INTRODUCTION

Unreinforced masonry (URM) walls, which are widely perceived to possess no or very limited ductility, are expected to behave linearly elastically when subject to out-of-plane loading and fail in a brittle manner when the maximum horizontal resistance is reached. Experimental and analytical investigations undertaken by the authors in recent years revealed that URM parapet walls and simply supported walls subject to one-way out-of-plane bending were able to perform satisfactorily even when the maximum resistance of the walls has been overcome by the seismic forces (Doherty, *et al.*, 2002, Griffith, *et al.*, 2005, Griffith, *et. al.*, 2004, Lam, *et al.*, 2003). Such walls in the post-elastic condition are able to displace out-of-plane up to the limit of overturning which is mainly a function of the wall thickness. Consequently, the risk of the wall collapsing is dependent on the displacement demand of the floor excitations which is natural period dependent and can be represented by the elastic displacement response spectrum of the floor.

This paper presents results obtained from more recent investigations on URM walls which are supported on four sides and hence subject to two-way bending. It was observed from quasi-static testing with loads applied through an air-bag that there can be significant irrecoverable displacement of the wall when the horizontal force is removed (behaviour not observed with walls subject to one-way bending). A mathematical model has been developed to simulate this hysteretic behaviour. The hysteretic model was then subject to non-linear time-history analyses based on various Australian site classes (AS/NZS 1170.4 Draft no.D5212-5.1, 2005) and the filtering effects of a six-storey building (Klopp and Griffith, 1993). A parametric study was then undertaken to investigate the effects of varying the parameters of the hysteretic model and to compare with the displacement demand of linear elastic systems as represented by the displacement floor spectra.

1. MODELLING OF FORCE-DISPLACEMENT BEHAVIOUR OF CRACKED URM WALLS

Cantilever (parapet) walls and walls supported only at the top and bottom edges (vertical bending) can be represented as equivalent single-degree-of-freedom (sdof) systems as shown in Figure 1. The horizontal applied force at the threshold of rocking can be calculated by simple statics assuming that the wall has cracked and hence the tensile strength of the mortar can be neglected. Similarly, the displacement at the threshold of overturning can be obtained from a simple kinematic model assuming rigid-body behaviour (Doherty *et al.*, 2002). The classical force-displacement (F-Δ) behaviour of the equivalent sdof rocking rigid-body is shown in Figure 1. F is defined as the total force applied to the wall and Δ is the effective displacement which is defined as 2/3 of the maximum displacement at mid height (in order that direct comparisons can be made with the elastic displacement response spectrum). The F-Δ relationship has to be modified as shown in Figure 2 to account for the effects of deformation in the wall at highly stressed locations (eg. at the pivotal edge) due to significant pre-compression from gravitational loading.

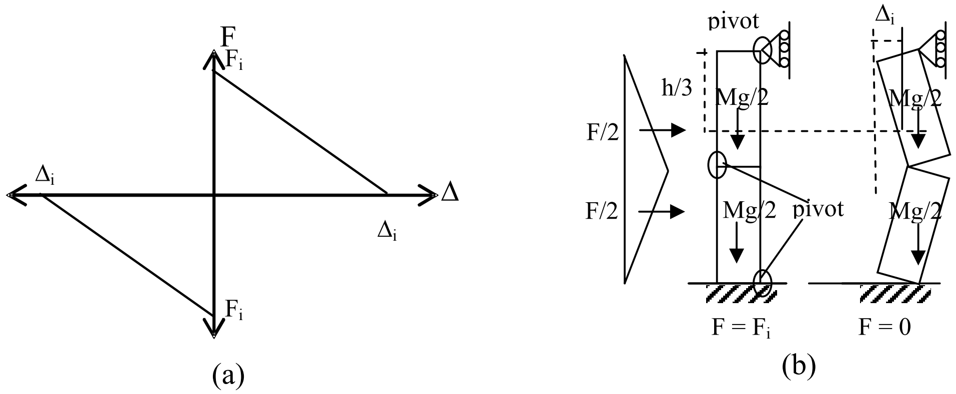


Figure 1 Bi-linear F-Δ relationship, (a) the F-Δ curve, (b) rigid wall assumption

The F-Δ relationship defining the behaviour with one-way bending as shown in Figure 2 can be represented algebraically by a hyperbolic tangent defined by force F_i and slopes K_{ini} and m_i. The F-Δ relationship is elastic since the behaviour on loading, un-loading and re-loading are similar as represented by equation (1a).

$$F = c\tanh(b\Delta) - ab\Delta \qquad (1a)$$

Parameters a, b and c in equation (1) are related to F_i, m_i and K_{ini} as follows :

$$c = F_i \qquad (1b)$$

$$b = \frac{K_{ini} - m_i}{c} \qquad (1c)$$

$$a = -\frac{m_i}{b} \qquad (1d)$$

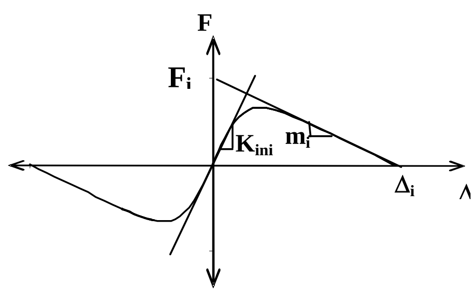


Figure 2 Hyperbolic force-displacement relationship

In contrast, the F-Δ relationship for walls supported on four boundaries, and hence subject to two-way bending, is inelastic as demonstrated by experiments undertaken by Vaculik *et al.* (2004) on dry-stack masonry walls subject to quasi-static out-of-plane loading applied through an air-bag. The tests enabled the total applied horizontal force to be measured along with the out-of-plane displacement during loading and unloading of the walls. It was observed that the displacement of the wall was not fully recoverable as the horizontal force was removed.

The F-Δ relationship can be described as "fully inelastic" when the displacement at the instance of unloading is irrecoverable. Such behaviour can be represented algebraically by a simple transformation of the origin as shown by equations (2a – 2d).

$$F' = c\tanh(b\Delta') - ab\Delta' \tag{2a}$$

and,

$$F' = F - F_o \tag{2b}$$

$$\Delta' = \Delta - \Delta_o \tag{2c}$$

where

$$c = m_i\Delta_o + F_i - F_o \tag{2d}$$

and parameters a and b are defined by equations (1c) & (1d). F_o and Δ_o represents the condition of the wall at the instance of un-loading.

It is assumed in equations (2a - 2d) that the behaviour of the wall at the instance of loading, or re-loading, can be represented by the same hyperbolic relationship (as for walls subject to one-way bending). The F-Δ relationships representing elastic and fully inelastic behaviour are shown schematically in Figures 3a & 3b.

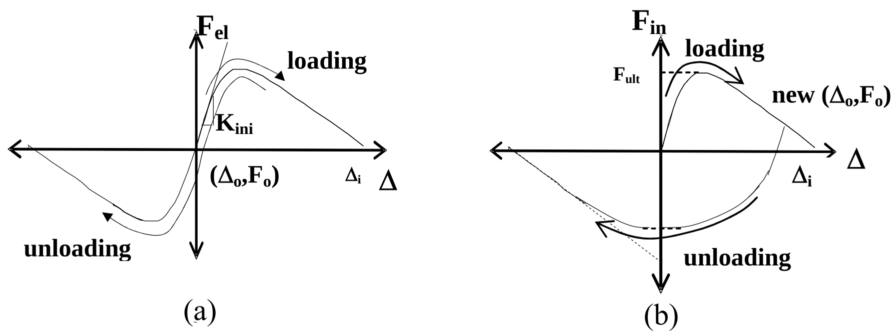


Figure 3 Force-displacement relationships, (a) elastic model, (b) fully inelastic model

The force-displacement behaviour of real URM walls subject to two-way bending is typically a combination of the elastic and fully-inelastic behaviour represented by Figures 3a & 3b. The parameter p_{el} is introduced to represent such hybrid behaviour by superimposing equation (1a) and equation (2a), as shown by equation (3).

$$F = p_{el}F_{el} + (1 - p_{el})F_{in} \tag{3}$$

where, F_{el} and F_{in} are the horizontal forces applied to the wall assuming elastic (p_{el} = 1.0) and fully inelastic (p_{el} = 0) behaviour respectively.

An example of the force-displacement relationships simulated by equation (3) is shown in Figures 4a – 4c for different values of p_{el}. This proposed model for URM walls subject to two-way bending was evaluated by comparison with experimental observations from tests on a dry-stack masonry specimen of length = 2180 mm, height = 960 mm and thickness = 55 mm and with vertical pre-compression of 0.046 MPa. Figure 5 shows the horizontal force applied to the wall (expressed in terms of the percentage of the self-weight of the wall) plotted against its mid-height displacement (expressed in terms of the ratio to the wall thickness). A reasonable match with the model relationship was obtained as p_{el} was set equal to 0.2.

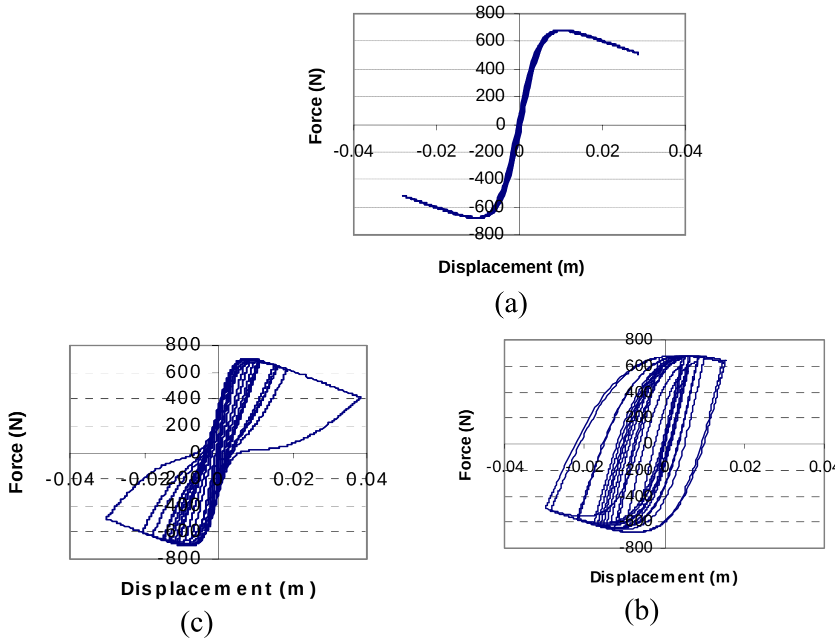


Figure 4 Force-displacement behaviour for walls with, (a) $p_{el} = 1$, (b) $p_{el} = 0.5$, (c) $p_{el} = 0$

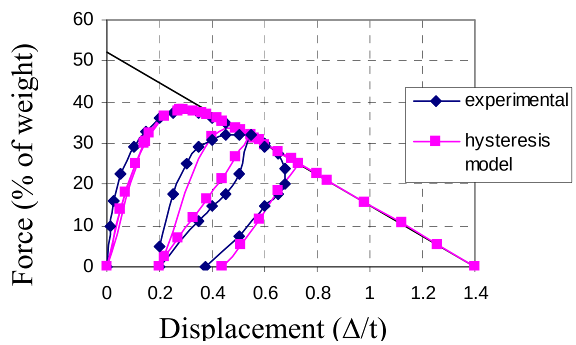
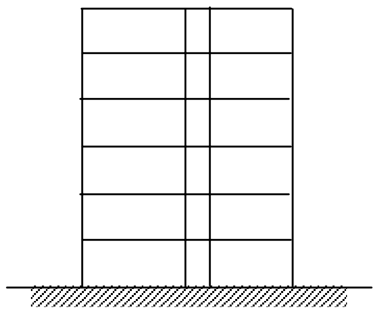


Figure 5 Comparison of F-Δ relationship from experimental result and proposed model

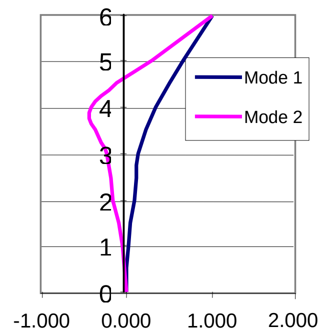
2. APPLIED EXCITATION

Three code-compatible generated accelerograms were used for time history analysis of URM walls. Accelerograms were generated by stochastic simulations (Lam *et al.*, 2000) and one-dimensional non-linear shear wave analyses (program SHAKE) based on earthquake scenarios producing peak ground velocity (PGV) of about 60 mm/sec. This intensity of ground shaking is consistent with a hazard factor of Z = 0.08 as stipulated for Melbourne, Sydney and Canberra for a return period of 500 years by the new Australian Standard (AS/NZS 1170.4, 2005). Accelerograms were generated to match with the design response spectra for rock, shallow and soft soil sites (site class B, C and D respectively). Refer Lam *et al.* (2005) for details of the simulated accelerograms.

A six-storey university-office building at the University of Adelaide, which was identified as the most critical out of a total of eleven buildings studied by Griffith *et al.* (2004), was subject to time-history analyses using the code-compatible accelerograms to obtain the acceleration time-histories for the upper floors. The analyses were based on natural periods and mode shapes as obtained from field tests undertaken by Klopp and Griffith (1993). Natural periods of 0.3 second and 0.17 second were measured for the first two modes (refer Figure 6).



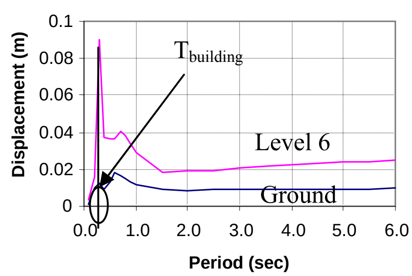
(a)



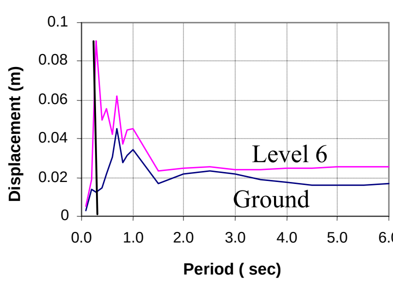
(b)

Figure 6 A six-storey university office building, (a) Building elevation view, (b) Building mode shapes, T_1 = 0.3 second, T_2 = 0.17 second (Klopp and Griffith, 1993)

The displacement response spectra calculated from the simulated motions for the top floor of the building are presented in Figure 7 along with the displacement response spectra of the ground. Very high spectral amplifications typified by resonant conditions are shown at the fundamental natural period of the building with elastic sdof systems predicted to displace up to 80 mm at resonance. However, the displacement demand is shown to decrease very rapidly with further increase in the natural period of the sdof system.



(a)



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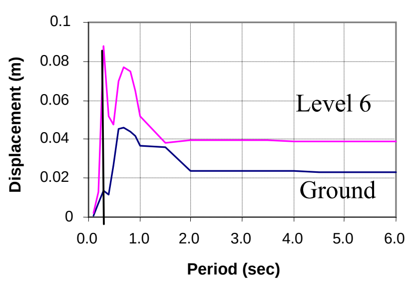


(c)

Figure 7 Displacement response spectra for top level of six-storey building subject to code-compatible earthquake ground motions for : (a) Site class B, (b) Site class C, and (c) Site class D

3. PARAMETRIC STUDIES

A model representing the dry stack masonry wall specimen introduced in Section 2 and with hysteretic behaviour presented in Figure 4 was then subject to non-linear time-history analyses based on motions simulated for the top floor of the six-storey building (described in Section 3 with a fundamental natural period of 0.3 second). The accelerograms were scaled in order that the walls would experience significant P-Δ effects associated with large displacements.

In the parametric study based on site class B conditions, the initial natural period of the wall was varied from 0.15 second to 0.7 second and the parameter p_{el} was also varied between the limits of 0 (fully inelastic behaviour) and 1 (elastic behaviour). The calculated response behaviour of the wall is presented in terms of the effective displacement in Figure 8. The effective wall displacement response only varied moderately with changes in the initial natural period of the wall assumed in the analyses unlike the corresponding linear elastic displacement response spectrum which predicts a displacement demand approximately double the maximum displacement calculated from the non-linear time-history analyses, due to resonance behaviour.

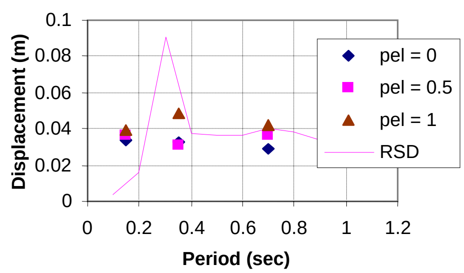
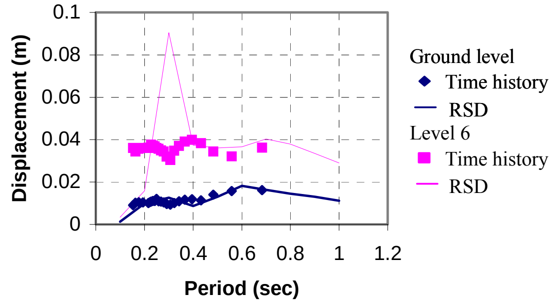
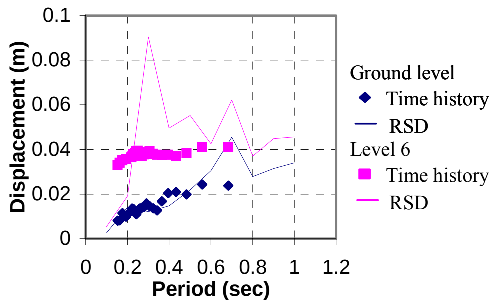


Figure 8 Maximum displacement of walls and displacement floor spectrum (site class B)

Further studies based on a wider range of site classes revealed similar trends except that when softer soil sites were considered, the calculated effective wall displacement was found to be more sensitive to changes in the initial natural period of the wall and closer to predictions from the linear elastic response spectrum (Figure 9). The response of walls at ground level has also been presented in Figure 9.



(a)



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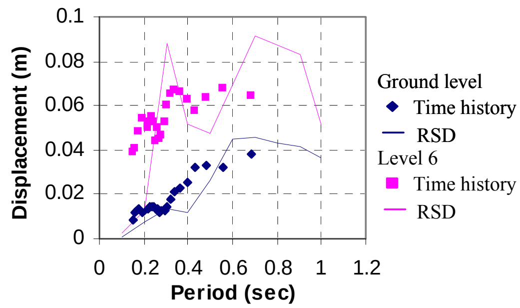

(c)

Figure 9 Comparison of the maximum displacement response of walls on the ground level and the top level, (a) Site class B, (b) Site class C, (c) Site class D

4. CLOSING REMARKS

A new force-displacement model characterising the out-of-plane response behaviour of unreinforced masonry walls subject to two-way bending has been presented. Parametric studies involving non-linear time-history analyses of the wall models were undertaken based on different site classes and the onerous conditions at the top floor of a six-storey building. The highest point on the linear elastic displacement response spectrum is shown to generally provide a conservative estimate for the maximum seismic displacement demand imposed on the URM walls.

5. REFERENCES

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