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Experimental study of shear wall and infilled frame on shake-table

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ABSTRACT: Two 3-d models of 4-storey high with 1:3 scale were tested on a 5m × 5m shaking table. One of the models tested was a reinforced concrete shear wall structure, while the other was a masonry infilled reinforced concrete frame structure with the same overall dimensions. The two models were designed to have similar static strengths and verified by static tests on 2-storey models. During the shake table tests, the models were subjected to increasing magnitudes of El-Centro earthquake excitations, from elastic stage initially until eventual failure of the structures. Their dynamic responses and seismic resistances were studied and compared to evaluate their relative effectiveness as earthquake resistant structures.

1 INTRODUCTION

Both shear walls and infilled frames have been used extensively in tall building structures (Beedle and Iyengar 1982) to resist lateral loads arising from winds and earthquakes. Each of these two types of construction has its own merits. Generally, a shear wall has higher lateral rigidity and strength. A high lateral rigidity is desirable when wind effects are considered. But during earthquakes, a higher lateral rigidity would lead to a higher seismic load on the structure because a stiffer structure tends to attract more seismic energy. Infilled frame, on the other hand, incorporates a more flexible frame and therefore attracts less seismic energy. Moreover, an infilled frame can dissipate significantly more energy (Liauw and Kwan 1985) because of the structural interaction at the panel-frame interfaces in the form of shear and slip, and local crushing and cracking of the infilled panels (Liauw and Kwan 1984). Another consideration, which is equally important, is that damages to shear walls during earthquakes tend to occur at the roots of the walls. Such damages could be very difficult to repair as the walls are carrying vertical loads. On the other hand, since damages to infilled frames are normally limited within the panels and the vertical loads are carried by the frame rather than by the panels, repairs to the structures after earthquakes are relatively easy. However, there has been the concern on the safety of infilled frames when brickworks are used for the infilled panels because the panels may fall out of plane when subjected to real seismic excitations.

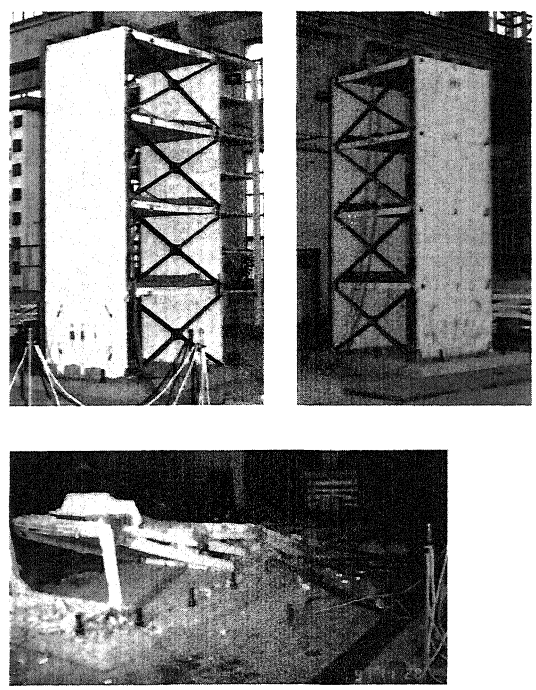

Fig. 1 From top left clockwise (a) Model 1 (b) Model 2 (c) Collapse of Model 2

There were separate studies in the past on these two different constructions, but no direct comparative study on them under seismic loads has ever been published. It is, therefore, interesting to compare experimentally a shear wall to an infilled frame of similar static strength when they are subjected to the same earthquake spectrum. This paper reports the experiment on two 1:3 scale 4-storey models with the following vertical structures: (1) a pair of solid reinforced concrete shear walls, and (2) a pair of reinforced concrete frames infilled with brick masonry. They were designed to have the same static lateral strengths, which were verified by static tests on 2-storey models of similar constructions. The two 4-storey models were tested on a 5 m x 5 m shake table, initially to test their elastic responses when subjected to scaled seismic spectrum and then to failure by gradually increasing the seismic loading.

2 THE MODELS AND THE EXPERIMENT

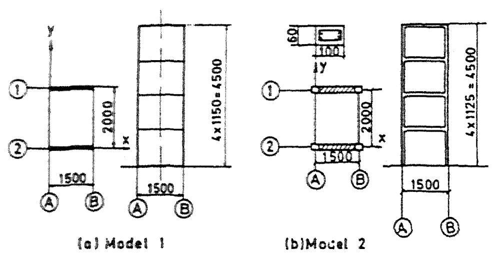

Fig. 2 Parameters (mm) of models

The two models tested are shown in Fig. 1. The structural parameters, Fig. 2, were chosen so that the models resemble a typical portion of a four storey building at 1/3 reduced scale. Each model consisted of a pair of parallel structures: either a pair of reinforced concrete shear walls (for Model 1) or a pair of reinforced concrete frames with brickwork infills (for Model 2). At each floor level, the two parallel structures were connected together by a reinforced concrete floor slab of 40mm thick. The shear walls of Model 1 were cast of 40 mm thick concrete with a cube strength of 10.2 MPa at the time of testing. Each shear wall was reinforced with seven nos. of 4 mm diameter mild steel bars with a yield strength of 238 MPa. The infilled frames of Model 2 were made of reinforced concrete frames and brickwork infills. Both the beams and the columns of the frames had the same section of 60 mm x 100 mm of concrete with a cube strength of 12.5 MPa at the time of testing, and reinforced with four nos. of 8 mm diameter mild steel reinforcement bars with a yield strength of 238 MPa. The brickworks for the infills were laid of model clay bricks with a dimension of 19 mm x 39 mm x 80 mm. The brick units have a compressive strength of 13.7 MPa and the mortar used for the joints has a compressive strength of 2.2 MPa. No connectors were provided to bond the brickwork panels and the frame together. The two models were designed to have approximately the same static lateral strength under the loading shown in Fig. 3. Separate static tests on the 2-storey models, revealed that the shear wall and the infilled frame models actually achieved static strengths of 72.5 kN and 56.8 kN respectively.

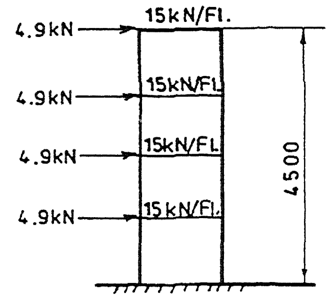

Fig. 3 Design static loads

Dynamic tests of the models were carried out on a shake table of 5 m x 5 m. Although the table can be excited in three horizontal degrees of freedom (i.e., two translations in perpendicular directions and one rotation about the vertical axis), only seismic excitations along the plane of the wall structures of the models were input to the table so that the models were subjected only to seismic loads within the planes of the wall structures. However, lateral effect amounting to 10~15% of the input loads was produced. The El Centro (1940,5,18S-E) seismic records were used as seismic excitations to the models. To account for the scaling factor of the models, the time scale of the seismic wave was compressed by a time factor of 1/1.732.

The shear wall model weighed 26.4 kN by itself. 15 kN of dead weight in the form of steel ingots were added on each storey of the model making the total weight 86.4 kN. On the other hand, the infilled frame model weighed 30.0 kN and after adding dead weight of 15 kN on each storey, the total weight of the model was 90.0 kN.

The responses of the models under the seismic excitations were measured by two accelerometers and one displacement

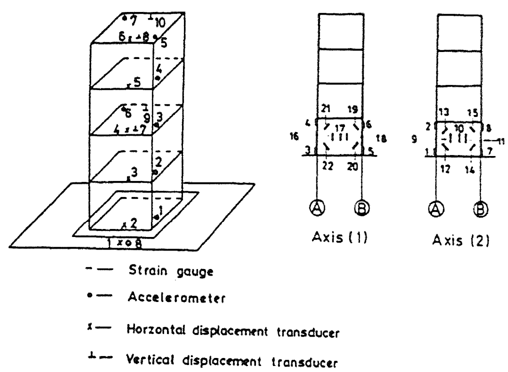

Fig. 4 Locations of instruments and strain gauges

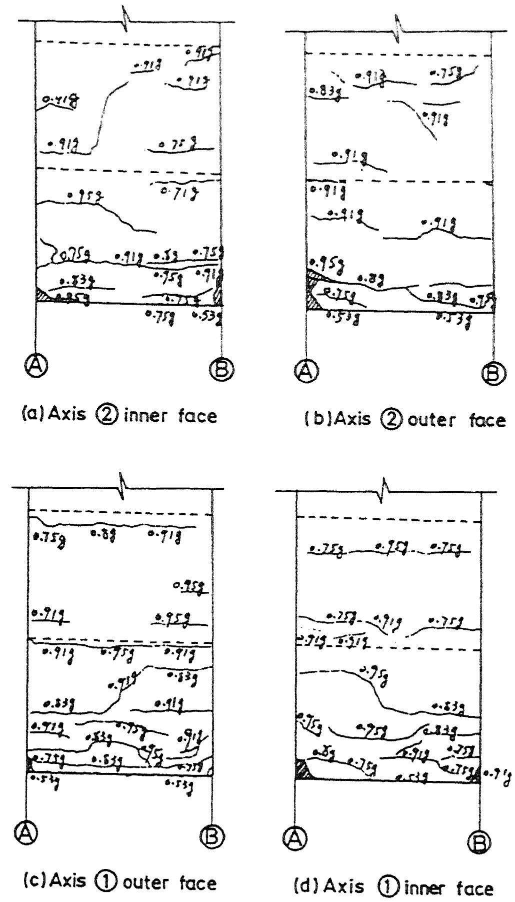

Fig. 5 Crack patterns of shear walls

tranducer on each storey, Fig. 4. For reference purposes, the responses of the table itself were also measured. Strain gauges were installed onto the steel reinforcement bars at the lowest stories of the models. All the transducer signals were logged by microprocessor controlled data-loggers and recorded in either analog or digital form by magnetic tapes.

During the shake table tests, increasing magnitudes of El-Centro earthquake excitations were applied to the models. Initially, a peak acceleration of 0.2 g was applied to the models. Then the models were subjected to increasing excitations by small increments of peak acceleration. Before and during intermediate stages of the shake table tests, the vibration modes, natural frequencies and damping ratios of the models were also measured and recorded to evaluated the changes in the structural characteristics. These are not included in this paper due to limited space.

3 RESULTS AND DISCUSSIONS

3.1 Damage characteristics of models

The cracks patterns are shown in Fig. 5. Shear wall (Model 1): The shear wall model started to crack at the corners of the lower two stories when the peak acceleration reached 0.38 g. From 0.53 g peak acceleration onwards, the cracks gradually extended towards the centroidal axis of the walls. When the peak acceleration reached 0.75 g, the cracks developed to such an extent that horizontal cracks cutting through the whole width of the walls were formed. However, the structure continued to withstand even stronger earthquake loads after the formation of such cracks. At 0.83 g peak acceleration, signs of concrete crushing and steel yielding at the roots of the walls were obvious. At the lower corners of the walls, the concrete had crushed to such extent that the steel reinforcement bars were exposed and the exposed reinforcement appeared to have fail laterally because of dowell action. Although the structure had been damaged seriously after the 0.83 g peak acceleration, it had not collapsed. In order to see if the structure could sustain a higher load, the peak acceleration was further increased to 0.95 g; but the model remained standing without collapse albeit being damaged very badly. The test ended at 0.95 g peak acceleration. Inspection of the shear wall structure after the test revealed that the shear walls were badly damaged by shearing at the bases of the walls. This failure mode was quite different from that of failing by bending under static load. Hence shear walls may fail under seismic load with a failure mode different from that under static load.

Infilled frame (Model 2): The infill frame model remained intact after 0.20 g and 0.30 g peak accelerations were applied to the structure. Separation between the infilled panels and the frame at the tension corners was obvious during the tests, but the infilled panels and the frame worked together as a composite structure. At 0.41 g peak acceleration, some fine horizontal and diagonal cracks appeared in the brickwork infills. At the same time, the gaps between the infilled panels and the frame had widened. When the peak acceleration reached 0.50 g, more diagonal cracks appeared in the brickwork panels. The frame, however, remained in good conditions. At 0.66 g peak acceleration, cracks started to appear at the lower and upper ends of the columns in the first storey, while the cracks in the brickwork panels widened dramatically indicating that the brickworks were very close to the point of total disintegration. However, when the peak acceleration was further increased to 0.81 g, the structure did not collapse, although the brickwork panels were very seriously damaged and one plastic hinge was formed at the top of a column of the first storey. Moreover, a serious situation was created, in which a masonry panel at the lowest storey was shifted slightly out of the plane of the frame by the lateral effect of the seismic load. Finally, when a peak acceleration of 0.835 g was applied to the structure, the brickwork panel which was shifted sideway ealier fell out-of-plane and the frame collapsed, bringing the floor slabs down together with the other frame.

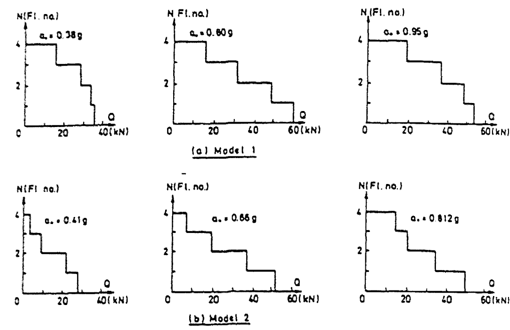

Fig. 6 Shear force profiles.

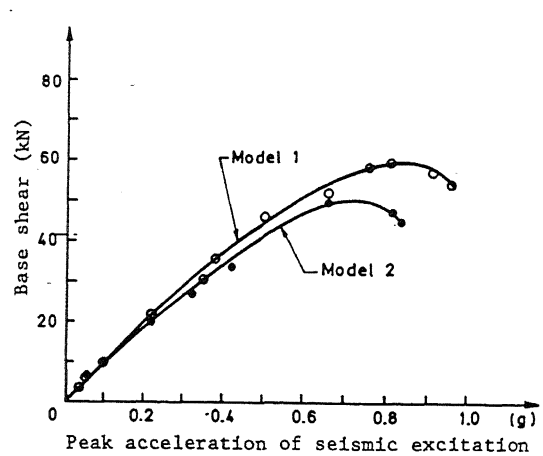

Fig. 7 Relation between base shear and peak acceleration input

3.2 Seismic resistance of models

From the acceleration response of the models, the shear force profiles in the structures under seismic excitation were evaluated as shown in Fig. 6. The variations of the base shears with the peak accelerations of the seismic excitation are shown in Fig. 7, which shows that in Model 1, a maximum dynamic base shear of 59.8 kN was reached at a peak acceleration of 0.83 g beyond which the structure may be considered to have reached the ultimate limit state. The equivalent static shear (it is taken as the total dead weight times the peak acceleration of the seismic excitation) corresponding to this acceleration was 86.4 kN × 0.83 = 71.7 kN. Comparing the dynamic base shear with the static shear, the dynamic base shear was 59.8 / 71.7 = 83 % of the static shear. In Model 2, a maximum base shear of 50.5 kN was reached at a peak acceleration of 0.66 g. The corresponding equivalent static shear was 90.0 kN × 0.66 = 59.4 kN. Hence the dynamic base shear was 50.5 / 59.4 = 85 % of the static shear. It may be said, therefore, that for this particular case, the two types of structures attracted more or less the same amount of dynamic loads. Comparison of the maximum dynamic base shears of the models with their static strengths revealed that the maximum base shears attained during earthquakes were only 59.8 / 72.5 = 83 % and 50.5 / 56.8 = 89 % of the static strengths for the shear wall model and the infilled frame model respectively. It is interesting to note, however, that the equivalent static base shears agree fairly closely with the static strengths: in Model 1, the static base shear was 99 % of the static strength, while in Model 2, the static base shear was 105 % of the static strength. Model 2 appeared to have withstood a marginally higher equivalent static base shear relative to the static strength than Model 1. Table 1 summarizes the afore-mentioned static and dynamic base shears.

Table 1 Static and dynamic strengths of the models.

Static and dynamic strength	Model 1	Model 2
max. base shear under seismic excitation	59.8 kN	50.5 kN
peak acceleration at which max. base shear occured	0.83 g	0.66 g
equivalent static base shear (dead weight times acceleration)	71.7 kN	59.4 kN
static strength	72.5 kN	56.8 kN

At the end of the tests, Model 1 did not collapse, but Model 2 collapsed because of a brickwork panel falling out-of-plane of the wall structure. It verifies the worry of some people about the out-of-plane failure of infilled frame structures wherein no connectors are provided to bond the infilled panels and the frames together. The shear walls are, on the other hand, not without problems. As observed from the shake table test of the shear wall model, shear walls tend to fail by shearing under seismic loads rather than by bending as in the static cases. It seems that the shear strengths of shear walls under cyclic loads could be significantly lower than those under static loads. More care in designing for the shear resistances of shear walls is required.

4 CONCLUSIONS

Shake table tests were carried out on a reinforced concrete shear wall model and a brickwork infilled reinforced concrete frame model to study their behaviours under seismic excitations, in particular their failure characteristics and shear resistances under strong earthquakes. A comparison has been attempted on their relative effectiveness as earthquake resistant structures. Since the seismic resistances of the two structures differ very little after taking account of their equivalent static shears or their static strengths, and the study was based on only a pair of models, it would be prematured to draw a definite conclusion on which one as a better earthquake resistant structure.

The study did show that both shear walls and infilled frames are quite effective in resisting earthquake loads. However, they are afflicted by the following problems. For the case of shear walls, the shear strengths of the walls under seismic excitations appear to be significantly lower than those under static loads, as indicated by the fact that the shear wall model failed under seismic load by shearing instead of bending as in the static case. Hence the failure modes of shear walls under strong earthquakes may be different from those observed from static tests. More care in designing for the shear resistances of shear walls seems to be required. For the case of infilled frames, the major problem is the possibility of the infilled panels falling out of the plane of the structures, especially under real earthquakes when there are multi-directional vibrations. It would seem prudent to provide shear connectors to bond the infilled panels with the frames and to prevent the infills from falling out of plane.

5 ACKNOWLEDGEMENT

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