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SHAKING TABLE STUDY OF BRICK MASONRY INFILLED FRAMES SUBJECTED TO SEISMIC EXCITATIONS

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SUMMARY

Seismic behaviour of infilled frames has been investigated experimentally and analytically. Steel frames infilled with brick walls were subjected to racking tests and inplane and out-of-plane dynamic shake table excitations. An analytical model has been developed and used to conduct a parametric study of the dynamic behaviour of cracked infilled frames, and the implications of the results of this study on the seismic design of infilled structures are discussed.

INTRODUCTION

The predominant and largely beneficial effect of masonry infills on the lateral response of structures has for long been recognized (Ref. 1). Various analytical methods have been developed (Refs. 2 - 5) but as yet the role of infills as earthquake resisting elements is often ignored by design engineers. This results not only in inaccurate analysis but in most cases gives rise to unrealistic designs. The fact that the effects of masonry infills on strength and stiffness is highly variable and dependent on workmanship, coupled with the lack of information on the seismic behaviour of masonry infills after cracking and crushing, has caused a degree of uncertainty about their role as structural elements. One of the main objectives of the current research at Imperial College was to bridge the existing gap between research and design in this area. A comprehensive literature review was conducted and reported elsewhere (Ref. 1). The biaxial shake table at Imperial College was used to apply a number of earthquakes to model scale steel frames infilled with reinforced and unreinforced brick masonry panels up to failure.

DESCRIPTION OF TESTS

A number of steel frames were fabricated from rectangular hollow sections (RHS). Two types of frames were used, designated F1 and F2. The height and length of both frames were 1300 and 1500 mm, respectively, and the RHS sections used were 60x40x2.5 mm and 90x50x2.9 mm, respectively. The strength and stiffness of frames F1 and F2 were measured as 600 and 2180 kg, and 17.5 and 41.8 kg/mm , respectively. Both these frames were infilled with brick panels of thickness 63 mm. The compressive strength of the mortar and a three-brick prism were measured as 29.0 and 56.5 kg/ cm2, respectively. The load deflection curves of the infilled frames are given in Figure 1. Bare frames and infilled frames were subjected to inplane dynamic excitations. A total weight of 870 kg was applied on top of all test specimens to simulate an inertial mass. The accelerations at the base and top of the frames were measured together with the relative displacement at the top of the frame with respect to the base, refereed to as the frame displacement. To investigate the effect of out-of-plane excitations four brick walls of different sizes, with and without reinforcement, were subjected to horizontal excitations perpendicular to the plane of walls as shown in Figure 2. The two small walls W1 and W2 and the two large walls W3 and W4 were identical except that in W1 and W4 horizontal reinforcement were laid in all bed joints. The width, height and thickness of the small walls were 1025 mm, 755 mm and 63 mm, and of large walls were 1860 mm, 1400 mm and 101 mm, respectively.

RESULTS

The peak displacement responses of frames F1, F2 and the infilled frame F1 due to the different earthquakes are summarized in Table 1. From these results it can be concluded that the presence of the infill has significantly reduced the displacement responses for all of the earthquake records even after cracking of the infill panel. The measured displacement response spectra of the infilled frame F1 after cracking are shown in Figure 3 for different amplitude of excitations. The initially measured natural frequencies of the transverse walls are given in Table 2 together with their cracking and collapse accelerations. The result indicate that the panels could sustain large out-of-plane accelerations even after the occurrence of cracks. This is believed to be due to the constraining effect of the frame (arching action). It can also be concluded that the insertion of horizontal reinforcement in the walls W1 and W4 has significantly enhanced their transverse strength.

MODELLING AND ANALYSIS

Hysteresis Behaviour of Cracked Infilled Frames   Energy absorption and strength deterioration are two important aspects of hysteresis behaviour of infilled frames in the context of earthquake engineering. In general the energy absorption of infilled frames can arise from the following causes: i) material damping which depends on the rate of loading ; ii) material failure such as cracking and crushing of the infill panel, or plastic hinges in the frame ; iii) friction at the panel surfaces of the infill ; iv) the impact resulting from rocking of the infill inside the frame. Energy absorption capacity in the first cycle of loading is usually much greater than in subsequent cycles because failure of infill material is insignificant in subsequent cycles and, consequently, the corresponding hysteresis curves are almost identical. Therefore the skeleton curves associated with these hysteresis curves may be defined as a function of the lateral deflection of the frame.

Cracked Model   A typical hysteresis cycle of a cracked infill frame is shown in Figure 4, where a cubic curve is suggested as the skeleton curve. The governing equation of this curve is

$$H = A.x + B.x^3 \qquad (1)$$

where H and x are the load and deflection, respectively.
Parameter A is equal to the initial stiffness of the cracked specimen as indicated in Figure 4, and parameter B accounts for the stiffness degradation as shown typically in Figure 5. So long as the model has not exceeded the last maximum deflection (Characteristic Deflection of the model designated as D and D´ in Figure 4), it remains elastic and governed by Equation (1). However, the model becomes inelastic after passing the characteristic points M and N in Figure 4 and new characteristic points are defined (M_1, M_2, ... in Figure 5), and parameter B

in Equation (1) is modified accordingly.

Analysis The load-deflection model discussed above was used in a nonlinear dynamic programme based on Newmark's method (Ref. 6) with ß equals 1/6. To calculate the response spectra, the initial stiffness was assumed as 50 kg/mm and the coordinates of points M and N in Figure 6 were assumed as (6.6 mm and 4000 kg) and (- 6.6 mm and - 4000 kg), respectively. Mass and damping ratio were assumed as 870 kg and 5%, respectively. The calculated response spectra are given in Figure 6 and the calculated and measured responses due to the Parkfield earthquake are illustrated in Figure 7. The analytical and experimental results of the peak displacement responses are also compared in Table 1 and it can be seen that the former are consistently greater than the latter, hence the analytical results are on the safe side.

Parametric Study A series of parametric studies were carried out using the analytical model, and the effects of variation of the following parameters were investigated: damping ratio, yield strength , earthquake intensity (defined as the ratio of peak accelaration of the simulated earthqake to the recorded earthquake) and mass. The results indicate that the damping ratio does not affect the response significantly as long as the displacement does not exceed the characteristic point (point M in Figure 4), otherwise it becomes significant. A reduction of the yield strength (H_0 in Figure 5) generally leads to an increase in displacement response, although the importance of this effect varies from one earthquake to another. The effects of variation of earthquake intensity on the "cracked model" and a perfect "elastic-plastic model" are illustrated in Figure 8. The ratio of the displacement response of the cracked model and the corresponding elastic-plastic model are given in Figure 9. From these results it can be concluded that i) In both these models the displacement response increases with a moderate slope as the earthquake intensity increases. There is usually a critical intensity beyond which the displacement response grows rapidly; ii) Results of the cracked model are consistently greater than those of the corresponding elastic-plastic model; iii) The ratio of the responses of the cracked model and the elastic-plastic model was below 2.5 for most of the earthquakes except at the critical intensities. Hence it can be concluded, from this study, that the seismic displacement of cracked infilled frames are 2.5 times the corresponding elastic plastic frames.

ASEISMIC DESIGN

Although infilled frames are much more ductile than ordinary brickwork structures they cannot be modelled simply as elastic-plastic systems because of their degradation of strength and stiffness under cyclic loading. As mentioned before it can be assumed that the displacement of the cracked model is approximately 2.5 times the corresponding displacement of the elastic-plastic model. Hence, it can be concluded that a cracked model with a ductility factor of μ corresponds to an elastic-plastic system with a ductility factor of $\mu/2.5$. The seismic spectral value of acceleration for elastic-plastic systems (in the constant velocity range with frequencies in the range of 0.2 Hz to 2.0 Hz) is inversely proportional to μ . Thus the seismic spectral acceleration of a cracked system is 2.5 times the corresponding elastic-plastic model. This means that the current seismic coefficients should be multiplied by 2.5 to account for the degrading nature of the cracked infilled frames, i.e. the seismic coefficient of infilled frames for an inelastic design (cracked design) C_c can be calculated as

$$C_c = 2.5\,C \qquad (2)$$

It has been shown (Ref. 7) that the seismic coefficient of infilled frames in the constant acceleration range (frequencies of 2 to 8 Hz) can be obtained from the following equation.

$$C_c = 1.8\ C \tag{3}$$

It is noticeable that the natural period of an infilled frame used for a cracked design differ from the one used in elastic (uncracked) design. The following expression was suggested (Ref. 7) for the effective natural period of a cracked infilled frame T_e with a ductility factor of μ.

$$T_e = 0.707\ (\sqrt{\mu} + 1)\ T_0 \tag{4}$$

where T_0 is the natural period of uncracked structure. The yield strength of the cracked infilled frames is assumed as the envelope of the hysteresis cycles and can be much less than the ultimate strength. As a result of cyclic racking tests it was suggested (Ref. 7) a value of 0.70 H_u for the yield strength, i.e.

$$H_y = 0.70\ H_u \tag{5}$$

where H_u is the ultimate strength obtained by cyclic testing.

CONCLUSIONS

1. The presence of infill significantly reduces the seismic displacement responses even after cracking of the infill panel.
2. The arching action of bounding frame and the presence of horizontal reinforcement significantly enhance the resistance of infill panels against out-of-plane excitations.
3. It is suggested that the existing codified provisions for seismic loadings are unsafe for inelastic (cracked) design of infilled frames. Appropriate modifications have been suggested.

ACKNOWLEDGEMENT

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REFERENCES

1. Moghaddam, H.A., Dowling, P.J., *The state of the Art in infilled frames*, ESEE report No. 87-2, Imperial College, London, Aug 1987.

2. Stafford-Smith, B.S., *Behaviour of square infilled frames, Am Soc Civ Eng*, 92, No. ST 1 (Feb), 1966, pp 381-403.

3. Riddington, J., Stafford-Smith, B., *Analysis of infilled frames subject to racking with design recommendations*, The Structural Engineer, 52, No. 6, 1977, pp 263-268.

4. Wood, R.H., *Plasticity, composite action and collapse design of unreinforced shear wall panels in frames*, Proc Institution of Civil Engineers, Part 2, 65, June 1978, 381-411.

5. Liauw, T.C., Kwan, K.H., *Plastic theory of non-integral infilled frames*, Proc Institution of Civil Engineers, Part 2, 75, Sept 1983, pp 379-396.

6. Newmark, N.N., Rosenblueth, *Fundamentals of earthquake engineering*, Prentice-Hall, 1971.

7. Moghaddam, H.A., *Seismic behaviour of infilled frames*, Ph.D thesis, Imperial College, London, 1988.

Table 1 - peak displacements of frames F1 and F2 and the infilled frame F1

earthquake	target peak base acceleration (g)	frame F1		frame F2		infilled frame F1				
						before cracking		after cracking		
		a_0	d	a_0	d	a_0	d	a_0	d	
									measured	calculated
Ancona 1972	.51	.50	21.2	.72	9.0	.51	.50	.60	1.46	2.80
Montenegro 1979	.46	.48	50.1	.43	25.5	.45	.40	.49	1.60	2.30
Tabas 1979	.92	.85	54.0	.86	42.5	.81	1.04	.87	2.20	3.15
Parkfield 1966	.52	.62	62.2	.56	24.0	.53	.85	.63	1.50	2.36
San Fernando at Pacoima dam 1971	1.14	1.22	66.0	1.29	57.5	1.26	3.00	1.21	3.30	3.52
San Fernando at Castaic old Ridge 1971	.32	.31	50.0	.38	15.4	.36	.80	.33	1.20	2.05
Gazli 1976	.76	.76	71.8	.76	42.5	.76	1.50	.77	2.60	2.91
El-Centro 1940	.34	.32	72.0	.39	17.0	.36	.70	.35	1.10	2.26

a_0 - maximum acceleration measured at the base of the frame in g.

d - maximum displacement of the frame

Table 2 - measured frequencies and acceleration responses of the transverse walls

wall	(1)	(2)	(3)
W1 (reinforced)	70	*	*
W2 (unreinforced)	68.5	9.60	†
W3 (unreinforced)	37	6.00	7.20
W4 (reinforced)	37	7.80	10.0

(1) - initially measured out-of-plane frequency, Hz
(2) - peak out-of-plane acceleration response of the wall just before cracking, g
(3) - peak out-of-plane acceleration response of the wall just before collapse, g
* - W1 neither cracked nor collapsed during these tests
† - no specific collapse acceleration could be identified

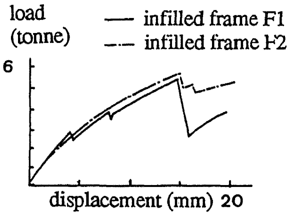

Figure 1 - load deflection behaviour of the infilled frames F1 and F2

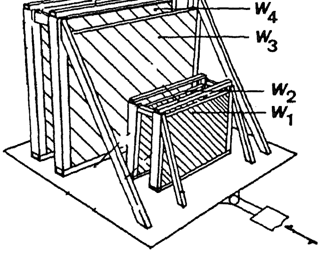

Figure 2 - transverse walls

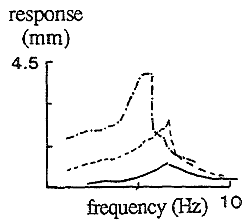

Figure 3 - experimental displacement response spectra for the following amplitudes of excitations (in g) : 0.15, 0.20 and 0.30

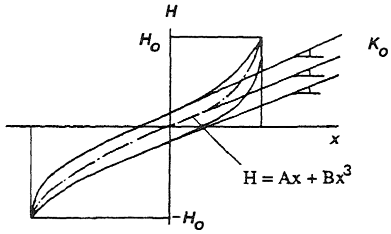

Figure 4 - a typical hysteresis cycle of a cracked infilled frame

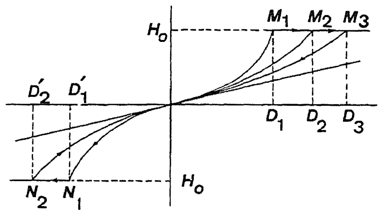

Figure 5 - cracked model

H_0 nominal ultimate strength

M_1, N_1, M_2, N_2, ... characteristic points of the cracked model

D_1, D'_1, D_2, D'_2, ... characteristic displacement of the cracked model

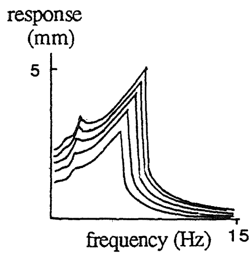

Figure 6 - calculated displacement response spectra for the following amplitudes of excitation (in g) : 0.10, 0.15, 0.20, 0.25 and 0.30

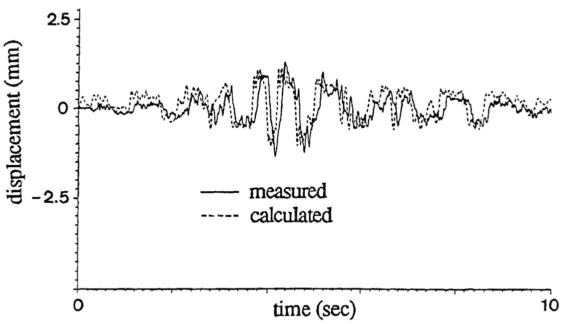

Figure 7 - a comparison of the calculated and measured responses of the infilled frame F2 to the Parkfield earthquake

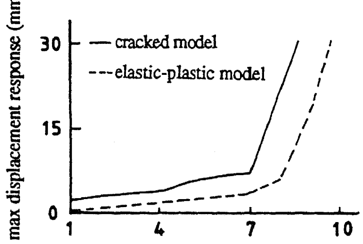

Figure 8 - a comparison of the behaviour of a cracked infilled frame and a perfect elastic plastic system subjected to an earthquake with varying intensity

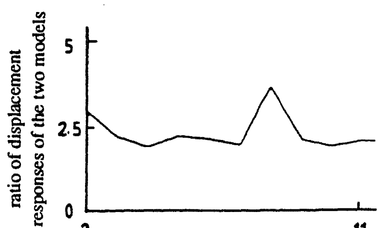

Figure 9 - ratio of displacement response of a cracked infilled frame and a perfect elastic plastic system subjected to an earthquake with varying intensity