TIME-HISTORY ANALYSIS FOR UNREINFORCED MASONRY WALLS IN TWO-WAY BENDING

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ABSTRACT

Unreinforced masonry (URM) walls have been traditionally perceived to possess very little displacement capacity with respect to out-of-plane loading. This view, which has arisen due to low tensile strength of URM and therefore small displacements at which such walls begin to crack, has led to the widespread misconception among engineers that such walls possess no ductility and perform poorly under earthquake loading. However, both theoretical and experimental research have in fact shown that URM walls have the capacity to undergo significant deformations before collapse occurs, due to the rigid block action and frictional resistance mechanisms present in such walls. In addition, walls supported at their vertical edges have been shown to possess good energy dissipation characteristics which is further beneficial to seismic resistance. This paper presents a time history analysis model developed for simulating the dynamic response of URM walls subjected to out-of-plane loading. The hysteresis model incorporated into the analysis is capable of representing the non-linear load versus displacement behaviour of walls with a range of boundary conditions that include one-way and two-way walls. The various parameters in the model can be calculated as a function of the properties of the URM wall, including the dimensions, material properties, axial loading and boundary support conditions. Comparisons of the analytical model with experimental shaketable tests show promising results.

KEYWORDS: unreinforced masonry, out-of-plane two-way bending, nonlinear time history analysis, hysteresis model, seismic response.

1 INTRODUCTION

Research into the seismic response of unreinforced masonry (URM) buildings (Abrams 2001; Bruneau 1994; Brunsdon 1994; Calvi 1999; Maffei et al. 2000) has highlighted the need for improvements in our understanding of the behaviour of URM buildings under earthquake loading and in the corresponding procedures for earthquake resistant design. Significant advances have been made in Australia in recent years in the design methodology used to calculate the out-of-plane load capacity of URM walls, with the development of a virtual work method (Lawrence and Marshall 2000), applicable two-way spanning wall panels with a range of boundary support conditions. The method, which has been adopted by the current version of the Australian Masonry Code AS-3700 (Standards Australia 2001) provides the ultimate load bearing capacity which is used in a force based design procedure of laterally loaded URM walls for both wind and earthquake loading. However, whilst a forced-based approach is appropriate in the design of walls for wind loading, it has been shown to be vastly conservative for the ultimate limit state design of URM walls under seismic loading (Magenes and Calvi 1997; Priestley 1985), because it effectively limits the allowable deformation response of the wall to the displacement at which ultimate strength is reached (typically 5-15 mm). By contrast, out-of-plane collapse of URM walls is well known to be governed by deformation limits rather than the load bearing capacity, with the ultimate displacement capacity of a panel typically being close to the its thickness (equal to

110 mm for standard Australian URM construction), which justifies a move away from traditional force based methods in favour of displacement based design.

Recent joint research by the University of Adelaide and University of Melbourne into seismic response of URM buildings of has led to the development of a nonlinear time history analysis for vertically spanning URM walls (Doherty et al. 2002). The nonlinear load-displacement relationship incorporated into this analysis comprised a semi-empirical trilinear elastic model based on rigid block theory, in which the input parameters could be easily determined from the wall's geometry, self-weight and vertical pre-compression. The accuracy of the dynamic analysis was validated by extensive shaketable testing (Doherty et al. 2002; Lam et al. 2003), which subsequently led to the development of a simplified displacement based design procedure.

The aim of current research efforts by the Universities of Adelaide and Melbourne is to extend the seismic assessment procedure previously developed for vertically spanning walls to include two-way walls, which include any class of panels supported by a combination of horizontal and vertical edges. As part of this research effort, two sets of experimental studies involving clay brick URM walls were performed. The first study involved quasi-static cyclic tests on eight full scale walls aimed at characterising the nonlinear load-displacement behaviour (Griffith et al. 2007); and in the second study, shaketable tests were performed using five half scale panels which validated the response observed in the quasistatic tests under true dynamic loading and provided dynamic data to aid the development of a nonlinear time history analysis (Vaculik and Griffith 2007). The most significant behavioural trends observed in these experimental tests included large deformation capacity in excess of the wall thickness, hysteretic energy dissipation (hysteresis loops) upon cyclic loading and strength and stiffness degradation which was both cycle and deformation dependent. The nonlinear inelastic nature of the two-way wall response was significantly different and more complex than that of vertically spanning one-way walls studied by Doherty et al. (2002), justifying the need for the development of a new time history analysis capable of simulating the nonlinear load-displacement behaviour of two-way panels.

This paper presents a time history analysis developed for modelling the dynamic response of URM panels to out-of-plane seismic loading. Whilst the model was developed specifically for two-way panels, it has an inherent ability to also simulate the behaviour of one-way vertically or horizontally spanning walls. This paper is structured into three parts: Section 2 will describe the nonlinear load-displacement model with an outline of the methods used to calculate the various input parameters, Section 3 discusses the transformations used in idealising the masonry panel as an equivalent SDOF system and Section 4 provides comparisons of the dynamic analysis with experimental data obtained by shaketable testing.

2 NONLINEAR LOAD-DISPLACEMENT MODEL

It is generally recognised that the dynamic response of nonlinear systems can be modelled using step by step time history analysis (THA) procedures with acceptable accuracy, provided that the model incorporates a good approximation of the system's dynamic properties, including the mass, damping and nonlinear load-displacement behaviour. The proposed load-displacement hysteresis model for the flexural response of two-way has been developed to simulate the full range of response of URM walls in bending, including pre-cracking elastic response, progressive strength and stiffness degradation during cracking and post-cracking response as a result of residual strength – trends which were observed in quasistatic cyclic tests performed as part of this research (Griffith et al. 2007). The model is based on superposition of three component hysteresis rules, as shown by Figure 1. These components account for the various internal resistance mechanisms present in a URM wall, with each exhibiting a unique mode of load-displacement hysteresis. These include a residual elastic component (Figure 1a), residual inelastic component (Figure 1b) and a degradable component (Figure 1c), with the first two accounting for the behaviour of the masonry panel in its cracked state, whilst the third component represents the additional strength the masonry gains from the presence of mortar bond which degrades as the system undergoes damage due to deformation beyond its elastic limit. These three components can be defined by a total of 9 input parameters (5 if the panel is modelled in its cracked state). A brief overview of these component hysteresis rules and methods used to calculate their input parameters will now be given.

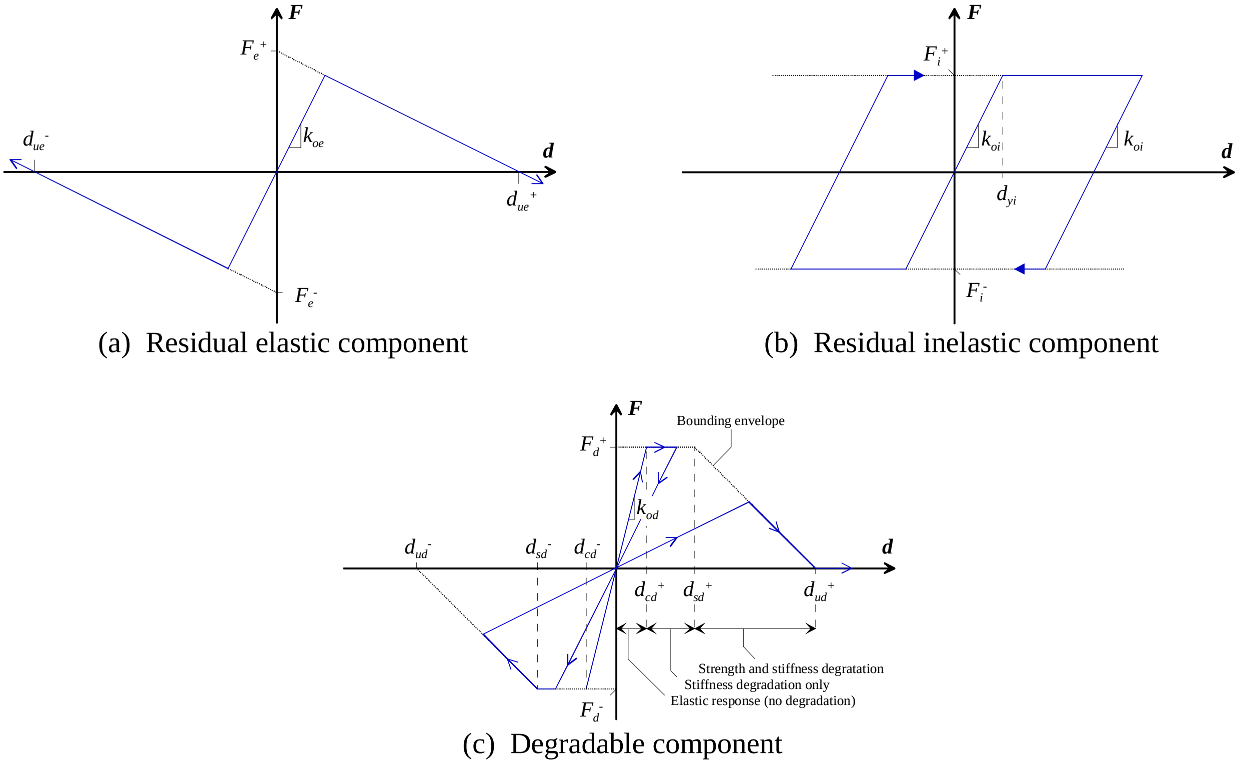
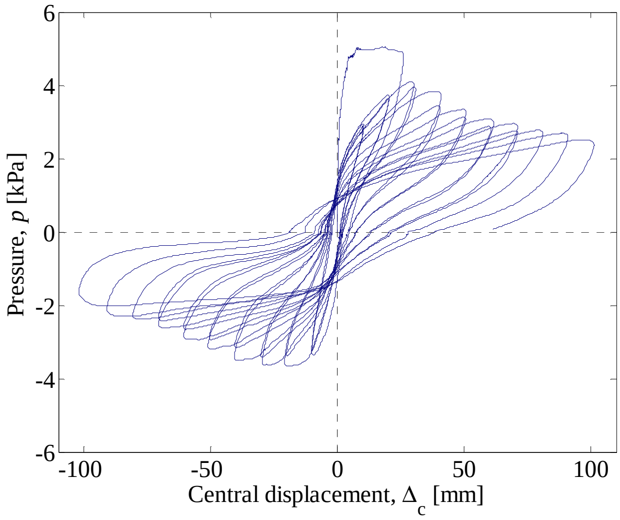


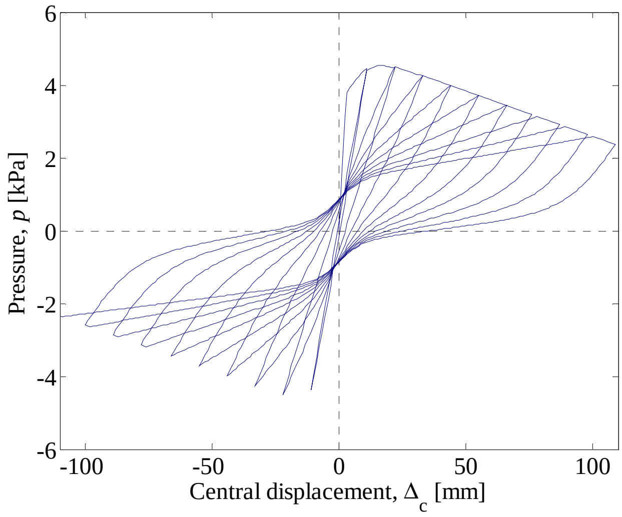
Figure 1. Component rules in the proposed hysteresis model

The residual elastic rule (Figure 1a) is similar to the elastic trilinear rule proposed by Doherty et al. (2002) for panels in vertical one-way bending and represents the elastic resistance provided by normal flexural stresses acting upon the bed joints in a cracked panel. It is modelled as a bilinear elastic rule with a negative post yield slope to account for a reduction in the panel's load resistance arising from rigid body stability effects. The residual elastic strength F_e can be calculated using the virtual work method by accounting only for bed joint flexural resistance along horizontal and diagonal crack lines. The instability displacement d_{ue} can be obtained using rigid body theory by taking into account the shape of the subpanels involved in the collapse mechanism (refer to Figure 5). The residual inelastic rule (Figure 1b) represents the inelastic shear frictional resistance of bed joints within a cracked panel. The rule is modelled as a simple elastoplastic hysteresis, which with a constant post-yield stiffness assumes that there is no loss or gain in the frictional resistance with increasing panel displacement. The author believes this to be a reasonable assumption for the displacement range of interest. The residual inelastic strength F_i may be obtained using the virtual work method by accounting solely for the inelastic frictional moment resistance along vertical and diagonal crack lines. Due to the complex internal resistance mechanics of two-way URM panels and joint degradation along the cracked bond interface, it is difficult to propose a fully rational method for calculating the initial lateral stiffness of cracked masonry to establish parameters k_{oe} and k_{oi}. The author has found, however, that the softening effect of crack formation can be roughly accounted for by applying a stiffness reduction multiplier (of approximately 0.1) to the initial elastic stiffness of the panel in its uncracked state, which in the case of two-way panels can be determined from elastic plate theory. The residual stiffness calculated using this approach can be subsequently distributed between the initial stiffness in the residual elastic and inelastic rules (k_{oe} and k_{oi}). Note that in the remaining figures in this paper, the residual elastic and inelastic rules are represented using hyperbolic functions which give a smooth transition zone between the initial stiffness slope and the 'post-yield' slope and provide a greater numerical stability in the step-by-step dynamic analysis procedure.

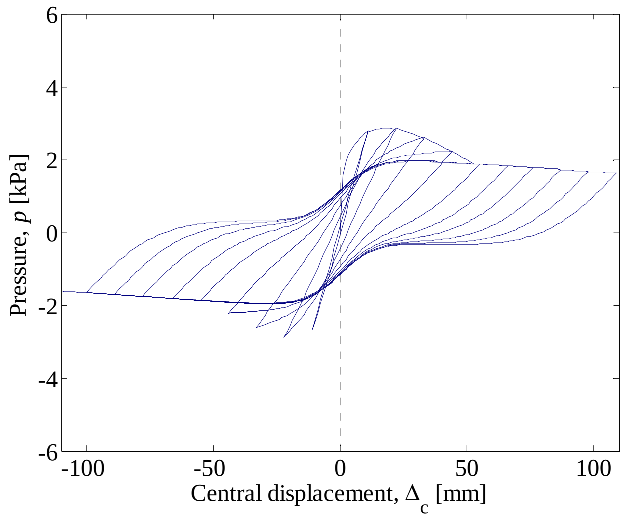
The degradable rule (Figure 1c) accounts for the additional strength the panel obtains from bond strength which diminishes as the panel develops progressive cracking. In the proposed form of the hysteresis rule, strength and stiffness degradation are modelled as being displacement dependent. The degradable strength F_d can be



(a) Experimental response



(b) Hysteresis model with parameters chosen to fit experimental response

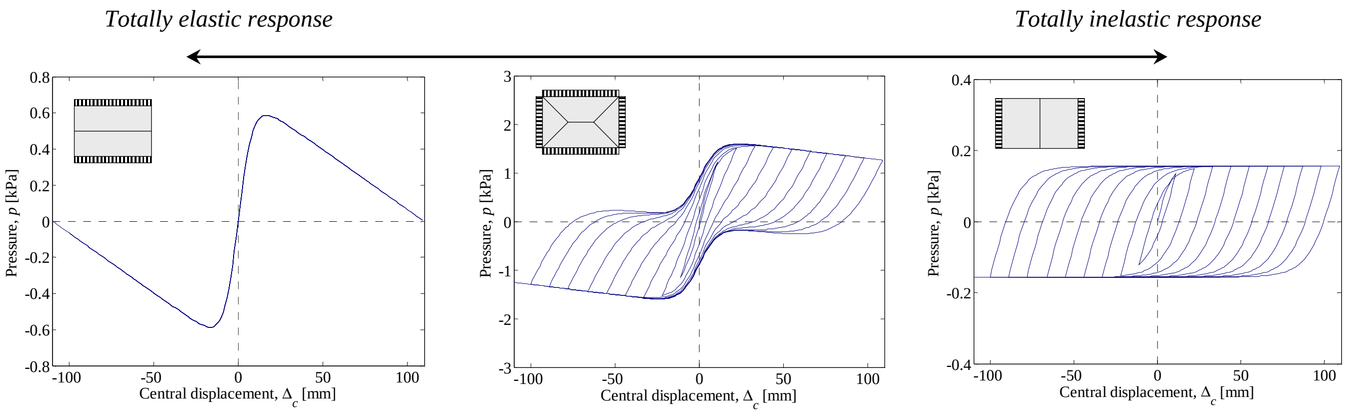


(c) Hysteresis model with parameters calculated from panel's physical properties

Figure 2. Comparison of hysteresis model with experimental response (panel s3)

calculated as the difference between the ultimate load capacity and the total residual load capacity (F_e + F_i). Similarly, the initial stiffness k_{od} is the difference between the initial elastic stiffness and the total residual stiffness (k_{oe} + k_{oi}). The displacements d_{sd} and d_{ud} which define the strength envelope depend on complex modes of internal stress redistribution within the panel as a result of progressive cracking and need to be determined empirically. The reader is referred to Griffith et al. (2007) for experimental test results which demonstrate typical cyclic strength envelopes of two-way panels.

Superposition of the three component hysteresis rules results in an overall load-displacement hysteresis model which is in reasonably good agreement with the general load-displacement trends observed in the quasistatic cyclic tests. An example of such a comparison is shown in Figure 2 for a clay brick panel (panel s3, Griffith et al. 2007) with dimensions 4080 × 2494 × 110 mm (length × height × thickness), full moment restraint at its vertical edges and simple support at the horizontal edges, an asymmetrically positioned opening and 0.1 MPa axial pre-compression at the top edge. In this example, the cyclic experimental response (Figure 2a) is compared to the hysteresis model for two cases. In the first case (Figure 2b), the input parameters for the model



(a) Vertically spanning one-way panel (F_e = 0.725 kPa, F_i = 0)

(b) Two-way panel supported at all four edges (F_e = 0.830, F_i = 0.893 kPa)

(c) Horizontally spanning one-way panel (F_e = 0, F_i = 0.156 kPa)

Figure 3. Examples showing the influence of a panel's boundary conditions on the residual hysteretic response

were chosen manually in order to demonstrate the model's ability to provide a good fit of the experimental curves. In the second case (Figure 2c), the input parameters were calculated a priori using the physical characteristics of the panel with analytical methods for strength, displacement and stiffness calculation as outlined previously. Whilst, in the author's opinion the parameters calculated a priori still provide a good representation of the general hysteretic response measured by experiment, the example highlights the sensitivity of the overall shape of the curves to the analytical tools used to calculate the model's input parameters.

A further significant feature of the proposed hysteresis model is the versatility in its capability to simulate panels with any degree of elastic character (i.e. hysteretic energy dissipation). For example, in the post-cracked state a panel with a high L/H aspect ratio will respond predominantly through elastic resistance mechanisms and exhibit low hysteretic energy dissipation, whilst a panel with a low L/H aspect ratio will respond predominantly through inelastic mechanisms and thus exhibit high hysteretic energy dissipation. The advantage of the proposed model is that the degree of overall elasticity of the masonry panel is inherently accounted for by the relative magnitudes of the input parameters for the post-cracking elastic strength F_e and the inelastic strength F_i which may be calculated using the virtual work method. Consequently, the model has the capability to simulate not only two-way panels, but also one-way horizontally and vertically spanning panels (Figure 3).

3 DYNAMIC ANALYSIS USING THE SUBSTITUTE STRUCTURE APPROACH

In dynamic modelling of multi-degree-of-freedom (MDOF) systems which respond in a dominant vibrational mode, a substitute structure approach may be implemented. In this approach the MDOF system whose properties including mass and displacement are spatially distributed is idealised as an equivalent single-degree-of-freedom (SDOF) system with a lumped mass and displacement. URM walls in out-of-plane bending fit this requirement, because following cracking they tend to deform in a predominant deflected shape defined by the wall's crack pattern. It is generally accepted that the collapse mechanisms of URM panels in bending can be idealised as a series of flat subpanels with rotational deformations being concentrated along hinge lines (Figure 4). In the Australian Masonry Standard AS-3700 virtual work procedure, these collapse mechanisms are determined directly from the panel's geometry and boundary support conditions. Several examples of such collapse mechanisms are shown by Figure 5 for various support arrangements.

Apart from the underlying assumption of a dominant vibrational mode, the transformation of the dynamic properties of a MDOF system to an equivalent SDOF system is based on the criteria that the MDOF and SDOF systems have equal base shear and perform an equal amount of energy. Based on these conditions, explicit

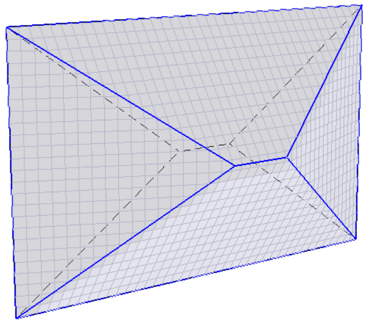
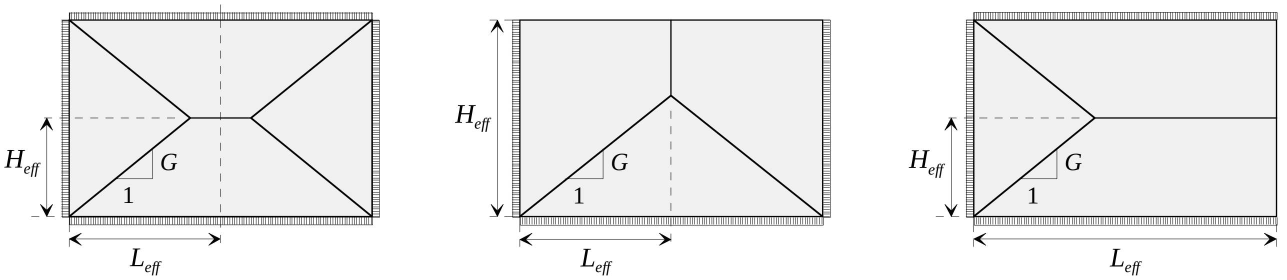


Figure 4. Three dimensional visualisation of the idealised displaced shape of a two-way spanning wall supported at all four edges



(a) Panel supported at all four edges

(b) Panel free at one horizontal edge

(c) Panel free at one vertical edge

Figure 5. Effective length and height of two-way panels for various boundary support arrangements

expressions have been derived to perform these transformations for the general AS-3700 collapse mechanisms for solid two-way panels, as follows.

$$\frac{\Delta_{dyn}}{\Delta_c} = \frac{2\beta - 1}{3\beta - 1} \qquad (3.1)$$

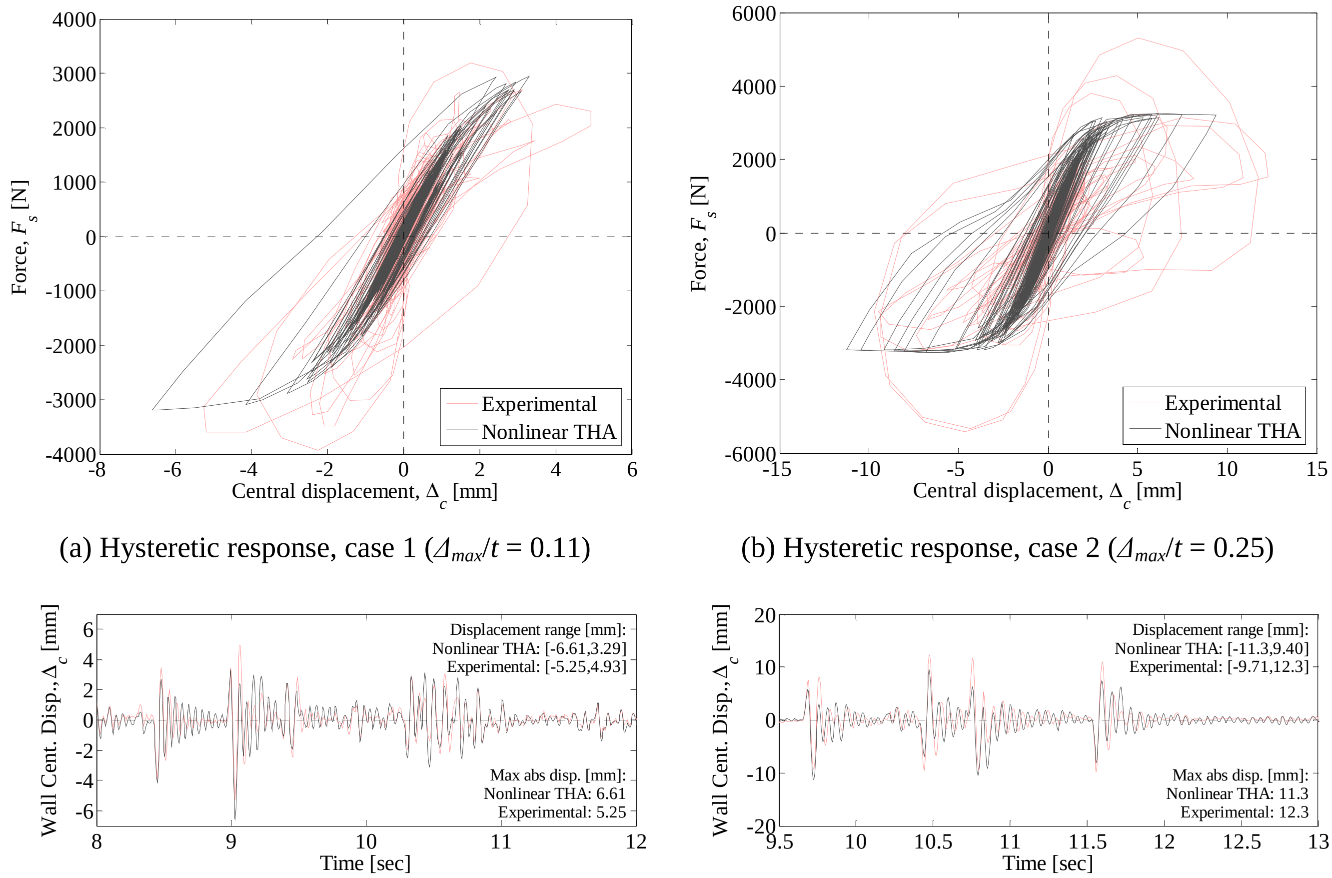
$$\frac{M_{dyn}}{M} = \frac{1 - 6\beta + 9\beta^2}{-6\beta + 12\beta^2} \qquad (3.2)$$

Equation (3.1) relates the effective dynamic displacement of the SDOF system Δ_{dyn}, to the maximum displacement along the actual panel Δ_c; and equation (3.2) relates the effective dynamic mass of the SDOF system M_{dyn}, to the actual mass of the panel M. The parameter β is related to the effective aspect ratio of the panel, α, which takes into account the diagonal crack slope G, and the effective dimensions of the panel L_{eff} and H_{eff}, as illustrated by Figure 5. These effective dimensions L_{eff} and H_{eff} are taken as the full span when only one span end is supported, or as half the span when both span ends are supported. Parameters α and β are calculated using the following expressions.

$$\alpha = G\frac{L_{eff}}{H_{eff}} \qquad (3.3)$$

$$\beta = \max\left(\alpha, \frac{1}{\alpha}\right) \qquad (3.4)$$

Because the parameter β may assume values ranging from 1 to infinity, the ratio Δ_{dyn}/Δ_c obtained by equation (3.1) ranges between 1/2 (when β = 1) and approaches 2/3 when β becomes sufficiently large. Similarly, the ratio M_{dyn}/M as given by equation (3.2) ranges between 2/3 (when β = 1) and approaches 3/4 when β becomes sufficiently large. The ratios at the limiting case when β approaches infinity are equal to those determined by Doherty et al. (2002) and effectively represent a scenario whereby the wall become sufficiently long to be treated as a one-way spanning wall. Once the panel's dynamic properties are transformed to an equivalent



(a) Hysteretic response, case 1 (Δ_{max}/t = 0.11) (b) Hysteretic response, case 2 (Δ_{max}/t = 0.25)

(c) Displacement-time trace, case 1 (Δ_{max}/t = 0.11) (d) Displacement-time trace, case 2 (Δ_{max}/t = 0.25)

Figure 6. Comparison of dynamic analysis with experimental response for earthquake excitation

SDOF system using the methods outlined herein, the dynamic time history analysis can be performed by solving the equation of motion for a given loading or acceleration history using standard step by step procedures, such as the constant acceleration method or the linear acceleration method. It is important to note that the computed displacement history of the SDOF system using the effective dynamic displacement Δ_{dyn} needs to be transformed back into the MDOF domain using equation (3.1) in order to obtain the central wall displacement Δ_c.

4 VALIDATION OF DYNAMIC ANALYSIS USING EXPERIMENTAL DATA

The accuracy of the proposed time history analysis was scrutinised by comparing the predicted dynamic response to experimental data obtained by shaketable tests (Vaculik and Griffith 2007). This experimental study was performed on five URM panels whose configurations were designed to represent half scale replicas of panels used in the test study involving quasistatic cyclic loading using airbags (Griffith et al. 2007). Throughout the shaketable tests, measurements of displacements and accelerations were taken at key locations along the panels and the support frame, which were subsequently used to determine the average wall and support accelerations. Dynamic analyses of the test walls were conducted using the proposed nonlinear load-displacement model (Section 2) and the equivalent SDOF system transformations (Section 3), with the input acceleration histories taken as the average support accelerations measured during the shaketable tests.

Figure 6 shows a comparison of two shaketable test runs and the corresponding dynamic analyses, which the author considers to be representative of the general performance of the time history analysis. The solid masonry panel chosen for this example (panel d1, Vaculik and Griffith 2007) had dimensions 1840 × 1232 × 50 mm (length × height × thickness) with simple supports at the horizontal edges and full moment restraint at the vertical edges and 0.1 MPa axial load applied at the top of the wall. Input parameters for the load-displacement model were calculated a priori using the panel's physical characteristics, with material properties including the Young's modulus of elasticity and coefficient of friction obtained from material tests on the test masonry. At the stage at which the presented dynamic tests were performed, the panel had already been previously cracked and exhibited a full cracking pattern consistent with the typical idealised collapse mechanism (refer to Figure 4 and Figure 5a). Since the wall was already cracked, the degradable component of the load-displacement model (Figure 1c) was assumed to be completely degraded ($F_d = 0$). The viscous damping ratio was taken as 0.02.

As shown by Figure 6, the performance of the time history analysis is quite favourable. The two test cases shown by the figure were chosen to demonstrate the performance of the time history analysis at two different magnitudes of displacement. In the first case (Figure 6a and c) the ratio of the maximum central displacement and the wall thickness (Δ_c/t) was 0.11, whilst in the second case (Figure 6b and d) ratio was 0.25. In test case 1, the maximum deformation predicted by the time history analysis was 6.61 mm, which only slightly overpredicts the experimental response of 5.25 mm. In test case 2, the results are also favourable with a predicted maximum deformation of 11.3 mm being quite close to the experimental response of 12.3 mm. Furthermore, the displacement time trace plots in the vicinity of the maximum deformation (Figure 6c and d) show that the time history analysis also managed to capture the general waveforms of the experimental response. It has been observed however, that the accuracy of the analysis was quite dependent on a good estimate of the initial stiffness of cracked masonry panel. It is seen from the hysteresis plots (Figure 6a and b) that in these particular cases, the initial cracked stiffness (k_{oe} and k_{oi} in Figure 1) closely resembled that of the experimental response. However, this apparent sensitivity of the displacement response on the input parameters of the model underlines the importance of accuracy in the methods used to calculate these parameters, especially the ultimate and residual load capacities and lateral stiffness of the panel in the post-cracked state.

5 SUMMARY

An original hysteresis model together with a substitute structure approach for conducting a dynamic time history analysis of two-way URM walls subjected to out-of-plane earthquake loading has been presented. The proposed hysteresis model has the capability to simulate the full range of nonlinear inelastic response of such panels, including their behaviour before and after cracking. Comparisons with quasistatic cyclic load tests on full scale URM panels show that the model has the capability to simulate the main hysteretic trends, but that the reliability of the model is dependent on the accuracy of the methods used to calculate its input parameters. The dynamic analysis also shows promising results, with the numerically computed displacement response being in good agreement with experimental shaketable tests for realistic earthquake motions. It is expected that this work will ultimately lead to the development of a simplified displacement-based procedure for the seismic design and assessment of one-way and two-way URM walls subjected to out-of-plane loading.

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