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SHAKING TABLE TEST OF PRECAST CONCRETE WALL STRUCTURE

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ABSTRACT

This paper presents the results of test on the precast concrete large panel building model using a shaking table facility. 1/3.3-scaled 3-story PC box specimen was made and tested under simulated earthquake motions. The employed input acceleration was the one recorded as Taft N21E component. The peak ground acceleration (PGA) of Taft N21E component was scaled depending on the desired level of seismic severity. The test results show that the collapse of the specimen occured through the rocking motions of superstructure at horizontal joints of the first floor. Elastic limit of test specimen was turned out 0.6g, which is 5 times of the maximum acceleration specified in Korean seismic design code.

KEYWORDS

precast concrete large panel building; shaking table; Taft N21E component; horizontal joint; rocking motion; elastic limit

INTRODUCTION

In recent years, precast concrete (PC) panel buildings have become more widely applied on Korea. The development of panel strucrtures is related to the production technology and the structural system depending on the type of the system and especially on the connection system. HDLP (Hyun Dai Large Panel) system is the PC panel system in Korea, which combines the walls by horizontal and vertical joints. The vertical tie in horizontal joints is connected by the joint box, which can minimize the erection errors.

Since the typical PC systems have weak connections, they cannot behave in such a way of monolithic reinforced concrete structures. Even if the structural response of the PC system to seismic excitation is substantially different and failure sequence, it has not been well documented yet. The object of this paper is to verify the seismic behavors of the HDLP system under seismic excitation using the shaking table.

TEST PROGRAM

Description of Test Specimen

Test specimen was 1/3.3-scaled 3-story PC box structure, which was composed of precast components produced in the factory. All connections of the test specimen were of the wet joint type with panels providing forming for the poured joints. Vertical continuity is provided by small cast in placed columns with continuous reinforcing in vertical joints between wall panels, and a single bar extending from the top and bottom at each end of the wall panels with joint boxes. Wall panels were reinforced for temperature and shrinkage using a mesh of deformed wire placed at the center of the wall panel. Layout of test specimen and details of the connections were illustrated in Fig. 1. To avoid the twist of the test specimen with seismic excitation, test specimen was modeled symmetrically. The materials of test specimen were scaled down in accordance with the scale factors of the test model.

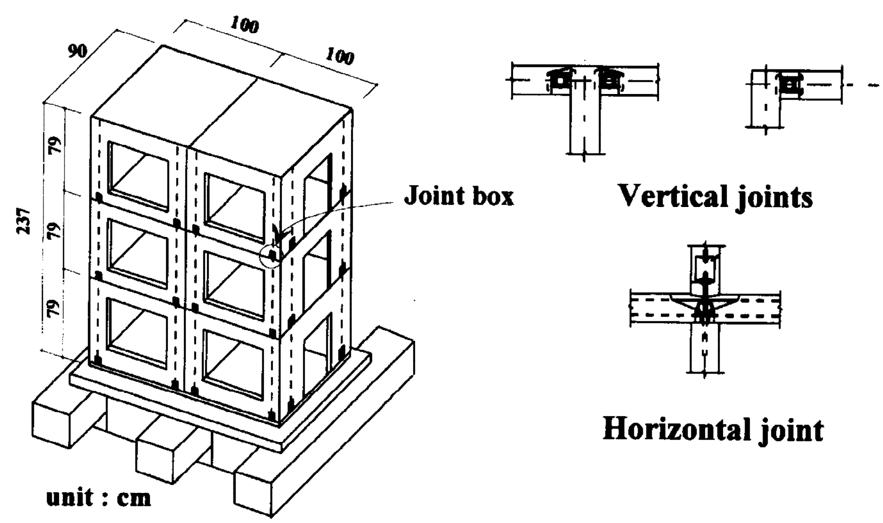

Fig. 1 Test specimen layout

Test Procedure

To acquire the data of deformation, strain and acceleration responses in the test specimen with seismic excitation, the equipments, such as LVDTs, potentiometers, strain gages, accelerometers were instrumented at the test specimen. Major point to check the deformations and strains of the test specimen were horizontal joints of each floors.

The movement of the shaking table was controlled by a computer, and white noise signal and earthquake loading with variable frequency and amplitude might be input as a model base acceleration. The test were conducted using the scaled N21E component of the Taft earthquake with increasing the amplitude step by step to simulate its different intensity (0.06g, 0.12g, 0.25g, 0.4g, 0.6g, 0.8g, 1.0g, 1.2g, 1.4g).

To properly simulate the dynamic inertia effects, the addition of mass plates was necessary (Sabnis et al, 1983). Lead mas plates were placed so that the inertia effect would be distributed to the model as it would be in the real structural without adding undue stffiness or strength to the joint.

Table 1 Artificial mass calculation

Type	Prototype	Model	α
Ratio	1	$\frac{1}{n^3} + \alpha = \frac{1}{n^2}$	$n = 3.3(ratio)$ $\alpha = \frac{1}{n^2} - \frac{1}{n^3} = \frac{(n-1)}{n^3} = 0.064$

The natural vibration periods for the model were determined by an free vibration test and ambient vibration test. In the free vibratiion test, the model was allowed to vibrate in its natural modes by pulling on the model at each level, releasing it, and recording the vibration of accelerometers that were attached to the model. In the ambient vibration test, the earthquake input to the model in several magnitudes was controled by simply scaling up the voltage input to the power amplifier.

TEST RESULTS

General Observations

The model was taken into the non-linear range during the 0.8g with rocking motions in the horizontal joints. The initial cracks appeared in the horizontal joints at 0.12g and were spreaded to the overall horizontal jonts. The test specimen was failed in 1.4g by crushing of the joint box in horizontal joints without any cracks in vertical joints and wall panels. Rocking of the wall system above the lower horizontal joint was predominant during the test. Shear-slip behavior couldn't be detected. The concentrated compression induced by rocking had crushed concrete at the wall ends.

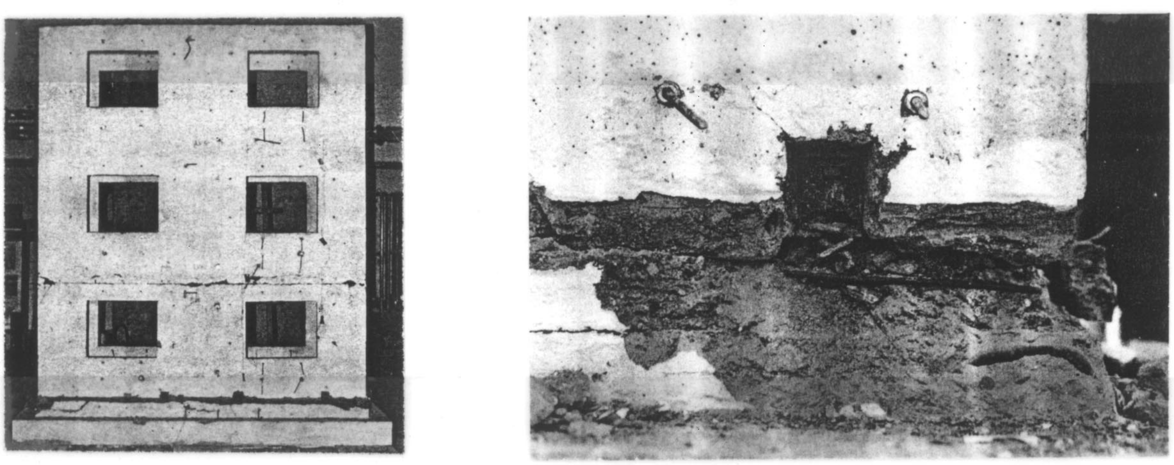

Fig. 2 Damage of horizontal joint

Acceleration Response

The acceleration responses of the test model in each levels are presented in Fig.5. Acceleration responses of the test model in each floors increased with increment of input accelerations to the 1.2g level. But at the 0.8g level, the acceleration responses of the roof and 3 floor were partially decreased with increment of input earthquake magnitude. This means that the test model might be behavied inelstically in this level.

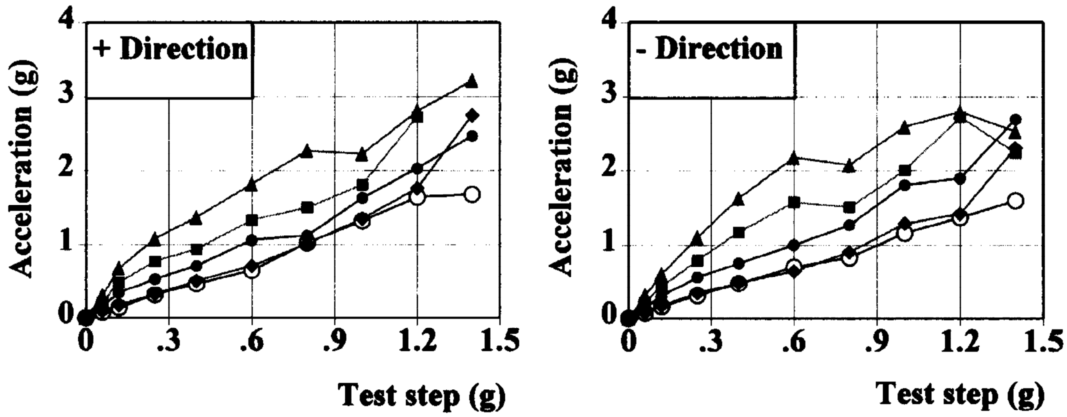

Fig. 3 Acceleration response of the test specimen

Base Shear-Displacement Response

The horizontal displacements of each floors in the test model was measured by LVDTs. The base shear-horizontal displacement curves of the test model are presented in Fig. 4. In 0.8g magnitude, the hysteretic loops of the test model were deformed to the inelastic shapes, but the properly inelastic shapes on the hysteretic loops were shown in 1.4g with rupture of the test model. Fig. 5 presents the strain response of the vertical tie bars in horizontal joints. The vertical tie bars yielded in 0.8g by the rocking motions on the horizontal joint.

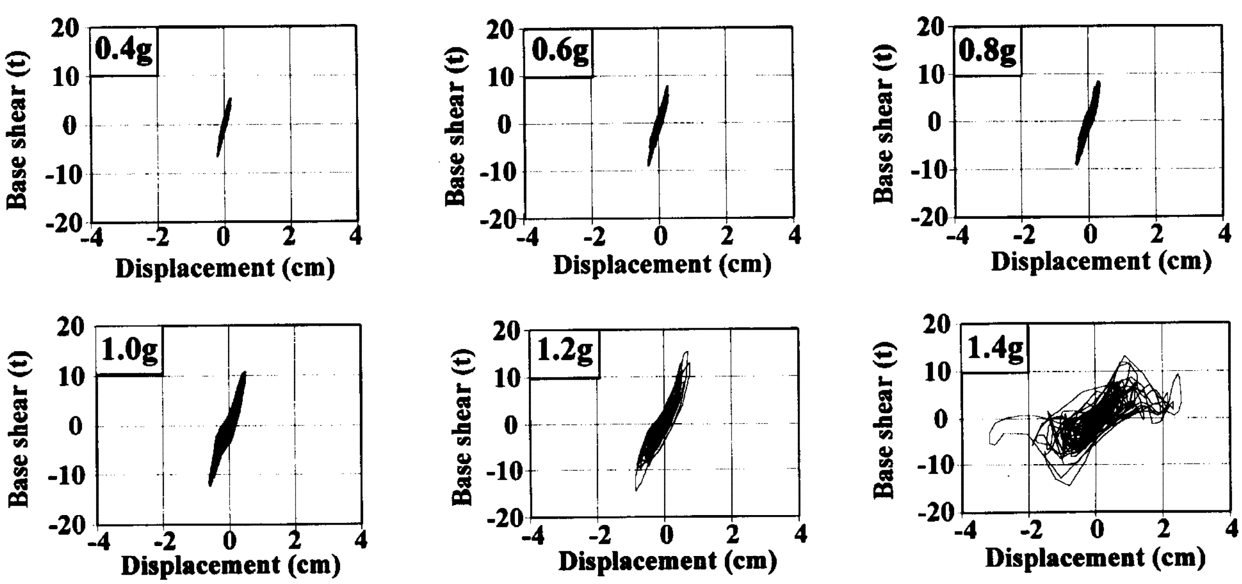

Fig. 4 Base shear vs. top displacement curve

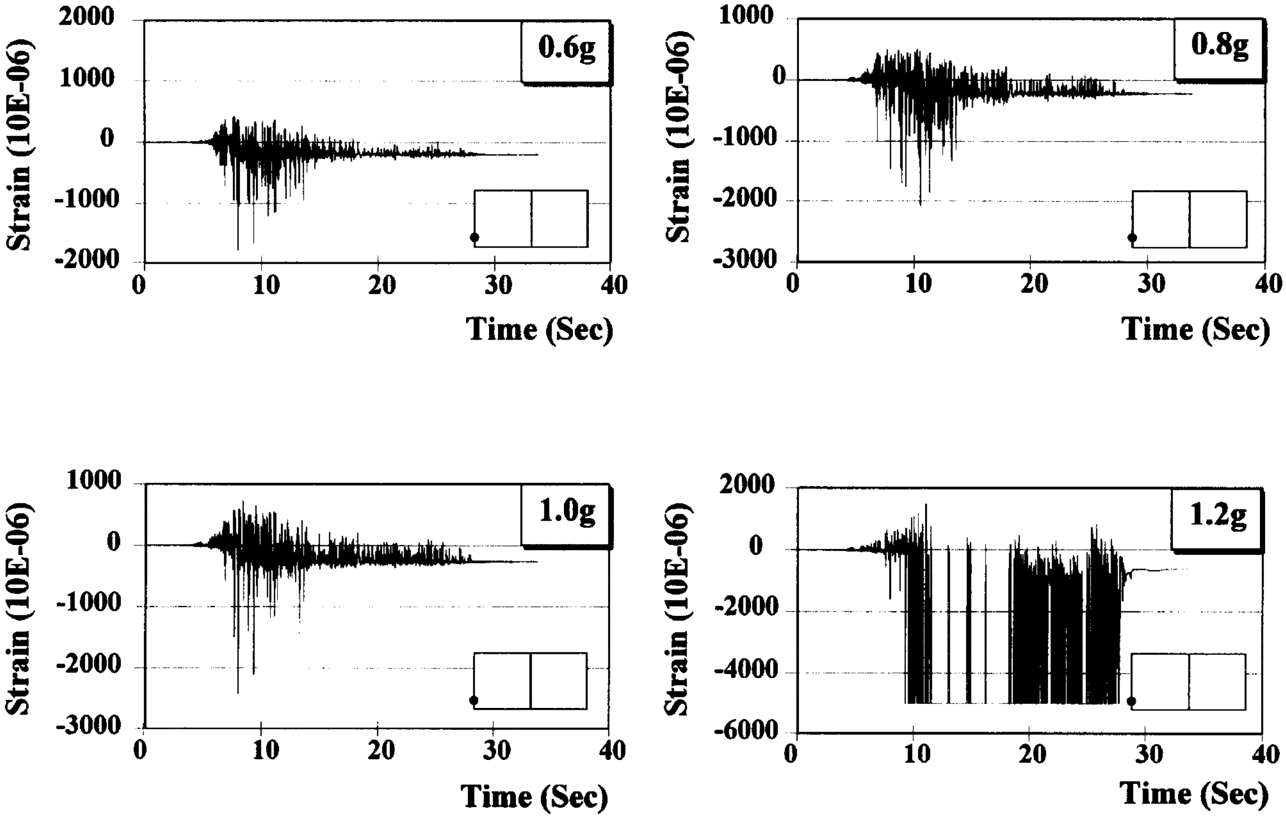

Fig. 5 Strain response of vertical tie bar (1st floor)

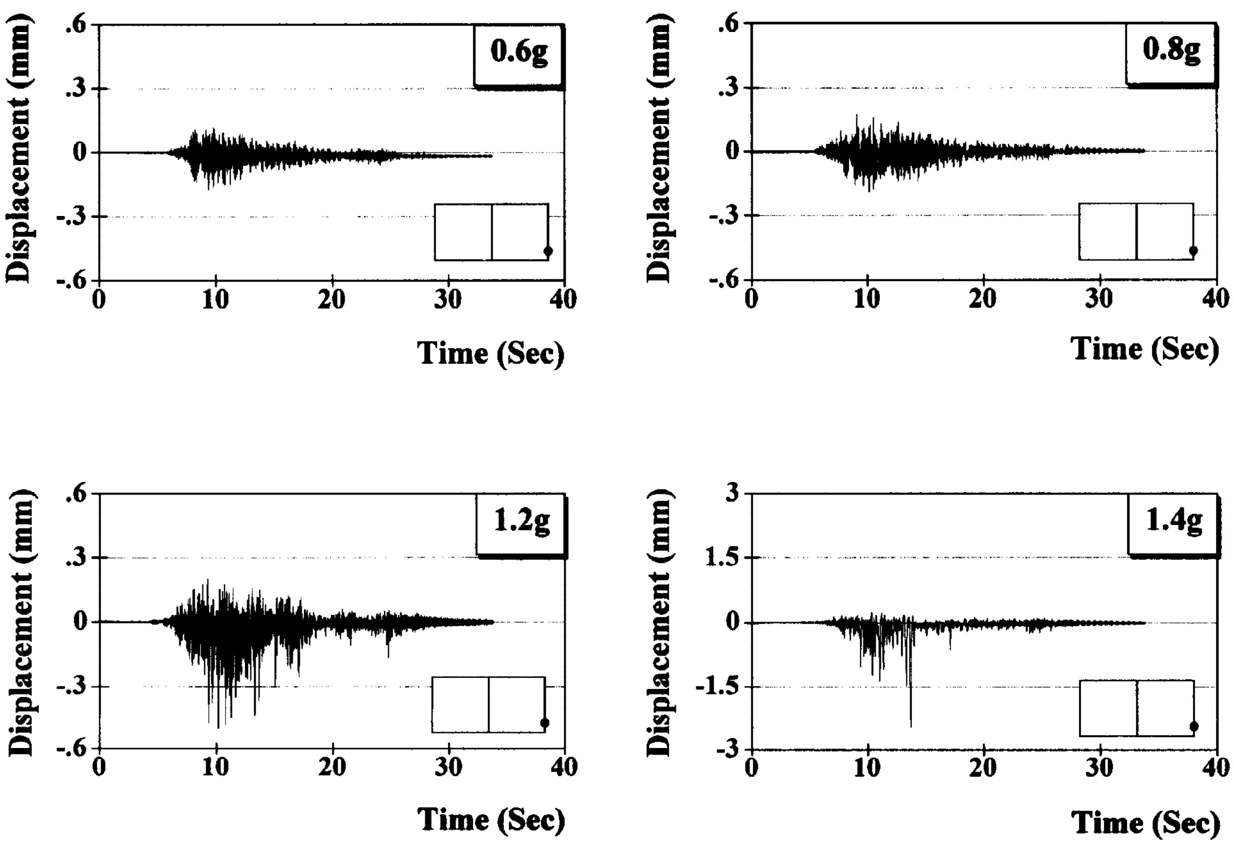

Fig. 6 Rocking response of horizontal joint (1st floor)

Periods, Damping Ratios and Stiffness

To verify the periods, damping ratio and stiffness of the test model, the free vibration test was conducted for the earthquake steps. After the model was pulled back and released, the acceleration at each floor level of the model were measured and recorded. From the decay curve, the frequency content was computed. The damping ratio was also approximated using the decay of the accelerogram. In the ambient test, "white nose" was employed to determine the free vibration frequency of test model. The test results are presented in Table 2 and Table 3. These tables show that natural periods of the test model which can be converted to 0.11 ~1.62 sec of the true size model.

Damping calculations from the given data are approximated at best, since it is seen from the frequency domain results that the acceleration versus time traces are made of signal of several dominant frequencies. The estimated results for damping computed from the test results are given in Table 3. The values of damping in Table 3 lies within the range of values which was from 0.05 to 0.064.

Table 2 Periods of test specimen

Test	Initial step	0.06 g	0.12 g	0.25 g	0.40 g	0.60 g	0.80 g	1.00 g	1.20 g
Free Vibration Test	0.060	0.065	0.065	0.067	0.070	0.073	-	-	-
Ambient Test	0.062	0.067	0.070	0.070	0.089	0.089	0.090	0.089	0.089

Table 3 Free vibration test results

Step	U_1	U_n	n	δ	ζ	K
0.06 g	0.039	0.009	5	0.367	0.058	1
0.12 g	0.035	0.005	6	0.389	0.062	1
0.25 g	0.039	0.009	6	0.293	0.050	0.97
0.40 g	0.030	0.004	6	0.463	0.064	0.86
0.60 g	0.032	0.008	5	0.347	0.055	0.8

U_1 : Initial displacement, U_n : nth displacement $\delta = \frac{1}{n} \ln\left(\frac{U_1}{U_n}\right)$

δ : Logarithmic decrement of damping, ζ : Damping ratio $\zeta = \frac{\delta}{2\pi}$

K : Stffiness ratio

CONCLUSIONS

1/3.3-scaled 3-story PC box specimen was made and tested, to study the seismic behavors of the precast concrete wall structure under seismic excitation using the shaking table. From the study of the present experimental results, it can be concluded that :

The dynamic response of the model showed the failure mechanism of the system to be the crushing of the horizontal joints by rocking without any damages in vertical joints and panels.

Elastic limit of the test model was turned out 0.6g, which is 5 times of the maximum acceleration specified in Korean seismic design code.

The base shear coefficients were considerably higher than the 0.07 value derived from the Korean code for equivalent lateral force.

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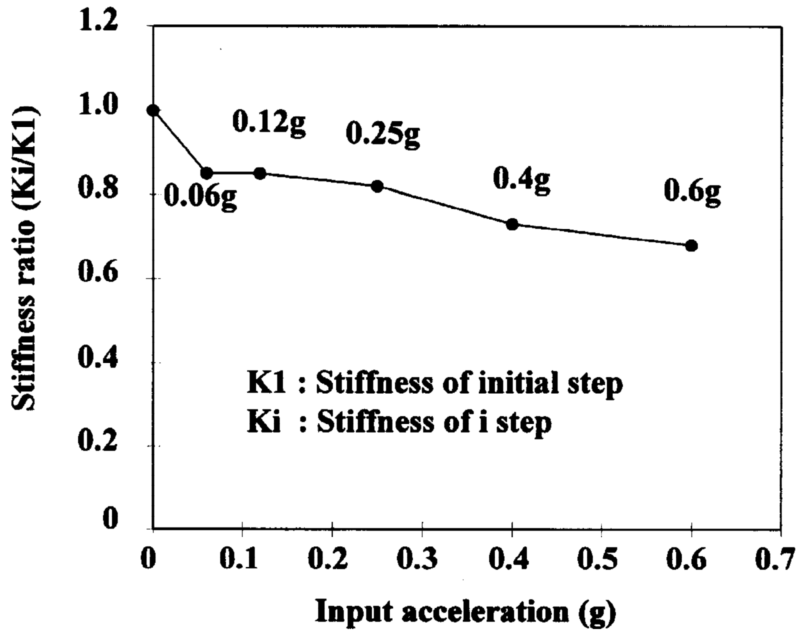

Fig. 7 Stiffness of test specimen

Shear Coefficient Ratio

The strength of this test relative to the commonly used "equivalent static design load" could be judged by the true base shear coefficient. Table 4 presents the true base shear coefficient of each earthquake steps. The value of the design shear coefficient for the Korean code would be approximately 0.07 for PC wall structure. The base shear coefficients in table were considerably higher than the 0.07 value derived from the Korean code for the equavelent lateral forces. This means that the test model might have designed very strongly.

Table 4 Coefficient of base shear vs. weight

		0.06 g	0.12 g	0.25 g	0.40 g	0.60 g	0.80 g	1.00 g	1.20 g	1.40 g
Base shear	(+)	1.29	2.83	4.45	5.42	7.95	8.43	11.02	15.30	13.18
	(-)	-2.43	-2.43	-4.56	-6.67	-8.97	-8.99	-14.55	-14.55	-14.54
$\frac{V}{W}$	(+)	0.16	0.36	0.57	0.69	1.01	1.07	1.40	1.95	1.68
	(-)	0.31	0.31	0.58	0.85	1.14	1.14	1.85	1.85	1.85

* Total weight (W) = 7.86t