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## STRUCTURAL ASPECTS, PART IV

**EXPERIMENTAL FACILITIES AND INVESTIGATION OF STRUCTURES AND MODELS**

**DETERMINISTIC DYNAMIC PROPERTIES AND BEHAVIOUR OF STRUCTURES**

**VIBRATION MEASUREMENT OF FULL SCALE STRUCTURES**

**DYNAMIC BEHAVIOUR OF STRUCTURAL MATERIALS AND COMPONENTS**

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# BIAXIAL SHAKING TABLE STUDY OF A R/C FRAME

By M. G. Oliva[I] and R. W. Clough[II]

## SUMMARY

Experimental earthquake testing of a large scale reinforced concrete frame with inelastic biaxial response was undertaken on the University of California shaking table. Comparison of results with response measured in a previous uniaxial test showed decreased capacity and greater stiffness degradation in the rectangular columns under biaxial loading. Evaluation of local bending mechanisms demonstrated a considerable degree of biaxial interaction when non-linear response occurred.

## INTRODUCTION

In the event of a major earthquake, large lateral forces will be induced in the structural framework of a typical building. The dynamic nature of the loading may produce biaxial inelastic bending in the columns at one moment, and then reverse the moments as well as the contribution to axial loading due to overturning effects, all within a fraction of a second. Yet, most buildings are designed only on the basis of static lateral loads (specified by code), applied independently to frames oriented parallel with the two principal axes of the structure.

Can a design based on such simplistic concepts resist the actual combination of peak bending moments and axial loads developed in the members, considering the entire history of deformation? While the structure is in the elastic state, the two biaxial concurrent loadings induce no interaction between the responses along their two axes; hence the code assumption may be valid for linear behavior. However, if inelastic response due to loading along one axis changes the resisting mechanism for motion along the other axis, then load independence between the axes ceases and the code design procedure would be highly questionable. The extent that such biaxial coupling occurs in the earthquake response of real structures is presently a matter of conjecture, and it was the purpose of this research to shed some light on this question.

Although the results of static biaxial tests on square columns with constant axial loads have shown varying amounts of interaction (1) (2) (3), they may not reflect the total influence that coupling could have when all load components vary randomly during inelastic earthquake response. Aktan, et al (4), tested a square column with lumped mass under earthquake motion and reported unexpected permanent displacement drift due to biaxial interaction effects when the yield displacement was exceeded by a factor of two or more. Jirsa (5) reviewed various other biaxial test programs and summarized similar results and conclusions.

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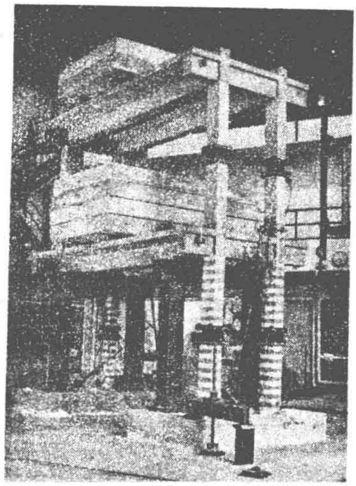
All of the reported tests either neglected the effects of axial load or considered only square column sections. Varying axial load may have serious effects on the strain history of the reinforcing and on the instantaneous effective concrete section. Square columns have no specific principal axes of flexural rigidity and would be expected to exhibit less biaxial response coupling than rectangular columns. The very nature of the rectangular column, with different moments of inertia along its principal axes, can be expected to induce exaggerated interaction effects under biaxial loading.

Fig. 1 Model Test Structure On the Shaking Table



## EXPERIMENTAL STUDY

In the present investigation, this biaxial response interaction mechanism was studied by means of experimental testing of a two-story reinforced concrete frame on the University of California, Earthquake Engineering Research Center's 20 ft. square shaking table. The seven-tenths scale model was subjected to intense earthquake motions applied at a skew angle relative to the structure's principal axes, thereby inducing significant biaxial column bending and overturning moments. The test structure, shown in Fig. 1, is identical to a frame tested previously under uniaxial motion applied along the model's major principal axis (6). Results of the previous tests thus serve as a control for comparison with the biaxial response of the frame in the present study.

Overall dimensions of the frame are illustrated in Fig. 2; the 7/10 scale allowed the use of normal reinforced concrete materials and fabrication procedures, and avoided the problems associated with modeling nonlinear behavior at small scales. The frame has a single bay in each direction; the four columns are connected by longitudinal and transverse 'T' beams cast integrally with the floor slab at each level. The columns have four

5/8 in. (1.6 cm.) diameter longitudinal reinforcing bars with 1/4 in. stirrups at 1-3/8 in. spacing, as shown in Fig. 3. The model was mounted on the shaking table with its longitudinal axis at a twenty-five degree angle to the horizontal excitation axis of the shaking table. A plan view of the frame and table layout is shown in Fig. 4; also indicated are the pretest first mode vibration frequencies along the longitudinal "stiff" axis and the transverse "weak" axis.

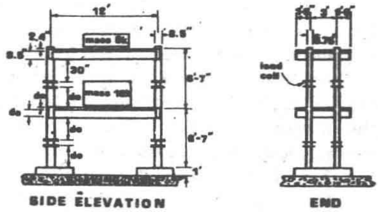



Fig. 2 Test Structure Dimensions


Fig. 3 Column Dimensions

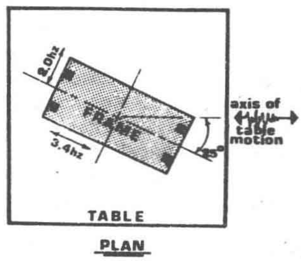



Fig. 4 Orientation on Table

The major excitation signal applied to the structure by the shaking table (Fig. 5) was derived from the Taft, California 1952 earthquake accelerogram. The maximum velocity applied in the strongest intensity test was 32.5 in/sec. (82.6 cm. sec.); the corresponding peak table acceleration was .7g and the maximum table displacement was 5 in. (12.7 cm.). More than one hundred forty transducers of various types were used to monitor the shaking table motion and frame response; measured quantities included accelerations, displacements, column forces, member curvatures and strains in reinforcing bars.

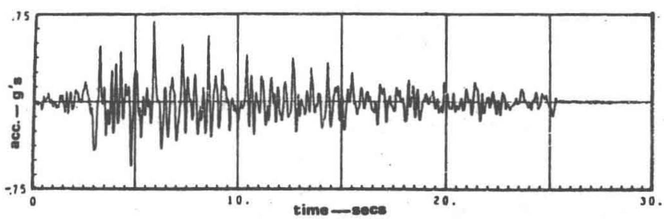



Fig. 5 Taft Earthquake - Table Acceleration History

## DYNAMIC RESPONSE BEHAVIOR

The biaxial nature of the response displacements is apparent in the trace of the motion shown in Fig. 6 as viewed from above. Indicated displacements were measured at the top of the first floor column shown as a darkened rectangle at the far left in Fig. 4 and show motions relative to the shaking table. The column's strong and weak axes coincide with the longitudinal and transverse axes of the frame. The maximum displacement loop toward the lower right in Figure 6 is in a direction nearly perpendicular to the axis of table motion; that axis is 25 degrees counter-clockwise from the horizontal plot axis.

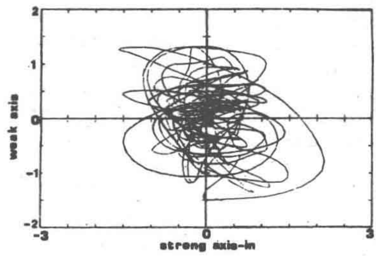



Fig. 6 Displacement Trace of Top of Column

If the first floor longitudinal component of this biaxial response (measured on Test Structure RCF5) is compared with the first floor displacements measured in the previous equivalent uniaxial tests (Structure RCF2) the two records are remarkably similar. The only major difference between the longitudinal displacement histories of the two frames is the increased degradation of stiffness resulting from biaxial damage. This degradation is evidenced by the increased first mode longitudinal vibration period, shown marked on the response spectrum for the test motion (Fig. 7). The uniaxial test frame (RCF2) showed a smaller change of period (0.32 to 0.49 sec.) in the corresponding test, thus demonstrating a lesser degree of damage.

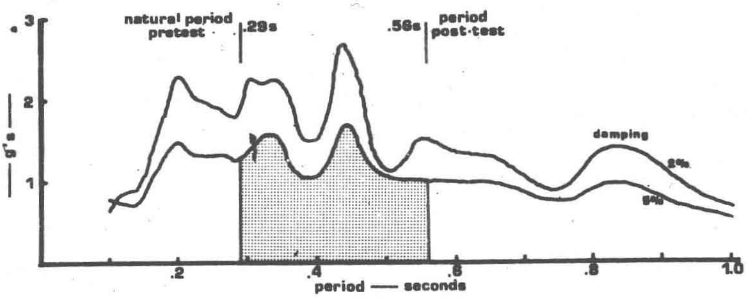



Fig. 7 Pseudo-acceleration Response Spectrum of Table Motion and Variation of Model Period

Most of the inelastic deformation and damage to the structure occurred at the extremities of the lower columns, with spalling and concrete crushing initiated at the corners (Figure 8). Greater visible column damage was apparent in the present biaxial frame than was seen in the earlier uniaxial test. However, there was virtually no visible beam damage in the present frame, whereas cracking was evident through the full depth of the beams in the previous tests; hence it may be inferred that less force was transferred between beams and columns in the biaxial test.

The smaller column forces developed in the biaxial test (RCF5), as compared with the uniaxial test (RCF2) are apparent in Fig. 9, where column shear along the longitudinal axis is plotted against longitudinal displacement of the test structure. Though the initial column stiffness (k) was

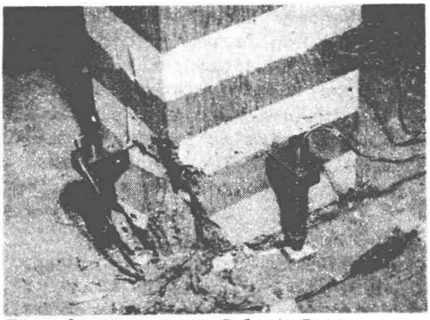


Fig. 8 Damage at Column Base

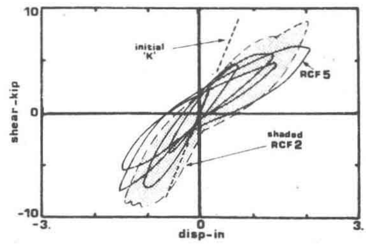


Fig. 9 Column Force-Displacement

identical for both frames, greater stiffness degradation is apparent in the biaxial loading test of RCF5. Moreover, the multiaxial load combination obviously decreases the yield capacity of the frame below that available in uniaxial loading.

## EVALUATION OF RESPONSE MECHANISM

Detailed studies of the local column response in terms of the moment-curvature relationship indicate a considerable amount of coupling occurring between the response in the two axes. Figures 10a and b are plots of the moment-curvature history at the column base during a period of intense motion.

(a) North-South Axis (b) East-West Axis

Fig. 10 Moment-Curvature Variation at Column Base

The bending response is separated into components along the major (north-south) axis and minor (east-west) axis. Positive moments correspond with compression on the south and west column faces respectively. Letters indicate corresponding points in time on each of the plots. The interval plotted extends from 5.68 seconds to 6.38 seconds in the earthquake record, but similar behavior, indicating significant strong axis influence on weak axis moments is evident in the entire response history after the first large displacement excursion at 3.14 seconds.

The nature of the response interaction becomes understandable if the moment-curvature results of Figs. 10a and b are considered in conjunction with the axial load variation, with the cracking condition of the confined and cover concrete, and with the reinforcing bar strain history and instantaneous stiffness. Prior to the start of the interval at 5.68 seconds, the reinforcing bars already have permanent residual strains ranging from 0.3% to 1.0% and have been strained to a maximum of 1.5%. The concrete has developed open residual cracks through the entire concrete cross section (in the absence of significant compression forces.) Loss of bond between concrete and steel was detected over at least a 3 in. length on one of the bars and the steel strains listed above justify assumption of bond loss over segments of all of the bars at the column-to-footing joint. Calculated concrete strains indicate that crushing of some of the concrete cover has occurred at the two corners on the north column face.

In the interval shown between points 'b' and 'c', for instance, the weak axis east-west bending plot has a plateau of low apparent stiffness while the north-south (strong) component is unloading from a southerly peak at 'b'. The north-south component bending moment changes from positive 145 in.-k (16.4kN-m) to a value of zero, while the east-west component increases slightly from positive 11 in.-k (1.2kN-m) to 21 in.-k(2.4kN-m). During the short 'b-c' period (0.12 sec.) the axial load increases from 3.0 kips (13kN) to 21.8 kips (97kN) as a result of changing overturning moments.

At point 'b', under low axial load and high residual bar strains, the entire column was cracked open and the bar in the south-west corner was yielding in compression, while the bar in the southeast corner was near compressive yielding. By the time of point 'c', the open crack along the north face of the column resulting from the south moment at 'b' had closed at its west corner under the increased axial load and relatively constant west moment. The reinforcing bar at the northwest corner was yielding in compression, the southwest bar was under elastic compression and the southeast bar changed from near compressive yield to tension. The large change of strain in the southeast bar and the yielding northwest bar caused a rotation in the column about an axis running roughly through the northeast and southwest corners.

Thus an apparent east-west component of rotation occurred while the column was under constant moment, and produced the flat segment in the east-west moment curvature diagram. Similar changes in apparent stiffness have been detected when the concrete cover at a corner reached it's crushing strain, effectively reducing the section size along both axes.

Unexpected deformations also occurred when decreasing axial loads affected the yield strain in specified bars, while other bars remained elastic under constant moment.

## CONCLUSION

Studies of the test data have verified that the resisting capacities of the structure are reduced under multiaxial loading as expected; also a marked degree of response interaction has been demonstrated when non-linear motion involving changes in the section stiffness occurred. In the test structure which has rectangular columns with distinct strong and weak axes, the biaxial coupling occurred predominantly in the form of a significant strong axis influence on weak axis response.

Correlation studies are now being undertaken to compare the experimental test results with predictions from various types of computer analysis. Methods considered include combining 2-D frame analyses along separate axes and using 3-D degrading stiffness modeling of the entire structure. The vast amount of test data that has been obtained during this investigation warrants extensive study, and the final report on the project will not be completed for several months.

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A NAIVE MODEL FOR NONLINEAR RESPONSE OF REINFORCED CONCRETE BUILDINGS

By

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SUMMARY

A simple and economical model is introduced for the calculation of the nonlinear displacement-response histories of multi-story structures subjected to strong earthquakes. A structure is idealized as a mass connected to a rigid bar that in turn is connected to the ground by a hinge and a rotational spring. The calculated responses are compared with measured experimental results from dynamic testing of eight small-scale ten-story model structures. Satisfactory correlation between the analytical and experimental results has been observed.

INTRODUCTION

Structures designed according to current engineering practice in the U.S. are expected to develop nonlinear deformations when subjected to strong ground motions. Although nonlinear analysis of structures is a complicated and lengthy process, with the help of sophisticated digital computers successful analytical models have been developed for this purpose [7, 9]. Because of the involved data preparation procedures and, at times, due to lack of confidence in complicated programs (which cannot be checked easily) these models have not been utilized by the engineer in practice who needs a simple model which can be easily used for several possible alternative designs.

This paper introduces a simple nonlinear model (called the Q-Model) to calculate the seismic displacement-response histories of multi-story reinforced concrete structures. Measured response histories of eight small-scale ten-story structures are used to evaluate the results of the model.

DESCRIPTION OF THE MODEL

The idea of representing a mutli-degree-of-freedom system by a "single-degree" system with some generalized mass, stiffness, and damping has been used for elastic structural systems. The extension of such idealization for inelastic problems has been viewed with some caution because of the changing stiffness properties and, therefore, dynamic properties of inelastic systems.

Current engineering practice encourages the designer to proportion the columns of a structure such that they experience only limited yielding during the design earthquake. Experimental results from testing of reinforced concrete structures designed according to this criterion indicate that the deflected shape will tend to remain essentially unchanged as nonlinear deformations are developed [1,3,4,5]. Furthermore, displacement responses have been shown to be dominated by the first mode. Therefore, a multi-story structure with the above properties can be reduced to a single-degree system with some source of hysteretic energy dissipation.

Equivalent Mass. The Q-Model is shown in Fig. 1. The governing dynamic differential equation can be described as [2].

$$\alpha_m M_t \ddot{x} + \alpha_\ell K x = -\alpha_\ell M_t \ddot{y} \qquad (1)$$

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where

$M_t$ = total mass of the MDOF system

K = stiffness of the MDOF system (overall stiffness defined in terms of a particular lateral force and a particular horizontal displacement)

x = lateral displacement of the mass of the SDOF oscillator with respect to its base

$\alpha_\ell = (\sum_{r=1}^{j} M_r \phi_r) / M_t$

$\alpha_m = (\sum_{r=1}^{j} M_r \phi_r^2) / M_t$

r = numeral identifying level in MDOF system

j = total number of levels in MDOF system

$M_r$ = mass at level r

$\phi_r$ = ratio of assumed displacement at level r to that at level j.

$\ddot{y}$ = acceleration of the base.

To simplify the equation, both sides are divided by $\alpha_\ell$, and a damping force is added.

$$M_e \ddot{x} + C\dot{x} + Kx = -M_t \ddot{y} \tag{2}$$

in which $M_e = (\alpha_m/\alpha_\ell) M_t$, equivalent mass; and C = viscous damping coefficent.

Stiffness Properties. To define the stiffness characteristics, assumptions are made about the primary force-deformation relationship and stiffness variations for unloading and load-reversal stages. The primary curve is directly related to the stiffness of the multi-story structure and is obtained from a static analysis of the structure for a set of monotonically increasing lateral forces applied at floor levels (Fig. 1). The lateral force at a given level is proportional to the mass and height at that level. The primary curve is then approximated by a bilinear curve. One possible set of rules for such approximation is given in Reference 11.

The assumptions about stiffness variations upon unloading and subsequent loadings are included in a simple hysteresis model described by four rules. Appendix A in Reference 11 describes the details of the hysteresis model.

Corresponding to each point of the primary curve there is a lateral deflected shape for the multi-story structure. The shape corresponding to the beak point of the idealized binary curve is assumed to represent the vibration shape of the structure. The height of the mass in the Q-Model is assumed to be

$$L_e = \frac{\sum_{r=1}^{j} M_r \phi_r h_r}{\sum_{r=1}^{j} M_r \phi_r}$$

in which $h_r$ = the height of level r from base.

Solution Technique. With an arbitrary damping factor of 2%. Equation 1 was integrated using Newmark's β-method [6]. The value of β was taken as 0.25.

## MODEL STRUCTURES

Eight small-scale ten-story reinforced concrete model structures were analyzed using the Q-Model. Four of these (MF1, MF2, H1, and H2) consisted of only two frames. Each of the other four (FW1, FW2, FW3, and FW4) comprised two frames as well as a central shear wall. The structures were subjected to simulated earthquakes at the University of Illinois at Urbana.

The input motion was applied to the structures in horizontal direction and parallel to the strong axis of each building. Structures MF1, MF2, H1, H2, FW1, and FW4 were subjected to a simulated north-south component of El Centro, 1940. The input motion for the other two was modeled after a north-east component of Taft 1952. All but one of the structures (H2) were subjected to three motions with increasing intensity from one run to the other. The first run for each case corresponded to the "design earthquake"