U.S. - JAPAN COOPERATIVE RESEARCH ON R/C FULL SCALE BUILDING TEST
Part 1 Single-Degree-of-Freedom Pseudo-Dynamic Test

Takashi Kaminosono (I)
Makoto Watabe (I)
Shin Okamoto (I)
Shinsuke Nakata (I)
Manabu Yoshimura (I)
Presenting Author: Takashi Kaminosono

SUMMARY

A full scale R/C seven story structure was tested under recommendations and resolutions of the Planning Group Meeting and of the Joint Technical Coordinating Committee of the U.S.-Japan Cooperative Research Program Utilizing Large Scale Testing Facilities. This paper presents an overview of the full scale test structure and its test method. The single-degree-of-freedom (SDF) pseudo-dynamic (SPD) test method was employed in this test. Effectiveness of the SPD test method was verified by comparing the results of numerical response analyses with the test results.

INTRODUCTION

A Planning Group for "U.S.-Japan Cooperative Research Program Utilizing Large Scale Testing Facilities" was organized in 1977 and met four times from 1977 to 1979. In those meeting, recomendations for the cooperative research program utilizing large scale testing facilities was developed. The overall objective of the recommended program was to improve seismic safety practices through studies to determine the relationship among full scale tests, small scale tests, component tests, and analytical studies.

Plans of large scale test on various types of structures were proposed in the meeting, and the reinforced concrete structure was chosen for the first phase of this joint program. To implement this joint program, a committee, named the Joint Technical Coordinating Committee (JTCC), was organized. Members of the committee discussed various details of the test program including the design of the full scale structure and the test procedures. H. Umemura (University of Tokyo) and J. Penzien (University of California at Barkeley) served as the co-chairman, and M. Watabe (Building Research Institute) and R.D. Hanson (University of Michigan) as the coordinater of JTCC.

The full scale test structure was designed in accordance with design procedures practiced in both Japan and U.S.A. The pseudo-dynamic test system was employed in the earthquake response test of this structure.

(I) Building Research Institute, Ministry of Construction, Tsukuba, Ibaraki, JAPAN

TEST STRUCTURE

First, members of the test structure were designed based upon the present building specifications of both U.S. (ACI, UBC) and Japan (New Aseismic Code) as well as preliminary analyses. The ultimate base shear coefficient was larger than 0.4. However, considering the results of response analysis and the capacity of testing facilities, the sections were considerably less reinforced than those conforming with U.S. and Japanese practices. Finally, the ultimate base shear coefficient obtained from the preliminary analysis was 0.23.

Dimmensions

Fig. 1 illustrates the test structure. The structure is a seven story reinforced concrete building, which is 21.75 m in total height and 272 m^2 in floor area. The story height is 3.75 m for the first story and 3.0 m for the second through seventh stories. The cross section of the columns and beams is 500 mm x 500 mm and 300 mm x 500 mm respectively. The transverse beams have the cross section of 300 mm x 450 mm. The structure has a shear wall of 200 mm in thickness in the middle frame parallel to the loading direction (Frame B in Fig. 1). The wall was considered to be the primary lateral load resisting element. Shear walls of 150 mm in thickness were also arrayed in the exterior frames perpendicular to the loading direction (Frames 1 and 4 in Fig. 1). The walls, isolated from the surrounding columns, were expected to restrain out-of-plane deformation of the structure during loading.

Details of Reinforcement

Fig. 2 illustrates the details of reinforcement in frames and slabs. Table 1 tabulates the arrangement of reinforcing bars in columns, beams and shear wall. Boundary columns attached to the shear wall were heavily reinforced in the first and second stories in order to ensure sufficient ductility of the wall. Closed hoops and cross ties, therefore, were arranged with a pitch of 100 mm respectively. The shear wall did not have any boundary beam in its own plane.

Material Properties

Concrete was mixed so that the compressive strength would arrive at 270 kg/cm^2 after twenty eight days. Reinforcing bars of SD35, equivalent to Grade 60, were used.

SINGLE-DEGREE-OF-FREEDOM PSEUDO-DYNAMIC TEST

In a pseudo-dynamic test of a multi-degree-of-freedom (MDF) system, external force distribution is complex and varies randomly with time. Performing pseudo-dynamic test of a MDF system is not suitable for obtaining data most effective to immediate use. For this reason, it was decided to test the structure as a SDF system with external force distributed in inverted triangular mode.

A MDF system can be reduced to an equivalent SDF system if the mode of deflection is assumed. Table 2 shows the equations of motion of the equivalent SDF system reduced from the MDF system. Static frame analysis of the test structure was carried out to find out an appropriate mode of deflection for the equivalent SDF system. An inverted triangular lateral force distribution was used for the analysis. The results, shown in Fig. 3, indicate that the deflection mode does not change significantly regardless of the magnitude of the force; the mode is nearly identical in elastic inelastic, and mechanism ranges. Table 3 demonstrates the variation of single-degree equivalent mass derived from the mode of deflection at each load level. The equivalent mass does not change significantly. For this reason, the average of deflection modes was employed in this test. Table 4 shows the mode of deflection, mass and external force distribution assumed in this test. The assumed deflection patterns and those obtained from test at several load levels, shown in Fig. 4, are also in good agreement.

Procedure of the Pseudo-Dynamic Test

Fig. 5 illustrates the load control system. In the pseudo-dynamic test, the computer on-line system controlled only the displacements of two actuators at the roof level. By using the measured force at roof level, input forces to the other actuators were so determined - independently from the on-line system - that the overall distribution of external forces would be held inverted triangular. Table 5 shows the testing procedure.

Dynamic Response Analysis

In order to reproduce the building response obtained from the SPD-4 test, the single-degree pseudo-dynamic response analysis (SPD analysis) was performed, in which the same procedures employed in the SPD tests was applied. In addition to the SPD analysis, the multi-degree (pseudo-)dynamic response analysis (MPD analysis) was performed to investigate the response of the MDF building.

The relationship of base shear and roof-level displacement is shown in Fig. 6. The maximum response displacements for SPD test and SPD analysis are quite agreeable although there are considerable differences in the maximum base shear and the energy absorption; i.e. smaller base shear and larger energy absorption in SPD analysis. The reason of this is believed to be the fact that the underestimate in the base shear for SPD analysis compensates the overestimate in the energy absorption, which leads to the good agreement in the maximum displacement. A MPD analysis indicates a pronounced higher mode effect which was not observed in the other two cases. However, the maximum response displacement agrees well with that of the SPD analysis.

The response roof-level displacement time histories (Fig. 7) demonstrate the same tendency as shown in Fig. 6: the wave form of the roof-level displacements of the SPD and MPD analyses are in good agreement although slight discrepancies are observed between these two and the SPD test.

The response base shear time histories (Fig. 8) demonstrate that the SPD test and SPD analysis are in good agreement with the same extent of the differences as observed in Fig. 7. Note that the base shear time history of the MPD analysis is affected significantly by higher modes, especially by the second mode oscillation.

The response base moment time histories (Fig. 9) indicate that the SPD and MPD analyses are in good agreement, but slightly different from the SPD test. Note that the base moment time history is similar in general tendency to the roof level displacement time history.

Through the comparisons of the responses of the test and two analyses, it is made clear that the SPD analysis is capable of adequately reproducing the results and that the MPD analysis produces the response results similar to the SPD analysis for the displacement and base moment time histories but does not produce for the base shear time history.

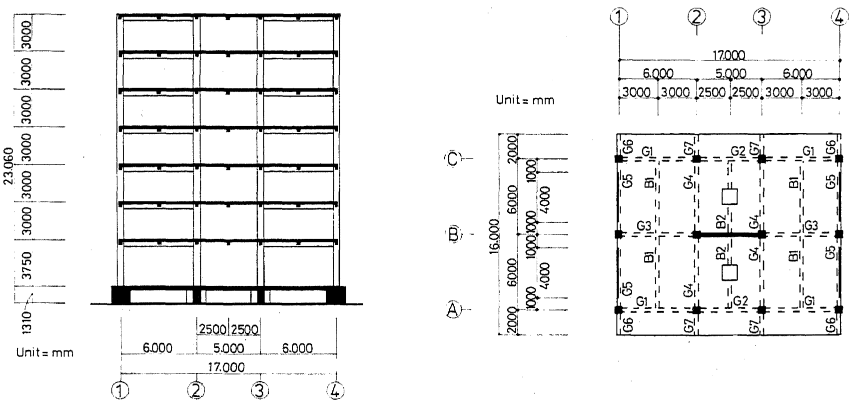
Concluding Remarks

The major findings of this paper focusing on the evaluation of the validity of the SPD test are summarized as follows;

1) The deflection mode assumed to reduce the degree of freedom was found reasonable.
2) The SPD analysis is capable of reproducing the SPD test results with sufficient accuracy.
3) Good agreement between the responses of the SPD and MPD analyses demonstrates that the SPD test process employed for the full scale R/C seven story building test has produced almost the same response displacement and base moment time histories as those to be obtained if the MPD test process had been employed.

REFERENCE

Okamoto, S., et al.,"A Progress Report on the Full-Seismic Experiment of a Seven Story Reinforced Concrete Building - part of the U.S.-Japan Cooperative Program", B.R.I. Research paper No. 94, JAPAN, March 1982.



a) Elevation b) Plan

Fig. 1 Test Specimen

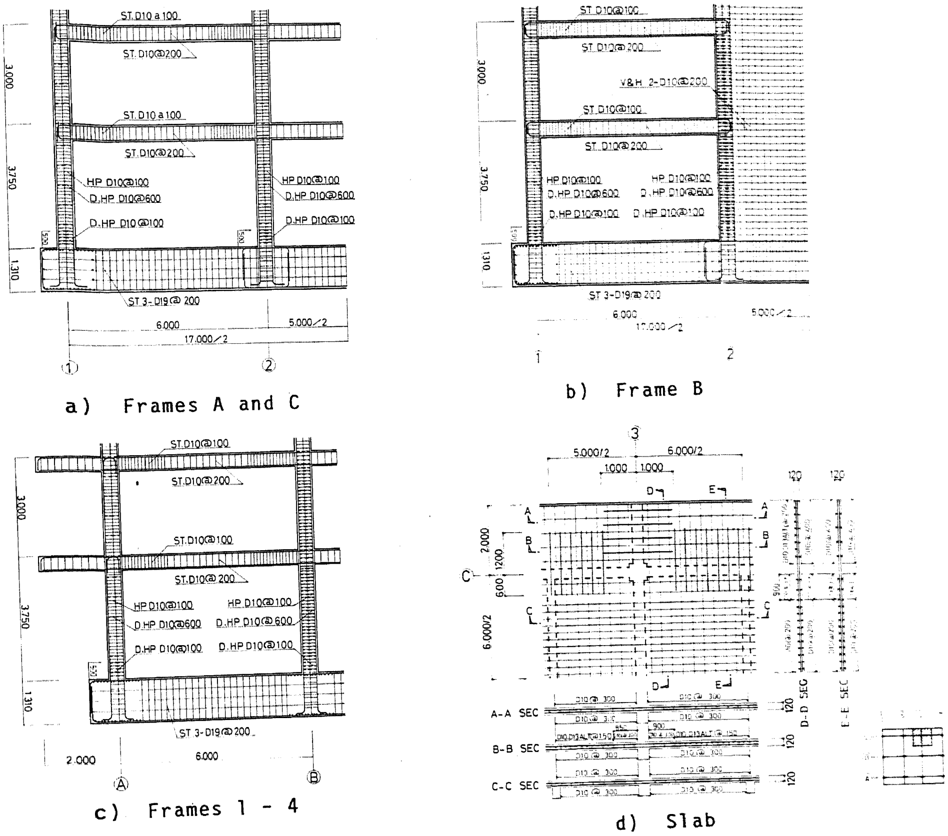


Fig. 2 Detailes of Reinforcements

Table 1 Arrangement of Reinforcing Bars

COLUMN		BEAM	END	CENTER	WALL	
B x D	500 X 500	b x D	300 X 500 *2		Thickness	200 mm
Main Bars	8 - D22	Top	3 - D19	2 - D19	Length	5000 mm
Hoop	D10 @100	Bottom	2 - D19	3 - D19	Reinf.	V&H 2-D10@200
Cross Ties	D10 @600 *1	Stirrup	D10 @100	D10 @200	Column	500 x 500

*1 Cross ties are D10 @100 in Boundary Column at 1-st and 2-nd story

*2 Dimension of Transverse Beam is 300 X 450

(Unit = mm)

Table 2 Equation of SDF System

$$\tilde{m}\ddot{\tilde{x}} + \tilde{p} = -\tilde{m}\ddot{x}_0$$

where: $\tilde{m} = \Sigma MrUr$ (equivalent mass)

$\beta = \Sigma MrUr/\Sigma MrUr^2$ (mode participation factor)

$\tilde{x} = (1/\beta)\ x$ (equivalent displacement)

$\tilde{p} = \Sigma PrUr$ (equivalent external shear)

$x = f(t)$ is a function varying with time

Mr = mass at the r-th floor level

Pr = external lateral force at the r-th floor level

$\ddot{X}r$ = acceleration at the r-th floor level

$\ddot{x}_0$ = ground acceleration

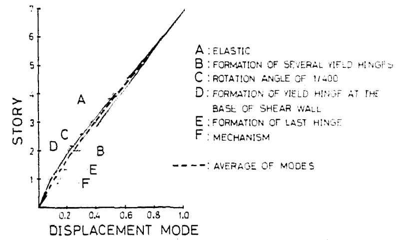


Fig. 3 Deflection Modes

Table 3 Equivlent Mass

LOAD POINT	A	B	C	D	E	F	AVERAGE
EQUIV. MASS	1.418	1.418	1.421	1.424	1.421	1.419	1.422

Table 4 Mode, Mass, and Force

$$(Ur) = \begin{pmatrix} 1.00 \\ 0.85 \\ 0.696 \\ 0.540 \\ 0.384 \\ 0.234 \\ 0.102 \end{pmatrix} \quad (Mr) = \begin{pmatrix} 0.156 \\ 0.173 \\ 0.173 \\ 0.173 \\ 0.173 \\ 0.173 \\ 0.187 \end{pmatrix} \frac{t.sec^2}{cm} \quad (Pr) = \begin{pmatrix} 21.75 \\ 18.75 \\ 15.75 \\ 12.75 \\ 9.75 \\ 6.75 \\ 3.75 \end{pmatrix} p\ ton$$

(Ur) : Mode of deflection

(Mr) : Mass of each floor

(Pr) : Lateral force of each floor

Table 5 Procedure of SPD Test

INPUT: ${}_n\tilde{X},\ {}_{n-1}\tilde{X},\ {}_n\tilde{P},\ {}_n\ddot{X}_0$

CALCULATION: ${}_{n+1}\tilde{X} = 2\cdot{}_n\tilde{X} - {}_{n+1}\tilde{X} - \frac{{}_n\tilde{P}}{\tilde{M}}(\Delta T)^2 - {}_n\ddot{X}_0(\Delta T)^2$

${}_{n+1}X_7 = 1.422\cdot{}_{n+1}\tilde{X}$

LOADING: ${}_nX_7 \Longrightarrow {}_{n+1}X_7$

MEASURING: ${}_{n+1}F_1 \sim {}_{n+1}F_7$

CONVERSION: ${}_{n+1}Q = \sum_{r=1}^{7} {}_{n+1}F_r$

${}_{n+1}\tilde{P} = \frac{1}{1.457}\cdot{}_{n+1}Q$

$\tilde{M}$: EQUIVALENT MASS

$n\tilde{X}$: EQUIVALENT DISPLACEMENT n-step

$n\tilde{P}$: EQUIVALENT FORCE n-step

nXr : DISPLACEMENT r-floor n-step

nFr : FORCE r-floor n-step

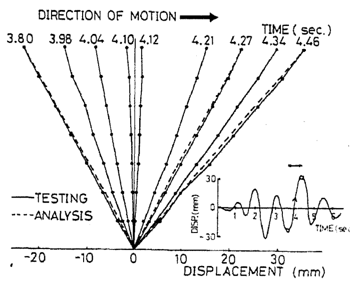


Fig. 5 Loading System

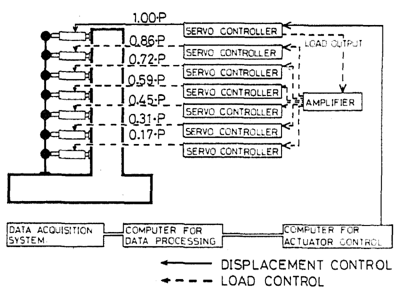
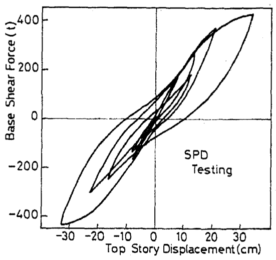
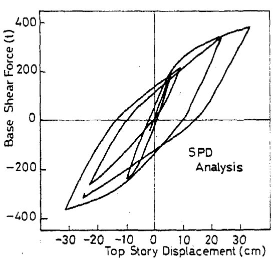


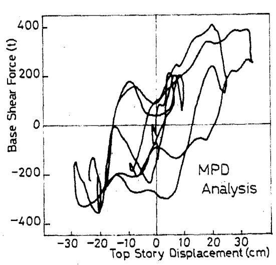
Fig. 4 Deflection Patterns



a) SPD Testing



b) SPD Analysis



c) MPD Analysis

Fig. 6 Base Shear vs. RFL Displacement

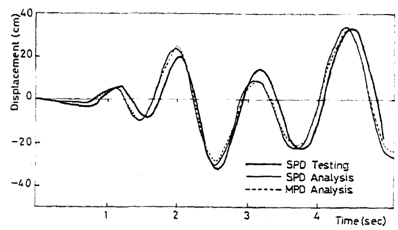


Fig. 7 RFL Displacement Time History

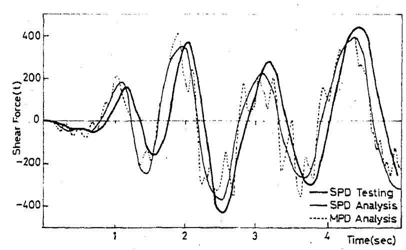


Fig. 8 Base Shear Time History

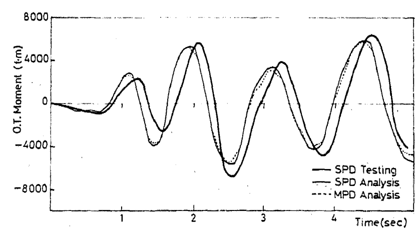


Fig. 9 Base Moment Time History