Modelling of the Lateral Load behavior of Unreinforced Masonry Walls using the software Atena 2D

I. Movilă& G. M. Atanasiu
Technical University 'Gheorghe Asachi' of Iasi, Romania

F. A. Zahn
HTWG Konstanz, University of Applied Sciences, Germany

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SUMMARY:
In regions where strong ground motions are anticipated, it is not economical to design shear wall buildings to remain elastic. Therefore, inelastic deformations are required as a mean of reducing the seismic demands. This paper contains some results of the study regarding the lateral load behaviour of unreinforced masonry (URM) walls using ATENA 2D nonlinear finite element software. The results were compared with experimental data obtained by Ötes and Löring (2003).The main purpose of the study is the comparison between the results obtained analytically and those given by experimental testing. Afterwards, finding a new equivalent material for modelling the behaviour of the masonry, some new numerical studies are performed in order to simplify the modelling. The results based on the proposed numerical equivalent model could be useful for further investigations.

Keywords: masonry, nonlinear, assessment

1. INTRODUCTION

Starting from the simplest shelters to modern reinforced masonry buildings, masonry as a construction material has a long history. It remains one of the most popular building materials because of the advantages in terms of material availability, ease of construction, high compressive strength, sound proofing, fire resistance, and low maintenance. Despite its wide use in structural engineering, masonry is still the least understood building material in terms of strength and deformation characteristics due to its heterogeneity.

Consisting of blocks with vertical and horizontal joints filled with mortar, masonry is characterized by complex constitutive laws. The asymmetry in tension and compression, the anisotropy in the horizontal versus the vertical loading direction, high non-linearity in compression and brittle cracking, poor load bearing capacity due to bending or shear resulting from earthquake ground motion make the analysis of masonry structures a difficult and arduous task.

Displacement based performance concepts are used for the purpose of determining the expected performance of a building under an expected seismic action. Due to the hypothesis that masonry is a brittle material, with limited deformation capacity, displacement based concepts have been mainly developed for reinforced concrete and steel structures. It should be mentioned that displacement based analysis of masonry buildings is allowed by many codes including Eurocode 8, FEMA 356, and Italian and Mexican national earthquake codes.

The seismic vulnerability of masonry buildings depends strongly on their resistance to shear forces. When out-of plane failure is impeded by using suitable devices, the structural reliability can be predicted and suitable strengthening techniques can be provided on the basis of the known in-plane behaviour. Hence, there is a great interest to model and test the shear responses of building elements subjected to horizontal cyclic loading conditions.

2. OBJECTIVES OF THE PAPER

Practical analysis of unreinforced masonry buildings for design or assessment purposes is usually carried out using static analysis which involves two dimensional (2D) models and is based on isotropic homogeneous linear elastic behaviour.

The available tools for URM include finite element models based on isotropic/orthotropic homogeneous nonlinear material behaviour for masonry units and on the nonlinear behaviour of the joints between masonry units. Due to high computational cost and high analytical skills for modelling, these tools are used mainly for research purposes.

In this paper, a micro non-linear model using the ATENA 2D (Cervenka 2011) finite element code is discussed with respect to its ability to simulate the in-plane behaviour of unreinforced masonry walls. Of the large number of analyses that were carried out during this study to investigate the influence of the great number of parameters representing the material properties, only those for which reliable experimental results were available for comparison are presented in this paper.

3. FUNDAMENTALS OF PUSH-OVER ANALYSES

The pushover analysis procedures can be divided into linear procedures (linear static and linear dynamic) and nonlinear procedures (nonlinear static and nonlinear dynamic). For assessing masonry structures exposed to seismic hazard, it is necessary to perform quasi-static collapse analysis before taking into account the dynamic collapse. For this purpose the non-linear static procedures are adopted since the main advantage with respect to the linear procedures is that they take into account the effects of nonlinear material response (strength, stiffness, deformation capacity) and therefore, the computed internal forces and deformations will be more realistic approximations of those expected during a dynamic excitation. Because only the first mode of vibration is considered, these methods are not suitable for irregular buildings for which higher modes become important.

In a nonlinear static procedure considering inelastic material response, the structural model includes directly the nonlinear force-deformation characteristics of individual components and elements. The nonlinear force-deformation characteristic of the building is represented by a pushover curve, obtained by subjecting the building model to permanent vertical loads and gradually increasing lateral forces or increasing displacements. The equivalent static lateral loads approximately represent earthquake induced forces and are distributed over the building height according to the first mode of vibration. A plot of the total base shear versus top displacement of a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis carried out up to failure allows the determination of the collapse load and the ductility capacity. The maximum displacements which can occur during a given earthquake are determined using either highly damped or inelastic displacement response spectra.

The purpose of a pushover analysis is to evaluate the expected performance of structural systems in order to subsequently compare it with the required performance levels.

The seismic performance of shear walls is also affected by some basic wall characteristics, such as the wall stiffness. When the structure loses stiffness during non-linear analysis as a result of increased load or lateral displacement, the fundamental natural period increases and therefore the base shear developed during an earthquake decreases.

The large number of influencing factors, such as material properties, dimensions, anisotropy of bricks and mortar, joint width, arrangement of bed and head joints and also quality of workmanship, makes the assessment of masonry buildings extremely difficult.

4. NUMERICAL SIMULATIONS

4.1. Modelling Aspects

When dealing with masonry, two main approaches can be adopted to perform different types of numerical analysis: macro-modelling and micro-modelling. Masonry has orthotropic material properties due to the presence of the mortar joints that are acting as planes of weakness. The use of hollow masonry units and partial grouting of voids increase the degree of complexity of the material characteristics. In macro-modelling, masonry is considered as being a homogenized body using the global properties of masonry. For the macro-modelling method a coarser mesh (fewer elements) is required and hence the computation time is reduced. Also, macro-modelling of masonry is advantageous when the global behaviour of the structure is the main interest of the study. Since masonry elements represent the homogenized properties of masonry, each element should comprise at least some portion of masonry units and mortar.

In contrast, in the micro-modelling approach the behaviour of unreinforced masonry walls is obtained based on assuming that the masonry bricks, the mortar and their interface are three separate elements and therefore these individual constituents are explicitly modelled. In the micro-model approach, it is possible to characterize mortar, bricks and their interfaces separately, adopting appropriate constitutive laws for each component so that their different mechanical behaviour is considered. The micro-model is probably the best tool available to analyze and understand the behaviour of masonry and in particular to assess its local response, but compared to macro-modelling it requires a large computational effort.

Since the objective of this paper is to describe the behaviour of masonry walls in order to find an equivalent material for macro modelling masonry, the micro-modelling approach has been employed in a first set of analyses. The masonry walls are modelled as being composed by bricks and joints using the software ATENA 2D which is a finite element code developed for nonlinear analysis of reinforced concrete structures.

4.1.1. Modelling of the bricks

Each brick is modelled as being an individual micro-element with the properties of the “3D Nonlinear Cementitious2” material. This fracture-plastic material model combines constitutive models for tensile (fracturing) and compressive (plastic) behaviour being described in detail by Cervenka and Jendele (2010). The main material laws developed for this model are presented in Fig.4.1. and 4.2.

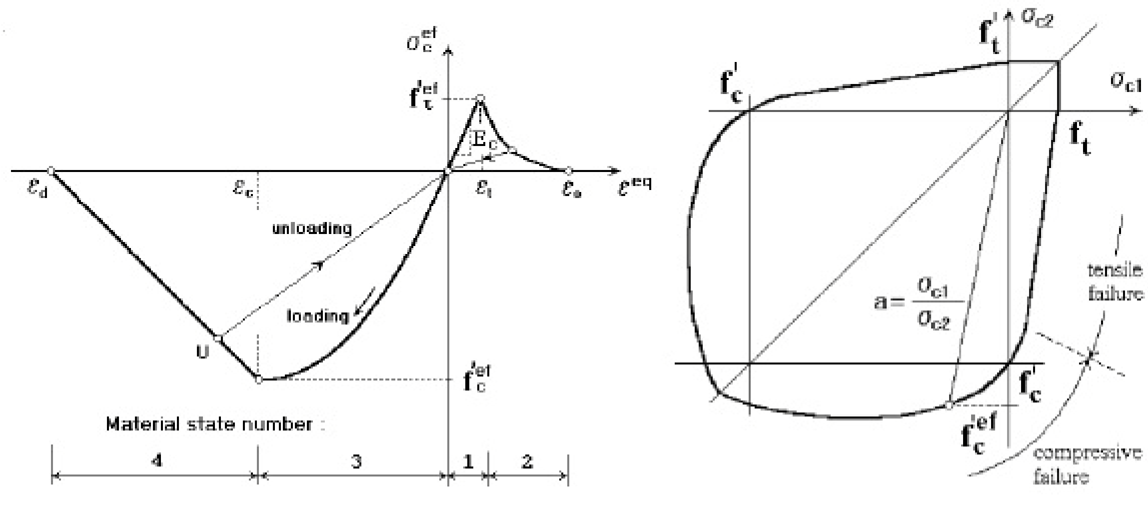

Figure 4.1. Uni-axial stress-strain law for concrete (left) and biaxial failure function for concrete (right) (from Cervenka 2012)

This material has a hardening regime before the compressive strength is reached as shown in Fig 4.2.

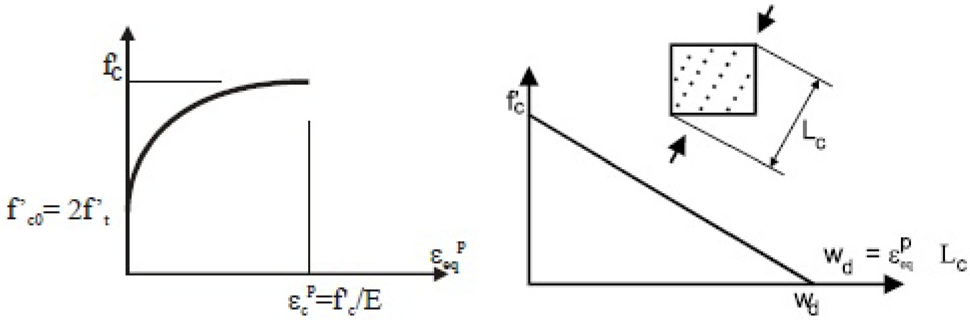

Figure 4.2. Compressive hardening/softening and compressive characteristic length (from Cervenka 2010)

An incremental formulation is used instead of a total formulation for the fracturing part of the model, therefore this material can be used in creep calculations or when it is necessary to change material properties during the analysis as in the present study. This formulation leads to the following set of non-linear equations:

$$K(p) \cdot \Delta p = q - f(p) \tag{4.1}$$

where q is the vector of total applied joint loads, f(p) is the vector of internal joint forces, Δp is the deformation increment due to the loading increment, p are the deformations of structure prior to the present load increment, K(p) is the stiffness matrix, relating loading increments to deformation increments.

4.1.2. Modelling of the joints

The joints represent the connections between the micro-elements. In ATENA 2D there are three different types of connections: no connection, rigid connection and Mohr-Coulomb connection represented by the “2D Interface Material”. The latter type of connection was adopted to represent the joints. Interface material describes the physical properties of contact between the micro-elements consisting in shear cohesion c and the friction coefficient μ from the dry friction (Mohr Coulomb) model. The maximum shear stress is limited by the linear relation given in Eqn. 4.2, where σ is the value of the interface compressive stress, considered positive.

$$\tau = c + \mu \cdot \sigma \tag{4.2}$$

This is also called cohesive limit or standard coulomb friction model which assumes that two materials support the same shear stress and no relative motion takes place if the equivalent frictional stress τ is less than the critical stress τ_{crit} which is proportional to the contact pressure. The failure surface for the interface elements is shown in Fig.4.3.

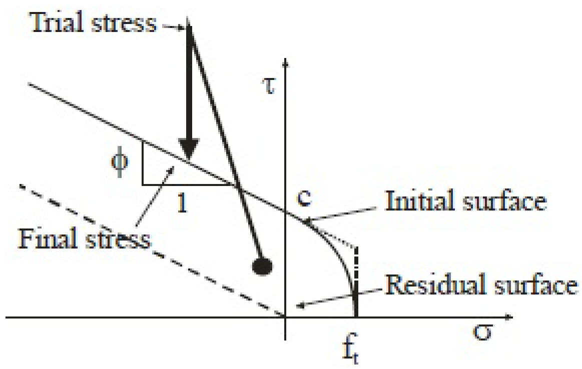

Figure 4.3. Failure surface for interface elements (from Cervenka 2010)

Also, for this material a tensile strength f_t can be defined in addition to the stiffness coefficients in order to be compliant with the finite element approach of the surrounding elements. Interface model behaviour in shear and tension is shown in Fig.4.4. The interface elements are defined as line elements having an assigned width. The width of the interface elements can be different from the width of the adjoining micro-elements and acts as multiplier on the Mohr-Coulomb law.

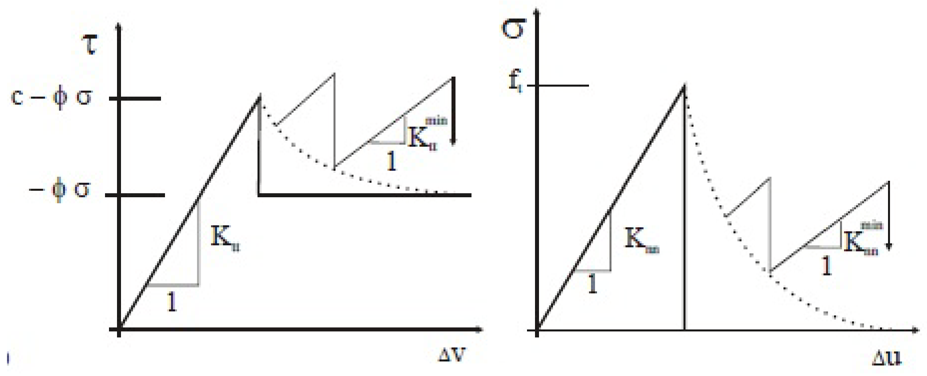

Figure 4.4. Interface model behaviour in shear (left) and tension (right) (from Cervenka 2010)

4.2. Scenario for Numerical Investigations

For the study presented here, wall V10 was selected from the experimental results from tests reported by Ötes and Löring (2003). The bricks used were PP2 – 0,4, solid gas concrete bricks having the dimensions 498/300/248 mm. For this wall a thin bed mortar (DM) was utilized. Experimental result and the numerical representation of the wall are shown in Fig. 4.5.

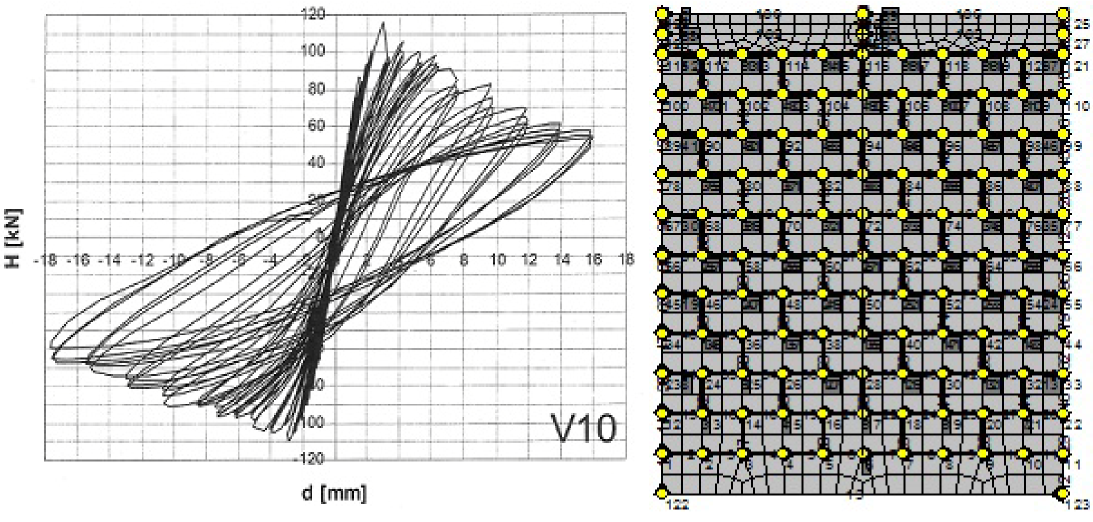

Figure 4.5. Experimental results (left) and numerical model in ATENA 2D (right)

The corresponding material properties adopted in this paper are presented in Table 4.1. For the properties of the masonry values reported by Ötes and Löring (2003) were adopted. Due to the lack of information regarding the material properties of the constitutive elements, the elastic modulus of the bricks was adopted from Table 2 presented in the study by Höveling, Steinborn, and Schöps (2009). Also, the value of Poisson's ratio for this type of brick was taken from Table 20 presented in the same study. The shear modulus was computed using the following formula:

$$G = \frac{E}{2 \cdot (1+\nu)} \tag{4.3}$$

where G, E and ν represent the shear modulus, elastic modulus and Poisson ratio, respectively.

Table 4.1. Material properties for the masonry wall and its constituents

		PP2 – DM wall (masonry properties)	PP2 brick (brick properties)	DM mortar
Unit weight	[kN/m^3]	5,0	5,0	[-]
E - Modulus	[MPa]	1500	1250	[-]
G - Modulus	[MPa]	400	565	[-]
Poisson's ratio (ν)	[-]	0,2	0,11	[-]
Friction coefficient (μ)	[-]	0,65	[-]	0.65
Compressive strength (ft)	[MPa]	2,3	2,8	16,7
Tensile strength (fc)	[MPa]	0,04	0,13	[-]

4.2.1. Parametric study considering masonry properties

The first approach was to assign the masonry properties to the brick elements due the fact that the axial stiffness and strength of the bricks must represent the stiffness and strength of the masonry since interface elements are approximately infinitely stiff and strong for axial compression. The same principle was considered by Beyer and Dazio (2008). For this wall with thin bed mortar joints this is a realistic assumption.

The top and bottom reinforced concrete beams attached to the test unit were represented using "Plane Stress Elastic Isotropic" material having the elastic modulus 10 times greater than the elastic modulus of the micro-elements in order to preserve the accuracy of the computations. The bed joints interface material has the same width as the brick elements, 300 mm, while for the head joints this is reduced to 100 mm taking into consideration that in practice these joints are not fully grouted. The rigidity of the interface material is obtained using the following formulae:

$$K_{nn} = \frac{E}{t} = \frac{1500}{0.01} = 150000\ MN/m^3 \tag{4.4}$$

$$K_{tt} = \frac{G}{t} = \frac{400}{0.01} = 40000\ MN/m^3 \tag{4.5}$$

where *E* and *G* are the minimum elastic and shear modulus, respectively of the surrounding material and t represents the thickness of the interface zones, assumed to be 10 mm in this study.

Typically, in almost all studies the tensile strength of the mortar is considered to be smaller than the tensile strength of the bricks. Here, a parametric study was carried out using four numerical models (NM) in which the tensile strength of the mortar had the values 0.03, 0.02, 0.01, and 0.005 MN/m^2. Since the cohesion was not specified in the report by Ötes and Löring (2003) it too was determined analytically. In order to satisfy the convergence criterion, cohesion must fulfil the relation:

$$c \geq f_t \cdot \mu \tag{4.6}$$

where c and μ are the same parameters presented in paragraph 4.1.2 and f_t represents the tensile strength of the mortar. The material properties used in numerical models are represented in Table 4.2.

Table 4.2. Material properties input in ATENA 2D considering the properties of the masonry

Topology	Contact - Joint					Brick - Elements				
Material	2D Interface					3D Non Linear Cementitious2				
Parameters	K_{nn} MN/m^3	K_{tt} MN/m^3	f_t MPa	c MPa	μ	E MPa	G MPa	ν	f_t MPa	f_c MPa
FEA NM 1	150000	40000	0.005	0.0035	0.65	1500	400	0.2	0.04	2.3
FEA NM 2	150000	40000	0.01	0.007	0.65	1500	400	0.2	0.04	2.3
FEA NM 3	150000	40000	0.02	0.015	0.65	1500	400	0.2	0.04	2.3
FEA NM 4	150000	40000	0.03	0.02	0.65	1500	400	0.2	0.04	2.3

The results of the four analytical models described above are compared with the experimental results from Ötes and Löring (2003) in Fig 4.6.

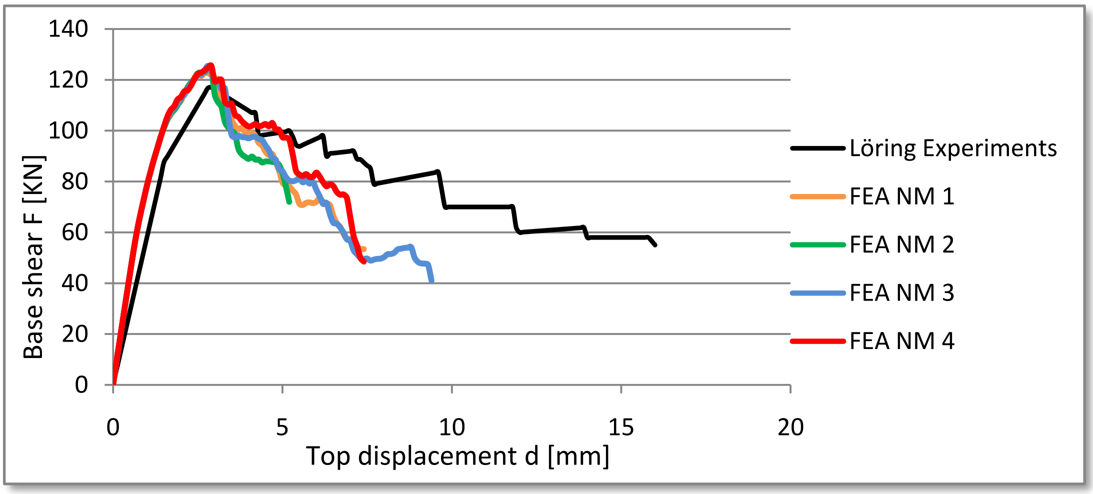

Figure 4.6. Force-displacement curves considering the properties of the masonry

It can be seen that for all the assumptions for the material the behaviour within the elastic domain is identical and moreover is very similar to the experimental force-displacement curve up to the peak value of the applied force. For both, the numerical models and the experimental results, the applied force reaches the maximum value at about the same value of lateral displacement. This value is about 7 % greater in the numerical models. In the descending portion of the force-displacement diagram all four numerical models exhibit a similar loss of rigidity. The best results were obtained assigning the maximum allowable tensile strength to the mortar (FEA NM 4).

4.2.2. Parametric study considering the properties of the bricks

The second approach was in compliance with the applied principles of the first one, the only difference is that the elements representing the bricks were modelled using the material properties of the bricks, as shown in Table 4.3. This parametric study also consisted of four numerical models in which the tensile strength of the mortar has the values 0.12, 0.08, 0.04, and 0.02 MN/m^2. This variation of the tensile strength leads to changes of the values for corresponding cohesion as presented in Eq. 4.6.The rigidity of the interface material is obtained using the same formulae:

$$K_{nn} = \frac{E}{t} = \frac{1250}{0.01} = 125000\ MN/m^3 \tag{4.7}$$

$$K_{tt} = \frac{G}{t} = \frac{565}{0.01} = 56500\ MN/m^3 \tag{4.8}$$

Table 4.3. Material properties input in ATENA 2D considering the properties of the brick

Topology	Contact - Joint					Brick - Elements				
Material	2D Interface					3D Non Linear Cementitious2				
Parameters	K_{nn} MN/m^3	K_{tt} MN/m^3	f_t MPa	c MPa	μ	E MPa	G MPa	ν	f_t MPa	f_c MPa
FEA NM 5	125000	56500	0.02	0.015	0.65	1250	565	0.11	0.13	2.8
FEA NM 6	125000	56500	0.04	0.04	0.65	1250	565	0.11	0.13	2.8
FEA NM 7	125000	56500	0.08	0.055	0.65	1250	565	0.11	0.13	2.8
FEA NM 8	125000	56500	0.12	0.08	0.65	1250	565	0.11	0.13	2.8

The force-displacement curves obtained from these four numerical models are compared with the experimental curve reported by Ötes and Löring (2003) in Fig. 4.7.

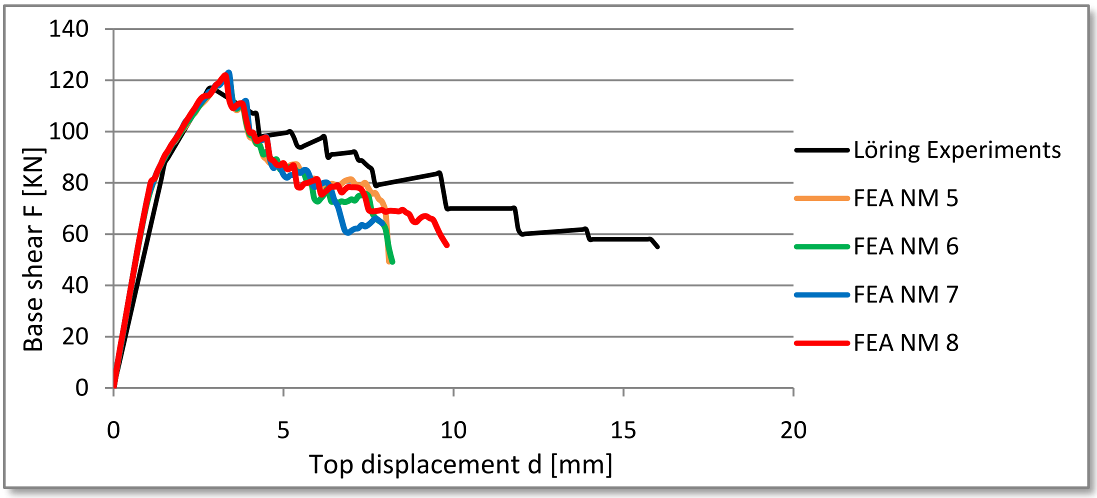

Figure 4.7. Force-displacement curves considering the properties of the brick

Satisfactory results were also obtained using this approach. The analytical behaviour of the wall resulting from the analytical models is almost identical to that of the test wall up to the maximum value of the applied force. It can be seen that for displacements up to about 5 mm the adopted numerical models predict the experimental results well and exhibit very similar results.

The geometry and layout of joints play an essential role in the response of the wall and in the mechanism of failure. For this wall a diagonal sliding shear failure was observed in the test reported by Ötes and Löring (2003). Fig. 4.8 illustrates this behaviour for the analytical model of the wall.

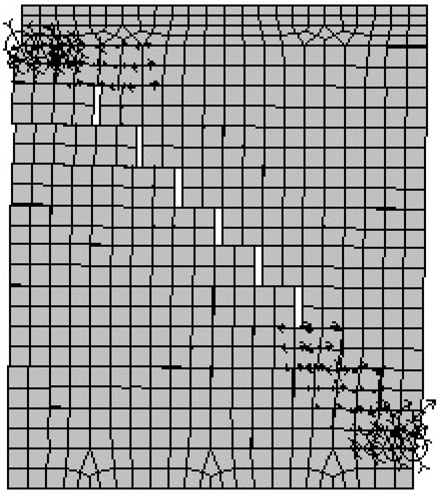

Figure 4.8. Displaced shape (magnifying factor = 10) and principal strain trajectories

4.3. EQUIVALENT MATERIAL FOR MACRO-MODELLING MASONRY WALLS

Taking into account the large amount of labour and time for managing the experimental process work, brick by brick and for defining joints characteristics, and using the approach with the best results in the previous chapter, an equivalent material was derived in order to macro-model the wall. This macro model consists of only a few "macro bricks" with the same aspect ratio of 2,5 as the actual bricks. The dimensions for the "macro bricks" were chosen as 1250/300/625 mm. The equivalent models (EM) are presented in Fig 4.9. Two types of bond were modelled: stack bond masonry and rolling bond masonry. The "macro bricks" are connected using the same interface material as in the micro model FEA NM 8 (see Table 4.3 and 4.4).

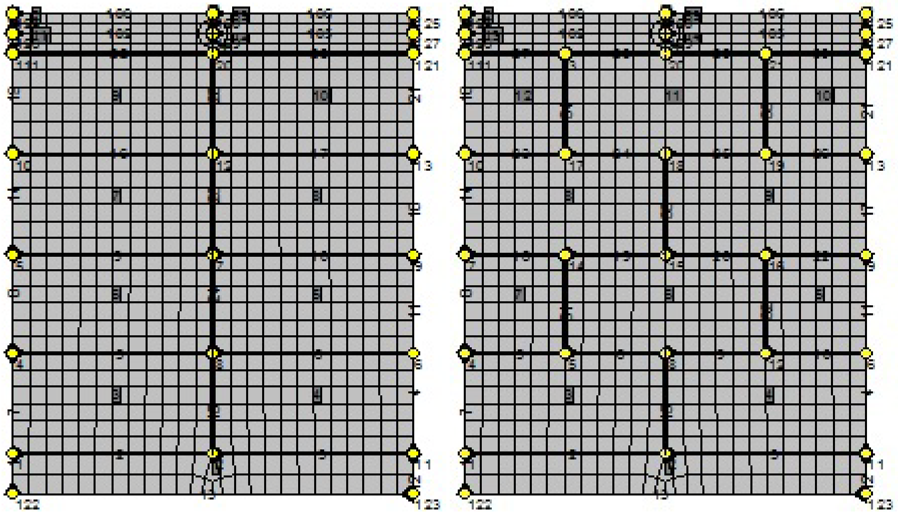

Figure 4.9. Equivalent model in ATENA 2D for stack bond masonry (left) and rolling bond masonry (right)

It was found that the stack bond masonry model developed an exceedingly high rigidity in the linear domain and that the peak value of the lateral force was two times greater than the experimental value. Better results were obtained using the rolling bond masonry model. In order to calibrate the model, several numerical models were analysed in which the varying parameters were the tensile and compressive strength of the "macro bricks". It was found that a good correlation between the material properties and the dimensions of the "macro brick" can be obtained by dividing both, the tensile and the compressive strength of the actual bricks by the value of the aspect ratio, in this case 2,5. The material properties that resulted in the best prediction of the test wall behaviour are summarized in Table 4.4.

Table 4.4. Material properties input in ATENA 2D for the equivalent "macro brick" model

Topology	Contact - Joint					"Macro Brick" elements				
Material	2D Interface					3D Non Linear Cementitious2				
Parameters	K_{nn} MN/m^3	K_{tt} MN/m^3	f_t MPa	c MPa	μ	E MPa	G MPa	ν	f_t MPa	f_c MPa
FEA EM	125000	56500	0.12	0.08	0.65	1250	565	0.11	0.052	1.12

The force displacements curve obtained from the equivalent model (EM) is compared with the experimental curve and with the one obtained from the micro model NM 8 in Fig. 4.10.

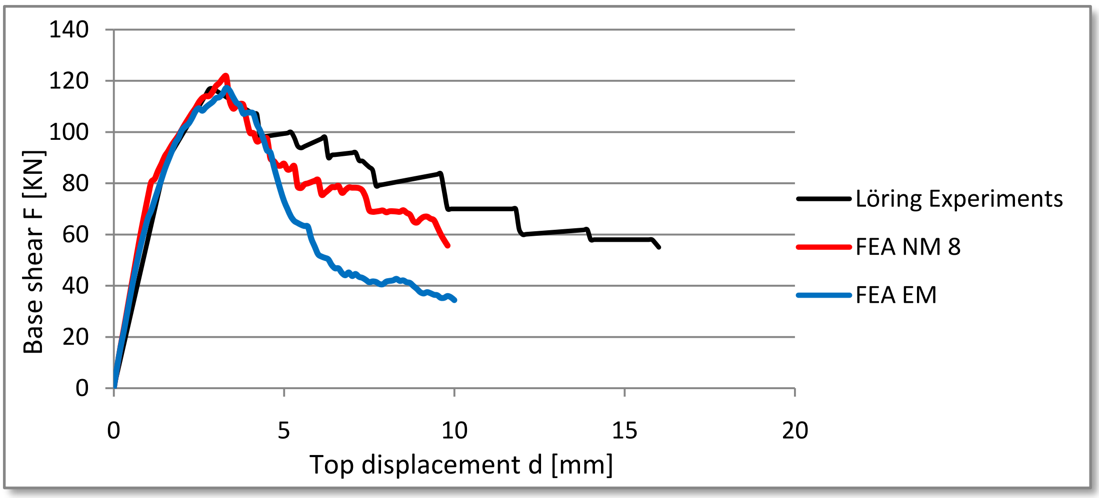

Figure 4.10. Force-displacement curve for the equivalent "macro brick" model

Up to a displacement equal to about 5 mm both, the equivalent “macro brick” model and the micro-model lead to almost identical force-displacement curves that agree very well with the experimental curve. For larger displacements, however the equivalent “macro brick” model over-estimates the loss of lateral load resistance of the wall.

5. CONCLUSIONS

In this paper, a micro-model for the in-plane behaviour of URM walls is presented using the general purpose finite element code, ATENA 2D. In the proposed models, bricks and joints are assumed to be separate elements and the behaviour of the brick elements obeys concrete plastic-damage models developed based on the concrete fracture-energy concept. A comparison between the numerical study and experimental results reported by Ötes and Löring (2003) is also given. It is shown that the finite element models were able to predict effectively the behaviour of masonry walls using commercially available software, a conclusion that is significant for the assessment of existing URM structures.

Within a parametric study using the numerical model it could be confirmed that better results are obtained using for the micro-elements the properties of the bricks instead of those of the masonry. Finally, the paper proposes an equivalent “macro brick” model that requires less computation time is easier to implement.

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