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Inelastic earthquake response of space reinforced concrete frames

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ABSTRACT: The dynamic experiments of models are carried out on shaking table. The dimensions of two test models, with three stories, two spans and two bays, are in one fifteeth scale of the prototype structure. The first test model is designed as structural system having plane eccentricities by noncoincidence of centers of mass and stiffness. In order to study the inelastic response of reinforced concrete frame under bi-directional ground motion, the second test model is under an earthquake simulation with a diagonal direction of base excitation. The analysis of earthquake response is performed. It is indicated that the experimental and analytical results in the elastic and inelastis stages are in good agreement.

1 INTRODUCTION

When the internal forces and deformations of the structures are usually far over the linear elastic range under a severe earthquake, their true three dimensional response may differ significantly from that predicted by using elastic analysis. This difference can be particularly acute when the distribution of the lateral load resisting elements, or centers(or both), is such that eccentricities in plane induce an unequal demand on different resisting elements. From structural damage investigations performed after many major earthquakes, it has been observed that plane eccentricity by noncoincidence of centers of mass and stiffness could lead to very dramatic inelastic response. These investigations also show that it is very difficult to predict this damage by elastic analysis. Satisfactory aseismic design and accurate prediction of response of reinforced concrete structures to strong ground motion may require a more realistic consideration of the true three-dimensional response. The reduced member capacities under multiaxial loading must satify response demands which may be increased by three-dimensional motion. Less attention has been paid to determining how inelasticity induced by multiaxial loading might change the nature of response in a structural system.

In order to gain a better understanding of dynamic behavior of space reinforeed concrete frame due to an earthquake excitation and to verify the mechanical model and analysis method, two structure models were tested on shaking table.

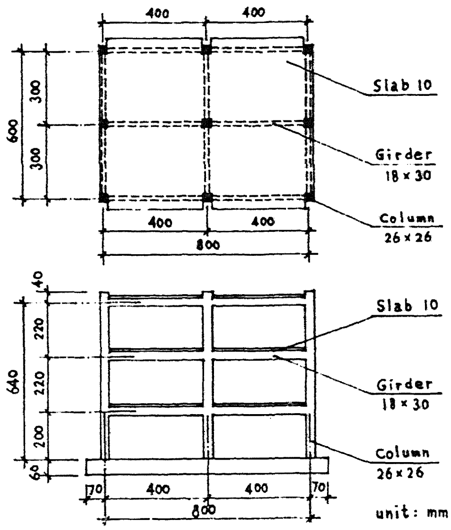

Figure 1. Test structure Dimensions.

2 OUTLINE OF EXPERIMENTAL PROGRAM

The dimensions of two three story models with two spans and two bays are in one fifteeth scale of the prototype frame structures. Designations of two test models are used as FR1 and FR2 respectively. The overall configuration of the models is shown in Fig.1. The elements of girders and columns were connected in site at the joint regions. The sections of columns and girders are 2.6 ×2.6 cm and 1.8×3.0 cm respectively. The first test model FR1 is designed as structural system having plane eccentricities created by noncoincidence of centers of mass and stiffness under a single directional earthquake simulation. In order to study the inelastic response of reinforced concrete frame under bi-directional ground motion, the second test model FR2 is under an earthquake simulation with a diagonal direction of base excitation. The model FR2 was mounted on the shaking table with its longitudinal axis at a sixty degree angle to the horizontal excitation axis of shaking table.

The columns have four 2.2 mm diameter longitudinal reinforcing bars with 0.9 mm stirrups at 8 mm spacing. The girders have four 1.2 mm diameter longitudinal reinforcing bars with 0.9 mm stirrups at 10 mm spacing. The yielding strength of ϕ 2.2 and ϕ 1.2 mm is 331 and 345 N/mm^2 respectively. The cubic strength of microconcrete is 23.5 N/mm^2. The elastic modulus of microconcrete is 2.35×10^4 N/mm^2. The two models are designed as weak beam-strong column system, in which $\Sigma Mc/\Sigma Mb = 2.3-4.6$ where ΣMc and ΣMb are the total ultimate moment of the beams and columns at the joint respectively. The flexural steel was welded at the joint regions for continuity and integrity.

The tests were carried out on an electro-magnetic shaking table. Story weights were made of lead, and were 60 kg at the top story level and 120 kg at all other story level. Instrumentation of a test model was arranged so that absolut accelerations and displacement in plane at each story level and base were measured. The primary test for each model was an earthquake simulation for which a single direction of base motion was modelled after a measured earthquake acceleration record. Base acceleration for earthquake simulation was modelled after the N -S component of El -Centro 1940 earthquake. Time scale of simulation was compressed by a factor of 8 so that reasonable ratio of base-motion to test structure frequencies would result. The experiments were divided into three stages: (1) the elastic stage befor the cracks appeared; (2) the inelastic stage after the cracks appeared; (3) the failure stage after the yield of model.

In order to investigate and compare the dynamic characteristics of the models during the different experimental stages, complementary test was conducted to measure response to ambient vibration and the response to random base motion by white noise process with a small amplitude.

3 THEORETICAL ANALYSIS

In analysis the following assumptions are used:

1. The stiffness in the plane of the horizontal floor is infinite.
2. Trilinear degrading type is used for representing the characteristics of the

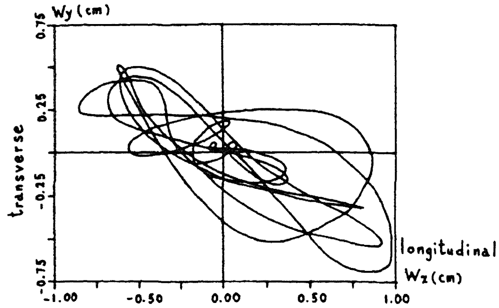

Figure 2. Displacement trace of column top of third floor for FR1.

restoring force of bending moment and curvature. This restoring force model are extended from one-dimensional degrading type of trilinear hysteresis rule by using the associated plastic flow rule and Mroz's(1976) hardening rule(Du 1990). In this model the biaxial coupling is considered on base of the experimental results. The rationality of this model is verified with the test results of reinforced concrete columns under arbitarity paths.

3. The influence of the bar slippage in joint on response of the structure is considered. The restoring force model of the slippage rotation spring at the end of the element is used (Shen 1982).
4. When the bending moment of the element reaches the yielding moment, the plastic hing zone will be formed and be concentrated at the ends of the element. The element can be considered as an element of variable stiffness with a rigid zone. The length of plastic hinge zone is increased with the development of plastic deformation, but it will be unchanged during unloading and reversed loading.

4 TEST RESULTS AND COMPARISON WITH CALCULATED RESPONSES

The displacement trace of column top of third floor for model FR1 during simulation with peak base acceleration 2.23g is shown in Fig.2.The maximum displacement loop is toward in a direction, which is 35 degree counter-clockwise from the axis of the table motion. Fig.3 shows the traces for displacement of column top of third floor and base shear force in model FR2 during simulation with peak base acceleration 1.87g. The maximum displacement loop is toward in a direction nearly parallel to the axis of the table motion. From those figures it is indicated that the bi-axial nature of response is apparent in two models.

Measured variations of maximum base shear force and top displacement for FR1 and FR2 at the different experimental stages are shown in Fig.4 and Fig.5 respectively. In those

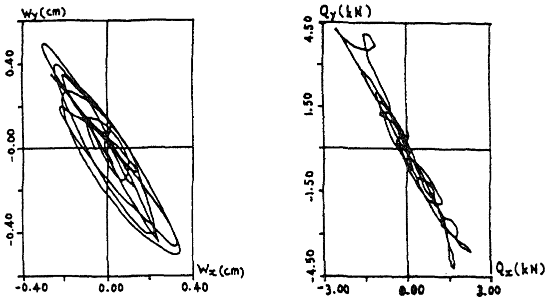

Figure 3. Measured response traces of FR2.

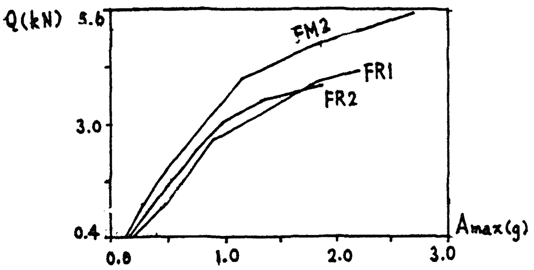

Figure 4. Relation between base shear force and peak base acceleration.

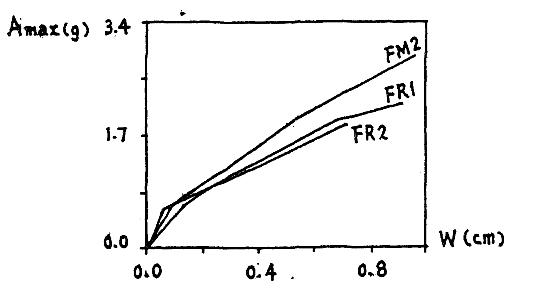

Figure 5. Relation between top displacement and peak base acceleration.

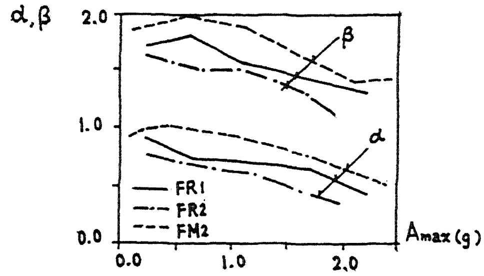

Figure 6. Variation of dynamic magnification factor with peak base acceleration.

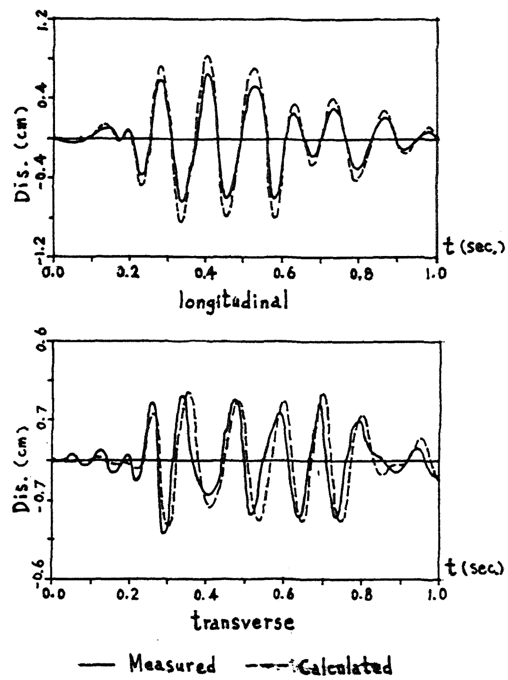

Figure 7. Top displacement history of FR1.

figures the bi-axial response quantities measured on test models FR1 and FR2 are compared with maximum response quantities measured in the previous equivalent uniaxial test of model FM2. Maximum response quantities are in the longitudinal direction for FR1 and FM2, and in the diagonal direction for FR2 respectively. It is shown that maximum base shear forces in the bi-axial tests (FR1 and FR2) are obviously smaller as compared with the uniaxial test (FM2), but the maximum top displacement of FR1 and FR2 are larger than that of FM2. The major difference between maximum response quantities of bi-axial and uniaxial tests is the increased degradation of stiffness resulting from bi-axial damage. Though the initial stiffness was indentical for those frames, greater stiffness degradation was apparent in the bi-axial test of FR1 and FR2. This degradation was evidenced by the increased first mode longitudinal vibration period. Moreover, the multi-axial load combination obviously decreases the yeild capacity of the frame below that available in uniaxial loading. Measured dynamic magnification factor β for top acceleration and base shear force coefficient α in different test stages are shown in Fig.6. It is shown that the dynamic magnification factor β and the coefficient α decreased as cracks and inelastic deformation of models appeared and developed.

The time-dependent curves of longitudinal

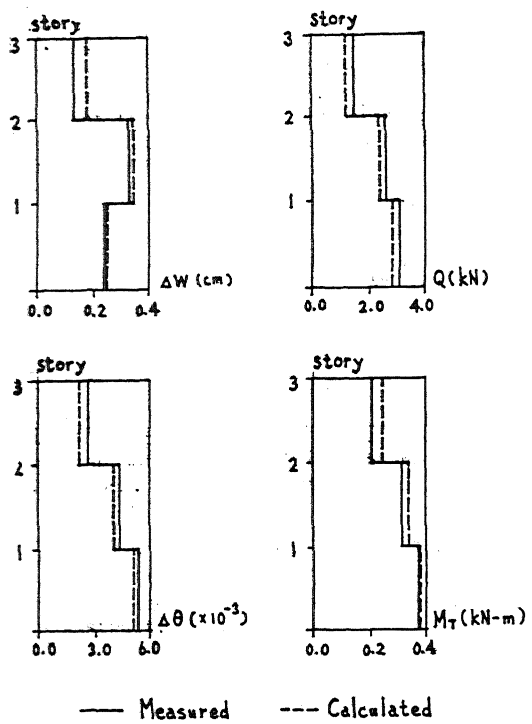

Figure 8. Measured and calculated maximum response quantities for FR1.

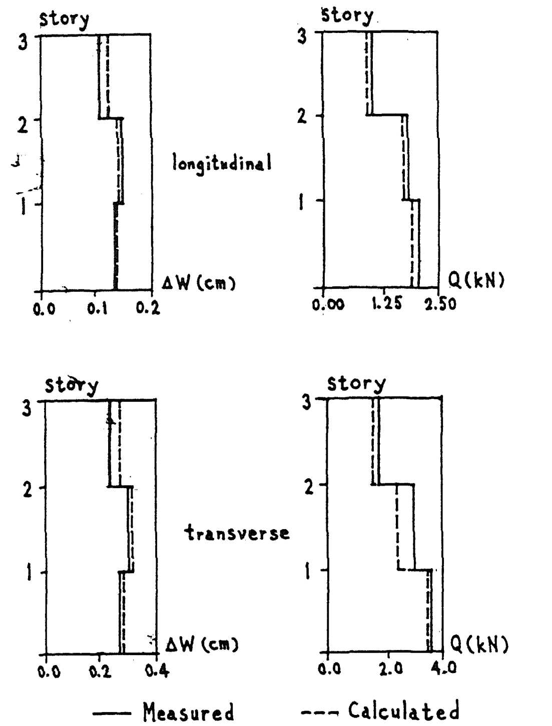

Figure 10. Measured and calculated maximum response quantities for FR2.

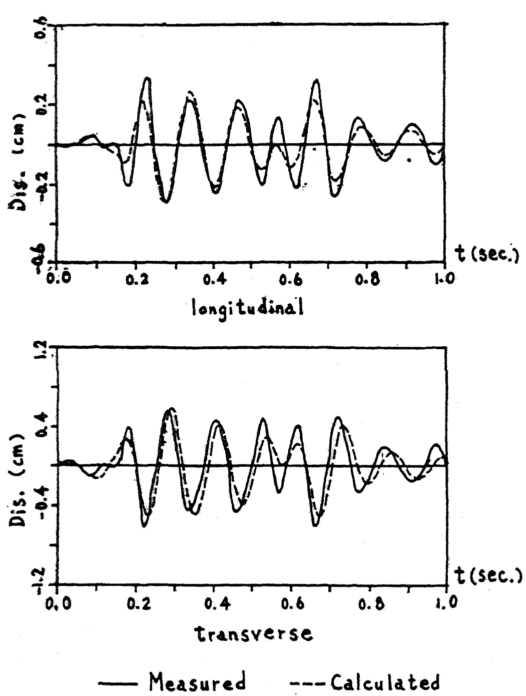

Figure 9. Top displacement history of FR2.

and transverse displacement on the third floor of FR1 during simulation with peak base acceleration 1.89 g are shown in Fig.7. Measured and calculated maximum response quantities of FR1 during the same simulation also are shown in Fig.8. In those figures the solid line denotes measured results and the dash line denotes calculated results by the bi-axial inelastic response analysis. Fig.9 shows the time-dependent curves of longitudinal and transverse displacement on the third floor of FR2 during simulation with peak base acceleration 1.87 g. Measured and calculated maximum response quantities in the longitudinal and transvers directions for FR2 during the same simulation also are shown in Fig.10. From those figures it is indicated that the experimental and analytical results in the bi-axial inelastic earthquake response are in good agreement.

Most of the inelastic deformation and damage to the structures FR1 and FR2 occured at the extremities of the first story columns, especially at the corner columns. Greater visible column damage was apparent in the present bi-axial frames than had be seen in the earlier uniaxial test. However, there was virtually no visible beam damage in the present frames, whereas cracking was evident through the full depth of the beams in the previous uniaxial test(FM2). It is shown that

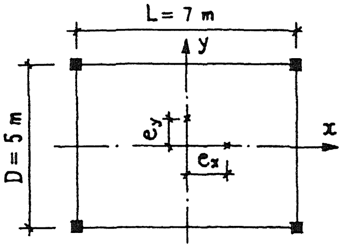

Figure 11. Plan of single story frame

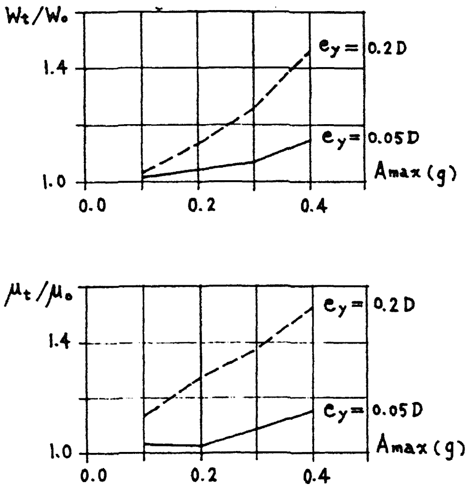

Figure 12. Influence of eccentricity on top displacement and ductility demand of columns.

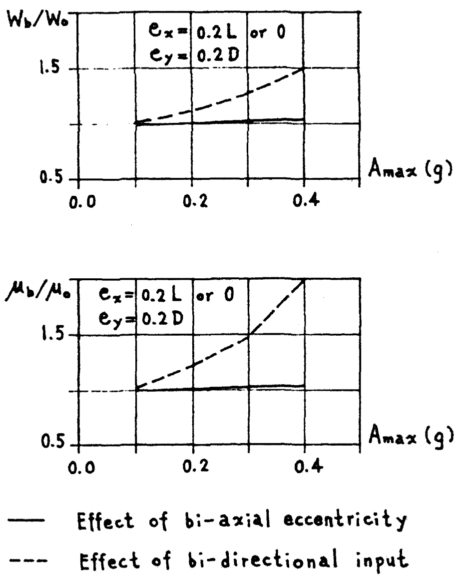

Figure 13. Effect of bi-axial eccentricity and bi-directional ground motion.

even if the frame building is designed by the strong column-weak beam system, in the case of bi-axial inelaste response the columns still may become the weak part of a frame, especially in the corner columns.

5 DISCUSSION

In order to investigate the earthquake response of the structure with eccentricity the inelastic analysis of a single story frame with one span is carried out as shown in Fig.11. The story height of frame is 3.5m. The first 8 second acceleration records of EL-Centro 1940 NS and EW direction ground motions are used in calculation. The relationship between relative maximum displacements and peak base accelerations is shown in Fig.12 in which w_t and w_o are the maximum displacements for the frame with and without eccentricity respectively. Figure 12 also shows the relationship between relative ductility factors of column and peak base accelerations. In this figure μ_t and μ_o are the ductility factors of columns for the frame with and without eccentricity respectively. It is shown that the relative maximum displacement and ductility factor of column greatly increase with increasing peak base acceleration. The influence of torsionally coupled vibration on the displacement response and ductility factor of columns increases with increasing relative eccentricity of mass.

Figure 13 shows the effect of bi-axial eccentricity and bi-directional ground motion on the maximum displacement response and ductility demands of columns. The solid line represents the relation of maximum response quantities between the structures with bi-axial and uni-axial eccentricities. The dash line represents the relation of maximum response quantities between the structures under bi-directional and single-directional ground motions. It is indicated that the displacement response and ductility demand of structure with bi-axial eccentricity are almost identical by comparison to the structure with uni-axial eccentricity. But the displacement response and ductility demand of structure under bi-directional ground motion obviously increase by comparison to the structure under single-directional ground motion with increasing peak acceleration.

Figure 14 shows the maximum story drift under single-directional and bi-directional ground motions for six stories reinforced concrete frame building with one span and one bay. The story height, span and bay of the frame are 3.3 m,6.0 m and 6.0 m respectively. The reinforced concrete frame was designed as strong beam-weak column system. The first 8 second acceleration records of El-Centro 1940 NS and EW directions with peak acceleration 0.348g and 0.182g are used in calculation. It

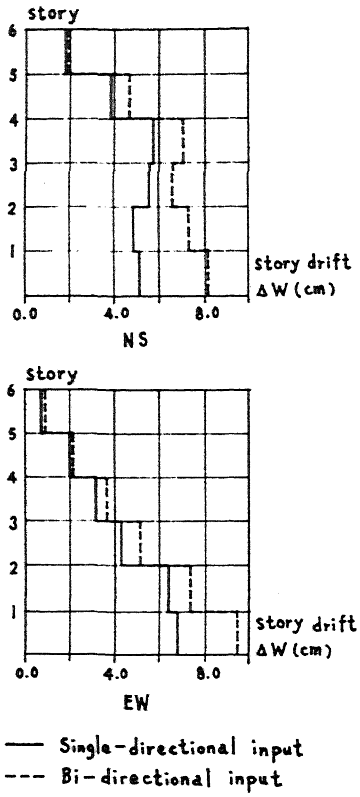

Figure 14. Bi-directional effect of seismic action on strong beam-weak column frame.

is shown that the story drift of structure under bi-directional ground motion obviously greater than that of structure under single-directional ground motion. Maximum drift of the first story in the NS and EW directions under bi-directional ground motion is 57% and 39% larger than that of the structure under single-direction ground motion respectively. Though current building design codes allow the response along each of a structure's principal axes to be computed independently, the results will be in error if the columns of structure yield. The error is caused by changes in the overall mode of response in addition to increased deformation induced by earlier yielding and reduced stiffness. It is of great importance to consider bi-directional effect of seismic action on structural behavior. However, the actual behavior of structures during earthquakes is very complicated and seismic forces in the transverse direction may have significant effects on seismic performance of structures.

6 CONCLUSION

Yielding of columns with bi-axial moment creates greater stiffness degradation than that of uniaxial tests. The simultaneous multi-axial loading can significantly change the nature of earthquake response of a frame. The mutual effect of bi-axial bending of column after yielding significantly influences on the inelastic earthquake response in the cases of mass eccentricity and bi-directional ground motion for symmetric reinforced concrete frame structures. The deformation demands generated by the earthquake response become concentrated within the columns of the frame due to early yielding induced by bi-axial loading. The deformation response demands induced by an earthquake are increased when members must resist bi-axial loads. The loads induced in the corner columns of the frame by bi-axial flexure and varying axial load casued by overturning moment are never identical. One individual column would reach yield before the others, and the overall lateral stiffness of the frame system would become unsymmetric. The unsymmetrical stiffness invitably led to a torsion in the frame and increased bi-axial flexure in the corner columns.

In this paper the multi-dimensional restoring force models of members with consideration of mutual effect of bi-axial stiffness after yielding for inelastic bi-axial response analysis is suggested. The rationality of the restoring force models is verified by the test results. The earthquake response of two test models are calculated and compared with the measured results. It is shown that the experimental and analytical results in the inelastic stage are in good agreement.

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