Out-of-plane seismic response of stone masonry walls: Experimental and analytical study of real piers

T. Ferreira, R. Vicente, H. Varum & A. Costa
University of Aveiro, Portugal

A. A. Costa
Faculty of Engineering of the University of Oporto

SUMMARY:
This paper presents the application of an existing simplified displacement-based procedure to the characterization of the nonlinear force-displacement relationship for the out-of-plane behaviour of unreinforced traditional masonry walls. According to this procedure, tri-linear models based on three different energy based criteria were constructed and confronted with three experimental tests on existing stone masonry constructions. Moreover, a brief introduction is presented regarding the main characteristics of the *in situ* cyclic testing recently carried out using distributed loads, as well as results obtained during the experimental campaigns performed. The comparison between the experimental and the analytical results are presented and discussed.

Keywords: URM walls, out-of-plane, seismic assessment, displacement-based procedure, in situ test

1. INTRODUCTION

Recent earthquakes around the world, including the Azores archipelago (Portugal), have identified the out-of-plane collapse of URM walls as one of the most important failure modes. This suggests that existing URM walls may be vulnerable to future earthquakes and should be studied for their seismic resistance. Thus, during the last years, several studies focused on the seismic response of masonry structures as well as on its structural retrofitting. A wide range of approaches from analytical modelling to extensive experimental and numerical methods have been used, where micro-models as well as single and multi-degree-freedom (MDOF) macro-models have been applied in this kind of analysis (Kelly (1995); Lam *et al.*(1995); and Blaikie & Davey (2002)).

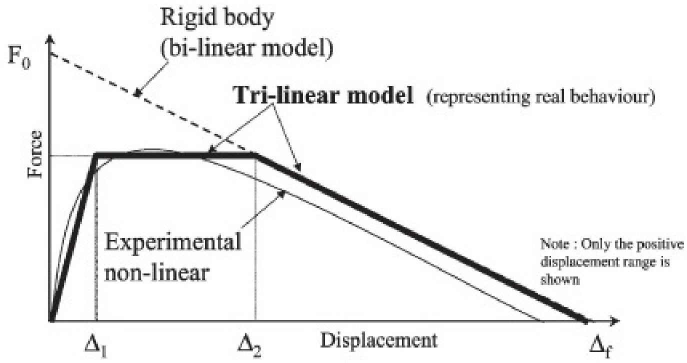
For simplicity, the behaviour of a wall subjected to out-of-plane forces may be modelled as a SDOF system (Doherty *et al.*, 2002; Derakhshan *et al.*, 2009). In fact, several researchers have studied the out-of-plane behaviour URM walls using this formulation and actually, all the nonlinearities associated with the material and construction practice of the URM walls, and the uncertainties involved in the selection of appropriate parameters for a complex numerical analysis,may render micro-modelling ineffective (Derakhshan *et al.*, 2009). In this paper, a SDOF idealization of the rocking behaviour of URM walls based on their force-displacement (F-Δ) relationship is presented and described. This idealization was developed and is applied to URM cantilever walls.

2. PRESENTATION OF THE SIMPLIFIED DISPLACEMENT-BASED PROCEDURE

2.1. Introduction

The simplified displacement-based procedure developed and applied in this work is based on the tri-linear *F-Δ* relationship originally described by Lam *et al.* (2003). This original formulation was based on statics assuming rigid body behaviour of the wall and neglecting the effects of its deformation and degradation. Based on this *F*-Δ relationship, a tri-linear model can be constructed for a one-way out-of-plane URM wall, knowing its mass, boundary conditions, overburden and dimensions.

To construct the tri-linear model, two ratios Δ_1/Δ_2 and Δ_2/Δ_f are used in conjunction with the bi-linear rigid body model of the wall. The displacement values Δ_1 and Δ_2 control respectively the initial stiffness reduction and strength reduction and Δ_f represents the maximum stable displacement (see Figure 1).



Tri-linear model parameters

Condition	Δ_1/Δ_f	Δ_2/Δ_f
New	6%	28%
Moderate	13%	40%
Severe	20%	50%

Figure 1. Bi-linear and tri-linear force displacement models (adapted from (Lam *et al.*, 2003))

In summary, Δ_1, Δ_2, Δ_f and Δ_0 are the only input parameters needed to define the tri-linear *F*-Δ relationship forming the macro SDOF model. Values for *F*-Δ and F_0 are first determined to construct the bi-linear spine based on the wall dimensions, boundary conditions and overburden loading conditions. The final tri-linear relationship is then defined according to representative values of Δ_1 and Δ_2 which account for the real non-linear behaviour of the wall (Lam *et al.*, 2003).

2.2. Construction of the bi-linear rigid body model

In this section the static response of a freestanding unreinforced masonry wall – cantilever wall - is described and its representation as an equivalent single degree-of-freedom system developed.

Considering rigid-body behaviour of a freestanding wall, it is possible to describe its behaviour using basic principles of static equilibrium. Considering the overturning equilibrium of the wall about the pivot point *O* located at the base of the wall, it is possible to obtain the force of incipient rocking (F_0) according to Eq. (1):

$$F_0 = \frac{3M \times g \times t}{8x} \tag{1}$$

where *M* is the total mass of the wall, *g*, the gravity acceleration, *t*, the wall thickness and *x*, the height of the resultant of the distributed load. For the cases which the wall slenderness ratio is low and the resultant of the horizontal distributed forces is applied at mid-height of the wall (Figure 2(b)) – as in the case of airbag testing (see section 3.1, test 1) - the value of the force of incipient rocking should be incremented due to the development of a compression strut located at the base of the wall. The value of this compression force may be calculated using Eqn. (2):

$$R = F_{max} \times A_v \times \tan(\theta) \tag{2}$$

where F_{max} is the maximum force attained during the experimental test; A_v, the horizontal cross-section area; and θ, the masonry angle of the internal friction (θ=38º, as mentioned by Betti & Vignoli, (2008)). Figure 2 shows the forces involved in the formulations analytically described by Eqn. (1) and Eqn. (2).

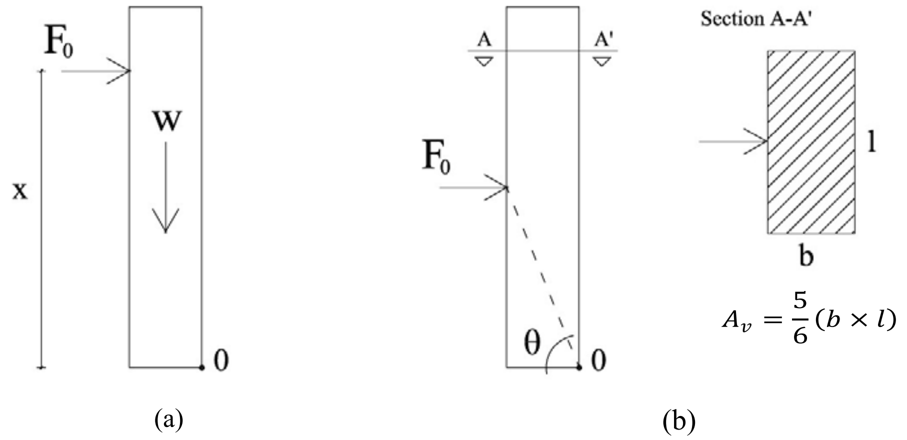


Figure 2. Forces and reactions on rigid URM parapet walls

Note that it is expected that the cantilever wall shown in Figure 2 experiences instability when Δ_f equals or exceeds 2/3 of the wall thickness, *b*. This displacement value corresponds to the moment when the centre of gravity of wall is vertical above the pivoting point, 0, and the resistance of the wall to overturning is zero. In this work, only the thickness of the outside lifer of the wall was used as maximum stability displacement, Δ_f. The use of this value is properly reasoned in Alexandre A. Costa (2012).

2.3. Calibration of parameters Δ_1 and Δ_2 based on energy criteria

In order calibrate the displacement limit values Δ_1 and $\Delta_2$based on experimental tests, three energy based criteria were defined. The first criterion considers the energy balance to the offset value corresponding to the maximum experimental envelope (in strength), d_{Fmax}. Figure 3 shows schematically the first energy based criterion.

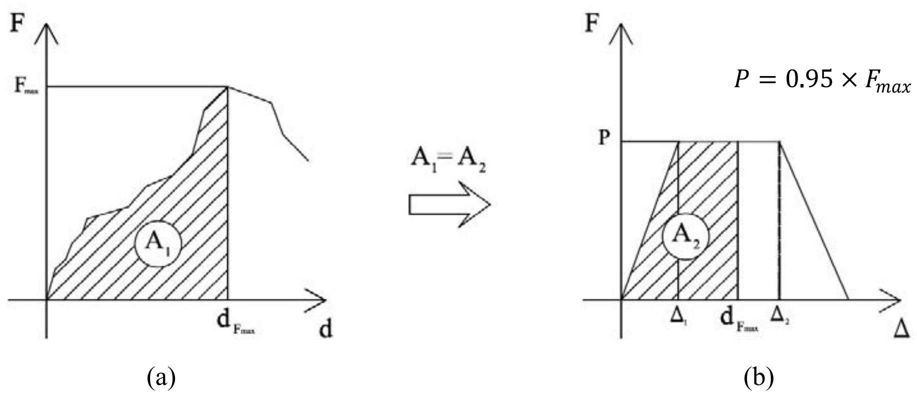
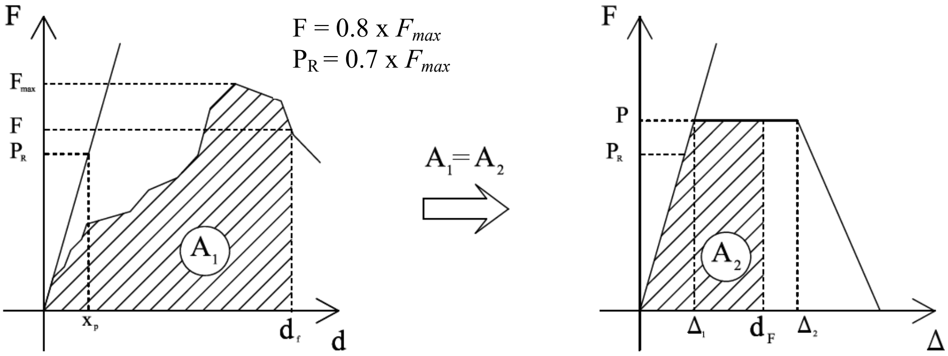


Figure 3. Energy balance to the offset value d_{Fmax}: (a) Experimental envelope; (b) Tri-linear approximation

Note that in this formulation the maximum strength value considerer in the tri-linear model, P, is equal to 95% of the maximum experimental strength value attained, F_{max}.

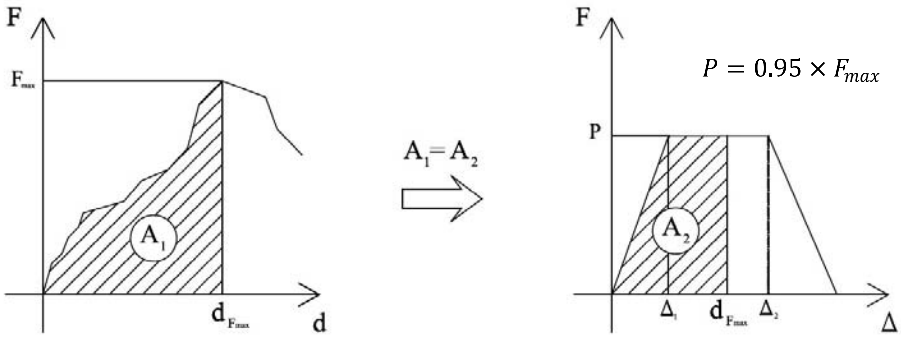
The second formulation is based on two assumptions: (i) the initial stiffness value is set at 70% of the maximum experimental strength values (according to NTC(2008)); and (ii) the values of $F_{máx}$ and P – the nonlinear *plateau* - are defined considering the energy balance to the offset value corresponding to 20% loss of strength after maximum experimental force value is reached ($F=0.8xF_{máx}$); it is possible to define, in an approximate way, Δ_1 and Δ_2. Figure 4 shows the energy based criterion described above.



(a) (b)

Figure 4. Energy balance to the offset value corresponding to 20% loss of strength after F_{max}: (a) Experimental envelope; (b) Tri-linear approximation

Finally, the third formulation consider the energy balance to the offset value of Δ_2, setting the initial stiffness at 70% of the maximum experimental strength value, F_{max} (NTC, 2008). As in the second formulation, variable P is defined here through energy balance ($A_1=A_2$). Figure 5 presents this third and last energy based criterion.



(a) (b)

Figure 5. Energy balance to the offset value Δ_2, setting the stiffness at 70% of the maximum experimental strength value, F_{max}: (a) Experimental envelope; (b) Tri-linear approximation

3. APPLICATION OF THE SIMPLIFIED DISPLACEMENT-BASED PROCEDURE

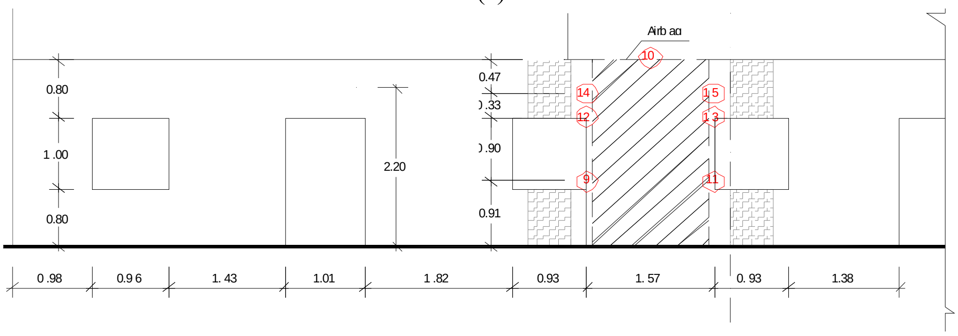
3.1 Experimental tests: presentation and description

This section briefly presents the three experimental tests, previously carried out by the authors and used in this work on the application of the simplified displacement-based procedure presented on section 2.

The first experimental test used as a case study was carried out on typical masonry houses from Azores, constituted by double leaf stone masonry with poor infill – also known as "*sacco*" masonry. Although these buildings normally present 1, 2 or 3 floors at maximum, they are seismically very vulnerable, as proved by the earthquake that hit the archipelago on the 9th July 1998. The test was performed using airbags on each side of the wall in order to mobilize its out-of-plane response under quasi-static cyclic loads.

Figure 6 presents the *in situ* implementation for tests and the operation scheme of the airbags on the wall.

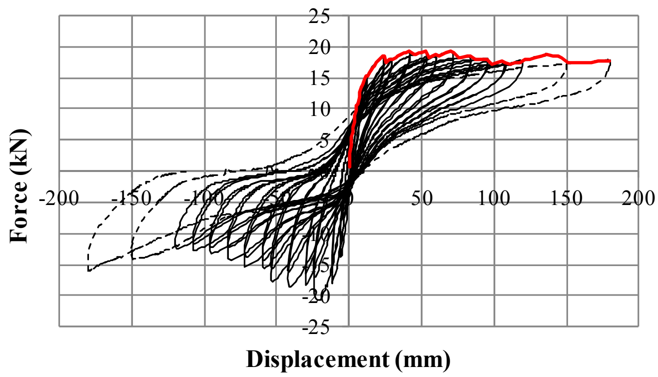
(a)



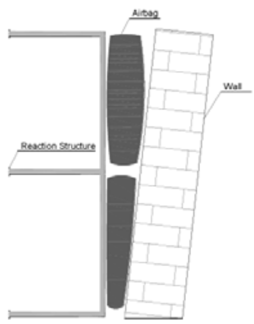
(b)

Figure 6. (a) *In situ* test; (b) façade and monitored points

In this test, the airbag pressure was slowly increased from 0 to the maximum value of about 6.4 kPa for which a maximum horizontal top displacement value of approximately 180 mm was reached. At this displacement point (monitored point 10; Figure 6 (b)), the test ended due to setup limitations in terms of maximum displacement. Figure 7 shows the experimental force-displacement envelope.



(a)



(b)

Figure 7. (a) Experimental envelope of test 1: distributed loads; (b) extreme displacement situation in the test

The remaining two *in situ* experimental tests used for the calibration of the analytical proposal were carried out on two traditional Azorean masonry houses located in the Faial island of Azores. This experimental campaign aimed to characterize the out-of-plane behaviour of stone masonry walls and strengthening solutions recommended for post-earthquake interventions. The tests were performed using an actuator placed on top of the wall which was used to mobilize its out-of-plane behaviour. Figure 8 presents the F-Δ envelopes obtained for both tests.

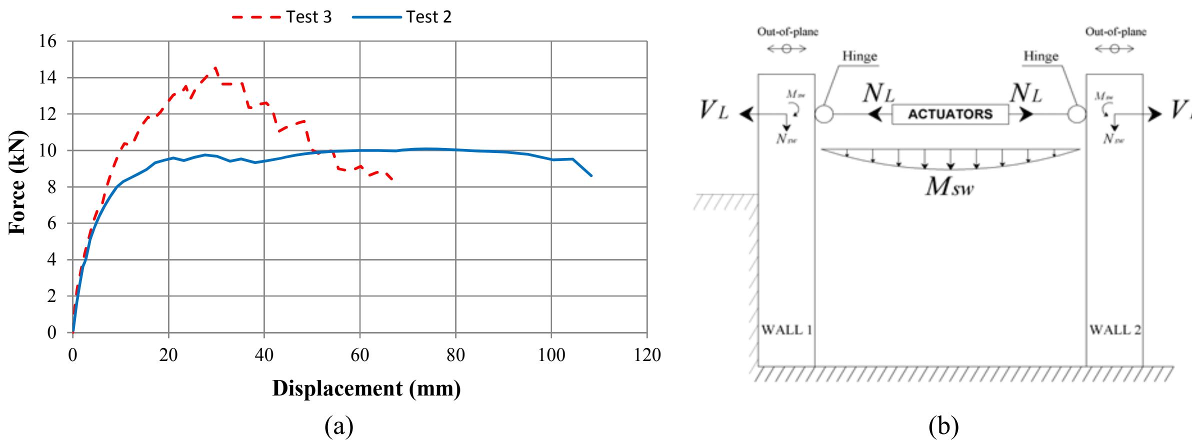


Figure 8. (a) Experimental envelopes of test 2 and test 3: point load at the top; (b) experimental test setup

For more information on the last mentioned experimental tests please see Costa *et al.* (2011).

3.3 Application of the tri-linear model

For the application of the proposal presented in section 2, the specific weight of 19 kN/m^3 was used. This value was chosen according to the recommended by Costa (2002) for this type of material and is consistent with the Italian Code (NTC, 2008) for materials with these characteristics.

For test 1, the geometry used was that presented in Figure 8 (b). Applying Equation (1) for a force resultant at 1.305 m, a force of incipient rocking (F_0) of 19.09 kN is obtained. As exposed in section 2.2, in order to taking into account the low slenderness ratio of the wall, this value of force of incipient rocking obtained should be corrected by the sum of a compression force given by Equation (2). As exposed before, considering an angle of the internal friction of 38^o (according to Betti & Vignoli, (2008)) and an horizontal cross section, A_v, of 1.05 m^2, for a maximum experimental force, F_{max}, of 19.24 kN, this compression force, R, is equal to 15.73 kN and consequently, the new force of incipient rocking, F_0, of 34.82 kN is obtained.

For test 2, and considering the geometrical configuration presented in Costa *et al.* (2011) – with a resultant applied at the high of 2.22 m –, a value of force of incipient rocking, F_0, of 13.61 kN is obtained. And finally for test three, considering a resultant at the high of 2.20 m, a value of force of incipient rocking F_0 of 18.41 kN is obtained. Note that these two values were obtained taking into account the partial participation of the lintels on the out-of-plane response of the pier by the development of shear forces on the lintels, V:

$$V = \tau_0 \times A_v \tag{3}$$

According to the Italian code (NTC, 2008) the value of the initial shear strength, τ_0, ranges between 20 and 30 kPa for the case of “*sacco*” stone masonry. According to the exposed, for case 2 and case 3, the force of incipient rocking was obtained by the sum of the value calculated through Equation (1) with the base shear force, V, affected with a correction factor, η, of 0.25 which regulates the participation of the lintels on the global response of the pier.

Figure 9 presents the confrontation between the experimental results and the tri-linear model developed, considering the three energy based criteria presented in section 2.3.

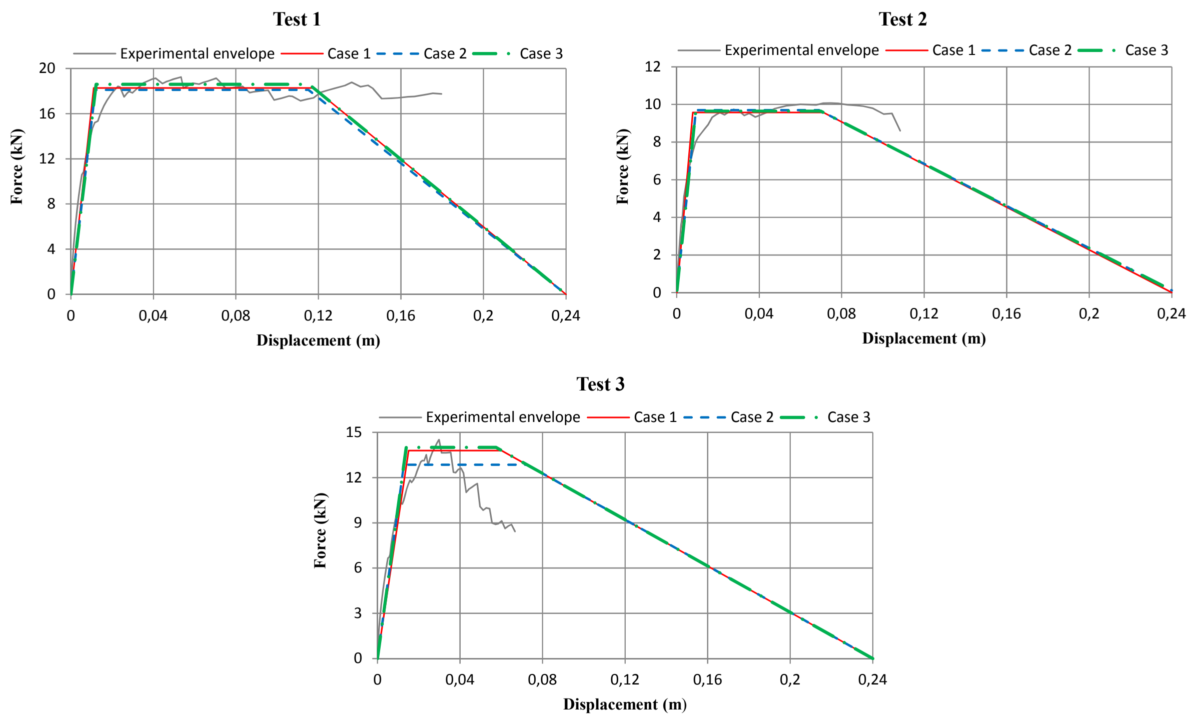


Figure 9. Experimental envelope vs. tri-linear model considering the three different energy based criteria

Table 1 presents the values of force and displacement obtained from the application above.

Table 1. Tri-linear model force and displacement parameters obtained for different energy based criteria

Energy based criterion 1

Test #	F_{max} (kN)	Δ_1 (mm)	Δ_2 (mm)	Δ_f (mm)	Δ_1/Δ_f (%)	Δ_2/Δ_f (%)
1	18.28	11.10	117.58	0.24	4.63	48.99
2	9.57	7.84	71.23	0.24	3.27	29.68
3	13.79	15.03	60.20	0.24	6.26	25.08

Energy based criterion 2

Test #	F_{max} (kN)	Δ_1 (mm)	Δ_2 (mm)	Δ_f (mm)	Δ_1/Δ_f (%)	Δ_2/Δ_f (%)
1	18.11	11.88	115.18	0.24	4.95	47.99
2	9.70	9.32	68.96	0.24	3.88	28.73
3	12.85	12.79	72.49	0.24	5.33	30.20

Energy based criterion 3

Test #	F_{max} (kN)	Δ_1 (mm)	Δ_2 (mm)	Δ_f (mm)	Δ_1/Δ_f (%)	Δ_2/Δ_f (%)
1	18.60	12.20	115.41	0.24	5.08	48.09
2	9.65	9.27	69.84	0.24	3.86	29.10
3	14.00	13.93	57.50	0.24	5.80	23.96

From the analysis of Table 1, it is possible to indicate new displacement values Δ_1 and Δ_2, calibrated for "sacco" stone masonry. As expected, for test 1 the ratio of $\Delta_2/\ \Delta_f$ cannot be directly compared with the same ratio obtained from test 2 and test 3. However, and comparing individually each one of these ratios, it is possible to state that these values vary significantly from the values suggested by other authors - (Doherty *et al.*, 2002); (Derakhshan *et al.*, 2009) and (Derakhshan & Ingham, 2008) – (see confrontation in Table 2). Values ranging between 4% and 6% seem reasonable for ratio $\Delta_1/\ \Delta_f$. Taking into account only test 1, values of $\Delta_2/\ \Delta_f$ about 50% seem appropriate. On the other hand, considering test 2 and test 3, this ratio decreases for values ranging between 30% and 40%.

Table 2 resumes the confrontation between the values obtained in this work and the values proposed in the literature.

Table 2. Different proposed $\Delta_i/\ \Delta_f$ ratios for the tri-linear model parameters

	Parameter	
Reference	$\Delta_1/\ \Delta_f$	$\Delta_2/\ \Delta_f$
Doherty et al. (2002)	13%	40%
Derakhshan, Ingham, and Griffith (2009)	1%	25%
Derakhshan and Ingham (2008)	2%	60%
URM wall (for distributed loads)	**4%-6%**	**50%**
URM wall (for point loads at the top)	**4%-6%**	**30%-40%**

In summary, the authors would like to note that in order to calibrate these and other tri-linear parameters, a new experimental laboratory test campaign is being prepared. Additionally, this new test campaign will allow to investigate the influence of the test setup and the number of displacement cycles per test over the tri-linear model.

4. FINAL REMARKS

The importance of out-of-plane seismic response of URM walls was emphasized with a particular reference to the characteristics of URM buildings in Azores, Portugal. The application of a simplified displacement-based procedure has been presented. According to this procedure, tri-linear models based on three different energy base criteria were constructed in order to match the results experimentally obtained in three experimental tests. Moreover, the presentation of a recent in situ experimental campaign carried out has been reported, where the application of cyclic loads resorting to an airbag system.

The parameters obtained to construct the tri-linear model were then confronted with values proposed in the literature. Due to their high dispersion, the results obtained need extra validation resourcing to laboratory testing, which will be the next step to be taken.

ACKNOWLEDGEMENTS

This work refers to research made with a financial contribution of the Portuguese Foundation for Science and Technology (FCT). The authors also thank the support of the Regional Government of Azores and the work of the technicians of the Laboratory of Earthquake and Structural Engineering in the experimental activity reported.

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