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# COMPARATIVE RESPONSES OF STRUCTURES SUBJECT TO THE 1985 MEXICO AND THE 1940 IMPERIAL VALLEY (CALIFORNIA) EARTHQUAKES

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## SUMMARY

Two-dimensional dynamic elastic response history analyses were performed on hypothetical six-story reinforced concrete buildings under ground motions recorded in the 1985 Mexico and the 1940 Imperial Valley (California) earthquakes. The buildings' structural schemes were altered in two orthogonal directions to obtain a number of different fundamental periods of vibration. The results of the elastic analyses are compared in this paper.

Two-dimensional dynamic inelastic response history analyses were also performed on the buildings considered, under the 1985 Mexico ground motion used for the elastic analyses. The results of elastic and inelastic analyses are also compared.

## THE 1985 MEXICO EARTHQUAKE

On September 19, 1985, Mexico City was hit by what is believed to have been the most damaging earthquake in its recorded history. At the Secretariat of Communications and Transportation (SCT Station) in the city, instruments measured over 20 repetitions of almost steady-state pulses of between 5 and 20 percent of gravity (Fig. 1). The ground acceleration record of Fig. 1 has a number of distinct characteristics with potential influence on structural

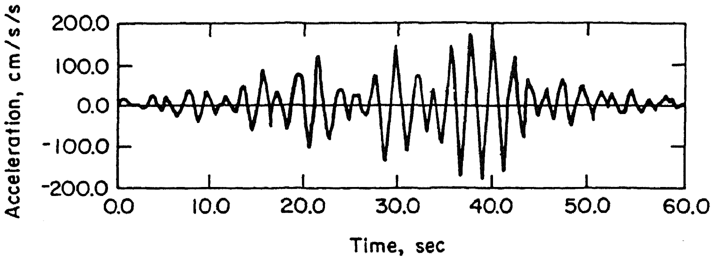

Fig. 1 S60E component of ground accelerations determined from N and E components measured at SCT in Mexico City

response. Firstly, the ground motion is very regular. Secondly, the predominant period of the ground motion is an uncharacteristically long two seconds. Thirdly, the peak ground acceleration is a relatively high 0.2g. Fourthly, the duration is unusually long. Indeed, the ground motions experienced in Mexico City in the earthquake of 1985 were unique with respect to intensity, regularity, frequency, and duration, making the earthquake "selectively devastating." This uniqueness was due to the well-known soil condition of the valley of Mexico (Ref. 1).

## RESPONSE OF BUILDINGS

The basic structure selected for study is a six-story building, regular in shape, with five 4 m stories, a bottom story of 4.9 m, and a penthouse. Floors consist of 230 mm thick flat plates with 9.75 m square bays.

Four bracing schemes are investigated, as might be done during a preliminary design. Scheme 1 combines ductile or special moment frames in one direction with load-bearing shearwalls in the orthogonal direction. Schemes 2 through 4 also utilize ductile or special moment frames in the E-W direction. The lateral load-resisting system in the N-S direction consists of a Building Frame System (an essentially complete space frame provides support for gravity loads; resistance to lateral load is provided by shearwalls or braced frames) in Scheme 2, a Dual System* in Scheme 3, and ductile or special moment frames in Scheme 4. Details of the schemes are reported in (Ref. 2).

Structural members comprising the four schemes were sized in conformance with the 1988 edition of the Uniform Building Code (UBC, Ref. 3) for gravity loads and Zone 4 (highest seismic zone including much of California) seismic forces. An importance factor I of 1.0 (standard occupancy structure) was assumed. Note that the ductile or special moment frames in the E-W direction are differently sized in Scheme 1 and in Schemes 2, 3, 4 because of a specific UBC requirement that where a structure has a Bearing Wall System in only one direction, the value of response modification factor** used for design in the orthogonal direction shall not be greater than that for the Bearing Wall System.

Dynamic elastic response history analyses using the computer program DRAIN-2D (Ref. 4) were performed on the six lateral load resisting systems of the four structural schemes described above (the systems in the E-W direction of Scheme 2, 3 and 4 are identical) under the first 44 seconds of the S60E component of the SCT, Mexico City, 1985 ground motion shown in Fig. 1. The system analyzed had fundamentals periods of vibration ranging from 0.55 to 2.14 seconds, as determined from eigenvalue analyses of two-dimensional models of the lateral load resisting systems in the two orthogonal directions (Refs. 2, 4).

Five percent of critical camping in the fundamental and second modes was assumed. The three schemes with the longest periods were also analyzed assuming 10% and 20% of critical damping.

Figure 2 shows a plot of the computed top deflections against fundamental periods. As expected, as the fundamental period of the structure approaches the

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*An essentially complete space frame provides support for gravity loads. Resistance to lateral load is provided by (a) a specially detailed moment resisting space frame which is capable of resisting at least 25% of the base shear, (b) shearwalls or braced frames. The two systems are designed to resist lateral load in proportion to their relative rigidities.

**The UBC specified values of this factor are 6, 8, 12 and 12 for Schemes 1, 2, 3 and 4, respectively, in the N-S direction.

predominant two-second period of the ground motion, the elastic response increases dramatically in magnitude. While higher damping reduces response, even at a very high damping of 20% of critical, the response of the buildings having fundamental periods close to two seconds shows drift in the range of 1.5%.

## COMPARATIVE RESPONSE TO 1940 IMPERIAL VALLEY EARTHQUAKE

Figure 3 shows a comparison of relative pseudo-velocity response spectra (corresponding to 2% of critical damping) for three ground motions: SCT, Mexico City, 1985, S60E; El Centro, 1940, N-S; and El Centro, 1940, E-W. It is obvious that the energy associated with the SCT ground motion is concentrated in the two-second and longer period range. The El Centro, N-S motion exhibits a much less pronounced peak at a period of about 1 sec. The El Centro, E-W motion exhibits a slightly ascending broad-band spectrum.

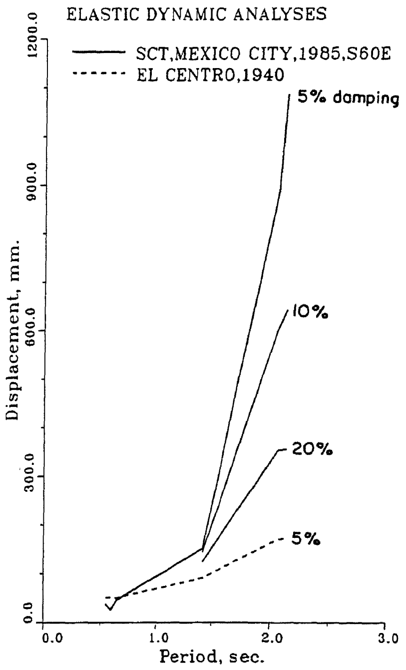

Fig. 2 Calculated elastic displacements of multi-degree-of-freedom systems subjected to SCT, 1985, S60E and El Centro, 1940 ground motions.

Elastic response history analyses, in addition to the ones described in the preceding section, were carried out for the various structural schemes along each principal direction. The ground motion used was the first ten seconds of the El Centro, 1940, N-S record for the short-period systems (fundamental periods: 0.55, 0.59 and 0.64 sec.), and the first ten seconds of the El Centro, 1940, E-W record for the long-period systems (fundamental periods: 1.42, 2.07 and 2.14 sec.). Past experience has indicated these ground motions to be near-critical in the respective period ranges considered (Ref. 5). Each ground motion was scaled to a peak ground acceleration of 0.19g, the same as that for the SCT, Mexico City, 1985, S60E component. The computed top deflections are plotted against fundamental periods in dotted lines in Fig. 3.

Interestingly, for the same amount of damping, the SCT, 1985, S60E ground motion with a peak acceleration of 0.19g excited much higher response in the long-period structures than the El Centro, 1940, E-W motion scaled to a peak acceleration also of 0.19g. While scaling on the basis of peak ground acceleration may not be the most rational, the effect of the pronounced spectral peak of the SCT, Mexico City, 1985, S60E motion is evident in the elastic response of structures to that motion. The 1940 El Centro motions being free from such pronounced spectral peaks, no similar resonant effect is evident from the elastic response of structures to these motions.

## INELASTIC RESPONSE

Although the above investigations confirm resonance of ground motion and structure and also demonstrate the role played by viscous damping in such resonance, it does not account for inelastic structural response.

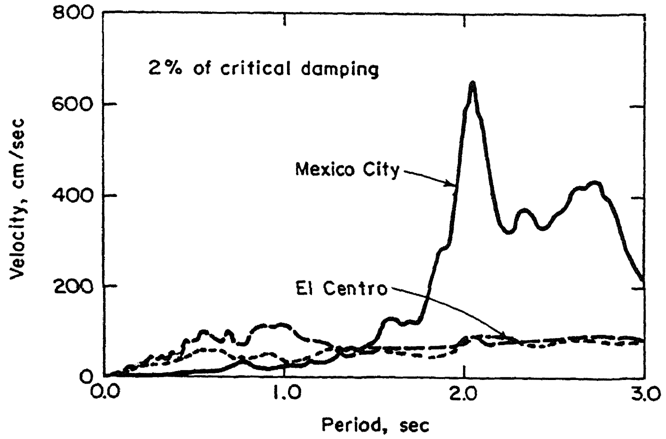

Fig. 3 Comparison of velocity responses: El Centro, 1940 and SCT, Mexico City 1985, S60E component.

There are two aspects of inelastic response that are of concern to the present investigation. Firstly, the period of a reinforced concrete structure progressively lengthens as it suffers inelastic deformations in certain locations while responding to an earthquake (Ref. 6). Secondly, inelastic hysteresis has an effect similar to damping on structural response to an earthquake (Ref. 5).

Dynamic inelastic response history analyses were performed on the orthogonal lateral load resisting systems of the buildings considered under the first 44 seconds of the S60E component of the SCT, Mexico City, 1985 ground motion. The program DRAIN-2D (Ref. 4) was used for these analyses also.

Program DRAIN-2D accounts for inelastic effects by allowing the formation of concentrated "point hinges" at the ends of elements where the moments equal or exceed the specified yield moments. The moment versus end rotation characteristics of elements are defined in terms of a basic bilinear relationship which develops into a hysteretic loop with unloading and reloading stiffness decreasing in loading cycles subsequent to yielding. The modified Takeda Model (Ref. 7) developed for reinforced concrete, was utilized in the program to represent the above characteristics.

The strength levels (yield moments) assigned to the ends of beams and columns were in accordance with factored bending moments at these locations caused by Zone 4 seismic forces. The strength of each column stack was kept uniform at the value chosen for the base of that particular column stack. The strength of all the beams at a particular floor level were also made equal to the largest factored bending moment anywhere along that line of beams. The assigned positive and negative flexural capacities were equal at every critical section.

Figure 4 shows a plot of the computed top deflections against fundamental periods. It is evident that yielding at certain locations decreases the response of the building considered to the SCT ground motion in both orthogonal directions. The response remains strong, however, even for 10% damping. The calculated roof level drifts in the N-S and E-W directions of Scheme 4 are 2.0% and 2.1%,respectively, for 5% damping. The corresponding values for 10% damping are 1.7% and 1.8%, respectively.

It should also be noted that unlike in Fig. 2 where elastic response undergoes a sudden and dramatic increase around the dominant ground motion period of 2 sec., the inelastic response of Fig. 4 shows a much more gradual increase in response as the period approaches the critical 2 sec. value. The response is already quite strong at a period of around 1.4 sec. This is the direct consequence of a lengthening of period with accumulating inelastic deformations. Figure 4 explains why buildings with small amplitude fundamental periods in the range of about 0.75 sec. to about 2 sec. suffered most of the observed damage in the 1985 earthquake in Mexico City. Buildings in a much narrower period range only would appear to be vulnerable on the basis of elastic response.

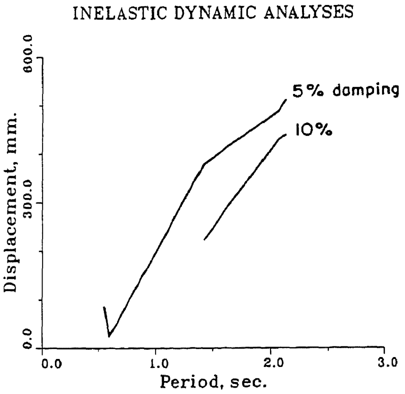

Fig. 4 Calculated inelastic displacements of multi-degree-of-freedom systems subjected to SCT, 1985, S60E ground motion.

## CONCLUSIONS

Two-dimensional dynamic elastic response history analyses performed on hypothetical six-story reinforced concete buildings with varying fundamental periods showed that for the same amount of damping and the same ground motion intensity, the SCT, 1985, S60E ground motion excited much higher response in the long-period structures than El Centro, 1940,E-W motion. This graphically illustrated the phenomenon of ground-structure resonance.

Two-dimensional dynamic inelastic response history analyses showed that yielding at certain locations decreased the response of the critically excited long-period structural systems. Also, the lengthening of initial fundamental periods due to accumulating inelastic deformations made structures having a wider range of initial fundamental periods vulnerable to the Mexico City ground motion, than indicated by elastic analyses.

## REFERENCES

1. Institute of Engineering, National University of Mexico, "Effects of the September 19, 1985 Earthquake in the Buildings of Mexico City," Preliminary Report, Mexico, D.F., (1985).

2. Forbes, M. A., Ghosh, S. K., and Corley, W. G., "Applications of the 1985 Blue Book to a Reinforced Concrete Building," Proceedings, Structural Engineers Association of California Convention, Coronado, California, (1985).

3. International Conference of Building Officials, Uniform Building Code, Whittier, California, (1988).

4. Ghosh, S, K., and Corley, W. G,"Behavior of Reinforced Concrete Framing Systems," The Mexico Earthquake-1985: Factors Involved and Lessons Learned, (Proceedings of an International Conference held in Mexico City, Mexico, September 1986), American Society of Civil Engineers, New York, N.Y., (1987).

5. Derecho, A. T., Ghosh,S. K., Iqbal, M., Freskakis, G. N., and Fintel, M., "Structural Walls in Earthquake-Resistant Buildings--Dynamic Analysis of Isolated Structural Walls--Parametric Studies," Final Report to the National Science Foundation, Portland Cement Association, Skokie, Illinois, (1978).

6. Bertero, V. V., Aktan, A. E., Charney, F. A., and Sause, R, "U.S.-Japan Cooperative Research Program: Earthquake Simulation Tests and Associated Studies of a 1/5th-scale Model of a 7-Story R/C Frame-Wall Test Structure," Report No. UCB/EERC-84/05, University of California, Berkeley, California, (1984).

7. Kanaan, A. E., and Powell, G. H., "A General Purpose Computer Program for Inelastic Dynamic Response of Plane Structures," Report No. EERC 73-22, University of California, Berkeley, California, (1975).

8. Takeda, T., Sozen, M. A., and Nielsen, N. N, "Reinforced Concrete Response to Simulated Earthquake," Journal of the Structural Division, ASCE, 96(ST12), 2557-2573, (1970).