Seismic assessment of masonry structures – Multi-scale numerical modelling

Y. Le Pape, A. Anthoine, P. Pegon
ELSA, IPSC, Joint Research Centre, European Commission, I-21020 Ispra (VA) Italy

ABSTRACT: This paper presents an overview of the modelling work performed in the framework of the SEISPROTEC project dealing with the failure of masonry walls. The mortar is represented by means of joint elements and the interface model, together with its mechanical characteristics is presented. Preliminary numerical results obtained for the various scales considered in the project (structural scale/walls, mesoscale/ wallettes and microscale/cells) are commented.

1 INTRODUCTION: A MULTI-SCALE APPROACH

The Joint Research Centre of the European Commission launched in 1999 an institutional research project on seismic assessment of masonry structures (SEISPROTEC), which is focussed on the protection of cultural heritage structures (Pinto et al. 2001). Among the pursued objectives, this project intends to allow for the calibration/development of advanced numerical models for masonry with particular emphasis on the in-plane cyclic behaviour. The project also includes an experimental program presented in an accompanying paper (Molina et al. 2001).

The numerical work intends to provide an in-depth study of the in-plane behaviour of masonry wall performed at different scales. It is strongly connected with the experimental work. Three scales are actually considered: microscopic, macroscopic and structural. Each level will be treated in some more detail in the following. However, it shall be emphasized that the goal pursued in this particular study is to enlighten the existing connections between these modelling levels. Indeed, literature confirms that such gap can be bridged (see e.g. (Anthoine 1995) for the derivation of elastic characteristics through homogenisation theory).

The modelling of masonry generally falls into two distinct classes: *(i)* the so-called micromechanical models (sometimes referred as discrete models) accounting for the morphology of masonry: each basic masonry constituent (i.e. brick, mortar joint, eventual reinforcement: steel bars, cement grouting, resin injection…) are considered separately. However, the micro-mechanical approach refers as well to simplified models where some constituents are omitted or combined. Lourenço (1996) has previously shown that adopting a simplified modelling strategy in which mortar unit and brick/mortar interface are combined in a single joint element, allows a drastic reduction of the computational time without losing accuracy. As a preliminary overview, units are schematised as linear elastic and interfaces are treated as unilateral frictional constraints. The micro-mechanical approach shall be seen as the best modelling scale to understand the salient features of the in-plane behaviour of masonry panels. However, such modelling cannot reasonably be applied to deal with structure larger than a single wall. For instance, the modelling of complete infilled-storeys buildings definitely requires adopting another standpoint at a macro-scale level. *(ii)* The macro-mechanical models (also called homogeneous or continuous models) account for the behaviour of a typical relevant unit cell by establishing a direct constitutive law between the average stress and strain states (see e.g. (Anthoine 1992)). Such relationship shall be obtained either by adopting a phenomenological standpoint assuming for instance the masonry as a *"no-tension material"*, or using homogenisation techniques. Obtaining a consistent continuous anisotropic inelastic model has been proved to be a difficult task due to the materials intrinsic properties and to the lack of comprehensive experimental results. The SEISPROTEC project intends to complete an accurate database obtained both by numerical and experimental means, so as to provide informations about the mechanical behaviour at different scales. Three types of tests are considered, as shown in Fig. 1: *(i)* at the structural level, in-plane cyclic behaviour of a shear wall, *(ii)* at a "mesoscale", large assemblages, also called "wallettes", with inclined bed-joints, tested under biaxial loading conditions, and finally, *(iii)* at the lowest scale, study of the behaviour of a basic cell under the assumptions of periodic medium. All numerical computations and developments are performed with CAST3M (formerly known as CASTEM 2000) FEM code.

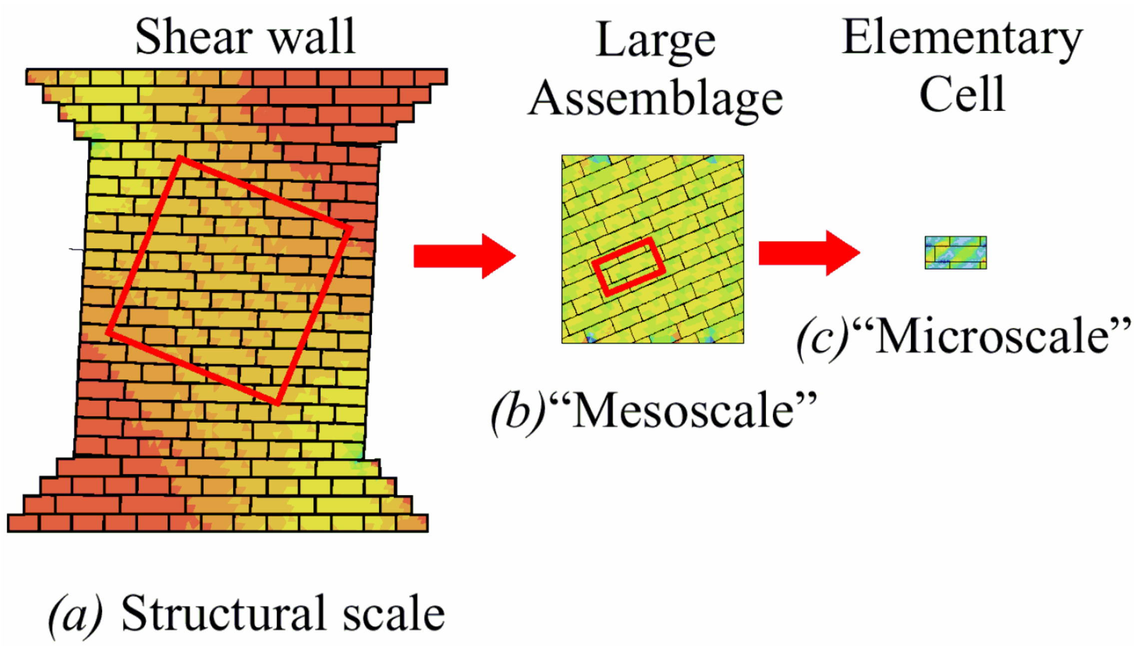

Figure 1: Scales of modelling

Note that *(iii)* can only be tested by numerical means, due to the "impossibility" of making an adequate testing rig able to reproduce the periodic conditions. Tests *(i)* and *(ii)* will also be performed by experimental means thanks to ELSA full-scale testing facilities (pseudo-dynamics testing procedure/reaction wall).

The micro-mechanical approach may therefore be seen as an efficient tool in order to develop a macro-model. Therefore, as a primary approach, the effort has fully been concentrated on the micro-modelling.

2 THE JOINT MODEL

The new model is an improvement of the so-called "soft-joint" already implemented in CAST3M. In this model, though a usual Mohr-Coulomb frictional yield locus is still assumed, the flow rule is mainly governed by the normal stress applied to the joint (see (Anthoine and Pegon 1996)). In tensile regime, i.e. in the vertex domain, no plastic shear strain rate is assumed, though the residual cohesion decreases linearly with the residual tensile strength governed by the softening in tension. Conversely, while the stress tensor lies in the compression domain, i.e. within the edge, solely shear failure may occur. A free dilation plastic flow rule is adopted. Shear behaviour may exhibit softening, so that the friction angle decreases to a residual value while the tensile strength is kept constant. The integration scheme is explicit. In any case, unloading is elastic. The following mechanical features has been added:

– crushing;
– both shear and tensile flow in the vertex domain;
– both shear and compression flow in the edge of the domain;
– possible damage affecting the behaviour while unloading;

– coupled effects between the damage variables in both normal and tangential directions.

Plastic yielding is thus governed by a composite yield locus featuring a tension cut-off, a usual frictional criterion and a compression cap assumed as another simple cut-off. Each of them is practically controlled by three uniaxial evolutions featuring respectively the observed or assumed behaviours in pure tension, pure shear and pure compression. The residual tensile strength, i.e. the location of the current tensile cut-off, and the edge of the friction cone are simply coupled using a homothetic rule which ratio is based on the initial values of the two mentioned previous characteristics. Shear behaviour is also governed by a homothetic rule with respect to the current state of normal stress. Free dilation property is still assumed.

This model has been developed with a practical phenomenological standpoint in order to create a simple, though consistent tool, which generality might be helpful for treating further related applications.

The Fig. 2 sketches a typical response induced by controlled displacements made in two steps. First, a normal jump is imposed **1**. Once the critical normal strain e_{cr} is reached, the tensile behaviour simply follows the softening branch. The tensile cut-off has moved with respect to the residual tensile strength $s_{r,1}$. Coupling the tensile cut-off with the Mohr-Coulomb shear criterion implies that the current cohesion c has also moved from its initial value c_0 to a residual value, namely $c_{r,1}$. Then, a tangential displacement is imposed **2**. Due to the previous tensile degradation, the shear stiffness has decreased. Once the shear-softening regime is activated, shear/tension damage coupling may provoke a tensile relaxation.

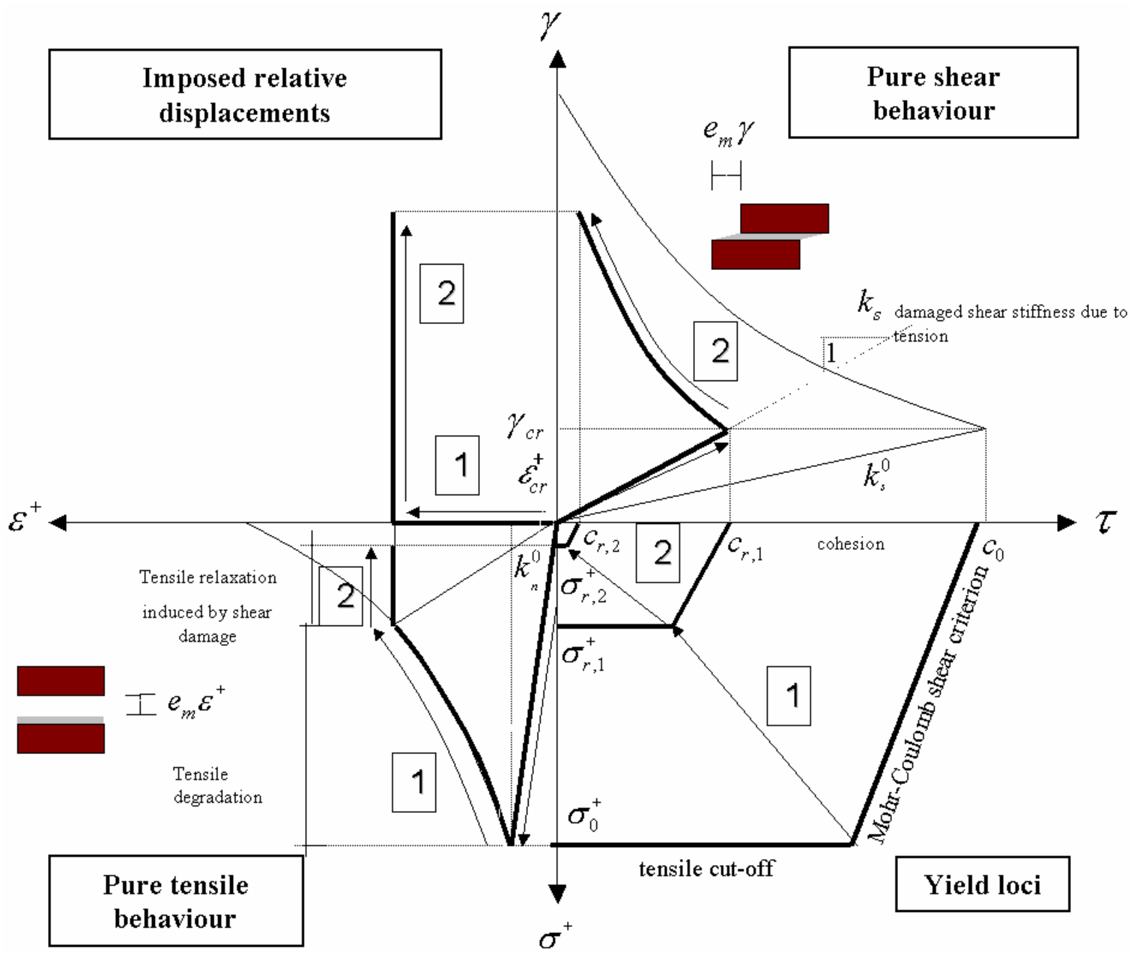

Figure 2: The joint model

3 MATERIALS AND MECHANICAL PROPERTIES

As any composite material, the mechanical properties of masonry depend on the characteristics of each component and also, on the manner they are assembled together. One shall keep in mind that the global properties of a masonry wall are strongly dependent on the skill of the bricklayer's mate. Large scattering effects may be observed even for walls made with the same basic constituents.

3.1 Bricks

Clay bricks are one of the oldest materials used by man. The evolution of the drying/ firing process has led to more and more standardized products along centuries, so that traditional handwork procedure was finally replaced by machinery during the Industrial Revolution (see e.g. (Lourenço 1996)). However, *"the properties of fired bricks vary considerably even for bricks of the same batch."* (Page 1978). As a general comment, bricks exhibit elastic-brittle behaviour. Isotropy/anisotropy depends on the geometry of the brick (solid unit/hollow tile).

The type of bricks used in SEISPROTEC project has been chosen upon two criteria: first, the quality of the manufacturing process to ensure homogeneous properties of the units (as far as one may obtain nowadays with reasonable cost!), well-documented physical and mechanical properties. These are detailed in (Binda et al. 1995).

The bricks are homogeneous solid units manufactured by RDB Terrecotte S.p.A under the reference: Borgonovo, type "a mano", serie "rosso classico". They are typically used in retrofitting of historical buildings. Nominal dimensions are: width 250 [mm], height 55 [mm], depth 120 [mm]. Numerical computations have been performed using the following parameters: elastic Young's modulus 2.1 [GPa], Poisson's ratio 0.15.

3.2 Mortar

The generic name mortar is used in various ways to qualify combinations of cement, lime and sand. The mechanical properties may therefore vary considerably depending on the constituents used. Mortar exhibits non-linear time-dependent stress-strain characteristics (Page 1978). The analysis of mortar properties found in historical monuments indicate they have generally suffered of time effects, such as desegregation due to water infiltration, so that they usually can be considered as a poor quality material. While retrofitting this type of buildings, the state of the art and the traditional habits of bricklayer's mate suggest to respect the following ratio: hydraulic lime and sand in volumetric ratio of one third.

Mortar elastic properties have been obtained using similar experimental procedures as the one performed for the units. The properties of the mortar aged of 90 days are very low: about a tenth of the characteristics of the bricks as far as the elastic modulus and the compression strength are regarded. Rather large scattered results are obtained on the transversal LVDT, so that it is difficult to give a good estimation of the Poisson's ratio.

The values adopted for the numerical computations are: elastic Young's modulus 0.25 [GPa], Poisson's ratio 0.18.

3.3 Bond

The properties of the mortar/unit bond find their roots in various physical/chemical phenomenons: covalent/polar covalent, electrostatic bonds. Cohesion and adhesion are dependent on physical aspects such the water abortion from the bricks, so that their porosity is of premium importance: the presence of a thin hydrated layer is probably prevalent upon the bond properties. Geometrical aspects such as asperities give also a contribution to the mentioned local properties of the bond.

The modelling of masonry requires to obtain bond micromechanical properties under shear and tensile loading. Numerous types of tests have been proposed in the literature (see Anthoine (1992)). However, they generally fall in two categories: full-scale masonry tests (e.g. panels, shear wall, wallettes...) or small assemblages (e.g. couplets, triplets, stack piers...).

As it appears in (Binda et al. 1995), the tensile bond strength (~0.05 Mpa) is very low and its value cannot be expected to be obtain rather than as a rough estimation. The tests were performed in load-control. The jump of normal displacement in the joint can be estimated between 0.05 and 0.20 [mm]. Very few indications are given except from the tensile strength. Thus, the modelling of tensile behaviour will require to make the following further assumptions: *(i)* the bond behaviour is brittle-elastic; *(ii)* the post-peak regime is controlled by an exponential softening branch mainly governed by mode I fracture energy.

The knowledge of the behaviour under shear is of premium importance to understand the behaviour of masonry under seismic/cyclic actions. Typical direct shear tests on three high stack “piers” indicate that the failure mode is governed by a cohesive/frictional law and is therefore dependent on the normal stress applied to the joint. According to the results of Binda et al. (1995) the value of the usual Coulomb’s can be chosen as: cohesion=0.3Mpa and friction angle=30°.

The post-peak regime describes a softening branch down to a residual value. Large tangential relative displacements about 10 [mm] were reached. If residual Mohr-Coulomb’s characteristics are sought, it appears that the loss of friction angle is between 5 and 10 degrees and that the residual cohesion drops under 0.1 [MPa].

4 THE WALL LEVEL

The numerical computations are performed in 2D. The boundary conditions and the loading history are so as to remain the closest as possible to the experimental conditions (see Figure in (Molina et al. 2001)): the wall base is fixed, the vertical load is applied through a rigid beam. Then, its horizontal displacement is controlled up to a value of 5 [mm]. No rotation of the upper-beam is allowed. The vertical preloading is ensured by the wall self-weight plus an additional uniformly distributed load on the upper part of the wall. The average vertical pressure is around 0.1 [MPa]. Such value is relevant of the usual loading of masonry walls and permits to activate the shear failure at low levels. Attention should be paid to the fact that the additional vertical load is uniformly distributed only at the beginning of the test. As soon as the shear load is applied, the distribution of the vertical load is affected by the formation of a diagonal strut, so that normal stresses are concentrated on the left part of the testing beam. The computation are first performed assuming a non-softening behaviour and no damage coupling between traction and compression.

The results given in section 3 indicate that the mean values of the cohesion c, the tensile strength f_t and the friction angle j can reasonably be estimated respectively by 0.3 [MPa], 0.05 [MPa] and 30[^{o}]. Keeping f_t and j constant, the ratio $c \tan j / f_t$ is increased from a value of 1 to 10. From the model described in section 2, such ratio describes the location of the edge of the friction cone with respect with the location of the cut-off. The value of ten is close from the one obtained using the numerical values mentioned above. However, due to the scattered experimental values, a pessimistic ratio around 3 could also be determined. Fig. 3 shows the upper-beam vertical displacements together with the applied shear forces plotted versus the horizontal displacement for different values of the ratio $c \tan j / f_t$. Two very different failure modes can occur. The failure mode B is initiated by tensile cracks at the top and the bottom of the wall at the early stage of loading. For high values of $c \tan j / f_t$, local shear cracks are not activated so that the initial process keeps on evolving. A diagonal strut permits the equilibrium with the external forces. It shall be noticed that for the purpose of the parametric study, the compression properties are kept linear elastic. Otherwise, if non-linear properties are introduced, the wall behaviour would be governed by the crushing of the toes and therefore the global rotational mechanism observed on Fig. 3 would be stopped. Conversely, for low values of $c \tan j / f_t$, local shear cracks appear so that the central joints are fully open (mechanism A). The diagonal compression strut is split so that the internal forces flow between the two lateral sizes of the wall by shearing of the bed joints. Depending on the relative value of the cohesion with respect to the tensile strength, failure mode A may be activated at various stages of the horizontal loading. If it is admitted that the ratio $c \tan j / f_t$ may vary from a value of 3 to ten, then the numerical results indicate that the ratio between the increasing of vertical displacement due to the horizontal loading with respect with its initial value due to the vertical preloading, may take various values in a range of 1 to 5. The computed maximal horizontal loads exhibit a dispersion around 40%.

The other parameters of joint model such as the shape of the softening curves in traction and shear or the amplitude of the residual forces only slightly modify the loading curves.

This section contains also numerical results obtained after a cyclic loading. The initial vertical preloading is equal to 150[kN] and is kept constant. The horizontal displacement is imposed

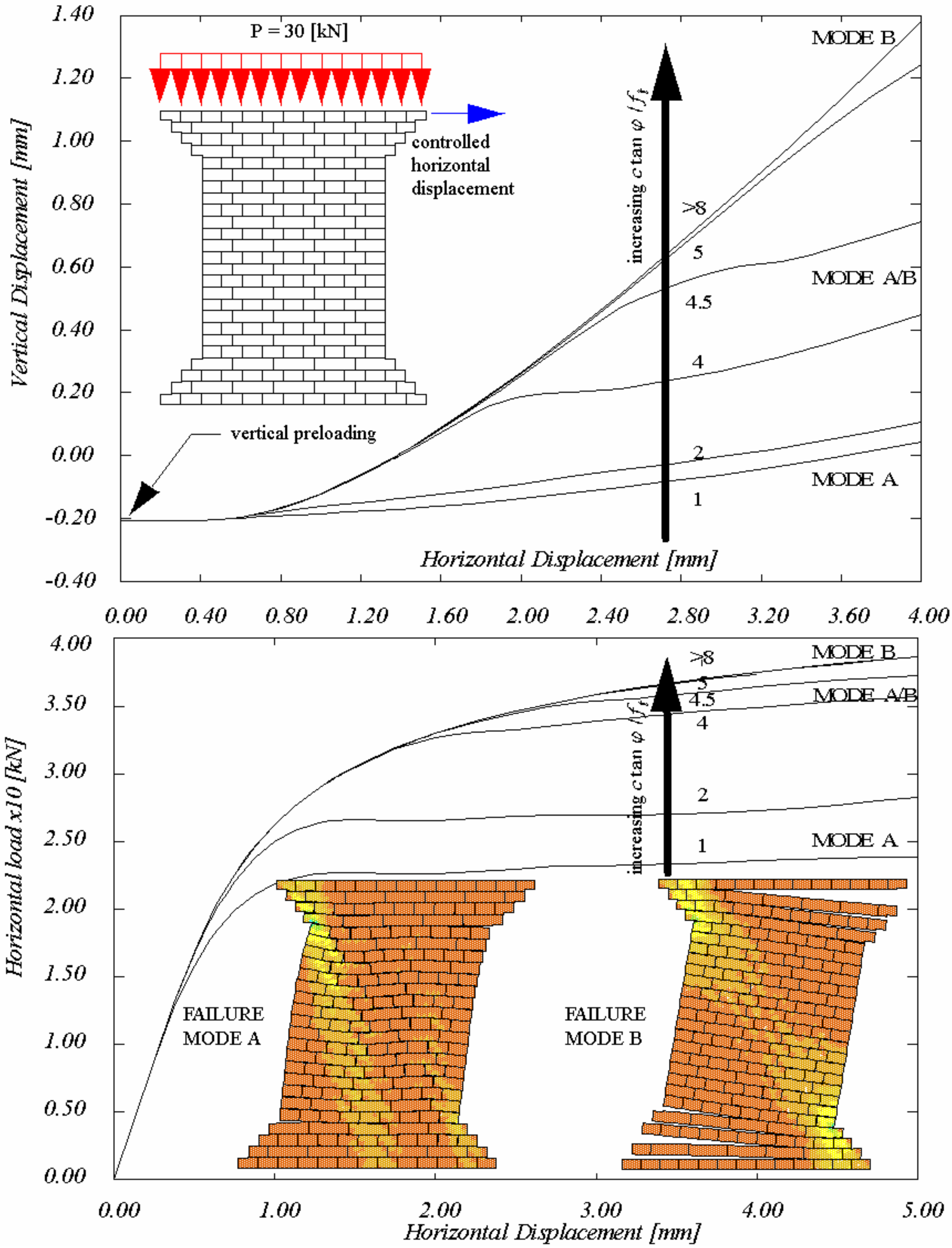

Figure 3: Upper-beam vertical displacements and shear force - Wall failure modes.

with respect to the following sequence: +1.00 [mm], -2.00 [mm], +3.00 [mm], -4.00 [mm] and +5.00 [mm]. Fig. 4 sketches the response of the wall. The upper left part of the figure shows the computed horizontal shear load versus the horizontal displacement of the upper beam: very small hysteretic behaviour is observed. The upper right part of the figure sketches the trajectory of the upper-beam. The computed behaviour does not exhibit any apparent volumetric increasing of the wall due to dilation or local rearrangement of the bricks. Such behaviour finds its roots within the adopted constitutive law of the joints: the unloading response of the joint is governed by a damage elastic modulus; no irreversible deformations within the joints are assumed. A further modification of the model, allowing for plastic unloading is thus required.

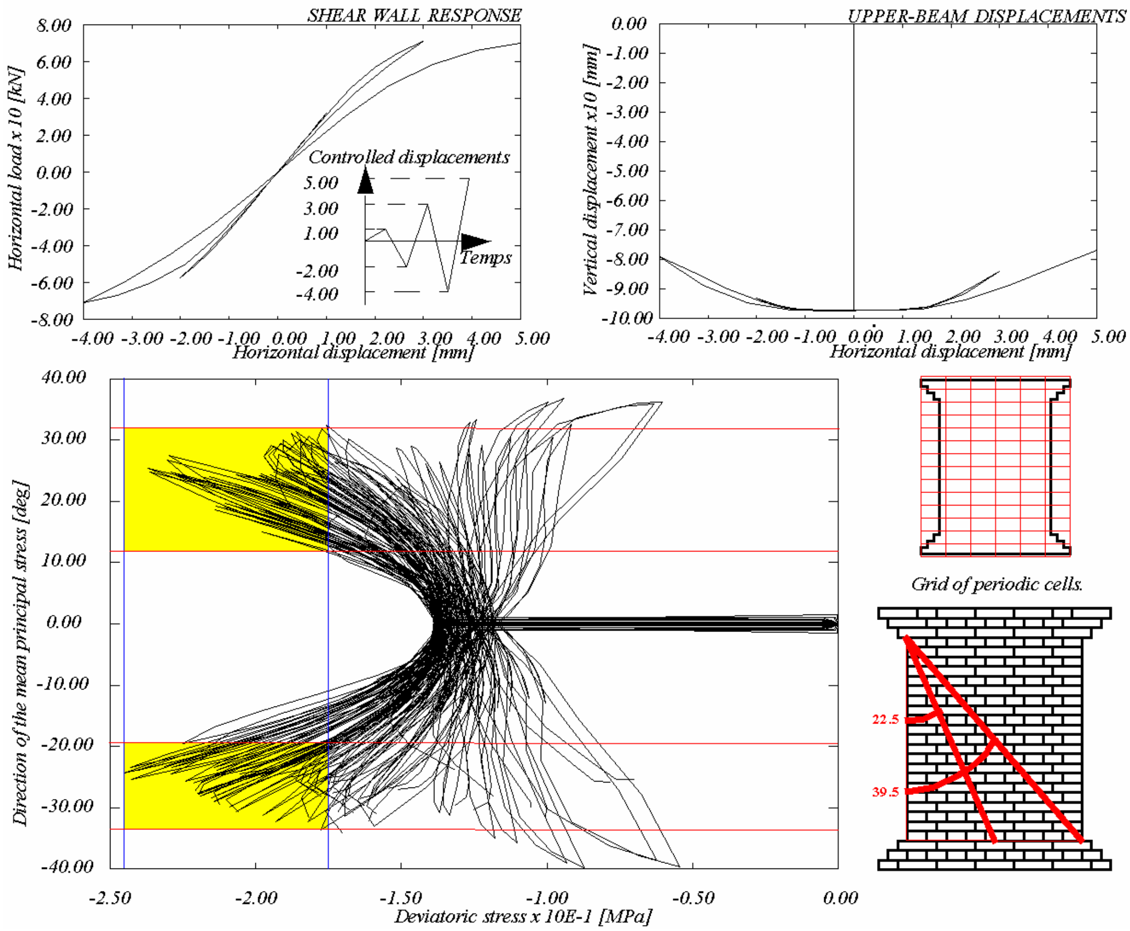

Figure 4: Cyclic response of the wall and Back-computed homogenized principal directions

In order to get a rough idea of the local behaviours within the wall, some typical periodic cells have been studied. Their respective locations are shown on Fig. 4. The homogenized stress/strain states are computed using formulae given by Anthoine and Pegon (1996). It is then possible to compute the principal direction of this mean principal stress/strain for instance defined with respect with the vertical. It should be reminded that the knowledge of the directions of the principal stress/strain for the large wall is of premium importance in the inclination of the bed joints of the large assemblages (wallettes). From its original zero value induced by the vertical preloading, the direction oscillates with the horizontal displacement of the upper-beam. Let it be noted that the computations of the mean principal directions give roughly the same results if calculated on the strain or the stress. Since it is not possible to change the principal directions of the large assemblage tests, it seems obvious to try to give an orientation to the “wallettes” that is relevant of what really occurs at the structural level. Therefore, it is proposed to hold the principal directions corresponding to high stress level: i.e. when the cell is already damaged. As can be seen in the lower part of Fig. 4, it seems possible to discriminate the relevant directions to be studied according to a criterion on the deviatoric stress level: the corresponding principal directions are in a range of 10 to 35 degrees with respect to the vertical.

5 THE WALLETTE LEVEL

As mentioned in previous section, it is not possible to ensure the same homogenized stress path on the large assemblages since this kind of tests fixes the principal directions. Even though, and in order to remain coherent with the previous analysis, a “wallette” made with the same materials is tested. The inclination of the bed joint is taken equal to 22.5 degrees. The loading is performed through the knowledge of the vertical displacement of the upper right corner up to a value of 1.5 [mm]. The lateral expansion of the wall is free. The boundary conditions are treated so as to account for the low friction due to the Teflon sheet placed between the panel and the external testing rig.

The aim of this specific study is to check how the homogenized stress/strain histories derive, or not, from the knowledge of simple external quantities such as the value of the imposed displacement or the loads applied to the external sides of the panel. Thus, three periodic cells located at random within the panel are considered (see Fig.5). The results obtained on the three cells are qualitatively rather similar (see Fig. 6): little shear strain, and roughly linear relationships between the vertical normal strain and the horizontal normal strain. The mean principal stress map (see Fig. 5) indicates that the obtained field is rather periodic. However, they are quantitatively significantly different.

An error indicator is computed to check if the knowledge of the imposed vertical displacement permits to obtain the “real” homogenized strain within the cells. This error indicator is computed using the following rule:

$$Err = \frac{|(du_v)/l - E_{yy}|}{|(du_v)/l|} \tag{1}$$

where du_v, l and E_{yy} denote respectively the vertical displacement of the upper-right corner, the height of the wall and the homogenized vertical normal strain. Depending upon the location of the cell, the computed error may reach up to 7%. This result does not prove that the total displacement is a bad index for computing the homogenized strain state. But, it seems that the size of the cell is not large enough to obtain a constant result.

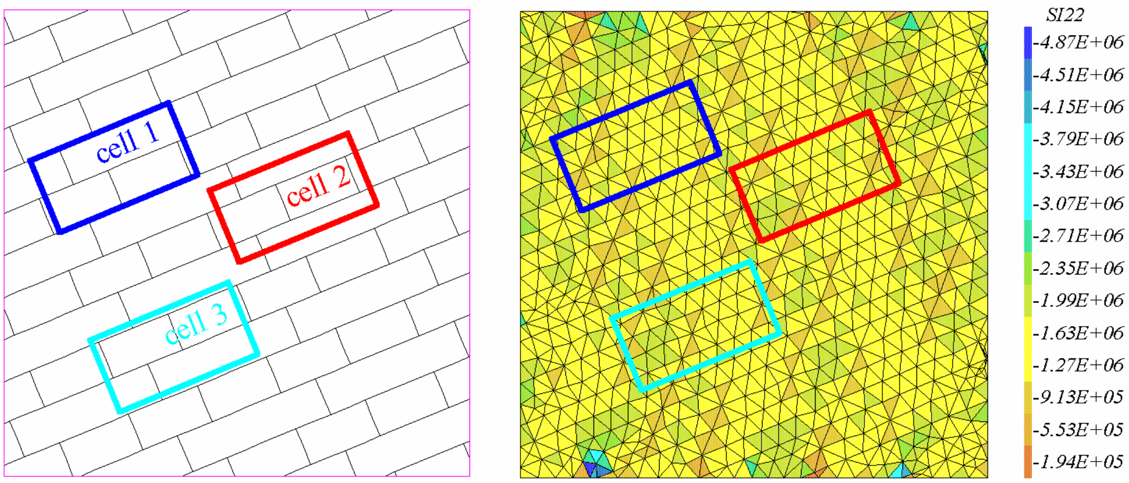

Figure 5: Joints mesh - Periodic cells - Mean principal compression stress field.

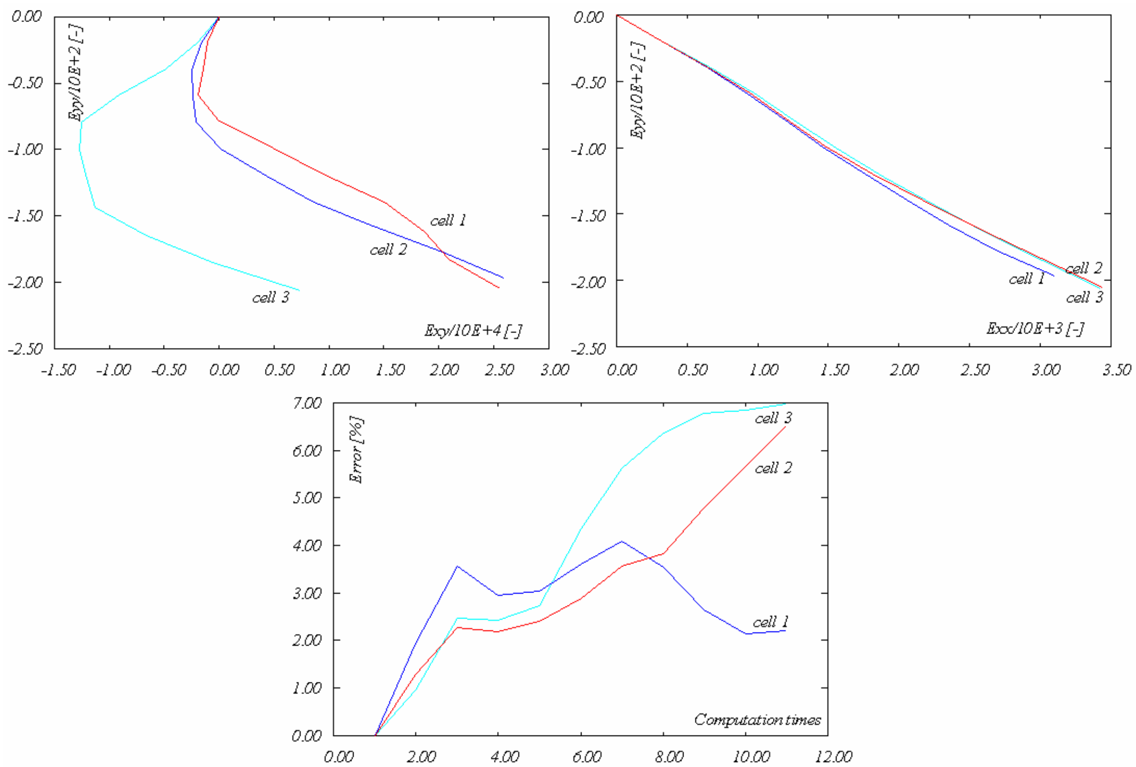

Figure 6: Vertical versus shear and horizontal strain, evolution of the error.

6 THE CELL LEVEL

Typical results at the cell level can already be found in Anthoine and Pegon (1996). Note that this level of modelling can be used to assess the results obtained on the wallettes because the periodicity condition can be perfectly applied (see Pegon and Anthoine (1997)) and no boundary effects can impair the solution.

This level of modelling is also interesting for studying 3D effects. In fact, the wall and the wallettes levels already involve too much elements for performing accurate and cheap 3D analyses. This cell level is thus an opportunity to study the possible splitting mechanism of the wall and more generally all the phenomenon for which the thickness of the wall should be taken into account.

Regarding the possibility of obtaining 3D solutions, it is worth to recall that, due to the difference of Poisson ration between the brick and the mortar, the brick is in a state of bi-traction (in the horizontal plane) when a vertical force is applied. Pegon and Anthoine (1997) showed that, if both mortar and bricks are modelled with continuum elements, the 2D results obtained with the generalized plane strain assumption should be preferred to the ones obtained with the plane stress assumption. The benefit of using the generalized plane strain assumption almost disappears when the mortar is represented by standard joint elements. Since these elements transmit only traction/compression or shear, no lateral effect could be induced when only compression is imposed.

7 CONCLUSION AND FURTHER WORK

An overview of the numerical work performed in relation with the SEISPROTEC project was presented. The masonry is modelled using a micro-mechanical Finite Element approach: the bricks are represented by continuum element whereas the mortar is modelled by joint elements. The joint model for the mortar has been presented and the importance of the traction cap has been illustrated by the computation of the wall structure. In fact, depending of the relative size of this cap, two very different mechanisms are likely to occur when the wall is loaded monotonically: the first involves unbonding at the top and bottom toes and only one compression strut and large opening of bed joints nears the toes of the wall. The second involves strong shearing along the bed joints, a heavier degradation of the central part of the wall and the formation of two compression struts. The computation on the walls allows also to individuate, by means of convenient averaging, a field representation of the principal directions of the strain and the stress during monotonic or cyclic test. If the model is correlated correctly with the experimental results this allows to decide what are the best bed joint orientation to consider on the wallettes.

The tests on the wallettes are expected to give homogeneous results about the behaviour of masonry. Here the modelling allows to verify up to which point this assumption is true. In fact the boundary conditions introduce slight heterogeneity for both the average magnitude of interest but also affect the effective principal directions of the loading. This can be assessed by performing computations at the level of the cells where perfect periodicity can be imposed.

The future work is firstly to correctly represent the results of the tests on the bare wall. This will be performed on the base of the monotonic and the cyclic tests that are currently performed. The modelling of the joints could be improved following two research lines: the introduction of a more complex model, still using conventional joint elements and the introduction of enhanced elements allowing to generate (between others) Poisson's effect on the bricks. Since they exhibit cracks, the bricks should also be modelled non-linearly. Vertical fictitious joints could be added at the centre of the bricks as proposed by Lourenço (1996). Unfortunately, this solution does not seem to work properly in this case so that the use of continuum crack models is now considered.

Since the next tests involve renforcing solutions, the second step of the future numerical work is to be able to model the effects of these interventions. For the time being, joints elements have been implemented in order to model the interface between the wall and the renforcing material that could be in turn modelled using a convenient 2D layer for the 2D analyses or 3D shell or solid elements for 3D (homogenisation) analyses.

8 REFERENCES

Anthoine, A. 1992. In-plane behaviour of masonry: a literature review. *EUR-Report No.13840 EN*, JRC-Ispra, 64 p.

Anthoine, A. 1995. Derivation of the in-plane elastic characteristics of masonry through homogenization theory, *Int. J. Solids Structures*, Vol. 32, pp 137-163.

Anthoine, A. and Pegon, P. 1996. Numerical analysis and modelling of the damage and softening of brick masonry, In: *Numerical analysis and modelling of composite materials, edited by J. Ball, Blackie Academic and Professional*, ISBN 0 794 0239 6, pp. 152-184.

Binda, L., Tiraboschi, C., Mirabella Roberti, G., Baronio, G. and Cardani, G. 1995. Experimental and numerical investigation on the brick masonry building prototype, report 5: measuring masonry material properties: detailed results from an extensive experimental research, *Politecnico di Milano, Dipartimento di Ingegneria Strutturale*.

Lourenço, P.B. 1996. Computational strategies for masonry structures. Ph.D. thesis of Delft University of Technology, Delft University Press.

Molina, J., Le Pape, Y. and Pegon, P. 2001. Seismic Assessment of Masonry Structures – Experimental Program. *These Proceedings*.

Page, A.W. 1978. Finite element model for masonry. *Journal of the Structural Division*, ASCE, Vol. 104(ST8), *Proceeding Paper 13957*, pp. 1267-1285.

Pinto, A. Molina, J., Pegon, P. and Renda, V. 2001. Protection of the Cultural Heritage at the ELSA Laboratory. *These Proceedings*.

Pegon, P. and Anthoine, A. 1997. Numerical strategies for solving continuum damage problems with softening: application to the homogenization of masonry. *Computers & Structures*, Vol. 64(1-4), pp 623-642.