TORSIONAL EFFECTS ON THE INELASTIC SEISMIC RESPONSE OF STRUCTURES

by

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Abstract

To evaluate inelastic torsional response of buildings due to different parameters such as unsymmetrical distribution of mass or lateral load resisting elements in the plan of the structure or inelastic behavior of resisting elements and loss of the resistance of such an element during an earthquake, a full three-dimensional nonlinear dynamic analysis is a powerful tool to evaluate such a nonlinear response.

The results of nonlinear dynamic analyses of two actual steel moment frame buildings that were damaged during the 1994 Northridge earthquake are subjected to recorded ground motions from the Northridge and Loma Prieta earthquakes. The importance of different parameters such as discontinuity of lateral resisting elements (setbacks), unsymmetrical distribution of mass or resistance in the plan of structure, intensity and frequency content of the earthquake ground motions, accidental eccentricity as prescribed by code and the effect of geometric nonlinearity (P-Delta) on the inelastic lateral-torsional response of the structures is discussed. Response parameters considered include lateral story displacement, Interstory drift index, plastic hinge rotation demand and torsional rotation of each floor.

The analysis procedures use three-dimensional nonlinear dynamic analytical models developed for the *PERFORM 3-D* computer program.

Study of the results for different models with different eccentricities clearly shows the effect of inelastic torsion in comparison with elastic torsion on the response of structures. The torsional rotation of floors considered as a main parameter of the torsional response of the building has an average increase of 30 to 60 percent for

material nonlinearity. By adding geometric nonlinearity (P-Delta), this increases to as much as 70 to 100 percent of elastic torsional rotation. This clearly shows the inelastic torsional response of structures may be significantly underestimated by a linear dynamic analysis, especially for large value of mass or stiffness eccentricity and intensity of the ground motion.

Chapter 1: Introduction

During earthquake ground motions, structures usually will experience torsional vibration in addition to lateral oscillations. One of the main sources for the torsional response of structures is unsymmetrical distribution of mass or lateral load resisting elements in the plan of the structure which is usually referred to mass or stiffness eccentricity. Different types of torsional response can happen in symmetrical structures in case of non-uniform ground motion along the foundation of the structure or inelastic behavior of resisting elements and loss of the resistance of such an element. This research is mainly focused on the last case which can happen even during moderate earthquake ground motion. In case of extreme or even moderate earthquakes, most structures behave inelastically. Because of this inelastic behavior, the coupled lateral-torsional vibrations of the structure can be significantly higher than those predicted by the linear-elastic analysis. As soon as one of the lateral resisting elements yields, the position of the center of stiffness will change and this can induce a significant change in the eccentricity of the whole structure.

The goal of this research is to study the effects of different types of torsion and especially inelastic torsion due to material or geometric (P-Delta effect) nonlinearities on the inelastic dynamic response of structures. A three-dimensional nonlinear dynamic analysis is a powerful tool to evaluate such a nonlinear response. The effect of different parameters on the nonlinear torsional response of buildings such as intensity and frequency content of the ground motion, stiffness and strength asymmetry, planwise distribution of mass and different amount of viscous damping will be also studied.

Chapter 2: Previous Works

Investigations of the torsional response and lateral-torsional coupling of symmetrical or asymmetrical buildings has been conducted during the last four decades and researchers have developed different rational methods for determining the torsional effects of the earthquake including accidental torsion.

In 1969, Newmark [16] developed a rational basis for determining the torsional earthquake effects in symmetrical single-story buildings and suggested design recommendations considering the effects of building size, period of vibration and type of framing on the necessary design eccentricities for seismic forces to compare with the design eccentricity of 5 percent of the maximum building dimension used in uniform building code.

The approach was to develop an estimate of torsional ground motions from a consideration of measured strong ground motions assumed to propagate as a wave. From these motions an estimate was made of a torsional response spectrum. By determining the combination of torsional and flexural responses, the relative responses of several typical building configurations with differences in frequencies were computed and values of eccentricity in terms of building width and the wave propagation velocity were determined. The results of the study indicated that the design eccentricity should vary with the natural frequency of the building and the transit time of the wave motion across the base of the building. Also in general, an accidental eccentricity of 5 percent of the longer plan dimension required by code seemed reasonable only for framed buildings having fundamental periods exceeding about 0.6 seconds or shear wall building with periods greater than about 1.0 second. Accidental eccentricity of about 10 or 15 percent would be reasonable for shorter fundamental periods.

Later in 1971, Newmark [17] categorized the main cause of the accidental eccentricity into the three groups, rotational component of ground motion about a vertical axis [16], the differences between assumed and actual stiffness and masses and asymmetrical patterns of nonlinear force-deformation relation. Results of this study showed that the accidental eccentricity based on his previous study [16] would probably be too conservative if the rotational component of the ground motion about a vertical axis were the only cause of accidental torsion.

A number of researches conducted in the 70's [19, 20, 21, 22] all dealt with coupled lateral torsional linear response of the single-story buildings or tall buildings modeled as a cantilever shear beam. These studies showed that a strong coupling effect occurred when corresponding natural frequencies are close together, even though eccentricities were small.

In 1982, Kung and Pecknold [18] presented a detail study of the effect of the ground motion characteristics, especially its multi-directional character on the seismic response of one-way and two-way torsionally coupled elastic structural systems including single-story and multi-story models with stiffness eccentricity. Other important characteristics of earthquake ground motions like frequency content, time-varying intensity, duration and lack of spatial correlation of ground motion components causing torsional excitation were addressed in this study. Results of this investigation showed for one-way torsionally coupled systems, an increase in torsional response and a reduction in translational response when the uncoupled torsional and translational frequencies are nearly equal. Compared to uncoupled translational response, the maximum root mean square displacement responses of the periphery of the single-story model were increased by about 40 percent for an eccentricity of 6 percent of the floor dimension and this displacement response for a two-way coupled system with equal eccentricities of 8 percent each at the corner of floor diaphragm increased by about 75 percent.

In the last twenty years, the inelastic torsional response and lateral-torsional coupling of symmetrical or asymmetrical structures for both static and dynamic loads received more attention and many studies were conducted in this field.

These studies can be categorized into the following three groups:

First, those studies which dealt with the inelastic torsional response of idealized one-story structures for different system parameters and their extension to the practical case of multistory buildings. [3], [6], [11], [13], [14], [23], [24], [25], [26]

The second group of researches was to study the effects of plan asymmetry in light of the story shear and torque response histories of different structural configurations. Based on this method, these shear and torque combinations were bounded in this space by a surface corresponding to the different collapse or plastic mechanisms of each story. [4], [7], [12], [27], [28] In the last group, different types of approximate three-dimensional analyses were conducted to incorporate inelastic and nonlinear effects due to lateral-torsional coupling in the models. Although a full three-dimensional nonlinear dynamic analysis could be a powerful tool to evaluate such an inelastic response, the large cost of such nonlinear analyses was a reason to use approximate methods. [5], [7], [15]

In 1990 and 1991, Goel and Chopra [3], [25] presented the earthquake response of idealized single-story asymmetric buildings for a wide range of system parameters. Those buildings were symmetric about X-axis. The ground motion acting in Y-direction was selected to be a half-cycle displacement pulse, because there is a close relationship between the response of a system to such a simple ground motion and actual earthquake ground motion. By comparing these responses with those of corresponding symmetric system, the effects of lateral-torsional coupling on building deformations and ductility demand were identified. Detailed parametric study of the inelastic earthquake response of single-story asymmetric building models was investigated by other researchers as well [24], [26].

The effect of torsion on the three-dimensional linear-elastic and nonlinear-inelastic seismic response of multi-story buildings was another research study conducted in 1990 by Sedarat and Bertero [15]. To investigate the effective parameters on lateral-torsional response of structures, including accidental eccentricity, the following two different analyses of a seven-story reinforced concrete frame-wall structure were conducted:

- 3-D linear-elastic static and dynamic analyses utilizing the computer program *ETABS*, and
- 3-D nonlinear static analysis by the computer program *FACTS* developed at the University of California, Berkeley utilizing its nonlinear three-dimensional reinforced concrete beam column element

Based on the results of these analyses, a realistic model of a seven-story structure was developed and it is shown that real nonlinear torsional response of the structure may be significantly underestimated by just a linear-elastic dynamic analysis.

Wong and Tso [23] in 1994 evaluated the inelastic seismic response of torsionally unbalanced structural systems with strength distributed using elastic response spectrum analysis. Accidental torsion was incorporated to the model by applying static floor torques or shifting the location of the center of mass.

In 1995, R. Bertero [11] showed that for a special class of buildings, the reduction in the building strength resulting from inelastic torsion could be obtained using the classical theorems of plastic analysis. By analyzing an auxiliary structure, a simplified formula for the reduction in strength due to inelastic torsion was obtained. A simplified global model of the three-dimensional (3D) seismic behavior was utilized for this purpose even though several 3D effects such as 3D flexure-torsion interaction and the local concentration of seismic demand due to multistory effects that could not be considered by the simplified approach could increase the local seismic demand.

This simplified three-dimensional model of multi-story buildings had the following characteristics: the center of mass of every floor lay on one vertical line; lateral loads were resisted by planes of strength without interruption along the height of the building and strengths and loads along the height of the building were such that plastic mechanisms were obtained for each plane of strength. With these characteristics, the plastic analysis of this model could be conducted by considering the idealized one-story building.

Later in the same year, De La Llera and Chopra [27] presented a procedure to estimate accidental torsion effects in seismic design of single-story buildings. In this procedure, by calculating the ratio between fundamental frequencies of uncoupled torsional and lateral motions of the building and having plan dimensions, the increase in displacement at the edge of the building resulting from all sources of accidental torsion were estimated and then total displacements and amplified forces on interior resisting planes were calculated. This procedure eliminates three-dimensional static or dynamic analyses to account for accidental torsion effects in building design

The inelastic seismic behavior of asymmetric multistory buildings emphasizing the use of story shear and torque histories was another research of Chopra and De La Llera et al [4] in 1996. The following six different structural characteristics and their effect on the torsional response of the buildings were analyzed: strength of orthogonal resisting planes, stiffness asymmetry, strength asymmetry, planwise distribution of strength, number of resisting planes and intensity of the ground motion component in the two orthogonal directions. As a result of these analyses several techniques and conceptual guidelines were developed to correct the planwise unbalance in deformation demands of asymmetric structures. The two most important were to increase the torsional capacity of the system by introducing resisting planes in the orthogonal direction and to modify the stiffness and strength distribution to localize yielding in selected resisting planes. The fundamental idea of this research was to study the effects of plan asymmetry by the story shear and torque response histories of different structural configurations. These histories were represented in the force space by the story shears Vx and Vy and story torque T at each time instant as one point. These shear and torque combinations were bounded in this space by a surface defined by the set of story shear and torque combinations corresponding to the different collapse mechanisms that could be developed in the story.

About two years later, Carlson and Hall et al [5] used an approximate three dimensional analysis by coupling of planar moment resisting frame to investigate the three dimensional nonlinear response of a tall, flexible, asymmetric steel building subjected to ground motions from the Northridge earthquake. Many inelastic and nonlinear effects were incorporated in the model, including weld fracture. The building model employed planar frames with four degrees of freedom at each frame joint, beam-end and column-end rotations and horizontal and vertical translations. The cross section of each member was divided into fibers with a nonlinear axial stress-strain relation. A special nonlinear interstory damping with a cap was used to ensure that the damping forces did not become unrealistically large. Finally, the planar frames were joined together by constraints that forced the average displacement of in-plane translational dofs at a floor to be compatible with the corresponding rigid body motion determined from the dofs of the master node for that floor.

In 1998, Postelnicu, Gabor and Zamfirescu [12] used simplified methods to evaluate torsional effects. The first type of simplified method was based on the global plastic mechanisms of structures, such as those proposed by R. Bertero [11]. In this method, the base shear-torque surfaces were to be used together with the corresponding displacement surfaces in order to obtain reliable control of the structure from the point of view of the sensitivity to the torsional effects.

In the same year, Humar and Kumar [13] and [14] continued the study of the inelastic torsional response of single- and multistory buildings by consideration of effects of both the natural and accidental torsion. It was shown that, given the complexity of inelastic response, particularly that of multistory buildings, the results of single-story buildings could be used for multistory buildings that are asymmetric in plan, but otherwise fairly regular.

The issue of how the relative contribution of structural elements in the planes orthogonal to ground motion, affects the torsional response of inelastic structures has been the subject of continuing study since 1999.

In 2000, Stathopoulos and Angnostopoulos [6] examined the inelastic seismic torsional response of simple structures by means of shear-beam type models as well as with plastic hinge idealization of a one-story building. Mass eccentric versus stiffness eccentric structures, effects of different types of motions and double eccentricities were examined with the shear-beam type model.

One year later, De La Llera and Chopra [7] studied the three-dimensional inelastic earthquake response of a seven-story reinforced concrete building during the 1994 Northridge earthquake. The objectives of this investigation were as follows: to understand the inelastic behavior of the building using recorded motions and to propose a simplified model that could explain the lateral-torsional coupling observed in this nominally symmetric building. Later, response results of a simplified inelastic stick model that used the story-shear and torque surfaces were compared with the results obtained from a conventional elastic three-dimensional building model. These results suggested that damage in the building occurred in the first few cycles of the response, and building showed markedly inelastic torsional behavior in spite of its nominal symmetry in plan. Such torsional behavior could also occur in other symmetric-plan buildings with strong perimeter frames. Since this building underwent coupled lateral-torsional motions and significant damage during the earthquake, an inelastic simplified idealization three-dimensional model was developed.

In this model, a single column like element connecting two consecutive floors represented the stiffness and strength properties of a building story. This single element model (SEM) allowed three degrees of freedom at each node, two horizontal displacements and one rotation, corresponding to the degrees of freedom of the rigid floors connected by the element. However, the structural response was assumed to be symmetric about the Y-axis. The inelastic properties of the SEM would be defined by the corresponding story-shear and torque surface, defined as the locus of pairs of shear force and torque values that, when applied statically to the story, produce its collapse [4].

In the same year, Lin, Chopra and Del La Llera [28] developed and extended the simplified analysis procedure [27] to consider accidental torsion in building design to compare with measured accidental torsion determined from motions of 12 multi-story nominally-symmetric plan buildings with different structural system recorded during different earthquakes. Results showed that the simplified procedure provided a good estimate of the measured value for buildings with the ratio between the fundamental frequencies of uncoupled torsional and lateral motions not close to 1.0. For buildings with this ratio very close to 1.0, the simplified procedure could be conservative. Based on analytical and probabilistic investigation of the increase in response of elastic structures due to accidental torsion, this design procedure may also be applied to inelastic design of buildings.

Chapter 3: Scope of the Research

The purpose of this study is to evaluate the inelastic torsional response of steel moment frame buildings due to different parameters such as unsymmetrical distribution of mass and/or lateral load resisting elements in the plan of the structure or inelastic behavior and geometric nonlinearity (P-Delta) of resisting elements and loss of the resistance of such an element during an earthquake. In order to have meaningful results and conclusions, a thorough and detailed investigation must be carried out.

The following is an outline of the tasks that were conducted in order to achieve this goal.

1. Modeling and analysis of two buildings with different conditions and different type of irregularities including mass and stiffness eccentricities in two perpendicular directions

2. Evaluation of their inelastic seismic response to the following:

- Different actual recorded ground motions with different characteristic such as intensity and frequency content
- Different amount of viscous damping
- Different amount of torsion by moving the center of mass in regard to the center of stiffness for each floor
- Different amount of material strength for low-rise building
- Geometric nonlinearity or P-Delta effect

3. Compare the results of the study to the recorded response of the actual buildings and to free field ground motions recorded in the San Fernando Valley.

4. Investigate the adequacy of code provision design for accidental torsion by comparison to the results of the 3D dynamic analyses both linear and nonlinear.

Chapter 4: Description of the Nonlinear Features of the Model

For modeling of inelastic beams and columns, the main concern is:

- Force-Deformation Relationship. A beam or column member exerts forces on the adjacent members and connections and also has deformations that contribute to the displacements of the complete structure, so it is important to have a reasonably accurate force-deformation relationship, so that the forces and deformations are both calculated with reasonable accuracy.

Based on the above condition, there are basic components with different degree of accuracy in the *PERFORM 3-D* [10] and *SAP2000* [2] computer programs to model beam and column. The description of the nonlinear features of each program is as follows:

PERFORM 3-D

Beam-type Element: For modeling of the beam element, the beam-type basic components include Moment Hinge rotational Model, Moment Hinge Curvature Model, Inelastic fiber segment and FEMA-273/356 Model using curvature hinge which is the one utilized in this study. A major advantage of this model is that FEMA-273 gives specific properties, including end rotation capacities. A curvature hinge is essentially the same as rotation hinge, but in the action-deformation relationship the deformation is curvature rather than hinge rotation. Figure 4.1 shows a length of an inelastic beam and its moment-curvature relationship. Program models that as a rigid-plastic hinge component and an elastic beam component.

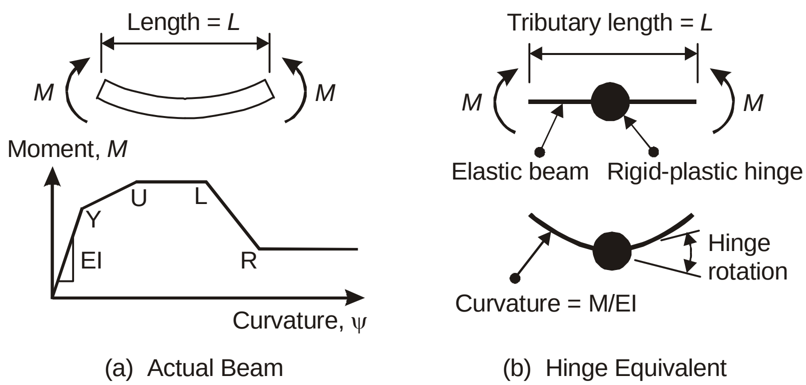


Figure 4.1: Moment-Curvature Relationship of Beams

The stiffness of the elastic beam component is the initial stiffness of the actual beam and the deformation of this beam accounts for the elastic part of the deformation. The rigid-plastic hinge then accounts for the plastic part of the total deformation.

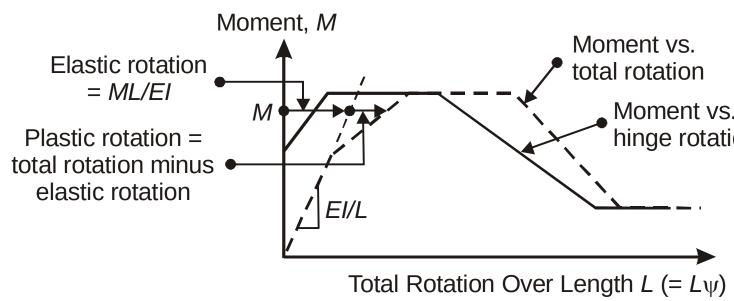


Figure 4.2: Moment-Total Rotation Relationship of Beams

The dashed line in figure 4.2 shows the relationship between the bending moment and the total rotation over the tributary length L. There are a couple of types to model inelastic beams as follows: Chord Rotation Model, Plastic Hinge Model and Plastic Zone Model. Chord rotation model is the one utilized in this study. In this model, each beam has two of these FEMA-273 components, to allow for the case where the strengths are different at the two ends of the elements. *PERFORM 3-D* converts each component to a plastic curvature hinge and an elastic beam segment and calculates the properties of the plastic hinge component to give the required relationship between end moment and end rotation based on the hysteretic behavior of steel. A curvature hinge requires a moment-curvature relationship and a tributary length. *PERFORM 3-D* uses the tributary length to convert the moment-curvature relationship to an equivalent moment-rotation relationship for the hinge. This tributary length of each hinge is 1/3 of the FEMA component length or 1/6 of the clear length between end zones for a symmetrical element.

The cord rotation model applies to a symmetrical beam member with equal strengths at both ends and an inflection point approximately at mid span. A W-shape steel beam satisfies these conditions (Fig.4.3).

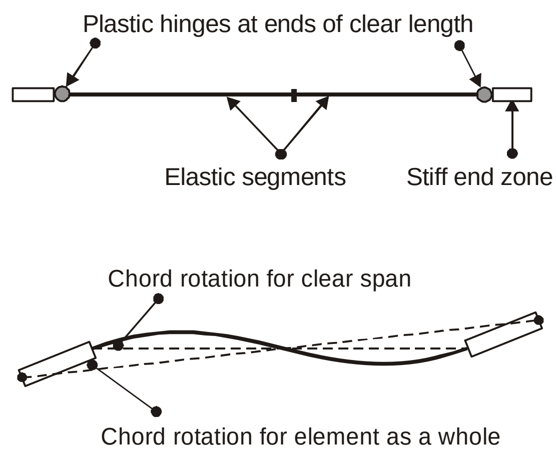


Figure 4.3: Cord Rotation of Beams

Hysteresis loops: The type of action-deformation relationship and Hysteresis loop that might be expected for a real structure member is shown in Figure 4.4:

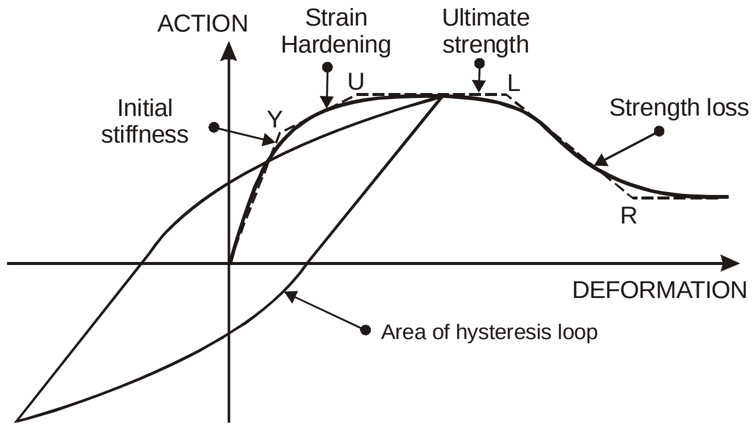


Figure 4.4: Hysteresis loops of Beams

The intent of the program action-deformation model is to capture the essential aspects of this behavior, namely the initial stiffness, strain hardening, ultimate strength and strength loss, as shown. The main intent of the hysteresis loop model is to capture the dissipation energy (the area of the loop). This is affected by stiffness degradation under cyclic load. For the case with no stiffness degradation, program assumes hysteresis loop as shown below (Fig.4.5 and Fig.4.6). The loops in these figures can be modified to account for unsymmetrical properties as well [10].

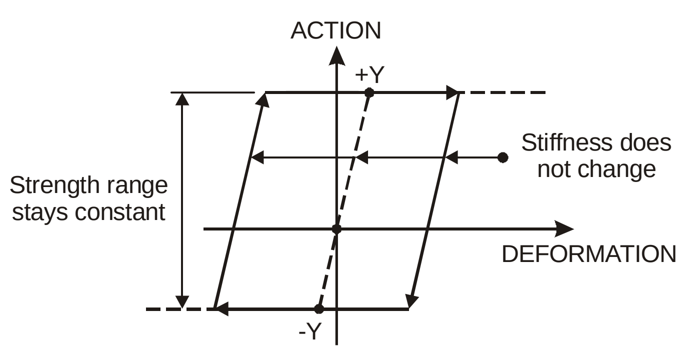


Figure 4.5: Non-Degraded Loop for E-P-P Behavior

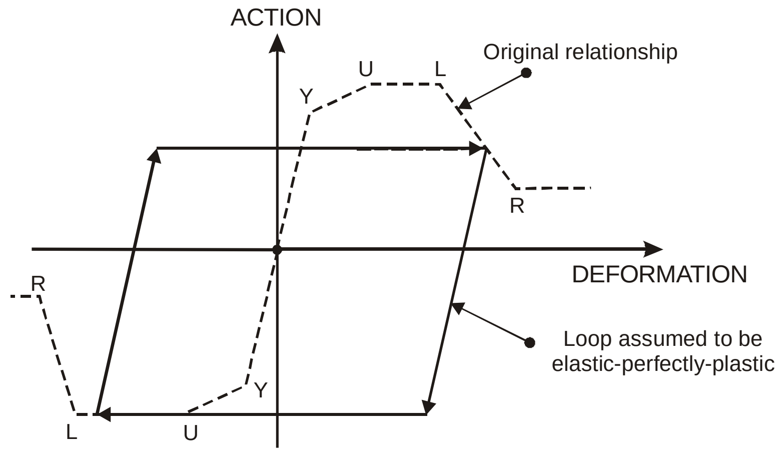


Figure 4.6: Non-Degraded Loop after Strength Loss

Column-type Element: The column-type basic components include the same model types of beam but instead of regular rotation or curvature hinge, a hinge with P-M-M interaction was used to include the effect of axial force and weak-axis moment. A rigid-plastic hinge with P-M-M interaction is conceptually similar to a moment plastic hinge. The major differences are as follows:

- A P-M-M hinge needs a yield (interaction) surface to define when yield occurs and what happens after yield.
- When a hinge yields it deforms axially as well as rotationally. It is easy to visualize rotation at zero-length moment hinge, but it is not so easy to visualize axial deformation along a zero length axial hinge. Mathematically, however, the two are the same.
- For a rotation hinge, bending deformation is rotation, and axial deformation is axial displacement across the hinge. For a curvature hinge, bending deformation is curvature and axial deformation is axial strain.

This yield (interaction) surface assumes a symmetrical cross section for bending. In principle it is possible to use yield surfaces for unsymmetrical sections, but the yield surface can become complex. The yield surface defines the strength of the material under biaxial stress. Plasticity theory uses the same surface to define the behavior of the material after it yields. The details of the theory are as follows:

- If the stress point moves around the yield surface, the material stays in a yielded state. This means that the stresses can change after yield even though the material is E-P-P.
- Figure 4.7 shows a yielded state defined by stresses σ_{1A} and σ_{2A}. Suppose that strain increments $\Delta\varepsilon_1$ and $\Delta\varepsilon_2$ are imposed, causing the stresses to change σ_{1B} and σ_{2B}. Based on plasticity theory, some of the strain increment is an elastic increment and the remainder is plastic flow. The elastic part of the strain causes the change in stress.

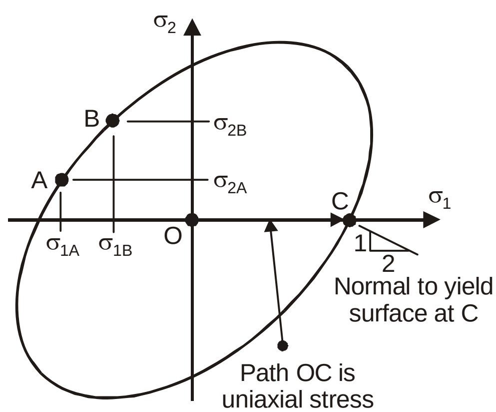


Figure 4.7: The Yield Surface for P-M-M Hinge

- Plasticity theory can be extended to P-M interaction in a column, where the axial force and bending moment interact with each other. For the E-P-P case, the yield surface is now the P-M strength interaction surface for column cross section.

<u>*Modeling of Damping*</u>**:** For dynamic analysis of elastic structures, it is a common practice to specify elastic viscous damping to account for energy dissipation. In dynamic analysis of inelastic structures, that elastic energy dissipation is still present and is added to the energy dissipation due to inelastic behavior. Inelastic energy dissipation is modeled directly in a nonlinear dynamic analysis. For modeling of elastic energy dissipation, elastic viscous damping is still a powerful tool, however, since the analysis does not make use of natural modes of vibration, it is not practical to assume modal damping. *PERFORM 3-D* uses Rayleigh damping in the form of "αM+βK" model, which assumes that the structure has a constant damping matrix, $\underline{C}$, given by:

$$\underline{C}= \alpha\underline{M}+\beta\underline{K} \qquad (4\text{-}1)$$

Where $\underline{M}$ is the structure mass matrix, $\underline{K}$ is the initial elastic stiffness matrix; α and β are multiplying factors.

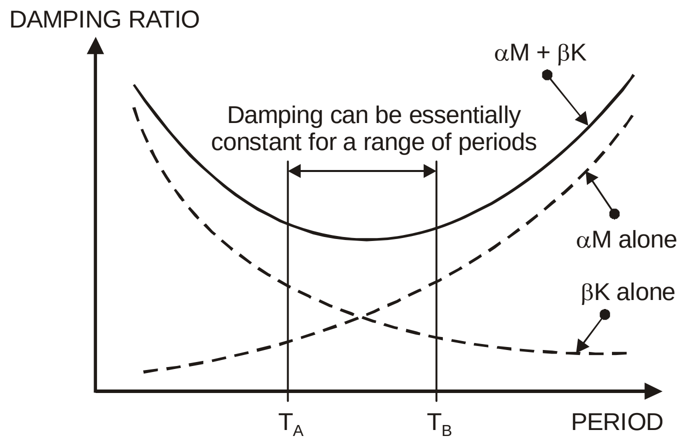
In terms of modal damping, it can be shown that $\alpha\underline{M}$ damping corresponds to more damping in lower (longer period) modes and less damping in higher (shorter period) modes, with the relationship:

$$\xi_i = \alpha \frac{T_i}{\pi} \qquad (4\text{-}2)$$

Where T_i= period of mode "i" and ξ_i = proportion of critical damping in this mode. It can also be shown that $\beta\underline{K}$ damping corresponds to less damping in lower modes and more damping in higher modes, with the relationship:

$$\xi_i = \beta \frac{\pi}{T_i} \tag{4-3}$$

By combining α<u>M</u> and β<u>K</u> damping it is possible to have almost constant damping over a significant range of periods, as indicated in Figure 4.8.



<u>Figure 4.8: The Rayleigh damping</u>

This type of damping is often used for step-by-step dynamic analyses of linear structures using coupled equations (as distinct from uncoupled modes). In this case the longer period, T_B, is usually the first mode period (or close to it) and the shorter period, T_A, corresponds to some higher mode.

In *PERFORM 3-D* procedure, instead of specifying values for α and β, two period ratios of T_A / T_1 and T_B / T_1 and the damping percentages at these ratios should be specified. Then the values of α and β will be calculated. T_1 is the period of first mode.

It is apparent that Rayleigh damping can be specified exactly at only two periods in order to solve for α and β in the above equations. To specify T_A and T_B for linear and nonlinear analysis, T_B / T_1 is typically close to 1.0 and T_A / T_1 is smaller than 1.0.

A typical choice for T_B and T_A is using T_1 and T_2 as a period of 1st and 2nd mode respectively. This choice produces nearly constant damping from T_A/ T_2=1 to T_B/ T_1=1. For this purpose, the elastic damping was kept essentially constant over that range of periods for all analyses by specifying damping of the choice at the ratios of T_A / T_1 and T_B / T_1 about 0.35 and 1.0 respectively. Considering T_2 / T_1 has a range from 0.32 to 0.37 based on Tables 2 and 4 for these two different case studies, models can have exact specified damping at 1st and 2nd mode periods. (See Figure 4.9)

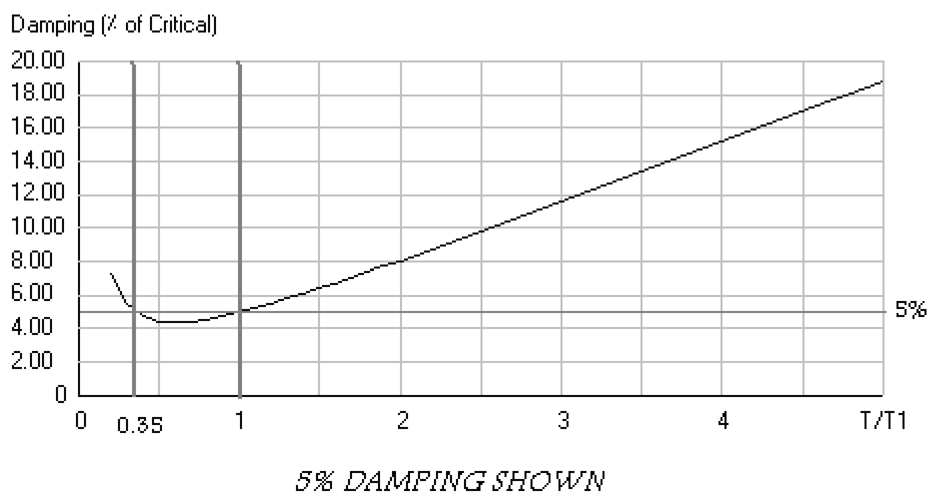


Figure 4.9: Damping versus Period Ratios

SAP2000 Nonlinear Version 8.0

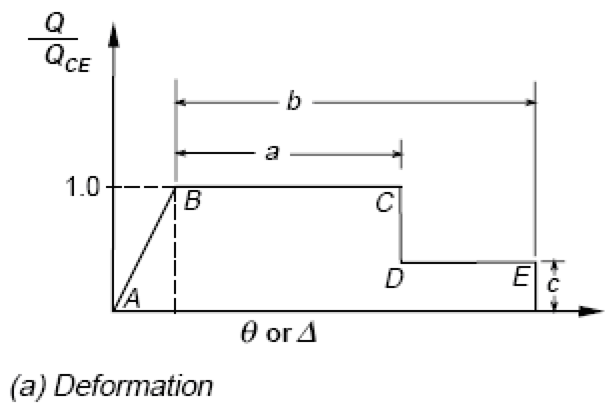
Yielding and post-yielding behavior of beam elements can be modeled using discrete plastic hinges which is the one was utilized in this study. Other nonlinear element is NL-link element, which is usually used to model other types of nonlinear behavior such as nonlinear viscous damper, gap, hook and biaxial shear plasticity or friction-pendulum. Plastic hinges can be assigned to a frame element at any location along the element. In a similar manner as the *PERFORM 3-D***,** FEMA-273 element, default hinge properties are provided based on FEMA-273 criteria with 3 percent strain hardening. Hinges only affect the behavior of the structure in nonlinear analysis. Normally the hinge properties for each of the six degrees of freedom are uncoupled from each other. However, in order to model column-type hinges, there is an option to specify coupled axial-force/ biaxial-moment behavior called P-M2-M3 hinge. For the P-M2-M3 hinge, an interaction (yield) surface in three-dimensional P-M2-M3 space is specified that represents where yielding first occurs for different combinations of axial force P, minor moment M2, and major moment M3.

The surface is specified as a set of P-M curves, where P is the axial force and M is the moment at a particular angle in the M2-M3 plane and it must be convex. The default hinge property of *SAP2000* used the P-M2-M3 curve as being the same as the uniaxial M3 curve.

In direct integration of the full equation of motion without use of modal superposition, the damping in the structure is modeled using a full damping matrix. Unlike modal damping, this allows coupling between the modes to be considered. Direct–integration damping uses the proportional damping of the whole structure as its source. As done for *PERFORM 3-D*, the damping matrix for that is calculated as a linear combination of the stiffness matrix and the mass matrix scaled with α and β coefficients.

FEMA-273 Criteria for Modeling of Nonlinear Steel Elements

As mentioned before, FEMA-273 [31] provides all specific properties including end rotation capacities to define FEMA-273 basic component model. Based on this guideline, the details of all segments of the generalized load-deformation curve for component of steel moment frames, as defined in Figure 4.10 and Table 5-4 of FEMA-273 [31], may be used. This curve may be modified by assuming a strain-hardening slope of 3 percent of the elastic slope.



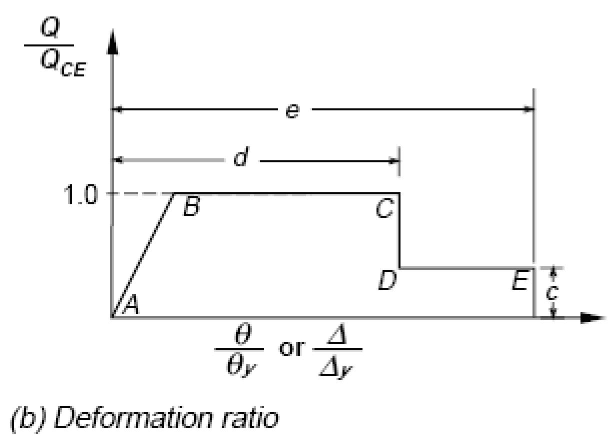


Figure 4.10: Generalized Load – Deformation Curve

The parameters Q and Q_{CE} are generalized component load and expected strength for the component. For beams and columns, θ is the plastic rotation of the beam or column, θ_y is the rotation at yield, Δ is displacement and Δ_y is yield displacement. The parameters "c", "d" and "e" of the second graph for curvature hinge model of FEMA-273 utilized in this study are equal to 0.6, 10 and 12 respectively.

Figure 4.11 defines chord rotation for beams. The cord rotation may be estimated by adding the yield rotation θ_y to the plastic rotation or to be equal to the story drift.

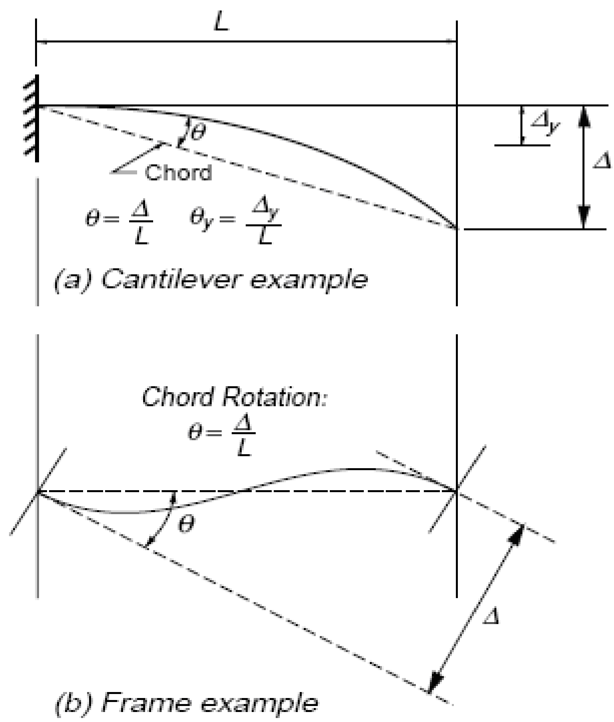


Figure 4.11: Definition of Chord Rotation

Based on assumption of a point of contraflexure at mid-length of the beam or column, the equations for θ_y are:

Beams: $\theta_y = Z\,F_{ye}\,l_b / 6EI_b$ (4-4)

$$Q_{CE} = M_{CE} = ZF_{ye} \qquad (4\text{-}5)$$

Columns: $\theta_y = Z\,F_{ye}\,l_c\,(1 - P/P_{ye}) / 6EI_c$ (4-6)

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}\,(1 - P/P_{ye}) < ZF_{ye} \qquad (4\text{-}7)$$

Where

E = Modulus of Elasticity, ksi

F_{ye} = Expected Yield Strength of the Material, ksi

I = Moment of Inertia, in^4

Z = Plastic Section Modulus, in^3

Chapter 5: Inelastic Torsional Response of Buildings

A. Case 1: Two-story Building (AAA Bldg.)

For the first building, a two-story steel office building which was severely damaged during the *Northridge earthquake (1994)* and investigated in reference [1] and [32], was chosen and analyzed using the programs *PERFORM 3-D* and *SAP2000* nonlinear version 8. A comparison between the nonlinear results of both programs was made.

a. Specifications of the Building

The building was designed and constructed in 1991. It has a rectangular plan that is 96 feet in the east-west direction by 101.5 feet in north-south direction as shown in Fig. 5.1. The lateral resistant system was provided by single bay, ductile steel moment frames located on each side of the building. Elevations of these four rigid frames are shown in Fig. 5.2. The building is almost symmetrical with the exception of second floor offsets on the north and east elevations. At the offset locations, the ductile steel moment frames are set back at the second story and girders support the upper story columns vertically. The lateral loads are transferred from the set back frames to the exterior frames through the second floor diaphragm. Although these setbacks do not look to have been a significant factor affecting the seismic response of the building, changing of the stiffness of lateral resisting frames at the north and east side of the building at second floor, could be a source of torsional irregularities. Also the configuration of a single, one bay frame on each side of the building results in limited redundancy that would introduce a significant torsion into the structure during the seismic response, particularly if inelastic behavior occurs.

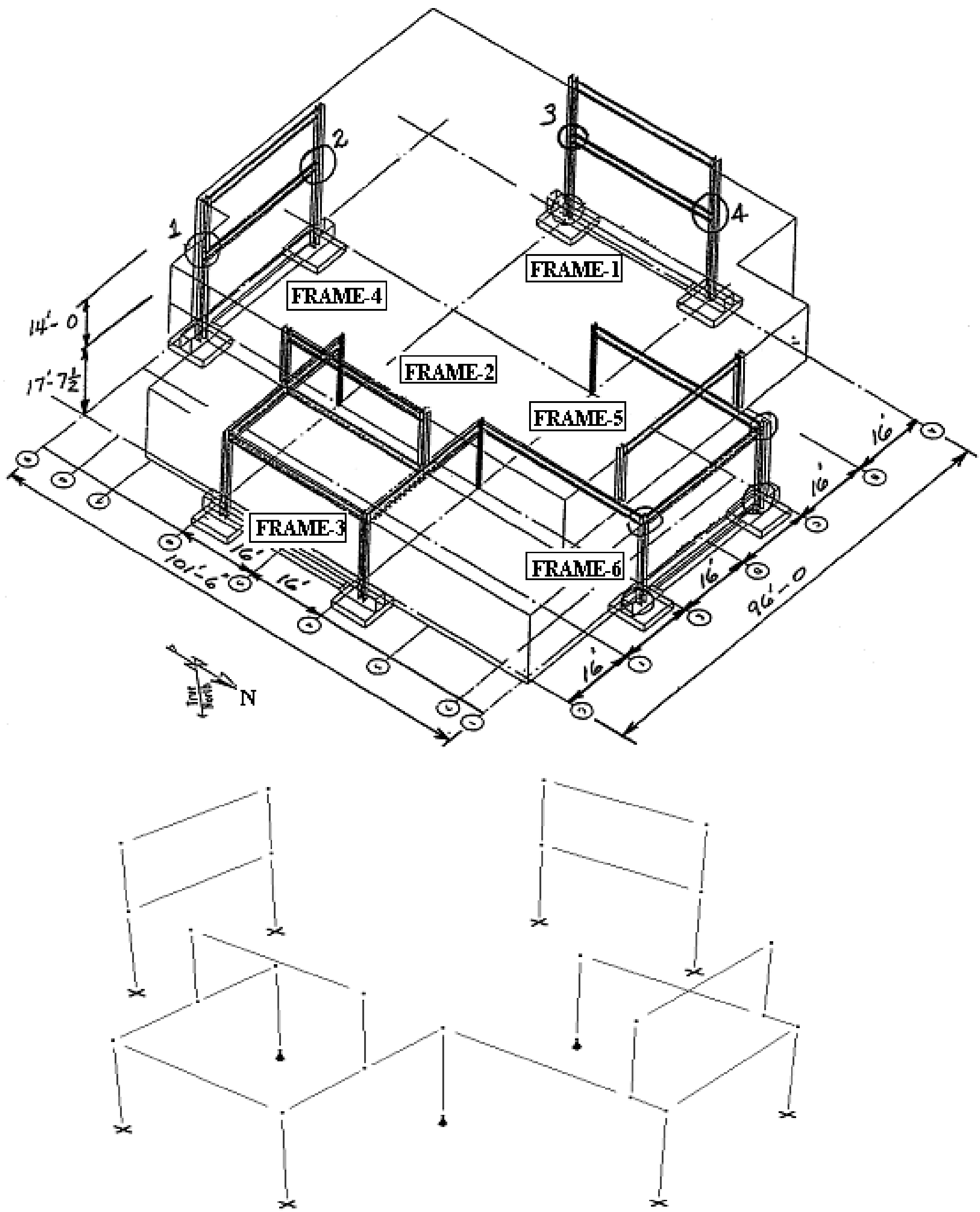


Figure 5.1: AAA Building 3-D Elevation

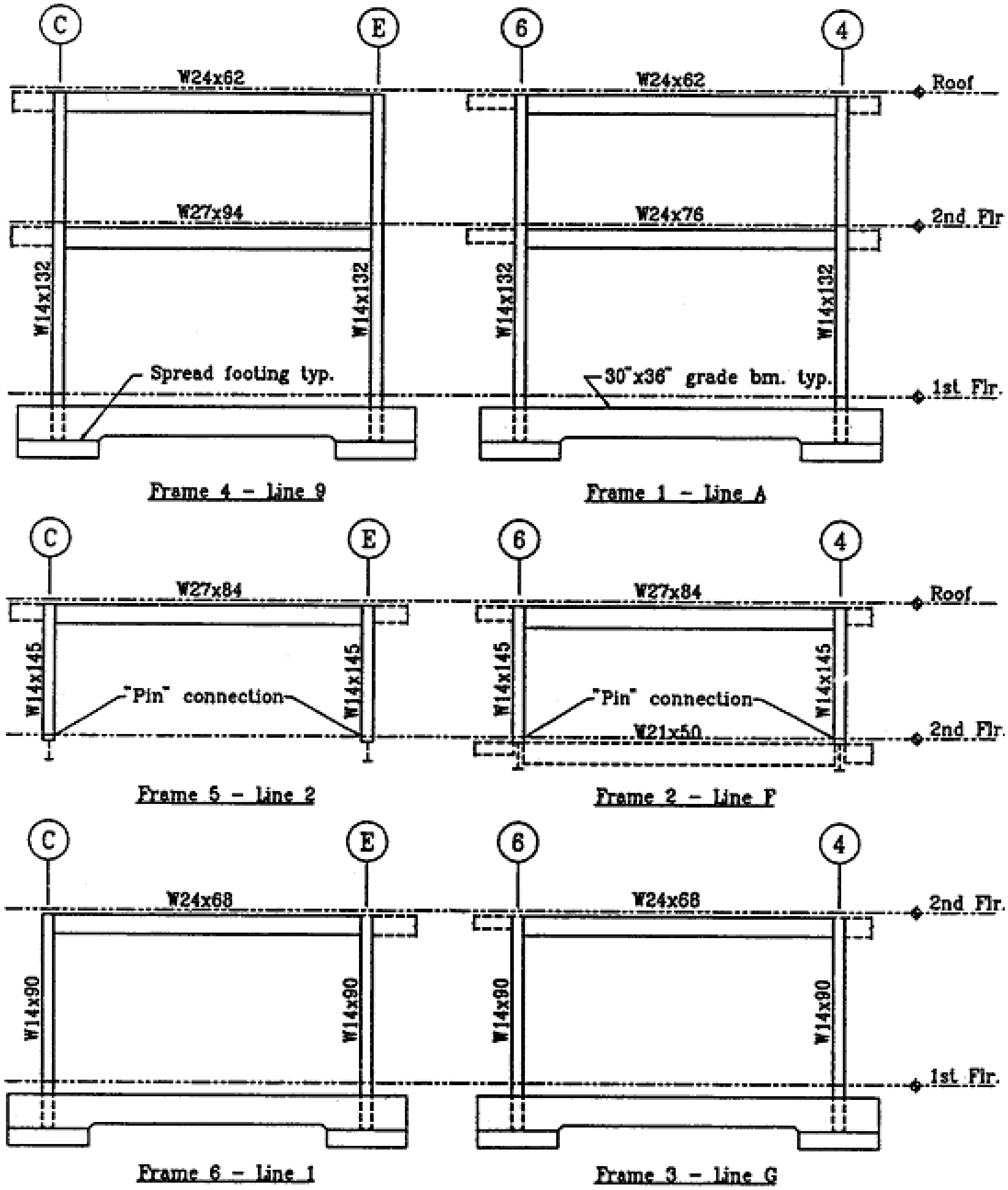


Figure 5.2: Elevations of AAA Building

The ductile moment frames are of conventional, Pre-Northridge construction and the beam to column connections utilize full penetration welded flanges with shear tabs and A325 friction bolts provided for shear transfer.

The second floor and roof are constructed using a 20 gage, 3 inch steel deck fastened to the frames with puddle welds, with 3 inches of light weight concrete fill. The exterior surfaces are faced with thin set brick veneer attached with an adhesive to metal studs.

The soils are classified as brown medium to fine silty sand becoming denser and including some gravel to a depth of 40 feet. This soil condition is classified as type S2 under uniform building code. At the time of construction, each footing excavation was inspected and densified. Foundations are reinforced concrete single footings connected by three-foot deep concrete grade beams. However, the grade beam shear reinforcing was only #4 stirrups at 18 inch spacing, which is not representative of ductile detailing.

b. Description of the Model and Analysis

As mentioned before, initial modeling and inelastic analysis were conducted using *PERFORM 3-D* and *SAP2000* nonlinear version 8 computer programs. As the first step, a model was developed based on all characteristics of the building. Even though the type of steel is A36 (Fy=36 ksi), the actual yield value was determined to be 47.5 ksi as measured in the lab and this value was used in all models except the one for strength irregularity which has the combination of both steel yield values. Sections used for columns and beams are W14, W24 and W27 respectively. The total effective seismic dead load [1] is 908 kips at the second floor level and 684 kips at the roof level for a total of 1592 kips. The related mass of each floor used in dynamic analysis is 28.2 k-S^2/ft and 21.2 k-S^2/ft for second floor and roof respectively and mass moment of inertia of 4.99E4 k-S^2-ft and 3.00E4 k-S^2-ft for second floor and roof respectively. The center of mass (C.O.M) coordinates of second floor and roof are (47.2, 52) and (39.4, 48.4) feet respectively measured from southeast corner of the building as a reference point based on uniform distribution of mass at each floor (Table 1). Based on linear 3-D analyses for a unit lateral load, the center of stiffness (C.O.S) coordinates are (44.5, 44.5) and (39, 39.5) feet from reference point for second floor and roof respectively (Table 1). The locations of the center of mass and stiffness at each floor were shown in Fig.5.3.

<table>
<tr><th>STORY</th><th>WEIGHT</th><th>AREA</th><th rowspan="2">Ai Xi</th><th rowspan="2">Ai Yi</th><th colspan="2">CENTER OF MASS (ft)</th><th colspan="2">CENTER OF STIFFNESS (ft)</th></tr>
<tr><th>No.</th><th>kips</th><th>ft^2</th><th>X = Ai Xi / A</th><th>Y = Ai Yi / A</th><th>X</th><th>Y</th></tr>
<tr><td>ROOF</td><td>684.00</td><td>7040</td><td>276992</td><td>340992</td><td>39.345</td><td>48.44</td><td>39.00</td><td>39.50</td></tr>
<tr><td>2ND</td><td>908.00</td><td>8544</td><td>403456</td><td>444360</td><td>47.22</td><td>52.01</td><td>44.50</td><td>44.50</td></tr>
<tr><td colspan="9">CENTER OF MASS W/ 5% ACC. TORSION
ROOF: X = 39.345 + 0.05 X 80 =43.345 ft , Y= 48.44 + 0.05 X 96= 53.24 ft
2ND FLOOR : X = 47.22 + 0.05 X 96 = 52.02 ft , Y= 52.01 + 0.05 X 101.5 = 57.09 ft</td></tr>
<tr><td colspan="9">AREA OF ROOF = 27.5 X 16 X 16 = 7040 ft^2
AREA OF 2ND FLOOR = 32 X 16 X 16 + 5.5 X 64 = 8544 ft^2
X (ROOF C.O.M) = [(20 X 32 + 3 X 40 + 4 X 72)(16 X 16) + 8 X 16 X 68] / 7040 = 276992 / 7040 =39.345 ft
Y (ROOF C.O.M) = [(20 X 40 + 4 X 88 + 3 X 56)(16 X 16) + 8 X 16 X 24] / 7040 = 340992 / 7040 =48.44 ft

X (2ND C.O.M) = [(20 X 32 + 3 X 72 + 5 X 88 + 4 X 48)(16 X 16) + 5.5 X 64 X 64] /8544 = 403456 /8544 =47.22 ft
Y (2ND C.O.M) = [(24 X 48 + 3 X 56 + 5 X 56)(16 X 16) + 5.5 X 64 X 98.75] / 8544 = 444360 /8544 =52.01 ft</td></tr>
</table>

TABLE 1: Mass and center of Mass for each floor of AAA Building

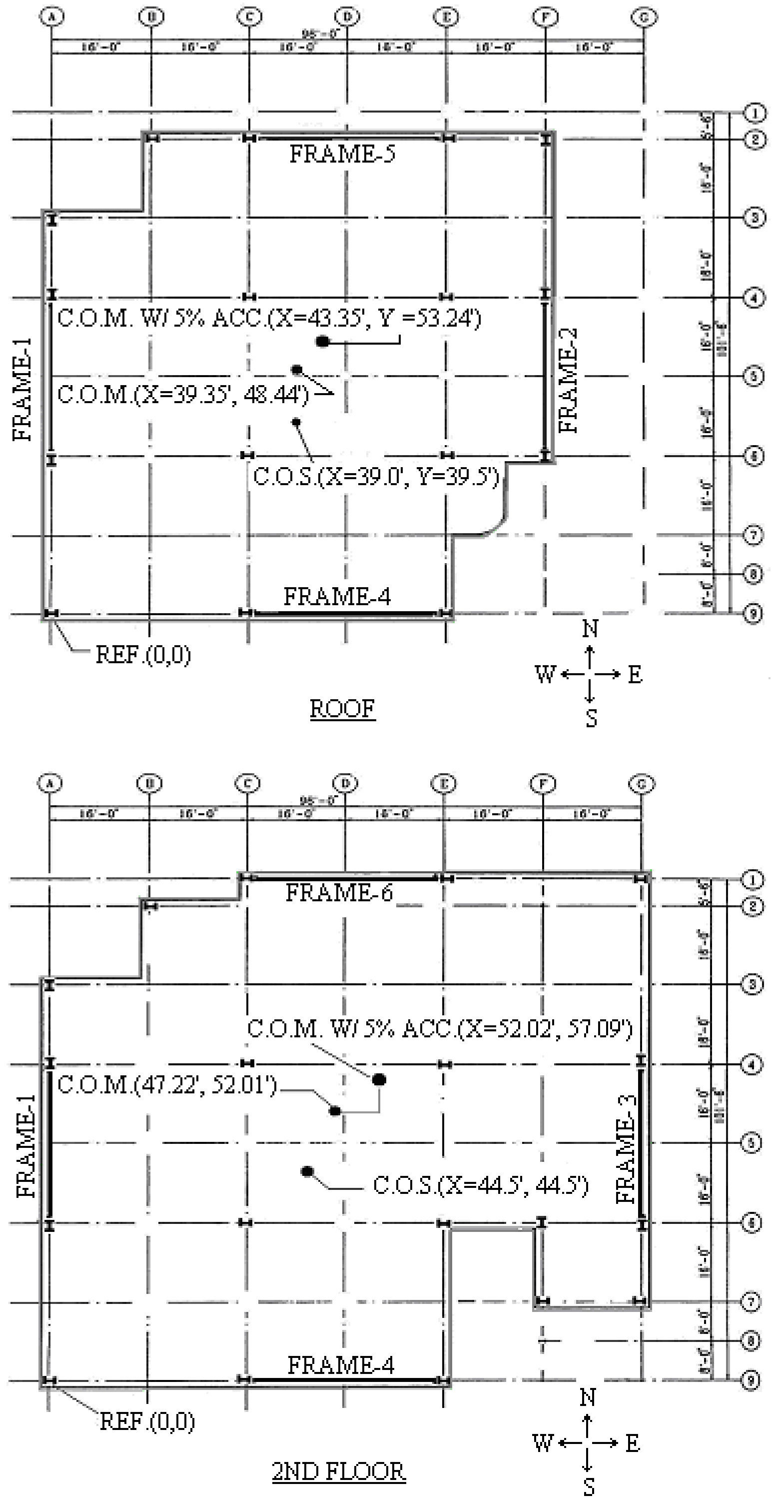


Figure 5.3: Center of Mass and Stiffness AAA Building

Dynamic analyses were conducted with time histories recorded during the 1994 Northridge earthquake (M6.7). Three acceleration records from this earthquake obtained in the San Fernando Valley and utilized for this case study are Newhall, Canoga and Oxnard. The Newhall record obtained at the Newhall fire station has the peak ground acceleration (PGA) of 0.589g and 0.582g for N-S and E-W directions respectively. The corresponding values for peak spectral acceleration (PSA) are 2.2g and 2.6g. The PGA and the PSA of the Canoga record are (0.388g, 1.5g) and (0.35g, 1.3g) respectively for N-S and E-W directions. The Oxnard record has the PGA of 0.41g and 0.32g and the PSA of 1.4g and 1.4g for N-S and E-W directions respectively. The time histories of acceleration, velocity and spectral acceleration of the two components for each record are shown in Fig.5.4, Fig.5.5 and Fig.5.6 respectively. For each analysis, orthogonal acceleration components of each record were applied simultaneously to the base of the structure.

To investigate the effect of torsion on the response of this building, three models with different eccentricities considering both elastic and inelastic behavior were utilized and comparisons among their responses were made. For the first model, moving the C.O.M to the C.O.S location resulted in minimum torsional response of building. The second model considered the actual torsion of the building for both directions due to the differences between C.O.M and C.O.S. locations at both floors. And in the last model 5% accidental torsion based on the Uniform Building Code [29] requirement was added to the actual torsion of the building to magnify the effect of torsion. See Table-1 for detail calculation of the center of mass and stiffness. Utilizing these three models, the effects of +/- acceleration for each pair of N-S and E-W records were considered and based on the results; the pair producing the maximum torsion was chosen.

In order to study the effect of strength irregularity on torsional response of this building, Actual torsion model with inelastic behavior considering strength irregularity was utilized. For this purpose, yield strength value of steel beams and columns for Frame-1 and Frame-4 at west and south elevations set to the actual yield measured value of 47.5 ksi; however the nominal yield strength value of 36 ksi was used for the steel beams and columns of the other frames in order to maximize torsional response due to strength irregularity.

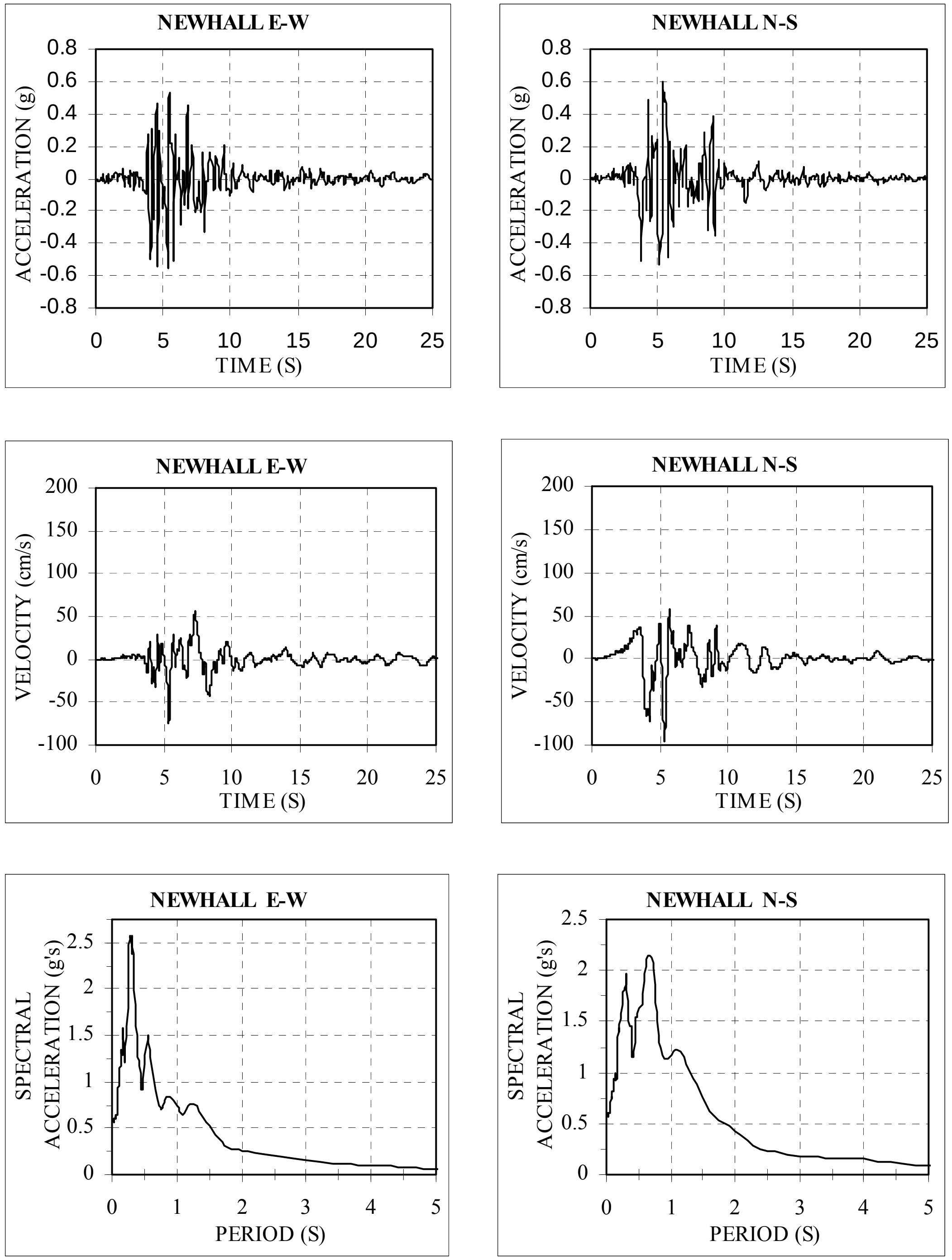


Figure 5.4: Newhall Records from Northridge Earthquake

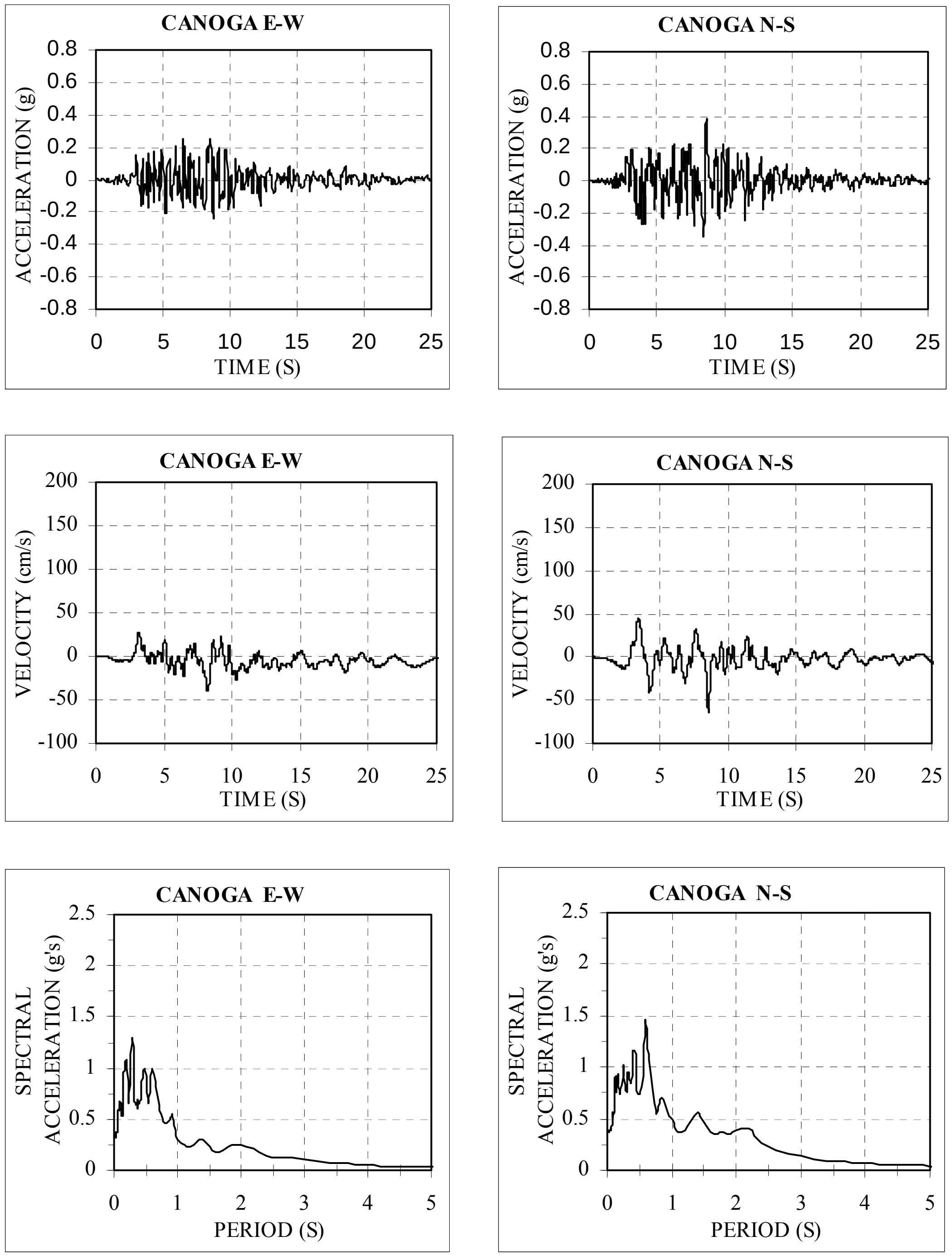


Figure 5.5: Canoga Records from Northridge Earthquake

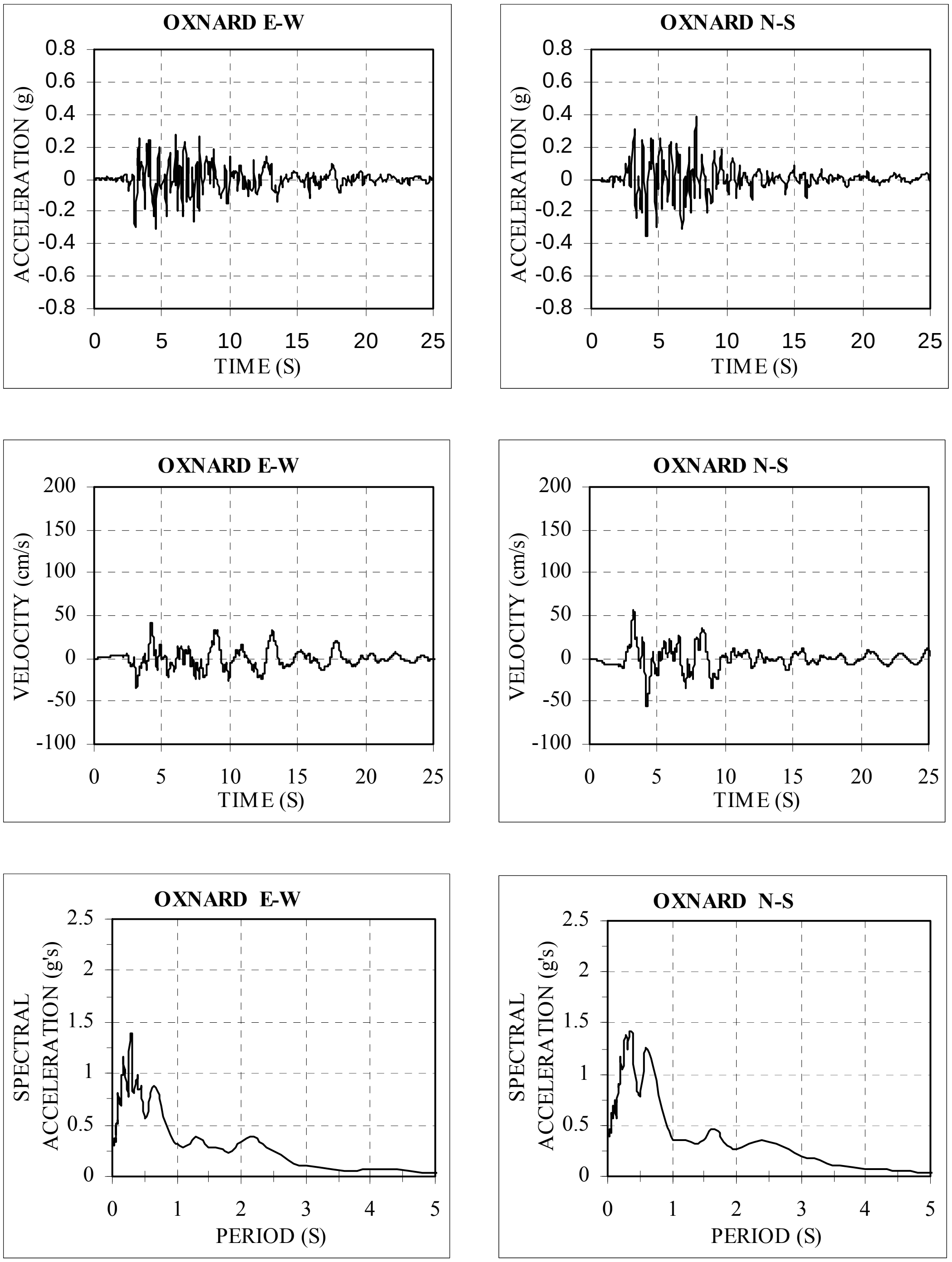
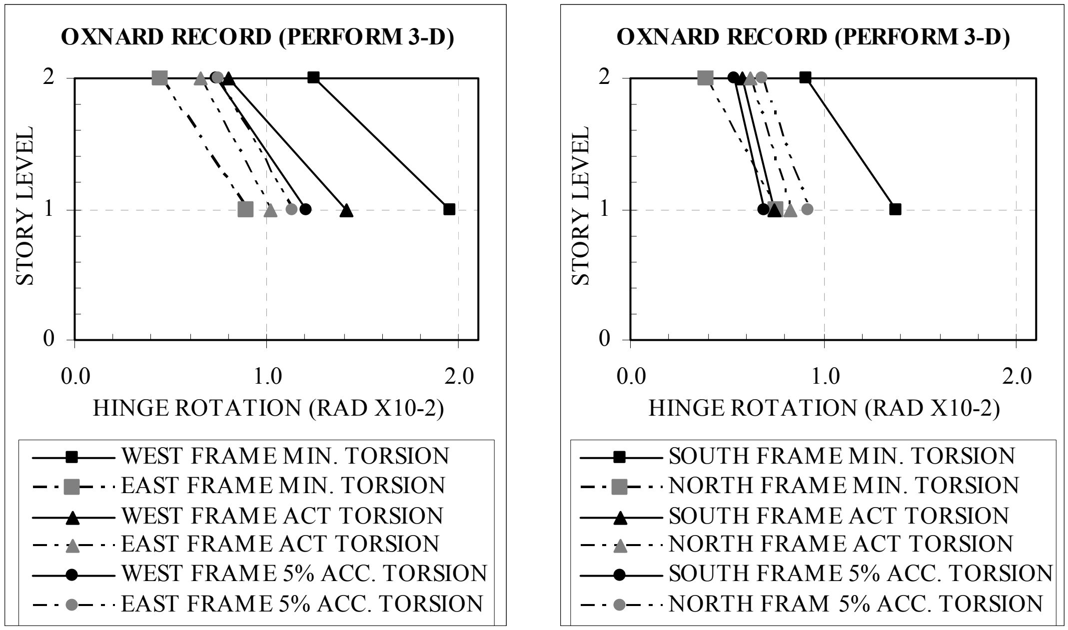


Figure 5.6: Oxnard Records from Northridge Earthquake

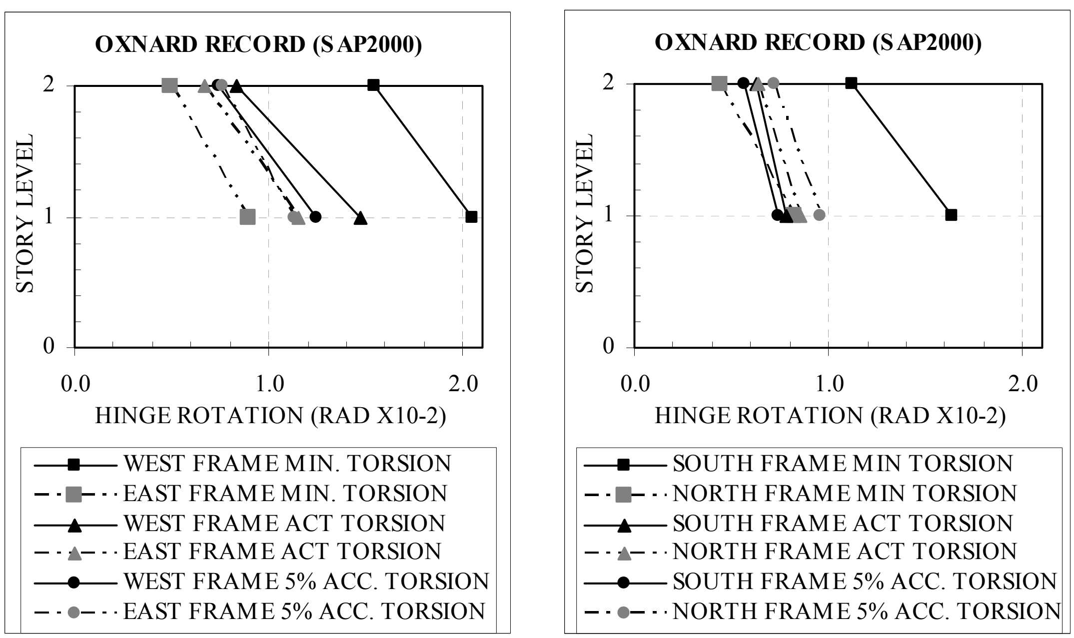
c. Verification of the *PERFORM 3-D* Results

The verification of the *PERFORM 3-D* results is an important task to show the validity of this study. For this purpose, the *SAP2000* nonlinear version 8 with capability of modeling inelastic moment hinges for beams and P-M-M hinges for columns was utilized. Responses of AAA building to N-S and E-W components of Oxnard record applied separately were investigated in this section. Two response parameters were considered for this comparison, namely; total hinge rotation and story displacement. The following models were considered: a) Nonlinear model with actual eccentricity, b) Linear model with actual eccentricity, c) Nonlinear model with minimum eccentricity and d) Nonlinear model with actual plus 5% code eccentricities. The results of these studies are shown in figures 5.7 through 5.12.

Comparison of the first set of graphs (Fig.5.7) related to the envelope of the maximum elastic plus plastic hinge rotation for the nonlinear models at different elevations shows a very good agreement between *PERFORM 3-D* and *SAP2000* results. For instance at east elevation, total hinge rotations for minimum eccentricity model are 0.5 and 0.90 radx10^{-2} respectively for roof and 2^{nd} floor with *SAP2000* which compares to 0.45 and 0.90 radx10^{-2} for *PERFORM 3-D*. By increasing the eccentricity using the other models, this pair of total hinge rotations became (0.70 and 1.15) radx10^{-2} for the actual torsion model and (0.75 and 1.13) radx10^{-2} for the actual plus 5% accidental torsion with *SAP2000.* These were in comparison with (0.75, 1.05) and (0.75, 1.15) radx10^{-2} for *PERFORM 3-D*. The total hinge rotations of east elevation show the difference of 0 to 10 percent between these two programs.



PERFORM 3-D



SAP2000

Figure 5.7: Envelope of Maximum Hinge Rotation, PERFORM 3-D versus SAP2000, OXNARD Record

The investigation of other elevations shows that the difference for 90 percent of all cases is less than 2% and for the remaining 10 percent the difference is varying from 5 to 15 percent.

The results of torsional rotation of 2nd floor and roof and maximum displacement of west and east elevations shown in Figures 5.8 and 5.9, are close enough for *PERFORM 3-D* and *SAP2000* with the difference being less than 1%. Figure 5.10 shows the moment versus hinge rotation of 2nd floor beam at the west elevation for the model with actual eccentricity and again the results of two programs indicate good agreement. Figures 5.11 and 5.12 show the time-history plot of roof displacement at west elevation for actual torsion and actual plus 5% accidental torsion cases. The results including residual displacements due to nonlinear behavior are very close and also show a good agreement.

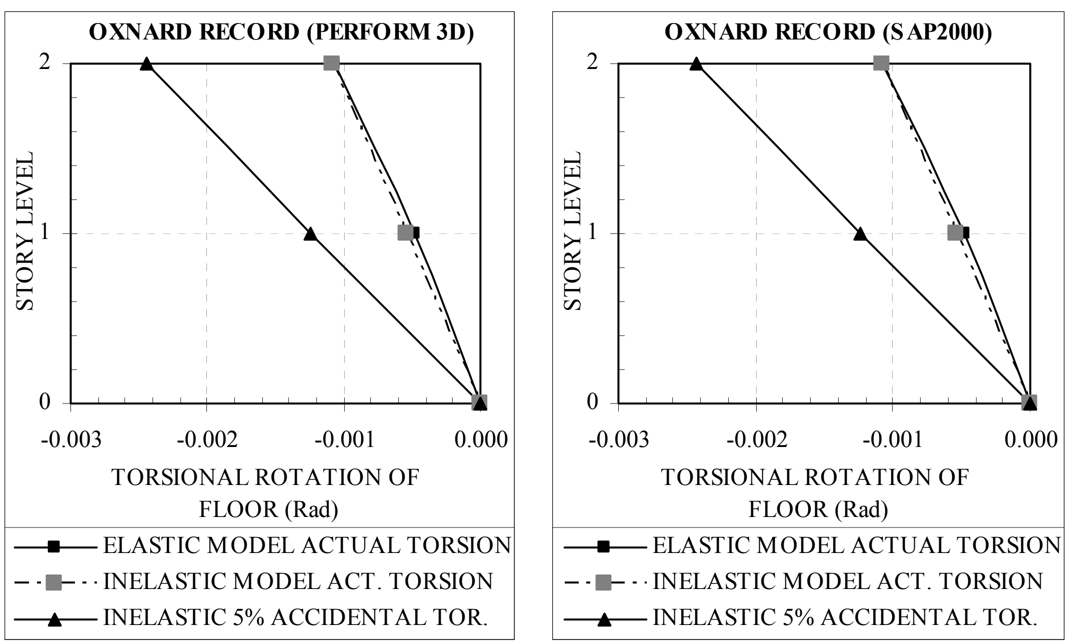
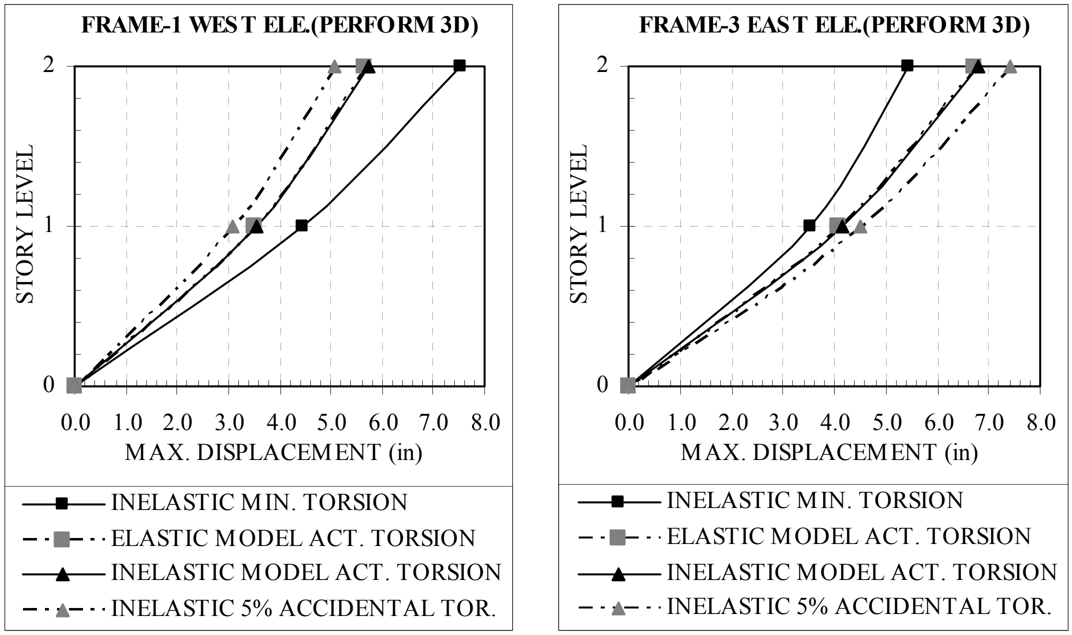
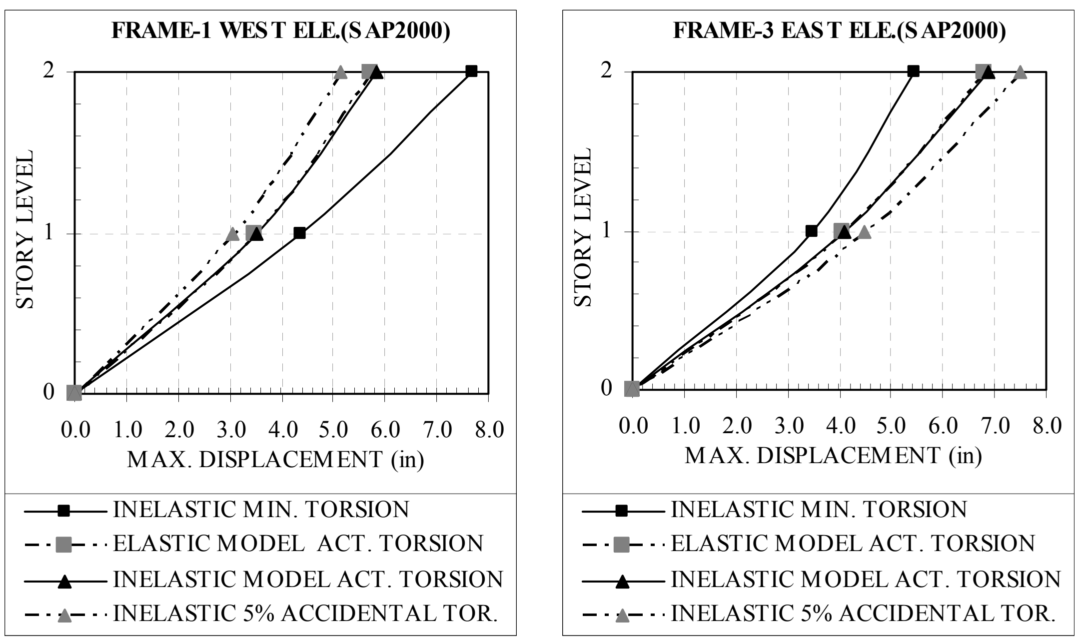


Figure 5.8: Torsional Rotation of 2nd Floor and Roof, PERFORM 3-D versus SAP2000, OXNARD Record



PERFORM 3-D



SAP2000

Figure 5.9 –Maximum Frame Displacement PERFORM 3-D versus SAP2000, OXNARD Record

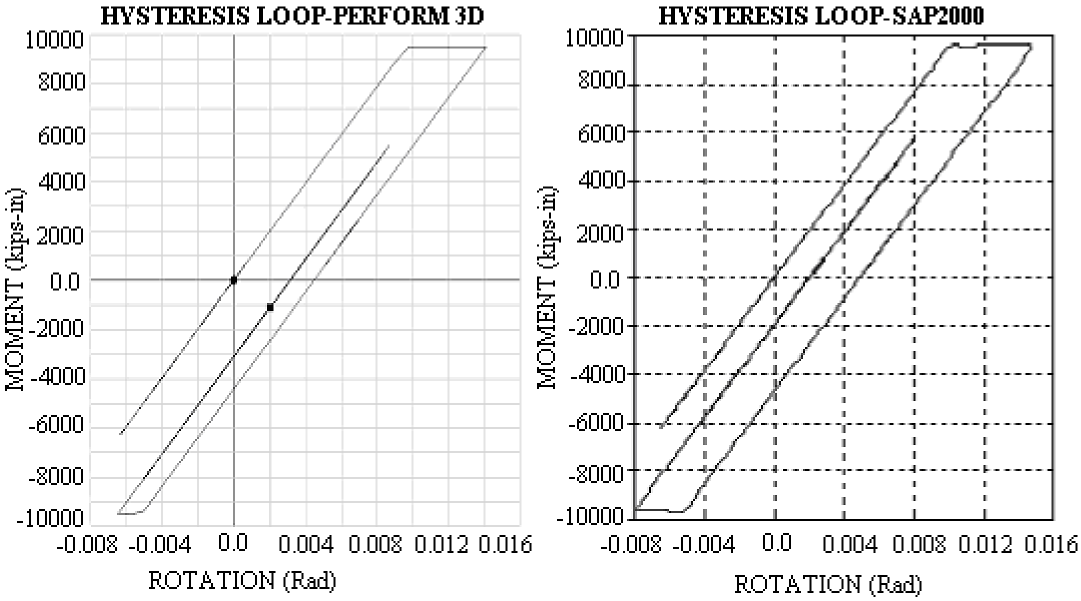
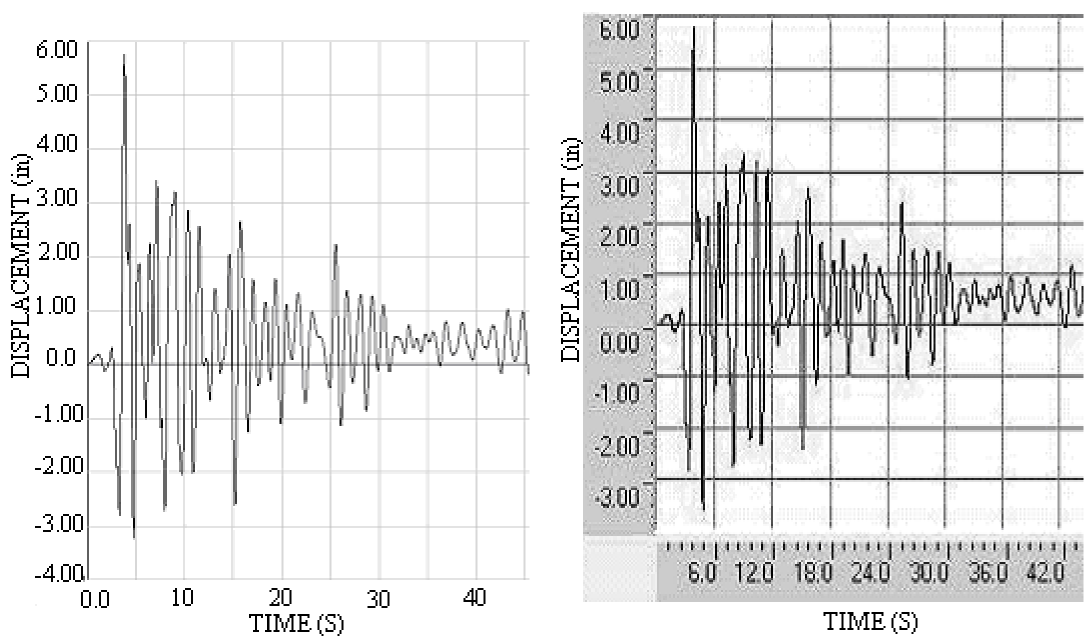


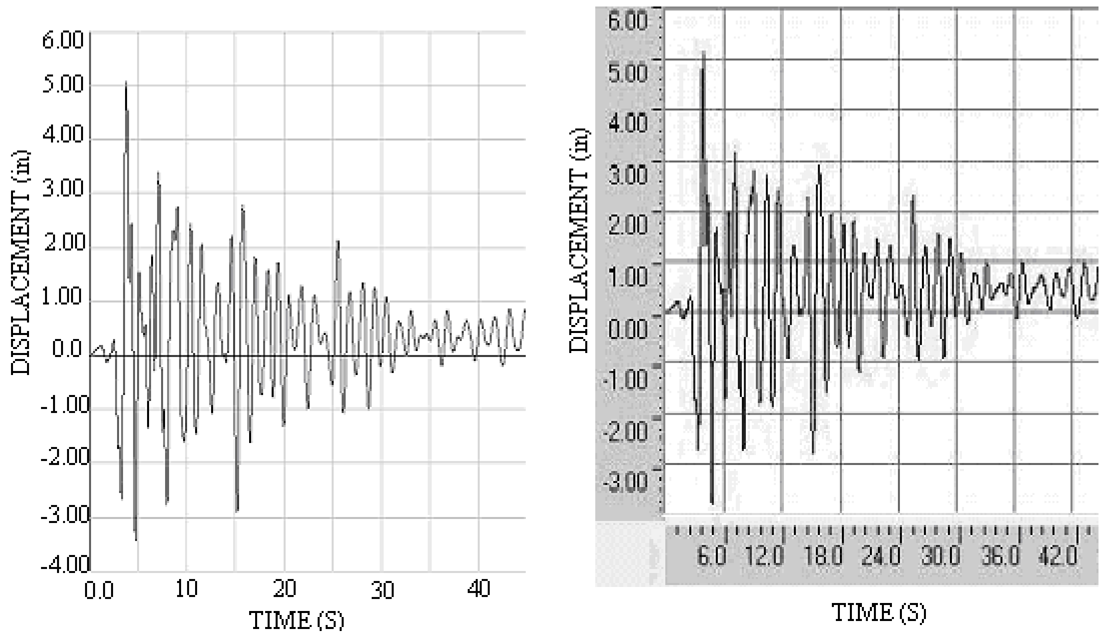
Figure 5.10 – Moment versus Hinge Rotation of 2nd Floor Beam, West Elevation, Actual Eccentricity Case, PERFORM 3-D versus SAP2000, OXNARD Record



PERFORM 3-D

SAP2000

Figure 5.11 – Time-history Displacement of Roof, West Elevation, PERFORM 3-D versus SAP2000, Actual Eccentricity Case, OXNARD Record



PERFORM 3-D SAP2000

Figure 5.12 – Time-history Displacement of Roof, West Elevation, PERFORM 3-D versus SAP2000, 5% Accidental Torsion Case, OXNARD Record

As this investigation shows, the results of these two programs have a very good agreement. This validates the *PERFORM 3-D* program for use in this study.

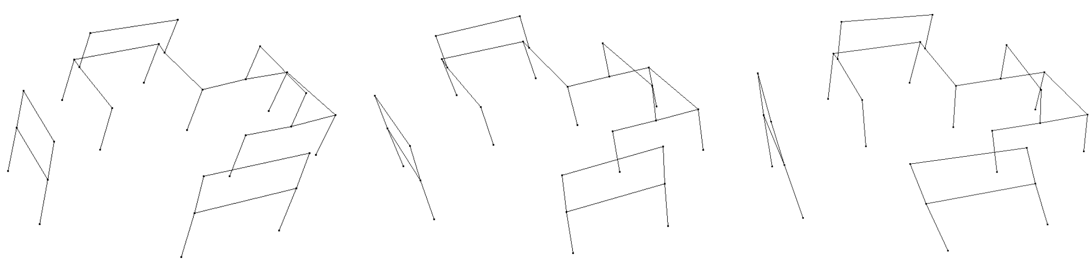
d. Discussion of the Results

The nonlinear dynamic responses of AAA building to three different records from Northridge earthquake are summarized in Table 2 and shown in figures 5.13 through 5.34. Modal characteristics of the building for the first six modes have been summarized and shown in Table 2 and Figure 5.13. The periods in seconds and modal participation factors of the first three modes of vibration based on the initial stiffness of members are (1.23, 92.3%) for 1st mode at N-S direction, (1.19, 91.4%) for 1st mode at E-W direction and (0.75, 1.95%) for 1st torsional mode of vibration.

The periods of the first three modes of vibration are close enough to the periods of reference [1].

Mode No.	PERIOD (S)	Modal Participation E-W Direction	Modal Participation N-S Direction	Cumulative Mode For E-W	Cumulative Mode For N-S	Relative Direction
1	1.23E+00	0.0025	0.9226	0.0025	0.9226	N-S
2	1.19E+00	0.9141	0.0027	0.9167	0.9253	E-W
3	7.47E-01	0.0195	0.0002	0.9361	0.9255	Torsional
4	4.66E-01	0.0309	0.031	0.9671	0.9565	N-S
5	3.83E-01	0.0265	0.0396	0.9936	0.9961	E-W
6	2.43E-01	0.0064	0.0039	1	1	Torsional

TABLE 2 – Period and Mass Contribution of each Mode for 2–Story AAA Building



1ST Mode of Vibration 2ND Mode of Vibration 3RD Mode of Vibration

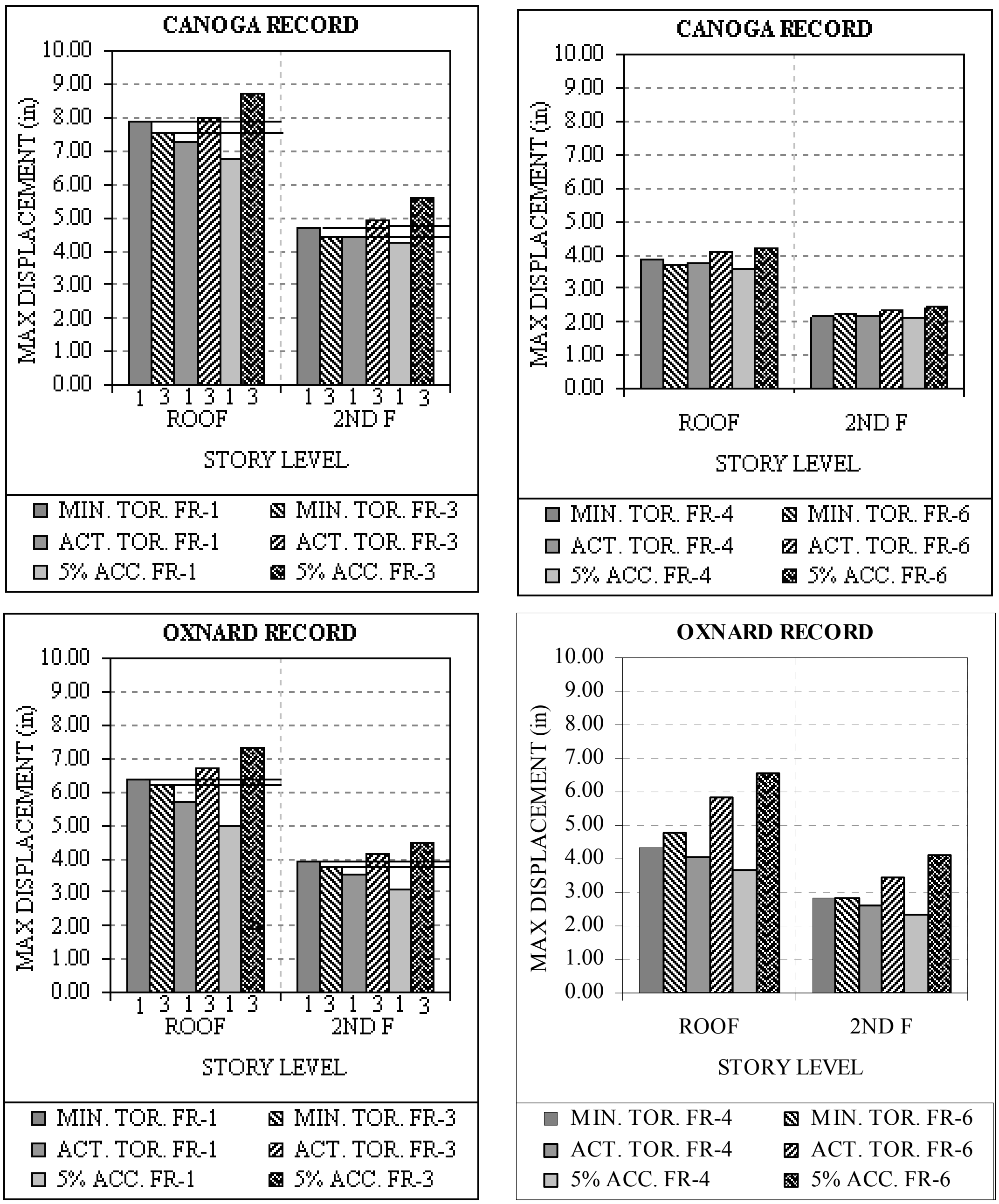
Figure 5.13 –Mode Shape, AAA Building

As discussed before, the AAA building at 2nd floor has a large setback at east and north elevations. The east frame has a setback of 16'-0" whereas the north frame has a setback of 5'-6". There are also re-entrant corners at the northwest and southeast corners of the building. Due to these plan irregularities, the stiffness of the moment frames at east and north elevations reduces and that causes considerable torsional effect which can be observed through the building responses. Comparison of the plan irregularities of this building with the 1997 uniform building code shows that these plan irregularities meet the code requirements for plan structural irregularities of type 2 (Re-entrant corners) and 4 (Out-of-plan offsets) and since this building is a 2-story irregular structure not more than five stories or 65 feet in height, could be analyzed and designed using any of lateral-force procedure.

The responses of the building to two moderate earthquake records of Canoga and Oxnard show the same pattern for both records. See graphs on Fig.5.14.

The results of N-S and E-W components of each record applied simultaneously shows that N-S component has much stronger responses than E-W component which can be seen on acceleration response spectra of each record as well. By comparing the results of column bar graphs, the effect of actual and actual plus 5% accidental torsion on the maximum inelastic displacement can be studied. For Canoga record with larger responses in N-S direction, Frame-3 that suffers severe damage, the maximum displacement increase due to torsion at roof is approximately (8.71-7.53)/7.53=1.18/7.53= 15.7%. The average changes are 7.0% and 15% for actual torsion case and actual plus 5% accidental torsion case respectively compared to minimum torsion case for the roof. Accidental torsion alone has an average change of about 8% on maximum displacement of the roof.

For the N-S direction of the Oxnard record, the maximum displacement increase due to torsion at roof for Frame-3 reaches approximately to (7.33-6.25)/6.25=1.08/6.25= 17.3%. The average changes are 9.0%, 20% for actual torsion case and actual plus 5% accidental torsion case respectively compared to minimum torsion case for the roof and Accidental torsion alone has an average change of about 11% on maximum displacement of the roof. Results of these graphs have been summarized in form of a table later on in the conclusion chapter.



N-S E-W

Figure 5.14 – Comparison of the Maximum Displacement of the Building, Inelastic Models, CANOGA and OXNARD Record

Figure 5.15 and 5.16 show the maximum displacements of all inelastic and elastic models for Canoga and Oxnard records respectively. Starting with minimum torsion model, for inelastic behavior the maximum displacements at roof for Frame-1 at west elevation and Frame-3 at east elevation are 7.90 and 7.53 inches for Canoga and 6.38 and 6.25 inches respectively for Oxnard record and for Frame-4 at south elevation and Frame-6 at north elevation are 3.91 and 3.75 inches for Canoga and 4.36 and 4.76 inches respectively for Oxnard record. For elastic behavior the maximum displacements at roof for Frame-1 and Frame-3 are 6.95 and 7.24 inches for Canoga and 6.14 and 6.12 inches respectively for Oxnard record and for Frame-4 and Frame-6 are 4.00 and 3.86 inches for Canoga and 4.65 and 4.80 inches respectively for Oxnard record.

For actual torsion model, displacements at roof for inelastic behavior of Frame-1 and Frame-3 become 7.28 and 8.00 inches for Canoga and 5.72 and 6.74 inches respectively for Oxnard record and for Frame-4 and Frame-6 become 3.78 and 4.11 inches for Canoga and 4.04 and 5.83 inches for Oxnard record. For elastic behavior the maximum displacements at roof for Frame-1 and Frame-3 become 7.42 and 7.93 inches for Canoga and 5.65 and 6.65 inches respectively for Oxnard record and for Frame-4 and Frame-6 become 3.62 and 4.31 inches for Canoga and 4.32 and 6.33 inches respectively for Oxnard record.

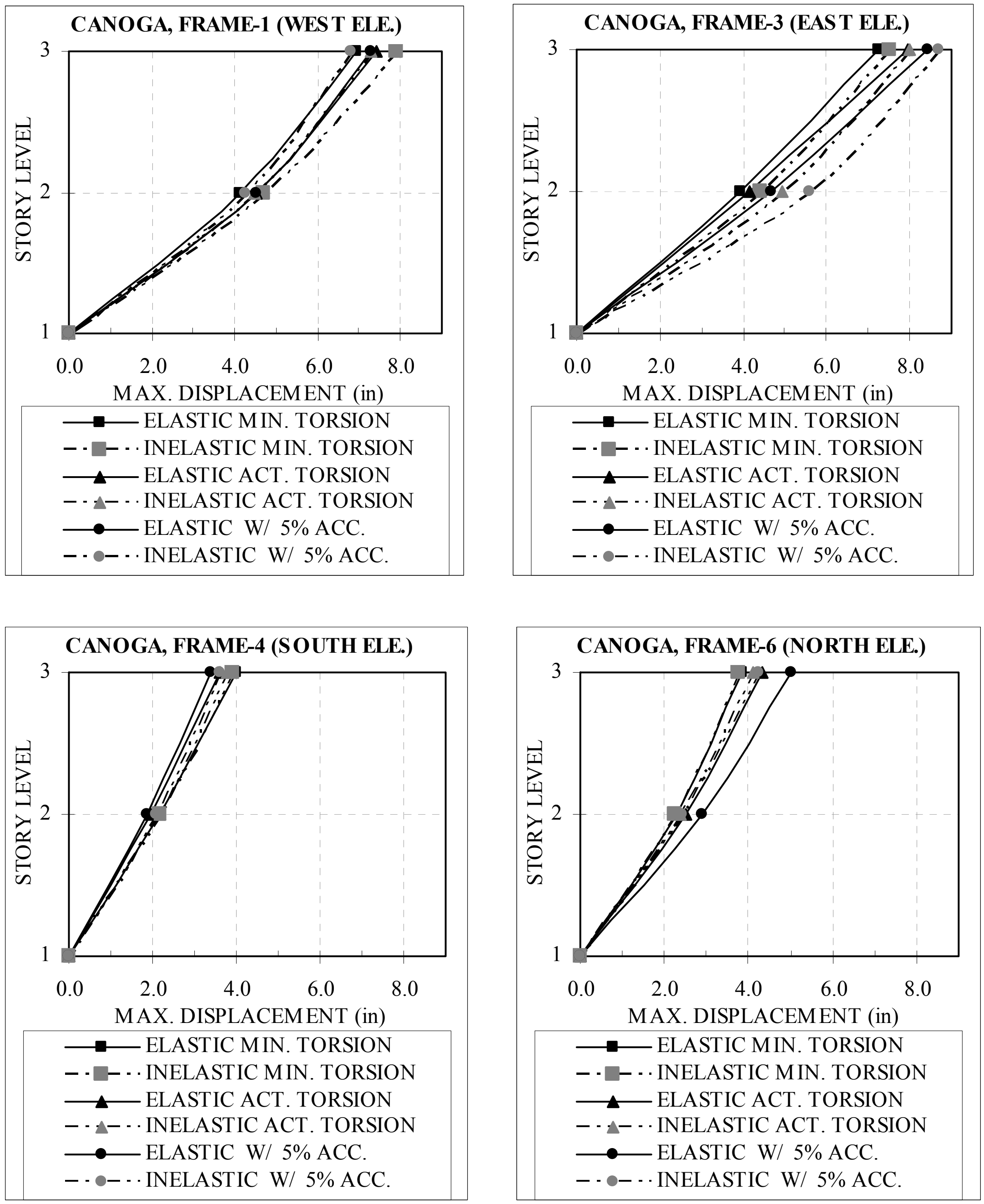


Figure 5.15 -MAX Displacement of the Building, CANOGA Record

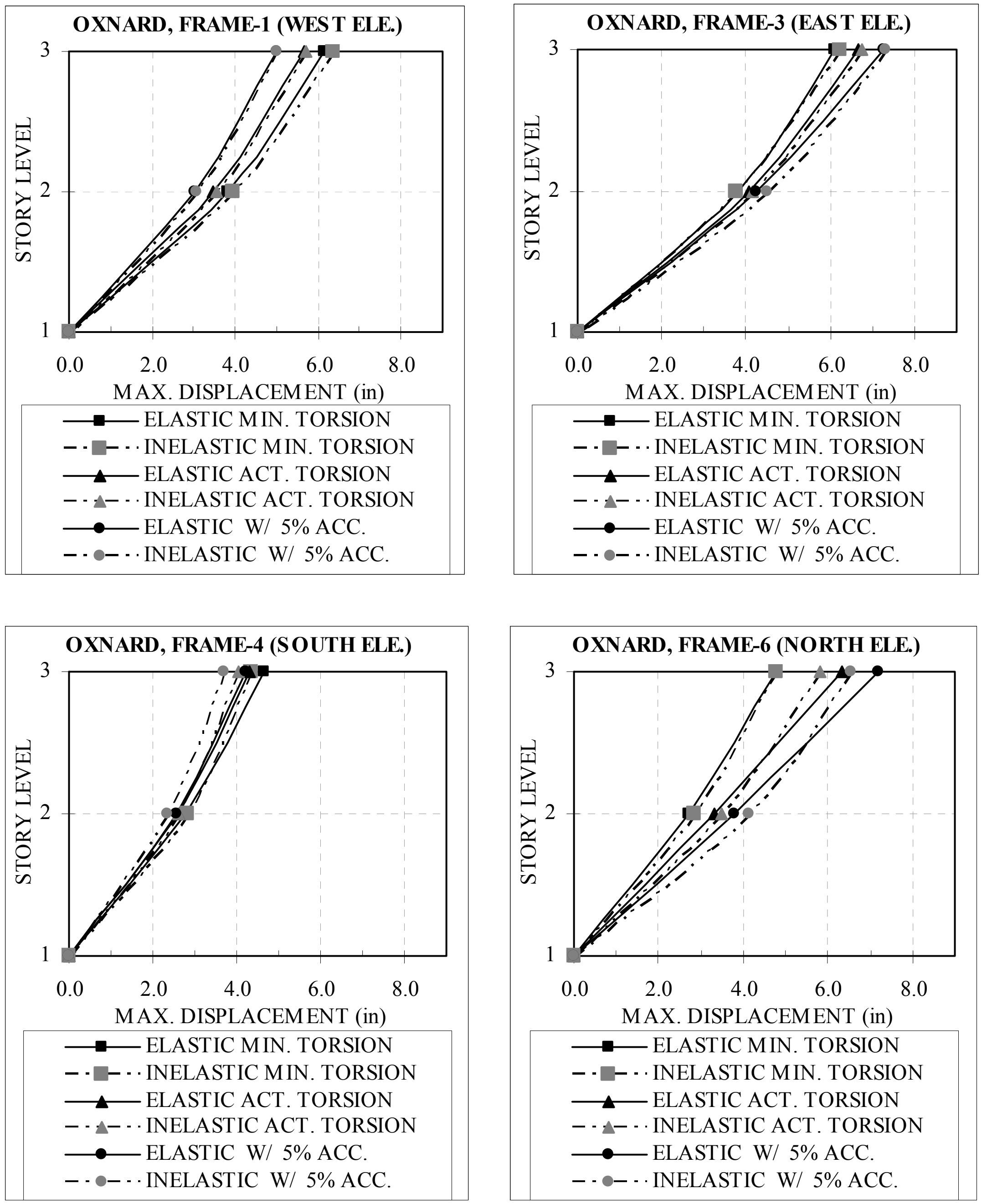
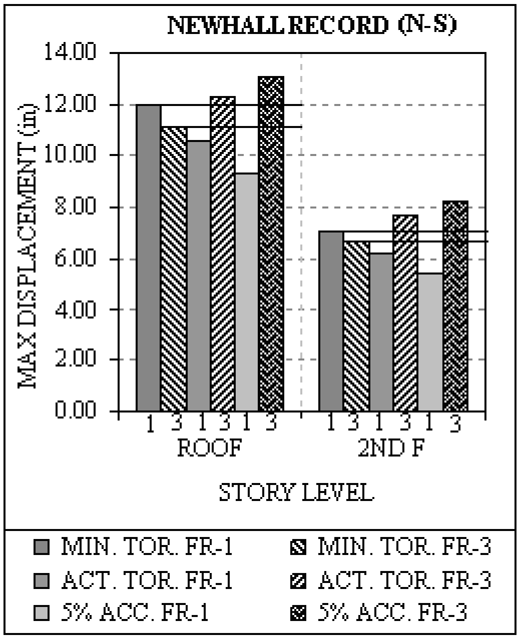


Figure 5.16 -MAX Displacement of the Building, OXNARD Record

In the model with 5% accidental torsion plus actual torsion, displacements at roof for inelastic behavior of Frame-1 and Frame-3 become 6.80 and 8.71 inches for Canoga and 5.00 and 7.33 inches respectively for Oxnard record and for Frame-4 and Frame-6 become 3.62 and 4.22 inches for Canoga and 3.68 and 6.53 inches for Oxnard record. For elastic behavior the maximum displacements at roof for Frame-1 and Frame-3 become 7.30 and 8.44 inches for Canoga and 4.99 and 7.28 inches respectively for Oxnard record and for Frame-4 and Frame-6 become 3.41 and 5.00 inches for Canoga and 4.23 and 7.21 inches respectively for Oxnard record.

Considering the frame with largest torsional effect, (Frame-3, east elevation), the actual inelastic torsion adds 6.2% and 7.8% to the displacement and the 5% accidental torsion adds an additional 9.5% and 9.5% for Canoga and Oxnard records respectively. For elastic behavior, the actual torsion adds 9.5% and 8.7% to the displacement and the 5% accidental torsion adds an additional 7.0% and 10.3% for Canoga and Oxnard records respectively. These displacements show a considerable amount of torsion at both directions for Oxnard record and only N-S direction for Canoga record.

The response of building to Newhall record with higher peak spectral accelerations in both directions is much higher but still follows the same general pattern of responses. The results of column bar graphs on Figure 5.17 show the effect of actual and actual plus 5% accidental torsion on maximum inelastic displacement. For Frame-3, the change due to torsion at roof is approximately (13.10-11.16)/11.16=1.94/11.16= 17.4%. This is the frame that suffers sever damages. The average changes are 11.0% and 20% for actual torsion case and actual plus 5% accidental torsion case respectively compared to minimum torsion case for the roof. Accidental torsion alone has an average change of about 9% on maximum displacement of the roof.



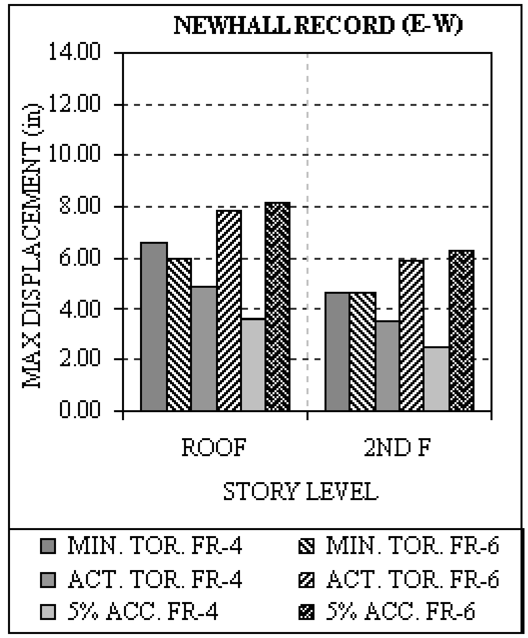


Figure 5.17 – Comparison of the Maximum Displacement of the Building, Inelastic Models, NEWHALL Record

Figure 5.18 shows the maximum displacements of each frame for all inelastic and elastic models. Note that the maximum displacements for elastic cases are higher than the inelastic cases but the changes between each inelastic cases, are higher than elastic cases. Starting with minimum torsion model, the maximum displacements at roof for Frame-1 at west elevation and Frame-3 at east elevation are 12.0 and 11.16 inches for inelastic and 16.9 and 16.6 inches respectively for elastic behavior and for Frame-4 at south elevation and Frame-6 at north elevation are 6.6 and 6.0 inches for inelastic and 10.6 and 11.0 inches respectively for elastic behavior.

For actual torsion model, displacements at roof for Frame-1 and Frame-3 become 10.6 and 12.3 inches for inelastic and 16.3 and 17.2 inches respectively for elastic behavior and for Frame-4 and Frame-6 become 4.87 and 7.87 inches for inelastic and 9.8 and 11.8 inches for elastic behavior.

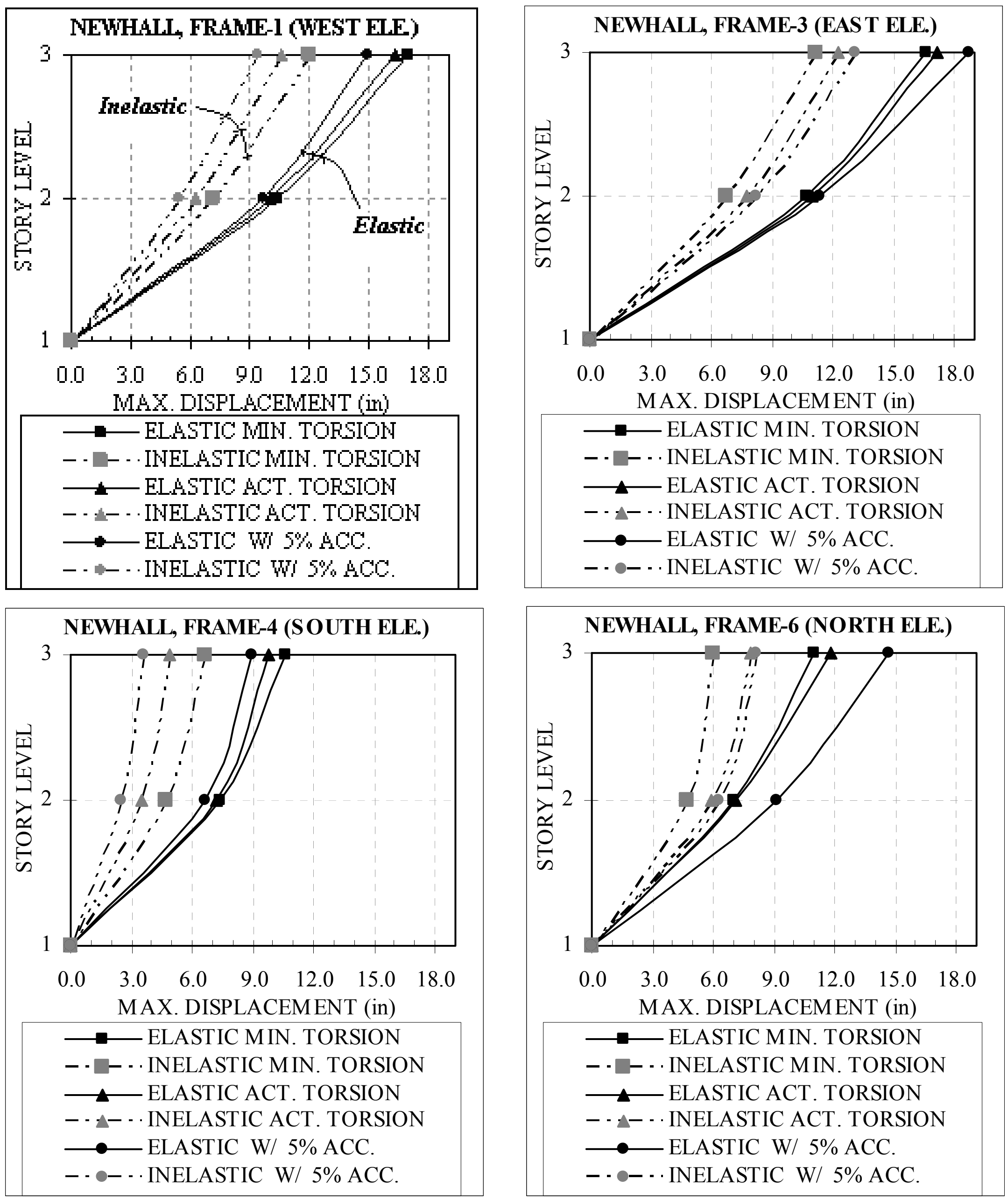


Figure 5.18 -MAX Displacement of the Building, NEWHALL Record

In the model with 5% accidental torsion plus actual torsion, displacements at roof for Frame-1 and Frame-3 become 9.34 and 13.1 inches for inelastic and 14.9 and 18.72 inches respectively for elastic behavior and for Frame-4 and Frame-6 become 3.57 and 8.15 inches for inelastic and 8.94 and 14.7 inches for elastic behavior. Considering the frame with largest torsional effect, (Frame-3, east elevation), the actual inelastic torsion adds 10.2% to the displacement and the 5% accidental torsion adds an additional 7.2% for Newhall record. For elastic behavior, the actual torsion adds 3.6% to the displacement and the 5% accidental torsion adds an additional 9.2% for Newhall record.

The graphs in Figures 5.19 through 5.21 are showing the amount of torsional rotation of each floor for linear and nonlinear cases as the main subject of this study. Comparison of any linear and nonlinear model shows the amount of inelastic torsion due to material nonlinearity. Figure 5.19 shows considerable amount of inelastic torsion of roof and 2nd floor for Canoga record.

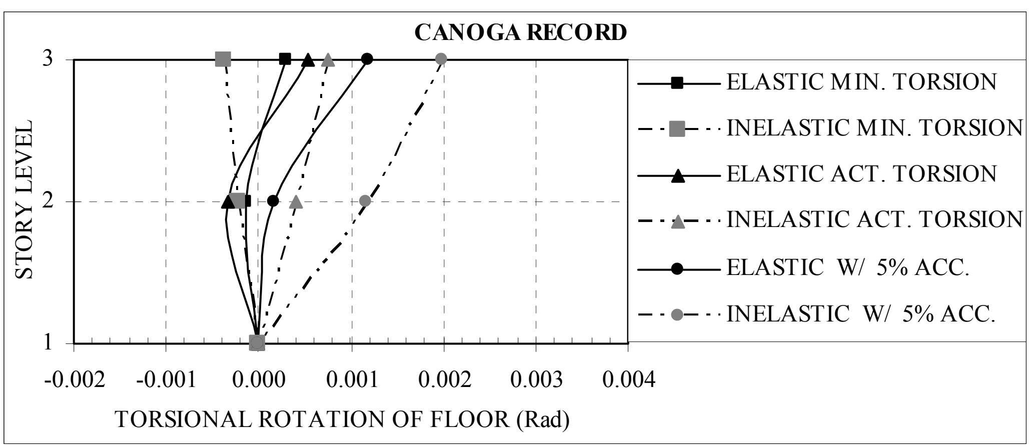


Figure 5.19 –Torsional Rotation of 2nd Floor and Roof, CANOGA Record

The amount at roof is approximately (0.75-0.53)e-3/0.53e-3=0.22/0.53= 42% increase to the torsional rotation for actual torsion case and about (1.99-1.19)e-3/1.19e-3=0.80/1.19= 67% increase to the torsional rotation for actual plus 5% accidental torsion. Accidental torsion has a big increase on torsional rotation and reaches to 125% and 165% for elastic and inelastic torsion of the roof. Since elastic torsional rotation at 2nd is very low, inelastic torsional rotation causes very large increase (>100%) to the torsional rotation of actual and actual plus 5% accidental torsion cases.

Inelastic torsion for Oxnard record (Fig.5.20) is not considerable and only causes about 8 to 22 percent increase to torsional rotation at 2nd floor. For Newhall record (Fig.5.21), inelastic torsion is considerable for both actual and actual plus 5% accidental torsion cases at 2nd floor and only actual torsion case at roof. The amount is about to 47 and 67 percent increase to torsional rotation for actual torsion and actual plus 5% accidental torsion cases respectively at the 2nd floor and 88 percent increase to torsional rotation for actual torsion case at roof.

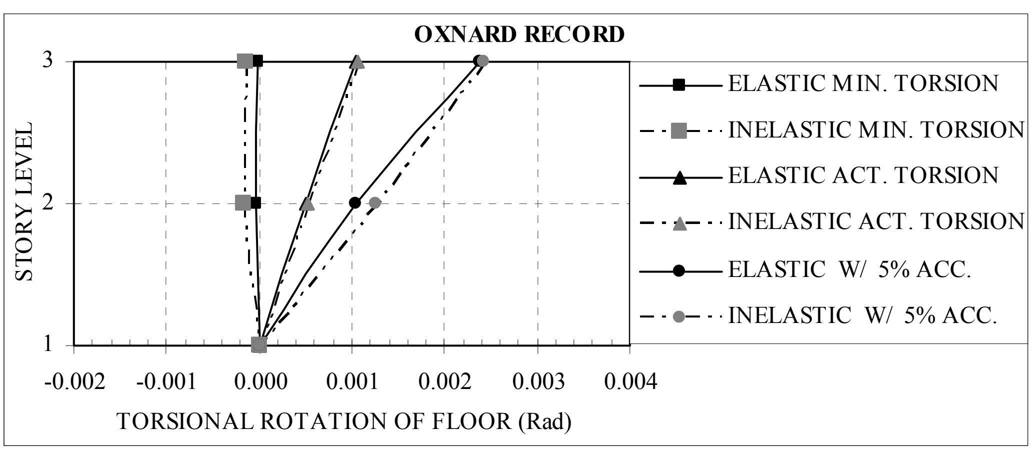


Figure 5.20 - Torsional Rotation of 2nd Floor and Roof, OXNARD Record

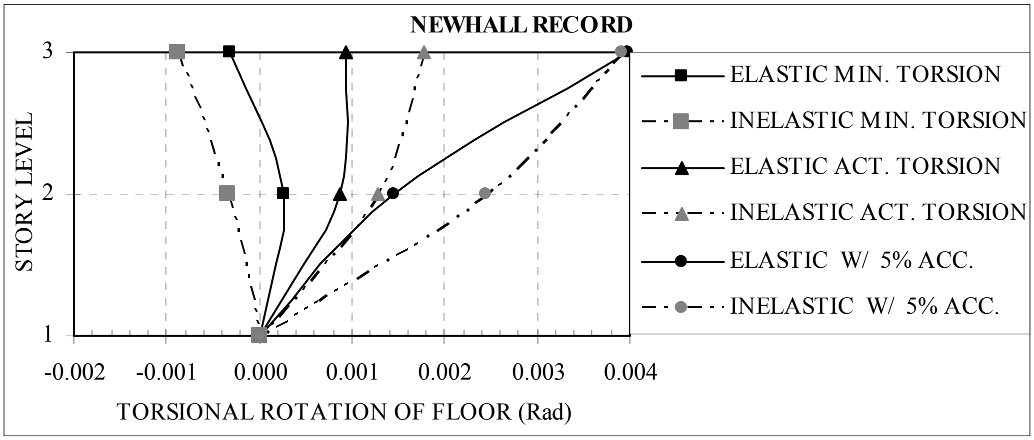


Figure 5.21 - Torsional Rotation of 2nd Floor and Roof, NEWHALL Record

Comparison of the graphs for Canoga and Newhall records (Fig.5.19 and Fig.5.21) with Oxnard record (Fig. 5.20) shows that for minimum torsion case, the amount of torsional rotation is not zero for both elastic and inelastic cases. For elastic case, the actual eccentricity of earthquake motion causes a torsional rotation. However for inelastic case, that may be the indication of strength eccentricity. For this purpose and in order to study the effect of strength irregularity on torsional response of this building, a modified Actual torsion model including strength irregularity for inelastic behavior only, was utilized. As it mentioned before, for this model yield strength value of steel beams and columns for Frame-1 and Frame-4 at west and south elevations are set to the actual measured yield strength value of 47.5 ksi; however the nominal yield strength value of 36 ksi was used for the steel beams and columns of the other frames.

The comparison of two models with and without strength eccentricity for each floor in Figure 5.21a shows the effect of strength irregularity on the inelastic torsional rotation. This effect has a considerable increase to the torsional rotation of most cases. For Canoga record, the increase at roof is approximately (1.38-0.75)e-3/0.75e-3= 0.63/0.75= 83% and for 2nd floor is about (1.14-0.41)e-3 /0.41e-3= 0.73/.41= 179% due to strength eccentricity which shows a very large increase for both floors. For Oxnard record, the increase at roof and 2nd floor are approximately 10% and 59% respectively due to strength eccentricity which still shows a large increase at 2nd floor. For Newhall record, the torsional rotation increases by about 28% at roof and by 26% at 2nd floor. Results of this study suggest that the strength eccentricity has a larger effect on the moderate earthquake records than the one with higher peak acceleration and also this effect is much larger at 2nd floor than roof for those moderate earthquake records.

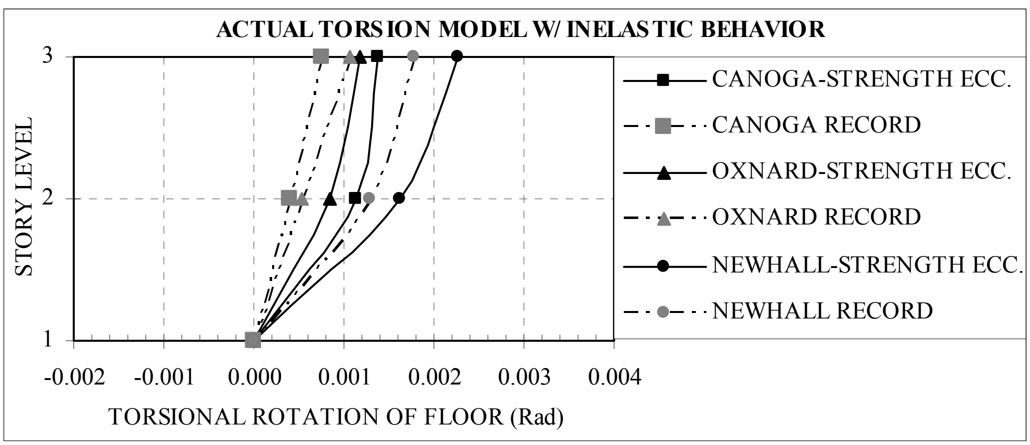


Figure 5.21a - Torsional Rotation of 2nd Floor and Roof, Actual Torsion Model with Inelastic Behavior Considering the Strength Eccentricity

The envelopes of the maximum elastic plus plastic (total) hinge rotations are shown in Figures 5.22 thru 5.27 for beams and columns. According to FEMA 273 guidelines, the maximum plastic hinge rotation relative to the expected performance level of the building is limited to 0.004 radian for Immediate Occupancy level with minimal or no damage to the structural elements, 0.025 radian for Life Safety level with extensive damage to the structural and nonstructural components and 0.043 radians for Collapse Prevention level with failure of nonstructural components and large permanent drifts but functioning of load bearing walls and columns.

The total hinge rotations of beams for Canoga and Oxnard records are shown in figures 5.22 and 5.23. For Frame-3 the maximum rotation is 0.009 and 0.008 radians at roof and 0.015 and 0.012 radians at second floor for Canoga and Oxnard records respectively. Frame-1 has the maximum total hinge rotation for both records and reaches to 0.014 radians at roof and 0.022 radians at second floor. The elastic hinge rotation for second floor W24x76 beam based on formula (4-4) is 0.01 radians, so the maximum plastic hinge rotation for these two records is 0.012 radians which is within the second deformation limit of FEMA 273.

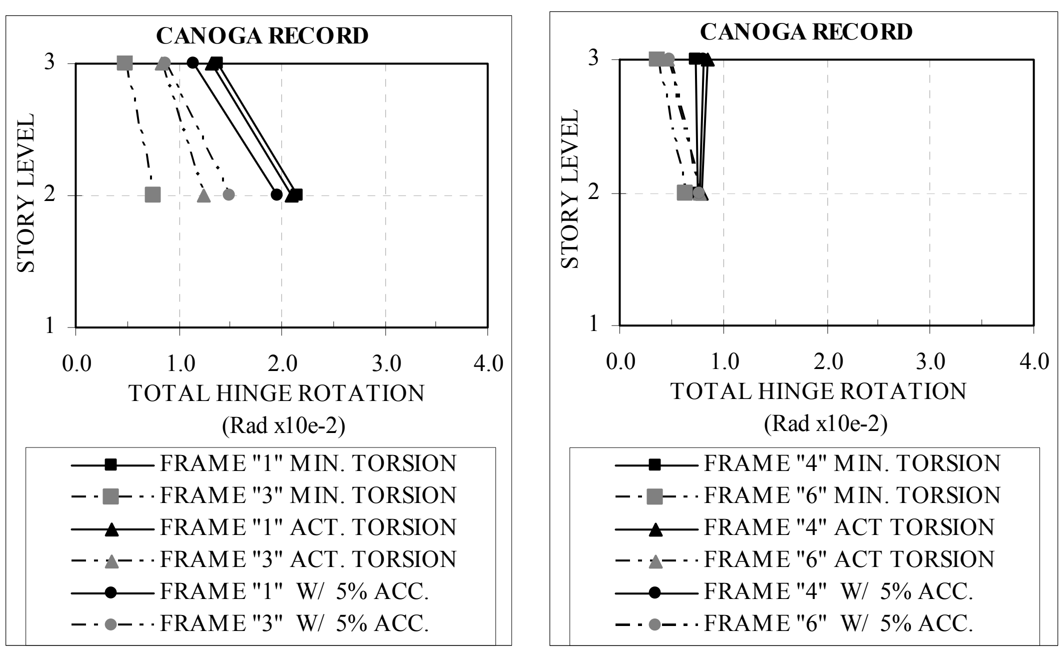


Figure 5.22 –Envelope of Maximum Hinge Rotation of Beams, CANOGA Record

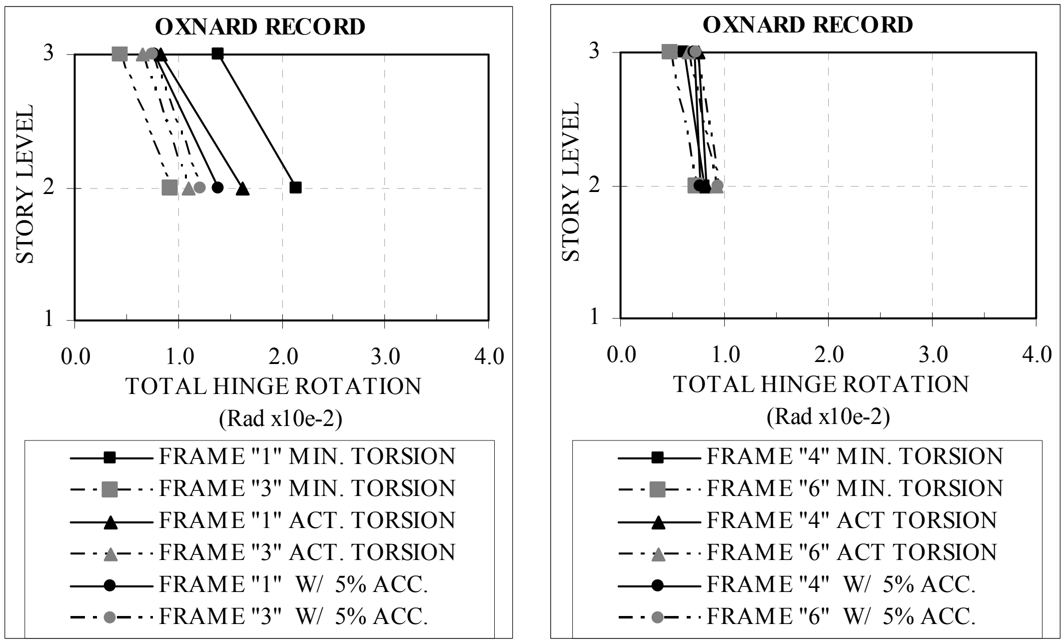


Figure 5.23 –Envelope of Maximum Hinge Rotation of Beams, OXNARD Record

For Newhall record (Fig.5.24), the total hinge rotations of beams are higher than two other records and it reaches to the 0.034 radians at 2nd floor beam and 0.025 radians at roof for Frame-1. For Frame-3 the maximum rotation is 0.03 and 0.019 radians at 2nd floor and roof for actual plus 5% accidental torsion case. Considering 0.01 radians as elastic hinge rotation of 2nd floor W24x76 beam, the maximum plastic hinge rotation for the Newhall record is 0.024 radians which is very close to the 2nd deformation limit of FEMA 273.

The total hinge rotations of columns for Canoga and Oxnard records (figures 5.25 and 5.26) for Frame-3 reach to 0.0027 and 0.0021 radians as a maximum for those two records respectively and it is more than 1.5 times of the hinge rotation for Frame-1. The maximum total hinge rotation of Frame-3 at the base for Newhall record (Fig.5.27) is 0.0403 radians for accidental plus actual torsion case.

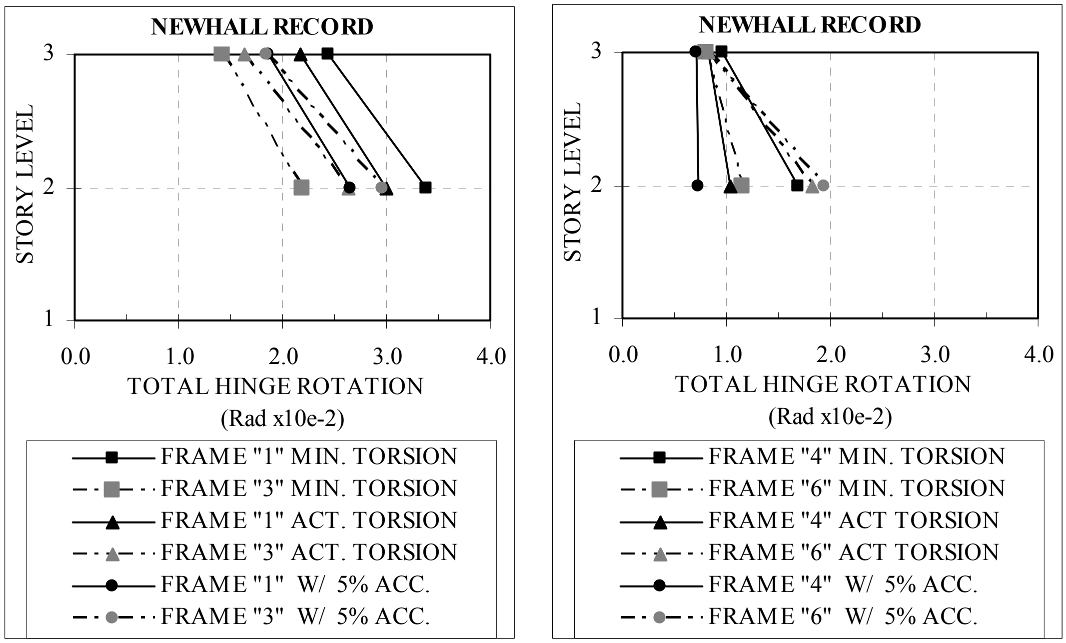


Figure 5.24 –Envelope of Maximum Hinge Rotation of Beams, NEWHALL Record

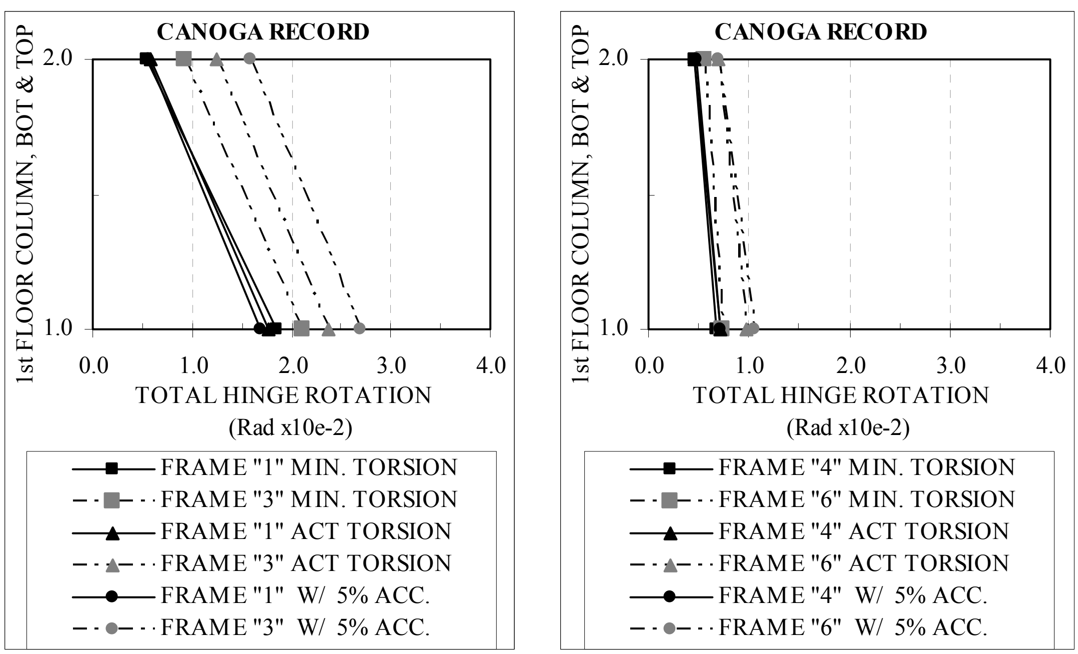


Figure 5.25 –Envelope of Maximum Hinge Rotation of Columns, CANOGA Record

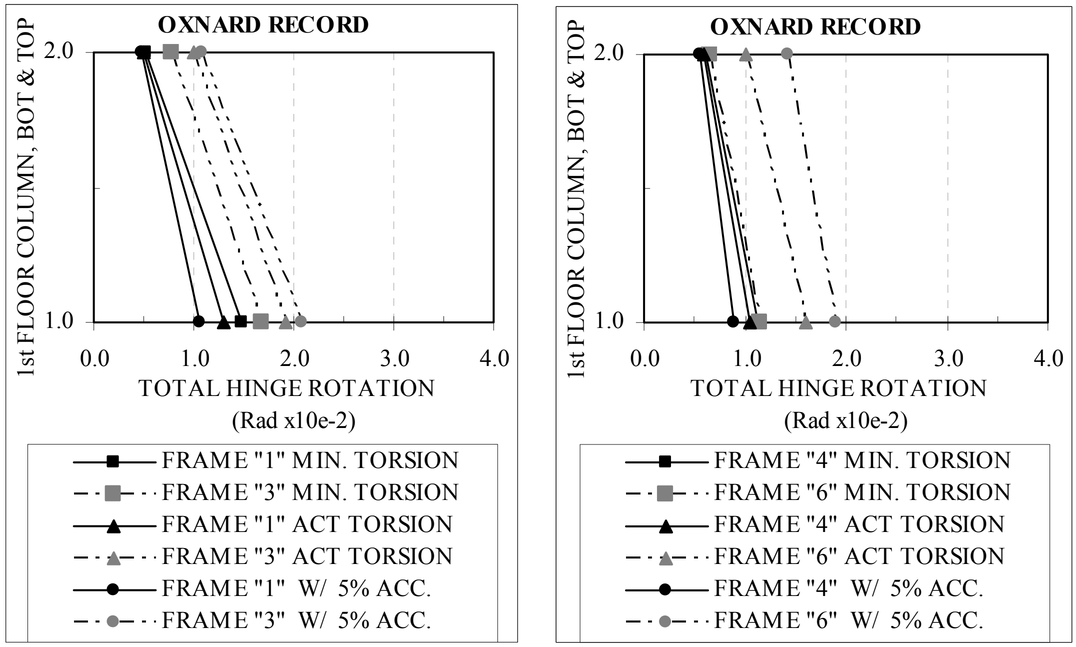


Figure 5.26 –Envelope of Maximum Hinge Rotation of Columns, OXNARD Record

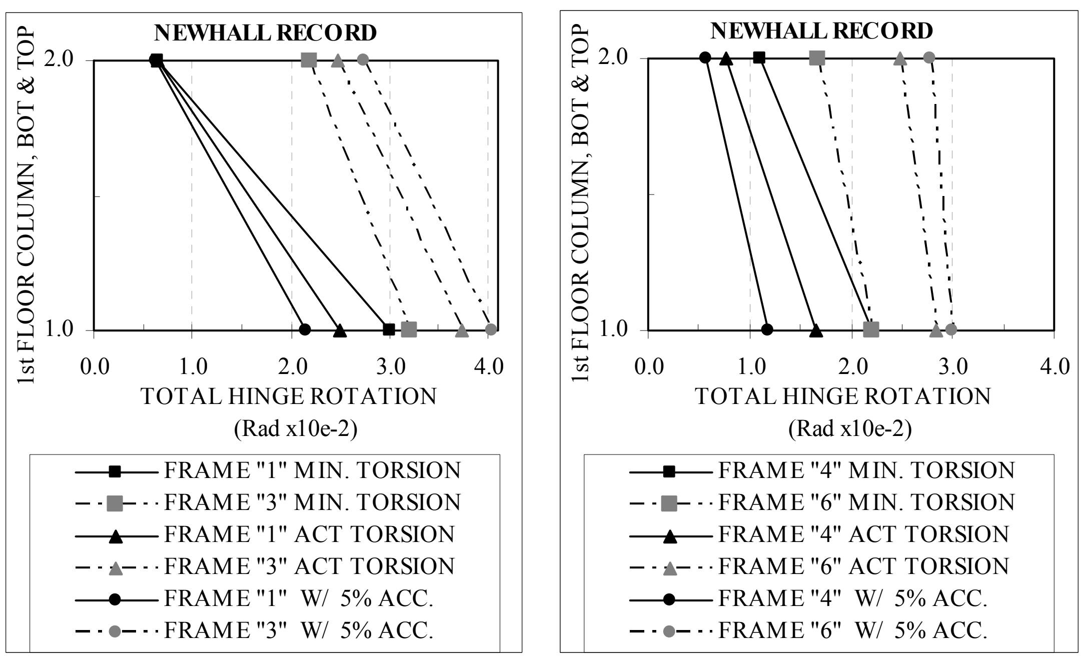


Figure 5.27 –Envelope of Maximum Hinge Rotation of Columns, NEWHALL Record

Figures 5.28 and 5.29 are representing the hysteresis curve for two beams at second floor and roof and a column under the Newhall record. The plastic hinge location and maximum rotation are shown in Figure 5.30.

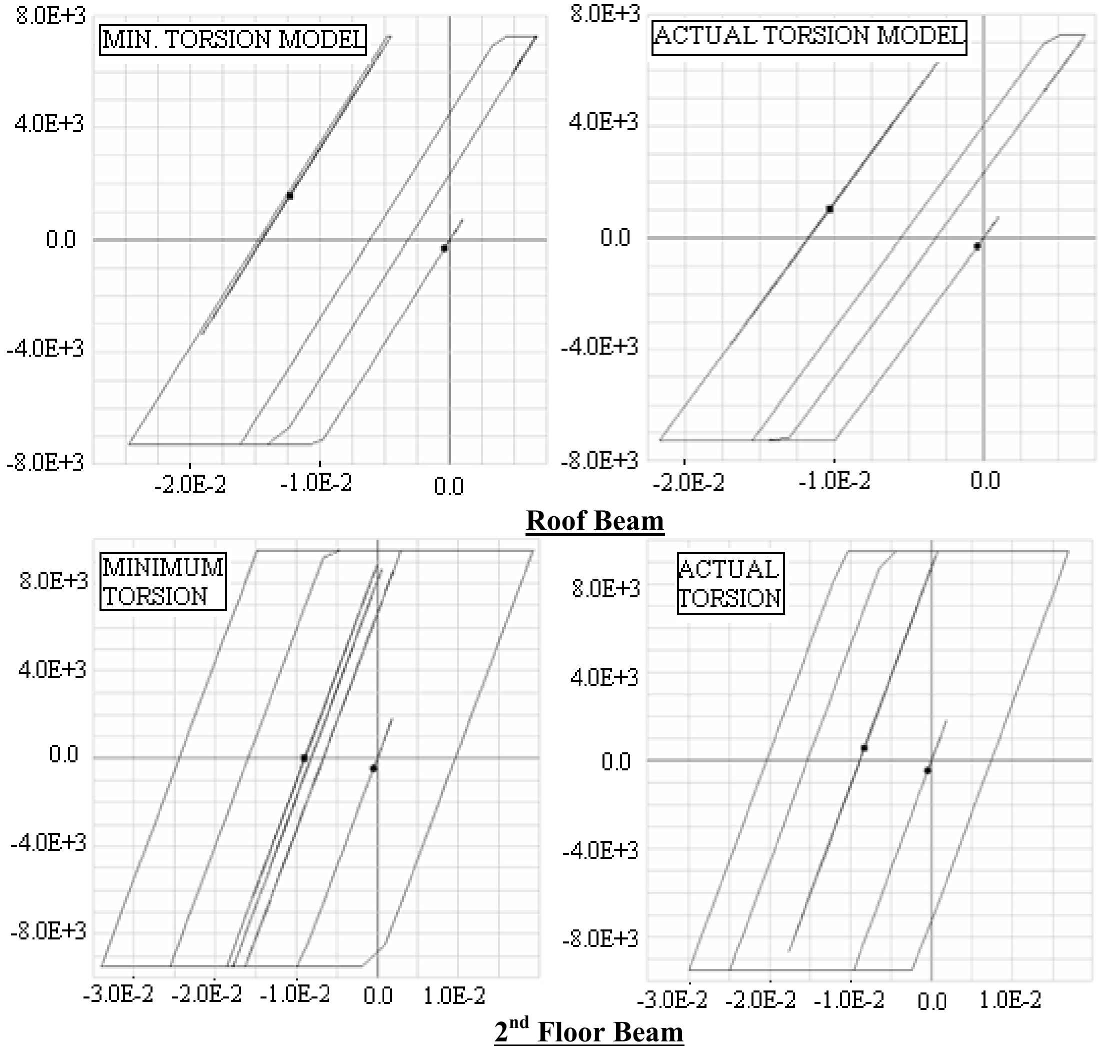
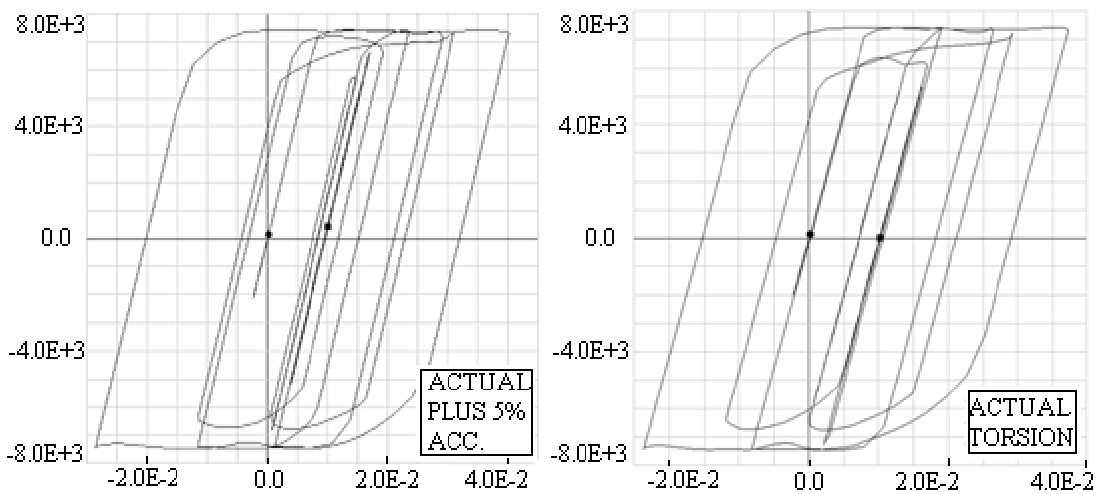


Figure 5.28 –Moment versus Hinge Rotation of Beams, Frame-1, Newhall Record

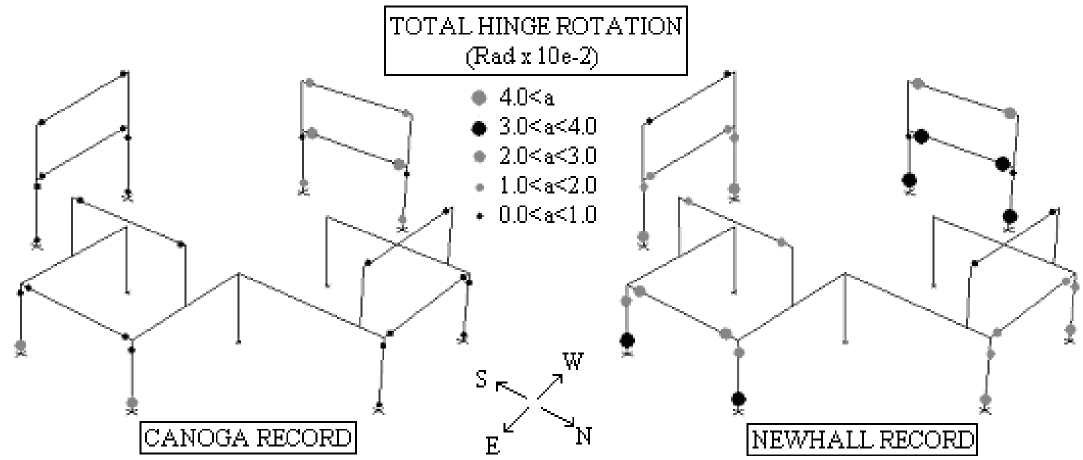
Moment versus hinge rotation hysteresis curves of the most critical cases for beams and columns are shown in Figures 5.28 and 5.29 for Newhall record and compared to actual model case. For beams (Figure 5.28), Frame-1 at the west elevation has the maximum hinge rotation for 2nd floor and roof for minimum torsion model. The comparison between the minimum and actual torsion model shows the effect of torsion on plastic hinge rotation. For columns (Figure 5.29), Frame-3 at east elevation reaches to the maximum hinge rotation for 1st floor column at base for actual plus 5% accidental torsion model. The amount of the maximum hinge rotation for actual torsion model reaches to about 94% of the actual plus 5% accidental torsion model.



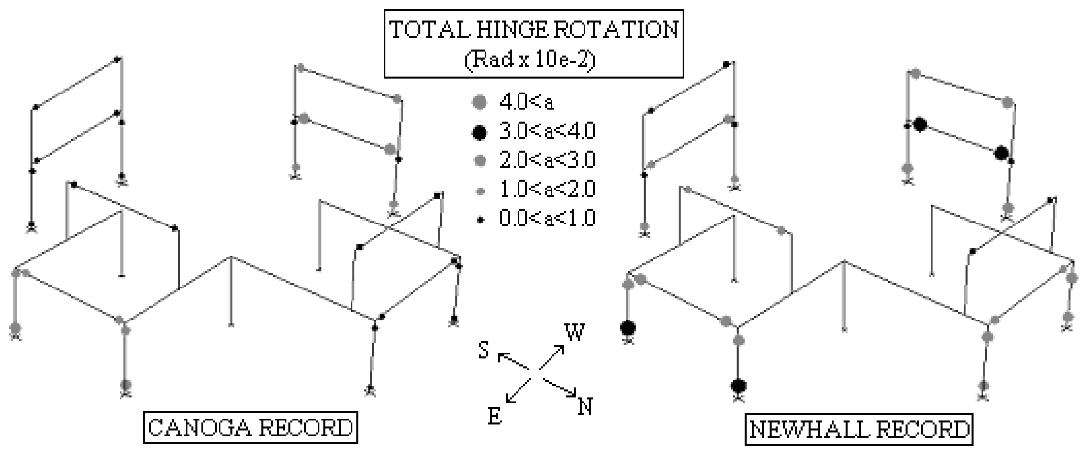
1st Floor Column

Figure 5.29 –Moment versus Hinge Rotation of Column, Frame-3, Newhall Record

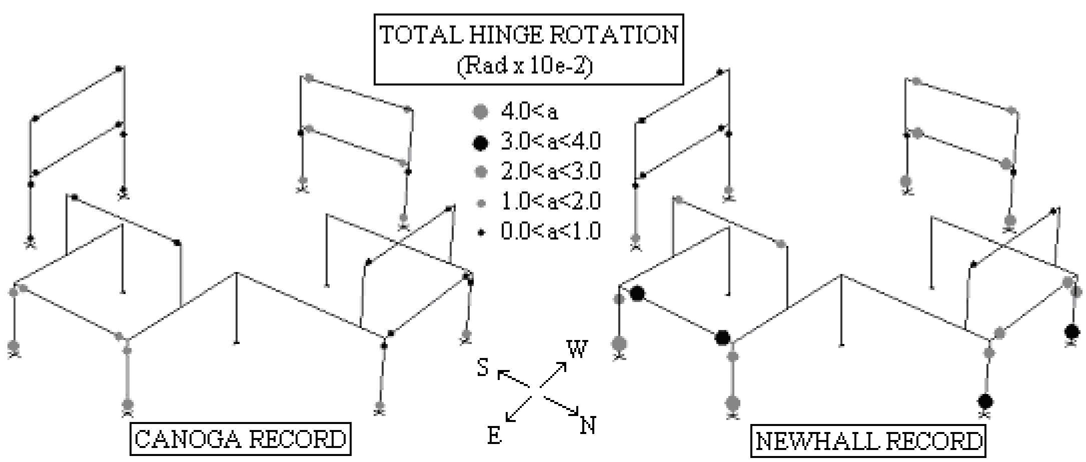
Figure 5.30 is dealing with the plastic hinge mechanism of the building for Canoga and Newhall records for all models and shows the effect of torsion on plastic hinge rotation. Comparison of different mechanisms especially for Newhall record shows that by adding torsion to the model, there is not significant change to the plastic hinge mechanism of 2nd floor to roof columns and roof beams. However, adding torsion to the system cause an increase to total hinge rotation of 1st floor columns at base and 2nd floor beams on North and east elevation and a decrease of total hinge rotation of beam and columns on west elevation.



Minimum Torsion case



Actual Torsion case



Actual Plus 5% Accidental Torsion case

Figure 5.30–Plastic Hinge Mechanism of Frames

As it discussed before, the N-S component of each record has much stronger responses than E-W component and as a result, plastic hinge mechanism on west and east elevations frames are more serious than frames on the other elevations. Plastic hinge mechanism for the frame on east elevation shows that an increase in the torsion will cause an increase in the plastic hinge rotation.

Figures 5.31 through 5.34 show the time-history displacement of different frame elevations for Newhall record to investigate the residual displacement. Residual displacement of frames occurs after forming plastic hinge mechanism.

For all models, residual displacements for frames at east and west elevations are much higher than those on south and north elevations. For Frame-1 on west elevation, residual displacements start from 1.2 and 2.5 inches at 2^{nd} floor and roof for minimum torsion model and decrease to 1.0 and 2.0 inches for two other models. For Frame-3 on east elevation, residual displacements increase from 1.7 and 2.2 inches for minimum torsion model and reach to 2.0 and 2.5 inches at 2^{nd} floor and roof for actual plus 5% accidental torsion model as a maximum residual displacement of the structure. For two other frames on North and south elevations, residual displacements are low and for Frame-6 reaches to 1.0 inch as a maximum.

The real residual displacement measured after earthquake for 2^{nd} floor was more than three inches to the south and the west [1] but the result of different analyses shows the maximum of 2.0 inches for those cases with torsion which is about 66 percent of the measured displacement. The failure of some welds instead of plastic hinge mechanism could be a good reason for this difference.

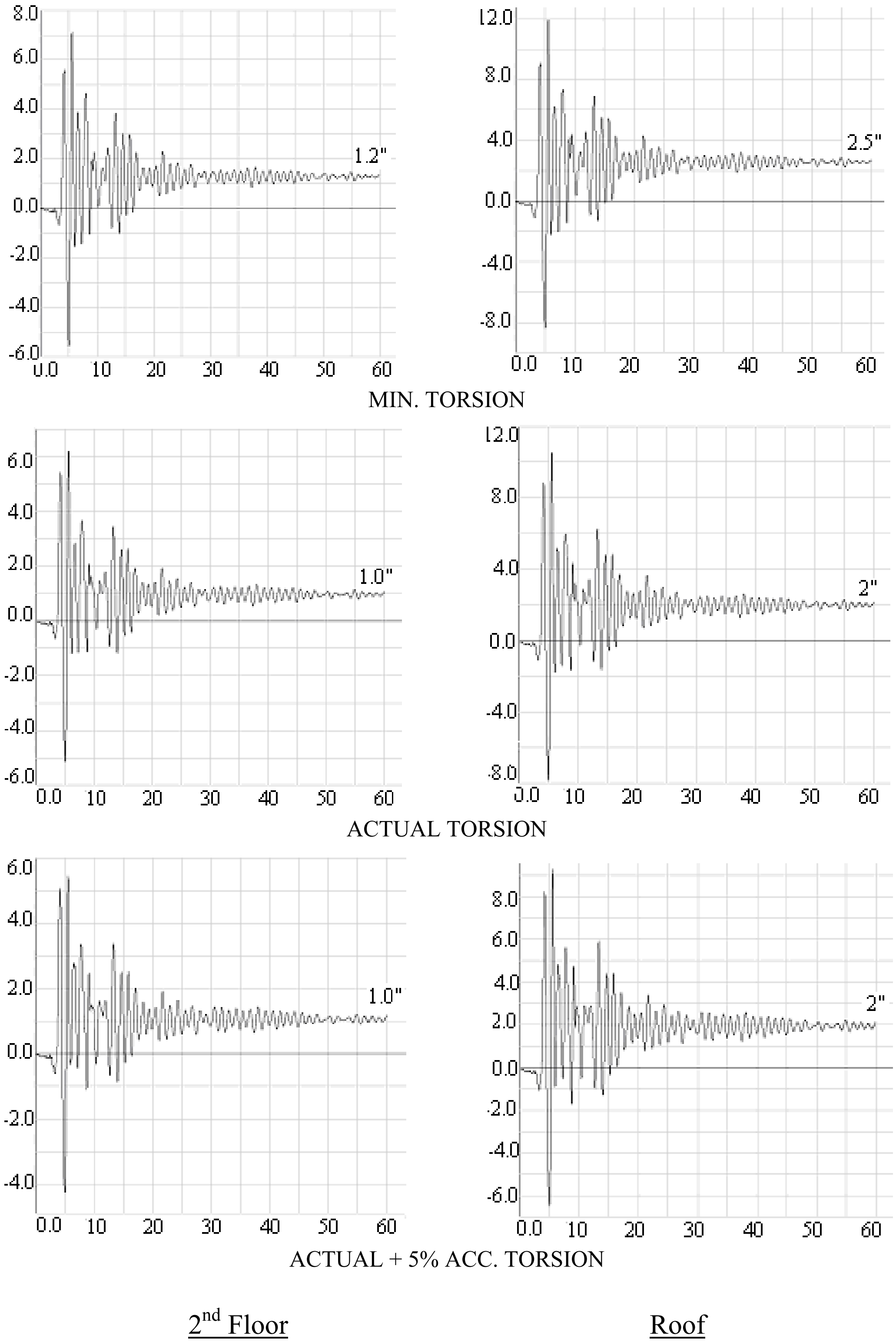
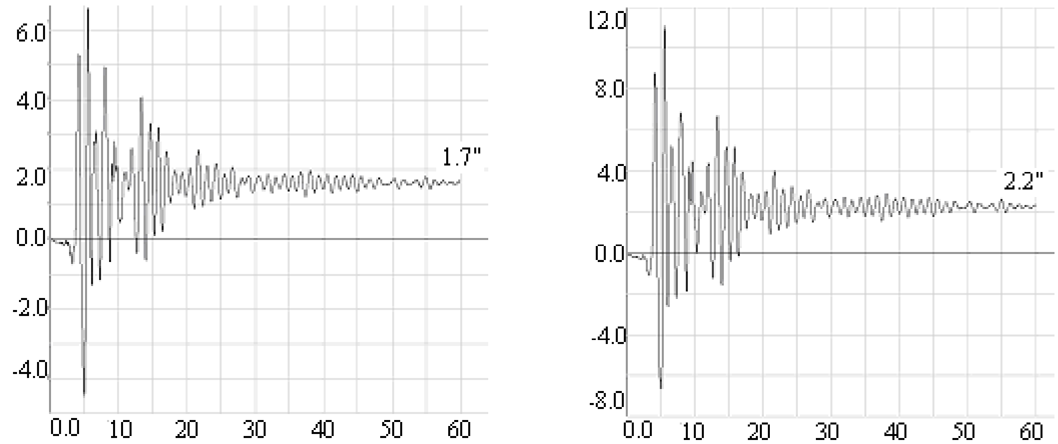
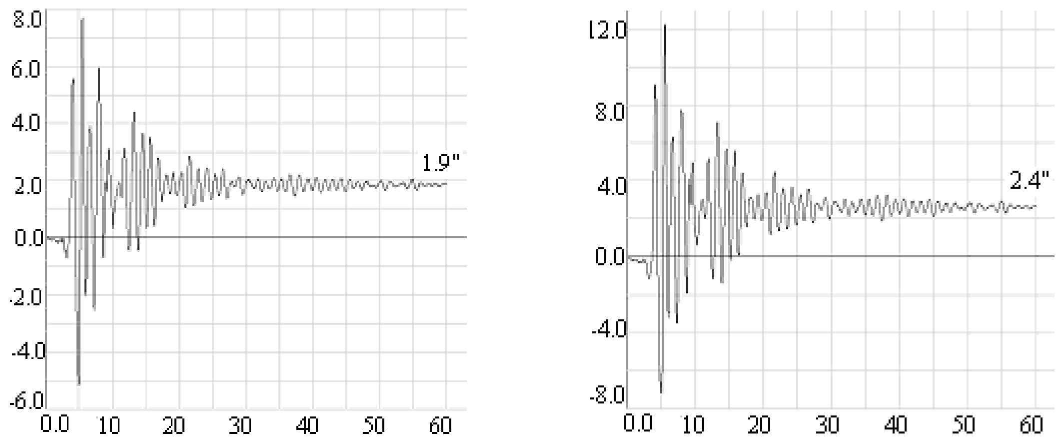


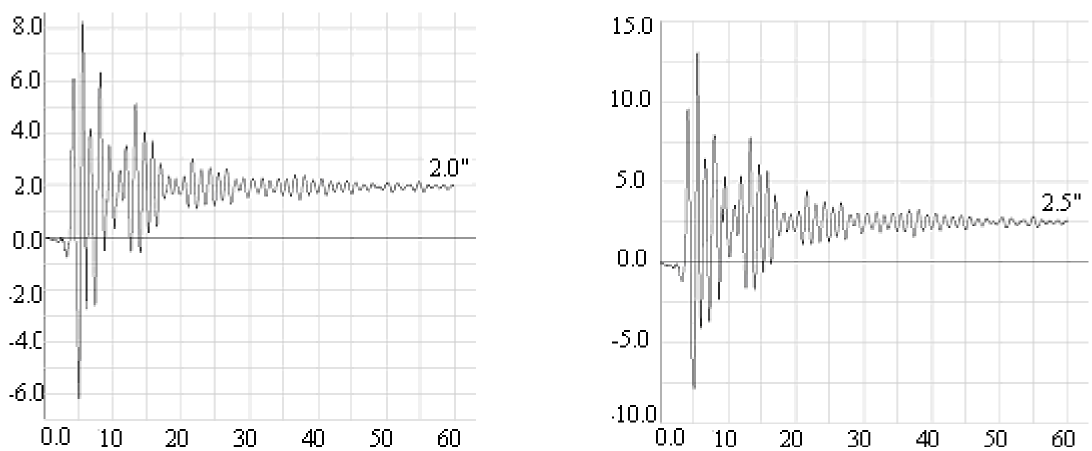
Figure 5.31 – Displacement Time-history of 2nd Floor & Roof for Inelastic Models, West Elevation Frame-1, NEWHALL Record



MIN. TORSION



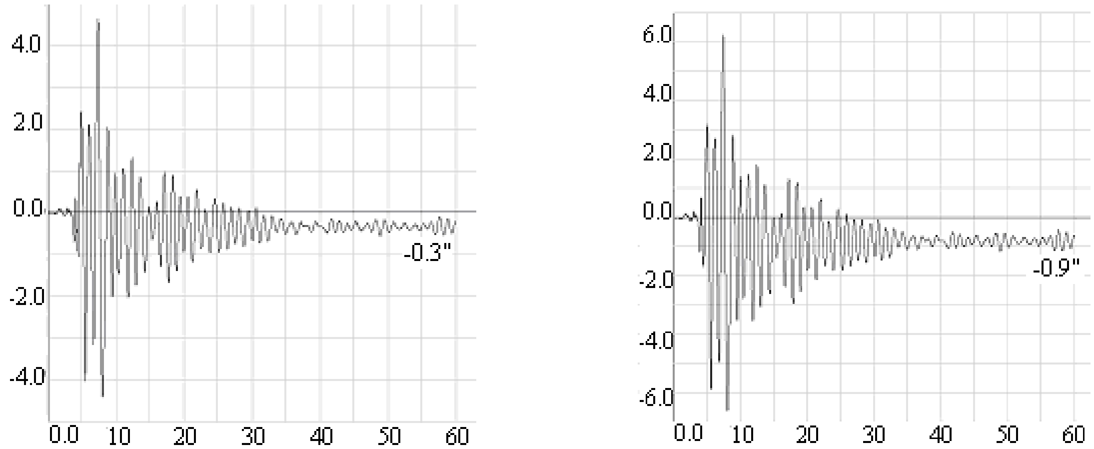
ACTUAL TORSION



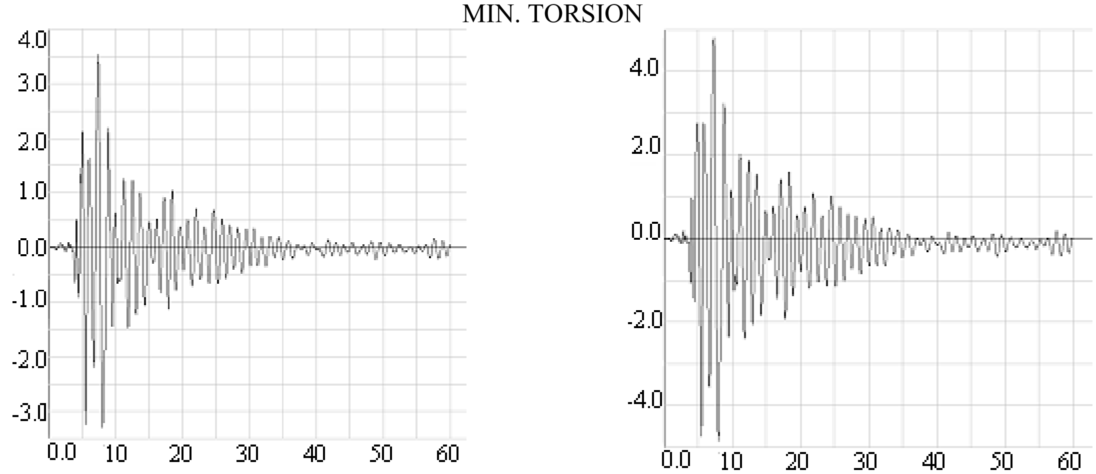
ACTUAL + 5% ACC. TORSION

2nd Floor   Roof

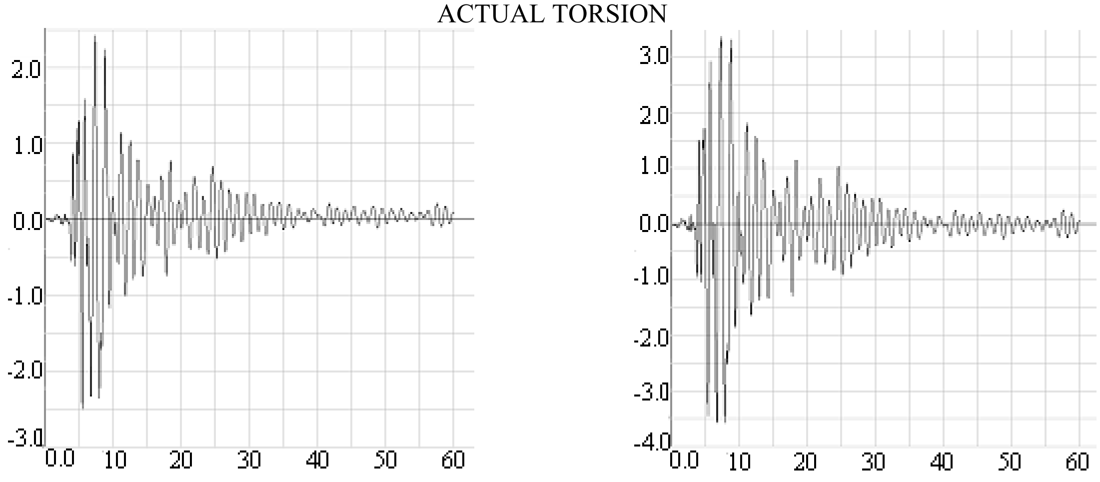
Figure 5.32 – Displacement Time-history of 2nd Floor & Roof for Inelastic Models, East Elevation Frame-3, NEWHALL Record



MIN. TORSION



ACTUAL TORSION



ACTUAL + 5% ACC. TORSION

2nd Floor   Roof

Figure 5.33 – Displacement Time-history of 2nd Floor & Roof for Inelastic Models, South Elevation Frame-4, NEWHALL Record

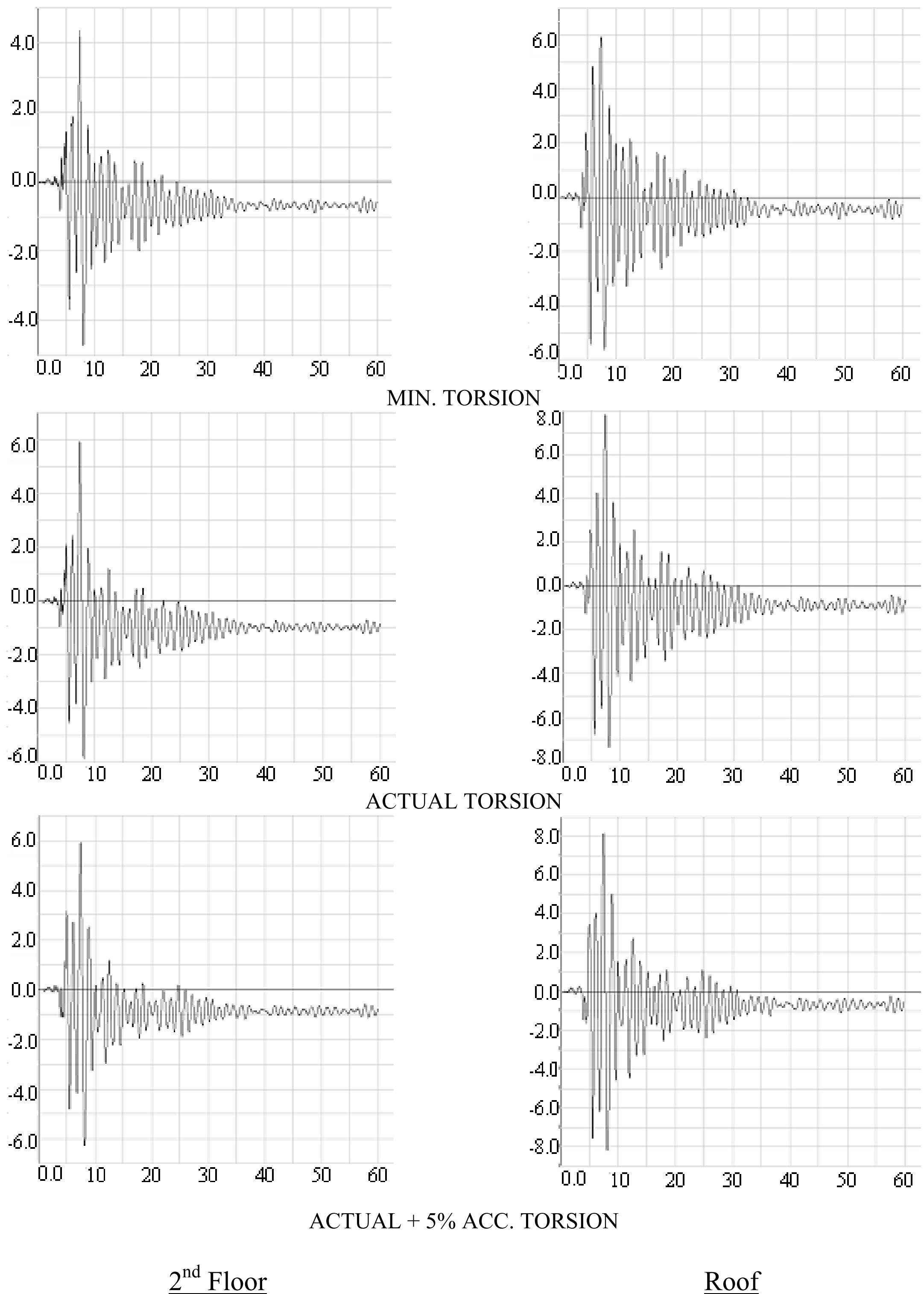


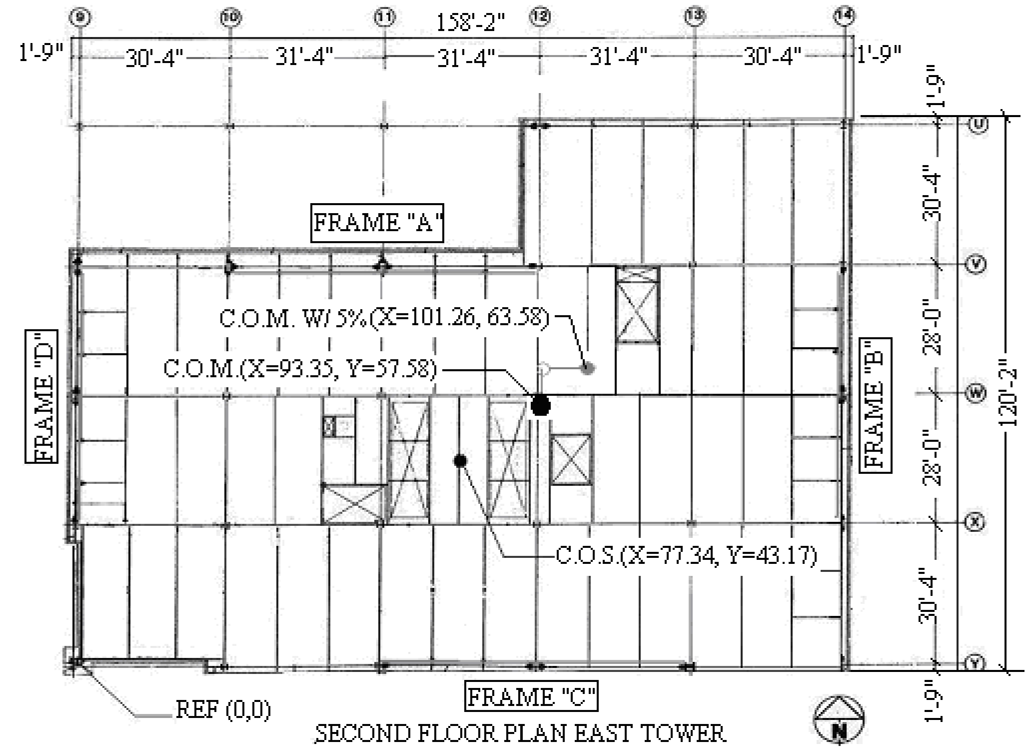
Figure 5.34 – Displacement Time-history of 2nd Floor & Roof for Inelastic Models, North Elevation Frame-6, NEWHALL Record

B. Case 2: 18-story Building (CANOGA Bldg.)

The next building, a 18-story steel building which was slightly damaged during the *Northridge earthquake (1994)* and already investigated in reference [8], was chosen and modeled by *PERFORM 3-D* and a comparison between nonlinear response of *PERFORM 3-D* and the actual damages pointed out in reference [8] was made.

a. Specifications of the Building

The building is one of the two eighteen-story steel structure towers (nineteen stories with penthouse) having an overall height of 248'-4" from ground level to the penthouse roof. It is located in the Woodland Hills region of the San Fernando Valley. The building was designed in 1984 for the lateral force requirements of the 1982 Uniform Building Code. The plan for a typical floor in east tower is shown in Figure 5.35. The dimensions of the rectangular plan are 120'-2" in the N-S direction and 158'-2" in E-W direction. Lateral resistance is provided by four, two-bay moment resistant frames. Three of these, frames B, C and D, are located on the perimeter of the structure. However, frame A is located one bay, 30'-4", inside the north face of the building. This creates an about 25% eccentricity of mass and stiffness for the motion in E-W direction which could have a significant effect on the dynamic response of the building. Elevations of the moment frames with member sizes are shown in Figure 5.36. In this study, the base of the structure is assumed to be fixed at the ground level. Typical floor construction is 3 1/2" light weight concrete fill over a 20 gage metal deck.



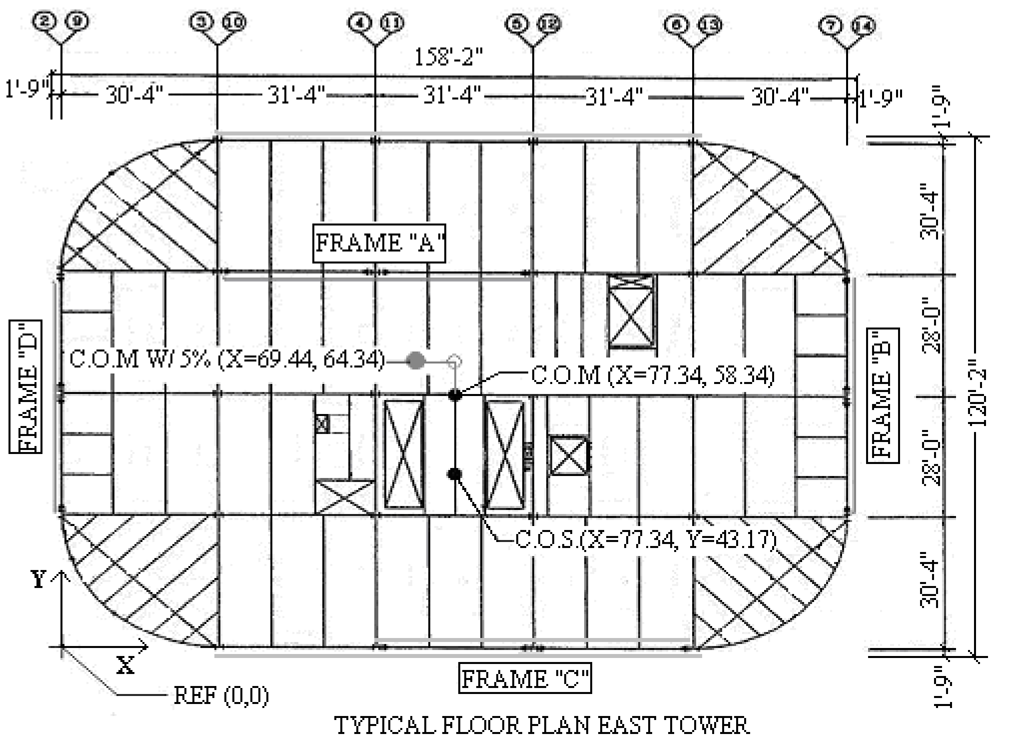


Figure 5.35 - Typical Plan of CANOGA Building

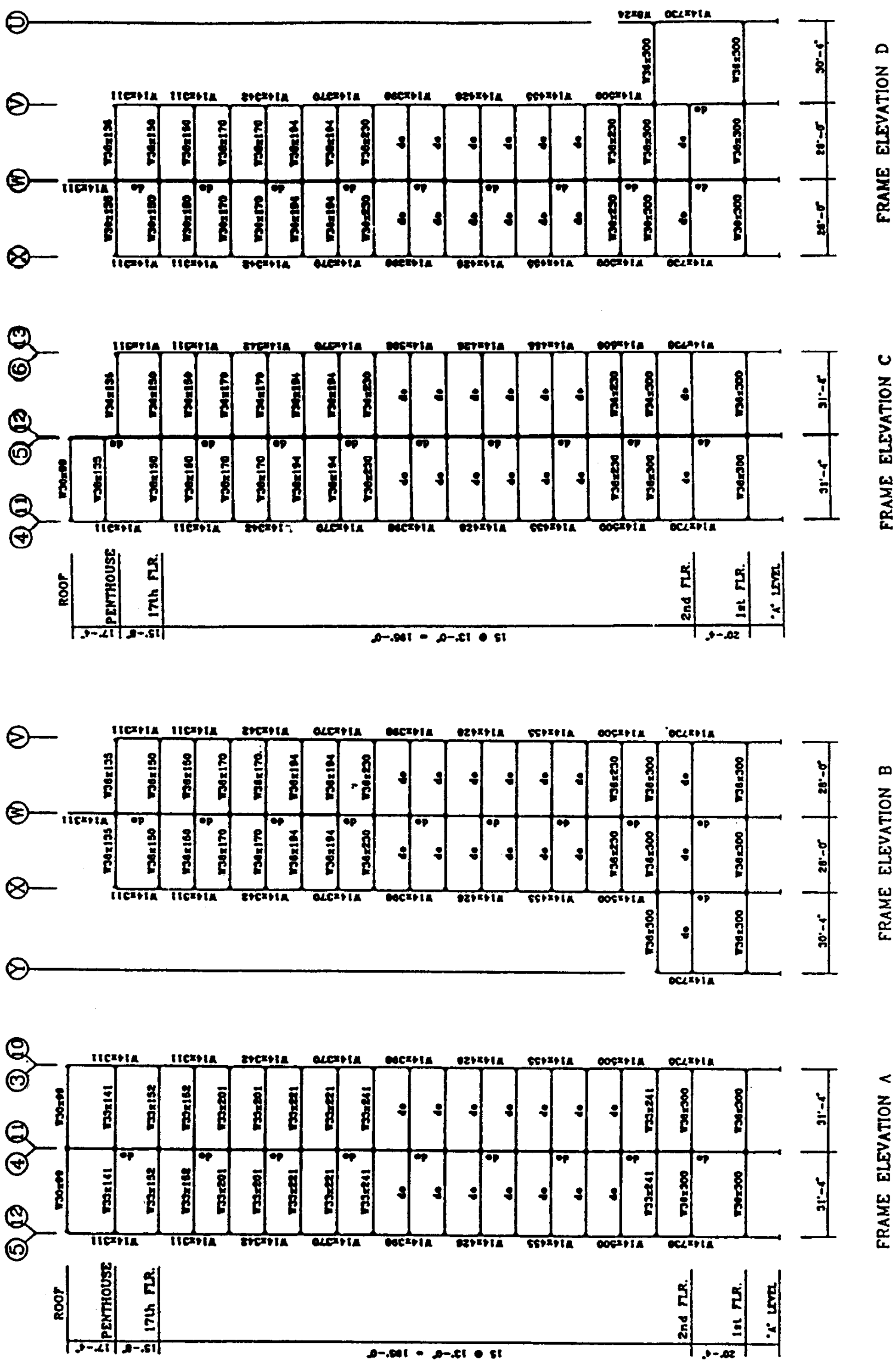


Figure 5.36 - Elevations of Moment Frames of CANOGA Building

A typical welded moment connection in this building consists of full penetration weld between the beam and column flanges and bolted shear tab on the web of beam welded to the column flange which is a typical connection for that period of time.

b. Description of the Model and Analysis

Initial modeling and inelastic analysis of the east tower of this building were conducted using the *PERFORM 3-D* computer program. At the first step, a model was made based on all characteristics of the building. The types of steel are A36 (Fy=36 ksi) for beams and A572-50 (Fy=50 ksi) for columns. Different kinds of W-Sections were used for columns and beams (Fig.5.36). The mass of each floor used in the dynamic analyses are shown in Table-3.

Dynamic analyses were conducted with time-histories recorded during the Northridge earthquake. The two nearest records to the site are those obtained at Canoga and Oxnard, therefore these records were selected and used as the primary records in the analyses. Another record from the Northridge earthquake utilized for this study is the Newhall record. A 3-D elevation of this model was shown in Figure 5.37.

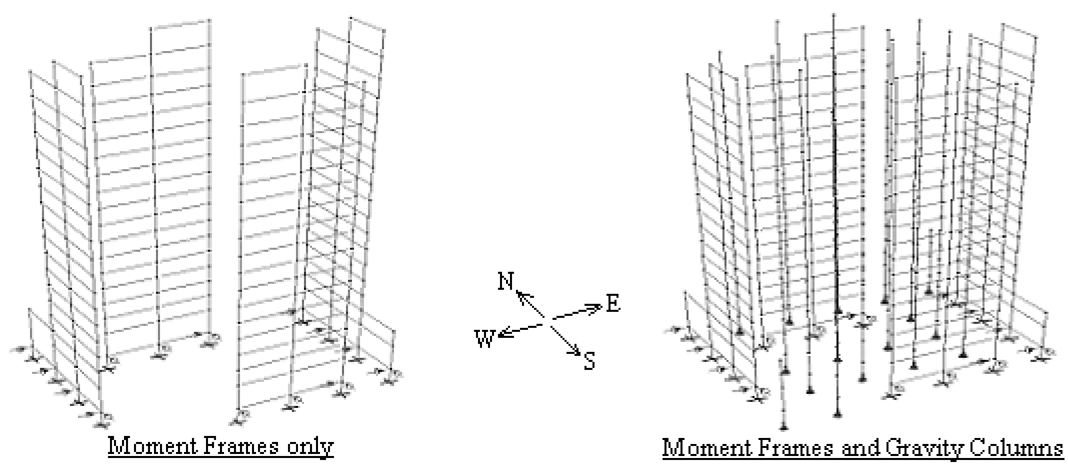


Figure 5.37 - 3-D model Elevation of CANOGA Building

STORY	Δh (in)	MASS (kips-S^2/ft)	I_m (kips-S^2-ft)
ROOF	208.00	28.28	56619
PENTHOUSE	188.00	58.53	183049
17th	156.00	46.56	145614
16th	156.00	46.39	145082
14^{th}-15^{th}	156.00	46.67	145958
13th	156.00	46.79	146333
11^{th} -12^{th}	156.00	47.01	147021
9^{th} -10^{th}	156.00	47.07	147209
7^{th} -8^{th}	156.00	47.29	147897
6th	156.00	47.40	148241
5th	156.00	48.93	153025
4th	156.00	49.58	155058
3rd	156.00	49.64	155246
2nd	244.00	45.14	135818

STORY	WEIGHT	AREA	Ai Xi	Ai Yi	CENTER OF MASS (ft)		CENTER OF STIFFNESS (ft)	
No.	kips	Ft^2			X = Ai Xi / A	Y = Ai Yi / A	X	Y
3^{RD}- Roof	VAR.	RECT.	-	-	158.17/2-1.75=77.34	120.17/2-1.75=58.34	77.34	43.17
2^{ND} Floor	1453.50	14529	1381708	862005	95.1-1.75=93.35	59.33-1.75=57.58	77.34	43.17

CENTER OF MASS W/ 5% ACC. TORSION

3^{RD} - Roof : X = 77.34 - 0.05 X 158.17 =69.44 ft , Y= 58.34 + 0.05 X 120.17 = 64.34 ft

2^{ND} Floor : X = 93.35 + 0.05 X 158.17 = 101.26 ft , Y= 57.58 + 0.05 X 120.17 = 63.58 ft

TABLE 3 – Mass and Center of Mass for each floor of 18–Story CANOGA Building

To investigate the effect of torsion on the response of this building, five models with different eccentricities considering both elastic and inelastic behavior were utilized and comparisons among their responses were made. For the first model, by moving of the C.O.M of each floor to the C.O.S, minimum torsional response of the building was investigated.

The second model considered the actual torsion of the building for both directions due to the differences between C.O.M and C.O.S. locations at each floor. For the third and fourth models, 5% accidental torsion based on the uniform building code requirements at only N-S (Y-Y or Y-DIR on graphs) and E-W (X-X or X-DIR on graphs) directions was added respectively to the actual torsion of the building to magnify the effect of torsion. For the last model, 5% accidental torsion was added to the actual torsion in both directions (X-Y on graphs). See Table-3 for information about the center of mass and stiffness of each floor.

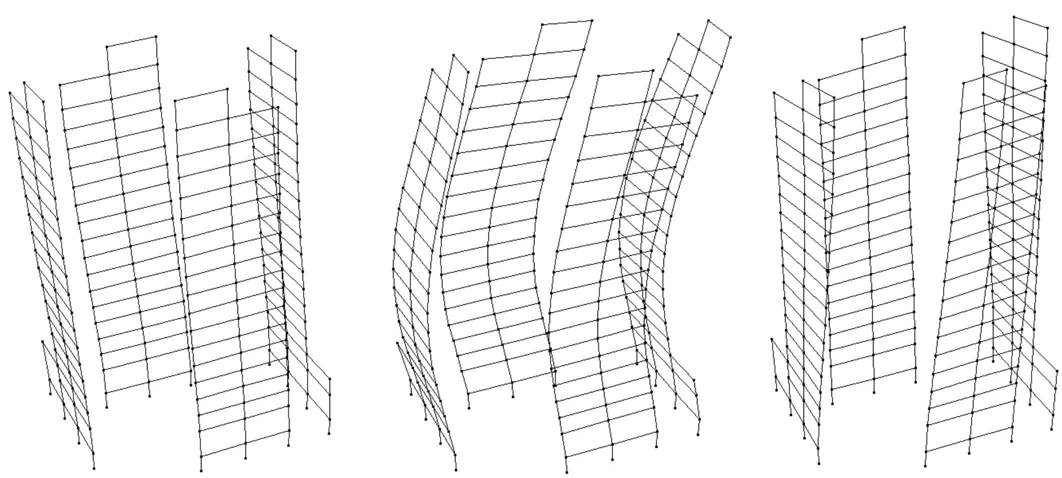
In utilization of these five models, the effect of +/- acceleration for each pair of N-S and E-W records were considered and based on the results, the pair producing the maximum torsion was chosen. Note that Frame "B" and Frame "D" are in the N-S direction and Frame "A" and Frame "C" are in E-W direction.

c. Discussion of the Results

The nonlinear dynamic response parameters of Canoga building to three different records from Northridge earthquake are shown in figures 5.38 through 5.61. Modal characteristics of the building for first eight modes have been summarized and shown in Table 4 and Figure 5.38. The periods per second and modal participation factors of the first five modes of vibration based on the initial stiffness of members are (4.03, 78.5%) for first mode at E-W direction, (3.81, 76%) for first mode at N-S direction, (2.29, 3%) for first rotational mode of vibration, (1.42, 12.2%) for second mode at E-W direction and (1.34, 13.3%) for second mode at N-S direction. Modal periods of the model are close enough to the periods of reference [8]. As discussed before, Frame "A" has been located one bay, 30'-4", in from the north face of the building and this creates an eccentricity of mass and stiffness in the E-W direction.

Mode No.	PERIOD (S)	Modal Participation E-W Direction	Modal Participation N-S Direction	Relative Direction
1	4.03	0.7851	0.0003	E-W
2	3.81	0.0003	0.7608	N-S
3	2.29	0.03	0.005	ROT
4	1.42	0.1221	0	E-W
5	1.34	0	0.1331	N-S
6	0.842	0.0408	0	E-W
7	0.8	0.0075	0	ROT
8	0.79	0	0.0462	N-S

TABLE 4 – Period and Mass Contribution of each Mode for 17–Story CANOGA Building



1st Mode of Vibration   2nd Mode of Vibration   1st Rotational Mode

Figure 5.38 – Mode Shapes, CANOGA Building

The responses of the building to two moderate earthquake records, Canoga and Oxnard, are summarized in graphs below (Fig. 5.39):

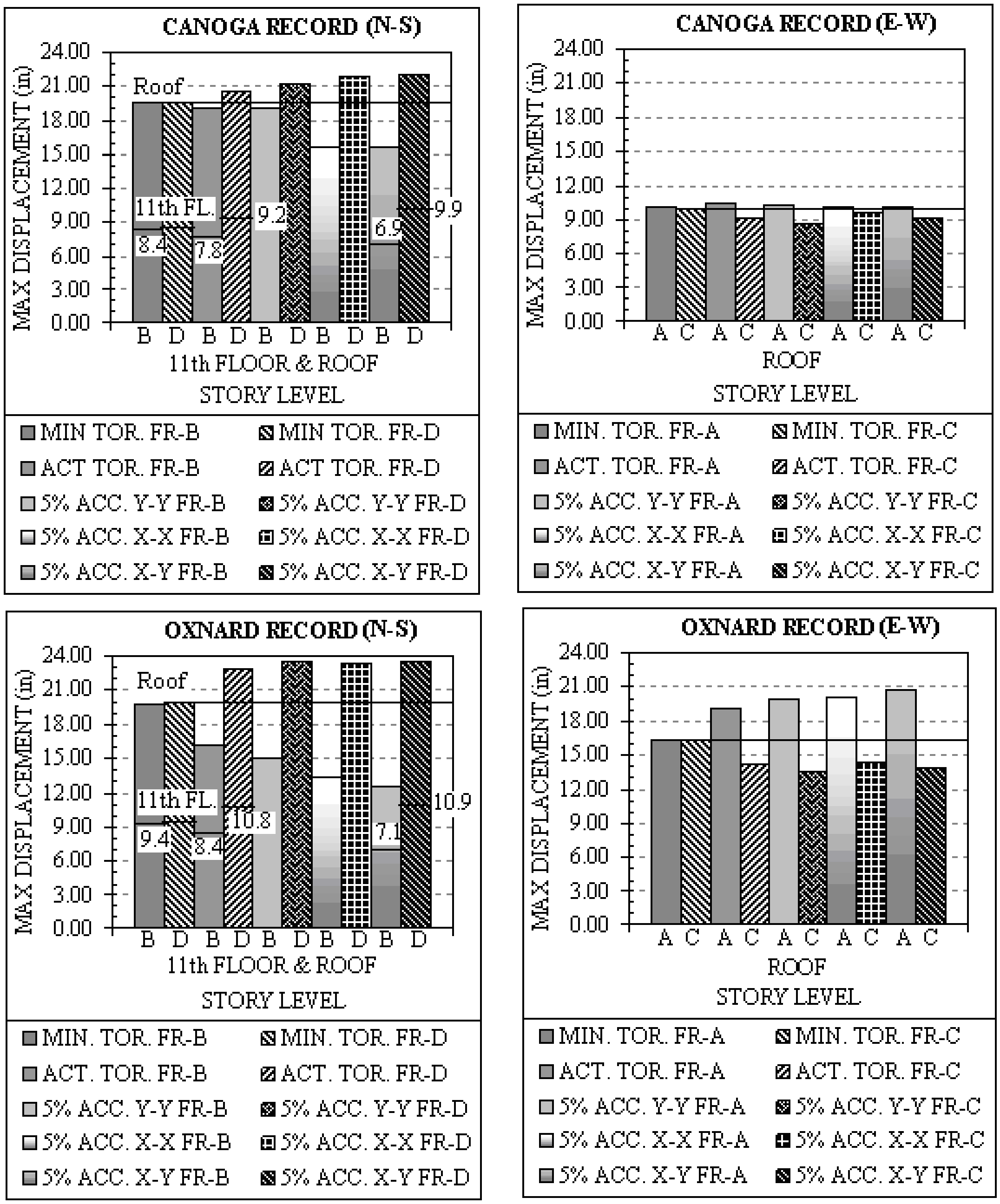


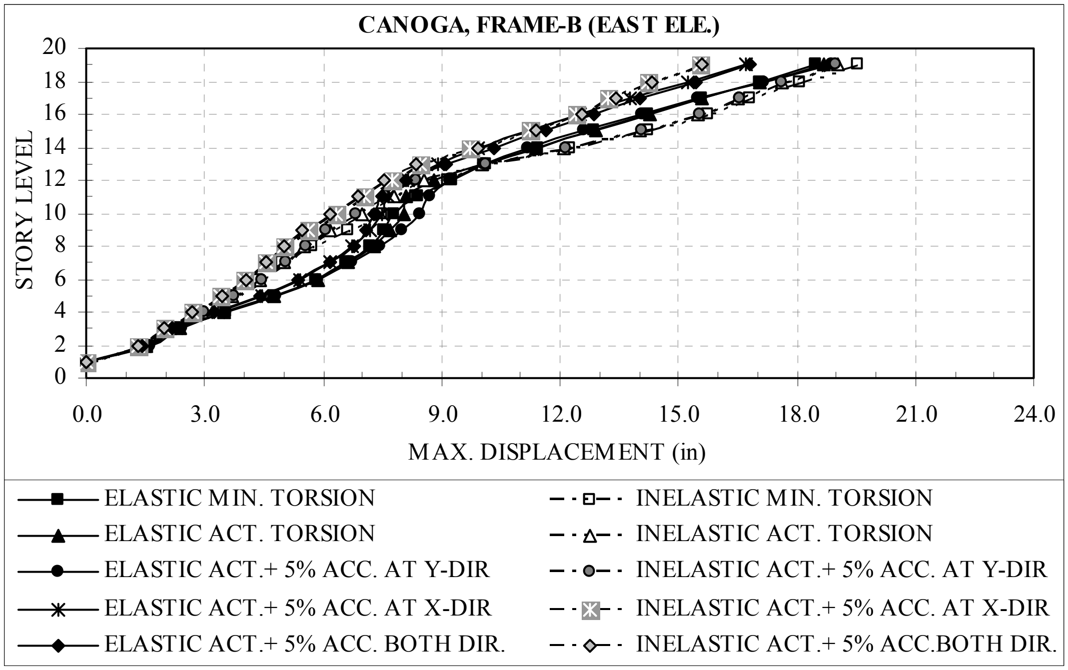
Figure 5.39 – Comparison of the Maximum Displacement of the Building, Inelastic Models, CANOGA and OXNARD Record

The results of N-S and E-W components of each record applied simultaneously shows that N-S component has much stronger responses than E-W component which can be seen on acceleration response spectra of each record as well. By comparing the results of column bar graphs, the effect of actual and actual plus 5% accidental torsion on maximum inelastic displacement can be studied. For Canoga record with larger responses in N-S direction, Frame "D" that suffers the most, the maximum displacement increase due to torsion at roof is approximately (21.99-19.59)/19.59= 2.4/19.59= 12.3%. The average changes between frames "B" and "D" are 4% and 16% for actual torsion case and actual plus 5% accidental torsion for the "both directions" case respectively compared to minimum torsion case. Accidental torsion alone has about average change of 12% on maximum displacement of the roof. These average changes for the 11th floor (mid-height) are 7%, 17% and 10% respectively. For N-S direction of the Oxnard record, the average changes in displacement of the N-S frames at roof level are 16%, 27% and 12% for actual torsion case and actual plus 5% accidental torsion in "both directions" case compared to minimum torsion case and accidental torsion alone respectively which show much higher percentage in compare to Canoga record. These average changes for the 11th floor are 12%, 19% and 8% respectively. Results of these graphs have been summarized in form of a table in chapter eight.

Figures 5.40 through 5.43 show the maximum displacements of all elastic and inelastic models for Canoga and Oxnard records. Starting with the inelastic minimum torsion model, the maximum displacements at roof for Frame “B” at east elevation and Frame “D” at west elevation are the same and equal to *19.6* for Canoga (Fig.5.40 and Fig.5.41) and *19.8* inches for Oxnard (Fig.5.42 and Fig.5.43) records. Frame “A” at north elevation and Frame “C” at south elevation have the same maximum displacement too and equal to *10.0* and *16.3* inches respectively for each of those two records. In inelastic actual torsion model, the maximum displacement at roof for frame “B” and “D” become *19.0* and *20.63* inches for Canoga record and *16.11* and *22.84* inches for Oxnard record and for frame “A” and “C” become *10.38* and *9.09* inches for Canoga record and *19.03* and *14.27* inches for Oxnard record which show a considerable amount of Torsion especially for Oxnard record.

In inelastic model with actual plus 5% accidental torsion only at N-S direction (Y-Y direction), the maximum displacement at roof for frame “B” and “D” become *18.99* and *21.2* inches respectively for Canoga record and *15* and *23.47* inches respectively for Oxnard record which all show large amount of Torsion. For frame “A” and “C” roof displacements become *10.35* and *8.64* inches for Canoga record and *19.85* and *13.57* inches for Oxnard record. Next model which is inelastic model with actual plus 5% accidental torsion only at E-W direction (X-X direction), the maximum displacement at roof for frame “B” and “D” become *15.56* and *21.93* inches respectively for Canoga record and *13.36* and *23.31* inches respectively for Oxnard record. For frame “A” and “C” roof displacements become *10.16* and *9.56* inches for Canoga record and *20.04* and *14.31* inches for Oxnard record.

In inelastic model with actual plus 5% accidental torsion at both directions (X-Y direction), the maximum displacement at roof for frame “B” and “D” become *15.62* and *21.99* inches respectively for Canoga record and *12.51* and *23.47* inches respectively for Oxnard record. For frame “A” and “C” roof displacements become *10.2* and *9.1* inches for Canoga record and *20.77* and *13.84* inches for Oxnard record.



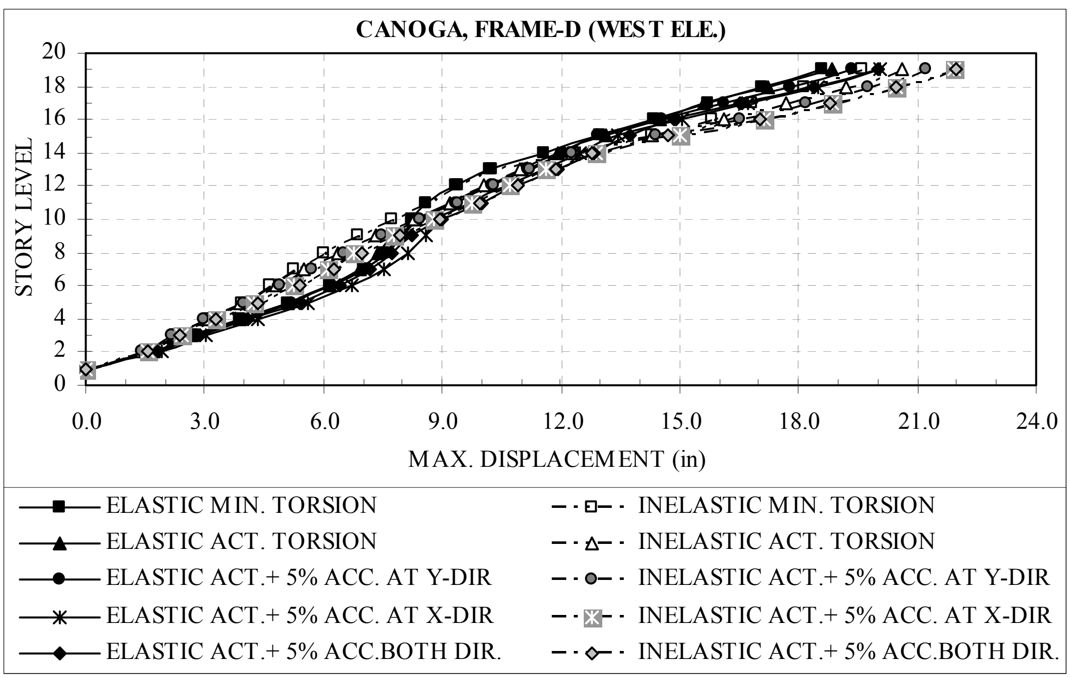
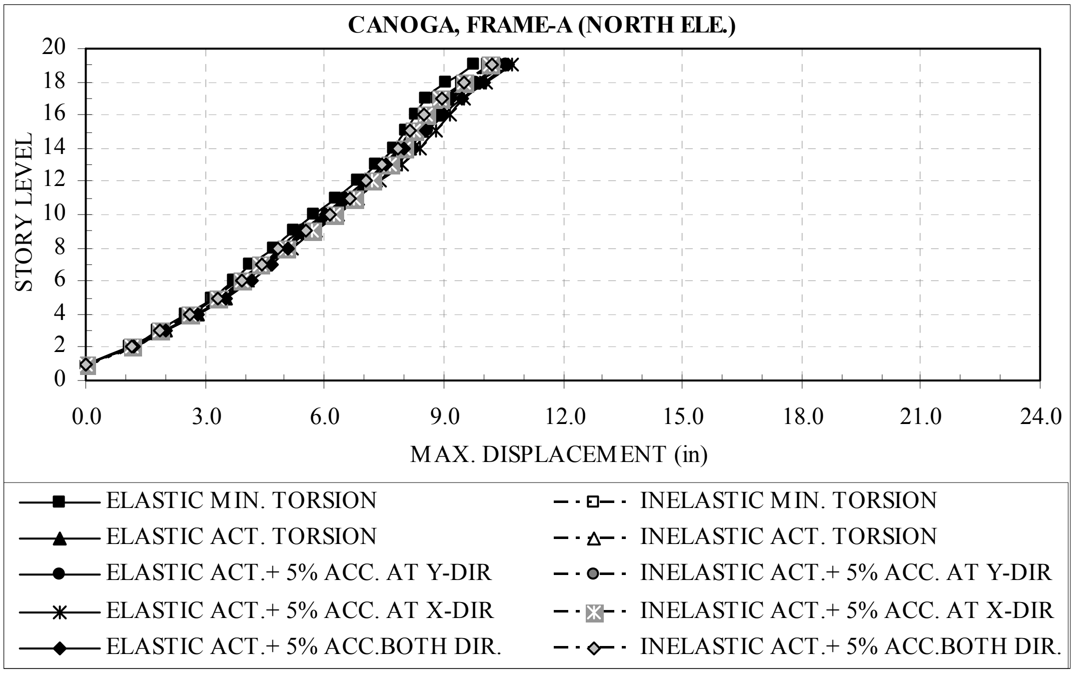


Figure 5.40 -MAX Displacements of Building in N-S Direction, CANOGA Record

Comparison among models with 5% accidental torsion shows that displacements are close to each other at each frame (especially 5% accidental torsion at N-S direction to 5% accidental torsion at both directions) which makes 5% accidental torsion at both directions model as a good envelope for all three of them.



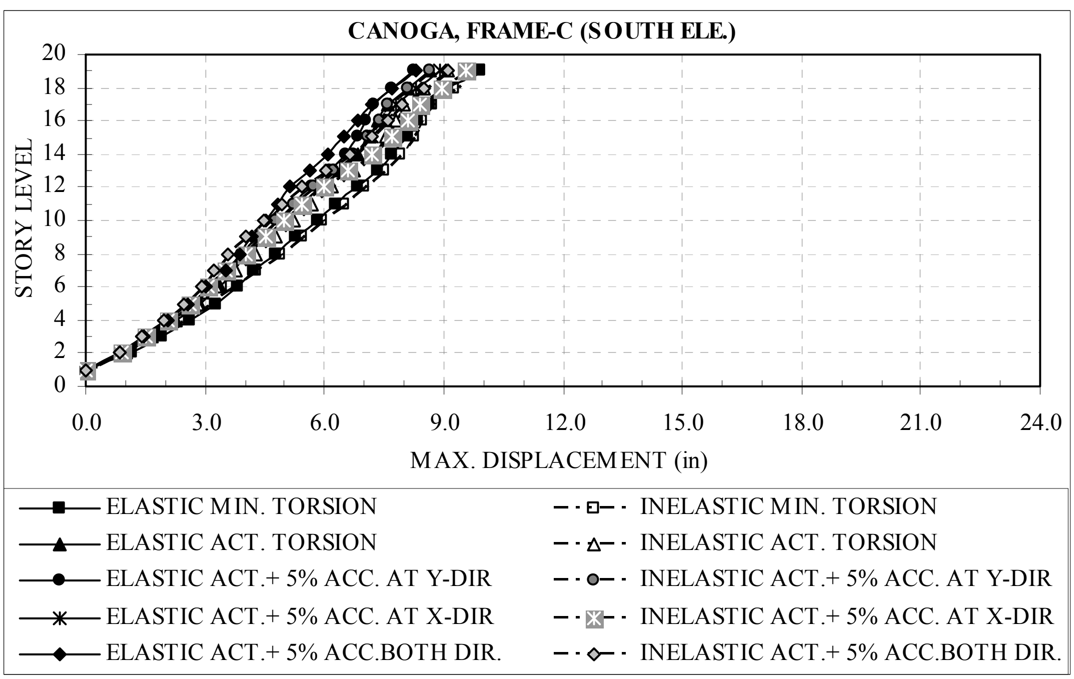
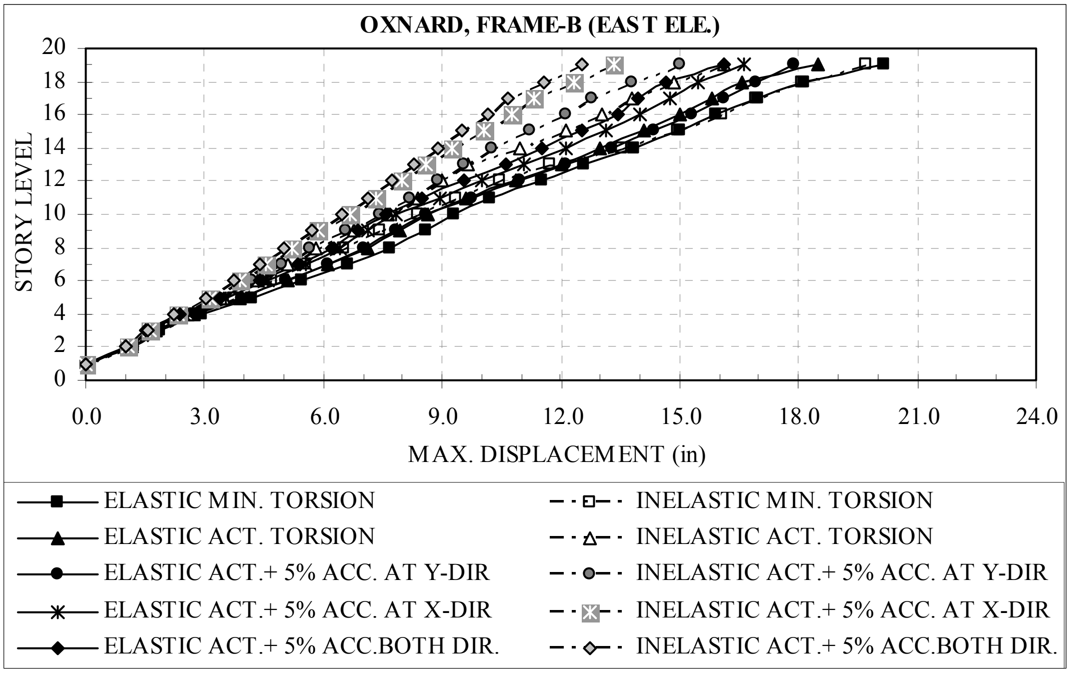


Figure 5.41 -MAX Displacements of Building in E-W Direction, CANOGA Record



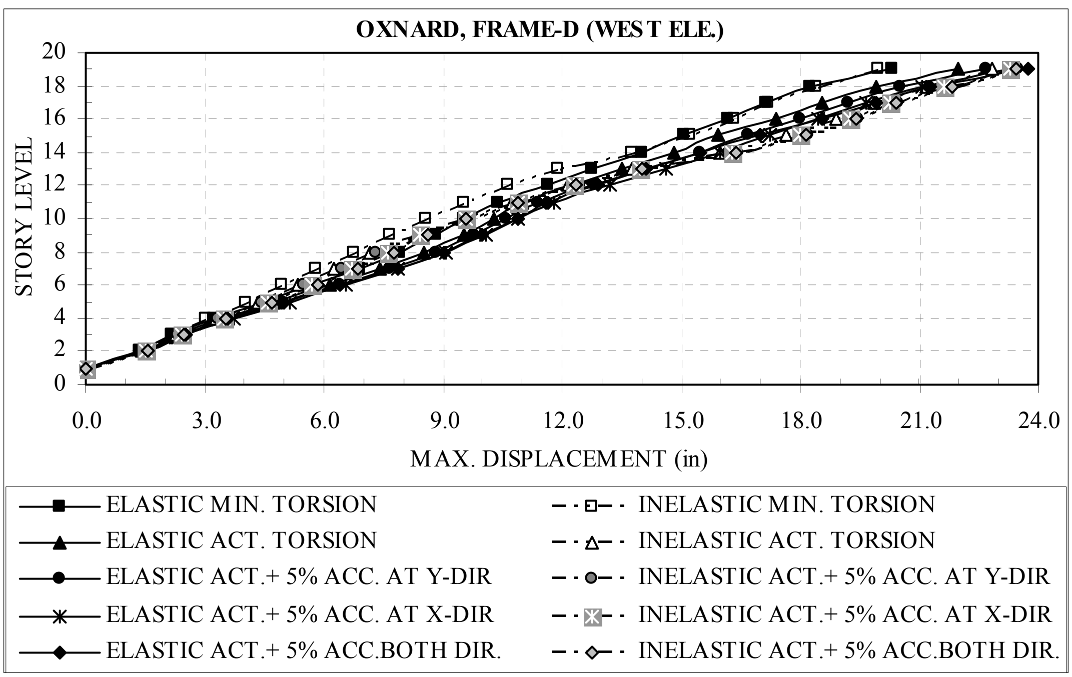
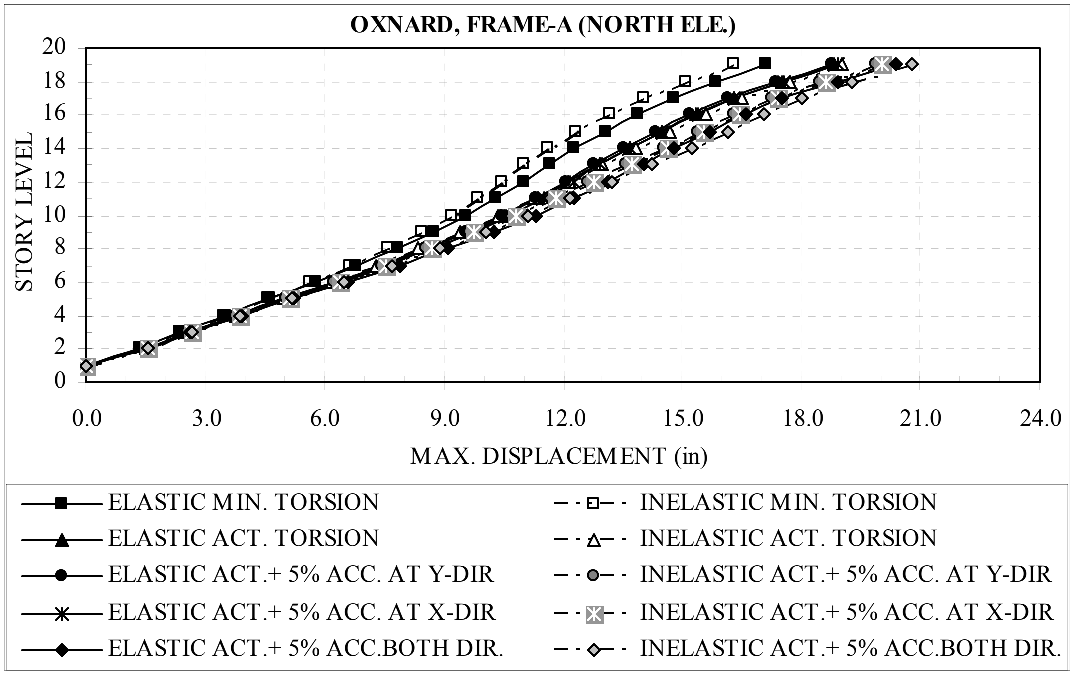


Figure 5.42 -MAX Displacements of Building in N-S Direction, OXNARD Record



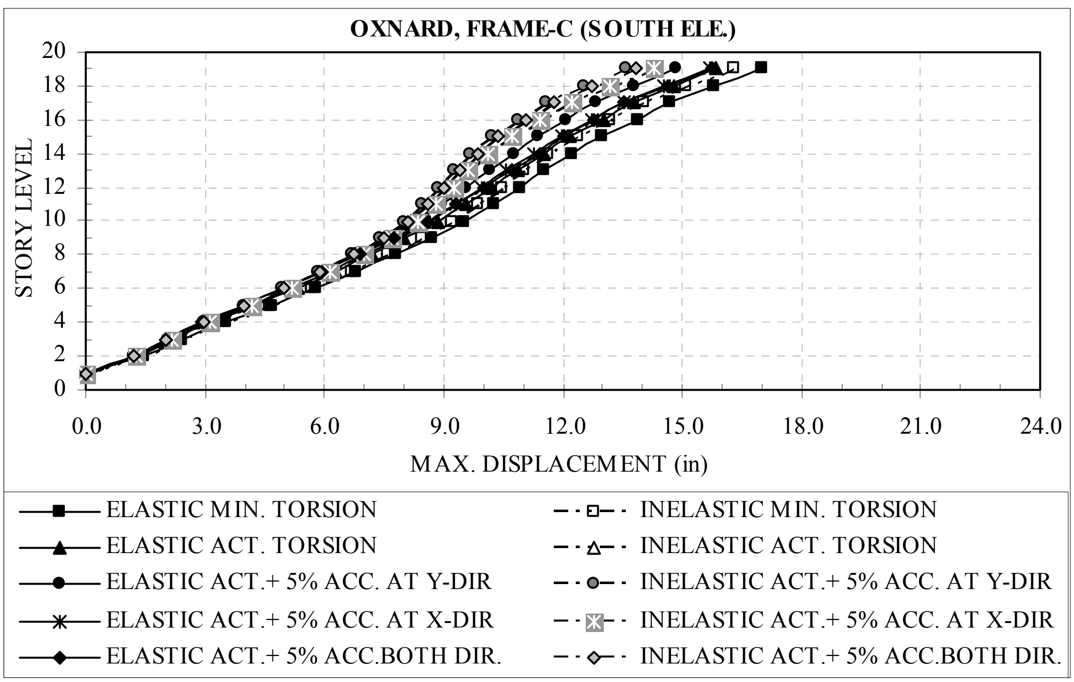


Figure 5.43 -MAX Displacements of Building in E-W Direction, OXNARD Record

Comparison between maximum displacements of elastic and inelastic models shows that torsional response has stronger effects on inelastic models than elastic.

For instance, the minimum torsion model shows 5.3% difference between inelastic and elastic roof displacements of Frame "D" at west elevation for N-S component of Canoga record and that increases to 9.4% and 10% for actual torsion case and actual plus 5% accidental torsion in both directions case respectively.

The response of building to Newhall record with higher peak spectral accelerations in both directions is much higher but still it follows the same general pattern of responses. The results of column bar graphs on Figure 5.44 show the effect of actual and actual plus 5% accidental torsion on maximum inelastic displacement.

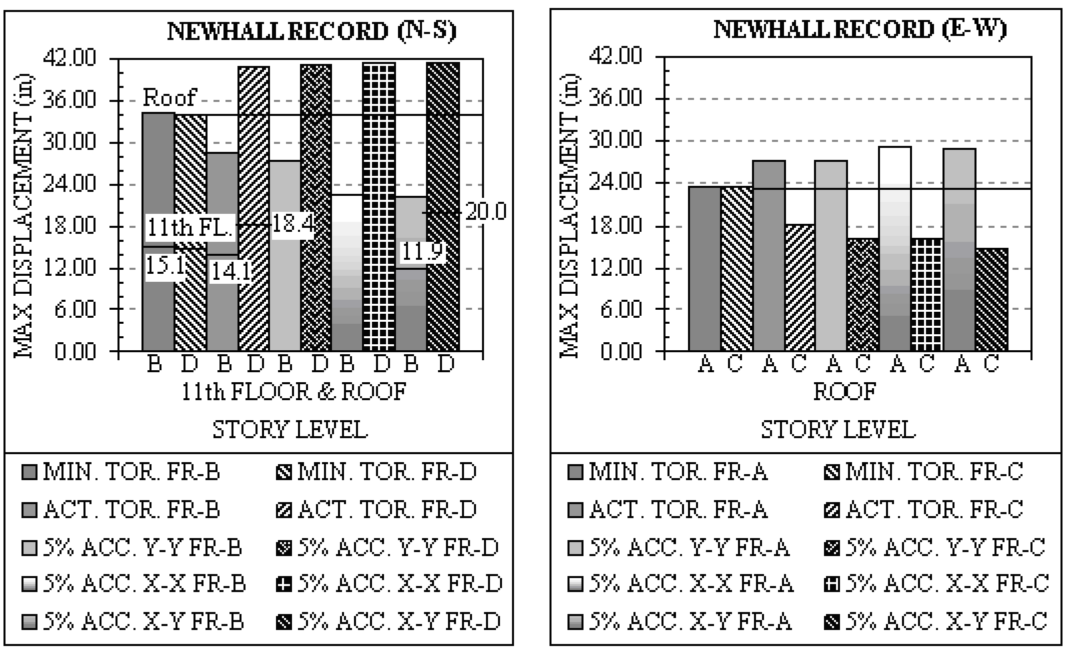
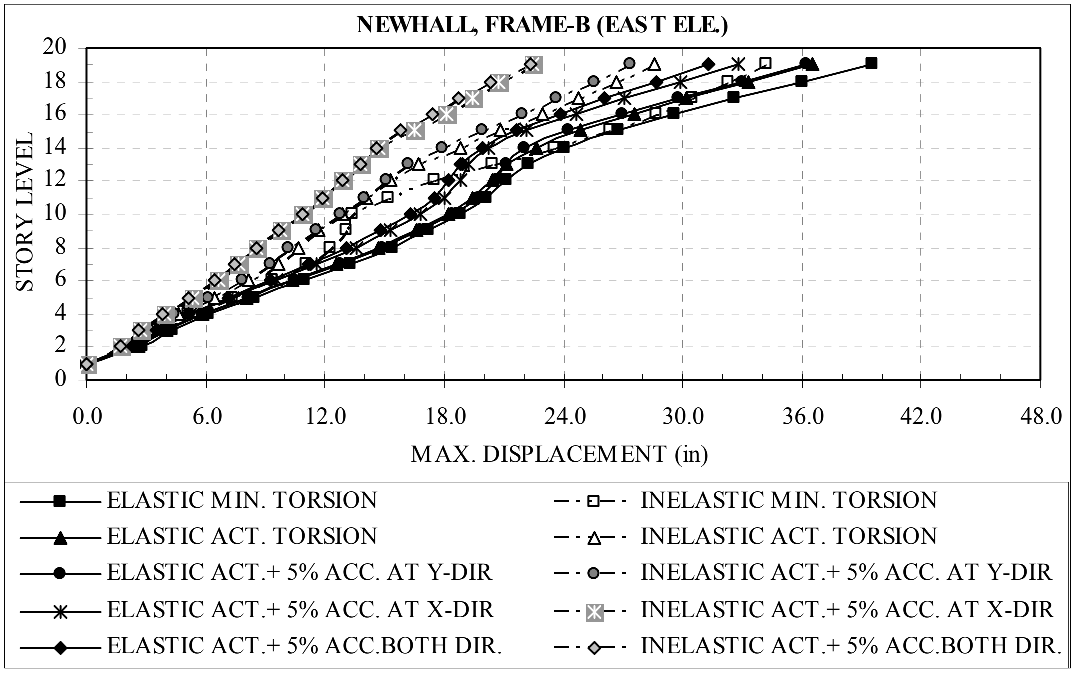


Figure 5.44 – Comparison of the Maximum Displacement of the Building, Inelastic Models, NEWHALL Record

For Newhall record with larger responses in N-S direction, Frame "D" that suffers the most, the maximum displacement increase due to torsion at roof is approximately (41.30-33.99)/33.99= 7.31/33.99= 21.5%. Considering the response at the roof level in N-S direction under the simultaneous components of Newhall record, the average changes of frames "B" and "D" are 18% and 28% for actual torsion case and actual plus 5% accidental torsion for the "both directions" case respectively compared to minimum torsion case. Accidental torsion alone has about average change of 11% on maximum displacement of the roof. These average changes for the 11th floor (mid-height) are 10%, 22.5% and 12.5% respectively. Comparison of the results between roof and 11th floor shows a decrease of 5 to 8 percent for average changes of frames "B" and "D" along the height of the structure. Note that the effect of torsion is not following the same pattern for two opposite frames and there are different percentages of changes from one model to another one. For instance, the decrease and increase of maximum displacement of Frame "B" and Frame "D" from minimum torsion to actual torsion model is about 16.4% and 20.1% respectively. However the percentages of decrease and increase are different from minimum torsion model to actual plus 5% accidental torsion in both directions and they are about 35% and 22% respectively for Frame "B" and Frame "D". Moving the center of mass toward Frame "D" for 5% accidental torsion model cause to shorten the lever torsional arm for Frame "D" and decrease the percentage of displacement changes for this frame.

Figure 5.45 and 5.46 shows the maximum displacements of different models for Newhall record. The maximum displacements at roof for Frame “B” and frame “D” from *34.0* inches for inelastic minimum torsion model change to *28.59* and *40.83* inches respectively for inelastic actual torsion model and for Frame “A” and frame “C” from *23.63* inches change to *27.36* and *18.09* inches respectively .



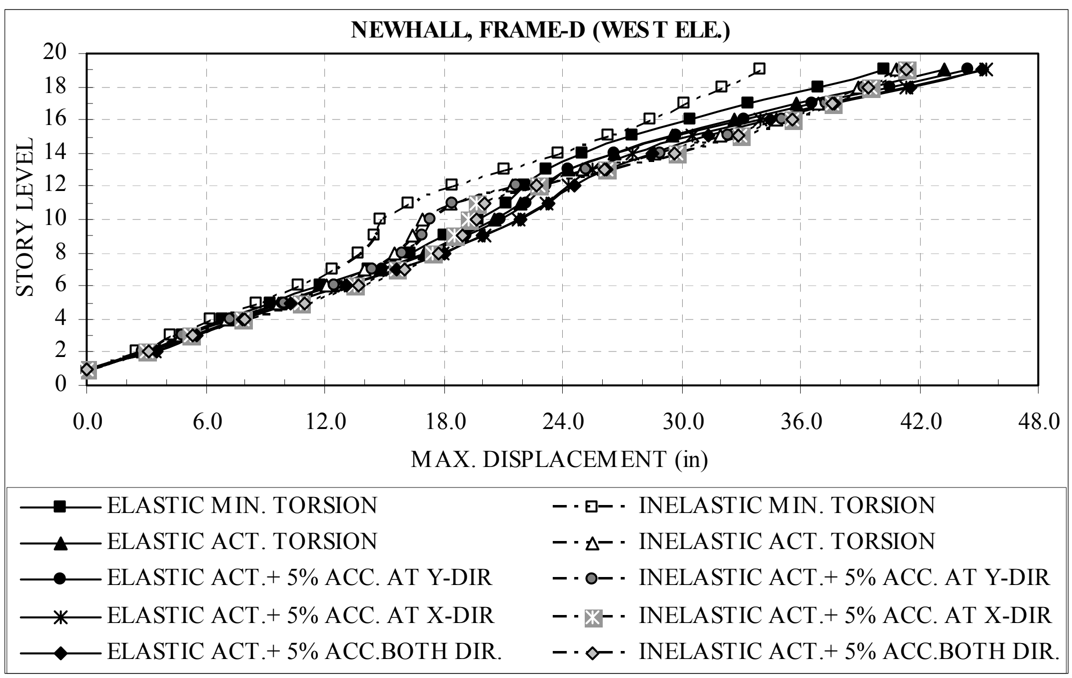
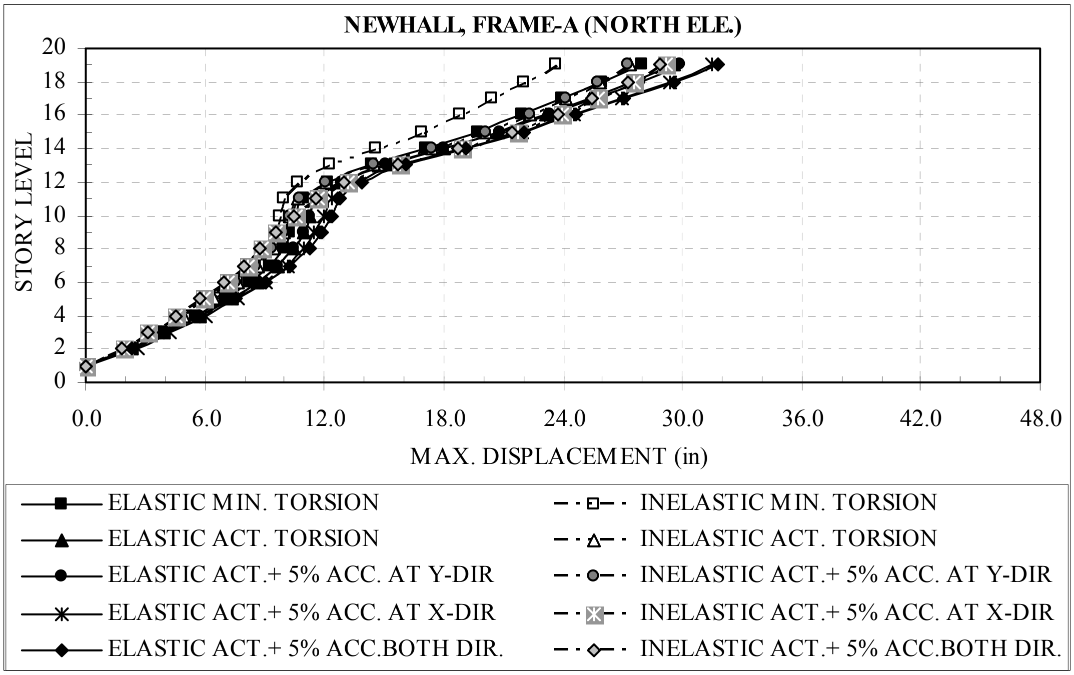


Figure 5.45 -MAX Displacements of Building in N-S Direction, NEWHALL Record



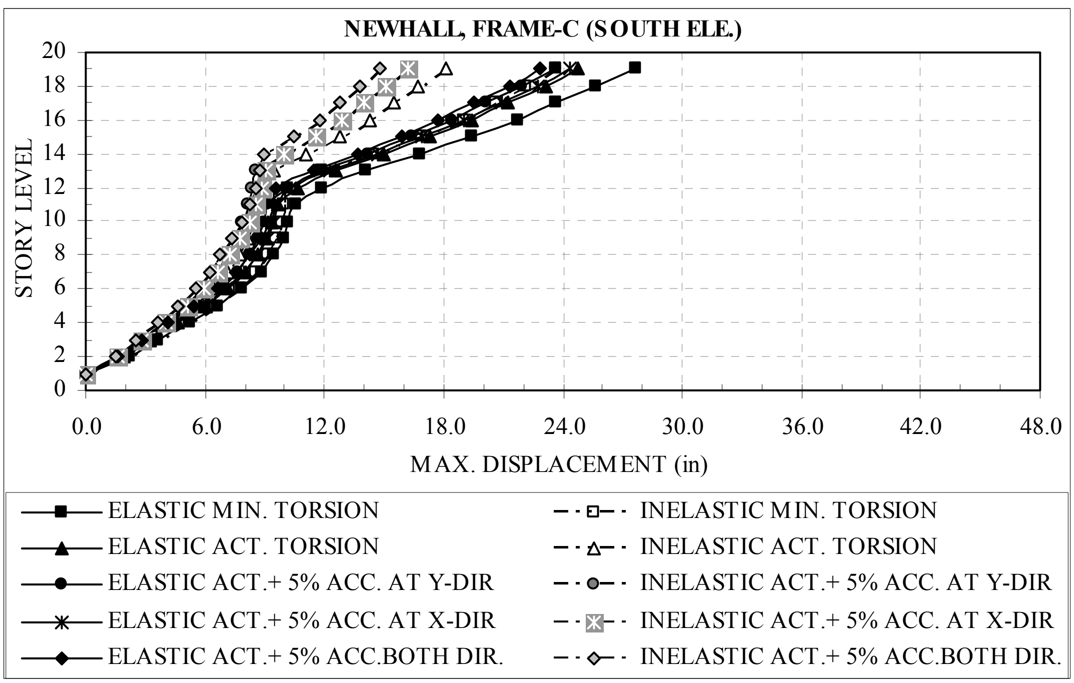


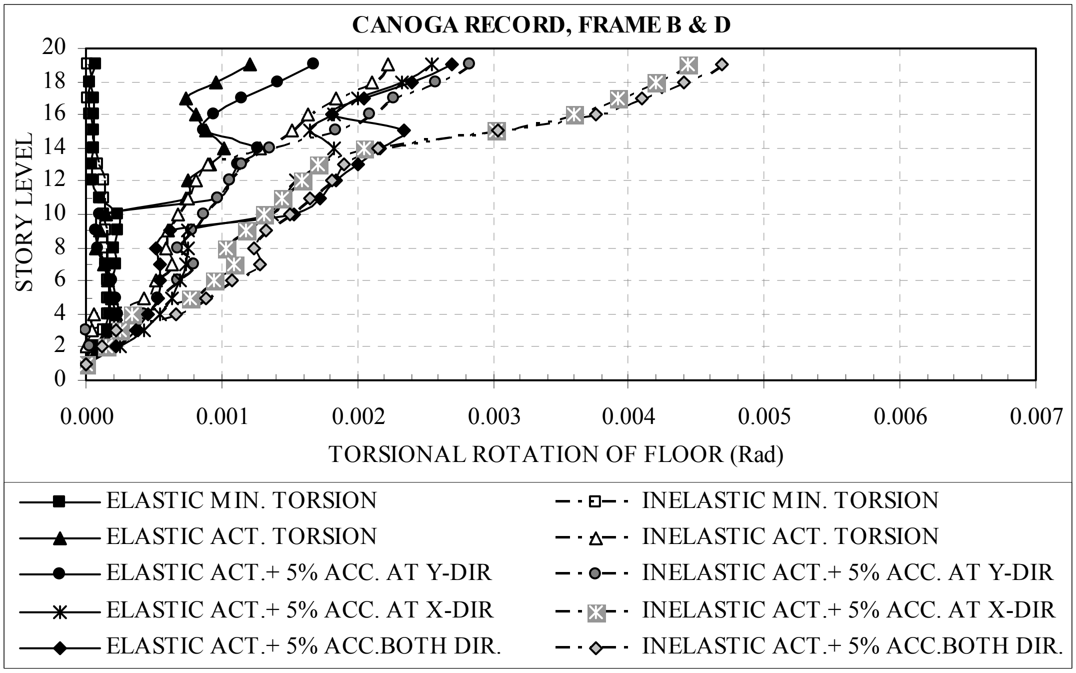
Figure 5.46- MAX Displacement of Building in E-W Direction, NEWHALL Record

For two inelastic models with actual plus 5% accidental torsion only at N-S and E-W directions, the maximum displacement at roof for frames “B” and “D” become *27.35* and *41.13* inches and *22.46* and *41.35* inches respectively which shows large amount of torsion for second model. For frames “A” and “C” roof displacements become *27.25* and *16.24* inches and *29.14* and *16.18* inches respectively for these two models.

In inelastic model with actual plus 5% accidental torsion at both directions, the maximum displacements at roof become *22.36* and *41.3* inches respectively for frames “B” and “D” and *28.83* and *14.77* inches respectively for Frame “A” and Frame “C”.

The results of these three representations of 5% accidental torsion in Figures 5.45 and 5.46 shows that displacement for two models of 5% accidental torsion at N-S and 5% accidental torsion at both directions are very close and 5% accidental torsion at both directions model is a good envelope for all three of them.

By investigation of the graphs in Figures 5.47 through 5.50 and comparing the results for linear and nonlinear cases, the effect of inelastic torsion can be observed. Graphs in Figures 5.47 through 5.49 are showing the amount of torsional rotation of each floor for linear and nonlinear cases for three records. Comparison of any linear and nonlinear model shows the amount of inelastic torsion due to material nonlinearity.



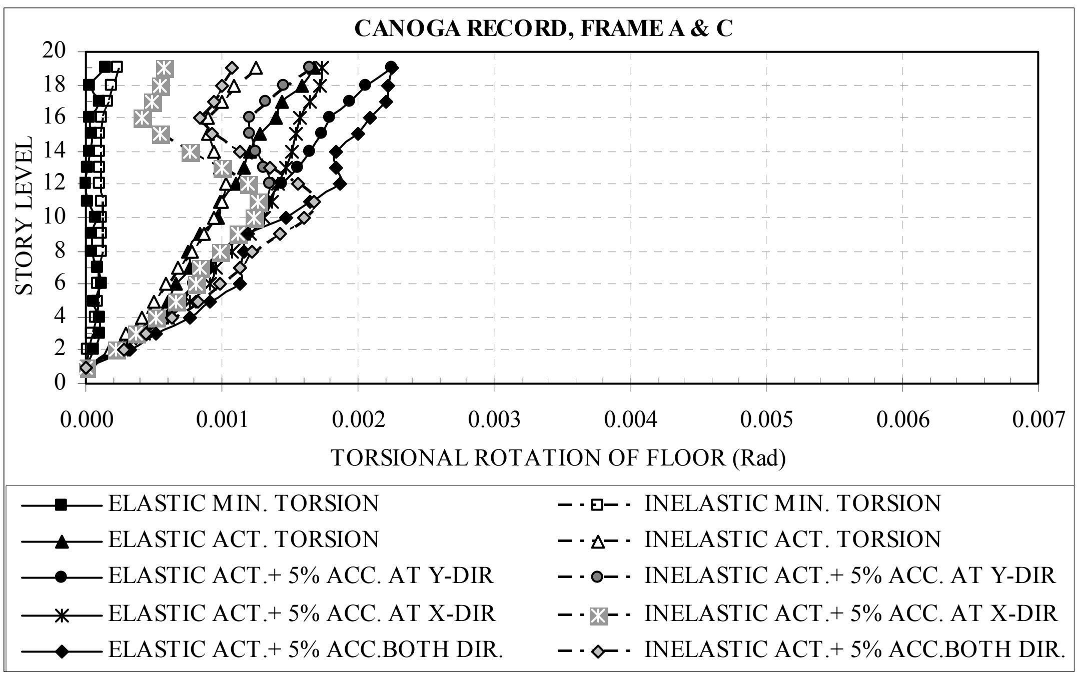


Figure 5.47 –Torsional Rotation of Building, CANOGA Record

Figure 5.47 shows considerable amount of inelastic torsion from 14th floor through the roof for Canoga record. This amount for roof is about to 83 and 74 percent increase to torsional rotation for actual torsion and actual plus 5% accidental torsion in both directions cases respectively. The three representations of accidental torsion have serious effects on torsional rotation and increase it by 38%, 110% and 122% for elastic and 27%, 100% and 110% for inelastic torsion respectively for accidental torsion of N-S, E-W and both directions of the roof. Graph for torsional rotation of Frame "B" and "D" shows a sudden increase at 9th floor level for linear cases. Study of time-history displacement of those two frames clarifies that there is a time shift to the peak displacement at 9th floor for Frame "D" which causes a sudden increase in corresponding torsional rotation. Effect of higher modes on response of upper floors of the structure could be the primary reason for that time shift.

Torsional response of Oxnard record (Fig.5.48) is considerable and graph shows serious amount of inelastic torsion from 10th floor to the roof. This amount at roof is about to 91 and 43 percent increase to torsional rotation of actual torsion and actual plus 5% accidental torsion in both directions cases respectively. All three types of accidental torsions have serious effects on torsional response of Oxnard record and increase it by 36%, 90% and 117% for elastic and 26%, 48% and 63% for inelastic torsion at roof for accidental torsion of N-S, E-W and both directions respectively.

Figure 5.49 shows considerable amount of inelastic torsion from 3rd floor through the roof for Newhall record. For roof, inelastic torsional rotation has about to 81 and 36 percent increase for actual torsion and actual plus 5% accidental torsion in both directions cases respectively. Effects of these three representations of accidental torsion on torsional rotation are considerable and increase it by 22%, 86% and 106%

for elastic and 13%, 54% and 55% for inelastic torsion respectively for accidental torsion of N-S, E-W and both directions of the roof.

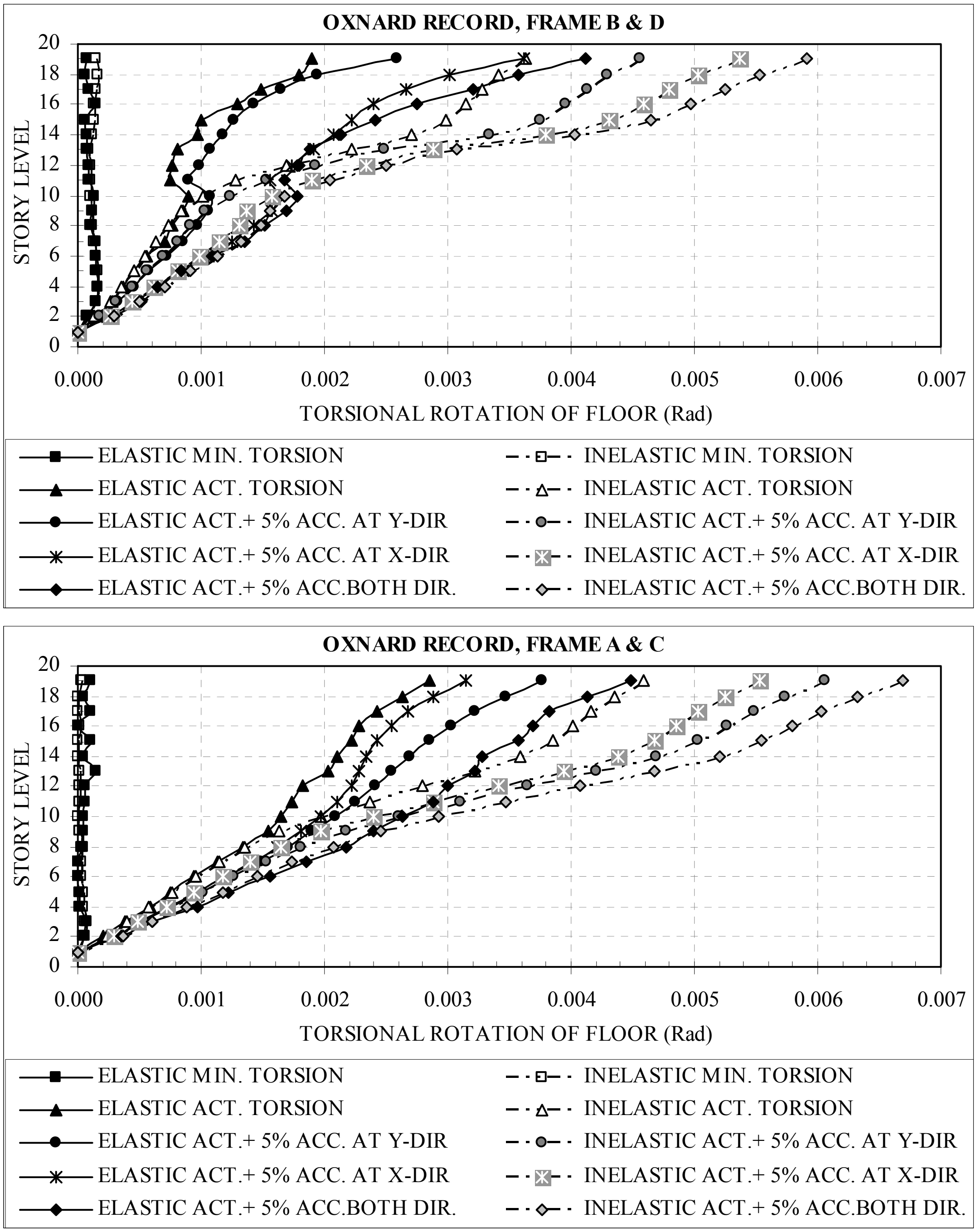
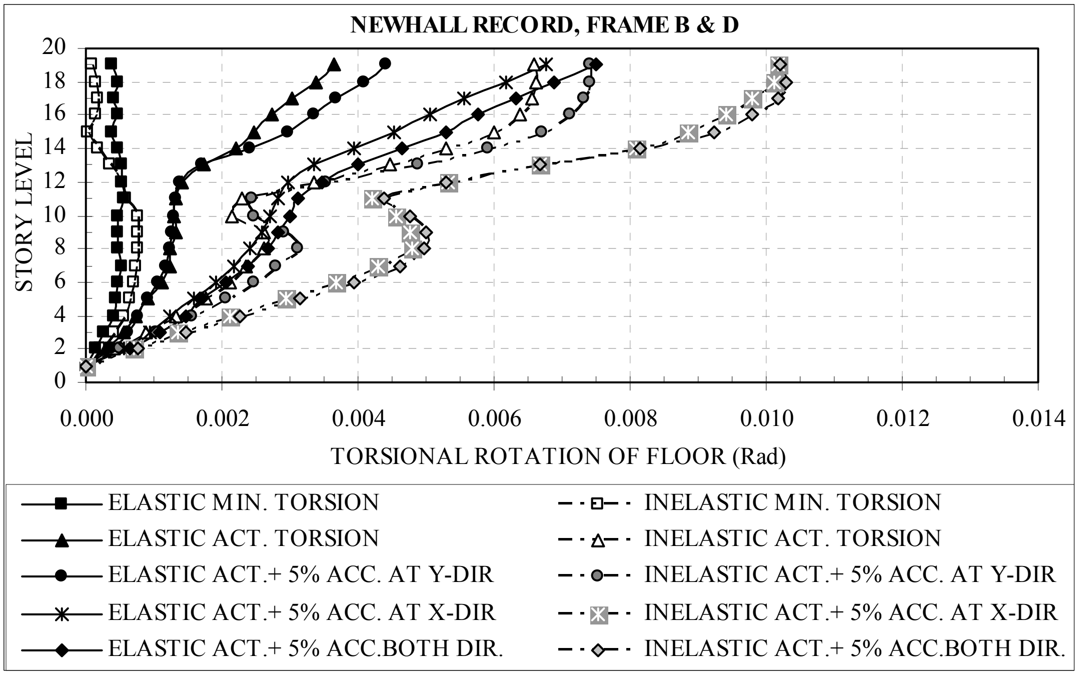


Figure 5.48 - Torsional Rotation of Building, OXNARD Record



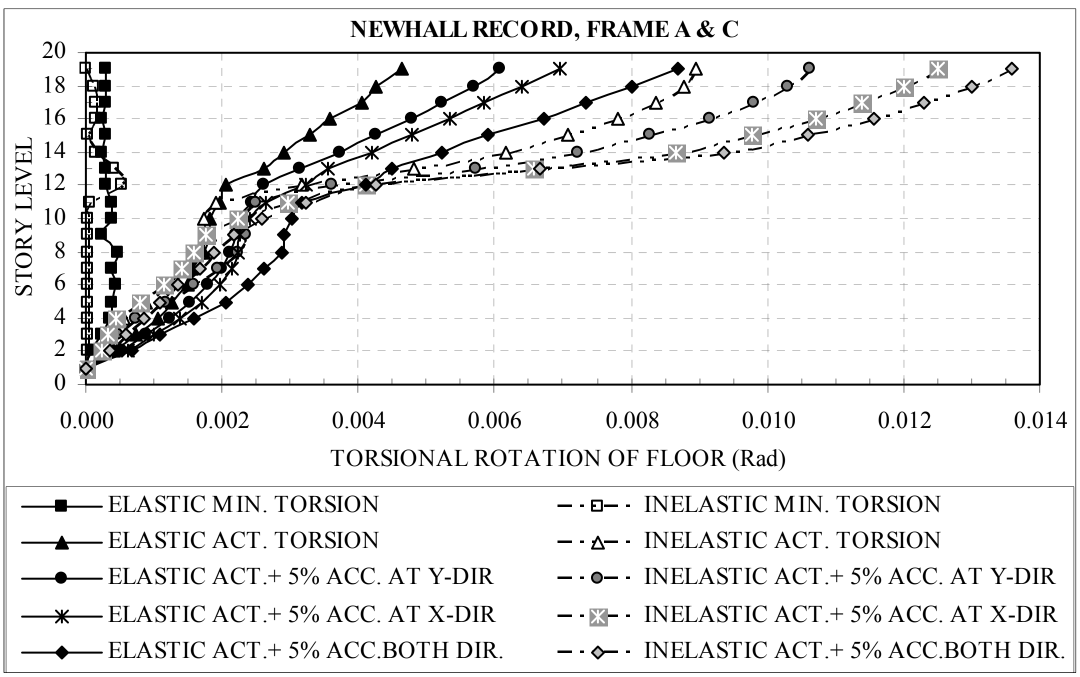
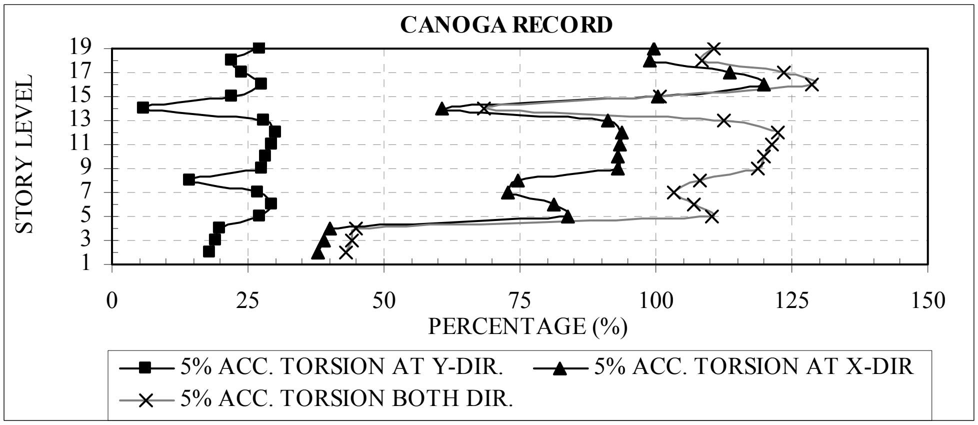


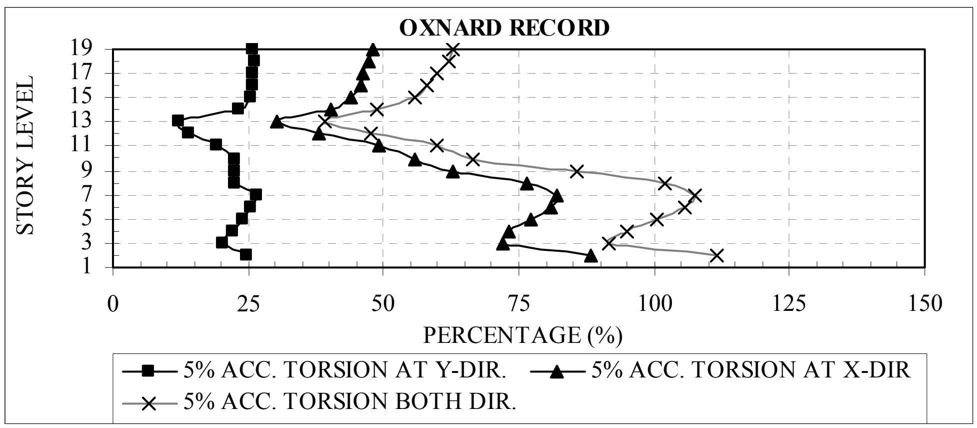
Figure 5.49 - Torsional Rotation of Building, NEWHALL Record

The percentages of the increase for torsional rotation due to material nonlinearity can be summarized as followed. This is done by comparing the inelastic response to the corresponding elastic response. For Canoga record, increase of 25 to 120 percent of torsional rotation is happening from 14-story to the roof and maximum increase happens for four top floors. Note that the increase for middle floors (5^{th} to 10^{th} floors) are unrealistic and it happens because of very small amount of elastic torsional rotation in compare to inelastic torsion. For Oxnard record, there is an enormous increase of torsional rotation from 10-story to the roof and it reaches to 200% at 15^{th} floor. For Newhall, the average increase is about 90 percent and it is almost covering all floors from 4^{th} to roof.

Figure 5.50 shows the increase of torsional rotation due to three different conditions of accidental torsion. Effect of two cases of accidental at X-direction and at both directions is much higher than the Y-direction and the average even reaches to 100% for Canoga record.

Figures 5.51 through 5.53 show the Interstory drift or IDI of different frames of Canoga building for three different models for inelastic case. Result for different frames shows that the Interstory drift is much higher for Frames "B" and "D" than Frames "A" and "C" for Canoga and Oxnard records. Newhall record shows much higher Interstory drift than Canoga and Oxnard and it is about the same for all frames at minimum torsion case. Other cases clearly show the effect of torsion on Interstory drift by increasing the drift for Frame "A" and "D" (lateral and torsional responses in same directions) and decreasing the drift for Frame "B" and "C (lateral and torsional responses in opposite directions). Peak of Interstory drift for Newhall record happens from floor thirteen through seventeen for most of the cases.





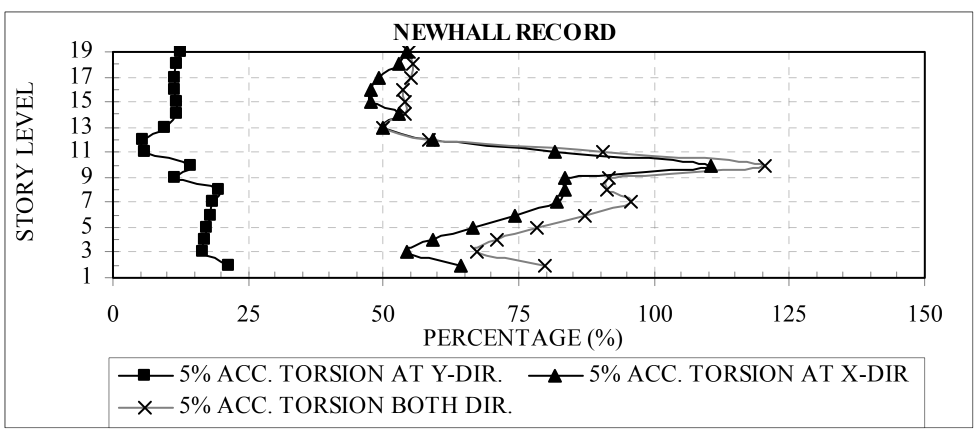


Figure 5.50 –Increase of the Torsional Rotation of Building due to 5% Accidental Torsion, Nonlinear Cases

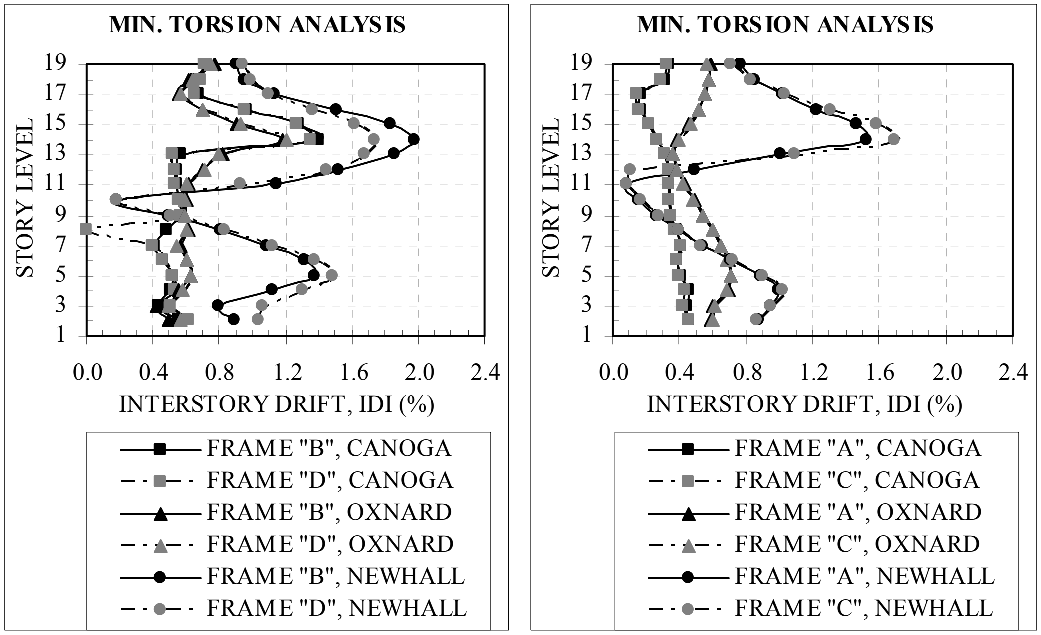


Figure 5.51 –Interstory Drift, IDI of Inelastic Analysis, Minimum Torsion

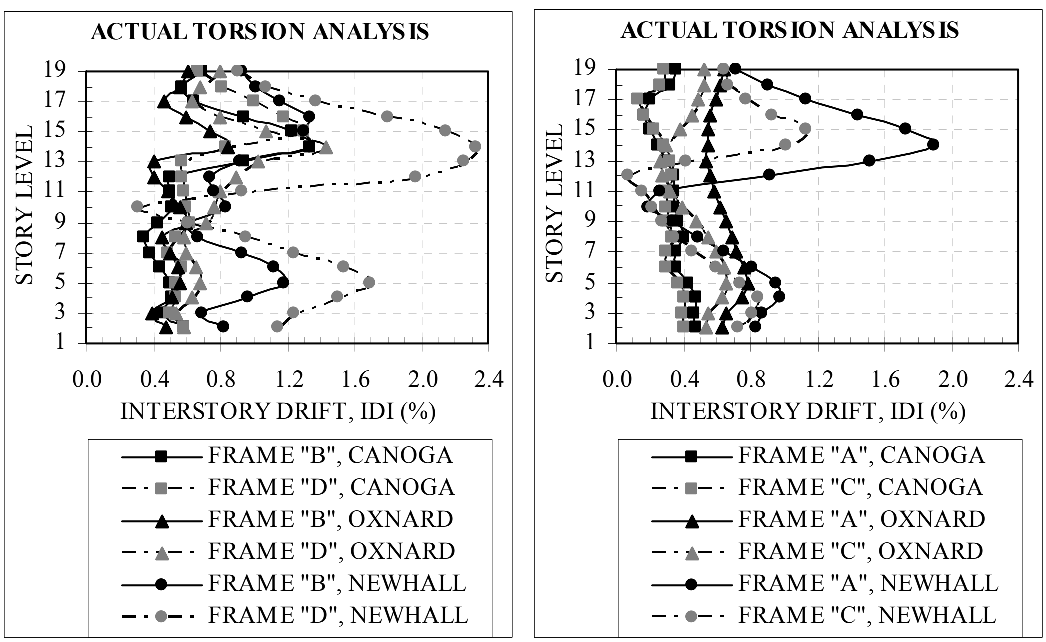


Figure 5.52 –Interstory Drift, IDI of Inelastic Analysis, Actual Torsion

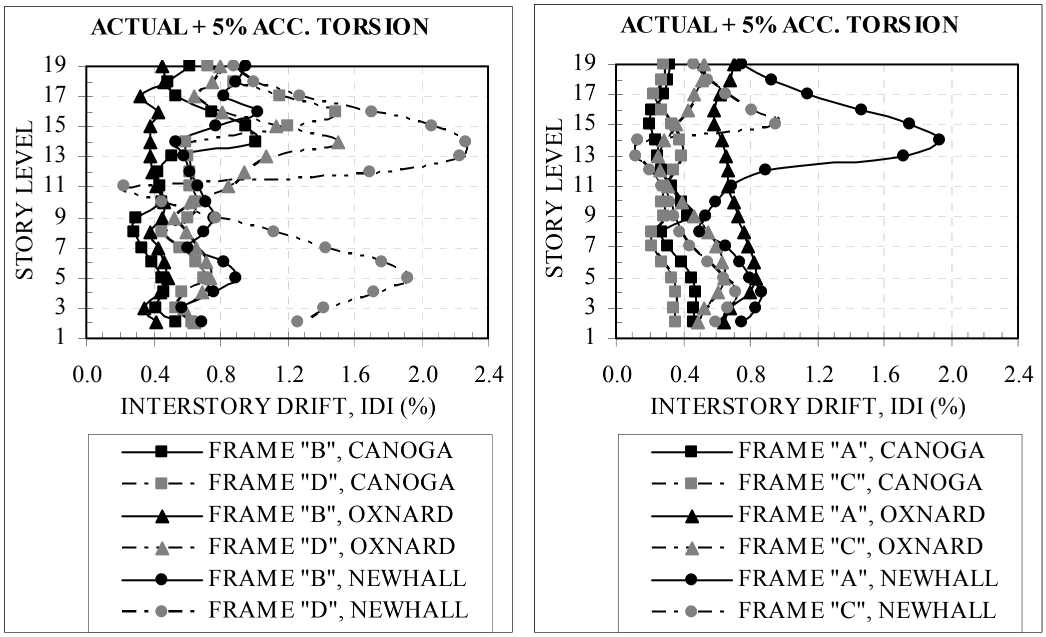


Figure 5.53 –Interstory Drift, IDI of Inelastic Analysis, Actual + 5% Acc. Torsion

The amount of elastic plus plastic (total) hinge rotations of beams for different records are shown in figures 5.54 through 5.56. The total hinge rotations of beams for Canoga and Oxnard records are shown on figures 5.54 and 5.55. Frames "B" and "D" have much higher hinge rotation than "A" and "C" and it reaches to the maximum rotation of 0.0186 radians at 14^{th} floor for Canoga record and 0.0145 radians at 12^{th} floor for Oxnard record for Frame "D". Besides, beams on 9^{th} through 16^{th} floor have the higher hinge rotation. The elastic hinge rotation for both 12^{th} and 14^{th} floor W36x194 and W36x170 beams based on formula (4-4) is 0.0044 radians, so the maximum plastic hinge rotation for these two records is 0.0142 radians which is within the 2^{nd} deformation limit of FEMA 273.

For Newhall record (Fig.5.56), the total hinge rotations of beams are higher than two other records and it reaches to the 0.023 radians at 4^{th}, 13^{th} and 14^{th} floor of frame "D". Considering 0.0044 radians as elastic hinge rotation of 14^{th} floor W36x170 beam, the maximum plastic hinge rotation for the Newhall record is 0.0186 radians which is again within the deformation limit of FEMA 273.

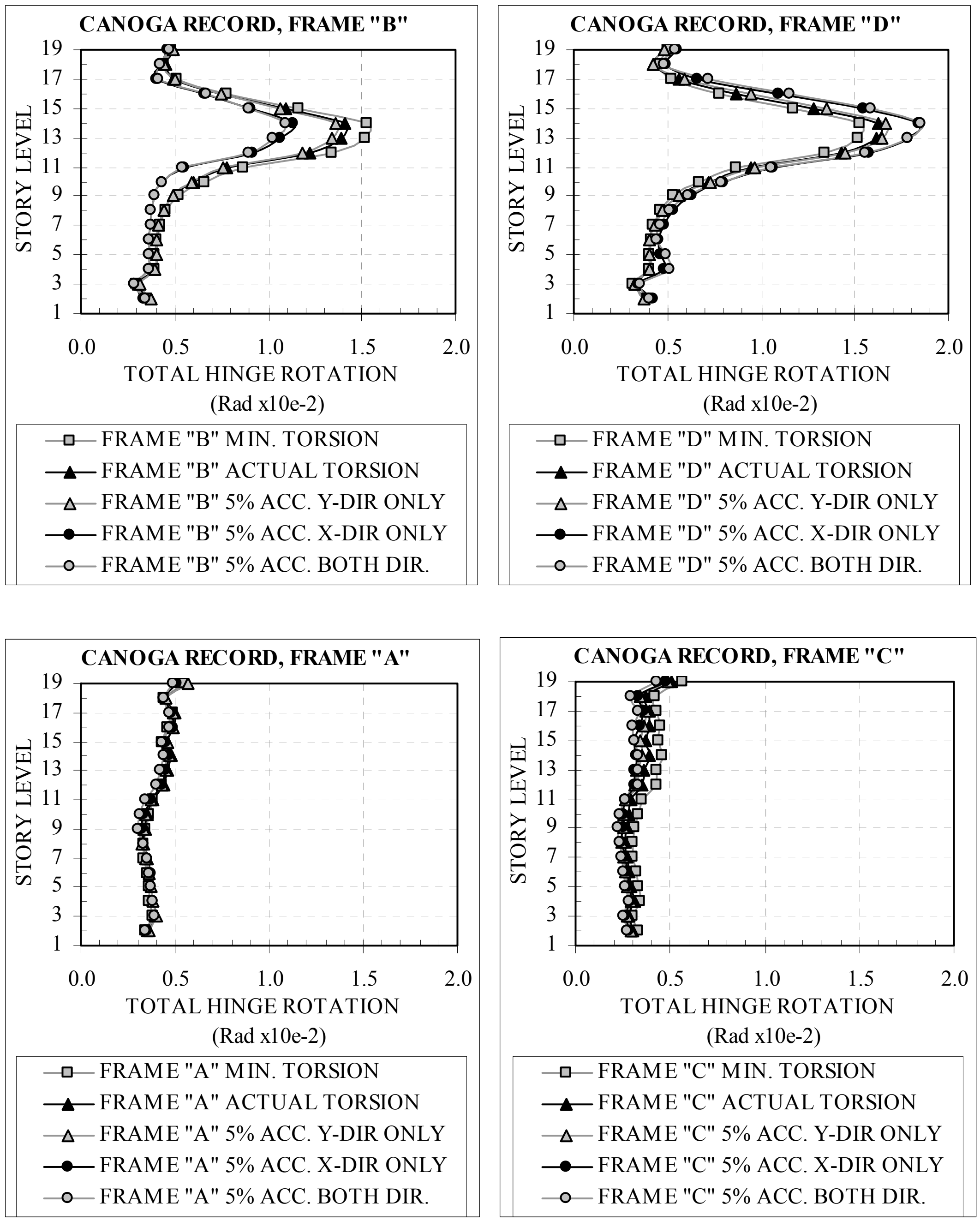


Figure 5.54 –Envelope of Maximum Hinge Rotation of Beams, CANOGA Record

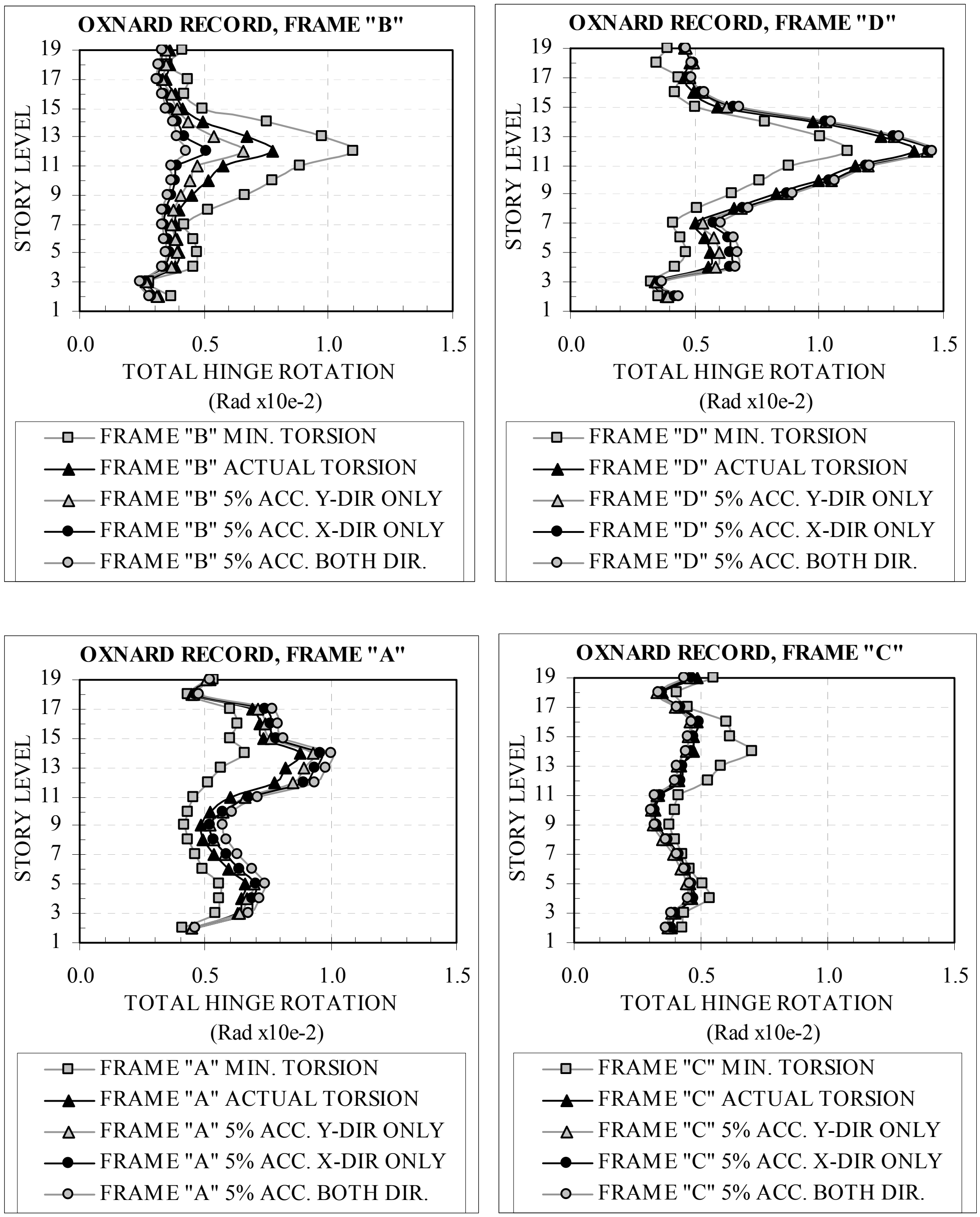


Figure 5.55 –Envelope of Maximum Hinge Rotation of Beams, OXNARD Record

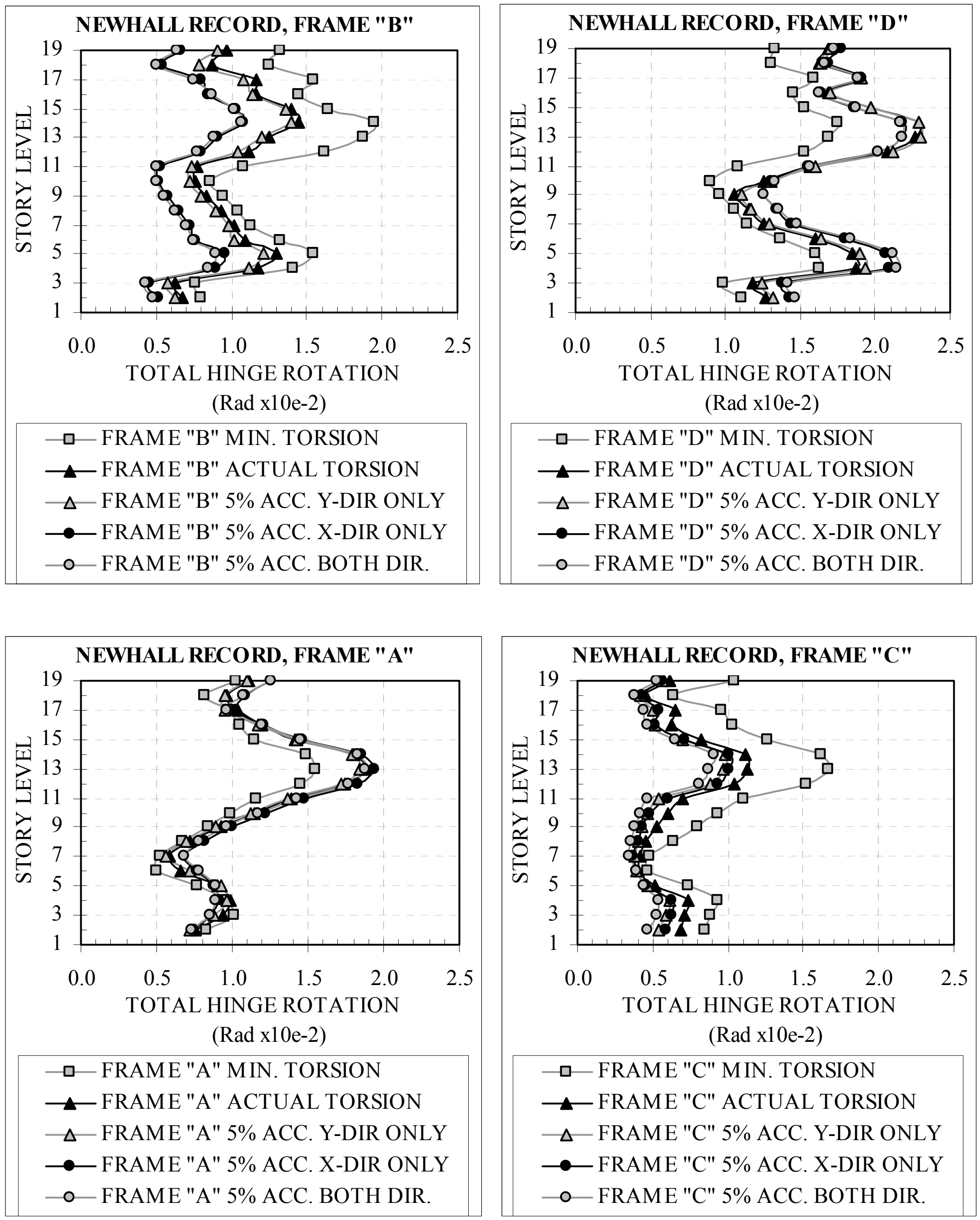


Figure 5.56 –Envelope of Maximum Hinge Rotation of Beams, NEWHALL Record

Figures 5.57 through 5.59 show the time-history displacements of different frame elevations for all three records to investigate the residual displacement. Residual or permanent displacement of frames occurs after forming plastic hinge mechanism.

For all models, residual displacements for frames at east and west elevations (Frame "B" and "D") are much higher than those on south and north elevations. For Frame "B" at east elevation, residual displacements start from 6.5 and 3.0 inches at roof for minimum torsion model of Canoga (Fig.5.57) and Oxnard (Fig.5.58) records and decrease to 3.5 and 0.0 inches for actual torsion model with 5% accidental torsion. For Frame "D" on west elevation, residual displacements increase from 6.0 and 3.0 inches for minimum torsion model of Canoga and Oxnard records and reach to 8.0 and 5.0 inches at roof for actual plus 5% accidental torsion model as a maximum residual displacement of these two records. The amount of residual displacements for actual torsion model of Canoga record is 6.8" which is very close to the real residual displacement of 6" measured after earthquake. For two other frames on North and south elevations, residual displacements are very low and for Frame "A" reaches to 1.0 inch as a maximum for Canoga record.

For Newhall record (Fig. 5.59), residual displacements are much higher for all frames than two other records. Residual displacements increase from 19.0 to the maximum of 20.5 inches in Frame "D" and from 8 to 10.0 inches in Frame "A" for accidental torsion and decrease from 9.0 to 4.5 in Frame "B" and from 2.0 to minimum of 1.0 inches for accidental torsion as maximum and minimum residual displacements of Newhall record. Amount of maximum residual displacement for Newhall record is much higher than two other records and reaches to 2.5 times of Canoga record.

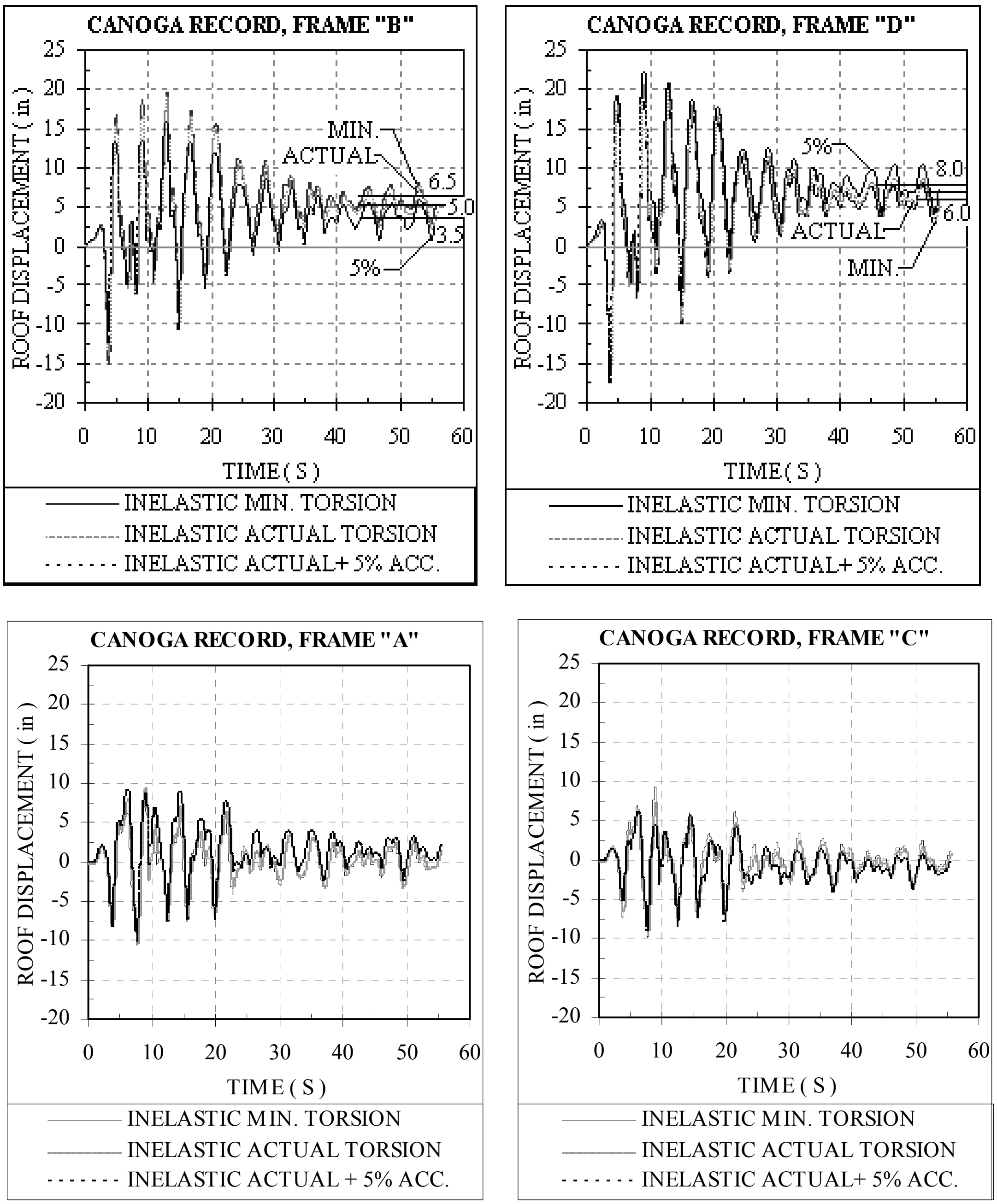


Figure 5.57 –Time-history Plot of Roof Displacement, N-S and E-W Directions, CANOGA Record

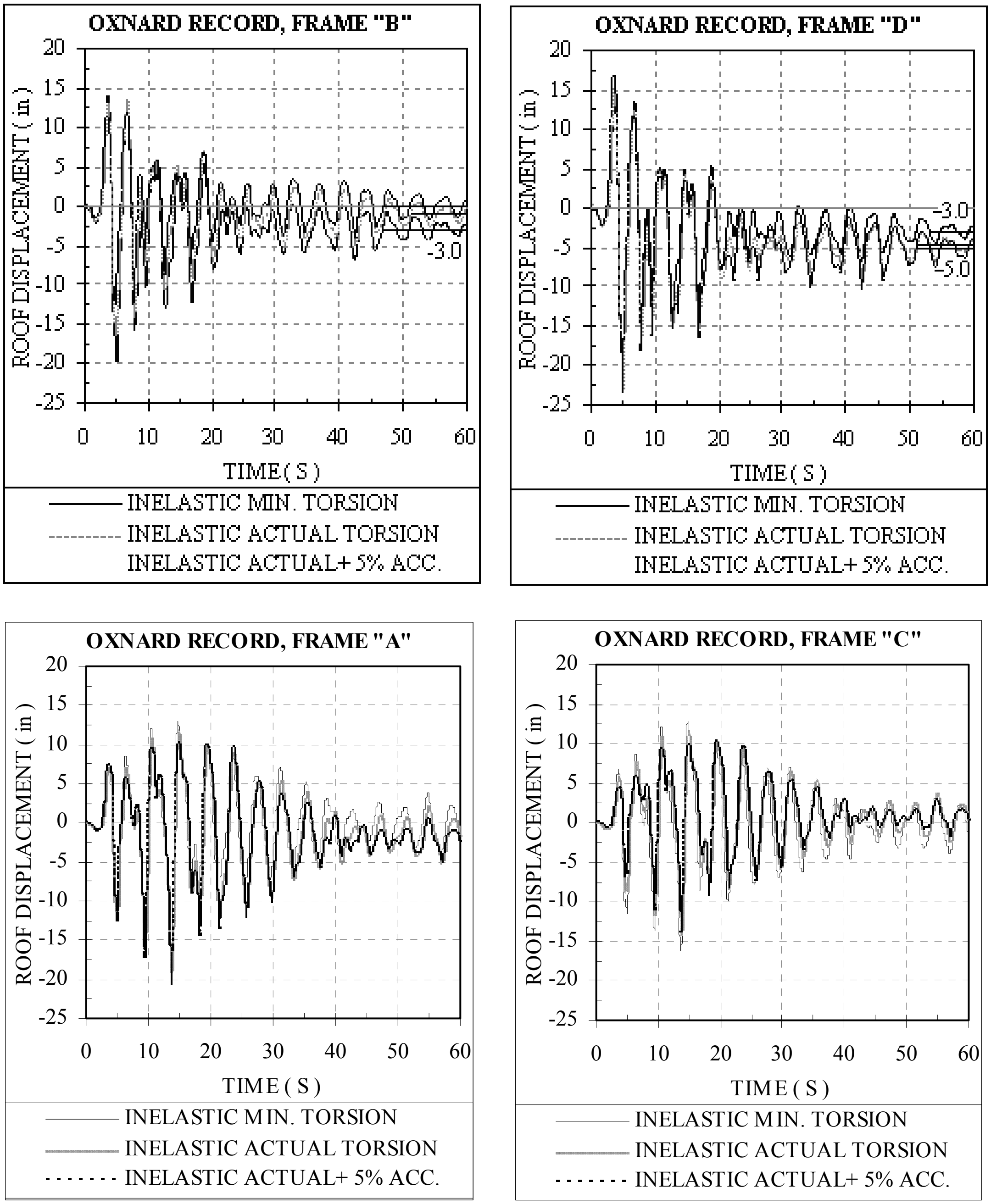


Figure 5.58 –Time-history Plot of Roof Displacement, N-S and E-W Directions, OXNARD Record

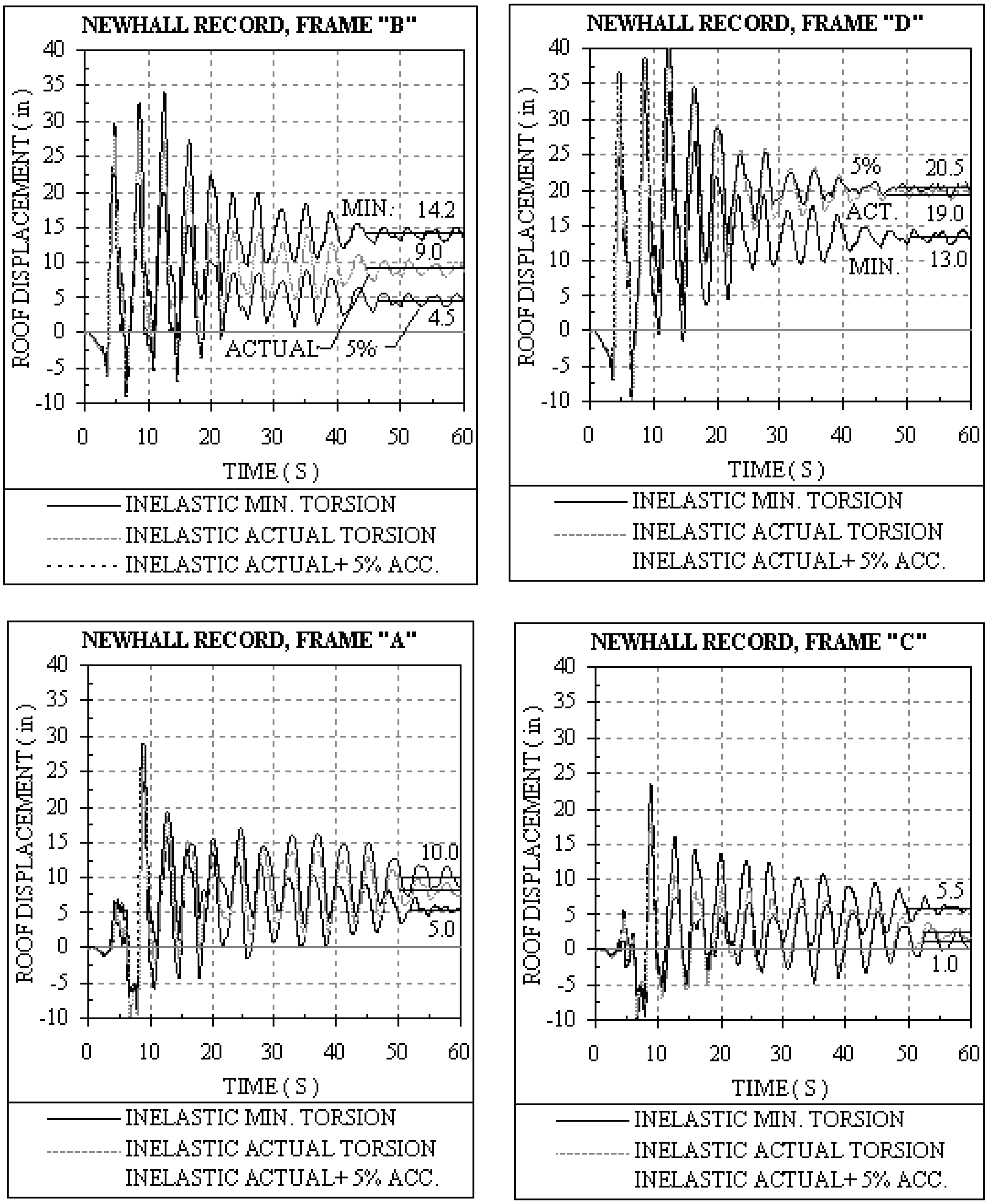
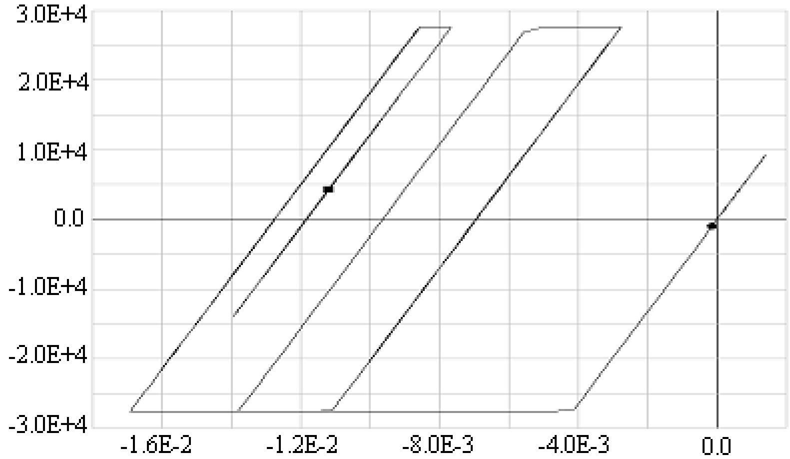
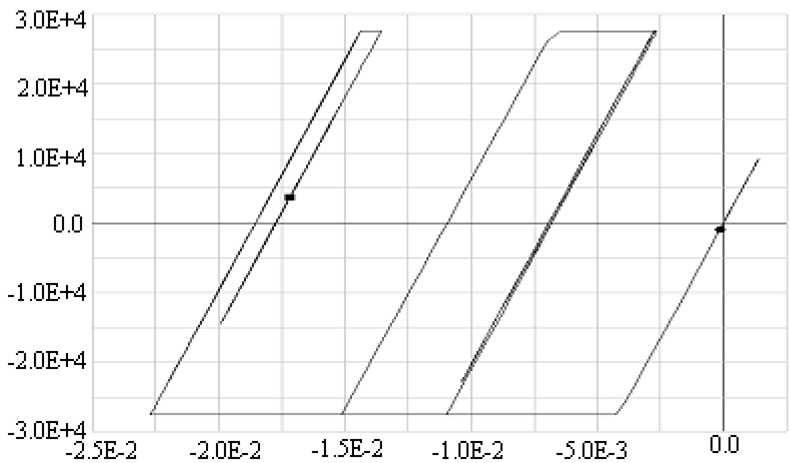


Figure 5.59 –Time-history Plot of Roof Displacement, N-S and E-W Directions, NEWHALL Record

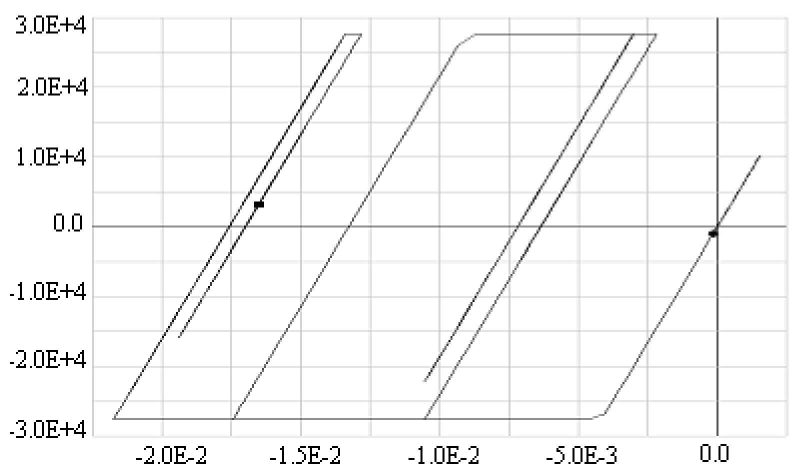
Figure 5.60 compares moment versus hinge rotation hysteresis loops of W36x194 beam on 13^{th} floor of frame "D" at different models for Newhall record.



Min. Torsion



Actual Torsion



5% Acc. Torsion in Both Directions

Figure 5.60 –Moment versus Hinge Rotation, 13th Floor, W36X194, Newhall Record

Chapter 6: Inelastic Torsional Response of Building due to Material and Geometric (P-Delta Effect) Nonlinearities

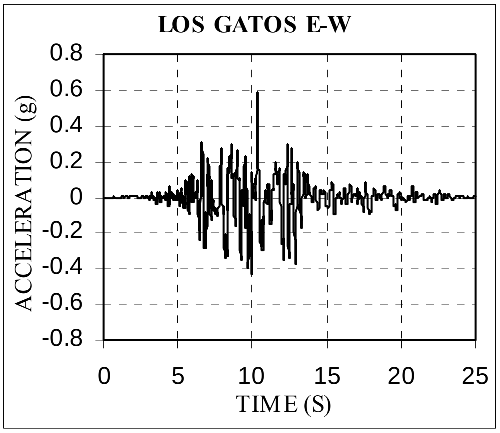
A. 18-story Building Case (CANOGA Bldg.)

The purpose of the study presented in this chapter is to evaluate the inelastic torsional response of the *CANOGA* Building due to material and geometric nonlinearity. Studies in previous chapters showed the effect of material nonlinearity to increase the torsional response of the structure relative to linear models and in this chapter by adding geometric nonlinearity, known as P-Delta effect to the different models of the 18-story steel *CANOGA* building, the combined nonlinearity effects on inelastic torsional response will be investigated.

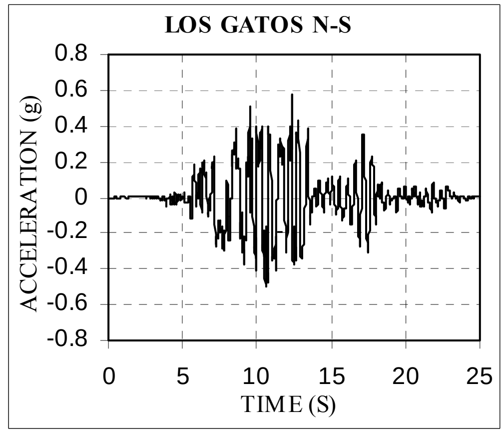
B. Description of the Model and Analysis

As discussed before, initial modeling and inelastic analyses of this building were conducted using the *PERFORM 3-D* computer program. In order to have P-Delta analysis, all gravity loads of the floors and roof were applied to moment frames and gravity columns (Fig.5.37).

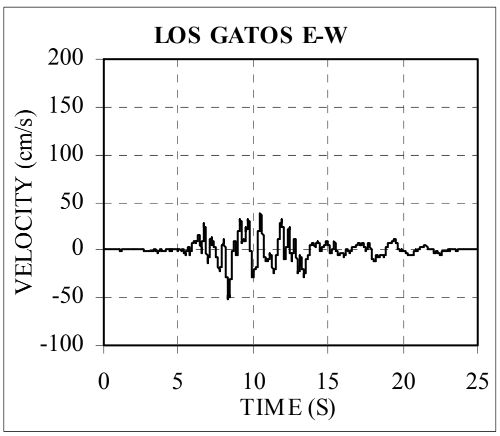
For dynamic analysis of each model, three pairs of time histories recorded during the Northridge and Loma Prieta (1989) earthquakes were utilized for this case study. Two records from the Northridge earthquake are the Newhall record used in previous analyses (Fig.5.4) and the Rinaldi record and the additional record from the Loma Prieta earthquake is Los Gatos (Fig.6.1 and Fig.6.2). It should be emphasized that these are strong motion records, but are not related to the actual building.



1- ACCELERATION



1- ACCELERATION



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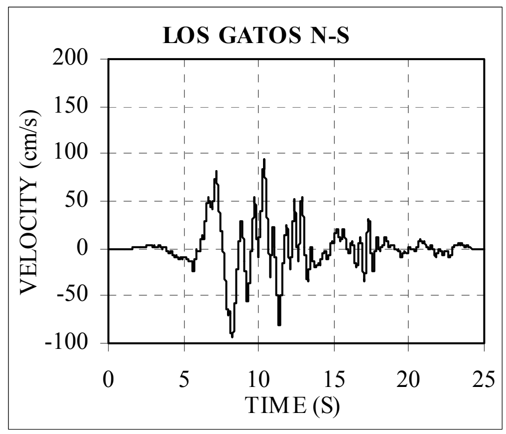
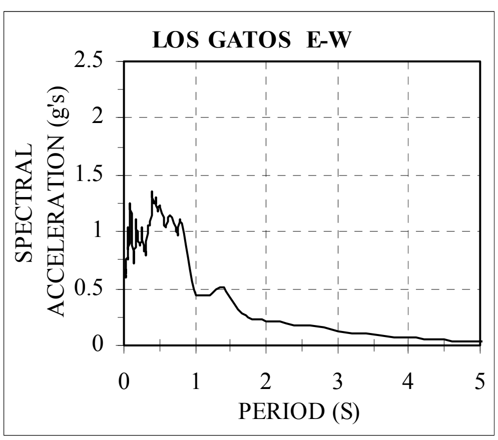
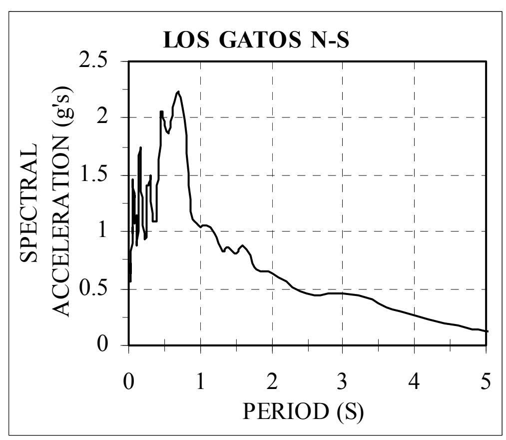


2- VELOCITY (cm/s)



3- SPECTRAL ACC.

E-W



3- SPECTRAL ACC.

N-S

Figure 6.1 – Los Gatos Records from Loma Prieta Earthquake

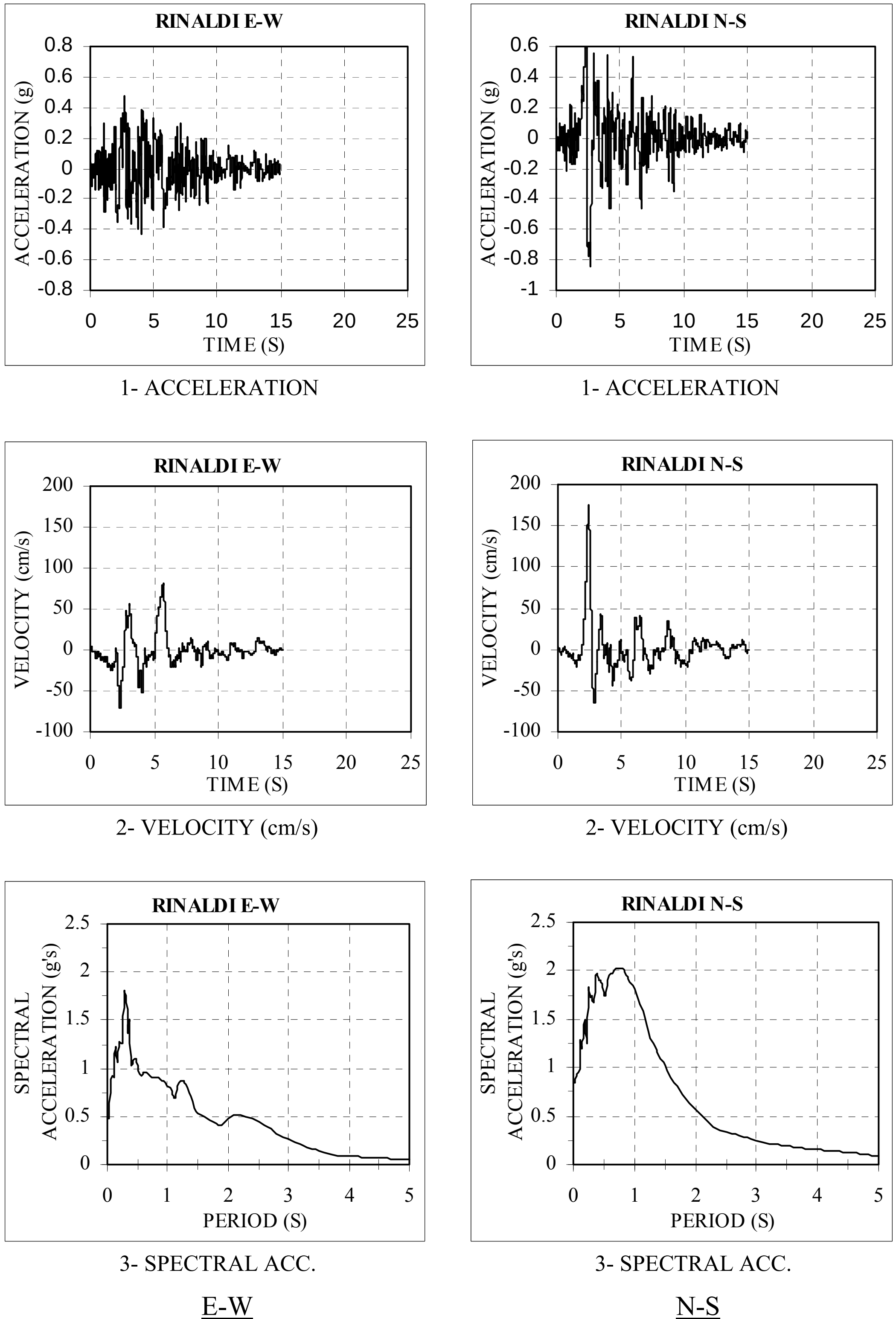


Figure 6.2 – Rinaldi Records from Northridge Earthquake

The Newhall and Los Gatos records both have a high PGA of about 0.6g in both directions and although there are some similarities between frequency content of the N-S component of both records, the frequency content of E-W components are totally different. Study of the spectral accelerations of both records at a few points of interest such as periods of first and second modes in each direction (about 4.0 and 1.35 second for the CANOGA building) shows that the response of the structure to the Los Gatos record might have been more significant for N-S component than E-W component in comparison to the Newhall record. However, both components of each record will be applied simultaneously to the model and the inelastic response of the model will be a combination of both components.

The Rinaldi record has a very high PGA about 0.82g in the N-S direction and lower PGA of about 0.45g at E-W direction. The frequency contents of N-S and E-W components are both comparable with other two records, however, the duration of the Rinaldi record is shorter in comparison with the other two.

To investigate the effect of inelastic torsion plus P-Delta effect on the response of this building, the same three models from previous study with different eccentricities considering only inelastic behavior were utilized. For the first model, by moving of the C.O.M of each floor to the C.O.S, minimum torsional response of the building was investigated. The second model considered the torsion of the building for both directions due to the actual differences between locations of C.O.M and C.O.S. at each floor. For the last model, 5% accidental torsion at both directions was added to the actual torsion of the building to magnify the effect of torsion (Table-3).

For each model there are at least two analyses for each record considering the model with and without P-Delta effect. For the Actual Torsion model, there are some extra analyses considering different critical damping ratios with and without P-Delta effect. In utilization of these three models, the effect of +/- acceleration for each pair of N-S and E-W records were considered and based on the results, the pair producing the maximum torsion was chosen.

Note that to determine residual displacements from maximum displacement time-history curve, a short time interval of few seconds from the end portion of the time-history curve was chosen, then the maximum and the minimum of all displacements for that time interval have been evaluated. The absolute average of the maximum and minimum displacements is then taken as the residual displacement of this displacement time-history.

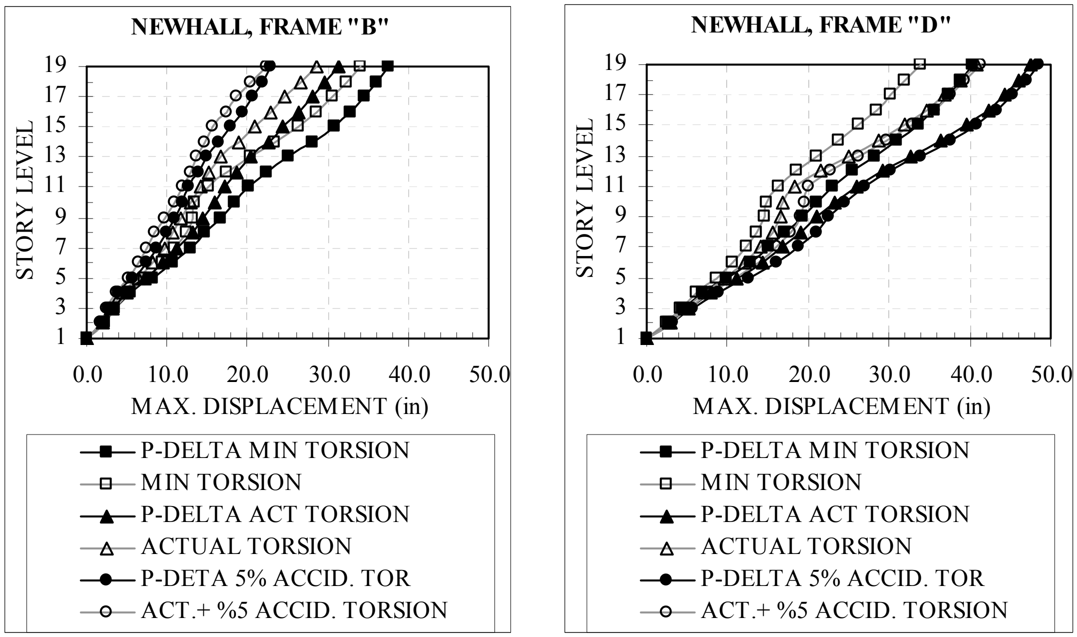
C. Discussion of the Results

The nonlinear dynamic response parameters of the Canoga building to three different records from Northridge and Loma Prieta earthquakes considering the P-Delta effect are shown in Figures 6.3 through 6.31. Modal characteristics of the building for the first eight modes have been summarized and shown in Table 4 and Figure 5.38. The periods per second and modal participation factors of the first five modes of vibration based on the initial stiffness of the members are as follows: (4.03, 78.5%) for first mode in E-W direction, (3.81, 76%) for first mode in N-S direction, (2.29, 3%) for first rotational mode of vibration, (1.42, 12.2%) for second mode in E-W direction and (1.34, 13.3%) for second mode in N-S direction.

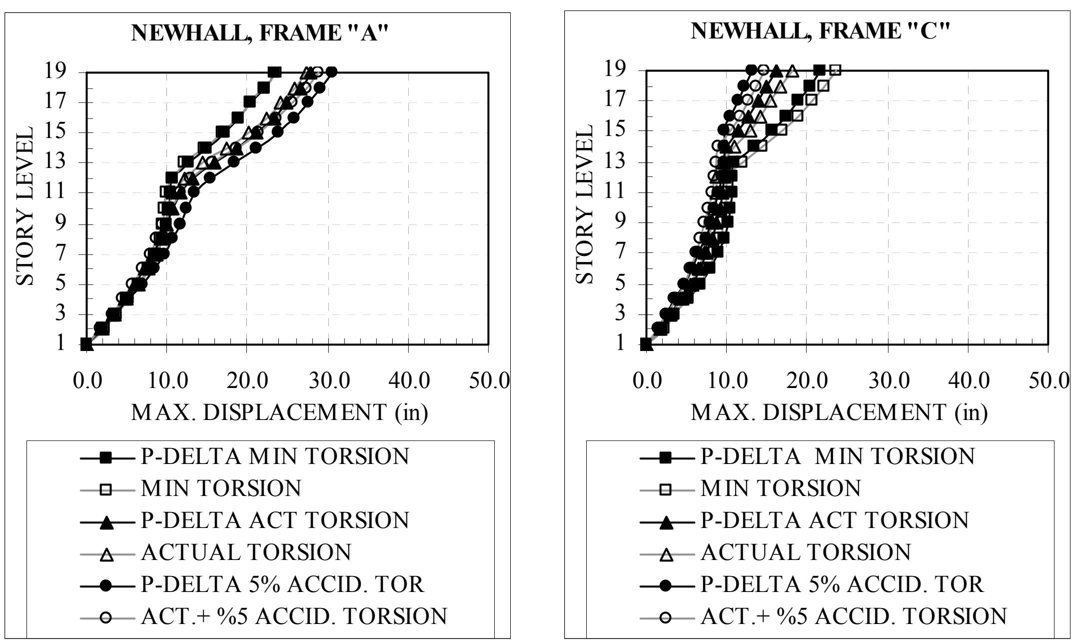
Starting with the Newhall record, the set of graphs on Fig.6.3 and Fig.6.4 show the maximum and residual (permanent) displacements of each model with and without P-Delta effect in N-S and E-W directions.

By comparing the results of each graph, the effect of P-Delta on maximum and residual displacement of each model can be studied. For N-S direction at roof, the average changes for simultaneous components of Newhall record are 14%, 13% and 10% on maximum displacement and 53%, 47% and 55% for residual displacement for minimum torsion, actual torsion and actual plus 5% accidental torsion models respectively. The above results clearly show that the effect of P-Delta on residual displacement is much higher than on maximum displacement. Results of these graphs have been summarized in form of a table later on in the conclusion chapter.

For Frame "B" at the east elevation, both maximum and residual displacements of the frame decrease by increasing of the eccentricity. Plus, analyses with P-Delta have a larger displacement than the one without P-Delta. For Frame "D" at west elevation, the maximum and residual displacements show that the 5% accidental torsion model with P-Delta effect has the maximum response which is 49 and 30 inches respectively. The difference between responses for analyses with and without P-Delta for Frame "D" is significant and reaches to 18 and 51 percent for maximum and residual displacement respectively. For the critical response, the displacements for frame "C" at south elevation are much lower than the others but for frame "A" at north elevation follow the same pattern as frame "D".

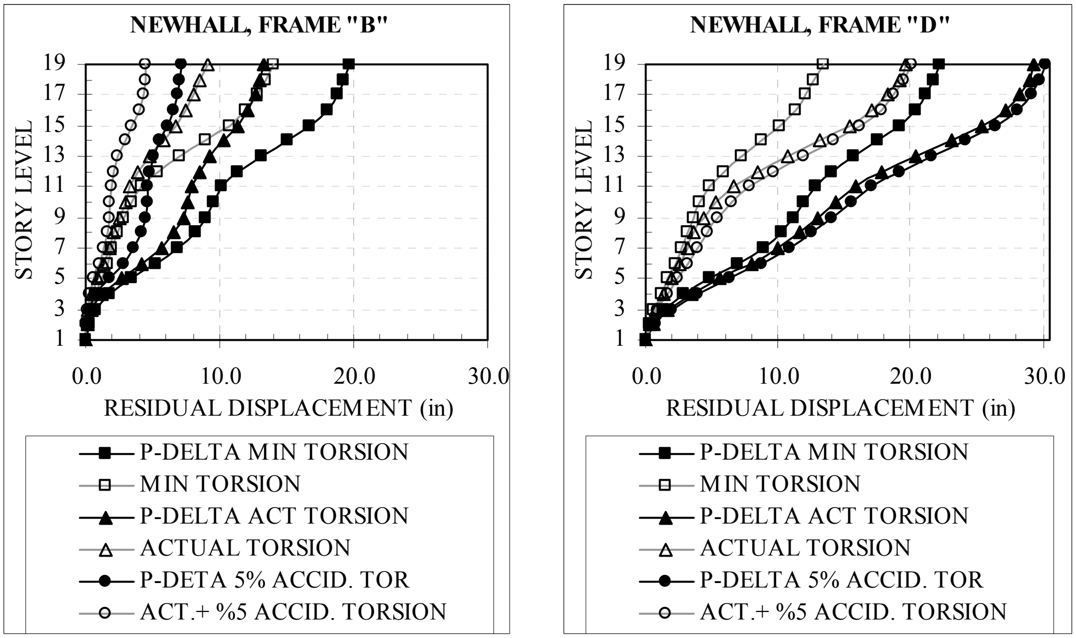


N-S: Frame-B and Frame-D

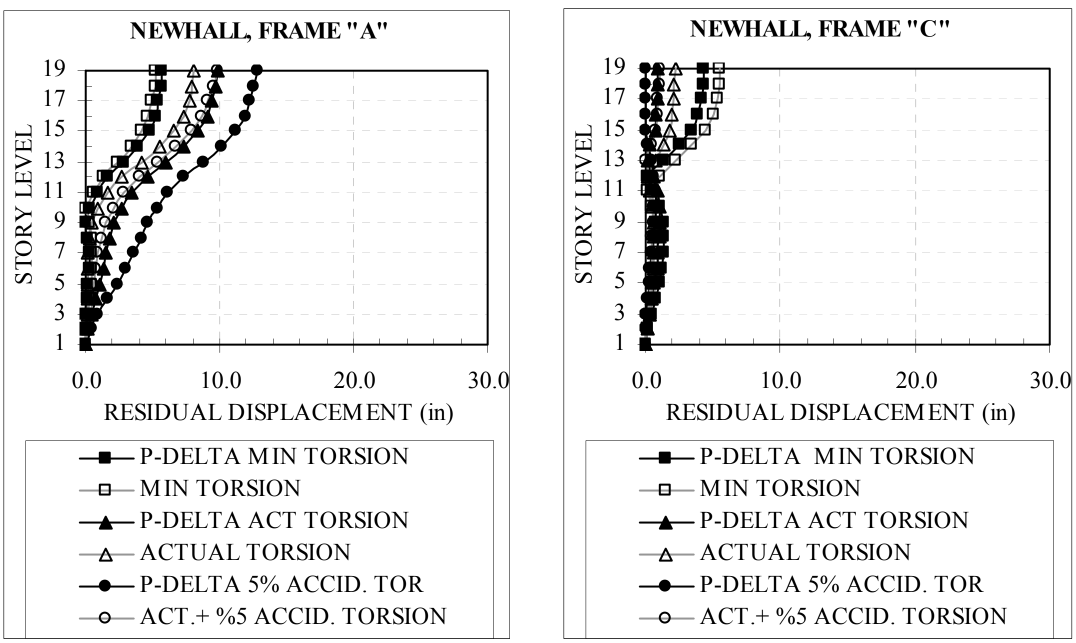


E-W: Frame-A and Frame-C

Figure 6.3 –MAX. Displacement of Building, NEWHALL Record



N-S: Frame-B and Frame-D

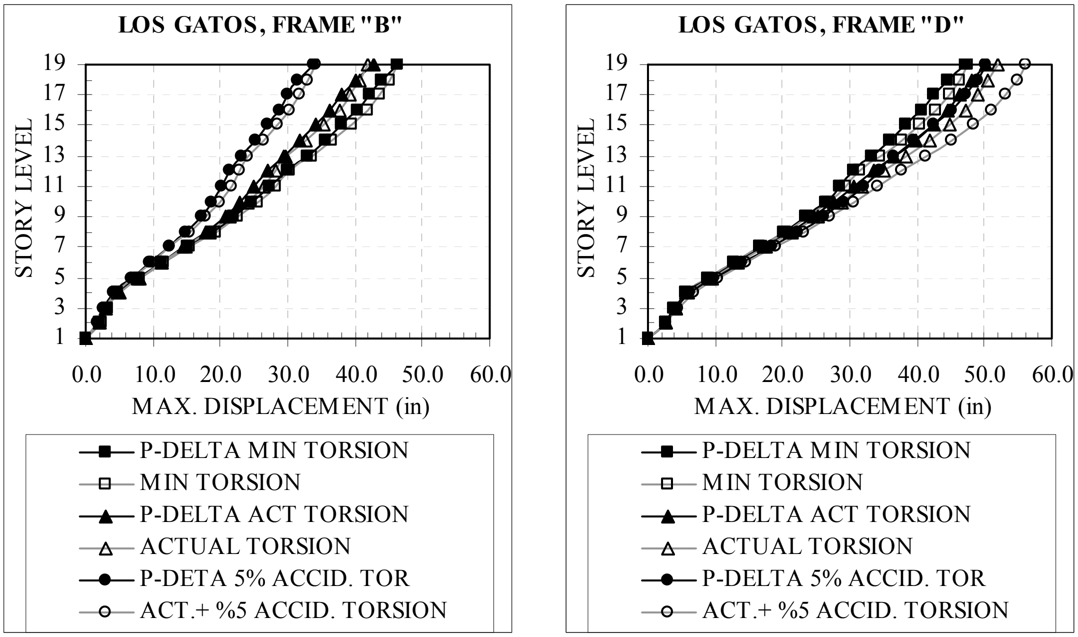


E-W: Frame-A and Frame-C

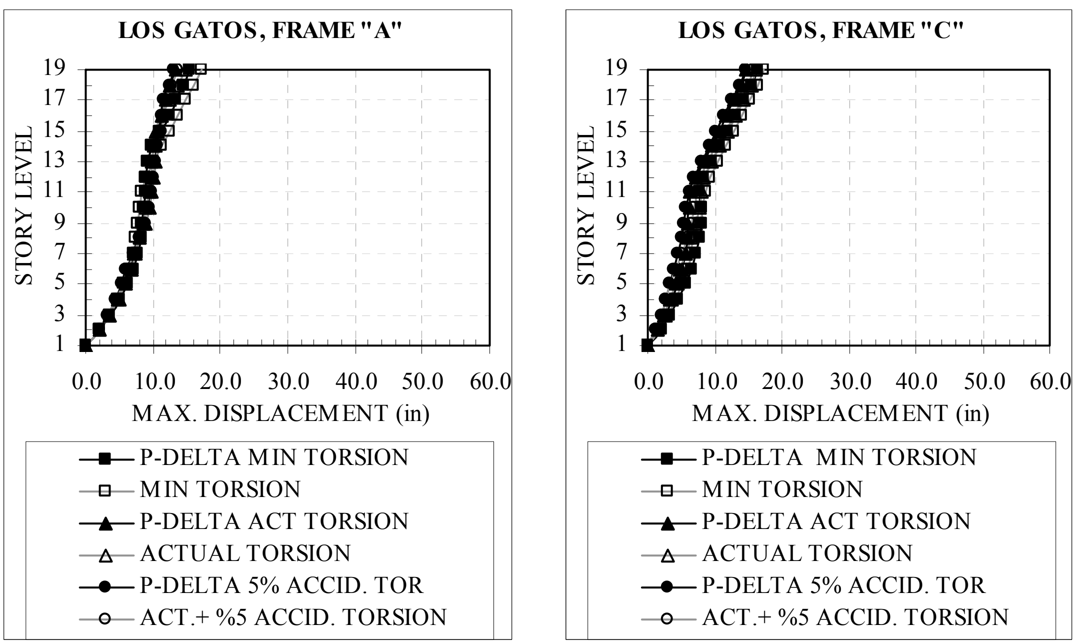
Figure 6.4 –Permanent Residual Displacement of Building, NEWHALL Record

For the Los Gatos record, the set of graphs on Fig.6.5 and Fig.6.6 show the maximum and residual (permanent) displacements of each model with and without P-Delta effect in N-S and E-W directions. Note that the results of N-S and E-W components of Los Gatos record shows that N-S component has much stronger responses than E-W component which can be seen on acceleration response spectra of the record as well.

Study of the results also shows that there is a major difference between the responses for maximum displacement (Fig.6.5) and residual displacement (Fig.6.6). Comparison of Frame "B" at east elevation and Frame "D" at west elevation shows that for the residual displacements, adding P-Delta effect to the system causes higher displacement responses as expected and correlates well with the results from Newhall records. But the displacements for maximum response are totally different and graphs with P-Delta effect have lower response than others. It seems the P-Delta effect shifts one of the torsional or translational responses peaks and they are not happening at the same time, which makes the total response lower. Besides the E-W component of Los Gatos has a much lower spectral acceleration response in compare to Newhall record. Displacement graphs for Frame "A" and "C" show that difference as well.

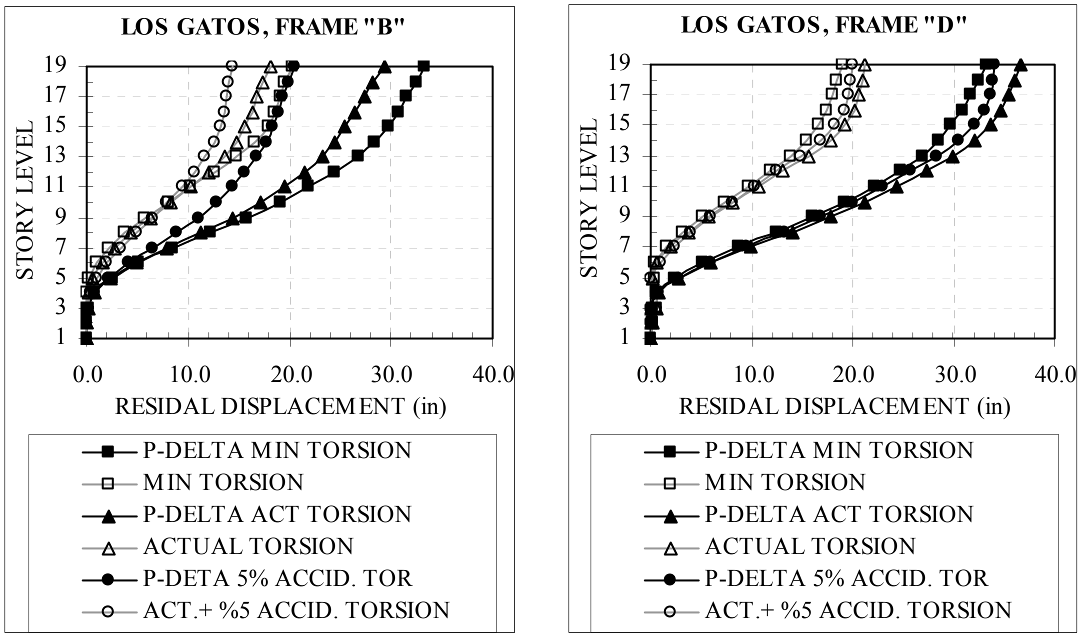


N-S: Frame-B and Frame-D

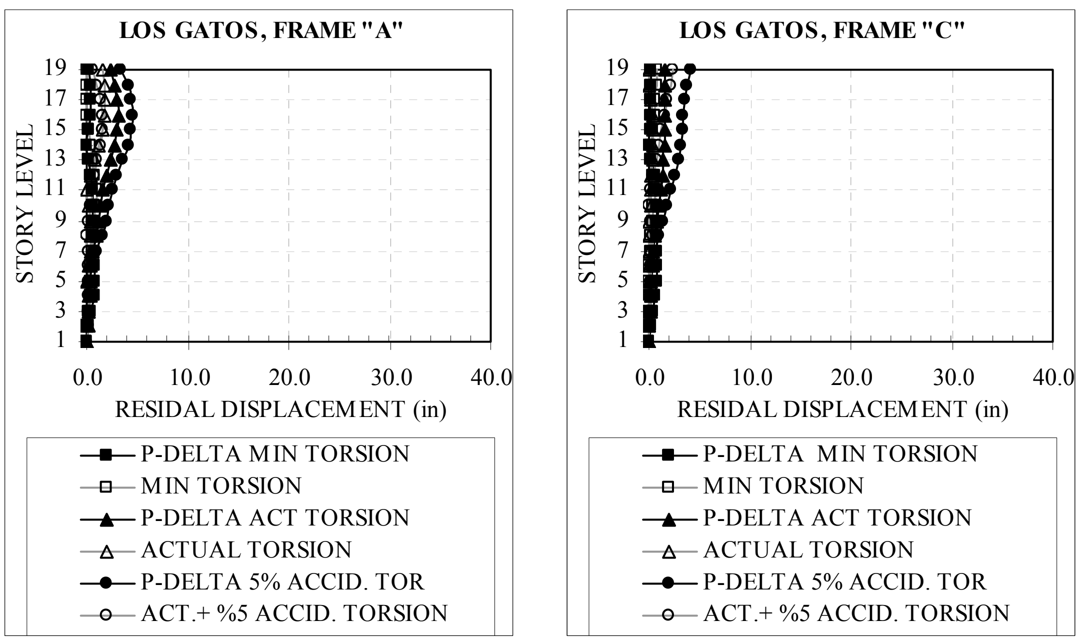


E-W: Frame-A and Frame-C

Figure 6.5 -MAX Displacement of Building, LOS GATOS Record



N-S: Frame-B and Frame-D



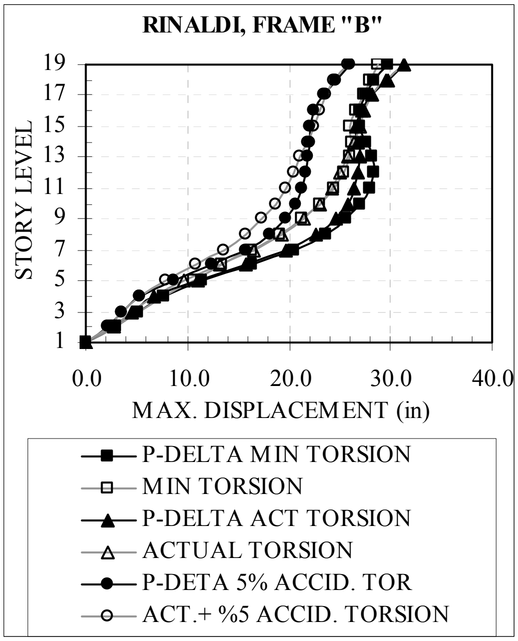
E-W: Frame-A and Frame-C

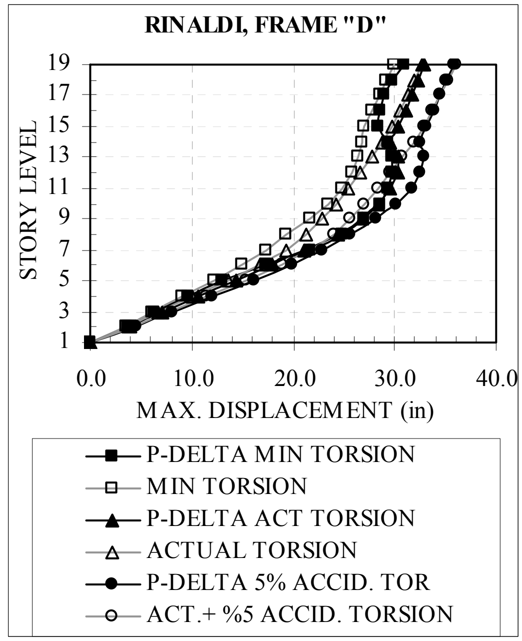
Figure 6.6 –Permanent Residual Displacement of Building, LOS GATOS Record

For the Rinaldi record, the set of graphs in Fig.6.7 and Fig.6.8 show the maximum and residual (permanent) displacements of each model with and without P-Delta effect in N-S and E-W directions.

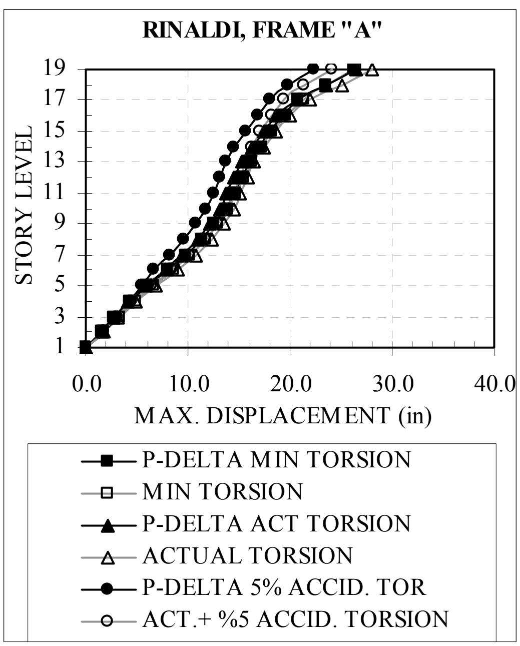
By comparing the results of each graph, the effect of P-Delta on maximum and residual displacement of each model can be studied. The graph for residual displacements of Frame "B" at east elevation and Frame "D" at west elevation shows that adding P-Delta effect to the system causes higher displacement responses for all floors as expected. The average changes for simultaneous components of Rinaldi record at the roof level are 20%, 26% and 30% for the residual displacement due to minimum torsion, actual torsion and actual plus 5% accidental torsion respectively. The maximum for the average changes of residual displacement happens at middle floors (6^{th} through 11^{th} floor) and reach to 76%, 82% and 82% due to minimum torsion, actual torsion and actual plus 5% accidental torsion respectively. But the graph for maximum displacements in N-S direction shows the different pattern and P-Delta has either negligible or much lower effect on maximum displacement in compare to residual displacement. The maximum for the average changes happens at middle floors (8^{th} through 11^{th} floor) and reach to 26%, 15% and 11% due to minimum torsion, actual torsion and actual plus 5% accidental torsion respectively.

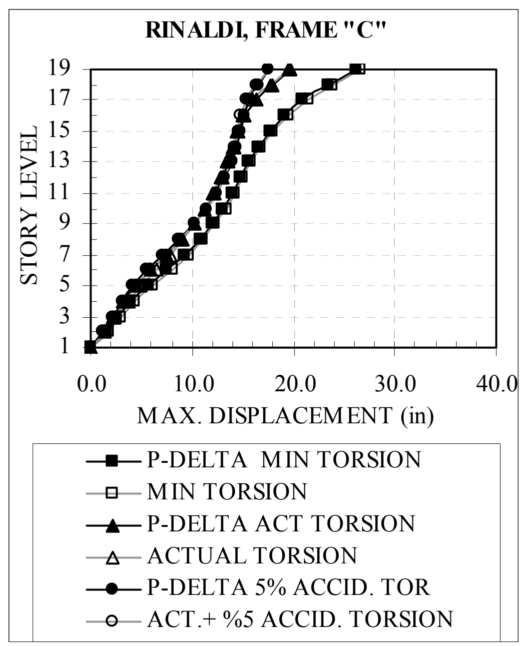
The simultaneous components of Rinaldi records has a much lower spectral acceleration response in E-W direction as compared to N-S direction and the maximum for the average changes of residual displacement only reaches to 35% for all three cases. The displacement graphs of Frame "A" and "C" show this difference.





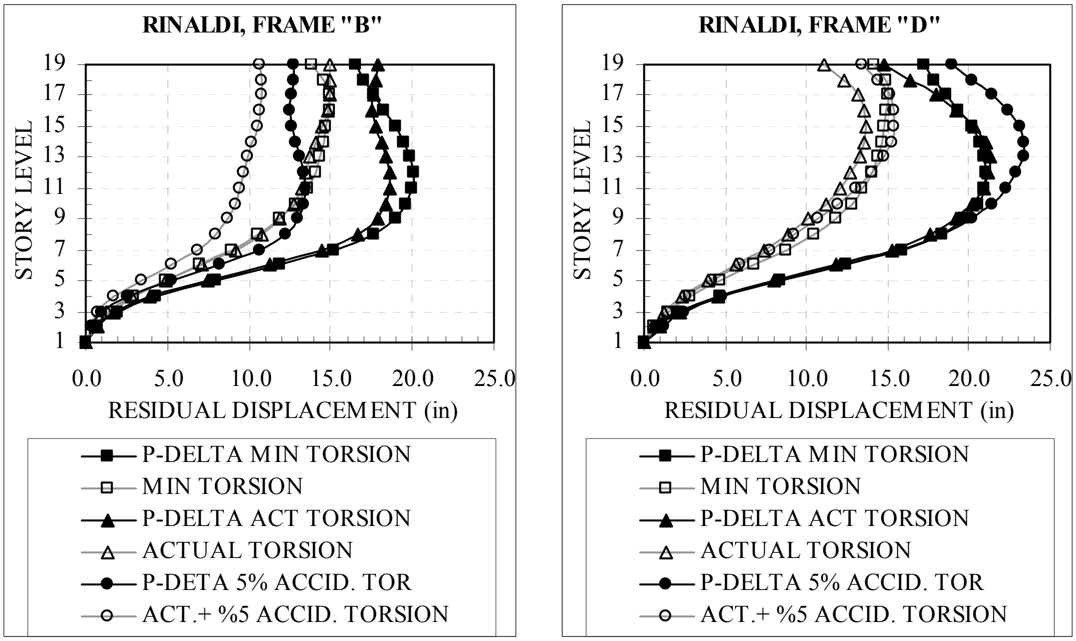
<u>N-S: Frame-B and Frame-D</u>



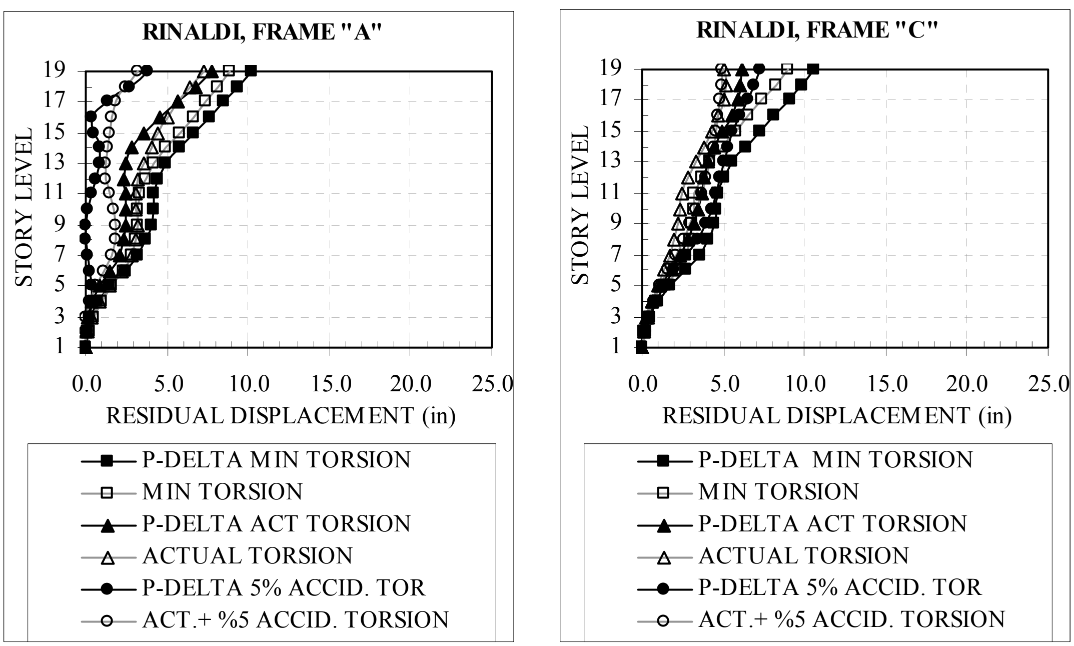


<u>E-W: Frame-A and Frame-C</u>

<u>Figure 6.7 -MAX Displacement of Building, RINALDI Record</u>



N-S: Frame-B and Frame-D



E-W: Frame-A and Frame-C

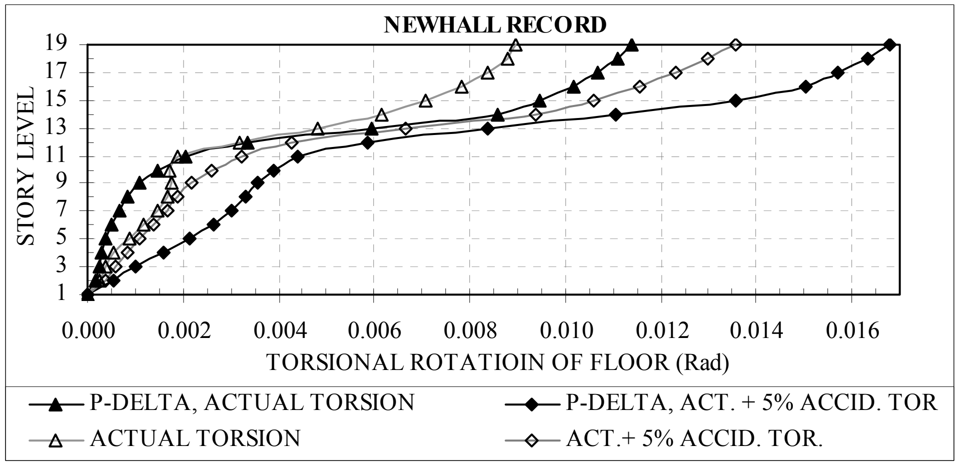
Figure 6.8 –Permanent Residual Displacement of Building, RINALDI Record

The next sets of graphs, Fig.6.9 through Fig.6.14, are considering the torsional rotation of each model and the effect of P-Delta. As of the displacement results, there is a good conformance between the maximum and permanent torsional rotations of the Newhall record and adding P-Delta effect to the system causes higher torsional response. The permanent torsional rotations of the Los Gatos and Rinaldi records have a good conformance for each of the models as well; however the maximum torsional rotations of Los Gatos and Rinaldi are different.

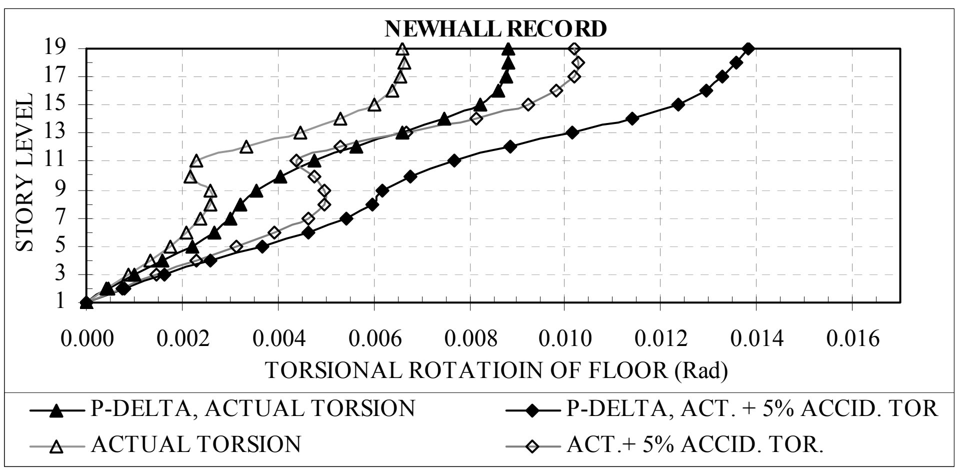
For the Newhall record, maximum torsional differential rotation of building for Frames "A&C" and "B&D" shows increase from 5 to 50 percent due to P-Delta effect which is a significant increase. This increase to the torsional rotation is about 20 to 25 percent for top 7 stories and decrease to about 5 to 10 percent for lower stories of Frame "B&D" and increase to about 40 to 50 percent for lower stories of Frame "A&C" (Fig.6.9).

For permanent torsional differential rotation of the building, Frames "A&C" and "B&D" show increases from 35 to 80 percent due to P-Delta effect which is a very significant. This increase to the torsional rotation is about 35 to 40 percent for top 7 stories and increases to about 80 percent for lower stories of Frames "A&C" and to about 65 percent of Frames "B&D" (Fig.6.10).

For Newhall record, The increase of torsional rotation at roof for actual torsion case due to P-Delta effect is about to 25.1 and 34.2 percent for maximum and residual torsional rotations respectively and for actual plus 5% accidental torsion case is about 26.1 and 32.5 percent for maximum and residual torsional rotations.

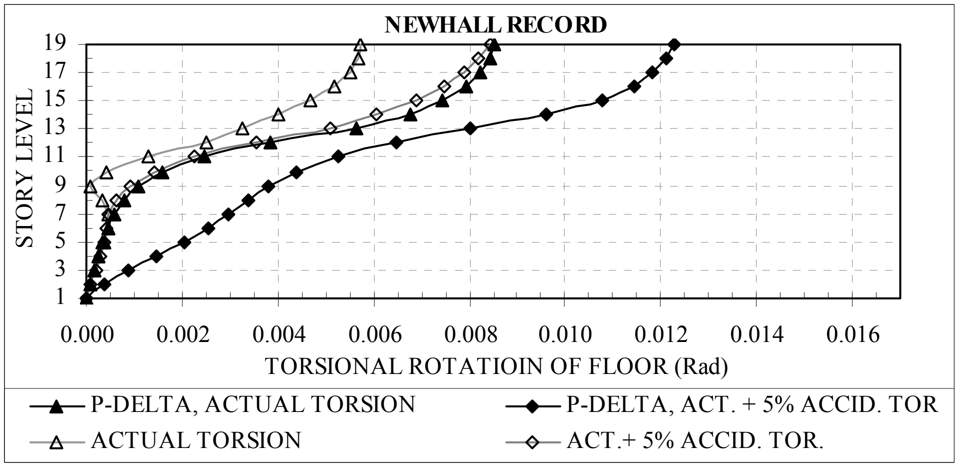


E-W: FRAME A & C

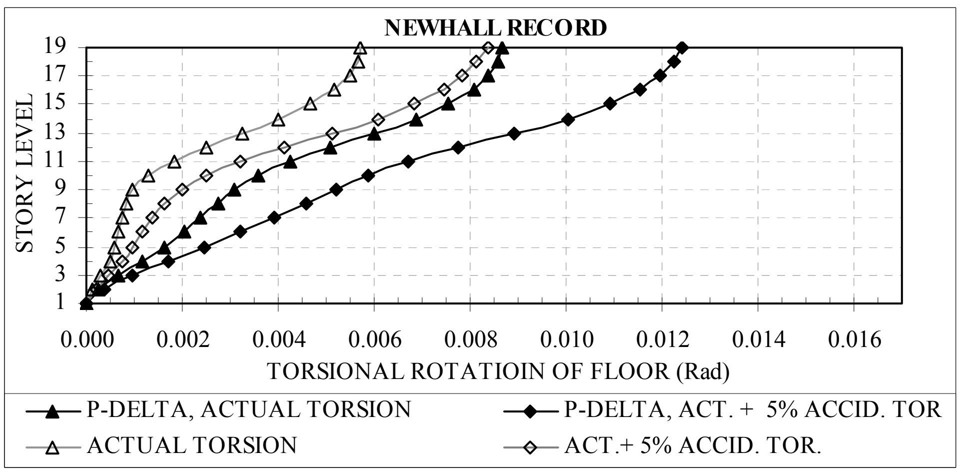


N-S: FRAME B & D

Figure 6.9 –Comparison of the Torsional Rotation of Building, NEWHALL Record



E-W: FRAME A & C

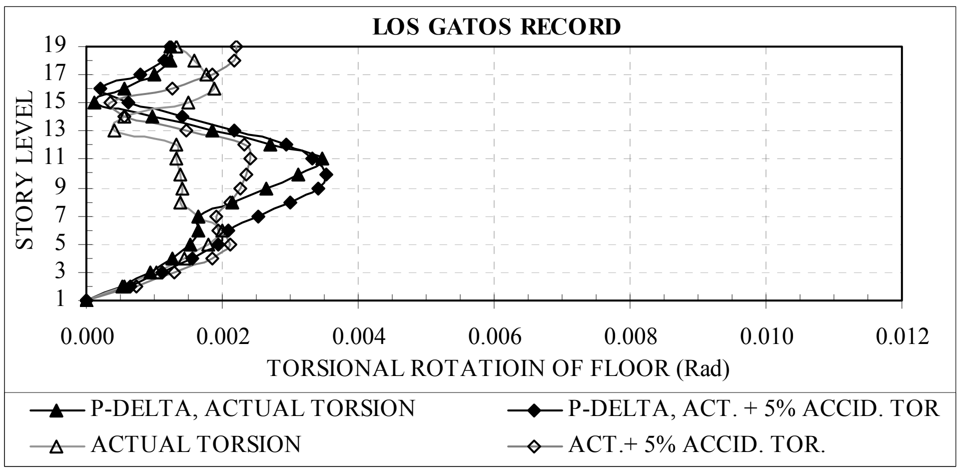


N-S: FRAME B & D

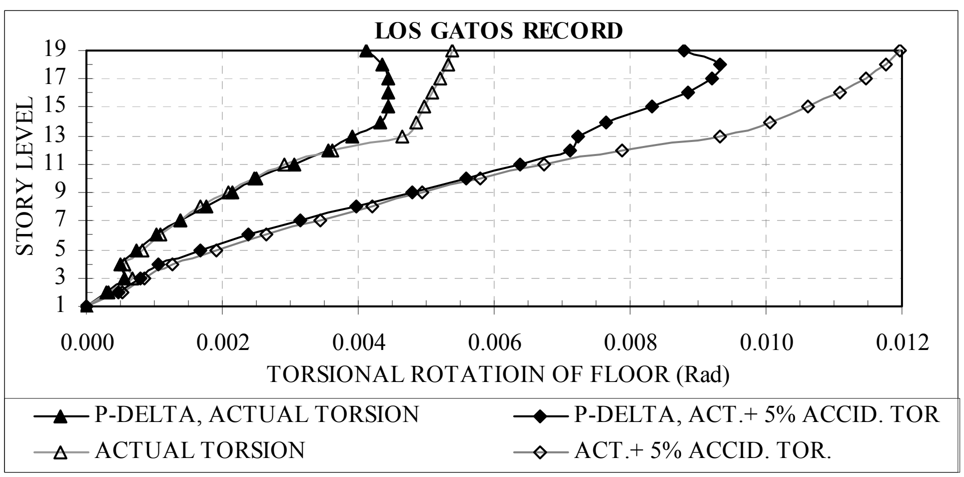
Figure 6.10 –Comparison of the Torsional Rotation of Building Based on Permanent Residual Displacement, NEWHALL Record

For the Los Gatos record, instant maximum and permanent torsional rotation of building for Frames "A&C" is low for all cases (Fig.6.11 and 6.12). That shows the pick rotational torsion of each floor does not happen at the maximum pick displacement of frames "A" and "C". However, for each floor diaphragm, the pick torsional rotation can be measured through frames "B" and "D".

For Frames "B&D", unlike Newhall record, the maximum torsional rotation decrease for all stories due to P-Delta effect. That could be happened because the pick torsional rotation is a function of interaction among P-Delta effect, seismic ground motion and frame displacement, it is possible that P-Delta acts against initial deflection and reduces the pick torsional rotation.



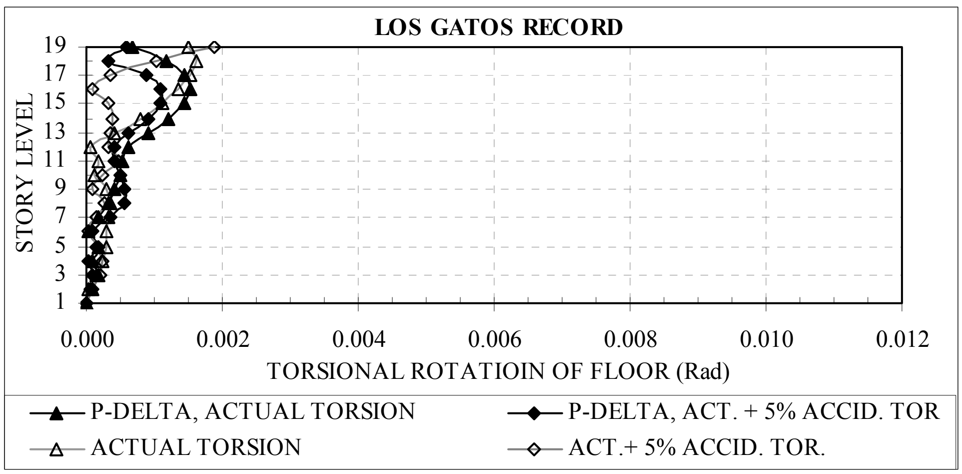
E-W: FRAME A & C



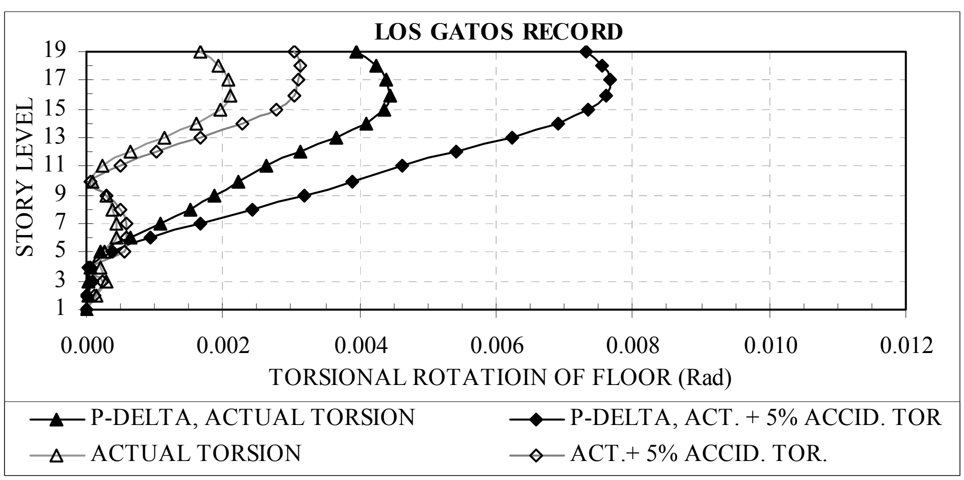
N-S: FRAME B & D

Figure 6.11 –Comparison of the Torsional Rotation of Building, Los Gatos Record

However; the permanent torsional differential rotation of building shows the increase of about 40 to 90 percent from 5th floor to the roof and the difference between Actual Torsion model and Actual plus 5% Accidental model is minor. The lower 5 floors have almost no torsional rotations (Fig.6.11 and 6.12). Torsional rotation increase for the roof is about to 58 percent for both actual torsion and actual plus 5% accidental torsion cases for permanent residual torsional rotation.



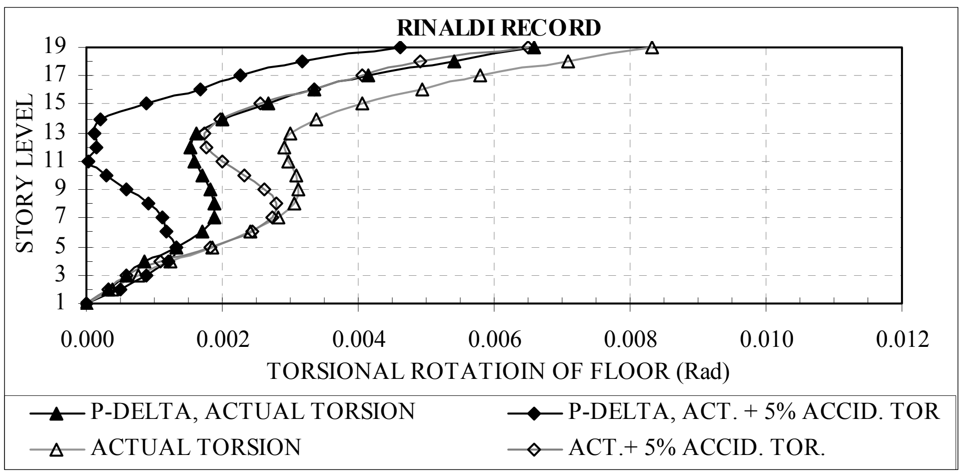
E-W: FRAME A & C



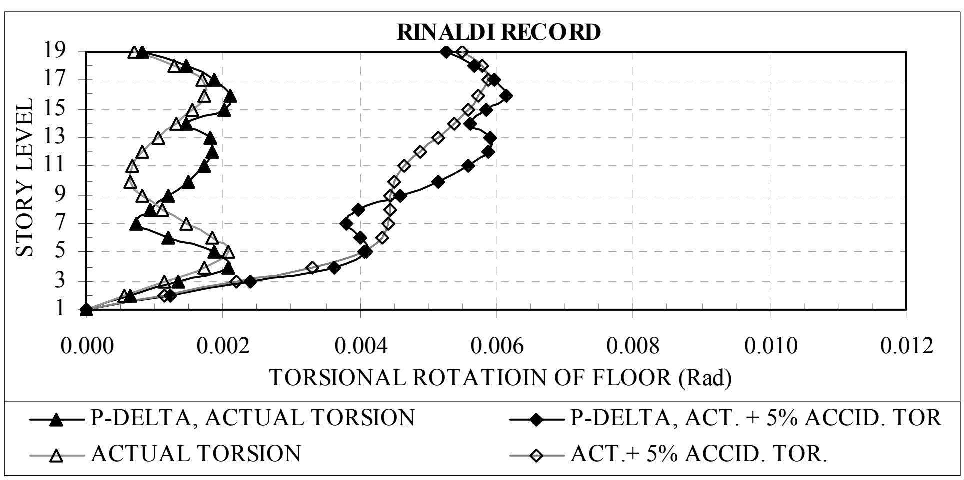
N-S: FRAME B & D

Figure 6.12 –Comparison of the Torsional Rotation of Building Based on Permanent Residual Displacement, Los Gatos Record

For the Rinaldi record, maximum torsional rotation graph of building for Frames "A&C" shows decrease for all stories due to P-Delta effect and as it discussed before, that could be happened because it is possible that P-Delta acts against initial deflection and reduces the pick torsional rotation. This rotation for Frames "B&D" is only negative for 5th through 8th floors and for the rest of the floors varies from 10 to 60 percent (Fig.6.13).



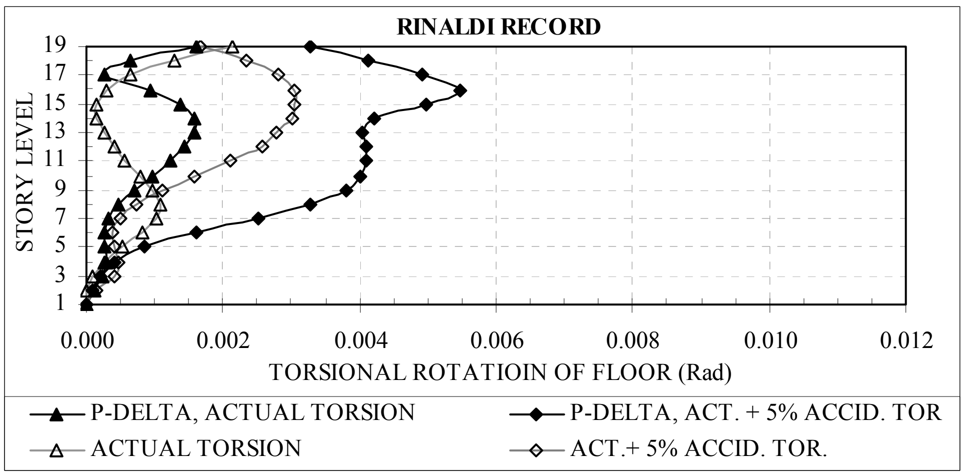
E-W: FRAME A & C



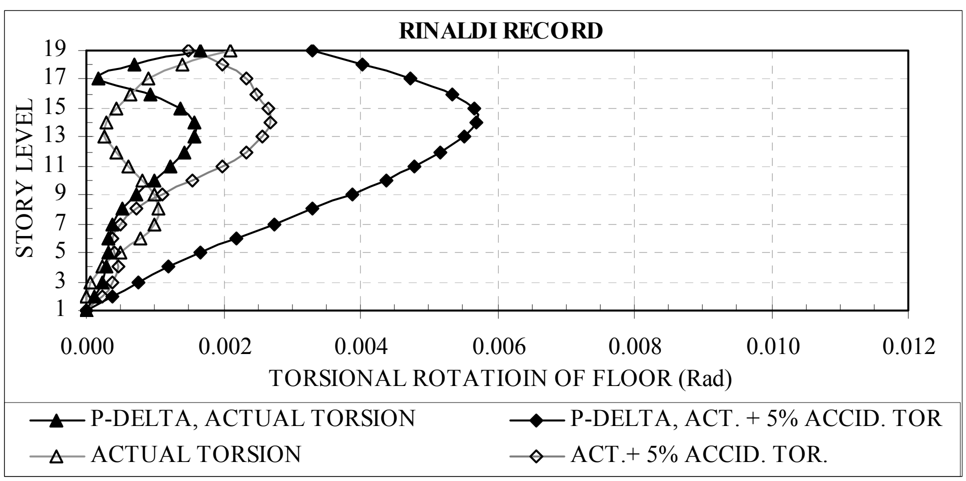
N-S: FRAME B & D

Figure 6.13 –Comparison of the Torsional Rotation of Building, RINALDI Record

The permanent torsional differential rotation of building shows about 20 to 90 percent increase from 10th floor to 16th floor for all frames and the difference between actual torsion and actual plus 5% accidental torsion models is major and adding 5% accidental torsion to the model causes a shift to the peak rotation from 7th floor to 14th floor. The lower 5 floors have almost no torsional rotations (Fig.6.14).



E-W: FRAME A & C



N-S: FRAME B & D

Figure 6.14 –Comparison of the Torsional Rotation of Building Based on Permanent Residual Displacement, RINALDI Record

Figures 6.15 through 6.17 present the plastic hinge mechanism of the building for the Canoga record for all models and show the effect of torsion and P-Delta on plastic hinge rotation. Comparison of different mechanisms shows that there is no plastic hinge mechanism for Frame "A" and "C" and they remain elastic for all different cases. As it mentioned before, the N-S component of Canoga record has much stronger responses than E-W component and as a result, plastic hinge mechanism on west and east elevations frames are more serious than frames on the other elevations. However adding torsion to the system causes an increase to the total (elastic plus plastic) hinge rotation of 4th floor through 7th floor and 13th floor through 15th floor of Frame "D" and a decrease to the total hinge rotation of 7th floor through 12th floor of Frame "B". Adding P-Delta to the system does not have a significant effect on upper floor and only increase total hinge rotation of lower floors (3rd through 6th floors).

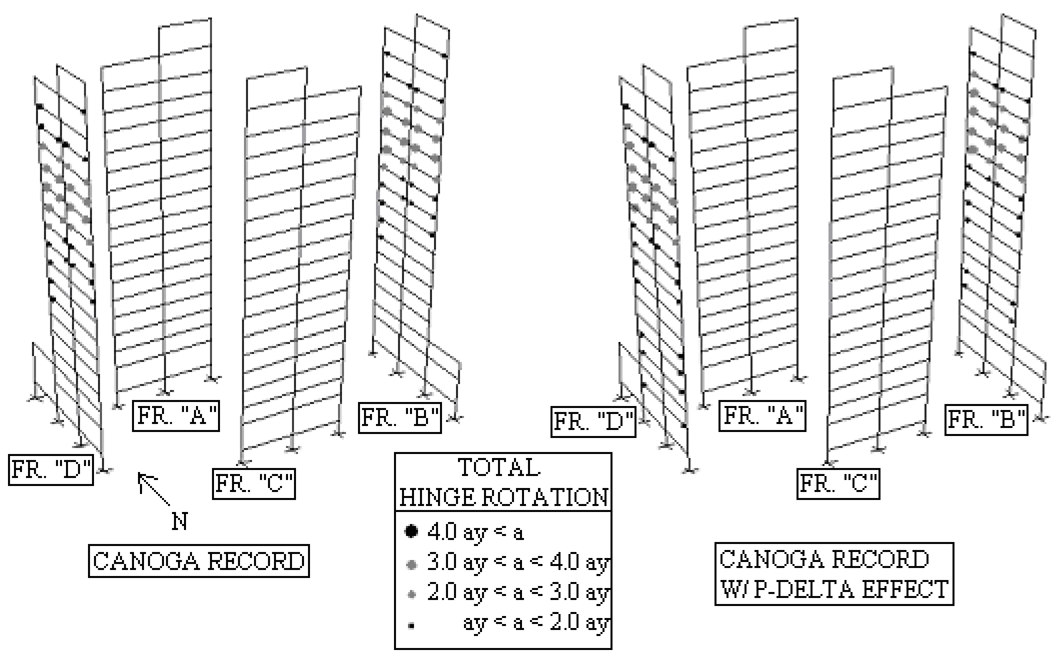


Figure 6.15 –Total Hinge Rotation, Minimum Torsion Case, Canoga Record

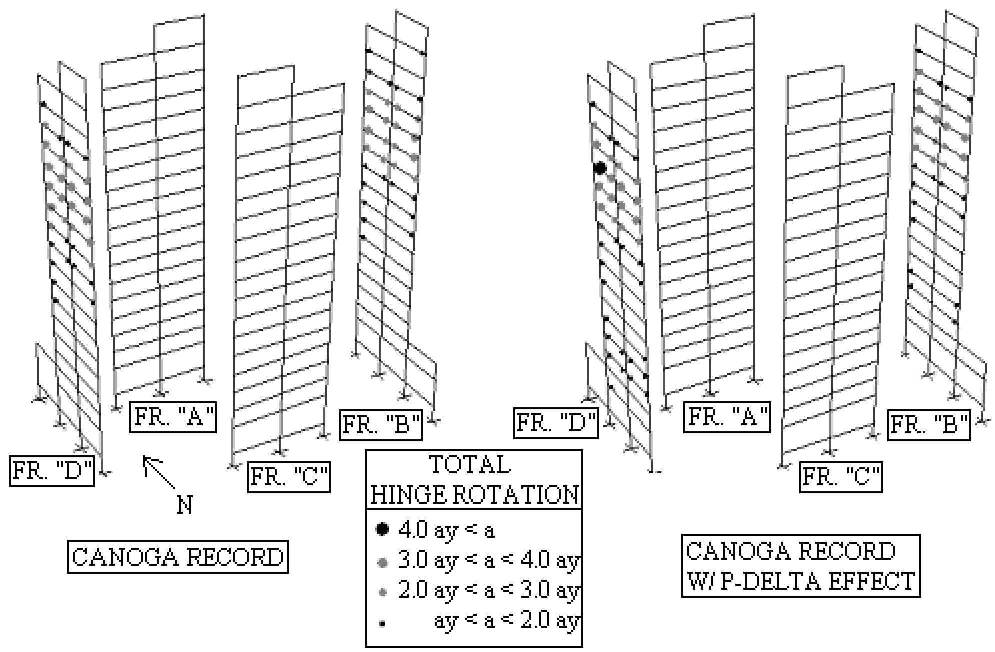


Figure 6.16 –Total Hinge Rotation, Actual Torsion Case, Canoga Record

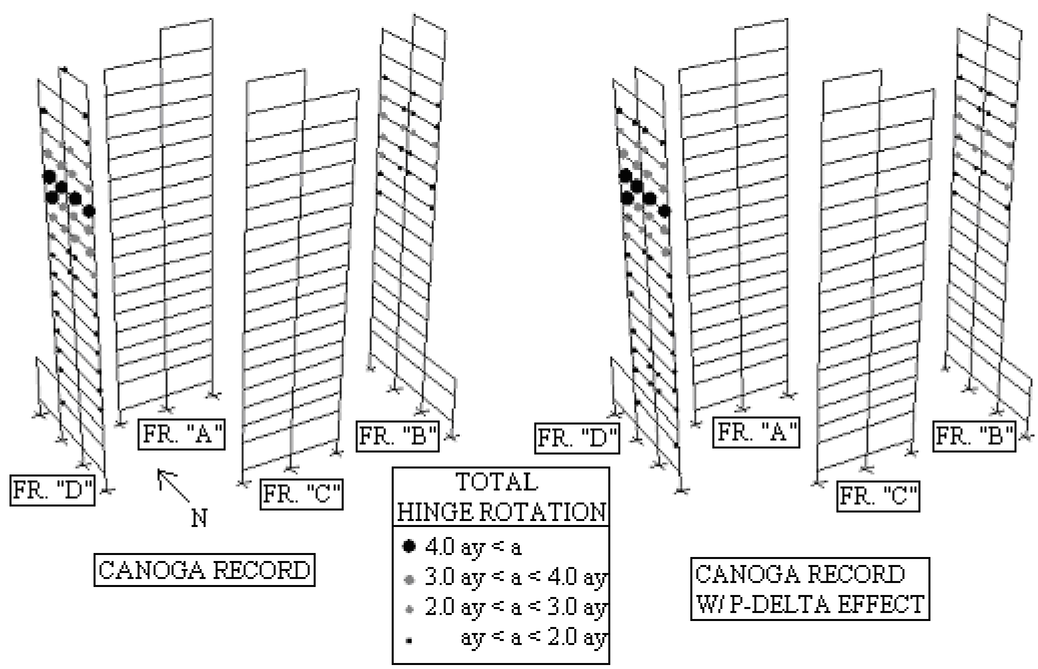


Figure 6.17 –Total Hinge Rotation of Frames, Actual Plus 5% Accidental Torsion Due to Eccentricity at Both Directions, Canoga Record

Comparison of actual damage happened to the building during Northridge earthquake (Fig.6.18) with plastic hinge mechanism for Canoga record with and without P-Delta shows a very good agreement for frame "D" (Fig. 6.16 and 6.17).

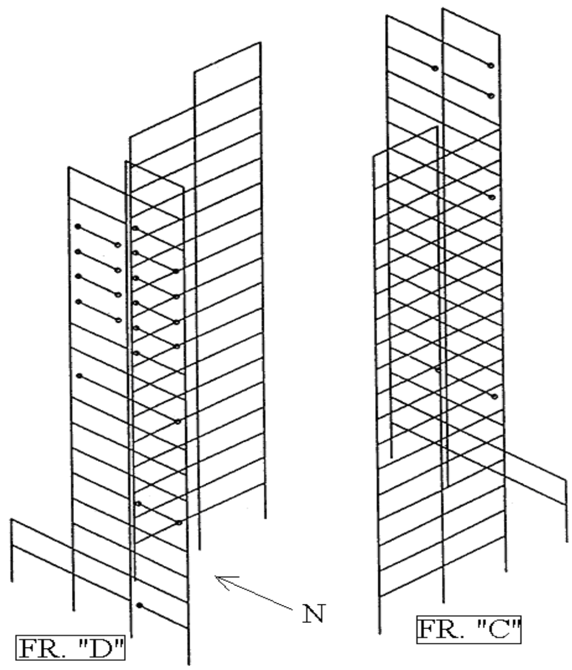


Figure 6.18 - Actual Location of Weld Failures

Figures 6.19 through 6.27 give the plastic hinge mechanism of the building for all other three records and show the effect of P-Delta on the plastic hinge rotation. Comparison of different mechanisms shows that there is not a significant increase due to P-Delta effects on frames "A" and "C" for any of these records.

Figures 6.19 through 6.21 show the plastic hinge mechanism of Newhall record for all three models and adding P-Delta to the model causes an increase of total hinge rotation in the 3rd through 7th floors and a decrease of total hinge rotation in the16th floor through the roof for frames "B" and "D".

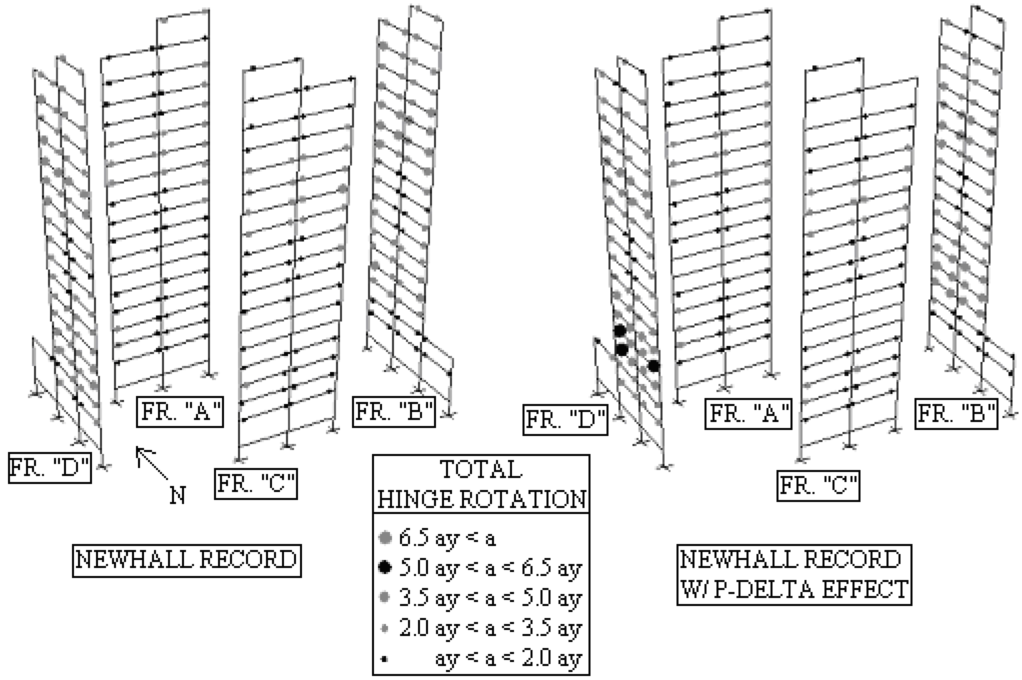


Figure 6.19 –Comparison between Total Hinge Rotations of Frames, Minimum Torsion Case, Newhall Record

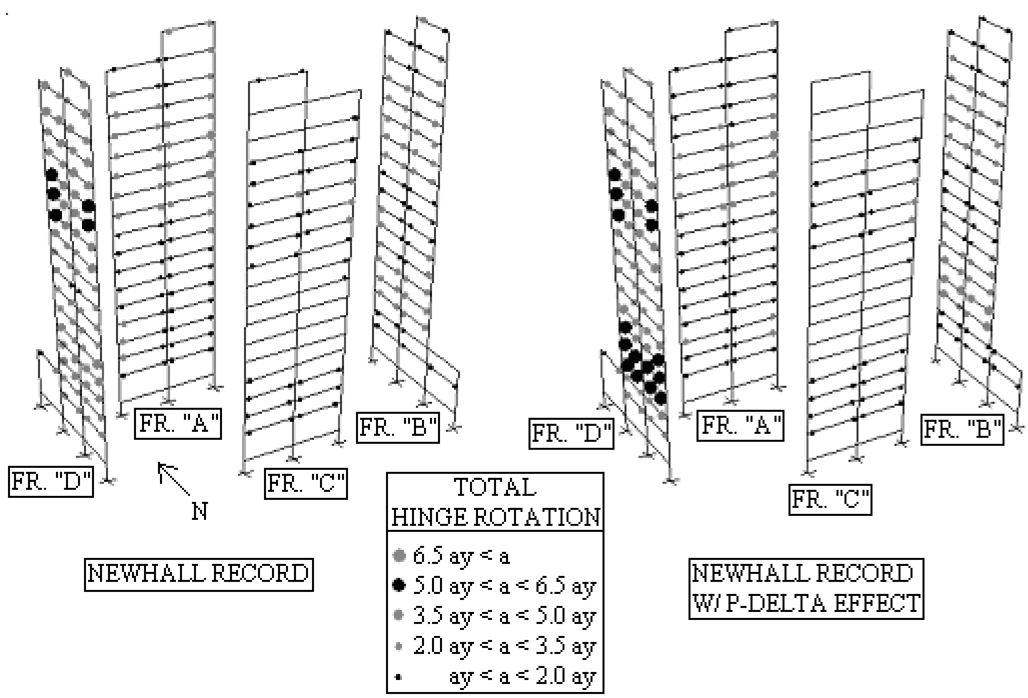


Figure 6.20 – Comparison between Total Hinge Rotations of Frames, Actual Torsion Case, Newhall Record

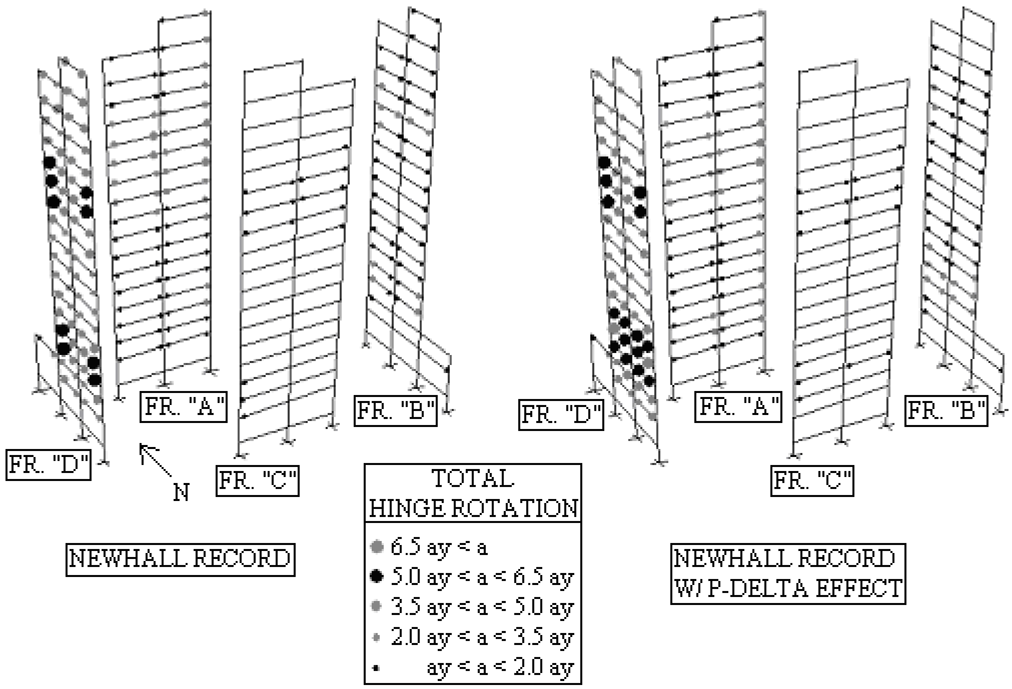


Figure 6.21 – Comparison between Total Hinge Rotations, Actual Plus 5% Accidental Torsion Due to Eccentricity at Both Directions, Newhall Record

Figures 6.22 through 6.24 are dealing with the plastic hinge mechanism of the building for Los Gatos record for all models and show the effect of P-Delta on plastic hinge rotation. For Los Gatos record, there is an increase for the 5th through 9th floors and a decrease for the 17th floor through the roof for frames "B" and "D". For Actual plus 5% accidental torsion Model (Fig.6.24), P-Delta has a negative effect on total hinge rotation of 10th floor through 12th floor of Frame "D" and decrease the amount of plastic hinge rotation.

The plastic hinge mechanism of Rinaldi record shows an increase for 3rd through 7th floor and a decrease for 9th through 13th floor for frames "B" and "D" (Figures 6.25 through 6.27). For Actual plus 5% accidental torsion Model (Fig.6.27), P-Delta has a considerable effect on total hinge rotation of lower floor columns.

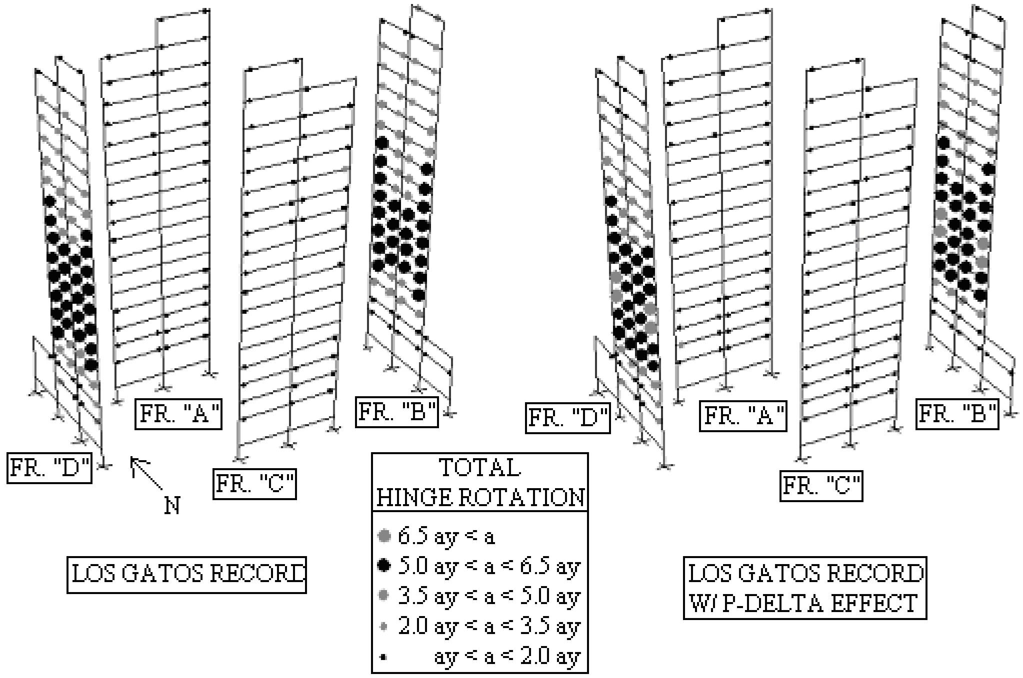


Figure 6.22–Comparison between Total Hinge Rotations of Frames, Minimum Torsion Case, Los Gatos Record

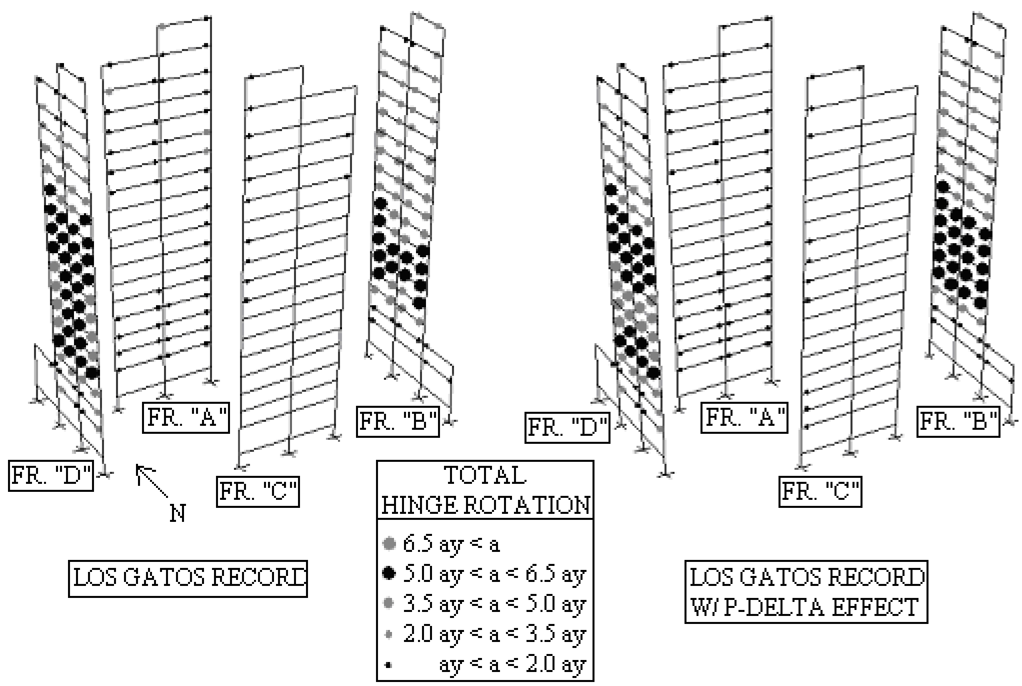


Figure 6.23 – Comparison between Total Hinge Rotations of Frames, Actual Torsion Case, Los Gatos Record

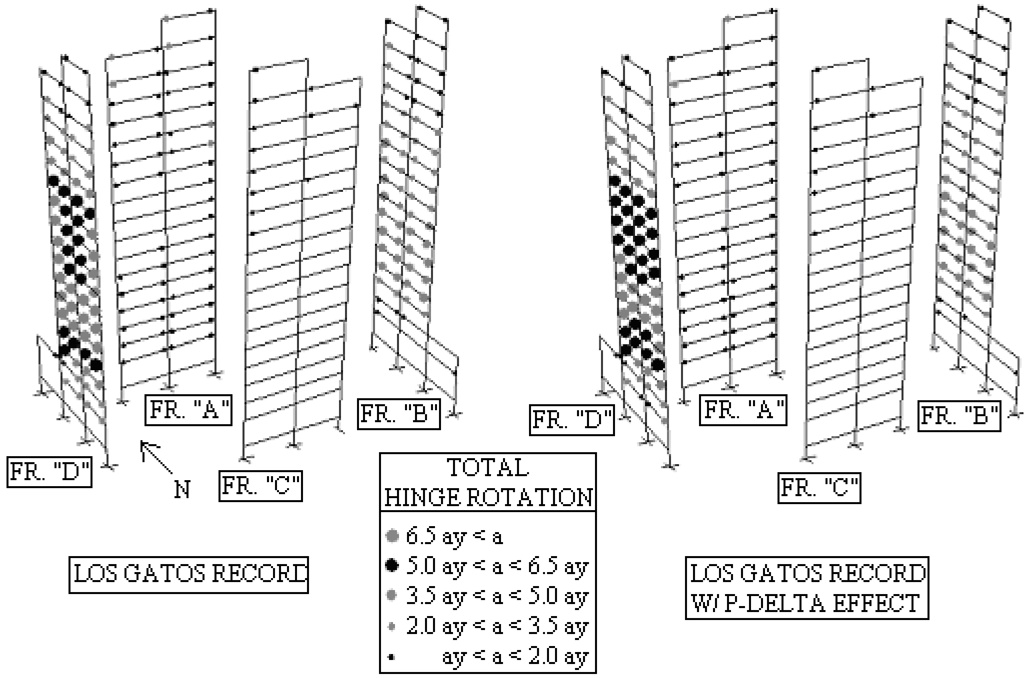


Figure 6.24 – Comparison between Total Hinge Rotations of Frames, Actual Plus 5% Accidental Torsion Due to Eccentricity at Both Directions, Los Gatos Record

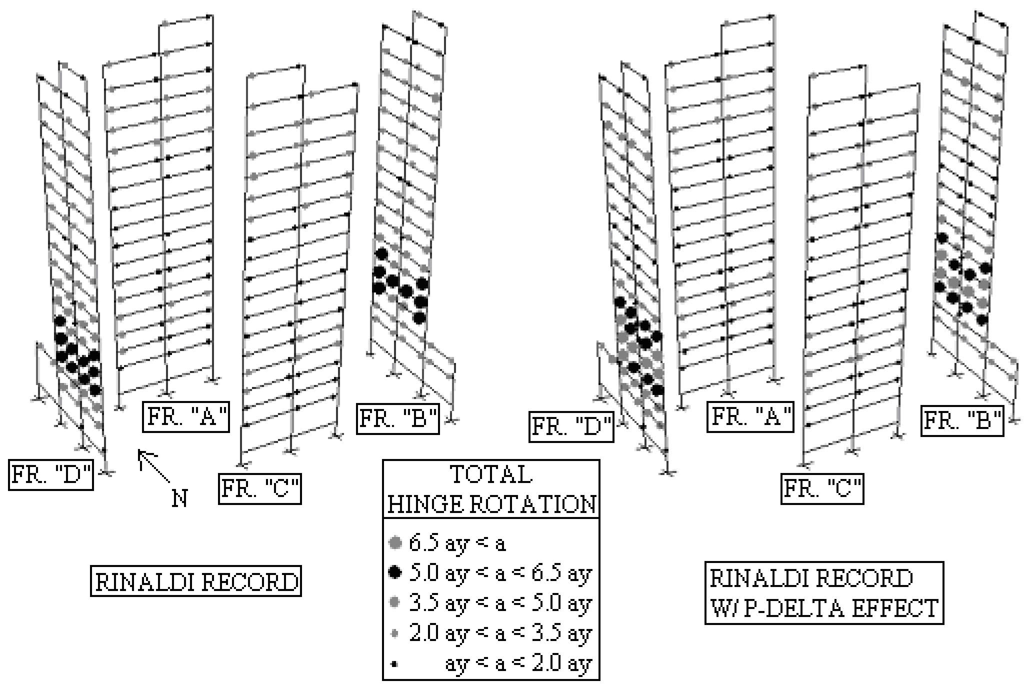


Figure 6.25–Comparison between Total Hinge Rotations, Minimum Torsion Case, Rinaldi Record

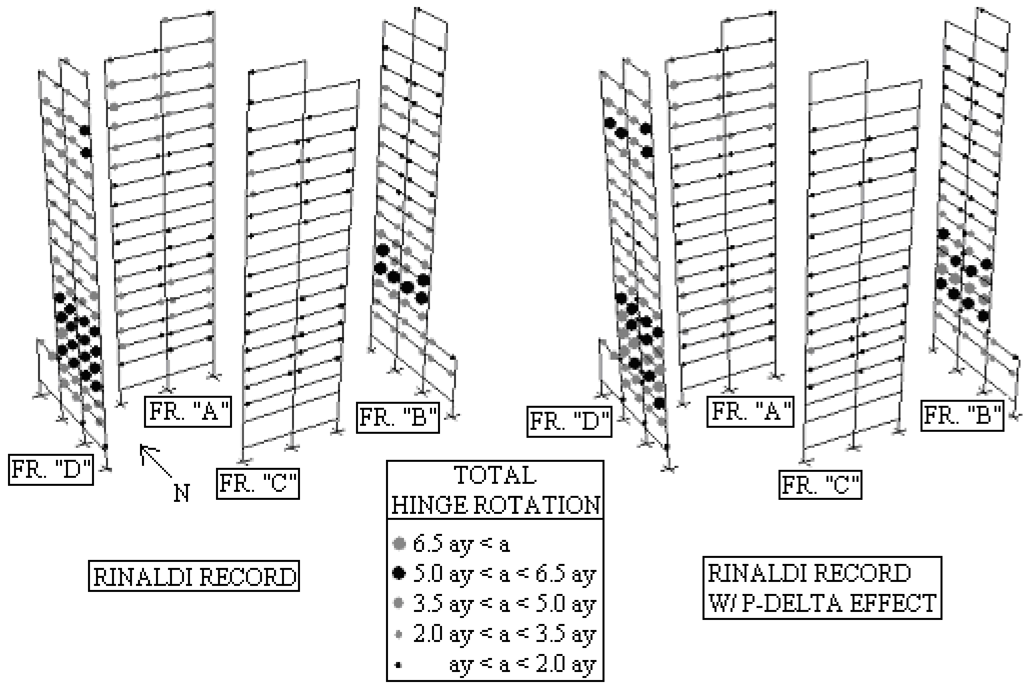


Figure 6.26 – Comparison between Total Hinge Rotations of Frames, Actual Torsion Case, Rinaldi Record

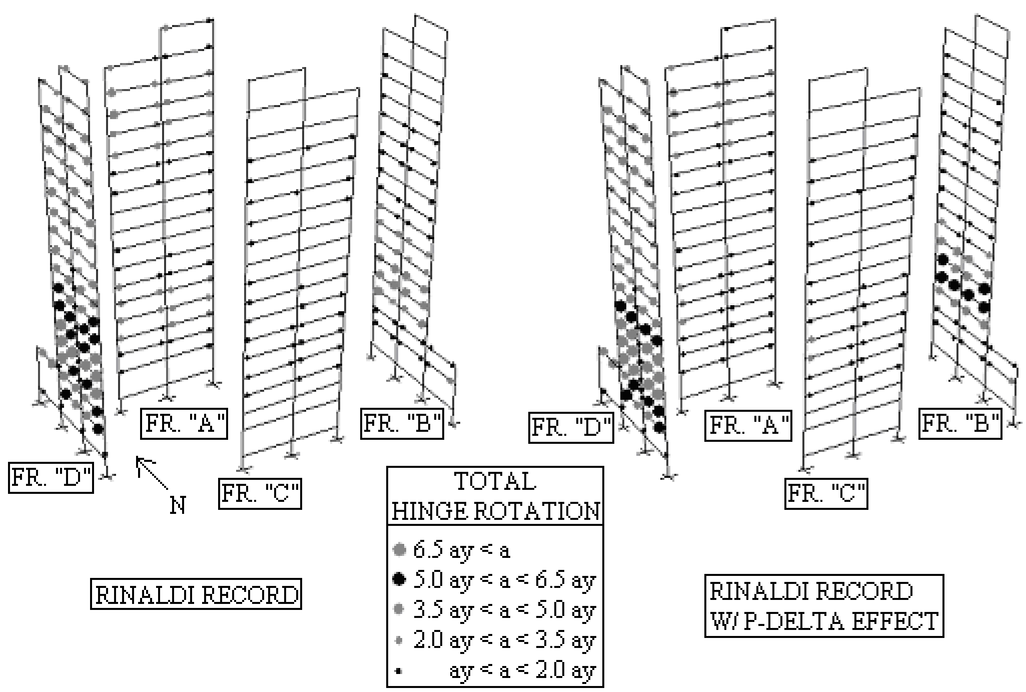


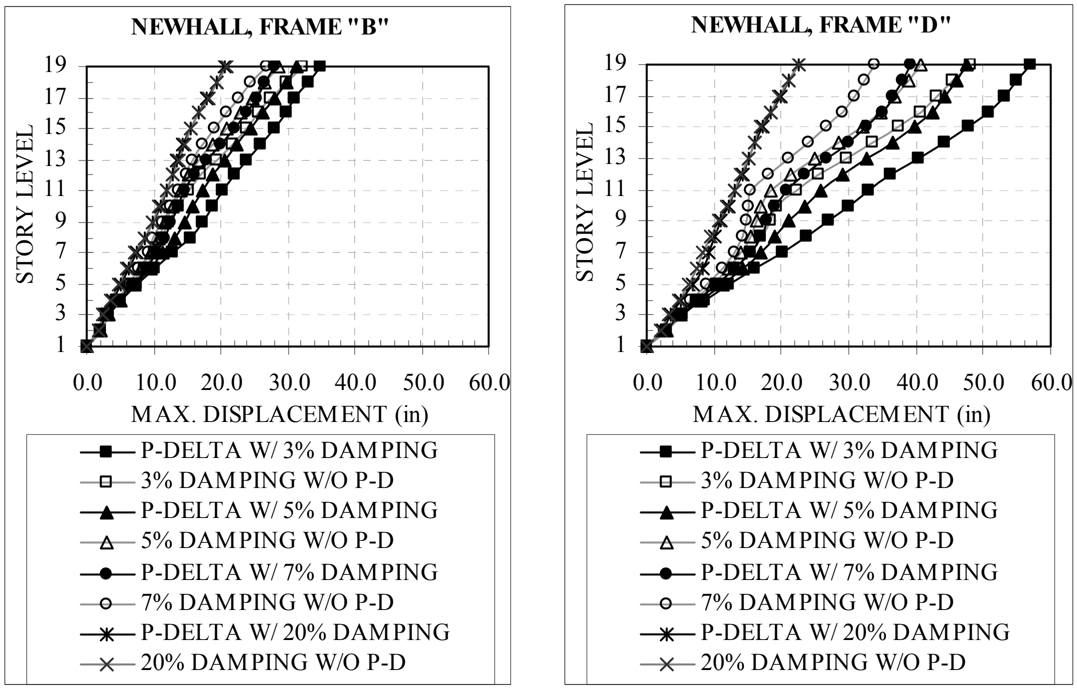
Figure 6.27 – Comparison between Total Hinge Rotations of Frames, Actual Plus 5% Accidental Torsion Due to Eccentricity at Both Directions, Rinaldi Record

D. Effect of Damping

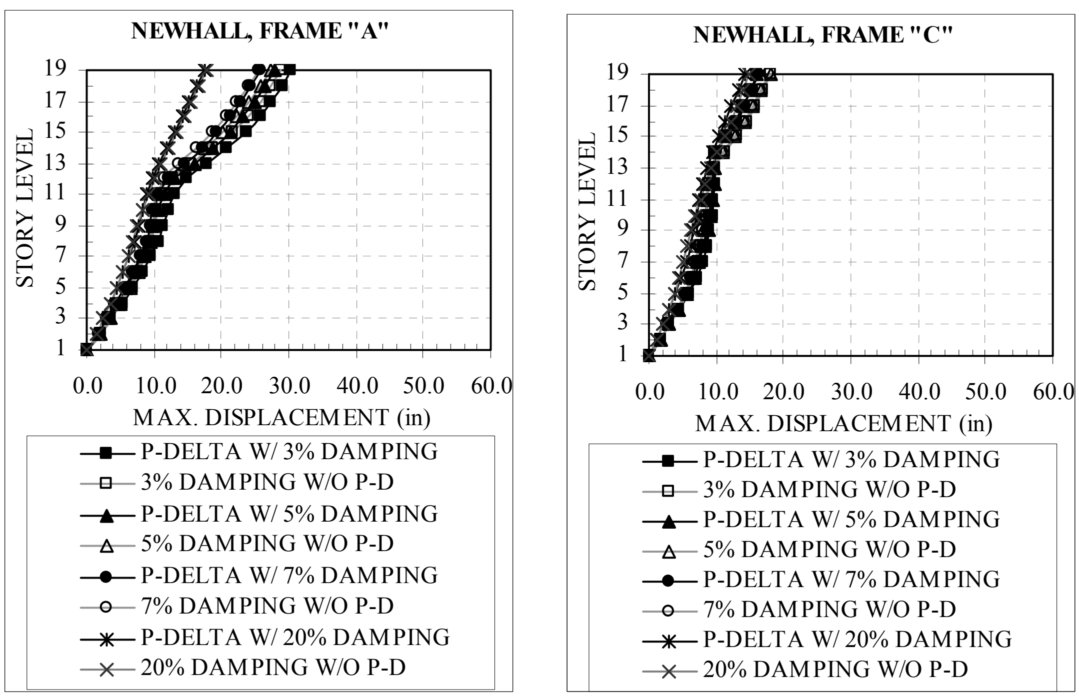
The next series of graphs in Fig.6.28 through Fig.6.31 consider the Actual Torsion model with different amounts of critical damping ratios of 3%, 5%, 7% and 20% utilizing the Newhall record and its effect on displacement and torsional rotation of the building. The first set of graphs shown in Fig.6.28 and Fig.6.29 consider the maximum and residual (permanent) displacements of the model with and without P-Delta effect in N-S and E-W directions.

Beginning with the Frame "B" at east elevation and Frame "D" at west elevation, the maximum and residual displacements are following similar pattern and by increasing the damping, displacements of the frames decrease as expected. In addition, analyses with P-Delta have an increased displacement. The difference between analysis response with and without P-Delta is significant and varies from 20 to 40 percent for maximum and residual displacement respectively. Comparison among analyses with different amount of damping ratio shows that by increasing the damping, P-Delta effect reduces and even reaches to zero for damping ratio of 20% for maximum displacement graphs. The displacements in frames "A" and “C” do not show much difference by changing the damping ratios from 3% to 7% and only for frame "A" with 20% damping ratio, the change is considerable.

Second set of graphs in Fig.6.30 and Fig.6.31 are dealing with the maximum and residual torsional rotation of the model with and without P-Delta effect in N-S and E-W directions. The results show a good conformance between instant maximum and permanent torsional rotations for different damping ratios.

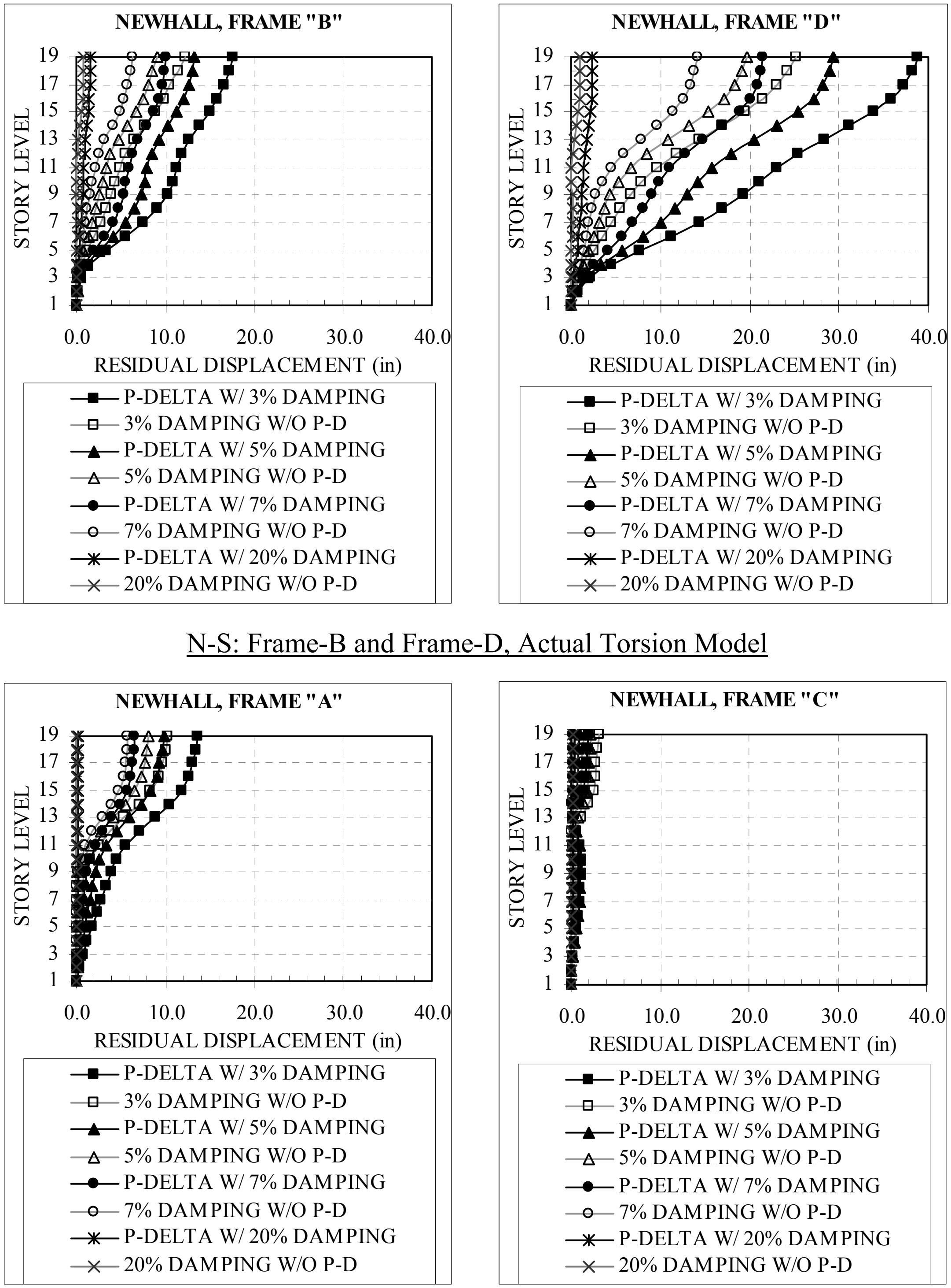


N-S: Frame-B and Frame-D, Actual Torsion Model



E-W: Frame-A and Frame-C, Actual Torsion Model

Figure 6.28 -MAX Displacement of Building in N-S and E-W Directions for Different Percentage of Damping, Newhall Record

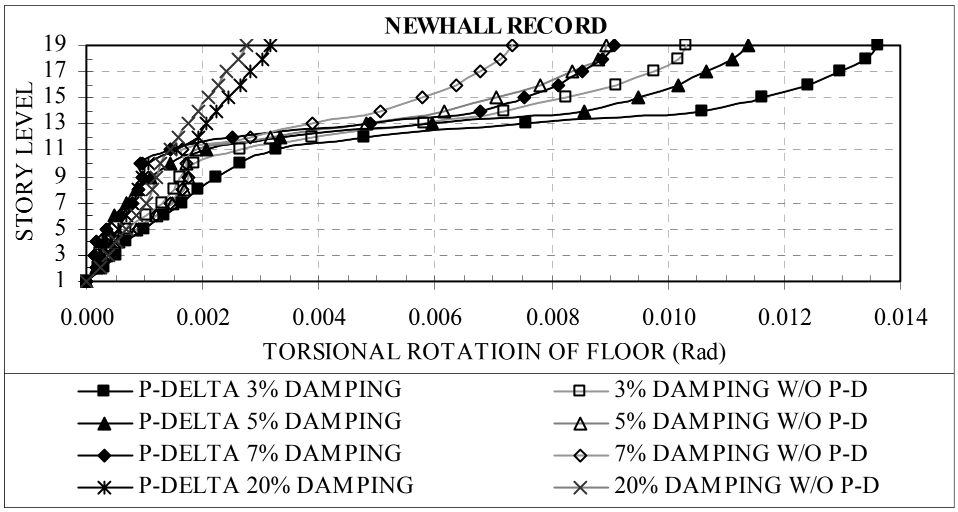


N-S: Frame-B and Frame-D, Actual Torsion Model

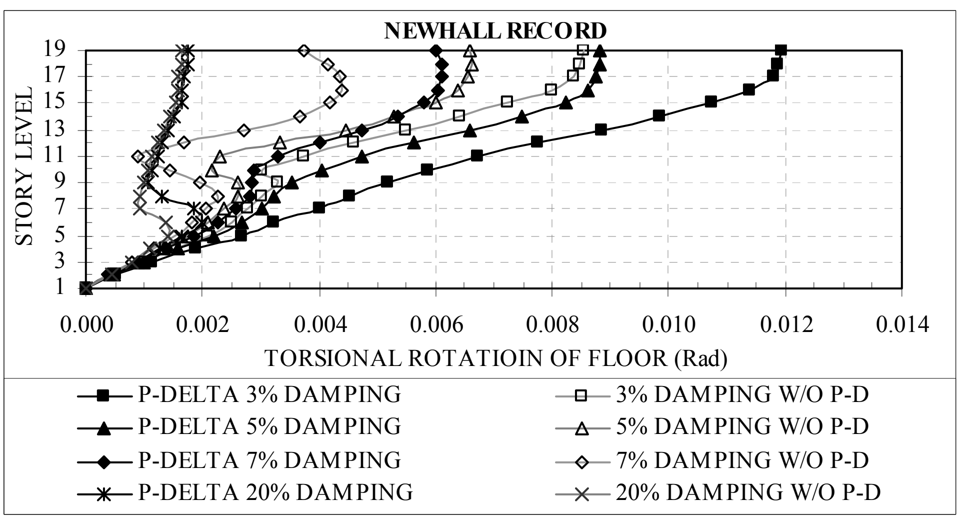
E-W: Frame-A and Frame-C, Actual Torsion Model

Figure 6.29 –Residual (Permanent) Displacement of Building in N-S and E-W Directions for Different Percentage of Damping, Newhall Record

The maximum torsional differential rotation of building for Frames "B&D" shows average increase of 30 percent for first three cases and about 6 percent for 20% damping case due to P-Delta effect, however this increase to the rotation for frames "A&C" reaches to 25 percent and happened only in the eight top stories.



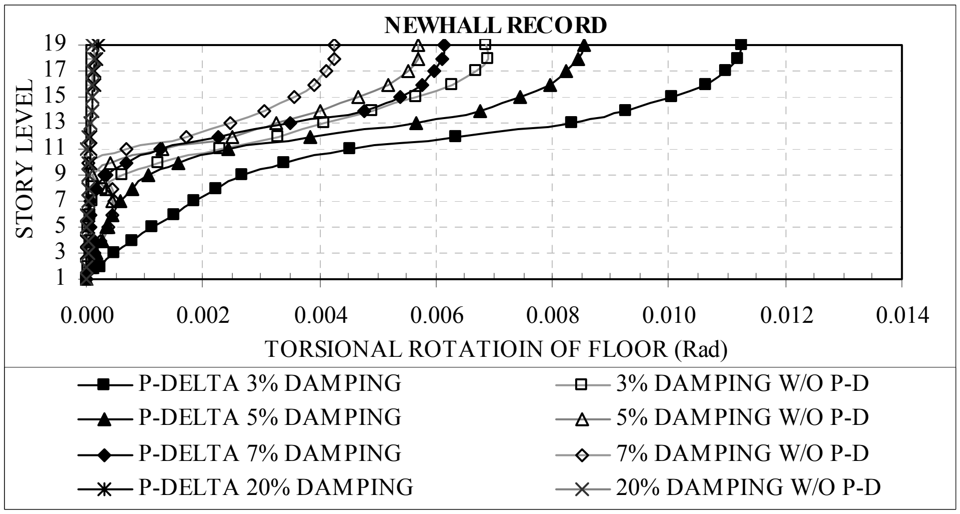
FRAME A & C, Actual Torsion Model



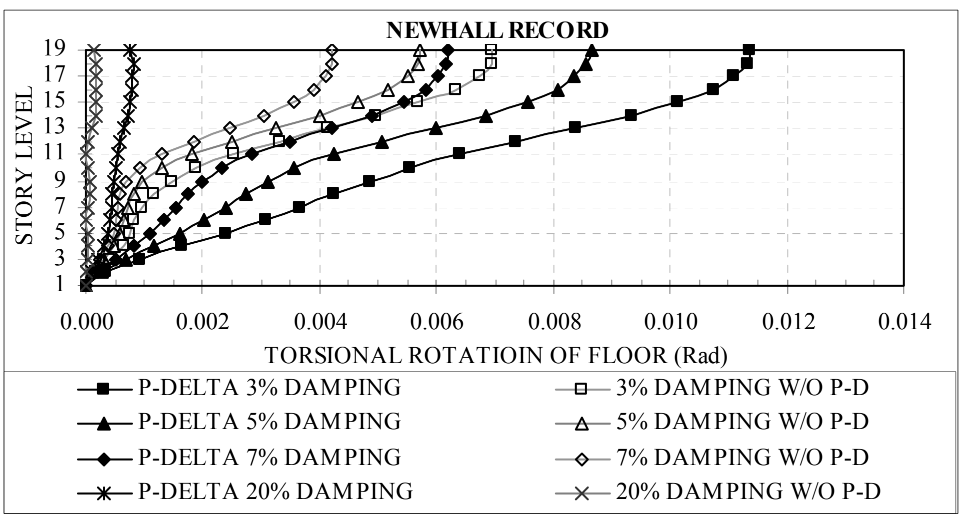
FRAME B & D, Actual Torsion Model

Figure 6.30–Comparison of the Torsional Rotation of Building Based on Maximum Displacement for Different Percentage of Damping, Newhall Record

For permanent torsional differential rotation of building, Frames "A&C" and "B&D" shows increase from 35 to 60 percent for first three cases due to P-Delta effect which is again, a very significant increase. This increase to the torsional rotation for 20% damping case for Frames "A&C" is not considerable.



FRAME A & C, Actual Torsion Model



FRAME B & D, Actual Torsion Model

Figure 6.31 –Comparison of the Torsional Rotation of Building Based on Permanent Displacement for Different Percentage of Damping, NEWHALL Record

Chapter 7: The 1997 Uniform Building Code Requirements for Torsional Design of Low and High Rise Buildings

A. Design Criteria

The 1997 Uniform Building Code [29] requires the following to consider torsion:

"For horizontal distribution of shear at non-flexible diaphragms, the mass at each level shall be assumed to be displaced from the calculated center of the mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered. Provisions shall be made for the increased shears resulting from horizontal torsion of non-flexible diaphragm. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral force at levels above that story and the vertical-resisting elements in that story plus an accidental torsional moment which shall be determined by assuming the mass is displaced a distance equal to 5 percent of plan dimension as mentioned above. Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of structure. In this case, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor A_x, determining from the following formula:

$$A_x = [\delta_{max} / 1.2\delta_{avg}]^2 < 3.0$$

δ_{avg}: The average of the displacements at the extreme points of the structure at level x

δ_{max}: The maximum displacement at level x

Besides, in seismic zones 2, 3 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes that is called the orthogonal effects. This requirement may be satisfied by applying 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic force in the perpendicular direction."

The SEAOC Blue Book code [30] clarifies the accidental eccentricity for the condition of applied concurrent forces in two orthogonal directions:

"Where forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass should be applied for only one of the orthogonal direction at a time, but shall be applied in the direction that produces the greater effect."

The other related requirement of the 1997 Uniform Building Code for torsional response is the determination of the maximum inelastic response displacement:

"The maximum inelastic response displacement Δ_m shall be computed as follows:

$$\Delta_m = 0.7\ R\ \Delta_s$$

Δ_s is the elastic static or dynamic deformation including translational and torsional deflection. Alternatively, Δ_m may be computed directly by nonlinear time history analysis."

B. Comparison of the Results of Code and Inelastic Torsional Response Analysis

Based on the 1997 Uniform Building Code's requirement mentioned before, linear static and dynamic response spectrum analyses of the AAA and Canoga buildings respectively were conducted using the PERFORM 3-D program. Both buildings are steel moment frame in seismic zone 4 and according to seismic zone map for California, the distances to the closest fault for both are more than 5 kilometer. Orthogonal effects for both buildings are considered by applying the set of 100-30 percent seismic forces for any of two perpendicular directions. The center of mass was displaced a distance equal to 5% of the plan dimension perpendicular to the direction of seismic force based on code. According to UBC 1997, for any dynamic time-history analysis a minimum of three Near-field ground motion records should be considered and response of the strongest ground motion (Newhall record) used for design purpose.

In order to compare the maximum inelastic displacements, Δ_m, and the resultant inelastic torsional rotation of UBC 1997 and nonlinear time-history analysis of actual plus 5% accidental torsion model with Newhall record, one set of analyses considering code linear static or response spectrum magnified by 0.7 R coefficient was conducted.

To check the existence of the torsional irregularity, the amplification factor A_x was determined and shown on Figure 7.1:

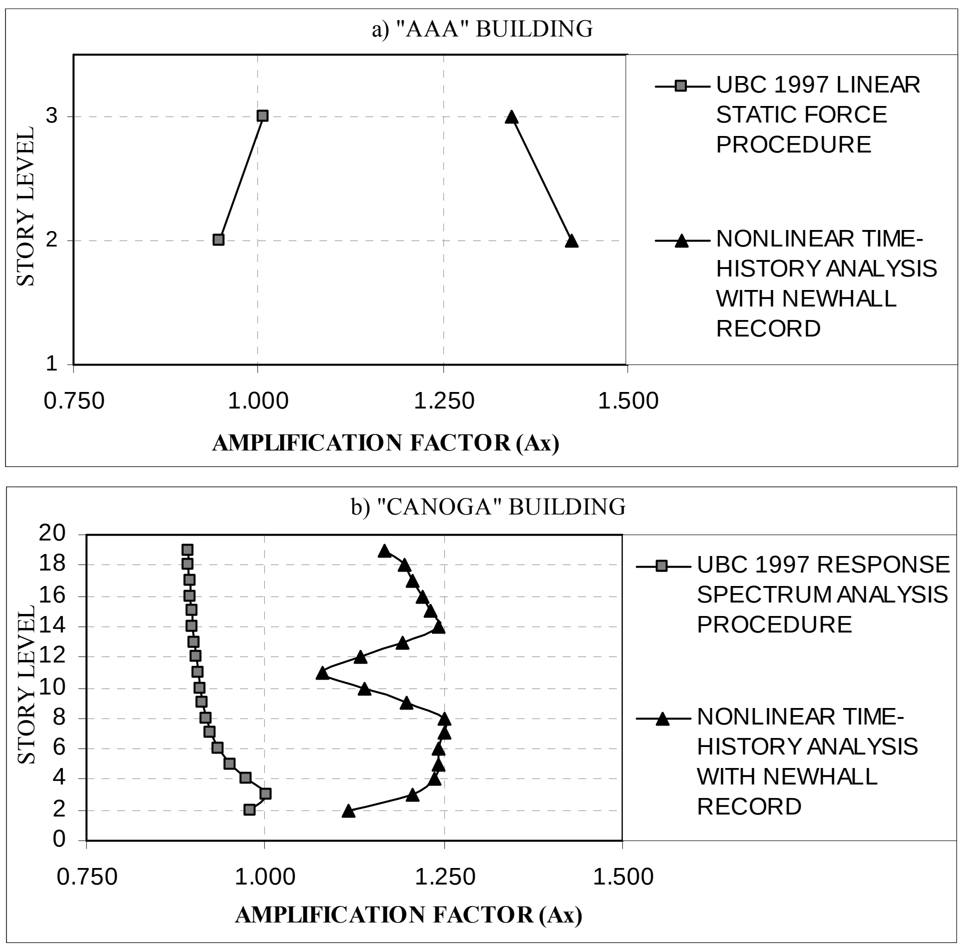
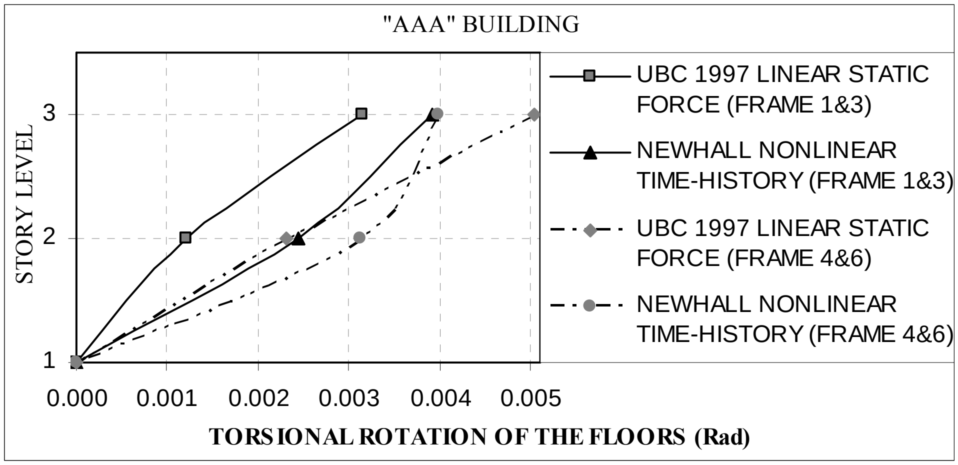


Figure 7.1–Accidental Torsion Amplification Factor

Code linear static or response spectrum analyses do not show any torsional irregularities and amplification factor is less than or equal one for almost all cases for "AAA" and "CANOGA" buildings which means 5 percent accidental eccentricity is adequate for the code analyses. However, calculation of the amplification factor for nonlinear dynamic analysis with Newhall record using the code procedure shows an average of 40 percent increase of accidental eccentricity for the AAA building (5%*1.4=7% accidental eccentricity) and about 20 percent increase excluding 10^{th} to 12^{th} floors for the CANOGA building (5%*1.2=6% accidental eccentricity) which is a considerable increase for accidental eccentricity that code analysis does not consider.

Comparison of the results for nonlinear torsional rotation of the floor between code analysis and nonlinear dynamic analysis with the Newhall record for the AAA building is shown in Figure 7.2. Increases of torsional rotations from code analysis to nonlinear dynamic analysis are 35 to 100 percent for different frames at the 2^{nd} floor, but for the roof this differences reduce and they are either negative or around 25 percent for different directions.

Based on the results for this building, code design criteria estimates average of total torsional rotation for two directions correctly for the roof and underestimates it by 35% for frames 4 & 6 and by 100% for frames 1 & 3 at the 2^{nd} floor due to the strong nonlinear behavior of the 1^{st} story.



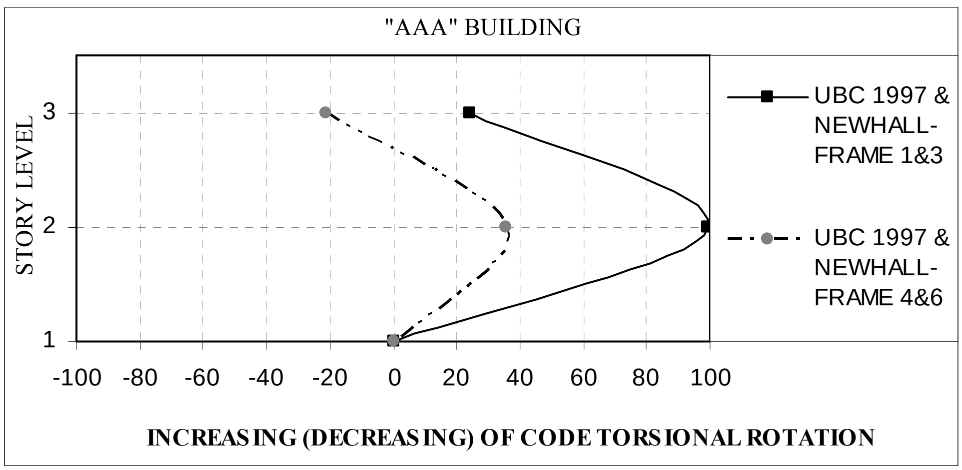
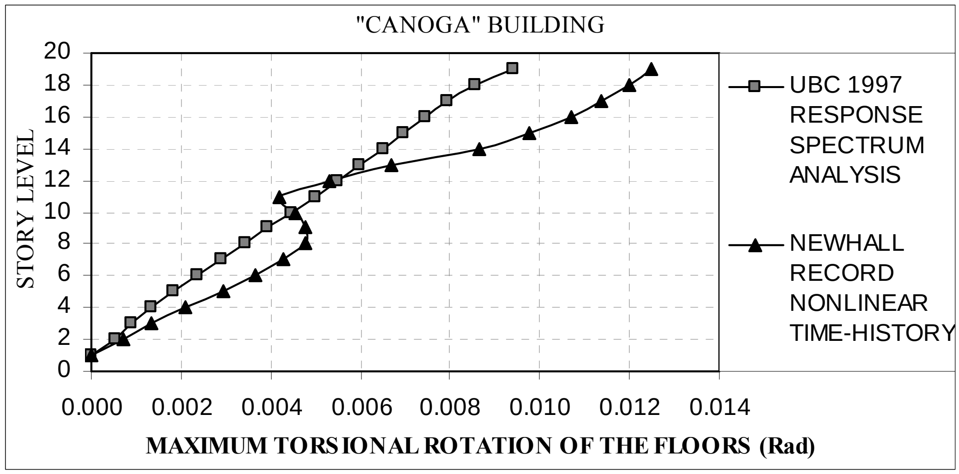


Figure 7.2 –UBC 1997 vs. Newhall Record, "AAA" Building

Comparison of the results for nonlinear torsional rotation of code analyses and nonlinear dynamic analysis with Newhall record for the CANOGA building shown in Figure 7.3 indicates Increases of 40 to 50 percent of torsional rotations from code analysis to nonlinear dynamic analysis for most of the floors excluding 9th thru 13th floors which have percentages that vary from -16% to 21%.

Based on the results for CANOGA building, the code design underestimates total torsional rotation for most of the floors about 40 to 50 percent which is a very considerable difference that needs to be addressed by code.



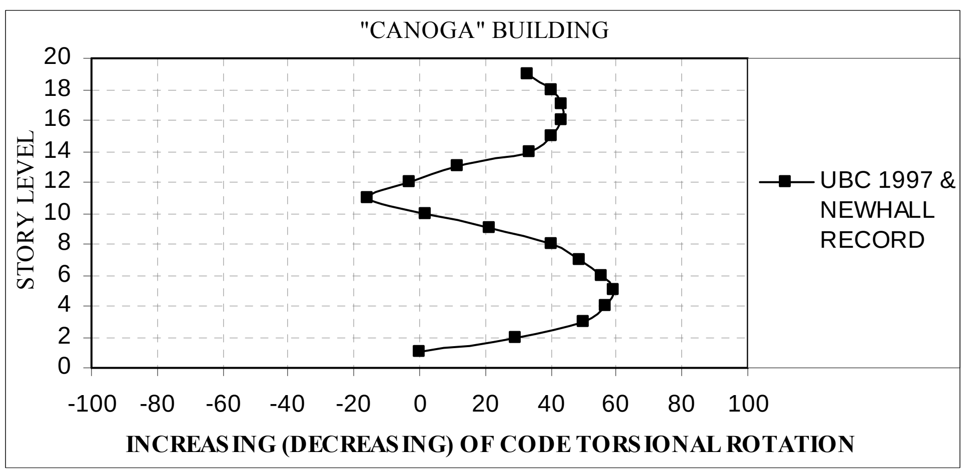


Figure 7.3 –UBC 1997 vs. Newhall Record, "CANOGA" Building

Chapter 8: Summary and Conclusion

A. Summary

The purpose of this study was to determine inelastic torsional seismic response of steel moment frame structures due to different parameters and nonlinearities and evaluate a correlation factor between elastic and inelastic torsional responses of the structure, and ultimately to arrive at design recommendation to include the effect of nonlinearity in accidental eccentricity. The first step was selecting two actual steel moment frame buildings damaged in Northridge earthquake with two and eighteen stories to represent low and high-rise buildings respectively for this study. These buildings were subjected to couple of Near-Field ground motions with different intensities and frequency contents from Northridge and Loma Prieta earthquakes and their performances to these earthquake records compared to real damages reported from Northridge earthquake to verify inelastic response of three-dimensional nonlinear dynamic model of the buildings. Detail investigations include the evaluation of elastic and the inelastic torsional seismic response of structure to Near-Field ground motion for different parameters and conditions and verification of the results as follows:

i. Effect of increasing eccentricity

ii. Effect of material nonlinearity on torsional response

iii. Geometrical effects (P-Δ) of high-rise building

iv. Strength eccentricity effect of low-rise building

v. Influence of different amount of critical damping ratios

vi. Comparison with UBS'97 requirements

vii. Verification of FEMA-273 nonlinear modeling criteria

In general, the inelastic torsional responses of both buildings were found to be noticeably different from the linear torsional elastic responses. In addition, the P-Δ effect has a very significant participation to the torsional response of the high-rise building.

The findings of this study can be summarized and compared as follows:

a) Effect of Increasing Eccentricity

As it discussed before, for this purpose, three different models representing three conditions of minimum torsion, actual torsion and actual plus 5% accidental torsion have been considered. Results of the roof displacements for these different models of "AAA" and "CANOGA" buildings with different time history earthquake records for elastic dynamic analyses have been summarized in Table 5:

		Actual Torsion compared to Minimum Torsion	Actual plus 5% Accidental Torsion compared to Minimum Torsion	Actual plus 5% Accidental Torsion compared to Actual Torsion
Canoga record	**18-story**	1.2%	8.3%	7.1%
	2-story	8%	11%	3%
Oxnard record	**18-story**	8.4%	18.6%	10.2%
	2-story	8.3%	18.8%	10.5%
Newhall record	**18-story**	7.6%	16.6%	9%
	2-story	3.6%	12.3%	8.7%

TABLE 5 – Percentage Increases of elastic Roof Displacement for a Low Rise and High Rise Building

Comparison of the results for Oxnard record shows that the percentage increases of roof displacement for both cases of actual and actual plus 5% accidental torsions are similar to each other for the low and high rise buildings of study. This percentage increase for Newhall record is similar to each other only for 5% accidental torsion alone between two buildings. However, the difference between the increases of roof displacement of two buildings is considerable for Canoga record and although low rise building reaches to larger percentage increase for actual plus 5% accidental torsion case, the high rise building has larger increase for 5% accidental torsion alone. In general for elastic dynamic analysis, there is a good agreement between the results of low and high rise buildings considered in this study. Also comparison of the results of this study with the results of three low and mid rise steel moment frame buildings from reference [28] shows a good agreement as well and percentage increase in displacement for 5% accidental torsion alone is less than or approximately equal to 10 percent for all cases.

Results of the roof displacements for the different models of "AAA" and "CANOGA" buildings with different time history earthquake records for inelastic dynamic analyses have been summarized in Table 6.

Unlike the elastic analyses, results for inelastic analyses show that the difference of percentage increase for roof displacement between low and high rise buildings of the study is considerable and the high rise building has a larger increase for most cases and even the difference reaches to about 77% for Oxnard record. However, percentage increase in displacement for 5% accidental torsion alone is very close for the low and high rise buildings of the study and has range from 8% to 12% for different time history earthquake records.

		Actual Torsion compared to Minimum Torsion	Actual plus 5% Accidental Torsion compared to Minimum Torsion	Actual plus 5% Accidental Torsion compared to Actual Torsion
Canoga record	**18-story**	4%	16%	12%
	2-story	7%	15%	8%
Oxnard record	**18-story**	16%	27%	12%
	2-story	9%	20%	11%
Newhall record	**18-story**	18%	28%	11%
	2-story	11%	20%	9%

TABLE 6 – Percentage Increases of inelastic Roof Displacement for a Low Rise and High Rise Building

b) Effect of Material Nonlinearity on Torsional Response

The percentage increases of the torsional rotation for different models of "AAA" and "CANOGA" buildings with different time history earthquake records have been summarized in Table 7 and 8. These increases have been calculated for the effect of inelastic behavior due to material nonlinearity and accidental torsion and the interaction between them for the two buildings considered in this study. The first two columns of each table from the left show the increase of torsional rotation for actual and actual plus 5% accidental torsion models due to material nonlinearity compared to elastic response and the second two columns compare the effect of 5% accidental torsion on the increase of torsional rotation for elastic and inelastic response of the two buildings to actual torsion model.

"AAA" BLD'G	Due to Material Nonlinearity				Due to 5% Accidental Torsion			
	Actual Torsion		Actual plus 5% Accidental Torsion		Elastic		Inelastic	
	2^{nd} Fl	Roof	2^{nd} Fl	Roof	2^{nd} Fl	Roof	2^{nd} Fl	Roof
Canoga record	N/S	42	N/S	67	N/S	125	180	165
Oxnard record	8	2	22	2	112	130	140	129
Newhall record	47	88	67	1	68	323	90	121

TABLE 7 – Percentage Increase of Torsional Rotation due to Material Nonlinearity and %5 Accidental torsion,
2–Story AAA Building

"CANOGA" BLD'G	Due to Material Nonlinearity				Due to 5% Accidental Torsion			
	Actual Torsion		Actual plus 5% Accidental Torsion		Elastic		Inelastic	
	11th	Roof	11th	Roof	11th	Roof	11th	Roof
Canoga record	-14	83	-4	74	98	122	121	110
Oxnard record	70	91	22	43	123	117	60	63
Newhall record	75	81	41	36	136	106	90	55

TABLE 8 – Percentage Increase of Torsional Rotation due to Material Nonlinearity and %5 Accidental torsion,
18–Story CANOGA Building

Comparison of the results shows that the material nonlinearity has a very considerable effect on the torsional rotation of both Actual and Actual plus 5% models especially on higher floors and increases the torsional rotation of both buildings of the study. Although, the percentage increase of torsional rotation due to nonlinearity is much higher for high rise building of the study than the low rise and for some cases this increase for low-rise building is about zero. For high-rise building at roof, actual torsion model has a very high percentage of increase for all records due to nonlinearity (81 to 91 percent). The inclusion of 5% accidental torsion has a very large effect on torsional rotation especially for elastic models and this effect is much higher for low-rise building.

Study of the plastic hinge mechanism for the most severe frames of each building shows a positive correlation between torsion and plastic hinge rotation and an increase in the torsion will cause an increase in the plastic hinge rotation as well. This helps to explain the increase of torsional rotation due to material nonlinearity.

Although the inelastic torsional rotation of each floor is a very important parameter to study nonlinear torsional response of the building, but in order to include the inelastic torsion into the linear models for design purpose, the evaluation of the related eccentricities is significantly important and could be a very useful tool for design of the building with torsion. For this purpose, nonlinear torsional moment and lateral load of low and high rise buildings of study for Newhall record at peak inelastic torsional rotation of each floor were evaluated through PERFORM 3-D outputs and additional eccentricity calculated as a percentage of the building dimension. In order to have a better tool to compare between Actual and Actual plus 5% Accidental torsion models, the additional eccentricities were presented only as a percentage of the building dimension in N-S direction for both buildings.

Results in form of the graph are shown in Fig.8.1 and Fig.8.2:

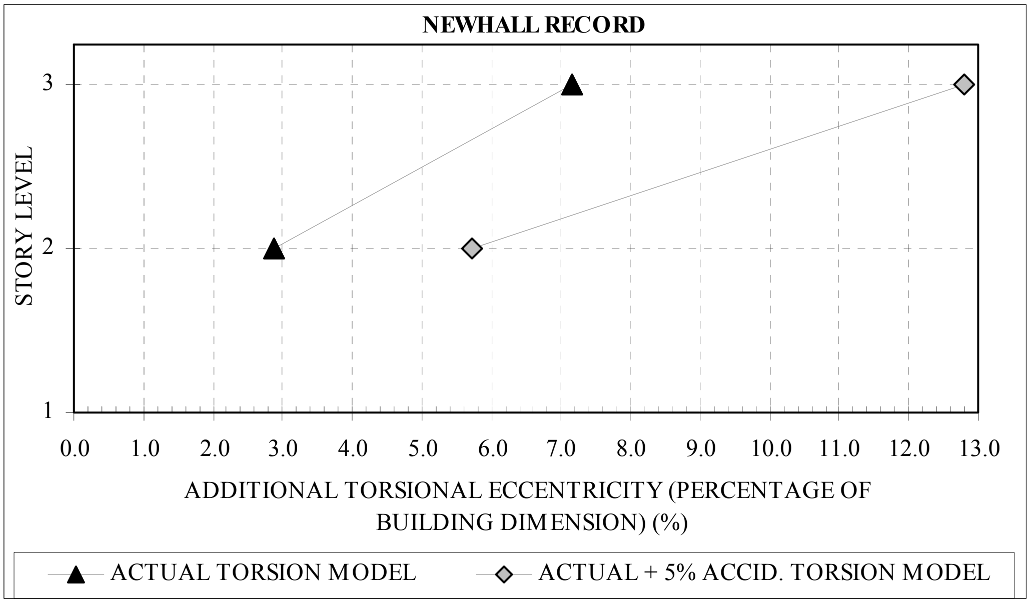


Figure 8.1 –Additional Torsional Eccentricity due to Material Nonlinearity (Percentage of the Building Dimension in N-S direction) (%), AAA Building

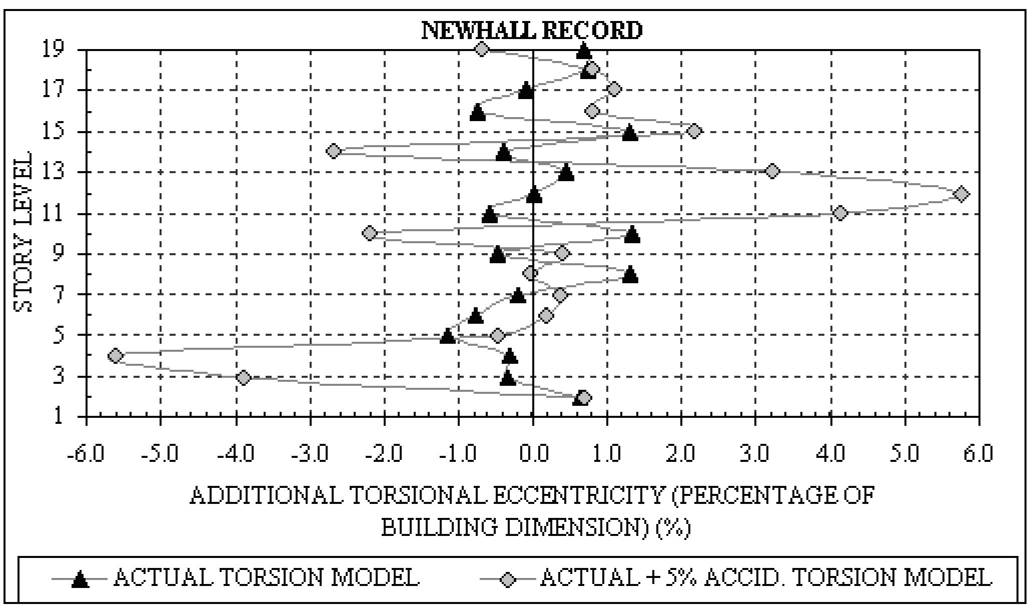


Figure 8.2 –Additional Torsional Eccentricity due to Material Nonlinearity (Percentage of the Building Dimension in N-S direction) (%), CANOGA Building

Study of the results shows that the additional torsional eccentricity due to material nonlinearity is considerable for all models of AAA building and vary from 3 to 13 percent for different floors. For instance, the results suggest adding extra eccentricity of 3 to 7 percent of building dimension to linear Actual torsion model to include the effect of material nonlinearity.

For the high rise CANOGA building, the additional torsional eccentricity due to material nonlinearity is not consistent over the height of the building. Since much of the seismic energy for a tall building is in frequencies corresponding to the higher modes of vibration, that lowers seismic response of some floors and cause an inconsistency of torsional eccentricity. Results shown in Figure 8.2 indicate no additional eccentricity for first five floors of the building and then varies from 0.0 to 1.25 percent of building dimension for Actual model and 0.0 to even 5.75 percent at one floor for Actual plus 5% Accidental torsion model for the fifth floor to roof.

c) Geometrical Nonlinearity Effect (P-Delta) of High Rise Building

Increase of the roof displacements and torsional rotation due to P-Delta effect for three different records and models representing three conditions of minimum torsion, actual torsion and actual plus 5% accidental torsion for "CANOGA" building have been summarized in Table 9 and 10.

Results of the roof displacement clearly show that the effect of P-Delta on residual displacement is much higher than maximum displacement and the increase for maximum displacement reaches only to 14% for Newhall record. However for residual displacement, it is different and for Newhall and Los Gatos it reaches to an average of 52 and 65 percent respectively. This average increase for Rinaldi is about to 25 percent.

"CANOGA" BLD'G	Minimum Torsion		Actual Torsion		Actual + 5% Accidental Torsion	
	Max.	**Residual**	**Max.**	**Residual**	**Max.**	**Residual**
Newhall record	14	53	13	47	10	55
Los Gatos record	N/S	70	N/S	67	N/S	57
Rinaldi record	3.5	20	1	26	N/S	30

TABLE 9 – Increase of Roof Displacement (%) Due to P-Delta effect, 18–Story CANOGA Building

Also results for Los Gatos record show that there is a major difference between the responses for maximum displacement and residual displacement. Comparison of most sever frames at east and west elevation shows that for the residual displacements, adding P-Delta effect to the system causes higher displacement responses as expected and correlates well with the results from Newhall records. But the displacements for maximum response are totally different and graphs with P-Delta effect have lower response than others. It seems the P-Delta effect shifts one of the torsional or translational responses peaks and they are not happening at the same time, which makes the total response lower.

Comparison of the results for torsional rotation (Table 10) shows that the P-Delta has a considerable effect on the maximum and residual torsional rotation of Actual and Actual plus 5% models for Newhall record and only residual torsional rotation of Los Gatos record. The maximum torsional rotation for Los Gatos and

Rinaldi records decrease for almost all stories due to P-Delta effect. That could be happened because the peak torsional rotation is a function of interaction among P-Delta effect, seismic ground motion and frame displacement, it is possible that P-Delta acts against initial deflection and reduces the peak torsional rotation.

"CANOGA" BLD'G	Actual Torsion		Actual + 5% Accidental Torsion	
	Max.	Residual	Max.	Residual
Newhall record	25.1	34.2	26.1	32.5
Los Gatos record	N/S	58	N/S	58
Rinaldi record	16	-21	-4	55

TABLE 10 – Percentage Increase of Roof Torsional Rotation (%) Due to P-Delta effect, 18–Story CANOGA Building

Study of the plastic hinge mechanism for the most severe frames of each building shows a positive correlation between torsion and plastic hinge rotation as well, however adding P-Delta to the system does not have a significant effect on upper floors and only increase the total hinge rotation of lower floors (3rd through 6th floors).

Additional eccentricities due to the material and geometrical nonlinear torsion for Newhall record were calculated as a percentage of the building dimension. In order to have a better tool to compare between Actual and Actual plus 5% Accidental torsion models, the additional eccentricity was presented only as a percentage of the building dimension in N-S direction.

Results in form of the graph are shown in Figure 8.3:

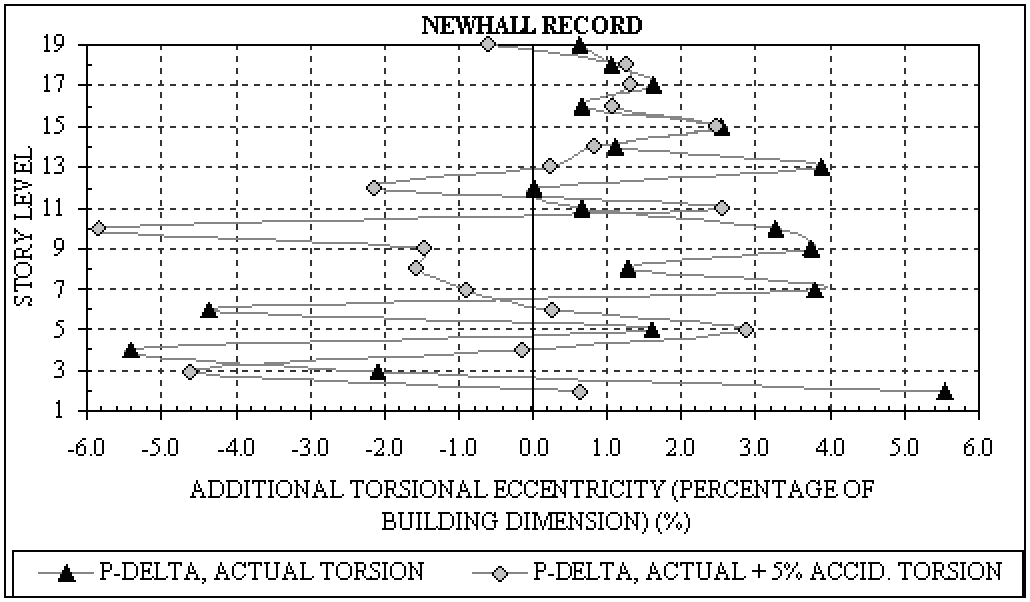


Figure 8.3 –Additional Torsional Eccentricity due to Material and Geometrical Nonlinearity (Percentage of the Building Dimension) (%), CANOGA Building

Study of the results shows that the additional torsional eccentricity due to combination of material and geometrical nonlinearity is not uniform over the height of the building due to participation of higher modes of vibration. Results shown in Figure 8.3 indicate almost no additional eccentricity for first five floors of the building for both models. However, the actual torsion model shows much larger eccentricities than material nonlinearity alone from seventh floor thru the roof and for most floors varies from 1.0 to 4.00 percent of building dimension. Actual plus 5% Accidental torsion model has additional torsional eccentricities from 11th floor thru the roof and vary from 0.25 to even 2.5 percent over the height.

d) Strength Eccentricity effect of Low-rise Building

As it discussed before, to study the effect of strength irregularity on torsional response of AAA building, Actual torsion model with inelastic behavior considering strength irregularity thru the combination of nominal and actual yield strength value for steel frame members was utilized.

The results of inelastic torsional rotation clearly show that adding strength eccentricity to the model has a considerable increase to the torsional rotation for most cases. Also results suggest that the strength eccentricity has a larger effect with moderate earthquake records than the one with higher peak acceleration and this effect is much larger at 2nd floor than roof for those moderate earthquake records.

e) Influence of Different Amount of Critical Damping Ratios

For this purpose, the Actual Torsion model with different amounts of critical damping ratios of 3%, 5%, 7% and 20% utilizing the Newhall record and its effect on displacement and torsional rotation of NEWHALL building was considered.

Results of the study show that by increasing the damping, displacements of the frames decrease as expected. In addition, analyses with P-Delta have an increased displacement. The difference between analysis response with and without P-Delta is significant and varies from 20 to 40 percent for maximum and residual displacement respectively. Comparison among analyses with different amount of damping ratio shows that by increasing the damping, P-Delta effect reduces and even reaches to zero for damping ratio of 20% for maximum displacement graphs. The results for torsional rotation show a good conformance between instant maximum and permanent torsional rotations for different damping ratios and only for the case with 20% damping ratio, the increase to the torsional rotation is negligible.

f) Comparison with UBC'97 Requirements

As has been discussed in the previous chapter, 5% accidental eccentricity based on UBC'97 for all types of buildings in general is to consider the differences between assumed and actual center of stiffness and mass locations and also the effect of other parameters such as the rotational component of ground motion. The other parameters such as inelastic and geometric nonlinearities are not included. Note that the CANOGA Building is 230' in height to the roof and this is just short of 240' limitation for static analysis. In this study for code analyses, response spectrum analysis was used instead of static analysis.

Results of this study based on code requirements suggest an increase to accidental eccentricity for inelastic effect using amplification factor A_x as defined in UBC'97. This amplification factor is about 1.4 for AAA building and about 1.2 for CANOGA building excluding 10^{th} to 12^{th} floors based on the results of previous chapter. These results suggest using an accidental eccentricity of 7 and 6 percent for the low-rise and high-rise buildings considered in this study respectively in lieu of 5 percent accidental eccentricity of code.

f) Verification of FEMA-273 Nonlinear Modeling Criteria

In order to verify the results of this study, inelastic responses of the 3-D model need to be compared to the measured responses and/or damage of actual buildings during the Northridge earthquake. Also FEMA-273 criteria need to be satisfied as well. According to FEMA 273 guidelines, maximum plastic rotation of the hinge relative to the expected performance level of the building is limited to 0.004 radian for Immediate Occupancy level with minimal or no damage to the structural elements, 0.025 radian for Life Safety level with extensive damage to the structural and nonstructural components and 0.043 radians for Collapse Prevention level with failure of nonstructural components and large permanent drifts but functioning of load bearing walls and columns.

Study of the results for the AAA building shows that the plastic hinge rotations for most severe beams and columns is about 0.024 and 0.0318 radians respectively which pass the second deformation limit of FEMA 273 for columns and reach to the Collapse Prevention level. This has a good agreement with the actual condition of the building after earthquake. Comparison between the real residual displacements measured after earthquake for second floor and the result of inelastic analyses shows the difference of about 33 percent between two displacements. The failure of some welds instead of plastic hinge mechanism could be a good reason for this difference. Although the building met the life safety criteria, it had to be demolished and a new building constructed in its place.

Investigation of the results for the CANOGA building shows that the plastic hinge mechanism happens only at beams and plastic rotation reaches to 0.0142 radians for most severe beam which is within the second deformation limit of FEMA 273. Comparison of actual damage of the building during the Northridge earthquake with plastic hinge mechanism for Canoga record with and without P-Delta effect shows a very good agreement for frame "D" at west elevation and acceptable agreement for frame "B" at east elevation. Also the amount of residual displacements for actual torsion model of Canoga record at roof is 6.8" which is very close to the actual residual displacement of 6" measured after earthquake.

In general, both 3-D models show a good agreement with actual damages happened to the buildings during Northridge earthquake.

B. Conclusions and Recommendations

The purpose of this study was to develop a rational basis for evaluation of nonlinear torsional response of buildings and utilized that basis to comment on the question of extrapolating elastic torsional design procedures to the inelastic range and to assist in identifying the limitations of code torsional design procedures. The following conclusions and recommendation were made:

1. Increasing eccentricity has a much larger effect on lateral displacement of inelastic model that elastic model. Also comparison between low and high rise buildings of the study shows that the high rise building has a much larger percentage increase for inelastic models than low rise building in general. However, percentage increase in displacement for 5% accidental torsion alone is very close for both buildings and has a range from 8% to 12% for different time history earthquake records.
2. Material nonlinearity has a considerable effect on torsional rotation of both low and high rise buildings of the study. However, the percentage increase of torsional rotation is much higher for high rise building. The inclusion of 5%

accidental torsion has a very large effect on torsional rotation of both elastic and inelastic models, although this effect is much higher for elastic models of low rise building of the study.

3. Study of the plastic hinge mechanism for the most severe frames of each building shows a positive correlation between torsion and plastic hinge rotation and an increase in the torsion will cause an increase in the plastic hinge rotation as well. This helps to explain the increase of torsional rotation due to material nonlinearity.
4. In order to include the inelastic torsional response into the linear models for design purpose, additional eccentricity due to material nonlinearity can be evaluated and added to the linear model as a percentage of the building dimension. This additional eccentricity is very considerable for the low rise building of this study and varies from 3 to 13 percent for second floor and roof. However, this additional eccentricity is not consistent over the height of the high rise building of the study due to participation of higher frequency modes of vibration to transmit seismic energy. Results show no additional eccentricity for first five floors of the building and vary from 0.0 to the maximum of 5.75 percent for the fifth floor thru the roof. This additional eccentricity can be added to accidental eccentricity to include the effect of material nonlinearity.
5. Study of the roof displacement show that the effect of P-Delta on residual displacement is much higher than its effect on maximum displacement and

the increase reaches to average of 47% for three different records. P-Delta effect in some cases decreases the displacement for maximum response due to shifting of one of the torsional or translational responses peak.

6. P-Delta effect has a considerable increasing effect on the maximum and residual torsional rotation of the building for most seismic records. However, in few cases it decreases the torsional response because the peak torsional rotation is a function of interaction among P-Delta effect, seismic ground motion and frame displacement, it is possible that P-Delta acts against initial deflection and reduces the peak torsional rotation.
7. P-Delta effect does not have a significant effect on upper floors plastic hinge mechanism and only increase the total hinge rotation of the lower floors (3rd to 6th floor).
8. The additional torsional eccentricity due to combination of material and geometrical nonlinearities is much larger than material nonlinearity alone over the height of the building.
9. Study of the strength eccentricity on the torsional response of the low rise building of the study shows a considerable increase of the torsional rotation. Besides this has much larger effect with moderate earthquake records at second floor. This indicates that adding strength above that required may not always be wise if the structure experiences inelastic response.
10. Amount of critical damping ratio has a considerable effect on torsional response of the building as well. By increasing the damping, response of the

structure decreases and also increasing of damping reduces the P-Delta effect and even reaches to zero for damping ratio of 20%.

11. Results of this study based on code requirements suggest an increase to accidental eccentricity for inelastic effect using amplification factor A_x as defined in UBC'97.
12. 3-D models of both buildings give a good indication of the actual damages that happened to the buildings during Northridge earthquake.

C. Future Research

Possible options to extend this study include the following: 1) Implementation of the model on a mid-rise building. 2) Changing of the overall model characteristics to consider different structural systems or different types of material.

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Appendix A:
Effect of Strength Eccentricity

Comparison of the torsional rotation graphs for Canoga and Newhall records (Fig.5.19 and Fig.5.21) with Oxnard record (Fig.5.20) shows that for minimum torsion case, the amount of inelastic torsional rotation is not zero and that may be the indication of strength eccentricity. For this purpose and in order to study the effect of strength irregularity on torsional response of this building, a modified Actual torsion model including strength irregularity for inelastic behavior only, was utilized. As it mentioned before, for this model yield strength value of steel beams and columns for Frame-1 and Frame-4 at west and south elevations are set to the actual measured yield strength value of 47.5 ksi; however the nominal yield strength value of 36 ksi was used for the steel beams and columns of the other frames.

Study of two models with and without strength eccentricity for each floor in Figure A.1 shows the effect of strength irregularity on the maximum inelastic displacement. The strength irregularity has a much larger effect at second floor displacement than roof. For Canoga record, the maximum displacements at second floor for Frame-1 at west elevation and Frame-4 at south elevation decrease from 4.46 and 2.24 inches to 4.30 and 2.21 inches respectively and for Frame-3 at east elevation and Frame-6 at north elevation increase from 4.93 and 2.36 inches to 5.61 and 2.64 inches (13.8% increase) respectively.

For Oxnard record, the maximum displacements at second floor for Frame-1 at west elevation and Frame-4 at south elevation decrease from 3.55 and 2.62 inches to 3.40 and 2.38 inches respectively and for Frame-3 at east elevation and Frame-6 at north elevation increase from 4.16 and 3.47 inches to 4.37 and 3.82 inches (4.8% increase) respectively.

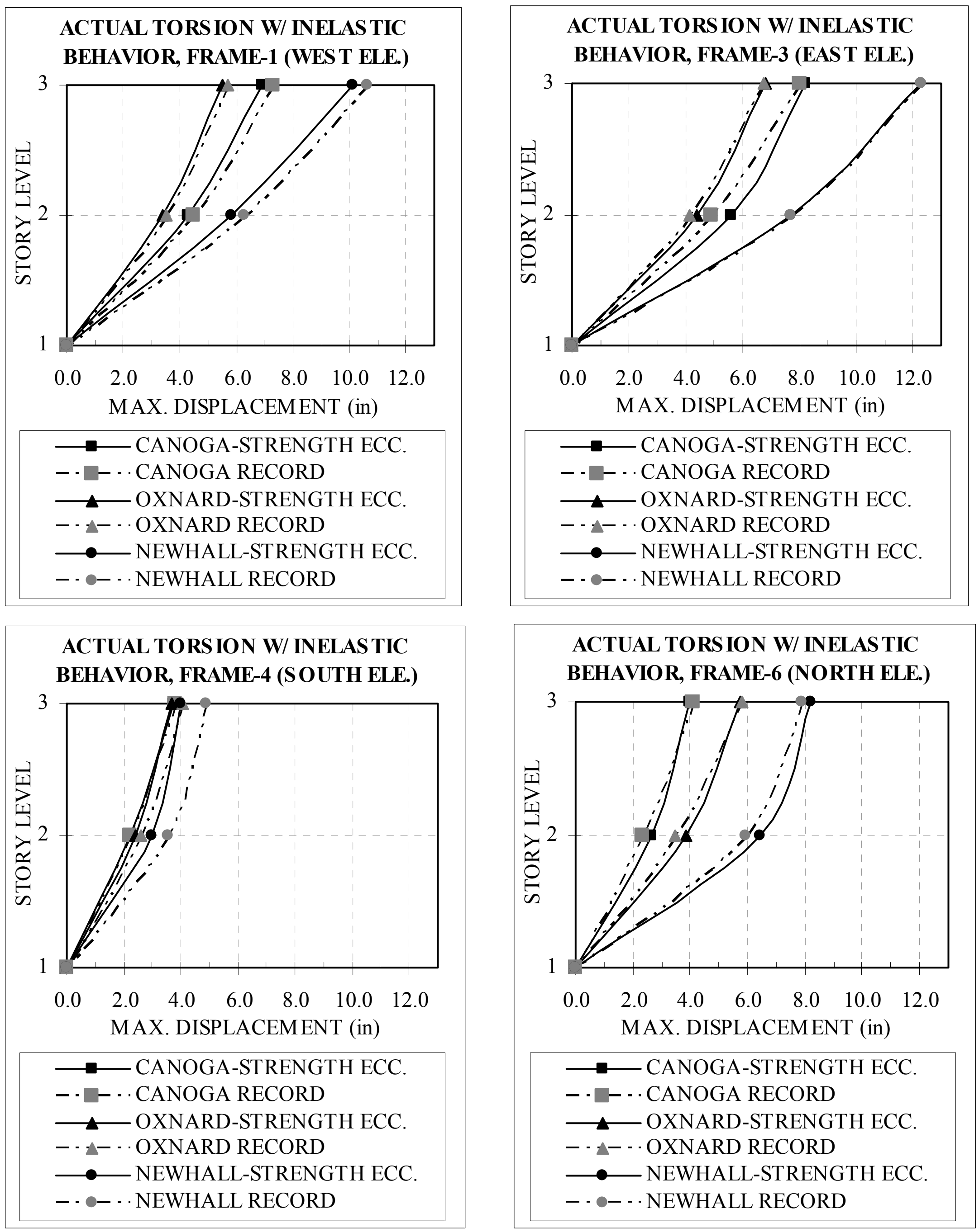


Figure A.1 - MAX Displacement of 2nd Floor and Roof, Actual Torsion Model with Inelastic Behavior Considering the Strength Eccentricity

Considering the frame with largest torsional effect, (Frame-3, east elevation), the strength eccentricity adds 14% and 3% to the displacement of second floor and roof respectively for Canoga record and 5% and 1% to the displacement of second floor and roof respectively for Oxnard records.

For Newhall record, the maximum displacements at second floor for Frame-1 at west elevation and Frame-4 at south elevation decrease from 6.25 and 3.53 inches to 5.85 and 2.96 inches respectively. The increase of displacement for Frame-3 at east elevation is negligible but for Frame-6 at north elevation increase from 5.94 to 6.41 inches (7.9% increase) respectively.

The comparison of two models with and without strength eccentricity for each floor in Figure A.2 shows the effect of strength irregularity on the inelastic torsional rotation. This effect has a considerable increase to the torsional rotation of most cases. For Canoga record, the increase at roof is approximately (1.38-0.75)e-3/0.75e-3= 0.63/0.75= 83% and for 2nd floor is about (1.14-0.41) e-3 /0.41e-3= 0.73/.41= 179% due to strength eccentricity which shows a very large increase for both floors. For Oxnard record, the increase at roof and second floor are approximately 10% and 59% respectively due to strength eccentricity which still shows a large increase at second floor. For Newhall record, the torsional rotation increases by about 28% at roof and by 26% at second floor. Results of this study suggest that the strength eccentricity has a larger effect on the moderate earthquake records than the one with higher peak acceleration and also this effect is much larger at second floor than roof for those moderate earthquake records.

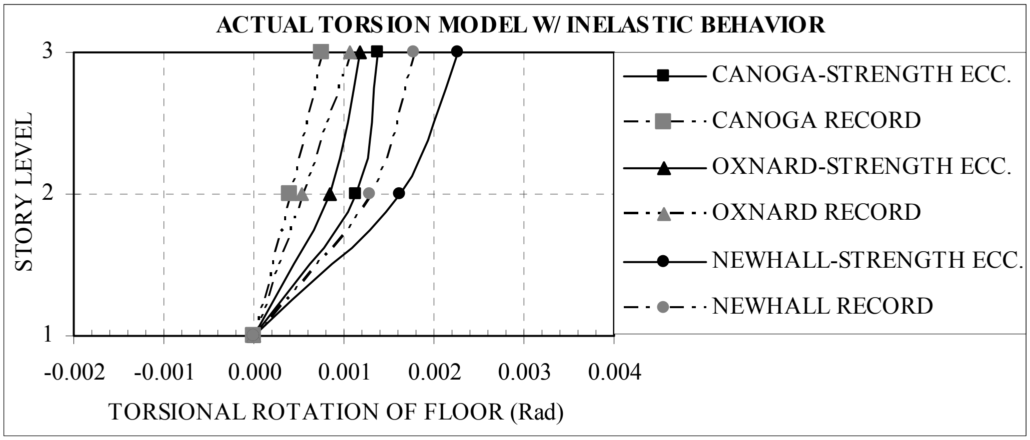


Figure A.2 - Torsional Rotation of 2nd Floor and Roof, Actual Torsion Model with Inelastic Behavior Considering the Strength Eccentricity

The envelope of the maximum elastic plus plastic (total) hinge rotations are shown in Figures A.3 and A.4 for beams and columns.

The total hinge rotations of beams with and without strength eccentricity are shown on Figure A.3. The strength eccentricity does not have a considerable effect on total hinge rotation of beams for Frame-4 and Frmae-6 of most cases. However, this effect is considerable For Frame-3 and increases the total hinge rotation about 68% and 16% respectively for second floor and roof for Canoga record. The increase for Oxnard record is negligible but for Newhall record reaches to 17% and 23% respectively for second floor and roof.

The total hinge rotations of columns with and without strength eccentricity are shown on Figure A.4. The strength eccentricity has a considerable effect on the total hinge rotation of columns for Frame-3 and Frmae-6 and increase of the total hinge rotation reaches to about 60% and 64% respectively for Canoga and Oxnard records at top of the column. The increase for Newhall record is about 25%.

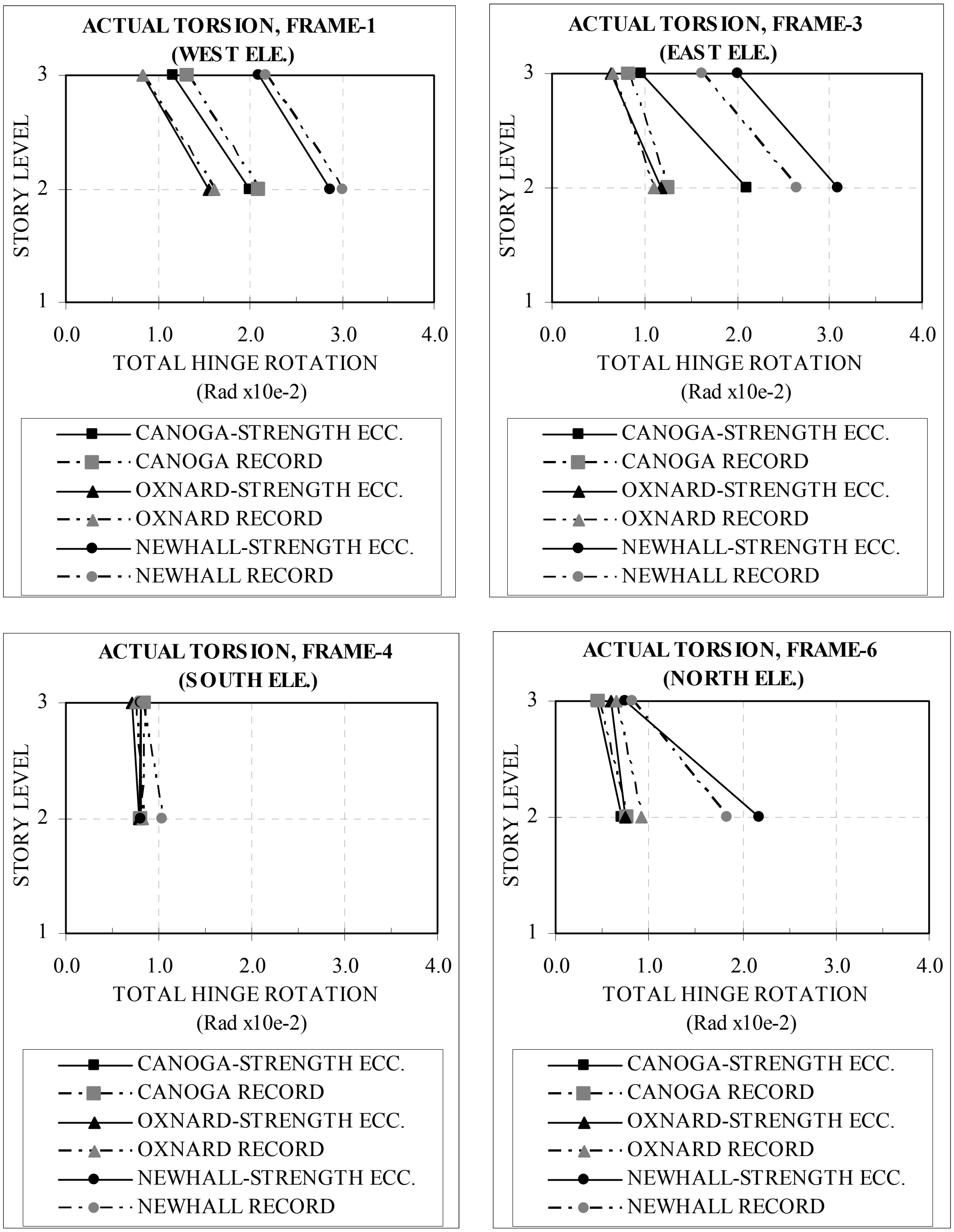


Figure A.3 - Envelope of Maximum Hinge Rotation of Beams, Actual Torsion Model Considering the Strength Eccentricity

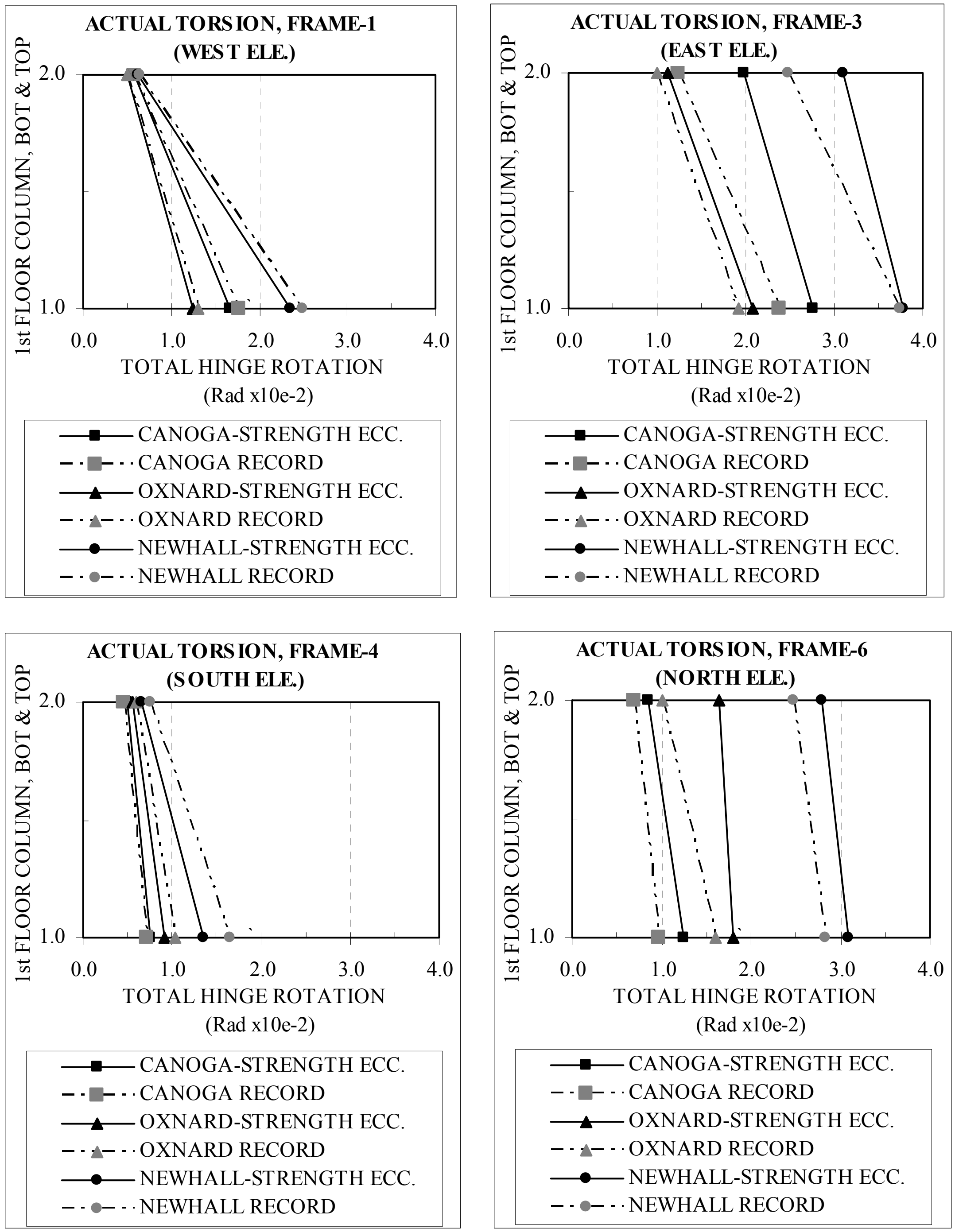
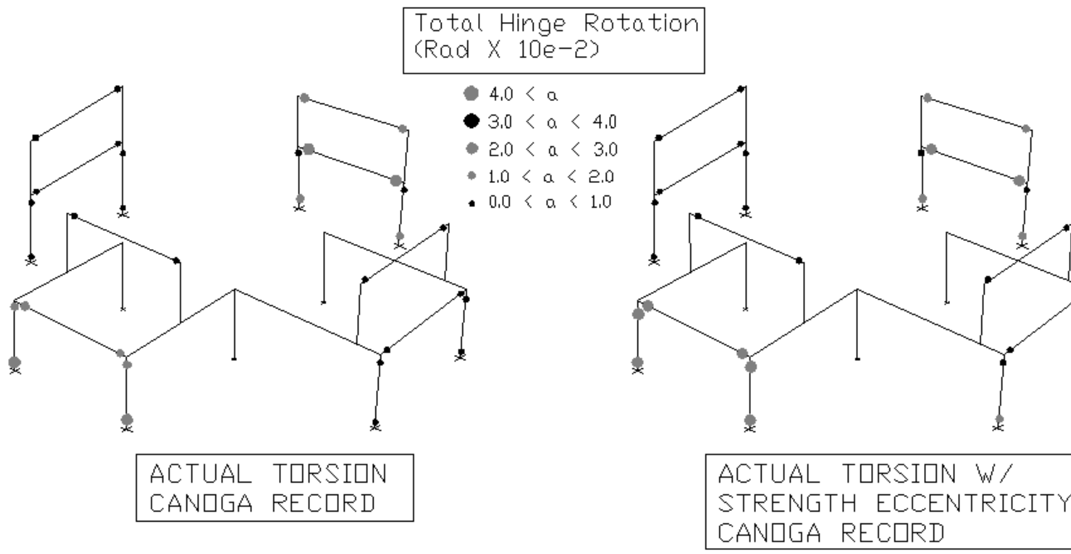
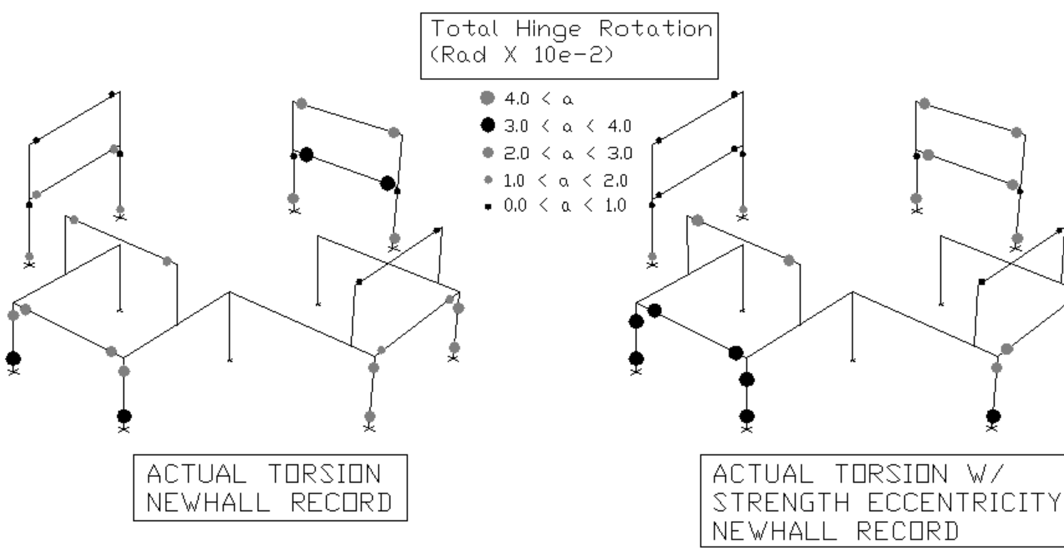


Figure A.4 - Envelope of Maximum Hinge Rotation of columns, Actual Torsion Model Considering the Strength Eccentricity

Figure A.5 is comparing the plastic hinge mechanism of the building for Canoga and Newhall records with and without strength eccentricity.



Canoga Record

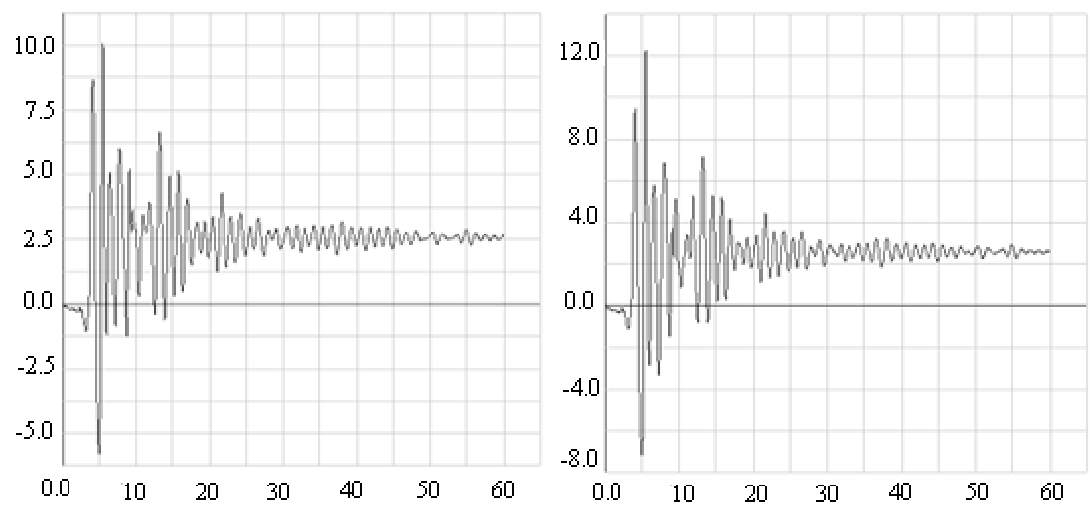


Newhall Record

Figure A.5 – Plastic Hinge Mechanism of Frames

Comparison of different mechanisms especially for Newhall record shows that by adding strength eccentricity to the model, there is not significant change to the plastic hinge mechanism of second floor to roof columns and roof beams. However, strength eccentricity cause an increase to total hinge rotation of second floor beams and first floor columns at base and at top respectively for North and east elevation and a decrease of total hinge rotation of second floor beam at west elevation.

Figure A.6 shows the time-history displacement of actual model with strength eccentricity for Newhall record to investigate the residual displacement. Comparison of the model with strength irregularity (Fig.A.6) with the one without strength irregularity (Fig 5.31 and 5.32) shows that the strength eccentricity does not have a considerable effect on the residual displacement and it only changes from 2.4 to 2.5 inches for Frame-3 at east elevation.



WEST ELEVATION- FRAME-1 EAST ELEVATION- FRAME-3

Figure A.6 – Displacement Time-history of Roof for Actual Model with Strength Eccentricity, NEWHALL Record