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FINITE ELEMENT ANALYSIS FOR THE ASSESMENT OF BUCKLING BEHAVIOUR OF MASONRY WALLS

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ABSTRACT

Masonry load bearing wall subjected to vertical concentric and eccentric loading may collapse through instability. In this Paper the buckling behavior of masonry load bearing wall of different slenderness ratio were investigated via testing a series of scale masonry wall subjected to concentric and eccentric vertical loading. A total of thirty six masonry walls were tested in the Laboratory of Technical University of Catalonia (UPC), which was the basis of this numerical study. For better understanding of buckling failure of the masonry load bearing wall, a numerical finite element model was developed based on the simplified micro model approach. The numerical model was calibrated by using those results found from the experimental study. The influence of tensile strength of units, nonlinear behavior of interface element, slenderness ratio and various end conditions have been investigated together with the effect of different end eccentricity of vertical load.

Keywords: *Buckling failure, Eccentric load, Masonry load bearing wall, Micro-modeling, Slenderness ratio.*

1. INTRODUCTION

Slender masonry load bearing wall subjected to vertical centric and eccentric loading may collapse through instability. This takes place if the compressive strength of the material is not reached any cross-section of the member. Otherwise, the failure occurs of the masonry wall due to crushing of the material itself. The failure mechanisms of masonry walls subjected to vertical loads are well documented, but a review of research carried out on masonry walls shows that due to the scarcity of test data there is no comprehensive method has been available for analyzing the complete load deformation relationships for slender walls of any chosen geometric configuration, material properties and load combination up to the collapse. Such an analysis requires consideration of the effects of both geometric and material nonlinearity. Chapman and Slatford (1957) obtained closed form solutions for the load deformation behavior of brittle elastic wall by assuming that masonry material has no tensile strength and that cracking occurs whenever a tensile stress would develop. Shalin (1978) reviewed the results of analysis carried out by a number of authors and presented experimental evidence in support of the calculations. Further work was carried out by Sawko and Towler (1982) who proposed a numerical procedure for calculating the failure load of a no-tension material wall. Some analytical solutions also have been worked out for linear elastic material with or without tensile strength. More recently, an analytical solution has been carried out by Romano et al. (1993), considering no tension bearing masonry with a monomial stress–strain relationship in compression. Parland et al. (1982) proposed a method for determining buckling failure load of a slender wall, taking into account the effect of tension stress field which exists between the cracked joints. However, the linear elastic materials were used in this analysis. Even though numerous experiments have been conducted on the strength and the failure modes of individual masonry wall under vertical loads, and several formulas have been proposed for the strength of a masonry wall corresponding to certain failure mode based on these experimental research results, no comprehensive theory is available to explain the interactions of different failure modes and the corresponding load-displacement relationship of masonry load bearing wall with effects of slenderness ratio and eccentricity of vertical loads, Payne et al. (1990). Yokel (1971) developed an analytical formula to determine the critical load of prismatic elements that, because of a very low tensile strength, have cracked sections. The study was based on a prismatic rectangular section, consisting of an elastic material, with a linear relationship between stress and strain and did not develop resistance to traction. A numerical model for the analysis of structural members under eccentric compression is presented by Vassilev et al. (2009). The equilibrium is formulated in the deformed state and takes account of the effect of deflections on the bearing capacity. The micro-modelling strategy, is considered at present as one of the most accurate tools available to model the behaviour of masonry structures, and has been adopted in the present research in order to carry out the needed numerical simulations. Micro-modelling allows, in particular, an appropriate simulation of the buckling response taking into account joint tensile cracking in combination with masonry crushing in compression. The tensile failure this phenomenon has been well identified, Page (1978). For shear failure, a

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softening process is also observed as degradation of the cohesion in Coulomb friction models. For compressive failure, softening behavior is highly dependent upon the boundary conditions in the experiments and the size of the specimen. Experimental concrete data provided by Schultz et al. (2009) indicated that the behavior in uniaxial compression is governed by both local and continuum fracturing processes.

In this study, the predictions of the ultimate capacity of walls obtained by means of micro-modelling approach and compared with experimental results obtained from the experimental study in structural engineering laboratory of Technical University of Catalonia (UPC). Moreover, results obtained in the parametric studies by considering different end support condition and effect of tensile stress on failure loads of walls were investigated. Conclusions are drawn on the relative importance of tensile strength, non-linear geometrical and material properties with different end eccentricity.

2. ADOPTED MODELING STRATEGY

The numerical simulation presented is performed with the well-known micro-model proposed by Lourenco & Rots (1997) requires more specific software oriented to masonry analysis. For all cases, micro-models assume 2D plain-stress and a hinged-hinged configuration. The hinges are modeled by means of stiff triangular objects placed at the bottom and at top of the wall, whose end vertex is allowed to freely rotate. In addition, a minimum eccentricity of 1mm is always applied in order to account for possible irregularities of the wall geometry of the load positioning. Basically, the model assigns an elastic behavior to the units whereas masonry inelastic behavior is transferred to the joints. This analysis was performed with DIANA software. The integration schemes used are 2x2 points Gauss integration for the continuum elements and 3 points Lobato integration for the interface elements. An interface allows discontinuities in the displacement field and its behavior is described in terms of a relation between the traction t and relative displacement u across the interface. In the multisurface interface model for the masonry proposed by Lourenco and Rots, the quantity of traction and displacement is denoted as generalized stress σ and generalized strain, ε. In this case the elastic constitutive relation between stresses and strain is given by:

$$\sigma = D\varepsilon \tag{1}$$

For 2D configuration $D = diag\{k_n, k_s\}, \sigma = (\sigma, \tau)^\tau \ and \varepsilon = (u_n, u_s)$

Where, n and s is the normal and shear components respectively. The terms in the elastic stiffness matrix can be obtained from the properties of both masonry components and thickness of the joint as:

$$k_n = \frac{E_u E_m}{t_m(E_u - E_m)}; k_s = \frac{G_u G_m}{t_m(G_u - G_m)} \tag{2}$$

Where, E_u and E_m = Young's moduli of units and mortar, respectively; G_u and G_m = Shear moduli units and mortar, respectively and t_m = thickness of the mortar joint.

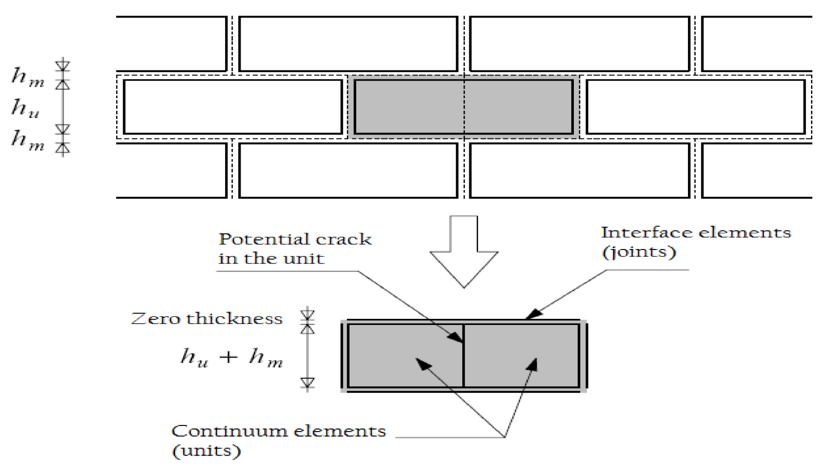

Figure 1 Modeling strategy. Units (u), which are expanded in both directions by the mortar thickness, are modeled with continuum elements. Mortar joints (m) and potential cracks in the units are modeled with zero-thickness interface elements

The interface model includes a compressive cap where the complete inelastic behavior of masonry in compression is lumped. This is a phenomenological representation of masonry crushing because the failure process in compression explained by the microstructure of units and mortar and the interaction between them. In the model the failure mechanism represented in such way that the global stress strain diagram is captured. The model was justified by Lobato et al. and found that the model is efficiently able to reproduce the experimental results. For this reason, the proposed micro-model was selected by the author to simulate the wall for buckling failure.

3. MODEL DESCRIPTION

In the numerical simulation, the units were modelled by using plain-stress continuum 8-node elements and for the mortar joints adopted 6-node zero-thickness line interface elements. In addition, hinges are modelled by means of stiff triangular objects. Each unit was modelled with 12 x 3 elements. The geometry and meshing of the wall for slenderness ratio 6 and eccentricity 0, t/6 and t/3 are shown in the Figure 2. For all cases, micro-models of wall considered hinged-hinged configuration. The hinges are modelled by means of stiff triangular objects placed at the bottom and at top of the wall, whose end vertex is allowed to freely rotate. The vertical load was applied concentrically and eccentrically as unit deformation. The boundary condition and loading configuration are also shown in the Figure 2.

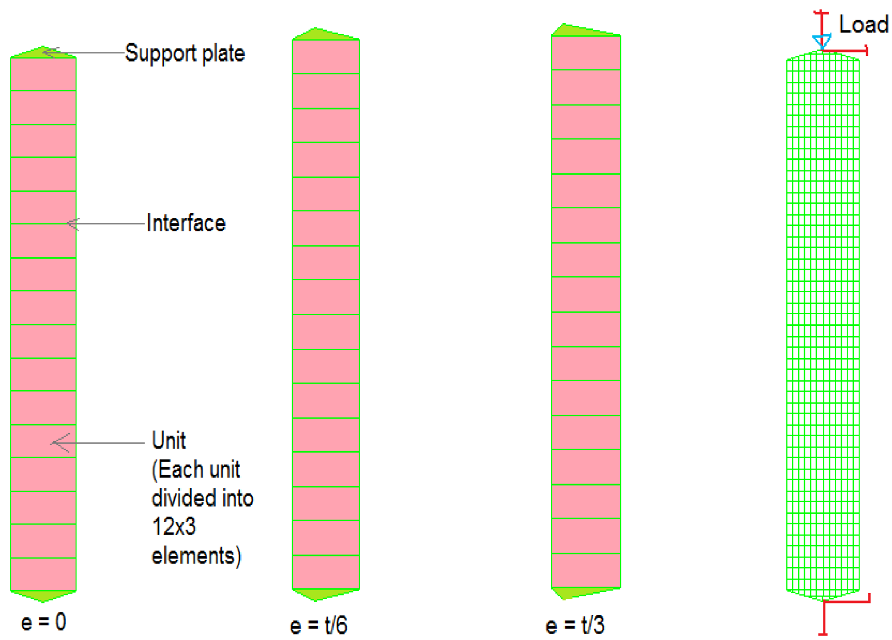

Figure 2 Geometry, meshing and load of walls for slenderness ratio 6

3.1 Material Properties

The material parameters used for the numerical simulation are shown in the Table 1. Some parameters such as $G^I{}_f$ and C_S have been taken directly from the previous research The fracture energy for mode I, $G^I{}_f$ have been taken from the test carried out by Van der Pluijm (1992) and for the parameter of shape of elliptical cap C_S a value of 9 has been adopted from Lourenco (1996). The interface elastic stiffness values were calculated from thickness of the joint h_j, the Young's moduli of unit and joint E_u and E_j, respectively, and the shear moduli of unit and joint G_u and G_j, respectively as CUR (1994):

$$k_n = \frac{E_u E_j}{h_j(E_u - E_j)}; k_t = \frac{G_u G_j}{h_j(G_u - G_j)} \tag{3}$$

The different strength values f_f , c and f_m have been obtained from the experimental study carried out in UPC (2009). The compressive fracture energy G_{fc} and equivalent relative displacement k_p calculated according to Model Code 90 and Eurocode 6, respectively by using followings formula (Lourenco, 1996):

$$G_{fc} = 15 + 0.43 f_m - 0.0036 f_m{}^2 ; k_p = \{ 0.002 - f_m (\frac{1}{E_u} + \frac{1}{k_n (h_u + h_j)}) \} \qquad (4)$$

Table 1 Material parameters adopted for numerical analysis

Components	Parameter	Symbol	Units	Values
Brick	Elastic modulus	E_u	N/mm^2	4800
	Poison ratio	v	-	0.15
	Tensile strength	f_{tb}	N/mm^2	3.95
	Normal stiffness	k_n	N/mm^2	2800
Joint	Shear stiffness	k_t	N/mm^2	1900
	Bond tensile strength	f_t	N/mm^2	0.554
	Mode – I fracture energy	$G^I{}_f$	Nmm/mm^2	0.02
	Cohesion	c	-	0.45
	Mode – II fracture energy	$G^{II}{}_f$	Nmm/mm^2	0.175
	Angle of internal friction	$tan\varphi$	-	0.812
	Angle of dilatancy	$tan\Psi$	-	0.009
	Compressive strength of masonry	f_m	N/mm^2	14.20
	Compressive fracture energy	G_{fc}	Nmm/mm^2	20.38

3.2 Validation of Model

The micro-models were validated next by a comparison with experimental results obtained from UPC (2010). Usually, experiments on load bearing walls have been adopted by the masonry community as the most common axial load test and the tensile capacity of masonry has been neglected. As a result, the clear understanding of the buckling characteristics of masonry load bearing walls under concentric and eccentric vertical load was absent. In this study, special attention is given to the load bearing wall tests carried out in the UPC (2010), because most of the parameters necessary to characterize the material model are available from micro-experiments. The main concern of this work was, to demonstrate the ability of the model to capture the behaviour observed in the experiments and close quantitative reproduction of the experimental results. The large number and variability of the material parameters necessary to characterize the developed model permits to adopt a set of parameters suitable to closely fit the experimental capacity.

3.3 Results of Numerical Simulation

The comparison between the experimental and numerical collapse load obtained from the micro- model is presented in the Figure 3. The above figure shows that the experimental behavior is satisfactorily reproduced and the collapse load estimated within a 15 % range of the experimental values. The micro-modeling approach is being able to provide a very satisfactory estimation of the experimental capacity of the walls particularly, for the case with e = 0. The average errors are 7.85%, 12.6% and 11.93% for the eccentricity of 0, t/6 and t/3 respectively. For all cases, one tendency is clear that with the increasing of slenderness ratio and application of load eccentricity the capacity of the wall decreased.

4. PARAMETRIC ANALYSYS

For this parametric study, vertical load is applied with different eccentricity and different boundary conditions such as hinge-hinge, hinge-fixed and fixed-fixed configurations. Four variables are investigated in the parametric study, namely, wall slenderness, loading conditions, boundary conditions and effect of tensile strength.

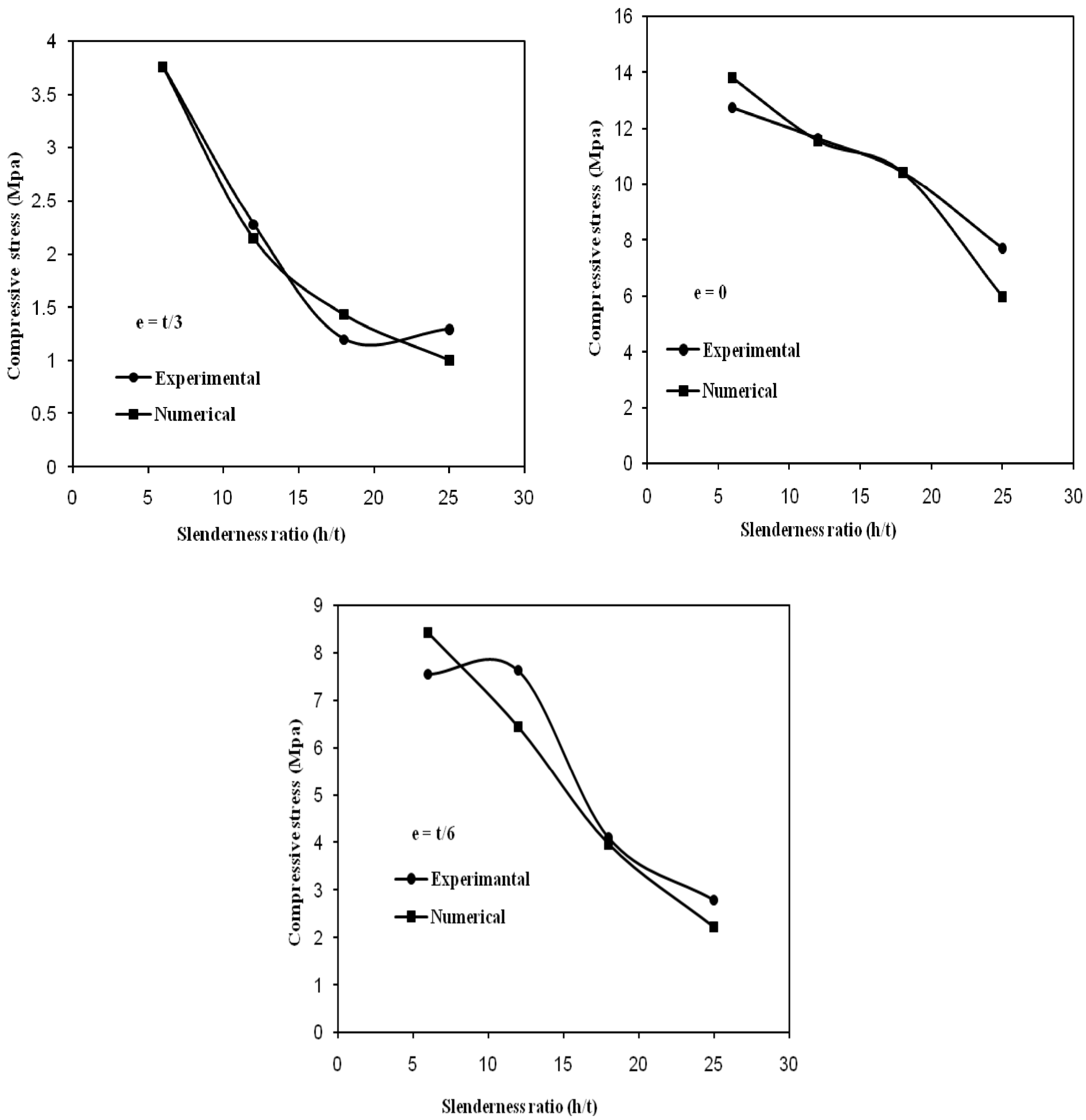

Figure 3 Comparison of compressive stress for different load eccentricity

4.1 Effect of Boundary Condition

In this study an investigation was carried out to better understanding the effect of boundary conditions on the strength and buckling behavior of masonry load bearing walls by using hinge-hinge, hinge-fixed and fixed-fixed configurations. The results of collapse load for different end conditions are shown in the Figure 4.

The above figure shows that due to the change of end condition from hinge-hinge to fixed-fixed, the ultimate capacity of wall increases significantly. This increment has small value for lower slenderness ratio and rate of increment increased with the increasing of slenderness ratio. In the case of eccentricity,e = 0 and slenderness ratio 25, when the support changes from hinge-hinge to hinge-fixed and fixed-fixed, the ultimate load capacity of wall increase more than two times and very close to three times respectively and for slenderness ratio 18 capacity increase around 1.5 times of hinge-hinge support. The capacity of the wall increases significantly for both cases of hinge-fixed and fixed-fixed support for eccentricity = t/6, however, the higher increment obtained for fixed-fixed support. Also found the similar tendency of higher increment rate for higher slenderness ratio. The ultimate capacity for slenderness ratio 25 increased three times and five times for hinge-fixed and fixed-fixed supports respectively and about two times and around four times respectively when consider slenderness ratio 18. For the other slenderness ratio the capacity increases within the ranges between 1.2 to 2 times of hinge-

hinge support. For the case of eccentricity t/3, the increment of capacity is higher than the other two eccentricities. The ultimate load increased around ten times for fixed-fixed support in cases of both slenderness ratio 25 and 18 respectively, while for hinge-fixed support about four times in cases of both slenderness ratio 25 and 18 respectively. On the other hand, the collapse load increased 4 to 6 times for slenderness ratio 6 and 12 with hinge-fixed and fixed-fixed end conditions respectively when compared with hing-hinge support.

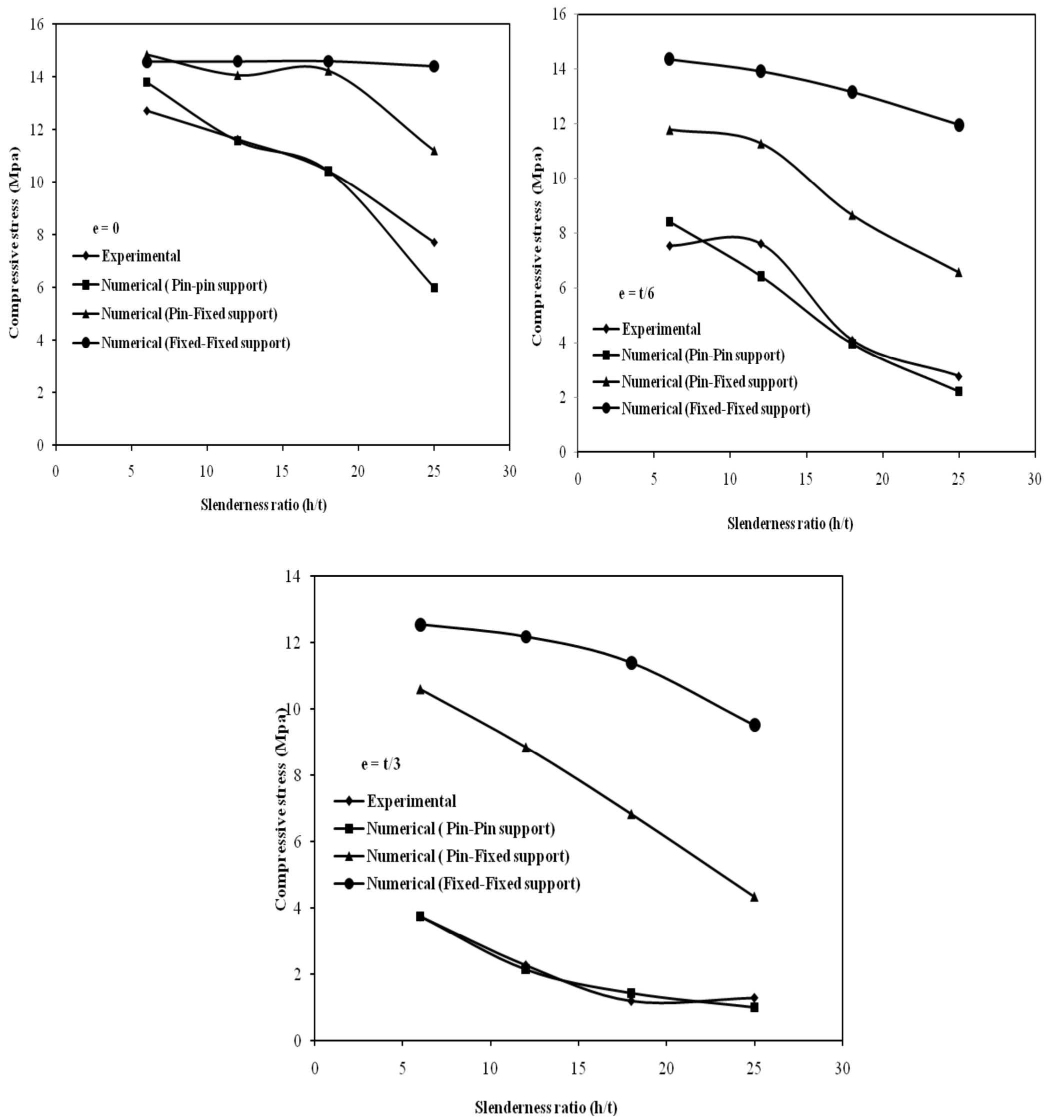

Figure 4 Capacity of wall for different end conditions with e = 0, t/6 and t/3

4.2 Effect of Tensile Stress

For better understanding the effect of tensile strength of masonry on the collapse load and buckling behavior, the parametric analysis was carried out by using tensile strength, f_t of 0.001, 0.284, 0.568, 0.852, 1.136 and 1.42 MPa which was 1%, 2%, 4%, 6%, 8% and 10% of masonry prism compressive strength respectively. The analysis was performed for the boundary conditions of hinge-hinge, hinge-fixed and fixed-fixed. The results obtained from this analysis are shown in the following figures below.

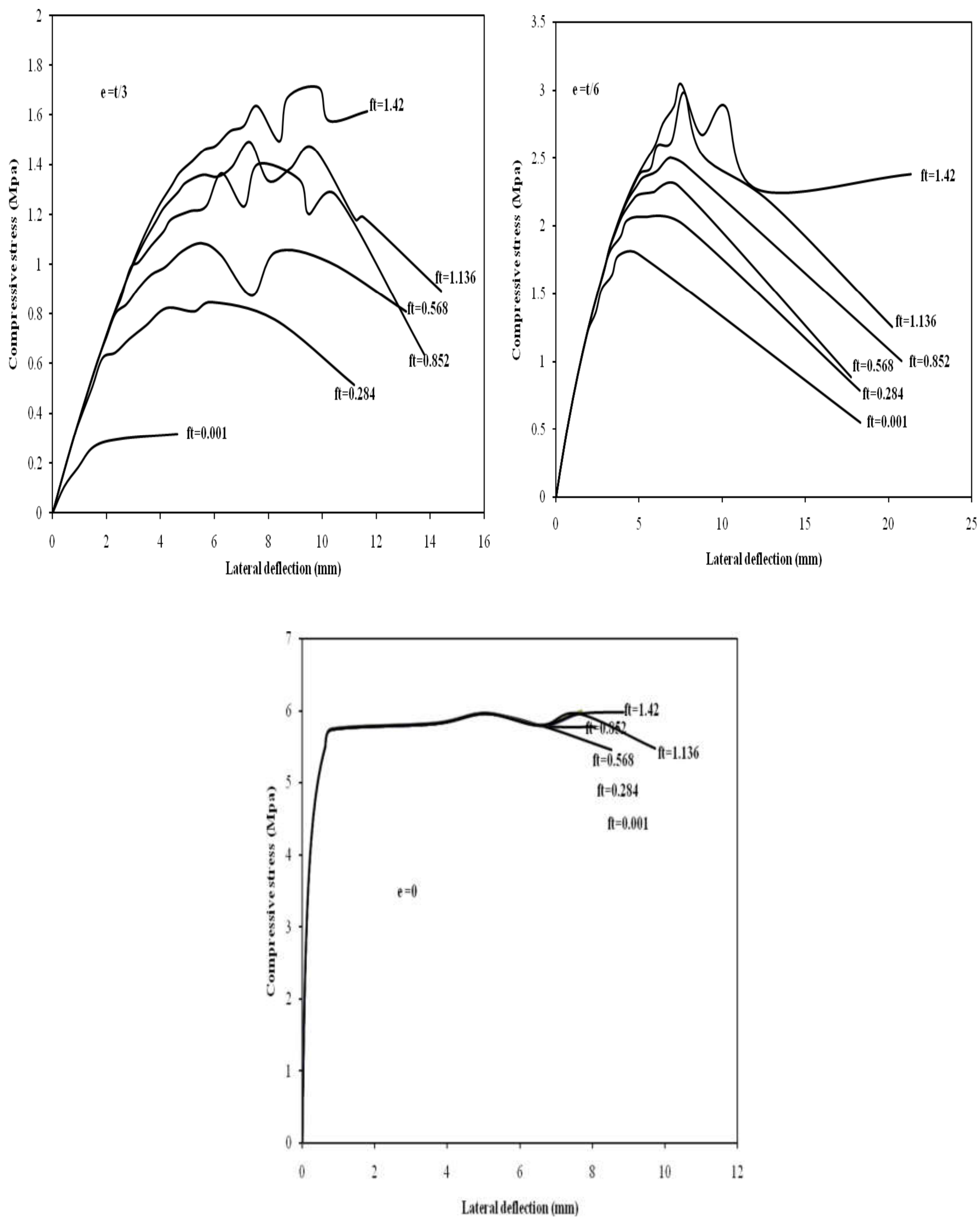

Figure 5 Load-deflection curve (hinge-hinge support) of different tensile strength for slenderness ratio 25

The Figure 5 shows that if the tensile strength varies from 0.001 to 1.42 MPa, the compressive stress of masonry increased from 0.31 to 1.49 MPa, 1.79 to 2.98 MPa and 5.96 to 5.98 MPa for the eccentricity t/3, t/6 and 0, respectively. The influence of tensile strength on ultimate capacity of masonry wall is very high in the case of higher load eccentricity and the influence decreases with the decreasing of load eccentricity.

In case hinge-hinge support, Figure 6 shows that the tensile strength has low effect on compressive strength for low eccentricity. The ultimate load increased 0.93 MPa and 1.40 MPa when the tensile strength of masonry varies from 1% to 10% of masonry prism compressive strength for eccentricity of load t/6 and t/3, respectively while the no effect of tensile strength on capacity in the case of eccentricity equal to zero.

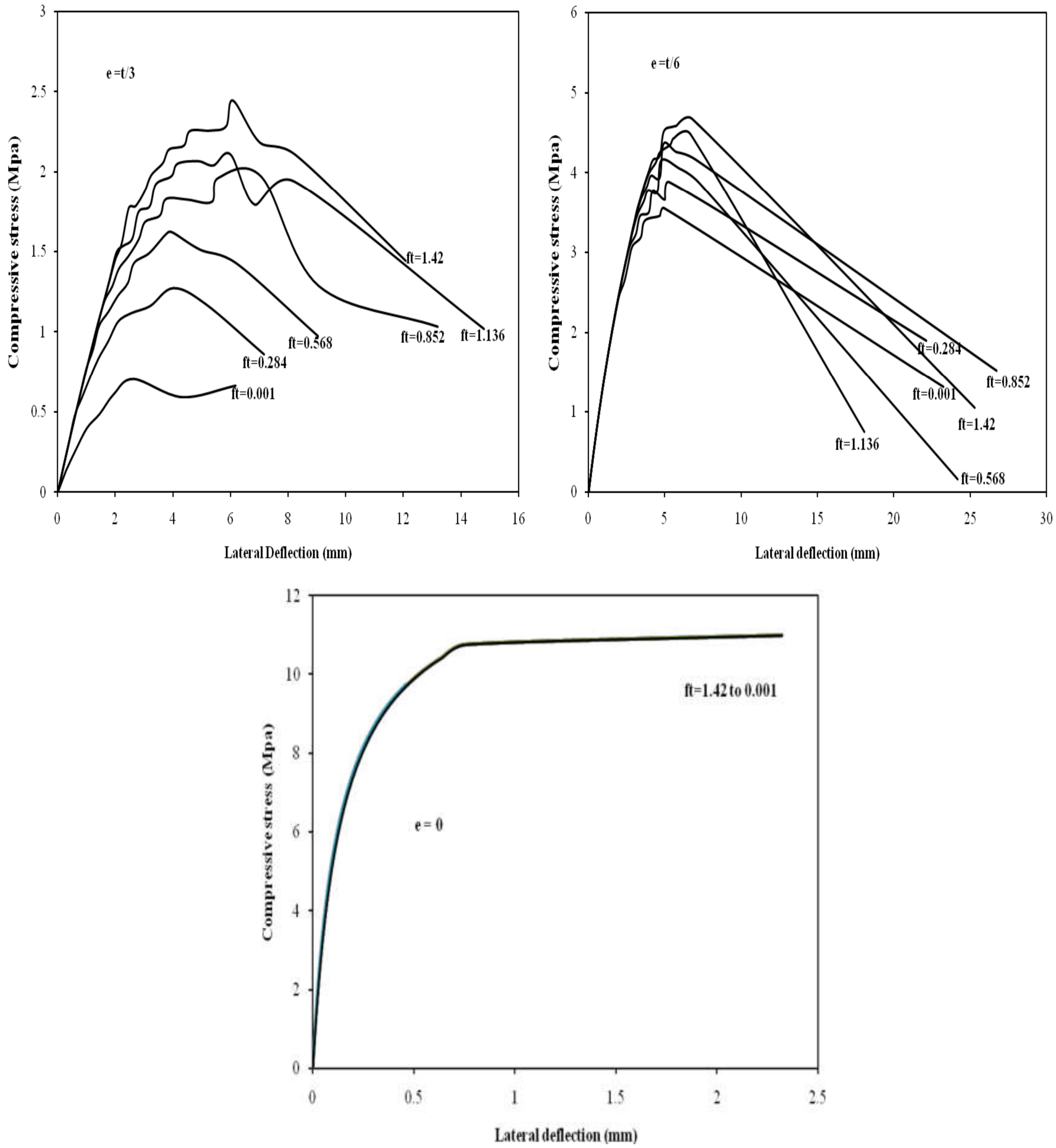

Figure 6 Load-deflection curve (hinge-hinge support) of different tensile strength for slenderness ratio 18

Figure 7 shows that when the tensile strength changes from 0.001 to 1.42 MPa for eccentricity e = t/3, the failure load changes from 1.23 to 3.28 MPa with irregular variation of lateral deflection. On the other hand the failure load chages very little from 6.25 MPa to 6.75 MPa in the case of eccentricity e = t/6 when the tensile strength variations are the same mentioned above. Moreover, the the variation of tensile strength has negligible effect on the ultimate load in the case of zero eccentricity, very similar to the zero eccentricity (slenderness ration 18) shown in the figure 6.

5. CONCLUSIONS

A set of experimental tests on the buckling failure of masonry walls has been numerically simulated by means of simplified micro-modeling approach. The micro-model described the nonlinear response of masonry in compression in an indirect way by localizing it to the units. In all cases, the non-linear response in tension is localized to the joints. The simplified micro-models afford a satisfactory prediction of the ultimate load of walls taking into account the buckling behavior. Simulations carried out by the micro-model provide the best fits for the test results with an acceptable error. It must be noted that some difference with respect to the experimental results is unavoidable because of the influence of possible non-reported accidental eccentricities. In the case of fixed support, the load capacity increased 2 to 6 times higher than hinge support depending on slenderness ratio and eccentricity. The capacity of wall for hinge-fixed support lies between the both end hinge and both end fixed support. In the case of hinge-hinge support with high eccentricity, the influence of tensile strength is higher than the other support conditions. Most of the cases, negligible effect was found for null eccentricity. The influence of tensile strength follow a common tendency from higher to lower values when the support condition and load eccentricity moves from hinge to fixed and higher to lower eccentricity, respectively.

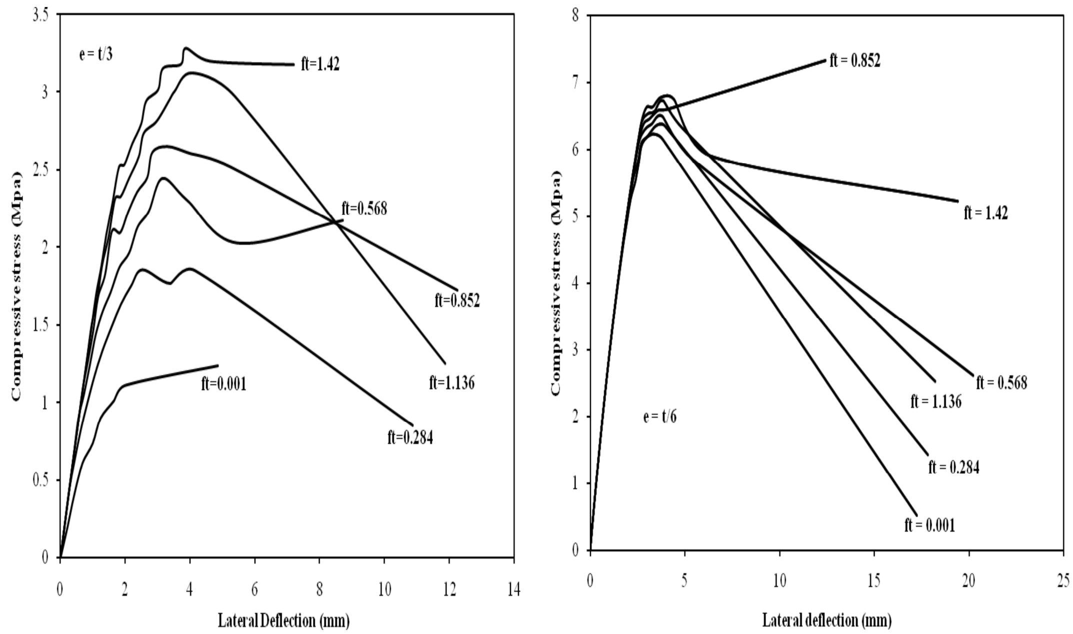

Figure 7 Load-deflection curve (hinge-hinge support) of different tensile strength for slenderness ratio 12

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