Investigation of coupled lateral-torsional response in multistorey buildings

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ABSTRACT: The dynamic behaviour of an existing building that responded severely during service level earthquakes is presented in this paper. The building is a thirteenth floor, "regular" space frame structure. The recorded responses of the building during different earthquakes were characterized by long duration, narrow bandwidth motions with strong amplitude modulation; by large translational and torsional accelerations; by large amplifications of the input ground motions; and by slow decay of the building's dynamic responses. Records were studied to obtain the building's dynamic properties and response envelopes. The causes of the severe response are identified from these studies. Three-dimensional linear and nonlinear models of the building were developed to match the recorded response of the structure. Parametric studies are performed on the analytical models to study the effects of material and geometric nonlinearities, accidental eccentricities, bi-directional input ground motions and energy dissipation capacity in the response of the building. Results indicate that the severity of the torsional response in a eccentric multi-story structure is strongly influenced by the level of inelastic behavior, level of eccentricity, ground motion characteristics and the structure's energy dissipation capacity.

INTRODUCTION

The lateral-torsional coupled behavior of structures has been the subject of a large number of studies. Investigations of this behavior have usually been undertaken using highly simplified linear or nonlinear computer models. With the extensive installation of strong motion instruments in structures around the world, it has become possible to monitor the actual three-dimensional behavior of buildings during earthquake events and to study lateral-torsional coupling in these structures under different conditions.

A structure that exhibited strong lateral-torsional coupling in its recorded response, among other response characteristics of interest, is an apparently regular thirteenth floor office building, located in San Jose, California, Fig. 1. The building is instrumented with twenty-two unidirectional strong motion accelerographs, positioned at five different levels (ground, 2, 7, 12 and 14 floors) at the NW, SW and SE corners of the square frame plan (Lines B and 12, Fig. 1). The structural system consists of steel moment resistant space frames. A strong moment resisting frame, Lines 12-A-B, is located around a lighter moment resisting (frame. A much lighter frame is located outside the strong frame. Lines 12--- and A-B.

Several earthquake records have been obtained in this structure. The three most intense responses recorded to date are those obtained during the Morgan Hill earthquake of April 12, 1984 (M = 6.2), the M. Lewis earthquake of March 31, 1986 (M = 5.8) and the October 17, 1989 Loma Prieta earthquake (M = 7.1). Peak horizontal ground accelerations recorded for these events at the base of the building were 4, 4 and 11 g, respectively. The building sub-structurally amplified these base motions so that the maximum structural accelerations during the earthquakes were 17, 32 and 36 g, respectively. Motion at the structure during all the earthquakes caused widespread damage to contents and disruption of services. The response records exhibit a very strongly modulated pallem and locally indicate that the structure experienced substantial torsion, Figs. 2, 3 and 4. Another characteristic of the responses shown in these figures is that the structure continued to vibrate vigorously for more than 30 seconds. The input motion was much shorter in duration and maximum structural responses occurred generally long after the end of the strong motion portion of the base excitation.
Considerable insight into the effect of small eccentricities in nearly regular multi-story space frames can be obtained considering simple three degree of freedom crane systems. The dynamic characteristics of a one story frame can be obtained by formulating the eigenvalue problem at the structure's center of mass (CM) and solving for the total global stiffness.

Figure 1: Building plan and framing.

Figure 2: Twelfth floor acceleration records. EW direction. Mt. Lewis earthquake.

Figure 3: Roof SW corner relative displacements. Mt. Lewis earthquake.

The general eigenvalue problem presented here cannot be solved in closed form because its characteristic equation. Nevertheless, this equation can be solved for the special case of identical translational stiffness in orthogonal directions. Then $K_x = K_y = K$. The solution for this special problem, with a single translational stiffness $K$, will provide insight into the more general problem.

The eigenvalue solution for this system can be expressed as:

$$
O_{1,2,3} = \frac{1}{2} \left( \left( \frac{e_x}{r} \right)^2 + \left( \frac{e_y}{r} \right)^2 \right)
\times \sqrt{\left( \left( \left( \frac{e_x}{r} \right)^2 + \left( \frac{e_y}{r} \right)^2 \right)^2 - 4 \left( \frac{e_x}{r} \right)^2 \right)}
$$

$$
O_0 = 1
$$

where $e^2 = e_x^2 + e_y^2$ is a measure of global eccentricity. $\text{O}_n = \left( \frac{\omega_n}{\omega} \right)^2$ is ratio of coupled to uncoupled frequencies, $\omega = K/\mu$, and $e_x^2 = K/K$ is the ratio of torsional to translational stiffness at the center of the frame.

These equations indicate that for similar uncoupled torsional and translational periods (values $e_x/r$ close to one), and small static eccentricities, the three coupled natural periods of the system could be extremely close.
Figure 4: Eleventh floor N5 relative displacements (SW comer) and torsion (EW records). Loma Prieta earthquake.

The modes shapes that correspond to the eigenvalue result have the following form if $e_u \neq 0$ and $e_t \neq 0$:

$$\Phi = \begin{bmatrix} \frac{1}{\sqrt{\lambda}} & 0 & \frac{1}{\sqrt{\lambda}} \\ \frac{1}{\sqrt{\lambda}} & 1 & \frac{1}{\sqrt{\lambda}} \\ \frac{1}{\sqrt{\lambda}} & 0 & \frac{1}{\sqrt{\lambda}} \end{bmatrix}$$

(4)

Figure 5: Ratio al uncoupled translational and torsional periods for a regular one story structure.

The mode that corresponds to $\Phi_n$ ($n = 1$) is a pure translational mode; the predominant direction of the mode is skewed relative to the reference axes, according to the global eccentricity. Close, torsional and translational uncoupled periods are typically observed in systems that have uniform distribution of stiffness in plan (Newmark, 1969). For regular space frame structures these periods can then be quite close. It can be shown that for one story frames or multi-story frames with only three degrees of freedom per story [chain systems] the ratio of translational to torsional uncoupled period can be obtained by Equation 5 if the y-term has the following characteristics: a) multiple columns evenly distributed; b) uniform distribution of mass; c) coincident center of mass and stiffness; d) all columns with the same stiffness in a given direction; e) shear behavior with three degrees of freedom per story [two horizontal translations and one in-plane rotation at the center of mass], d) all vertical elements with negligible torsional stiffness, and e) radius of gyration defined in terms of the external column position and distributed mass.

$$\left(\frac{T_{tx}}{T_{tr}}\right)^2 = \frac{\left(\frac{k_x}{d_x}\right)^2(N_x^2 - 1) + \left(\frac{k_y}{d_y}\right)^2(N_y^2 - 1)}{(N_x - 1)^2(N_x^2 + (N_x - 1)^2)}$$

(5)

where: $N_x$ or $y$ is the number of column lines in the x or y direction, $d_x$ or $y$ is the spadget between consecutive columns ID the x or y direction, $T_x$ is the uncoupled translational period in the x direction, and $T_{tr}$ is the uncoupled torsional period. For the special case of a square building with equal structural y-t.ems in both directions, the formula can be simplified as follows:

$$\left(\frac{T_{tx}}{T_{tr}}\right)^2 = \frac{(N_x - 1)(N_x + 1)}{(N_x - 1)^2}$$

(6)

It can be seen in Fig. 5 that this ratio quickly approaches one as the number of columns increases. For regular frames with an even distribution stiffness in plan and LumaM eccentricities the following assumption can be made $(c/r) < 1$ and $(c/r) \approx 1$. So Equation 2 and 4 can be approximated by:

Figure 6: Eleventh floor motion Mt. Lewis event. Input ground motion 0-100 seconds. EW relative displacement, SW comer. a) First three modes, low damping model (1 %), b) First mode, low damping model. c) Second mode, low damping model. d) First three modes, moderate damping model (5 %). Model — — Record .......
where: the mode shapes are normalized so that \( \Phi_1 \Phi_1 = 1 \), [see also Kelly (1990)].

Finally, the coupled natural frequencies of the system can be found from Equation 7 making use again of the assumption of small eccentricities:

\[
\omega_1 \approx \omega \left( 1 - \frac{\epsilon}{2} \right) ; \quad \omega_2 \approx \omega ; \quad \omega_3 \approx \omega \left( 1 + \frac{\epsilon}{2} \right),
\]

(9)

The close agreement of the predominant periods and the three-dimensional characteristics of the modes shape, will produce responses that can increase substantially the severity of the linear response of the coupled system (see Berceck (1991)). Also the time histories of the response are strangely modulated (beating behavior).

3 BUILDING RECORDING RESPONSE

The recorded response of the San Francisco building have been studied extensively. For example, Borousch and Mahin (1989, 1990, 1991). Basic dynamic properties and response envelopes were obtained directly from the records. The dynamic properties of the building are presented in Table 1. The maximum response envelopes obtained during the three earthquakes studies are presented in Table 2. Figures 2 through 4 show ID time histories obtained in the building.

It can be concluded, from the analysis of the response records, that the building presents a rather flexible, and structurally simple system with relatively low damping. The predominant period was found to be near 2.3 seconds and modal damping was believed to be below 3% of critical. Because of the similar frame structural characteristics in both directions and the even distribution of stiffness in plan, the predominant periods of the system are quite close. The closeness of the modes together with small eccentricities present in the structure produced the strongly coupled lateral-torsional behavior observed in the records. Because of the spatial characteristics of the building frame, the coupling affects both directions and the rotation for most of the modes studied. The eccentricity that produces the torsionally coupled response can be associated with the irregular distribution of the mass and Cramming irregularities caused by a greater number of structural and nonstructural elements on the west and south sides of the building.

<table>
<thead>
<tr>
<th>Predominant</th>
<th>Period (sec.)</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EW</td>
<td>2.15-2.20</td>
<td>2-3</td>
</tr>
<tr>
<td>NS</td>
<td>2.05-2.10</td>
<td>2-4</td>
</tr>
<tr>
<td>Tonion</td>
<td>Third</td>
<td>1.70</td>
</tr>
<tr>
<td>EW</td>
<td>Fourth</td>
<td>0.65-0.75</td>
</tr>
<tr>
<td>NS</td>
<td>FICth</td>
<td>0.60-0.70</td>
</tr>
</tbody>
</table>

(a) Defined as the ratio of the peak acceleration at a location to the corresponding acceleration at the ground. (b) At recording position. (c) Maximum difference between recordings at same building side.

The structure responded strangely to these relatively minor earthquakes, usually believed that high intensity of the structural response cause of the building's relatively low damping, the three-dimensional modes of the building constructively reinforcing one another during portions of the motion, the input duration, the possible resonance effect on the building caused by the close maleh of the dy. Dynamic characteristics of the site and the structure and the relatively large flexibility of the structure.

4 ANALYTICAL MODELLING AND PARAMETRIC STUDIES

Three-dimensional linear and non-linear numerical models of the building structure were developed to simulate the recorded responses. Static and dynamic analyses were performed. The dynamic analyses consider unidirectional as well as bidirectional input motions with and without torsional input excitations. Sev-
eral parameters were monitored during the analyses: maximum displacement, interstory drifts, base shear, maximum ductility demands, maximum cumulative ductility and element corees. The final model had 2418 elements.

Linear models

The model was developed using inCormation from building plans and site aequaltions. A good match was obtained when the models included the center-to-center member dimensions [i.e., no rigid panel], were included to model the 8xibility of the beam-column joints, mass magnitude and distribution as estimated from structural plans, nominal element properties, and a modal damping ratio typically associated with structural frames responding in the linear range (1-3% of critical). By further adjusting the actual mass distribution and incorporating the deck contribution to the beam stiffness, computed global results were virtually identical to recorded values.

Frem analyses of different loading conditions it was concluded that in order to reproduce the building's response, both horizontal components of the ground records should be included. Bi-directional effects accounted for nearly 22% of the response in orthogonal direction. Tonional input motion had a small effect on the overall response of the structure.

The damping ratio did have an important effect on the response of the model. Because the rapid fluctuations of spectral accelerations present in lightly damaged items, the response over the low damping models used in the study were very sensitive to modeling uncertainties that influence the period estimates.

Also the models showed a strong sensitivity to the position of the center of mass. Increasing the eccentricity by 5% of the building's largest plan dimension reduced displacements and shear forces at both directions, by a maximum 0/36% and increases floor rotations by nearly 144% and base torque by 160%. The ratio of maximum base torque to maximum base shear was increased by 180% when the additional eccentricity was included. The modal coupling was quite sensitive to design model parameters. Small changes of stiffness to one direction reduce the coupling, to some modal components; by nearly 75%. This demonstrates the difficulty to reproduce the coupled behavior of the structure.

The linear model confirmed that the severity of the response is caused by the modal interaction of the three dimensional truduru modes. Figure 6, for example, shows the response of a model with relatively low damping to the first 40 seconds of the M. Lewis earthquake. In pan (a) of the figure the response of the analytical model with 1% viscous damping is compared with the recorded motion. Parts (b) and (e) of the figure show the first and second mode contribution to the displacement in the EW direction. Here it can be seen that the first and second modes individually have slightly attenuated responses after about 30 seconds of motion. However, the two modes go in and out of phase, resulting in constructive and destructive interference that produces a large dip in the combined response at second 30 and an increase in response up to second 60.

An analysis was also performed considering 5% viscous damping. Here (Fig. 6d), the response of the individual modes attenuate so quickly that virtually no beating under free vibration can occur and little significant motion occurs after 35 seconds.

4.2 Nonlinear models

The building studied did act suffer significant inelastic behavior; so the nonlinear charactererics of the models were not lit to any of the observed responses. Nevertheless the nonlinear model was used to study the effect of nonlinearities, additional eccentricities, damping and ground motion characteristics on the global response of the system. The element developed by Riahi et al. (1978) were used in the analysis. These elements are three dimensional beam-column with a multidimensional interaction yield surface. For (P, M'' M = M).}

Initially a nearly triangular static lateral loading was applied to the structure. The load deCormation curve lowered that, due to the pattern of yielding, tonionalJ rotation, can grow more rapidly than the displacements at the center of mass in the direction of loading. Figs. 7 and 8. Nevertheless, after severe yield the yield orientation has occurred the displacement grows much faster than tonioDa! rotations (energy dissipating mechanism mainly involved). In other words, apparent coupling between translations and rotations is a multi-Jory. Irreducing highly dependable on the story of loading. Ineluctable distribution and level of inelastic behavior.

For the dynamic studies five earthquake records were considered: the recorded base building records during the Morgan Hill (1984). M. Lewis (1986) and Loma Prieta (1989) events (because of their period characteristics these are considered as a 10ft. site record.), as well as the Mexico SCT (1985) and El Centro (1940) events. 80th horizontal components of these earthquakes were used in all the analyses. The record were called to different areas of effective peak acceleration to obtain the response of the
found that the ratio or in-plane maximum translational displacement at the center of the structure tends to decrease as the eccentricity of the force increases.

Analyses using different ground motions indicate that the properties of the input motion have a strong effect on the response of the models. These differences were more pronounced for elastic than inelastic responses.

It was found that the ratio or in-plane torsional rotations to lateral displacements (Maapparent coupling) could increase or decrease depending on the level of inelastic behavior and the characteristics of the input ground motion. Nevertheless, some scatter was found in the results. This indicates that more analyses are needed to identify a trend on the response and its relation to the observations.

The effect of decoupling of torsional and lateral motions made on the basis or the static load to collapse studies and the effect on input motion predominant direction observed from the recorded torsional response of the building and simple linear models studied by Boroschek (1991).

Results from the analyses that considered added mass eccentricities indicate that, contrary to what was found for models subjected to unidirectional input, displacements at the center of mass (or at a fixed point on the story plan) could decrease or increase depending on the building's characteristics and the properties of the input ground motion. An increase in eccentricity, from 0 to 10% or the maximum building dimension, had the effect of increasing the maximum rotational ductility demand on the strong frame girders (maximum increment was 22%) and reducing the cumulative ductility demand on the strong frame elements (maximum reduction was 56%).

Similarly, the variation of the elastic model's response for the strong frame girders increased (up to 40%) or decreased (25%) for the different ground motions studied. Similar trends were observed for the inter-story drifts.

Also, the addition of eccentricity had the effect of redistributing seismic demands between orthogonal directions. In some cases, the maximum ductility demand for a given earthquake changed from one direction to the orthogonal direction, when the additional eccentricity was included.

5 CONCLUDING REMARKS

The results of this investigation agree with findings from the analyses of simple structures developed in other investigations. In general, the existence of torsional behavior in nearly regular space frames has the effect of increasing the stress or ductility demands in elements located far away from the center of rotation and changes the maximum translational displacements. These effects are more severe for elastic structures than inelastic structures and are highly dependent on the characteristics of the input ground motion.

REFERENCES


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