NONLINEAR EARTHQUAKE RESPONSE TEST OF TORSIONALLY COUPLED REINFORCED CONCRETE BUILDING FRAMES

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SUMMARY

The horizontal and torsional motions are coupled in the response of building subjected to the earthquake ground motion if the center of story resistance does not coincide with the center of floor mass. In the recent strong earthquake in Japan, some reinforced concrete buildings have severely damaged due to the eccentricity between center of mass and that of resistance. The objective of this paper is carry out the earthquake response test of one story reinforced concrete building frames with eccentricities subjected to the one-component strong earthquake ground motion.

INTRODUCTION

In 1968 Tokachi-Offshore Earthquake(Ref.1) and in 1978 Miyagiken-Offshore Earthquake(Ref.2), some reinforced concrete buildings which had the eccentricities between the center of story resistance and that of floor mass. In the Japanese Seismic Standard Code in 1981, the provisions concerning the eccentricities were newly incorporated according to the empirical damage of earthquake and the results of many studies on the earthquake response analysis and test of torsionally coupled reinforced concrete buildings(Ref. 3-7), however, there is much left to study. In this paper, the inelastic behavior of torsionally coupled reinforced concrete building frames are investigated by the earthquake response test, so called Computer-Actuator On-line test, and cyclic loading test.

OUTLINE OF TEST

Test Specimen

The specimens are one-bay one-story and 1/4 scaled reinforced concrete frames with eccentricities which consist of a rigid deck supported on four columns and/or one shear wall. Four frames were tested (Table 1), namely, one frame with non-eccentricity(frame ID; EFU-01), two frames with eccentricity in the X-direction (EFU-11,EWU-11) and one frame with eccentricities in both X- and Y-direction (EFU-21), respectively.

According to the Japanese Seismic Code, the eccentricities of buildings were defined as follows;

$$R_{ex} = e_y \,/\, {}_rE_x \quad , \quad R_{ey} = e_x \,/\, {}_rE_y \qquad \text{.................(1)}$$

where; e_x, e_y = distance between the center of story resistance and that of floor mass in X- and Y-direction, respectively.

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r^E_x , r^E_y = radius of gyration of the roof deck

In the Japanese Seismic Code, the upper limited eccenctricity for not giving extra increasing of design strength is 0.15. The eccentricities of specimens became Rex=0.54, Rey=0.0 for EFU-11, Rex=3.34, Rey=0.0 for EWU-11 and Rex=Rey=0.18 for EFU-21, respectively. The relationships between the eccentricities of specimens and the Japanese Code ones are shown in Fig.1. And furthermore, the ones of which the buildings were severly damaged in 1978 Miyagiken-Offshore Earthquake (Ref.8) are also shown in this figure.

Outline of specimen is shown in fig.2 and the detail is shown in Fig.3. The size of roof deck is 150x150 cm, floor deck's one is 150x200 cm and thickness of both are 20 cm. The size of column is 10x10 cm and 15x15 cm, respectively. The thickness of shear wall is 4 cm and the length is 100 cm. Clear height of columns and wall are 90 cm. Shear span ratio (a/D) of columns are 3.0 and 4.5. The reinforcing steel bars were welded to the end plates to avoid the effect of bond deterioration between concrete and reinforcing bars at the end of members. The concrete is normal weight one and in mixing, small aggregates (ø=10 and 3 mm) were used. The compressive average strength was about 240 kg/cm^2. The specifications of used reinforcing bars were D10(SD10) D6(SD30),4ø(SR24) and 2.6ø(SR24), and the tensile yield strength were about 3600,3900,3000,3000 kg/cm^2 , respectively. According to the AIJ Building Code (Ref.9), the columns were designed so that the shear failure did not occur prior to the flexural yielding, on the other hand, the shear wall was designed so that the shear failure occured at first.

On-line Test and Cyclic Loading Test

Response test was carried out by the Computer-Actuator On-line System developed at the Institute of Industrial Science, University of Tokyo (Ref. 10). A similar system was recently developed for the U.S.- Japan joint aseismic research program at Building Research Institute, Ministry of Construction, Japanese Government. The parameters for the response test are shown in Table 2. The uncoupled initial periods of all frames in ground motion input direction were assumed to be 0.2 sec. The amplitude of the ground motion acceleration was modified so that the ratio of lateral story strength of frames to the peak acceleration value (Ky/Kg) became about 1.0- 1.2. For the earthquake ground motion, the NS component of 1968 Hachinohe acceleration record was used, and it was applied to the X-direction of specimens. The central finite difference method was used for the numerical integration, and the time interval is 0.01 sec. Though the duration time was 12 sec, it took about five hours to finish the test because of the observation of cracks ,etc.

After the On-line test, the cyclic loading test was carried out in order to investigate the frame behavior in the inelastic large deflection stage. The one-cycled displacement from 3 cm to 5 cm was applied to the center of roof deck until the buckling of reinforcing bars will take place.

Loading System

Test setup is shown in Fig.4. The specimen consists of roof deck, columns, shear wall and floor deck. They were connected rigidly with one another by the high tension bolts, and the floor deck was fixed to the testing floor by PC bolts(30ø). Response displacement was applied to the center of roof deck through steel beam by electro-hydraulic actuator. The capacity is $\pm$30 tons for load and $\pm$15 cm for displacement. At the center of the roof deck, bearing was installed so that the gyration of the deck could not be restricted. Axial stress of columns and shear wall were not applied except the dead load of members and that of roof deck. Two digital input type transducers (Accuracy; $\pm$ 0.01mm) of displacement were installed at both the other side of actuator and the perpendicular side of actuator connected to the center of roof deck by the flexial amber steel wires. The former transducer was used to control the movement of the actuator. For other measurements, namely, torsional angle of roof deck and vertical displacement of columns and wall, inductance type transducers (Accuracy; $\pm$0.5%/FS) were used.

RESULTS OF TEST

Maximum Response Displacement

Response results are shown in Table 2. In this Table, maximum response displacement of the center of story mass, maximum response shear force and ratio of k_y/k_g obtained by the test were presented. It was recognized that the larger the eccentricities of frames, the smaller the ultimate coupled shear story strength in test than those uncoupled before test, especially, this tendency was remarkable in the frame with shear wall (EWU-11). The peak-to-peak average periods varying with time became about 2 seconds in EWU-11 which was ten times the uncoupled one before test, and about 0.5 second in other frames which were about three times the uncoupled ones. By these results, it was clear that the reduction of torsional coupled story shear strength and stiffness became larger according to the increasing of the eccentricities of frames. The maximum response displacement of frame became 1.7 - 2.3 cm (Rotational angle;R=1/56 - 1/39) under the k_y/k_g from 0.5 to 1.3.

Restoring Force Characteristics

Fig. 5 shows the relationships between shear force and story displacement the solid line is by on-line test and the dotted line is by cyclic loading test. Both hysteretic loops presented the energy absorption shapes in large deflection stage as well as in small deflection stage. And furthermore, the frame EWU-11 which had a shear wall didnot take place the shear failure of wall. As mentioned above, the torsionally coupled story shear strength became smaller than that of uncoupled frames with increasing the eccentricity, especially, in frame EWU-11 (Fig. 5), the coupled strength is about a half of the uncouped one. It was also observed that the larger the eccentricity, the smaller the limited displacement for collapse defined as the buckling of reinforcing bars.

Displacement of Members and Frames

Fig. 6 shows the examples of transition of frames according to the various displacement stage. In this figure, both on-line test results and cyclic loading test results are shown together. In order to make clear the state of gyration, torsional angle (Θ) was drawn by ten times larger than the observed one. The shapes of each frame presented the first vibrational mode. The positions of the gyration centers were different with one another, namely, the larger the eccentricity, the smaller the distance between the center of gyration and the center of mass. Fig. 7 shows the relationships between torsional angle (θ) of the roof deck about a vertical axis through the center of mass and the lateral displacement of the center of mass(X_1). According to increasing of the displacement, the torsional angle became larger, and this tendency increased with increasing the eccentricity. Fig. 8 shows the relationshpes between displacement members and that of center of mass. In this figure,it it was apparent that because of torsional coupling, column displacements could be considerably amplified, namely, in frame EWU-11, the displacement of corner columns became about two times that of center of mass, on the contrary, the displacement of wall became a half. This was resulted that the tosional angle increased as the eccentricity increased.

Crack Pattern of Columns and Shear Wall

The crack pattern of the tosionally uncoupled frame (EFU-01) was dominated by the flexural failure at the top and bottom of columns. This frame had the by the torsionally coupled effect, and furthermore, at the mid height of columns the inclined shear cracks caused by torsion took place. In frame EWU-11, the limited displacement of collapse was about 3 cm (R=1/30), which was smaller than torsionally uncoupled frame (EFU-01).

CONCLUDING REMARKS

The principal conclusions of this paper concerning the torsionally coupled single -story reinforced concrete frames subjected to one-component earthquake ground motion and cyclic loading are as follows;

(1) This testing system appears to be useful for the nonlinear earthquake response analysis of torsionally coupled reinforced concrete building frames, while the revision is necessary to be applicable to the multidegree system.

(2) The displacement of corner columns can be considerably amplified with increasing the eccentricities of frames, and can be damaged severly comparing to the torsionally uncoupled frames.

(3) The stiffness and ultimate story strength of torsionally coupled frames become smaller than the uncoupled one which are calculated before test.

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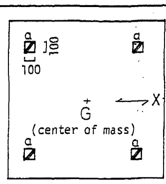
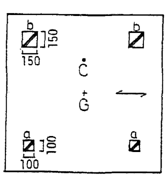
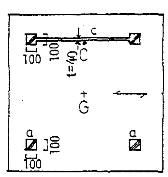
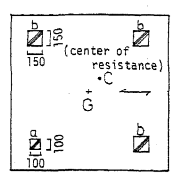
Table 1 Test specimens

FRAME ID	EFU-01	EFU-11	EWU-11	EFU-21
LOCATION OF WALL & COLUMN				
FRAME TYPE	FOUR COLUMNS		THREE COLUMNS & ONE SHEAR WALL	FOUR COLUMNS
LOADING DIRECTION	X-DIRECTION			
TORTIONAL COUPLE	UNCOUPLED	COUPLED TO X-DIRECTION		COUPLED TO X-,Y-DIRECTION

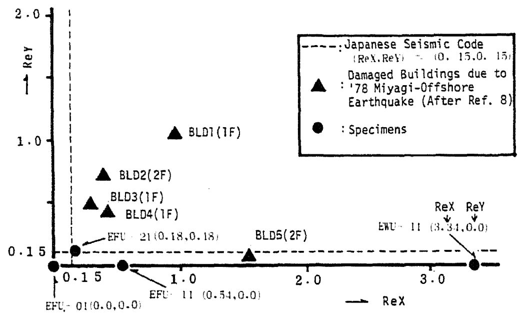


Fig.1 Comparison of eccentricities

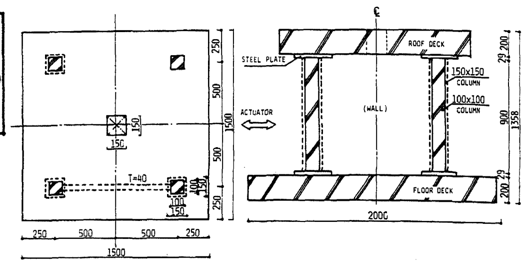
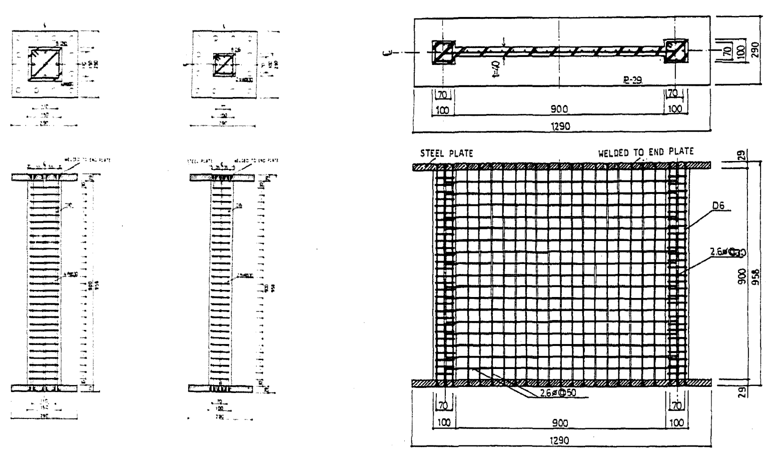


Fig.2 Outline of specimen



(a) Column (15x15cm) (b) Column (10x10cm) (c) Shear wall

Fig.3 Detail of specimen

Table 2 Test variables and results of response test

Frame ID	Stiffness Kx (Ky) (t/cm)	Assumed Valiables: Period (sec) Tx *1 (Ty)	Period (sec) T_1 T_2 T_3 *2	Mass (t·cm·sec²) M	Ultimate Strength Qx *3 (Qy) (t)	ky/kg *3	Results of Response Test: Max. Load Q (t)	ky/kg	Max. response displacement X_1 (cm)	Rotational angle (R)
EFU-01	11.5 (11.5)	0.2 (0.2)	0.2 — —	0.01167	4.2 (4.2)	1.21	4.37	1.26	2.33	1/39
EFU-11	34.9 (34.9)	0.2 (0.2)	0.26 0.15 —	0.03538	8.5 (8.5)	1.22	7.40	1.05	2.04	1/44
EWU-11	405.8 (11.5)	0.2 (1.19)	1.00 0.16 —	0.4111	13.8 (4.2)	0.95	6.63	0.46	1.68	1/54
EFU-21	46.6 (46.6)	0.2 (0.2)	0.22 0.20 0.16	0.04724	10.1 (10.1)	1.20	10.52	1.25	1.62	1/56

*1 Torsionaly uncoupled period *3 Torsional uncoupled ultimate strength
*2 Torsionally coupled period

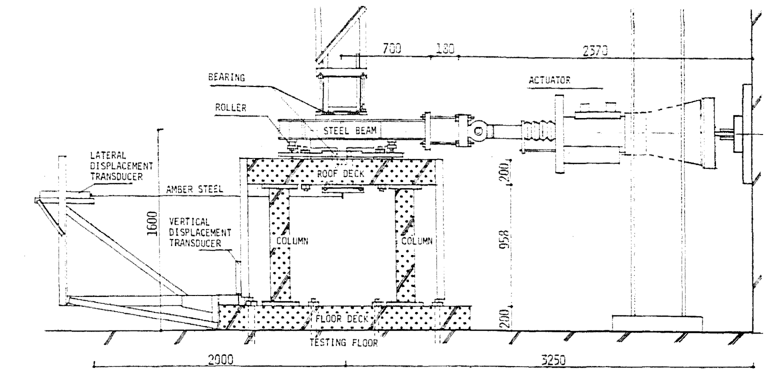


Fig. 4 Test setup

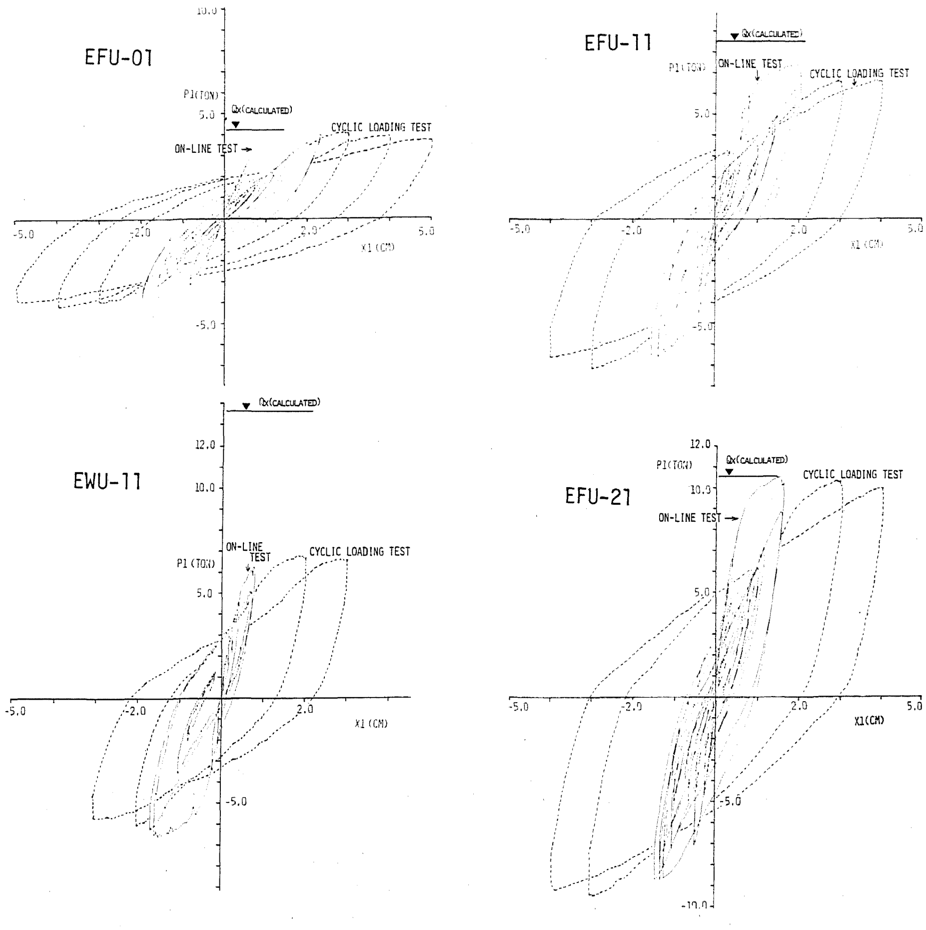


Fig. 5 Restoring characteristics

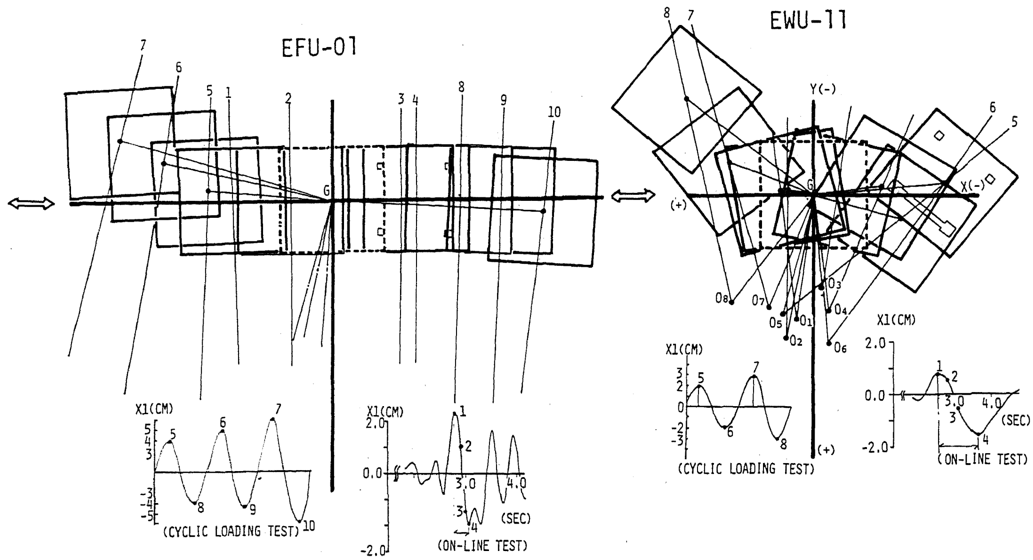


Fig. 6 Transition according to the displacement

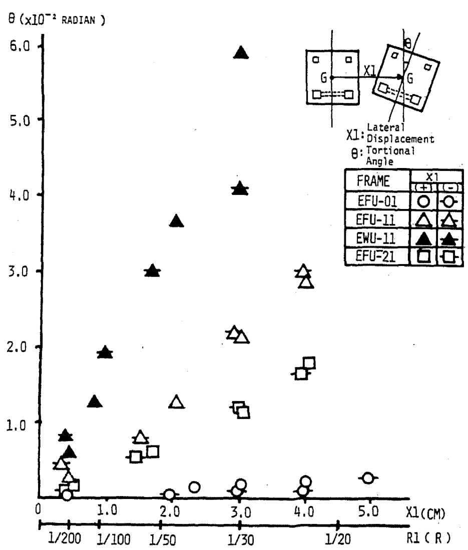


Fig. 7 Torsional angle vs displacement of frame

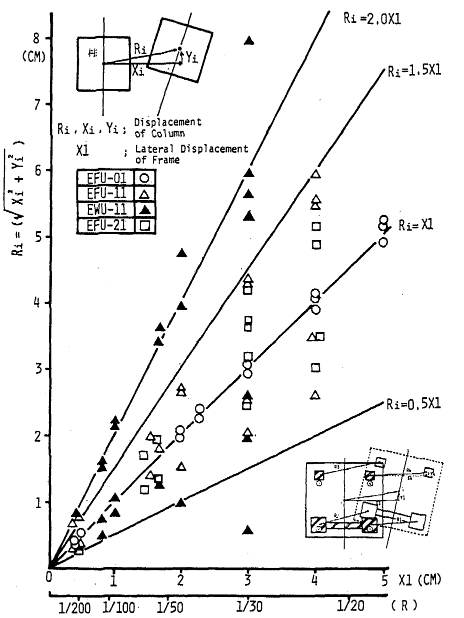


Fig. 8 Displacement of member vs displacement of frame