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Dynamic and static collapse tests of reinforced-concrete columns

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Abstract

The reinforced-concrete column damage pattern differences between static cyclic loading tests and dynamic shaking-table tests were observed. In this study, the spacings of shear reinforcement bars in columns were variable. Four types of shear reinforcement bar spacing are used. As a result of the tests, it was verified that the maximum horizontal strengths of columns, the deformation capacities and the damage patterns were changed with the shear reinforcement bar spacings, and also with testing method differences between statics and dynamics. Furthermore, $P-\delta$ effects were discussed.

1. Introduction

Many studies concerning the ultimate horizontal strengths of columns have already been conducted. However, most of these studies were carried out by the static cyclic loading of one column, or pseudodynamic loadings. In this investigation, realistic dynamic loadings which would simulate earthquake responses of structures, were done by the use of a shaking table, and the dynamic damage characteristics of columns were observed. A small test structure of one storey and one bay had four columns. These columns were made of reinforced concrete. Four kinds of spacing of shear reinforcement bars were used. In addition to the shaking-table test, a static cyclic loading test was made for comparison. The time variations in the damage patterns, the ultimate strength and the storey deformation were observed.

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2. Test methods

Four kinds of reinforced-concrete (RC) column were tested on the shaking table. The shaking-table test set-up is shown in Fig. 1. In these test

Fig. 1. Test set-up.

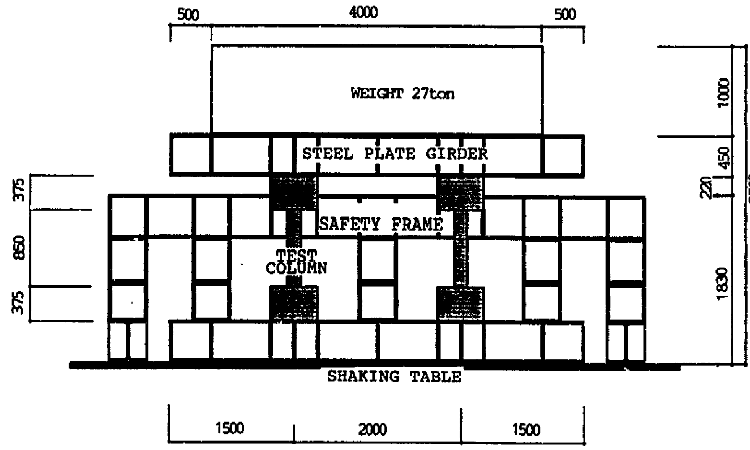

Fig. 2. Test structure.

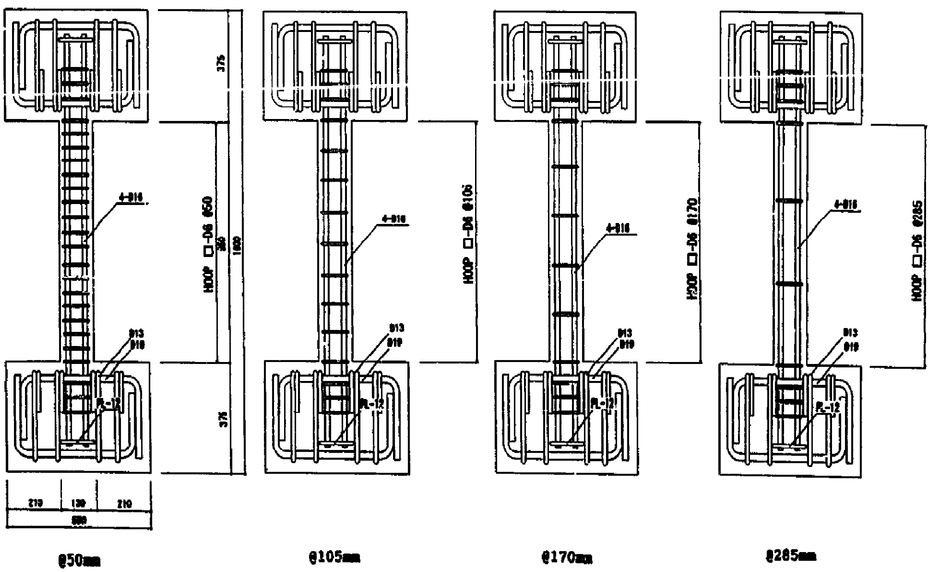

Fig. 3. Reinforcement steel bar distributions.

series, the shaking table was driven by the strong earthquake motion of 1968 Tokachioki Hachinohe E-W, which was made in the use of the fast Fourier transform bandpass filter, 0.4–50 Hz. The real time scale was adopted.

The one-storey-by-one-bay test structure was assembled on the shaking table. Four columns had a section of 13 cm by 13 cm, and a height of 85 cm. The diameters of the four main steel bars were 16 mm. The columns were fixed in a steel roof girder and steel base frames with bolts. The roof of the test structure was made of a steel frame. On the roof, three concrete masses were fixed. The total mass of the roof was 27.8 ton.

The spans of this test structure were 2 m in the loading direction, and 2 m in the perpendicular direction. In order to mitigate the shock load of the falling roof on the shaking table during damage excitation, the roof support frame was installed inside the test structure. Therefore the roof of mass 27.8 ton fell about 15 cm in damage excitation. Four types of column with shear reinforcement bar spacings of 50, 105, 170 and 285 mm were tested. The outline of the test structure is shown in Fig. 2. The details of columns are shown in Fig. 3, and the physical properties of columns are a concrete compressive strength of 230–300 kg cm^{-2}, and a steel bar tensile strength of 3600 kg cm^{-2}.

In a static cyclic loading test, pull-type oil jacks which were connected in the roof frame were used. Two jacks were set on the right-hand side; the others were set on the left-hand side. The cyclic loading levels were 1/1000, 1/200, 1/100, 1/50 and 1/20 in storey deformation angles. After the cyclic test, the load increased gradually until the column collapsed. The one-dimensional shaking table at the National Research Institute for Earth, Science and Disaster Prevention was used in this test. The dimensions of the shaking table are 12 m by 12 m. The table mass is 180 ton. The maximum power is 360 tonf. The maximum velocity and displacement are 75 cm s^{-1} and 24 cm respectively. The acceleration levels of these tests were 0.5G and the velocity was 70 cm s^{-1} approximately. The collapse responses of the test columns were observed by video records and measured with various sensors.

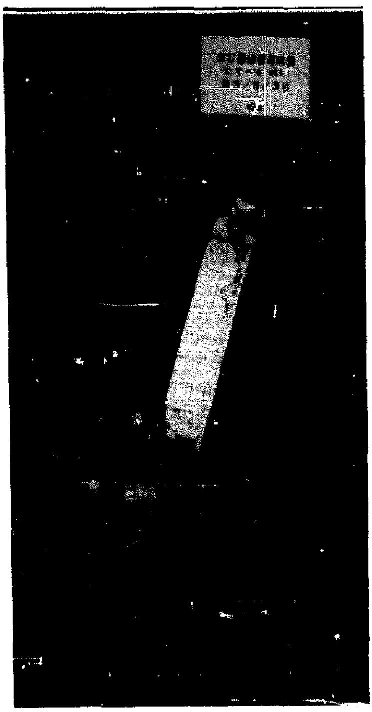

Fig. 4. 50 mm static loading test damage.

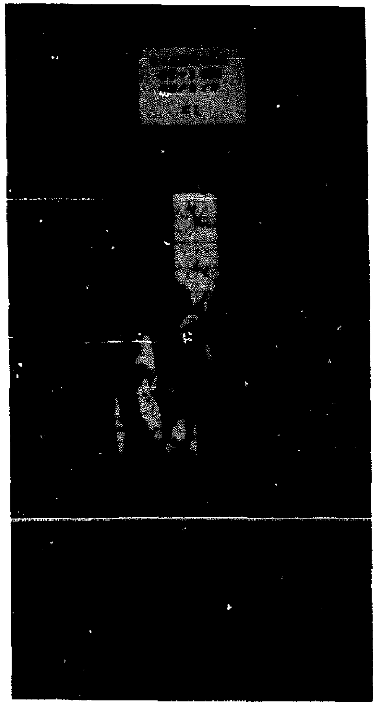

Fig. 5. 285 mm static loading test damage.

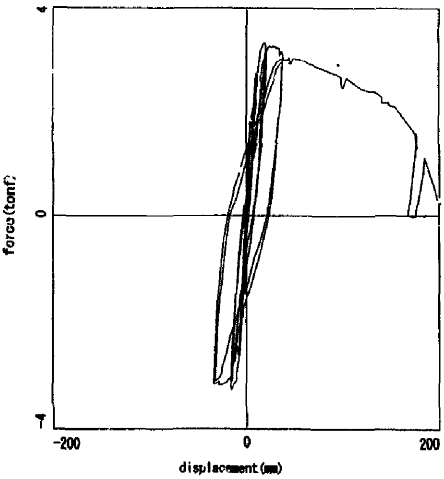

Fig. 6. 50 mm load–storey displacement loop.

3. Static test results

Two kinds of column were tested. In the static cyclic load test with columns of reinforcement bar spacing 285 mm, the shear cracks appeared in the region between about 10 and 40 cm from the two column ends on cyclic loading with a deformation angle of 1/100. With a cyclic loading of 1/50, the shear cracks increased diagonally, and the main steel bars of the two columns in X-1 frame buckled at the midheight, with separation of concrete and main steel bars; the roof fell down at last. In the static cyclic loading test with columns of reinforcement bar spacing 50 mm, the bending cracks concentrated in the region from the two column ends up to 10 or 20 cm positions with cyclic loadings of 1/50 and 1/25. With a cyclic loading of 1/5, the compressive crushes of two column ends began immediately. Finally, the concrete of the two column ends was crushed completely, and the main steel bars buckled. The crack sketches of these loadings are shown in Figs. 4 and 5. The load–displacement hysteresis loops of these two static cyclic loading tests are shown in Figs. 6 and 7.

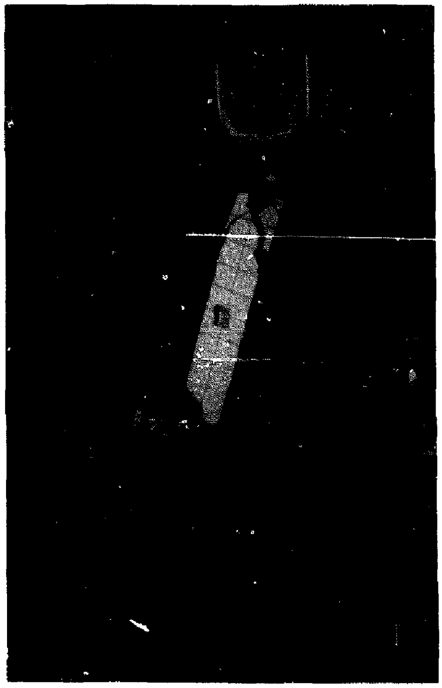

Fig. 8. 50 mm dynamic test damage.

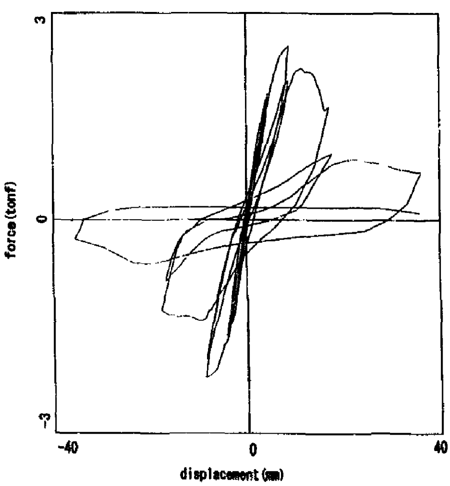

Fig. 7. 285 mm load–storey displacement loop.

4. Dynamic test results

Four kinds of column were tested on the shaking table. The natural frequency of the elastic regions of each test structure was 4.5 Hz approximately. The damping ratios in the elastic regions were estimated to be about 5% by AR model analysis. No distinct differences between the four test structures were found. The plastic region dominant frequencies of each test structure were about 3 Hz at the time just before collapse. The four test structures collapsed at the beginning of the main shocks. The damaged columns are shown in Figs. 8–11. Acceleration–storey displacement hysteresis loops of the four shaking table tests are shown in Figs. 12–15.

In the shaking-table test with columns with a reinforcement bar spacing of 285 mm, initial shear cracks grew immediately, and three columns were cut near the column tops, and the other was cut near a column bottom. The main steel bars buckled.

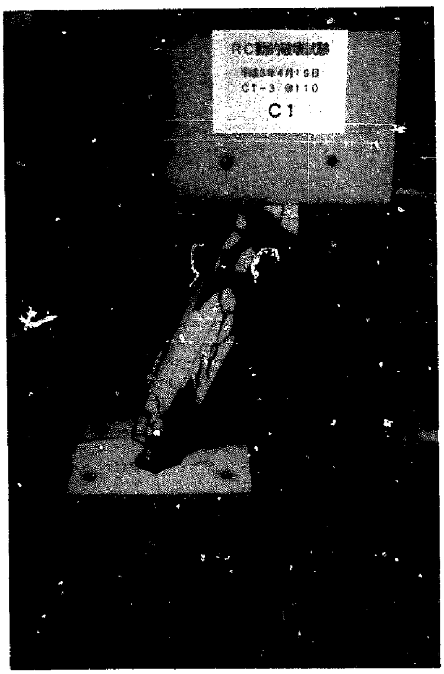

Fig. 9. 105 mm dynamic test damage.

Fig. 10. 170 mm dynamic test damage.

In the test with a reinforcement bar spacing of 170 mm, the initial crack types were the same as in the 285 mm test case. However, the columns collapsed during the next opposite acceleration with the separation of concrete and steel bars, and with the crushing of core concrete.

In the test with a reinforcement bar spacing of 105 mm, shear bending cracks appeared near two column ends. These cracks increased and collapsed with the separation of concrete, and with buckling of the main steel bars.

In the test with a reinforcement bar spacing of 50 mm, initial shear bending cracks appeared in two column ends. These cracks grew gradually, and the compressive concrete crushed with main steel bar buckling. The crush region of columns was limited in the column ends.

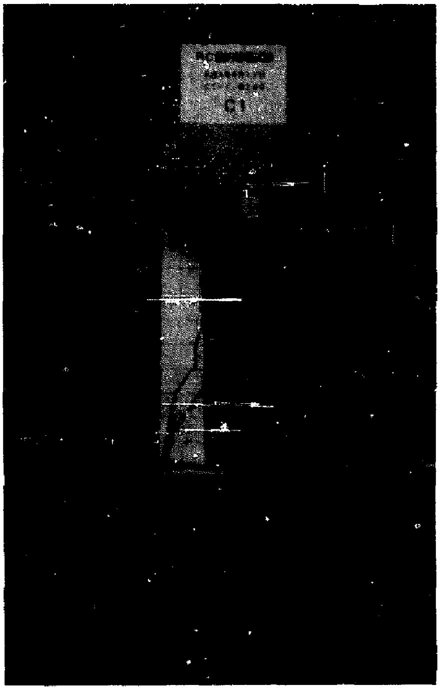

Fig. 11. 285 mm dynamic test damage.

5. P–δ effects

The measured acceleration–storey displacement hysteresis loops in Fig. 12 show the effects of the roof mass, i.e. P–δ effects. A simple model with rigid girders, shown in Fig. 16, is considered. The equations for this model (Minowa, 1985) are expressed as follows:

$$F_u \cos\theta + F_v \sin\theta = Q$$

$$F_u \sin\theta + F_v \cos\theta = N$$

where

$$\cos\theta = \frac{h+v}{[(h+v)^2+u^2]^{1/2}} \quad \sin\theta = \frac{u}{[(h+v)^2+u^2]^{1/2}}$$

$$Q = 2[k_b(u\cos\theta - v\sin\theta) + c_b(\dot{u}\cos\theta - \dot{v}\sin\theta)]$$

$$N = 2(k_a\{[(h+v)^2+u^2]^{1/2} - h\} + c_a(\dot{v}\cos\theta + \dot{u}\sin\theta))$$

$$F_u = -m(\ddot{u} + \ddot{u}_g)$$

$$F_v = -m(\ddot{v} + \ddot{v}_g + g)$$

Using these expressions, the following equations are obtained:

$$m(\ddot{u}+\ddot{u}_g) - m(\ddot{v}+\ddot{v}_g+g)\frac{u}{h+v} + f(u) = 0$$

$$m(\ddot{u}+\ddot{u}_g)\frac{u}{h+v} + m(\ddot{v}+\ddot{v}_g+g) + k_a\{[(h+v)^2+u^2]^{1/2} - h\}\frac{[(h+v)^2+u^2]^{1/2}}{h+v} = 0$$

By this P–δ analysis, the RC column shaking-table test data were discussed. Judging from the test data in Fig. 5, the restoring force characteristic $F(u)$ of RC columns was assumed to be as in

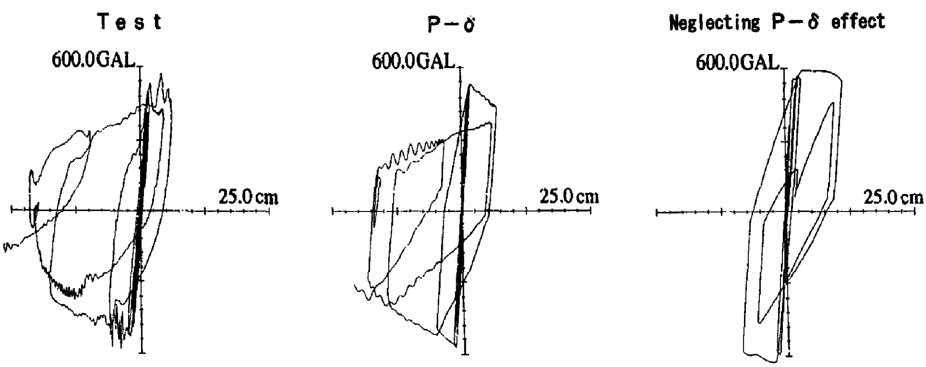

Fig. 12. 50 mm acceleration–storey displacement loop.

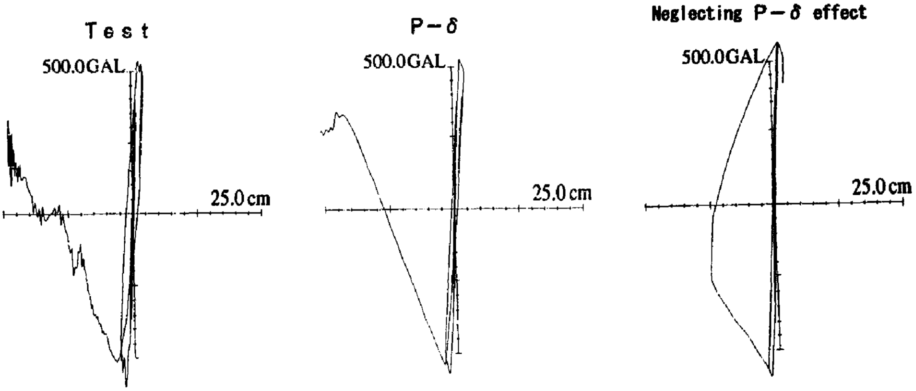

Fig. 13. 105 mm acceleration–storey displacement loop.

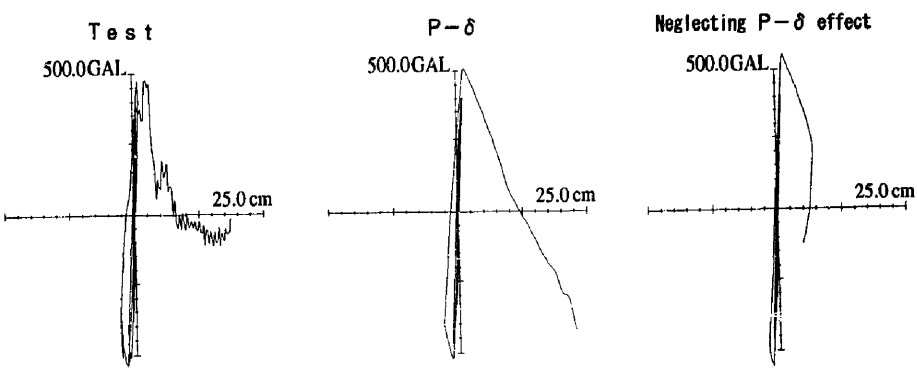

Fig. 14. 170 mm acceleration–storey displacement loop.

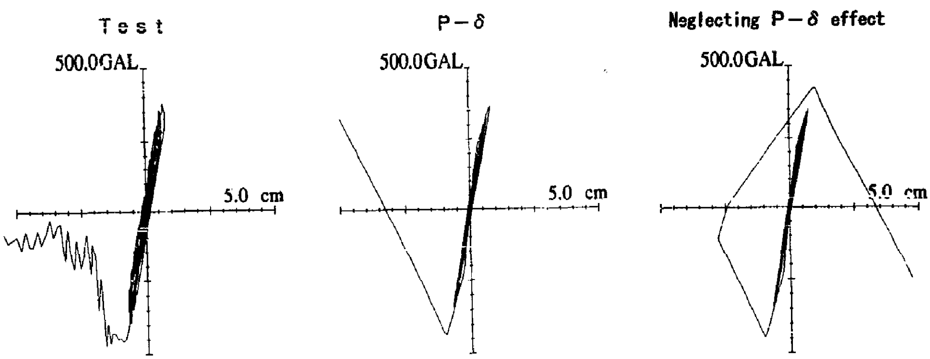

Fig. 15. 285 mm acceleration–storey displacement loop.

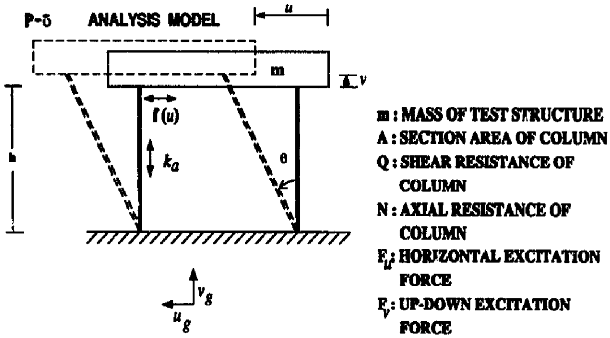

Fig. 16. P–δ analysis model.

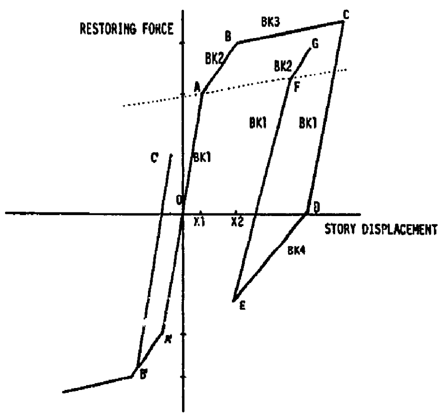

Fig. 17. RC restoring force model.

Fig. 17. In the cases of these test structures, the member end rotations were considered to be negligibly small by the effects of high rigidity girders.

In order to search for the optimal parameter combinations, the quadratic errors between measured time histories and computer simulation time histories were calculated for the various parameter values of damping ζ_u, K_2, K_3, e_2 and Q_p in Fig. 10. The column height h of 75 cm, inertia mass m of 0.027 ton s^2 cm^{-1}, the axial rigidity K_a of 750 ton cm^{-1}, and damping ζ_v of 1% were used for these simulations. Figs. 5–8 show the acceleration and deformation Lissajous figures which gave good agreement between shaking-table tests and computer simulations. Table 1 indicates the parameter values which were used for these computer simulations.

Table 1
Calculation parameters

Hoop spacing (mm)	K_2 (tonf cm^{-1})	e_2 (cm)	K_3 (tonf cm^{-1})	Q_p (ton)	ζ_u (%)
50	7	1.65	0.1	−2	1.85
110	7	1.65	−0.7	−2	1.80
170	6	1.5	−0.9	−3	4.0
285	8	1.0	−5.0	−5	1.5

6. Conclusion

The dynamic maximum horizontal strengths and the dynamic deformation capacities almost agreed with the static values. However, the damage patterns were changed with the shear reinforcement bar spacing, and also with the test method differences between statics and dynamics.

P–δ effects are important for the collapse properties of columns.

Acknowledgment

This experiment was started at the suggestion of Professor Heki Shibata, University of Tokyo. The authors wish to express their appreciation to him.

Reference

C. Minowa, One storey frame collapse tests by using the two dimensional shaking table (horizontal and up-down), NRCDP Rep. 34, 1985.