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SHAKE-TABLE TESTING OF SMALL SCALE STRUCTURAL WALLS

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SUMMARY

The preliminary work and first results of an ongoing experimental programme investigating the response of reinforced concrete walls under earthquake loading is discussed in this paper. A modified similitude relation for small-scale reinforced concrete dynamic modelling is presented. Based on the chosen model parameters, the design of 1:5 isolated reinforced concrete walls with reduced shear reinforcement is given. The test-rig set-up and the earthquake input signals suitable for the wall model and shake-table are also discussed. Preliminary observations regarding stiffness, strength and failure mode of the reinforced concrete wall models are given. Experimental results from both shake-table and cyclic pilot tests are compared.

INTRODUCTION

Observations from the field study of earthquakes indicate that a level of drift control higher than that demonstrated by moment resisting frames is necessary in order to avoid excessive non-structural damage. This is emphasized by the extent of non-structural damage sustained by reinforced concrete structures subjected to strong ground motion, primarily due to excessive storey displacements. Stiff shear resisting members such as 'shear walls' not only enhance the integrity of the load bearing and non-structural components but also can reduce damage to service ducts and other contents of the structure.

Present code requirements for earthquake-resistant design often underestimate the ductility of reinforced concrete walls. This is manifested by the increase in design base shear coefficient imposed on buildings with walls. The reasons for this undue conservatism can be attributed to an attempt to avoid observed brittle modes of failure in walls designed in accordance with the code provisions for flexural behaviour.

A design-oriented research programme (Ref.1) is underway, to verify conclusions drawn from previous static testing at Imperial College (Ref.2,3) and to provide improved detailing guidelines and enhanced ductility and energy dissipation capacity. The principal conclusions to be verified include:

(a) The inadequacy of the 'truss analogy' model and the merits of the proposed design for shear based on the 'compressive force path' approach.
(b) The validity of the assumption that a triaxial stress condition prevails in the lower compressive zone and that the ultimate strength is independent of the concrete strength and loading history.

(c) The postulate that failure of walls is due to the development of tensile stresses in the compressive zone, as a result of change in the compressive path direction, stress concentrations at the tip of cracks and loss of bond between tensile reinforcement and concrete.

All the experiments are performed on 1:5 and 1:2.5 scale models. In the first part of this paper, the development of a small scale dynamic modelling procedure for reinforced concrete walls is discussed together with the design of the experiment and support frame, and choice of shake-table input signals. In the second part, the results obtained from a pilot shake-table experiment are compared with cyclic tests performed at the same scale.

This work is part of an ongoing analytical and experimental research programme into the behaviour of reinforced concrete walls subjected to earthquake loading.

MODELLING PROCEDURE

Small scale model analysis is based on dimensional analysis. Given the basic model ratios, the similitude relationships can be determined by using Buckingham theorem (Ref.4). For shake-table testing, the three arbitrarily chosen model ratios define the geometric, force and dynamic similitude relationships. The technological limitations of small shake-tables usually govern the first two relationships.

Force Similitude   The size of the model is limited by the maximum shake-table force and hence the geometric scale factor l is determined first. Force scaling is necessary to reproduce stresses of interest and as a result it is influenced by material properties such as the stress-strain relationship, shear transfer, creep, bond, cracking and crushing. In order to avoid material scaling effects, the same concrete mix is used throughout the experimental programme. For the same reason, specially manufactured model reinforcement (Ref.5) is utilized. As a result, the elastic properties scaling factor e is chosen as unity. However, it has been demonstrated analytically (Ref.6) that a 30% variation in the concrete elastic moduli will influence the overall stiffness and result in fundamental frequency variations of up to 15%. This implies that final "tuning" of the experimental programme and choice of scaled input signal is better performed after material properties have been measured. Self-weight gravity (or inertia) forces in shear walls are generally negligible compared to the building gravity forces to be resisted and hence the density scale factor p which determines these forces can be neglected.

Simulation of the actual boundary conditions in a structural member depends entirely on the structural system used in the prototype, but for the research purposes of this programme, isolated wall boundary conditions are employed.

Dynamic Similitude   In small scale modelling, the exact and simultaneous reproduction of stiffness and inertia forces is impossible. However, since p can be neglected, the similitude relationship which uses p as the third variable model scale factor can be used. This factor when used with l and e influences only time-depended quantities including the velocity. Since in a material such as concrete the velocity of loading and propagation of cracks is an important dynamic parameter, it is proposed to select the velocity scale factor v as the third modelling parameter.

Choice of Model Parameters   The geometric and dynamic model parameters must yield model quantities within the shake-table capabilities. For horizontal excitation the maximum dynamic force of the Imperial College shake-table is 48 KN and the maximum payload is 4895 Kg at a height of 1 meter. The frequency range is 1-40Hz and the minimum time interval dt that would allow an adequate number of data acquisition channels is 0.005 seconds. Preliminary analysis of a full size isolated reinforced concrete wall of aspect ratio 2 yielded a strength of about 1000 KN, hence

a model scale force factor of 25 can be chosen. Since the elastic properties scale factor was chosen as unity, it follows that l=5. For v=1 the time t and acceleration scale factor a would be 5 and 1/5 respectively. In order to avoid very high accelerations for the pilot experiment, the inertia mass was chosen to be 2000Kg and a=1.

Design of Test Rig   The design of the test-rig shown in Figure 1 was intended to satisfy the following requirements :

(a) Support the 2000Kg mass at a centre of gravity of 1 meter or less above the table platform.
(b) Allow free translation and rotation in the direction of shaking to satisfy the isolated wall boundary conditions.
(c) Prevent all out-of-plane degrees of freedom.
(d) Be stable during all stages of assembly and testing including impact loading due to brittle wall failure.

Additionally, the test-rig lateral stiffness (in the absence of the wall and with the mass locked) was designed to coincide with the expected wall stiffness. With this approach, time-history matching is performed on the test-rig without damaging reinforced concrete specimens.
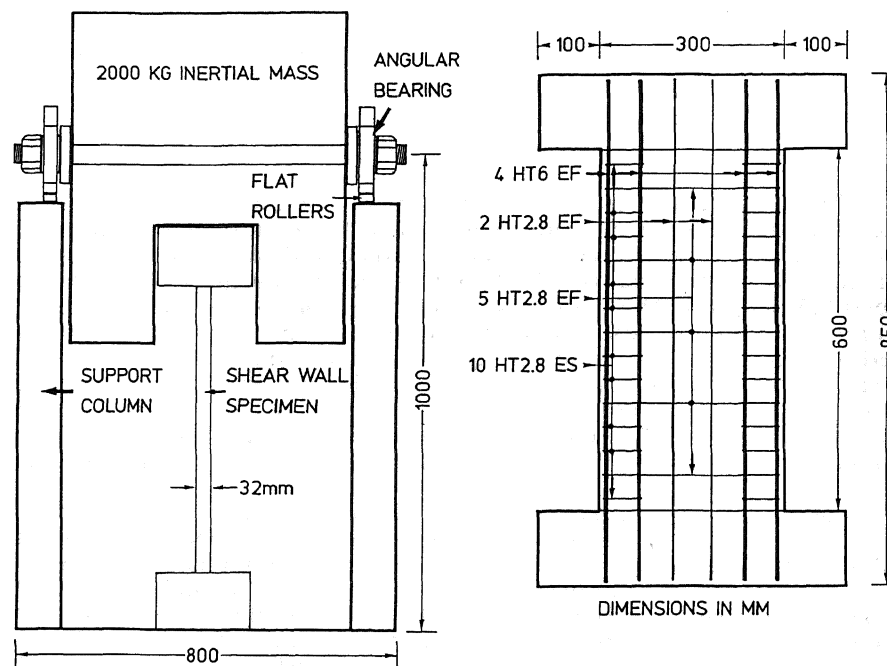

Figure 1. Shake-table test-rig arrangement and 1:5 model wall.

Design of Model Wall and Beams   The beams are designed to provide fixity at the bottom, transfer the load uniformly to the wall, provide anchorage for the longitudinal wall reinforcement and prevent uplifting of the mass. The wall dimensions are as shown in figure 1 and since the load is applied at the centre of gravity of the inertia mass, the effective aspect ratio is 3. The estimated flexural capacity of the wall is 19 KNm and the expected maximum mass acceleration is 1.1g. Shear reinforcement provided is at least 25% less than required by EC8, NZS 3101 and UBC 83, and was based on the design of walls with aspect ratio 2 previously tested at Imperial College (Ref.2).

Choice of Input Signal   Careful consideration is given to a number of parameters relating to the characteristics of the input signal which can be chosen from a large number of earthquake records available from the Imperial College strong-motion data bank. The length of the record should not exceed 1024 points due to computer limitations. The response of the wall in prototype quantities was expected to be in the range of 2-2.5Hz and accounting for an inevitable decrease in frequency due to stiffness deterioration, the predominant frequencies required in the 5% damping acceleration elastic response spectra should be between 1-2.5Hz. Response accelerations in the expected range of frequencies should not exceed 1.1g at ultimate load testing. Table 1 shows the strong motion characteristics after time history matching. The records used were scaled in time from 0.02 to 0.005 seconds and were matched at an acceleration gain of 25-100%.

Table 1.   Strong motion characteristics

SM RECORD	DATE	CODE	MAX. BASE ACCEL. (g)	MAX. RESPONSE ACCEL. (g)
El Centro	19May40	(EC)	0.335	1.1
Parkfield	28Jun66	(PA)	0.499	1.6
Montenegro	15Apr79	(MO)	0.440	1.9
San Fernando	09Feb71	(SA)	1.121	3.0

EXPERIMENTAL RESULTS

Stiffness   The overall stiffness of the wall (SW1) was much lower than expected by simplified analytical and even more sophisticated finite element programs. The predictions, based on the uncracked section and code design elastic properties, yield top wall stiffness of the order of 22 KN/mm while the initial experimental values do not exceed 5.5 KN/mm. The overall wall cross-section axial elastic modulus was checked by loading virgin specimens. The values obtained are about half those predicted by code equations. Another significant contribution to the lateral displacement grossly underestimated by the elastic analysis is the contribution of shear deformations. Base rotations of the concrete beam, the test rig and shake-table platform can also amplify the lateral displacement and while the first two rotations have been shown to be insignificant, the platform rotations were not evaluated.

It is noteworthy that in recent research in the nuclear industry on small scale shear wall models (Ref.7), a 3 to 4 times decrease in the expected stiffness triggered a series of further credibility tests. The stiffness of the uncracked section of the 1:2.5 scale models tested at Imperial College (Ref.3) was about half the analytical predictions. There seems to exist a relation between the thickness of the section as determined by the scale and the elastic properties of concrete This, though it does not influence significantly the ultimate behaviour, it affects the stiffness and hence the dynamic characteristics of scaled models.

Stiffness deterioration   As a result of the reduced stiffness, the chosen earthquakes for the pilot experiment were not as effective as expected. However, significant stiffness deterioration occurred before reaching steel yield limits. This is demonstrated by the drop in the response frequency as obtained by the frequency analyser (Table 2). Due to high frequency noise originating from metal to metal contact, the acceleration records have been filtered so as to remove parasitic peaks. Unfortunately, these peaks triggered the shake-table emergency shut down mechanism during the San Fernando earthquake and hence the peak acceleration could not be obtained. High energy harmonic frequencies were used to break the wall at very high displacements.

Table 2   Stiffness deterioration for shake-table test.

RECORD	GAIN	MAX. WALL DISPL.mm	MAX. WALL ACCEL.(g)	RESPONSE FREQ. Hz BEFORE	AFTER
EC	25%	0.62	0.09	8.0	6.0
EC	50%	1.38	0.190	6.0	5.6
EC	100%	3.44	0.4	5.6	5.2
PA	50%	2.11	0.266	5.2	---
MO	50%	2.65	0.333	4.8	4.8
MO	100%	4.50	0.476	4.8	4.8
SA	100%	6.91	-----	4.8	4.0
SA	100%	7.62	-----	4.0	3.6
Harmonics @	3.4Hz	12.2	1.2	3.6	3.4

Damping   An attempt to calculate damping was made by swinging a pendulum from a constant height. The decay of the ensuing free vibration was used to estimate the damping coefficient. One of the disadvantages of this method, however, is that the energy input by the pendulum is very low and hence the motion is at very low excitation levels and cannot therefore represent accurately the real damping of the component. This highlights a common error in estimating dynamic characteristics from forced vibrations. On the one hand, if the amplitude is too small, linear elastic values are obtained . On the other hand, damage of the model will increase if large amplitude vibrations are used to assess damping.
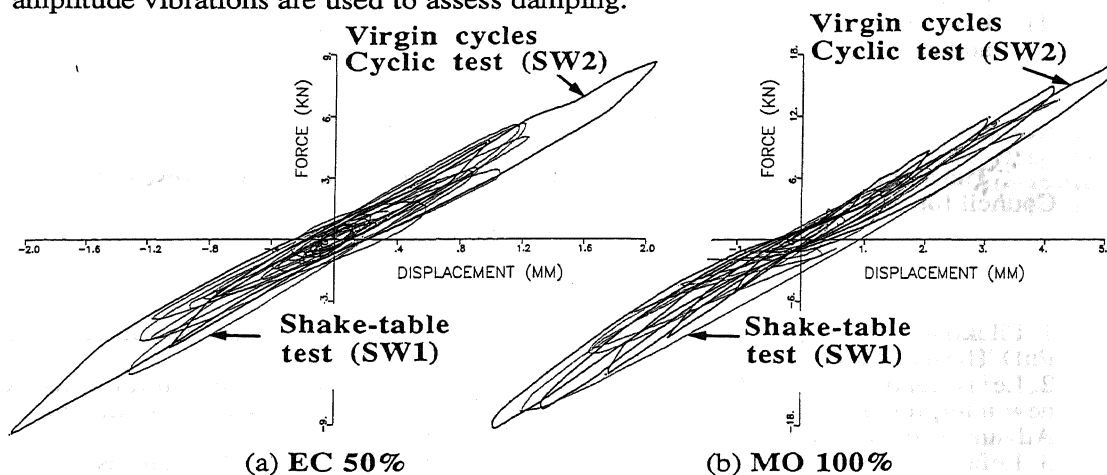

(a) EC 50%   (b) MO 100%

Figure 2. Cyclic and shake-table force vs displacement results.

Comparison with cyclic testing   A cyclic test (SW2) was performed on an identical wall with similar boundary conditions (Ref.1). The displacement-controlled load was applied at the same height as the inertia mass centre of gravity. A large number of load reversals was applied at low displacement levels in order to investigate the stiffness deterioration that would occur before the steel yield limits. It has been observed that the stiffness deterioration is insignificant prior to yielding, provided that the previous maximum displacement has not been exceeded. This is not valid after yielding and the number of load reversals was reduced so as to avoid excessive accumulation of plastic strains in the reinforcement. By converting the wall response acceleration into an equivalent static force, a comparison between the cyclic and shake-table tests can be made. Before the yield, the cyclic 'virgin' loops agree very well with the filtered shake-table loops as shown in Figure 2a and 2b.

Ultimate Behaviour   The ultimate displacements though much higher than expected by finite element programs, compare very well with the cyclic experiments. The maximum force achieved is in close agreement with the predicted ultimate force in flexure. The failure mode was obviously flexural, by snapping of the tensile reinforcement at the bottom of the wall at the point where the bars have been strain-gauged. The same type of failure was observed during cyclic experiments and the snapped bars have undergone very significant plastic strain reversal. Further investigation of ultimate behaviour is underway on scale 1:2.5 models subject to a large number of load cycles.

CONCLUSIONS

(a)   The geometric scale factor l of shake-table models is a function of technological constrains imposed by the table characteristics. For reinforced concrete small scale modelling it is proposed that the other two independent scale factors are e for elastic properties and v for velocity.

(b)   A suitably designed test-rig can be used for time history matching, hence avoiding the use of actual specimens at low excitation levels that may cause some stiffness deterioration.

(c)   The large reduction in experimental stiffness as compared with analytical predictions can partly be accounted for by the reduction in concrete axial stiffness. This seems to be related to the thickness of the specimen, probably due to the effect of confinement.

(d)   Damping values obtained by low level excitation cannot represent the real damping of an inelastic system.

(e)   The ultimate displacements of both shake-table and cyclic tests are significantly higher than predicted by finite element programs.

(f)   Failure was in both cases in flexure by snapping of the tensile reinforcement and not in shear as predicted by code calculations.

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