

# **Proceedings of the Seventh World Conference on Earthquake Engineering**

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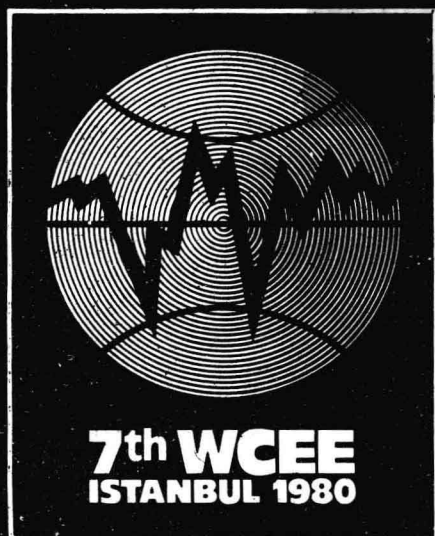
**Structural Aspects**

**Part 4**

September 8-13, 1980

# PROCEEDINGS OF THE SEVENTH WORLD CONFERENCE ON EARTHQUAKE ENGINEERING

September 8-13, 1980  
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VOLUME

7

## 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

## TABLE OF CONTENTS

BIAXIAL SHAKING TABLE STUDY OF A R/C FRAME .....	1
M.G. Oliva, R.W. Clough	
A NAIVE MODEL FOR NONLINEAR RESPONSE OF REINFORCED CONCRETE BUILDINGS .....	8
Mehdi Saidi, Mete Sözen	
DUCTILITY AND STRENGTH OF REINFORCED CONCRETE COLUMNS WITH SPIRALS OF HOOPS UNDER SEISMIC LOADING .....	15
R. Park, M.J.N. Priestley, W.D. Gill, R.T. Potangaroa	
SEISMIC BEHAVIOR OF PRECAST WALLS COUPLED THROUGH VERTICAL CONNECTIONS.....	23
Peter Mueller, James M. Becker	
DELAYING SHEAR STRENGTH DECAY IN REINFORCED CONCRETE FLEXURAL MEMBERS UNDER LARGE LOAD REVERSALS .....	31
Charles F. Scribner, James K. Wight	
INELASTIC BEHAVIOR OF WIDE-FLANGE BEAM-COLUMNS UNDER CONSTANT VERTICAL AND TWO-DIMENSIONAL ALTERNATING HORIZONTAL LOADS /.....	39
Chiaki Matsui, Shasuke Morino, Keigo Tsuda	
STRENGTH AND HYSTERESIS CHARACTERISTICS OF STEEL-REINFORCED CONCRETE BEAM-COLUMNS WITH BASE .....	47
Takeo Naka, Koji Morita, Masahiko Tachibana	
TENSION LAP SPLICES UNDER SEVERE LOAD REVERSALS .....	55
J.D. Aristizabal-Ochoa, A.E.Fiorato, W.G. Corley	
SLOSHING OF LIQUIDS IN RIGID ANNULAR CYLINDRICAL AND TORUS TANKS DUE TO SEISMIC GROUND MOTIONS .....	63
M. Aslam, W.G. Godden, D.T. Scalise	
MECHANISM OF CONFINEMENT IN TIED COLUMNS .....	71
Shamim A. Sheikh, S.M. Üzümeri	
FORCED VIBRATION TESTS AT A NUCLEAR POWER PLANT .....	79
K. Hanada, A. Kato, M. Kanae	
LARGE SCALE TESTS FOR THE HYSTERESIS BEHAVIOR OF INCLINED BRACING MEMBERS .....	87
Heinrich Gugerli, Subhash C. Goel	
SPATIAL SUBASSEMBLAGE TESTS FOR R/C FRAMES TO DYNAMIC CYCLIC LOADING .....	95
Dumitru Vasilescu	
DUCTILITY BEHAVIOR OF REINFORCED CONCRETE FRAMES .....	99
L.R. Gupta, Brijesh Chandra	
EARTHQUAKE SIMULATOR STUDY OF A STEEL FRAME SMALL-SCALE MODEL .....	103
Russell S. Mills, Helmut Krawinkler	

STATIC AND SEISMIC MODEL BEHAVIOUR OF A NEW TYPE OF MIXED STRUCTURE FOR REINFORCED CONCRETE INDUSTRIAL STORIED BUILDINGS ....	107
C. Mihai, M. Manolovici, St. Carlan, St. Marinescu, M. Itrcovici	
SEISMIC BEHAVIOR OF MASONRY BUILDINGS .....	111
Pedro A. Hidalgo, Hugh D. Mc Niven	
AN INVESTIGATION OF THE SEISMIC BEHAVIOR AND REINFORCEMENT REQUIREMENTS FOR SINGLE-STORY MASONRY HOUSES .....	119
R.W. Clough, P. Gü lkan, R.L.Mayes	
ECCENTRIC SEISMIC BRACING OF STEEL FRAMES .....	127
Egor P. Popov	
EARTHQUAKE-INDUCED PERMANENT DISPLACEMENTS IN SMALL SHEAR-TYPE STEEL STRUCTURES .....	133
Dixon Rea, Alvar M. Kabe	
SEISMICALLY LOADED HOLDING DOWN BOLTS .....	141
R. Shepherd	
EXPERIMENTAL SEISMIC VERIFICATION OF ASSEMBLED PREFABRICATED STRUCTURES .....	149
A. Cantoldi, M. Casirati, P. Pezzoli	
LARGE-SCALE EARTHQUAKE SIMULATION TABLES .....	157
J.D. Aristizabal-Ochoa, A.J. Clark	
SHAKING TABLE TEST OF STEEL FRAMES .....	165
Takeshi Nakamura, Nozomu Yeshida, Satoshi Iwai Hidehiro Takai	
EXPERIMENTAL STUDY ON STRENGTHENING REINFORCED CONCRETE STRUCTURE BY ADDING SHEAR WALL .....	173
Yoichi Higashi, Toneo Endo, Masamichi Ohkubo, Yasushi Shimizu	
A COMPARISON OF THREE LABORATORY TEST METHODS USED TO DETERMINE THE SHEAR RESISTANCE OF MASONRY WALLS .....	181
A. Bernardini, C. Modena, V. Turnsek, U. Vescovi	
A NEW DYNAMIC CONTROL 4-MOMENT VIBRATION GENERATOR .....	185
Shohei Hayashi	
RIGIDITY AND STRENGTH OF REINFORCED CONCRETE STRUCTURE WITH SPANDRELS AND WING WALLS .....	189
Setsuro Nomura, Kazuhide Sato	
REPEATED HORIZONTAL DISPLACEMENTS OF INFILLED FRAMES HAVING DIFFERENT STIFFNESS AND CONNECTION SYSTEMS-EXPERIMENTAL ANALYSIS .....	193
Alberto Parducci, Marco Mezzi	

SOME ANALYSES ON MECHANISMS TO DECREASE EARTHQUAKE EFFECTS TO BUILDING STRUCTURES (PART 7, STEEL DAMPERS FOR TALL BUILDINGS) .....	197
Kiyoo Matsushita, Masanori Izumi, Hirozo Mihashi, Tatsuo Sasaki, Noriaki Nomura	
INELASTIC EARTHQUAKE RESPONSE OF TALL R.C. BRIDGE PIERS WITH EMPHASIS ON ULTIMATE STATES .....	201
H. Goto, T. Kasai, N. Imanishi	
A STUDY ON OVERTURNING VIBRATION OF RIGID STRUCTURES .....	205
N. Ogawa	
RESTORING FORCE CHARACTERISTICS OF RC WALLS WITH OPENINGS AND REINFORCING METHODS .....	209
Hajime Umemura, Hiroyuki Aoyama, Youji Hosokawa	
STRUCTURAL TEST AND ANALYSIS ON THE SEISMIC BEHAVIOR OF THE REINFORCED CONCRETE REACTOR BUILDING .....	217
Takashi Uchida, Nobutsugu Ohmori, Tsunehisa Tsugawa, Akira Endoh	
PSEUDO-DYANAMIC TESTS ON A 2-STORY STEEL FRAME BY COMPUTER-LOAD TEST APPARATUS HYBRID SYSTEM .....	225
Koichi Takanashi, kuniaki Udagawa, Hisashi Tanaka	
EFFECTS OF WELDED BAND PLATES ON ASEISMIC CHARACTERISTICS OF REINFORCED CONCRETE COLUMNS .....	233
Takashi Arakawa	
SEISMIC BEHAVIOR OF INELASTIC MEMBERS OF BRACED FRAME STRUCTURE .....	241
Teizo Fujiwara	
ELASTO-PLASTIC BEHAVIOR OF STEEL FRAME WITH STEEL-CONCRETE COMPOSITE BEAM UNDER CYCLIC HORIZONTAL LOADING .....	249
M. Yamada, B. Tsuji, H. Adachi	
DYANAMIC COLLAPSE TESTS OF REINFORCED CONCRETE FRAME STRUCTURES WITH A COLUMN SUBJECTED TO HIGH COMPRESSIVE STRESS .....	257
Hiroaki Eto, Toshikazu Takeda	
USING TUNED-MASS DAMPERS TO REDUCE SEISMIC RESPONSE .....	265
J.R. Sladek, R.E. Klingner	
EARTHQUAKE-INDUCED PERMANENT DISPLACEMENTS IN MODEL REINFORCED EARTH WALLS .....	273
Dixon Rea, William E. Wolfe	
ANALYTICAL AND EXPERIMENTAL INVESTIGATION ON SEISMIC BEHAVIOR OF FRAME-PANEL TYPE NONSTRUCTURAL WALLS .....	281
Isao Sakamoto	
BEHAVIOUR OF REINFORCED CONCRETE BEAM-COLUMN JOINTS UNDER CYCLIC LOADING .....	289
P. Gavrilovic, M.Velkov, D.Jurukovski, D.Mamucevski	

REPAIRED REINFORCED CONCRETE MEMBERS AND JOINTS UNDER CYCLIC LOADING .....	297
Miodrag Velkov, P.M. Gavrilovic	
HYSTRETIC CHARACTERISTICS OF BEAM-TO-COLUMN CONNECTIONS IN STEEL SEINFORCED CONCRETE STRUCTURES .....	305
Koichi Minami, Yasushi Nishimura	
INELASTIC RESPONSE OF STEEL FRAMES SUBJECT TO MULTICOMPONET EARTHQUAKES .....	309
Erdoğan Uzgider	
RECENT WORK ON THE DYNAMIC BEHAVIOUR OF TALL BUILDINGS AT VARIOUS AMPLITUDES .....	313
B.R. Ellis, A.P. Jeary	
STATIC AND DYNAMIC RESPONSE ANALYSIS OF A R/C BUILDING DAMAGED BY MIYAGI-KEN-OKI EARTHQUAKE OF JUNE, 1978 .....	317
Tomoya Nagasaka	
INFLUENCE OF LOCAL AND LATERAL BUCKLING ON INELASTIC BEHAVIOR OF STEEL FRAMES .....	321
Chiaki Matsui, Takashi Yoshizumi	
THE RESPONSE OF VETERANS HOSPITAL' BUILDING 41 IN THE SAN FERNANDO EARTHQUAKE .....	325
Avigdor Rutenberg, Paul C. Jennings, George W. Housner	
INELASTIC, CYCLIC BEHAVIOR OF REINFORCED CONCRETE FRAME-WALL STRUCTURES SUBJECTED TO LATERAL FORCES .....	333
Ikuro Yamaguchi, Shunsuke Sugano, Yasuo Higashibata, Toshio Nagashima	
FIELD TEST OF FINITE ELEMENT CONCRETE DAM ANALYSIS .....	341
Ronald F. Scott, J. Brent Hoerner	
PRECAST CONCRETE BRACED FRAMES INCORPORATING LOAD-LIMITING ENERGY-DISSIPATING DEVICES IN AN EARTHQUAKE RESISTANT BUILDING ....	349
C.D. Matthewson, R.A. Davey	
PERFORMANCE OF THE BUILDING OF FACULTY OF ENGINEERING, TOHOKU UNIVERSITY DURING THE 1978 MIYAGI-KEN-OKI EARTHQUAKE .....	357
Toshio Shiga, Akenori Shibata, Junichi Shibuya, Junichi Takahashi	
SLIDING OF BRIDGE ABUTMENTS DURING EARTHQUAKE .....	365
He Duxin, Gao Jinying	
VERTICAL EARTHQUAKE LOAD ON CHIMNEYS .....	373
Qian Peifeng, Su Wunzao, Yang Yadi	

STATIC AND DYNAMIC ANALYSES OF A REINFORCED CONCRETE HOTEL BUILDING DAMAGED DURING THE OITA EARTHQUAKE OF APRIL 21, 1975 .....	381
Koji Yoshimura, Kenji Kikuchi	
MATHEMATICAL MODELLING OF HYSTERESIS LOOPS FOR REINFORCED CONCRETE COLUMNS .....	389
Shinsuke Nakata, Terry Sproul, Joseph Penzien	
DAMAGE OF WELDED STEEL STRUCTURES DUE TO STRONG EARTHQUAKES .....	397
Kiyoshi Kaneta, Isao Kohzu	
DYNAMIC BEHAVIOR OF PIPELINE DURING THE MIYAGI-KEN-OKI EARTHQUAKE IN 1978 .....	405
Jiro Miyauchi, Jun Tsujimoto	
DYNAMIC RESPONSE OF A REINFORCED CONCRETE COLUMN MODEL .....	413
F. Vestroni, D. Capecchi, G. Rega	
NONLINEAR CYCLIC BEHAVIOR OF REINFORCED CONCRETE PLANE STRESS MEMBERS .....	
Haluk Aktan, Movses J. Kaldjian	
OSCILLATION DECREMENTS OF BUILDINGS AND STRUCTURES .....	429
G.A. Shapiro, V.F. Zakharov	
DAMPING ESTIMATION FROM FULL-SCALE CYCLIC TESTING OF A FIVE-STORY STEEL FRAME .....	433
Jean-Guy Beliveau, Michel Favillier	
MEASUREMENTS AND CALCULATIONS OF NATURAL FREQUENCIES AND COUPLED BENDING-TORSION MODES OF HIGHRISE BUILDINGS.....	437
Eberhard Luz, Siegfried Gurr	
AMBIENT VIBRATION SURVEY OF A 325-METER HIGH MAST .....	441
Bao Zhi-Wen, Qiu Zong-Lian, Lai Jin-Yan	
PSEUDO-DYNAMIC TESTS ON FRAMES INCLUDING HIGH STRENGTH BOLTED CONNECTIONS .....	445
Koichi Takanashi, Hidetake Taniguchi	
EFFECT OF CLADDING ON BUILDING RESPONSE TO MODERATE GROUND MOTION .....	449
Barry J. Goodno, Kenneth M. Will, Hafsteinn Palsson	
SEISMIC RESPONSE OF HYSTERETIC DEGRADING STRUCTURES .....	457
Thomas T. Baber, Yi-Kwei Wen	
DRY FRICTION DAMPING OF MULTISTORY STRUCTURES .....	465
W.O. Keightley	
VIBRO-TESTING OF MULTI-STOREY FRAMED BUILDINGS .....	472
Yu. Simon, V. A. Zakarian, R. A. Badalian	

A FIELD WORK INVESTIGATION OF STEEL BUILDING DAMAGE DUE TO THE 1978 MIYAGIKEN-OKI EARTHQUAKE .....	479
Ben Kato, Atsuo Tanaka, Hiroyuki Yamanouchi	
HYSTERETIC ENERGY SPECTRA IN SEISMIC DESIGN .....	487
W.E. McKeivitt, D.L. Anderson, N.D. Nathan, S. Cherry	
INELASTIC BEHAVIOR OF BUILDING SYSTEMS SUBJECTED TO THREE-DIMENSIONAL EARTHQUAKE MOTIONS .....	495
Franklin Y. Cheng, Prasert Kitipitayangkul	
EFFECTS OF LOCALIZED FAILURE IN EVALUATING ULTIMATE CAPACITY OF R/C BUILDINGS, .....	503
H. Takizawa	
TEST OF RC SHEAR WALLS SUBJECTED TO BI-AXIAL LOADING .....	511
Hiroyuki Aoyama, Manabu Yoshimura	
VIBRATION TESTS ON REINFORCED CONCRETE TOWERS FOR MICROWAVE TELECOMMUNICATION .....	519
Yuji Sato, Yukio Sawabe	
STRUCTURAL DAMAGE AND STIFFNESS DEGRADATION OF BUILDINGS CAUSED BY SEVERE EARTHQUAKES .....	527
Junji Ogawa, Yoshihiro Abe	
VIBRATION TESTS ON MANY TYPES OF BASE AND BUILDING MODELS SET ON REAL GROUND .....	535
Soichi Kawamura, Koji Kitazawa, Nagahide Kani, Masatashi Hisano	
NECESSARY AND SUFFICIENT CONDITIONS FOR PROPERTIES OF FRAMED SHEAR WALLS AFTER CRACKING .....	543
Y. Sonobe, M. Yokoyama, H. Imai	
DAMPING DETERMINATION FROM FULL-SCALE EXPERIMENTS .....	551
Trifun Paskalov, Ljubomir Taskov, Boris Taskovski	
CONVENTIONAL METHOD FOR ESTIMATING THE FLOOR RESPONSE PROPERTIES OF ELASTIC-AND ELASTO-PLASTIC SUB-STRUCTURE SYSTEMS ....	559
K. Suzuki, S. Aoki	
DYNAMIC BEHAVIOR OF LARGE PANEL CONNECTIONS .....	563
Nina Avramidou Maio	
NON SYMMETRIC RESPONSE OF SYMMETRIC R/C STRUCTURES TO BIAXIAL SEISMIC INPUTS .....	567
Alberto Parducci, Marco Mezzi	



# BIAXIAL SHAKING TABLE STUDY OF A R/C FRAME

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## 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.

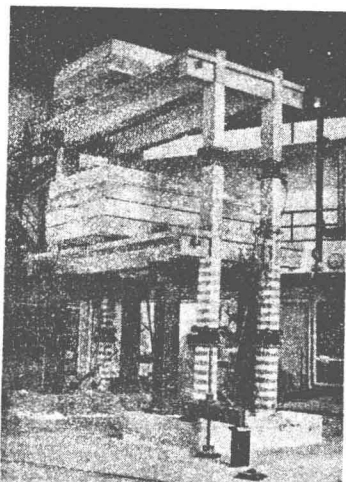
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All of the reported tests either neglected the effects of axial load or considered only square column sections. Varying axial load may have serious effects on the strain history of the reinforcing and on the instantaneous effective concrete section. 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.

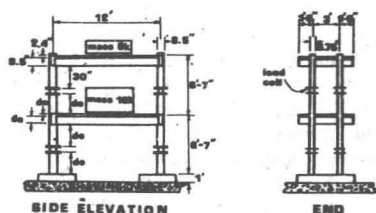


Fig. 2 Test Structure Dimensions



Fig. 3 Column Dimensions

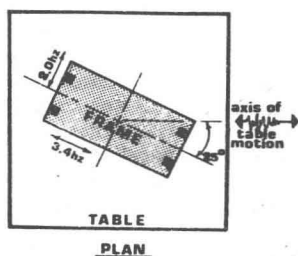


Fig. 4 Orientation on Table

The major excitation signal applied to the structure by the shaking table (Fig. 5) was derived from the Taft, California 1952 earthquake accelerometerogram. 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.

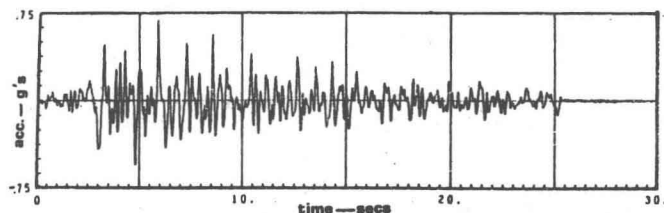


Fig. 5 Taft Earthquake - Table Acceleration History

## 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.

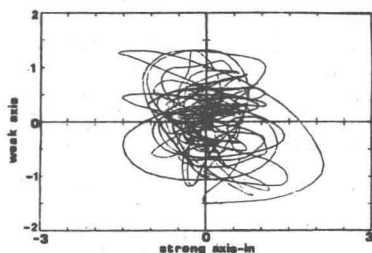


Fig. 6 Displacement Trace of  
Top of Column

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.

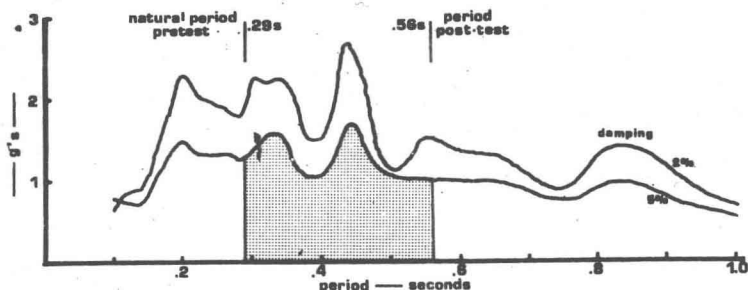


Fig. 7 Pseudo-acceleration Response Spectrum of Table  
Motion and Variation of Model Period

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

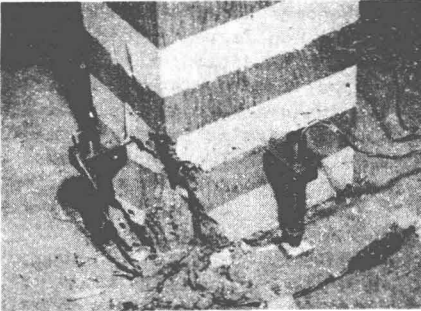


Fig. 8 Damage at Column Base

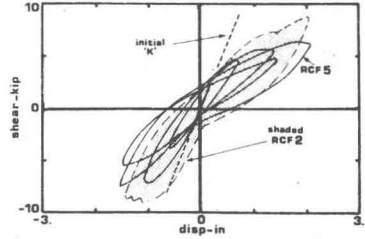
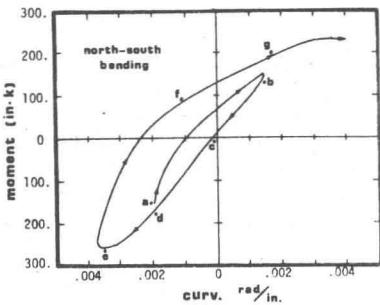


Fig. 9 Column Force-Displacement

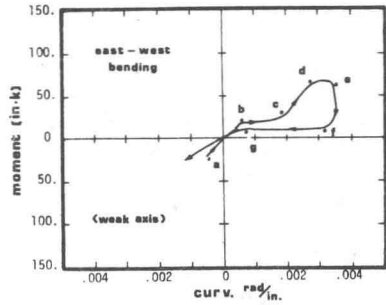
identical for both frames, greater stiffness degradation is apparent in the biaxial loading test of RCF5. Moreover, the multiaxial load combination obviously decreases the yield capacity of the frame below that available in uniaxial loading.

#### 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 its 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 multi-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].

$$\alpha_m M_t \ddot{x} + \alpha_k K_o = -\alpha_t M_t \ddot{y} \quad (1)$$

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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  
 $\alpha_l = \left( \sum_{r=1}^j M_r \phi_r \right) / M_t$   
 $\alpha_m = \left( \sum_{r=1}^j M_r \phi_r^2 \right) / M_t$   
 $r$  = 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  $\alpha_l$ , and a damping force is added.

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

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

**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 peak 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

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

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  $\beta$ -method [6]. The value of  $\beta$  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"