Evaluation of Code Accidental Torsional Provisions Using Strong Motion Records from Regular Buildings

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ABSTRACT

A procedure was developed for evaluating building code provisions for accidental torsion from analysis of recorded motions of nominally symmetric-plan buildings during earthquakes. This procedure was applied to the motions of a three story office building in Richmond (CSMIP station nº 58506), California recorded during the Loma Prieta earthquake. The results for this particular earthquake show that the prescribed code accidental eccentricity value of 0.05b predicts reasonably well the torsional effects experienced by the structure.

INTRODUCTION

Building codes require that the effects of torsion be considered by applying the equivalent lateral forces at a distance e_d from the center of rigidity (CR), resulting in story torques in addition to shears and overturning moments. The design eccentricity, e_d, specified in U.S. codes and design recommendations [1, 2] is of the form $e_d = e_s \pm 0.05b$, where e_s is the static stiffness eccentricity—i.e. the distance between the center of mass (CM) and CR— and b is the plan dimension of the building perperdicular to the direction of ground motion. The first term, e_s, is intended to account for the coupled lateral torsional response of the building arising from lack of symmetry in plan. The additional $\pm 0.05b$, known as *accidental eccentricity*, is introduced to account for eccentricities due to discrepancies between the mass, stiffness, and strength distributions used in analysis and true distributions at the time of an earthquake; torsional vibrations induced by a rotational component of ground motion; and other sources of torsion not considered explicitly in analysis.

Most of the research investigations of coupled lateral-torsional response of buildings, including the work aimed towards evaluating the adequacy of torsional provisions in building codes, have been concerned with structures with asymmetric plan. Perhaps there are two major reasons. Firstly, buildings with asymmetric floor plan tend to suffer greater damage. Secondly, the dynamics of asymmetric plan buildings are amenable to analytical study; elastic as well as inelastic systems have been investigated [3, 4, e.g].

On the other hand, the subject of accidental eccentricity is not amenable to investigation by traditional analytical approaches. Standard dynamic analyses cannot predict torsion in symmetric-plan buildings. However, it has been possible to investigate analitically the torsional response of such buildings due to rotational ground motion [5]. These studies are based on ground motion assumptions which so far have not been verified for lack of suitable ground motion records. Therefore, analysis of recorded motions of nominally-symmetric-plan buildings during earthquakes would be the most direct means of developing an understanding of the torsional responses of such buildings and for evaluation of building code provisions for accidental torsion.

BUILDING CONSIDERED

Ideal for the purposes of the investigation would be buildings with nominally symmetric floor plans, rigid floor diaphragms, and negligible soil structure interaction effects, for wich three independent components of acceleration have been recorded at the ground level and at each floor. A three-story office building in Richmond, California (CSMIP station nº 58506) satisfies these requirements. Records of motions of the this building during the Loma Prieta earthquake are available.

A typical framming plan of this steel structure is shown in Figure 1. It consists of moment-resisting frames 1 and 7 in the Y-direction. Between frame lines 3 and 6, frames A and C are also designed for lateral load resistance. All other frames with semi-rigid connections are designed to carry only gravity loads.

The floor decking system is formed by a steel corrugated metal sheet filled with lightweight concrete. The roof deck is lighter but has additional insulating concrete. The foundation system consists of rectangular column footings interconnected by grade beams. In the Y-direction only footings for columns of frames 1 and 7 are inter-connected.

RECORDED MOTIONS

The locations of the accelerographs in the building are shown in Figure 2. The strong motion records obtained from the Loma Prieta earthquake are shown in Figure 3. The peak accelerations at the ground level are $0.083g$ in the X-direction and $0.11g$ in the Y-direction. These motions were amplified to $0.31g$ and $0.27g$, respectively at the roof level. The building experienced no damage during the earthquake.

The vibration frequencies and shapes of the first mode in the X-direction, the first mode in the Y-direction, and the first torsional mode were determined from the recorded motions. The results are summarized in Table 1.

STRUCTURAL IDEALIZATION

The building was idealized for analysis by the ETABS computer program, wherein floor diaphragms are assumed to be rigid and the building mass is lumped at the floor levels. The building was treated as fixed at the level defined by the slab on grade. Each frame was modeled with appropiate beam-column joints: moment resistant (or rigid) connections and semi-rigid connections. The latter were divided into two groups: connections of column flanges with beams were modeled as rigid and connections of column webs with beams as pinned.

The vibration frequencies and mode shapes of the idealized system computed by ETABS program are also presented in Table 1. The agreement between these results and the values obtained from the recorded responses is satisfactory.

DYNAMIC ECCENTRICITY

First, consider the simplest possible problem: a one-story, nominally-symmetric-plan building with a rigid roof-diaphragm with accelerations $a_1(t), a_2(t)$, and $a_3(t)$ of the roof recorded during an earthquake (Figure 4). From these records, $a_x(t)$ and $a_y(t)$, the x and y acceleration components at the CM, and $a_\theta(t)$, the torsional acceleration of the diaphragm (Figure 4) can be determined. The associated lateral forces are ma_x and ma_y in the x and y directions, respectively, and the associated torque is $I_p a_\theta$, where m is the mass of the roof, and I_p is the polar moment of inertia of the distributed mass about the CM. These forces are statically equivalent to each of the following force sets: (1) ma_x at the CM and ma_y at eccentricity e_x (Figure 5) where

$$e_x(t) = I_p a_\theta(t)/ma_y(t) \tag{1}$$

and (2) ma_y at the CM and ma_x at eccentricity e_y where

$$e_y(t) = I_p a_\theta(t)/ma_x(t) \tag{2}$$

The time-dependent quantities $e_x(t)$ and $e_y(t)$ may be interpreted as the instantaneous accidental eccentricities. They can be computed from the accelerations $a_x(t)$, $a_y(t)$, and $a_\theta(t)$, determined from the recorded motions. This approach to evaluate the accidental eccentricities is very appealing as it is based exclusively on recorded motions and does not require modeling of the structure or estimates of its stiffness properties. The only structural properties needed are the mass and polar moment of inertia of the roof.

This procedure for one-story systems can be extended to determine the accidental eccentricities for each floor of a multistory building with rigid floor diaphragms. The shears and torques in the j^{th} story can be determined by using simple statics from the floor inertia forces which are known from the floor masses and recorded accelerations.

$$V_{xj}(t) = \sum_{i=j}^{N} m_i a_{xi}(t) \tag{3}$$

$$V_{yj}(t) = \sum_{i=j}^{N} m_i a_{yi}(t) \tag{4}$$

$$T_j(t) = \sum_{i=j}^{N} I_{pi} a_{\theta i}(t) \tag{5}$$

Thus the accidental eccentricities at the j^{th} floor are defined by equations (6) and (7).

$$e_{xj}(t) = \frac{T_j(t)}{V_{yj}(t)} \tag{6}$$

$$e_{yj}(t) = \frac{T_j(t)}{V_{xj}(t)} \tag{7}$$

Required in evaluating the accidental eccentricities of a multistory building are the accelerations $a_{xi}(t)$, $a_{yi}(t)$ and $a_{\theta i}(t)$ at each floor.

From the recorded motions shown in Figure 3 these accidental eccentricities were computed for the selected building. The results for the first floor are presented in Figure 6 wherein the base shear and base torque are presented together with accidental eccentricities $e_{x1}(t)$ and $e_{y1}(t)$. These computed eccentricity values grossly exceed the code value of $0.05b$ intermittently during the earthquake. However, this result does not imply that the code provisions are deficient because the largest peaks in the eccentricity-time plot are usually associated with small values of base shear. Therefore, a large value for the accidental eccentricity by itself is not meaningful and should be considered in conjunction with the instantaneous base shear value; i.e. the combined effects of shear and torque should be considered in evaluating the code provisions.

STORY SHEARS AND TORQUE

The base shears and torque at each instant of time are given by equations (3) to (5) with $j = 1$. These forces were computed for the selected building from its floor floor masses and the recorded accelerations (Figure 3). Each point in Figure 7a represents the values of V_{x1} and T_1 at a particular instant of time, similarly V_{y1} and T_1 values are presented in Figure 7b.

According to the Uniform Building Code [6] the base shear is given by equation (8) :

$$V = \frac{ZIC}{R_w} W \tag{8}$$

For the selected building, estimates of the fundamental periods were obtained earlier from the recorded motions: $Tx = 0.59sec$ and $Ty = 0.76sec$. For analysis in the X-direction the code formula leads to a base shear of $V_{x1} = 161.2kips$, which combined with an eccentricity of $0.05b$ where $b = 77ft$, leads to a base torque $T_1 = 621kip - ft$. The code values of V_{x1} and T_1 are identified in Figure 7a; V_{y1} and T_1 are shown in Figure 7b. Only the dead weights were included in W in calculating code values of base shear.

In order to evaluate the code accidental torsional provisions, these code forces are amplified by a factor chosen to increase the base shear value to the peak value of $V_{x1}(t)$ determined from equation (3). The amplified code values are identified in Figure 7a. Each point on the straight line *AB* represents a combination of base shear and torque that produces the same member force as the amplified code combination, denoted by point *C* (Figure 7a). In particular, point *A* denotes the value of base shear alone (without any torque) that produces the same member force. For the selected building the straight lines *AB* and *A'D'* denote combinations of base shear and torque which produce the same bending moment (and shear force) in column 4 (Figure 1) in the first story. This is the element with the largest bending moment (and shear force) under the forces corresponding to point *C*. Straight lines *A'B'* and *AD* denote combinations of base shear and torque which produce the same bending moment (and shear force) in column 22 (Figure 1) at the first story. The straight lines *AB, AD, A'B'* and *A'D'* represent "equivalent" code limits. The corresponding results for analysis in the *Y*-direction are presented in Figure 7b.

At a particular time instant the base shear and torque computed from the recorded motions (equations (3) to (5), represents a point on the base shear-torque diagram. A point outside the "equivalent" code region represents a combination of base shear and torque which, together with the code specified heightwise distribution of lateral forces, produces member forces in the first story column 4 wich exceed the amplified code forces associated with point *C*. Such a condition would suggest that the accidental eccentricities should be increased beyond the code specified value of $0.05b$. For the selected building and recorded motions all combinations of V_{y1}-T_1 and V_{x1} - T_1, except the one denoted by **a**, fall within the "equivalent" code limits (Figures 7a and 7b). This one point falls only slightly outside the "equivalent" code limit indicating that the accidental eccentricity of $0.05b$ seems satisfactory.

For the particular recorded response of this building during the Loma Prieta earthquake, the torsional effects are so small that it may not be necessary to consider accidental eccentricity at all. Figure 7 indicates that only a couple of points fall outside the "equivalent" code limit with zero accidental eccentricity.

MEMBER FORCES

Additionally, the member forces associated with the recorded motions can be compared with those resulting from the amplified code forces defined earlier. At each time instant during the earthquake, the member forces are determined by static analysis of the building , using the ETABS computer program, subjected to the floor inertia forces $m_j a_{xj}(t), m_j a_{yj}(t)$ and $I_{pj} a_{\theta j}(t)$. The results of such static analyses are plotted in Figures 8 and 9, where the shear force and bending moment in the first story columns 22 and 18 are presented. Also shown is the member force associated with the amplified code forces defined by the base shear-torque combination of point *C* in Figure 7. It is seen that the column 18 forces during the earthquake remain below the amplified code values. The same is true for column 22 forces, except for one peak (This peak corresponds to point **a** in Figure 7). Even at this peak the actual member force exceeds the amplified code value only slightly. Thus the accidental eccentricity of $0.05b$ seems to be satisfactory in representing the torsional response of the building during the Loma Prieta earthquake.

Also shown in Figures 8 and 9 , are the member forces associated with the amplified code forces modified for zero accidental eccentricity. These lower values are slightly exceeded for both columns only by a single peak, suggesting that the torsional response of this building during the Loma Prieta earthquake is so small that it may not be necessary to consider accidental eccentricity at all.

CONCLUSIONS

A procedure has been developed for evaluating building code provisions for accidental torsion from analysis of recorded motions of nominally symmetric-plan buildings during earthquakes. This procedure has been applied to the motions of a three story office building in Richmond, California recorded during the Loma Prieta earthquake. It is demonstrated that the code accidental eccentricity of $0.05b$ seems to be satisfactory in representing the torsional motion of this building during this particular earthquake. Furthermore it appears that the accidental eccentricity need not even be considered in this case. Because the building analyzed is almost perfectly symmetric, these conclusions may not apply to all nominally-symmetric buildings for which the code accidental eccentricity is intended.

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[5] N. M. Newmark and E. Rosenblueth. *Fundamentals of Earthquake Engineering*. Prentice-Hall, Inc., Englewood Cliffs, N.J., 1971.

[6] Uniform Building Code. International Conference of Building Officials (ICBO), 1988.

	X-lateral mode		Y-lateral mode		Torsional mode	
	Recorded	Computed	Recorded	Computed	Recorded	Computed
Frequency (Hz)	1.317	1.321	1.672	1.657	2.242	2.212
Mode Shape						
Floor 3	1	1	1	1	1	1
Floor 2	.75	.73	.69	.77	.73	.76
Floor 1	.46	.38	.35	.44	.42	.42

Table 1: Vibration frequencies and shapes of first three modes from recorded motions and computed using structural model

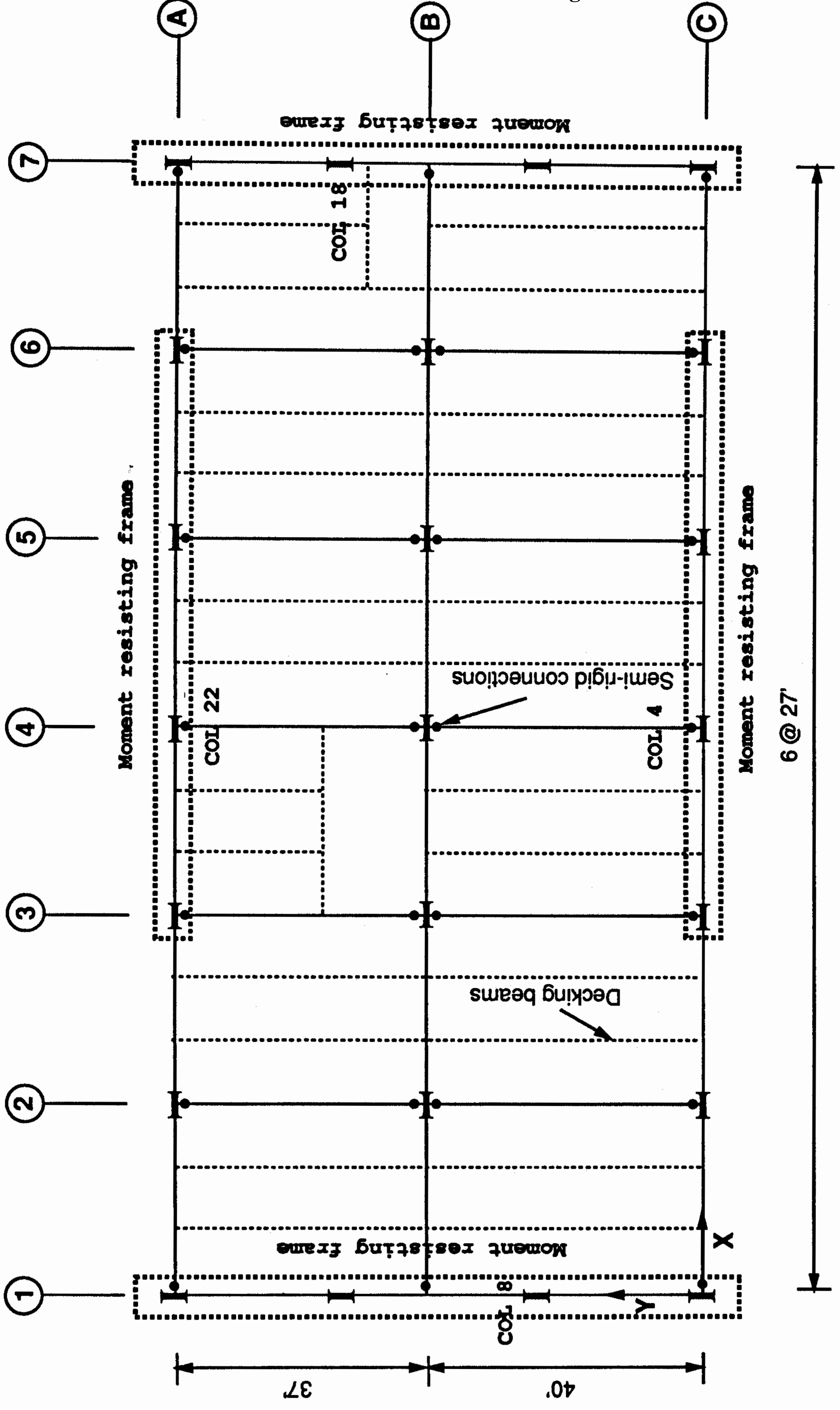

Figure 1: Framing plan

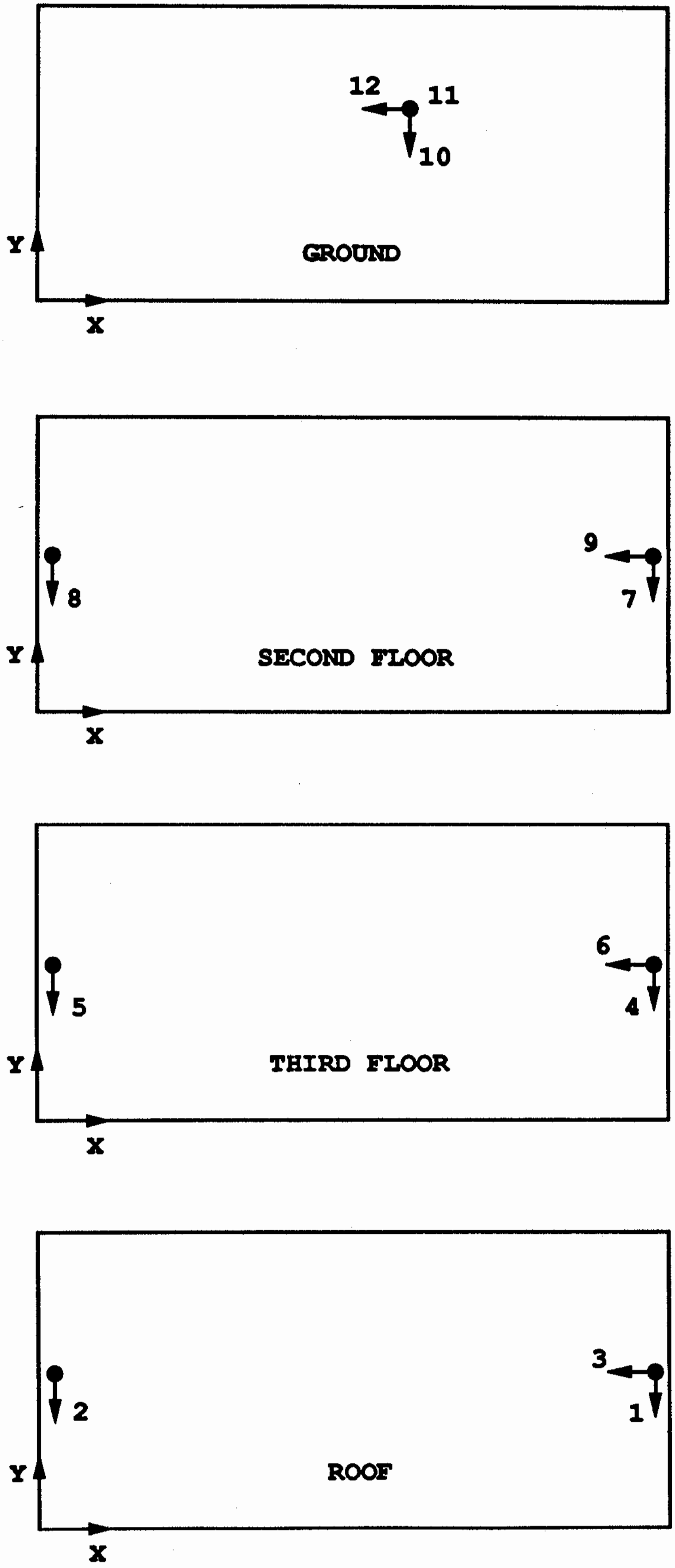

Figure 2: Instrument locations

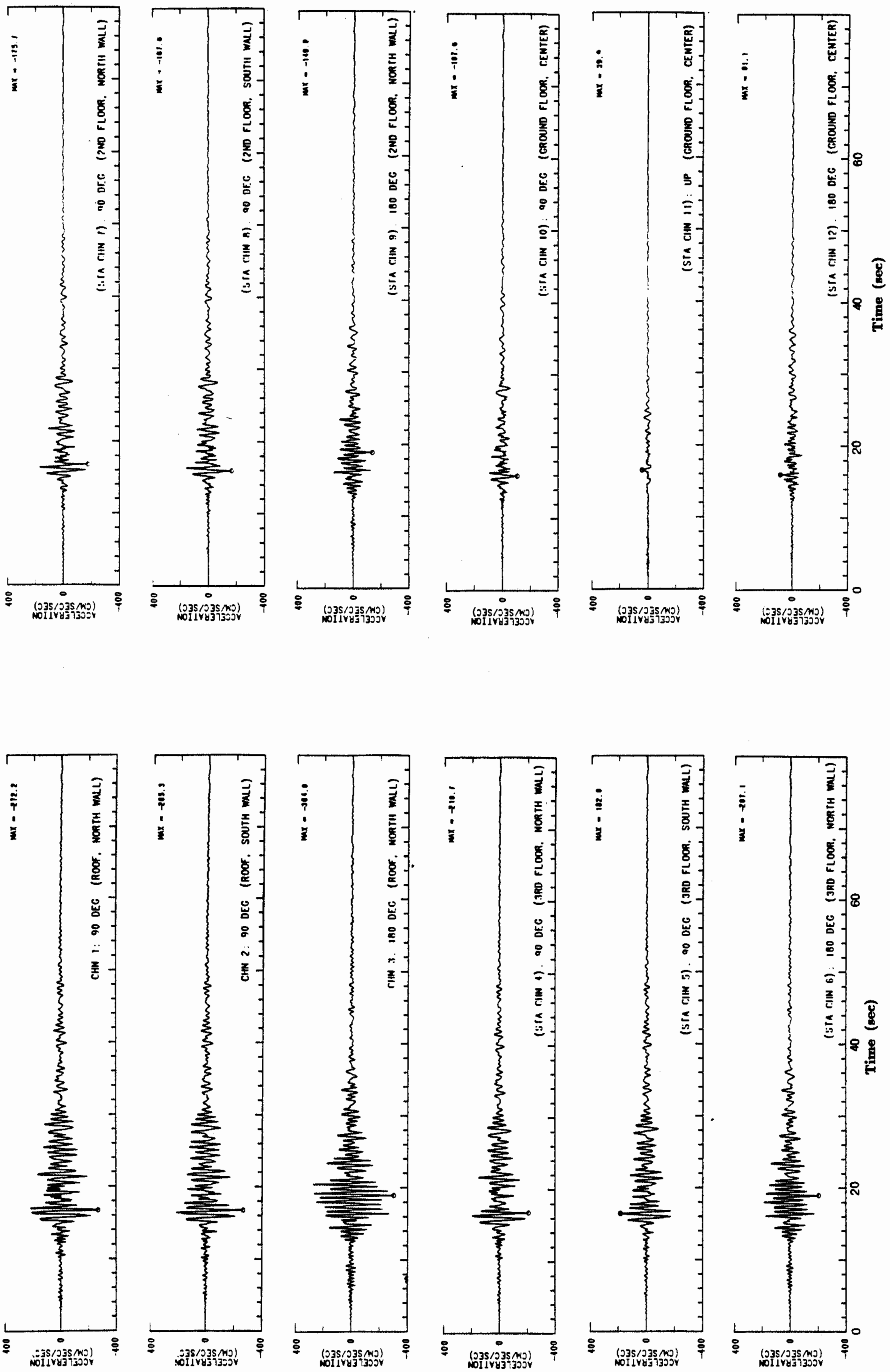

Figure 3: Recorded motions during Loma Prieta earthquake

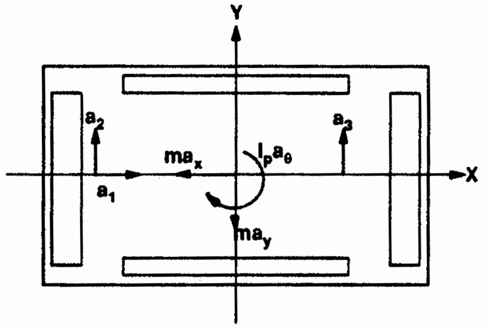

Figure 4: Recorded accelerations and floor forces

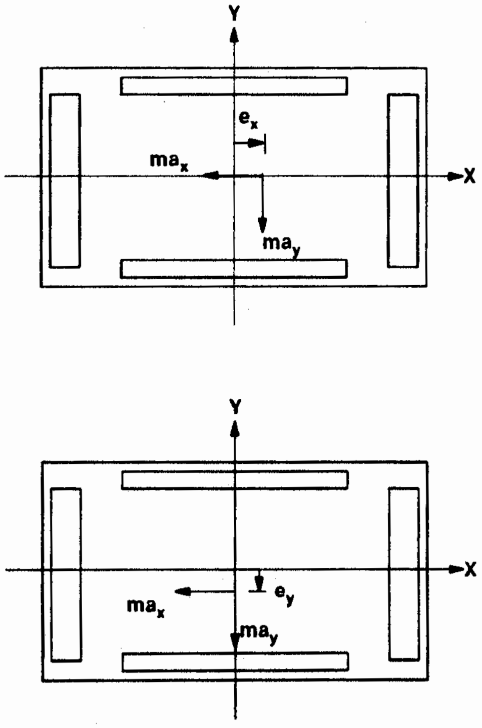

Figure 5: Accidental eccentricities, e_x and e_y

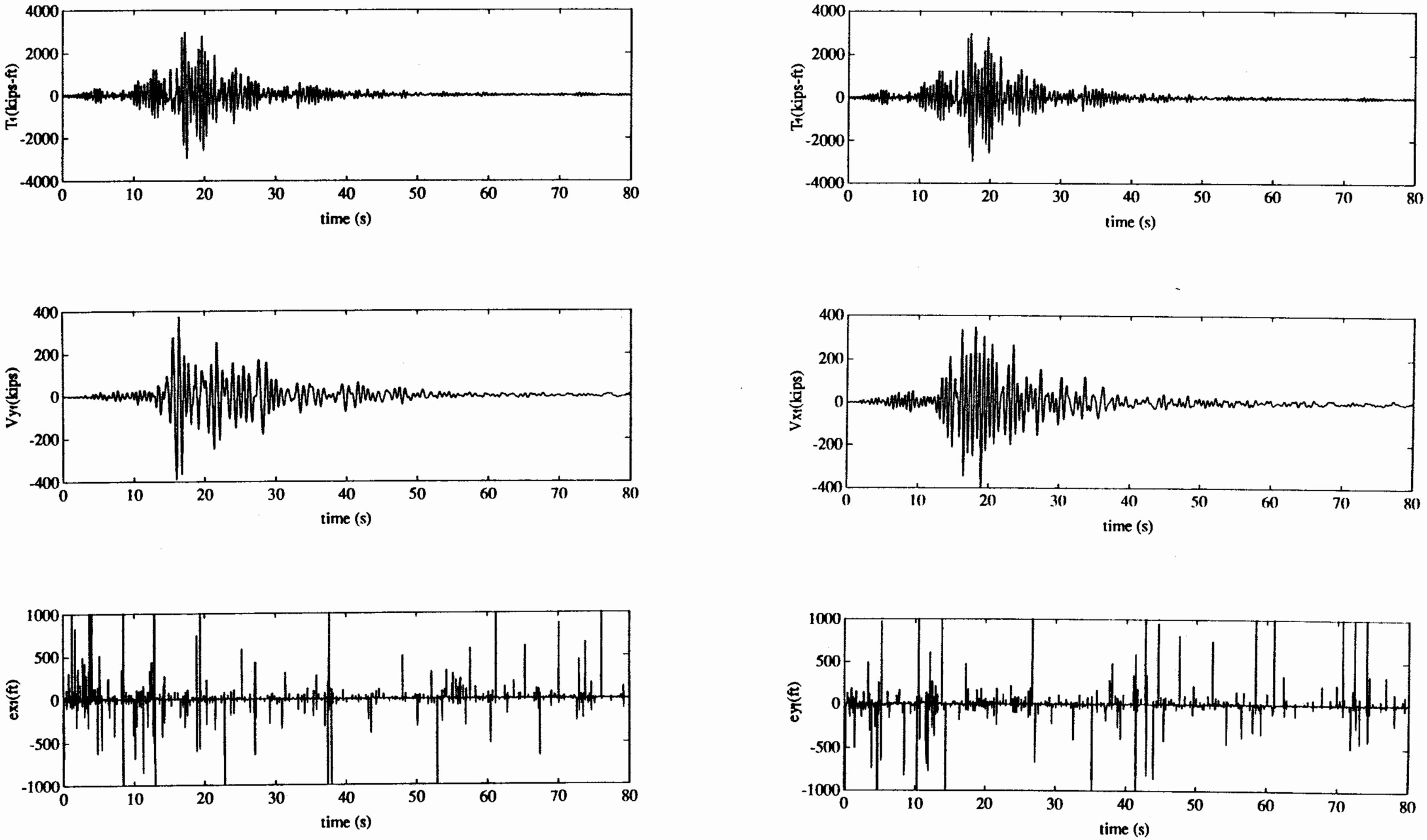

Figure 6: Base shears, base torque and first floor accidental eccentricities computed from recorded accelerations

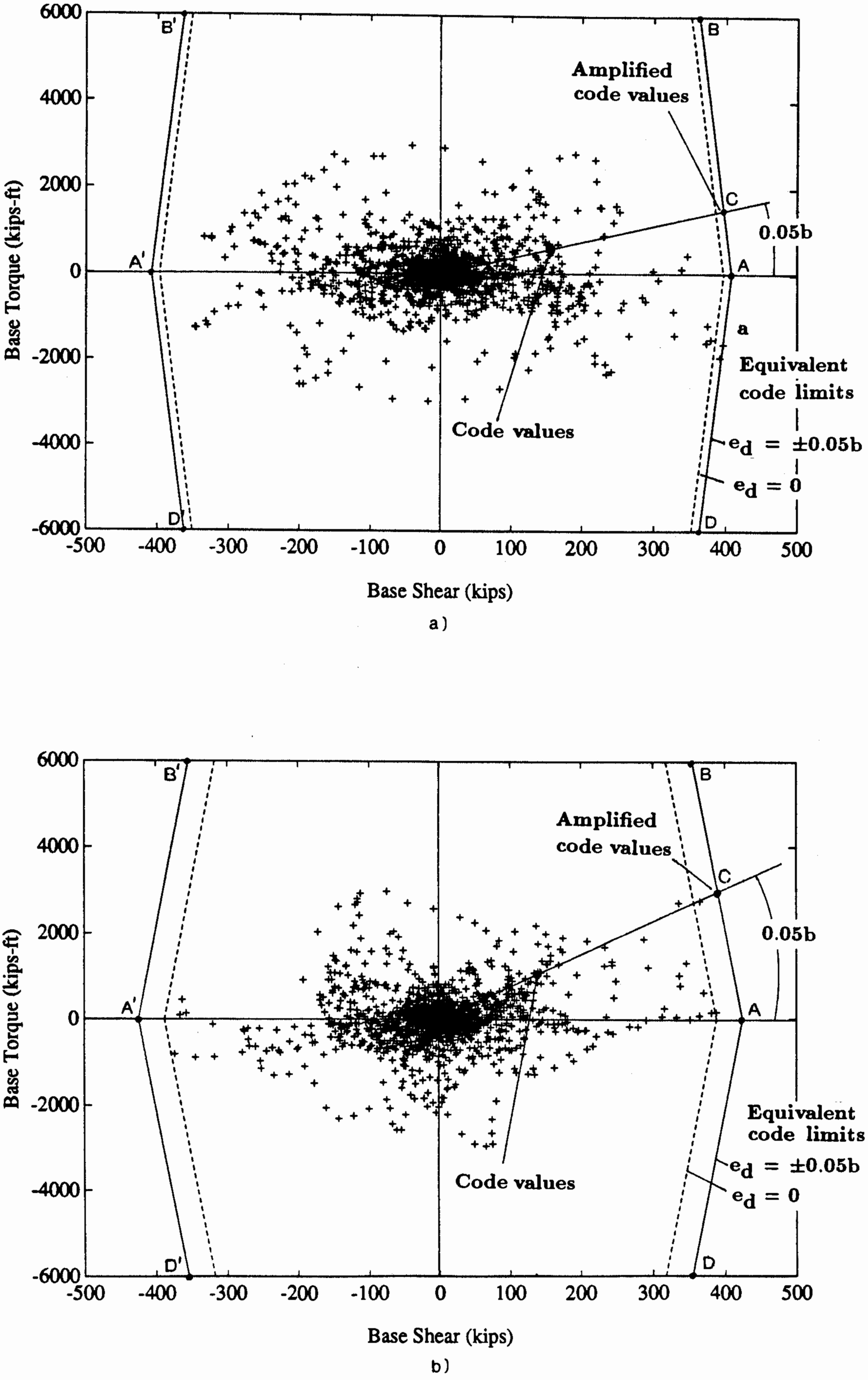

Figure 7: Comparison of dynamic base shear and base torque with code values, and "equivalent" code limits

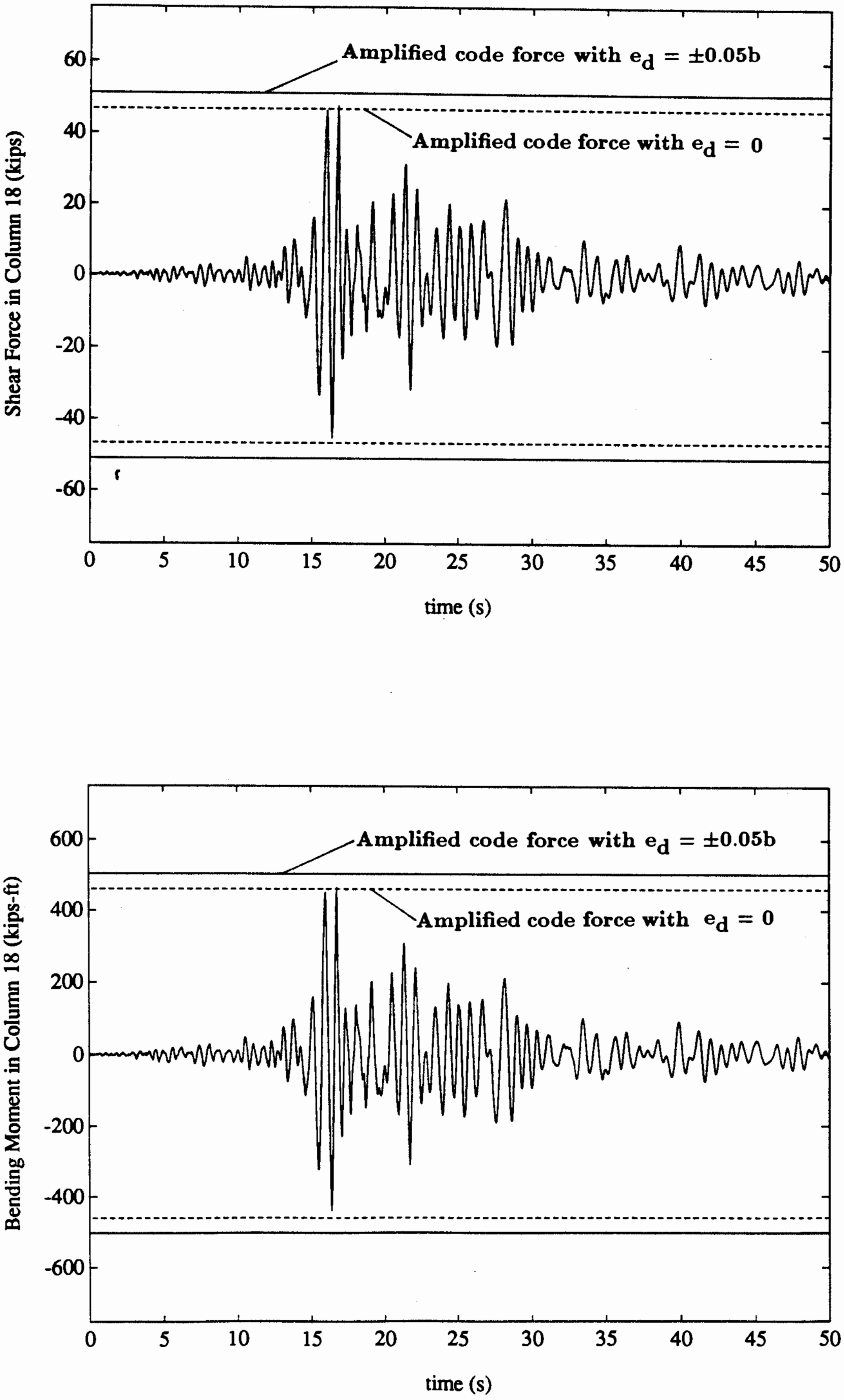

Figure 8: Comparison of earthquake-induced forces in column 18 with amplified code values

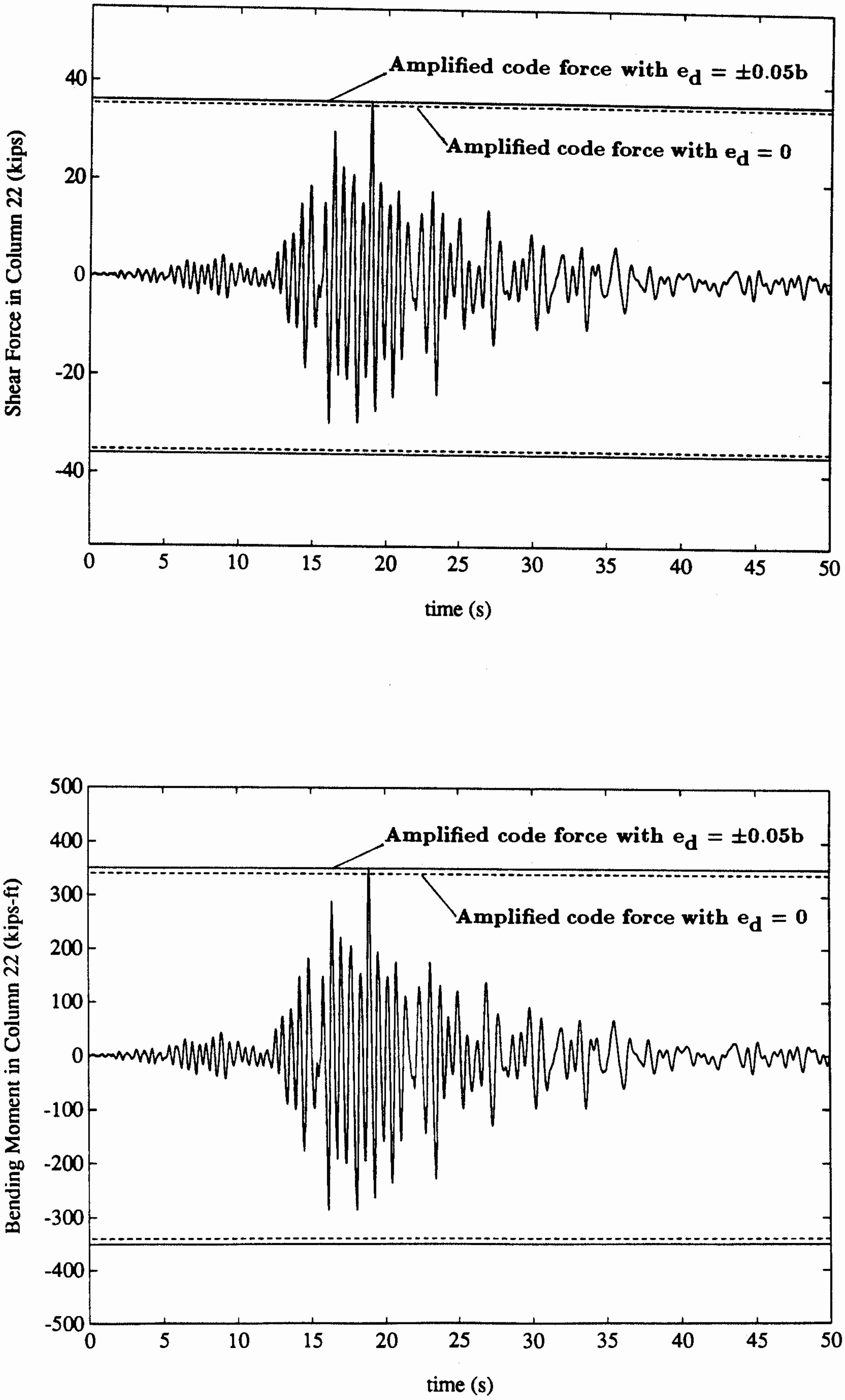

Figure 9: Comparison of earthquake-induced forces in column 22 with amplified code values