13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1802

INVESTIGATION OF TORSIONAL RESPONSE IN A SHAKETABLE MODEL

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SUMMARY

The purpose of this study was to investigate the inelastic response of one-story, symmetric- and asymmetric-plan steel moment-frame to biaxial lateral earthquake ground motions. Through the combined experimental and analytical study, the lateral-torsional response of the system was studied for eight different configurations of mass, strength, and stiffness eccentricity. The primary goals of this study were to examine the adequacy of current building code torsional design assumptions and the ability of analytical software to predict inelastic response, for both a model tuned to the measured dynamic properties of the actual structure and a model based on common modeling assumptions.

INTRODUCTION

The complete inelastic response of structures is of great interest because collapse is prevented during earthquakes primarily by the dissipation of energy that occurs after yielding. As long as the structure doesn't collapse, it has not failed, and this is achieved by dissipating the energy imparted to the structure by the ground through the inelastic, hysteretic behavior of the structure. In structures with low damping, this hysteretic behavior is the primary mechanism of energy dissipation. Thus, understanding the seismic behavior of structures in the inelastic region is the first step in an effective design.

One mechanism that is of particular interest in the seismic behavior of structures is torsion. Torsion is, in essence, the twisting of the structure, and when coupled with the lateral response, the forces and displacements of the various resisting elements are different from those experienced when the structure responds only in the planar directions. Torsion is the result when the mass distribution and/or stiffness distribution of a structure are not symmetric with respect to both planar axes of motion. Due to

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architectural constraints, buildings are typically designed with the aforementioned asymmetries. However, even when they are designed symmetrically, in actuality, asymmetries are present as a result of the imprecise nature of construction, the variability of the objects and people occupying the structure, etc. Thus, lateral-torsional coupling, to some degree, is a mechanism always present in the seismic response of a structure.

Overall, the structure tested and analyzed in this study was a low-period, acceleration-sensitive system. This fact increases the importance of using biaxial ground motions as Riddell and Santa-Maria (2) and Correnza and Hutchinson (1) both indicate that accurate assessment of the response of short-period systems can be achieved only by using bi-directional analyses. They show that bi-directional ground motions have the largest impact on low-period systems, and the impact increases as the eccentricity increases. The eccentricity typically found in the configurations in this study is roughly 0.4 times the radius of gyration, which is considered a moderate degree of eccentricity. Riddell and Santa-Maria also indicate that low-period systems can achieve inelastic response significantly greater than elastic response. For intermediate- and long-period systems, lateral deformations are not as affected by yielding so that elastic and inelastic systems experience, on average, essentially the same lateral deformation.

This study aims to first and foremost begin to fill part of the void in the realm of the experimental, shaketable investigation of the inelastic torsional response of structures. The planar distributions of mass, strength, and stiffness are each potentially independent factors in the seismic torsional response. Thus, this research study utilizes a model that contains any or all of the three types of eccentricity. This experimental part of this study was performed on the Triaxial Earthquake and Shock Simulator (TESS) located at CERL in Champaign, Illinois, which has full triaxial motion capability. An experimental study of this nature is a natural and necessary companion to the analytical work that has been done thus far in the seismic behavior of structures.

EXPERIMENTAL PROGRAM

Test Structure

A simple one-story system was chosen as appropriate to lay the groundwork for further experimental studies of more complex structures. The structure used in this study is basically a diaphragm, approximately two and a half meters on a side, supported by four circular steel pipe columns, one and a half meters in length, as shown in Figures 1 and 2. The diaphragm is composed of four W12x65 beams on the perimeter, with concrete used to fill the void in the center. Eight octagonal and eight rectangular steel masses were available to be attached to the slab in various configurations in order to provide dead load and mass asymmetry. The pipe columns have base- and top-plates welded to the bottoms and tops, respectively, of the columns to provide attachment points to the shaketable and to the diaphragm. The diaphragm was designed to be used throughout the entire sequence of tests, while the columns, having plastically deformed, were replaced after each model test sequence was complete. Circular pipe columns were chosen because of their complete plan symmetry and their reduced sensitivity to torsional buckling and, thus, greater stability. The focus of this study is the lateral-torsional behavior of the system as a whole, not the local behavior of the columns; non-circular columns would only add unnecessary complexity to the analysis of the system. Figure 1 depicts in particular the model setup used in Test Configuration 4, in which all of the masses were placed on the east half of the structure, resulting in a mass asymmetry along the Y-Axis. Figure 2 depicts the model in Test Configuration 3, in which all of the masses were placed on the northeast corner of the structure, resulting in a mass asymmetry along both the X-Axis and Y-Axis.

Figure 1. Test Structure with One-Way Mass Asymmetry

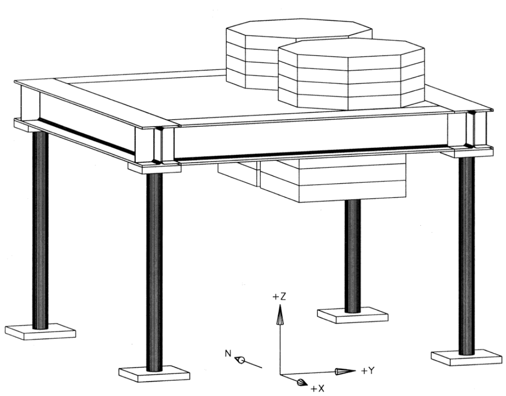
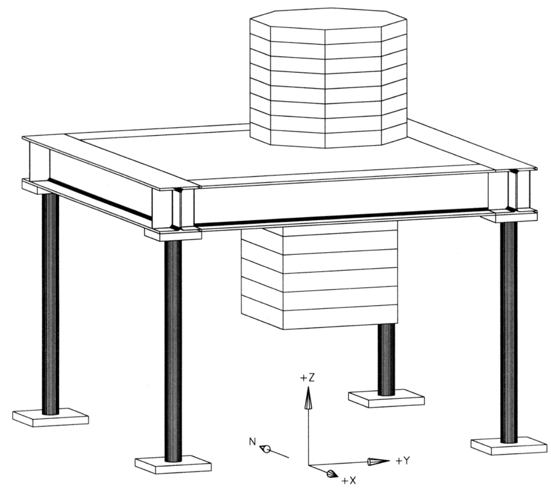


Figure 2. Test Structure with Two-Way Mass Asymmetry



Testing Procedures

In all, eight different model test sequences were studied, as shown in Table 1. The configurations consist of different combinations of varying parameters: one-quarter asymmetric mass, one-half asymmetric mass, symmetric mass, asymmetric and symmetric column strength, and concentric lateral bracing. The mass asymmetries were achieved simply by placing the masses all on one-quarter or one-half of the diaphragm. In all cases, half of the masses were attached above and half below the diaphragm to keep the vertical mass center reasonably near the center of the diaphragm. The strength asymmetry in the structure was achieved by pairing two sets of columns which have similar stiffness but different yield strengths. The stiffness asymmetry was achieved through the use of asymmetrical lateral bracing.

Table 1. Test Configuration Summary

Test Configuration	Input Motion	Mass Distribution	Stiffness Distribution	Strength Distribution
1	Uniaxial	Symmetric	Symmetric	Symmetric
2	Biaxial	Symmetric	Symmetric	Symmetric
3	Biaxial	Asymmetric [X] Asymmetric [Y]	Symmetric	Symmetric
4	Biaxial	Asymmetric [X]	Symmetric	Symmetric
5	Biaxial	Asymmetric [Y]	Symmetric	Asymmetric [Y]
6	Biaxial	Asymmetric [Y]	Asymmetric [Y]	Asymmetric [Y]
7	Biaxial	1/2 Asymmetric [Y]	Asymmetric [Y]	Asymmetric [X] Asymmetric [Y]
8	Biaxial	Asymmetric [X] Asymmetric [Y]	Symmetric	Symmetric

Although varying for each Test Configuration, the general dynamic properties of the structure were as follows: weight, $W \approx 35$ Kips; radius of gyration, $\rho \approx 38.5$ inches; uncoupled lateral frequency, $\omega_X \approx \omega_Y \approx 3.5$ Hertz; braced $\omega_Y \approx 8.0$ Hertz; uncoupled torsional to lateral frequency ratio, $\Omega \approx 1.8$; $0.5\% <$ damping ratio, $\xi < 1.5\%$; mass eccentricity, $e_M \approx 0.4\rho$; strength eccentricity, $e_P \approx 0.3\rho$; stiffness eccentricity, $e_S \approx 0.15\rho$.

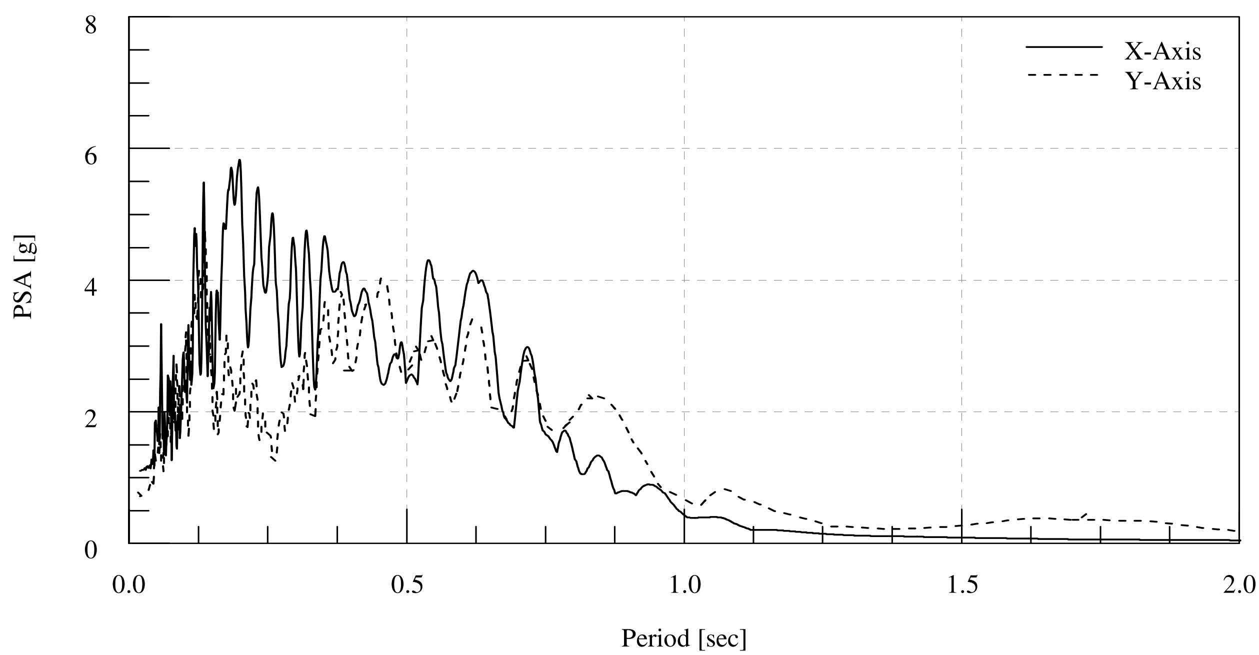
Following construction, the model was placed on the shaking table for the simulations. Approximately 72 channels of data were recorded in each test. Response quantities measured included accelerations and displacements of the table and slab in both planar directions, and strains in the columns.

The test sequences for each model configuration were nearly identical. Preliminary tests were performed before any earthquake simulations in order to determine the natural frequencies and damping characteristics of the structure. White noise, sine decay, and sine sweep tests were performed once in each planar axis, and once in the yaw-axis, for a total of nine preliminary tests for each model configuration.

For the configuration shown in Figure 1, the structure had modal frequencies of 3.47 Hz in the X direction, 3.42 in the Y direction, and 7.26 Hz about the Z-Axis. The measured damping ratios were 0.47% in the X direction, 0.87% in the Y direction, and 0.68% about the Z-Axis.

Following the preliminary tests, the structure was subjected to earthquake simulations. The 230 [X] and 140 [Y] degree acceleration components from the 1979 Imperial Valley earthquake recorded at Bonds Corner were chosen as the base earthquake motions based on the large response spectrum magnitudes near the natural frequencies of the structure. The goal in choosing the motions was to achieve a ductility in the response of the structure in the neighborhood of four to five. To this end, the motions were modified using a combination of scaling and filtering, and then checked in finite element simulations, in order to produce the desired ductility while not violating the performance limits of the shaketable. This procedure produced a set of reference ground motion accelerograms to be used on the structure. Figure 3 shows elastic response spectra for a typical set of reference ground motions.

Figure 3. Elastic Response Spectra – Reference Ground Motions [100%]



The structure was first subjected to low-level earthquake tests, typically at 10-25 percent of the reference. The low-level tests were performed using first the X-Axis input motion, then the Y-Axis input motion, followed by both axes simultaneously. Next, the structure was subjected to the full-scale accelerograms. Typically, the shaketable displacement limits were such that the earthquake accelerograms could be scaled up by 50% in order to perform a subsequent simulation. When possible, this was done, followed by white noise tests to analyze any changes in the natural frequency of the structure. This general sequence of preliminary tests and earthquake simulations was followed for each of the eight different model configurations.

Inelastic Structural Response

Figure 4 shows the inelastic force vs. displacement and torsional moment vs. rotation responses, which are no longer tight and linear but now are taking on a fuller shape indicating inelastic behavior, for Test Configuration 2 with complete mass symmetry. The peak displacements of the structure were 2.86 inches in the X direction and 1.24 inches in the Y direction, which are both larger than the yield displacements of 0.45 inches and 0.49 inches, respectively. The peak rotation of the structure was, at 2.85 x 10^{-3} radians, less than the yield rotation of 0.0064 radians. The Peak Ground Accelerations (PGAs) for this simulation were 1.080 g in the X direction and 0.699 g in the Y direction.

Figure 4. Force vs. Displacement and Torsional Moment vs. Rotation Inelastic Response – Symmetric Mass

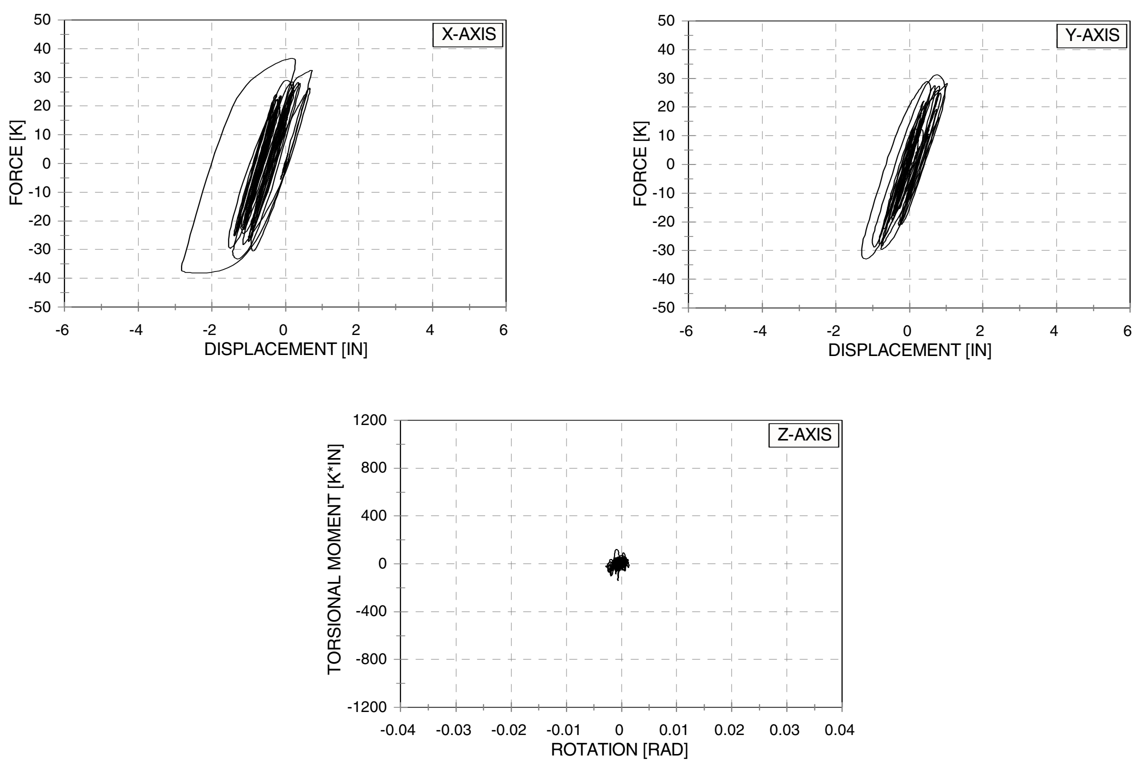
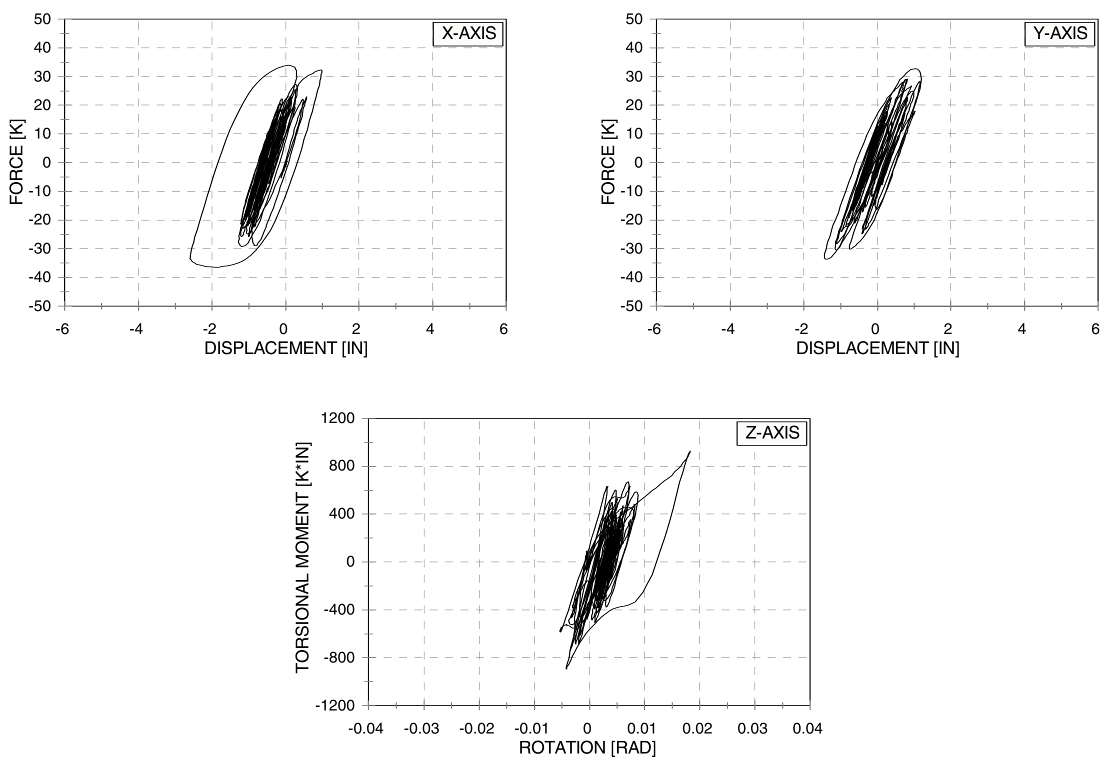


Figure 6 shows the inelastic force vs. displacement and torsional moment vs. rotation responses for Test Configuration 4 with one-way mass asymmetry about the X-Axis. The peak displacements of the structure were 2.59 inches in the X direction and 1.44 inches in the Y direction, which are both larger than the yield displacements of 0.42 inches and 0.44 inches, respectively. The peak rotation of the structure was, at

18.26 x 10^{-3} radians, less than the yield rotation of 6.2 x 10^{-3} radians. For this simulation, the X-Axis PGA was 1.115 g, while the Y-Axis PGA was 0.711 g.

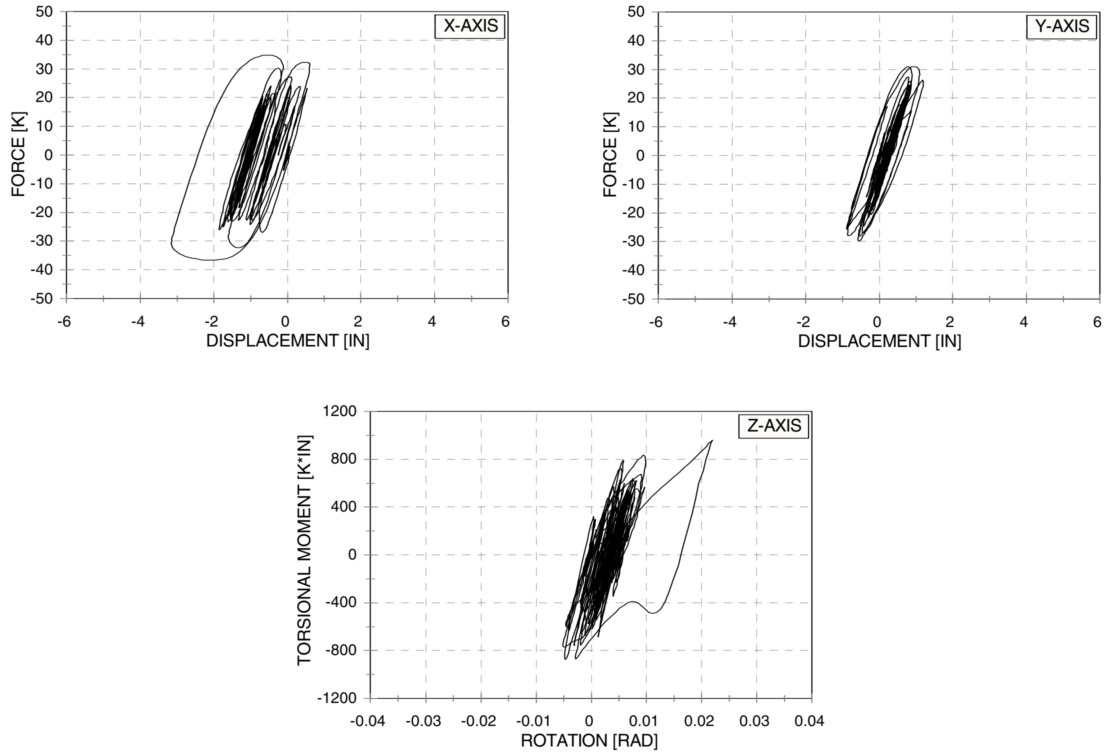
Figure 6 shows the inelastic force vs. displacement and torsional moment vs. rotation responses for Test Configuration 3 with two-way mass asymmetry about the X-Axis and Y-Axis. The peak displacements of the structure were 3.17 inches in the X direction and 1.18 inches in the Y direction, which are both larger than the yield displacements of 0.47 inches and 0.53 inches, respectively. The peak rotation of the structure was, at 21.94 x 10^{-3} radians, less than the yield rotation of 6.8 x 10^{-3} radians. For this simulation, the X-Axis PGA was 1.299 g, while the Y-Axis PGA was 0.797 g.

Figure 5. Force vs. Displacement and Torsional Moment vs. Rotation Inelastic Response – One-Way Asymmetric Mass [X-Axis]



Overall, the Y-Axis inelastic response curves are not as full as for the X-Axis, due to the fact that the Y-Axis ground motion has a lesser amplitude than the X-Axis motion. Further, the torsional response of the two-way asymmetric system is not significantly different from the one-way system. This is again because the amplitude of the Y-Axis ground motion is less than the X-Axis motion, and the additional torsional response in the two-way system is due to the eccentricity about the Y-Axis. Although the test structure experiences a rotational ductility of nearly 3 for both the one-way and two-way mass asymmetric cases, the peak torsional moment is roughly 2/3 of the yield moment. The simultaneous lateral motion of the structure interacts with the torsional motion by producing a large amount of column yielding that would be present even without the torsion, allowing the structure to experience much larger rotations than it would when subjected to torsion alone. Each of the four columns provides strength and stiffness in both the X and Y directions. The stiffness of the model in the X and Y directions working together produces the torsional stiffness of the structure. When one of the columns yields, it loses stiffness in both directions and consequently loses stiffness torsionally as well.

Figure 6. Force vs. Displacement and Torsional Moment vs. Rotation
Inelastic Response – Two-Way Asymmetric Mass



Applicability to Seismic Design

The applicability of the experimental results to the behavior of code-designed structures is important to establish before detailing the experimental results. Overall, the peak accelerations of the ground motions used in the inelastic simulations were larger than those specified by the *Seismic Provisions* for design. However, this is a product of the “backwards” approach that was taken in designing the test matrix. For this study, the structure was designed first and the ground motions were then selected and modified in order to achieve the desired displacement ductilities, rather than vice versa. Thus, making a direct comparison between the ground motions used in this study and design ground motions specified by the *Seismic Provisions* is not a good measure of the applicability to code-designed structures. A better indicator of the similarity to code-designed structures is the ductility experienced by the test structure or the effective Strength Reduction Factor. An effective Strength Reduction Factor for a particular simulation is computed as the ratio of the elastic base shear force to the yield base shear. The Seismic Provisions specifies a Strength Reduction Factor (R) of 8 for Special Steel Moment Frames and 3.5 for Ordinary Steel Moment Frames. The Strength Reduction Factors for the inelastic simulations performed in this study generally fall within this range. Only two simulations possessed an effective Strength Reduction Factor larger than 8, and in both cases the values are roughly 9.

U_X/U_Y and U_θ/U_X Ratio

Two common design methods found in the *Seismic Provisions* are the Equivalent Lateral Force procedure and the Modal Response Spectrum procedure. Both of these methods use elastic analysis to design for inelastic response. The use of these methods to accurately predict inelastic response then is based on the basic assumptions that U_Y/U_X, or the ratio of the two peak lateral displacements, and U_θ/U_X, or the ratio of the peak rotational displacement to the peak lateral displacement, remain constant regardless of whether the structure responds elastically or inelastically, and also regardless of the amount of structural asymmetry. Shown in Table 2 are the lateral displacement ratio and torsional to lateral displacement ratio for four different configurations.

Table 2. Lateral Displacement Ratio and Torsional to Lateral Displacement Ratio

Configuration	TC	U_Y / U_X	U_θ / U_X
Symmetric Mass 25% Biaxial	2	0.73	N/A
Symmetric Mass 100% Biaxial	2	0.43	N/A
Symmetric Mass 150% Biaxial	2	0.80	N/A
Asymmetric Mass [X-Axis] 10% Biaxial	4	0.86	0.29
Asymmetric Mass [X-Axis] 100% Biaxial	4	0.51	0.35
Asymmetric Mass [X-Axis] 150% Biaxial	4	0.85	0.35
Asymmetric Mass [X & Y-Axis] 10% Biaxial	3	0.96	0.28
Asymmetric Mass [X & Y-Axis] 100% Biaxial	3	0.37	0.33
Asymmetric Mass [X & Y-Axis] 100% Biaxial	8	0.64	0.32
Asymmetric Mass [X & Y-Axis] 135% Biaxial	8	1.04	0.30
Asymmetric Mass [Y-Axis] Strength Asymmetry [Y-Axis] 10% Biaxial	5	0.29	0.14
Asymmetric Mass [Y-Axis] Strength Asymmetry [Y-Axis] 100% Biaxial	5	0.35	0.31
Asymmetric Mass [Y-Axis] Strength Asymmetry [Y-Axis] 150% Biaxial	5	0.44	0.24

The U_Y / U_X *ratio* remains constant, or somewhat close to constant, during the change from elastic to inelastic response. However, this result does depend significantly on the system achieving a significant degree of yielding in both directions. This appears to hold true for systems with no mass eccentricity, one-way mass eccentricity, two-way mass eccentricity, and lateral bracing. The small, but non-trivial, differences in the ground motions used for the various configurations also lend additional validation to the results. However, the configuration with a strength eccentricity and no bracing clearly did not exhibit similar response results. This type of eccentricity tends to produce non-uniform yielding, which is critical for the U_Y / U_X *ratio* to remain constant.

The U_θ / U_Y *ratio* increases roughly 20% in moving from elastic response to inelastic response. Results indicate that the increase is applicable for both one-way and two-way mass eccentric structures. The increase also appears to be independent of the magnitude of the inelastic ground motions, as long as the ground motions are sufficient to produce relatively uniform yielding in both translational directions. The results for the systems featuring lateral bracing do not directly support these conclusions, but it is not felt that enough data exists for braced systems to contradict the conclusions either. For the system with a strength eccentricity, however, the U_θ / U_Y *ratio* roughly doubles in moving from elastic response to inelastic response. The non-uniform column yielding due to the strength eccentricity likely produced significant instantaneous stiffness eccentricities, resulting in noticeable increases in torsional response. This type of eccentricity presents particular difficulties for design because the resulting instantaneous stiffness eccentricities are not present during elastic response.

ANALYTICAL WORK

Finite Element Modeling

The primary focus of seismic design is to resist large-scale earthquakes, in which the structure will respond well into the inelastic region, and thus it is important to be able to accurately predict inelastic structural response. However, it is also important in designing or assessing a building to consider the response to lower level earthquakes, which would be more likely to occur during the lifetime of the building but might not produce significant yielding, if any at all.

Finite element analyses were performed for a selected group of earthquake simulations in an effort to accurately predict the structural response of the system for each test configuration. Initially, the nonlinear analysis program Abaqus was chosen to carry out the analyses. However, due to the considerable amount of computing time required in some cases, a second program was chosen, Drain-3DX, to provide, in addition to response predictions, a comparison with Abaqus regarding prediction accuracy and the overall features and limitations of each program and their potential impact on the analysis solutions. Abaqus is a very flexible and powerful commercial program, which can model geometric and material nonlinearities. It includes a built-in pipe section for beam elements, which was very useful in modeling the columns, and rigid elements are also available. Drain-3DX is primarily a research program, and is simpler and faster. It also can model geometric and material nonlinearities, but has a limited number of nonlinear elements and does not feature rigid elements. Both programs include distributed plasticity type elements, which allows the designation of the elastic-plastic element behavior through stress-strain data; thus, it is not necessary to determine the locations of plastic hinges in advance.

The basic Finite Element Model used in this study consisted of approximately 100 nodes and 100 elements of various types. Each column was modeled with 20 elements, which were able to deform inelastically. The cyclic inelastic behavior of the column elements was defined based on uniaxial tension test data performed on column coupons, and the theoretical cyclic model was of a combined isotropic-kinematic hardening type in Abaqus and isotropic hardening in Drain-3DX. The diaphragm was modeled in Abaqus using 16 rigid quadrilateral elements, while in Drain-3DX the same effect was achieved by slaving the rotations of all diaphragm nodes to the center node. The diaphragm height was positioned at the same height as the test structure diaphragm center-of-mass. Non-rigid links were used in the model to connect the top of the 60 inch columns to the rigid diaphragm. These links responded elastically, and were useful in that their cross-sectional properties could be adjusted to "tune" the modal frequencies of the Finite Element Model to the actual measured modal frequencies of the test structure.

Finite Element Analysis Results

Typical elastic and inelastic analytical vs. experimental response results are shown in Figures 7 and 8 for the configuration with one-way mass asymmetry about the X-Axis. Figure 7 shows the elastic force vs. displacement and moment vs. rotation response. For both displacements and the rotation, the frequency content of the analytical responses matches the test data very well. The peak displacements of the

Figure 7. Analytical and Experimental Displacement and Rotation vs. Time
Elastic Response – One-Way Asymmetric Mass (X-Axis)

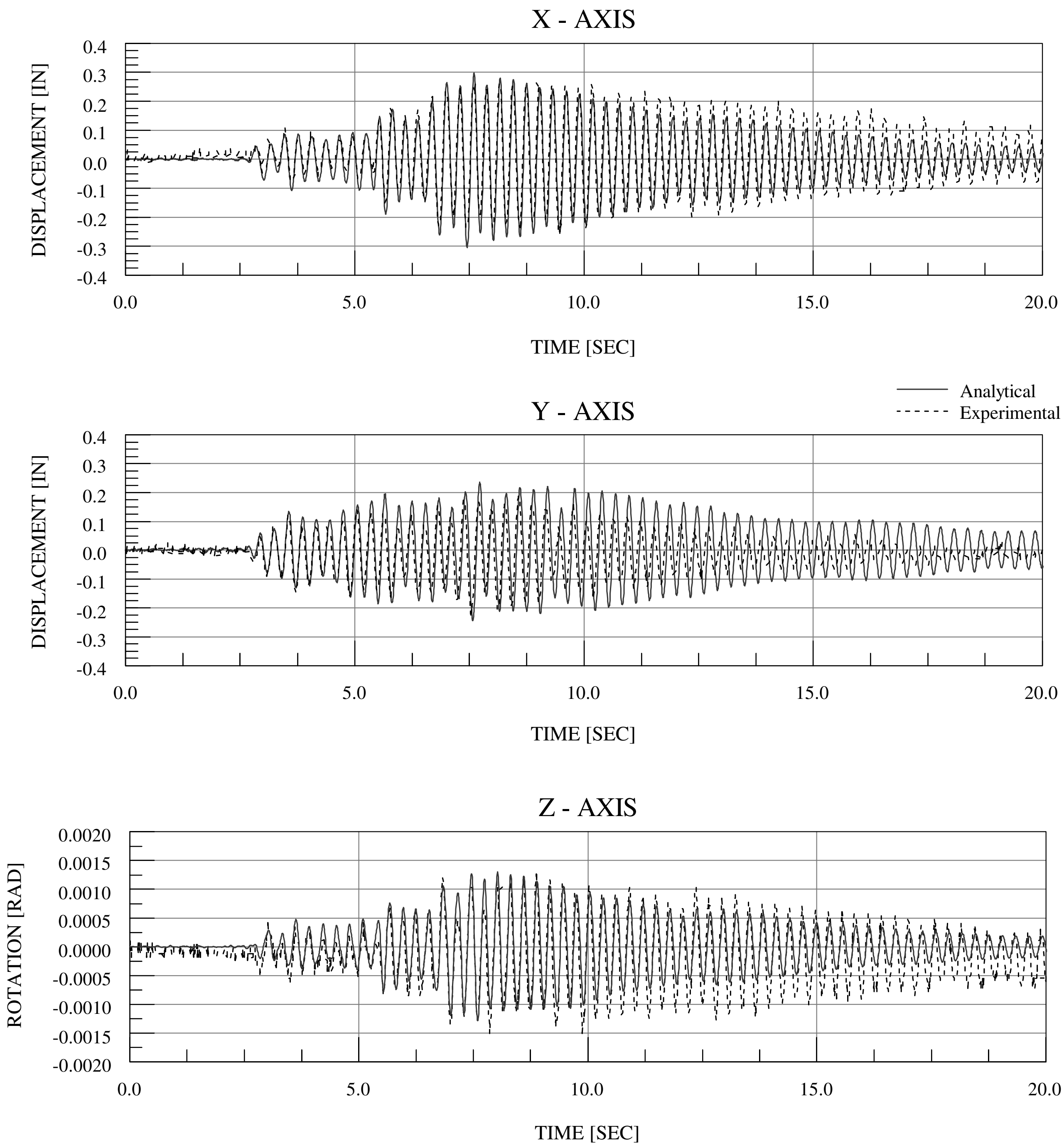
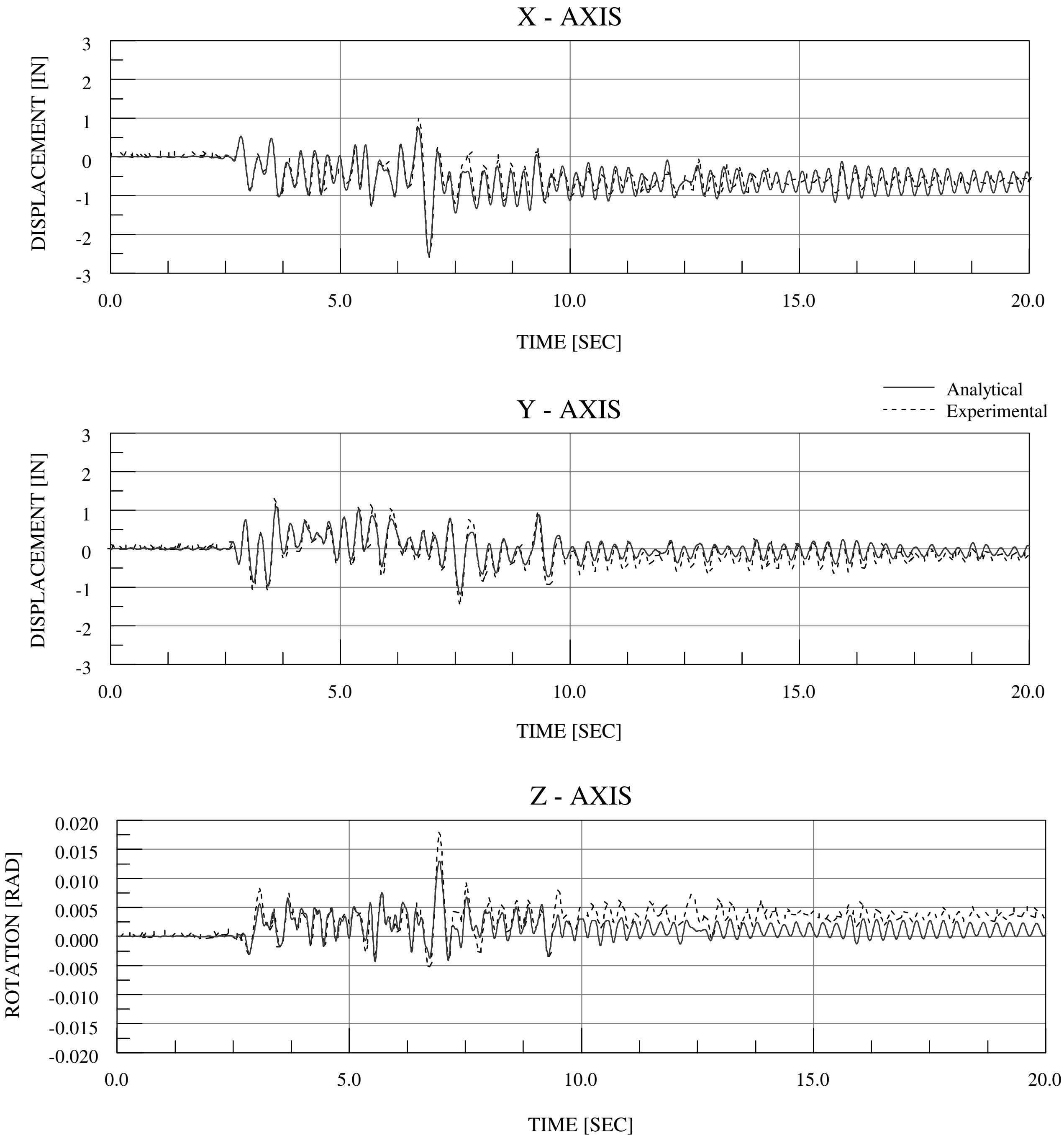


Figure 8. Analytical and Experimental Displacement and Rotation vs. Time
Inelastic Response – 100% Biaxial Ground Motions – One-Way Asymmetric Mass (X-Axis)



analytical model match the test data fairly well, agreeing to within 10% in the X direction, within 5% in the Y direction, and within 15% about the Z-Axis. In Figure 8, the response histories can be seen for the first inelastic test in this configuration, with 100% biaxial ground motions. In the X direction, the peak displacements match very well, agreeing to within 3%. For both the Y direction motion and the rotation, the analytical model predicts smaller peak displacements at a number of points in the time history and a lesser degree of permanent deformation. The peak displacements in the Y direction agree to within only 20%, and the rotation differs from the test data by nearly 30%.

Finite Element Analysis Conclusions
In the finite element analysis of elastic displacement response, no one configuration proved more difficult to predict than the others. With one exception, the predictions for the inelastic responses of each of the configurations using were equally accurate. Overall, the analytical predictions of the elastic displacement response were accurate to within about 7.5%, while the predictions of the inelastic response were accurate to within about 14.5%. A number of the configurations featured two inelastic simulations, with the structure possessing pre-existing inelastic damage and typically amplified ground motions during the second simulation, and the peak displacement predictions proved to be equally accurate as those predicted for the initial inelastic simulation.

The elastic torsional response for the more complicated force-resisting systems, including those systems with lateral bracing and with two different types of columns, is more difficult to predict. This trend appears to continue for the inelastic response, as Configurations 5 – 7, which each feature either concentric lateral bracing or two different types of columns, or both, appear on average to be more difficult to predict rotationally than Configurations 3, 4, and 8, having only mass eccentricity. Overall, the analytical predictions of the elastic rotational response were accurate to within about 42%, while the predictions of the inelastic response were accurate to within about 31%. The peak rotations, for both elastic and inelastic simulations, were consistently underestimated by the analytical analyses.
Overall, the Abaqus and Drain-3DX analyses predicted equally well the measured peak displacements for both elastic and inelastic response and for a variety of structural configurations. Drain-3DX is slightly more accurate in predicting the peak inelastic rotations than Abaqus, although both were reasonably accurate. Despite the increased modeling capabilities of Abaqus, including a much more general and detailed post-yield material hardening model, Drain-3DX proved to be equally capable, and in some instances more capable, in its analytical performance. In addition, the Abaqus analyses required significantly more computing time and power to complete.

Design Assumptions in Finite Element Analysis
Despite the good agreement, in most cases, between the analytical response and the experimental response, the finite element models used in these analyses must be examined for their degree of applicability to models that would typically be used during design. Four specific elements of the finite element models used are examined: the Non-Rigid Links, the column and concentric bracing material behavior, the material damping, the exclusion of accidental eccentricity, and the length of the concentric bracing.

Modal Frequencies
In the previous comparisons between analytical response predictions and the experimental data, a Finite Element Model was used that was "tuned" to the measured modal frequencies of the test structure. This was achieved by varying the cross-sectional dimensions of the Non-Rigid Links to adjust the translational modal frequencies, and by altering the rotational mass moment of inertia to adjust the rotational modal frequency. One other factor impacting the modal frequencies in two configurations was the length of the diagonal bracing in the analytical model, which was also adjusted to "tune" the analytical model. In a design situation, the actual modal frequencies of the structure would not be known in advance when the

response predictions were being made. The rotational moment of inertia would be based on the known geometry of the structure, and two likely design alternatives to the "tuned" Non-Rigid Links in the finite element model would be: first, extending the columns to the height of the diaphragm; and second, incorporating elastic Non-Rigid Links with some pre-determined cross section, in this case having twice the stiffness of the columns. These two cases resulted in the design modal frequencies, as averaged over all of the test configurations, differing from the measured frequencies by 7.4% and 5.0%, respectively.

As shown in Table 3, the "design models" were about half as accurate as the "tuned" model for elastic response. Small differences between the design and actual modal frequencies lead to large differences between predicted and actual elastic response.

Table 3. Elastic Response with Design Assumptions – Non-Rigid Links

Model	% Difference from Experimental		
	Displacements	Rotations	Overall
Tuned	7.5	42	19
Case 1 [Links with Same Stiffness as Columns]	33	43	36
Case 2 [Links with Twice Column Stiffness]	28	59	37

As shown in Table 4, the Case 2 Model was more accurate than the Case 1 Model in predicting displacements, while the two Cases were equally accurate in predicting rotations. The Case 2 Model correctly models the amount of material that can deform inelastically, while the Case 1 Model includes 10-15% more material.

Table 4. Inelastic Response with Design Assumptions – Non-Rigid Links

Model	% Difference from Experimental		
	Displacements	Rotations	Overall
Tuned	15	28	19
Case 1 [Links with Same Stiffness as Columns]	43	32	39
Case 2 [Links with Twice Column Stiffness]	20	33	24

Material Models

In defining the material properties of the columns and diagonal bracing, tension tests were performed with steel coupons to determine the actual stress-strain behavior of each of the column types. Despite the fact that steel in each of these column groups was specified to the manufacturer as A36 or Grade 50 steel, each batch had noticeably different stress-strain properties, as shown in Table 5.

In a design situation, the actual material properties would not be known in advance to the designer, and most likely idealized properties would be assumed. Two likely design assumptions for the stress-strain behavior would be an elastic-perfectly plastic material model, with no material hardening, or an elastic-plastic model in which material hardening was designated. As shown in Table 6, results using a linear hardening material model were more accurate than using an elastic-perfectly plastic model in predicting displacements. The addition of a simple strain-hardening model to an analytical model yields significant improvement in prediction accuracy.

Table 5. Comparison Between Finite Element Design and Actual Material Models

Test	Column Type	Actual		Nominal	
		F_Y [ksi]	F_U [ksi]	F_Y [ksi]	F_U [ksi]
1 – 4	4" Extra-Strong	37.5	58.5	36.0	58.0
8	4" Extra-Strong	29.9	55.2	36.0	58.0
5 & 7	5" Standard	48.0	62.1	36.0	58.0
5 & 7	4" Double Extra-Strong	42.0	73.4	36.0	58.0
6	4" Extra-Strong	48.1	56.6	36.0	58.0
6 & 7	Lateral Bracing	63.5	79.9	50.0	65.0

Table 6. Inelastic Response with Design Assumptions – Material Model

Model	% Difference from Experimental		
	Displacements	Rotations	Overall
Tuned	15	28	19
Elastic – Plastic with Linear Hardening	29	23	27
Elastic – Perfectly Plastic	43	24	37

Conclusions

The results found by using a "tuned" analytical model proved to be consistent with results using a more simplified analytical model with "design" assumptions. When analyzing elastic response, any differences in the modal frequencies of the analytical model and actual structure can produce larger errors in the predicted response. When analyzing inelastic response, differences in the modal frequencies become less important, while the amount of the structure able to deform plastically becomes more important to model accurately. Changing the "effective length" of the diagonal bracing altered the modal frequency in the direction of the bracing, and as a result had a noticeable impact on the elastic peak response. However, altering this parameter had no significant impact on the inelastic response. Using an analytical model with some degree of strain hardening results in response predictions that are significantly more accurate than an analytical model with an elastic – perfectly plastic material model.

SUMMARY

The purpose of this study was to investigate the inelastic response of one-story, symmetric- and asymmetric-plan steel moment-frame to biaxial lateral earthquake ground motions. Through the combined experimental and analytical study, the lateral-torsional response of the system was studied for eight different configurations of mass, strength, and stiffness eccentricity. The primary goals of this study were to examine the adequacy of current building code torsional design assumptions and the ability of analytical software to predict inelastic response, for both a model tuned to the measured dynamic properties of the actual structure and a model based on common modeling assumptions.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support of the Construction Engineering Research Laboratory located in Champaign, Illlinois. Any findings or conclusions expressed in this paper are those of the authors and not necessarily those of the sponsor.

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