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Structural Response Including Vertical Component of Ground Motion

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STRUCTURAL RESPONSE INCLUDING VERTICAL COMPONENT OF GROUND MOTION

by

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the

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THESIS APPROVAL

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TITLE: STRUCTURAL RESPONSE INCLUDING VERTICAL COMPONENT OF GROUND MOTION

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Evidence indicates that the vertical component of ground motion is more significant than previously thought, especially for near fault events. However, many design codes do not reflect the importance of the vertical component of ground motion. Therefore, the purpose of this thesis is to determine what effects the vertical component of ground motion has on a structure by way of comparison. Specifically, structural response due to the lateral components of ground acceleration is compared to structural response due to all three components of ground acceleration. Structural response includes the following parameters: story drift; axial force; shear; torsion; and bending moment. Variables are fundamental period of vibration, ground motion record, and presence of cross-bracing. Through nonlinear dynamic time history analysis, it is shown that the vertical component of ground motion greatly affects axial force response for these short-period frames. However, the story drift is unaffected for the short, medium, and long-period frames. Other parameters show varying degrees of dependence or independence in relation to the vertical component of ground motion.

ACKNOWLEDGMENTS

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CHAPTER 1

INTRODUCTION

The purpose of this section is to familiarize the reader with my motivation for investigating structural response to strong vertical ground motion. First, I discuss the appropriate background material, hopefully demonstrating the importance of the topic. Second, I review available literature. Lastly, I specify the objectives of my thesis and explain how they may improve our knowledge of the subject.

Background

The design of structures to withstand seismic loading is primarily governed by horizontal ground motion, and the effects of vertical ground motion have long been deemed unimportant or secondary. However, an emerging body of evidence suggests that vertical ground motions have great destructive potential, especially for certain site conditions.

The ratio of vertical spectral acceleration to horizontal spectral acceleration (V/H) is a strong function of natural period, local site conditions, and source-to-site distance. It is a weak function of magnitude, faulting mechanism, and sediment depth (Bozorgnia & Campbell, 2004). At short periods (0.05-0.4 sec), V/H can be as high as 1.8, and the ratio is generally lower than ½ for medium periods (0.4-0.8 sec) (Bozorgnia & Campbell, 2004). The largest short-period V/H values occur on stiff soils at short epicentral distances, and the largest long-period (greater than 0.8 sec) V/H values occur on hard rock where they may be as high as 0.7 (Bozorgnia, Campbell, & Niazi, Vertical Ground Motion: Characteristics, Relationship with

Horizontal Component, and Building-Code Implications, 1999). This apparent amplitude in V/H is due to the large contrast in shear-wave velocity at the rock/soil interface that causes the vertical component of S-waves to be converted to P-waves as they travel through said interface (Silva, 1997).

The most common design practice is to take the vertical spectral acceleration as 2/3 of the horizontal spectral acceleration. This is the approach used by FEMA, for example (Bozorgnia & Campbell, 2004). However, this 2/3 rule of thumb is inaccurate for near-source moderate and large earthquakes (Friedland, Power, & Mayes, 1997). In fact, V/H may exceed unity (Friedland, Power, & Mayes, 1997)(Bozorgnia, Campbell, & Niazi, Vertical Ground Motion: Characteristics, Relationship with Horizontal Component, and Building-Code Implications, 1999)(Bozorgnia & Campbell, 2004) (Button, Cronin, & Mayes, 2002). The reason for this traditional underestimation may be attributed to the fact that regression in the context of attenuation relations was performed for the entire range of epicentral distances and magnitudes rather than focusing on distinct intervals; therefore, the results are biased (Papazoglou & Elnashai, 1996). Other agencies such as the US Atomic Energy Commission, the European Building Code, and the Unified Building Code recognize that V/H varies with period, though neither UBC-97 nor IBC-2000 offer guidance on a vertical design spectrum (Bozorgnia & Campbell, 2004). It can be seen that 2/3 is un-conservative for short periods and long periods and generally conservative for medium periods.

There have been two arguments against the importance of vertical ground motion in the past: the peaks of vertical strong-motion have low energy content; and properly designed structures already contain a large factor of safety in the vertical direction. These arguments are easily refuted. It is contended that the relationship between structural and excitation periods are more important than energy content, and field evidence demonstrates that even sound structures may fail due to vertical strong-motion (Papazoglou & Elnashai, 1996).

Structural response due to vertical ground motion has been thoroughly studied by Papazoglou and Elnashai (1996). They argue that a structure may fail due to strong vertical ground motion. Failure mechanisms include direct compression or tension and reduction in shear or flexure capacity. As far as specific structural parts are concerned, interior columns are more vulnerable than exterior columns because the former are not designed to withstand overturning forces, and intermediate and top stories are more likely to undergo tensile deformations. Concrete columns are particularly susceptible to a reduction in shear due to large tensile forces and reduction in flexure capacity due to large compressive forces. Interestingly, the vertical motion does not significantly influence transverse response parameters like inter-story drift. They show that the vertical response amplification is higher than corresponding horizontal and is not influenced significantly by building height. There are two reasons for this: damping in the vertical direction is less due to absence of an efficient energy dissipating mechanism; and there is a quasi-resonant response for a wide range of building frames due to large stiffness in the vertical direction and high-frequency pulses from vertical ground motion. In contrast, it has been shown by others that the vertical acceleration experienced by upper stories is greater than the acceleration experienced at the base by factors ranging from 1.1 to 6.4 (Bozorgnia, Mahin, & Brady, Vertical Response of Twelve Structures Recorded during the Northridge Earthquake, 1998).

Many engineers recognize the importance of accounting for vertical ground motion in design, and they argue for implementation. Friedland, Power, and Mayes recommend considering vertical ground motions in bridge design in higher seismic zones for certain types of construction (1997). Bozorgnia, Campbell, and Niazi believe that modified spectra must be used since using 2/3 for V/H is un-conservative at short and long periods, but un-conservative at medium periods (Vertical Ground Motion: Characteristics, Relationship with Horizontal Component, and Building-Code Implications, 1999). Finally Papazoglou and Elnashai think simple procedures for the inclusion of the vertical component in design are urgently needed (1996).

Literature Review

Four recent earthquakes have provided unprecedented levels of information on vertical ground motion: Kalamata 1986, Loma Prieta 1989, Northridge 1994, and Kobe 1995. Papazoglou and Elnashai studied these events, and they found ample field evidence of damage from vertical ground motion (1996). The Kalamata earthquake was a shallow near-field event, with M_s = 5.7, epicenter less than 9km from town, and a focal depth of 7 km. These characteristics made it susceptible to amplified V/H ratios; the ratios were in fact as high as 1.26. Evidence here of failure due to strong vertical ground motion included an RC pedestal cracked at mid-height, signifying possible tensile failure. Also, there were a high number of symmetric compression and shear-compression failures in columns and shear walls even in buildings where bending failure was expected. In Northridge, where V/H was as high as 1.79, several columns in the third story of the Holiday Inn Hotel sustained structural damage. Since the RC frame vibrated in first mode and there was no torsion, it is likely that the columns failed due to reduced shear capacity. The authors go on to explain that a larger reduction in column shear capacity is expected for higher stories because they undergo a larger relative change in preexisting axial force at least for vibration in the first vertical mode. Crack patterns were observed in the beam-column connections and column webs in many steel moment resisting frames. The authors hypothesize that the beam vibrations due to the vertical ground motion exaggerated rotational demand imposed by the horizontal ground motion. At the La Cienega-Venice Undercrossing, a pier collapsed. This failure is attributed by the authors to instantaneous reduction of shear strength and fluctuation of axial loads. The conditions of the Kobe earthquake are unique in that large V/H ratios and PGA occurred even at large epicentral distances (≥ 45 km). Here failure was observed in steel box column members. Since there is no bending deformation of the plates comprising the box column, the authors conclude that the axial response was primarily tensile. Additionally members of a steel mega-truss were severed. Because no bending exists in a truss system, the damage may be attributed to vertical ground motion. Bridges failed too as evidenced symmetric outward buckling of longitudinal reinforcement and crushing of concrete at mid-height of the piers. Bending rotations were limited or nonexistent in the crushed areas, again indicating axial response.

In addition to studying field evidence, Papazoglou and Elnashai performed analyses on multiple buildings. Looking at RC and steel buildings, they found that vertical periods were not significantly influenced by building height or lateral stiffness. Furthermore the authors analyzed lumped MDOF structural models using bilinear stiffness characteristics for RC and found that strong vertical motion could induce column tension. In another nonlinear dynamic analysis of an 8-story, 3-bay moment resisting RC frame designed according to UBC standards, net tensile forces and deformations were observed. A separate 3-D nonlinear analysis of the aforementioned La Cienega-Venice Undercrossing yielded interesting results. The peak horizontal and vertical accelerations experienced by the structure during the Northridge earthquake were 0.3g and 0.22g respectively. So, V/H was not particularly large. Despite the small ratio of V/H the model predicted shear failure by a margin of 15 percent for two piers, one of which actually failed during the Northridge earthquake. The time histories showed that biaxial shear response peaked when axial force was at a minimum, and this confirmed the authors' initial hypothesis that failure was caused by reduced shear capacity induced by strong vertical ground motion. They also modeled three piers from the Hanshin Expressway which was damaged during the Kobe earthquake. Failure of this structure could not be convincingly attributed to shear or flexure by inspection. Their dynamic analysis demonstrated that shear demand exceeded capacity, but the piers did not actually fail due to shear. So, they examined the flexural behavior and found that bending demand never exceeded capacity. Finally they scrutinized the axial force behavior. It was observed that axial force response fluctuated greatly, up to

70% of the static load. They concluded that the fluctuations caused the concrete cover to spall and the longitudinal reinforcement to buckle, further reducing the shear and bending capacity.

However, Ambraseys and Douglas found that the effect of vertical ground motion on horizontal response is small for most realistic models (2003). They studied elastic SDOF bending and hinged models under the combined effects of gravity loads and horizontal and vertical ground motion. They felt that in order to understand complex systems they must first study simple systems. From their literature search, they discovered that previous studies had two shortcomings. First, many studies of the effect of vertical ground motion used white noise representations of ground motion which adequately estimate the importance of vertical ground motion but need to be confirmed with actual ground motion records. Second, the studies that did use actual ground motion records relied heavily on the El Centro recordings which are not as complete nor as intense as other, more recent ground motion records. To avoid these shortcomings, the authors used 186 strong-motion records from 42 earthquakes. All records met the following criteria: $M_s \geq 5.8$, distance to surface projection of rupture $d \leq 15$km and focal depth $h \leq 20$km. They found that some vertical records induced instability in SDOF bending models with finite vertical stiffness and a load ratio of 0.3 to 0.5. For hinged systems, no recorded vertical ground motions induced instability for realistic column length and horizontal and vertical periods.

Armed with extensive data from the Northridge earthquake, Bozorgnia, Mahin, and Brady studied the vertical response of twelve structures (Bozorgnia, Mahin, & Brady, Vertical Response of Twelve Structures Recorded during the Northridge Earthquake, 1998). The studied structures had at least two vertical component sensors at two different levels and vary from 2 to 14 stories. Some were concrete, and others steel. Three even had base isolation. All were located within a distance of 8 to 71 km of the fault. None of the structures were subjected to severe vertical ground accelerations; the vertical accelerations at the base ranged from 2 to 22 % g. However, the largest vertical accelerations recorded above the base were 52, 43, and 23.7 % g. So, the authors compared the vertical acceleration at the base of a given structure to the vertical acceleration experienced at an upper level. They found that the vertical response of each of the 12 structures was amplified. One seismic isolated building experienced an amplification of 1.8-2.3. The vertical response of a steel structure was amplified by a factor of 1.5-2.67. A concrete structure experienced an amplification of 2-3.4. All structural periods fell within a range of 0.075 to 0.26 sec. The authors recognized that this makes them more susceptible to vertical ground motion, especially for regions near the fault.

A study of bridges by Button, Cronin, and Mayes provides further insight into the effect of vertical ground motion (2002). The authors performed dynamic analyses on a group of representative highway bridges and recommended when vertical motion should be explicitly included in design, when the effects can be adequately accounted for by changing code load combinations, and when vertical motions may be safely ignored. For comparison among the bridges they used a ratio of the difference between three-component and two component response divided by the dead load response. This ratio decreased as fault distance increased. Soil site conditions and a magnitude 7.5 event produced the highest ratios for pier axial force for all distances. For distances less than 10 km rock site conditions produced the highest ratios for deck shear at the pier and moment at the mid-span. The authors discovered that the early arrival of strong vertical ground motion has little effect on the bridge response compared to the strong vertical ground motion arriving at the same time as the horizontal ground motion. They learned this from time history analyses. The authors came to many important conclusions: the impact of vertical ground motion increases greatly as the bridge site gets closer to the fault; the horizontal response is not significantly influenced by the vertical component of motion; and bridges with the greatest percentage of modal mass attributed to periods near the peak of the vertical response spectrum are affected the most by vertical ground motions. They recommend ignoring the effect of vertical ground motion for bridges located more than 50 km away from an active fault. For bridges less than 10 km away from an active fault, a site specific study is required, and the CQC modal combination and SRSS directional combination methods should be used in a linear analysis to determine vertical design forces. For bridges at an intermediate distance, a site specific study may be performed to determine the effects of the vertical ground motion.

Objectives

The purpose of my thesis is to investigate structural response to strong vertical ground motion. Specifically, I compare structural response to lateral ground accelerations (X and Y) against structural response to all orthogonal components of ground accelerations (X, Y, and Z). In other words, what difference does the inclusion of the vertical component make in a time history analysis? In addition, I investigate torsion effects introduced by cross-bracing in response to all three orthogonal components of ground acceleration. Here are the parameters of interest:

- Story drift
- Axial force at the base of a given column
- Shear at the ends of a given column
- Torsion
- Bending moment at the ends of a given column

As mentioned in the background and literature review, previous studies show that strong vertical ground motion has little effect on the lateral response of a structure, but axial force response is greatly exasperated by such motion. I hope to confirm these findings. Additionally, I want to see if strong vertical ground motion influences bending moments in beams and columns. Lastly, how does cross-bracing affect the response of a structure? Intuition tells me that the addition of cross-bracing will alter story drift, torsion, and base shear. In summary, here are the variables of interest:

- Fundamental period of vibration
- Ground motion record
- Cross-bracing

Previous research indicates that short-period and long-period structures are most affected by strong vertical ground motion. Therefore, I investigate three structures with varying periods. The results may show that certain parameters are perioddependent. In order to limit the scope of the thesis, I am only investigating steel moment-resisting frames with and without cross-bracing.

CHAPTER 2

PROCEDURE

In this section, I explain how I conducted my analysis. I briefly describe the computer program that I used. Then I discuss rationale concerning selection of ground motions, including information about the ground motions themselves. Next I summarize the analytical steps. I conclude the section with descriptions of the structural models.

About SeismoStruct

I used SeismoStruct for the analysis. It is a finite element software package distributed freely by SeismoSoft for non-commercial purposes. The program considers geometric nonlinearity and material inelasticity. It is capable of performing seven different analysis types, though I only used nonlinear acceleration time history, Eigen value, and static.

Ground Motions

I selected near-fault ground motions exclusively, and all records came from the PEER NGA database. Kalkan and Graizer provided guidance (2007). A variety of V/H ratios were represented: values ranged from 0.50 to 3.77. However, it must be stated that exploring the effect of the V/H ratio on structural response is beyond the scope of this thesis. Structural response to ground motion is complicated, and to examine the V/H ratio in isolation is an oversimplification of structural response. I scaled all records such that the greatest peak ground acceleration (PGA) was 2 g. For example, the 1992 Cape Mendocino record, as recorded from the Cape

Mendocino station, has PGAs of 1.50, 1.04, and 0.75 g in the two horizontal and vertical directions, respectively. So, I multiplied all three records by a factor of 1.33 so that the PGAs became 2.00, 1.38, and 1.00 g. I did this to insure inelastic response of the structures. The following two tables display important characteristics of the ground motions, including moment magnitude, PGAs, V/H, and closest distance to fault. Please note that the closest distance to fault is less than 15 km for each record. For more information, visit http://peer.berkeley.edu/nga/Appendix.

Table 1

Summary of Ground Motions: Acceleration Information

Year	Event	Station	PGA (g)			V/H
			Hor. 1	Hor. 2	Vert.	
1992	Cape Mendocino	Cape Mendocino	1.50	1.04	0.75	0.50
1999	Chi-Chi, Taiwan-06	TCU079	0.77	0.62	0.58	0.75
1990	Manjil, Iran	Abbar	0.51	0.50	0.54	1.05
1994	Northridge-01	Arleta	0.34	0.31	0.55	1.61
1985	Nahanni, Canada	Site 1	0.98	1.10	2.09	1.90
1979	Imperial Valley-06	El Centro Array #6	0.41	0.44	1.66	3.77

Table 2

Summary of Ground Motions: Moment Magnitude and Closest Distance

Year	Event	Station	M_w	Distance (km)
1992	Cape Mendocino	Cape Mendocino	7.0	7.0
1999	Chi-Chi, Taiwan-06	TCU079	6.3	10.1
1990	Manjil, Iran	Abbar	7.4	12.6
1994	Northridge-01	Arleta	6.7	8.7
1985	Nahanni, Canada	Site 1	6.8	9.6
1979	Imperial Valley-06	El Centro Array #6	6.5	1.4

Analytical Procedure

My method is best explained through an example. First, I analyzed the short-period frame using the X and Y, or horizontal, components of the 1992 Cape Mendocino earthquake. Then I recorded the results. Next, I re-analyzed the short-period frame using all three components (X, Y, and Z) of the 1992 Cape Mendocino earthquake. Finally I analyzed the braced short-period frame using all three components of the 1992 Cape Mendocino Earthquake. I repeated this process for the remaining five ground motions. Then I repeated all of the previous steps for the medium-period and the long-period frames.

The time steps for the ground motion records vary from 0.005 to 0.02 sec. In order to shorten analysis time, I set the program to report results for every 0.02 sec. To clarify, if a record had time steps of 0.005 sec, SeismoStruct used 0.005 sec time steps, but the program only displayed results every 0.02 sec.

As mentioned previously, I desired inelastic response in each analysis. To accomplish this, I told the program to notify me whenever the strain in a steel element exceeded 0.00124. This is the yield strain of steel:

$$\varepsilon_y = \frac{f_y}{E} = \frac{36}{29000} = 0.00124$$

In most analyses, several members yielded, sometimes more than once.

Structural Models

I designed the structural models to represent a range of natural periods. One structure has a short natural period (less than 0.4 sec); one structure has a medium natural period (0.4-0.8 sec); and one structure has a long natural period (greater than 0.8 sec). As indicated by the literature search, these periods interest structural and earthquake engineers. All of the structures are steel moment-resisting frames. Each member of each structure is square. Therefore, the members have the same bending properties. In other words, there is no weak axis. The models are simply dimensioned. Each bay is 10 ft by 10 ft, and each story is 10 ft tall. So, a given frame consists of cubes. I added stories to increase the natural period to a desired value. Design was guided by the strong column/weak beam idea. A dead load of 20 psf was added to each story. I wanted each floor to act as a slab. Unfortunately SeismoStruct does not model slabs. However the program gives a suggestion for modeling slab action: use truss elements to connect opposing corner nodes of a floor. The help menu gives the equivalent stiffness relating the slab to the truss elements as well as modeling tips. The relationship between the stiffness of one brace and the slab is

$$\frac{EA}{L} = 0.35\left\{\frac{1}{\frac{L^3}{12EI} + \frac{L}{AG}}\right\}$$

where

E = Young's modulus (ksi)

A = Cross-sectional area (in^2)

I = Moment of Inertia (in^3)

G = Shear modulus (ksi)

The left side of the equation represents the axial stiffness of the truss element, and the right side of the equation represents the stiffness of the concrete slab. The length and modulus of elasticity of the steel brace are fixed for all models, so I had to solve the equation for the cross sectional are of the brace. I used 6,000 psi concrete with a thickness of 5.3431 in and Poisson's ratio equal to 0.2. The length of the concrete slab is 10 ft, same as the width of the frame. For the truss element, E = 29,000 ksi and L = 169.71 inches.

The modulus of elasticity of the concrete is

$$E = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{6{,}000} = 4{,}415.2\ ksi$$

And the shear modulus is

$$G = \frac{E}{1 + 2v} = \frac{4{,}415.2}{1 + 2(0.2)} = 1{,}839.7\ ksi$$

The moment of inertia of the slab is given by

$$I = \frac{1}{12} Lt^3 = \frac{1}{12}(120)(5.3431)^3 = 1{,}525.4\ in^3$$

Therefore, the cross-sectional area of one truss element is equal to

$$A = 0.35\left(\frac{169.71}{29{,}000}\right)\left\{\frac{1}{\frac{(120)^3}{12(4{,}415.2)(1{,}525.4)} + \frac{120}{(120 * 5.3431)(1{,}839.7)}}\right\} = 0.095344\ in^2$$

In all, there are three basic structural models. For expedience, I will refer to them as short-period frame, medium-period frame, and long-period frame. Each of the frames has a braced version of itself. The bracing is on every story and only one side. It is moment-resisting cross-bracing. The braced models will be referred to as short-period frame with bracing, etc.

Steel Model

SeismoStruct includes a number of default material models. For the steel, I used a bilinear model with kinematic strain hardening. By doing so, I was able to model material inelasticity. There are five material properties: modulus of elasticity, yield strength, strain-hardening parameter, specific weight, and fracture strain. I kept the default settings, except for yield strength; I used 36 ksi steel. The strain-hardening parameter is the ratio between post-yield stiffness and initial elastic stiffness. In other words, it accounts for the inelasticity. In SeismoStruct, a fracture strain of zero means that the material cannot fracture. See Table 3 for a list of material properties.

Table 3

Steel Properties

Modulus of Elasticity (ksi)	29006.51
Yield Strength (ksi)	36
Strain Hardening Parameter (-)	0.005
Specific Weight (kip/in^3)	0.0002873
Fracture Strain (in/in)	0

Short-Period Frame

Figure 1 displays a perspective view of the short-period frame without any bracing. The arrows located at the supports of the structures indicate applied ground accelerations. The large blocks located at each support indicate restraints. All supports are fixed. Each beam and column consists of four elements or five nodes. Each end element is as long as 15% of the total member length. This subdivision system is the default setting of the SeismoStruct program. The truss elements across the top of the frame cause slab-action.

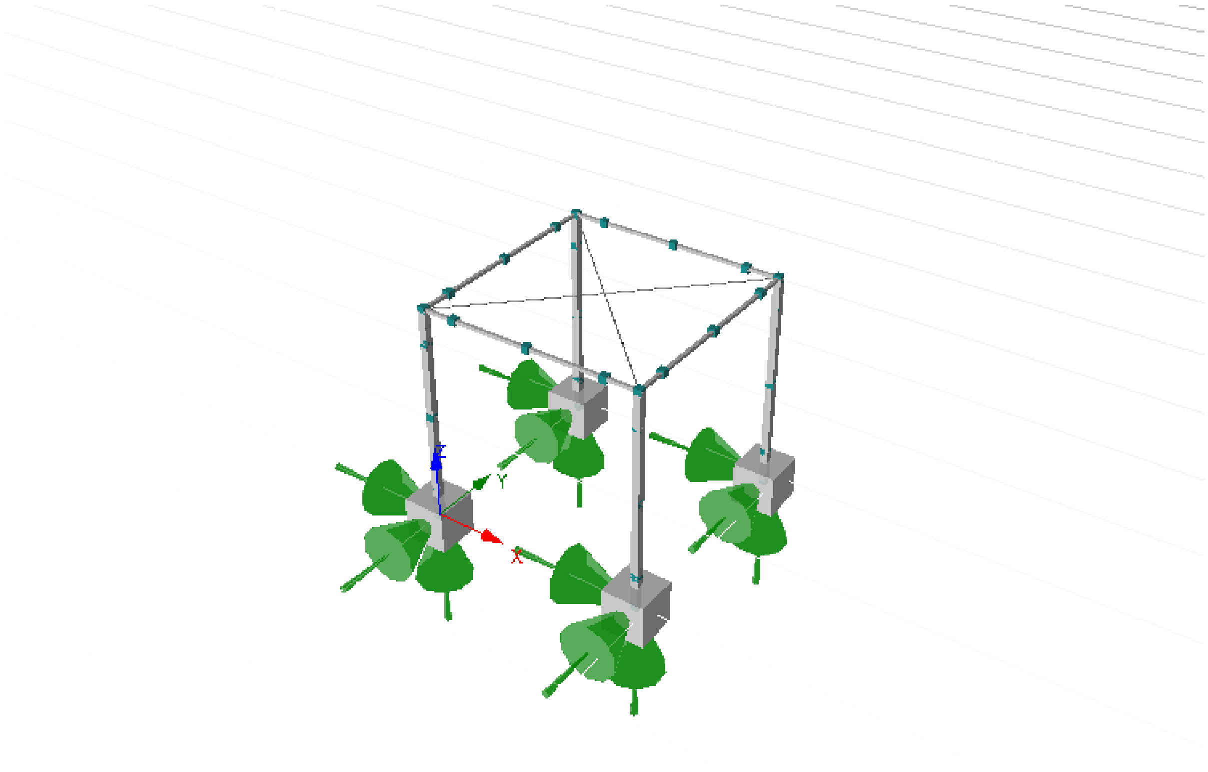

Figure 1. Short-Period Frame

Table 4 displays the structural dimensions of the short-period frame. Please note that 1st story brace is only included in the short-period frame with brace structural model. Therefore, the 1st story brace is not visible in Figure 1.

Table 4

Structural Dimensions of Short-Period Frame

	Length (in)	Area (in^2)
1st Story Column	120	16
1st Story Beam	120	3.24
1st Story Brace	169.71	2.25

Medium-Period Frame

Figure 2 displays the medium-period frame analytical model. It is similar to the short-period frame in that each bay and frame is 10 ft by 10 ft. However, this frame is 30 ft tall rather than 10 ft tall. I had to change beam and column dimensions to account for increased structural weight. This data is shown in Table 5.

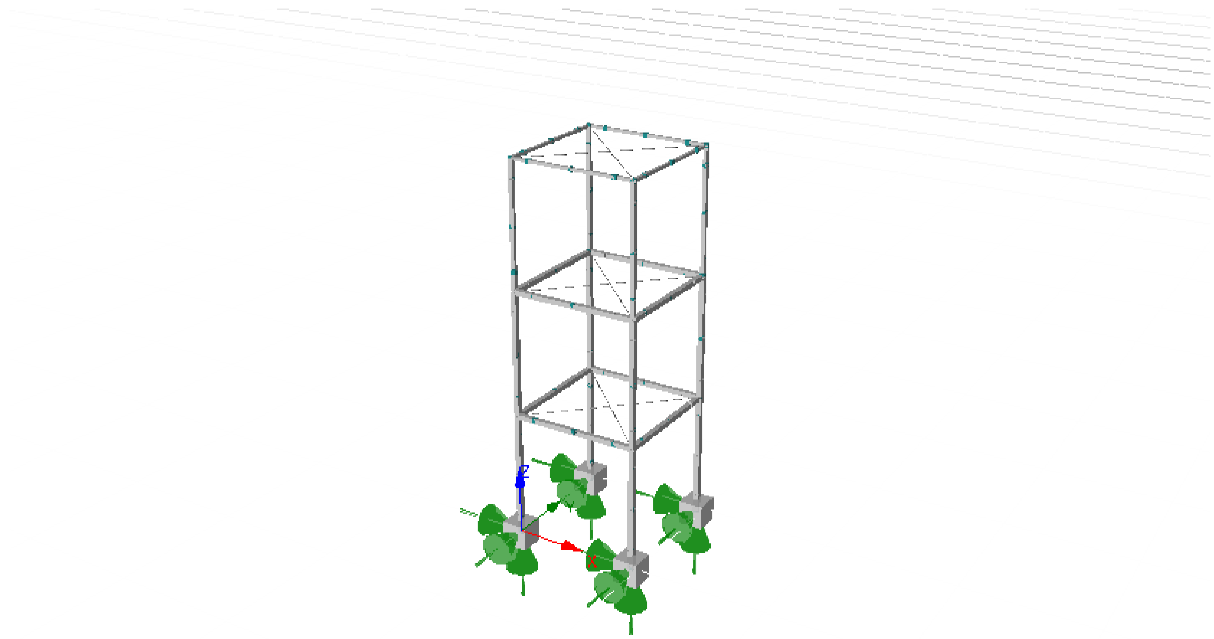

Figure 2. Medium-Period Frame

Table 5

Structural Dimensions of Medium-Period Frame

	Length (in)	Area (in^2)
1st Story Column	120	25
2nd Story Column	120	25
3rd Story Column	120	16
1st Story Beam	120	22.09
2nd Story Beam	120	22.09
3rd Story Beam	120	7.84
1st Story Brace	169.71	4
2nd Story Brace	169.71	4
3rd Story Brace	169.71	1

Long-Period Frame

Figure 3 displays the analytical model of the long-period frame. It is six stories tall with a total height of 60 ft. See Table 6 for structural details. Similar to the previous models, truss elements provide slab action at each floor. Unlike the other models, the long-period frame with cross-bracing only has cross-bracing at the first story level. This is explained in the results section.

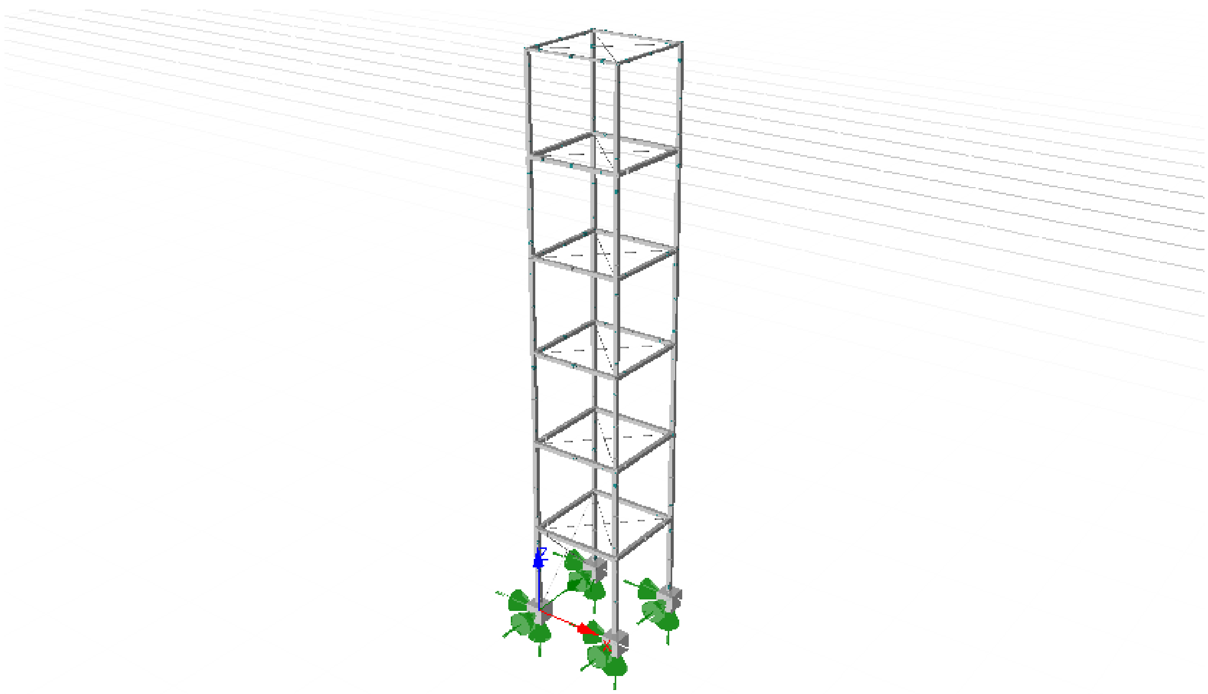

Figure 3. Long-Period Frame

Table 6

Structural Dimensions of Long-Period Frame

	Length (in)	Area (in^2)
1st Story Column	120	36
2nd Story Column	120	36
3rd Story Column	120	30.25
4th Story Column	120	30.25
5th Story Column	120	30.25
6th Story Column	120	25
1st Story Beam	120	30.25
2nd Story Beam	120	30.25
3rd Story Beam	120	25
4th Story Beam	120	25
5th Story Beam	120	25
6th Story Beam	120	9
1st Story Brace	169.71	1

CHAPTER 3

RESULTS

Short-Period Frame

Eigenvalue Analysis

The natural period of the frame was 0.25 sec for the first mode of vibration. However, the effective modal mass was 78 % in the X-direction, indicating a mixed mode response.

Time History Analysis

Figure 4 displays the maximum percent story drift for the short-period frame in the X- and Y-directions. It is divided into three sections, one for each analysis: XY; XYZ; and XYZ with cross-bracing. Each data point represents the maximum percent story drift obtained from time history analysis for a given earthquake record. The red circles and bars are the averages and standard deviations of a given set of data. To examine the inclusion of vertical ground motion, I use a relative percent difference:

$$\text{Change (\%)} = \frac{\text{(Mean Value for XYZ)-(Mean Value for XY)}}{\text{(Mean Value for XY)}} * 100\%$$

I examine the effect of cross-bracing similarly:

$$\text{Change (\%)} = \frac{\text{(Mean Value for XYZ w/ Bracing)-(Mean Value for XYZ)}}{\text{(Mean Value for XYZ)}} * 100\%$$

The differences between the XY and XYZ analyses were -1.35 and 1.90 % in the X- and Y- directions, respectively. The differences between the XYZ and XYZ w/

bracing analyses were 28.3 and -26.3 % in the X- and Y-directions, respectively. Note that the cross-bracing lies in the y-plane. In other words, the maximum percent story drift increased by 28.3 % in the direction perpendicular to the plane of the cross-bracing and decreased by 26.3 % in the direction parallel to the plane of the cross-bracing. Therefore, the inclusion of the vertical component of ground motion had little effect on story drift. As discussed above, cross-bracing significantly increased story drift in the direction perpendicular to the plane of the cross-bracing and significantly decreased story drift in the direction parallel to the plane of the cross-bracing.

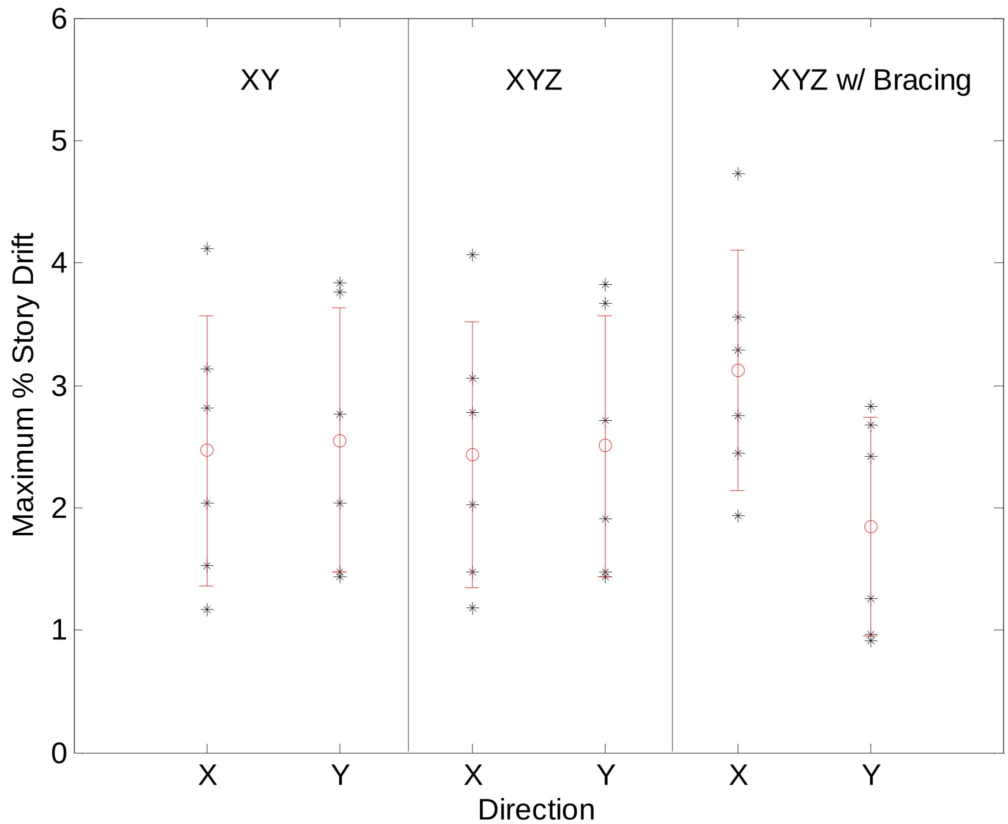

Figure 4. Max. % Story Drift: Short-Period Frame

Please note that I calculated the story drift based on the relative change in position from one particular node to the base node. Also, I observed column shear, bending moment, axial force, and torsion for a given column. Figure 5 displays the column and node under consideration. For the medium-period and long-period frames, I considered the same column and node in addition to the columns and nodes above them.

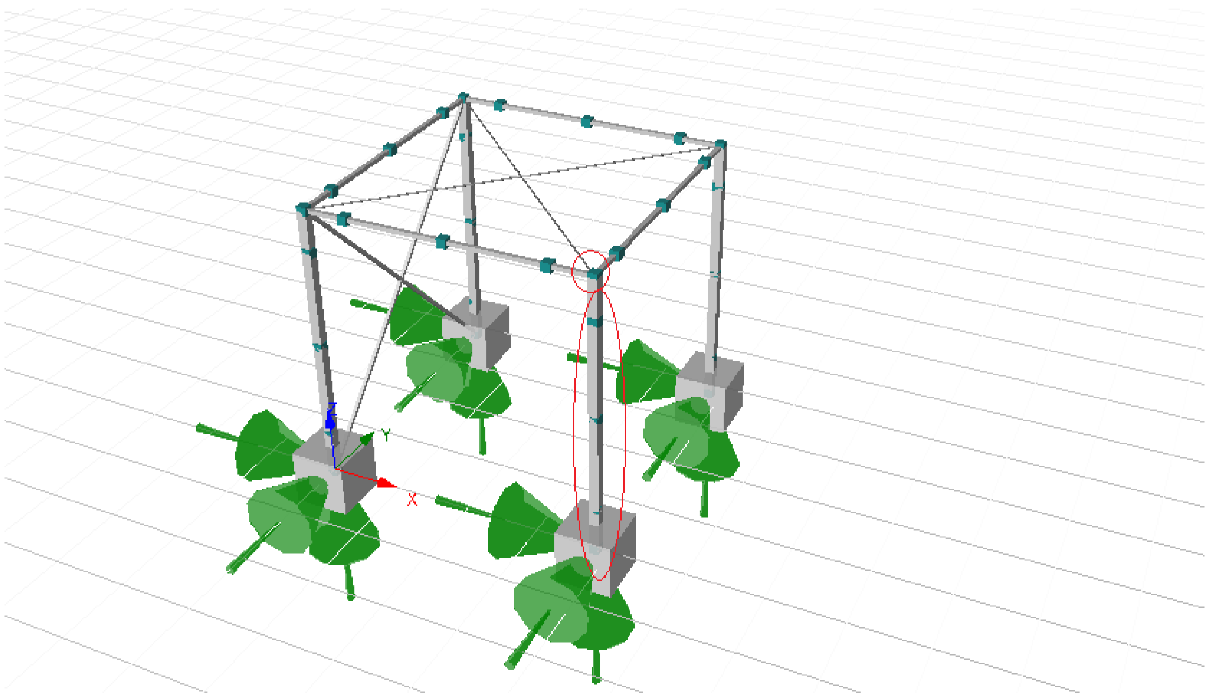

Figure 5. Column and Node under Consideration

Figure 6 displays the maximum and minimum axial forces for the short-period frame. The axial force in the column due to dead load was -1.12 kips, and I subtracted this value from the minimum and maximum axial forces from time history analysis. The differences in minimum and maximum axial force between the XY and XYZ analyses were 140 and 212 %, respectively. I.e. the compressive force increased by 140 %, and the tensile force increased by 212 % with the inclusion of the vertical component of ground motion. For the XYZ and XYZ with bracing analyses, the differences in minimum and maximum axial force were -2.49 and -0.236 %, respectively. Inclusion of the vertical component of ground motion greatly affected the axial force response. Conversely, the addition of cross-bracing negligibly affected axial force response.

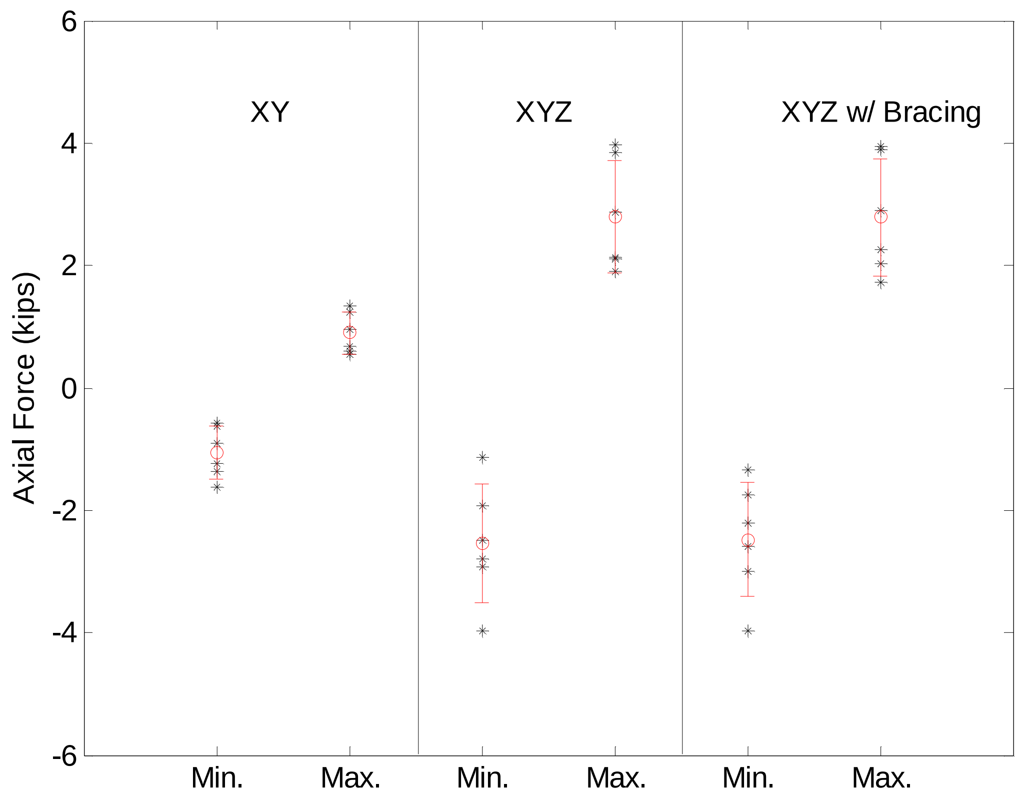

Figure 6. Min. and Max. Axial Force: Short-Period Frame

Figure 7 and Figure 8 display the absolute maximum shear at the column ends, i.e., the base of the column and the 1st story. The differences between the XY and XYZ analyses for base shear were -0.338 and 1.22 % in the X- and Y-directions, respectively. For the XYZ and XYZ with bracing analyses, the differences were -

24.3 and 20.4 % in the X- and Y-directions, respectively. Looking at first story shear, differences between the XY and XYZ analyses were less than 1 % in both directions. Differences between the XYZ and XYZ with bracing analyses were -31.6 and 21.3 % in the X- and Y-directions, respectively. So, the inclusion of the vertical component of ground motion had almost no effect on shear response. However, the addition of cross-bracing caused a significant decrease in shear response in the X-direction and a significant increase in the Y-direction. This was true for both column ends.

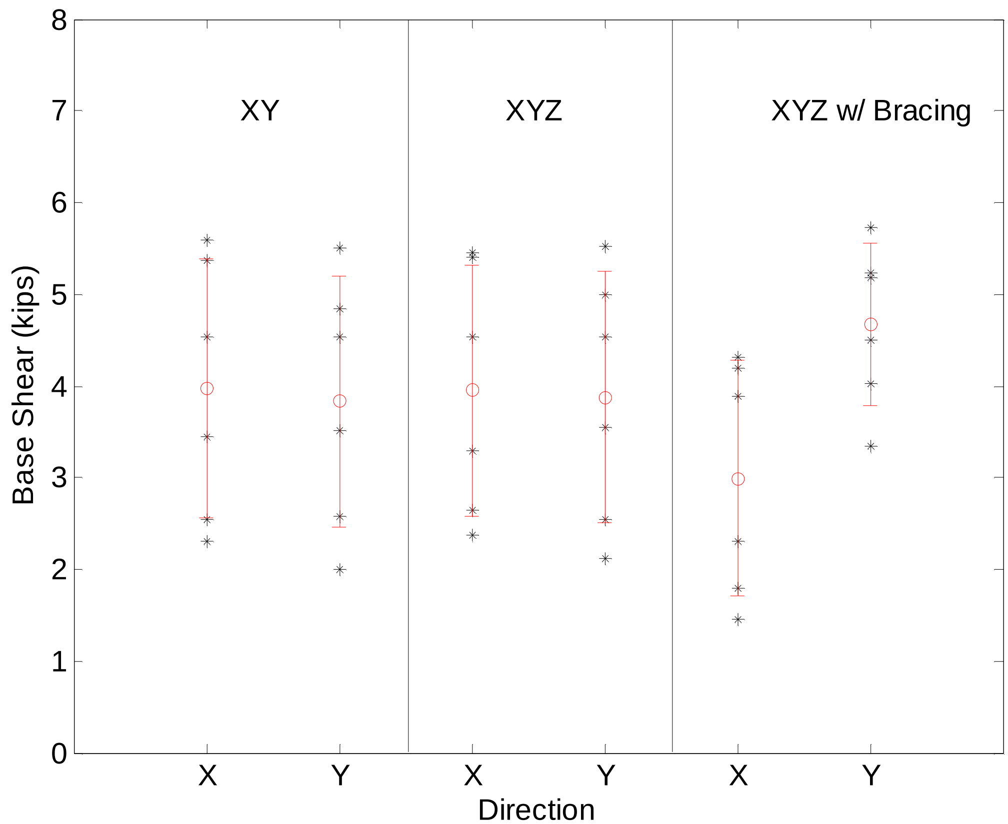

Figure 7. Max. Base Shear: Short-Period Frame

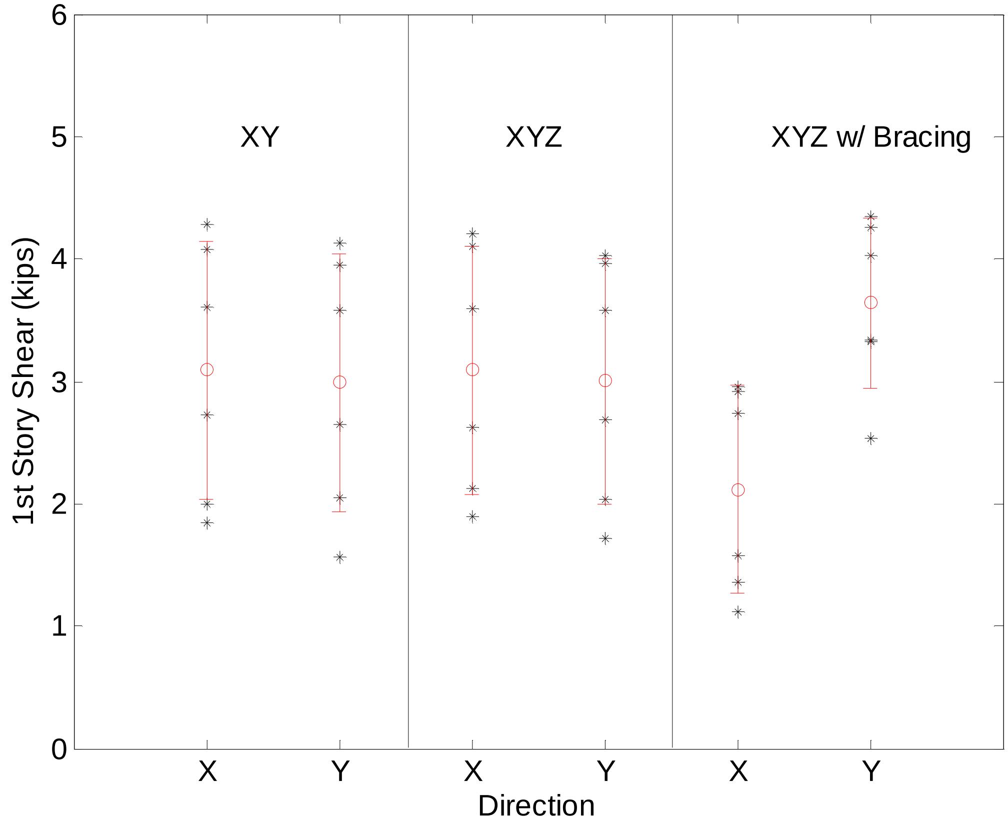

Figure 8. Max. 1st Story Shear: Short Period Frame

Figure 9 displays the absolute maximum torsion in the column under consideration. The difference between the XY and XYZ analyses was -1.87 %, and the difference between the XYZ and XYZ with bracing analyses was 2.32 %. Neither the inclusion of the vertical component of ground motion nor the addition of cross-bracing significantly affected the torsion response.

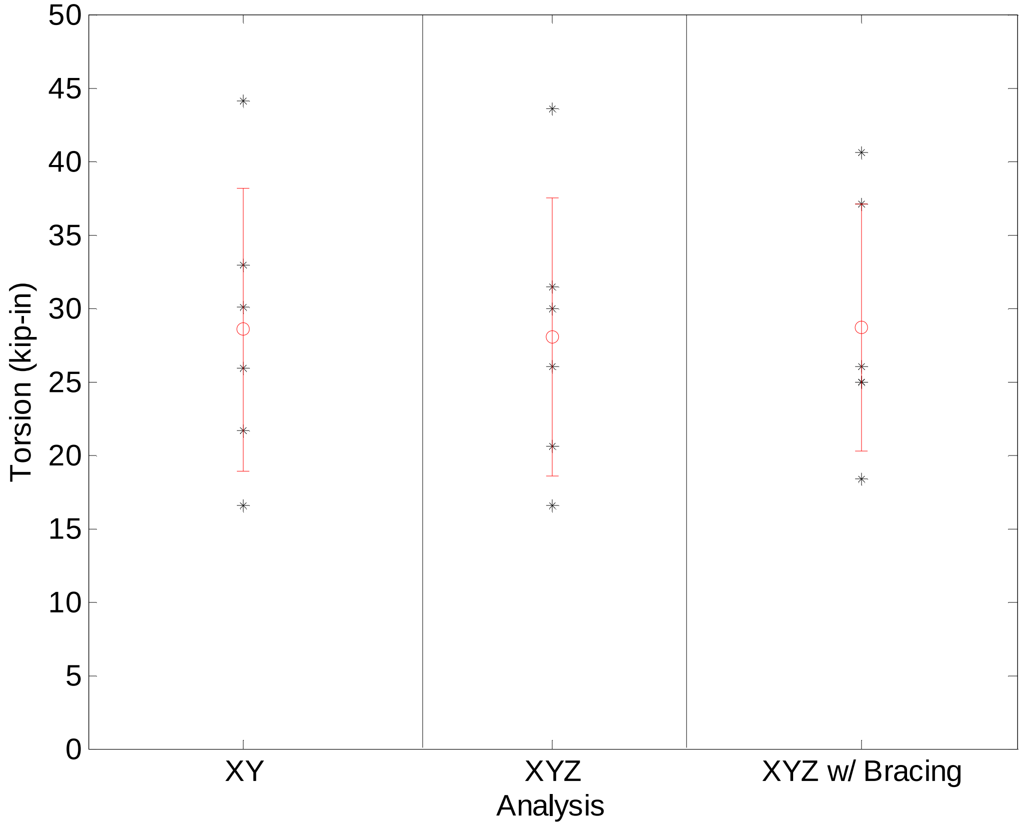

Figure 9. Torsion: Short-Period Frame

Figure 10 and Figure 11 display the absolute maximum bending moments at the column ends. Since this is a one-story frame, these moments are at the base and height H=120". At the base, the differences between the XY and XYZ analyses were less than 1 % and -5.04 % in the X- and Y-directions, respectively. For the XYZ and XYZ with bracing analyses, the differences were 24.0 and -23.2 % in the X- and Y-directions respectively. At H=120, the differences between the XY and XYZ analyses were 15.3 and 12.2 % in the X- and Y-directions, respectively. For the XYZ and XYZ with bracing analyses, the differences were 10.3 and -7.96 % in the X- and Y-directions respectively. As seen in the figures, the maximum bending moment is considerably smaller at H=120" than it is at the base. To summarize, at the base of the column, inclusion of the vertical component of ground motion had little effect on the bending moment response. However, at H=120", bending moment response increased in both directions. The addition of cross-bracing caused a significant increase in the bending moment about the X-axis and a significant decrease in the bending moment about the Y-axis. Remember that the longitudinal axis of the cross-bracing is parallel to the Y-axis and perpendicular to the X-axis.

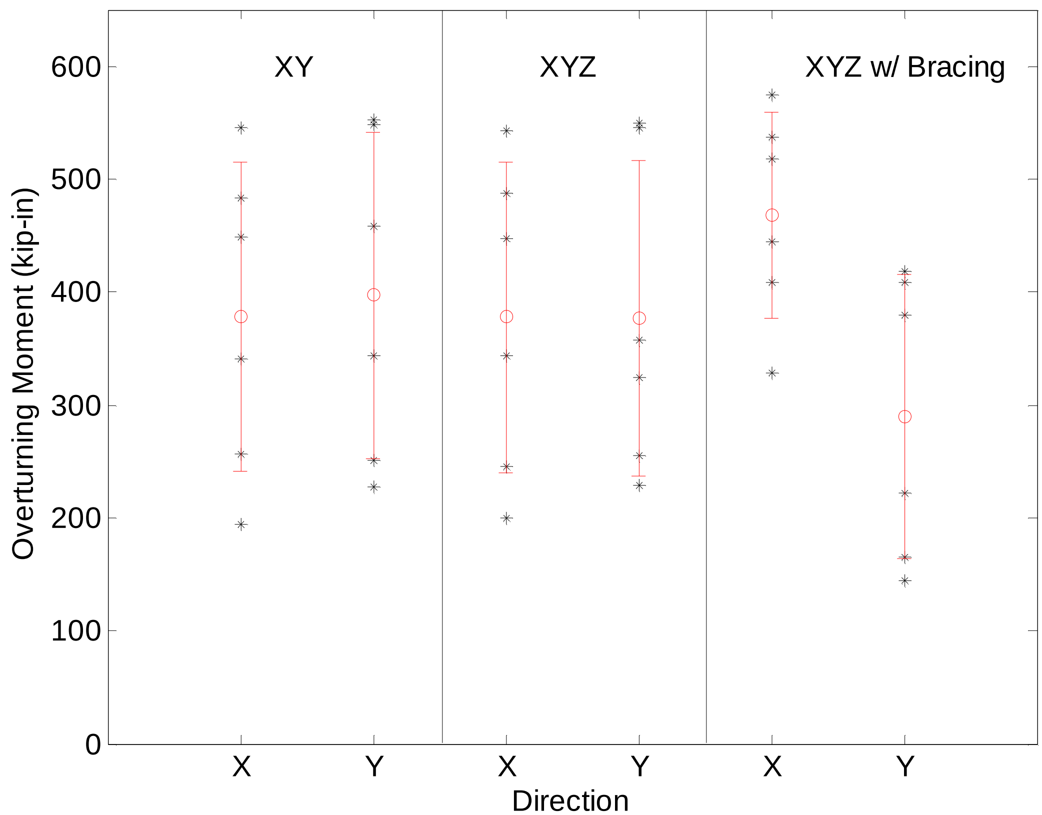

Figure 10. Max. Overturning Moment: Short-Period Frame

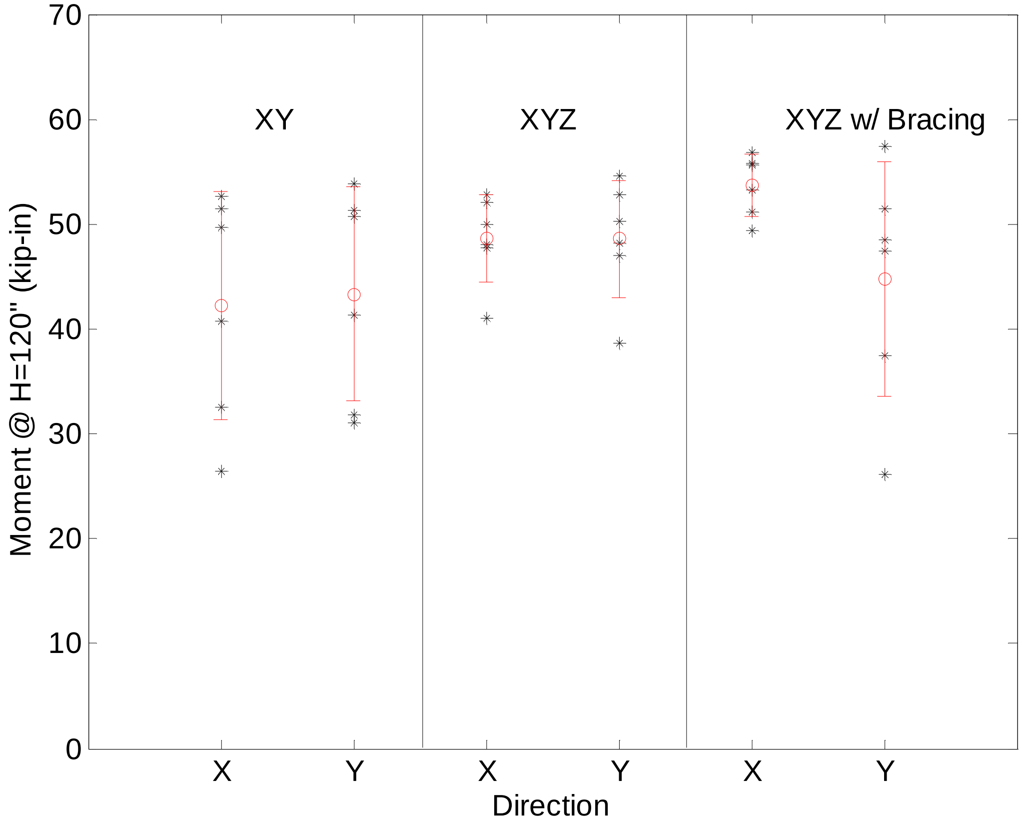

Figure 11. Max. M at H=120": Short-Period Frame

Medium-Period Frame

I had to make one deviation from the established procedure for the medium-period frame. Analysis with the Chi-Chi ground motion record at 2 g resulted in an unstable structure. Therefore, I scaled the record down to 1.5 g. This resulted in stable structural model that had significant inelastic response.

Since this structure is much larger than the short-period frame, the number of figures detailing structural response becomes cumbersome.

Eigenvalue Analysis

The natural period of the frame was 0.45 sec for the first mode of vibration. However, the effective modal mass was 73 % in the Y-direction, indicating a mixed mode response.

Time History Analysis

Table 7 displays the relative percent change in story drifts. Maximum percent story drift changed by less than 1 % in all instances with the vertical component of ground motion added. Cross-bracing caused notable increases in maximum percent story drift in the X-direction and notable decreases in the Y-direction. This is similar to the results of the short-period frame.

Table 7

Change (%) in Story Drifts: Medium-Period Frame

		XY vs. XYZ		XYZ vs. XYZ w/ Bracing	
Direction		X	Y	X	Y
Change (%)	1st Story	-0.40	-0.52	19.11	-21.55
	2nd Story	-0.67	0.17	14.09	-27.37
	3rd Story	-0.25	0.73	11.08	-26.45

The minimum and maximum axial force increased by 10.3 and 7.99 %, respectively, with the inclusion of the vertical component of ground motion. I.e. maximum compression and tension both increased. However, this change is nowhere near as dramatic as that seen in the short-period frame. The minimum and maximum axial force decreased by 13.9 and 12.4 %, respectively, when I added cross-bracing. The dead load at the base of the column was subtracted from the minimum and maximum axial force values.

Table 8 displays the percent change in absolute maximum column shear. At the base and heights H=120” and 360”, there was little change with the inclusion of the vertical component of ground motion. However, at H=480”, i.e. the top of the structure, shear response increased considerably in both directions. The changes due to cross-bracing were mixed; all stories experienced reduced shear in the X-direction. However, shear force increased in the Y-direction for the lower stories and decreased slightly for the upper stories.

Table 8

Change (%) in Max. Column Shear: Medium-Period Frame

		XY vs. XYZ		XYZ vs. XYZ w/ Bracing	
Direction		X	Y	X	Y
Change (%)	H=0	2.31	3.44	-16.74	13.30
	H=120"	2.06	4.04	-22.47	17.07
	H=360"	-0.03	12.81	-19.00	-2.16
	H=480"	11.36	20.76	-30.71	-7.45

Table 9 displays the changes in absolute maximum torsion response. Interestingly, torsion increased with story height with the inclusion of the vertical component of ground motion. Torsion increased drastically with cross-bracing.

Table 9

Change (%) in Max. Column Torsion: Medium-Period Frame

		XY vs. XYZ	XYZ vs. XYZ w/ Bracing
Change (%)	1st Story	18.86	578.54
	2nd Story	38.00	468.68
	3rd Story	286.85	170.00

Table 10 displays the percent change in absolute maximum bending moment in the column under consideration. The inclusion of the vertical component of ground motion altered bending moment response by less than 1 % in all instances. Absolute maximum bending moment increased about the X-axis and decreased about the Y-axis with the addition of cross-bracing. This result is similar to the short-period frame.

Table 10

Change (%) in Max. Column Bending Moment: Medium-Period Frame

		XY vs. XYZ		XYZ vs. XYZ w/ Bracing	
Direction		X	Y	X	Y
Change (%)	H=0	-0.06	0.57	15.16	-18.24
	H=120"	-0.11	-0.89	17.77	-21.14
	H=360"	-0.49	0.00	16.56	-17.64
	H=480"	0.75	0.87	12.82	-10.76

Long-Period Frame

Again, I had to deviate from the established procedure for the long-period frame. The structure was unstable during the Imperial Valley ground motion record at 2 g. Consequently, I scaled the record down so that the PGA was 1.5 g. The structure exhibited inelastic response without failure. Also, the structure was unstable for analysis with the Chi-Chi ground motion record, even with low PGAs. Therefore, I only neglected to use that record. Rather, I used five of the six ground motion records for analysis of the long-period frame.

I made another noticeable change by only using cross-bracing at the first story. I experimented with many different configurations, and I continually ran into the same problem: structural model instability. The solution for a particular time step would not converge. The final design was the only successful one out of a half dozen. So, although it does not reflect the previous structural models, it does introduce a degree of eccentricity.

Eigenvalue Analysis

The natural period of the frame was 0.80 sec for the first mode of vibration, and the effective modal mass was 93 % in the X-direction.

Time History Analysis

Table 11 displays the percent change in the absolute maximum percent story drifts. The percent change was less than one percent for nearly all stories in all directions when the vertical component of ground motion was included in analysis. Results varied considerably for the analysis with cross-bracing. No conclusions may be drawn regarding the effects of cross-bracing for the long-period structure. However, the data shows that the vertical component of ground motion had no effect on story drift.

Table 11

Change (%) in Story Drifts: Long-Period Frame

		XY vs. XYZ		XYZ vs. XYZ w/ Bracing	
Direction		X	Y	X	Y
Change (%)	1st Story	-0.36	0.15	4.22	-20.59
	2nd Story	-0.15	0.10	0.38	-6.09
	3rd Story	0.00	-0.25	0.57	-3.60
	4th Story	0.07	0.18	-1.23	6.46
	5th Story	-0.25	-1.13	3.13	16.15
	6th Story	-0.56	-0.41	2.24	22.82

The average minimum and maximum axial force increased by 9.99 and 5.46 %, respectively, when comparing the XYZ analysis to the XY analysis. Minimum axial force increased by 11.6 %, and the maximum axial force decreased by 1.45 % when comparing analysis with cross-bracing to the XYZ analysis. The dead load at the column was subtracted from the maximum and minimum axial forces.

Table 12 displays the percent change in absolute maximum column shear. For the most part, the greatest changes in shear occurred at the higher stories, i.e. stories 4 and up. This may be due to the change in stiffness above the second floor.

Table 12

Change (%) in Max. Column Shear: Long-Period Frame

		XY vs. XYZ		XYZ vs. XYZ w/ Bracing	
Direction		X	Y	X	Y
Change (%)	H=0	0.13	-1.18	-10.61	6.75
	H=120"	1.77	-1.04	5.43	1.84
	H=240"	10.45	4.67	2.23	2.97
	H=360"	9.80	5.70	5.45	-0.87
	H=480"	12.24	1.70	13.54	0.28
	H=600"	9.95	11.76	21.82	2.07
	H=720"	13.10	10.50	15.55	3.38

Table 13

Change (%) in Max. Column Torsion: Long-Period Frame

		XY vs. XYZ	XYZ vs. XYZ w/ Bracing
Change (%)	1st Story	13.45	127.28
	2nd Story	8.29	118.80
	3rd Story	19.25	223.90
	4th Story	12.23	73.50
	5th Story	25.51	22.90
	6th Story	14.50	34.19

Table 13 indicates moderate increases in absolute maximum torsion with the inclusion of the vertical component of ground motion. Cross-bracing drastically increased torsion in the first four stories. However, for the fifth and sixth stories, the data was inconclusively scattered. Figure 12 provides an example.

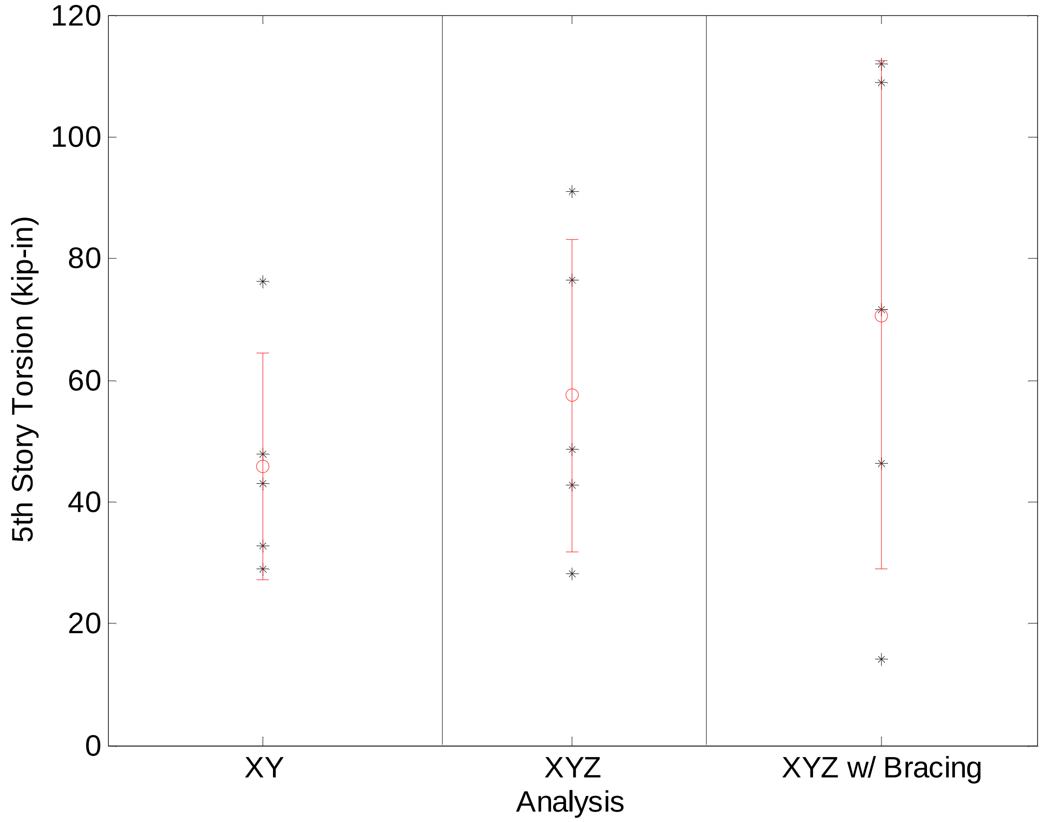

Figure 12. Max. Column Torsion: 5th Story: Long-Period Frame

Table 14 displays the percent change in absolute maximum column bending moment. The greatest changes occurred in the upper stories for each comparison. The inclusion of the vertical component of ground motion had little effect on bending moment response. Comparing XYZ analysis to XYZ with bracing, bending moment increased about the Y-axis.

Table 14

Change (%) in Max. Column Bending Moment: Long-Period Frame

		XY vs. XYZ		XYZ vs. XYZ w/ Bracing	
Direction		X	Y	X	Y
Change (%)	H=0	-0.66	-0.20	4.11	-10.30
	H=120"	-0.08	0.33	0.41	11.54
	H=240"	-0.32	-0.61	0.66	1.79
	H=360"	1.78	0.20	2.59	9.97
	H=480"	-1.93	-1.05	2.49	17.23
	H=600"	0.37	1.00	5.12	19.34
	H=720"	5.53	8.16	1.10	17.36

CHAPTER 4

DISCUSSION

The results of the analysis of the short-period frame provide the clearest trends. The vertical component of ground motion had no effect on story drift. However, axial force response increased drastically with a change as high as 200 %. The change in shear at the column ends was negligible. Likewise, there was no change in column torsion. At the base of the column, the absolute maximum bending moment did not significantly change, although there was a slight increase at the other column end on the order of 10 %. Cross-bracing increased story drift in the direction perpendicular to the plane of the cross-bracing by nearly 30 %; it decreased story drift in the direction parallel to the plane of the cross-bracing to a similar degree. The axial force response did not change. The shear force at the column ends decreased significantly in the X-direction, i.e. perpendicular to the cross-bracing and increased by similar values in the Y-direction. Changes were around 20 to 30 %. There was no change in torsion in the column. Finally, bending moment at the column ends increased about the X-axis, i.e. in direction of the cross-bracing and decreased about the Y-axis. The changes were more significant at the base of the column.

Looking at the medium-period frame, the vertical component of ground motion had no effect on story drift. Axial force response increased by as much as 10 %, though this change pales in comparison to the change in the short-period frame. Shear response hardly changed with an exception at H=480”, i.e. the very top of the structure. There, absolute maximum shear force increased by about 10 %.

Absolute maximum column torsion increased significantly with the height of the structure. In the third story, the increase was nearly 300 %. However, the stiffness of the structure changed at that story, and this could be the cause of such a large change. The absolute maximum bending moments at the column ends did not change with the inclusion of the vertical component of ground motion. As with the short-period frame, cross-bracing increased story drift in the direction perpendicular to the cross-bracing and decreased story drift in the parallel direction. The increases varied from 10 to 20 %, and the decreases varied from 20 to 30 %. Axial force response decreased by about 10 %. Shear at the column ends decreased 15 to 30 % in the X-direction. Column torsion increased largely at all stories. In fact, the increase was nearly 600 % for the first story. Bending moment about the X-axis increased 10 to 20 % and decreased by the same magnitude about the Y-axis.

Lastly, the vertical component of ground motion had no effect on story drift for the long-period frame. Axial force response increased less than 10 %. Changes in shear at the column ends were negligible at the base and H=120". However, at greater heights, shear increased as much as 10 %. Based on data from the other structures, perhaps, the increases are a result of multiple changes in stiffness due to decreased column and beam cross-sectional area. Bending moment response did not change with the exception of a small increase at H=720". Unlike the previous structures, there was no pattern for change in story drift due to cross-bracing. The results were mixed. The minimum axial force increased by about 10 % while the maximum hardly changed. Shear force generally increased by less than 20 % in the X-direction. In the Y-direction, it increased by less than 5 %. The column torsion increased greatly at some stories and moderately at others. Though, these results may be unreliable due to a large scattering of the data. For the most part, column bending moment changed very little about the X-axis, but it increased 10 to 20 % about the Y-axis.

The purpose of this thesis was to examine the effects of the vertical component of ground motion and cross-bracing. Though I covered many different structural responses, two trends are significant. First, for each of the three structures, the maximum story drift was unaffected by the inclusion of the vertical component of ground motion. Second, for the short-period frame, inclusion of the vertical component of ground motion caused large increases in the axial force response.

In future studies, I suggest normalizing the earthquake records in some other fashion in order to lessen the deviation of the data. Perhaps using energy rather than PGA would result in a better grouping of data. Also, examining structures designed according to IBC or another code may result in more practical conclusions.

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