U.S. - JAPAN COOPERATIVE RESEARCH ON R/C FULL SCALE BUILDING TEST
Part 2 Damage Aspects and Response Properties Before Repair Works

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SUMMARY

Earthquake response of the full scale R/C seven story structure was examined by employing the single-degree-of-freedom pseudo-dynamic (SPD) test method. This paper presents the results of test conducted before repair works. The test consisted of four stages, from small input magnitude to large one. The results include the displacement histories, base shear force vs. roof floor displacement curves and observed damages. The structure showed ductile hysteresis behavior, and damage was not concentrated on any particular story.

INTRODUCTION

There have been numerous number of tests conducted around the world for the purpose of examining the earthquake behavior of building structures. Most of those tests were either tests of structural members and subassemblages or tests of small scale structural models, and their loading was quasi-static. Based upon those findings, various procedures to evaluate the seismic safety of building structures have been developed. However, the ultimate seismic capacity of building structures is influenced by many factors such as three dimentional effect, scale effect, and construction accuracy, and, as a result, the response of building structures under earthquakes is very complicated. In those conditions, it is considered that the earthquake response test of full scale structural models is needed to evaluate the ultimate seismic capacity of building structures.

Test on a full scale R/C seven story building structure was conducted as part of the U.S.-Japan Cooperative Research Program. The primary objectives of this test were 1) to observe the behavior of the total building system, 2) to estimate the true margin of safety of the structure and 3) to calibrate the accuracy of techniques for analyzing structures under static or dynamic loading. The test was also expected to provide fundamental data on the current seismic design. The test structure was designed according to the recommendations of the Joint Technical Coordinating Committee (JTCC) organized for this cooperative program. The test sequence was determined to satisfy the above objectives.

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TEST STRUCTURE AND LOADING SYSTEM

The test structure, a full scale R/C seven story building structure, consisted of three three-bay frames parallel to the loading direction, and four two-bay frames perpendicular to the loading direction. The central frame parallel to the loading direction had a shear wall in the central bay continuous from the first to the seventh story. The structure was clamped to the reaction floor, and external lateral force was applied by means of eight actuators as shown in Fig. 1. Two actuators, with capacity of ±100 ton load and ±1000 mm stroke, were set at the roof level, whereas an actuator, with capacity of ±100 ton load and ±500 mm stroke, was placed at the other floor levels. To apply force to the floor, a steel yoke was extended from the head of the actuator to the loading points of the floor. Details of the structure are described in reference (1). The SPD test, also described in reference (1) was employed to observe the behavior of the sturcture under the earthquake.

TEST SEQUENCE

Table 1 tabulates the sequence of the test program before repair works. The test program consists of four subprograms; they are 1) vibration test (VT), 2) each floor level loading (FLL) test, 3) static loading (SL) test, and 4) SPD test. Vibration test was carried out in two ways: a free vibration test and a forced vibration test. Each floor level loading test was conducted in order to obtain the flexibility matrix of the structure. Static loading test was conducted in order to obtain the fundamental data of the structure in the elastic stage.

Pseudo-Dynamic Test

In order to observe the response and damage propagation of the structure under various amplitudes of input ground motion, four tests, labelled SPD-1 through SPD-4, were executed by the SPD test method. Table 1 indicates the target maximum drift angle (the average from the foundation to roof), the maximum input ground acceleration, and the ground motion record designation of each test. The amplitude of the input ground motion was determined by preliminary analysis so that the maximum drift angle would be limited to 1/1000, 1/400, 3/400, and 1/75 for the test SPD-1 to SPD-4.

Fig. 2 shows the input ground motion in SPD-1 and SPD-2. The input ground motions in SPD-3 and SPD-4 are illustrated in Figs. 3 and 4 respectively.

RESULTS OF FLL AND SL TESTS

In the intact stage, the natural periods of the first, second, and third modes are 0.448, 0.125, and 0.0663 sec. from test FLL-1 and 0.468 sec., 0.123 sec., and 0.0602 sec. from the preliminary analysis. Agreement between the FLL test and analysis is satisfactory. The natural periods of test FLL-2 after test SPD-4 were measured 0.887, 0.171. and 0.0885 sec. for the first, second, and third modes respectively.

RESULTS OF SPD TEST

Table 2 tabulates the input motion, the maximum displacement at the roof level, the maximum base shear, and the period of each SPD test.

SPD-1

Test SPD-1 with very small amplitude was performed in order to evaluate the accuracy of the equivalent SPD test method. Fig. 6 shows experimental (the dotted line) and analytical (the solid line) time histories of the displacement at the roof level. Correlation between the two lines in Fig. 6 verifies the ability of the numerical analysis to simulate the experimental behavior. After 0.6 second, the SPD test was continued in a free vibration mode. The natural period of the SDF system was found to be 0.43 second, which coincides with the fundamental natural period predicted by the vibration test and the FLL test. No cracks were observed after this test.

SPD-2

The maximum roof displacement, the drift angle, and the maximum base shear were 32.5 mm, 1/670, and 226 ton, respectively. As evidenced in Fig. 8, correlation between the test and the numerical pre-test analysis is excellent during the first 2 seconds of ground motion. However, this analytical curve (the solid line) deviates significantly from the experimental curve (the dotted line) once the time exceeds 2 seconds. From the comparison between the two hysteretic characteristics (Fig. 9), one used in the pre-test analysis and the other derived from the test SPD-2, the analytical cracking load was shifted from 91 ton to 153 ton (Table 3) and the structure was reanalyzed. The post-test analytical curve (the chained line in Fig. 8) reasonably duplicates the experimental curve. Therefore, the hysteresis used in post-test analysis was employed in the succeeding numerical analysis. A free vibration test using the SPD method was conducted at the end of this test, and the natural period was mesured 0.55 second. Cracks developed in the first and second story shear wall, and some cracks were observed in several beams and slabs.

SPD-3

By using the modified hysteretic characteristics, numerical analysis was performed many times to find an input ground motion which would satisfy the JTCC requirements. Eventually the artificial wave based on Taft earthquake of 1952 was selected, and the maximum ground acceleration was set at 320 gal so that the maximum drift angle would be limited to 3/400. The maximum responses were 238 mm, 1/91, and 411 ton. The displacement time history at the roof level obtained from this test is very similar to that obtained from the analysis. The natural period of 1.15 second was found from the SPD test in a free vibration mode. Existing cracks widened, and new flexural as well as shear cracks were observed in the boundary columns and the shear wall panel. Concrete spalling started at the base of the boundary columns and at the wall ends of beams connected to the shear wall. Flexural cracks appeared in the transverse beams at the wall end. Cracks radiating from the edge boundary columns developed in slabs due to the upward displacement of the shear wall.

SPD-4

The responses were 342 mm, 1/64 and 439 ton. The maximum base shear (439 ton) of the structure was much larger than the calculated one (280 ton) in the preliminary nonlinear analysis. Little decrease in the resistance of the structure was observed even at a drift angle of 1/64. Final crack patterns is shown in Fig. 16. Damage of the test structure is briefly described as follows:

a) Major cracking of the shear wall was concentrated in the first story. Cracking in other parts of the shear wall was light.
b) Cracks in the first three stories indicated flexural behavior of the wall. Under simulated earthquake response, a top story drift of 1/64 was obtained with no shear failure in the wall.
c) Significant crushing and spalling of compressive concrete were observed in beams connected to the shear wall. In addition, spalling occurred at the base of the first story boundary columns.
d) Crack patterns in slabs near the boundary columns indicated upward displacement of the shear wall boundary columns.
e) No severe damage was observed in beam-to-column connections despite the relatively low amount of shear reinforcement provided in the connections.

During the SPD test in a free vibration mode, which was executed in the last part of test SPD-4, the natural period was 1.48 second. This natural period was three times as long as that in the intact stage. This natural period is 1.67 times longer than that of test VT-2 and FLL-2. The discrepancy of these natural periods was caused by the difference in vibration amplitude.

CONCLUDING REMARKS

Major findings of this paper are summarized as follows.

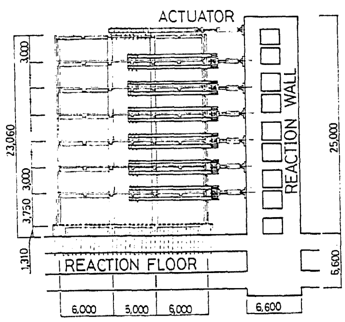
1. The fundamental natural period of the structure was 0.43 sec. in the intact stage, but was lengthened in accordance with the level of damage that the structure sustained. At the end of Test SPD-4, by which time the structure had undergone sever damage, the natural period was 1.48 sec., more than three times as long as the initial natural period.
2. The maximum base shear carried by the structure observed in Test SPD-4 was 439 ton, and it was much larger than the calculated one in the preliminary nonlinear analysis. It is considered that the reason of this discrepancy is the underestimation of the effects of slabs and transverse beams.
3. The maximum drift angle obtained during Test SPD-4 was 1/64, at which the structure showed ductile behavior since the shear wall, the major lateral load resisting component, was damaged in a flexural mode.

Table 1 Test Sequence

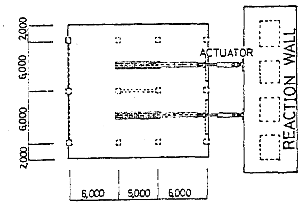
VT - 1	Free & forced vibration tests
FLL- 1	Each floor level loading test
SL - 1	Static loading test
SPD- 1	Pseudo-dynamic test (1/7000) Modified MIYAGIKEN-OKI NS 23.5gal
SPD- 2	Pseudo-dynamic test (1/400) Modified MIYAGIKEN-OKI NS 105gal
SPD- 3	Pseudo-dynamic test (3/400) Modified TAFT EW 1952 320gal
SPD- 4	Pseudo-dynamic test (1/75) TOKACHIOKI HACHINOHE EW 350gal
FLL- 2	Each floor level loading test
VT - 2	Free & forced vibration tests

Table 2 Test Results

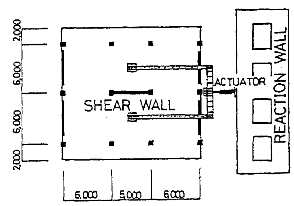
	SPD-1	SPD-2	SPD-3	SPD-4
Period (sec)	0.43	0.55	1.15	1.36
Input motion (gal)	23.5	105.	320.	350.
Max. disp. (mm)	2.52.	32.5.	238.	342.
Max. Shear (ton)	31.5	226.	411.	439.



a) Section of Test Structure

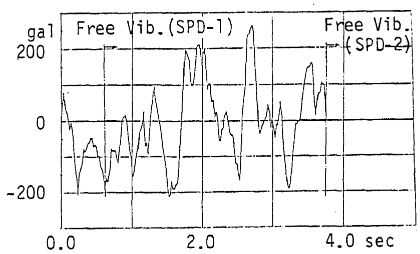


b) Top Floor



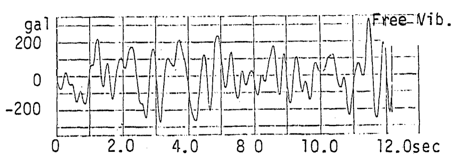
c) Other Floors

Fig. 1 Set Up of Test Structure



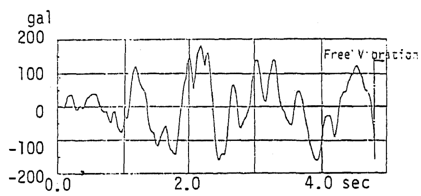
Input Motion (SPD-1 x0.088)
(SPD-2 x0.396)

Fig. 2 Input Motion (SPD-1,2)



Input Motion (SPD-3 x0.962)

Fig. 3 Input Motion (SPD-3)



Input Motion (SPD-4 x1.914)

Fig. 4 Input Motion (SPD-4)

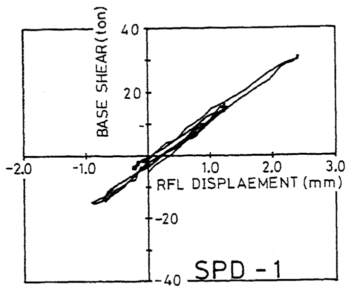


Fig. 5 Base Shear vs. RFL Displacement (SPD-1)

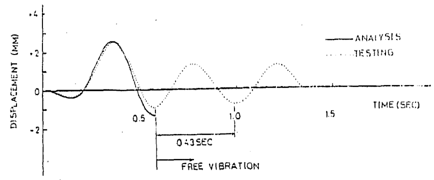


Fig. 6 RFL Displacement Time History (SPD-1)

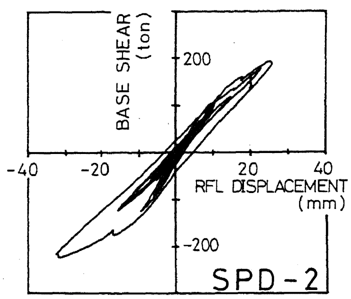


Fig. 7 Base shear vs. RFL Displacement (SPD-2)

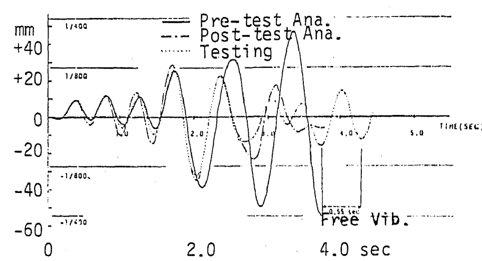


Fig. 8 RFL Displacement Time History (SPD-2)

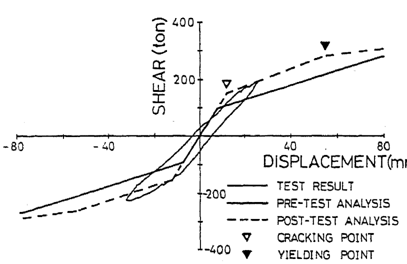


Fig. 9 Hysteretic Characteristics used in Pre- & Post- Analyses

Table 3 Values of Hysteretic Characteristics

	PRE-TEST ANALYSIS	POST-TEST ANALYSIS
INITIAL STIFFNESS (ton/cm)	129.0	129.0
CRACKING SHEAR FORCE (ton)	91.0	153.0
YIELDING SHEAR FORCE (ton)	274.0	274.0
YIELDING DISPLACEMENT (cm)	7.76	5.44

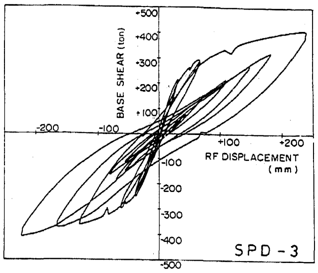


Fig.10 Base Shear vs.
RFL Displacement (SPD-3)

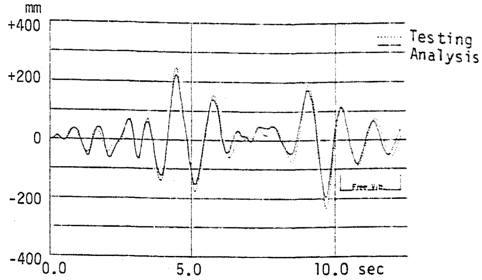


Fig.11 RFL Displacement
Time HIstory (SPD-3)

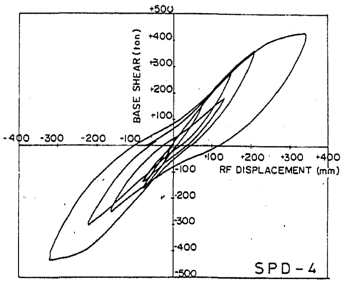


Fig.12 Base Shear vs.
RFL Displacement (SPD-4)

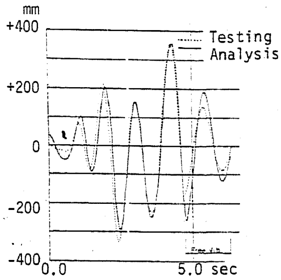
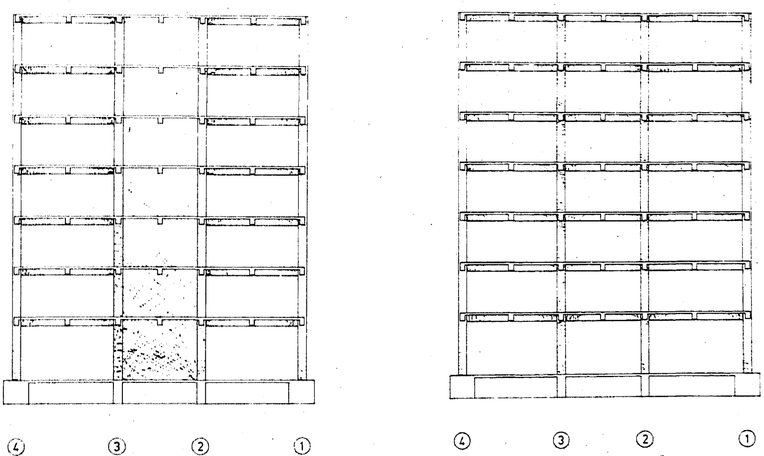


Fig.13 RFL Displacement
Time HIstory (SPD-4)



a) Frame-B

b) Frame-A

Fig.14 Crack Patterns after SPD-4

REFERENCES

(1) Kaminosono, T., et al.,"U.S.-Japan Cooperative Research on R/C Full Scale Building Test - part 1," The 7th WCEE, U.S.A., 1984.

(2) JTCC,"Summary Report on the Seismic Test of a Seven Story Reinforced Concrete Building", Materials for the Fourth JTCC Meeting, 1983.

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