Earthquake Simulator Testing of Scale Model Structures

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This paper presents an overview of the engineering aspects involved with dynamic testing of scale-model structures, the reasons for conducting such tests, and a summary of the results of earthquake simulator tests which were conducted on two separate 1/5-scale models of a 3 -storey reinforced concrete frame building.

Introduction

Even though there have been a number of recent Australian earthquakes greater in Richter magnitude than ML 5 (Everingham, 1982; Rynn et al., 1987), most buildings in Australia have not been designed to resist earthquake forces. In fact, the first Australian Earthquake Code, AS 2121 (SAA, 1979), was not published until 1979 and was a consequence of the 1968 earthquake (ML 6.9) in Western Australia which devastated the town of Meckering. This code was based largely on the 1974 edition of the report entitled Recommended Lateral Force Requirements and Commentary published by the Seismology Committee of the Structural Engineers Association of California (SEAOC, 1974) and represented the state-of-the-art at that time. It was, however, based largely on the results of Californian earthquake research and the observed behaviour of buildings there during earthquakes. Not withstanding AS 2121's publication in 1979, it was not until 1984 that the code was used, and even then only in one State and part of another.

The application of an American based earthquake code raises the question of whether buildings designed and built in accordance with Australian standards behave much differently under earthquake loading than their American counterparts (Griffith, 1992). Of special concern is the level of ductility provided by the current design practice since the recommended R factors in the code rely upon ductility. These values are based largely on the documented response of a wide variety of buildings during earthquakes in California. In the absence of this type of data for Australian buildings, it is necessary to conduct experiments to address this question.

Hence, a pilot study was conducted at the University of Adelaide to establish whether earthquake simulator tests could be carried out successfully with the facilities available in the Civil Engineering Department. However, the results of this preliminary research were also expected to: (1) establish whether a comprehensive research program into the behaviour of Australian designed reinforced concrete buildings subject to earthquake loads was needed; (2) experimentally determine the failure mode for a low-rise reinforced concrete moment resisting frame structure subject to earthquake loading; and (3) provide and estimate of the elastic reserve strength provided by the current design details for these types of structures.

Experimental models

This research was conducted in the Civil Engineering Department's structural testing laboratory at the University of Adelaide. The project consisted of testing two reinforced concrete frame structures, which were designed in accordance with the Australian Standard for concrete structures AS 3600 (SAA, 1988). The test structures were designed to be 1/5-scale replicas of a section out of a typical reinforced concrete moment-resisting frame building. The models were each 2.4 m tall, three storey singlebay portal frame structures spanning 1.2 m in each horizontal direction (Figure 1). The first model (Model A) was designed and constructed to comply with the concrete detailing requirements for ductile buildings in seismic zone A in Australia (SAA, 1979) which require no special detailing requirements for earthquake loads. The second model (Model B) was designed and constructed to comply with the concrete detailing requirements for ductile buildings in seismic zone 1 in Australia. These requirements call for extra stirrups (i.e. decreased stirrup spacing) in the beams and columns in the region of beam-column joints. The stirrup spacing for the two models is shown in Figure 2.

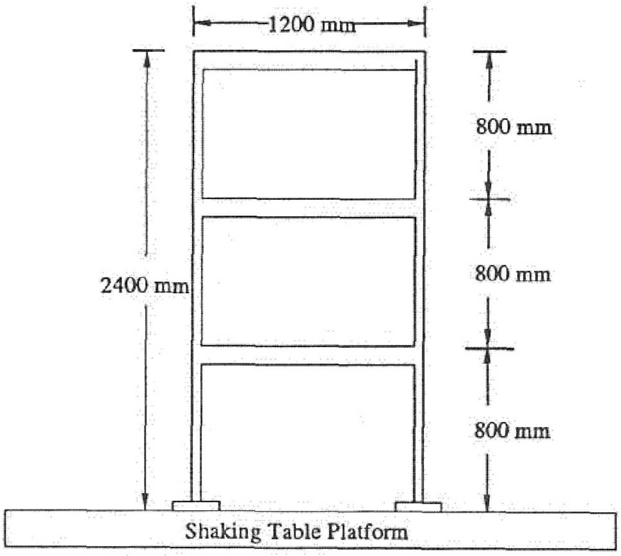
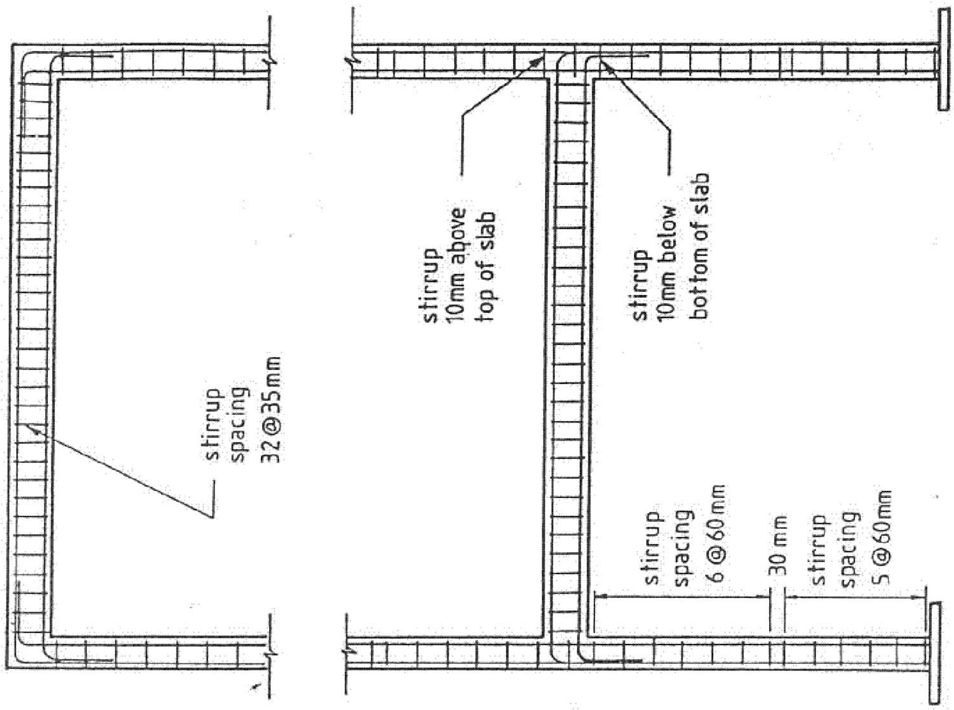


Figure 1. - Elevation of 1/5-scale model test structure

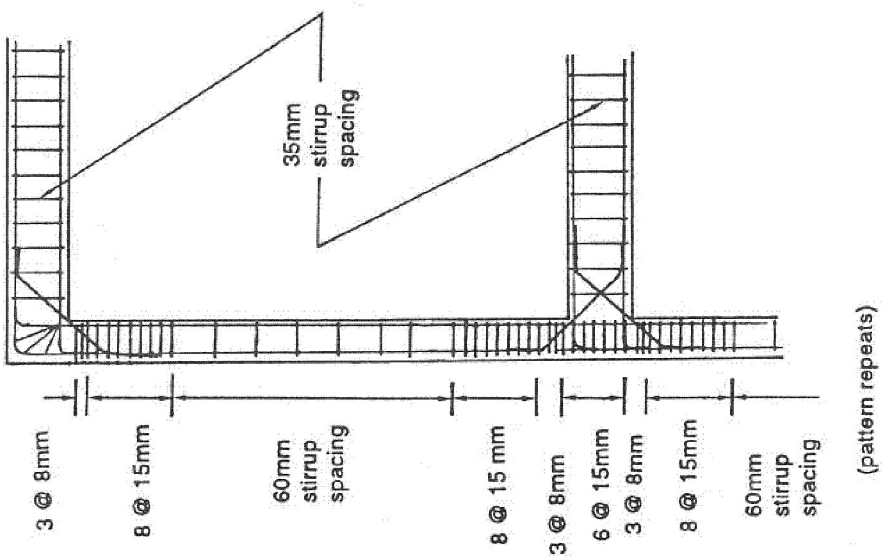
Model testing program

Each model was subjected to a testing program which consisted of a series of progressively increasing earthquake tests. A free-vibration test was conducted before and after every earthquake test in order to measure the change in natural frequencies and damping of the models. This in turn provided an indication of the decrease in stiffness of each model and a measure of the damage suffered during each earthquake test. Although Australian strong ground motion records may be used in future testing, the North/South component of the 1940 El Centro, California, strong ground motion earthquake record (Read et al., 1974) was used for these tests in view of its historical use as a benchmark for testing.

Model A Test results Model A was subjected to a free-vibration test before conducting the first earthquake test from which the first mode frequency was 3.1 Hz and the damping ratio was 1.7%. This was in good agreement with what would be expected for `uncracked' concrete structures (Newmark and Hall, 1982).
The 1/5-scale structure was then subjected to a sequence of four $\sqrt{5}$ time-scaled El Centro earthquake tests. The peak amplitude of the earthquake input was increased in each successive test, starting with a peak ground acceleration (PGA) of 0.23 g and finishing with a PGA of 0.39 g test. The results for all earthquake tests are summarised by plots of the inertia force and storey shear profiles (Figure 3). Interestingly, the irregularity of the inertia force profile for the first earthquake test (PGA 0.23 g) corresponded to a drop in the fundamental frequency from 3.1 Hz to 1.6 Hz. This was attributed to the cracking suffered by the model during this test. It was also noted that the damping of the cracked structure was 6%, in good agreement with the 5% value normally used for design. The results of free-vibration tests conducted



(a) - Model A Ligature Details for Earthquake Zones 0 and A



(b) - Model B Ligature Details for Earthquake Zone 1

Figure 2 - Steel Reinforcing Details for Model Test Structures

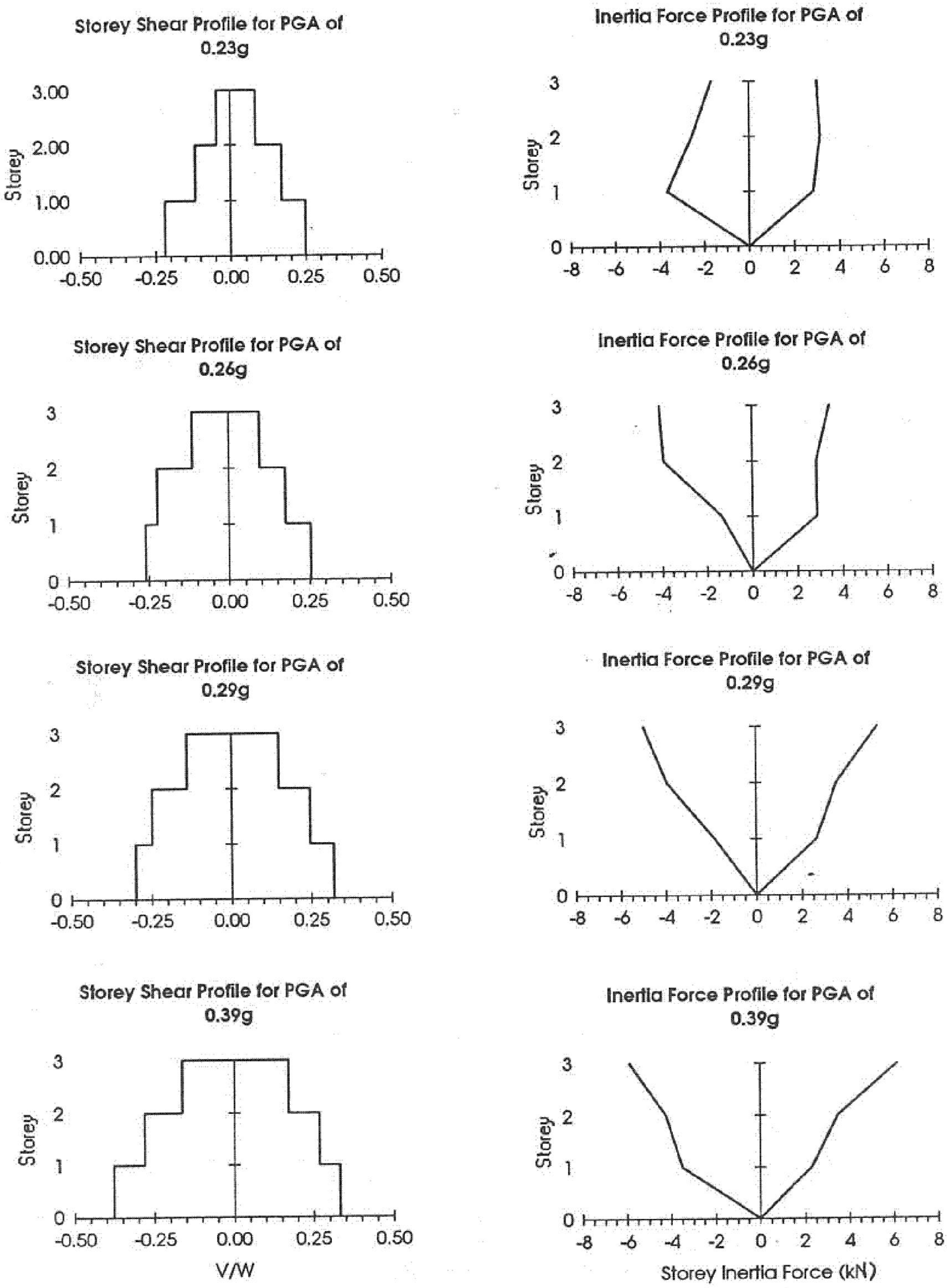


Figure 3 - Model A Test Results

after each subsequent earthquake test indicated that there were no further drops in the fundamental frequency of the structure.

To estimate the magnitude of base shear force which would cause yielding to occur, the peak base shear force (V), normalised by the weight of the structure (W), was plotted against peak shaking table acceleration (PGA, Figure 4). It was observed that the shear increased nearly linearly for all tests with a peak ground acceleration magnitude less than 0.3 g. However, the last test (PGA 0.39 g) caused a peak base shear force which deviated from the linear part of the curve in Figure 4. One possible explanation for the decreased structural response was that the structure had undergone a change in period, however, results of the subsequent snap-back test indicated no such change in period. Hence, this deviation was attributed to yielding of the structure, most probably in the region of the first storey beam-column joints.

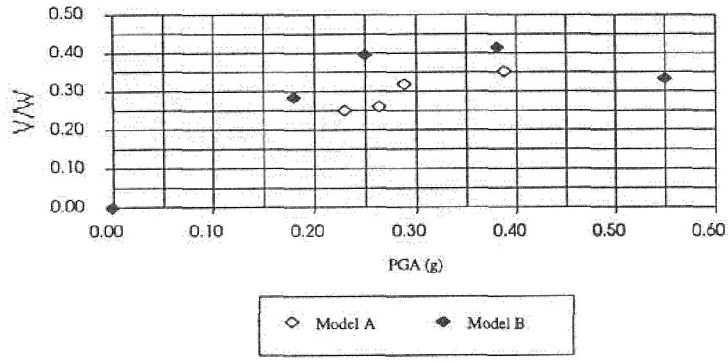


Figure 4. Normalised peak base shear vs. peak shaking table acceleration for models A & B.

Model B test results The natural frequency of the first mode of vibration for Model B was found to be 2.4 Hz with a damping ratio of 4.8% before any earthquake tests. The 4.8% damping ratio was somewhat higher than normal in a well-constructed reinforced concrete frame in an uncracked condition, indicating that some cracking may already have occurred in the structure at this stage.

Model B was then subjected to a sequence of four √5 time-scaled El Centro earthquake tests, starting with a PGA 0.18 g test and finishing with a PGA 0.55 g test which caused failure of the ground floor columns approximately 100 mm below the bottom of the first floor beams. The maximum story shear and acceleration profiles for each of these tests are shown in Figure 5. In contrast to the inertia force profiles for Model A, the inertia force profiles are relatively symmetrical which possibly reflects the minimal change in frequency and damping during the first three earthquake tests. However, the last earthquake test (PGA 0.55 g) caused bending failure of the ground floor columns, resulting in the irregular inertia profile shown in Figure 5.

To estimate the value of maximum base shear force which would cause yielding of the structure, the normalised peak base shear force (V/W) was plotted against peak shaking table acceleration (PGA) and is shown in Figure 4. As was observed for the results of Model A, the curve is nearly linear for the tests with PGA $\leq$ 0.30g, however, the peak base shear measured in last test (PGA 0.55g) deviated significantly from the line through the first three data points in Figure 4. In fact, this test was terminated prematurely as the main reinforcing bars in each of the ground floor columns failed in bending (Figure 6). It may be assumed that the peak base shear required for yielding and failure of the model was less than the 0.55 g recorded during this test. The test left the structure in a greatly weakened state and no further testing was possible.

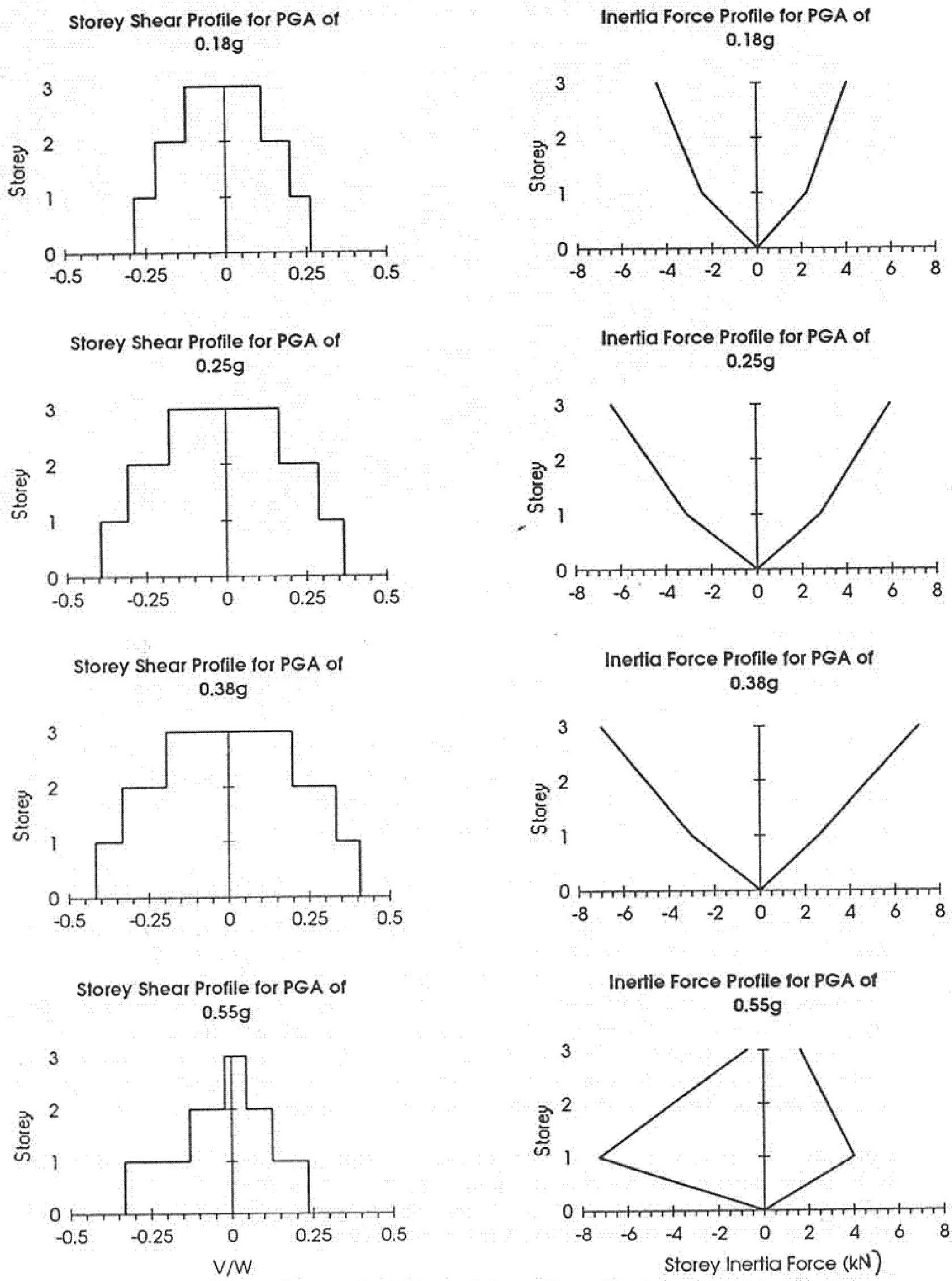


Figure 5 - Model B Test Results

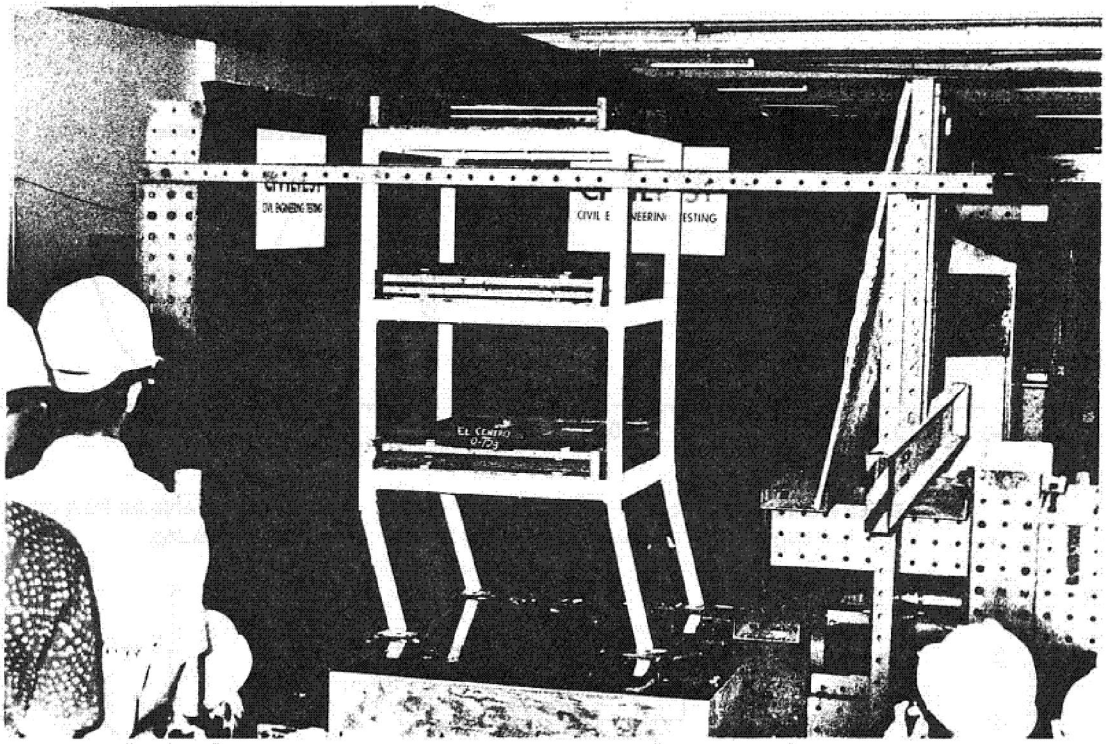


Figure 6.- Ground floor column failures after PGA 0.55 g test.

Discussion of test results

The behaviour of the lightly reinforced concrete frames was particularly good in the elastic range of response which for the models appeared to be for earthquake input having a PGA ≤ 0.25 g. The magnitude of acceleration required to initiate yielding in the model was greater than might normally be attributed to Australian buildings and was a consequence of the inherent strength of buildings designed to resist vertical dead and live loads where earthquake loads are not the critical design consideration.

On the other hand, substantial discrepancies were observed between the so-called overstrength factor reported by other researchers and those calculated from these test results. The `over-strength' factor Ω is defined as the ratio of the actual structural yield level divided by the code-prescribed yield level. An over-strength value $\Omega = 2.3$ has been reported by Uang (1991) for ductile moment frames in the United States. Estimates of $\Omega = 1.0$ and 1.6 were made for Models A and B, respectively, by dividing the bending moment in the ground floor columns corresponding to the yield level shear force by the code-predicted yield level moment. This large discrepancy clearly warrants further investigation to determine whether the difference is due to significantly different levels of ductility between American and Australian concrete frame structures.

Comparing the plots of normalised peak base shear (Figure 4) for Model A and Model B, it can be seen that the AS 3600 detailing requirements for ductile frames in earthquake zone 1 do not enhance the strength substantially, although they do appear to provide for some increased ductility over the zone A details.

Finally, the results of these tests indicate quite clearly that the failure mode is a catastrophic one. Since the member sizes are controlled by the vertical dead and live loads and column spacings are commonly stretched to the limit, Australian moment frames tend to have `strong beams' and `weak columns', thereby ensuring such a failure mode.

Conclusions

The research reported here involved earthquake simulator testing of two three-storey reinforced concrete frame scale model structures and showed that satisfactory results can be obtained through small-scale model testing using existing facilities of the Civil Engineering Department's structural testing laboratory. In addition, and more importantly, the following conclusions can be drawn based on the results of these tests:

(1) both model structures behaved well at elastic levels of earthquake loading;
(2) both models were at least as strong as the code calculations predicted, with the overstrength factor Ω for Model A being about 1.0 and Ω for Model B being about 1.6. However, both overstrength ratios were significantly lower than American ratios reported for ductile frame structures, implying that Australian detailing levels may not be comparable with those used in the United States, at least with regard to earthquake loads;
(3) both structures exhibited only limited amounts of ductility with plastic hinges forming in the ground floor columns leading to eventual catastrophic collapse of the structures.

Clearly, the collapse mechanism which is possible in this type of structure is not desirable. Unfortunately, structures in low to moderately active earthquake zones tend to have `weak' columns and `strong' beams. Methods of detailing which would prevent this and still allow for large span frames must be addressed. A second area requiring further investigation concerns the question of ductility and the corresponding energy dissipation capacity of lightly reinforced concrete structures, including moment resisting frames. A study is currently in progress at the University of Adelaide to investigate the second of these questions. It is hoped that the answers to these questions will enable engineers to better design concrete structures to resist earthquake forces in Australia.

Acknowledgments

The research reported herein was conducted as a final year student project and was supported by The University of Adelaide's Civil Engineering Department, the Cement and Concrete Association, and the Steel Reinforcement Promotion Group. The views and conclusions expressed in this report are those of the author and not necessarily those of the sponsors.

The author would like to thank the students (Messrs. P. Bayliss, J. Doherty, S. Reynolds, D. Tonge, and J. Walters) for their invaluable contribution toward the project. The technical staff of the Civil Engineering Department also provided valuable assistance and advice over the course of the experimental work. Finally, thanks must go to Mr. Doug Amey of the Cement and Concrete Association and Mr David Tubb of Aquila Steel for their assistance in providing the materials necessary for building the model structures.

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