SEISMIC BEHAVIOR OF PRECAST CONCRETE LARGE PANEL BUILDINGS USING A SMALL SHAKING TABLE

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SUMMARY

This paper presents the results of tests on precast concrete large panel building models using a small capacity shaking table facility. Simple, single stack shear walls are tested in stages to failure and the results compared to nonlinear dynamic analysis. Both the slip and the rocking mechanisms of failure that had been predicted analytically were observed in these tests.

INTRODUCTION

Large panel industrialized concrete buildings have become steadily more viable forms of construction in the United States in the last ten to twenty years. Their increasing use and extension into the more active seismic regions of the country requires an understanding of their behavior under cyclic dynamic loading. Experimental data of this type, although meager, are available for some large panel systems used in other parts of the world. For the typical precast concrete large panel systems used in North American practice this type of information is lacking. The main reason for this critical situation is the relatively high costs associated with full scale testing.

In order to overcome these difficulties and to shed some light on the behavior of large panel systems, a small-scale modeling technique has been developed at Drexel University over the past few years to study such important structural aspects of these bearing wall structures as progressive collapse (Ref. 1-3) and earthquake resistance (Ref. 4-7). In addition to quasi-static cyclic tests on small scale models, tests on a small shaking table facility (Ref. 8) that has been built for this purpose have been conducted.

Analytical investigations(9-10) have pointed out to the weakness of the horizontal wall-to-floor joints in large panel systems. These analytical and also other experimental investigations have found that the non-linear inelastic behavior of simple shear walls is concentrated in the connection region and that the connection regions act as precracked planes. Simple shear wall structures subjected to seismic loading were found to possess two sources of inelastic action: joint opening and closing (known as rocking) and shear slip. No experimental results have been presented to date to confirm these analytically predicted behavioral modes. The present paper discusses experimental results obtained to characterize the behavior of these structural systems.

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EXPERIMENTAL PROCEDURE

The small scale modeling technique was chosen to study the behavior of the simple shear wall studied analytically at MIT (Refs. 9 and 10) under earthquake loading. Since the major goal of the research effort was to study the nonlinear inelastic behavior of the model, it was necessary to use model materials that adequately duplicate the prototype properties. Microconcrete and appropriately annealed steel wires were chosen for this purpose. A dimensional analysis (Ref. 11) using the three independent variables: modulus of elasticity, gravitational acceleration and length is shown in Table 1. The scale factor for modulus of elasticity and gravitational acceleration was taken as unity. The length scale was restricted by two major considerations: adequate construction detail in the joints and the ability of the testing facility. The Drexel shaking table (Fig. 2) consists of an electro-magnetic shaker attached to a 2 inch aluminum slab riding on roller bearings. The system is capable of reaching a maximum force of 7.6 kN (1700 lbs) in the random mode. A scale of 1/32 was chosen for the model which is shown in Figure 3. Mass simulation was artificially maintained by attaching small custom made lead plates to the wall panels and floor slabs (Fig. 3).

Prior to subjecting the model to earthquake loading, a series of tests were carried out to determine the static and dynamic properties of the model. Flexibility influence coefficients were determined by applying small elastic loads to each floor level and measuring the deflections using LVDT's. Using the measured flexibilities the model frequencies and mode shapes were determined by solving the eigenvalue problem. In addition, the vibration frequencies of the model were determined using a free vibration pull-back test. The vibration was measured using accelerometers attached at each floor level. A digitized representation of the acceleration signal was analyzed in the frequency domain by use of the Fast Fourier Transform (FFT) technique.

The earthquake simulation tests were conducted using the properly scaled N-S component of the 1940 El Centro earthquake as the input base acceleration. The model was instrumented using LVDT's and accelerometers at the various floor levels. Reference locations are shown in Figure 4. Two LVDT's were oriented vertically in the fore and aft position across the horizontal joint to measure the rocking motion of the stacked wall panels. All other LVDT's and accelerometers were oriented horizontally. During the test, the real time instrument signals were recorded in analog form on a reel-to-reel tape recorder. To process the data, the recorded signal was played back at a reduced rate, digitized and stored on a microcomputer system. Time domain and frequency domain results were plotted on an x-y recorder for visual inspection and comparisons.

TEST RESULTS

The first test performed on the uncracked model was the flexibility matrix determination. Using these results, the solution to the eigenvalue problem gave a first mode frequency of 41.7 Hz. The free vibration pull-back test which followed showed typical acceleration decay curves as shown in Figure 5. Analysis of this data in the frequency domain gave the first mode frequency to be in the band from 41.6 to 45.8 Hz which coincides with the result from the flexibility coefficient determination. At higher modes this correlation broke down. For the second mode the eigenvalue problem solution gave 101 Hz as compared to 81 Hz for the pull-back test. For this model the eigenvalue solution from the flexibility data appears to give a good indication only of the fundamental frequency.

The model was taken into the non-linear range during the first earthquake test. During this first test the maximum base acceleration recorded was 0.24g. Floor displacements at levels 2 thru 5 were recorded relative to the model base level. As an example of the results, Figure 6 shows the level 5 displacement versus time curve. Levels 2 thru 4 displacements followed the same waveform but with a different amplitude scale. Table 2, Column 1, summarizes the maximum response recorded on each instrument during the test.

The rocking motion was analyzed as a rocking rotation, θ, defined as follows:

$$\theta = \tan^{-1} \frac{b_1 - b_2}{\ell}$$

where, b_1 = rear rocking displacement
b_2 = front rocking displacement
ℓ = length of the shear wall

For this earthquake test, the rotation was recorded across the base joint. The rotation versus time calculations are shown in Figure 7 with a maximum rotation of 0.00427°.

The frequency of the cracked wall was then determined by repeating the test used to determine the flexibility coefficients along with the pull back test. From the flexibility data a first mode frequency of 31.7 Hz was computed. The pull back test showed the first mode frequency to be in the band from 30 Hz to 32.5 Hz.

The level of the earthquake was then increased in two steps to .41 g and then to 1.05 g maximum base acceleration. A comparison of the maximum instrument readings for the three magnitudes of input is given in Table 2. The amplitude versus time curves for the level 5 displacement and rocking rotation of the 1.05 g earthquake are shown in Figures 8 and 9, respectively. As expected, the largest drop in frequency is seen in the 1.05 g earthquake test as the structure softened due to cracking. The dominant frequency in the level 5 acceleration spectrum was 35 Hz for the 0.24 g magnitude compared to spectral peaks at 13 Hz and 21 Hz for the 1.05 g magnitude earthquake.

A plot of the maximum floor displacements for each earthquake magnitude is shown in Figure 10. In all three cases deviation from the normal bending mode is observed. The deviation was caused by an apparent slip at either the base or at level one of the structure. The magnitude of the slip at level 1 was estimated as 1.88 mm (0.071 in.) for the 1.05 g magnitude earthquake. This scales up to 57.6 mm (2.27 in.) in the prototype. The global slip observed at level 1 occurred in the joint directly above the first wall panel where the bearing pad is located. The increased rocking caused crushing of the joint and the panel corners at both ends of the wall. These observed failure mechanisms are shown in Figure 11.

A comparison of the MIT analysis to the small scale model test is summarized in Table 3. The MIT analysis used a .25 g magnitude earthquake therefore, the comparison was made to the .24 g magnitude test on the small scale model. When scaled up to the prototype the small scale model's initial frequency was approximately 25% higher than the analytical frequency. This would indicate that the small scale model was stiffer than the MIT analytical model. After the small scale model was cracked however, it was shown that its frequency dropped to 34 Hz which scales to 6 Hz in the prototype. This is only 2% higher than the MIT result. The maximum deflection of the model was lower than the MIT prediction by 6%. The MIT analysis showed global slip to occur at level 4 of the model. The level where slip occurred in the physical model was undoubtedly affected by variables such as strength of materials, workmanship and panel smoothness.

CONCLUSIONS

The dynamic response of the small-scale model showed the failure mechanisms of the system to be the slip and the crushing of the panel corners caused by rocking. The behavior was remarkably close to the behavior predicted by an MIT analysis. Further tests on two identical models are currently underway on the Drexel University shaking table. Future tests will have additional instrumentation to enable the determination of slip at all floor levels.

Increasing the earthquake magnitude resulted in considerable softening of the structure as damage in the horizontal connections increased. In the case of the El-Centro input, the model response was amplified to a greater extent as the structure softened. The damaged structure remained standing even after a 1.05 g magnitude earthquake mostly due to the fact that a large amount of energy was absorbed by means of inelastic slip at level 1. The major damage in the structure was accumulated at the failed joint only. The remainder of the structure went essentially undamaged.

The tests thus far have demonstrated that the small scale modeling technique is a powerful tool useful in predicting the behavior and the mechanisms of failure of precast concrete large panel systems.

ACKNOWLEDGEMENTS

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Table 1 - Summary of Scale Factors for Earthquake Response of the Model

		Dimension	Scale Factors: Artificial Mass Simulation	Scale Factors: Small Scale Model
1	2	3	4	5
Loading	Force, P	F	$S_E S_\ell^2$	$(1/32)^2$
	Pressure, q	FL^{-2}	S_E	1
	Acceleration, a	LT^{-2}	1	1
	Gravitational Acceleration, g	LT^{-2}	1	1
	Velocity, v	LT^{-1}	$S_\ell^{1/2}$	$1/\sqrt{32}$
	Time, t	T	$S_\ell^{1/2}$	$1/\sqrt{32}$
ometry	Linear Dimension, ℓ	L	S_ℓ	1/32
	Displacement, δ	L	S_ℓ	1/32
	Frequency, ω	T^{-1}	$S_\ell^{-1/2}$	$\sqrt{32}$
Material Properties	Modulus, E	FL^{-2}	S_E	1
	Stress, σ	FL^{-2}	S_E	1
	Strain, ε		1	1
	Poisson's Ratio, ν		1	1
	Mass Density, ρ	$FL^{-4}T^2$	*	$32\ \rho_{conc}$
	Energy, EN	FL	$S_E S_\ell^3$	$(1/32)^3$

$* \left(\frac{g\rho\ell}{E}\right)_m = \left(\frac{g\rho\ell}{E}\right)_p$

Table 2 - Maximum Instrument Readings for 3 Earthquake Magnitudes

ITEM \ MAGNITUDE	1	2	3
	(1)	(2)	(3)
Base Acceleration	0.24 g	0.41 g	1.05 g
Table Displacement***	0.53 mm	1.04 mm	3.28 mm
Level 5 Displacement	0.245 mm	0.488 mm	3.45 mm
Level 4 Displacement	0.186 mm	0.381 mm	+
Level 3 Displacement	0.137 mm	0.320 mm	2.78 mm
Level 2 Displacement	0.107 mm	0.269 mm	2.59 mm
Level 1 Displacement	+	+	0.41 mm
Front Rocking	0.0106 mm*	0.027 mm*	0.36 mm**
Rear Rocking	-0.0065 mm*	-0.016 mm*	-0.44 mm**
Max. Rocking Rotation	0.0043°	0.0108°	0.193°

\+ not measured
* recorded at base
** recorded at level 1
***displacement relative to laboratory floor

Table 3 - Small Scale Model Comparison with MIT Five Story Structure (0.5% Reinforcement Ratio)

	MIT Analysis	Drexel University Tests: Model	Drexel University Tests: Prototype	% Difference
Peak Acceleration	.25 g	.24 g	.24 g	-4%
Initial Natural Frequency	5.88 Hz	42 Hz	7.42	+25%
Maximum Deflection	8.3 mm	.245 mm	7.84 mm	-6%
Slip	3.4 mm	0 to .09 mm	0 to 2.9 mm	-
Slip Level Location	4	Base or level 1	Base or level 1	-

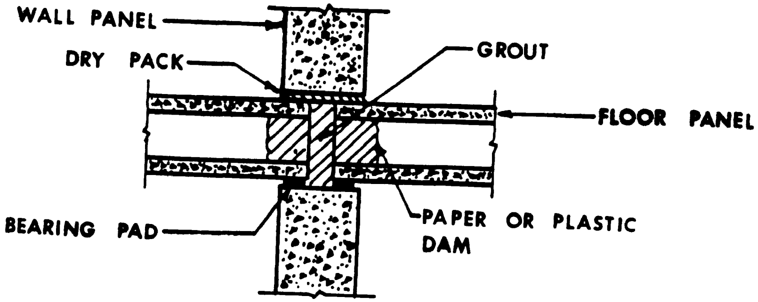

Fig.1- Platform Type Horizontal Joint.

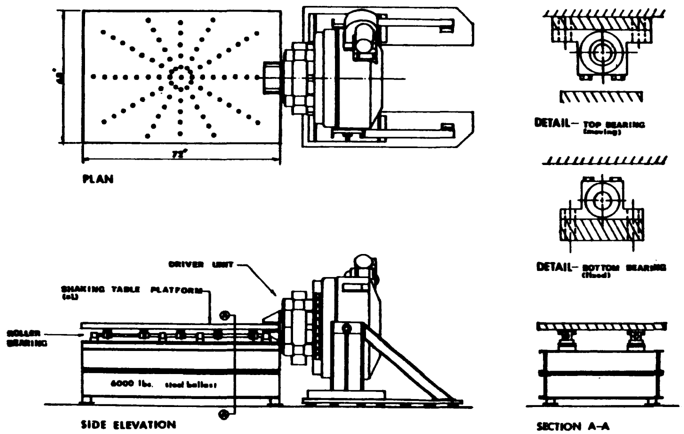

Fig.2- Drexel University Shaking Table.

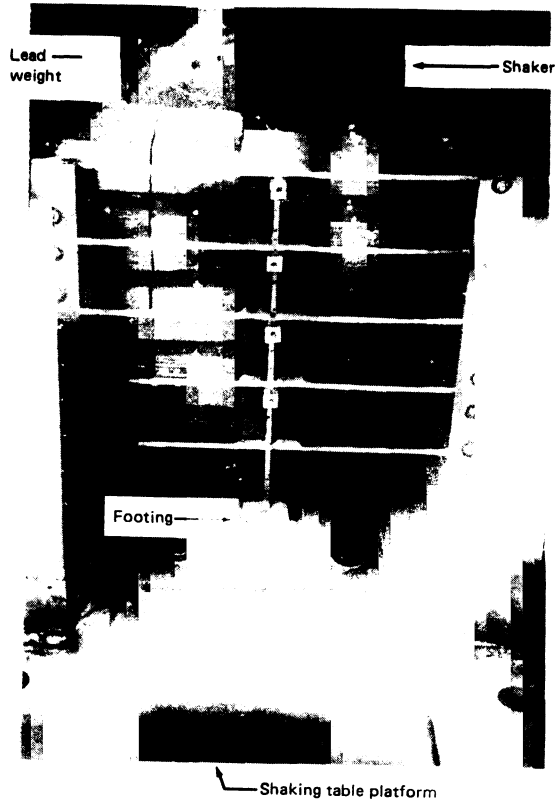

Fig.3- Photograph of the Model
Prior to Testing.

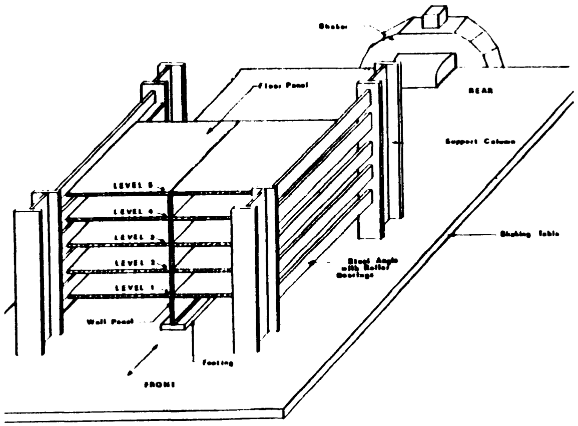

Fig.4- Reference Locations on Model.

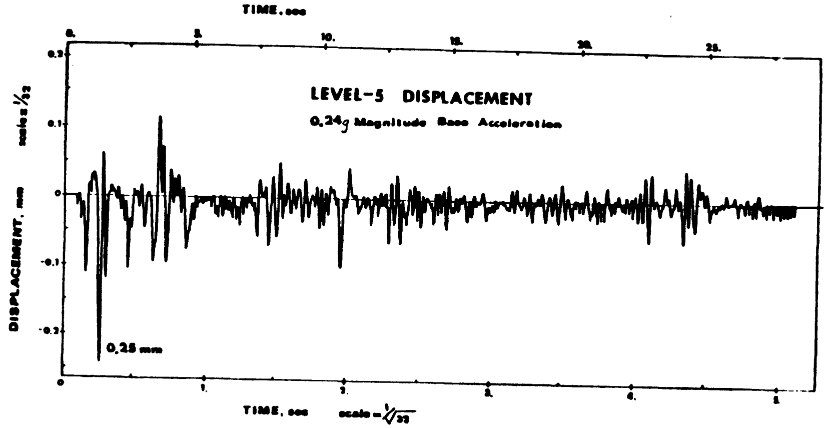

Fig.6- Measured Level 5 Displacement for 0.24g Magnitude Earthquake.

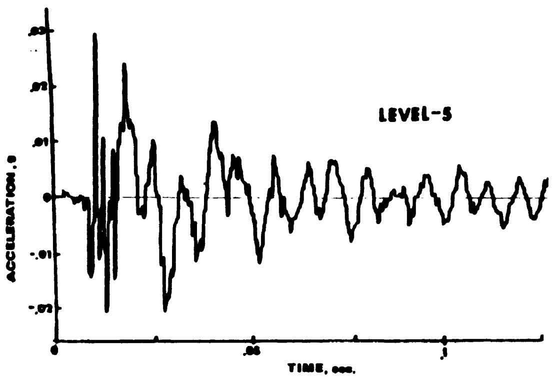

Fig.5- Typical Decay Curve Due to Pull-Back Test.

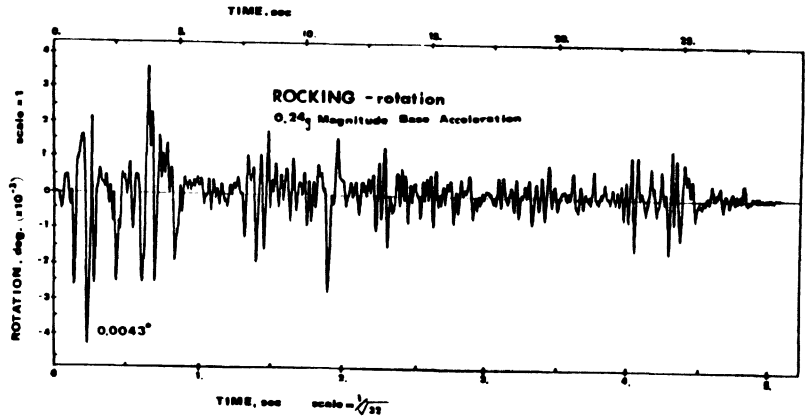

Fig.7- Rocking Rotation at the Base Level for 0.24g Magnitude Earthquake.

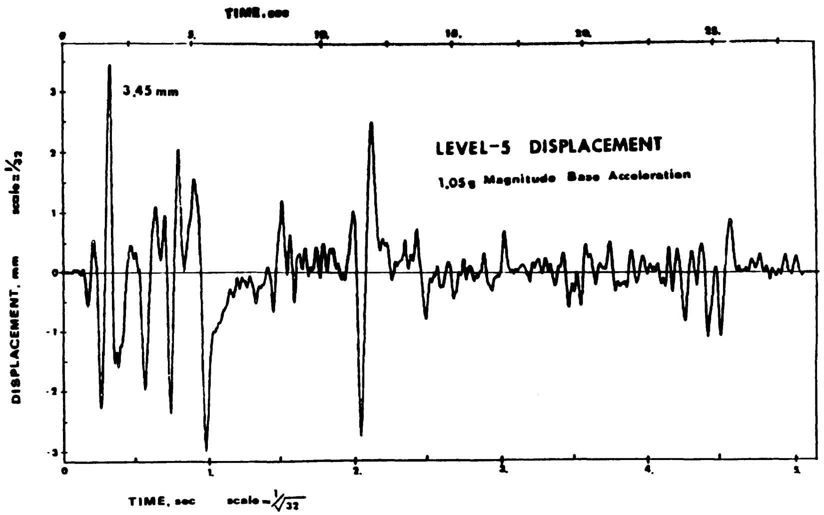

Fig.8- Measured Level 5 Displacement for 1.05g Magnitude Earthquake.

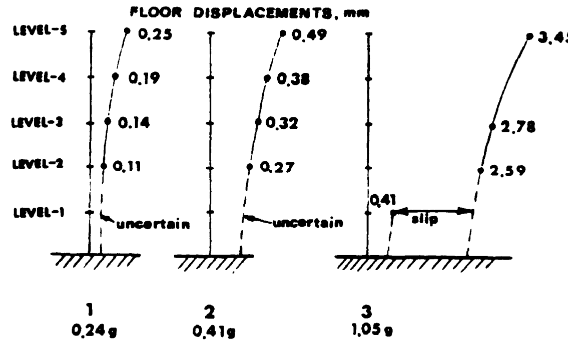

Fig.10- Maximum Floor Displacement for Each Earthquake Magnitude.

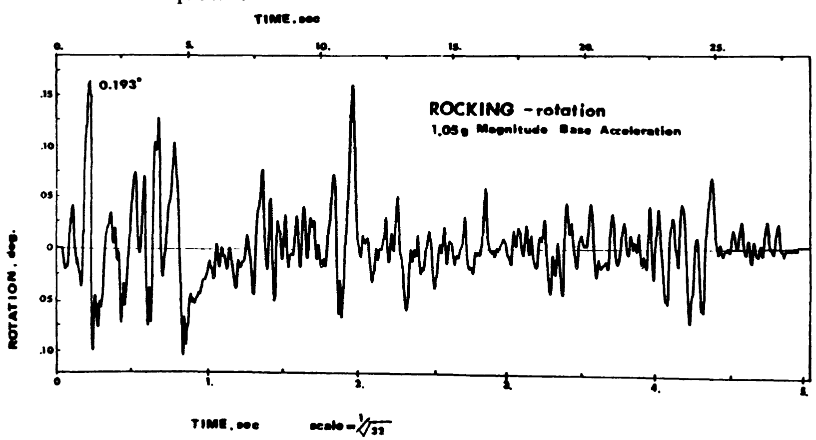

Fig.9- Rocking Rotation at Level 1 for 1.05g Magnitude Earthquake.

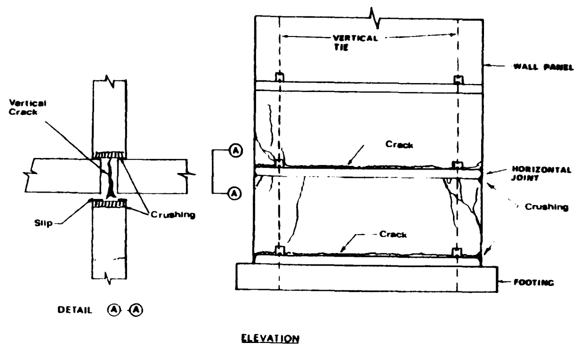

Fig.10- Observed Failure Mechanisms of the Small Scale Model.