \\ 1.185 \\ 1.526 \\ 1.668 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmEW} := F_m \cdot 0.05 w_{NS} = \begin{pmatrix} 330.628 \\ 630.142 \\ 969.473 \\ 1.333\times 10^3 \\ 1.716\times 10^3 \\ 1.877\times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$SS := \begin{pmatrix} F_{m_1} + F_{m_2} + F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_2} + F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_5} + F_{m_6} \\ F_{m_6} \end{pmatrix} = \begin{pmatrix} 914.202 \\ 870.118 \\ 786.099 \\ 656.836 \\ 479.102 \\ 250.262 \end{pmatrix}$$ Story Shears(k)

Joint Masses for Dynamic Model

$$W = \begin{pmatrix} 2.212\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 1.916\times 10^{3} \end{pmatrix} \quad \text{(kip)} \qquad W_{penthouse} := \begin{pmatrix} W_1 \\ W_2 \\ W_3 \\ W_4 \\ W_5 \\ W_{withpenthouse} \end{pmatrix} = \begin{pmatrix} 2.212\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.597\times 10^{3} \end{pmatrix} \quad \text{kips}$$

$$W_{outpenthouse} := \begin{pmatrix} W_1 \\ W_2 \\ W_3 \\ W_4 \\ W_5 \\ W_{withoutpenthouse} \end{pmatrix} = \begin{pmatrix} 2.212\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 2.16\times 10^{3} \\ 1.854\times 10^{3} \end{pmatrix} \quad \text{kips}$$

$$M_{penthouse} := \frac{W_{penthouse}}{32.2} = \begin{pmatrix} 68.708 \\ 67.093 \\ 67.093 \\ 67.093 \\ 67.093 \\ 80.64 \end{pmatrix} \qquad M_{outpenthouse} := \frac{W_{outpenthouse}}{32.2} = \begin{pmatrix} 68.708 \\ 67.093 \\ 67.093 \\ 67.093 \\ 67.093 \\ 57.581 \end{pmatrix}$$

$$m_{penthouse} := \frac{M_{penthouse}}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 2.897\times 10^{-3} \\ 2.829\times 10^{-3} \\ 2.829\times 10^{-3} \\ 2.829\times 10^{-3} \\ 2.829\times 10^{-3} \\ 3.4\times 10^{-3} \end{pmatrix}$$

$$m_{outpenthouse} := \frac{M_{outpenthouse}}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 2.897 \times 10^{-3} \\ 2.829 \times 10^{-3} \\ 2.829 \times 10^{-3} \\ 2.829 \times 10^{-3} \\ 2.829 \times 10^{-3} \\ 2.428 \times 10^{-3} \end{pmatrix}$$

Determine joint masses based tributary area

$$A_c := \left(\frac{30}{2} + 2\right) \cdot \left(\frac{30}{2} + 2\right) = 289 \qquad A_{e\_NS} := \left(\frac{30}{2} + 2\right) \cdot 30 = 510$$

Note : NS means the walls running from north to south (East and West Faces)

$$A_{e\_EW} := \left(\frac{30}{2} + 2\right) \cdot 30 = 510 \qquad A_i := 30 \cdot 30 = 900$$

$$m_c := m_{outpenthouse} \cdot A_c = \begin{pmatrix} 0.837 \\ 0.818 \\ 0.818 \\ 0.818 \\ 0.818 \\ 0.702 \end{pmatrix} \qquad m_{e\_NS} := m_{outpenthouse} \cdot A_{e\_NS} = \begin{pmatrix} 1.478 \\ 1.443 \\ 1.443 \\ 1.443 \\ 1.443 \\ 1.238 \end{pmatrix}$$

$$m_{e\_EW} := m_{outpenthouse} \cdot A_{e\_EW} = \begin{pmatrix} 1.478 \\ 1.443 \\ 1.443 \\ 1.443 \\ 1.443 \\ 1.238 \end{pmatrix} \qquad m_i := A_i \cdot m_{outpenthouse} = \begin{pmatrix} 2.607 \\ 2.546 \\ 2.546 \\ 2.546 \\ 2.546 \\ 2.185 \end{pmatrix}$$

$$m_{ipc} := A_i \cdot m_{outpenthouse} \cdot \frac{3}{4} + A_i \cdot m_{penthouse} \cdot \frac{1}{4} = \begin{pmatrix} 2.607 \\ 2.546 \\ 2.546 \\ 2.546 \\ 2.546 \\ 2.404 \end{pmatrix}$$

$$m_{ipe} := A_i \cdot m_{outpenthouse} \cdot \frac{2}{4} + A_i \cdot m_{penthouse} \cdot \frac{2}{4} = \begin{pmatrix} 2.607 \\ 2.546 \\ 2.546 \\ 2.546 \\ 2.546 \\ 2.623 \end{pmatrix}$$

Moment Frame Direction

$CH_1 = 18 \quad CH_2 = 13 \quad CH_3 = 13 \quad CH_4 = 13 \quad CH_5 = 13 \quad CH_6 = 13$

$H_{total} := \sum_{i=1}^{3} CH_i = 44 \qquad I_e := 1 \qquad C_{s\_min} := 0.044 S_{ds} \cdot I_e = 0.061$

$R_m := 8 \qquad C_t := 0.028 \qquad x := 0.8 \qquad C_{dm} := 5.5 \qquad \Omega_{om} := 3$

$T_{am} := C_t \cdot H_{total}^{\,x} = 0.578 \qquad C_u := 1.4 \qquad T_m := C_u \cdot T_{am} = 0.809$

I get the period T from the model in SAP2000.

$T_{mEW} := 2.11$

$$C_{s1} := \frac{S_{ds}}{\left(\frac{R_m}{I_e}\right)} = 0.174 \qquad C_{s2} := \frac{S_{d1}}{T_{mEW}\left(\frac{R_m}{I_e}\right)} = 0.046$$

$C_{sm} := C_{s2}$

$$H := \begin{pmatrix} 18 \\ 31 \\ 44 \\ 57 \\ 70 \\ 83 \end{pmatrix}$$

$V_m := C_{sm} \cdot W_{total} = 582.517 \qquad k := 1 + \frac{(T_{mEW} - 0.5)}{2} = 1.805$

$$CVT := \sum_{j=1}^{6} \left[W_j \cdot \left(H_j\right)^k \right] = 1.686 \times 10^7$$

$i := 1..6$

$$CV_i := \frac{W_i \cdot \left(H_i\right)^k}{CVT} \qquad CV = \begin{pmatrix} 0.024 \\ 0.063 \\ 0.119 \\ 0.189 \\ 0.274 \\ 0.331 \end{pmatrix} \qquad F_m := CV \cdot V_m = \begin{pmatrix} 14.095 \\ 36.718 \\ 69.089 \\ 110.238 \\ 159.727 \\ 192.649 \end{pmatrix}$$

storyforce(k)

$$w_{mEW} := \frac{F_m}{w_{EW}} = \begin{pmatrix} 0.094 \\ 0.245 \\ 0.461 \\ 0.735 \\ 1.065 \\ 1.284 \end{pmatrix}$$ Dist story force(k/ft)

$$M_{TmEW} := F_m \cdot 0.05 w_{EW} = \begin{pmatrix} 105.714 \\ 275.388 \\ 518.167 \\ 826.783 \\ 1.198 \times 10^3 \\ 1.445 \times 10^3 \end{pmatrix}$$ Story Torsional Moment (k-ft)

$$SS := \begin{pmatrix} F_{m_1} + F_{m_2} + F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_2} + F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_5} + F_{m_6} \\ F_{m_6} \end{pmatrix} = \begin{pmatrix} 582.517 \\ 568.421 \\ 531.703 \\ 462.614 \\ 352.376 \\ 192.649 \end{pmatrix}$$ Story Shears(k)

$T_{mNS} := 1.9\text{4}$

$$C_{s1} := \frac{S_{ds}}{\left(\frac{R_m}{I_e}\right)} = 0.174 \qquad C_{s2} := \frac{S_{d1}}{T_{mNS}\left(\frac{R_m}{I_e}\right)} = 0.05 \qquad H := \begin{pmatrix} 18 \\ 31 \\ 44 \\ 57 \\ 70 \\ 83 \end{pmatrix}$$

$C_{sm} := C_{s2}$

$$V_m := C_{sm} \cdot W_{total} = 633.562 \qquad k := 1 + \frac{(T_{mNS} - 0.5)}{2} = 1.72$$

$$CVT := \sum_{j=1}^{6} \left[W_j \cdot (H_j)^k \right] = 1.188 \times 10^7$$

$i := 1..6$

$$CV_i := \frac{W_i \cdot (H_i)^k}{CVT} \qquad CV = \begin{pmatrix} 0.027 \\ 0.067 \\ 0.122 \\ 0.191 \\ 0.271 \\ 0.322 \end{pmatrix} \qquad F_m := CV \cdot V_m = \begin{pmatrix} 17.022 \\ 42.34 \\ 77.33 \\ 120.702 \\ 171.862 \\ 204.306 \end{pmatrix}$$

storyforce(k)

$$w_{mNS} := \frac{F_m}{w_{NS}} = \begin{pmatrix} 0.113 \\ 0.282 \\ 0.516 \\ 0.805 \\ 1.146 \\ 1.362 \end{pmatrix}$$

Dist story force(k/ft)

$$M_{TmNS} := F_m \cdot 0.05 w_{NS} = \begin{pmatrix} 127.664 \\ 317.551 \\ 579.976 \\ 905.267 \\ 1.289 \times 10^3 \\ 1.532 \times 10^3 \end{pmatrix}$$

Story Torsional Moment (k-ft)

$$SS := \begin{pmatrix} F_{m_1} + F_{m_2} + F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_2} + F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_3} + F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_4} + F_{m_5} + F_{m_6} \\ F_{m_5} + F_{m_6} \\ F_{m_6} \end{pmatrix} = \begin{pmatrix} 633.562 \\ 616.54 \\ 574.2 \\ 496.87 \\ 376.167 \\ 204.306 \end{pmatrix}$$

Story Shears(k)

$ORIGIN \equiv 1$

Preliminary Sizing Calculations---6 story

1) Column

We set column splices 4 ft above 4th floor for each column.

Column Height

$CH_1 := 18$ $\quad CH_2 := 13$ $\quad CH_3 := 13$ $\quad CH_4 := 13$ $\quad CH_5 := 13$ $\quad CH_6 := 13$

Building Dimension

$N - S \qquad w_{NS} := 150$

$W - E \qquad w_{EW} := 150$

Building Dimensions including cladding

$w_{NS\_Clad} := w_{NS} + 4 = 154 \quad ft$

$w_{EW\_Clad} := w_{EW} + 4 = 154 \quad ft$

Tributary area

$A_c := \left(\frac{30}{2} + 2\right)\cdot\left(\frac{30}{2} + 2\right) = 289 \qquad A_{e\_NS} := \left(\frac{30}{2} + 2\right)\cdot 30 = 510$

$A_{e\_EW} := \left(\frac{30}{2} + 2\right)\cdot 30 = 510 \qquad A_i := 30\cdot 30 = 900$

Dead Load

Material weights subject to change. Basic weight assumptions listed here:

Column Weight $\quad w_{col} := 0.15$ $\qquad$ HVAC/Mech/Plumbing $\quad w_{mech} := 0.007$

Floor Weight $\quad w_{floor} := 0.067$ $\qquad$ Partition Weight $\quad w_{par} := 0.02$

Roof Weight $\quad w_{.roof} := 0.066$ $\qquad$ Penthouse Mechnical/Electronic equipment $\quad w_{Penthouse} := 0.047$

Cladding Weight $\quad w_{clad} := 0.025$

42in. tall parapet at the roof

Parapet Height $\quad H_{par} := 3.5$

Column Weight

$i := 1..6$

$$COL := CH\cdot w_{col}\cdot 36 = \begin{array}{c} w \\ \begin{pmatrix} 97.2 \\ 70.2 \\ 70.2 \\ 70.2 \\ 70.2 \\ 70.2 \end{pmatrix} \end{array} \quad kips$$

Cladding Weight

$$CLAD := w_{clad} \cdot CH \cdot \left(w_{EW\_Clad} + w_{NS\_Clad}\right) \cdot 2 = \begin{pmatrix} 277.2 \\ 200.2 \\ 200.2 \\ 200.2 \\ 200.2 \\ 200.2 \end{pmatrix} \quad kips$$

$$CLAD_6 := CLAD_6 + w_{clad} \cdot H_{par} \cdot \left[\left(w_{EW\_Clad} + w_{NS\_Clad}\right) \cdot 2\right] = 254.1 \quad kips$$

Dead Load

$$DD := \left[\left(w_{floor} + w_{mech} + w_{par}\right) \cdot \left(w_{NS} \cdot w_{EW}\right)\right] + CLAD + COL = \begin{pmatrix} 2.489 \times 10^3 \\ 2.385 \times 10^3 \\ 2.385 \times 10^3 \\ 2.385 \times 10^3 \\ 2.385 \times 10^3 \\ 2.439 \times 10^3 \end{pmatrix} \quad kips$$

$$DL := \frac{DD}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 0.105 \\ 0.101 \\ 0.101 \\ 0.101 \\ 0.101 \\ 0.103 \end{pmatrix} \quad ksf$$

without the consideration of penthouse load

$$DD_6 := DD_6 + \left(w_{Penthouse} - w_{mech}\right) \cdot \left(w_{NS} \cdot w_{EW}\right) = 3.339 \times 10^3 \quad kips$$

$$DD = \begin{pmatrix} 2.489 \times 10^3 \\ 2.385 \times 10^3 \\ 2.385 \times 10^3 \\ 2.385 \times 10^3 \\ 2.385 \times 10^3 \\ 3.339 \times 10^3 \end{pmatrix} \quad kips$$

$$DLP := \frac{DD}{w_{NS\_Clad} \cdot w_{EW\_Clad}} = \begin{pmatrix} 0.105 \\ 0.101 \\ 0.101 \\ 0.101 \\ 0.101 \\ 0.141 \end{pmatrix} \quad ksf$$

Live Load

$$LL := \begin{pmatrix} 0.05 \\ 0.05 \\ 0.05 \\ 0.05 \\ 0.05 \\ 0.05 \end{pmatrix} \quad ksf$$

Roof Live Load

Effective Length factor K

We assume the factor K is 2 for unbraced column.

$$K_{unbraced} := 2$$

Effective Length
Corner Column, E-W edge Column, Internal Column,N-S edge Column

$$L_{unbraced} := K_{unbraced} \cdot CH = \begin{pmatrix} 36 \\ 26 \\ 26 \\ 26 \\ 26 \\ 26 \end{pmatrix} \quad ft$$

Select the column size based on load combination 2

Select column above 4th floor

Interior columns out of Penthouse range

$$P_{int4} := \sum_{i=4}^{5} \left[\left(1.2 DL_i + 1.6 LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot A_i = 494.84 \quad kips$$

Interior columns within Penthouse range

$$P_{int4pc} := \sum_{i=4}^{5} \left[\left(1.2 DL_i + 1.6 LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot 0.75 A_i + \left(1.2 DLP_6 + 0.5 LL_6\right) \cdot 0.25 A_i$$

$P_{int4pc} = 505.086$ kips Penthouse Conner

$$P_{int4pe} := \sum_{i=4}^{5} \left[\left(1.2\,DL_i + 1.6\,LL_i\right)\cdot A_i\right] + \left(1.2\,DL_6 + 0.5\,LL_6\right)\cdot 0.5\,A_i + \left(1.2\,DLP_6 + 0.5\,LL_6\right)\cdot 0.5\,A_i$$

$P_{int4pe} = 515.332$ kips Penthouse Edge

$P_{e\_NS5} := \dfrac{A_{e\_NS}}{A_i}\cdot P_{int4} = 280.409$ kips $\quad P_{e\_EW5} := \dfrac{A_{e\_EW}}{A_i}\cdot P_{int4} = 280.409$ kips

$P_{c5} := \dfrac{A_c}{A_i}\cdot P_{int4} = 158.899$ kips

Internal Column W14X90

Internal Penthouse Conner W14X90

Internal Penthouse Edge W14X90

E-W edge Column W14X68

Select column above 1st floor

N-S edge Column W14X68

Interior columns out of Penthouse range

Corner Column W14X61

$$P_{int1} := \sum_{i=1}^{5} \left[\left(1.2\,DL_i + 1.6\,LL_i\right)\cdot A_i\right] + \left(1.2\,DL_6 + 0.5\,LL_6\right)\cdot A_i = 1.041\times 10^3 \text{ kips}$$

Interior columns within Penthouse range

$$P_{int1pc} := \sum_{i=1}^{5} \left[\left(1.2\,DL_i + 1.6\,LL_i\right)\cdot A_i\right] + \left(1.2\,DL_6 + 0.5\,LL_6\right)\cdot 0.75\,A_i + \left(1.2\,DLP_6 + 0.5\,LL_6\right)\cdot 0.25\,A_i$$

$P_{int1pc} = 1.052\times 10^3$ kips Penthouse Conner

$$P_{int1pe} := \sum_{i=1}^{5} \left[\left(1.2\,DL_i + 1.6\,LL_i\right)\cdot A_i\right] + \left(1.2\,DL_6 + 0.5\,LL_6\right)\cdot 0.5\,A_i + \left(1.2\,DLP_6 + 0.5\,LL_6\right)\cdot 0.5\,A_i$$

$P_{int1pe} = 1.062\times 10^3$ kips Penthouse Edge

$P_{e\_NS1} := \dfrac{A_{e\_NS}}{A_i}\cdot P_{int1} = 590.161$ kips $\quad P_{e\_EW1} := \dfrac{A_{e\_EW}}{A_i}\cdot P_{int1} = 590.161$ kips

$P_{c1} := \dfrac{A_c}{A_i}\cdot P_{int1} = 334.425$ kips

Internal Column W14X193

Internal Penthouse Conner W14X193

Internal Penthouse Edge W14X193

E-W edge Column W14X120

Check External Column Size Under Seismic Combination

N-S edge Column W14X120

Check Seismic Combination $F_y := 50$

Corner Column W14X90

Column above 4th floor

Interior columns out of Penthouse range

$$P_{int4s} := \sum_{i=4}^{5} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot A_i = 440.84 \quad \text{kips}$$

$$A_{int4sr} := \frac{P_{int4s}}{0.4 F_y} = 22.042 \quad in^2$$

Interior columns within Penthouse range

$$P_{intpc4s} := \sum_{i=4}^{5} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot 0.75 A_i + \left(1.2 DLP_6 + 0.5 LL_6\right) \cdot 0.25 A_i$$

$P_{intpc4s} = 451.086$ kips Penthouse Conner $A_{intpc4sr} := \frac{P_{intpc4s}}{0.4 F_y} = 22.554 \quad in^2$

$$P_{intpe4s} := \sum_{i=4}^{5} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot 0.5 A_i + \left(1.2 DLP_6 + 0.5 LL_6\right) \cdot 0.5 A_i$$

$P_{intpe4s} = 461.332$ kips Penthouse Edge $A_{intpe4sr} := \frac{P_{intpe4s}}{0.4 F_y} = 23.067 \quad in^2$

$P_{e\_NS4s} := \frac{A_{e\_NS}}{A_i} \cdot P_{int4s} = 249.809$ kips $P_{e\_EW4s} := \frac{A_{e\_EW}}{A_i} \cdot P_{int4s} = 249.809$ kips

$A_{e\_NS4sr} := \frac{P_{e\_NS4s}}{0.4 F_y} = 12.49 \quad in^2$ $A_{e\_EW4sr} := \frac{P_{e\_EW4s}}{0.4 F_y} = 12.49 \quad in^2$

$P_{c4s} := \frac{A_c}{A_i} \cdot P_{int4s} = 141.559$ kips $A_{c4sr} := \frac{P_{c4s}}{0.4 F_y} = 7.078 \quad in^2$

Internal Column W14X90 A=26.5 in^2

OK!

Internal Penthouse Conner W14X90 A=26.5 in^2

Internal Penthouse Edge W14X90 A=26.5 in^2

E-W edge Column W14X68 A=20 in^2

N-S edge Column W14X68 A=20 in^2

Corner Column W14X61 A=17.9 in^2

Column above 1st floor

Interior columns out of Penthouse range

$$P_{int1s} := \sum_{i=1}^{5} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot A_i = 906.461 \quad \text{kips}$$

$$A_{int1sr} := \frac{P_{int1s}}{0.4 F_y} = 45.323 \quad in^2$$

Interior columns within Penthouse range

$$P_{intpc1s} := \sum_{i=1}^{5} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot 0.75 A_i + \left(1.2 DLP_6 + 0.5 LL_6\right) \cdot 0.25 A_i$$

$P_{intpc1s} = 916.707$ kips Penthouse Conner $A_{intpc1sr} := \frac{P_{intpc1s}}{0.4 F_y} = 45.835 \quad in^2$

$$P_{intpe1s} := \sum_{i=1}^{5} \left[\left(1.2 DL_i + 1 \cdot LL_i\right) \cdot A_i\right] + \left(1.2 DL_6 + 0.5 LL_6\right) \cdot 0.5 A_i + \left(1.2 DLP_6 + 0.5 LL_6\right) \cdot 0.5 A_i$$

$P_{intpe1s} = 926.954$ kips Penthouse Edge $A_{intpe1sr} := \frac{P_{intpe1s}}{0.4 F_y} = 46.348 \quad in^2$

$P_{e\_NS1s} := \frac{A_{e\_NS}}{A_i} \cdot P_{int1s} = 513.661$ kips $P_{e\_EW1s} := \frac{A_{e\_EW}}{A_i} \cdot P_{int1s} = 513.661$ kips

$A_{e\_NS1sr} := \frac{P_{e\_NS1s}}{0.4 F_y} = 25.683 \quad in^2$ $A_{e\_EW1sr} := \frac{P_{e\_EW1s}}{0.4 F_y} = 25.683 \quad in^2$

$P_{c1s} := \frac{A_c}{A_i} \cdot P_{int1s} = 291.075$ kips $A_{c1sr} := \frac{P_{c1s}}{0.4 F_y} = 14.554 \quad in^2$

Internal Column W14X193 A=56.8 in^2

OK!

Internal Penthouse Conner W14X193 A=56.8 in^2

Internal Penthouse Edge W14X193 A=56.8 in^2

E-W edge Column W14X120 A=35.3 in^2

2) Girders and Beams

N-S edge Column W14X120 A=35.3 in^2

Tributary area

Corner Column W14X90 A=26.5 in^2

$A_{GNSedge} := (30 + 2) \cdot 30 = 960$ $A_{GEWedge} := (30) \cdot \left(\frac{30}{2} + 2\right) = 510$

$A_{Gcenter} := 30 \cdot 30 = 900$ $A_{Gcorner} := \left(\frac{30}{2} + 2\right) \cdot (30 + 2) = 544$

3 beams per girder. Spaced at 10' on center.

$A_{BNSedge} := (30)\cdot(5+2) = 210$ $\qquad$ $A_{BEWedge} := (30+2)\cdot 10 = 320$

$A_{Bcenter} := 10\cdot 30 = 300$ $\qquad$ $A_{Bcorner} := (5+2)\cdot(30+2) = 224$

Select the girder size based on load combination 2

We assume Dead Load, Live Load and Roof Live Load are uniform load in the girders.

Girders

$$DL_{GNSedge} := \frac{DL\cdot A_{GNSedge}}{30} = \begin{pmatrix} 3.359 \\ 3.219 \\ 3.219 \\ 3.219 \\ 3.219 \\ 3.291 \end{pmatrix} \text{klf} \qquad DL_{GEWedge} := \frac{DL\cdot A_{GEWedge}}{30} = \begin{pmatrix} 1.784 \\ 1.71 \\ 1.71 \\ 1.71 \\ 1.71 \\ 1.749 \end{pmatrix} \text{klf}$$

$$DL_{GCenter} := \frac{DL\cdot A_{Gcenter}}{30} = \begin{pmatrix} 3.149 \\ 3.017 \\ 3.017 \\ 3.017 \\ 3.017 \\ 3.086 \end{pmatrix} \text{klf} \qquad DL_{GCorner} := \frac{DL\cdot A_{Gcorner}}{30} = \begin{pmatrix} 1.903 \\ 1.824 \\ 1.824 \\ 1.824 \\ 1.824 \\ 1.865 \end{pmatrix} \text{klf}$$

$$DL_{GCenterpe} := \frac{DLP\cdot 0.5A_{Gcenter} + DL\cdot 0.5A_{Gcenter}}{30} \qquad DL_{GCenterpe} = \begin{pmatrix} 3.149 \\ 3.017 \\ 3.017 \\ 3.017 \\ 3.017 \\ 3.655 \end{pmatrix} \text{klf}$$

$$DL_{GCenterpc} := \frac{DLP\cdot A_{Gcenter}}{30} = \begin{pmatrix} 3.149 \\ 3.017 \\ 3.017 \\ 3.017 \\ 3.017 \\ 4.224 \end{pmatrix} \text{klf}$$

$$LL_{GNSedge} := \frac{LL \cdot A_{GNSedge}}{30} = \begin{pmatrix} 1.6 \\ 1.6 \\ 1.6 \\ 1.6 \\ 1.6 \\ 1.6 \end{pmatrix} \text{ klf} \qquad LL_{GEWedge} := \frac{LL \cdot A_{GEWedge}}{30} = \begin{pmatrix} 0.85 \\ 0.85 \\ 0.85 \\ 0.85 \\ 0.85 \\ 0.85 \end{pmatrix} \text{ klf}$$

$$LL_{GCenter} := \frac{LL \cdot A_{Gcenter}}{30} = \begin{pmatrix} 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \end{pmatrix} \text{ klf} \qquad LL_{Gcorner} := \frac{LL \cdot A_{Gcorner}}{30} = \begin{pmatrix} 0.907 \\ 0.907 \\ 0.907 \\ 0.907 \\ 0.907 \\ 0.907 \end{pmatrix} \text{ klf}$$

$$uniformload_{GNSedge} := 1.2 \cdot DL_{GNSedge} + 1.6 \cdot LL_{GNSedge}$$

$$uniformload_{GNSedge_6} := 1.2 DL_{GNSedge_6} + 0.5 LL_{GNSedge_6}$$

$$uniformload_{GEWedge} := 1.2 \cdot DL_{GEWedge} + 1.6 \cdot LL_{GEWedge}$$

$$uniformload_{GEWedge_6} := 1.2 DL_{GEWedge_6} + 0.5 LL_{GEWedge_6}$$

$$uniformload_{GCenter} := 1.2 \cdot DL_{GCenter} + 1.6 \cdot LL_{GCenter}$$

$$uniformload_{GCenter_6} := 1.2 DL_{GCenter_6} + 0.5 LL_{GCenter_6}$$

$$uniformload_{GCenterpe} := 1.2 \cdot DL_{GCenterpe} + 1.6 \cdot LL_{GCenter}$$

$$uniformload_{GCenterpe_6} := 1.2 DL_{GCenterpe_6} + 0.5 LL_{GCenter_6}$$

$$uniformload_{GCenterpc} := 1.2 \cdot DL_{GCenterpc} + 1.6 \cdot LL_{GCenter}$$

$$uniformload_{GCenterpc_6} := 1.2 DL_{GCenterpc_6} + 0.5 LL_{GCenter_6}$$

$$uniformload_{GCorner} := 1.2 \cdot DL_{GCorner} + 1.6 \cdot LL_{Gcorner}$$

$$uniformload_{GCorner_6} := 1.2 DL_{GCorner_6} + 0.5 LL_{Gcorner_6}$$

$$uniformload_{GNSedge} = \begin{pmatrix} 6.591 \\ 6.422 \\ 6.422 \\ 6.422 \\ 6.422 \\ 4.75 \end{pmatrix} \qquad uniformload_{GEWedge} = \begin{pmatrix} 3.501 \\ 3.412 \\ 3.412 \\ 3.412 \\ 3.412 \\ 2.523 \end{pmatrix}$$

$$uniformload_{GCenter} = \begin{pmatrix} 6.179 \\ 6.021 \\ 6.021 \\ 6.021 \\ 6.021 \\ 4.453 \end{pmatrix} \qquad uniformload_{GCorner} = \begin{pmatrix} 3.735 \\ 3.639 \\ 3.639 \\ 3.639 \\ 3.639 \\ 2.691 \end{pmatrix}$$

$$uniformload_{GCenterpe} = \begin{pmatrix} 6.179 \\ 6.021 \\ 6.021 \\ 6.021 \\ 6.021 \\ 5.136 \end{pmatrix} \qquad uniformload_{GCenterpc} = \begin{pmatrix} 6.179 \\ 6.021 \\ 6.021 \\ 6.021 \\ 6.021 \\ 5.819 \end{pmatrix}$$

Assume the girders are simple supported, the Max Moment is 1/8XqXL^2X2/3 in the midspan for moment frame while /8XqXL^2 for other girders.

$L_{span} := 3($

$$M_{uGNS} := \frac{1}{8}\cdot uniformload_{GNSedge}\cdot L_{span}{}^{2} = \begin{pmatrix} 741.458 \\ 722.514 \\ 722.514 \\ 722.514 \\ 722.514 \\ 534.332 \end{pmatrix} \qquad \phi M_{nGNS} := \begin{pmatrix} 750 \\ 750 \\ 750 \\ 750 \\ 750 \\ 540 \end{pmatrix}\cdot kip\cdot ft$$

W24x7(
W24x7(
W24x7(
W24x7(
W24x7(
W21x6.

$$M_{uGEW} := \frac{2}{3}\cdot\frac{1}{8}\cdot uniformload_{GEWedge}\cdot L_{span}{}^{2} = \begin{pmatrix} 262.6 \\ 255.89 \\ 255.89 \\ 255.89 \\ 255.89 \\ 189.243 \end{pmatrix} \qquad \phi M_{nGEW} := \begin{pmatrix} 274 \\ 274 \\ 274 \\ 274 \\ 274 \\ 203 \end{pmatrix}\cdot kip\cdot ft$$

W16x4
W16x4
W16x4
W16x4
W16x4
W16x3

$$M_{uGCenter} := \frac{1}{8}\cdot uniformload_{GCenter}\cdot L_{span}{}^{2} = \begin{pmatrix} 695.117 \\ 677.357 \\ 677.357 \\ 677.357 \\ 677.357 \\ 500.936 \end{pmatrix} \qquad \phi M_{nGCen} := \begin{pmatrix} 750 \\ 750 \\ 750 \\ 750 \\ 750 \\ 540 \end{pmatrix}\cdot kip\cdot ft$$

W24x7(
W24x7(
W24x7(
W24x7(
W24x7(
W21x6.

$$M_{uGCorner} := \frac{2}{3}\cdot\frac{1}{8}\cdot uniformload_{GCorner}\cdot L_{span}^{\,2} = \begin{pmatrix} 280.106 \\ 272.95 \\ 272.95 \\ 272.95 \\ 272.95 \\ 201.859 \end{pmatrix} \qquad \phi M_{nGCor} := \begin{pmatrix} 294 \\ 294 \\ 294 \\ 294 \\ 294 \\ 203 \end{pmatrix}\cdot kip\cdot ft$$

W18x4
W18x4
W18x4
W18x4
W18x4
W16x3

$$M_{uGCenterpe} := \frac{1}{8}\cdot uniformload_{GCenterpe}\cdot L_{span}^{\,2} = \begin{pmatrix} 695.117 \\ 677.357 \\ 677.357 \\ 677.357 \\ 677.357 \\ 577.783 \end{pmatrix} \qquad \phi M_{nGCenpe} := \begin{pmatrix} 750 \\ 750 \\ 750 \\ 750 \\ 750 \\ 574 \end{pmatrix}\cdot kip\cdot ft$$

W24x7
W24x7
W24x7
W24x7
W24x7
W24x6

$$M_{uGCenterpc} := \frac{1}{8}\cdot uniformload_{GCenterpc}\cdot L_{span}^{\,2} = \begin{pmatrix} 695.117 \\ 677.357 \\ 677.357 \\ 677.357 \\ 677.357 \\ 654.63 \end{pmatrix} \qquad \phi M_{nGCenpc} := \begin{pmatrix} 750 \\ 750 \\ 750 \\ 750 \\ 750 \\ 750 \end{pmatrix}\cdot kip\cdot ft$$

W24x7
W24x7
W24x7
W24x7
W24x7
W24x7

Choose appropriate Girder sizes from AISC Steel Construction Manual

Design Tables: 3-2

W24x76 for all NS-edge Girders, except roof. Use W21x62 for roof NS-edge Girders

W16x40 for all EW-edge Girders, except roof. Use W16x31 for roof EW-edge Girders

W24x76 for all Center Girders, except roof. Use W21x62 for roof Center Girders

W24x76 for all Center Penthouse Edge Girders, except roof. Use W24x62 for roof Center Girders

W24x76 for all Center Penthouse Center Girders.

W18x40 for all Corner Girders, except roof. Use W16x31 for roof Corner Girders

Beams

$$DL_{BNSedge} := \frac{DL\cdot A_{BNSedge}}{30} = \begin{pmatrix} 0.735 \\ 0.704 \\ 0.704 \\ 0.704 \\ 0.704 \\ 0.72 \end{pmatrix} klf \qquad DL_{BEWedge} := \frac{DL\cdot A_{BEWedge}}{30} = \begin{pmatrix} 1.12 \\ 1.073 \\ 1.073 \\ 1.073 \\ 1.073 \\ 1.097 \end{pmatrix} klf$$

$$DL_{BCenter} := \frac{DL \cdot A_{Bcenter}}{30} = \begin{pmatrix} 1.05 \\ 1.006 \\ 1.006 \\ 1.006 \\ 1.006 \\ 1.029 \end{pmatrix} \quad klf$$

$$DL_{BCorner} := \frac{DL \cdot A_{Bcorner}}{30} = \begin{pmatrix} 0.784 \\ 0.751 \\ 0.751 \\ 0.751 \\ 0.751 \\ 0.768 \end{pmatrix} \quad klf$$

$$DL_{BCenterpe} := \frac{DL \cdot 0.5A_{Bcenter} + DLP \cdot 0.5A_{Bcenter}}{30} = \begin{pmatrix} 1.05 \\ 1.006 \\ 1.006 \\ 1.006 \\ 1.006 \\ 1.218 \end{pmatrix} \quad klf$$

$$DL_{BCenterpc} := \frac{DLP \cdot A_{Bcenter}}{30} = \begin{pmatrix} 1.05 \\ 1.006 \\ 1.006 \\ 1.006 \\ 1.006 \\ 1.408 \end{pmatrix} \quad klf$$

$$LL_{BNSedge} := \frac{LL \cdot A_{BNSedge}}{30} = \begin{pmatrix} 0.35 \\ 0.35 \\ 0.35 \\ 0.35 \\ 0.35 \\ 0.35 \end{pmatrix} \quad klf$$

$$LL_{BEWedge} := \frac{LL \cdot A_{BEWedge}}{30} = \begin{pmatrix} 0.533 \\ 0.533 \\ 0.533 \\ 0.533 \\ 0.533 \\ 0.533 \end{pmatrix} \quad klf$$

$$LL_{BCenter} := \frac{LL \cdot A_{Bcenter}}{30} = \begin{pmatrix} 0.5 \\ 0.5 \\ 0.5 \\ 0.5 \\ 0.5 \\ 0.5 \end{pmatrix} \quad klf$$

$$LL_{Bcorner} := \frac{LL \cdot A_{Bcorner}}{30} = \begin{pmatrix} 0.373 \\ 0.373 \\ 0.373 \\ 0.373 \\ 0.373 \\ 0.373 \end{pmatrix} \quad klf$$

$$uniformload_{BNSedge} := 1.2 \cdot DL_{BNSedge} + 1.6 \cdot LL_{BNSedge}$$

$$uniformload_{BNSedge_6} := 1.2 \cdot DL_{BNSedge_6} + 0.5 LL_{BNSedge_6}$$

$$uniformload_{BEWedge} := 1.2 \cdot DL_{BEWedge} + 1.6 \cdot LL_{BEWedge}$$

$$uniformload_{BEWedge_6} := 1.2 \cdot DL_{BEWedge_6} + 0.5 LL_{BEWedge_6}$$

$$uniformload_{BCenter} := 1.2 \cdot DL_{BCenter} + 1.6 \cdot LL_{BCenter}$$

$$uniformload_{BCenter_6} := 1.2 \cdot DL_{BCenter_6} + 0.5 LL_{BCenter_6}$$

$$uniformload_{BCenterpc} := 1.2 \cdot DL_{BCenterpc} + 1.6 \cdot LL_{BCenter}$$

$$uniformload_{BCenterpc_6} := 1.2 \cdot DL_{BCenterpc_6} + 0.5 LL_{BCenter_6}$$

$$uniformload_{BCenterpe} := 1.2 \cdot DL_{BCenterpe} + 1.6 \cdot LL_{BCenter}$$

$$uniformload_{BCenterpe_6} := 1.2 \cdot DL_{BCenterpe_6} + 0.5 LL_{BCenter_6}$$

$$uniformload_{BCorner} := 1.2 \cdot DL_{BCorner} + 1.6 \cdot LL_{Bcorner}$$

$$uniformload_{BCorner_6} := 1.2 \cdot DL_{BCorner_6} + 0.5 LL_{Bcorner_6}$$

$$uniformload_{BNSedge} = \begin{pmatrix} 1.442 \\ 1.405 \\ 1.405 \\ 1.405 \\ 1.405 \\ 1.039 \end{pmatrix} \qquad uniformload_{BEWedge} = \begin{pmatrix} 2.197 \\ 2.141 \\ 2.141 \\ 2.141 \\ 2.141 \\ 1.583 \end{pmatrix}$$

$$uniformload_{BCenter} = \begin{pmatrix} 2.06 \\ 2.007 \\ 2.007 \\ 2.007 \\ 2.007 \\ 1.484 \end{pmatrix} \qquad uniformload_{BCorner} = \begin{pmatrix} 1.538 \\ 1.499 \\ 1.499 \\ 1.499 \\ 1.499 \\ 1.108 \end{pmatrix}$$

$$uniformload_{BCenterpc} = \begin{pmatrix} 2.06 \\ 2.007 \\ 2.007 \\ 2.007 \\ 2.007 \\ 1.94 \end{pmatrix} \qquad uniformload_{BCenterpe} = \begin{pmatrix} 2.06 \\ 2.007 \\ 2.007 \\ 2.007 \\ 2.007 \\ 1.712 \end{pmatrix}$$

Assume the girders are simple supported, the Max Moment is 1/8XqXL^2 in the midspan.

$L_{span} := 3($

$$M_{uBNS} := \frac{1}{8}\cdot uniformload_{BNSedge}\cdot L_{span}^{2} = \begin{pmatrix} 162.194 \\ 158.05 \\ 158.05 \\ 158.05 \\ 158.05 \\ 116.885 \end{pmatrix} \qquad \phi M_{nGNS} := \begin{pmatrix} 177 \\ 177 \\ 177 \\ 177 \\ 177 \\ 125 \end{pmatrix}\cdot kip\cdot ft$$

W14x3(

W14x2

$$M_{uBEW} := \frac{1}{8}\cdot uniformload_{BEWedge}\cdot L_{span}^{2} = \begin{pmatrix} 247.153 \\ 240.838 \\ 240.838 \\ 240.838 \\ 240.838 \\ 178.111 \end{pmatrix} \qquad \phi M_{nGEW} := \begin{pmatrix} 249 \\ 249 \\ 249 \\ 249 \\ 249 \\ 203 \end{pmatrix}\cdot kip\cdot ft$$

W18x3

W16x3

$$M_{uBCen} := \frac{1}{8}\cdot uniformload_{BCenter}\cdot L_{span}^{2} = \begin{pmatrix} 231.706 \\ 225.786 \\ 225.786 \\ 225.786 \\ 225.786 \\ 166.979 \end{pmatrix} \qquad \phi M_{nGCen} := \begin{pmatrix} 249 \\ 249 \\ 249 \\ 249 \\ 249 \\ 203 \end{pmatrix}\cdot kip\cdot ft$$

W18x3

W16x3

$$M_{uBCor} := \frac{1}{8}\cdot uniformload_{BCorner}\cdot L_{span}^{2} = \begin{pmatrix} 173.007 \\ 168.587 \\ 168.587 \\ 168.587 \\ 168.587 \\ 124.677 \end{pmatrix} \qquad \phi M_{nGCor} := \begin{pmatrix} 177 \\ 177 \\ 177 \\ 177 \\ 177 \\ 125 \end{pmatrix}\cdot kip\cdot ft$$

W14x3(

W14x2

$$M_{uBCenpc} := \frac{1}{8}\cdot uniformload_{BCenterpc}\cdot L_{span}^{2} = \begin{pmatrix} 231.706 \\ 225.786 \\ 225.786 \\ 225.786 \\ 225.786 \\ 218.21 \end{pmatrix} \qquad \phi M_{nGCenpc} := \begin{pmatrix} 249 \\ 249 \\ 249 \\ 249 \\ 249 \\ 249 \end{pmatrix}\cdot kip\cdot ft$$

W18x3

W18x3

$$M_{uBCenpe} := \frac{1}{8}\cdot uniformload_{BCenterpe}\cdot L_{span}{}^{2} = \begin{pmatrix} 231.706 \\ 225.786 \\ 225.786 \\ 225.786 \\ 225.786 \\ 192.594 \end{pmatrix} \qquad \phi M_{nCCenpe} := \begin{pmatrix} 249 \\ 249 \\ 249 \\ 249 \\ 249 \\ 203 \end{pmatrix} \cdot kip\cdot ft$$

W18x3

W16x3

*Reminder - All beams run from North to South

W14x30 for all NS-edge beams, except roof. Use W14x22 for roof NS-edge beams.

W18x35 for all EW-edge beams, except roof. Use W16x31 for roof EW-edge beams.

W18x35 for all Center beams, except roof. Use W16x31 for roof Center beams

W18x35 for all Center Penthouse Beams.

W18x35 for all Edge Penthouse Beams, except roof. Use W16x31 for roof Penthouse Beams

W14x30 for all Corner beams, except roof. Use W14x22 for roof Corner beams

ORIGIN≡ 1

WEAK BEAM STRONG COLUMN CHECK

Check column-beam relationships per ANSI/AISC 358 Section 5.4

Check 3 story building NS direction

The Capacity of column in 3 story building NS direction- joint 1

Column size W14X193 Beam size W27X94

$P_{uc} := 91.9$ kip $A_g := 56.8$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 27.6$ in

$F_y := 50$ ksi $Z_{xt} := 355$ in^3 $Z_{xb} := 355$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.174 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 1

$DL := 0.125 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 203$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $d_{cl} := 15.5$ in $d_{cr} := 15.7$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 311.7 \quad in$$

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.739$ kip $V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.042 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.772 \times 10^4 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.506$ Larger than 1 ok!

The Capacity of column in 3 story building NS direction- joint 2

Column size W14X193 Beam size W27X94

$P_{uc} := 63.2$ kip $\quad A_g := 56.8$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := h_t$ $\quad d_b := 27.6$ in

$F_y := 50$ ksi $\quad Z_{xt} := 355$ in^3 $\quad Z_{xb} := 355$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.217 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 2

$DL := 0.125 \ \frac{kip}{ft}$ $\quad LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \ \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 203$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $\quad d_{cl} := 15.5$ in $\quad d_{cr} := 15.7$ in $\quad L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 311.7 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.739 \quad kip \qquad V_{RBSprime} := 0 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.042 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.772 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.521$$

Larger than 1 ok!

The Capacity of column in 3 story building NS direction- joint 3

Column size W14X193 Beam size W18X40

$P_{uc} := 34.5$ kip $\quad A_g := 56.8$ in^2 $\quad h_t := 0$ in $\quad h_b := \frac{13}{2} \cdot 12 = 78$ in $\quad d_b := 17.9$ in

$F_y := 50$ ksi $\quad Z_{xt} := 0$ in^3 $\quad Z_{xb} := 355$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 1.981 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building NS direction- joint 3

$DL := 0.125 \ \frac{kip}{ft} \quad LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \ \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1 \quad F_u := 65$ ksi $\quad Z_{RBS} := 55.3$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 3.498 \times 10^3$ kip − in

$S_h := 10.3245$ in $\quad d_{cl} := 15.5$ in $\quad d_{cr} := 15.7$ in $\quad L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 323.751 \quad \text{in}$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 24.052 \quad \text{kip} \qquad V_{RBSprime} := 0 \quad \text{kip}$$

$$\Sigma M_{uv} := (V_{RBS} + V_{RBSprime}) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 434.737 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = \blacksquare \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.666$$ Larger than 1 ok!

The Capacity of column in 3 story building NS direction- joint 4

Column size W14X211 Beam size W27X94

$P_{uc} := 257.1$ kip $A_g := 62$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 27.6$ in

$F_y := 50$ ksi $Z_{xt} := 390$ in^3 $Z_{xb} := 390$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.345 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 4

$DL := 0.125 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 203$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $d_{cl} := 15.7$ in $d_{cr} := 15.7$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 311.6 \quad in$$

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.765$ kip $V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 80.059$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 3.989 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.967 \times 10^4 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.465$ Larger than 1 ok!

The Capacity of column in 3 story building NS direction- joint 5

Column size W14X211 Beam size W27X94

$P_{uc} := 171.8$ kip $A_g := 62$ in^2 $h_t := \frac{13}{2}\cdot 12 = 78$ in $h_b := h_t$ $d_b := 27.6$ in

$F_y := 50$ ksi $Z_{xt} := 390$ in^3 $Z_{xb} := 390$ in^3

$$\Sigma M_{pc} := Z_{xt}\cdot\left(F_y - \frac{P_{uc}}{A_g}\right)\cdot\left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb}\cdot\left(F_y - \frac{P_{uc}}{A_g}\right)\cdot\left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.476\times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building NS direction- joint 5

$DL := 0.125\ \frac{\text{kip}}{\text{ft}}$ $LL := 0.0625\ \frac{\text{kip}}{\text{ft}}$

$w_u := 1.2\cdot DL + 0.5\cdot LL = 0.181\ \frac{\text{kip}}{\text{ft}}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 203$ in^3 $C_{pr} := \frac{F_y + F_u}{2\cdot F_y} = 1.15$

$M_{pr} := C_{pr}\cdot R_y\cdot F_y\cdot Z_{RBS} = 1.284\times 10^4$ kip – in

$S_h := 16.35$ in $d_{cl} := 15.7$ in $d_{cr} := 15.7$ in $L := 30\cdot 12 = 360$ in

$$L_h := L - 2\cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 311.6 \quad \text{in}$$

$$V_{RBS} := \frac{2\cdot M_{pr}}{L_h} + \frac{w_u\cdot L_h}{2\cdot 12} = 84.765 \ \text{kip} \qquad V_{RBSprime} := \frac{2\cdot M_{pr}}{L_h} - \frac{w_u\cdot L_h}{2\cdot 12} = 80.059 \ \text{kip}$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right)\cdot\left(S_h + \frac{d_{cl}}{2}\right) = 3.989\times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2\cdot M_{pr} + \Sigma M_{uv} = 2.967\times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.509$$

Larger than 1 ok!

The Capacity of column in 3 story building NS direction- joint 6

Column size W14X211 Beam size W18X40

$P_{uc} := 93.4$ kip $A_g := 62$ in^2 $h_t := 0$ in $h_b := \frac{13}{2}\cdot 12 = 78$ in $d_b := 17.9$ in

$F_y := 50$ ksi $Z_{xt} := 0$ in^3 $Z_{xb} := 390$ in^3

$$\Sigma M_{pc} := Z_{xt}\cdot\left(F_y - \frac{P_{uc}}{A_g}\right)\cdot\left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb}\cdot\left(F_y - \frac{P_{uc}}{A_g}\right)\cdot\left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 2.136\times 10^4 \quad kip-in$$

The Capacity of beam in 3 story building NS direction- joint 6

$DL := 0.125 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2\cdot DL + 0.5\cdot LL = 0.181 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 55.3$ in^3 $C_{pr} := \frac{F_y + F_u}{2\cdot F_y} = 1.15$

$M_{pr} := C_{pr}\cdot R_y\cdot F_y\cdot Z_{RBS} = 3.498\times 10^3$ kip – in

$S_h := 10.3245$ in $d_{cl} := 15.7$ in $d_{cr} := 15.7$ in $L := 30\cdot 12 = 360$ in

$$L_h := L - 2\cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 323.651 \quad in$$

$$V_{RBS} := \frac{2\cdot M_{pr}}{L_h} + \frac{w_u\cdot L_h}{2\cdot 12} = 24.058 \quad kip \qquad V_{RBSprime} := \frac{2\cdot M_{pr}}{L_h} - \frac{w_u\cdot L_h}{2\cdot 12} = 19.17 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right)\cdot\left(S_h + \frac{d_{cl}}{2}\right) = 785.654 \quad kip-in$$

$$\Sigma M_{pb} := 2\cdot M_{pr} + \Sigma M_{uv} = 7.781\times 10^3 \quad kip-in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.746$$ Larger than 1 ok!

Check 3 story building EW direction

The Capacity of column in 3 story building EW direction- joint 4

Column size W14X159 Beam size W24X55

$P_{uc} := 257.7$ kip $A_g := 46.7$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.6$ in

$F_y := 50$ ksi $Z_{xt} := 287$ in^3 $Z_{xb} := 287$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.008 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building EW direction- joint 4

$DL := 0.125 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 99.4$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 6.287 \times 10^3$ kip – in

$S_h := 13.056$ in $d_{cl} := 15.5$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 318.638 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 41.868 \quad kip \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 37.056 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 1.642 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 1.422 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.116$$

Larger than 1 ok!

The Capacity of column in 3 story building EW direction- joint 5

Column size W14X159 Beam size W24X55

$P_{uc} := 173.9$ kip $A_g := 46.7$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.6$ in

$F_y := 50$ ksi $Z_{xt} := 287$ in^3 $Z_{xb} := 287$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.13 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building EW direction- joint 5

$DL := 0.125 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 99.4$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 6.287 \times 10^3$ kip – in

$S_h := 13.056$ in $d_{cl} := 15.5$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 318.638 \quad in$$

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 41.868$ kip $V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 37.056$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 1.642 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 1.422 \times 10^4 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.202$ Larger than 1 ok!

The Capacity of column in 3 story building EW direction- joint 6

Column size W14X159 Beam size W18X35

$P_{uc} := 93.1$ kip $A_g := 46.7$ in^2 $h_t := 0$ in $h_b := \frac{13}{2} \cdot 12 = 78$ in $d_b := 17.7$ in

$F_y := 50$ ksi $Z_{xt} := 0$ in^3 $Z_{xb} := 287$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 1.554 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building EW direction- joint 6

$DL := 0.125 \frac{\text{kip}}{\text{ft}}$ $LL := 0.0625 \frac{\text{kip}}{\text{ft}}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.181 \frac{\text{kip}}{\text{ft}}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 49.4$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 3.125 \times 10^3$ kip – in

$S_h := 10.2375$ in $d_{cl} := 15.5$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 324.275 \quad \text{in}$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 21.72 \quad \text{kip} \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 16.822 \quad \text{kip}$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 693.274 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 6.942 \times 10^3 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.239$$

Larger than 1 ok!

Check 6 story building NS direction

The Capacity of column in 6 story building NS direction- joint 1

Column size W14X257 Beam size W30X108

$P_{uc} := 301.8$ kip $\quad A_g := 75.6$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := \frac{18}{2} \cdot 12 = 108$ in $\quad d_b := 29.8$ in

$F_y := 50$ ksi $\quad Z_{xt} := 487$ in^3 $\quad Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.369 \times 10^4 \quad kip - in$$

The Capacity of beam in 6 story building NS direction- joint 1

$DL := 0.131 \frac{kip}{ft}$ $\quad LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.189 \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 252.6$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip – in

$S_h := 17.475$ in $\quad d_{cl} := 16.4$ in $\quad d_{cr} := 16.4$ in $\quad L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 308.65$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 105.956$ kip $\quad V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := (V_{RBS} + V_{RBSprime}) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.72 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.467 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.548$$ Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 2

Column size W14X257 Beam size W30X108

$P_{uc} := 238.3$ kip $A_g := 75.6$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 29.8$ in

$F_y := 50$ ksi $Z_{xt} := 487$ in^3 $Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.64 \times 10^4 \quad kip - in$$

The Capacity of beam in 6 story building NS direction- joint 2

$DL := 0.126 \ \frac{kip}{ft}$ $LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 252.6$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip – in

$S_h := 17.475$ in $d_{cl} := 15.5$ in $d_{cr} := 15.7$ in $L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 309.45$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 105.617$ kip $V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.664 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.462 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.629$$ Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 3

Column size W14X257 Beam size W27X94

$P_{uc} := 178.3$ kip $\quad A_g := 75.6$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := \frac{13}{2} \cdot 12 = 78$ in $\quad d_b := 27.6$ in

$F_y := 50$ ksi $\quad Z_{xt} := 487$ in^3 $\quad Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.638 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 3

$DL := 0.126 \frac{kip}{ft}$ $\quad LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 203$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $\quad d_{cl} := 16.4$ in $\quad d_{cr} := 16.4$ in $\quad L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 310.9$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.965$ kip $\quad V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.086 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.777 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.03$$ Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 4

Column size W14X176 Beam size W24X76

$P_{uc} := 122.3$ kip $A_g := 51.8$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.7$ in

$F_y := 50$ ksi $Z_{xt} := 320$ in^3 $Z_{xb} := 320$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.595 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 4

$DL := 0.126 \frac{kip}{ft}$ $LL := 0.062\frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 141.1$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 8.925 \times 10^3$ kip − in

$S_h := 14.356$ in $d_{cl} := 15.2$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 316.187 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 58.86 \quad kip$$ $V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 1.292 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 1.914 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.878$$ Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 5

Column size W14X176 Beam size W24X62

$P_{uc} := 73.5$ kip $A_g := 51.8$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.7$ in

$F_y := 50$ ksi $Z_{xt} := 320$ in^3 $Z_{xb} := 320$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.666 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 5

$DL := 0.126$ $\frac{kip}{ft}$ $LL := 0.0625$ $\frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183$ $\frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 112.1$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 7.09 \times 10^3$ kip − in

$S_h := 13.1115$ in $d_{cl} := 15.2$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 318.677 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 46.926 \quad kip$$

$V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 971.906 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 1.515 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.42$$

Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 6

Column size W14X176 Beam size W18X40

$P_{uc} := 34.6$ kip $A_g := 51.8$ in^2 $h_t := 0$ in $h_b := \frac{13}{2} \cdot 12 = 78$ in $d_b := 17.9$ in

$F_y := 50$ ksi $Z_{xt} := 0$ in^3 $Z_{xb} := 320$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 1.783 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 6

$DL := 0.128 \ \frac{kip}{ft}$ $LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.186 \ \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 55.3$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 3.498 \times 10^3$ kip – in

$S_h := 10.3245$ in $d_{cl} := 15.2$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 324.251 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 24.085 \quad kip$$

$V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := (V_{RBS} + V_{RBSprime}) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 431.704 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 7.427 \times 10^3 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.401$$

Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 7

Column size W14X257 Beam size W30X108

$P_{uc} := 555.3$ kip $A_g := 75.6$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := \frac{18}{2} \cdot 12 = 108$ in $d_b := 29.8$ in

$F_y := 50$ ksi $Z_{xt} := 487$ in^3 $Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.978 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 7

$DL := 0.128 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.186 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 252.6$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip – in

$S_h := 17.475$ in $d_{cl} := 16.4$ in $d_{cr} := 16.4$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 308.65 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 105.918 \quad kip \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 101.138 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 5.316 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.727 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.336$$

Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 8

Column size W14X257 Beam size W30X108

$P_{uc} := 456.4$ kip $\quad A_g := 75.6$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := h_t$ $\quad d_b := 29.8$ in

$F_y := 50$ ksi $\quad Z_{xt} := 487$ in^3 $\quad Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.293 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building NS direction- joint 8

$DL := 0.126 \ \frac{\text{kip}}{\text{ft}} \quad LL := 0.0625 \ \frac{\text{kip}}{\text{ft}}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{\text{kip}}{\text{ft}}$

$F_y = 50$ ksi $\quad R_y := 1.1 \quad F_u := 65$ ksi $\quad Z_{RBS} := 252.6$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4 \quad \text{kip} - \text{in}$

$S_h := 17.475$ in $\quad d_{cl} := 15.5$ in $\quad d_{cr} := 15.7$ in $\quad L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 309.45$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 105.617$ kip $\quad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 100.903$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 5.209 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.716 \times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.424$$ Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 9

Column size W14X257 Beam size W27X94

$P_{uc} := 359.7$ kip $A_g := 75.6$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := \frac{13}{2} \cdot 12 = 78$ in $d_b := 27.6$ in

$F_y := 50$ ksi $Z_{xt} := 487$ in^3 $Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.354 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building NS direction- joint 9

$DL := 0.126 \ \frac{kip}{ft}$ $LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 203$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $d_{cl} := 16.4$ in $d_{cr} := 16.4$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 310.9 \quad \text{in}$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.965 \text{ kip} \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 80.229 \text{ kip}$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 4.056 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.974 \times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.8$$

Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 10

Column size W14X159 Beam size W24X76

$P_{uc} := 263.1$ kip $A_g := 46.7$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.7$ in

$F_y := 50$ ksi $Z_{xt} := 287$ in^3 $Z_{xb} := 287$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.003 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building NS direction- joint 4

$DL := 0.126 \ \frac{kip}{ft}$ $LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 141.1$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 8.925 \times 10^3$ kip – in

$S_h := 14.3565$ in $d_{cl} := 15$ in $d_{cr} := 15$ in $L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 316.287$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 58.843$ kip $V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 54.024$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.467 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.032 \times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.478$$ Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 11

Column size W14X159 Beam size W24X62

$P_{uc} := 178.4 \quad kip \qquad A_g := 46.7 \quad in^2 \qquad h_t := \frac{13}{2} \cdot 12 = 78 \quad in \qquad h_b := h_t \qquad d_b := 23.7 \quad in$

$F_y := 50 \quad ksi \qquad Z_{xt} := 287 \quad in^3 \qquad Z_{xb} := 287 \quad in^3$

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g} \right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}} \right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g} \right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}} \right) = 3.126 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 5

$DL := 0.126 \ \frac{kip}{ft} \qquad LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50 \quad ksi \qquad R_y := 1.1 \qquad F_u := 65 \quad ksi \qquad Z_{RBS} := 112.1 \quad in^3 \qquad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 7.09 \times 10^3 \quad kip - in$

$S_h := 13.1115 \quad in \qquad d_{cl} := 15 \quad in \qquad d_{cr} := 15 \quad in \qquad L := 30 \cdot 12 = 360 \quad in$

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 318.777 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 46.913 \quad kip \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 42.056 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime} \right) \cdot \left(S_h + \frac{d_{cl}}{2} \right) = 1.834 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 1.601 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.952$$

Larger than 1 ok!

The Capacity of column in 6 story building NS direction- joint 12

Column size W14X159 Beam size W18X40

$P_{uc} := 96.2$ kip $\quad A_g := 46.7$ in^2 $\quad h_t := 0$ in $\quad h_b := \frac{13}{2} \cdot 12 = 78$ in $\quad d_b := 17.9$ in

$F_y := 50$ ksi $\quad Z_{xt} := 0$ in^3 $\quad Z_{xb} := 287$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 1.554 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 6

$DL := 0.128 \frac{kip}{ft}$ $\quad LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.186 \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 55.3$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 3.498 \times 10^3$ kip – in

$S_h := 10.3245$ in $\quad d_{cl} := 15$ in $\quad d_{cr} := 15$ in $\quad L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 324.351$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 24.079$ kip $\quad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 19.056$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 768.861 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 7.764 \times 10^3 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.002$$

Larger than 1 ok!

Check 6 story building EW direction

The Capacity of column in 6 story building EW direction- joint 1

Column size W14X257 Beam size W30X108

$P_{uc} := 220.5$ kip $A_g := 75.6$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := \frac{18}{2} \cdot 12 = 108$ in $d_b := 29.8$ in

$F_y := 50$ ksi $Z_{xt} := 487$ in^3 $Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.494 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building EW direction- joint 1

$DL := 0.131 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.189 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 252.6$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip – in

$S_h := 17.475$ in $d_{cl} := 16.4$ in $d_{cr} := 17.1$ in $L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 308.3$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 106.071$ kip $V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.723 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.468 \times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.584$$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 2

Column size W14X257 Beam size W30X108

$P_{uc} := 179.8$ kip $A_g := 75.6$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 29.8$ in

$F_y := 50$ ksi $Z_{xt} := 487$ in^3 $Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.734 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building EW direction- joint 2

$DL := 0.126 \ \frac{\text{kip}}{\text{ft}}$ $LL := 0.0625 \ \frac{\text{kip}}{\text{ft}}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{\text{kip}}{\text{ft}}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 252.6$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip − in

$S_h := 17.475$ in $d_{cl} := 16.4$ in $d_{cr} := 17.1$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 308.3 \quad \text{in}$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 105.994 \quad \text{kip} \qquad V_{RBSprime} := 0 \quad \text{kip}$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.721 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.468 \times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.654$$

Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 3

Column size W14X257 Beam size W27X94

$P_{uc} := 140.6$ kip $\quad A_g := 75.6$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := \frac{13}{2} \cdot 12 = 78$ in $\quad d_b := 27.6$ in

$F_y := 50$ ksi $\quad Z_{xt} := 487$ in^3 $\quad Z_{xb} := 487$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 5.697 \times 10^4 \quad kip - in$$

The Capacity of beam in 6 story building EW direction- joint 3

$DL := 0.126 \ \frac{kip}{ft}$ $\quad LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 203$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $\quad d_{cl} := 16.4$ in $\quad d_{cr} := 17.1$ in $\quad L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 310.55 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 85.056 \quad kip \qquad V_{RBSprime} := 0 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.088 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.777 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.052$$

Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 4

Column size W14X176 Beam size W27X94

$P_{uc} := 101.9$ kip $\quad A_g := 51.8$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := h_t$ $\quad d_b := 27.6$ in

$F_y := 50$ ksi $\quad Z_{xt} := 320$ in^3 $\quad Z_{xb} := 320$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.735 \times 10^4 \quad kip - in$$

The Capacity of beam in 6 story building EW direction- joint 4

$DL := 0.126 \frac{kip}{ft}$ $\quad LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 203$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $\quad d_{cl} := 15.2$ in $\quad d_{cr} := 15.2$ in $\quad L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 312.1 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.657 \quad kip \qquad V_{RBSprime} := 0 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.028 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.771 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.348$$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 5

Column size W14X176 Beam size W24X76

$P_{uc} := 63.8$ kip $A_g := 51.8$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.9$ in

$F_y := 50$ ksi $Z_{xt} := 320$ in^3 $Z_{xb} := 320$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 3.686 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building EW direction- joint 5

$DL := 0.126 \ \frac{kip}{ft}$ $LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 141.1$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 8.925 \times 10^3$ kip – in

$S_h := 14.3565$ in $d_{cl} := 15.2$ in $d_{cr} := 15.2$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 316.087 \quad in$$

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 58.877$ kip $V_{RBSprime} := 0$ kip

$$\Sigma M_{uv} := (V_{RBS} + V_{RBSprime}) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 1.293 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 1.914 \times 10^4 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.926$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 6

Column size W14X176 Beam size W18X40

$P_{uc} := 34.6$ kip $A_g := 51.8$ in^2 $h_t := 0$ in $h_b := \frac{13}{2} \cdot 12 = 78$ in $d_b := 17.9$ in

$F_y := 50$ ksi $Z_{xt} := 0$ in^3 $Z_{xb} := 320$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 1.783 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 6

$DL := 0.128 \frac{kip}{ft}$ $LL := 0.062 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.186 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 55.3$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 3.498 \times 10^3$ kip – in

$S_h := 10.324$ in $d_{cl} := 15.2$ in $d_{cr} := 15.2$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 324.151 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 24.09 \quad kip \qquad V_{RBSprime} := 0 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 431.809 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 7.427 \times 10^3 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.401$$

Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 7

Column size W14X311 Beam size W30X108

$P_{uc} := 517.7$ kip $\quad A_g := 91.4$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := \frac{18}{2} \cdot 12 = 108$ in $\quad d_b := 29.8$ in

$F_y := 50$ ksi $\quad Z_{xt} := 603$ in^3 $\quad Z_{xb} := 603$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 6.406 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building EW direction- joint 7

$DL := 0.131 \frac{kip}{ft} \quad LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.189 \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1 \quad F_u := 65$ ksi $\quad Z_{RBS} := 252.6$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip – in

$S_h := 17.475$ in $\quad d_{cl} := 17.1$ in $\quad d_{cr} := 17.1$ in $\quad L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 307.95$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 106.186$ kip $\quad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 101.341$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 5.401 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.735 \times 10^4 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.715$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 8

Column size W14X311 Beam size W30X108

$P_{uc} := 423.4$ kip $\quad A_g := 91.4$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := h_t$ $\quad d_b := 29.8$ in

$F_y := 50$ ksi $\quad Z_{xt} := 603$ in^3 $\quad Z_{xb} := 603$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 6.763 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building EW direction- joint 8

$DL := 0.126 \frac{kip}{ft}$ $\quad LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1$ $\quad F_u := 65$ ksi $\quad Z_{RBS} := 252.6$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.598 \times 10^4$ kip – in

$S_h := 17.475$ in $\quad d_{cl} := 17.1$ in $\quad d_{cr} := 17.1$ in $\quad L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 307.95$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 106.109$ kip $\quad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 101.418$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 5.401 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 3.735 \times 10^4 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.811$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 9

Column size W14X311 Beam size W27X94

$P_{uc} := 334$ kip $\quad A_g := 91.4$ in^2 $\quad h_t := \frac{13}{2} \cdot 12 = 78$ in $\quad h_b := \frac{13}{2} \cdot 12 = 78$ in $\quad d_b := 27.6$ in

$F_y := 50$ ksi $\quad Z_{xt} := 603$ in^3 $\quad Z_{xb} := 603$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 6.791 \times 10^4 \quad kip - in$$

The Capacity of beam in 6 story building EW direction- joint 9

$DL := 0.126 \ \frac{kip}{ft} \quad LL := 0.0625 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \ \frac{kip}{ft}$

$F_y = 50$ ksi $\quad R_y := 1.1 \quad F_u := 65$ ksi $\quad Z_{RBS} := 203$ in^3 $\quad C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $\quad d_{cl} := 17.1$ in $\quad d_{cr} := 17.1$ in $\quad L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 310.2 \quad in$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 85.147 \quad kip \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 80.421 \quad kip$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 4.123 \times 10^3 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.98 \times 10^4 \quad kip - in$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.279$$

Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 10

Column size W14X211 Beam size W27X94

$P_{uc} := 245.4$ kip $A_g := 62$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 27.6$ in

$F_y := 50$ ksi $Z_{xt} := 390$ in^3 $Z_{xb} := 390$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.363 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 6 story building EW direction- joint 10

$DL := 0.126 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 203$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 1.284 \times 10^4$ kip – in

$S_h := 16.35$ in $d_{cl} := 15.7$ in $d_{cr} := 15.7$ in $L := 30 \cdot 12 = 360$ in

$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 311.6$ in

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 84.785$ kip $V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 80.038$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 3.989 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.967 \times 10^4 \quad \text{kip} - \text{in}$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 1.471$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 11

Column size W14X176 Beam size W24X76

$P_{uc} := 164.1$ kip $A_g := 62$ in^2 $h_t := \frac{13}{2} \cdot 12 = 78$ in $h_b := h_t$ $d_b := 23.9$ in

$F_y := 50$ ksi $Z_{xt} := 390$ in^3 $Z_{xb} := 390$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 4.362 \times 10^4 \quad \text{kip} - \text{in}$$

The Capacity of beam in 3 story building EW direction- joint 11

$DL := 0.126 \frac{kip}{ft}$ $LL := 0.0625 \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.183 \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 141.1$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 8.925 \times 10^3$ kip – in

$S_h := 14.3565$ in $d_{cl} := 15.2$ in $d_{cr} := 15.2$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 316.087 \quad \text{in}$$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 58.877 \quad \text{kip} \qquad V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 54.061 \quad \text{kip}$$

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 2.48 \times 10^3 \quad \text{kip} - \text{in}$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 2.033 \times 10^4 \quad \text{kip} - \text{in}$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.146$$ Larger than 1 ok!

The Capacity of column in 6 story building EW direction- joint 12

Column size W14X176 Beam size W18X40

$P_{uc} := 89$ kip $A_g := 62$ in^2 $h_t := 0$ in $h_b := \frac{13}{2} \cdot 12 = 78$ in $d_b := 17.9$ in

$F_y := 50$ ksi $Z_{xt} := 0$ in^3 $Z_{xb} := 390$ in^3

$$\Sigma M_{pc} := Z_{xt} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_t}{h_t - \frac{d_b}{2}}\right) + Z_{xb} \cdot \left(F_y - \frac{P_{uc}}{A_g}\right) \cdot \left(\frac{h_b}{h_b - \frac{d_b}{2}}\right) = 2.14 \times 10^4 \quad kip - in$$

The Capacity of beam in 3 story building NS direction- joint 12

$DL := 0.128 \ \frac{kip}{ft}$ $LL := 0.062 \ \frac{kip}{ft}$

$w_u := 1.2 \cdot DL + 0.5 \cdot LL = 0.186 \ \frac{kip}{ft}$

$F_y = 50$ ksi $R_y := 1.1$ $F_u := 65$ ksi $Z_{RBS} := 55.3$ in^3 $C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 3.498 \times 10^3$ kip − in

$S_h := 10.324$ in $d_{cl} := 15.2$ in $d_{cr} := 15.2$ in $L := 30 \cdot 12 = 360$ in

$$L_h := L - 2 \cdot S_h - \frac{d_{cl}}{2} - \frac{d_{cr}}{2} = 324.151 \quad in$$

$V_{RBS} := \frac{2 \cdot M_{pr}}{L_h} + \frac{w_u \cdot L_h}{2 \cdot 12} = 24.09$ kip $V_{RBSprime} := \frac{2 \cdot M_{pr}}{L_h} - \frac{w_u \cdot L_h}{2 \cdot 12} = 19.071$ kip

$$\Sigma M_{uv} := \left(V_{RBS} + V_{RBSprime}\right) \cdot \left(S_h + \frac{d_{cl}}{2}\right) = 773.651 \quad kip - in$$

$$\Sigma M_{pb} := 2 \cdot M_{pr} + \Sigma M_{uv} = 7.769 \times 10^3 \quad kip - in$$

$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} = 2.754$ Larger than 1 ok!

ORIGIN $\equiv 1$

PANEL ZONE CHECK

Check Panel Zone Shear Strength per AISC Seismic Provisions Section E3.6e

Check 3 story building NS direction-joint 1

Column Shear in 3 story building NS direction-joint 1

$M_{pr} := 1.284\times 10^4$ kip – in $\quad V_{RBS} := 84.73$ kip $\quad V_{RBSprime} := 80.03$ kip

$S_h := 16.3$ in $\quad h_b := 13\cdot 12 = 156$ in $\quad h_t := 13\cdot 12 = 156$ in $\quad d_b := 27.$ in $\quad t_f := 0.74$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 1.423\times 10^4$ kip – in $\quad M_{fprime} := 0$ kip – in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 91.189 \text{ kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 438.525 \text{ kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X193

$A_g := 56.$ in^2 $\quad F_y := 5$ ksi

$0.75\cdot A_g\cdot F_y = 2.13\times 10^3$ kip

$\phi R_{\upsilon 1} := 41$ kip $\quad \phi R_{\upsilon 2} := 293$ kip – in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 520.159 \text{ kip}$$

We don't need the column-web doubler plate.

Check 3 story building NS direction-joint 2

Column Shear in 3 story building NS direction-joint 2

$M_{pr} := 1.284\times 10^4$ kip – in $\quad V_{RBS} := 84.73$ kip $\quad V_{RBSprime} := 80.03$ kip

$S_h := 16.3$ in $\quad h_b := 13\cdot 12 = 156$ in $\quad h_t := 13\cdot 12 = 156$ in $\quad d_b := 27.$ in $\quad t_f := 0.74$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 1.423\times 10^4$ kip – in $\quad M_{fprime} := 0$ kip – in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 91.189 \text{ kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 438.525 \text{ kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X193

$A_g := 56.8$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.13 \times 10^3$ kip

$\phi R_{\upsilon 1} := 414$ kip $\phi R_{\upsilon 2} := 2930$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 520.159 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 3 story building NS direction-joint 3

Column Shear in 3 story building NS direction-joint 3

$M_{pr} := 3.498 \times 10^3$ kip − in $V_{RBS} := 24.05$ kip $V_{RBSprime} := 19.16$ kip

$S_h := 10.324$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 0 \cdot 12 = 0$ in $d_b := 17.9$ in $t_f := 0.525$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 3.746 \times 10^3$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 48.03 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 167.586 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X193

$A_g := 56.8$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.13 \times 10^3$ kip

$\phi R_{\upsilon 1} := 414$ kip $\phi R_{\upsilon 2} := 2930$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 577.687 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 3 story building NS direction-joint 4

Column Shear in 3 story building NS direction-joint 4

$M_{pr} := 1.284 \times 10^4$ kip − in $V_{RBS} := 84.76$ kip $V_{RBSprime} := 80.05$ kip

$S_h := 16.3$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 27.$ in $t_f := 0.745$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.423 \times 10^4$ kip − in $M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.415 \times 10^4$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 181.89 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 874.705 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X211

$A_g := 62 \quad in^2 \qquad F_y := 50 \quad ksi$

$0.75 \cdot A_g \cdot F_y = 2.325\times 10^3 \quad kip$

$\phi R_{\upsilon 1} := 462 \quad kip \qquad \phi R_{\upsilon 2} := 3460 \quad kip - in$

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 587.362 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X211 and W27X94 Beam

$d_z := 0.282 \cdot 90 \qquad d_w := 0.14 \cdot 90 \qquad t_w := 0.98 \quad in$

$\frac{d_z}{90} + \frac{d_w}{90} = 0.422 \quad in \qquad$ Smaller than tw OK

$d_c := 15.7 \quad in \qquad b_{cf} := 15.8 \quad in \qquad t_{cf} := 1.56 \quad in$

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.611 \quad \text{in}$$

Use a 3/4 in thick doubler plate.

Check 3 story building NS direction-joint 5

Column Shear in 3 story building NS direction-joint 5

$M_{pr} := 1.284\times 10^4 \quad kip - in \qquad V_{RBS} := 84.76 \quad kip \qquad V_{RBSprime} := 80.05 \quad kip$

$S_h := 16.35 \quad in \qquad h_b := 13 \cdot 12 = 156 \quad in \qquad h_t := 13 \cdot 12 = 156 \quad in \qquad d_b := 27.0 \quad in \qquad t_f := 0.745 \quad in$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.423\times 10^4 \quad kip - in \qquad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.415\times 10^4 \quad kip - in$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 181.89 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 874.705 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X211

$A_g := 62 \quad in^2 \quad F_y := 50 \quad ksi$

$0.75 \cdot A_g \cdot F_y = 2.325 \times 10^3 \quad kip$

$\phi R_{\upsilon 1} := 462 \quad kip \quad \phi R_{\upsilon 2} := 3460 \quad kip - in$

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 587.362 \quad kip$$

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X211 and W27X94 Beam

$d_z := 0.282 \cdot 90 \quad d_w := 0.14 \cdot 90 \quad t_w := 0.98 \quad in$

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.422 \quad in \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 15.7 \ in \quad b_{cf} := 15.8 \quad in \quad t_{cf} := 1.56 \quad in$

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.611 \quad in$$

Use a 3/4 in thick doubler plate.

Check 3 story building NS direction-joint 6

Column Shear in 3 story building NS direction-joint 6

$M_{pr} := 3.498 \times 10^3 \quad kip - in \quad V_{RBS} := 24.058 \quad kip \quad V_{RBSprime} := 19.17 \quad kip$

$S_h := 10.324 \quad in \quad h_b := 13 \cdot 12 = 156 \quad in \quad h_t := 0 \cdot 12 = 0 \quad in \quad d_b := 17.9 \quad in \quad t_f := 0.525 \quad in$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 3.746 \times 10^3 \quad kip - in \quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 3.696 \times 10^3 \quad kip - in$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 95.414 \quad kip$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 332.92 \quad kip$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X211

$A_g := 62 \quad in^2 \quad F_y := 50 \quad ksi$

$0.75 \cdot A_g \cdot F_y = 2.325 \times 10^3$ kip

$\phi R_{\upsilon 1} := 462$ kip $\quad \phi R_{\upsilon 2} := 3460$ kip $-$ in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 655.296 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 3 story building EW direction-joint 4

Column Shear in 3 story building EW direction-joint 4

$M_{pr} := 6.287 \times 10^3$ kip $-$ in $\quad V_{RBS} := 41.868$ kip $\quad V_{RBSprime} := 37.050$ kip

$S_h := 13.050$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 23.6$ in $\quad t_f := 0.505$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 6.834 \times 10^3$ kip $-$ in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 6.771 \times 10^3$ kip $-$ in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 87.208 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 501.856 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X159

$A_g := 46.7$ in^2 $\quad F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 1.751 \times 10^3$ kip

$\phi R_{\upsilon 1} := 335$ kip $\quad \phi R_{\upsilon 2} := 1990$ kip $-$ in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 419.322 \quad \text{kip}$$

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X159 and W24X55 Beam

$d_z := 0.251 \cdot 90$ $\quad d_w := 0.14 \cdot 90$ $\quad t_w := 0.745$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.391 \quad \text{in}$$ Smaller than tw OK

$d_c := 15$ in $\quad b_{cf} := 15.6$ in $\quad t_{cf} := 1.19$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.183 \quad \text{in}$$

Use a 1/4 in thick doubler plate.

Check 3 story building EW direction-joint 5

Column Shear in 3 story building EW direction-joint 5

$M_{pr} := 6.287 \times 10^3$ kip − in $\quad V_{RBS} := 41.86$ kip $\quad V_{RBSprime} := 37.05$ kip

$S_h := 13.05$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 23.6$ in $\quad t_f := 0.505$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 6.834 \times 10^3$ kip − in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 6.771 \times 10^3$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 87.208 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 501.856 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X159

$A_g := 46.7$ in^2 $\quad F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 1.751 \times 10^3$ kip

$\phi R_{\upsilon 1} := 335$ kip $\quad \phi R_{\upsilon 2} := 1990$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 419.322 \quad \text{kip}$$

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X159 and W24X55 Beam

$d_z := 0.251 \cdot 90$ $\quad d_w := 0.14 \cdot 90$ $\quad t_w := 0.745$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.391 \quad \text{in} \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 15$ in $\quad b_{cf} := 15.6$ in $\quad t_{cf} := 1.19$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.183 \quad \text{in}$$

Use a 1/4 in thick doubler plate.

Check 3 story building EW direction-joint 6

Column Shear in 3 story building EW direction-joint 6

$M_{pr} := 3.125\times 10^3$ kip − in $V_{RBS} := 21.7$ kip $V_{RBSprime} := 16.82$ kip

$S_h := 10.324$ in $h_b := 13\cdot 12 = 156$ in $h_t := 0\cdot 12 = 0$ in $d_b := 17.$ in $t_f := 0.42$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 3.349\times 10^3$ kip − in $M_{fprime} := M_{pr} + V_{RBSprime}\cdot S_h = 3.299\times 10^3$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 85.23 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 299.6 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X159

$A_g := 46.$ in^2 $F_y := 5($ ksi

$0.75\cdot A_g\cdot F_y = 1.751\times 10^3$ kip

$\phi R_{\upsilon 1} := 33$ kip $\phi R_{\upsilon 2} := 199($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 447.429 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 1

Column Shear in 6 story building NS direction-joint 1

$M_{pr} := 1.598\times 10^4$ kip − in $V_{RBS} := 105.95$ kip $V_{RBSprime} := 101.$ kip

$S_h := 17.47$ in $h_b := 18\cdot 12 = 216$ in $h_t := 13\cdot 12 = 156$ in $d_b := 29.$ in $t_f := 0.7($ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 1.783\times 10^4$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 95.869 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 518.166 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.($ in^2 $F_y := 5($ ksi

$0.75\cdot A_g\cdot F_y = 2.835\times 10^3$ kip

$\phi R_{\upsilon 1} := 581$ kip $\phi R_{\upsilon 2} := 514($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 753.483 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 2

Column Shear in 6 story building NS direction-joint 2

$M_{pr} := 1.598\times 10^4$ kip − in $V_{RBS} := 105.61$ kip $V_{RBSprime} := 100.90$ kip

$S_h := 17.47$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 29.8$ in $t_f := 0.7($ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.783\times 10^4$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 114.267 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 499.564 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.($ in^2 $F_y := 5($ ksi

$0.75 \cdot A_g \cdot F_y = 2.835\times 10^3$ kip

$\phi R_{\upsilon 1} := 581$ kip $\phi R_{\upsilon 2} := 514($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 753.483 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 3

Column Shear in 6 story building NS direction-joint 3

$M_{pr} := 1.284\times 10^4$ kip − in $V_{RBS} := 84.96$ kip $V_{RBSprime} := 80.22$ kip

$S_h := 17.302$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 29.5$ in $t_f := 0.6$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.431\times 10^4$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 91.731 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 403.599 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.6 \quad in^2 \qquad F_y := 50 \quad ksi$

$0.75 \cdot A_g \cdot F_y = 2.835 \times 10^3 \quad kip$

$\phi R_{\upsilon 1} := 581 \quad kip \qquad \phi R_{\upsilon 2} := 5140 \quad kip - in$

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 755.237 \quad kip$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 4

Column Shear in 6 story building NS direction-joint 4

$M_{pr} := 8.925 \times 10^3 \quad kip - in \qquad V_{RBS} := 58.86 \quad kip \qquad V_{RBSprime} := 54.043 \quad kip$

$S_h := 14.356 \quad in \qquad h_b := 13 \cdot 12 = 156 \quad in \qquad h_t := 13 \cdot 12 = 156 \quad in \qquad d_b := 23.7 \quad in \qquad t_f := 0.68 \quad in$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 9.77 \times 10^3 \quad kip - in \qquad M_{fprime} := 0 \quad kip - in$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 62.628 \quad kip$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 361.786 \quad kip$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X176

$A_g := 51.8 \quad in^2 \qquad F_y := 50 \quad ksi$

$0.75 \cdot A_g \cdot F_y = 1.942 \times 10^3 \quad kip$

$\phi R_{\upsilon 1} := 378 \quad kip \qquad \phi R_{\upsilon 2} := 2420 \quad kip - in$

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 480.11 \quad kip$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 5

Column Shear in 6 story building NS direction-joint 5

$M_{pr} := 7.09\times 10^3$ kip − in $\quad V_{RBS} := 46.92($ kip $\quad V_{RBSprime} := 42.07$ kip

$S_h := 13.111$ in $\quad h_b := 13\cdot 12 = 156$ in $\quad h_t := 13\cdot 12 = 156$ in $\quad d_b := 23.7$ in $\quad t_f := 0.5$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 7.705\times 10^3$ kip − in $\quad M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 49.393 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 284.024 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X176

$A_g := 51.8$ in^2 $\quad F_y := 50$ ksi

$0.75\cdot A_g\cdot F_y = 1.942\times 10^3$ kip

$\phi R_{\upsilon 1} := 378$ kip $\quad \phi R_{\upsilon 2} := 242($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 480.11 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 6

Column Shear in 6 story building NS direction-joint 6

$M_{pr} := 3.498\times 10^3$ kip − in $\quad V_{RBS} := 24.08$ kip $\quad V_{RBSprime} := 19.06$ kip

$S_h := 10.324$ in $\quad h_b := 13\cdot 12 = 156$ in $\quad h_t := 0\cdot 12 = 0$ in $\quad d_b := 17.9$ in $\quad t_f := 0.525$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 3.747\times 10^3$ kip − in $\quad M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 48.034 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 167.601 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X176

$A_g := 51.8$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 1.942 \times 10^3$ kip

$\phi R_{\upsilon 1} := 378$ kip $\phi R_{\upsilon 2} := 2420$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 513.196 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building NS direction-joint 7

Column Shear in 6 story building NS direction-joint 7

$M_{pr} := 1.598 \times 10^4$ kip − in $V_{RBS} := 105.91$ kip $V_{RBSprime} := 101.13$ kip

$S_h := 17.47$ in $h_b := 18 \cdot 12 = 216$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 29.8$ in $t_f := 0.76$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.783 \times 10^4$ kip − in $M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.775 \times 10^4$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 191.281 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 1.034 \times 10^3 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.6$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.835 \times 10^3$ kip

$\phi R_{\upsilon 1} := 581$ kip $\phi R_{\upsilon 2} := 5140$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 753.483 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X257 and W30X108 Beam

$d_z := 0.314 \cdot 90$ $d_w := 0.14 \cdot 90$ $t_w := 1.18$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.454 \quad \text{in} \qquad \text{Smaller than tw} \qquad \text{OK}$$

$d_c := 16.4$ in $b_{cf} := 16$ in $t_{cf} := 1.89$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^{2}\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.571 \quad \text{in}$$

Use a 3/4 in thick doubler plate.

Check 6 story building NS direction-joint 8

Column Shear in 6 story building NS direction-joint 8

$M_{pr} := 1.598 \times 10^4$ kip – in $\quad V_{RBS} := 105.61$ kip $\quad V_{RBSprime} := 100.90$ kip

$S_h := 17.47$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 29.8$ in $\quad t_f := 0.76$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.783 \times 10^4$ kip – in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.774 \times 10^4$ kip – in

$$V_c := \frac{M_f + M_{fprime}}{\dfrac{h_b}{2} + \dfrac{h_t}{2}} = 228.006 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 996.82 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.6$ in^2 $\quad F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.835 \times 10^3$ kip

$\phi R_{v1} := 581$ kip $\quad \phi R_{v2} := 5140$ kip – in

$$\phi R_n := \phi R_{v1} + \frac{\phi R_{v2}}{d_b} = 753.483 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X257 and W30X99 Beam

$d_z := 0.314 \cdot 90$ $\quad d_w := 0.14 \cdot 90$ $\quad t_w := 1.18$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.454 \quad \text{in} \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 16.4$ in $\quad b_{cf} := 16$ in $\quad t_{cf} := 1.89$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^{2}\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.495 \quad \text{in}$$

Use a 1/2 in thick doubler plate.

Check 6 story building NS direction-joint 9

Column Shear in 6 story building NS direction-joint 9

$M_{pr} := 1.284\times 10^4$ kip − in $\quad V_{RBS} := 84.96$ kip $\quad V_{RBSprime} := 80.22$ kip

$S_h := 16.3$ in $\quad h_b := 13\cdot 12 = 156$ in $\quad h_t := 13\cdot 12 = 156$ in $\quad d_b := 27.$ in $\quad t_f := 0.74$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 1.423\times 10^4$ kip − in $\quad M_{fprime} := M_{pr} + V_{RBSprime}\cdot S_h = 1.415\times 10^4$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 181.929 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 874.892 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.$ in^2 $\quad F_y := 5$ ksi

$0.75\cdot A_g\cdot F_y = 2.835\times 10^3$ kip

$\phi R_{\upsilon 1} := 58$ kip $\quad \phi R_{\upsilon 2} := 514$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 767.232 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X257 and W30X90 Beam

$d_z := 0.314\cdot 9$ $\quad d_w := 0.14\cdot 9$ $\quad t_w := 1.1$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.454 \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 16.$ in $\quad b_{cf} := 1$ in $\quad t_{cf} := 1.8$ in

$$t_p := \left[R_u - \frac{0.6\cdot F_y\cdot\left(3\cdot b_{cf}\cdot t_{cf}^{\,2}\right)}{d_b}\right]\cdot\left(\frac{1}{0.6\cdot F_y\cdot d_c}\right) - t_w = 0.219 \quad \text{in}$$

Use a 1/4 in thick doubler plate.

Check 6 story building NS direction-joint 10

Column Shear in 6 story building NS direction-joint 10

$M_{pr} := 8.925\times 10^3$ kip − in $\quad V_{RBS} := 58.84$ kip $\quad V_{RBSprime} := 54.02$ kip

$S_h := 14.356$ in $\quad h_b := 13\cdot 12 = 156$ in $\quad h_t := 13\cdot 12 = 156$ in $\quad d_b := 23.$ in $\quad t_f := 0.6$ in

$M_f := M_{pr} + V_{RBS}\cdot S_h = 9.77\times 10^3$ kip − in $\quad M_{fprime} := M_{pr} + V_{RBSprime}\cdot S_h = 9.701\times 10^3$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 124.81 \text{ kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 720.992 \text{ kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X159

$A_g := 46.$ in^2 $\quad F_y := 50$ ksi

$0.75\cdot A_g\cdot F_y = 1.751\times 10^3$ kip

$\phi R_{\upsilon 1} := 33$ kip $\quad \phi R_{\upsilon 2} := 199$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 418.966 \text{ kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X159 and W24X76 Beam

$d_z := 0.25\cdot 90$ $\quad d_w := 0.14\cdot 90$ $\quad t_w := 0.74$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.39 \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 15$ in $\quad b_{cf} := 15.$ in $\quad t_{cf} := 1.1$ in

$$t_p := \left[R_u - \frac{0.6\cdot F_y\cdot\left(3\cdot b_{cf}\cdot t_{cf}^2\right)}{d_b}\right]\cdot\left(\frac{1}{0.6\cdot F_y\cdot d_c}\right) - t_w = 0.671 \text{ in}$$

Use a 3/4 in thick doubler plate.

Check 6 story building NS direction-joint 11

Column Shear in 6 story building NS direction-joint 11

$M_{pr} := 7.09\times 10^3$ kip − in $\quad V_{RBS} := 46.91$ kip $\quad V_{RBSprime} := 42.05$ kip

$S_h := 13.111$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 23.7$ in $\quad t_f := 0.59$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 7.705 \times 10^3$ kip − in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 7.641 \times 10^3$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 98.375 \text{ kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 565.689 \text{ kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X159

$A_g := 46.7$ in^2 $\quad F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 1.751 \times 10^3$ kip

$\phi R_{\upsilon 1} := 335$ kip $\quad \phi R_{\upsilon 2} := 1990$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 418.966 \text{ kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X1159 and W24X62 Beam

$d_z := 0.25 \cdot 90$ $\quad d_w := 0.14 \cdot 90$ $\quad t_w := 0.745$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.39$$ Smaller than tw OK

$d_c := 15$ in $\quad b_{cf} := 15.6$ in $\quad t_{cf} := 1.19$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.326 \text{ in}$$

Use a 1/2 in thick doubler plate.

Check 6 story building NS direction-joint 12

Column Shear in 6 story building NS direction-joint 12

$M_{pr} := 3.498 \times 10^3$ kip − in $\quad V_{RBS} := 24.079$ kip $\quad V_{RBSprime} := 19.056$ kip

$S_h := 10.324$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 0 \cdot 12 = 0$ in $\quad d_b := 17.9$ in $\quad t_f := 0.525$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 3.747 \times 10^3$ kip − in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 3.695 \times 10^3$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 95.402 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 332.877 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X159

$A_g := 46.7$ in^2 $\quad F_y := 5($ ksi

$0.75 \cdot A_g \cdot F_y = 1.751 \times 10^3$ kip

$\phi R_{\upsilon 1} := 335$ kip $\quad \phi R_{\upsilon 2} := 199($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 446.173 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 1

Column Shear in 6 story building EW direction-joint 1

$M_{pr} := 1.598 \times 10^4$ kip − in $\quad V_{RBS} := 106.07$ kip $\quad V_{RBSprime} := 101.2$ kip

$S_h := 17.47$ in $\quad h_b := 18 \cdot 12 = 216$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 29.8$ in $\quad t_f := 0.7($ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.783 \times 10^4$ kip − in $\quad M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 95.88 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 518.225 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.6$ in^2 $\quad F_y := 5($ ksi

$0.75 \cdot A_g \cdot F_y = 2.835 \times 10^3$ kip

$\phi R_{\upsilon 1} := 581$ kip $\quad \phi R_{\upsilon 2} := 514($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 753.483 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 2

Column Shear in 6 story building EW direction-joint 2

$M_{pr} := 1.598\times 10^{4}$ kip − in $V_{RBS} := 105.99$ kip $V_{RBSprime} := 101.29$ kip

$S_h := 17.47$ in $h_b := 13\cdot 12 = 156$ in $h_t := 13\cdot 12 = 156$ in $d_b := 29.8$ in $t_f := 0.76$

$M_f := M_{pr} + V_{RBS}\cdot S_h = 1.783\times 10^{4}$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 114.309 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 499.749 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.6$ in^2 $F_y := 50$ ksi

$0.75\cdot A_g\cdot F_y = 2.835\times 10^{3}$ kip

$\phi R_{\upsilon 1} := 581$ kip $\phi R_{\upsilon 2} := 5140$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 753.483 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 3

Column Shear in 6 story building EW direction-joint 3

$M_{pr} := 1.284\times 10^{4}$ kip − in $V_{RBS} := 85.05$ kip $V_{RBSprime} := 80.32$ kip

$S_h := 16.35$ in $h_b := 13\cdot 12 = 156$ in $h_t := 13\cdot 12 = 156$ in $d_b := 27.6$ in $t_f := 0.745$

$M_f := M_{pr} + V_{RBS}\cdot S_h = 1.423\times 10^{4}$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 91.222 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 438.685 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X257

$A_g := 75.($ in^2 $\quad F_y := 5($ ksi

$0.75 \cdot A_g \cdot F_y = 2.835 \times 10^3$ kip

$\phi R_{\upsilon 1} := 581$ kip $\quad \phi R_{\upsilon 2} := 514($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 767.232 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 4

Column Shear in 6 story building EW direction-joint 4

$M_{pr} := 1.284 \times 10^4$ kip − in $\quad V_{RBS} := 84.65$ kip $\quad V_{RBSprime} := 79.90$ kip

$S_h := 16.3$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 27.($ in $\quad t_f := 0.74$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.422 \times 10^4$ kip − in $\quad M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 91.18 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 438.484 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X176

$A_g := 51.8$ in^2 $\quad F_y := 5($ ksi

$0.75 \cdot A_g \cdot F_y = 1.942 \times 10^3$ kip

$\phi R_{\upsilon 1} := 378$ kip $\quad \phi R_{\upsilon 2} := 242($ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 465.681 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 5

Column Shear in 6 story building EW direction-joint 5

$M_{pr} := 8.925\times 10^{3}$ kip − in $V_{RBS} := 58.87$ kip $V_{RBSprime} := 54.06$ kip

$S_h := 14.356$ in $h_b := 13\cdot 12 = 156$ in $h_t := 13\cdot 12 = 156$ in $d_b := 23.9$ in $t_f := 0.68$

$M_f := M_{pr} + V_{RBS}\cdot S_h = 9.77\times 10^{3}$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 62.63 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 358.14 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X176

$A_g := 51.8$ in^2 $F_y := 50$ ksi

$0.75\cdot A_g\cdot F_y = 1.942\times 10^{3}$ kip

$\phi R_{\upsilon 1} := 378$ kip $\phi R_{\upsilon 2} := 2420$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 479.255 \quad \text{kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 6

Column Shear in 6 story building EW direction-joint 6

$M_{pr} := 3.498\times 10^{3}$ kip − in $V_{RBS} := 24.09$ kip $V_{RBSprime} := 19.07$ kip

$S_h := 10.324$ in $h_b := 13\cdot 12 = ■$ in $h_t := 0\cdot 12 = 0$ in $d_b := 17.9$ in $t_f := 0.525$

$M_f := M_{pr} + V_{RBS}\cdot S_h = 3.747\times 10^{3}$ kip − in $M_{fprime} := 0$ kip − in

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 48.035 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 167.604 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X176

$A_g := 51.8$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 1.942 \times 10^3$ kip

$\phi R_{\upsilon 1} := 378$ kip $\phi R_{\upsilon 2} := 2420$ kip – in

$\phi R_n := \phi R_{\upsilon 1} + \dfrac{\phi R_{\upsilon 2}}{d_b} = 513.196$ kip

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 7

Column Shear in 6 story building EW direction-joint 7

$M_{pr} := 1.598 \times 10^4$ kip – in $V_{RBS} := 106.18$ kip $V_{RBSprime} := 101.34$ kip

$S_h := 17.47$ in $h_b := 18 \cdot 12 = 216$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 29.8$ in $t_f := 0.76$ in

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.784 \times 10^4$ kip – in $M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.775 \times 10^4$

$$V_c := \frac{M_f + M_{fprime}}{\dfrac{h_b}{2} + \dfrac{h_t}{2}} = 191.325 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 1.034 \times 10^3 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X311

$A_g := 91.4$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 3.428 \times 10^3$ kip

$\phi R_{\upsilon 1} := 723$ kip $\phi R_{\upsilon 2} := 7450$ kip – in

$\phi R_n := \phi R_{\upsilon 1} + \dfrac{\phi R_{\upsilon 2}}{d_b} = 973$ kip

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X311 and W30X108 Beam

$d_z := 0.314 \cdot 90$ $d_w := 0.14 \cdot 90$ $t_w := 1.41$ in

$\dfrac{d_z}{90} + \dfrac{d_w}{90} = 0.454$ in Smaller than tw OK

$d_c := 17.1$ in $b_{cf} := 16.2$ in $t_{cf} := 2.26$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^{2}\right)}{d_b} \right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c} \right) - t_w = 0.119 \quad \text{in}$$

Use a 1/4 in thick doubler plate.

Check 6 story building EW direction-joint 8

Column Shear in 6 story building EW direction-joint 8

$M_{pr} := 1.598\times 10^{4}$ kip – in $\quad V_{RBS} := 106.10$ kip $\quad V_{RBSprime} := 101.41$ kip

$S_h := 17.47$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 29.8$ in $\quad t_f := 0.76$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.783\times 10^{4}$ kip – in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.775\times 10^{4}$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 228.119 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 997.313 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X311

$A_g := 91.4$ in^2 $\quad F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 3.428\times 10^{3}$ kip

$\phi R_{\upsilon 1} := 723$ kip $\quad \phi R_{\upsilon 2} := 7450$ kip – in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 973 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X311 and W30X108 Beam

$d_z := 0.314 \cdot 90$ $\quad d_w := 0.14 \cdot 90$ $\quad t_w := 1.41$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.454 \quad \text{in} \qquad \text{Smaller than tw} \qquad \text{OK}$$

$d_c := 17.1$ in $\quad b_{cf} := 16.2$ in $\quad t_{cf} := 2.26$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^{2}\right)}{d_b} \right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c} \right) - t_w = 0.047 \quad \text{in}$$

Use a 1/4 in thick doubler plate.

Check 6 story building EW direction-joint 9

Column Shear in 6 story building EW direction-joint 9

$M_{pr} := 1.284\times 10^4$ kip – in $V_{RBS} := 85.14$ kip $V_{RBSprime} := 80.42$ kip

$S_h := 16.3$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 27.$ in $t_f := 0.74$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.423\times 10^4$ kip – in $M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.415\times 10^4$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 181.968 \text{ kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 875.08 \text{ kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X311

$A_g := 91.$ in^2 $F_y := 5$ ksi

$0.75 \cdot A_g \cdot F_y = 3.428\times 10^3$ kip

$\phi R_{\upsilon 1} := 72$ kip $\phi R_{\upsilon 2} := 745$ kip – in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 992.928 \text{ kip}$$

We don't need the column-web doubler plate.

Check 6 story building EW direction-joint 10

Column Shear in 6 story building EW direction-joint 10

$M_{pr} := 1.284\times 10^4$ kip – in $V_{RBS} := 84.65$ kip $V_{RBSprime} := 79.90$ kip

$S_h := 16.3$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 13 \cdot 12 = 156$ in $d_b := 27.$ in $t_f := 0.74$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 1.422\times 10^4$ kip – in $M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 1.415\times 10^4$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 181.862 \text{ kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 874.572 \text{ kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X211

$A_g := 62$ in^2 $\quad F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.325 \times 10^3$ kip

$\phi R_{\upsilon 1} := 462$ kip $\quad \phi R_{\upsilon 2} := 3460$ kip – in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 587.362 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X211 and W27X94 Beam

$d_z := 0.282 \cdot 90$ $\quad d_w := 0.14 \cdot 90$ $\quad t_w := 0.98$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.422 \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 15.7$ in $\quad b_{cf} := 15.8$ in $\quad t_{cf} := 1.56$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.611 \quad \text{in}$$

Use a 3/4 in thick doubler plate.

Check 6 story building NS direction-joint 11

Column Shear in 6 story building NS direction-joint 11

$M_{pr} := 8.925 \times 10^3$ kip – in $\quad V_{RBS} := 58.87$ kip $\quad V_{RBSprime} := 54.06$ kip

$S_h := 14.356$ in $\quad h_b := 13 \cdot 12 = 156$ in $\quad h_t := 13 \cdot 12 = 156$ in $\quad d_b := 23.9$ in $\quad t_f := 0.68$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 9.77 \times 10^3$ kip – in $\quad M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 9.701 \times 10^3$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 124.817 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 713.745 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X211

$A_g := 62$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.325\times 10^3$ kip

$\phi R_{\upsilon 1} := 462$ kip $\phi R_{\upsilon 2} := 3460$ kip − in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 606.77 \quad \text{kip}$$

We need the column-web doubler plate.

Size Web Doubler Plate

From Table 4-2 of AISC Seismic Design Manual for Column W14X211 and W24X76 Beam

$d_z := 0.25 \cdot 90$ $d_w := 0.14 \cdot 90$ $t_w := 0.83$ in

$$\frac{d_z}{90} + \frac{d_w}{90} = 0.39 \qquad \text{Smaller than tw} \quad \text{OK}$$

$d_c := 15.7$ in $b_{cf} := 15.8$ in $t_{cf} := 1.56$ in

$$t_p := \left[R_u - \frac{0.6 \cdot F_y \cdot \left(3 \cdot b_{cf} \cdot t_{cf}^2\right)}{d_b}\right] \cdot \left(\frac{1}{0.6 \cdot F_y \cdot d_c}\right) - t_w = 0.378 \quad \text{in}$$

Use a 1/2 in thick doubler plate.

Check 6 story building EW direction-joint 12

Column Shear in 6 story building EW direction-joint 12

$M_{pr} := 3.498\times 10^3$ kip − in $V_{RBS} := 24.09$ kip $V_{RBSprime} := 19.07$ kip

$S_h := 10.324$ in $h_b := 13 \cdot 12 = 156$ in $h_t := 0 \cdot 12 = 0$ in $d_b := 17.9$ in $t_f := 0.525$

$M_f := M_{pr} + V_{RBS} \cdot S_h = 3.747\times 10^3$ kip − in $M_{fprime} := M_{pr} + V_{RBSprime} \cdot S_h = 3.695\times 10^3$

$$V_c := \frac{M_f + M_{fprime}}{\frac{h_b}{2} + \frac{h_t}{2}} = 95.405 \quad \text{kip}$$

The Required Strength of the Panel Zone

$$R_u := \frac{M_f + M_{fprime}}{d_b - t_f} - V_c = 332.889 \quad \text{kip}$$

According to Table 4-2 of AISC Seismic Design Manual for Column W14X211

$A_g := 62$ in^2 $F_y := 50$ ksi

$0.75 \cdot A_g \cdot F_y = 2.325\times 10^3$ kip

$\phi R_{\upsilon 1} := 462$ kip $\quad \phi R_{\upsilon 2} := 3460$ kip $-$ in

$$\phi R_n := \phi R_{\upsilon 1} + \frac{\phi R_{\upsilon 2}}{d_b} = 655.296 \text{ kip}$$

We don't need the column-web doubler plate.

Appendix B

Table B-1: Collected Result for 3 story MF as Beam--Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor Moment Frame Rotation(EXT)	2nd Floor Moment Frame Rotation(EXT)	3rd Floor Moment Frame Rotation(EXT)
FF01-1	5.80%	7.40%	8.60%	0.08	0.05	0.02	0.07	0.05	0.02	0.067	0.079	0.084
FF13-1	3.20%	2.90%	4.00%	0.09	0.06	0.02	0.06	0.04	0.02	0.034	0.038	0.063
FF14-1	1.50%	2.50%	4.10%	0.08	0.06	0.02	0.07	0.05	0.02	0.007	0.019	0.027
FF14-2	8.70%	10.30%	10.70%	0.09	0.05	0.02	0.06	0.04	0.02	0.104	0.107	0.105
FF15-2	3.60%	3.60%	3.50%	0.08	0.05	0.02	0.06	0.04	0.02	0.025	0.023	0.034
FF19-1	1.70%	2.10%	2.60%	0.09	0.05	0.02	0.06	0.04	0.02	0.007	0.009	0.015
FF21-2	1.10%	1.20%	1.60%	0.08	0.05	0.02	0.06	0.04	0.02	0.008	0.004	0.018
FF22-1	1.50%	2.00%	2.30%	0.08	0.05	0.02	0.07	0.04	0.02	0.024	0.013	0.030
FF22-2	2.60%	2.70%	6.30%	0.09	0.05	0.02	0.07	0.05	0.03	0.011	0.022	0.064
NF02-2	0.89%	1.40%	2.40%	0.09	0.05	0.02	0.07	0.05	0.03	0.035	0.042	0.065
NF05-1	2.50%	3.50%	3.80%	0.09	0.05	0.02	0.06	0.04	0.02	0.041	0.034	0.056
NF05-2	1.20%	1.90%	2.70%	0.09	0.05	0.02	0.06	0.04	0.02	0.028	0.024	0.040
NF16-1	4.70%	6.30%	9.40%	0.09	0.05	0.02	0.07	0.05	0.02	0.034	0.071	0.097
NF16-2	1.70%	2.40%	4.90%	0.09	0.06	0.02	0.07	0.05	0.02	0.008	0.020	0.038
NF17-2	3.60%	4.70%	5.10%	0.09	0.05	0.02	0.07	0.04	0.02	0.029	0.038	0.063
NF21-2	2.70%	4.20%	6.00%	0.09	0.06	0.02	0.07	0.05	0.02	0.025	0.066	0.072
NF22-1	3.10%	4.00%	4.20%	0.09	0.05	0.02	0.06	0.04	0.02	0.025	0.027	0.029
NF25-2	3.60%	3.80%	3.70%	0.09	0.05	0.02	0.07	0.05	0.02	0.028	0.028	0.037
NF27-2	2.70%	3.30%	4.80%	0.09	0.05	0.02	0.06	0.04	0.02	0.017	0.026	0.039
NF28-1	1.80%	2.60%	3.60%	0.09	0.05	0.02	0.07	0.05	0.03	0.083	0.083	0.097

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	0.070	0.082	0.101	0.778	0.892	3.069	0.708	0.756	3.158	1.109	1.269	0.982
FF13-1	0.032	0.036	0.043	0.892	0.831	3.125	0.794	0.797	3.228	0.698	0.762	1.223
FF14-1	0.005	0.020	0.043	0.669	0.903	2.658	0.578	0.622	2.786	1.164	1.423	1.584
FF14-2	0.106	0.109	0.119	0.764	0.858	3.258	0.733	0.833	3.256	0.728	0.727	0.797
FF15-2	0.028	0.021	0.033	0.981	0.864	2.381	0.800	0.836	2.394	0.630	0.506	0.639
FF19-1	0.008	0.012	0.023	0.642	0.631	2.186	0.511	0.536	2.214	0.395	0.844	0.567
FF21-2	0.005	0.001	0.013	0.581	0.525	2.347	0.486	0.486	2.156	0.319	0.338	0.726
FF22-1	0.021	0.009	0.020	0.597	0.767	2.300	0.544	0.569	2.267	0.576	0.646	0.817
FF22-2	0.013	0.024	0.081	0.797	0.867	2.742	0.583	0.653	2.847	0.910	1.317	1.612
NF02-2	0.033	0.039	0.045	0.664	0.650	3.175	0.603	0.608	3.372	0.732	0.693	1.175
NF05-1	0.038	0.031	0.039	0.636	0.708	2.794	0.683	0.728	2.978	0.840	0.855	0.751
NF05-2	0.025	0.021	0.027	0.703	0.819	2.300	0.592	0.650	2.406	0.604	1.000	0.791
NF16-1	0.037	0.073	0.111	0.739	0.969	3.108	0.708	0.819	3.036	1.343	2.028	1.519
NF16-2	0.010	0.022	0.062	0.786	0.858	3.222	0.569	0.614	3.278	0.827	0.971	1.497
NF17-2	0.032	0.040	0.061	0.872	0.944	3.269	0.733	0.750	3.278	1.215	0.736	1.266
NF21-2	0.024	0.063	0.075	0.883	0.906	3.294	0.639	0.789	3.436	1.536	1.434	1.536
NF22-1	0.027	0.029	0.044	0.586	0.642	2.794	0.514	0.578	2.794	0.582	0.729	1.145
NF25-2	0.030	0.025	0.036	0.758	0.789	2.606	0.686	0.706	2.667	1.021	1.280	1.228
NF27-2	0.019	0.028	0.059	0.708	0.725	3.086	0.667	0.686	3.086	0.596	0.570	0.842
NF28-1	0.080	0.080	0.081	0.600	0.681	3.089	0.744	0.725	3.286	0.711	1.118	1.388

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)
FF01-1	0.815	0.896	1.167	3.30%	5.10%	5.20%	0.912	35.33	85.50	34.07	317.13	385.72	137.65
FF13-1	0.717	0.679	1.054	0.15%	0.28%	0.21%	0.934	171.92	185.34	96.65	112.01	115.68	52.51
FF14-1	1.124	1.341	1.427	0.18%	1.20%	0.86%	1.069	36.79	89.30	55.89	11.44	41.45	25.63
FF14-2	0.820	0.738	0.846	5.30%	8.00%	8.20%	0.782	56.22	54.04	16.17	629.35	632.62	192.76
FF15-2	0.626	0.670	0.549	0.23%	1.00%	1.30%	0.793	54.07	59.47	31.96	153.05	141.38	32.33
FF19-1	0.563	0.539	0.664	0.28%	0.26%	0.16%	0.634	33.69	36.32	17.19	26.83	42.09	13.71
FF21-2	0.435	0.650	0.743	0.07%	0.42%	0.49%	0.669	27.12	12.44	15.13	8.68	1.95	6.19
FF22-1	0.819	0.658	0.572	1.30%	1.50%	0.95%	0.914	131.07	65.68	31.03	13.22	16.50	18.81
FF22-2	1.096	0.970	1.460	0.75%	1.10%	2.80%	1.055	12.41	54.61	40.45	76.39	133.01	104.69
NF02-2	1.205	1.100	1.636	0.89%	1.80%	2.20%	0.695	86.63	178.24	78.08	91.51	74.99	30.34
NF05-1	0.968	0.777	0.941	1.20%	0.86%	0.64%	0.989	178.24	141.60	74.15	127.38	129.66	45.53
NF05-2	0.860	0.688	0.854	0.41%	0.36%	0.49%	0.924	95.11	88.91	54.73	54.05	61.49	22.83
NF16-1	1.200	1.152	1.239	2.70%	4.90%	5.60%	0.993	16.13	64.49	52.81	103.17	294.33	110.85
NF16-2	0.636	1.072	1.591	0.02%	0.15%	0.68%	0.968	31.21	105.58	47.69	31.84	93.56	61.18
NF17-2	1.023	0.763	1.318	0.93%	1.00%	0.18%	0.988	183.69	250.37	123.58	53.74	77.74	57.79
NF21-2	1.107	0.985	1.131	0.87%	2.80%	2.40%	0.99	165.83	466.56	149.05	68.59	107.37	51.96
NF22-1	0.629	0.724	1.081	1.00%	1.40%	1.50%	0.809	43.87	44.49	22.93	77.68	88.53	29.46
NF25-2	0.932	0.998	1.334	0.39%	0.21%	0.11%	0.598	239.80	197.48	59.16	64.68	77.00	26.49
NF27-2	0.808	0.926	0.935	0.33%	0.41%	0.49%	0.895	76.95	120.85	52.54	81.45	117.81	78.54
NF28-1	0.863	1.065	1.716	4.30%	6.50%	6.70%	0.718	448.80	478.45	150.92	35.44	39.69	17.94

Table B-2: Collected Result for 3 story MF as beam--Horizontal+ Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor Moment Frame Rotation(EXT)	2nd Floor Moment Frame Rotation(EXT)	3rd Floor Moment Frame Rotation(EXT)
FF01-1	5.80%	7.40%	8.60%	0.10	0.07	0.03	0.12	0.08	0.04	0.067	0.079	0.082
FF13-1	2.90%	2.60%	3.70%	0.14	0.09	0.04	0.06	0.04	0.02	0.031	0.040	0.066
FF14-1	1.50%	2.50%	4.10%	0.10	0.06	0.02	0.07	0.05	0.02	0.008	0.019	0.030
FF14-2	8.70%	10.30%	10.70%	0.10	0.06	0.02	0.06	0.04	0.02	0.099	0.102	0.100
FF15-2	3.60%	3.60%	3.60%	0.15	0.10	0.04	0.06	0.04	0.02	0.025	0.023	0.034
FF19-1	1.70%	2.20%	2.60%	0.09	0.06	0.02	0.09	0.06	0.03	0.007	0.010	0.016
FF21-2	1.10%	1.20%	1.60%	0.08	0.06	0.02	0.17	0.11	0.04	0.008	0.004	0.017
FF22-1	1.50%	2.00%	2.30%	0.11	0.07	0.03	0.12	0.08	0.04	0.023	0.015	0.036
FF22-2	2.60%	2.70%	6.30%	0.10	0.06	0.03	0.12	0.09	0.04	0.011	0.022	0.067
NF02-2	0.93%	1.50%	2.40%	0.10	0.06	0.02	0.14	0.10	0.05	0.036	0.042	0.066
NF05-1	2.60%	3.60%	3.90%	0.14	0.09	0.03	0.16	0.11	0.04	0.040	0.035	0.052
NF05-2	1.30%	2.10%	2.80%	0.12	0.08	0.03	0.16	0.11	0.04	0.026	0.023	0.039
NF16-1	4.60%	6.10%	9.40%	0.13	0.08	0.03	0.27	0.21	0.11	0.033	0.068	0.098
NF16-2	1.60%	2.30%	4.70%	0.11	0.08	0.03	0.18	0.13	0.07	0.008	0.021	0.036
NF17-2	3.60%	4.70%	5.10%	0.09	0.06	0.03	0.15	0.11	0.06	0.029	0.037	0.063
NF21-2	2.70%	4.10%	5.90%	0.10	0.07	0.03	0.15	0.10	0.05	0.025	0.067	0.075
NF22-1	3.10%	4.00%	4.20%	0.10	0.06	0.03	0.11	0.08	0.04	0.026	0.027	0.028
NF25-2	3.60%	3.80%	3.70%	0.10	0.06	0.02	0.10	0.07	0.04	0.027	0.027	0.037
NF27-2	2.60%	3.20%	4.80%	0.10	0.06	0.02	0.10	0.07	0.04	0.016	0.026	0.039
NF28-1	1.80%	2.60%	3.50%	0.09	0.06	0.02	0.11	0.07	0.04	0.082	0.082	0.097

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	0.070	0.082	0.101	0.850	0.939	3.936	0.750	0.828	3.869	1.491	2.043	2.276
FF13-1	0.029	0.037	0.042	1.242	1.442	5.514	1.500	1.889	5.394	8.218	9.924	7.163
FF14-1	0.005	0.020	0.041	0.864	0.928	3.303	0.828	1.017	3.258	5.000	4.817	4.120
FF14-2	0.101	0.105	0.114	1.092	0.994	3.539	1.078	1.167	3.656	4.955	4.799	4.115
FF15-2	0.028	0.021	0.033	1.286	1.194	4.256	1.147	0.836	4.006	4.716	5.065	6.629
FF19-1	0.009	0.012	0.023	0.669	0.675	2.658	0.556	0.586	2.664	0.715	0.909	1.383
FF21-2	0.005	0.001	0.013	0.583	0.711	2.339	0.753	1.028	2.442	3.289	4.078	2.313
FF22-1	0.022	0.012	0.023	0.794	1.103	3.836	0.778	0.936	3.517	2.880	3.759	4.604
FF22-2	0.013	0.025	0.079	0.889	0.903	3.769	0.714	0.864	3.708	2.740	3.461	3.851
NF02-2	0.033	0.039	0.045	0.758	0.714	3.308	0.683	0.692	3.439	3.607	3.854	3.281
NF05-1	0.037	0.032	0.041	1.381	1.394	3.222	1.250	1.297	3.194	5.660	5.963	3.854
NF05-2	0.023	0.021	0.029	0.956	1.150	3.072	1.025	1.267	2.994	5.449	5.675	4.109
NF16-1	0.035	0.070	0.110	0.861	1.061	3.458	0.853	1.139	3.489	4.712	6.010	5.332
NF16-2	0.010	0.022	0.056	0.847	1.006	3.417	0.811	1.031	3.297	3.852	3.218	3.412
NF17-2	0.031	0.040	0.062	0.944	0.997	3.867	0.869	1.014	4.133	3.104	3.297	4.836
NF21-2	0.023	0.064	0.078	0.897	0.867	3.950	0.764	0.794	4.164	2.771	2.628	4.067
NF22-1	0.027	0.029	0.052	0.719	0.792	3.417	0.717	0.836	3.519	3.581	3.170	3.403
NF25-2	0.030	0.026	0.036	0.769	0.817	2.844	0.744	0.803	2.839	1.688	2.098	1.800
NF27-2	0.018	0.028	0.056	0.733	0.789	3.231	0.672	0.761	3.236	1.335	1.407	1.870
NF28-1	0.080	0.080	0.080	0.683	0.694	3.347	0.722	0.742	3.428	2.371	2.467	2.123

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)
FF01-1	2.004	2.052	2.729	3.30%	5.10%	5.20%	0.919	35.36	84.45	31.92	317.39	382.79
FF13-1	9.415	15.016	7.463	0.26%	0.49%	0.43%	1.243	152.16	168.55	90.65	105.60	127.87
FF14-1	6.500	8.758	6.338	0.18%	1.20%	0.86%	1.090	39.24	91.97	59.97	11.87	41.32
FF14-2	6.471	8.751	6.305	5.30%	8.00%	8.20%	0.841	52.28	52.77	16.37	581.36	582.00
FF15-2	6.419	6.178	5.611	0.26%	0.96%	1.10%	0.920	54.22	59.57	39.27	155.15	143.24
FF19-1	1.013	1.042	1.948	0.27%	0.27%	0.17%	0.643	34.07	38.26	17.46	27.32	41.86
FF21-2	5.608	8.923	3.591	0.07%	0.42%	0.48%	0.809	26.98	13.08	15.00	8.54	1.99
FF22-1	2.895	3.796	3.965	1.30%	1.60%	1.10%	0.988	124.22	69.41	33.26	14.43	23.01
FF22-2	2.669	4.076	4.105	0.74%	1.10%	2.70%	1.042	11.69	54.27	44.77	76.12	133.76
NF02-2	2.801	3.952	4.373	0.85%	1.80%	2.10%	0.699	84.84	178.64	80.52	94.64	78.31
NF05-1	7.138	8.331	3.779	1.10%	0.73%	0.57%	1.163	179.84	145.59	66.72	137.61	143.62
NF05-2	6.683	8.535	4.206	0.29%	0.20%	0.36%	1.173	92.46	96.79	56.19	51.24	65.27
NF16-1	1.200	1.152	1.239	2.60%	4.70%	5.50%	1.131	17.42	64.93	54.85	104.19	289.50
NF16-2	5.265	6.665	5.609	0.00%	0.22%	0.55%	1.000	31.95	110.04	46.52	31.65	93.86
NF17-2	4.208	6.287	5.730	0.91%	1.00%	0.16%	0.956	184.16	248.16	125.17	54.57	77.63
NF21-2	3.943	3.019	4.759	0.92%	2.90%	2.60%	1.022	171.35	469.41	155.51	66.74	107.52
NF22-1	3.395	4.891	3.632	1.00%	1.50%	1.60%	0.859	45.60	44.94	20.98	80.32	90.41
NF25-2	1.702	2.440	2.855	0.35%	0.15%	0.05%	0.665	237.66	195.40	57.69	63.16	75.07
NF27-2	1.921	2.358	2.067	0.28%	0.33%	0.40%	0.912	77.01	127.73	57.89	79.18	114.03
NF28-1	2.230	3.355	3.269	4.20%	6.50%	6.70%	0.708	447.04	477.36	150.81	35.45	39.82

Table B-3: Collected Result for 3 story MF as Girder--Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor Moment Frame Rotation(EXT)	2nd Floor Moment Frame Rotation(EXT)	3rd Floor Moment Frame Rotation(EXT)
FF01-1	5.30%	6.60%	8.20%	0.05	0.04	0.02	0.11	0.09	0.06	0.047	0.065	0.069
FF13-1	1.90%	2.60%	2.90%	0.05	0.04	0.02	0.11	0.09	0.06	0.047	0.075	0.095
FF14-1	1.70%	2.70%	3.40%	0.05	0.04	0.02	0.11	0.09	0.06	0.005	0.015	0.028
FF14-2	4.40%	5.00%	5.70%	0.05	0.04	0.02	0.10	0.09	0.06	0.032	0.032	0.039
FF15-2	2.00%	3.00%	3.60%	0.05	0.04	0.02	0.11	0.08	0.06	0.033	0.044	0.067
FF19-1	2.80%	3.30%	3.00%	0.05	0.03	0.02	0.11	0.08	0.06	0.037	0.037	0.040
FF21-2	1.10%	1.30%	1.90%	0.04	0.03	0.02	0.11	0.08	0.06	0.008	0.008	0.023
FF22-1	1.60%	2.50%	2.90%	0.04	0.03	0.02	0.10	0.08	0.06	0.018	0.013	0.025
FF22-2	2.50%	2.70%	5.60%	0.05	0.04	0.02	0.11	0.09	0.06	0.006	0.016	0.036
NF02-2	0.70%	1.10%	1.20%	0.05	0.03	0.02	0.11	0.09	0.06	0.063	0.071	0.078
NF05-1	4.50%	6.00%	6.50%	0.05	0.03	0.02	0.11	0.08	0.06	0.035	0.044	0.043
NF05-2	4.70%	5.50%	5.50%	0.05	0.04	0.02	0.12	0.08	0.06	0.035	0.033	0.032
NF16-1	8.70%	9.70%	11.20%	0.05	0.04	0.02	0.11	0.11	0.09	0.084	0.091	0.099
NF16-2	1.50%	2.10%	4.20%	0.05	0.04	0.03	0.11	0.09	0.06	0.007	0.019	0.033
NF17-2	3.60%	3.90%	4.30%	0.05	0.03	0.02	0.11	0.08	0.06	0.020	0.026	0.062
NF21-2	2.30%	3.60%	4.80%	0.05	0.04	0.02	0.11	0.08	0.06	0.011	0.042	0.063
NF22-1	3.00%	4.90%	5.80%	0.05	0.04	0.02	0.11	0.09	0.07	0.020	0.032	0.035
NF25-2	7.10%	8.50%	8.70%	0.05	0.03	0.02	0.11	0.08	0.06	0.073	0.077	0.070
NF27-2	1.80%	2.90%	3.30%	0.05	0.03	0.02	0.10	0.08	0.05	0.025	0.037	0.061
NF28-1	3.10%	3.40%	3.60%	0.04	0.03	0.01	0.10	0.09	0.06	0.099	0.100	0.107

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	0.064	0.080	0.094	4.792	5.000	8.239	1.489	1.647	3.647	0.435	0.651	1.911
FF13-1	0.049	0.078	0.084	3.333	3.622	8.797	1.497	1.589	3.719	0.877	0.701	2.136
FF14-1	0.011	0.021	0.032	3.328	4.086	8.272	1.286	1.353	3.272	1.119	1.113	2.778
FF14-2	0.048	0.048	0.068	4.425	4.481	7.486	1.469	1.436	3.769	0.679	0.722	2.324
FF15-2	0.035	0.046	0.043	3.486	3.900	8.289	1.469	0.836	3.825	0.782	0.698	2.569
FF19-1	0.038	0.038	0.029	3.636	3.622	6.942	1.233	1.225	3.128	0.364	0.422	1.353
FF21-2	0.007	0.008	0.016	2.325	2.822	6.631	1.036	1.194	2.808	0.561	0.524	2.403
FF22-1	0.016	0.017	0.026	3.247	3.561	6.958	1.153	1.175	2.947	1.244	0.591	1.147
FF22-2	0.016	0.027	0.074	3.708	4.194	8.425	1.197	1.267	4.172	1.122	1.363	1.811
NF02-2	0.064	0.073	0.070	2.436	2.547	4.525	1.256	1.314	3.792	0.402	0.555	0.727
NF05-1	0.053	0.064	0.074	4.508	4.856	7.481	1.503	1.478	3.389	0.813	0.696	1.734
NF05-2	0.054	0.050	0.068	4.406	4.444	8.856	1.556	1.553	4.100	0.882	0.665	3.11
NF16-1	0.100	0.107	0.131	5.508	6.042	9.542	1.681	1.542	3.344	1.54	1.78	1.897
NF16-2	0.010	0.022	0.046	3.033	4.050	8.811	1.083	1.181	3.114	0.689	0.905	3.157
NF17-2	0.032	0.039	0.041	4.311	4.342	7.281	1.317	1.347	3.556	0.711	0.732	1.436
NF21-2	0.024	0.041	0.059	3.867	4.331	7.783	1.256	1.453	3.808	0.881	0.912	2.964
NF22-1	0.033	0.052	0.072	3.847	4.836	7.803	1.131	1.303	4.467	0.96	0.651	2.501
NF25-2	0.088	0.093	0.096	5.008	5.275	6.878	1.489	1.394	3.583	0.268	0.632	0.785
NF27-2	0.025	0.038	0.039	3.328	3.667	6.431	1.217	1.314	3.494	0.555	0.658	1.5
NF28-1	0.104	0.104	0.098	3.833	3.681	6.175	1.478	1.400	3.572	0.477	0.565	0.591

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)
FF01-1	0.62	0.979	0.775	3.20%	5.40%	5.70%	0.869	9.67	30.70	28.55	143.07	220.79	121.20
FF13-1	0.855	0.565	1.006	2.60%	4.30%	4.60%	0.857	100.29	167.31	104.49	17.22	52.16	38.99
FF14-1	0.788	0.702	0.99	0.59%	1.50%	1.20%	1.072	3.93	11.44	20.92	9.82	27.18	22.56
FF14-2	0.507	0.397	0.547	0.22%	0.11%	0.20%	0.728	58.89	50.94	31.51	128.89	140.48	88.06
FF15-2	0.474	0.411	0.689	1.10%	1.80%	2.00%	0.921	18.79	40.24	53.93	86.54	100.70	44.31
FF19-1	0.271	0.417	0.455	0.83%	1.30%	1.30%	0.579	67.41	78.94	35.67	61.62	65.74	24.41
FF21-2	0.391	0.538	0.839	0.41%	0.66%	0.64%	0.731	6.75	9.09	12.24	16.92	20.27	17.62
FF22-1	0.587	0.52	0.581	0.52%	0.66%	0.34%	0.85	36.43	23.42	8.88	31.20	38.94	27.81
FF22-2	0.596	0.655	1.411	0.50%	0.64%	1.20%	1.114	2.22	14.12	37.28	41.12	77.55	83.09
NF02-2	0.576	0.633	0.826	2.70%	4.60%	4.90%	0.577	122.82	156.78	76.31	28.52	32.07	18.36
NF05-1	0.47	0.534	0.517	1.90%	3.10%	3.30%	0.763	24.97	31.42	24.60	146.96	204.19	113.71
NF05-2	0.391	0.474	1.049	0.78%	1.10%	1.20%	0.725	48.78	40.22	26.13	128.36	121.01	69.48
NF16-1	0.903	0.874	0.738	5.50%	8.20%	8.20%	1.04	12.49	18.23	23.23	235.00	245.71	145.18
NF16-2	0.39	0.69	0.799	0.24%	0.44%	0.12%	0.948	5.98	22.13	25.63	25.61	67.17	51.52
NF17-2	0.492	0.444	0.804	0.15%	0.21%	1.20%	1.165	81.37	105.43	91.45	42.12	59.45	34.28
NF21-2	0.942	0.807	0.72	0.00%	1.40%	2.00%	1.044	19.30	80.61	88.85	25.62	79.39	53.89
NF22-1	0.487	0.556	1.395	1.20%	2.30%	2.50%	0.684	10.98	24.53	18.59	23.76	55.75	46.95
NF25-2	0.466	0.71	0.706	4.50%	7.00%	7.20%	0.53	11.33	12.62	5.87	190.04	215.82	106.47
NF27-2	0.375	0.438	0.524	0.63%	1.10%	1.20%	0.759	43.51	55.50	43.26	47.97	81.34	45.35
NF28-1	0.726	0.489	0.952	5.00%	7.60%	7.80%	0.568	231.39	240.43	116.76	53.24	46.10	25.67

Table B-4: Collected Result for 3 story MF as Girder--Horizontal+ Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor Moment Frame Rotation(EXT)	2nd Floor Moment Frame Rotation(EXT)	3rd Floor Moment Frame Rotation(EXT)
FF01-1	5.20%	6.60%	8.10%	0.06	0.05	0.02	0.17	0.14	0.10	0.045	0.063	0.064
FF13-1	1.80%	2.50%	2.90%	0.09	0.05	0.02	0.24	0.22	0.13	0.032	0.040	0.069
FF14-1	1.70%	2.60%	3.30%	0.06	0.04	0.03	0.18	0.17	0.11	0.005	0.016	0.026
FF14-2	4.40%	5.00%	5.70%	0.06	0.04	0.02	0.18	0.17	0.11	0.032	0.032	0.041
FF15-2	1.60%	2.70%	3.50%	0.10	0.06	0.03	0.24	0.20	0.14	0.080	0.086	0.092
FF19-1	2.70%	3.20%	3.00%	0.05	0.03	0.02	0.12	0.11	0.07	0.036	0.036	0.042
FF21-2	1.10%	1.30%	1.80%	0.09	0.06	0.03	0.29	0.27	0.17	0.009	0.008	0.023
FF22-1	1.60%	2.40%	2.90%	0.07	0.05	0.02	0.18	0.17	0.11	0.021	0.018	0.032
FF22-2	2.30%	2.50%	5.30%	0.07	0.05	0.02	0.16	0.16	0.11	0.006	0.019	0.034
NF02-2	0.71%	1.10%	1.20%	0.05	0.03	0.02	0.15	0.13	0.08	0.063	0.070	0.081
NF05-1	5.20%	6.60%	6.90%	0.06	0.04	0.02	0.16	0.15	0.10	0.046	0.059	0.043
NF05-2	4.80%	5.80%	6.10%	0.07	0.05	0.03	0.16	0.16	0.10	0.041	0.039	0.042
NF16-1	8.70%	9.80%	11.20%	0.06	0.04	0.02	0.20	0.18	0.12	0.085	0.092	0.100
NF16-2	1.50%	2.10%	4.30%	0.06	0.05	0.03	0.15	0.15	0.09	0.006	0.016	0.029
NF17-2	3.60%	3.90%	4.30%	0.06	0.04	0.02	0.17	0.15	0.10	0.020	0.026	0.064
NF21-2	2.30%	3.70%	4.80%	0.08	0.06	0.03	0.24	0.23	0.15	0.011	0.043	0.065
NF22-1	2.90%	4.80%	5.70%	0.05	0.04	0.02	0.14	0.14	0.10	0.022	0.037	0.037
NF25-2	8.00%	9.40%	9.70%	0.06	0.04	0.02	0.14	0.13	0.09	0.083	0.086	0.080
NF27-2	1.70%	2.80%	3.10%	0.07	0.04	0.02	0.16	0.13	0.09	0.030	0.045	0.072
NF28-1	3.10%	3.50%	3.70%	0.06	0.04	0.02	0.16	0.15	0.10	0.098	0.097	0.105

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	0.063	0.080	0.095	5.333	5.644	9.061	1.931	1.997	5.225	2.977	2.496	2.445
FF13-1	0.032	0.038	0.056	6.461	6.536	9.314	2.964	3.489	7.989	6.661	7.124	3.526
FF14-1	0.011	0.018	0.031	3.986	3.722	8.486	1.853	1.914	4.317	2.378	2.177	3.571
FF14-2	0.051	0.048	0.064	5.269	5.033	8.308	2.333	2.189	5.039	2.034	2.238	2.386
FF15-2	0.083	0.090	0.083	4.972	5.289	10.197	2.878	3.097	6.725	5.912	6.277	4.242
FF19-1	0.038	0.038	0.030	3.867	3.714	6.656	1.347	1.394	3.317	1.223	1.256	1.431
FF21-2	0.007	0.006	0.020	3.678	3.522	11.350	2.053	2.275	6.406	3.818	4.330	6.385
FF22-1	0.016	0.019	0.032	3.481	3.867	6.497	1.997	1.953	5.072	4.192	3.676	3.827
FF22-2	0.015	0.024	0.081	4.822	5.336	9.750	2.200	2.294	6.583	3.689	4.199	3.685
NF02-2	0.063	0.072	0.070	2.514	2.625	4.833	1.600	1.519	3.789	1.602	1.801	1.766
NF05-1	0.064	0.073	0.075	5.403	5.978	7.731	2.183	2.208	4.228	2.513	3.133	3.052
NF05-2	0.060	0.059	0.073	5.358	5.572	8.719	2.378	2.006	4.539	2.478	3.368	3.843
NF16-1	0.101	0.109	0.129	5.675	6.678	9.344	2.147	2.183	4.642	2.559	2.521	2.513
NF16-2	0.010	0.022	0.045	3.161	4.139	8.300	1.978	2.122	4.325	2.197	1.908	3.073
NF17-2	0.032	0.040	0.046	4.464	4.544	7.781	1.933	2.150	5.864	2.717	2.681	2.945
NF21-2	0.022	0.040	0.066	3.892	5.475	10.903	2.125	2.719	7.225	2.606	3.599	5.497
NF22-1	0.034	0.050	0.072	4.614	4.842	8.283	2.114	2.153	4.675	2.002	1.987	3.156
NF25-2	0.098	0.102	0.106	5.608	5.961	7.964	1.731	1.683	5.114	2.159	2.426	1.251
NF27-2	0.029	0.042	0.047	3.761	4.114	6.856	1.839	2.267	5.192	2.828	2.799	2.549
NF28-1	0.102	0.102	0.101	4.375	4.392	7.244	1.600	1.669	3.592	2.825	2.714	1.325

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)
FF01-1	2.17	2.192	3.731	3.20%	5.30%	5.60%	0.866	9.56	29.76	24.42	142.33	219.52	129.93
FF13-1	5.788	5.723	5.412	0.53%	0.80%	1.00%	1.034	91.25	107.70	89.39	37.26	54.26	34.28
FF14-1	3.715	4.343	4.113	0.57%	1.40%	1.20%	1.095	4.94	17.64	20.27	11.01	23.20	22.63
FF14-2	3.681	4.338	4.063	0.20%	0.16%	0.24%	0.734	57.85	51.61	32.42	137.73	136.80	93.68
FF15-2	5.531	4.68	4.66	4.40%	6.60%	6.80%	0.782	190.27	227.27	119.84	19.58	24.60	12.40
FF19-1	1.351	1.438	1.124	0.86%	1.30%	1.40%	0.542	66.22	76.66	34.59	60.58	64.55	25.02
FF21-2	5.059	5.312	5.649	0.35%	0.56%	0.53%	0.855	9.38	9.68	21.91	19.36	20.49	20.81
FF22-1	3.45	3.177	3.939	0.52%	0.68%	0.48%	0.935	40.57	27.77	9.54	30.53	36.84	46.52
FF22-2	3.996	3.435	5.414	0.38%	0.51%	0.90%	1.096	2.02	18.80	38.11	47.29	63.12	76.90
NF02-2	2.792	3.187	2.355	2.60%	4.40%	4.80%	0.615	120.74	153.76	76.34	29.22	33.27	19.42
NF05-1	3.53	3.435	3.261	3.20%	5.20%	5.50%	0.786	11.64	14.25	7.08	160.35	191.42	99.34
NF05-2	3.875	4.26	2.891	1.60%	2.50%	2.60%	0.689	40.82	29.67	27.53	146.52	155.62	80.30
NF16-1	3.607	3.808	4.686	5.70%	8.50%	8.40%	1.046	12.24	18.98	22.33	246.84	245.31	141.20
NF16-2	3.376	3.12	3.273	0.18%	0.36%	0.04%	0.972	5.33	21.13	28.35	24.52	65.60	52.71
NF17-2	3.734	3.902	3.515	0.31%	0.06%	0.89%	1.137	76.44	104.53	87.09	45.72	64.02	40.01
NF21-2	4.373	4.682	5.435	0.02%	1.40%	2.00%	1.109	25.51	82.36	107.12	29.65	74.01	54.86
NF22-1	4.261	3.385	3.245	1.20%	2.20%	2.40%	0.644	12.88	32.44	23.58	27.34	54.22	49.29
NF25-2	1.597	1.679	2.377	5.10%	8.00%	8.10%	0.593	12.66	14.32	6.93	196.53	228.19	112.79
NF27-2	1.718	1.949	2.766	0.94%	1.50%	1.70%	0.829	62.25	86.57	63.99	41.57	67.61	40.31
NF28-1	1.838	1.935	2.425	4.90%	7.50%	7.70%	0.6	233.16	237.82	116.00	53.19	47.54	29.71

Table B-5: Collected Result for 3 Story Braced Frame-- Chevron--Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor BRB Normalized Axial Deformation(EXT)	2nd Floor BRB Normalized Axial Deformation(EXT)	3rd Floor BRB Normalized Axial Deformation(EXT)
FF01-1	4.10%	2.80%	1.80%	0.39	0.18	0.05	0.41	0.20	0.06	26.326	20.115	14.605
FF13-1	1.70%	1.30%	1.80%	0.33	0.17	0.05	0.43	0.21	0.07	13.542	10.320	14.467
FF14-1	1.90%	1.60%	1.40%	0.32	0.15	0.04	0.39	0.20	0.08	13.126	11.255	12.471
FF14-2	2.20%	1.60%	1.40%	0.31	0.14	0.03	0.37	0.19	0.07	16.284	11.715	10.943
FF15-2	0.72%	0.37%	0.42%	0.31	0.16	0.04	0.40	0.21	0.07	15.195	10.730	8.652
FF19-1	1.60%	1.10%	0.64%	0.29	0.13	0.03	0.34	0.17	0.05	13.268	10.400	7.948
FF21-2	0.87%	0.78%	0.45%	0.29	0.14	0.03	0.32	0.17	0.05	7.016	7.125	5.786
FF22-1	1.30%	1.10%	1.10%	0.31	0.15	0.04	0.36	0.19	0.06	12.089	10.160	10.795
FF22-2	1.90%	1.90%	1.90%	0.35	0.17	0.04	0.41	0.21	0.07	12.242	14.490	13.914
NF02-2	0.53%	0.20%	0.17%	0.31	0.14	0.03	0.38	0.18	0.05	21.711	15.300	10.286
NF05-1	5.40%	4.10%	3.00%	0.37	0.16	0.03	0.35	0.19	0.06	34.332	25.010	20.262
NF05-2	2.30%	1.60%	0.96%	0.31	0.14	0.04	0.39	0.20	0.06	15.326	11.275	8.171
NF16-1	3.90%	4.20%	4.00%	0.40	0.18	0.05	0.37	0.19	0.06	24.874	26.720	26.114
NF16-2	2.80%	2.20%	2.50%	0.36	0.17	0.05	0.45	0.25	0.09	19.826	16.125	17.305
NF17-2	3.10%	2.80%	2.60%	0.36	0.16	0.03	0.43	0.21	0.07	20.263	18.380	17.852
NF21-2	2.90%	2.30%	2.00%	0.35	0.17	0.05	0.45	0.21	0.07	18.158	15.275	17.024
NF22-1	3.00%	1.80%	0.91%	0.28	0.14	0.04	0.37	0.19	0.06	18.689	12.030	6.986
NF25-2	0.63%	0.93%	0.98%	0.29	0.13	0.02	0.35	0.17	0.05	7.447	8.820	9.224
NF27-2	4.20%	3.60%	2.90%	0.37	0.16	0.03	0.38	0.19	0.06	28.847	23.510	18.967
NF28-1	2.50%	1.50%	0.73%	0.27	0.13	0.03	0.38	0.18	0.05	16.311	10.090	5.519

Ground Motion	1st Floor BRB Normalized Axial Deformation(INT)	2nd Floor BRB Normalized Axial Deformation(INT)	3rd Floor BRB Normalized Axial Deformation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	21.237	14.405	16.424	4.597	6.186	6.286	6.592	6.906	7.250	3.323	3.623	2.938
FF13-1	21.668	18.015	14.671	5.094	5.028	6.381	6.897	7.406	7.583	5.260	5.482	3.899
FF14-1	7.916	5.185	9.276	3.961	4.311	6.150	6.503	6.822	7.075	5.365	5.104	4.486
FF14-2	9.179	6.420	8.819	3.825	4.394	5.361	6.725	7.064	7.364	3.218	3.695	3.501
FF15-2	23.174	18.490	17.624	4.333	4.603	5.231	6.658	6.986	7.233	4.337	4.700	3.505
FF19-1	7.795	8.800	8.581	4.361	4.811	4.878	6.492	6.747	7.044	3.018	1.674	2.943
FF21-2	3.268	5.015	7.600	2.506	3.528	4.094	6.486	6.722	7.036	2.343	1.889	2.643
FF22-1	8.711	7.420	10.671	4.328	4.333	5.833	6.506	6.772	7.075	4.462	3.287	3.629
FF22-2	15.295	13.250	13.448	4.253	6.067	6.975	6.506	6.781	7.147	5.685	4.470	4.291
NF02-2	24.221	18.745	16.271	2.053	2.169	4.289	6.383	6.614	6.972	2.932	1.775	3.378
NF05-1	30.105	21.215	11.795	3.083	3.431	4.225	6.503	6.789	7.161	2.833	2.891	3.273
NF05-2	11.368	6.885	9.095	2.633	4.333	4.956	6.539	6.842	7.178	2.868	4.777	2.882
NF16-1	20.358	20.080	16.114	3.411	3.658	6.383	6.700	7.058	7.314	4.383	5.542	5.172
NF16-2	12.305	13.140	10.443	3.922	5.633	6.267	6.528	6.836	7.108	4.388	6.427	4.801
NF17-2	14.763	12.145	10.010	4.161	4.467	4.803	6.497	6.750	7.128	3.697	3.349	3.344
NF21-2	17.779	18.840	15.600	3.006	4.872	6.544	6.686	7.025	7.206	4.364	5.338	4.985
NF22-1	15.600	7.190	11.219	2.472	4.147	5.739	6.392	6.572	6.906	2.583	7.489	5.716
NF25-2	12.495	8.290	5.843	3.814	4.436	4.806	6.439	6.647	6.936	2.201	2.184	1.623
NF27-2	20.511	17.060	10.643	4.586	4.289	5.072	6.742	7.133	7.394	4.852	3.393	2.656
NF28-1	12.063	8.845	8.190	2.378	3.733	4.064	6.475	6.700	7.019	2.227	2.033	2.021

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)
FF01-1	0.282	0.475	0.597	0.15%	0.12%	0.21%	1.192	708.82	420.63	269.98	1264.00	589.37	266.58
FF13-1	0.471	0.780	0.928	0.41%	0.65%	0.67%	1.290	628.31	375.65	133.22	462.48	373.66	349.28
FF14-1	0.323	0.500	0.584	0.71%	0.67%	0.37%	1.127	381.05	264.75	181.51	223.48	178.89	147.83
FF14-2	0.278	0.467	0.557	0.54%	0.56%	0.46%	1.019	513.81	342.06	199.43	321.21	185.57	115.89
FF15-2	0.289	0.484	0.471	1.10%	1.40%	1.10%	1.256	496.22	280.60	235.38	177.94	124.66	92.84
FF19-1	0.173	0.291	0.309	0.54%	0.35%	0.02%	0.951	365.31	234.56	123.11	166.37	128.34	116.89
FF21-2	0.203	0.341	0.419	0.20%	0.09%	0.20%	0.928	153.71	146.70	79.08	94.89	87.83	104.73
FF22-1	0.258	0.480	0.509	0.45%	0.37%	0.15%	1.206	360.33	215.92	197.58	173.44	131.93	132.39
FF22-2	0.293	0.457	0.620	0.16%	0.02%	0.07%	1.424	519.94	355.36	189.61	658.84	483.18	328.25
NF02-2	0.147	0.287	0.305	0.40%	0.82%	1.10%	0.884	971.35	456.65	178.61	243.42	218.89	217.40
NF05-1	0.213	0.396	0.549	2.40%	2.90%	2.30%	1.136	792.65	564.62	287.14	625.25	260.16	73.70
NF05-2	0.201	0.313	0.497	0.41%	0.11%	0.20%	0.875	260.96	177.65	79.27	410.40	310.18	161.66
NF16-1	0.277	0.496	0.585	1.40%	1.80%	1.70%	1.296	707.14	563.88	382.90	361.99	323.32	202.91
NF16-2	0.500	0.851	1.044	0.22%	0.37%	0.36%	1.286	580.01	334.89	170.52	612.75	432.13	340.58
NF17-2	0.227	0.390	0.440	0.12%	0.59%	0.98%	1.219	724.98	606.09	321.46	698.39	394.89	152.08
NF21-2	0.417	0.718	0.671	0.18%	0.64%	0.96%	1.150	785.24	473.66	123.18	813.70	634.02	396.22
NF22-1	0.196	0.272	0.390	1.00%	0.60%	0.00%	1.018	371.11	216.55	132.21	373.62	251.74	170.82
NF25-2	0.145	0.255	0.359	0.66%	0.25%	0.10%	0.843	255.34	209.85	135.70	379.37	165.14	66.09
NF27-2	0.396	0.676	0.572	0.64%	1.10%	1.20%	1.028	376.95	286.18	224.68	1016.00	551.42	194.15
NF28-1	0.255	0.416	0.386	0.25%	0.36%	0.40%	0.873	175.51	118.22	51.35	622.48	341.77	124.86

Table B-6: Collected Result for 3 Story Braced Frame-- Chevron --Horizontal+ Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor BRB Normalized Axial Deformation(EXT)	2nd Floor BRB Normalized Axial Deformation(EXT)	3rd Floor BRB Normalized Axial Deformation(EXT)
FF01-1	4.10%	2.80%	1.80%	0.38	0.17	0.05	0.44	0.23	0.09	25.547	20.610	15.357
FF13-1	1.70%	1.30%	1.70%	0.41	0.24	0.12	0.64	0.47	0.24	13.453	11.160	14.429
FF14-1	1.70%	1.50%	1.40%	0.37	0.22	0.10	0.62	0.48	0.26	12.405	10.770	12.205
FF14-2	2.20%	1.60%	1.40%	0.34	0.17	0.05	0.46	0.26	0.13	15.758	11.365	11.800
FF15-2	0.69%	0.41%	0.42%	0.41	0.23	0.07	0.49	0.30	0.14	15.674	11.170	10.271
FF19-1	1.60%	1.10%	0.65%	0.30	0.14	0.03	0.35	0.19	0.06	13.374	10.355	7.948
FF21-2	0.81%	0.77%	0.49%	0.29	0.15	0.05	0.42	0.25	0.11	6.368	6.645	5.714
FF22-1	1.30%	1.10%	1.10%	0.34	0.16	0.05	0.52	0.30	0.14	11.126	9.570	10.152
FF22-2	1.90%	1.90%	1.80%	0.37	0.19	0.06	0.56	0.33	0.16	14.626	11.595	13.619
NF02-2	0.54%	0.21%	0.17%	0.33	0.15	0.05	0.46	0.24	0.09	21.521	14.850	10.462
NF05-1	5.30%	4.10%	3.00%	0.43	0.21	0.06	0.45	0.28	0.13	33.063	25.295	19.595
NF05-2	2.30%	1.60%	1.20%	0.41	0.23	0.10	0.50	0.30	0.14	14.842	11.455	10.610
NF16-1	3.90%	4.20%	4.00%	0.41	0.20	0.07	0.52	0.30	0.14	25.032	26.095	25.929
NF16-2	2.70%	2.20%	2.50%	0.37	0.20	0.08	0.49	0.30	0.12	19.489	15.865	17.081
NF17-2	3.10%	2.80%	2.40%	0.39	0.18	0.06	0.43	0.26	0.12	20.595	18.535	16.957
NF21-2	2.90%	2.30%	2.00%	0.36	0.17	0.05	0.48	0.24	0.09	18.716	15.425	17.229
NF22-1	3.00%	1.80%	0.91%	0.29	0.14	0.04	0.39	0.20	0.08	18.553	12.090	7.110
NF25-2	0.68%	0.92%	0.96%	0.30	0.13	0.04	0.36	0.19	0.07	7.695	8.610	9.095
NF27-2	4.20%	3.60%	2.90%	0.37	0.17	0.04	0.41	0.21	0.08	30.021	22.570	19.038
NF28-1	2.40%	1.50%	0.80%	0.28	0.13	0.03	0.41	0.21	0.07	16.363	10.230	6.281

Ground Motion	1st Floor BRB Normalized Axial Deformation(INT)	2nd Floor BRB Normalized Axial Deformation(INT)	3rd Floor BRB Normalized Axial Deformation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	22.105	14.530	16.319	3.656	5.975	6.689	14.414	14.608	15.358	4.649	4.612	3.497
FF13-1	21.611	18.330	14.962	5.389	5.842	7.164	11.925	11.944	13.414	10.156	11.103	11.729
FF14-1	7.026	4.875	8.943	3.858	4.206	6.131	11.975	12.314	12.733	6.784	7.188	10.188
FF14-2	9.295	6.365	8.824	3.531	4.233	5.603	21.219	21.367	21.808	4.550	4.265	4.596
FF15-2	22.279	18.890	17.776	3.858	4.125	4.964	16.006	16.389	16.644	9.259	9.055	9.700
FF19-1	7.800	8.780	8.657	4.386	4.872	5.006	8.875	9.175	9.947	3.400	2.203	2.665
FF21-2	3.321	4.740	7.257	2.319	3.236	4.103	7.706	7.958	8.397	5.068	4.066	6.460
FF22-1	7.595	7.020	9.595	3.892	4.242	5.419	16.006	16.375	17.469	5.848	5.472	5.690
FF22-2	16.805	10.895	11.700	4.703	4.881	6.500	17.956	18.597	19.900	7.718	5.665	7.698
NF02-2	24.279	19.365	16.176	1.819	2.375	4.058	9.272	9.425	9.972	2.987	3.790	3.591
NF05-1	30.247	21.065	12.038	4.028	3.561	4.469	9.181	9.511	10.339	6.298	6.618	8.106
NF05-2	11.642	6.720	9.343	2.856	4.219	5.658	9.206	9.517	10.333	5.767	8.147	9.580
NF16-1	20.421	19.970	16.043	3.228	4.111	6.442	9.392	9.803	11.011	7.324	5.748	6.974
NF16-2	12.305	12.855	9.971	4.203	5.481	6.561	8.544	8.875	9.706	9.855	5.748	5.997
NF17-2	14.505	11.565	9.457	4.525	4.783	5.422	12.969	12.897	12.261	4.785	3.947	6.482
NF21-2	17.853	19.040	15.686	3.150	4.939	6.533	7.367	7.544	7.781	6.140	4.401	5.157
NF22-1	15.742	7.045	11.219	2.461	4.231	5.811	9.747	9.972	10.389	4.175	8.265	6.405
NF25-2	12.463	8.185	6.133	3.969	4.467	4.700	9.750	9.908	11.036	3.553	2.572	2.888
NF27-2	19.326	17.940	10.819	4.872	4.269	5.364	14.914	14.889	15.625	4.934	4.434	3.819
NF28-1	11.374	9.220	8.548	2.644	3.769	4.456	11.381	11.494	12.042	3.136	2.236	2.360

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)
FF01-1	3.14	3.258	3.354	0.15%	0.16%	0.23%	1.189	706.55	422.89	278.36	1317.00	588.57	248.06
FF13-1	2.886	3.861	4.261	0.38%	0.58%	0.59%	1.272	617.56	377.92	131.57	465.40	342.97	331.34
FF14-1	2.615	2.607	3.733	0.60%	0.58%	0.32%	1.146	378.97	264.88	184.70	228.92	175.83	156.33
FF14-2	5.864	5.875	5.816	0.52%	0.54%	0.45%	0.991	514.35	340.75	203.16	333.88	185.08	108.75
FF15-2	4.817	5.484	5.112	1.10%	1.40%	1.20%	1.352	491.75	277.75	231.81	154.58	122.78	91.91
FF19-1	1.261	1.436	1.454	0.55%	0.35%	0.03%	0.935	371.77	235.58	123.77	165.83	128.03	118.06
FF21-2	1.136	1.788	2.217	0.17%	0.08%	0.17%	0.895	133.39	139.15	81.42	95.62	89.71	94.63
FF22-1	4.352	4.961	5.751	0.43%	0.36%	0.14%	1.426	374.77	224.08	205.42	162.51	117.32	119.23
FF22-2	5.073	5.837	6.595	0.12%	0.07%	0.19%	1.363	572.09	359.38	204.65	646.81	476.19	305.27
NF02-2	1.726	2.112	2.992	0.39%	0.83%	1.20%	0.831	950.09	421.24	175.96	242.04	214.42	217.04
NF05-1	2.005	2.795	3.417	2.40%	2.80%	2.20%	1.081	801.50	545.59	303.78	640.08	261.31	99.26
NF05-2	1.945	2.399	2.897	0.41%	0.11%	0.13%	1.239	252.77	169.87	102.42	409.32	300.08	173.04
NF16-1	1.741	2.935	3.415	1.40%	1.80%	1.60%	1.344	696.10	552.65	361.99	364.99	330.81	201.50
NF16-2	1.301	1.936	2.417	0.22%	0.34%	0.34%	1.295	581.13	338.27	179.46	596.65	426.33	341.09
NF17-2	2.618	2.805	2.607	0.09%	0.51%	0.86%	1.185	700.77	591.93	334.64	697.89	384.85	153.07
NF21-2	0.951	1.44	1.683	0.18%	0.64%	0.97%	1.199	787.78	472.22	124.06	787.50	626.96	389.03
NF22-1	1.51	1.693	1.693	1.00%	0.60%	0.01%	1.027	369.61	218.78	132.31	377.34	250.00	168.75
NF25-2	1.576	1.845	2.235	0.63%	0.26%	0.08%	0.818	245.33	209.92	136.00	378.66	158.91	77.08
NF27-2	3.537	3.529	3.414	0.64%	1.00%	1.20%	0.986	379.61	284.87	224.49	945.36	600.66	197.43
NF28-1	2.199	2.443	2.499	0.31%	0.40%	0.40%	0.916	178.52	117.53	61.50	620.17	350.43	127.29

Table B-7: Collected Result for 3 Story Braced Frame-- Single Diagonal--Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor BRB Nomalized Axial Deformation(EXT)	2nd Floor BRB Nomalized Axial Deformation(EXT)	3rd Floor BRB Nomalized Axial Deformation(EXT)
FF01-1	3.40%	2.30%	1.40%	0.35	0.33	0.04	0.64	0.23	0.17	12.611	10.489	11.458
FF13-1	1.60%	1.30%	2.10%	0.33	0.30	0.05	0.66	0.27	0.20	10.047	10.514	8.403
FF14-1	2.10%	1.60%	1.80%	0.30	0.28	0.05	0.56	0.23	0.17	8.089	5.714	6.639
FF14-2	2.70%	2.10%	1.80%	0.32	0.30	0.05	0.56	0.21	0.16	10.064	7.829	6.344
FF15-2	1.20%	1.10%	0.97%	0.32	0.29	0.05	0.61	0.21	0.17	11.161	10.154	8.697
FF19-1	2.30%	1.60%	1.20%	0.30	0.28	0.03	0.52	0.17	0.14	8.794	6.114	3.875
FF21-2	0.87%	0.70%	0.70%	0.27	0.25	0.03	0.50	0.19	0.14	3.017	2.546	2.683
FF22-1	1.30%	0.73%	1.40%	0.33	0.30	0.05	0.53	0.24	0.18	4.772	4.451	4.892
FF22-2	2.80%	2.40%	2.30%	0.35	0.33	0.06	0.63	0.27	0.19	10.525	9.203	8.244
NF02-2	0.24%	0.21%	0.25%	0.32	0.30	0.03	0.60	0.18	0.14	17.117	13.103	9.400
NF05-1	6.20%	4.90%	3.60%	0.37	0.35	0.03	0.52	0.18	0.14	23.822	19.634	13.183
NF05-2	2.20%	1.70%	0.93%	0.29	0.27	0.03	0.58	0.22	0.16	8.419	6.500	4.103
NF16-1	4.00%	4.50%	5.00%	0.38	0.36	0.04	0.56	0.21	0.16	15.158	17.840	18.808
NF16-2	2.10%	2.00%	2.60%	0.34	0.32	0.04	0.66	0.25	0.19	9.683	8.611	9.383
NF17-2	4.10%	3.50%	3.50%	0.36	0.34	0.04	0.67	0.23	0.17	15.592	13.709	12.953
NF21-2	3.10%	2.40%	2.60%	0.35	0.32	0.04	0.70	0.26	0.20	11.936	11.034	9.628
NF22-1	3.90%	2.30%	1.10%	0.29	0.27	0.03	0.50	0.20	0.15	14.769	8.857	4.500
NF25-2	1.10%	1.10%	1.20%	0.29	0.28	0.03	0.54	0.18	0.13	6.556	4.463	3.908
NF27-2	4.00%	3.80%	3.60%	0.36	0.33	0.04	0.61	0.22	0.17	15.328	15.040	13.350
NF28-1	2.80%	1.90%	1.10%	0.29	0.27	0.03	0.60	0.18	0.14	10.728	7.091	5.553

Ground Motion	1st Floor BRB Normalized Axial Deformation(INT)	2nd Floor BRB Normalized Axial Deformation(INT)	3rd Floor BRB Normalized Axial Deformation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	12.558	10.711	11.414	6.436	6.347	6.639	6.775	7.039	7.606	1.217	2.066	2.259
FF13-1	9.967	10.794	8.461	6.736	6.644	6.650	6.956	7.267	7.772	1.914	3.258	2.685
FF14-1	8.014	5.997	6.786	6.444	6.533	6.636	6.550	6.764	7.311	1.173	2.122	2.395
FF14-2	10.056	7.600	6.219	6.394	6.414	6.606	6.736	7.061	7.600	1.44	2.521	2.733
FF15-2	11.186	10.251	8.775	6.411	6.472	6.619	6.633	6.944	7.500	2.106	3.494	2.27
FF19-1	8.739	5.971	3.853	6.289	6.283	6.569	6.583	6.850	7.297	0.66	1.221	0.832
FF21-2	2.944	2.583	2.828	6.344	6.389	6.525	6.458	6.647	7.186	0.647	1.122	0.959
FF22-1	4.814	4.906	5.014	6.481	6.403	6.658	6.625	6.861	7.328	1.441	2.333	3.332
FF22-2	10.556	9.354	8.278	6.356	6.389	6.703	6.611	6.794	7.283	1.674	2.889	2.95
NF02-2	17.086	13.220	9.442	6.306	6.231	6.531	6.436	6.603	7.083	0.636	1.079	1.119
NF05-1	23.719	19.620	13.164	6.344	6.331	6.550	6.600	6.783	7.283	0.837	1.445	1.506
NF05-2	8.331	6.497	4.339	6.308	6.317	6.572	6.517	6.803	7.342	0.887	1.537	1.227
NF16-1	15.075	17.540	18.947	6.411	6.375	6.633	6.800	7.100	7.558	1.366	2.433	2.614
NF16-2	9.631	8.849	9.506	6.442	6.447	6.650	6.736	6.967	7.433	1.4	2.396	2.604
NF17-2	15.542	13.534	13.111	6.506	6.364	6.597	6.708	6.803	7.214	1.513	2.528	2.546
NF21-2	11.722	11.620	9.581	6.456	6.375	6.606	7.014	7.214	7.522	1.388	2.29	2.608
NF22-1	14.764	8.860	4.744	6.364	6.292	6.594	6.594	6.742	7.194	0.609	1.007	0.933
NF25-2	6.561	4.474	3.986	6.275	6.256	6.514	6.450	6.639	7.103	0.527	0.907	0.821
NF27-2	15.356	14.823	13.325	6.392	6.361	6.622	6.753	7.086	7.653	0.987	1.867	1.9
NF28-1	10.669	7.074	5.619	6.356	6.300	6.603	6.717	6.900	7.364	0.478	0.828	1.043

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)
FF01-1	0.379	0.662	1.051	0.41%	0.80%	1.10%	0.947	1560.00	1150.00	572.21	1946.00	942.03	556.70
FF13-1	0.677	1.037	1.517	0.80%	0.98%	0.99%	1.659	876.99	696.89	355.94	1136.00	810.43	651.46
FF14-1	0.399	0.736	0.839	0.75%	0.72%	0.41%	1.18	698.56	277.19	270.70	522.00	390.83	374.85
FF14-2	0.417	0.762	1.269	0.62%	0.89%	0.78%	1.061	803.29	486.22	190.57	1095.00	501.66	242.03
FF15-2	0.315	0.592	0.792	0.93%	1.10%	0.99%	0.773	574.00	1026.00	213.22	1509.00	373.38	562.63
FF19-1	0.26	0.405	0.595	0.85%	0.71%	0.28%	0.729	708.07	308.04	216.48	526.36	440.23	171.84
FF21-2	0.174	0.34	0.548	0.24%	0.24%	0.15%	0.806	363.48	119.38	175.33	192.61	297.47	83.77
FF22-1	0.476	0.781	1.464	0.06%	0.13%	0.06%	0.981	606.65	277.38	236.13	502.70	336.52	326.81
FF22-2	0.487	0.865	1.293	0.55%	0.61%	0.43%	1.343	969.17	731.12	365.03	1107.00	787.02	479.98
NF02-2	0.199	0.402	0.47	1.30%	1.40%	1.30%	0.93	1427.00	597.27	252.77	1040.00	815.67	336.03
NF05-1	0.284	0.369	0.561	2.90%	3.50%	2.70%	1.004	2004.00	678.38	665.03	1353.00	1449.00	236.50
NF05-2	0.329	0.613	0.971	0.22%	0.06%	0.09%	0.78	414.28	570.33	214.34	865.46	310.81	182.31
NF16-1	0.59	0.928	1.543	2.10%	2.80%	2.80%	1.049	1201.00	831.70	686.31	612.43	902.18	710.26
NF16-2	0.433	0.952	1.299	0.48%	0.54%	0.44%	0.97	871.44	1088.00	261.53	1502.00	599.24	895.87
NF17-2	0.492	0.816	0.942	1.30%	1.90%	2.00%	1.031	1718.00	838.06	569.48	1602.00	1123.00	415.53
NF21-2	0.486	0.772	0.971	0.42%	0.61%	0.61%	1.204	1401.00	1071.00	375.90	1623.00	1155.00	705.83
NF22-1	0.274	0.427	0.532	1.90%	1.20%	0.02%	0.758	1373.00	389.94	229.43	527.89	427.82	232.60
NF25-2	0.18	0.347	0.495	0.19%	0.01%	0.18%	0.72	652.49	354.13	234.03	619.52	387.00	193.22
NF27-2	0.422	0.716	0.916	1.00%	1.60%	1.70%	0.889	1450.00	913.12	597.19	1468.00	973.47	427.91
NF28-1	0.301	0.474	0.68	0.30%	0.57%	0.64%	0.727	661.50	880.95	190.77	1576.00	292.64	347.80

Table B-8: Collected Result for 3 Story Braced Frame-- Single Diagonal --Horizontal+ Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	1st Floor BRB Normalized Axial Deformation(EXT)	2nd Floor BRB Normalized Axial Deformation(EXT)	3rd Floor BRB Normalized Axial Deformation(EXT)
FF01-1	3.40%	2.30%	1.50%	0.38	0.36	0.05	0.70	0.25	0.18	12.728	10.406	11.369
FF13-1	1.60%	1.30%	2.10%	0.42	0.36	0.09	1.00	0.64	0.40	10.506	10.643	8.403
FF14-1	2.10%	1.60%	1.80%	0.37	0.33	0.07	0.60	0.35	0.24	8.000	5.740	6.669
FF14-2	2.60%	2.00%	1.70%	0.37	0.34	0.07	0.63	0.32	0.22	9.922	7.700	6.164
FF15-2	1.20%	1.00%	0.93%	0.33	0.30	0.09	0.81	0.39	0.28	11.219	10.217	9.003
FF19-1	2.30%	1.60%	1.20%	0.30	0.28	0.03	0.56	0.20	0.15	8.728	6.029	3.869
FF21-2	0.88%	0.68%	0.69%	0.33	0.29	0.06	0.70	0.47	0.27	3.083	2.754	2.672
FF22-1	1.20%	0.76%	1.40%	0.33	0.29	0.06	0.73	0.35	0.22	4.783	4.409	4.969
FF22-2	2.80%	2.40%	2.20%	0.35	0.32	0.07	0.72	0.32	0.23	10.528	9.089	8.125
NF02-2	0.24%	0.22%	0.25%	0.34	0.31	0.06	0.77	0.32	0.22	16.986	13.074	9.558
NF05-1	6.20%	4.90%	3.70%	0.43	0.39	0.07	0.79	0.43	0.29	23.797	19.240	13.689
NF05-2	2.30%	1.70%	0.93%	0.37	0.33	0.08	0.74	0.38	0.24	8.481	6.329	4.314
NF16-1	4.00%	4.50%	4.90%	0.39	0.36	0.08	0.83	0.49	0.29	15.200	17.869	18.722
NF16-2	2.20%	2.00%	2.60%	0.37	0.34	0.07	0.76	0.46	0.29	9.494	8.503	9.425
NF17-2	4.10%	3.50%	3.50%	0.37	0.34	0.06	0.66	0.32	0.21	15.553	13.649	12.881
NF21-2	3.10%	2.40%	2.60%	0.37	0.35	0.05	0.72	0.28	0.19	11.928	11.026	9.631
NF22-1	3.90%	2.30%	1.10%	0.30	0.28	0.05	0.51	0.19	0.14	14.750	8.851	4.397
NF25-2	1.10%	1.10%	1.20%	0.30	0.29	0.04	0.56	0.20	0.14	6.556	4.469	3.897
NF27-2	4.00%	3.80%	3.60%	0.38	0.35	0.05	0.62	0.24	0.17	15.217	14.931	13.186
NF28-1	2.90%	1.90%	1.10%	0.30	0.28	0.04	0.62	0.22	0.16	10.758	7.100	5.539

Ground Motion	1st Floor BRB Normalized Axial Deformation(INT)	2nd Floor BRB Normalized Axial Deformation(INT)	3rd Floor BRB Normalized Axial Deformation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)
FF01-1	12.644	10.603	11.325	21.856	23.247	31.581	14.442	14.661	15.281	3.550	3.836	3.952
FF13-1	10.542	11.066	8.614	13.758	15.178	20.983	12.050	12.275	13.744	4.830	7.515	6.491
FF14-1	7.956	6.017	6.817	11.319	11.372	13.156	11.981	12.353	12.942	3.962	4.853	5.014
FF14-2	9.986	7.631	6.156	11.611	11.961	13.189	11.844	12.019	12.408	3.370	4.776	4.314
FF15-2	11.211	10.303	8.731	18.778	18.625	22.867	15.906	16.239	16.950	4.898	6.020	5.268
FF19-1	8.667	5.943	3.867	8.072	8.075	9.369	8.939	9.281	10.022	1.145	1.474	1.518
FF21-2	2.997	2.831	2.894	7.614	7.633	8.181	7.625	7.931	8.489	2.683	4.296	3.917
FF22-1	4.775	4.829	5.144	16.608	17.161	21.936	16.022	16.442	17.406	3.591	4.301	3.936
FF22-2	10.514	9.363	8.072	20.103	19.733	25.353	18.019	18.692	19.967	3.635	4.736	4.989
NF02-2	17.014	13.343	9.461	8.750	8.814	9.836	9.261	9.411	10.053	3.202	4.536	4.929
NF05-1	23.750	19.334	13.411	9.008	8.975	9.958	9.289	9.689	10.478	3.924	6.247	6.253
NF05-2	8.419	6.254	4.558	9.011	9.042	9.878	9.297	9.644	10.369	3.684	6.527	5.455
NF16-1	15.164	17.580	18.944	9.561	9.489	9.992	9.486	9.894	11.303	3.699	5.682	5.571
NF16-2	9.444	8.771	9.489	8.506	8.572	8.925	8.511	8.856	9.836	2.962	4.951	5.802
NF17-2	15.531	13.466	13.008	14.428	14.419	15.083	12.914	12.850	12.333	3.086	3.626	3.062
NF21-2	11.686	11.534	9.567	18.439	18.175	19.389	13.078	13.233	13.214	3.764	4.672	3.569
NF22-1	14.725	8.906	4.681	9.158	8.953	8.644	8.844	8.850	9.031	2.658	3.640	3.432
NF25-2	6.544	4.434	3.981	9.233	9.308	10.181	9.767	9.911	11.253	2.013	2.433	2.680
NF27-2	15.167	14.720	13.225	24.831	25.872	38.603	14.811	14.817	15.564	3.390	3.931	3.494
NF28-1	10.711	7.069	5.556	11.808	11.756	12.419	11.367	11.511	12.350	2.841	2.726	2.249

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	Roof Horizontal Acceleration (g)	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)
FF01-1	3.051	3.194	3.317	0.37%	0.75%	1.00%	0.929	1564.00	1145.00	579.04	1931.00	952.12	552.88
FF13-1	3.131	4.339	5.156	0.90%	1.00%	1.00%	1.558	869.01	694.51	344.80	1174.00	805.41	654.36
FF14-1	2.524	3.34	3.498	0.74%	0.72%	0.44%	1.25	695.84	278.88	278.75	526.62	385.49	365.68
FF14-2	2.631	2.671	3.113	0.62%	0.87%	0.74%	1.039	805.32	479.55	194.57	1080.00	497.42	246.61
FF15-2	4.576	5.009	5.167	0.95%	1.20%	1.00%	0.962	568.77	1040.00	207.21	1527.00	366.25	559.64
FF19-1	1.224	1.356	1.607	0.84%	0.69%	0.28%	0.746	701.45	305.21	214.24	523.71	438.60	173.40
FF21-2	1.505	2.565	3.374	0.25%	0.24%	0.13%	0.883	345.01	111.17	171.42	191.79	320.16	92.49
FF22-1	4.285	4.958	5.952	0.06%	0.12%	0.04%	1.04	604.49	279.67	229.27	501.30	335.23	333.67
FF22-2	5.286	6.312	7.12	0.54%	0.59%	0.40%	1.379	963.95	722.33	362.87	1112.00	779.96	472.97
NF02-2	1.912	2.88	3.539	1.30%	1.40%	1.30%	0.921	1404.00	603.79	268.24	1036.00	823.52	343.06
NF05-1	2.341	3.044	4.018	2.90%	3.40%	2.70%	1.047	1998.00	655.86	689.67	1353.00	1408.00	244.78
NF05-2	2.246	2.952	3.551	0.21%	0.02%	0.13%	0.864	413.71	570.71	214.56	892.23	306.88	188.89
NF16-1	2.038	2.926	3.57	2.10%	2.80%	2.80%	1.073	1206.00	837.24	689.78	619.86	897.32	716.69
NF16-2	1.838	2.745	2.922	0.45%	0.51%	0.42%	1.097	873.18	1065.00	261.26	1517.00	601.86	881.29
NF17-2	2.829	3.104	2.95	1.30%	1.90%	2.00%	1.018	1718.00	834.59	574.30	1608.00	1119.00	411.95
NF21-2	2.882	3.204	3.346	0.41%	0.60%	0.59%	1.253	1392.00	1068.00	373.56	1620.00	1145.00	700.80
NF22-1	1.3	1.38	1.613	1.90%	1.20%	0.00%	0.862	1347.00	384.54	227.60	540.86	431.77	228.03
NF25-2	1.52	1.821	2.55	0.19%	0.01%	0.18%	0.757	651.97	353.62	235.37	621.03	384.41	192.80
NF27-2	3.661	3.845	3.871	1.00%	1.60%	1.70%	0.9	1448.00	903.93	591.03	1438.00	972.64	425.71
NF28-1	2.28	2.712	2.625	0.29%	0.57%	0.62%	0.71	658.92	884.70	192.90	1577.00	295.95	347.99

Table B-9: Collected Result for 6 story MF as Beam for Each Earthquake--Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Nomalized Column Axial Force(EXT)	2nd Floor Max Nomalized Column Axial Force(EXT)	3rd Floor Max Nomalized Column Axial Force(EXT)	4th Floor Max Nomalized Column Axial Force(EXT)	5th Floor Max Nomalized Column Axial Force(EXT)	6th Floor Max Nomalized Column Axial Force(EXT)
FF01-1	4.60%	4.00%	3.30%	4.80%	7.70%	10.10%	0.11	0.08	0.06	0.06	0.04	0.02
FF13-1	2.00%	1.80%	2.20%	2.70%	2.90%	2.90%	0.11	0.09	0.06	0.06	0.04	0.02
FF14-1	1.80%	1.50%	1.30%	1.60%	2.00%	3.50%	0.11	0.08	0.06	0.06	0.04	0.02
FF14-2	0.87%	1.00%	1.20%	1.70%	3.20%	3.70%	0.12	0.09	0.07	0.06	0.04	0.02
FF15-2	7.20%	8.40%	9.30%	9.90%	10.50%	11.10%	0.11	0.09	0.06	0.06	0.04	0.02
FF19-1	5.10%	5.70%	5.60%	5.80%	5.60%	5.50%	0.12	0.09	0.06	0.06	0.04	0.01
FF21-2	1.80%	2.30%	3.10%	3.90%	4.20%	4.50%	0.11	0.08	0.06	0.06	0.04	0.02
FF22-1	1.80%	2.10%	2.10%	2.20%	2.90%	3.50%	0.09	0.08	0.06	0.06	0.04	0.02
FF22-2	1.90%	1.90%	2.10%	2.90%	1.90%	4.10%	0.09	0.07	0.06	0.06	0.04	0.02
NF02-2	0.84%	0.87%	0.81%	0.96%	1.20%	1.40%	0.13	0.09	0.07	0.06	0.04	0.02
NF05-1	7.60%	8.60%	9.30%	10.10%	10.70%	11.00%	0.12	0.09	0.06	0.06	0.04	0.02
NF05-2	7.00%	7.20%	6.00%	4.80%	4.20%	4.20%	0.12	0.09	0.06	0.06	0.04	0.01
NF16-1	1.40%	1.90%	2.80%	2.50%	3.50%	5.30%	0.12	0.09	0.06	0.06	0.04	0.02
NF16-2	1.30%	1.70%	2.40%	2.40%	3.40%	4.30%	0.10	0.08	0.06	0.06	0.04	0.02
NF17-2	6.00%	6.10%	5.90%	7.10%	9.10%	10.10%	0.11	0.09	0.06	0.06	0.04	0.02
NF21-2	1.50%	1.90%	2.20%	2.60%	3.10%	3.00%	0.11	0.08	0.06	0.06	0.04	0.01
NF22-1	4.30%	4.10%	4.00%	4.20%	4.30%	4.80%	0.10	0.08	0.06	0.06	0.03	0.02
NF25-2	2.90%	2.70%	3.40%	4.20%	4.60%	4.40%	0.12	0.09	0.06	0.06	0.04	0.01
NF27-2	1.10%	0.96%	1.10%	1.90%	2.90%	3.00%	0.11	0.09	0.06	0.06	0.04	0.01
NF28-1	7.50%	8.40%	8.60%	9.00%	9.90%	10.70%	0.12	0.09	0.06	0.06	0.04	0.01

Ground Motion	1st Floor Max Nomalized Column Axial Force(INT)	2nd Floor Max Nomalized Column Axial Force(INT)	3rd Floor Max Nomalized Column Axial Force(INT)	4th Floor Max Nomalized Column Axial Force(INT)	5th Floor Max Nomalized Column Axial Force(INT)	6th Floor Max Nomalized Column Axial Force(INT)	1st Floor Moment frame Rotation(EXT)	2nd Floor Moment frame Rotation(EXT)	3rd Floor Moment frame Rotation(EXT)	4th Floor Moment frame Rotation(EXT)	5th Floor Moment frame Rotation(EXT)	6th Floor Moment frame Rotation(EXT)
FF01-1	0.14	0.12	0.09	0.10	0.07	0.03	0.030	0.013	0.016	0.059	0.075	0.089
FF13-1	0.15	0.12	0.09	0.10	0.07	0.03	0.000	0.001	0.004	0.031	0.086	0.099
FF14-1	0.14	0.11	0.08	0.10	0.06	0.03	0.002	0.000	0.000	0.011	0.020	0.020
FF14-2	0.14	0.12	0.09	0.10	0.06	0.03	0.075	0.080	0.088	0.092	0.123	0.134
FF15-2	0.15	0.12	0.09	0.10	0.07	0.03	0.076	0.076	0.088	0.095	0.102	0.098
FF19-1	0.15	0.12	0.09	0.10	0.06	0.03	0.042	0.037	0.037	0.039	0.037	0.029
FF21-2	0.14	0.11	0.09	0.10	0.06	0.03	0.002	0.007	0.015	0.022	0.024	0.029
FF22-1	0.14	0.11	0.09	0.10	0.06	0.03	0.002	0.002	0.003	0.006	0.015	0.032
FF22-2	0.14	0.11	0.08	0.10	0.07	0.03	0.004	0.009	0.009	0.013	0.025	0.032
NF02-2	0.14	0.11	0.08	0.10	0.06	0.03	0.067	0.075	0.087	0.096	0.105	0.107
NF05-1	0.14	0.12	0.09	0.10	0.06	0.03	0.079	0.079	0.088	0.097	0.099	0.096
NF05-2	0.15	0.12	0.09	0.10	0.06	0.03	0.070	0.053	0.032	0.026	0.021	0.014
NF16-1	0.14	0.11	0.09	0.10	0.07	0.03	0.023	0.012	0.015	0.022	0.026	0.031
NF16-2	0.14	0.11	0.09	0.10	0.07	0.03	0.007	0.005	0.005	0.013	0.023	0.027
NF17-2	0.14	0.12	0.09	0.10	0.06	0.03	0.055	0.039	0.047	0.070	0.089	0.087
NF21-2	0.14	0.11	0.09	0.10	0.07	0.03	0.000	0.000	0.003	0.008	0.011	0.017
NF22-1	0.14	0.11	0.09	0.10	0.06	0.03	0.023	0.020	0.019	0.023	0.019	0.025
NF25-2	0.15	0.12	0.09	0.10	0.07	0.03	0.044	0.030	0.023	0.029	0.028	0.027
NF27-2	0.14	0.11	0.09	0.10	0.07	0.03	0.082	0.080	0.083	0.099	0.100	0.103
NF28-1	0.15	0.12	0.09	0.10	0.06	0.03	0.078	0.080	0.082	0.089	0.097	0.095

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	4th Floor Moment Frame Rotation(INT)	5th Floor Moment Frame Rotation(INT)	6th Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)
FF01-1	0.038	0.020	0.025	0.070	0.089	0.117	3.586	3.000	3.339	4.500	5.244	7.392
FF13-1	0.006	0.008	0.012	0.033	0.088	0.093	2.214	2.417	2.769	3.381	3.414	5.389
FF14-1	0.004	0.002	0.005	0.012	0.020	0.035	1.944	1.747	2.228	2.856	3.431	5.997
FF14-2	0.082	0.087	0.096	0.097	0.128	0.131	2.978	3.131	3.181	3.425	3.706	5.950
FF15-2	0.087	0.087	0.100	0.109	0.118	0.121	4.464	4.539	5.033	5.336	5.650	6.689
FF19-1	0.055	0.048	0.050	0.055	0.055	0.054	3.914	3.831	4.006	4.403	4.703	5.869
FF21-2	0.009	0.014	0.024	0.033	0.037	0.038	2.364	2.606	3.150	3.764	4.189	5.481
FF22-1	0.009	0.008	0.010	0.016	0.023	0.030	2.383	2.381	2.656	3.258	3.714	5.883
FF22-2	0.008	0.012	0.015	0.014	0.025	0.042	2.342	2.217	2.842	3.178	3.489	6.014
NF02-2	0.073	0.082	0.094	0.101	0.108	0.102	2.778	3.067	3.231	2.947	2.864	3.578
NF05-1	0.091	0.090	0.101	0.111	0.115	0.120	4.506	4.578	4.978	5.325	5.542	6.592
NF05-2	0.081	0.064	0.041	0.037	0.034	0.036	4.200	4.086	3.878	3.989	4.058	5.406
NF16-1	0.027	0.016	0.019	0.023	0.034	0.062	2.169	2.644	2.931	3.253	4.331	6.297
NF16-2	0.009	0.008	0.012	0.019	0.029	0.044	1.994	2.408	2.678	3.286	3.950	5.842
NF17-2	0.066	0.051	0.060	0.082	0.104	0.111	4.017	3.975	4.258	4.539	5.289	6.572
NF21-2	0.005	0.007	0.010	0.018	0.022	0.022	2.103	2.328	2.664	3.269	3.714	4.983
NF22-1	0.032	0.028	0.028	0.034	0.031	0.051	3.242	3.169	3.369	3.847	4.028	6.072
NF25-2	0.051	0.035	0.027	0.037	0.037	0.037	2.889	2.753	3.211	3.906	4.122	5.586
NF27-2	0.090	0.087	0.090	0.105	0.103	0.097	3.258	3.183	3.108	3.175	3.575	5.375
NF28-1	0.089	0.091	0.093	0.102	0.112	0.118	4.683	4.803	4.919	5.294	5.661	6.767

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	1.014	1.033	1.164	1.558	1.900	3.756	1.054	1.081	0.936	0.724	1.389	1.321
FF13-1	0.622	0.617	0.694	1.139	1.531	3.053	0.467	0.919	0.729	0.899	1.092	1.342
FF14-1	0.564	0.533	0.669	0.972	1.269	2.953	0.804	0.601	0.814	0.699	0.975	1.249
FF14-2	0.822	0.825	0.967	1.397	1.692	3.303	1.025	0.685	0.857	0.835	1.243	1.247
FF15-2	0.842	0.825	1.028	1.342	1.708	3.494	0.688	0.547	0.646	0.673	0.777	1.201
FF19-1	1.072	1.147	1.331	1.681	1.997	3.294	0.579	0.333	0.383	0.391	0.602	0.733
FF21-2	0.550	0.556	0.675	1.031	1.186	2.642	0.357	0.326	0.418	0.435	0.837	0.866
FF22-1	0.567	0.631	0.733	1.006	1.289	2.617	0.706	0.681	0.874	0.726	0.755	1.972
FF22-2	0.644	0.658	0.808	1.103	1.322	2.619	0.845	0.907	0.923	0.826	1.729	1.767
NF02-2	0.711	0.703	0.803	1.183	1.478	2.981	1.077	0.610	0.376	0.487	0.621	0.892
NF05-1	0.786	0.764	0.911	1.367	1.592	3.433	1.088	0.905	0.506	0.661	0.617	0.961
NF05-2	0.881	0.994	1.058	1.353	1.567	2.733	0.717	0.416	0.593	0.777	0.589	0.871
NF16-1	0.756	0.767	0.894	1.247	1.442	3.636	1.126	0.756	1.096	0.908	1.785	1.203
NF16-2	0.589	0.586	0.731	1.039	1.258	2.706	0.538	1.123	0.661	0.969	1.573	1.684
NF17-2	0.817	0.872	1.042	1.439	1.692	3.439	0.700	0.644	0.687	0.648	1.110	1.375
NF21-2	0.511	0.525	0.653	0.944	1.186	2.417	0.169	0.297	0.246	0.423	0.603	0.685
NF22-1	0.642	0.636	0.803	1.089	1.289	3.028	0.544	0.546	0.881	0.982	0.946	1.368
NF25-2	1.047	1.081	1.167	1.461	1.653	2.861	0.487	0.317	0.339	0.645	0.710	0.605
NF27-2	0.772	0.839	1.058	1.389	1.650	3.533	0.938	0.886	0.544	0.800	0.757	1.221
NF28-1	1.000	1.081	1.222	1.686	1.997	3.631	0.582	0.550	0.525	0.455	0.760	0.775

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	1.038	0.732	1.034	1.045	1.056	1.432	1.10%	1.00%	0.84%	1.20%	2.70%	3.30%	0.897
FF13-1	0.735	1.256	0.537	1.036	0.669	1.019	0.06%	0.06%	0.22%	0.59%	4.10%	4.00%	0.868
FF14-1	1.399	0.790	0.670	0.673	0.690	1.499	0.03%	0.32%	0.46%	0.65%	0.78%	0.45%	0.899
FF14-2	1.070	0.914	1.060	0.674	1.057	1.730	6.30%	7.40%	7.90%	8.40%	8.70%	8.90%	0.783
FF15-2	0.571	0.536	0.486	0.563	0.588	0.947	6.90%	8.20%	9.10%	9.70%	10.10%	10.10%	0.726
FF19-1	0.591	0.618	0.511	0.519	0.613	0.607	2.50%	2.70%	2.70%	2.70%	2.60%	2.50%	0.515
FF21-2	0.800	0.566	0.504	0.910	0.509	1.034	0.72%	1.10%	1.50%	1.90%	2.00%	1.90%	0.681
FF22-1	0.906	0.759	0.629	0.603	0.700	0.819	0.16%	0.24%	0.22%	0.16%	0.08%	0.22%	0.902
FF22-2	1.332	1.024	1.031	0.836	0.995	0.852	0.19%	0.17%	0.17%	0.20%	0.01%	0.65%	0.946
NF02-2	0.970	0.857	0.596	0.796	0.872	0.766	5.10%	6.40%	7.20%	7.90%	8.50%	8.80%	0.563
NF05-1	1.483	0.726	0.537	0.823	0.697	1.638	5.90%	6.60%	7.00%	7.30%	7.70%	7.90%	0.649
NF05-2	1.259	0.594	0.532	0.340	0.547	0.438	3.30%	3.80%	3.40%	2.80%	2.20%	1.90%	0.727
NF16-1	1.252	0.726	0.739	1.093	0.920	1.179	0.59%	0.29%	0.05%	0.32%	0.94%	1.30%	0.923
NF16-2	0.879	1.303	1.163	0.989	1.048	1.428	0.50%	0.50%	0.63%	0.95%	1.20%	0.85%	0.88
NF17-2	1.143	0.778	0.647	0.606	0.645	1.040	4.30%	4.30%	4.30%	4.80%	6.00%	6.50%	0.816
NF21-2	0.312	0.370	0.312	0.425	0.525	0.895	0.09%	0.21%	0.36%	0.49%	0.59%	0.62%	0.603
NF22-1	0.957	0.623	0.742	0.420	0.670	1.103	2.40%	2.20%	2.10%	2.00%	2.00%	2.10%	0.834
NF25-2	0.478	0.575	0.425	0.404	1.187	1.246	2.70%	2.80%	2.40%	2.10%	1.80%	1.30%	0.572
NF27-2	0.913	0.659	0.567	0.626	1.071	1.154	6.50%	7.30%	7.60%	8.00%	8.20%	8.00%	0.887
NF28-1	0.742	0.677	0.631	0.692	0.534	0.875	6.40%	7.20%	7.40%	7.80%	8.10%	8.40%	0.575

Ground Motion	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	4th Floor RBS Energy Dissipation(EXT)	5th Floor RBS Energy Dissipation(EXT)	6th Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)	4th Floor RBS Energy Dissipation(INT)	5th Floor RBS Energy Dissipation(INT)	6th Floor RBS Energy Dissipation(INT)
FF01-1	67.15	30.29	50.24	60.05	79.83	56.58	240.38	229.38	180.63	231.56	248.81	134.13
FF13-1	0.01	3.44	24.28	70.41	135.52	71.67	18.77	23.72	25.42	107.46	140.91	83.56
FF14-1	3.88	0.00	0.27	24.65	34.08	36.51	29.20	9.45	13.99	42.63	56.77	24.74
FF14-2	270.51	301.06	310.75	335.90	363.19	189.29	28.34	30.28	31.32	47.14	72.91	41.69
FF15-2	5.17	1.10	2.34	5.43	33.53	18.21	254.78	238.05	252.14	215.21	212.34	120.72
FF19-1	129.64	89.36	73.40	62.37	46.93	17.18	508.41	451.19	390.04	305.28	212.29	85.02
FF21-2	3.83	9.43	34.48	52.54	61.30	20.89	24.12	30.39	36.42	28.52	38.02	30.77
FF22-1	7.84	9.59	12.21	43.90	27.36	21.79	14.97	34.76	29.41	30.58	51.42	44.41
FF22-2	2.85	11.27	25.83	8.84	18.11	22.85	50.72	87.09	74.42	73.35	89.21	73.08
NF02-2	296.85	336.76	341.56	262.92	241.56	114.56	32.39	37.13	37.71	29.56	27.44	13.01
NF05-1	37.94	37.72	35.50	30.55	23.90	16.96	672.44	662.35	655.68	540.77	423.92	184.81
NF05-2	175.02	78.80	43.54	31.34	33.54	20.48	384.43	342.15	144.56	104.65	56.81	20.10
NF16-1	29.52	25.68	30.70	8.50	28.11	23.78	145.57	127.28	124.05	119.49	120.60	81.61
NF16-2	24.02	35.27	20.71	66.07	80.13	47.38	65.28	40.23	26.36	34.70	61.39	45.26
NF17-2	36.47	27.64	24.88	34.05	31.05	17.68	671.14	377.54	437.88	411.43	432.38	229.10
NF21-2	0.01	0.08	3.24	9.23	15.92	6.67	25.04	23.55	29.72	31.30	51.95	31.11
NF22-1	20.96	25.89	41.36	35.55	23.75	13.38	56.74	50.24	44.56	37.68	24.61	29.90
NF25-2	62.36	32.21	39.99	39.06	25.12	8.49	284.92	225.67	155.95	152.73	126.12	63.76
NF27-2	384.75	345.55	269.28	351.90	303.77	142.42	44.72	41.99	36.88	37.98	38.02	20.75
NF28-1	41.17	35.67	29.08	28.53	32.38	17.34	626.22	566.17	461.13	401.16	392.91	198.62

Table B-10: Collected Result for 6 story MF as Beam for Each Earthquake--Horizontal + Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	4th Floor Max Normalized Column Axial Force(EXT)	5th Floor Max Normalized Column Axial Force(EXT)	6th Floor Max Normalized Column Axial Force(EXT)
FF01-1	4.80%	4.10%	3.40%	4.90%	7.70%	10.10%	0.12	0.10	0.08	0.09	0.06	0.03
FF13-1	1.70%	1.70%	2.10%	2.80%	3.30%	4.40%	0.20	0.18	0.15	0.19	0.11	0.05
FF14-1	1.60%	1.40%	1.20%	1.50%	1.90%	3.20%	0.12	0.10	0.08	0.10	0.07	0.03
FF14-2	1.60%	1.00%	1.10%	1.80%	3.00%	3.00%	0.17	0.15	0.12	0.14	0.09	0.05
FF15-2	4.70%	5.70%	7.30%	9.10%	11.00%	11.70%	0.17	0.13	0.10	0.11	0.07	0.04
FF19-1	5.10%	5.60%	5.40%	5.50%	5.30%	4.90%	0.14	0.10	0.07	0.07	0.04	0.02
FF21-2	1.80%	2.30%	3.10%	3.90%	4.30%	4.50%	0.16	0.13	0.11	0.12	0.08	0.04
FF22-1	1.80%	2.10%	2.10%	2.20%	3.00%	3.70%	0.14	0.13	0.10	0.10	0.07	0.03
FF22-2	2.00%	1.90%	2.10%	2.90%	2.10%	4.00%	0.14	0.14	0.12	0.12	0.08	0.03
NF02-2	0.85%	0.89%	0.82%	0.96%	1.20%	1.50%	0.15	0.12	0.09	0.10	0.07	0.03
NF05-1	6.10%	7.20%	8.10%	9.10%	9.80%	10.20%	0.15	0.12	0.09	0.09	0.06	0.03
NF05-2	6.60%	6.70%	6.30%	6.40%	6.20%	6.10%	0.14	0.11	0.09	0.10	0.07	0.04
NF16-1	1.80%	2.00%	2.80%	2.50%	3.30%	5.00%	0.15	0.12	0.09	0.12	0.09	0.04
NF16-2	1.30%	1.70%	2.50%	2.20%	3.50%	4.50%	0.13	0.12	0.10	0.12	0.08	0.03
NF17-2	5.50%	5.80%	5.20%	6.00%	8.00%	9.10%	0.14	0.11	0.09	0.09	0.06	0.03
NF21-2	1.50%	1.90%	2.20%	2.60%	3.00%	3.00%	0.11	0.10	0.08	0.08	0.05	0.03
NF22-1	4.20%	4.10%	4.10%	4.40%	4.60%	5.40%	0.12	0.11	0.08	0.09	0.05	0.03
NF25-2	3.40%	3.10%	3.30%	4.20%	4.60%	4.60%	0.13	0.10	0.07	0.07	0.05	0.02
NF27-2	1.10%	0.93%	1.10%	1.90%	2.70%	3.00%	0.14	0.11	0.07	0.07	0.05	0.02
NF28-1	7.10%	8.00%	8.00%	8.50%	9.50%	10.30%	0.12	0.09	0.07	0.07	0.05	0.03

Ground Motion	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	4th Floor Max Normalized Column Axial Force(INT)	5th Floor Max Normalized Column Axial Force(INT)	6th Floor Max Normalized Column Axial Force(INT)	1st Floor Moment Frame Rotation(EXT)	2nd Floor Moment Frame Rotation(EXT)	3rd Floor Moment Frame Rotation(EXT)	4th Floor Moment Frame Rotation(EXT)	5th Floor Moment Frame Rotation(EXT)	6th Floor Moment Frame Rotation(EXT)
FF01-1	0.22	0.19	0.15	0.17	0.11	0.06	0.031	0.013	0.017	0.058	0.076	0.090
FF13-1	0.36	0.34	0.31	0.36	0.23	0.11	0.002	0.000	0.005	0.034	0.088	0.095
FF14-1	0.28	0.24	0.20	0.24	0.17	0.09	0.002	0.000	0.000	0.012	0.020	0.022
FF14-2	0.34	0.30	0.25	0.33	0.23	0.11	0.046	0.038	0.068	0.086	0.110	0.124
FF15-2	0.31	0.27	0.22	0.29	0.19	0.10	0.040	0.041	0.072	0.099	0.111	0.107
FF19-1	0.17	0.14	0.12	0.16	0.11	0.06	0.041	0.035	0.034	0.037	0.034	0.034
FF21-2	0.37	0.31	0.25	0.30	0.20	0.10	0.002	0.007	0.015	0.022	0.027	0.029
FF22-1	0.31	0.27	0.22	0.27	0.19	0.09	0.003	0.002	0.004	0.010	0.021	0.032
FF22-2	0.32	0.29	0.24	0.31	0.21	0.10	0.004	0.009	0.010	0.019	0.030	0.038
NF02-2	0.27	0.24	0.21	0.26	0.17	0.08	0.069	0.076	0.088	0.097	0.106	0.106
NF05-1	0.33	0.29	0.24	0.30	0.19	0.08	0.063	0.065	0.076	0.087	0.092	0.089
NF05-2	0.35	0.30	0.26	0.36	0.24	0.09	0.066	0.055	0.049	0.046	0.043	0.040
NF16-1	0.28	0.25	0.21	0.25	0.19	0.08	0.022	0.013	0.016	0.025	0.032	0.048
NF16-2	0.26	0.23	0.19	0.24	0.16	0.07	0.007	0.005	0.006	0.013	0.024	0.027
NF17-2	0.29	0.26	0.22	0.28	0.21	0.10	0.049	0.035	0.036	0.057	0.080	0.076
NF21-2	0.30	0.27	0.22	0.27	0.19	0.10	0.000	0.000	0.003	0.007	0.010	0.018
NF22-1	0.27	0.24	0.20	0.26	0.18	0.08	0.024	0.021	0.021	0.026	0.025	0.029
NF25-2	0.22	0.19	0.16	0.20	0.13	0.07	0.037	0.028	0.021	0.028	0.030	0.029
NF27-2	0.20	0.17	0.13	0.15	0.10	0.05	0.076	0.075	0.077	0.088	0.089	0.098
NF28-1	0.23	0.20	0.18	0.23	0.18	0.09	0.074	0.075	0.075	0.084	0.093	0.091

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	4th Floor Moment Frame Rotation(INT)	5th Floor Moment Frame Rotation(INT)	6th Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)
FF01-1	0.040	0.020	0.026	0.070	0.089	0.116	3.675	3.161	3.647	4.772	5.733	7.106
FF13-1	0.005	0.007	0.013	0.033	0.088	0.090	2.383	2.700	3.192	5.525	5.342	8.844
FF14-1	0.004	0.002	0.005	0.012	0.019	0.033	1.900	1.878	2.353	3.328	3.694	5.944
FF14-2	0.053	0.046	0.075	0.091	0.115	0.121	3.006	2.800	3.361	4.319	5.003	6.400
FF15-2	0.052	0.054	0.083	0.111	0.125	0.135	3.939	4.022	4.644	6.281	6.742	9.025
FF19-1	0.054	0.046	0.046	0.052	0.048	0.041	3.981	3.797	4.006	4.553	4.811	5.808
FF21-2	0.008	0.014	0.023	0.033	0.038	0.040	2.461	2.617	3.150	3.908	4.756	6.011
FF22-1	0.009	0.008	0.008	0.016	0.028	0.044	2.469	2.650	2.692	3.944	5.211	7.583
FF22-2	0.007	0.012	0.015	0.017	0.025	0.039	2.289	2.378	3.283	4.253	4.336	6.517
NF02-2	0.076	0.083	0.095	0.102	0.109	0.100	2.869	3.136	3.339	3.058	3.181	3.789
NF05-1	0.073	0.075	0.087	0.100	0.106	0.113	4.139	4.094	4.558	5.644	6.197	7.728
NF05-2	0.076	0.066	0.062	0.061	0.060	0.068	4.497	4.450	4.619	5.664	5.594	6.439
NF16-1	0.025	0.016	0.018	0.026	0.036	0.054	2.583	3.111	3.272	4.317	4.989	6.817
NF16-2	0.010	0.009	0.014	0.021	0.033	0.048	2.236	2.514	3.147	4.058	4.986	6.453
NF17-2	0.061	0.045	0.047	0.069	0.094	0.099	3.989	3.836	4.086	4.903	5.036	6.389
NF21-2	0.006	0.007	0.011	0.018	0.021	0.022	2.106	2.364	2.708	3.353	3.789	5.217
NF22-1	0.033	0.029	0.030	0.037	0.037	0.064	3.328	3.339	3.675	4.458	5.014	7.481
NF25-2	0.043	0.032	0.026	0.037	0.038	0.045	3.172	3.039	3.475	4.239	4.536	5.911
NF27-2	0.083	0.082	0.083	0.093	0.092	0.090	3.069	3.056	2.936	3.242	4.028	5.814
NF28-1	0.084	0.085	0.086	0.097	0.108	0.114	4.567	4.653	4.769	5.639	6.056	6.883

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	1.022	1.039	1.256	1.819	2.375	4.542	2.453	2.146	2.471	4.475	4.396	5.330
FF13-1	0.922	1.317	1.953	2.850	4.772	6.506	5.848	11.306	10.452	14.694	14.065	10.250
FF14-1	0.664	0.794	1.267	1.956	2.608	4.550	5.035	6.252	7.000	8.927	8.642	6.620
FF14-2	1.208	1.300	1.783	2.606	3.372	6.800	7.962	10.113	11.180	11.668	12.951	8.845
FF15-2	0.894	1.186	1.692	2.531	4.092	8.164	6.277	7.134	7.298	12.441	9.805	7.015
FF19-1	1.117	1.194	1.372	1.767	2.006	3.739	0.786	1.189	1.647	4.493	3.985	2.530
FF21-2	0.706	0.933	1.203	1.983	2.253	6.850	4.911	6.144	5.952	8.491	10.610	9.179
FF22-1	0.719	1.047	1.364	2.292	3.933	4.808	3.389	3.170	4.240	10.936	10.143	5.727
FF22-2	0.872	1.022	1.611	2.531	3.600	5.417	3.490	3.357	5.140	10.422	7.881	6.641
NF02-2	0.717	0.753	1.206	1.947	2.136	4.403	2.636	3.430	4.206	6.130	5.346	3.513
NF05-1	1.069	1.189	1.736	2.800	3.583	5.136	4.946	4.734	6.647	12.061	9.486	4.085
NF05-2	1.044	1.164	1.689	3.044	3.828	5.744	5.323	6.163	7.034	14.149	10.367	4.898
NF16-1	0.894	1.064	1.472	2.569	3.625	5.872	4.127	6.397	6.292	11.875	10.630	6.069
NF16-2	0.778	0.900	1.375	1.864	3.106	4.989	5.362	5.499	6.185	10.802	10.582	5.081
NF17-2	0.844	0.967	1.400	2.544	4.094	5.536	3.822	4.204	6.082	17.042	8.900	4.445
NF21-2	0.567	0.747	1.097	1.406	1.814	6.161	3.308	3.530	4.494	5.208	7.524	6.029
NF22-1	0.786	1.011	1.431	1.842	2.247	5.961	3.744	3.815	4.482	4.450	5.692	7.744
NF25-2	1.014	1.056	1.253	1.672	1.967	4.006	1.820	1.911	2.312	3.312	3.488	3.440
NF27-2	0.831	0.942	1.275	1.747	2.367	4.189	1.537	1.924	1.821	5.473	4.840	3.289
NF28-1	1.047	1.142	1.386	1.994	2.228	4.447	2.287	2.165	4.034	6.460	4.510	3.714

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	2.327	2.627	3.357	4.289	5.539	4.399	1.40%	1.20%	1.00%	1.30%	2.70%	3.30%	1.179
FF13-1	7.072	12.033	12.432	15.445	19.960	12.733	0.37%	0.32%	0.35%	0.62%	3.90%	3.70%	1.545
FF14-1	5.169	5.798	10.331	9.979	11.742	11.473	0.09%	0.38%	0.52%	0.71%	0.87%	0.66%	1.6
FF14-2	7.029	9.218	16.010	14.877	17.719	15.345	3.50%	4.00%	4.50%	5.00%	5.50%	5.80%	1.499
FF15-2	4.602	5.063	8.663	9.147	14.475	13.763	3.10%	3.80%	4.90%	5.70%	6.30%	6.60%	1.159
FF19-1	0.854	1.101	1.451	3.109	2.906	5.902	1.20%	1.40%	1.30%	1.30%	1.10%	0.95%	0.734
FF21-2	4.442	5.009	6.012	6.576	7.771	13.649	0.62%	1.00%	1.40%	1.70%	1.90%	1.90%	1.625
FF22-1	3.596	4.974	4.244	7.953	12.785	10.285	0.21%	0.24%	0.21%	0.17%	0.16%	0.46%	1.39
FF22-2	4.586	3.608	5.571	8.783	12.497	10.925	0.23%	0.20%	0.19%	0.20%	0.06%	0.37%	1.648
NF02-2	3.976	5.087	7.308	10.234	9.902	10.771	5.30%	6.60%	7.30%	8.00%	8.60%	8.80%	0.913
NF05-1	5.382	5.549	10.885	13.423	15.953	10.809	4.80%	5.70%	6.20%	6.60%	7.10%	7.20%	0.84
NF05-2	4.417	4.864	7.595	14.830	16.466	13.596	3.90%	4.70%	4.90%	4.80%	4.60%	4.50%	1.017
NF16-1	4.697	6.016	9.092	10.768	14.960	9.612	0.05%	0.06%	0.25%	0.41%	0.70%	0.89%	1.204
NF16-2	4.618	5.543	6.360	9.999	10.239	9.595	0.54%	0.53%	0.66%	0.97%	1.10%	0.69%	1.236
NF17-2	2.990	3.657	5.553	10.290	15.266	12.252	3.50%	3.30%	3.20%	3.70%	4.80%	5.20%	1.104
NF21-2	3.753	3.941	6.022	5.918	7.229	12.928	0.01%	0.11%	0.27%	0.39%	0.48%	0.50%	1.474
NF22-1	2.832	3.812	5.336	4.932	5.765	9.215	2.60%	2.40%	2.30%	2.30%	2.30%	2.60%	1.025
NF25-2	1.365	1.594	2.882	3.762	4.453	6.544	1.20%	1.60%	1.50%	1.40%	1.20%	0.79%	0.901
NF27-2	1.961	1.998	2.927	3.780	5.381	4.259	5.80%	6.60%	6.70%	7.00%	7.20%	7.10%	0.976
NF28-1	1.475	2.480	2.688	5.204	7.107	9.915	6.00%	6.70%	6.90%	7.30%	7.70%	7.90%	1.115

Ground Motion	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	4th Floor RBS Energy Dissipation(EXT)	5th Floor RBS Energy Dissipation(EXT)	6th Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)	4th Floor RBS Energy Dissipation(INT)	5th Floor RBS Energy Dissipation(INT)	6th Floor RBS Energy Dissipation(INT)
FF01-1	63.08	25.56	46.82	63.29	80.36	56.26	252.84	235.63	189.54	236.55	262.30	142.83
FF13-1	8.15	0.88	25.02	102.94	176.63	102.33	40.57	28.24	27.49	106.93	145.85	85.33
FF14-1	5.93	0.00	0.27	31.51	32.57	36.34	23.74	10.37	13.41	36.72	48.45	21.35
FF14-2	166.05	121.57	350.93	390.58	377.76	189.67	61.34	34.13	42.33	38.50	47.81	34.71
FF15-2	23.61	24.99	31.31	75.26	69.00	36.30	364.36	386.65	579.56	547.73	476.59	253.51
FF19-1	196.88	150.97	121.30	108.03	75.62	32.86	475.77	415.54	338.01	252.28	155.68	59.43
FF21-2	1.74	8.83	31.67	47.53	61.68	20.86	25.35	33.12	42.51	38.52	50.97	34.29
FF22-1	11.56	8.65	12.12	37.85	51.45	34.83	16.97	30.12	33.12	29.53	49.82	51.46
FF22-2	2.37	7.88	23.87	18.56	24.23	27.02	51.58	92.04	84.81	89.72	89.08	68.15
NF02-2	305.46	336.61	339.43	266.01	246.17	111.63	33.46	37.97	38.08	30.10	28.06	13.08
NF05-1	28.44	29.85	29.70	28.69	30.16	17.80	463.24	516.27	543.07	486.44	414.84	186.96
NF05-2	108.20	24.52	19.09	15.26	12.20	18.57	408.94	419.42	325.21	262.39	188.94	84.04
NF16-1	15.32	22.77	36.94	12.59	38.50	33.39	142.40	139.04	120.40	137.51	134.40	94.05
NF16-2	29.11	38.63	30.82	70.16	81.17	49.53	65.44	37.20	30.13	42.05	71.75	50.18
NF17-2	48.50	39.97	22.06	31.96	38.66	20.20	665.42	370.71	357.66	365.10	373.31	198.14
NF21-2	0.01	0.16	3.26	8.73	15.75	9.29	27.12	25.17	32.54	30.26	52.09	30.66
NF22-1	17.89	24.70	42.62	42.85	32.23	15.63	54.61	52.12	43.49	37.92	30.60	43.45
NF25-2	45.86	32.77	33.34	33.39	25.12	10.92	386.18	277.41	197.80	187.72	154.41	72.63
NF27-2	378.84	362.62	300.33	332.66	290.38	139.84	40.23	39.41	42.77	32.68	31.55	20.68
NF28-1	38.56	31.68	26.01	29.05	32.39	17.38	603.66	528.73	430.33	399.63	384.65	191.12

Table B-11: Collected Result for 6 story MF as Girder for Each Earthquake--Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	4th Floor Max Normalized Column Axial Force(EXT)	5th Floor Max Normalized Column Axial Force(EXT)	6th Floor Max Normalized Column Axial Force(EXT)
FF01-1	5.10%	4.40%	3.90%	4.80%	6.90%	8.70%	0.13	0.11	0.08	0.08	0.05	0.02
FF13-1	1.60%	2.00%	2.50%	2.30%	2.40%	3.50%	0.14	0.11	0.08	0.09	0.05	0.02
FF14-1	1.70%	1.40%	1.60%	1.90%	2.30%	3.80%	0.12	0.10	0.07	0.08	0.05	0.02
FF14-2	0.87%	1.00%	1.10%	1.60%	3.30%	3.70%	0.14	0.11	0.08	0.09	0.05	0.02
FF15-2	4.70%	5.30%	5.00%	4.70%	4.50%	4.70%	0.13	0.11	0.08	0.08	0.05	0.02
FF19-1	5.00%	4.80%	4.30%	3.80%	3.50%	3.90%	0.14	0.11	0.08	0.09	0.05	0.02
FF21-2	2.10%	3.20%	3.60%	3.60%	3.20%	2.90%	0.13	0.11	0.08	0.08	0.05	0.02
FF22-1	1.70%	2.10%	2.00%	2.10%	2.30%	2.70%	0.12	0.10	0.08	0.08	0.05	0.03
FF22-2	1.70%	1.90%	2.10%	2.20%	1.80%	4.70%	0.11	0.10	0.08	0.09	0.05	0.02
NF02-2	1.10%	1.30%	0.78%	0.85%	0.98%	1.10%	0.15	0.12	0.09	0.09	0.05	0.02
NF05-1	6.70%	8.30%	8.90%	9.20%	9.30%	9.60%	0.14	0.11	0.08	0.08	0.05	0.02
NF05-2	6.20%	6.20%	5.50%	4.70%	3.90%	4.00%	0.14	0.11	0.08	0.08	0.05	0.02
NF16-1	2.10%	3.10%	2.90%	2.50%	3.40%	4.20%	0.14	0.11	0.08	0.09	0.05	0.02
NF16-2	1.50%	1.70%	2.60%	2.20%	3.20%	5.10%	0.13	0.11	0.08	0.09	0.05	0.02
NF17-2	4.30%	4.70%	5.20%	6.20%	6.90%	6.90%	0.13	0.11	0.08	0.09	0.05	0.02
NF21-2	4.50%	4.70%	4.30%	3.90%	4.20%	5.20%	0.13	0.10	0.08	0.08	0.05	0.02
NF22-1	3.40%	3.70%	3.50%	3.60%	4.20%	4.80%	0.12	0.10	0.07	0.07	0.04	0.02
NF25-2	2.90%	3.00%	3.60%	3.50%	2.90%	2.90%	0.14	0.11	0.08	0.08	0.05	0.02
NF27-2	1.50%	1.30%	1.20%	1.40%	1.90%	3.30%	0.14	0.11	0.08	0.09	0.05	0.02
NF28-1	7.30%	8.50%	8.60%	8.50%	8.70%	9.10%	0.14	0.11	0.08	0.09	0.05	0.02

Ground Motion	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	4th Floor Max Normalized Column Axial Force(INT)	5th Floor Max Normalized Column Axial Force(INT)	6th Floor Max Normalized Column Axial Force(INT)	1st Floor Moment frame Rotation(EXT)	2nd Floor Moment frame Rotation(EXT)	3rd Floor Moment frame Rotation(EXT)	4th Floor Moment frame Rotation(EXT)	5th Floor Moment frame Rotation(EXT)	6th Floor Moment frame Rotation(EXT)
FF01-1	0.13	0.11	0.09	0.09	0.06	0.03	0.034	0.022	0.024	0.046	0.070	0.077
FF13-1	0.13	0.11	0.08	0.09	0.06	0.03	0.002	0.007	0.012	0.025	0.028	0.036
FF14-1	0.12	0.10	0.08	0.09	0.06	0.03	0.002	0.001	0.003	0.001	0.007	0.013
FF14-2	0.13	0.11	0.08	0.09	0.06	0.03	0.063	0.075	0.068	0.027	0.053	0.068
FF15-2	0.13	0.11	0.09	0.09	0.06	0.03	0.037	0.036	0.030	0.028	0.025	0.028
FF19-1	0.13	0.11	0.08	0.09	0.06	0.03	0.068	0.070	0.065	0.040	0.035	0.037
FF21-2	0.13	0.10	0.08	0.09	0.06	0.03	0.011	0.017	0.019	0.016	0.011	0.015
FF22-1	0.12	0.10	0.08	0.09	0.06	0.03	0.004	0.006	0.003	0.008	0.011	0.023
FF22-2	0.11	0.10	0.08	0.09	0.06	0.03	0.004	0.010	0.007	0.005	0.017	0.032
NF02-2	0.12	0.10	0.07	0.08	0.05	0.03	0.070	0.064	0.043	0.034	0.037	0.046
NF05-1	0.13	0.11	0.08	0.09	0.06	0.03	0.072	0.080	0.084	0.085	0.087	0.087
NF05-2	0.13	0.10	0.08	0.09	0.06	0.03	0.058	0.042	0.033	0.023	0.020	0.015
NF16-1	0.12	0.11	0.08	0.09	0.06	0.03	0.013	0.015	0.019	0.022	0.025	0.037
NF16-2	0.12	0.11	0.09	0.09	0.06	0.03	0.010	0.010	0.011	0.014	0.022	0.032
NF17-2	0.13	0.11	0.08	0.09	0.06	0.03	0.029	0.032	0.040	0.056	0.061	0.045
NF21-2	0.12	0.10	0.08	0.09	0.06	0.03	0.031	0.028	0.020	0.023	0.029	0.053
NF22-1	0.12	0.10	0.09	0.09	0.06	0.03	0.020	0.018	0.014	0.020	0.020	0.024
NF25-2	0.13	0.11	0.09	0.10	0.06	0.03	0.037	0.030	0.025	0.015	0.020	0.025
NF27-2	0.13	0.10	0.08	0.09	0.06	0.03	0.037	0.038	0.042	0.042	0.070	0.081
NF28-1	0.13	0.11	0.08	0.09	0.05	0.03	0.078	0.085	0.076	0.075	0.082	0.076

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	4th Floor Moment Frame Rotation(INT)	5th Floor Moment Frame Rotation(INT)	6th Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (EXT)
FF01-1	0.044	0.030	0.033	0.059	0.082	0.102	3.775	3.383	3.719	4.311	4.814	7.786
FF13-1	0.006	0.014	0.015	0.029	0.031	0.034	2.247	2.772	2.831	2.881	3.644	7.639
FF14-1	0.006	0.004	0.007	0.009	0.018	0.041	2.186	1.875	2.500	2.783	3.811	7.844
FF14-2	0.068	0.082	0.074	0.032	0.058	0.054	2.894	3.203	2.783	3.500	3.564	7.306
FF15-2	0.048	0.047	0.040	0.037	0.036	0.050	3.881	3.914	3.897	3.719	4.256	7.128
FF19-1	0.075	0.077	0.071	0.047	0.038	0.036	3.797	3.522	3.503	3.275	3.872	6.061
FF21-2	0.019	0.025	0.028	0.024	0.021	0.023	2.694	2.969	3.278	3.253	3.517	5.403
FF22-1	0.011	0.009	0.010	0.010	0.015	0.027	2.428	2.436	2.669	2.744	3.264	9.283
FF22-2	0.007	0.013	0.015	0.007	0.023	0.058	2.300	2.428	2.936	2.125	3.514	8.161
NF02-2	0.077	0.070	0.049	0.039	0.040	0.034	2.944	2.889	2.594	2.403	2.350	4.658
NF05-1	0.083	0.091	0.096	0.098	0.101	0.106	4.261	4.592	4.833	4.917	5.481	8.567
NF05-2	0.069	0.055	0.044	0.032	0.031	0.037	4.086	4.069	3.894	3.683	3.914	7.319
NF16-1	0.019	0.023	0.022	0.025	0.032	0.040	2.836	3.064	2.931	2.897	4.094	6.842
NF16-2	0.014	0.014	0.017	0.017	0.031	0.064	2.292	2.681	3.039	2.728	3.914	8.008
NF17-2	0.038	0.041	0.054	0.067	0.073	0.073	3.542	3.667	4.139	4.322	4.667	8.017
NF21-2	0.040	0.037	0.029	0.033	0.041	0.061	3.617	3.572	3.511	3.525	4.211	7.711
NF22-1	0.029	0.026	0.023	0.029	0.032	0.057	3.136	3.156	3.281	3.322	4.161	7.850
NF25-2	0.042	0.035	0.029	0.022	0.022	0.024	2.928	2.947	3.361	3.219	3.686	5.811
NF27-2	0.042	0.044	0.048	0.049	0.074	0.071	2.528	2.625	2.625	2.542	3.478	6.875
NF28-1	0.089	0.102	0.088	0.086	0.096	0.100	4.658	4.964	4.822	4.753	5.325	6.706

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3) (INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	1.022	1.064	1.314	1.458	2.792	7.578	0.914	0.943	0.716	0.758	0.856	2.504
FF13-1	1.628	1.869	2.119	2.661	4.531	9.172	0.671	0.874	0.563	0.739	0.776	2.032
FF14-1	0.739	0.747	1.325	1.481	2.633	4.606	0.593	0.583	0.834	0.531	0.982	2.100
FF14-2	1.256	1.233	2.203	1.967	3.725	5.339	0.829	0.695	0.875	1.029	0.854	1.994
FF15-2	1.308	1.483	2.283	2.589	4.228	8.139	0.608	0.438	0.565	0.605	0.766	1.917
FF19-1	1.214	1.144	1.364	1.389	1.747	3.589	0.406	0.261	0.250	0.267	0.522	1.018
FF21-2	0.714	0.692	0.953	1.025	1.831	3.097	0.420	0.329	0.358	0.335	0.432	1.628
FF22-1	0.922	1.147	1.619	1.856	2.658	6.200	0.722	0.663	0.597	0.605	0.680	2.319
FF22-2	1.269	1.103	1.708	2.269	2.953	6.669	0.734	0.834	0.792	0.700	0.925	1.604
NF02-2	0.747	0.756	1.022	1.222	1.817	3.617	0.458	0.504	0.236	0.362	0.389	0.825
NF05-1	1.183	1.117	1.717	1.644	3.142	3.875	0.595	0.726	0.358	0.515	0.710	1.567
NF05-2	1.258	1.192	1.744	1.753	2.581	4.547	0.479	0.319	0.278	0.383	0.500	1.397
NF16-1	1.125	1.339	1.567	1.911	4.344	5.075	0.877	0.758	0.771	0.605	0.899	1.501
NF16-2	1.022	1.161	1.314	1.703	3.503	5.083	0.607	0.802	0.729	0.561	0.949	2.283
NF17-2	1.172	1.056	1.433	1.778	2.172	5.756	0.648	0.726	0.425	0.413	0.856	1.512
NF21-2	0.989	1.075	2.256	2.558	2.714	8.600	0.685	0.954	0.820	0.710	1.255	1.872
NF22-1	0.908	1.258	2.139	1.733	2.833	5.314	0.409	0.435	0.467	0.625	1.362	2.204
NF25-2	0.853	0.908	1.103	1.117	1.697	4.292	0.360	0.289	0.412	0.320	0.402	0.547
NF27-2	0.989	1.075	1.367	1.433	2.383	5.514	0.523	0.326	0.535	0.593	0.701	1.465
NF28-1	1.067	1.175	1.303	1.375	1.842	4.031	0.461	1.091	0.371	0.433	0.508	0.471

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	0.450	0.376	0.621	0.628	0.756	0.672	2.90%	2.60%	2.50%	3.10%	3.90%	4.80%	1.114
FF13-1	0.504	0.680	0.294	0.708	0.431	0.963	0.44%	0.69%	0.84%	0.73%	0.47%	0.46%	0.985
FF14-1	0.593	0.583	0.834	0.531	0.982	2.100	0.71%	0.64%	0.56%	0.60%	0.75%	0.89%	1.051
FF14-2	0.829	0.695	0.875	1.029	0.854	1.994	5.00%	6.10%	6.20%	5.80%	4.00%	4.20%	0.962
FF15-2	0.608	0.438	0.565	0.605	0.766	1.917	3.00%	3.10%	2.90%	2.70%	2.60%	2.60%	1.159
FF19-1	0.357	0.238	0.307	0.341	0.368	0.316	5.10%	5.50%	5.30%	4.70%	3.90%	3.50%	0.659
FF21-2	0.490	0.386	0.417	0.346	0.512	0.699	0.36%	0.49%	0.50%	0.46%	0.43%	0.43%	0.721
FF22-1	0.614	0.450	0.435	0.594	0.551	0.390	0.04%	0.15%	0.14%	0.07%	0.14%	0.23%	0.94
FF22-2	0.450	0.538	0.527	0.635	0.484	0.860	0.33%	0.30%	0.29%	0.26%	0.22%	0.96%	1.136
NF02-2	0.396	0.371	0.473	0.339	0.546	0.474	5.00%	5.10%	4.60%	3.70%	3.00%	2.90%	0.633
NF05-1	0.309	0.376	0.531	0.593	0.669	0.648	5.20%	6.20%	6.70%	7.00%	7.20%	7.30%	0.871
NF05-2	0.609	0.393	0.234	0.296	0.414	0.502	2.90%	3.50%	3.20%	2.70%	2.20%	2.10%	0.717
NF16-1	0.766	0.536	0.492	0.426	0.643	0.767	0.56%	0.52%	0.39%	0.37%	0.38%	0.01%	1.165
NF16-2	0.391	0.575	0.523	0.435	0.406	0.893	0.65%	0.83%	1.00%	1.20%	1.20%	0.49%	1.068
NF17-2	0.501	0.407	0.335	0.458	0.366	0.520	2.30%	2.50%	3.00%	3.50%	4.00%	3.90%	1.086
NF21-2	0.721	0.709	0.585	0.969	0.718	0.841	0.95%	0.67%	0.44%	0.12%	0.25%	0.60%	1.232
NF22-1	0.481	0.324	0.313	0.393	0.426	0.868	1.70%	1.60%	1.50%	1.50%	1.60%	1.80%	0.889
NF25-2	0.385	0.462	0.289	0.509	0.523	0.671	1.10%	1.20%	1.00%	0.77%	0.49%	0.39%	0.581
NF27-2	0.336	0.255	0.366	0.361	0.543	0.702	3.00%	3.30%	3.60%	4.00%	4.30%	4.40%	1.034
NF28-1	0.309	1.764	0.355	0.279	0.324	0.473	6.30%	7.10%	7.30%	7.30%	7.20%	7.30%	0.568

Ground Motion	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	4th Floor RBS Energy Dissipation(EXT)	5th Floor RBS Energy Dissipation(EXT)	6th Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)	4th Floor RBS Energy Dissipation(INT)	5th Floor RBS Energy Dissipation(INT)	6th Floor RBS Energy Dissipation(INT)
FF01-1	31.00	20.40	20.93	68.72	75.89	41.08	344.91	252.96	171.12	306.85	371.86	175.51
FF13-1	10.70	16.34	8.91	13.78	81.63	44.58	36.65	92.42	101.26	112.53	88.47	35.73
FF14-1	6.47	4.39	4.41	0.36	23.17	21.16	21.78	12.66	18.53	16.82	21.97	15.88
FF14-2	281.33	343.87	279.88	114.02	135.28	73.79	34.12	100.33	41.23	28.64	66.11	31.10
FF15-2	69.49	57.14	32.42	31.54	14.04	8.19	231.98	312.92	246.41	194.46	186.23	92.09
FF19-1	651.72	570.45	378.18	163.77	95.54	32.40	90.48	108.88	92.39	33.86	20.44	7.45
FF21-2	26.03	34.29	23.11	12.75	7.05	2.70	84.55	120.94	93.37	87.71	100.19	45.79
FF22-1	21.19	21.78	17.67	49.45	22.28	6.80	19.81	79.08	20.46	47.01	53.33	31.28
FF22-2	2.07	7.26	34.77	16.17	40.00	35.31	47.98	105.02	45.51	25.11	72.45	65.48
NF02-2	494.60	422.38	122.11	75.87	56.10	28.05	64.85	40.55	21.83	31.87	31.69	15.38
NF05-1	35.33	39.70	33.71	34.07	26.42	20.67	651.87	714.82	589.45	539.98	357.27	136.85
NF05-2	211.65	167.36	91.53	68.52	50.73	23.15	283.38	211.64	84.99	63.40	45.42	20.55
NF16-1	46.21	78.82	52.55	21.30	85.74	38.74	109.89	130.37	134.28	151.27	122.06	54.76
NF16-2	21.53	44.14	77.98	75.97	138.52	66.47	78.54	58.40	34.39	38.64	43.31	49.57
NF17-2	62.65	53.04	43.42	67.22	69.11	24.12	291.18	307.04	358.97	344.51	236.49	97.02
NF21-2	96.87	101.76	79.40	119.68	140.54	85.08	113.72	123.84	56.58	71.14	118.09	80.13
NF22-1	46.86	32.14	15.83	52.71	23.78	18.63	62.56	67.74	44.74	60.19	44.27	46.47
NF25-2	105.15	114.19	89.06	59.79	38.66	11.01	253.76	222.00	165.14	128.57	99.21	36.06
NF27-2	114.22	103.87	94.68	95.88	206.29	85.06	87.71	66.93	46.36	42.58	81.04	42.89
NF28-1	39.51	61.29	30.65	34.38	28.97	11.52	621.47	1068.00	473.75	511.56	411.52	167.37

Table B-12: Collected Result for 6 story MF as Girder for Each Earthquake--Horizontal + Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	4th Floor Max Normalized Column Axial Force(EXT)	5th Floor Max Normalized Column Axial Force(EXT)	6th Floor Max Normalized Column Axial Force(EXT)
FF01-1	5.30%	4.60%	4.10%	4.90%	6.80%	8.60%	0.16	0.14	0.11	0.11	0.07	0.04
FF13-1	1.50%	2.00%	2.40%	2.40%	2.20%	3.10%	0.20	0.19	0.15	0.15	0.10	0.05
FF14-1	1.60%	1.30%	1.50%	1.80%	2.20%	3.50%	0.14	0.12	0.09	0.09	0.06	0.03
FF14-2	0.83%	1.00%	1.30%	2.00%	2.90%	3.60%	0.19	0.16	0.13	0.13	0.08	0.03
FF15-2	4.10%	4.90%	4.90%	4.80%	4.80%	5.00%	0.23	0.19	0.15	0.15	0.10	0.04
FF19-1	4.90%	4.80%	4.30%	3.80%	3.50%	3.90%	0.15	0.12	0.09	0.09	0.06	0.02
FF21-2	2.10%	3.10%	3.50%	3.60%	3.10%	2.80%	0.14	0.11	0.09	0.09	0.06	0.03
FF22-1	1.70%	2.10%	2.00%	2.10%	2.30%	2.80%	0.16	0.13	0.11	0.11	0.07	0.04
FF22-2	1.60%	1.90%	2.20%	2.30%	1.40%	3.70%	0.15	0.15	0.13	0.13	0.08	0.04
NF02-2	0.92%	1.10%	0.76%	0.84%	0.98%	1.10%	0.16	0.13	0.10	0.10	0.06	0.02
NF05-1	6.30%	7.90%	8.50%	8.90%	9.00%	9.30%	0.16	0.13	0.10	0.10	0.06	0.03
NF05-2	6.00%	6.10%	5.40%	4.80%	4.40%	4.60%	0.15	0.12	0.09	0.09	0.06	0.02
NF16-1	2.00%	2.80%	2.80%	2.40%	3.40%	4.10%	0.22	0.18	0.14	0.13	0.09	0.03
NF16-2	1.50%	1.70%	2.50%	2.10%	3.10%	4.80%	0.16	0.14	0.11	0.11	0.07	0.03
NF17-2	4.20%	4.60%	5.10%	5.80%	6.30%	6.20%	0.15	0.13	0.09	0.10	0.06	0.03
NF21-2	4.50%	4.80%	4.70%	3.90%	4.50%	6.10%	0.19	0.16	0.14	0.15	0.09	0.05
NF22-1	3.50%	3.90%	3.60%	3.50%	4.10%	5.00%	0.14	0.13	0.11	0.11	0.07	0.03
NF25-2	3.00%	3.00%	3.60%	3.50%	3.00%	3.00%	0.16	0.13	0.09	0.09	0.06	0.03
NF27-2	1.50%	1.40%	1.30%	1.40%	2.10%	3.40%	0.19	0.15	0.11	0.11	0.07	0.03
NF28-1	7.00%	8.40%	8.40%	8.10%	8.40%	8.80%	0.16	0.13	0.10	0.10	0.06	0.02

Ground Motion	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	4th Floor Max Normalized Column Axial Force(INT)	5th Floor Max Normalized Column Axial Force(INT)	6th Floor Max Normalized Column Axial Force(INT)	1st Floor Moment Frame Rotation(EXT)	2nd Floor Moment Frame Rotation(EXT)	3rd Floor Moment Frame Rotation(EXT)	4th Floor Moment Frame Rotation(EXT)	5th Floor Moment Frame Rotation(EXT)	6th Floor Moment Frame Rotation(EXT)
FF01-1	0.22	0.18	0.14	0.15	0.11	0.06	0.036	0.022	0.024	0.047	0.068	0.074
FF13-1	0.32	0.25	0.19	0.21	0.16	0.08	0.001	0.006	0.011	0.027	0.042	0.054
FF14-1	0.22	0.19	0.17	0.18	0.14	0.05	0.002	0.001	0.003	0.001	0.008	0.011
FF14-2	0.29	0.26	0.22	0.24	0.18	0.06	0.058	0.069	0.074	0.038	0.067	0.079
FF15-2	0.29	0.25	0.21	0.22	0.15	0.07	0.030	0.032	0.033	0.027	0.039	0.050
FF19-1	0.16	0.13	0.10	0.12	0.08	0.04	0.066	0.066	0.060	0.033	0.029	0.032
FF21-2	0.16	0.14	0.12	0.11	0.10	0.04	0.010	0.017	0.018	0.015	0.010	0.016
FF22-1	0.25	0.21	0.17	0.18	0.12	0.06	0.005	0.005	0.006	0.007	0.014	0.026
FF22-2	0.22	0.19	0.17	0.19	0.12	0.06	0.005	0.011	0.008	0.008	0.025	0.044
NF02-2	0.16	0.14	0.12	0.13	0.10	0.04	0.070	0.064	0.048	0.034	0.039	0.049
NF05-1	0.22	0.19	0.16	0.16	0.13	0.05	0.068	0.075	0.080	0.081	0.086	0.086
NF05-2	0.25	0.21	0.17	0.17	0.13	0.05	0.058	0.042	0.034	0.027	0.026	0.022
NF16-1	0.25	0.22	0.18	0.18	0.14	0.06	0.016	0.017	0.022	0.024	0.032	0.036
NF16-2	0.23	0.19	0.15	0.17	0.12	0.05	0.011	0.011	0.012	0.014	0.022	0.037
NF17-2	0.20	0.16	0.14	0.15	0.11	0.05	0.027	0.031	0.037	0.047	0.052	0.038
NF21-2	0.31	0.27	0.23	0.25	0.16	0.08	0.031	0.030	0.027	0.025	0.037	0.067
NF22-1	0.24	0.20	0.17	0.16	0.11	0.05	0.023	0.020	0.019	0.023	0.026	0.034
NF25-2	0.17	0.14	0.10	0.12	0.09	0.05	0.034	0.029	0.024	0.016	0.020	0.024
NF27-2	0.18	0.16	0.13	0.14	0.09	0.05	0.036	0.036	0.040	0.037	0.071	0.082
NF28-1	0.18	0.15	0.12	0.15	0.11	0.05	0.077	0.083	0.075	0.071	0.079	0.074

Ground Motion	1st Floor Moment Frame Rotation(INT)	2nd Floor Moment Frame Rotation(INT)	3rd Floor Moment Frame Rotation(INT)	4th Floor Moment Frame Rotation(INT)	5th Floor Moment Frame Rotation(INT)	6th Floor Moment Frame Rotation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)
FF01-1	0.049	0.032	0.036	0.061	0.081	0.100	3.997	3.803	4.300	4.478	5.433	9.989
FF13-1	0.006	0.013	0.014	0.027	0.041	0.042	3.000	3.311	3.919	4.756	7.206	9.103
FF14-1	0.006	0.005	0.007	0.008	0.019	0.046	2.331	2.078	3.075	2.903	4.461	8.208
FF14-2	0.064	0.076	0.085	0.043	0.072	0.068	3.786	3.531	4.008	3.925	5.686	8.808
FF15-2	0.039	0.042	0.041	0.038	0.049	0.064	3.906	4.297	5.356	5.467	7.247	9.139
FF19-1	0.072	0.072	0.065	0.038	0.033	0.033	4.006	3.767	3.800	3.581	4.272	6.378
FF21-2	0.018	0.024	0.027	0.023	0.020	0.023	2.636	2.950	3.264	3.242	3.631	6.589
FF22-1	0.011	0.010	0.010	0.011	0.022	0.036	3.000	2.958	3.619	3.486	5.253	10.208
FF22-2	0.007	0.014	0.015	0.012	0.025	0.047	2.922	2.675	3.197	2.817	5.908	7.433
NF02-2	0.076	0.070	0.054	0.039	0.040	0.034	2.989	2.797	2.764	2.606	2.469	4.711
NF05-1	0.079	0.087	0.092	0.093	0.098	0.108	4.531	4.431	4.994	4.878	5.844	8.578
NF05-2	0.068	0.054	0.045	0.036	0.039	0.052	5.039	4.619	4.644	4.206	4.731	6.675
NF16-1	0.019	0.021	0.024	0.025	0.036	0.041	3.406	3.903	3.519	3.986	4.886	7.533
NF16-2	0.014	0.014	0.017	0.018	0.032	0.066	2.589	2.914	3.453	3.300	4.756	8.583
NF17-2	0.036	0.040	0.049	0.061	0.066	0.073	3.472	3.639	4.064	4.256	4.906	8.561
NF21-2	0.040	0.041	0.036	0.033	0.050	0.084	4.014	3.892	4.811	5.350	6.058	12.575
NF22-1	0.031	0.029	0.028	0.031	0.036	0.071	3.500	4.053	5.089	4.292	5.503	8.847
NF25-2	0.039	0.033	0.028	0.022	0.023	0.026	3.092	3.283	3.567	3.597	4.108	6.275
NF27-2	0.041	0.042	0.046	0.043	0.073	0.073	2.769	3.139	3.611	3.417	5.522	8.439
NF28-1	0.088	0.094	0.086	0.082	0.093	0.097	4.703	4.956	4.867	4.833	5.358	7.178

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	1.022	1.064	1.314	1.458	2.792	7.578	2.161	2.014	3.284	3.979	4.117	3.672
FF13-1	1.628	1.869	2.119	2.661	4.531	9.172	4.744	5.275	6.241	7.997	7.995	5.123
FF14-1	2.331	2.078	3.075	2.903	4.461	8.208	0.593	0.583	0.834	0.531	0.982	2.100
FF14-2	3.786	3.531	4.008	3.925	5.686	8.808	0.829	0.695	0.875	1.029	0.854	1.994
FF15-2	3.906	4.297	5.356	5.467	7.247	9.139	0.608	0.438	0.565	0.605	0.766	1.917
FF19-1	1.214	1.144	1.364	1.389	1.747	3.589	1.109	1.339	1.857	2.356	1.973	1.210
FF21-2	0.714	0.692	0.953	1.025	1.831	3.097	1.697	2.126	1.927	2.563	2.179	3.348
FF22-1	0.922	1.147	1.619	1.856	2.658	6.200	3.553	3.974	4.088	3.935	5.645	5.602
FF22-2	1.269	1.103	1.708	2.269	2.953	6.669	3.982	4.324	3.840	3.840	5.982	4.217
NF02-2	0.747	0.756	1.022	1.222	1.817	3.617	2.085	2.575	2.795	3.391	2.385	3.400
NF05-1	1.183	1.117	1.717	1.644	3.142	3.875	2.355	2.789	2.839	3.243	3.295	3.072
NF05-2	1.258	1.192	1.744	1.753	2.581	4.547	2.813	3.262	3.087	3.453	3.458	3.255
NF16-1	1.125	1.339	1.567	1.911	4.344	5.075	3.858	5.012	5.163	5.724	5.697	4.753
NF16-2	1.022	1.161	1.314	1.703	3.503	5.083	3.395	4.576	4.123	4.990	4.838	3.738
NF17-2	1.172	1.056	1.433	1.778	2.172	5.756	2.451	3.007	2.831	2.934	3.471	2.339
NF21-2	0.989	1.075	2.256	2.558	2.714	8.600	3.878	4.752	4.458	5.183	4.362	6.322
NF22-1	0.908	1.258	2.139	1.733	2.833	5.314	3.525	4.638	4.317	3.874	4.481	4.460
NF25-2	0.853	0.908	1.103	1.117	1.697	4.292	1.159	1.348	1.996	2.436	2.511	1.664
NF27-2	0.989	1.075	1.367	1.433	2.383	5.514	1.682	1.935	2.346	2.831	4.000	2.745
NF28-1	1.067	1.175	1.303	1.375	1.842	4.031	1.613	2.046	2.020	2.307	2.838	1.573

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	2.140	2.150	2.745	2.740	4.916	4.454	3.20%	2.80%	2.60%	3.10%	3.80%	4.80%	1.303
FF13-1	6.886	7.064	6.609	8.215	9.500	7.901	0.47%	0.63%	0.57%	0.31%	0.01%	0.13%	1.375
FF14-1	2.971	2.526	3.332	3.850	6.239	3.490	0.65%	0.60%	0.53%	0.58%	0.64%	0.69%	1.061
FF14-2	4.857	4.029	5.321	5.917	8.723	4.965	4.70%	6.00%	6.40%	6.20%	5.00%	5.20%	1.021
FF15-2	4.248	4.849	4.788	5.696	9.430	7.113	2.40%	2.40%	2.30%	2.30%	2.30%	2.40%	0.865
FF19-1	0.981	0.976	1.539	1.670	2.695	1.609	4.90%	5.20%	4.90%	4.20%	3.40%	3.10%	0.710
FF21-2	3.009	2.704	2.933	3.111	4.957	2.341	0.31%	0.43%	0.44%	0.39%	0.36%	0.35%	0.793
FF22-1	3.650	3.995	3.702	4.373	6.288	4.657	0.10%	0.18%	0.13%	0.06%	0.18%	0.31%	0.990
FF22-2	3.339	3.173	4.805	5.002	4.999	4.860	0.21%	0.24%	0.25%	0.20%	0.01%	0.29%	1.298
NF02-2	2.896	2.540	3.082	3.592	4.577	5.234	4.90%	5.00%	4.60%	3.90%	3.30%	3.20%	0.631
NF05-1	4.400	3.465	3.566	4.523	6.155	3.858	4.80%	5.80%	6.20%	6.50%	6.80%	6.90%	0.897
NF05-2	3.816	4.766	5.141	4.587	5.484	3.758	2.90%	3.60%	3.30%	2.90%	2.50%	2.40%	0.816
NF16-1	4.110	4.242	3.992	5.517	7.686	7.138	0.60%	0.54%	0.40%	0.35%	0.31%	0.04%	1.252
NF16-2	3.068	3.601	4.298	4.628	7.453	6.026	0.71%	0.88%	1.10%	1.20%	1.30%	0.69%	1.312
NF17-2	4.020	3.248	3.162	3.921	5.275	3.260	1.90%	2.10%	2.60%	3.00%	3.40%	3.40%	1.149
NF21-2	3.258	3.592	4.522	5.757	6.168	5.875	1.20%	1.00%	0.82%	0.62%	0.80%	0.79%	1.152
NF22-1	3.227	4.023	4.422	3.583	4.434	4.652	1.80%	1.70%	1.60%	1.60%	1.60%	1.80%	0.949
NF25-2	1.290	1.472	1.589	1.473	3.411	3.279	0.84%	0.87%	0.77%	0.58%	0.34%	0.25%	0.815
NF27-2	1.957	1.958	2.160	2.537	2.718	3.465	2.60%	2.80%	3.00%	3.20%	3.50%	3.70%	1.050
NF28-1	2.313	1.984	2.284	2.599	2.850	3.402	5.90%	6.70%	6.90%	6.90%	6.80%	7.00%	0.617

Ground Motion	1st Floor RBS Energy Dissipation(EXT)	2nd Floor RBS Energy Dissipation(EXT)	3rd Floor RBS Energy Dissipation(EXT)	4th Floor RBS Energy Dissipation(EXT)	5th Floor RBS Energy Dissipation(EXT)	6th Floor RBS Energy Dissipation(EXT)	1st Floor RBS Energy Dissipation(INT)	2nd Floor RBS Energy Dissipation(INT)	3rd Floor RBS Energy Dissipation(INT)	4th Floor RBS Energy Dissipation(INT)	5th Floor RBS Energy Dissipation(INT)	6th Floor RBS Energy Dissipation(INT)
FF01-1	30.74	18.83	15.63	63.15	73.38	43.32	399.98	259.52	182.52	308.67	369.95	172.38
FF13-1	4.24	14.71	24.34	51.49	123.98	61.14	24.22	59.92	78.33	100.45	114.81	49.19
FF14-1	9.03	5.09	3.92	0.46	27.97	23.87	22.00	14.45	13.77	17.44	22.61	17.40
FF14-2	230.65	312.50	300.11	123.88	260.72	102.99	23.85	49.63	21.64	31.10	66.02	28.21
FF15-2	80.61	80.63	65.73	33.51	45.41	21.62	228.84	284.53	241.34	197.32	231.63	144.61
FF19-1	612.86	506.29	332.64	140.22	88.79	28.32	104.15	119.84	94.43	29.26	18.79	6.39
FF21-2	26.35	34.41	23.01	12.74	7.51	2.26	82.14	119.64	91.12	87.94	112.14	44.90
FF22-1	35.63	29.62	30.69	46.22	38.62	16.69	22.13	69.33	30.38	43.36	69.75	36.46
FF22-2	17.92	10.45	42.71	18.52	57.69	45.19	48.32	100.25	51.45	40.11	75.00	77.37
NF02-2	488.01	425.48	163.69	76.30	59.81	36.20	68.37	42.81	22.66	25.84	25.28	11.47
NF05-1	33.13	37.04	32.58	31.60	33.79	21.88	645.78	716.53	608.56	567.91	400.27	143.02
NF05-2	199.51	151.08	89.46	87.60	69.32	28.70	274.72	199.14	94.74	68.62	52.27	31.90
NF16-1	41.28	81.04	55.88	40.10	81.74	38.62	118.46	138.44	144.52	153.16	162.73	56.84
NF16-2	28.74	50.80	77.79	78.82	146.94	69.49	83.88	51.21	33.90	39.50	47.86	51.40
NF17-2	78.23	78.77	62.60	94.26	77.76	25.43	258.51	273.46	300.36	281.59	197.23	114.56
NF21-2	94.16	101.42	108.83	103.95	113.20	73.71	122.80	121.08	70.09	74.59	162.04	92.89
NF22-1	48.39	39.12	26.83	47.29	35.66	28.54	77.71	82.65	59.88	70.99	52.09	55.39
NF25-2	102.65	109.07	86.87	61.31	39.85	12.65	255.17	216.34	154.20	136.75	104.10	38.21
NF27-2	112.37	106.20	96.48	88.42	219.20	91.59	109.48	96.77	71.13	67.91	99.27	50.12
NF28-1	56.19	76.88	38.71	33.23	28.70	11.69	617.63	586.59	478.79	500.71	415.36	172.32

Table B-13: Collected Result for 6 story Braced Frame for Each Earthquake-- Chevron --Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Column Axial Force(EXT)	2nd Floor Max Column Axial Force(EXT)	3rd Floor Max Column Axial Force(EXT)	4th Floor Max Column Axial Force(EXT)	5th Floor Max Column Axial Force(EXT)	6th Floor Max Column Axial Force(EXT)
FF01-1	4.60%	4.20%	3.80%	3.20%	2.50%	2.90%	0.20	0.15	0.11	0.11	0.06	0.02
FF13-1	2.00%	1.50%	1.00%	0.84%	0.70%	0.68%	0.17	0.13	0.10	0.10	0.05	0.01
FF14-1	1.10%	0.91%	0.91%	1.00%	1.20%	1.20%	0.17	0.13	0.09	0.09	0.05	0.01
FF14-2	3.70%	2.90%	2.20%	1.80%	1.50%	1.20%	0.21	0.16	0.11	0.11	0.05	0.01
FF15-2	3.30%	2.70%	2.30%	1.90%	1.60%	1.30%	0.20	0.15	0.10	0.10	0.05	0.01
FF19-1	3.40%	2.60%	2.00%	1.50%	0.99%	0.59%	0.19	0.14	0.10	0.09	0.05	0.01
FF21-2	2.60%	1.90%	1.40%	0.92%	0.76%	0.78%	0.16	0.12	0.09	0.09	0.05	0.01
FF22-1	2.40%	1.90%	1.60%	1.30%	1.30%	1.30%	0.17	0.13	0.10	0.10	0.05	0.01
FF22-2	1.40%	0.82%	0.29%	0.20%	0.40%	0.38%	0.17	0.13	0.09	0.10	0.05	0.01
NF02-2	1.80%	1.10%	0.47%	0.31%	0.28%	0.26%	0.19	0.14	0.09	0.09	0.05	0.01
NF05-1	3.60%	3.30%	2.90%	2.70%	2.40%	2.40%	0.19	0.15	0.10	0.10	0.05	0.01
NF05-2	4.70%	4.00%	3.50%	3.30%	3.20%	3.00%	0.20	0.15	0.11	0.10	0.05	0.01
NF16-1	3.90%	3.10%	2.80%	2.90%	3.20%	3.00%	0.20	0.15	0.11	0.11	0.06	0.01
NF16-2	2.10%	1.50%	1.20%	1.30%	1.40%	1.70%	0.17	0.13	0.09	0.10	0.05	0.01
NF17-2	2.90%	2.60%	2.40%	2.20%	1.90%	1.80%	0.20	0.15	0.10	0.10	0.05	0.01
NF21-2	4.80%	3.90%	3.20%	2.60%	2.00%	1.50%	0.18	0.14	0.10	0.10	0.05	0.01
NF22-1	5.20%	4.10%	3.30%	2.40%	2.00%	1.80%	0.17	0.13	0.09	0.10	0.05	0.01
NF25-2	4.50%	4.00%	3.40%	2.80%	2.10%	2.10%	0.19	0.14	0.10	0.10	0.05	0.01
NF27-2	2.10%	1.50%	0.96%	0.92%	0.86%	0.79%	0.20	0.15	0.10	0.10	0.05	0.01
NF28-1	4.80%	4.00%	3.30%	2.70%	2.30%	2.10%	0.20	0.15	0.10	0.10	0.05	0.01

Ground Motion	1st Floor Max Column Axial Force(INT)	2nd Floor Max Column Axial Force(INT)	3rd Floor Max Column Axial Force(INT)	4th Floor Max Column Axial Force(INT)	5th Floor Max Column Axial Force(INT)	6th Floor Max Column Axial Force(INT)	1st Floor BRB Max Normalized Axial Deformation (EXT)	2nd Floor BRB Max Normalized Axial Deformation (EXT)	3rd Floor BRB Max Normalized Axial Deformation (EXT)	4th Floor BRB Max Normalized Axial Deformation (EXT)	5th Floor BRB Max Normalized Axial Deformation (EXT)	6th Floor BRB Max Normalized Axial Deformation (EXT)
FF01-1	0.22	0.17	0.13	0.14	0.08	0.03	29.126	28.400	25.423	23.757	18.777	23.600
FF13-1	0.24	0.18	0.13	0.14	0.08	0.03	14.052	11.732	9.227	9.000	8.573	10.095
FF14-1	0.21	0.17	0.13	0.13	0.08	0.03	9.533	9.264	8.632	9.262	10.650	12.540
FF14-2	0.24	0.19	0.13	0.14	0.08	0.03	28.222	24.600	20.568	17.038	12.991	14.425
FF15-2	0.22	0.17	0.12	0.13	0.08	0.03	20.715	18.755	16.991	16.267	13.855	13.850
FF19-1	0.26	0.20	0.14	0.14	0.08	0.03	24.559	19.545	15.732	13.733	10.718	11.815
FF21-2	0.20	0.16	0.12	0.12	0.07	0.03	17.437	14.268	10.977	9.624	8.818	10.420
FF22-1	0.20	0.16	0.12	0.13	0.08	0.03	14.678	12.977	11.441	10.957	11.705	13.705
FF22-2	0.21	0.17	0.13	0.13	0.08	0.03	11.470	8.905	6.350	6.862	7.327	8.460
NF02-2	0.25	0.19	0.13	0.14	0.08	0.02	27.952	24.032	20.450	16.643	11.941	10.170
NF05-1	0.22	0.17	0.12	0.13	0.07	0.03	23.470	22.159	19.636	19.100	17.186	19.400
NF05-2	0.25	0.19	0.14	0.14	0.08	0.03	28.241	25.618	22.323	22.562	20.741	22.370
NF16-1	0.22	0.17	0.12	0.13	0.08	0.03	23.981	21.818	20.359	21.933	22.236	23.455
NF16-2	0.22	0.17	0.13	0.14	0.08	0.03	16.211	12.641	10.868	11.238	12.427	15.340
NF17-2	0.23	0.18	0.13	0.14	0.08	0.03	17.600	17.745	17.018	16.748	15.005	16.340
NF21-2	0.24	0.18	0.14	0.14	0.08	0.03	28.389	24.855	20.664	18.071	15.159	14.325
NF22-1	0.20	0.15	0.11	0.12	0.07	0.03	29.856	24.973	20.445	17.195	14.714	16.040
NF25-2	0.25	0.19	0.13	0.14	0.08	0.03	26.711	24.991	21.650	19.433	15.495	17.750
NF27-2	0.25	0.19	0.14	0.14	0.08	0.03	22.026	18.055	14.482	11.262	9.618	10.615
NF28-1	0.26	0.20	0.14	0.14	0.08	0.03	28.170	24.700	20.982	18.357	16.564	18.085

Ground Motion	1st Floor BRB Max Normalized Axial Deformation (INT)	2nd Floor BRB Max Normalized Axial Deformation (INT)	3rd Floor BRB Max Normalized Axial Deformation (INT)	4th Floor BRB Max Normalized Axial Deformation (INT)	5th Floor BRB Max Normalized Axial Deformation (INT)	6th Floor BRB Max Normalized Axial Deformation (INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)
FF01-1	17.622	15.555	13.955	11.310	11.955	12.170	5.978	6.242	5.592	5.944	6.053	6.456
FF13-1	17.515	17.823	16.900	16.905	14.918	16.520	5.431	5.381	4.814	5.617	6.003	6.089
FF14-1	11.593	10.105	8.691	8.295	8.595	11.045	5.567	5.294	4.936	5.322	5.683	6.583
FF14-2	35.689	32.559	27.714	26.386	20.532	24.350	4.078	4.656	4.969	5.375	5.625	6.431
FF15-2	20.744	18.182	15.359	13.148	10.536	10.545	5.614	5.736	5.658	6.186	6.139	6.467
FF19-1	30.656	29.332	26.514	25.495	21.005	24.625	5.194	5.233	5.369	6.428	5.792	6.114
FF21-2	8.422	5.527	4.241	6.205	7.723	10.085	5.350	5.461	4.394	4.986	5.194	5.831
FF22-1	9.193	6.791	4.977	5.110	7.082	10.785	4.744	4.972	4.592	4.672	5.586	6.161
FF22-2	16.615	14.027	11.986	12.400	13.759	14.535	5.303	5.564	5.197	5.886	6.181	6.450
NF02-2	35.693	32.377	27.945	25.095	20.441	20.600	4.931	4.833	4.211	4.347	4.544	4.906
NF05-1	13.319	12.614	10.877	10.010	7.741	8.140	5.589	5.406	5.000	5.100	5.225	5.381
NF05-2	19.330	16.414	13.736	13.552	12.227	11.860	5.086	4.456	4.519	4.475	5.000	5.303
NF16-1	17.952	15.241	12.286	10.505	11.659	11.570	5.339	5.783	5.672	6.147	5.647	6.342
NF16-2	16.648	14.164	11.850	10.967	10.923	10.425	5.642	6.019	5.739	6.392	6.244	6.178
NF17-2	18.922	17.114	13.345	11.162	9.482	9.160	5.194	5.394	5.242	5.919	6.000	6.097
NF21-2	20.385	16.427	13.014	10.738	13.073	15.955	4.600	4.725	5.428	5.542	5.886	6.211
NF22-1	23.341	17.968	13.932	9.276	5.455	5.065	3.858	3.586	3.472	4.014	5.161	5.856
NF25-2	18.993	16.595	13.818	11.124	9.923	9.885	4.311	5.164	4.683	5.019	5.533	5.608
NF27-2	33.252	29.764	24.909	22.038	19.545	20.080	6.358	5.972	5.433	5.769	5.750	6.025
NF28-1	22.281	19.182	15.655	14.071	13.859	13.825	4.056	4.256	4.628	5.244	5.961	6.214

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	5.869	5.839	5.892	5.942	5.981	6.297	3.522	1.456	2.251	2.303	2.070	1.981
FF13-1	5.869	5.842	5.892	5.936	5.975	6.269	2.384	1.482	1.413	2.316	1.950	1.091
FF14-1	5.872	5.842	5.894	5.942	5.981	6.278	3.516	1.859	2.008	1.699	1.935	2.174
FF14-2	5.864	5.836	5.892	5.936	5.978	6.267	1.921	2.139	1.953	2.296	1.615	1.310
FF15-2	5.881	5.850	5.897	5.942	5.981	6.264	2.524	1.852	1.648	1.865	1.794	0.713
FF19-1	5.861	5.828	5.878	5.928	5.969	6.256	1.534	1.139	1.065	1.513	1.405	1.247
FF21-2	5.864	5.831	5.881	5.922	5.953	6.256	2.045	1.649	1.018	1.947	1.184	1.545
FF22-1	5.886	5.861	5.919	5.961	5.989	6.286	1.866	2.617	2.582	1.773	1.755	1.151
FF22-2	5.872	5.842	5.889	5.942	5.983	6.267	2.497	1.950	1.925	2.191	1.745	2.157
NF02-2	5.856	5.822	5.872	5.914	5.950	6.250	2.263	1.424	1.117	1.205	1.505	1.482
NF05-1	5.861	5.831	5.875	5.925	5.967	6.264	3.974	2.154	1.346	1.078	1.142	1.541
NF05-2	5.861	5.828	5.872	5.919	5.964	6.256	3.800	1.789	1.213	1.454	1.083	1.173
NF16-1	5.869	5.844	5.903	5.953	5.989	6.272	4.069	1.790	1.629	1.955	1.786	2.048
NF16-2	5.886	5.875	5.933	5.981	6.011	6.286	3.021	2.104	1.807	2.435	1.824	1.875
NF17-2	5.872	5.850	5.908	5.958	5.997	6.278	3.096	1.545	1.551	2.053	1.352	1.379
NF21-2	5.875	5.847	5.908	5.958	5.997	6.300	3.502	2.333	2.533	2.228	1.760	1.475
NF22-1	5.878	5.833	5.919	5.958	6.000	6.261	4.418	1.789	2.334	1.479	1.774	1.531
NF25-2	5.858	5.822	5.872	5.917	5.964	6.250	2.416	1.514	1.098	1.173	1.200	0.994
NF27-2	5.869	5.844	5.897	5.939	5.975	6.281	3.682	2.350	2.131	2.229	1.255	1.675
NF28-1	5.861	5.828	5.878	5.922	5.956	6.244	1.737	0.860	0.878	1.067	0.874	0.860

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	0.209	0.259	0.368	0.489	0.606	0.692	0.51%	0.57%	0.67%	0.73%	0.71%	0.74%	0.443
FF13-1	0.167	0.233	0.307	0.361	0.417	0.423	0.76%	0.80%	0.87%	0.88%	0.86%	0.82%	0.452
FF14-1	0.233	0.259	0.344	0.419	0.487	0.440	0.03%	0.07%	0.11%	0.13%	0.13%	0.06%	0.383
FF14-2	0.185	0.240	0.312	0.369	0.402	0.407	0.06%	0.00%	0.12%	0.23%	0.33%	0.38%	0.430
FF15-2	0.280	0.322	0.481	0.570	0.672	0.367	0.40%	0.46%	0.46%	0.37%	0.21%	0.17%	0.424
FF19-1	0.176	0.192	0.262	0.334	0.397	0.276	0.47%	0.68%	0.93%	1.10%	1.20%	1.30%	0.414
FF21-2	0.086	0.117	0.155	0.188	0.221	0.349	0.55%	0.55%	0.46%	0.31%	0.16%	0.09%	0.395
FF22-1	0.138	0.170	0.223	0.278	0.332	0.492	1.00%	0.93%	0.76%	0.66%	0.62%	0.67%	0.421
FF22-2	0.253	0.261	0.327	0.440	0.554	0.434	0.06%	0.00%	0.22%	0.36%	0.36%	0.26%	0.479
NF02-2	0.123	0.193	0.255	0.306	0.352	0.346	0.92%	1.10%	1.20%	1.30%	1.50%	1.50%	0.398
NF05-1	0.204	0.220	0.275	0.325	0.373	0.353	0.50%	0.62%	0.71%	0.83%	0.93%	1.00%	0.415
NF05-2	0.183	0.210	0.276	0.360	0.445	0.301	0.38%	0.46%	0.37%	0.22%	0.10%	0.10%	0.452
NF16-1	0.204	0.235	0.307	0.414	0.501	0.507	1.20%	1.20%	1.30%	1.30%	1.30%	1.30%	0.473
NF16-2	0.153	0.207	0.280	0.334	0.369	0.533	0.05%	0.01%	0.01%	0.09%	0.16%	0.14%	0.447
NF17-2	0.155	0.244	0.361	0.474	0.564	0.585	0.57%	0.61%	0.50%	0.34%	0.17%	0.11%	0.423
NF21-2	0.272	0.281	0.369	0.480	0.581	0.668	0.07%	0.09%	0.03%	0.01%	0.08%	0.14%	0.499
NF22-1	0.181	0.253	0.329	0.390	0.451	0.388	1.80%	1.90%	1.90%	1.70%	1.30%	1.10%	0.350
NF25-2	0.148	0.210	0.274	0.336	0.389	0.309	0.10%	0.11%	0.12%	0.17%	0.26%	0.24%	0.405
NF27-2	0.156	0.238	0.301	0.350	0.398	0.506	0.05%	0.13%	0.28%	0.38%	0.46%	0.49%	0.435
NF28-1	0.198	0.237	0.302	0.375	0.418	0.246	0.14%	0.14%	0.05%	0.21%	0.29%	0.25%	0.440

Ground Motion	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	4th Floor BRB Energy Dissipation(EXT)	5th Floor BRB Energy Dissipation(EXT)	6th Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)	4th Floor BRB Energy Dissipation(INT)	5th Floor BRB Energy Dissipation(INT)	6th Floor BRB Energy Dissipation(INT)
FF01-1	460.96	230.66	190.58	204.65	159.48	120.98	349.15	250.75	178.24	132.79	82.04	80.90
FF13-1	328.25	223.52	182.05	139.20	88.05	61.26	219.14	145.66	117.40	98.31	68.51	53.49
FF14-1	106.15	69.78	46.72	47.36	38.20	33.73	191.87	129.12	104.70	112.89	87.14	76.44
FF14-2	511.51	333.69	220.17	135.23	64.35	67.20	932.20	548.78	415.74	325.98	203.57	108.36
FF15-2	720.93	542.18	393.70	271.42	117.53	71.55	153.20	98.15	72.94	70.19	61.07	42.42
FF19-1	1232.00	684.04	404.00	241.29	130.28	99.21	302.58	220.49	158.65	119.37	87.62	61.72
FF21-2	98.39	87.16	54.76	44.57	40.79	37.97	356.43	166.90	87.34	68.46	56.02	45.71
FF22-1	306.44	198.67	139.39	124.32	93.47	91.37	165.87	89.46	70.26	50.01	31.60	22.55
FF22-2	430.98	257.65	128.46	107.75	97.16	70.28	142.28	73.52	77.72	92.37	66.31	44.37
NF02-2	891.52	479.71	356.91	209.24	93.08	48.13	379.36	238.46	182.83	158.00	113.22	81.14
NF05-1	360.16	237.51	186.16	168.16	128.31	99.35	360.72	229.59	154.15	104.30	62.05	43.50
NF05-2	247.52	139.46	92.09	44.89	31.83	24.01	1079.00	764.12	550.13	413.05	289.56	199.39
NF16-1	382.80	260.29	236.72	188.02	144.88	106.86	430.20	210.03	127.36	97.48	89.82	56.42
NF16-2	208.44	132.42	96.71	89.02	83.67	64.66	378.23	215.93	137.39	120.59	69.88	69.46
NF17-2	650.79	364.85	282.71	201.58	124.19	79.43	174.25	144.91	90.92	66.71	30.63	35.74
NF21-2	384.12	244.16	186.84	121.90	108.34	81.71	641.92	368.32	291.85	192.86	122.10	79.48
NF22-1	277.46	225.69	207.56	168.16	91.11	67.36	458.01	178.17	91.87	43.20	35.46	27.86
NF25-2	204.56	146.55	93.84	56.65	43.65	31.74	1012.00	687.96	520.47	358.62	184.22	130.53
NF27-2	710.26	409.35	243.47	165.66	107.40	71.27	458.84	210.70	151.61	136.79	123.15	90.80
NF28-1	577.95	309.73	247.05	193.01	126.20	70.18	768.43	517.28	319.71	238.61	145.62	98.54

Table B-14: Collected Result for 6 story Braced Frame for Each Earthquake-- Chevron --Horizontal + Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	4th Floor Max Normalized Column Axial Force(EXT)	5th Floor Max Normalized Column Axial Force(EXT)	6th Floor Max Normalized Column Axial Force(EXT)
FF01-1	4.60%	4.20%	3.70%	3.20%	2.40%	2.80%	0.20	0.16	0.12	0.12	0.06	0.02
FF13-1	1.90%	1.40%	1.00%	0.85%	0.75%	0.76%	0.18	0.14	0.10	0.10	0.06	0.02
FF14-1	1.00%	0.85%	0.81%	0.94%	1.00%	1.10%	0.16	0.12	0.09	0.09	0.05	0.01
FF14-2	4.00%	3.20%	2.40%	2.00%	1.80%	1.50%	0.22	0.16	0.12	0.11	0.06	0.02
FF15-2	3.20%	2.60%	2.20%	1.80%	1.40%	1.20%	0.20	0.15	0.11	0.11	0.06	0.02
FF19-1	3.40%	2.60%	2.00%	1.50%	0.96%	0.56%	0.20	0.14	0.10	0.10	0.05	0.01
FF21-2	2.60%	1.90%	1.40%	0.96%	0.78%	0.81%	0.16	0.12	0.09	0.09	0.05	0.01
FF22-1	2.30%	1.90%	1.60%	1.30%	1.30%	1.30%	0.17	0.13	0.09	0.10	0.05	0.02
FF22-2	1.40%	0.87%	0.34%	0.25%	0.37%	0.41%	0.17	0.13	0.10	0.10	0.06	0.02
NF02-2	1.80%	1.10%	0.48%	0.33%	0.30%	0.26%	0.19	0.14	0.09	0.09	0.05	0.01
NF05-1	3.70%	3.30%	3.00%	2.80%	2.40%	2.30%	0.20	0.15	0.10	0.10	0.06	0.02
NF05-2	4.70%	4.00%	3.60%	3.40%	3.30%	3.00%	0.21	0.15	0.11	0.11	0.06	0.02
NF16-1	3.80%	3.00%	2.80%	2.70%	3.10%	2.90%	0.19	0.15	0.11	0.11	0.06	0.02
NF16-2	2.00%	1.40%	1.20%	1.20%	1.30%	1.60%	0.18	0.14	0.10	0.10	0.05	0.02
NF17-2	2.90%	2.50%	2.30%	2.10%	1.90%	1.80%	0.20	0.15	0.11	0.10	0.05	0.02
NF21-2	4.80%	4.00%	3.30%	2.70%	2.00%	1.60%	0.19	0.14	0.10	0.10	0.05	0.02
NF22-1	5.20%	4.10%	3.30%	2.40%	1.90%	1.70%	0.17	0.13	0.09	0.10	0.05	0.01
NF25-2	4.50%	4.00%	3.40%	2.80%	2.10%	2.10%	0.20	0.15	0.10	0.10	0.05	0.01
NF27-2	2.10%	1.50%	0.96%	0.81%	0.87%	0.80%	0.20	0.15	0.10	0.10	0.05	0.02
NF28-1	4.80%	4.00%	3.30%	2.70%	2.30%	2.10%	0.20	0.15	0.10	0.10	0.05	0.01

Ground Motion	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	4th Floor Max Normalized Column Axial Force(INT)	5th Floor Max Normalized Column Axial Force(INT)	6th Floor Max Normalized Column Axial Force(INT)	1st Floor Max Normalized BRB Axial Deformation(EXT)	2nd Floor Max Normalized BRB Axial Deformation(EXT)	3rd Floor Max Normalized BRB Axial Deformation(EXT)	4th Floor Max Normalized BRB Axial Deformation(EXT)	5th Floor Max Normalized BRB Axial Deformation(EXT)	6th Floor Max Normalized BRB Axial Deformation(EXT)
FF01-1	0.22	0.17	0.14	0.16	0.10	0.05	29.207	28.436	25.018	24.433	19.636	23.635
FF13-1	0.26	0.20	0.14	0.15	0.09	0.04	13.663	11.105	8.850	8.833	8.459	11.350
FF14-1	0.22	0.18	0.13	0.14	0.08	0.04	9.933	8.777	7.432	8.743	9.609	12.020
FF14-2	0.27	0.21	0.15	0.15	0.10	0.05	26.567	22.373	19.814	17.024	13.045	13.445
FF15-2	0.26	0.20	0.15	0.15	0.09	0.05	20.333	18.691	16.259	15.319	12.991	13.495
FF19-1	0.27	0.20	0.15	0.15	0.08	0.03	24.674	19.423	15.723	13.667	10.691	12.040
FF21-2	0.20	0.16	0.12	0.13	0.08	0.03	17.530	14.268	11.045	9.595	8.968	10.880
FF22-1	0.21	0.16	0.13	0.14	0.09	0.04	14.763	12.891	12.282	10.657	11.486	14.005
FF22-2	0.23	0.18	0.14	0.16	0.09	0.04	11.630	8.977	6.045	6.605	7.745	8.915
NF02-2	0.26	0.20	0.14	0.15	0.08	0.03	27.748	23.477	20.668	16.824	11.964	10.040
NF05-1	0.22	0.18	0.13	0.14	0.08	0.03	23.522	22.236	19.745	20.229	17.264	19.170
NF05-2	0.28	0.21	0.16	0.18	0.11	0.06	28.289	25.964	22.886	22.819	22.036	22.895
NF16-1	0.23	0.18	0.13	0.14	0.08	0.04	23.300	21.891	19.850	20.657	21.032	22.685
NF16-2	0.22	0.17	0.13	0.14	0.08	0.03	15.978	12.150	10.386	11.224	12.182	15.240
NF17-2	0.23	0.18	0.13	0.14	0.08	0.03	17.167	16.895	17.314	16.657	14.668	15.995
NF21-2	0.25	0.20	0.14	0.16	0.10	0.04	28.137	25.159	21.614	19.329	14.127	13.410
NF22-1	0.20	0.15	0.11	0.12	0.07	0.03	30.874	24.805	18.877	18.100	14.336	15.520
NF25-2	0.26	0.20	0.14	0.14	0.08	0.03	26.841	24.964	21.659	19.381	15.932	18.165
NF27-2	0.27	0.20	0.16	0.17	0.10	0.04	21.870	19.455	14.300	13.048	9.759	10.800
NF28-1	0.26	0.20	0.14	0.14	0.08	0.03	28.307	25.623	21.727	18.486	16.436	18.240

Ground Motion	1st Floor Max BRB Axial Deformation(INT)	2nd Floor Max BRB Axial Deformation(INT)	3rd Floor Max BRB Axial Deformation(INT)	4th Floor Max BRB Axial Deformation(INT)	5th Floor Max BRB Axial Deformation(INT)	6th Floor Max BRB Axial Deformation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)
FF01-1	17.044	15.086	13.755	11.200	10.855	12.340	6.811	6.581	6.142	6.883	7.008	8.492
FF13-1	17.119	17.532	15.945	15.643	14.600	15.790	6.169	5.864	5.722	5.783	6.306	7.508
FF14-1	12.081	10.532	9.118	8.914	8.700	11.635	5.992	5.239	4.822	5.292	5.578	6.372
FF14-2	35.037	32.064	25.805	23.671	20.459	23.140	4.483	4.975	5.094	5.800	5.714	7.106
FF15-2	22.048	18.886	15.850	13.329	10.727	11.730	6.819	7.608	6.572	6.561	6.344	6.903
FF19-1	30.830	29.764	26.768	25.833	21.309	24.535	5.217	5.533	5.656	6.533	6.144	6.439
FF21-2	8.378	5.309	4.123	5.667	7.464	9.625	5.292	5.519	4.414	4.847	5.183	6.103
FF22-1	8.656	6.691	4.391	4.781	6.545	11.125	5.572	5.525	5.222	4.517	5.425	6.672
FF22-2	18.248	13.200	11.859	11.971	14.023	14.605	7.836	5.411	5.383	5.581	6.794	6.961
NF02-2	35.878	32.841	27.473	24.681	20.309	20.725	5.003	4.733	4.042	4.747	4.917	5.136
NF05-1	13.519	12.773	11.559	9.595	7.964	7.650	6.503	5.764	4.892	5.378	5.639	5.758
NF05-2	18.719	16.277	14.273	14.343	12.605	12.675	5.231	5.556	5.553	4.914	5.728	6.972
NF16-1	17.919	15.550	12.559	9.662	11.555	10.950	5.928	6.381	5.836	6.008	5.997	7.347
NF16-2	17.041	14.609	12.027	11.490	10.668	11.590	6.131	6.175	6.089	6.472	6.167	6.681
NF17-2	19.481	16.927	13.809	11.443	9.295	9.315	5.042	5.408	5.583	6.158	6.094	6.267
NF21-2	21.463	16.841	13.123	10.376	12.650	15.520	5.661	5.117	5.758	5.531	6.025	6.803
NF22-1	22.411	18.595	15.173	8.319	4.900	5.040	4.269	4.414	4.175	5.478	5.256	5.694
NF25-2	18.711	16.668	13.995	11.181	9.959	10.140	4.881	5.058	4.644	4.931	5.747	6.089
NF27-2	34.033	28.718	25.382	20.538	20.168	21.660	7.350	6.714	6.561	6.281	6.231	7.156
NF28-1	22.011	18.982	15.545	13.529	13.282	13.840	4.261	4.483	4.667	5.294	5.964	6.492

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	15.167	15.069	15.133	15.189	15.208	15.500	6.413	3.146	2.974	3.498	3.037	3.622
FF13-1	9.814	10.083	10.075	10.069	10.069	12.353	10.107	6.067	4.327	6.674	5.349	6.457
FF14-1	10.764	10.478	10.586	10.675	10.733	12.242	5.796	3.256	5.476	4.976	3.902	4.371
FF14-2	13.936	13.494	13.644	13.767	13.842	16.167	7.371	6.594	9.748	8.966	5.955	5.716
FF15-2	14.733	14.606	14.644	14.636	14.594	19.581	8.227	4.691	6.017	4.731	5.010	7.530
FF19-1	8.042	8.114	8.150	8.189	8.219	8.564	1.699	1.203	1.404	1.254	1.562	1.643
FF21-2	7.303	7.353	7.403	7.433	7.453	8.297	6.192	3.689	3.505	4.604	3.775	3.408
FF22-1	10.881	10.933	11.011	11.083	11.158	14.497	6.649	4.144	3.674	4.079	4.333	4.630
FF22-2	11.081	11.139	11.214	11.281	11.336	14.783	7.002	3.047	3.746	3.738	4.266	3.673
NF02-2	7.300	7.381	7.417	7.456	7.489	9.033	2.172	1.984	2.212	2.203	2.918	2.375
NF05-1	8.289	8.083	8.133	8.203	8.253	9.500	4.755	4.470	4.464	4.007	3.623	3.486
NF05-2	17.094	16.336	16.511	16.639	16.667	25.592	5.391	4.074	5.146	6.435	5.841	6.792
NF16-1	9.525	9.364	9.444	9.511	9.553	9.986	6.401	4.589	3.817	3.656	3.949	5.167
NF16-2	8.750	8.617	8.678	8.742	8.794	9.114	4.886	3.014	3.635	3.119	4.846	3.537
NF17-2	8.525	8.547	8.614	8.675	8.719	9.608	7.122	2.791	2.825	3.265	4.168	4.566
NF21-2	16.922	16.939	17.042	17.131	17.186	16.017	6.512	4.157	3.815	5.257	3.459	4.463
NF22-1	8.417	8.269	8.364	8.422	8.478	9.764	4.832	4.192	5.713	4.865	2.790	2.998
NF25-2	9.867	9.633	9.717	9.794	9.836	9.736	2.162	1.711	1.587	2.262	1.905	2.013
NF27-2	12.042	11.556	11.692	11.814	11.881	15.883	3.542	2.177	2.997	3.407	2.676	2.578
NF28-1	8.553	8.378	8.461	8.542	8.600	12.669	2.478	2.951	3.511	2.216	3.309	3.472

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	4.916	4.874	4.969	5.069	5.149	4.009	0.46%	0.51%	0.62%	0.67%	0.67%	0.69%	0.457
FF13-1	2.450	2.537	2.625	2.744	2.823	4.498	0.73%	0.78%	0.84%	0.85%	0.82%	0.77%	0.477
FF14-1	2.955	2.959	3.107	3.225	3.327	3.606	0.04%	0.01%	0.00%	0.02%	0.04%	0.01%	0.420
FF14-2	4.763	4.806	4.935	5.111	5.447	5.356	0.11%	0.09%	0.01%	0.05%	0.11%	0.12%	0.427
FF15-2	5.294	5.315	5.267	5.199	5.155	6.966	0.45%	0.50%	0.47%	0.36%	0.15%	0.09%	0.463
FF19-1	1.357	1.425	1.448	1.471	1.498	1.131	0.48%	0.69%	0.95%	1.10%	1.20%	1.30%	0.404
FF21-2	1.260	1.272	1.460	1.656	1.815	2.014	0.55%	0.55%	0.47%	0.33%	0.19%	0.13%	0.406
FF22-1	3.159	3.083	3.226	3.426	3.586	4.037	0.98%	0.88%	0.71%	0.63%	0.60%	0.64%	0.438
FF22-2	3.295	3.254	3.457	3.669	3.826	4.630	0.09%	0.04%	0.19%	0.33%	0.33%	0.22%	0.446
NF02-2	1.356	1.350	1.508	1.658	1.785	2.146	0.91%	1.10%	1.20%	1.30%	1.50%	1.50%	0.387
NF05-1	1.860	1.839	1.952	2.055	2.148	2.131	0.51%	0.64%	0.73%	0.84%	0.92%	0.99%	0.409
NF05-2	7.428	6.904	7.330	7.760	8.139	8.885	0.25%	0.33%	0.27%	0.18%	0.05%	0.08%	0.448
NF16-1	3.304	3.814	4.226	4.622	4.973	3.800	1.10%	1.10%	1.20%	1.20%	1.20%	1.20%	0.465
NF16-2	2.418	2.779	3.032	3.285	3.513	2.529	0.10%	0.07%	0.05%	0.02%	0.08%	0.06%	0.429
NF17-2	1.597	1.840	1.866	1.923	1.964	2.173	0.59%	0.64%	0.54%	0.38%	0.20%	0.13%	0.418
NF21-2	7.138	7.439	7.519	7.609	7.686	4.634	0.09%	0.11%	0.05%	0.01%	0.04%	0.10%	0.469
NF22-1	1.984	2.134	2.317	2.496	2.658	2.546	1.80%	1.90%	1.80%	1.60%	1.20%	1.00%	0.327
NF25-2	2.350	2.226	2.296	2.380	2.442	2.129	0.10%	0.10%	0.11%	0.15%	0.24%	0.21%	0.399
NF27-2	3.348	3.183	3.262	3.331	3.379	4.737	0.07%	0.15%	0.31%	0.44%	0.49%	0.51%	0.442
NF28-1	1.769	1.653	1.729	1.819	1.874	3.068	0.14%	0.12%	0.07%	0.23%	0.31%	0.27%	0.440

Ground Motion	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	4th Floor BRB Energy Dissipation(EXT)	5th Floor BRB Energy Dissipation(EXT)	6th Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)	4th Floor BRB Energy Dissipation(INT)	5th Floor BRB Energy Dissipation(INT)	6th Floor BRB Energy Dissipation(INT)
FF01-1	505.01	257.68	193.24	200.28	167.15	120.81	327.77	239.19	168.64	125.02	62.60	79.82
FF13-1	322.97	204.37	170.25	139.14	92.18	61.34	211.72	143.57	113.16	98.59	70.69	50.87
FF14-1	106.32	67.77	45.13	44.35	34.68	32.65	197.47	142.46	113.76	111.29	86.48	78.66
FF14-2	451.86	294.54	226.01	161.60	64.18	69.22	979.50	551.03	409.81	324.35	207.52	124.71
FF15-2	744.74	514.49	376.81	281.23	113.89	67.97	146.16	92.45	71.25	67.28	54.07	44.16
FF19-1	1233.00	680.81	399.50	239.24	129.66	101.75	298.06	213.05	155.12	119.50	88.20	62.90
FF21-2	98.16	87.14	55.44	44.10	42.38	44.33	347.89	164.51	87.01	64.62	51.98	47.70
FF22-1	300.86	195.51	142.92	123.08	94.66	91.99	175.78	92.23	60.22	54.14	31.51	23.64
FF22-2	453.53	265.01	125.56	103.39	94.40	76.44	137.56	68.66	73.47	87.08	68.35	44.84
NF02-2	883.49	463.39	361.64	214.07	94.26	47.33	377.80	240.96	181.78	158.81	113.82	82.47
NF05-1	362.69	243.28	191.20	171.68	130.22	100.39	358.98	225.61	159.21	99.39	66.12	44.11
NF05-2	265.37	145.48	97.61	51.75	27.91	25.49	1010.00	735.30	527.74	432.94	274.40	211.68
NF16-1	380.94	261.64	226.63	190.61	144.65	109.60	435.62	209.54	131.22	99.69	92.22	56.96
NF16-2	212.24	129.70	100.18	87.46	81.98	65.84	373.79	213.71	139.45	115.74	69.78	69.19
NF17-2	640.74	368.86	280.20	209.55	124.16	73.03	173.37	150.44	86.58	66.42	29.55	35.76
NF21-2	366.73	244.46	181.78	131.12	103.34	71.11	688.93	376.92	295.77	184.04	126.72	85.67
NF22-1	274.48	229.09	205.17	182.14	89.00	65.50	419.30	190.51	114.62	41.62	34.01	24.98
NF25-2	202.53	145.76	95.87	57.62	44.42	32.23	999.44	692.96	529.07	359.41	178.61	123.05
NF27-2	719.69	444.21	240.64	179.69	107.45	64.78	455.60	211.07	154.06	131.22	123.50	89.31
NF28-1	581.31	309.00	242.85	198.48	130.55	68.46	785.57	502.61	311.62	240.69	142.28	98.06

Table B-15: Collected Result for 6 story Braced Frame for Each Earthquake-- Single Diagonal --Horizontal Only

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	4th Floor Max Normalized Column Axial Force(EXT)	5th Floor Max Normalized Column Axial Force(EXT)	6th Floor Max Normalized Column Axial Force(EXT)
FF01-1	4.40%	3.90%	3.80%	3.40%	3.00%	3.80%	0.22	0.13	0.13	0.09	0.08	0.01
FF13-1	1.80%	1.60%	1.30%	1.10%	1.00%	0.87%	0.20	0.12	0.11	0.09	0.07	0.01
FF14-1	0.94%	0.84%	0.76%	0.89%	1.30%	1.50%	0.19	0.11	0.11	0.08	0.07	0.01
FF14-2	2.60%	2.10%	1.40%	1.10%	0.95%	0.89%	0.23	0.14	0.13	0.10	0.08	0.01
FF15-2	2.90%	2.50%	2.20%	1.90%	1.60%	1.80%	0.23	0.13	0.12	0.09	0.08	0.01
FF19-1	4.20%	3.60%	3.00%	2.40%	1.70%	1.50%	0.23	0.13	0.12	0.09	0.08	0.01
FF21-2	3.40%	2.80%	2.20%	1.70%	1.10%	1.10%	0.21	0.12	0.11	0.08	0.07	0.01
FF22-1	2.60%	1.90%	1.60%	1.40%	1.80%	2.10%	0.19	0.11	0.11	0.09	0.08	0.01
FF22-2	1.30%	0.73%	0.59%	0.30%	0.56%	0.78%	0.18	0.11	0.10	0.08	0.07	0.01
NF02-2	0.58%	0.49%	0.45%	0.44%	0.46%	0.51%	0.21	0.12	0.11	0.08	0.07	0.01
NF05-1	3.80%	3.60%	3.50%	3.30%	3.40%	3.80%	0.22	0.13	0.12	0.09	0.08	0.01
NF05-2	5.60%	5.10%	4.40%	4.00%	4.00%	3.70%	0.23	0.14	0.13	0.09	0.08	0.01
NF16-1	3.90%	3.50%	2.90%	2.70%	2.90%	3.40%	0.22	0.13	0.12	0.09	0.08	0.01
NF16-2	2.10%	2.00%	1.60%	1.50%	1.70%	1.90%	0.20	0.12	0.11	0.09	0.07	0.01
NF17-2	3.30%	2.80%	2.80%	2.80%	3.00%	3.20%	0.23	0.14	0.13	0.09	0.08	0.01
NF21-2	5.20%	4.30%	3.80%	3.40%	3.20%	3.00%	0.22	0.12	0.12	0.09	0.08	0.01
NF22-1	4.80%	3.90%	3.40%	3.00%	2.60%	2.60%	0.21	0.13	0.12	0.09	0.08	0.01
NF25-2	4.00%	3.90%	3.70%	3.30%	2.70%	3.10%	0.23	0.13	0.12	0.09	0.08	0.01
NF27-2	1.20%	0.87%	0.77%	0.83%	0.81%	1.30%	0.22	0.13	0.13	0.09	0.08	0.01
NF28-1	5.00%	4.20%	3.60%	3.00%	2.50%	2.90%	0.23	0.13	0.12	0.09	0.08	0.01

Ground Motion	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	4th Floor Max Normalized Column Axial Force(INT)	5th Floor Max Normalized Column Axial Force(INT)	6th Floor Max Normalized Column Axial Force(INT)	1st Floor BRB Max Normalized Axial Deformation (EXT)	2nd Floor BRB Max Normalized Axial Deformation (EXT)	3rd Floor BRB Max Normalized Axial Deformation (EXT)	4th Floor BRB Max Normalized Axial Deformation (EXT)	5th Floor BRB Max Normalized Axial Deformation (EXT)	6th Floor BRB Max Normalized Axial Deformation (EXT)
FF01-1	0.20	0.19	0.12	0.17	0.07	0.05	18.651	14.616	14.470	12.808	11.139	14.255
FF13-1	0.21	0.20	0.12	0.17	0.07	0.05	9.560	7.903	8.724	8.822	8.947	10.708
FF14-1	0.20	0.18	0.12	0.16	0.07	0.05	7.723	4.743	3.292	3.414	4.863	5.739
FF14-2	0.21	0.20	0.12	0.17	0.07	0.05	27.435	22.357	19.638	16.051	13.187	15.361
FF15-2	0.21	0.20	0.12	0.16	0.07	0.05	13.821	10.324	8.835	7.395	6.058	6.471
FF19-1	0.23	0.22	0.12	0.17	0.07	0.05	18.847	15.014	13.362	11.535	10.784	13.058
FF21-2	0.18	0.17	0.11	0.15	0.07	0.04	14.272	10.619	8.070	6.405	4.061	4.832
FF22-1	0.18	0.16	0.11	0.15	0.07	0.05	11.077	7.292	5.822	5.297	6.437	7.529
FF22-2	0.19	0.17	0.11	0.15	0.07	0.05	9.458	6.592	5.684	4.695	5.587	6.616
NF02-2	0.23	0.22	0.12	0.17	0.07	0.05	29.181	23.192	20.686	19.114	15.739	14.868
NF05-1	0.21	0.20	0.12	0.17	0.07	0.05	16.160	13.514	13.303	12.386	12.584	14.205
NF05-2	0.23	0.21	0.12	0.18	0.07	0.05	23.842	19.503	16.778	15.008	14.750	13.963
NF16-1	0.21	0.19	0.12	0.16	0.07	0.05	16.558	13.127	11.038	10.057	10.829	12.755
NF16-2	0.21	0.19	0.12	0.16	0.07	0.05	8.763	7.541	6.049	5.541	6.276	7.189
NF17-2	0.22	0.20	0.12	0.17	0.07	0.05	13.809	10.503	10.714	10.659	10.942	11.863
NF21-2	0.22	0.20	0.12	0.17	0.07	0.05	22.293	16.489	14.522	13.014	11.929	11.421
NF22-1	0.19	0.17	0.10	0.14	0.07	0.04	20.414	14.803	12.714	11.105	9.650	9.811
NF25-2	0.22	0.21	0.12	0.17	0.07	0.05	17.119	14.935	14.176	12.565	10.034	11.311
NF27-2	0.22	0.21	0.13	0.18	0.07	0.05	25.595	20.049	17.289	14.732	14.766	13.900
NF28-1	0.23	0.22	0.13	0.18	0.07	0.05	21.523	16.149	13.595	11.343	9.276	10.753

Ground Motion	1st Floor BRB Max Normalized Axial Deformation (INT)	2nd Floor BRB Max Normalized Axial Deformation (INT)	3rd Floor BRB Max Normalized Axial Deformation (INT)	4th Floor BRB Max Normalized Axial Deformation (INT)	5th Floor BRB Max Normalized Axial Deformation (INT)	6th Floor BRB Max Normalized Axial Deformation (INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)
FF01-1	18.530	14.830	14.414	13.097	11.111	14.284	6.217	5.956	5.928	5.875	5.853	5.911
FF13-1	9.679	7.773	8.727	8.876	9.068	11.150	6.281	5.986	5.961	5.892	5.881	5.956
FF14-1	7.774	4.889	3.359	3.624	4.771	5.889	6.261	5.961	5.939	5.878	5.861	5.922
FF14-2	27.572	22.495	19.895	16.273	13.313	15.700	6.244	5.972	5.936	5.883	5.856	5.933
FF15-2	13.684	10.319	8.905	7.346	6.092	6.489	6.253	5.989	5.956	5.894	5.883	5.944
FF19-1	19.016	14.886	13.419	11.692	10.934	13.468	6.217	5.933	5.914	5.858	5.825	5.897
FF21-2	14.277	10.665	7.951	6.354	3.916	4.737	6.242	5.950	5.944	5.867	5.844	5.919
FF22-1	11.260	7.232	5.849	5.316	6.274	7.645	6.253	5.997	6.008	5.911	5.908	6.000
FF22-2	9.549	6.386	5.838	4.957	6.179	6.837	6.219	5.931	5.911	5.861	5.836	5.903
NF02-2	29.435	23.116	20.778	19.251	15.858	15.258	6.194	5.936	5.908	5.856	5.833	5.892
NF05-1	16.165	13.659	13.254	12.405	12.505	14.408	6.211	5.933	5.897	5.861	5.847	5.892
NF05-2	23.688	19.732	16.949	15.086	14.632	14.274	6.214	5.936	5.900	5.861	5.822	5.894
NF16-1	16.281	13.273	11.476	10.086	11.053	12.884	6.267	5.944	5.928	5.869	5.850	5.917
NF16-2	9.023	7.424	6.273	5.297	6.095	7.392	6.247	5.975	5.969	5.900	5.897	5.964
NF17-2	13.812	10.619	10.273	10.716	10.839	12.066	6.233	5.961	5.919	5.878	5.856	5.919
NF21-2	22.512	16.241	14.700	12.884	11.795	11.642	6.278	5.994	5.961	5.906	5.894	5.961
NF22-1	20.614	14.873	13.057	11.114	9.242	10.358	6.253	5.958	5.931	5.892	5.856	5.931
NF25-2	17.035	14.932	14.114	12.554	9.924	11.439	6.200	5.917	5.883	5.847	5.811	5.878
NF27-2	25.784	19.978	17.646	14.870	15.071	14.453	6.253	5.958	5.931	5.869	5.853	5.908
NF28-1	21.553	16.208	13.549	11.419	9.308	10.734	6.197	5.919	5.894	5.853	5.819	5.881

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)
FF01-1	6.289	5.983	5.944	5.892	5.844	5.869	0.189	0.272	0.216	0.178	0.138	0.107
FF13-1	6.261	5.975	5.936	5.892	5.844	5.872	0.351	0.379	0.325	0.280	0.220	0.192
FF14-1	6.261	5.967	5.933	5.883	5.836	5.864	0.184	0.201	0.176	0.138	0.123	0.096
FF14-2	6.258	5.972	5.939	5.889	5.842	5.867	0.196	0.257	0.223	0.171	0.153	0.112
FF15-2	6.269	5.989	5.944	5.900	5.858	5.878	0.228	0.303	0.256	0.195	0.169	0.112
FF19-1	6.261	5.958	5.922	5.872	5.828	5.861	0.094	0.129	0.107	0.083	0.067	0.058
FF21-2	6.267	5.967	5.933	5.883	5.836	5.864	0.147	0.143	0.136	0.095	0.082	0.065
FF22-1	6.281	5.983	5.947	5.903	5.858	5.878	0.253	0.346	0.303	0.235	0.208	0.165
FF22-2	6.328	5.994	5.956	5.903	5.850	5.872	0.291	0.265	0.234	0.181	0.134	0.136
NF02-2	6.250	5.956	5.917	5.872	5.825	5.858	0.094	0.123	0.101	0.081	0.072	0.050
NF05-1	6.253	5.964	5.925	5.883	5.839	5.867	0.159	0.150	0.116	0.094	0.079	0.077
NF05-2	6.275	5.981	5.942	5.889	5.836	5.867	0.135	0.141	0.127	0.101	0.084	0.065
NF16-1	6.286	5.992	5.950	5.897	5.842	5.867	0.234	0.219	0.195	0.136	0.107	0.092
NF16-2	6.333	5.983	5.947	5.903	5.864	5.883	0.282	0.223	0.217	0.169	0.152	0.134
NF17-2	6.306	5.994	5.958	5.900	5.847	5.872	0.194	0.367	0.300	0.234	0.186	0.116
NF21-2	6.300	6.003	5.967	5.917	5.867	5.883	0.200	0.260	0.213	0.176	0.155	0.140
NF22-1	6.253	5.978	5.931	5.886	5.842	5.867	0.105	0.142	0.127	0.104	0.082	0.070
NF25-2	6.247	5.953	5.917	5.869	5.822	5.858	0.099	0.114	0.096	0.074	0.059	0.041
NF27-2	6.286	5.986	5.947	5.897	5.847	5.869	0.205	0.258	0.216	0.168	0.141	0.124
NF28-1	6.242	5.958	5.922	5.875	5.825	5.858	0.107	0.153	0.130	0.094	0.079	0.066

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift	Roof Horizontal Acceleration (g)
FF01-1	0.173	0.263	0.224	0.176	0.132	0.106	1.30%	1.30%	1.40%	1.30%	1.20%	1.20%	0.633
FF13-1	0.316	0.570	0.460	0.360	0.268	0.166	1.10%	1.00%	0.99%	1.10%	1.20%	1.20%	0.614
FF14-1	0.240	0.265	0.220	0.168	0.121	0.081	0.40%	0.29%	0.19%	0.14%	0.17%	0.24%	0.639
FF14-2	0.219	0.224	0.198	0.161	0.129	0.068	1.20%	1.30%	1.40%	1.50%	1.50%	1.50%	0.606
FF15-2	0.251	0.282	0.239	0.191	0.149	0.107	0.65%	0.74%	0.77%	0.72%	0.64%	0.63%	0.55
FF19-1	0.144	0.187	0.162	0.125	0.091	0.058	0.02%	0.23%	0.42%	0.60%	0.77%	0.83%	0.437
FF21-2	0.141	0.150	0.122	0.094	0.073	0.055	1.50%	1.20%	0.86%	0.49%	0.09%	0.12%	0.602
FF22-1	0.251	0.282	0.248	0.199	0.152	0.094	1.40%	1.20%	1.00%	0.88%	0.85%	0.88%	0.557
FF22-2	0.345	0.291	0.250	0.200	0.152	0.099	0.23%	0.19%	0.22%	0.24%	0.17%	0.14%	0.702
NF02-2	0.105	0.209	0.170	0.129	0.093	0.049	2.70%	2.70%	2.70%	2.70%	2.80%	2.80%	0.453
NF05-1	0.219	0.162	0.138	0.108	0.085	0.059	1.50%	1.70%	1.70%	1.70%	1.60%	1.60%	0.593
NF05-2	0.123	0.141	0.121	0.101	0.083	0.060	0.23%	0.17%	0.05%	0.11%	0.20%	0.16%	0.51
NF16-1	0.241	0.425	0.353	0.270	0.194	0.115	1.70%	1.80%	1.70%	1.50%	1.50%	1.60%	0.764
NF16-2	0.356	0.318	0.279	0.213	0.164	0.100	0.43%	0.33%	0.19%	0.02%	0.16%	0.28%	0.6
NF17-2	0.256	0.368	0.309	0.233	0.167	0.096	0.02%	0.16%	0.24%	0.33%	0.31%	0.30%	0.565
NF21-2	0.246	0.294	0.259	0.189	0.128	0.105	1.20%	1.10%	1.00%	1.10%	1.20%	1.30%	0.796
NF22-1	0.123	0.136	0.109	0.087	0.065	0.052	2.30%	2.40%	2.30%	2.00%	1.80%	1.70%	0.494
NF25-2	0.085	0.102	0.089	0.067	0.049	0.036	1.60%	1.50%	1.30%	1.00%	0.71%	0.59%	0.453
NF27-2	0.251	0.240	0.206	0.157	0.117	0.090	1.40%	1.50%	1.50%	1.50%	1.60%	1.60%	0.674
NF28-1	0.076	0.121	0.100	0.077	0.056	0.036	0.05%	0.20%	0.32%	0.27%	0.24%	0.18%	0.413

Ground Motion	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	4th Floor BRB Energy Dissipation(EXT)	5th Floor BRB Energy Dissipation(EXT)	6th Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)	4th Floor BRB Energy Dissipation(INT)	5th Floor BRB Energy Dissipation(INT)	6th Floor BRB Energy Dissipation(INT)
FF01-1	870.51	771.66	335.59	458.11	158.84	276.04	1248.00	454.10	668.36	227.12	333.47	164.22
FF13-1	628.13	233.94	303.98	201.85	181.85	141.62	339.95	355.10	257.90	252.35	160.86	121.31
FF14-1	294.84	166.33	120.32	100.90	145.44	61.63	343.86	170.01	106.44	107.52	86.06	134.84
FF14-2	1197.00	1065.00	546.70	411.96	287.28	198.00	1825.00	720.82	768.22	421.49	230.98	203.57
FF15-2	193.99	1148.00	70.04	643.82	33.28	255.52	1882.00	87.29	930.64	50.58	376.76	28.08
FF19-1	1500.00	740.61	745.81	329.56	325.19	182.67	1429.00	961.95	535.81	541.16	206.99	210.55
FF21-2	189.92	436.57	91.10	173.94	41.51	118.90	740.74	107.14	298.51	61.49	145.24	34.70
FF22-1	285.56	234.04	82.29	108.99	92.02	112.26	401.53	116.76	145.75	76.59	132.02	83.01
FF22-2	397.50	264.85	127.40	109.54	72.29	96.68	515.65	160.50	186.83	86.20	129.87	63.15
NF02-2	951.96	712.37	553.13	344.93	361.36	84.34	1311.00	620.05	517.30	470.84	146.85	235.19
NF05-1	775.66	509.72	347.46	281.50	245.36	147.20	681.67	411.73	424.95	274.88	179.00	205.65
NF05-2	907.28	525.40	410.19	218.46	163.74	140.38	1171.00	549.36	343.42	217.70	174.43	135.94
NF16-1	760.35	731.65	254.09	291.74	200.64	145.00	973.47	346.37	507.28	229.08	192.81	165.60
NF16-2	426.26	351.46	215.72	171.62	168.44	67.27	606.24	279.37	235.00	180.33	114.28	145.65
NF17-2	1550.00	238.17	757.84	137.53	476.73	40.60	407.07	809.80	169.23	578.21	54.67	346.42
NF21-2	1634.00	244.18	682.68	161.93	286.28	128.15	426.98	844.27	180.63	483.92	150.02	190.44
NF22-1	821.42	421.41	172.84	189.82	80.13	106.05	490.59	270.02	340.56	125.91	143.64	76.29
NF25-2	869.45	467.12	474.23	244.54	206.57	126.30	749.26	543.91	357.09	339.83	158.09	148.08
NF27-2	857.40	750.03	449.57	340.27	297.67	163.67	1360.00	519.71	559.44	384.65	267.89	250.14
NF28-1	1898.00	708.40	594.29	356.44	239.57	150.53	1001.00	888.74	579.35	436.05	248.49	212.87

Table B-16: Collected Result for 6 story Braced Frame for Each Earthquake-- Single Diagonal --Horizontal + Vertical

Ground Motion	1st Floor Max Drift	2nd Floor Max Drift	3rd Floor Max Drift	4th Floor Max Drift	5th Floor Max Drift	6th Floor Max Drift	1st Floor Max Normalized Column Axial Force(EXT)	2nd Floor Max Normalized Column Axial Force(EXT)	3rd Floor Max Normalized Column Axial Force(EXT)	4th Floor Max Normalized Column Axial Force(EXT)	5th Floor Max Normalized Column Axial Force(EXT)	6th Floor Max Normalized Column Axial Force(EXT)
FF01-1	4.40%	3.90%	3.80%	3.40%	3.00%	3.80%	0.23	0.14	0.13	0.11	0.09	0.02
FF13-1	1.80%	1.50%	1.30%	1.10%	1.00%	0.90%	0.21	0.13	0.12	0.10	0.08	0.02
FF14-1	0.94%	0.84%	0.75%	0.89%	1.30%	1.50%	0.19	0.12	0.11	0.10	0.08	0.02
FF14-2	2.70%	2.20%	1.40%	1.10%	0.93%	0.87%	0.24	0.16	0.14	0.11	0.09	0.02
FF15-2	2.80%	2.50%	2.10%	1.90%	1.60%	1.80%	0.24	0.14	0.13	0.11	0.09	0.02
FF19-1	4.20%	3.60%	3.00%	2.40%	1.70%	1.50%	0.23	0.13	0.12	0.09	0.08	0.02
FF21-2	3.40%	2.80%	2.20%	1.70%	1.10%	1.10%	0.21	0.12	0.11	0.09	0.07	0.02
FF22-1	2.60%	1.90%	1.60%	1.40%	1.80%	2.10%	0.19	0.12	0.11	0.09	0.08	0.02
FF22-2	1.20%	0.61%	0.37%	0.22%	0.38%	0.60%	0.19	0.12	0.11	0.10	0.08	0.02
NF02-2	0.58%	0.49%	0.45%	0.44%	0.46%	0.51%	0.21	0.12	0.11	0.09	0.07	0.02
NF05-1	3.80%	3.60%	3.50%	3.30%	3.40%	3.80%	0.22	0.13	0.12	0.09	0.08	0.02
NF05-2	5.60%	5.10%	4.40%	4.00%	4.00%	3.70%	0.24	0.14	0.14	0.10	0.09	0.02
NF16-1	3.90%	3.50%	2.90%	2.60%	2.90%	3.40%	0.21	0.13	0.12	0.10	0.08	0.02
NF16-2	2.00%	2.00%	1.60%	1.40%	1.70%	1.90%	0.21	0.12	0.12	0.09	0.08	0.02
NF17-2	3.30%	2.80%	2.80%	2.80%	3.00%	3.20%	0.24	0.13	0.13	0.10	0.08	0.02
NF21-2	5.20%	4.30%	3.80%	3.40%	3.20%	3.00%	0.22	0.13	0.12	0.10	0.08	0.02
NF22-1	4.80%	3.90%	3.40%	3.00%	2.60%	2.60%	0.21	0.13	0.12	0.10	0.08	0.02
NF25-2	4.00%	3.90%	3.70%	3.30%	2.70%	3.10%	0.23	0.13	0.12	0.09	0.08	0.02
NF27-2	1.30%	0.87%	0.77%	0.83%	0.81%	1.30%	0.23	0.15	0.14	0.11	0.09	0.02
NF28-1	5.10%	4.20%	3.60%	3.00%	2.50%	2.90%	0.23	0.13	0.12	0.09	0.08	0.02

Ground Motion	1st Floor Max Normalized Column Axial Force(INT)	2nd Floor Max Normalized Column Axial Force(INT)	3rd Floor Max Normalized Column Axial Force(INT)	4th Floor Max Normalized Column Axial Force(INT)	5th Floor Max Normalized Column Axial Force(INT)	6th Floor Max Normalized Column Axial Force(INT)	1st Floor Max Normalized BRB Axial Deformation(EXT)	2nd Floor Max Normalized BRB Axial Deformation(EXT)	3rd Floor Max Normalized BRB Axial Deformation(EXT)	4th Floor Max Normalized BRB Axial Deformation(EXT)	5th Floor Max Normalized BRB Axial Deformation(EXT)	6th Floor Max Normalized BRB Axial Deformation(EXT)
FF01-1	0.22	0.20	0.14	0.18	0.11	0.07	18.721	14.686	14.503	12.811	11.184	14.134
FF13-1	0.24	0.22	0.14	0.18	0.09	0.06	9.574	7.895	8.684	8.711	8.876	10.621
FF14-1	0.22	0.20	0.13	0.17	0.09	0.06	7.728	4.735	3.284	3.405	4.866	5.763
FF14-2	0.26	0.24	0.14	0.19	0.10	0.06	27.356	22.259	19.570	16.022	13.087	15.226
FF15-2	0.28	0.25	0.16	0.22	0.11	0.07	13.902	10.357	8.868	7.486	6.116	6.421
FF19-1	0.24	0.23	0.13	0.18	0.08	0.05	18.798	14.970	13.327	11.511	10.779	13.037
FF21-2	0.19	0.17	0.12	0.16	0.08	0.05	14.300	10.641	8.086	6.419	4.074	4.805
FF22-1	0.21	0.19	0.13	0.17	0.10	0.07	11.074	7.292	5.868	5.292	6.474	7.574
FF22-2	0.22	0.20	0.14	0.19	0.09	0.06	10.340	7.705	6.827	5.786	6.234	7.447
NF02-2	0.24	0.23	0.14	0.19	0.08	0.05	29.230	23.219	20.697	19.119	15.732	14.861
NF05-1	0.22	0.20	0.13	0.18	0.09	0.05	16.133	13.503	13.295	12.365	12.550	14.216
NF05-2	0.24	0.23	0.14	0.19	0.09	0.06	23.851	19.519	16.776	14.997	14.734	13.976
NF16-1	0.23	0.21	0.13	0.18	0.09	0.06	16.563	13.132	11.038	10.043	10.821	12.753
NF16-2	0.21	0.19	0.12	0.17	0.09	0.05	8.602	7.411	5.914	5.408	6.153	7.068
NF17-2	0.22	0.20	0.12	0.17	0.08	0.05	13.784	10.486	10.697	10.643	10.939	11.839
NF21-2	0.24	0.21	0.14	0.19	0.11	0.07	22.216	16.411	14.427	12.965	11.811	11.326
NF22-1	0.20	0.18	0.12	0.16	0.08	0.05	20.374	14.781	12.719	11.159	9.671	9.821
NF25-2	0.24	0.22	0.13	0.18	0.08	0.06	17.133	14.951	14.181	12.565	10.037	11.308
NF27-2	0.25	0.23	0.15	0.21	0.10	0.07	25.537	20.000	17.219	14.657	14.661	13.818
NF28-1	0.23	0.22	0.13	0.17	0.09	0.06	21.530	16.170	13.603	11.346	9.279	10.739

Ground Motion	1st Floor Max Normalized BRB Axial Deformation(INT)	2nd Floor Max Normalized BRB Axial Deformation(INT)	3rd Floor Max Normalized BRB Axial Deformation(INT)	4th Floor Max Normalized BRB Axial Deformation(INT)	5th Floor Max Normalized BRB Axial Deformation(INT)	6th Floor Max Normalized BRB Axial Deformation(INT)	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(EXT)
FF01-1	18.626	14.854	14.427	13.092	11.118	14.103	33.242	23.039	23.050	22.886	22.767	25.078
FF13-1	9.686	7.789	8.668	8.759	9.003	11.055	13.769	9.889	9.897	9.933	10.017	9.811
FF14-1	7.765	4.897	3.359	3.611	4.774	5.892	12.817	10.519	10.489	10.419	10.325	10.675
FF14-2	27.481	22.430	19.816	16.138	13.245	15.516	18.256	17.975	17.744	17.264	16.714	19.289
FF15-2	13.753	10.373	8.965	7.403	6.116	6.437	27.847	21.078	21.178	21.322	21.325	22.058
FF19-1	18.970	14.851	13.386	11.673	10.932	13.453	8.481	7.956	7.964	7.958	7.947	7.922
FF21-2	14.307	10.686	7.968	6.368	3.929	4.726	8.250	7.286	7.225	7.222	7.211	7.225
FF22-1	11.258	7.230	5.892	5.316	6.276	7.674	21.550	10.903	10.892	10.831	10.828	10.836
FF22-2	10.602	7.446	6.803	6.141	6.863	7.637	21.861	11.111	11.078	11.072	11.058	11.061
NF02-2	29.488	23.151	20.792	19.262	15.839	15.247	9.050	7.236	7.225	7.211	7.228	7.264
NF05-1	16.142	13.643	13.246	12.408	12.479	14.411	9.406	8.156	8.128	8.064	8.033	8.222
NF05-2	23.693	19.741	16.946	15.095	14.645	14.276	9.939	8.556	8.497	8.453	8.367	8.633
NF16-1	16.279	13.270	11.478	10.089	11.053	12.866	9.942	9.369	9.319	9.317	9.286	9.431
NF16-2	8.851	7.273	6.149	5.192	5.979	7.268	9.186	8.708	8.686	8.631	8.611	8.750
NF17-2	13.809	10.605	10.265	10.700	10.824	12.055	9.483	8.378	8.356	8.336	8.317	8.422
NF21-2	22.451	16.127	14.592	12.830	11.695	11.542	25.822	22.214	22.256	22.336	22.300	22.647
NF22-1	20.560	14.873	13.068	11.149	9.271	10.363	9.611	8.331	8.281	8.267	8.217	8.392
NF25-2	17.060	14.949	14.116	12.530	9.916	11.445	9.692	9.700	9.678	9.631	9.556	9.883
NF27-2	25.751	19.914	17.562	14.789	15.005	14.387	39.625	12.208	12.144	11.992	11.839	12.556
NF28-1	21.565	16.224	13.549	11.403	9.305	10.742	13.633	8.572	8.517	8.439	8.367	8.550

Ground Motion	1st Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	2nd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	3rd Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	4th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	5th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	6th Floor Beam Max Normalized Deflection in the Midspan (10^-3)(INT)	1st Floor Vertical Acceleration(EXT)	2nd Floor Vertical Acceleration(EXT)	3rd Floor Vertical Acceleration(EXT)	4th Floor Vertical Acceleration(EXT)	5th Floor Vertical Acceleration(EXT)	6th Floor Vertical Acceleration(EXT)	Roof Horizontal Acceleration (g)
FF01-1	15.828	15.097	15.108	15.086	15.039	15.172	3.823	4.426	4.526	4.560	4.595	4.210	0.621
FF13-1	12.344	9.639	9.692	9.758	9.828	9.669	4.240	3.152	2.944	2.636	2.643	2.404	0.616
FF14-1	12.467	10.764	10.706	10.581	10.450	10.756	3.377	2.953	2.886	2.773	2.647	2.780	0.632
FF14-2	16.133	13.769	13.703	13.547	13.392	13.883	4.891	4.804	4.451	4.045	3.879	4.261	0.600
FF15-2	19.694	14.308	14.317	14.272	14.206	14.586	5.112	7.144	6.921	6.629	6.343	6.667	0.546
FF19-1	8.650	8.033	8.022	8.017	8.008	7.983	1.062	1.227	1.239	1.258	1.267	1.224	0.437
FF21-2	8.425	7.381	7.339	7.294	7.275	7.275	2.137	1.472	1.352	1.230	1.137	1.043	0.602
FF22-1	14.847	10.922	10.878	10.828	10.781	10.772	4.007	3.098	3.032	2.938	2.998	2.907	0.558
FF22-2	14.739	11.139	11.086	11.039	10.989	10.975	4.409	3.361	3.243	3.154	3.253	3.193	0.734
NF02-2	9.264	7.269	7.250	7.242	7.239	7.267	2.887	1.744	1.595	1.438	1.294	1.275	0.452
NF05-1	9.564	8.297	8.250	8.175	8.083	8.281	3.082	2.232	2.113	1.964	1.810	1.678	0.591
NF05-2	10.061	8.667	8.619	8.533	8.431	8.653	3.610	2.565	2.420	2.262	2.096	1.944	0.509
NF16-1	10.233	9.444	9.428	9.369	9.297	9.442	3.426	3.641	3.321	3.202	3.046	2.642	0.761
NF16-2	9.322	8.706	8.681	8.636	8.575	8.694	3.143	3.127	2.813	2.615	2.417	2.186	0.597
NF17-2	9.597	8.469	8.428	8.369	8.325	8.408	2.379	1.864	1.699	1.619	1.585	1.502	0.569
NF21-2	15.983	16.844	16.831	16.800	16.756	16.797	5.047	5.842	5.889	5.941	5.977	5.850	0.792
NF22-1	9.925	8.400	8.350	8.297	8.239	8.472	2.604	2.124	2.059	1.999	1.921	1.808	0.493
NF25-2	9.869	9.875	9.794	9.708	9.619	9.933	1.952	2.384	2.353	2.291	2.255	2.392	0.451
NF27-2	16.383	11.958	11.883	11.747	11.597	12.006	4.368	3.865	3.742	3.783	3.804	3.744	0.676
NF28-1	13.036	8.692	8.619	8.525	8.422	8.583	3.243	1.762	1.704	1.631	1.562	1.676	0.416

Ground Motion	1st Floor Vertical Acceleration(INT)	2nd Floor Vertical Acceleration(INT)	3rd Floor Vertical Acceleration(INT)	4th Floor Vertical Acceleration(INT)	5th Floor Vertical Acceleration(INT)	6th Floor Vertical Acceleration(INT)	1st Residual Story Drift	2nd Residual Story Drift	3rd Residual Story Drift	4th Residual Story Drift	5th Residual Story Drift	6th Residual Story Drift
FF01-1	4.998	6.809	6.535	6.162	5.781	5.409	1.30%	1.40%	1.40%	1.30%	1.20%	1.20%
FF13-1	5.115	4.427	4.025	3.501	2.962	2.779	1.10%	1.00%	1.00%	1.10%	1.20%	1.20%
FF14-1	4.548	3.945	3.676	3.322	2.965	3.024	0.39%	0.28%	0.19%	0.14%	0.16%	0.23%
FF14-2	7.982	6.283	5.758	5.107	4.644	4.829	1.20%	1.30%	1.40%	1.50%	1.50%	1.50%
FF15-2	8.673	6.960	6.731	6.463	6.197	5.826	0.62%	0.72%	0.75%	0.70%	0.63%	0.62%
FF19-1	1.230	1.192	1.191	1.202	1.236	1.191	0.03%	0.22%	0.41%	0.59%	0.76%	0.82%
FF21-2	2.910	2.023	1.869	1.662	1.433	1.281	1.50%	1.20%	0.86%	0.49%	0.09%	0.12%
FF22-1	4.282	2.975	2.906	2.835	2.817	2.661	1.40%	1.20%	1.00%	0.89%	0.86%	0.88%
FF22-2	4.598	3.182	3.100	3.009	2.936	2.788	0.46%	0.43%	0.46%	0.47%	0.39%	0.35%
NF02-2	4.325	2.350	2.084	1.815	1.596	1.398	2.70%	2.70%	2.70%	2.70%	2.80%	2.80%
NF05-1	4.632	2.851	2.617	2.349	2.104	1.881	1.50%	1.70%	1.70%	1.70%	1.60%	1.60%
NF05-2	5.489	3.442	3.154	2.817	2.489	2.196	0.23%	0.18%	0.05%	0.11%	0.20%	0.16%
NF16-1	4.384	4.363	3.933	3.401	3.025	2.552	1.70%	1.80%	1.70%	1.50%	1.50%	1.60%
NF16-2	3.941	3.976	3.481	2.880	2.299	2.013	0.39%	0.29%	0.15%	0.02%	0.20%	0.32%
NF17-2	2.846	2.303	2.103	1.896	1.674	1.623	0.01%	0.14%	0.23%	0.33%	0.30%	0.30%
NF21-2	5.994	7.184	7.178	7.189	7.213	6.976	1.20%	1.10%	1.00%	1.00%	1.20%	1.20%
NF22-1	3.216	2.518	2.400	2.257	2.115	1.940	2.30%	2.40%	2.30%	2.00%	1.80%	1.70%
NF25-2	2.154	2.555	2.513	2.428	2.312	2.442	1.60%	1.50%	1.30%	1.00%	0.71%	0.59%
NF27-2	6.526	4.108	3.908	3.698	3.488	3.478	1.40%	1.50%	1.50%	1.50%	1.60%	1.60%
NF28-1	3.649	1.791	1.733	1.656	1.580	1.657	0.05%	0.20%	0.33%	0.27%	0.24%	0.18%

Ground Motion	1st Floor BRB Energy Dissipation(EXT)	2nd Floor BRB Energy Dissipation(EXT)	3rd Floor BRB Energy Dissipation(EXT)	4th Floor BRB Energy Dissipation(EXT)	5th Floor BRB Energy Dissipation(EXT)	6th Floor BRB Energy Dissipation(EXT)	1st Floor BRB Energy Dissipation(INT)	2nd Floor BRB Energy Dissipation(INT)	3rd Floor BRB Energy Dissipation(INT)	4th Floor BRB Energy Dissipation(INT)	5th Floor BRB Energy Dissipation(INT)	6th Floor BRB Energy Dissipation(INT)
FF01-1	871.16	775.25	335.23	462.56	159.07	274.77	1248.00	451.58	671.75	224.96	336.93	160.97
FF13-1	620.24	228.83	297.82	196.54	180.43	140.30	331.92	345.78	252.34	247.61	158.87	122.39
FF14-1	295.89	165.05	120.28	101.08	145.38	61.43	342.52	170.83	106.25	107.31	86.06	135.28
FF14-2	1207.00	1058.00	552.99	408.77	285.26	194.28	1817.00	726.44	763.48	423.57	228.39	202.27
FF15-2	194.61	1143.00	69.86	645.86	33.38	255.99	1877.00	87.27	929.44	50.59	375.09	28.12
FF19-1	1501.00	740.60	744.55	329.77	324.13	182.72	1426.00	960.91	536.20	539.66	207.75	209.42
FF21-2	190.44	436.88	91.13	174.67	41.53	118.45	741.65	107.21	299.02	61.58	145.37	34.71
FF22-1	283.71	235.89	82.35	109.78	92.40	113.35	403.02	114.44	147.31	75.43	132.76	82.99
FF22-2	412.68	279.83	129.56	116.44	78.85	98.96	530.66	174.46	192.48	109.20	129.13	63.57
NF02-2	951.78	713.81	553.14	345.10	361.31	84.22	1316.00	619.90	517.75	471.02	146.43	235.20
NF05-1	772.87	509.07	347.74	281.25	244.64	147.27	681.49	411.56	425.06	275.45	178.45	205.94
NF05-2	908.62	524.54	410.48	218.92	164.03	139.99	1171.00	549.92	343.47	217.98	175.20	136.18
NF16-1	761.12	732.52	254.23	291.18	200.40	145.16	973.67	346.56	507.94	229.50	192.64	165.18
NF16-2	425.76	349.48	215.01	171.58	167.73	66.65	603.35	278.18	232.46	180.82	113.50	145.00
NF17-2	1554.00	235.57	757.10	136.84	477.69	40.51	404.79	813.87	169.42	578.42	54.42	346.76
NF21-2	1630.00	244.28	680.38	161.70	285.00	128.07	428.17	839.70	179.97	484.59	150.23	190.29
NF22-1	820.04	419.63	173.62	191.85	80.40	105.85	490.00	271.36	340.73	126.16	144.01	76.12
NF25-2	868.99	466.28	474.04	246.21	206.70	126.36	748.80	544.15	356.26	338.87	158.45	147.80
NF27-2	857.92	747.03	448.24	337.75	295.87	162.46	1359.00	519.31	556.05	382.16	266.51	248.86
NF28-1	1897.00	708.91	594.08	356.60	239.28	150.58	1001.00	889.24	579.17	434.74	248.38	212.77

Table B-17: Energy Dissipation – Three-Story MF as Beam --

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	61400	61100	47360	47130	-0.49%	-0.49%
FF13-1	92380	95290	68870	70280	3.15%	2.05%
FF14-1	31430	31990	18290	18390	1.78%	0.55%
FF14-2	55740	55610	44100	43880	-0.23%	-0.50%
FF15-2	66190	66150	49720	49010	-0.06%	-1.43%
FF19-1	28550	28760	18900	19080	0.74%	0.95%
FF21-2	11850	12210	4927	4973	3.04%	0.93%
FF22-1	19620	19980	12170	12470	1.83%	2.47%
FF22-2	52440	52710	38820	38950	0.51%	0.33%
NF02-2	27530	27820	22540	22720	1.05%	0.80%
NF05-1	62390	63170	49250	49750	1.25%	1.02%
NF05-2	33480	34020	24950	25180	1.61%	0.92%
NF16-1	54780	55720	40690	41090	1.72%	0.98%
NF16-2	59950	60280	42950	43050	0.55%	0.23%
NF17-2	97580	98260	75880	76070	0.70%	0.25%
NF21-2	66810	67130	51470	51700	0.48%	0.45%
NF22-1	14540	14150	8718	8961	-2.68%	2.79%
NF25-2	52860	52580	41670	41380	-0.53%	-0.70%
NF 27-2	87230	87440	64850	65010	0.24%	0.25%
NF28-1	40540	40740	30990	31100	0.49%	0.35%

Table B-18: Energy Dissipation-- Three-Story MF as Girder --

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	50990	52380	37760	38670	2.73%	2.41%
FF13-1	60580	70660	44750	49870	16.64%	11.44%
FF14-1	29160	30180	18530	18660	3.50%	0.70%
FF14-2	49560	50370	37850	37890	1.63%	0.11%
FF15-2	75690	80200	57780	59270	5.96%	2.58%
FF19-1	37100	37390	26940	27010	0.78%	0.26%
FF21-2	10900	14960	4648	6451	37.25%	38.79%
FF22-1	17850	19370	11280	11910	8.52%	5.59%
FF22-2	44920	46350	32040	32440	3.18%	1.25%
NF02-2	27830	28230	22430	22530	1.44%	0.45%
NF05-1	50420	52180	39750	40360	3.49%	1.53%
NF05-2	35960	37590	27730	28850	4.53%	4.04%
NF16-1	46970	49400	32130	33240	5.17%	3.45%
NF16-2	37440	38420	24090	24570	2.62%	1.99%
NF17-2	78770	81370	60560	61360	3.30%	1.32%
NF21-2	47760	52480	34690	37810	9.88%	8.99%
NF22-1	16870	16690	8668	8897	-1.07%	2.64%
NF25-2	48810	49740	40860	41160	1.91%	0.73%
NF 27-2	65330	67310	46890	47900	3.03%	2.15%
NF28-1	39450	40040	32370	32450	1.50%	0.25%

Table B-19: Energy Dissipation -- Three-Story Chevron --

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	138300	139000	120800	120900	0.51%	0.08%
FF13-1	95640	98400	80720	82560	2.89%	2.28%
FF14-1	44400	45500	35580	36170	2.48%	1.66%
FF14-2	32410	34790	26770	27110	7.34%	1.27%
FF15-2	58980	61530	45880	47190	4.32%	2.86%
FF19-1	41070	41180	31890	31890	0.27%	0.00%
FF21-2	15070	15280	10670	10840	1.39%	1.59%
FF22-1	34530	35910	28000	28390	4.00%	1.39%
FF22-2	87740	89040	74210	74390	1.48%	0.24%
NF02-2	28350	28540	24110	24130	0.67%	0.08%
NF05-1	52580	53530	45000	45530	1.81%	1.18%
NF05-2	28310	29710	22620	23450	4.95%	3.67%
NF16-1	92200	93240	77640	78300	1.13%	0.85%
NF16-2	92150	92240	78180	78120	0.10%	-0.08%
NF17-2	142200	144000	123000	123400	1.27%	0.33%
NF21-2	118200	118200	102400	120400	0.00%	17.58%
NF22-1	21260	21390	16560	16560	0.61%	0.00%
NF25-2	26640	27220	21280	21440	2.18%	0.75%
NF 27-2	130600	132200	112600	112800	1.23%	0.18%
NF28-1	23650	24250	18990	19070	2.54%	0.42%

Table B-20: Energy Dissipation -- Three-Story Single Diagonal –

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	140700	143300	121800	123300	1.85%	1.23%
FF13-1	102800	105600	86640	87850	2.72%	1.40%
FF14-1	41760	42580	32400	32620	1.96%	0.68%
FF14-2	36270	37120	29360	29590	2.34%	0.78%
FF15-2	58020	60840	44030	45110	4.86%	2.45%
FF19-1	45790	45980	36420	36410	0.41%	-0.03%
FF21-2	14570	14920	10160	10350	2.40%	1.87%
FF22-1	31790	33770	25150	25890	6.23%	2.94%
FF22-2	78220	80980	65670	66850	3.53%	1.80%
NF02-2	31590	31710	26530	26510	0.38%	-0.08%
NF05-1	53650	54380	44460	44660	1.36%	0.45%
NF05-2	27240	27910	22020	22170	2.46%	0.68%
NF16-1	94870	95580	78820	78990	0.75%	0.22%
NF16-2	91390	91840	77020	77150	0.49%	0.17%
NF17-2	138300	140600	118500	119000	1.66%	0.42%
NF21-2	121600	123300	104000	104800	1.40%	0.77%
NF22-1	21290	21510	16450	16350	1.03%	-0.61%
NF25-2	25360	26030	20780	20790	2.64%	0.05%
NF 27-2	135800	139800	116500	118200	2.95%	1.46%
NF28-1	32400	33250	26170	26330	2.62%	0.61%

Table B-21: Energy Dissipation – Six-Story MF as Beam –

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	144100	146100	106500	108200	1.39%	1.60%
FF13-1	98920	116000	69410	83680	17.27%	20.56%
FF14-1	31980	32840	17090	17730	2.69%	3.74%
FF14-2	136700	144400	101300	107800	5.63%	6.42%
FF15-2	150100	161500	114400	122300	7.59%	6.91%
FF19-1	212500	217300	169900	173900	2.26%	2.35%
FF21-2	51710	54480	34680	37120	5.36%	7.04%
FF22-1	36820	39070	22010	24060	6.11%	9.31%
FF22-2	65210	69460	43690	47720	6.52%	9.22%
NF02-2	80780	80770	66760	66560	-0.01%	-0.30%
NF05-1	85240	88290	63580	66900	3.58%	5.22%
NF05-2	113100	114200	88190	90310	0.97%	2.40%
NF16-1	90400	93880	63540	66740	3.85%	5.04%
NF16-2	79500	84240	52140	56620	5.96%	8.59%
NF17-2	145700	149700	111800	114800	2.75%	2.68%
NF21-2	30080	31400	17490	18800	4.39%	7.49%
NF22-1	57880	59580	34440	36430	2.94%	5.78%
NF25-2	146800	144200	122400	119700	-1.77%	-2.21%
NF 27-2	169200	167300	124900	123600	-1.12%	-1.04%
NF28-1	140500	140600	116700	116700	0.07%	0.00%

Table B-22: Energy Dissipation-- Six-Story MF as Girder --

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	171000	172500	128200	128900	0.88%	0.55%
FF13-1	79370	107700	49540	67320	35.69%	35.89%
FF14-1	31490	34950	15720	16570	10.99%	5.41%
FF14-2	127100	140000	94080	99930	10.15%	6.22%
FF15-2	157000	178100	121100	129400	13.44%	6.85%
FF19-1	231600	231800	187700	187600	0.09%	-0.05%
FF21-2	54060	55810	36990	37490	3.24%	1.35%
FF22-1	36240	39820	20930	23210	9.88%	10.89%
FF22-2	65100	71390	42030	46060	9.66%	9.59%
NF02-2	90150	89750	74660	73690	-0.44%	-1.30%
NF05-1	88550	91900	66230	67360	3.78%	1.71%
NF05-2	113200	114900	90660	89860	1.50%	-0.88%
NF16-1	103100	109700	72430	74780	6.40%	3.24%
NF16-2	87930	92810	57800	59820	5.55%	3.49%
NF17-2	154000	159700	118000	119500	3.70%	1.27%
NF21-2	131000	137600	96050	99630	5.04%	3.73%
NF22-1	56920	56910	36400	36930	-0.02%	1.46%
NF25-2	143500	143100	119400	118000	-0.28%	-1.17%
NF 27-2	175100	176900	127900	128100	1.03%	0.16%
NF28-1	149500	148900	124700	123000	-0.40%	-1.36%

Table B-23: Energy Dissipation -- Six-Story Chevron –

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	156300	159000	128700	129900	1.73%	0.93%
FF13-1	130500	132200	108600	109700	1.30%	1.01%
FF14-1	48360	49410	38520	38780	2.17%	0.67%
FF14-2	180100	182900	149100	149900	1.55%	0.54%
FF15-2	151100	154300	127500	128300	2.12%	0.63%
FF19-1	151500	151800	131100	131300	0.20%	0.15%
FF21-2	46630	47110	38290	38560	1.03%	0.71%
FF22-1	41660	42630	32160	32450	2.33%	0.90%
FF22-2	77500	78090	61750	61700	0.76%	-0.08%
NF02-2	80010	80210	68590	68630	0.25%	0.06%
NF05-1	87120	88340	72630	73440	1.40%	1.12%
NF05-2	86660	99430	72500	75060	14.74%	3.53%
NF16-1	125100	126600	102200	103200	1.20%	0.98%
NF16-2	120200	121000	98950	99520	0.67%	0.58%
NF17-2	169700	170400	145200	145300	0.41%	0.07%
NF21-2	151300	154000	123400	124000	1.78%	0.49%
NF22-1	68820	68470	31480	31380	-0.51%	-0.32%
NF25-2	113000	113600	97160	97290	0.53%	0.13%
NF 27-2	222600	225100	189500	190200	1.12%	0.37%
NF28-1	123300	123400	104100	104000	0.08%	-0.10%

Table B-24: Energy Dissipation -- Six-Story Single Diagonal –

Ground Motion	Total Energy(kip*in)		Dissipated Inelastic Energy(kip*in)		Total Energy Diffence	Dissipated Inelastic Energy Difference
	Hor	Hor+Ver	Hor	Hor+Ver		
FF01-1	148500	155400	122300	126400	4.65%	3.35%
FF13-1	115400	116800	94140	94030	1.21%	-0.12%
FF14-1	42820	44640	32950	33100	4.25%	0.46%
FF14-2	166300	172100	137300	139400	3.49%	1.53%
FF15-2	140200	147700	117700	121200	5.35%	2.97%
FF19-1	155500	155700	133800	133900	0.13%	0.07%
FF21-2	47510	47780	38620	38630	0.57%	0.03%
FF22-1	37050	39090	27360	28010	5.51%	2.38%
FF22-2	72520	75140	57160	58360	3.61%	2.10%
NF02-2	73220	73430	62350	62360	0.29%	0.02%
NF05-1	80970	81470	65910	65920	0.62%	0.02%
NF05-2	92930	93620	76900	76940	0.74%	0.05%
NF16-1	113000	113900	91060	91090	0.80%	0.03%
NF16-2	109300	110000	88820	88820	0.64%	0.00%
NF17-2	164800	165400	140500	140300	0.36%	-0.14%
NF21-2	138100	145200	111700	115200	5.14%	3.13%
NF22-1	67770	67270	28920	28910	-0.74%	-0.03%
NF25-2	108100	109200	93070	93070	1.02%	0.00%
NF 27-2	209000	214900	176300	178200	2.82%	1.08%
NF28-1	114000	114800	96800	96920	0.70%	0.12%