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Effect of Vertical Accelerations on Steel Frame Structures

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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 the 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 damage or collapse. Further research is needed to investigate the impact of vertical ground motion on seismic response of structures. In this paper, a six-story steel frame structure is chosen to investigate and represent this problem. Two special moment frames with reduced beam section (RBS) connections are designed by the equivalent lateral force (ELF) method based on ASCE 7-10 and analyzed using nonlinear dynamic analysis. In this study, a suite of 20 ground motions were selected from the far-field and near-field suites of recorded ground motions from FEMA P695 which include horizontal and vertical ground motions. 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. The impacts of the vertical accelerations had little impact on the lateral drift of the structure but had a significant impact on the axial forces in the columns.

Keywords: vertical ground motion, reduced beam section (RBS), ductility of steel structure, nonlinear analysis

1. Introduction

1.1 Research Motivation

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.

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 is limited. Iyengar and Shinozuka [1] did some investigation on the vertical ground motion by using a cantilever beam. Anderson and Bertero [2] did research on a ten story building which only had an unbraced, single bay frame.

1.2 Scope of Work

The purpose of this paper 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, two different kinds of steel frames are used in this paper. Both models are special moment frames designed based on the information given by Sabelli [3]. Both models in this study are located in Los Angeles. The design is completed by the equivalent lateral force (ELF) method according to ASCE 7-10 [4] along with SAP2000 [5] software. Finite element and nonlinear dynamic analysis are completed using Perform 3D [6].

The finite element modeling procedure for the mesh method of the beam, mass, and reduced beam section (RBS) connections will be discussed in this paper. A suite of twenty amplitude-scaled strong ground motions are selected to complete the nonlinear dynamic analysis on two different models. All the structural models are analyzed under two different loading cases: 1) Horizontal Only and 2) Horizontal plus Vertical.

2. Special Moment Frame (SMF) Design

2.1 Basic Building Information

The six story moment frame models are modified versions of the SAC building models [3]. Fig. 1 shows the plan view of the six story moment frame structure. The building is built as an office building. The height of first floor is 18 ft (5.5m) and the height of the remaining floors is 13 ft (4m). The plan dimensions of the building are 154 ft (47m) by 154 ft (47m). 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 30 ft (9m) by 30 ft (9m). A 12 ft (3.7m) tall penthouse on the roof of the building is represented as a dashed line rectangle. The dimensions of the penthouse are 30 ft (9m) by 60 ft (18m). In the east-west direction, each bay has two secondary beams running from north to south. The distance between them is 10 ft (3m). Four exterior frames are responsible for the seismic resistance.

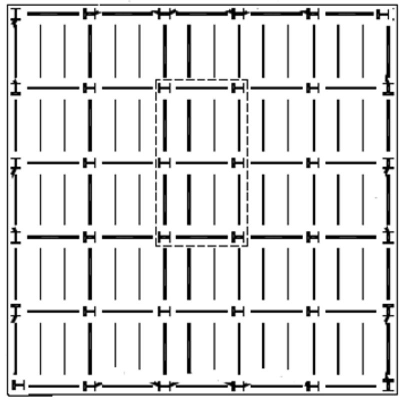

Fig. 1 – Plan view of six-story moment frame structure [3]

2.2 Equivalent lateral force (ELF) procedure for structure design

The six-story moment frame lateral resisting system design was accomplished with the assistance of SAP 2000 [5]. Cross sections and properties of columns and beams in the moment frame structure are listed in Tables 1 and 2, respectively. The moment frame as beam model means north-south direction moment frame while moment frame as girder model means east-west direction moment frame.

Table 1 – 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

Table 2 – 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

3. Modeling of Special Moment Frame (SMF)

3.1 Basic modeling conception

The moment frame structures were modeled in 3D models for design and as 2D models for the nonlinear dynamic analysis. One of the exterior frames of six-story structures is used to represent half of the structure. Figure 2 shows the six-story moment frame as beam model as an example. Columns are all considered fixed at the base. A ghost column is used to represent the remaining gravity columns tributary to the modeled frame. The ghost column carries the gravity load from the columns tributary to the modeled frame. 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 A992Fy50 which is the same as the frame element in the lateral load resisting system. The ghost column joints are constrained by the body constraint. 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 have a moment release.

Each beam has three segments which have same length, 10 ft (3m). 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 not in the model is transferred to become a point load assigned to the joints of ghost column. Mass is very important in the dynamic analysis model. The method of assigning masses to the beams will be detailed later. There is no mass assigned to the nodes of the ghost column.

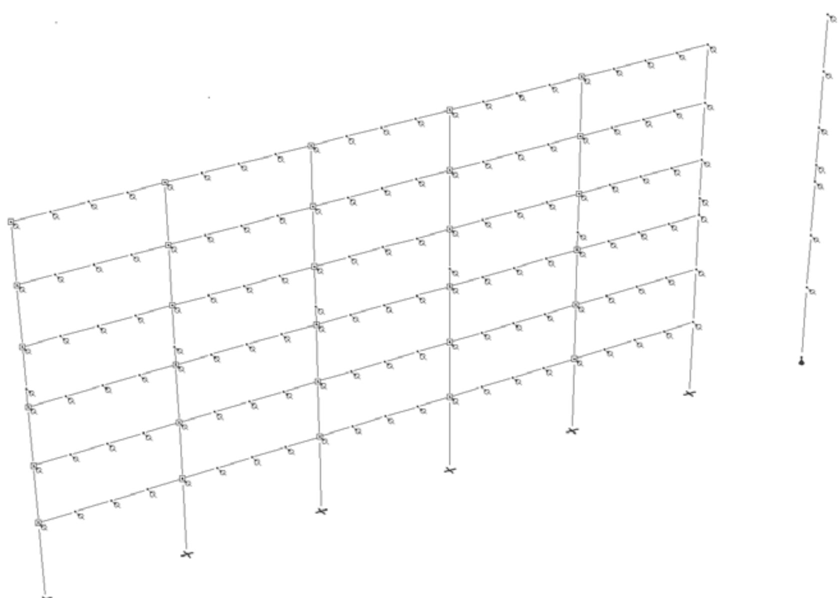

Fig. 2 – Six-story moment frame as beam analysis model

3.2 Mesh method 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. [7] 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. Fig. 3 shows the three different types of mesh models.

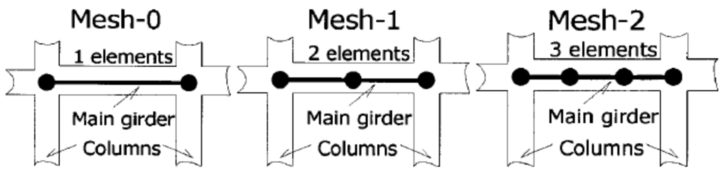

Fig. 3 – Three different type of mesh model [7]

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 shows the percentage of averaged error by using different kinds of models. The results show that lower buildings create more error while the mesh-2 method will give relatively accurate results. For this paper, the mesh-2 method was chosen to avoid the unnecessary error when the models are analyzed.

Table 3 – Percentage of averaged error using different models [7]

Building stories	Lumped mass			Consistent mass	
	Mesh-0	Mesh-1	Mesh-2	Mesh-0	Mesh-1
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
L-shape 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

3.2 Mass

The lumped mass method is commonly used in dynamic structural analysis. However, there will be significant errors in mass modeling if using the same modeling method for the horizontal and vertical degrees of freedom. Overestimation or underestimation of the lumped masses associated with vertical displacements in 2D frame models will lead to inaccurate modal periods and associated modal participation factors for dynamic response [8].

The mass modeling methods for the 2D models are shown in Fig. 4 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. Figs 5 and 6 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.

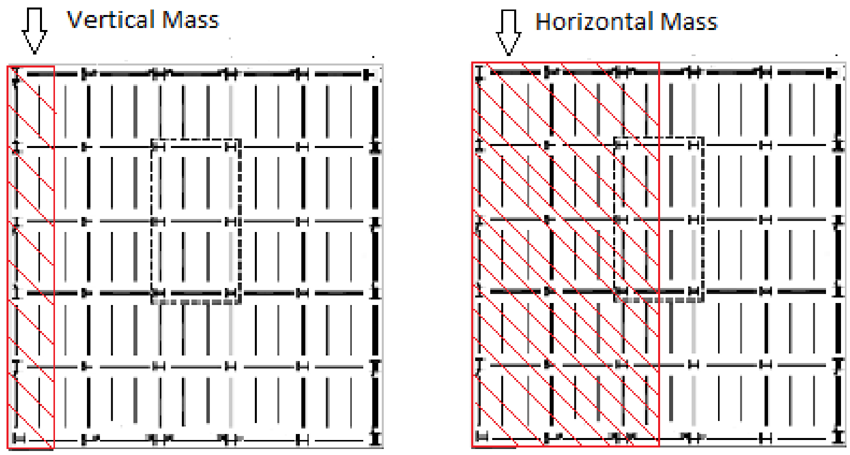

Fig. 4 – Tributary area for vertical mass and horizontal mass

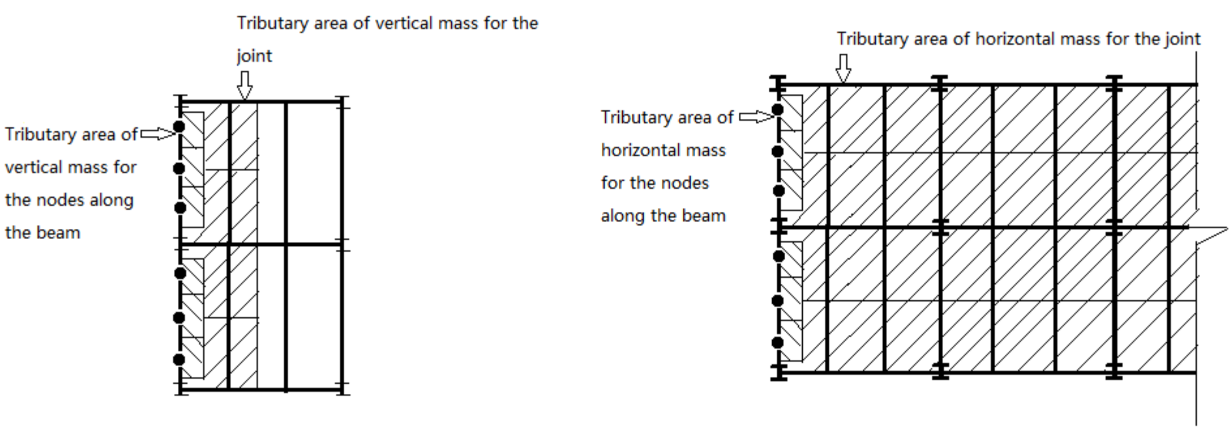

Fig. 5 – Detail of Tributary area for mass in the moment frame as beam model (two bays)

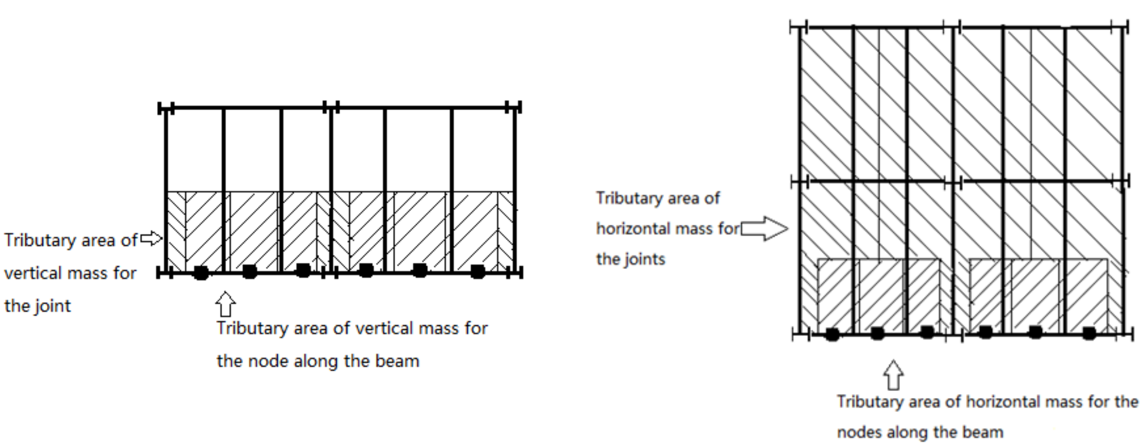

Fig. 6 – Detail of tributary area for mass in the moment frame as girder model (two bays)

3.3 Reduced beam section (RBS)

There are two major methods to move the plastic hinge away from the column face in moment frames. One is to increase the capacity of beam at the column face by putting cover plates on the beam flanges. The second 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. The strength properties and location of the RBS is according to a 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. Fig. 7 shows the simplified model for RBS.

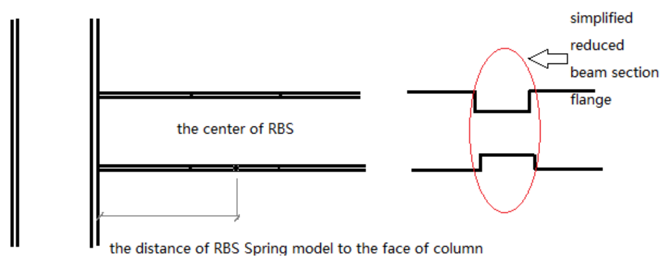

Fig. 7 – Simplified model for reduced beam section

The reduced beam section can be modeled as a plastic hinge which can be represented by a single-spring system. This single-spring system is defined by a hysteretic model confined within a force-displacement boundary according to FEMA 440a [9].

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) according to FEMA 440a [9] Table 3-2 to constitute the basic F-D relationship and strength loss. Table 4 shows the value used for the plastic hinge in the model.

Table 4 – 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 [9]. Cyclic degradation is also considered in this study. The energy degradation factor for the whole hysteretic loop is 0.7. Fig. 8 shows the relationship between moment and rotation for the plastic moment hinge.

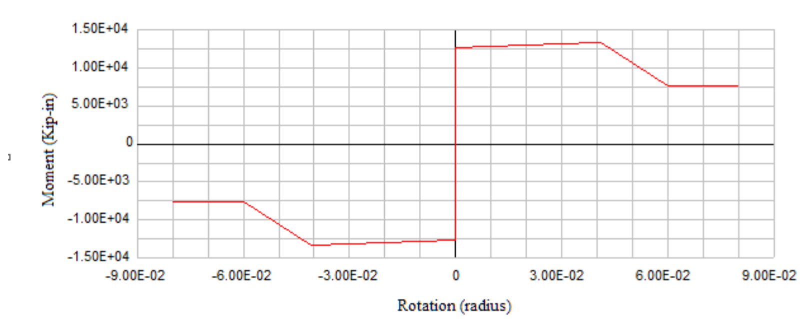

Fig. 8 – Relationship between moment and rotation for the plastic moment hinge

3.4 Earthquakes

Dynamic earthquake load cases including the horizontal earthquake and vertical earthquake are defined according to the 20 earthquake records which are scaled by the different 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. The selected earthquake records include both Far Field (FF) and Near Field (NF) records. Table 5 shows the basic information on the selected earthquake records [10] and the scale factors for the six-story structures. The amplitude scaling of the horizontal and vertical records is the same.

Table 5 – Basic information and scale factors for the selected earthquake records

Number	Name	V-H Ratio	Earthquake	Station	Group	LA6 scale factor
1	FF01-1	0.77	Northridge	Beverly Hills	2	3.10
2	FF13-1	0.92	Loma Prieta	Capitola	3	2.80
3	FF14-1	0.54	Loma Prieta	Gilroy Array #3	1	2.92
4	FF14-2	0.79	Loma Prieta	Gilroy Array #3	2	4.81
5	FF15-2	0.94	Manjil	Iran Transverse Comp	3	2.79
6	FF19-1	0.58	Chi-Chi	CHY101	1	2.30
7	FF21-2	0.94	San Fernando	USGS Station 135	3	5.80
8	FF22-1	0.59	Friuli	Tolmezzo	1	5.00
9	FF22-2	0.72	Friuli	Tolmezzo	2	5.20
10	NF02-2	1.25	Imperial Valley	USGS Station 5028	4	1.53
11	NF05-1	1.15	Loma Prieta	CDMG Station58065	4	2.99
12	NF05-2	1.17	Loma Prieta	CDMG Station58065	4	3.45
13	NF16-1	0.71	Imperial Valley	USGS Station 5054	2	3.28
14	NF16-2	0.51	Imperial Valley	USGS Station 5054	1	2.60
15	NF17-2	0.75	Imperial Valley	UNAM/UCSD Station 6621	2	4.80
16	NF21-2	0.98	Loma Prieta	CDMG Station 57007	3	4.00
17	NF22-1	0.54	Cape Mendocino	CDMG Station 89005	1	1.80
18	NF25-2	1.15	Kocaeli	Yarimca	4	2.17
19	NF27-2	0.90	Chi-Chi	TCU084	3	2.66
20	NF28-1	1.23	Denali	PS10	4	2.65

4. Result and Discussion

4.1 Modal analysis

Most of the time, structural engineers only concern themselves with a small number of modes because almost all of the horizontal effective mass will be included in the first few modes. However, in this study, vertical ground motions were included in the analysis, so the vertical modes which include effective vertical mass are very important. Fig. 9 shows the first horizontal mode shape which includes 83% effective horizontal mass and the first vertical mode shape which includes 25% of the effective vertical mass of the six-story moment frame as beam model.

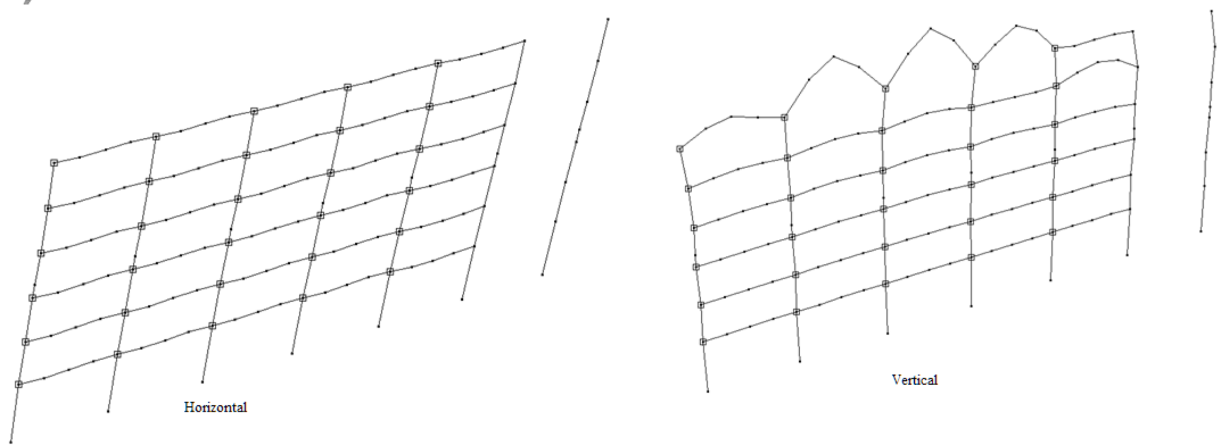

Fig. 9 – First horizontal and vertical mode shape of six-story moment frame as beam model

4.2 Maximum drift

The data of max drift for all the stories during the earthquake is calculated according to the nodal displacements for each floor. Table 6 shows the average value of maximum story drift for each story. 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 the basis between the two different loading cases. The average 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 similar. The impact of vertical acceleration on the steel moment frame is insignificant and limited.

Table 6 – Summary of maximum story drift for six-story moment frame

Story	Max Story Drift (MF as beam)			Max Story Drift (MF as girder)		
	Horizontal Only	Horizontal + Vertical	Average Absolute Difference	Horizontal Only	Horizontal + Vertical	Average Absolute Difference
1	3.43%	3.23%	12.20%	3.30%	3.20%	4.21%
2	3.66%	3.41%	5.37%	3.63%	3.57%	3.62%
3	3.84%	3.61%	4.78%	3.66%	3.64%	4.17%
4	4.26%	4.17%	5.43%	3.68%	3.65%	3.62%
5	4.89%	4.90%	7.47%	3.99%	3.92%	5.69%
6	5.56%	5.61%	10.20%	4.74%	4.67%	6.29%

4.3 Maximum 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. All the column forces in this study are normalized 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%. Figure 10 show the average value of maximum column axial force for each story between two different loading cases in the six –story moment frame as beam model. The six-story moment frame as girder model has similar results. The average value of column maximum normalized axial force for each story increases significant 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.

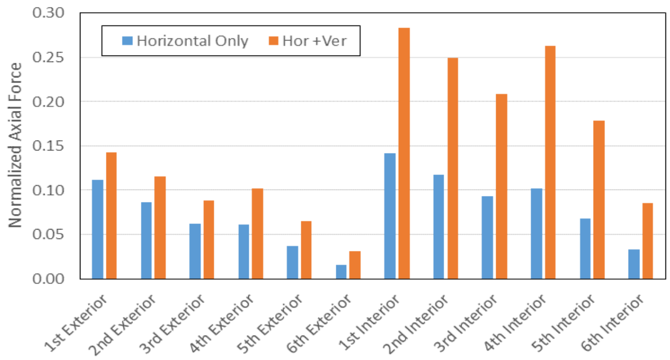

Fig. 10 – Max normalized story axial force for six-story moment frame as beam model

4.4 Roof horizontal acceleration

The maximum roof horizontal acceleration during the earthquake is calculated according to the nodal absolute acceleration. Tables 7 shows the average value of maximum roof horizontal acceleration under forty selected earthquakes including horizontal and vertical ground motions in the six-story moment frame models.

Table 7 – Summary of roof horizontal acceleration for the six-story models

Model	Roof Horizontal Acceleration		
	Horizontal Only (g)	Horizontal + Vertical (g)	Difference
6-Story MF as Beam	0.762	1.209	58.6%
6-Story MF as Girder	0.929	1.003	8.00%

The average value of maximum roof acceleration for the six-story moment as girder model is very similar. However, in the six-story moment frame as beam model, the roof maximum horizontal acceleration increased by 59% when the vertical and horizontal ground motions are added to the structure concurrently. Figures 11 and 12 show the time history response of roof absolute horizontal acceleration in the six-story moment frame as beam model and the six-story moment frame as girder model under FF-14-1 and NF-28-1, respectively. The reason the roof horizontal acceleration increases significantly in the six-story moment frame as beam model is because the first vertical mode period which is 0.102s is close to the first peak value in the response spectrum of earthquake FF-14-1 and NF-28-1 including horizontal and vertical acceleration. Fig. 13 shows the response spectrum of NF-28-1 (Horizontal + Vertical) for the six-story moment frame as beam model as an example.

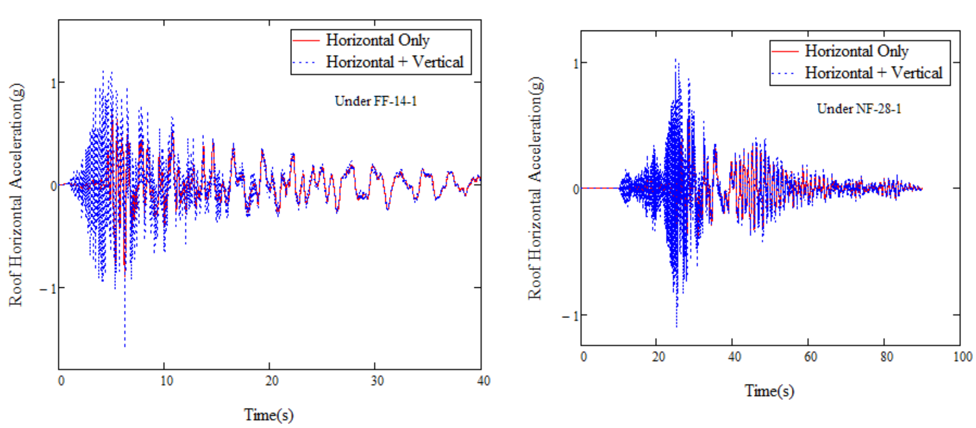

Fig. 11 – Time history response of roof absolute horizontal acceleration in the six-story moment frame as beam model under FF-14-1 and NF-28-1

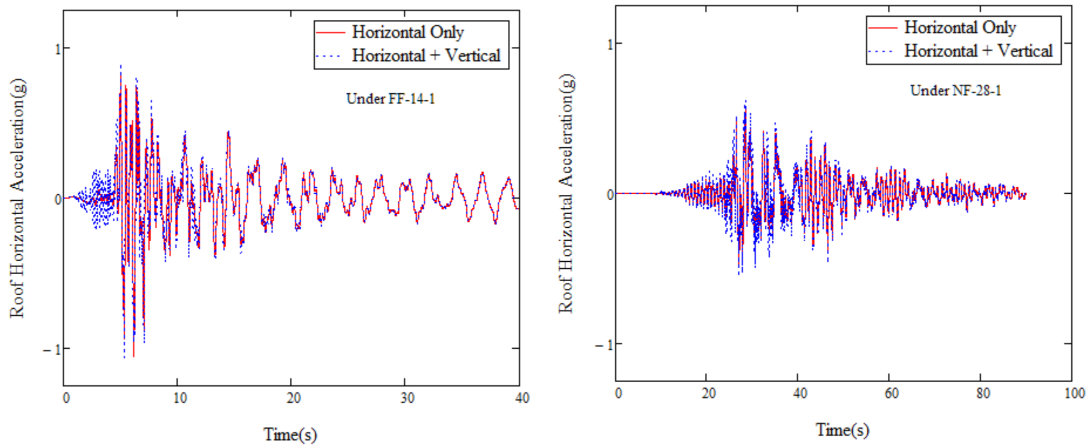

Fig. 12 – Time history response of roof absolute horizontal acceleration in the six-story moment frame as girder model under FF-14-1 and NF-28-1

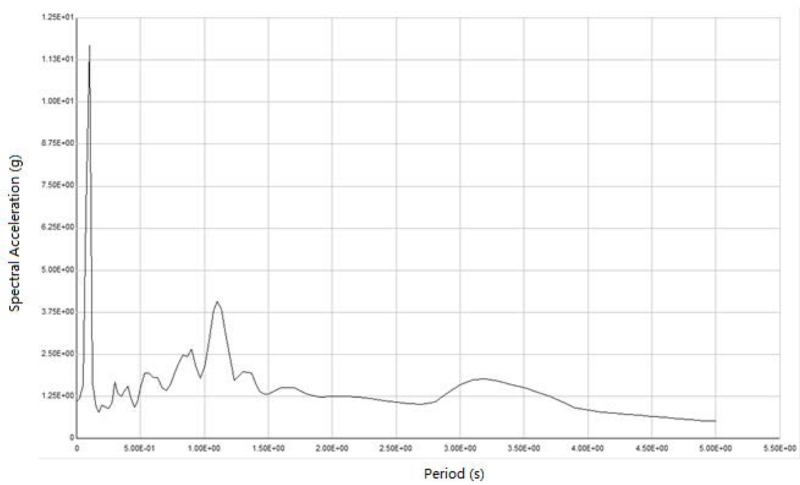

Fig. 13 – Response spectrum with 2% damping ratio under NF-28-1 (Horizontal + Vertical) in the six-story moment frame as beam model

4.5 Reduced beam section

The reduced beam section is one of the major inelastic elements which will absorb the energy from earthquake. The inelastic deformation of the exterior span and interior span are collected separately. Fig. 14 shows the box plot of maximum RBS rotation for each story under forty selected earthquakes including horizontal and vertical ground motions. The first six box plots represent the exterior spans while the remaining represent the interior spans. The median value in the six-story moment frames increase, especially in the upper three stories, when the vertical ground motions are added to the structures. There is some impact of vertical ground motion on these two six-story moment frame models. The impact of vertical ground motions is very subtle. The maximum deformation for each story may increase or decrease when including vertical earthquake excitation.

4. Conclusion

In general, the vertical ground motion has a minor effect on maximum drift. 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. The impact of vertical ground motion is insignificant on the roof horizontal acceleration for the six-story moment frame as girder model while it has significant effect on the six-story moment frame as beam model. The reason the roof horizontal acceleration increases significantly in the six-story moment as beam model is because the first vertical period of the structure is very close to peak value of the response spectrum. There is a decent impact of vertical ground motions on rotation of reduced beam sections, especially the reduced beam sections in the upper stories.

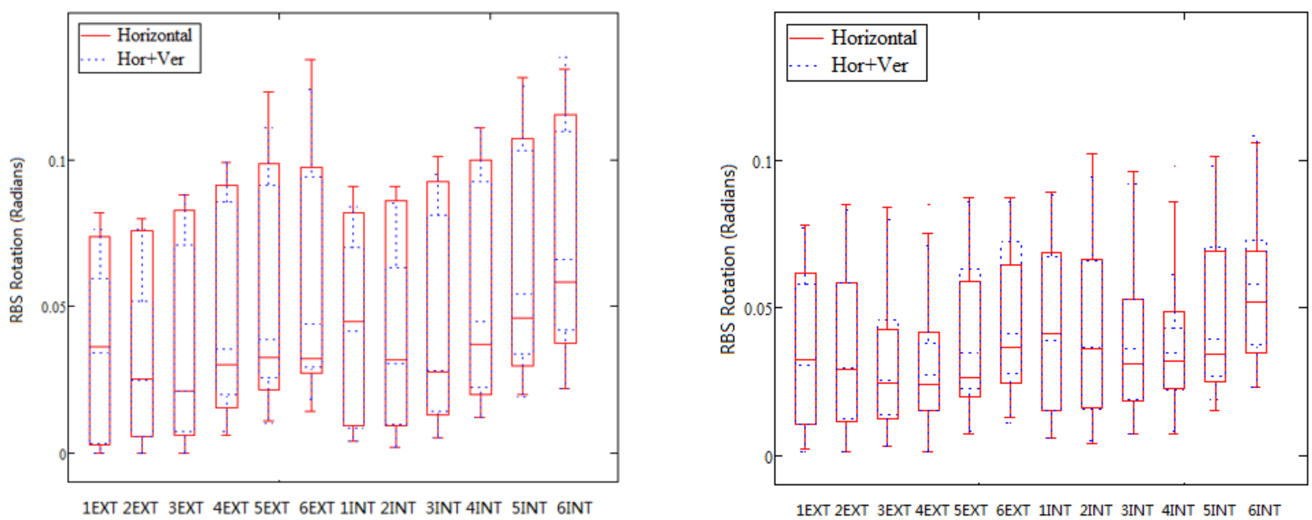

Fig. 14 – Box plot of max story RBS rotation for six-story MF as beam model and MF as girder model

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