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Shake-table tests of large-scale shear wall and infilled frame models

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- **In order to evaluate and compare the seismic performance of different types of wall construction, three 1/3 scale four-storey models of structural walls were tested dynamically to ultimate failure by subjecting each of them to a sequence of simulated earthquakes of progressively increasing magnitude on a 5 m × 5 m shaking table. The three models tested are respectively: a reinforced concrete shear wall structure; a masonry infilled reinforced-concrete frame structure; and a concrete infilled steel frame structure. They have the same overall dimensions and were designed to represent typical wall constructions. Before and at intervals during the shake-table tests, the models were also subjected to small amplitude vibration tests to measure their dynamic properties and the corresponding changes as the structures degraded. From the shake table and small amplitude vibration tests, the seismic behaviour, damage characteristics, structural degradations and dynamic shear strengths, etc., of the models are studied and compared. Two problems with the structural forms investigated are identified, and further studies are recommended.**

Introduction

Early day structural forms

In the early days, when buildings were relatively short, most buildings were constructed as beam-column frame structures. Frame structures can be designed easily to have sufficient ductility to sustain the large distortions to which they would be subjected during earthquakes. However, they are quite flexible and, although the structures themselves can usually survive the earthquakes with only minor damage, the large inter-storey distortions resulting from the high flexibility often lead to serious damage to the non-structures which may comprise more than half of the total value.¹ Moreover, the structural efficiency of a pure frame system is rather low and, as a result, for taller buildings, the required frame member sizes might become so big that the structural form would be impractical.² There are two common methods to reduce inter-storey drift and to increase structural efficiency: the first is to incorporate shear walls into the structural system; the second is to infill the frames

with masonry or concrete panels to form infilled frame structures. Both shear walls and infilled frames have been used quite extensively in tall buildings³⁻⁵ and each of these two types of wall construction has its own merits.

Shear walls

2. Shear walls are basically vertical cantilevered beam structures capable of taking both vertical and lateral loads. They have much higher lateral stiffness and strength than frames, and can be designed to have very good structural efficiency. Since they are more effective in restricting inter-storey distortions, they can provide much better damage control than frame structures.^{1,6} On the other hand, owing to their relatively high lateral stiffness, they also tend to attract larger amounts of seismic energy, resulting in larger seismic loads to be resisted. Fortunately, the larger seismic loads are not necessarily associated with larger stresses because of the higher efficiency of the structural system.^{1,6}

3. Although shear walls are often regarded and designed as deep reinforced-concrete beams, there are actually two important differences between shear walls and deep beams.^{7,8} Firstly, deep beams are normally supported at their bottom faces and loaded on top. Under such conditions, shear is carried mainly by concrete arch action. In contrast, shear walls are generally loaded through floor diaphragms and thus the applied loads are distributed along the wall sections. In such case, no effective arch action can develop. Secondly, deep beams are not normally subjected to axial loads, whereas the consideration of axial compression or tension may be important in shear walls. On account of these differences, ordinary deep beam theories should not be applied indiscriminately to shear walls.

4. Existing design methods⁹⁻¹¹ for estimating the flexural and shear strengths of shear walls are similar to those being applied to beams, and are based mostly on the 'non-linear beam theory' and the 'truss analogy' respectively. The non-linear beam theory treats the shear wall as a beam, assumes a linear axial strain distribution and uses the non-linear uniaxial stress-strain relations of the materials (the materials are assumed to be under uniaxial stress condition) for flexural strength analysis. On the other hand, the truss analogy assumes that the concrete can take up a portion of the shear and the shear in excess of that carried by

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the concrete alone is borne by the horizontal reinforcement as if the wall were an idealized truss structure. Unlike beams which would fail only in flexure or shear, shear walls may also fail by sliding along horizontal construction joints or slab-wall joints.^{7,12} This mode of failure has been analysed by the 'shear-friction theory' which postulates that the horizontal reinforcement is ineffective in preventing sliding shear failure, but the vertical reinforcement will contribute to the sliding shear strength by providing clamping forces to resist sliding.

5. Recent works have, however, revealed that the existing design methods do not accord strictly with the actual behaviour of shear walls. For instance, it has been found¹³ that the tested flexural strengths of shear walls are often significantly higher than predicted by the non-linear beam theory and such discrepancies can be explained only by considering the tri-axial stress conditions in the compression zones of the walls. Furthermore, it has been shown^{13,14} that the wall shear capacity as predicted by the truss analogy concept often overestimates considerably the wall shear capacity established by experiment and that the contribution of the horizontal reinforcement to shear capacity is much smaller than expected. It would seem, therefore, that the present code provisions should be revised¹⁵ and that further research is urgently needed.

6. Another important aspect, which somehow is often not considered in the aseismic design of shear walls, is that of alternating near ultimate loading.⁶ Observed earthquake damage suggests strongly that this is, in fact, a common occurrence. An excursion into the post-elastic range may lead to considerable residual strains in the tension reinforcement, thereby causing large cracks to be developed in the surrounding concrete which would remain open after the removal of the loads. This phenomenon can substantially reduce the shear capacities of the walls. However, up to now, most tests on shear walls were carried out statically. In order to study the true behaviour of the shear walls during earthquakes, more cyclic load tests, or even dynamic tests, should be carried out.

Infilled frames

7. Infilled frames are basically beam-column frame structures infilled with masonry or concrete panels. Acting compositely, the infilled panels greatly stiffen the frame, and when the infill panels crack, the frame prevents disintegration so that the infilled frame structure may resist substantially higher load before final collapse. Furthermore, progressive cracking and crushing of the infilled panels increase the deflection at which the structure fails and hence the ductility of the structure. As a result,

an infilled frame structure is stiffer and stronger than the frame plus the infilled panels acting independently, and is more ductile than the infills. Since infilled frames incorporate more flexible frames, they attract less seismic energy than shear walls. Moreover, infilled frames can also dissipate more energy because of shear and slip at the infill-frame interfaces, and local crushing and cracking of the infilled panels.

8. One other advantage of using infilled frames is that earthquake damage to infilled frames is relatively easier to repair than that to shear walls. Earthquake damage to shear walls tends to occur at the lower parts of the walls and is generally very difficult to repair because the walls are also carrying vertical loads. On the other hand, since damage to infilled frames is normally limited within the infilled panels and the vertical loads are carried by the frame members rather than the infills, repairs to infilled frames after earthquakes can usually be carried out without temporary supports. Nevertheless, there has also been concern on the safety of infilled frames when brickwork is used for the infilled panels because the panels may fall out of plane when subjected to seismic excitations normal to the panels.

9. Studies on infilled frames started even earlier than those on shear walls. In fact, the earliest 'shear wall' models tested were really infilled frames.¹⁶ Investigations of simple infilled frames, where the infills are not connected to the frames, have shown increases in lateral stiffness and strength of the structures as the infills act as diagonal bracing struts.^{17,18} If the infills are firmly connected to the frames, either by means of connectors or as monolithical structures, the stiffness and strength can be increased further.^{19,20} There are therefore two types of infilled frame: non-integral infilled frames in which the infilled panels are not connected to the frames; *integral infilled frames* in which the infilled panels are connected to the frames.

10. The lateral stiffness of non-integral infilled frames may be estimated by the 'equivalent diagonal strut method' which replaces the infilled panels with equivalent diagonal struts whose equivalent width may be taken as 1/3 of the diagonal length of the panels¹⁷ or more precisely established by experiment, taking into account the various structural parameters.²¹ On the other hand, the 'equivalent frame method' has been developed to estimate the lateral stiffness of integral infilled frames. Briefly, this method treats the portions of the panels bonded to the frame members as parts of the frame members, thus increasing the sizes of the frame members to reflect the stiffening effects of the infilled panels.²²

11. With regard to strength prediction, modern theories are all based on plastic

analysis. The first plastic theory was developed by Wood²³ in the 1970s. However, in this theory, non-integral infilled frames and integral infilled frames are not differentiated from each other and an empirical penalty factor has to be used to account for the wide discrepancies between theoretical values and experimental results. Later on, in the 1980s, Liauw and Kwan^{24,25} developed two separate plastic theories for non-integral and integral infilled frames. Their plastic theories take into account separation and slip at the infill-frame interfaces in the case of non-integral infilled frames and the finite shear strength of the connectors in the case of integral infilled frames, and have been shown to yield good agreement with the experimental results without the use of any empirical factors.

12. Both the monotonic and cyclic behaviour of infilled frames have been studied quite extensively.²⁶⁻²⁹ However, there have been relatively few dynamic tests. Hence, further research work should be concentrated on dynamic testing and analysis.

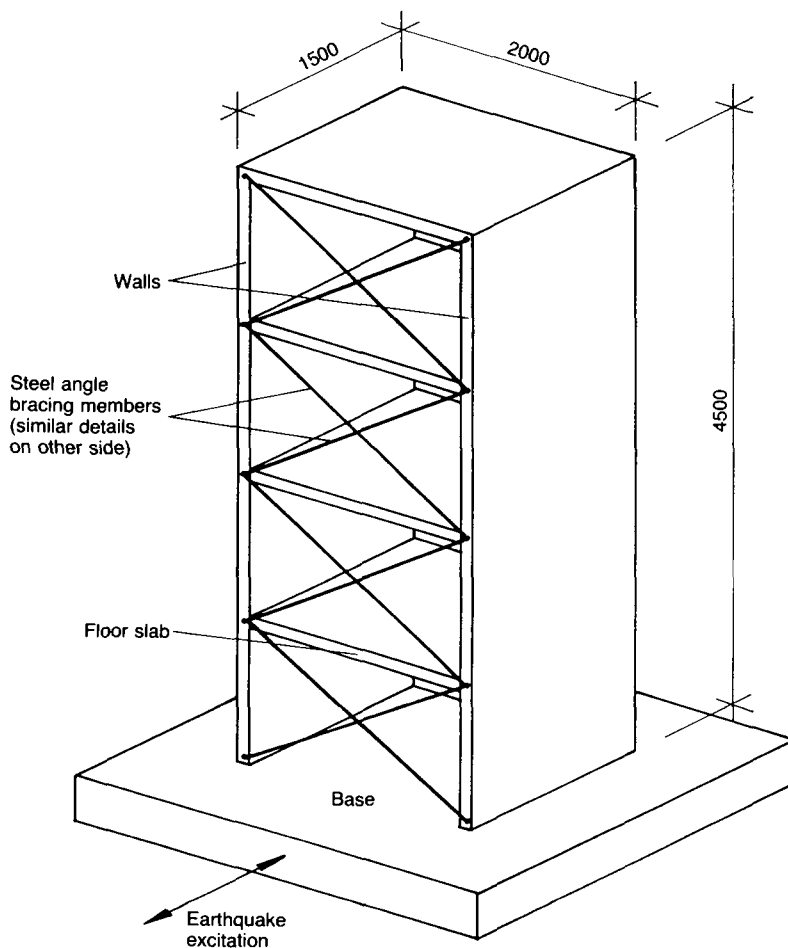
Present study

13. While there were separate studies in the past of shear walls and infilled frames, no

direct comparative study on them under seismic loads have ever been published. It is, therefore, interesting to compare experimentally shear walls to infilled frames of similar overall dimensions when they are subjected to earthquake motions. This Paper reports the experiment on three 1/3 scale four-storey models with the following vertical structures: a pair of reinforced concrete shear walls; a pair of reinforced concrete frames infilled with brick masonry; and a pair of steel frames infilled with concrete. They were tested on a 5 m × 5 m shaking table at the Institute of Engineering Mechanics, State Seismological Bureau of China.

14. The models were tested by subjecting them to a sequence of simulated earthquake motions of progressively increasing magnitude until failure. Before and at intervals during the shake-table tests, the models were subjected to a set of diagnostic tests to measure their dynamic properties and the corresponding changes when the models degraded. Another identical set of models was also constructed and tested for static behaviour so that the seismic performance of the models could be evaluated and compared relative to their respective static strengths

Fig. 1. General layout of models (all dimensions in mm)



Details of models tested

15. In Fig. 1, the general layout of the three models tested is shown. The structural parameters were chosen so that the models resemble typical portions of four-storey buildings at 1/3 reduced scale. Each model consisted of a pair of identical and parallel walls interconnected at the floor levels by concrete slabs. The three models tested are

- model 1: a reinforced concrete shear wall structure
- model 2: a masonry infilled reinforced concrete frame structure
- model 3: a concrete infilled steel frame structure.

The models have the same overall dimensions as follows: width of wall = 1.5 m; spacing between the two parallel walls = 2.0 m; storey height = 1.125 m; overall height = 4.5 m. The height to width ratios of the walls are equal to 3.0; therefore, the walls may be regarded as tall wall structures. All floor slabs were cast of 40 mm thick reinforced concrete. In order to increase the rigidity of the models in the direction perpendicular to the planes of the walls, two diagonal bracing members made of steel angles and tied to adjacent floors were provided at each side and at each storey of the models, as shown in Fig. 1. Each model was founded on a 200 mm thick reinforced concrete base where lifting hooks and fixing bolts were provided. The lifting hooks were for lifting the

models into place while the fixing bolts were for fixing the models onto the shaking table.

16. The shear walls of model 1, details of which are shown in Fig. 2, were cast of 40 mm thick concrete. Each wall was provided with vertical reinforcement consisting of sixteen 4 mm diameter mild steel bars evenly distributed in the wall section, and nominal horizontal reinforcement consisting of 4 mm diameter mild steel bars at a vertical spacing of 125 mm centres. Generous anchorage for the vertical reinforcement was provided by extending the reinforcement bars deep into the base. The walls were cast vertically storey by storey together with the floor slabs as in actual site construction. Material tests revealed that the cube strength of the concrete at the time of model testing was 10.2 MPa and the yield strength of the mild steel reinforcement was 238 MPa.

17. The infilled frames of model 2 (Fig. 3) were constructed of reinforced concrete frames and brickwork infills. Both the beams and columns of the frames have the same section of 60 mm × 100 mm and the same reinforcement details. The longitudinal reinforcement consisted of four 8 mm diameter mild steel bars, while the transverse reinforcement consisted of 4 mm diameter mild steel stirrups at 30 mm centre-to-centre spacing. As for the previous model, generous anchorage for the longitudinal reinforcement of the columns was provided by extending the reinforcement bars deep into the base. The frames were cast in situ storey by storey together with the floor slabs. Concrete cube strength at the time of model testing was 12.5 MPa. Yield strength of the mild steel reinforcement bars was 238 MPa. The brickwork for the infills consisted of model clay bricks with a dimension of 19 mm × 39 mm × 80 mm. Typical mortar joint thickness was about 2 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 together the brickwork panels and the frames. As such, this model is a non-integral infilled frame structure.

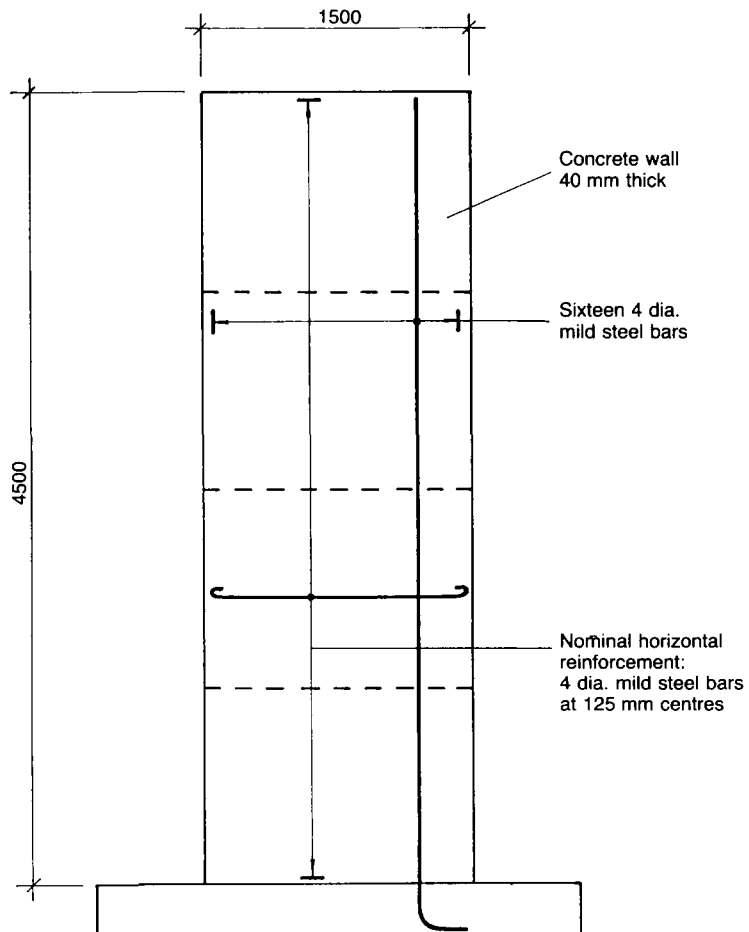
18. The infilled frames of model 3 (Fig. 4) were constructed by infilling steel frames with concrete panels. Both the beam and column members of the frames were made of 40 mm × 40 mm × 2.5 mm thick high yield steel square hollow sections. Originally, the frame members were designed as mild steel sections so that the static strength of the model would be similar to those of the other models, but owing to unavailability of suitable mild steel sections, high yield steel sections were finally used. Welding was used to join together the frame members, and the complete frame structure was connected to the base by four strong fixing bolts at each leg. The infilled concrete panels were 40 mm thick and contained nominal reinforcement,

comprising one layer of 4 mm diameter mild steel bars at 100 mm vertical spacing and 150 mm horizontal spacing. They were all cast in situ, with the steel frames at upright position. Material tests indicated that the cube strength of the infilling concrete was 4.3 MPa and the yield strength of the steel sections was 411 MPa. The infilled concrete panels were not bonded to the steel frames and, therefore, as for the previous infilled frame model, this model is also a non-integral infilled frame structure. Although no connectors were provided at the infill-frame interfaces, small steel plates were welded on to the frame members to prevent the infilled panels from falling out of plane.

19. Dead weight in the form of steel ingots were applied at the floor of each model to simulate the actions of vertical loads. The dead weight added on to each floor was 15.0 kN. On the other hand, the self-weight of models 1, 2 and 3 were measured to be 26.4 kN, 30.0 kN and 27.3 kN respectively. Added with the weight of steel ingots on each floor, the total weight, excluding the weight of the bases, of models 1, 2 and 3 were 86.4 kN, 90.0 kN and 87.3 kN respectively.

20. In order to measure the static strength of the models, another set of models made of

Fig. 2. Model 1: reinforced concrete shear walls (all dimensions in mm)



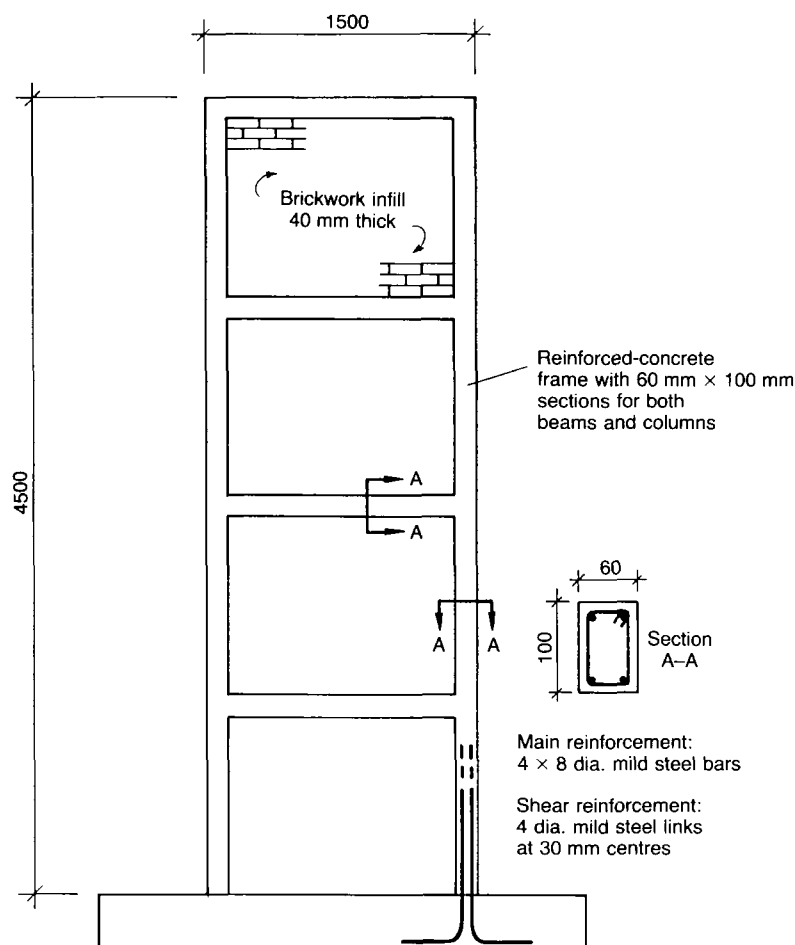


Fig. 3. Model 2: brickwork infilled reinforced concrete frames (all dimensions in mm)

the same materials and built by the same gang of workers was also constructed and tested statically. The same amount of vertical load as for the dynamically tested models was applied, and the models were tested by being subjected to monotonically increasing uniformly distributed lateral loads. The measured values of the total lateral loads when models 1, 2 and 3 failed were 72.5 kN, 56.8 kN, and 113.6 kN respectively.

Testing procedure

21. The simulated earthquake tests were conducted on the 5 m × 5 m shaking table of the Institute of Engineering Mechanics, State Seismological Bureau of China. This shaking table is currently the largest ever designed and built by China. It is hydraulically powered and is controlled by a MTS control system operating in conjunction with a VAX-11/730 computer. The shaking table is capable of operating within a frequency range of 0.5–30 Hz and the maximum excitation force that can be applied in one direction is 500 kN.

22. The models were each subjected to a sequence of earthquake tests. Although the shaking table can be excited in more than one

horizontal degree of freedom, only seismic excitations along the in-plane direction 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. The El Centro (1940.5.18:S-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 1/3. Initially, a simulated earthquake with a peak acceleration of 0.2 g was applied to the test model. Then the test was repeated several times, using simulated earthquakes of progressively increasing magnitude, with the peak acceleration of the excitation increased each time by approximately 0.1 g until the model failed or was no longer test worthy.

23. Before and after every two or three earthquake tests, a set of diagnostic tests were also carried out to measure the vibration modes, natural frequencies and damping ratios, etc., of the test model. This was to evaluate the dynamic characteristics and their changes as the model structure degraded. The diagnostic tests were conducted through the application of small amplitude sinusoidal excitations of fixed amplitude but varying frequency at the top of the test model so that the model was subjected to forced vibrations, whereupon the corresponding dynamic responses of the model structure were measured.

24. The horizontal acceleration and displacement responses of the models in the longitudinal direction (the direction parallel to the planes of the walls) were measured by two accelerometers and one displacement transducer on each storey. The accelerometers were located on the floor slabs adjacent to the walls while the displacement transducers were located at the centres of the floor slabs. In contrast, the transverse responses were measured by one accelerometer on the second floor and another on the fourth floor, which were both oriented perpendicular to the walls. In addition, vertical displacement transducers were also installed on the second and fourth floors. In Fig. 5 the overall arrangement of the accelerometers and the displacement transducers is shown. With regard to strain measurements, in model 1, strain gauges were installed on to the steel reinforcement bars at the wall–base junction. In model 2, the strain gauges were installed on to the vertical reinforcement in the columns of the lowest storey; while in model 3, the strain gauges were glued on to the surface of the steel column members at the lowest storey. Some other strain gauges were also fixed on to the concrete and masonry surfaces at the lowest storey. All the transducer signals were logged by microprocessor controlled data-loggers and were recorded on magnetic tapes for later data processing.

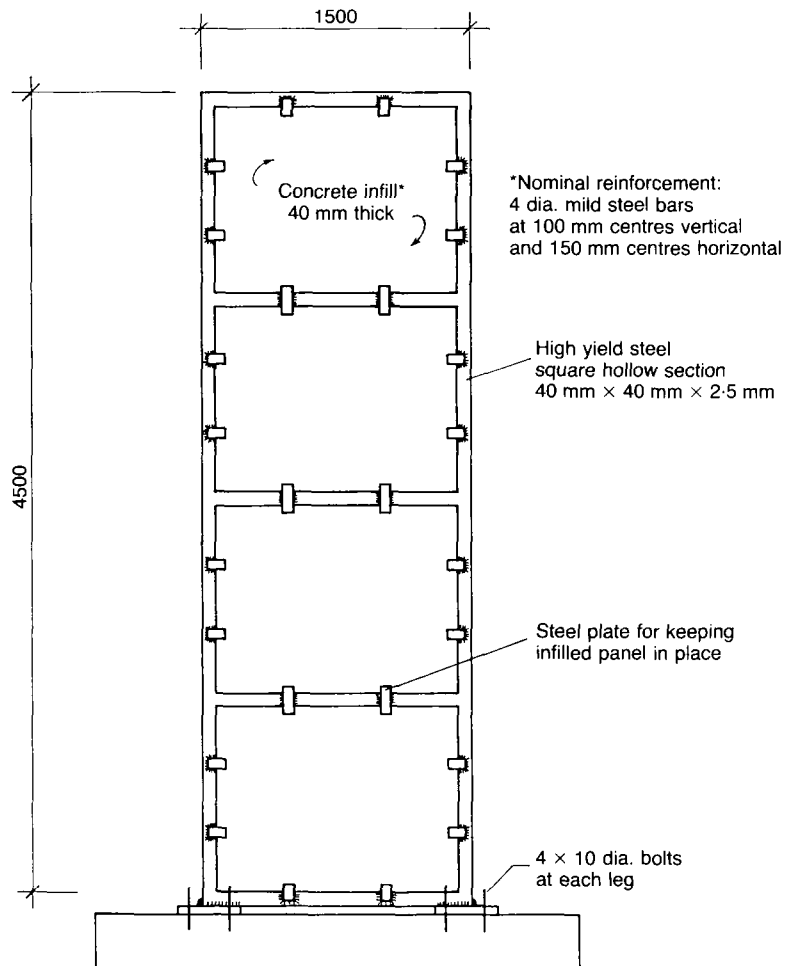
Seismic behaviour and damage characteristics

Model 1

25. Basically, the shear walls acted like vertical cantilevered beams during the earthquake tests. The walls started to crack at the corners of the lower two storeys when the peak acceleration of the applied earthquake reached $0.38 g$. As simulated earthquakes of progressively larger peak acceleration were applied, the previously formed cracks gradually extended horizontally towards the centroidal axis of the walls and more horizontal cracks appeared. 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 near the wall-base junctions. However, the structure continued to withstand even stronger earthquakes. After an earthquake with a peak acceleration of $0.83 g$ was applied, signs of concrete crushing and steel yielding at the roots of the walls were obvious. At the lowest corners of the walls, the concrete was so severely crushed that the reinforcement bars there were exposed. It could be seen that the vertical rebars at the roots of the walls had been shifted laterally because of dowel action and sliding of the walls relative to the base. Some of the vertical rebars there, particularly those near the edges of the walls, had even buckled. Although the structure had been seriously damaged after the $0.83 g$ peak acceleration earthquake test, it had not collapsed. In order to ascertain whether or not the structure could withstand a larger earthquake, the peak acceleration was increased further to $0.95 g$. After the $0.95 g$ earthquake, the model remained standing, although it was very badly damaged. The earthquake test was then ended. Inspection of the shear wall structure after completion of the earthquake tests revealed that the shear walls were badly damaged by shearing at the wall-base junctions. This sliding shear failure mode was quite different from the failure by bending that was observed during the static load test. Hence the failure mode of shear walls may be different under seismic load than under static load. In Fig. 6, the failure pattern of the model is shown.

Model 2

26. The infilled frame model remained intact after both $0.20 g$ and $0.30 g$ peak accelerations were applied to the structure. Separation between the infilled panels and the frames at the tension corners was obvious during the tests, but the infilled panels and the frames worked together as a composite structure, with the infilled panels acting as diagonal struts. 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 frames widened. When the peak acceleration reached $0.50 g$, more diagonal cracks appeared in the brickwork panels. The frames, however, remained in good condition. At $0.66 g$ peak acceleration, cracks started to appear on the columns of the lowest storey, while the cracks in the brickwork panels widened dramatically, indicating that the brickwork was very close to the point of total disintegration. However, when the peak acceleration was increased further 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 in 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 excitation. Finally, when an earthquake with a peak acceleration of $0.84 g$ was applied to the structure, the brickwork panel which was shifted sideways earlier fell out of the plane of the frame and the frame collapsed, bringing the floor slabs down together with the other frame. Among the models tested, this model was the only one which had totally collapsed during the earthquake tests. In Fig. 7, the failure pattern of the model before it collapsed is shown.

Fig. 4. Model 3:
concrete infilled steel
frames (all
dimensions in mm)

Fig. 5. Arrangement of accelerometers and displacement transducers

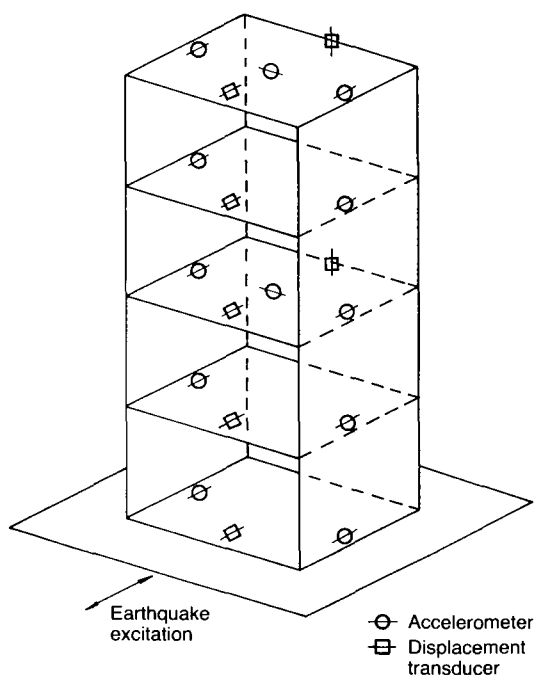
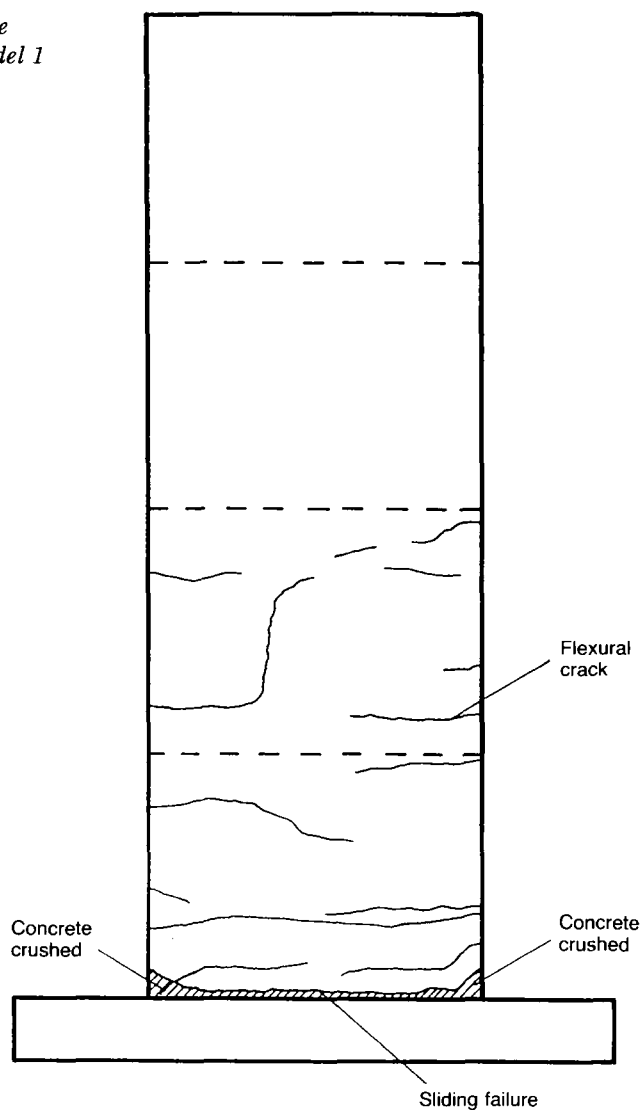


Fig. 6. Failure pattern of model 1



Model 3

27. Since the concrete infilled panels and the steel frames were not connected by any means, the infilled panels started to separate from the bounding frames during the first earthquake test when the peak acceleration applied was only 0.20 g. After the separation, the panels acted as diagonal struts, bracing the skeletal frame structure. When the peak acceleration reached 0.48 g, some fine inclined cracks began to appear on the infilled panels at the first and second storeys. At the same time, the gaps between the infilled panels and the frame members widened. When the peak acceleration reached 0.70 g, the previously formed cracks on the panels widened and more new diagonal cracks were formed. After a peak acceleration of 0.77 g was applied, the lower corners of the infilled panels at the first storey were crushed locally, leaving even wider gaps between the panel corners and the frames. However, the structure still worked as a composite structure, with the concrete panels acting as diagonal struts. When the peak acceleration was increased further to 0.94 g, all corners of the first storey concrete panels were crushed. Moreover, the column members in the first storey were bent permanently outwards, indicating the formation of plastic hinges at the ends of the columns. Some fine cracks also started to appear on the infilled panels at the third storey. When the peak acceleration reached 1.18 g, the previously formed cracks widened, more cracks appeared, all corners of the infilled panels in the first storey were very badly damaged and the permanent deformations of the first storey columns became very large, but the integrity of the structure still remained good. At this stage, the corners of the infilled panels on the second storey also started to crush. Furthermore, it was observed that the infilled panels in the lowest storey were rocking against the columns of the bounding frames during the earthquake test. In order to investigate how much more peak acceleration the structure could withstand, the peak acceleration was increased further to 1.27 g and then to 1.50 g. After these two earthquakes were applied, the structure was very badly damaged, but it did not collapse. It appeared during these two earthquake tests that the infilled panels at the first storey had lost the bracing capability and, as a result, the lowest storey had become a soft storey, thereby isolating the structure from the base motions. The earthquake test was then stopped. In Fig. 8, the failure pattern of the model is shown.

Dynamic characteristics

28. The first natural frequencies of vibrations in the in-plane direction of the walls and the corresponding damping ratios of the models as measured by the small amplitude forced

vibration tests before and at various stages during the earthquake tests are listed in Table 1. Since the models had roughly the same weight, the natural frequency values may be regarded as an indirect measure of the lateral stiffness of the models and the change in natural frequencies as an indication of the degree of stiffness degradation of the models when subjected to earthquake damage. From the natural frequency results tabulated in Table 1, it is seen that, initially, model 2 had the highest lateral stiffness, model 1 had a slightly lower stiffness than model 2, and model 3 had the lowest stiffness. The natural frequencies of the models gradually decreased as they degraded during the earthquake tests. Model 1 had the smallest variation in natural frequency throughout the earthquake tests, while the natural frequencies of models 2 and 3 decreased fairly rapidly as the peak accelerations of the simulated earthquakes applied to them increased. Eventually, after the models had been subjected to moderate earthquake damage, models 2 and 3 had much lower natural frequencies than model 1; model 3, in particular, had its first natural frequency at the failure stage reduced to only 0.27 of the initial value. Observations made during the earthquake tests suggested that the faster rates of stiffness degradation of the infilled frame models (models 2 and 3) were attributable to crushing of the corners of the infilled panels which led to gradual reduction of the bracing action of the panels.

29. The damping ratios of the models were obtained from the small amplitude forced vibration tests by using the half-power (frequency bandwidth) method.³⁰ From the results tabulat-

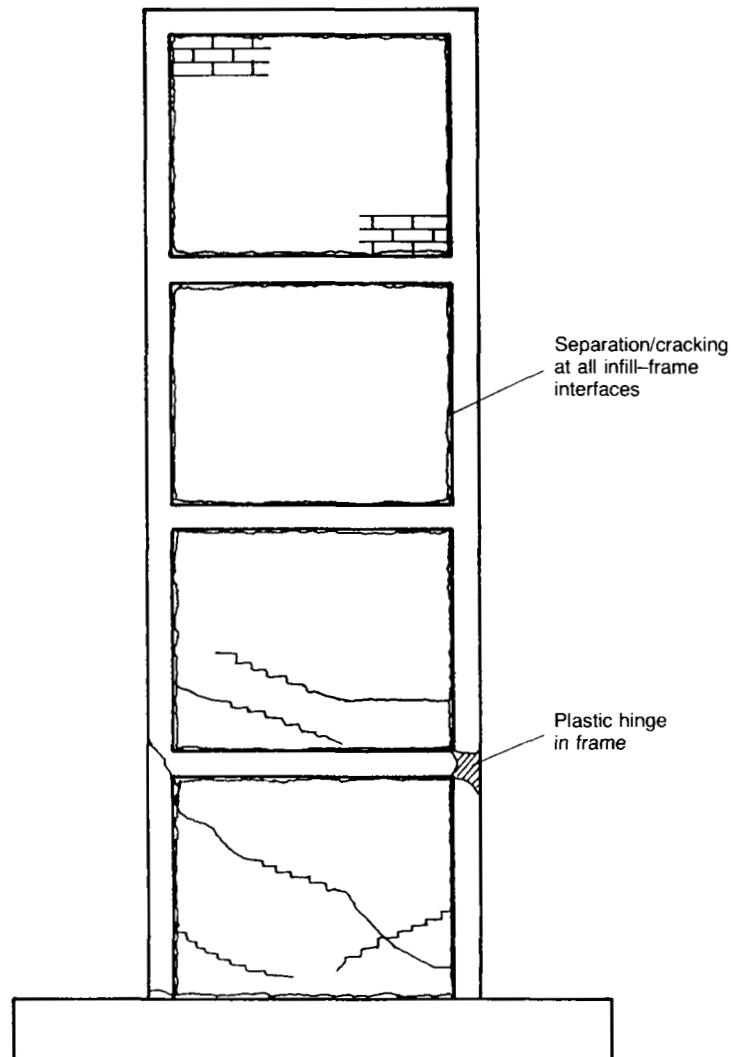


Fig. 7. Failure pattern of model 2

Condition of model	First natural frequency of vibration in in-plane direction of wall: Hz			Damping ratio: %		
	Model 1	Model 2	Model 3	Model 1	Model 2	Model 3
Initial state (before earthquake tests)	7.4	7.7	6.7	1.2	1.1	1.7
Elastic state (vibrated but not cracked)	6.9	6.8	5.0	1.6	1.5	1.5
Cracked state (cracked but not failed)	6.6	5.9	4.5	2.0	1.7	3.6
Failure state (badly damaged condition)	4.8	3.0	1.8	2.0	1.9	11.0

Table 1. Natural frequencies and damping ratios of models as measured by small amplitude vibration tests

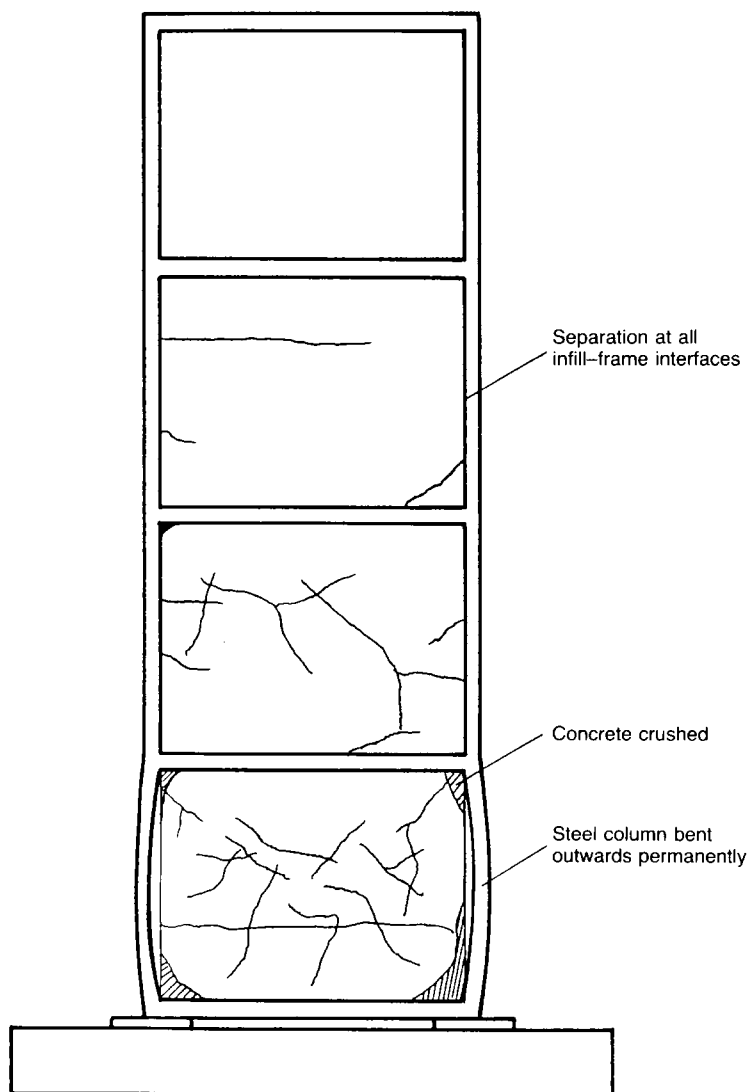


Fig. 8. Failure pattern of model 3

ed in Table 1, it can be seen that, initially, the damping ratios of models 1, 2 and 3 were equal to 1.2%, 1.1% and 1.7% respectively. As the models degraded, the corresponding damping ratios increased gradually to 2.0%, 1.9% and 11.0% respectively. It should be noted, however, that these damping ratios were applicable only under small amplitude vibrations because of the way they were obtained. The actual damping ratios of the structures when subjected to large amplitude vibrations during earthquakes should be larger than these values.

30. The mode shapes of the three models corresponding to the first vibration mode in the in-plane direction of the walls as measured by the small amplitude vibration tests are plotted in Fig. 9. From the results of model 1, it can be seen that the vibration modes of model 1 at the initial and failure states are generally curve-shaped, and that the maximum curvatures occur at the roots of the walls. Such curvatures, which were the results of flexural deformations, indicate that the shear walls acted like vertical

cantilevered beams, deflecting in both shear and flexure during the vibrations. Comparing the mode shapes at different states, it can also be seen that even when the shear wall structure approached the failure state, there was very little change in its vibration mode shape. The only noticeable change in the mode shape as the structure degraded was the slight increase in shear and bending deformations at the lower parts of the walls compared with those at the initial state. This was the result of the more rapid degradation near the roots of the walls. With regard to the mode shape results of the other two models, it can be seen from Fig. 9 that the mode shapes of both models 2 and 3 were quite linear at the initial state. The small curvatures of the mode shapes reveal that the flexural deflections were insignificant and that the two models acted like shear structures. Owing to a break-down of the transducers, the mode shape of model 2 at the failure state was not obtained. Nevertheless, from that of model 3, it can be seen that as the infilled frame structure degraded, its mode shape changed quite significantly, the shear deflection of the first storey increasing at a faster rate than those of the upper storeys until, at the failure state, the shear deflection of the first storey was much larger than those of the upper storeys. Although the mode shape of model 2 was not obtained, the other observations suggested that its mode shape at the failure state was similar to that of model 3. Hence it may be concluded that for the two infilled frame models tested, the stiffness of the lowest storey degraded much more rapidly than the upper storeys. It was the rapid degradation of the lowest storey which led to the large reduction in natural frequencies towards the failure state.

Comparison of seismic resistances

31. The dynamic loads applied to the models during the earthquake tests are best evaluated in terms of the maximum base shear induced by each simulated earthquake, which can be computed by the following three steps.

- From the acceleration response, the horizontal inertia forces acting on the floors is calculated by multiplying the floor masses with their respectively horizontal accelerations.
- The base shear is determined by summing up all the instantaneous horizontal inertia forces acting on the floors.
- The maximum base shear is taken as the maximum value of all base shear results recorded during the simulated earthquake.

Fig. 10 shows the maximum base shear so evaluated for each model plotted against the peak acceleration of the applied earthquake.

32. It can be seen from the base shear results that, for each model, the maximum base

shear induced during a simulated earthquake increased with the peak acceleration of the applied earthquake motions until a certain peak value of maximum base shear was reached; thereafter, the maximum base shear started decreasing, indicating that the model structure had then been very badly damaged and could take only an ever decreasing shear load when subjected to further earthquake excitations. From the results shown in Fig. 10, the following can be seen: in model 1, the maximum base shear reached a peak value of 59.8 kN at a peak acceleration of 0.83 g; in model 2, the maximum base shear reached a peak value of 50.5 kN at a peak acceleration of 0.66 g; in model 3, the maximum base shear reached a peak value of 116.4 kN at a peak acceleration of 0.94 g.

33. The state when the maximum base shear reached the peak value may be regarded as the ultimate limit state under earthquake attack because, beyond this state, the structure may be considered to have failed. Based on this philosophy, the peak value of maximum base shear reached at the ultimate limit state may be taken as the dynamic shear strength of the structure. The dynamic shear strengths of the models so defined are compared with the corresponding static shear strengths (measured by the static load tests carried out separately on the duplicate set of models) in Table 2. It is revealed that the ratios of dynamic shear strength to static shear strength were 0.82, 0.89 and 1.02 for models 1, 2 and 3 respectively. The ratio of dynamic shear strength to static shear strength was particularly low for model 1. This was on account of the fact that model 1 developed its full flexural strength under the static load case (it failed by flexure during the static load test), but was not capable of developing its full flexural strength during the earthquake tests because of sliding shear failure (it failed by sliding at the wall-base junctions during the earthquake tests). From this result, it may be concluded that the sliding shear strengths of shear walls can be significantly lower under dynamic/cyclic load than under static/monotonic load. For the other two models, the dynamic shear strengths were basically the same as the corresponding static shear strengths. The cumulative damage caused by the simulated earthquakes applied before the ultimate limit state was attained did not significantly reduce the shear strengths of the infilled frame models.

Discussion of results

34. At the end of the earthquake tests, models 1 and 3 did not collapse, but model 2 collapsed because a brickwork panel fell out of the plane of the wall structure. The collapse of model 2 verified the concern of some people about the out-of-plane failure of infilled frame structures wherein no connectors are provided

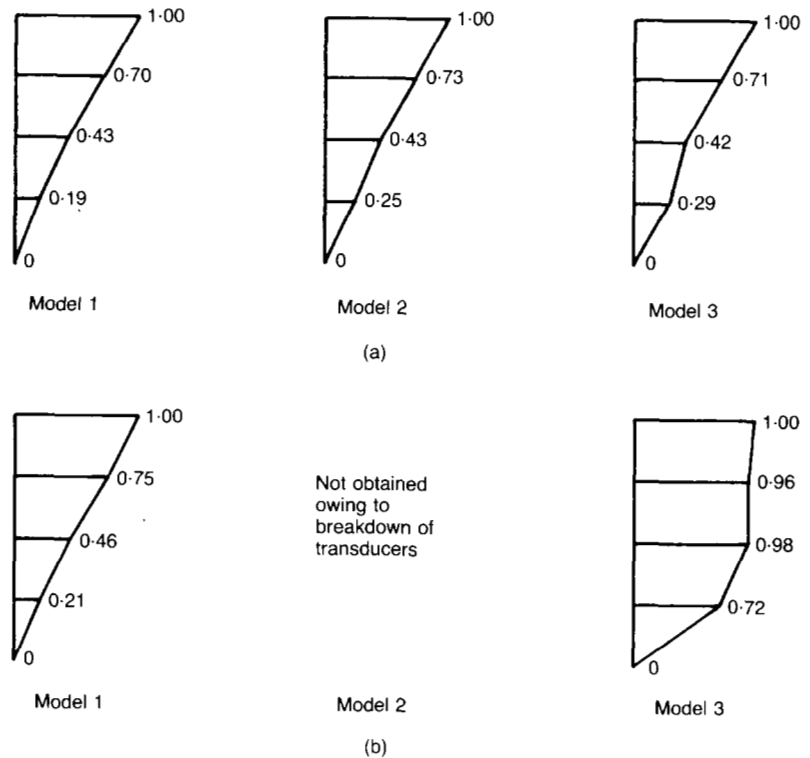


Fig. 9. Shape of first vibration mode in in-plane direction of walls: (a) at initial state; (b) at failure state

to bond together the infilled panels and the frames. Although the infilled concrete panels in model 3 did not fall out of plane, this was only because the infilled panels were held in place by steel plates welded on to the frame members, and if suitable measures for keeping the infilled panels inside the frames were not taken, the danger that infilled panels would fall out of plane would still exist. Generally speaking, therefore, non-integral infilled frames are dangerous if no measures are taken to keep the infilled panels in place. Since it may not be practical in actual construction to install steel plates on to the frame members to keep the

Fig. 10. Variation of maximum base shear with peak acceleration of applied earthquake

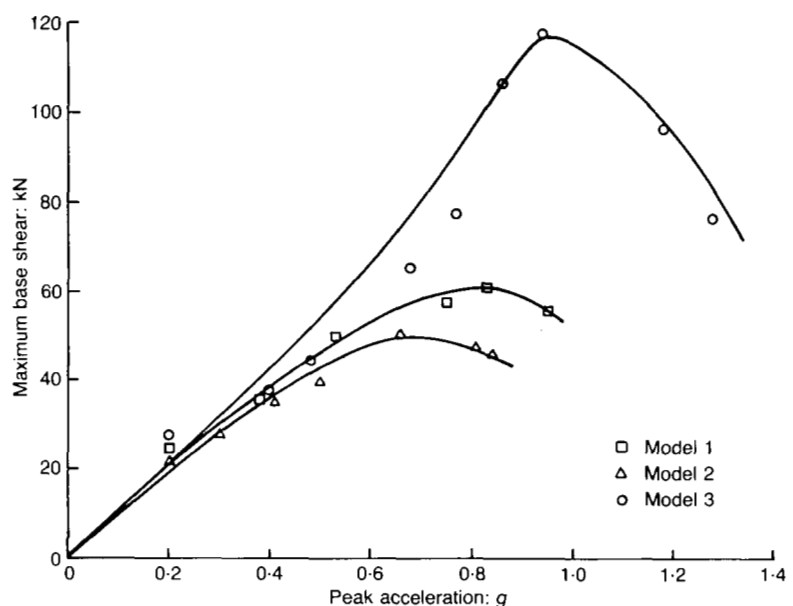


Table 2. Static and dynamic shear strengths of models

Property being compared	Model 1	Model 2	Model 3
Static shear strength: kN	72.5	56.8	113.6
Dynamic shear strength: kN	59.8	50.5	116.4
$\frac{\text{Dynamic shear strength}}{\text{Static shear strength}}$	0.82	0.89	1.02

infilled panels in place, it would be better if the infilled panels were bonded to the frames through the provision of connectors at the infill-frame interfaces. This would not only prevent the infilled panels from falling off but also increase the lateral stiffness and strength of the infilled frame structures. In other words, integral infilled frames are much better than non-integral infilled frames and the former should therefore be preferred.

35. Shear walls, on the other hand, are not without problems. As observed from the earthquake tests, model 1 failed under the earthquake loads by sliding along the wall-base joints rather than by flexure, as would have been expected from the results of the static load tests. The shear walls of model 1 have a height to width ratio of 3.0 and should therefore be regarded as tall shear walls.^{6,7} The occurrence of sliding shear failure in such a tall shear wall structure reveals the fact that sliding shear failure can occur not only in squat shear walls but also in taller shear walls. More care in designing for the sliding shear resistance of shear walls is required. This phenomenon of substantial reduction in sliding shear strength during earthquakes had, in fact, been recognized by Paulay⁶ early in the 1960s. Based partly on Paulay's study and partly on the observations made in the present shake-table tests, an explanation for the above phenomenon is given as follows. Initially, the shear walls behaved as vertical cantilevered beams. The bending moments acting on the walls during the earthquakes caused flexural cracks to be formed at both sides of the walls. As the earthquake loads increased, the flexural cracks gradually extended towards the centroidal axis of the walls until eventually some of the cracks cut through the whole width of the walls. Following a further increase in loading, the alternate near ultimate loading caused the vertical reinforcement to yield. Excursion of the vertical reinforcement into the post-elastic range led to considerable residual strains in the reinforcement and, consequently, to widening of the cracks in the surrounding concrete. As a result, on load reversal, the cracks did not close immediately. Before the cracks closed, the vertical reinforcement carried most of the vertical compression, thereby leading to possible compression yielding or buckling of the reinforcement bars. More importantly, the compression in the

vertical reinforcement significantly reduced the clamping forces which could provide friction to resist the horizontal shear. Furthermore, the large crack width drastically reduced the interlocking action of the aggregates, leaving behind only the dowel action of the vertical reinforcement bars as the last line of defence against sliding shear failure. More research studies on the quantification of the above effects are recommended.

Conclusions

36. Three large-scale models of shear walls and infilled frames were tested on a shake table by being subjected to simulated earthquakes of progressively increasing magnitude until ultimate failure. From the simulated earthquake tests, the seismic behaviour, damage characteristics, structural degradation and dynamic shear strengths, etc., of the models were studied. Many useful observations have been made. However, the comparison of the seismic performance of the different structural forms which was one of the objectives of the research study did not yield any definite conclusion as to which structural form is more effective, simply because the number of models tested was not sufficient for a fair comparison to be made. More shake-table tests are recommended, not just to compare more fairly the relative effectiveness of the different structural forms, but also to understand better the seismic behaviour of these structures.

37. The earthquake tests revealed that the two structural forms studied have the following problems. For the case of shear walls, the sliding shear strengths of the shear walls under dynamic/cyclic loads can be significantly lower than those under static/monotonic loads. As this phenomenon can occur not only in squat shear walls but also in taller walls (the tested shear wall model has a height to width ratio of 3.0 and yet it failed by sliding shear during the earthquake tests), more care should be taken in designing for the sliding shear strengths of reinforced concrete walls. For the case of infilled frames, the major problem is the possibility that the infilled panels will fall out of the planes of the frame structures (one of the infilled frame models tested actually failed by falling off an infilled panel). Suitable measures for keeping the infilled panels inside the bounding frames must be taken. For this purpose, it is suggested that all infilled panels should be bonded to the frames through the provision of connectors at the infill-frame interfaces. This would, in fact, also increase the lateral stiffness and strength of the infilled frame structures.

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