EVALUATION OF CODE-ACCIDENTAL TORSION PROVISIONS USING EARTHQUAKE RECORDS FROM THREE NOMINALLY SYMMETRIC-PLAN BUILDINGS

Juan Carlos De la Llera and Anil K. Chopra

Department of Civil Engineering
University of California at Berkeley

ABSTRACT

A procedure is presented for evaluating building code provisions for accidental torsion from analysis of recorded motions of nominally symmetric-plan buildings during earthquakes. This procedure is utilized to analyze the motions of three buildings recorded during recent California earthquakes. The results demonstrate that the accidental torsion specified by the Uniform Building Code is adequate in representing the torsion in the recorded motions of these three buildings. It is also concluded, although somewhat speculatively, that accidental torsion need not be considered in the design of many buildings.

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 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 perpendicular 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 building torsion arising from 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. Accidental torsion is to be considered in the design of buildings with asymmetric plans as well as symmetric plans; in the latter case, this is the total torsion to be considered.

Most of the research investigations of coupled lateral-torsional response of buildings, including the work aimed toward 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.

On the other hand, the subject of accidental torsion 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 analytically 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 provides 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. This is the approach adopted in this investigation.

BUILDINGS CONSIDERED AND RECORDED MOTIONS

Ideal for the purposes of this investigation would be buildings satisfying certain requirements--nominally-symmetric floor plans, rigid floor diaphragms, and negligible soil-structure interaction effects--that have experienced significant ground shaking, and three independent components of acceleration have been recorded at the ground level and at each floor. Three buildings which essentially satisfy the above requirements have been identified for the present study. A brief description of these three structures and their motions recorded during earthquakes is presented next.

BUILDING A

Identified as CSMIP Station No. 58506, this building is located in Richmond, California. A typical framing plan of this steel structure is shown in Fig. 1. The building has a nominally-symmetric floor plan. 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 interconnected. Additional information about this building is presented in the complete report.

Accelerographs recorded the motion of the building during the Loma Prieta earthquake (October, 1989), including three channels of horizontal motion at the second floor, third floor, and roof levels, and two channels of motion at the first (or ground) floor level. 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 structural damage during the earthquake.

BUILDING B

Identified as CSMIP Station No. 23511, this building is located in Pomona, California. This reinforced concrete frame building has two stories and a partial basement, and a light penthouse structure. The building has a nominally-symmetric floor plan, as indicated by its framing plan. The lateral force-resisting system in the building consists of peripheral columns interconnected by longitudinal and transverse beams. The "L"-shaped exterior corner columns as well as the interior columns in the building are not designed especially for earthquake resistance. The floor decking system is formed by a 6" concrete slab. The building also includes walls in the stairwell system--concrete walls in the basement and masonry walls in upper stories. Foundations of columns and interior walls are supported on piles. Additional information about this building is presented in the complete report.

Accelerographs recorded the motion of the building during the Whittier (October, 1987) and Upland (February, 1990) earthquakes, including three channels of horizontal motion at the second floor and roof levels and at the basement of the building. During the Whittier earthquake, the peak accelerations at the basement level were $0.046g$ in the X-direction and $0.05g$ in the Y-direction. These motions were amplified to $0.15g$ in both directions at the roof level. During the Upland earthquake, the peak accelerations at the ground level were $0.12g$ and $0.13g$ in the X- and Y-directions, respectively. These motions were amplified to $0.24g$ in the X-direction and $0.39g$ in the Y-direction at the roof level. The building experienced no structural damage during either earthquake.

BUILDING C

Identified as CSMIP Station No. 57562, this building is located in San Jose, California. The building considered is one of four similar wings around a central building. Each wing is isolated from the central building by a separation joint. A typical framing plan of this three-story steel structure is shown in Fig. 3. The triangular portion of the building (shown in dashed lines) is not part of any lateral moment-resisting frame of the structure. Thus, the building has a nominally-symmetric floor plan consisting of moment-resisting frames A, B, C, and D in the X-direction and frames 1 through 9 in the Y-direction. All other frames are designed to carry only gravity loads. The floor decking system is formed by a steel corrugated metal sheet filled with lightweight concrete. The foundation system consists of rectangular column footings interconnected by grade beams. Additional information about this building is available in the complete report.

Accelerographs recorded the motion of the building during the Loma Prieta earthquake, including three channels of horizontal motion at each of the roof, third, and first (ground) floor levels. The peak accelerations at the ground level are $0.2g$ in both lateral directions, X and Y. These motions were amplified to $0.58g$ in the X-direction and $0.68g$ in the Y-direction at the roof level. The building experienced no structural damage during the earthquake. The two horizontal components of acceleration and rotational acceleration at the second floor without any accelerographs were estimated using the procedure described in the complete report.

DYNAMIC ACCIDENTAL ECCENTRICITY

We first determine the accidental eccentricity for a nominally symmetric-plan building with rigid floor diaphragms directly from the recorded motions. At the i^{th} floor these recorded accelerations are denoted by $a_{1i}(t)$, $a_{2i}(t)$, and $a_{3i}(t)$, and such data are assumed to be available for all floors $i = 1, 2, \ldots, N$ (Fig. 4(a)). From the recorded motions of the i^{th} floor the X and Y acceleration components at the CM of the floor, $a_{Xi}(t)$ and $a_{Yi}(t)$, and the torsional acceleration, $a_{\theta i}(t)$, of the i^{th} floor diaphragm can be determined by a simple geometric transformation. The associated inertia forces are $m_i\, a_{Xi}(t)$ and $m_i\, a_{Yi}(t)$ in the X- and Y-directions, respectively, and the associated torque is $I_{pi}\, a_{\theta i}(t)$ where m_i is the i^{th} floor mass and I_{pi} is the polar moment of inertia of the i^{th} floor mass about the CM of the floor (Fig. 4(b)). The shears and torques in the j^{th} story are determined by simple statics from the floor inertia forces which are known from the floor masses and recorded accelerations: $V_{Xj}(t) = \sum_{i=j}^{N} m_i\, a_{Xi}(t)$, $V_{Yj}(t) = \sum_{i=j}^{N} m_i\, a_{Yi}(t)$, and $T_j(t) = \sum_{i=j}^{N} I_{pi}\, a_{\theta i}(t)$. These story shears and torque are statically equivalent to each of the following force sets: (1) V_{Xj} at the CM and V_{Yj} at eccentricity e_{Xj} (Fig. 4(c)) given by $e_{Xj}(t) = T_j(t)/V_{Yj}(t)$; and (2) V_{Yj} at the CM and V_{Xj} at eccentricity e_{Yj} given by $e_{Yj}(t) = T_j(t)/V_{Xj}(t)$. The time-dependent quantities $e_{Xj}(t)$ and $e_{Yj}(t)$ may be interpreted as the instantaneous accidental eccentricities for the j^{th} story.

From the recorded motions these accidental eccentricities were computed for the three selected buildings. The results for the first story of Building B during the Upland earthquake are presented in Fig. 5 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.

This approach to compute the accidental eccentricity is appealing because it is based exclusively on recorded motions and does not require idealization or analysis--static or dynamic--of the structure. However, the numerical results are not especially useful because the largest peaks in the eccentricity-time plot are usually associated with small values of the base shear, and can occur even during the trailing, weak portions of the building motions. 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. In order to consider the combined effects of shear and torque in evaluating the code provisions, however, static analysis of the structure becomes necessary.

STRUCTURAL IDEALIZATION

The natural vibration frequencies and modes of the buildings are computed and static analyses are performed at many time instants, but no dynamic analyses were necessary. For these analyses the three buildings were idealized consistent with the ETABS computer program where in the building mass is assumed to be lumped at the floor levels and the floor diaphragms are assumed to be rigid. The compatibility of axial deformations required in columns belonging to more than one moment-resisting frame is considered by analyzing each structure as a single three-dimensional frame with six degrees of freedom per joint (in contrast to the more common type of analysis that considers the structure as an assemblage of independent planar frames). A brief summary of the structural idealization for each building is presented next; additional details are available in the complete report.

BUILDING A

This building was treated as fixed at the level defined by the slab on grade. Each frame was modeled with appropriate 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 beam webs as pinned. Computed by the ETABS program, the natural vibration frequencies and shapes of the first mode in the X-direction, the first mode in the Y-direction, and the first torsional mode are presented in Table 1. These computed results are similar to the "actual" vibration properties in Table 1 determined from the recorded earthquake motions by the procedure described in the complete report.

BUILDING B

This building was treated as fixed at the level defined by the base of the columns because the pile foundations are very stiff. The structural idealization considers all structural elements, including those not intended to provide lateral resistance, such as the masonry walls in the stairwell system, because they may cause torsion of the building and contribute to its accidental eccentricity. The effective moment of inertia in the beams was calculated assuming cracked sections and including the contribution of the concrete slab. The actual variation of moment of inertia along the span was considered in modeling the tapered beams along axes 2, 3, 4, and 5 (Fig. 2). The effective moment of inertia in columns was calculated assuming gross section properties.

Computed by the ETABS program, the natural vibration frequencies and shapes of the first mode in the X-direction, first mode in the Y-direction, and first torsional mode are presented in Table 1. These computed results agree reasonable well with the "actual" vibration properties determined from the recorded earthquake motions. As expected, the computed vibration properties are closer to the "actual" values from the less intense Whittier earthquake motions than from the more intense Upland earthquake motions. The higher intensity of shaking during the Upland earthquake, combined with the stiffness degradation during the earlier Whittier earthquake, leads to lower vibration frequencies during the Upland earthquake.

BUILDING C

This building was treated as fixed at the level of the slab on grade. The structural idealization includes all structural elements, including those that provide little lateral resistance, such as the triangular portion of the building (Fig. 3), because they may cause torsion of the building and contribute to its accidental eccentricity. Each frame was modeled with appropriate beam-column connections: moment-resistant (or rigid) connections and pinned connections as defined in the original structural drawings of the building. Computed by the ETABS program, the natural vibration frequencies and shapes of the first mode in the X-direction, the first mode in the Y-direction, and the first torsional mode are presented in Table 1. These computed results are similar to the "actual" vibration properties in Table 1 determined from the recorded earthquake motions by the procedure described in the complete report.

BASE SHEAR AND BASE TORQUE

As mentioned in a preceding section, the combined effects of shear and torque must be considered in evaluating the accidental torsion provisions in building codes. For each of the three buildings the base shears $V_{X1}(t)$ and $V_{Y1}(t)$ and base torque $T_1(t)$ have already been computed from the recorded accelerations. Consistent with the code approach of two independent lateral-force analyses in two orthogonal directions, X and Y, we consider the combined effects of V_{Y1} and T_1 separately from the combined effects of V_{X1} and T_1; only the first pair is considered in the following presentation and the modification for the other pair is obvious. Figure 6 shows the base shear $V_{Y1}(t)$ and base torque $T_1(t)$ for Building A during the recorded earthquake wherein each point (+) denotes the combination of V_{Y1} and T_1 values at a particular time instant; there are as many points as the time instants considered. The point C in Fig. 6 identifies the code value of base shear $V_{code} = (ZIC/R_w)W$ and base torque which, for a nominally-symmetric building, is $T_{code} = (0.05b)\,V_{code}$. In computing the coefficient C, the fundamental vibration period T was taken equal to the "actual" value in Table 1, and R_w as 12. The fact that the base shear during the earthquake exceeds the code value of base shear at many time instants is consistent with the well known fact that the actual capacity of most buildings is much larger than the design base shear. In order to evaluate the code-accidental torsion provisions, we also show the point C_a which denotes the maximum value of actual base shear $(V_{Y1})_o = \max_t |V_{Y1}(t)|$ and $T_1 = (0.05b)(V_{Y1})_o$. However, it is by no means obvious whether the pair of actual forces $V_{Y1}(t)$ and $T_1(t)$ at a particular time instant is more or less "critical" to the structure than the amplified "code" forces denoted by C_a. Note that so far no structural analysis was necessary.

In order to resolve this issue, we determine all combinations of base shear and base torque which, when considered as static forces, produce the same member force as the amplified code forces denoted by C_a. These code-equivalent combinations shown, for example, in Fig. 6 for Building A are determined by static analysis of the building as follows:

1. The maximum value of base shear $V = (V_{Y1})_o$ determined from floor accelerations may be defined as the amplified "code" base shear.

2. Analyze the structure using a static code-type analysis considering: (a) base shear as given in Step 1; (b) heightwise distribution of lateral floor forces according to the code; and (c) accidental eccentricity, equal to $0.05b$ in the Uniform Building Code, in the most unfavorable direction for each element. The resulting base shear V and base torque T are shown as point C in Fig. 8(e). A member force computed by this analysis is defined as a member "design" force. The analysis required in Step 2 is shown conceptually in Fig. 8(a), where F_i ($i = 1,2,3$) are the lateral floor forces in the Y-direction, defined by Steps 2a and 2b. The resulting "design" shear V_{c1}^D in column 1 is obtained by applying the story lateral forces at a distance equal to $0.05b$ to the right of the CM. Analogously, the "design" shear V_{c2}^D in column 2 is obtained by applying the same floor forces at a distance of $0.05b$ to the left of the CM.

3. Determine the value of base shear and the associated lateral floor forces distributed over the building height according to the code which, applied at the CM (without any floor torques or eccentricity), produce the same member "design" force as determined in Step 2. This base shear is identified by points A_c and A'_c in Fig. 8(e). The analysis required in Step 3 is shown conceptually in Fig. 8(b). The building subjected to the lateral floor forces F_1, F_2, and F_3 of Steps 2a and 2b applied at the CM of the floors is analyzed to determine V_{c1}^S and V_{c2}^S, the shear forces in columns 1 and 2, respectively. The lateral forces F_i and base shear V multiplied by the ratio V_{ci}^D / V_{ci}^S ($i = 1,2$) acting alone (without any floor torques or eccentricity) would produce in column "i" the shear force V_{ci}^D, which is equal to the member "design" force determined in Step 2. In the case of column 1 this base shear, $V_{c1}^o = (V_{c1}^D / V_{c1}^S)\,V$, defines the points A_{c1} and A'_{c1} in Fig. 8(e). Similarly, $V_{c2}^o = (V_{c2}^D / V_{c2}^S)\,V$ defines the points A_{c2} and A'_{c2} in Fig. 8(e).

4. Determine the value of base torque and the associated floor torques distributed over the building height in the same proportion as the lateral floor forces which alone (without any lateral forces) produce the same "design" force in a selected member as determined in Step 2. This torque is identified by points B_c and B'_c in Fig. 8(e). The analysis required in Step 4 is shown conceptually in Fig. 8(c). The building subjected to story torques T_i, where $T_i = 0.05b\,F_i$ and F_i are known from Steps 2a and 2b, is analyzed to determine V_{c1}^T and V_{c2}^T, the shear forces in columns 1 and 2, respectively. The floor torques T_i and base torque T multiplied by the ratio V_{ci}^D / V_{ci}^T ($i = 1,2$) acting alone (without any lateral forces) would produce the "design" shear force V_{ci}^D in column "i." In the case of column 1 this base torque $T_{c1}^o = (V_{c1}^D / V_{c1}^T)\,T$ defines the points B_{c1} and B'_{c1} in Fig. 8(e). Similarly, $T_{c2}^o = (V_{c2}^D / V_{c2}^T)\,T$ defines the points B_{c2} and B'_{c2} in Fig. 8(e).

5. Each point on lines $A_{c1}B_{c1}$ and $A'_{c1}B'_{c1}$ denotes a combination of base shear and base torque, each being distributed over the building height according to the code (Steps 3 and 4) which produces the same member "design" force as determined in Step 2; hence, lines $A_{c1}B_{c1}$ and $A'_{c1}B'_{c1}$ are called "code-equivalent combinations" associated with column 1. Similarly, $A_{c2}B_{c2}$ and $A'_{c2}B'_{c2}$ are the "code-equivalent combinations" associated with column 2.

If at each time instant the "actual" base shear and base torque combination falls within the region enclosed by the code-equivalent limits, this implies that, during the earthquake, the force in the selected member did not exceed the "design" value determined in Step 2. Alternatively, such a situation indicates that the accidental eccentricity of $0.05b$ is conservative during the particular earthquake. Any point in the base shear-torque plot which falls outside the region enclosed by code-equivalent combination represents, at a particular time instant, a combination of base shear and base torque that produces in the selected member a force that is larger than its "design" value. Alternatively, this situation indicates that the accidental eccentricity of $0.05b$ is unconservative at that instant of time.

The above-described procedure was utilized to determine the code-equivalent combinations of base shear and base torque for Building A, and the results are presented in Fig. 6. Analysis for Y-lateral forces with the shear forces in columns 8 and 18 selected as the member "design" forces led to the code-equivalent combinations of Fig. 6(b). Similarly, analysis for X-lateral forces with the shear forces in columns 4 and 22 selected as the member "design" forces led to the code-equivalent combinations of Fig. 6(a). These results demonstrate that all points denoting "actual" values of base shear and base torque during the earthquake fall inside the region enclosed by the code-equivalent combinations with one exception: point A in Fig. 6(a), which indicates that only at that instant of time during the earthquake, the shear force in column 22 exceeds the "design" force. This observation is consistently confirmed by examining the code-equivalent limits for the "design" shear forces and bending moments in several other beams and columns. For the recorded response of Building A during the Loma Prieta earthquake, the torsional effects are so small that it may not be necessary to consider accidental eccentricity at all. Figure 6 indicates that very few points fall outside the region enclosed by the code-equivalent combinations with zero accidental eccentricity.

Figure 9 shows the dynamic base shear-torque values, and code-equivalent combinations determined from the motions of Building B recorded during the Whittier earthquake. Similar results for the Upland earthquake are presented in Fig. 10. Analysis for X-lateral forces with the shear forces in columns 2 and 29 selected as the member "design" forces led to the code-equivalent combinations of Figs. 9(a) and 10(a). Similar analysis for Y-lateral forces with the shear forces in columns 8 and 25 selected as the member "design" forces led to the code-equivalent combinations of Figs. 9(b) and 10(b). Only at two time instants during the Whittier earthquake does the "actual" shear force in column 29 exceed the "design" force. During the Upland earthquake, the "actual" forces in all columns remain below their respective design values. In fact, the design value with zero accidental eccentricity is exceeded only once, suggesting that it is not even necessary to consider any accidental eccentricity for the recorded response of Building B during the Upland earthquake.

The actual values of the Y component of the base shear and base torque for Building C during the Loma Prieta earthquake are presented in Fig. 7(b). This plot shows a trend towards the second and fourth quadrants which implies that the dynamic forces in structural elements located on the left side of the CM of the structure (Fig. 3), e.g. column 1, and more likely to exceed their "design" values. This speculation is confirmed in Fig. 7 which shows that at a few time instants the actual shear force in the first story exceeds the design value.

MEMBER FORCES

An alternative procedure to the one presented in the preceding section for evaluating the code-accidental torsion provisions is to compare the member "design" forces defined in Step 2 of the preceding section with the time history of the "actual" member forces during the earthquake. At each time instant the "actual" member forces during the earthquake are determined by static analysis of the building subjected to the floor inertia forces $m_j a_{xj}(t)$, $m_j a_{yj}(t)$, and $I_{pj} a_{\theta j}(t)$ at all floors, i.e. $j = 1, 2, \ldots, N$ (Fig. 8(d)). If at all time instants the "actual" member force is less than its "design" value, the accidental eccentricity of $0.05b$ can be interpreted to be conservative during the particular earthquake. Conversely, the accidental eccentricity of $0.05b$ is unconservative at those time instants when the "actual" member force exceeds the "design" value. The two procedures are equivalent for a symmetric one-story system but differ slightly for multistory buildings because the actual heightwise distribution of lateral forces computed from recorded accelerations and floor masses is not identical to the heightwise distribution of lateral forces specified by the code.

The time variation of the "actual" shear force in the first-story columns 22 and 18 of Building A during the Loma Prieta earthquake is presented in Fig. 11, together with the "design" values of these forces obtained by static analysis of the building for amplified code forces in the X-direction (for column 22) and in the Y-direction (for column 18). The "actual" values of these member forces do not exceed their "design" values based on the specified accidental eccentricity and barely exceed the design values ignoring this eccentricity. The results for shear force and bending moment in all columns support this conclusion.

The time variation of the "actual" shear force in the first-story columns 8 and 26 of Building B during the Whittier earthquake is presented in Fig. 13, together with the "design" values of these forces obtained by static analysis for amplified "code" forces in the X-direction (for column 26) and in the Y-direction (for column 8). Similar results obtained from the Upland earthquake records are presented in Fig. 14. For both earthquakes the "actual" values for these member forces do not exceed their "design" values based on the code-specified accidental eccentricity and barely exceed the design values ignoring this eccentricity. The results for shear force and bending moment in all columns in the building support this conclusion.

The time variation of the "actual" shear force in the first-story columns 1 and 8 of Building C during the Loma Prieta earthquake is presented in Fig. 12, together with the "design" values of these forces obtained by static analysis of the building for amplified code forces in the X-direction (for column 8) and in the Y-direction (for column 1). The "actual" value of the X-component of the shear force in the first-story column 8 does not exceed its "design" value based on the code-specified accidental eccentricity and barely exceeds the design value ignoring this eccentricity. The results for shear forces and bending moments in all columns associated with motion of the building in the X-direction support this conclusion. The "actual" value of the Y-component of the shear force in the first-story column 1 exceeds its "design" value for a small fraction of a second three times during the earthquake. The maximum value of the "actual" shear during the earthquake is 10% greater than its "design" value. These observations are representative of other columns at the left edge of the plan. The "actual" forces in columns located to the right of the CM remain below their "design" values throughout the earthquake.

Accidental torsion is seen to be more significant in the response of Building C than the other two buildings. This may be the result of several-factors: the natural vibration periods of the first three--two lateral and one torsional--vibration modes are very close to each other--a situation known from forced vibration tests to create strong coupling of lateral and torsional motions even in nominally-symmetric buildings; the torsional component of the ground motion is significant; and the restraint provided by the adjacent building may have contributed to accidental torsion.

For the three buildings and their motions during past earthquakes considered in this investigation, the actual member forces exceed their design values based on the UBC-specified accidental torsion by at most 10% for a small fraction of a second three times during an earthquake. These discrepancies between the design force and the actual force are small when considered in the context of the many larger approximations inherent in building code provisions, uncertainties in building idealization and material properties, etc. Thus, the accidental torsion provisions in building codes are satisfactory in representing the torsional motions of these three buildings during the particular earthquakes.

The design values of member forces determined by ignoring the accidental torsion are, of course, exceeded more often and by a larger amount. During the earthquakes considered, a member design force is exceeded once for a small fraction of a second by less than 3% in Building A, once for a small fraction of a second by less than 10% in Building B, and five times, each for a small fraction of a second, by less than 38% in Building C. Such increased force demand should not be a problem for most well designed buildings with nominally-symmetric floor plan for two reasons. Firstly, the overstrength relative to design values that is typical of most buildings would, for moderate ground motion, be sufficient for the building to withstand the increased force demand essentially within the elastic range. Secondly, even if the force demands exceeded structural capacity because of accidental torsion, the damaging effects of the very few and small inelastic excursions of very short duration would be very small.

During strong ground motions, most buildings would be expected to deform beyond the elastic range and accidental torsion may increase the ductility demand for some structural frames or elements of a building designed without considering accidental eccentricity. However, the results from analyzing the recorded motions of these three buildings considered suggest that the additional ductility demand should be small. Thus, if these three buildings were designed ignoring accidental eccentricity, but detailed for sufficient ductility for the design earthquake, their performance should not be adversely affected by accidental torsion.

Thus, it seems that accidental torsion need not be considered in the design of these three buildings for the recorded ground motions or amplified versions of these ground motions. Although extrapolating these observations to other situations is somewhat speculative, it is difficult to visualize that the design of nominally-symmetric buildings would be influenced significantly by accidental torsion, or that torsional response could be a significant contributor to the damage such a building may experience during an earthquake. Exceptions may occur if the natural vibration periods of the fundamental lateral and torsional modes of the building are very close to each other, or the torsional vibration period is much longer than the lateral period, or if the ground motion contained an unusually strong torsional component of ground motion.

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Table 1: Natural Vibration Periods and Modes Shapes for Buildings A, B and C

Vibration Properties	X-lateral mode		Y-lateral mode		Torsional mode	
	Recorded	Computed	Recorded	Computed	Recorded	Computed
	Building A: Loma Prieta Earthquake					
Period (sec)	0.63	0.60	0.74	0.76	0.46	0.45
Mode Shape						
Roof	1.00	1.00	1.00	1.00	1.00	1.00
3^{rd} Floor	0.71	0.77	0.72	0.73	0.72	0.76
2^{nd} Floor	0.39	0.57	0.39	0.38	0.40	0.43
	Building B: Whittier Earthquake					
Period (sec)	0.29	0.28	0.27	0.27	0.20	0.20
Mode Shape						
Roof	1.00	1.00	1.00	1.00	1.00	1.00
2^{nd} Floor	0.62	0.61	0.39 ?	0.60	0.57	0.64
	Building B: Upland Earthquake					
Period (sec)	0.30	0.28	0.28	0.27	0.21	0.20
Mode Shape						
Roof	1.00	1.00	1.00	1.00	1.00	1.00
2^{nd} Floor	0.64	0.61	0.55	0.60	0.52	0.64
	Building C: Loma Prieta Earthquake					
Period (sec)	0.67	0.70	0.69	0.69	0.69 - 0.65	0.67
Mode Shape						
Roof	1.00	1.00	1.00	1.00	1.00	1.00
3^{rd} Floor	0.80	0.70	0.70	0.67	0.67	0.66
2^{nd} Floor	0.44	0.33	0.33	0.30	0.31	0.30

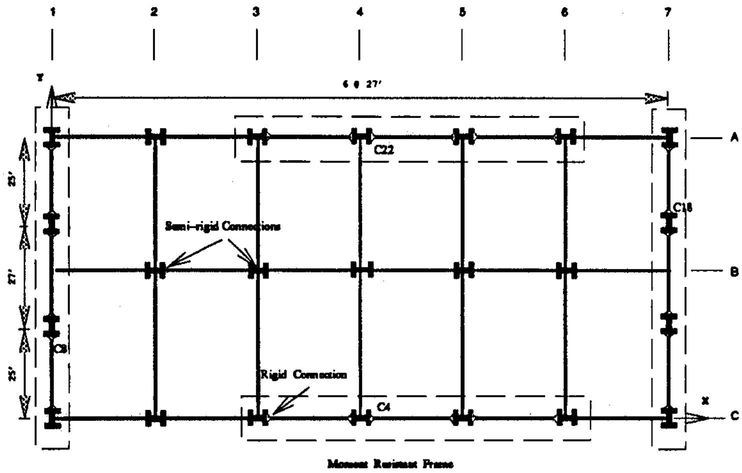


Figure 1 : Framing Plan of Building A

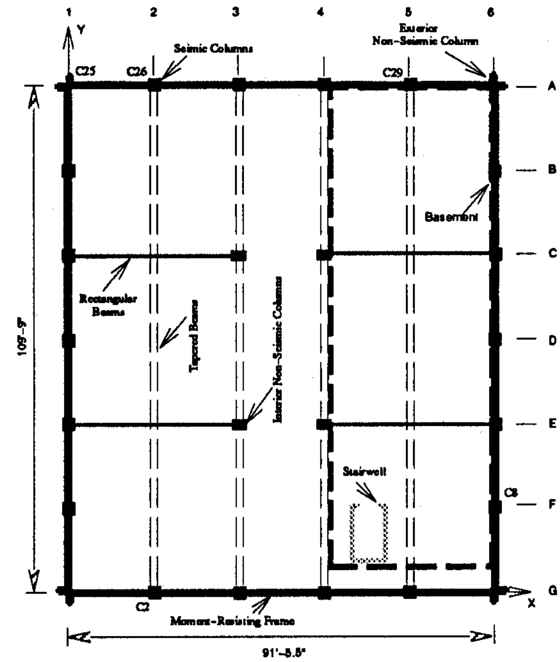


Figure 2 : Framing Plan of Building B

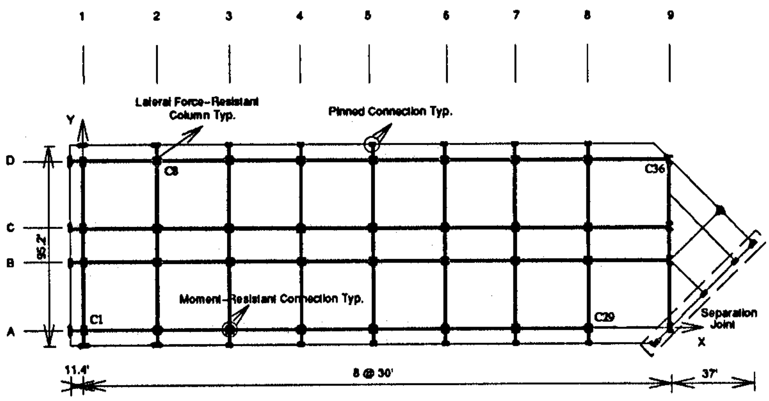
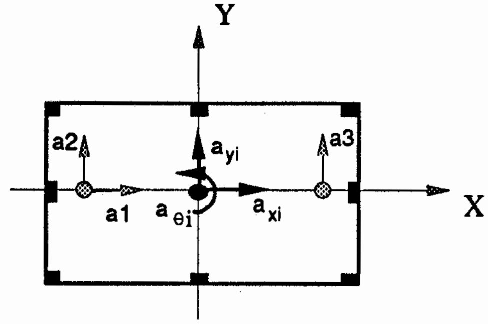
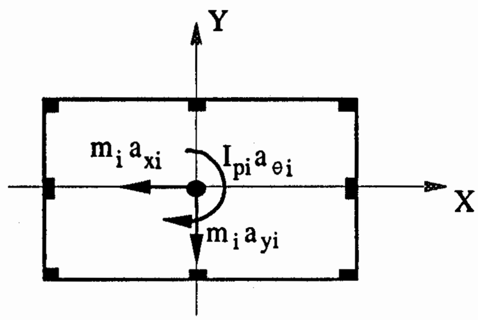


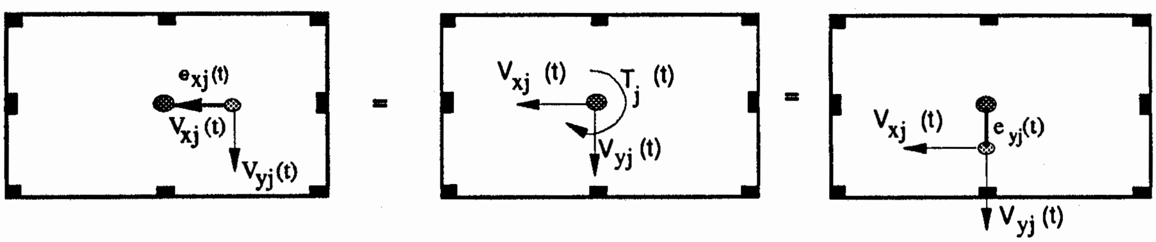
Figure 3 : Framing Plan of Building C



(a) Recorded Accelerations at ith Floor and Accelerations at the CM



(b) Inertia Forces at ith Floor



(c) Accidental Eccentricities for the jth Story

Figure 4 : Dynamic Accidental Eccentricity

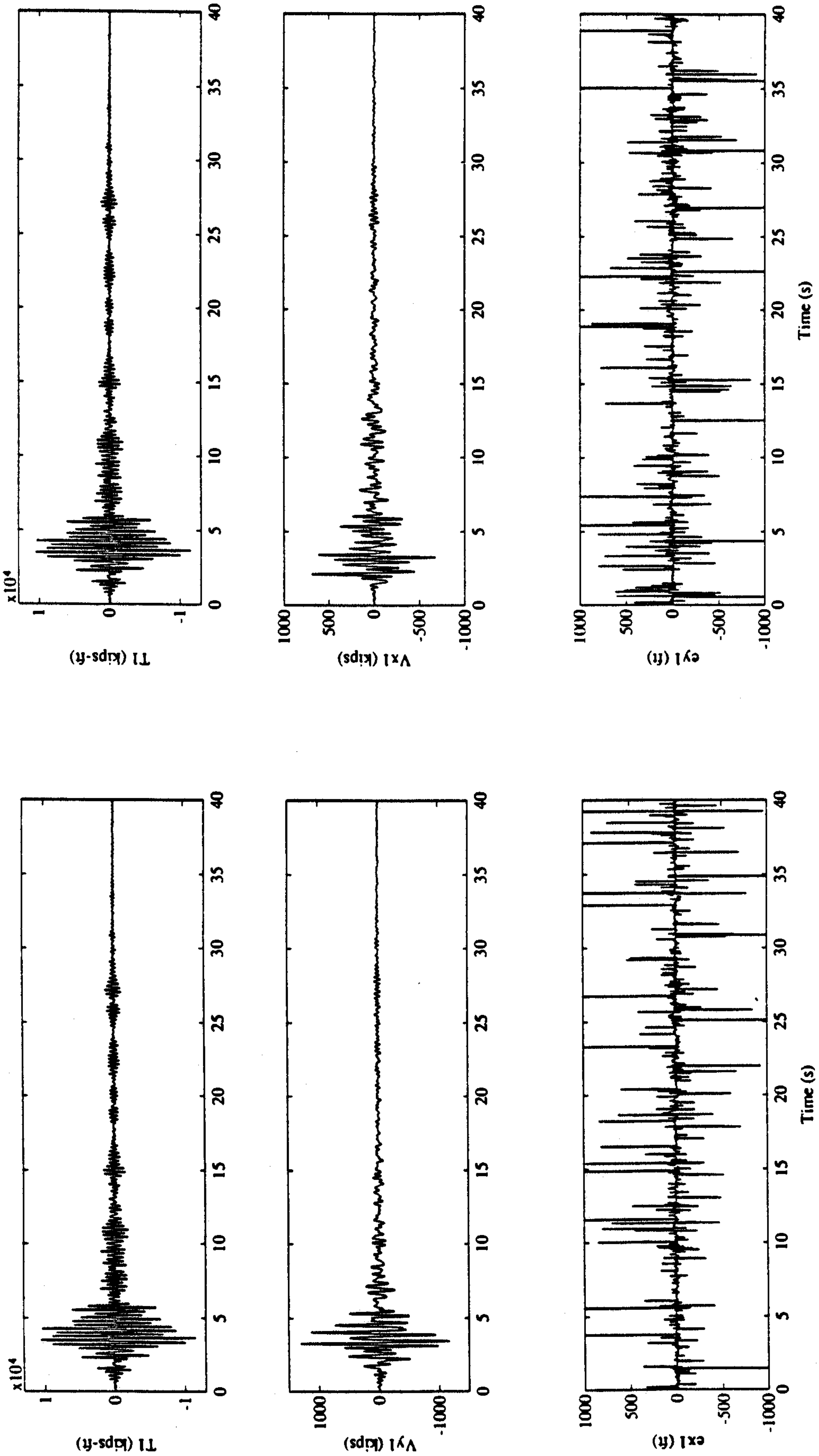


Figure 5: Base Shear, Base Torque and First Floor Accidental Eccentricities Computed from Recorded Accelerations in Building B During the Upland Earthquake

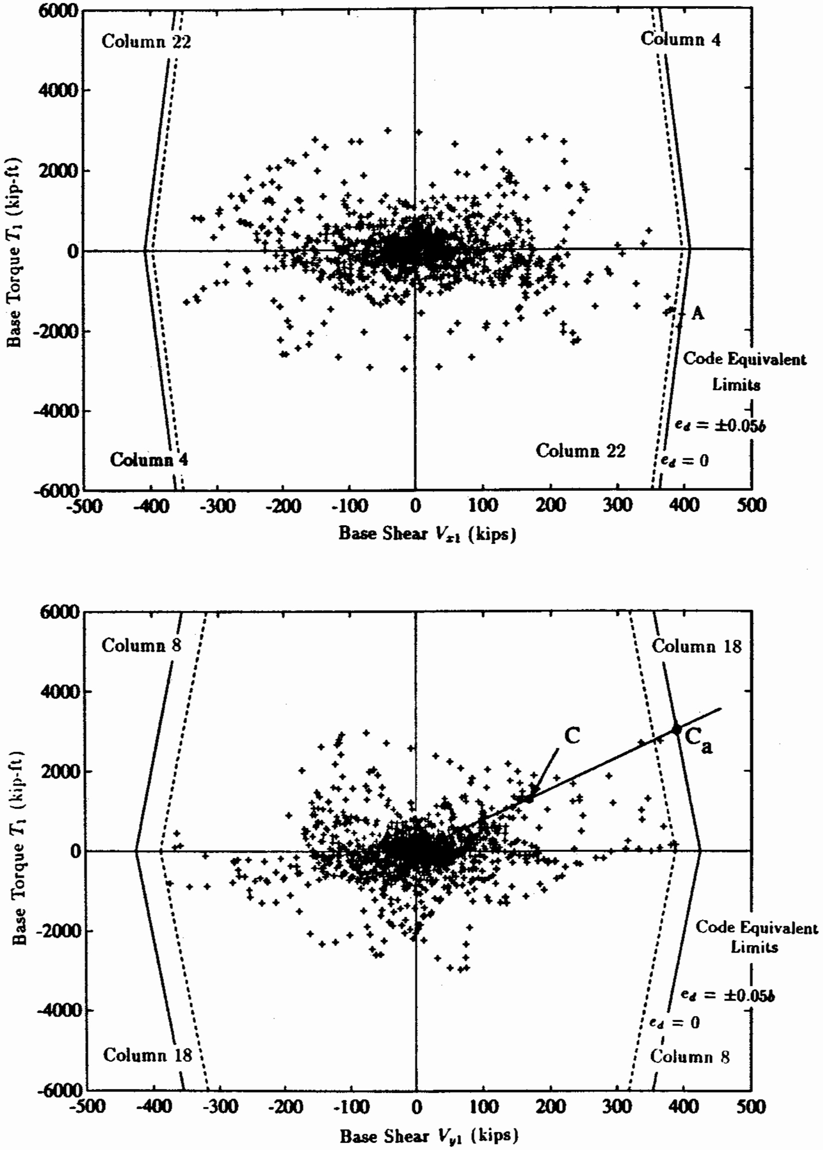


Figure 6: Comparison of Dynamic Base Shear, Base Torque and "Code Equivalent Combinations" in Building A During the Loma Prieta Earthquake

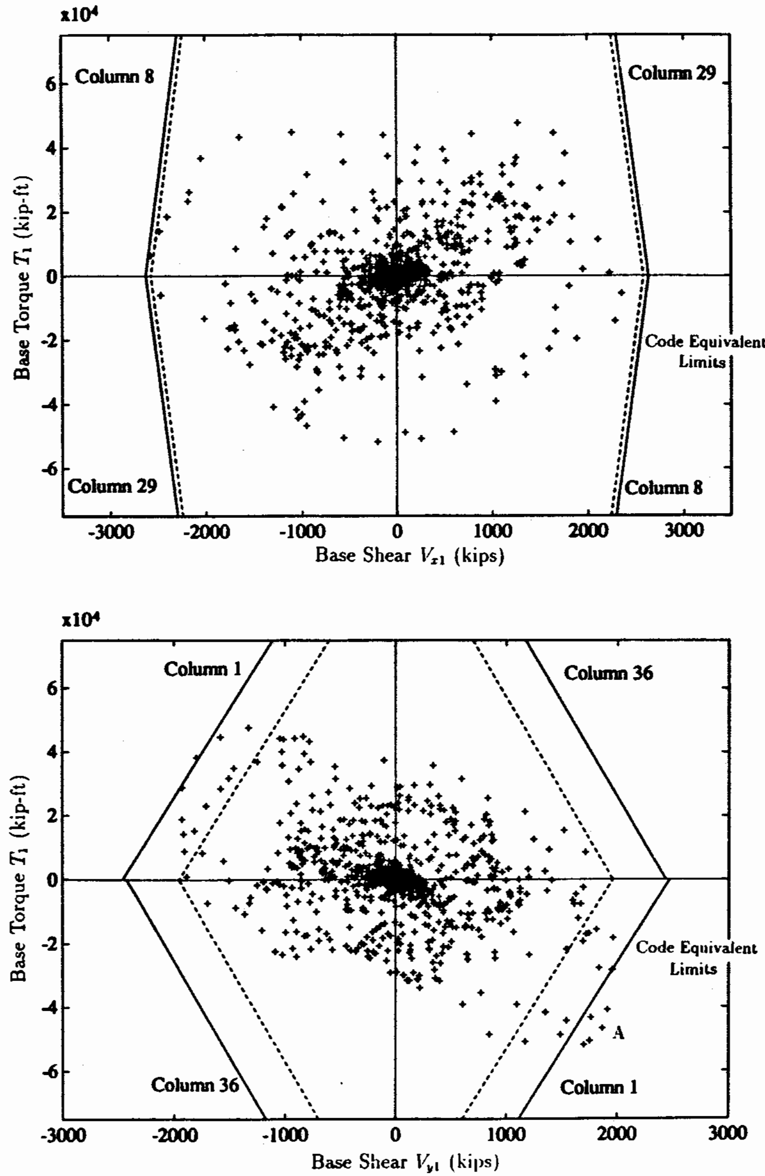


Figure 7: Comparison of Dynamic Base Shear, Base Torque and "Code Equivalent Combinations" in Building C During the Loma Prieta Earthquake

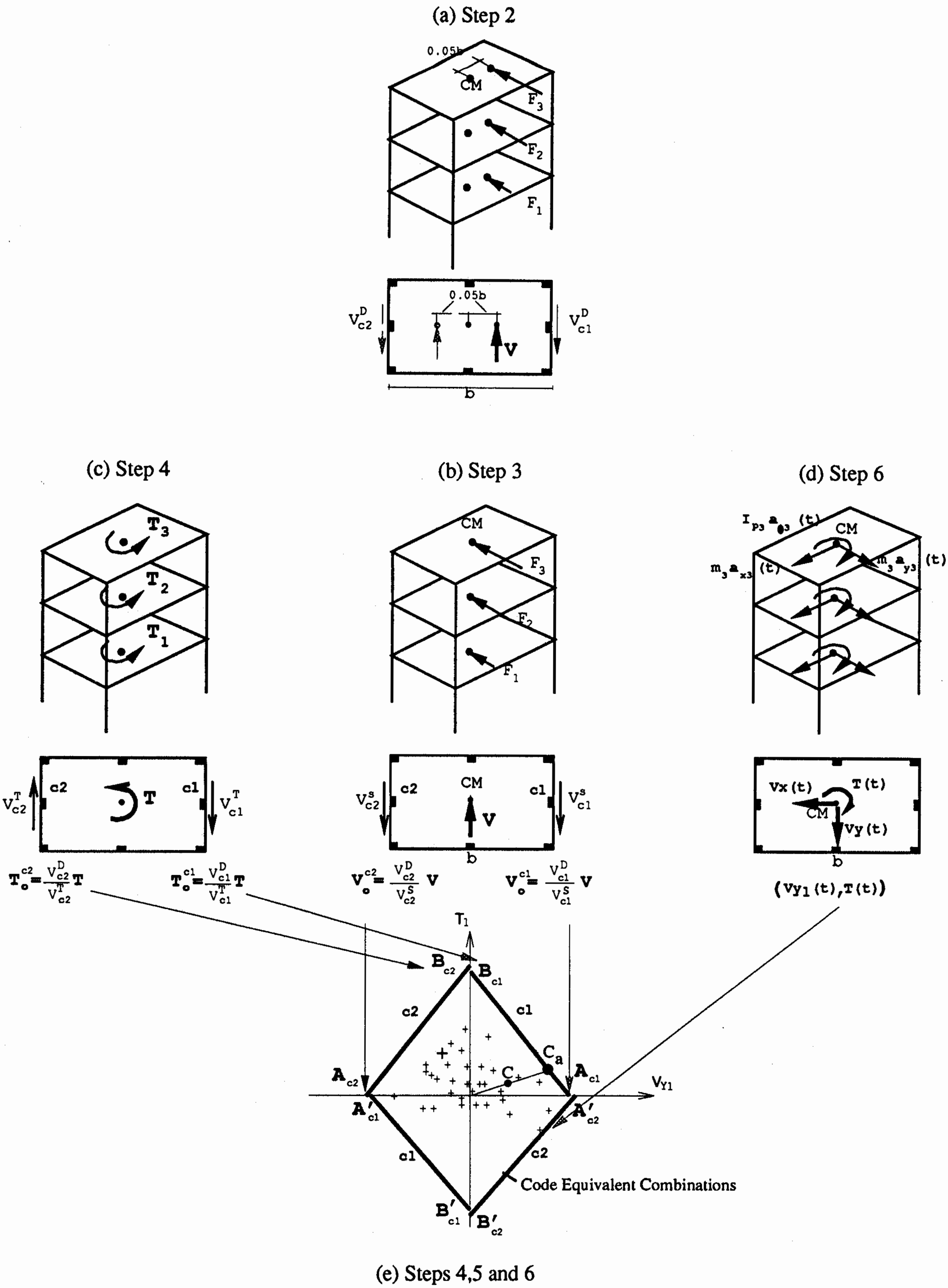


Figure 8: Computation of Design Member Forces, Base Shear, Base Torque, and Code Equivalent Limits

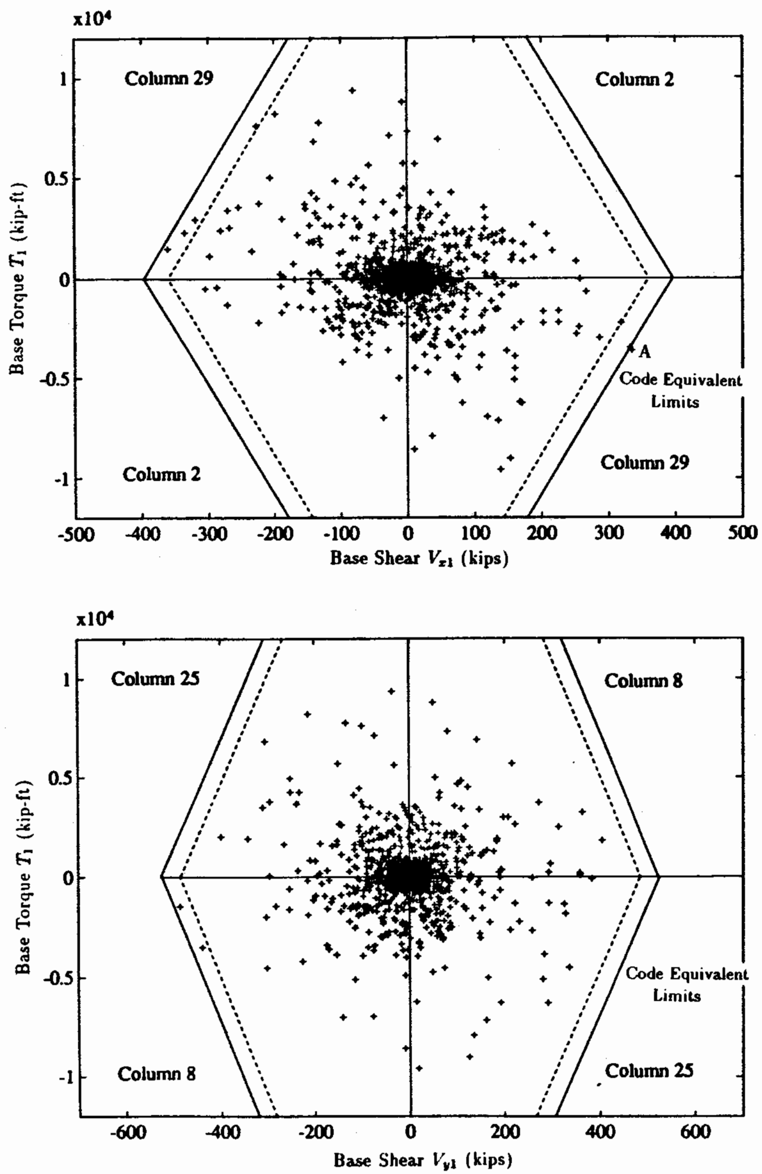


Figure 9: Comparison of Dynamic Base Shear, Base Torque and "Code Equivalent Combinations" in Building B During the Whittier Earthquake

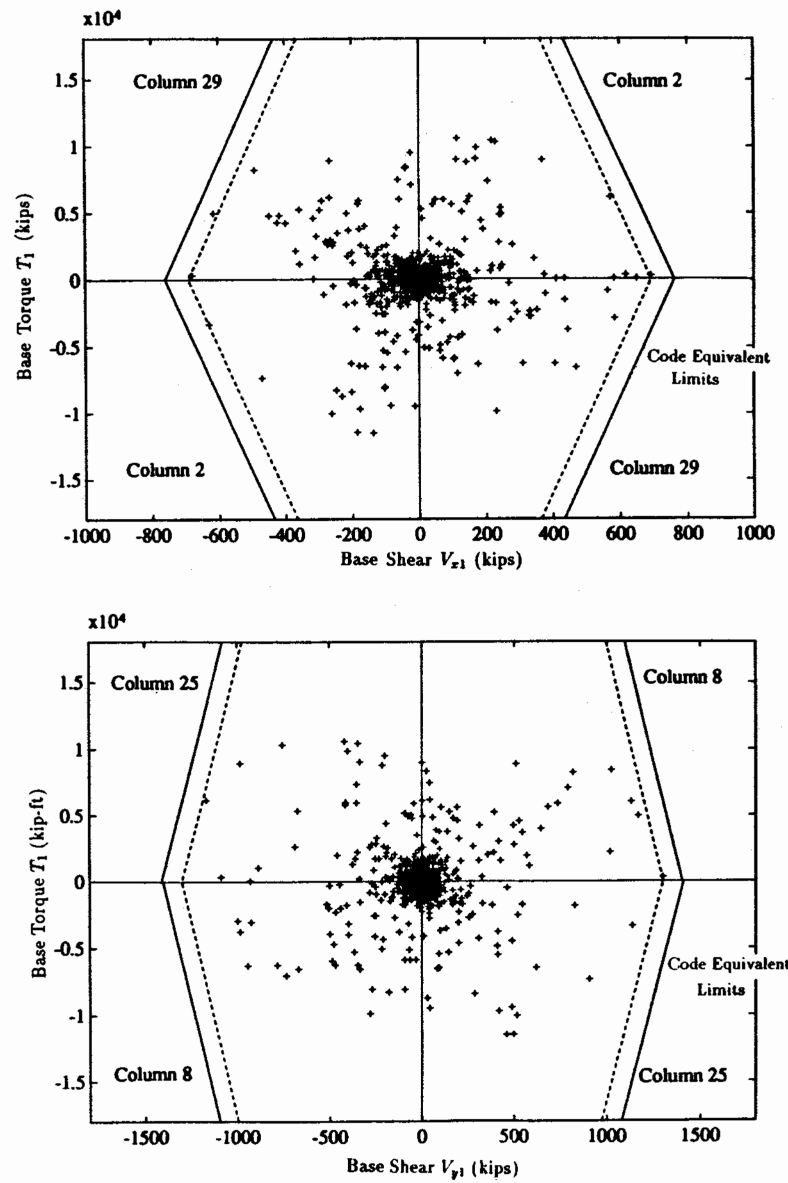


Figure 10: Comparison of Dynamic Base Shear, Base Torque and "Code Equivalent Combinations" in Building B During the Upland Earthquake

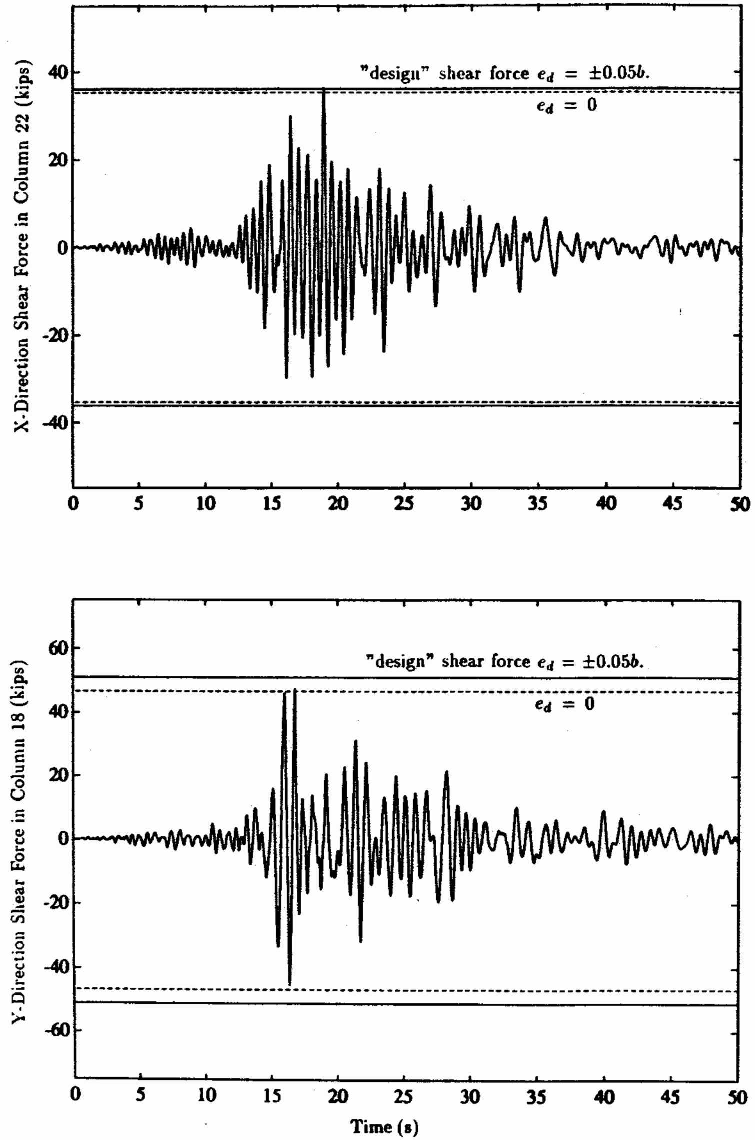


Figure 11: Comparison of Earthquake Induced Shears in Columns 22 and 18 with "Design" Shear Values, Building A (Loma Prieta Earthquake)

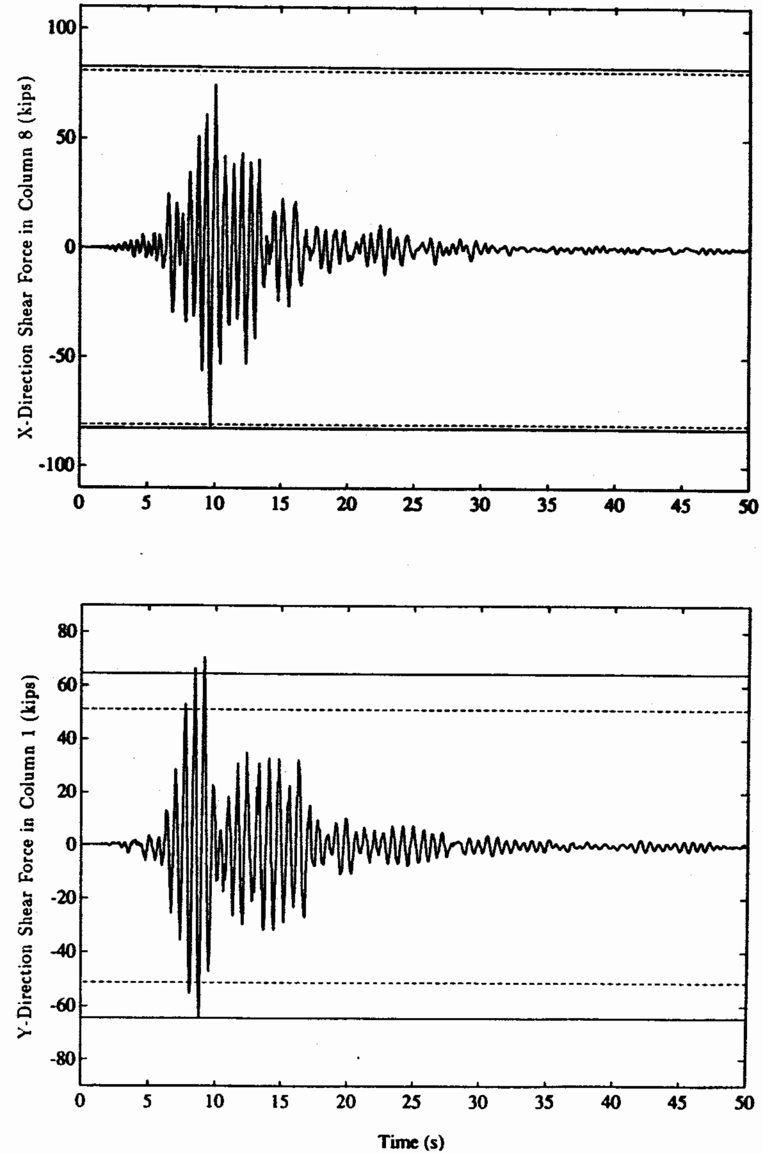


Figure 12: Comparison of Earthquake Induced Shears in Columns 8 and 1 with "Design" Shear Values, Building C (Loma Prieta Earthquake)

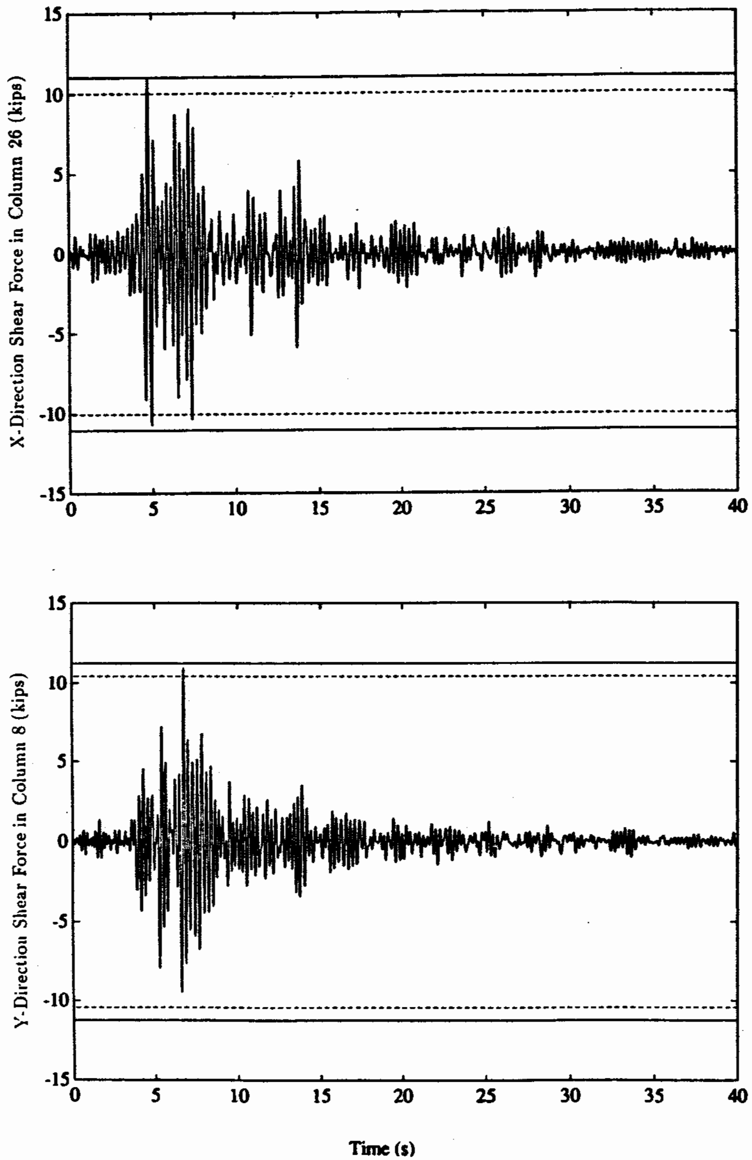


Figure 13: Comparison of Earthquake Induced Shears in Columns 26 and 8 with "Design" Shear Values, Building B (Whittier Earthquake)

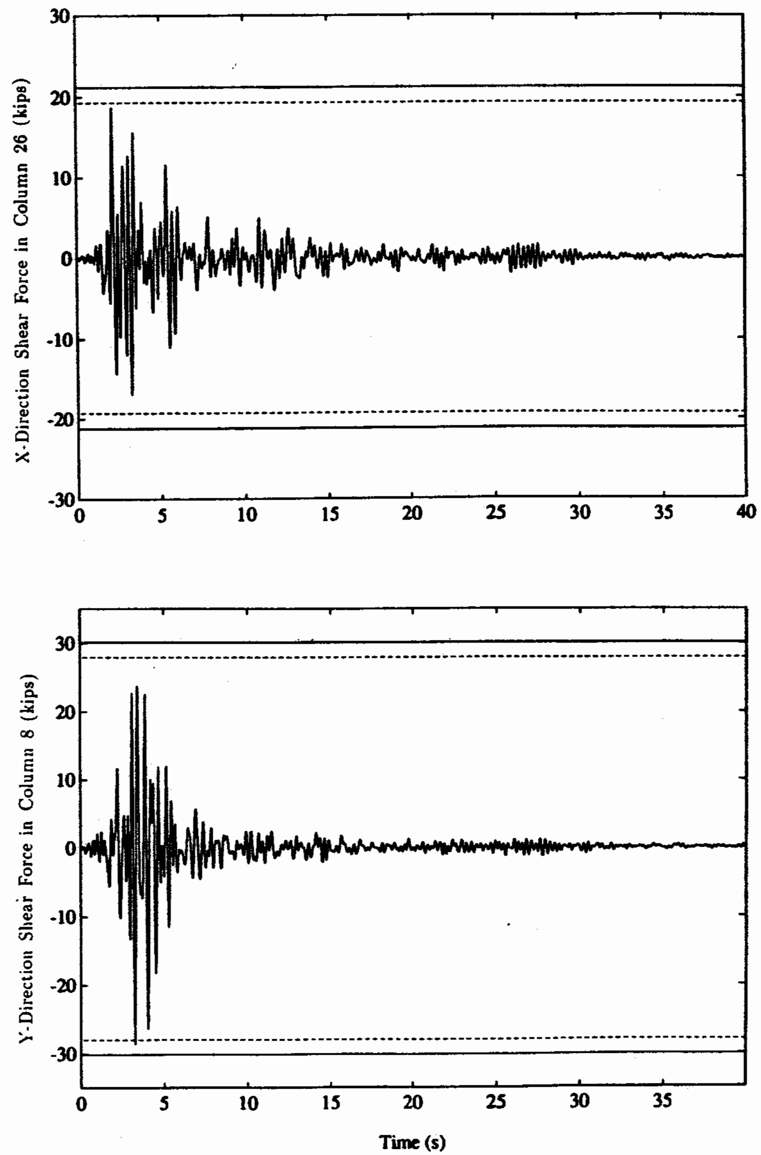


Figure 14: Comparison of Earthquake Induced Shears in Columns 26 and 8 with "Design" Shear Values, Building B (Upland Earthquake)