THREE-DIMENSIONAL INELASTIC RESPONSE OF AN RC BUILDING DURING THE NORTHRIKE EARTHQUAKE

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ABSTRACT: The three-dimensional inelastic earthquake response of a seven-story reinforced-concrete building during the 1994 Northridge earthquake is studied herein. The objectives of this investigation are as follows: (1) to understand the inelastic behavior of the building using recorded motions; and (2) to propose a simplified inelastic model that could explain the lateral-torsional coupling observed in this nominally symmetric building. Because several two-dimensional inelastic models of the building have been reported by other researchers, this paper focuses on the three-dimensional behavior of the structure. Response results of a simplified inelastic stick model that uses the story-shear and torque surfaces are compared with the results obtained from a conventional elastic three-dimensional building model. These results suggest that damage in the building occurred in the first few cycles of the response, and that the 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, and constitutes a rather new phenomenon that should be studied further. It is proposed herein that such behavior be foreseen in design by using the concept of the ultimate story-shear and torque surface.

INTRODUCTION

Because few examples of the measured inelastic response of instrumented buildings are available in the earthquake engineering literature, the building considered in this investigation (Fig. 1) has attracted the attention of researchers as a benchmark case for the calibration of earthquake analysis and design methods (Islam 1996; Loh and Lin 1996; Lynn et al. 1996; Moehle et al. 1997). These researchers developed planar inelastic models of the longitudinal frames of the building. The building is identified as California Strong Motion Instrumentation Program (CSMIP) station number 24386. It is a seven-story nominally symmetric reinforced-concrete (RC) structure (Fig. 1). The lateral force resisting system of the building consists of an RC perimeter frame with columns spaced approximately 5.8 m (19 ft) apart in the longitudinal direction (x) and 6.1 m (20 ft) apart in the transverse direction (y) (Fig. 1). These columns are typically 356 × 508 mm (14 × 20 in.) and are arranged about their weak bending axis in the longitudinal resisting planes A and D, and about their strong bending axis in the transverse resisting planes 1 and 9. Interior frames are intended for carrying gravitational loads only and do not incorporate any special seismic detailing. Spandrel beams of variable height, 572 mm (22.5 in.) typically, are used in the perimeter frames. The floor system used is a reinforced-concrete flat slab, 254 mm (10 in.) thick on the second floor, 216 mm (8.5 in.) thick on the third to seventh floors, and 203 mm (8 in.) thick at the roof. Further details of the structural configuration of the building as well as material properties may be found elsewhere (John 1973).

The building underwent serious structural damage in the beams, columns, and beam-column joints of the longitudinal frames A and D (Fig. 1); only minor structural damage occurred in the end bays of transverse frames 1 and 9. Seismic columns in frame A were severely damaged in shear between the fourth and fifth levels. The lack of appropriate confinement in these columns produced the subsequent buckling of longitudinal bars and the observed splitting failures. According to a damage report of the building (Islam 1996), minor to moderate shear cracks occurred in many beam column joints below the fifth level. Spandrel beams showed spalling and flexural cracks only at the bottom face, suggesting yielding of the bottom reinforcement only.

Nonstructural damage was also severe in the building. Partitions, doors, and windows in the east-west direction were damaged primarily between the fourth and fifth levels of the building (Islam 1996). Although there was a nominal 1 in. separation between the brick filler walls and frame D, cracks were observed near the corners of each wall, indicating that these walls may have participated to some extent in the earthquake response of the building. Such was also the case during the 1971 San Fernando earthquake (John 1973).

This building underwent significant torsional motions coupled with lateral motions, as documented in the next section. Planar analyses of the building reported previously are obviously not capable of predicting such coupled lateral-torsional motion. The objective of this paper is to develop simplified three-dimensional idealizations of the building that are capable of explaining the significant torsion experienced by this nominally symmetric structure.

MEASURED LATERAL TORSIONAL COUPLING

At the time of the earthquake, the building was instrumented with a total of 16 acceleration channels, four lateral channels (1, 13, 14, and 16) and one vertical channel at the ground (15), three lateral channels on the second and third floors (7, 8, and 12), two lateral channels on the sixth floor (4 and 10), and three lateral channels at the roof (2, 3, and 9). A selection of the accelerations recorded by these channels is presented in Fig. 2. Observe the shorter period motions of floors 1–3 (records 1–11), representative of the undamaged structure, and the longer period motions of floors 5–7 (records 3–9), representative of the damaged structure. The interstory deformations at selected time instants at the center of mass (CM) are shown in Fig. 3(a). Also presented are the maximum values of the translational and rotational components of the interstory drift. These plots demonstrate that the deformations were largest in the third and fourth stories, the two stories where earthquake induced yielding was concentrated. The largest drifts of...
2.1% and 1.5% in the x- and y-directions, respectively, occur in the third story; however, drifts exceed 1% between the first and fifth stories of the building in both lateral directions. Compare these drifts with the 1% value customarily chosen as a threshold to indicate the onset of structural damage in a frame structure. As a reference, the global roof-to-base drift was 1.16% and 1% in the x- and y-directions, respectively. Besides, building drifts are consistently larger in the x-direction.

From the recorded motions, the lateral accelerations of frame A (or D) due only to (1) the lateral motion of the building; and (2) the torsional motion of the floor plan are shown in Fig. 4. The latter has been scaled by a factor of two for easier visualization. The maximum acceleration along plane A (or D) due to torsion is 0.12 g, which is about 40% of that due to translation. Since the building plan is nominally symmetric, these significant torsional motions must be attributed primarily to the yielding of the structure, predominantly in one resisting plane (plane A), and would be expected to begin at the time damage is initiated. Indeed, looking at the traces of torsional motions of the fourth floor, it is apparent that torsional motion is significant starting at about 3.9 s; another episode of large rotation is seen at about 7.5 s. In the subsequent sections, we will investigate how this torsional behavior can be anticipated and accounted for during the design process.

PIECEWISE LINEAR IDENTIFICATION OF STRUCTURE

Before attempting the construction of a nonlinear three-dimensional model of the structure, it is useful to see how far we can get in representing the building response and damage through a sequence of identified time-varying linear models. This will enable us to identify the time variation in modal parameters, the effective modal damping of the structure, and to develop a benchmark model for comparison.

The building identification was performed in the time domain using a multiinput multioutput scheme and the eigensystem realization algorithm with data correlation (Juang 1994). Three linear models of the structure are identified for the three time windows shown in Fig. 5—\( w_1 = 0-5 \) s (elastic structure), \( w_2 = 4-15 \) s (inelastic structure), and \( w_3 = 14-30 \) s (residual structure). Linear modeling is reasonable for these three time windows because damage occurred only at a few instants of the building response and involved only short inelastic excursions.
The identified impulse response functions for the roof acceleration are presented in Fig. 6(a) for the three parametric models developed for the building; the excitation is a base acceleration of 10 mm/s² in the x-direction and the time scale starts at the initial time of each window. The correlation between the predicted and “measured” acceleration impulse response at the roof is good. The largest observed differences occur for model 2 and are due to the yielding of the structure during that time window. Shown in Fig. 6(b) is a comparison between the predicted and measured roof acceleration during the earthquake. Results of the piecewise linear identification show good accuracy for all time windows, implying that the identified linear models can be reliably used to compute the modal parameters of the structure.

A modal decomposition of the acceleration impulse response at the roof is presented in Fig. 7; the first four modal contributions for the time windows, \( w_1, w_2, \) and \( w_3 \), are presented. The modal frequencies and damping ratios shown in Fig. 7 are in agreement with those computed earlier from spectral analysis (De la Llera and Chopra 1997). These results show an important decrease in vibration frequencies due to damage of the structure. For instance, the fundamental frequency is reduced from 0.6 Hz to 0.5 Hz and a similar trend is observed for the other modes. The corresponding fundamental vibration mode is apparent in the three windows considered; it is characterized by an in-phase motion of all floors. On the other hand, the fundamental mode damping ratio of 13% during the first time window increases to 30% due to damage during the second time window, the strong phase of motion, and later decreases to 11% for the trailing part of the record.

Observe that two second modes of vibration of the structure, identified as 2a and 2b, are shown in Fig. 7. This pair of modes is a mathematical recognition of the broadening of the spectrum around the second modal frequency due probably to the yielding of the structure in that “mode.” Such yielding is also responsible for the large damping ratio observed in this mode. This can be confirmed by the x-direction mode shapes (Fig. 8), obtained from the relative acceleration values at different floors of the modal responses (Fig. 7). The modes are preserved among the different time windows considered, with the exception of the second mode shape (mode 2a) that presents important variation. This result provides another clue that damage in the building could have been associated with this mode. Indeed, the second mode shape for the second time window indicates a maximum deformation demand in the...
FIG. 7. First Four Modal Contributions to the Acceleration Impulse Response of Building Floors

"Measured" results also support the theory of an important second mode in the earthquake response of the structure. A more detailed observation of the modal contributions presented in Fig. 7 shows that the second mode has a significant contribution to the identified impulse roof acceleration of the building. The contribution of the second mode acceleration adds to those of the first mode at intermediate floors, unlike the situation for other building floors. A similar observation applies to the second time window, in which first mode accelerations are enhanced by second mode accelerations at intermediate floors. This leads to larger floor accelerations and interfloor deformations in these stories, as previously observed in Fig. 3.

**INELASTIC BUILDING MODEL**

Since this building underwent coupled lateral-torsional motions and significant damage during the earthquake, an inelastic three-dimensional analysis is developed. Instead of a detailed idealization that includes an inelastic model of each structural element, a simplified idealization of the building is developed. In this model, the stiffness and strength properties of a building story are represented by a single columnlike element connecting two consecutive floors. This single element model (SEM) allows 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 is assumed to be symmetric about the y-axis, a simplification that is supported by the results presented later.

The inelastic properties of the SEM will be defined by the corresponding story-shear and torque (SST) surface, defined as the locus of pairs of shear force and torque values that, when applied statically to the story, produce its collapse. Therefore, story shear and torque combinations developed during the dynamic response of the structure cannot fall outside this surface. The SST surface divides the story shear and torque space into an elastic (inside) and a statically impossible region (outside). Yielding occurs when the shear-torque pairs reach the SST surface.

The SEM is developed for the building considered in this study, as follows: The first step is to compute the individual capacities of the structural elements of the building. For instance, shown in Fig. 9 are the nominal shear capacities associated with shear- and flexural-type failures for the x-direction building columns. Such capacities were computed considering the gravitational axial load in each column, and the actual distribution of longitudinal and shear reinforcement. Elastoplastic and parabolic constitutive relationships were assumed for steel and concrete, respectively. Shear capacities
associated with flexural-type failure for the columns were estimated by assuming flexural hinges at the top and bottom of each element. Shear-type failure capacities were obtained, assuming diagonal cracking of the element and computing the shear resistance according to the ACI 318-95 code formula (ACI 1995). The latter nominal capacities have shown to be overly conservative in many cases, resulting in overstrength, as will be shown later for this building. Fig. 9 indicates that the shear capacities associated with shear failure are substantially smaller than the shear capacities associated with flexural failure. Thus, this building does not meet current design criteria requiring that brittle failures of columns must be avoided. Based on these results, shear failure of columns is expected before they are able to develop stable plastic hinges at the top and bottom ends. This analysis-based conclusion has been demonstrated by Lynn et al. (1996).

By using the shear capacities of Fig. 9, the parameters of the nominal SST surfaces were computed by methods described elsewhere (De la Llera and Chopra 1997), and are presented in Fig. 10, after appropriate scaling, as described later. These surfaces include approximately the effect of the level of yielding of frames in the y-direction, by assuming that a small percentage of the capacity of resisting planes in the y-direction is available to resist the torque produced in the x-direction motion of the building. Based on previous results (De la Llera and Chopra 1997), 10% of this capacity was assumed for the next analysis.

The story shears and torques developed in the building during the earthquake are calculated or measured from the recorded motions. For this purpose, the X and Y components of acceleration at the CM of the ith floor, \( a_x(t) \) and \( a_y(t) \), and the torsional acceleration, \( a_x(t) \), are determined by a simple geometric transformation of the motions recorded at that floor, assuming a rigid diaphragm. The associated inertia forces are \( m_i a_x(t) \) and \( m_i a_y(t) \) in the x- and y-directions, respectively. The associated torque is \( I_p a_y \), where \( m_i \) is the ith floor mass and \( I_p \) is the polar moment of inertia of the ith floor mass about the CM of the floor. The shears and torques in the ith story are computed by simple statics from the floor inertia forces

\[
V_x = \sum_{i=1}^{N} m_i a_i; \quad V_y = \sum_{i=1}^{N} m_i a_i; \quad T_i = \sum_{i=1}^{N} I_p a_y
\]  

Eq. (1) may be used directly to compute the measured story shears and torques if accelerations of all floors are recorded; however, only some of the building floors are instrumented.

The accelerations at the CM of noninstrumented floors can be computed by an interpolation or identification procedure. An interpolation procedure was preferred for this building with inelastic behavior (De la Llera and Chopra 1997). The procedure was calibrated using the two extra translational motions recorded on the sixth floor of the structure (channels 4 and 10) that were not used in the computation of inertia forces.

The measured story-shear and torque pairs are presented together with appropriately scaled SST surfaces in Fig. 10. With the goal of enforcing the condition of static admissibility, a physically motivated consideration was incorporated into the SST surfaces. These surfaces were scaled up by an overstrength factor defined as the ratio between the peak value of the measured story shear and the calculated nominal shear capacity of the story. Such overstrength comes from different sources, such as material overstrength and hardening, conservatism in the prediction of nominal shear capacities of structural members, and participation of nonstructural components. For the stories that remained elastic, as indicated by story shears and torque combinations remaining inside the SST surface, no correction was deemed necessary.

Observe in Fig. 10 that the SST surfaces and the story-shear and torque combinations are not forced to satisfy the static admissibility condition at all time instants. The only condition imposed to scale up the surfaces is that the calculated value of story shear capacity coincides with the maximum measured story shear. The results show that all shear-torque combinations automatically satisfy the static admissibility condition after the surface has been scaled up by a single factor. To illustrate this statement, the third- and fourth-story SST surfaces are shown enlarged in Fig. 11, which permits several observations. First, as mentioned before, essentially all story-shear and torque combinations lie inside the boundaries of the surfaces. Observe that a number of story-shear and torque combinations follow closely the SST surface; these combinations of story shears and torque have been encircled in the figure. Second, if the structure is yielding along the SST surface, at least some of the encircled points that lie on the surface should correspond to consecutive instants of time. If they did not, it would be unlikely that they correspond to a yielding excursion...
of the structure. For this reason, the time instants at which the story-shear and torque pairs occur are noted in Fig. 11.

Fig. 11 indicates that the longest inelastic excursion lasted about 0.12 s and occurred at about 8.4 s after the start of the record, and a plastic building mechanism was achieved in the third and fourth stories during the second train of pulses identified at that time in Figs. 2 and 4. This does not imply that there was no inelastic action in some resisting planes of the building before this time instant, such as the interval between 4 and 5 s, but that a building mechanism was not reached during that interval. Moreover, the few points on the SST surface indicate that there was essentially one or two inelastic excursions. This is consistent with the evidence of a brittle failure of columns of this building during the earthquake.

With the SST surfaces established, the inelastic properties of the SEM for each story are known. Furthermore, the elastic properties of these SEMs are selected to match ideally the structural properties at small motions, identified for the first time window $w_1$, 0–5 s. These properties were checked to be consistent with those obtained from motions of the building recorded during previous earthquakes (De la Llera and Chopra 1997). Hence, the stiffness properties of the SEM model were obtained by matching the identified fundamental frequency of the system of 0.6 Hz (Fig. 8) with that of the SEM in the $x$-direction.

The response of the resulting SEM model to the recorded motions at the base of the building was computed by MATLAB (MATLAB 1997). The base motion was computed from the ground accelerations recorded (Fig. 2) in channels 1, 14, and 16 by a simple geometric transformation assuming a rigid base. These analytically predicted responses are compared with the measured responses in Figs. 12–14. Such comparison for the measured floor accelerations (Fig. 12) shows that the SEM follows the measured acceleration traces reasonably well. In general, the predicted accelerations are slightly smaller than those measured. Shown in Fig. 13 are the measured (crosses) and predicted (circles) story-shear and story-torque combinations in the fourth story. Although a point-to-point resemblance is not obtained, certain general features of the measured plot, such as the magnitudes of maximum shear and torque and trends of specific combinations, are represented reasonably well by the model. Similarly, Fig. 14 shows a comparison between the measured and predicted floor displacements relative to the base of the building. In this figure, displacements of noninstrumented floors are also denoted as measured, although they were computed from recorded displacements by interpolation (De la Llera and Chopra 1997). The model predicts the displacement response quite accurately for the first 9 s; subsequently, the model is stiffer than the building, as indicated by the shorter period in the calculated response than the measured fundamental period. One possible explanation is that, although damage starts at about 4 s, the structure changes its properties at about $t = 9$ s (Fig. 14). Starting at this time instant, the brittle shear failure that was observed cannot be modeled accurately by the yielding single-element model.
IMPLICATIONS FOR BUILDING DESIGN

The single element model proposed herein has two important features useful in building design. First, it provides much better estimates of the inelastic displacement demand on the building compared to elaborate elastic models. Second, it provides a simple conceptual tool to understand the three-dimensional inelastic behavior of a building. Further, the model can be useful in defining, at early stages of the building design, a structural system with planwise distribution of strength that minimizes inelastic torsional motions.

In spite of the fact that the building considered is nominally symmetric in plan, the measured inelastic response of the structure shows significant torsional motions, causing more damage in one of the longitudinal resisting planes (plane A, Fig. 1). Structures like this building, which is predominantly a moment resisting perimeter frame, are likely to experience such markedly torsional behavior at collapse. Such is the case because the plastic mechanisms associated with a perimeter frame involve plastic rotation of the building plan about a resisting plane along the edge of the structure, far from the CM (Fig. 11). Yielding will almost never occur simultaneously in both resisting planes along parallel edges. Indeed, a minimal accidental eccentricity, caused in this case, for instance, from the insufficient spacing between the brick filler walls and frame D, or a small torsional component of base motion is enough to trigger such markedly inelastic torsional behavior.

A linear three-dimensional model of the structure, which is useful to verify serviceability limit conditions, is incapable of predicting the torsional response of a nominally symmetric building arising from unsymmetric yielding (De la Llera and Chopra 1997). Inelastic structural models are necessary for this purpose. However, it is impractical, for most engineering companies, to analyze three-dimensional nonlinear models that explicitly include every structural element. Nonlinear analysis of simplified inelastic models like the SEM provides an alternative that is also practical to predict the main features of the inelastic earthquake performance of a structure.

As with any other inelastic analysis procedure, the structural response of the SEM is sensitive to the correct estimation of shear capacities of the resisting planes. In this case study, since the response of the structure during the earthquake was recorded, the SEM capacities could be matched with the measured building capacities, thus reducing considerably the uncertainty of the structural model. Of course, this is not usually the case during building design, and it is advisable that the designer performs a sensitivity analysis for the inelastic response of the model. Moreover, the local failure of the fourth-story columns in this structure made the construction of the SEM simpler, since the mechanism produced was a story mechanism controlled by yielding of all story columns along a resisting plane. In current earthquake design philosophy, however, the inelastic behavior of the structure will tend to be distributed in height, leading to mechanisms of the strong column–weak girder type that involve yielding of elements in different stories. In that case, the capacities of the SEM should be based on the shear forces of resisting planes developed for those mechanisms. Other limitations of the SEM regarding high mode, axial, and overturning effects are those of a stick model. The main contribution of the SEM has been the incorporation of the elastic and inelastic behavior of the building plan through the story-shear and torque surfaces.

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