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COMPUTATIONAL SIMULATIONS FOR HOMOGENIZATED MASONRY STRUCTURES BY USING 3-D DAMAGE MODEL

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Abstract. *In this paper proposed the application of two-parameters damage model, based on non-linear finite element approach, to the analysis of masonry panels. Masonry is treated as a homogenized material, for which the material characteristics can be defined by using homogenization technique. The masonry panels subjected to shear loading are studied by using the proposed procedure within the framework of three-dimensional analyses. Finally, the model is validated with a comparison with experimental results available in the literature.*

1 INTRODUCTION

In recent years growing attention has been paid by researches in structural mechanics to masonry structures with the intent to provide theoretical and numerical tools for better understanding the complex mechanical behaviour of such structures. The complex mechanical behaviour of masonry structures depends strongly on the composite nature of masonry material. Masonry is constituted by blocks of natural or artificial material jointed by dry or mortar joints; the latter are the weakness – areas of such a composite material and notably affect the overall response of the assembly with a number of kinematical modes at joints such as sliding, opening – closing and dilatancy. The predictive modeling of the masonry structures behaviour, particularly in the non-linear range, represents a challenge due to their discrete and composite nature. An adequate computational model should include the fundamental mechanisms that characterize the masonry behaviour at failure, i.e:

- sliding along a bed or head joint at low values of normal stresses,
- cracking of the masonry units in direct tension,
- diagonal tensile cracking of masonry units at values of normal stress sufficient to develop frictional behaviour in the joints and
- splitting of units in tension as a result of mortar dilatancy at high values of normal stresses.

The damage theory model revealed to be a good choice to exploit in this area of structural mechanics due to especially to its efficiency. The main problems of this model are the definition of the damage evolution curve and the introduction in the model of the ortothropy. Mechanical properties of the masonry can be obtained from the behaviour of its constitutive materials (brick and mortar) through a homogenization technique. The mechanical parameters can also be calibrated using a micro-modeling approach.

This paper presents the application of a two-parameters, isotropic, damage model, based on the finite elements method, to simulate the ultimate response.

2 THE CONCEPTION OF DAMAGE MODEL

The nonlinear behaviour of masonry can be modelled using concepts of damage theory. In this case an adequate damage function is defined for taking into account different response of masonry under tension and compression states. Cracking can, therefore, be interpreted as a local damage effect, defined by the evolution of known material parameters and by one or several functions which control the onset and evolution of damage. The model takes into account all the important aspects which should be considered in the nonlinear analysis of masonry structures such as the effect of stiffness degradation due to mechanical effects and the problem of objectivity of the results with respect to the finite element mesh.

A useful concept for understanding the effect of damage is that of effective stress. The damaged $\boldsymbol{\sigma}_d$ and effective undamaged $\boldsymbol{\sigma}$ stress tensors are correlated, according to continuum damage mechanics, by the relation:

$$\boldsymbol{\sigma}_d = (1-d)\mathbf{D}\boldsymbol{\varepsilon} = (1-d)\,\boldsymbol{\sigma} \tag{1}$$

where d is a scalar value, ranging from 0 to 1 and representing the local damage parameter, **D** is the elastic stiffness matrix and $\boldsymbol{\varepsilon}$ is the strain tensor.

The damage function $g(\bar{\tau}, r)$ defines the limit of the region of undamaged response and is written at time t as:

$$\left(g(\bar{\tau}, r)\right)^t = (\bar{\tau})^t - (r)^t \le 0 \tag{2}$$

where the undamaged complementary energy norm is defined as:

$$(\bar{\tau})^t = \gamma\sqrt{2\Lambda^0\left((\boldsymbol{\sigma})^t\right)} \tag{3}$$

where $\Lambda^0(\underline{\sigma})$ is the elastic complementary energy
For Simo's damage model $\gamma = 1$.

$(r)^t$ in the damage function (2) is the current damage strength measured with an energy norm and can be given as:

$$(r)^t = \max\left\{(r)^0, (\bar{\tau})^t\right\} \tag{4}$$

where $(r)^0$ denotes the initial damage threshold of the material.

The initial damage threshold $(r)^0$, can be considered to carry out a similar function to the initial yield stress in an analysis involving an elasto-plastic material. However, in a damage analysis, the value of the damage threshold influences the degradation of the elastic modulus matrix. A value for $(r)^0$ may be obtained from:

$$(r)^0 = \frac{\sigma_d^t}{(E_o)^{1/2}} \tag{5}$$

where σ_d^t is the uniaxial tensile stress at which damage commences and E_0 is the undamaged Young's modulus. The damage criterion is enforced by computing the elastic complementary energy function as damage progresses:

$$\beta\left(\boldsymbol{\sigma}^T \mathbf{D}_e \boldsymbol{\sigma}\right)^{1/2} - (r)^t \le 0\,. \tag{6}$$

The damage flow rule defines the damage softening and is given by

$$\dot{d} = \dot{\mu}^t \frac{\partial\left(G\left(\bar{\tau}^t, d\right)^t\right)}{\partial\tau} \tag{7}$$

where $\dot{\mu} \ge 0$ is the damage consistency parameter and defines damage loading/unloading conditions according to the Kuhn-Tucker relations

$$\dot{\mu} \ge 0, g(\bar{\tau}, r) \le 0, \dot{\mu} g(\bar{\tau}, r) = 0 \tag{8}$$

In addition, to simplify the calculations in damage analysis, the damage multiplier $\dot{\mu}$ is defined so that

$$\dot{\mu} = \dot{r} \tag{9}$$

From the consistency of the damage condition in (2) it is given that

$$\dot{\bar{\tau}} = \dot{r} = \dot{\mu} \tag{10}$$

According to (10), the definition (3) we have

$$\dot{\mu} = \frac{\gamma^2}{\bar{\tau}} \underline{\boldsymbol{\sigma}}^T \mathbf{D}_e^{-1} \underline{\dot{\boldsymbol{\sigma}}} \tag{11}$$

$(\partial G / \partial \bar{\tau})^t$ defines the damage rate with respect to the undamaged elastic complementary norm. If the damage potential function G is assumed to be independent of d, substitution of (10) into (7) will lead to:

$$d = \mathbf{G} \tag{12}$$

with the undamaged condition being enforced so that

$$\left\{ \mathbf{G}\left(r^t\right)\Big|_{(r)^t=(r)^0} \right\} = 0 \tag{13}$$

Damage accumulation functions is given by:

$$\mathbf{G}\left((r)^t\right) = 1 - \frac{(r)^0 (1-A)}{(r)^t} - A \exp\left[B\left((r)^0 - (r)^t\right)\right]. \tag{14}$$

For no damage, $\mathbf{G}(r)^t = 0$. The characteristic material parameters, A and B, would generally be obtained from experimental data.

3 NUMERICAL ANALYSIS

The damage model described in the present paper has been implemented in numerical analysis of masonry panel subjected to shear loading (schematically reported in Fig.1). The applied load in the numerical simulation consisted in incremental shear loading. The masonry panel is characterized by the following geometrical parameters:

H = 96 cm , L = 104 cm , W = 12 cm.

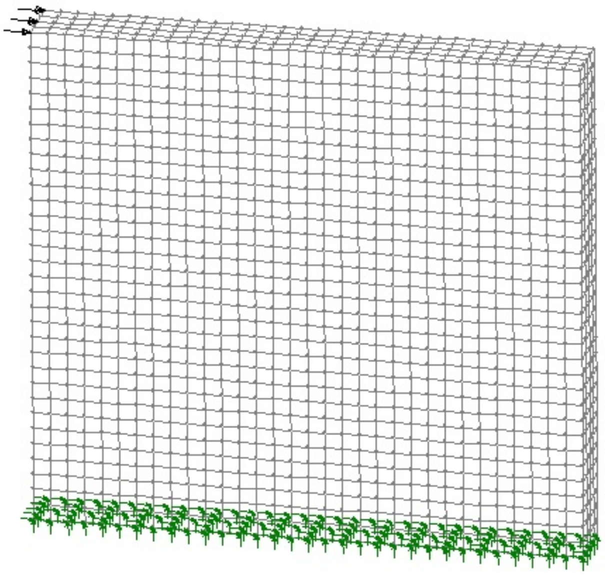

Figure 1 Finite element mesh and load condition for analysed masonry panel

The values of the mechanical parameters used in the numerical analysis to describe the masonry behaviour are summarized in Table 1, 2. Meshes obtained adopting 8-node solid elements are considered for the computations.

The comparison between numerical and experimental (taken from technical literature) load- – displacement diagrams is shown in Figure 2. The numerical analysis reproduces satisfactorily the experimental behaviour. The trend of the experimental diagrams shows that masonry panel should not completely collapsed at the end of the experiment.

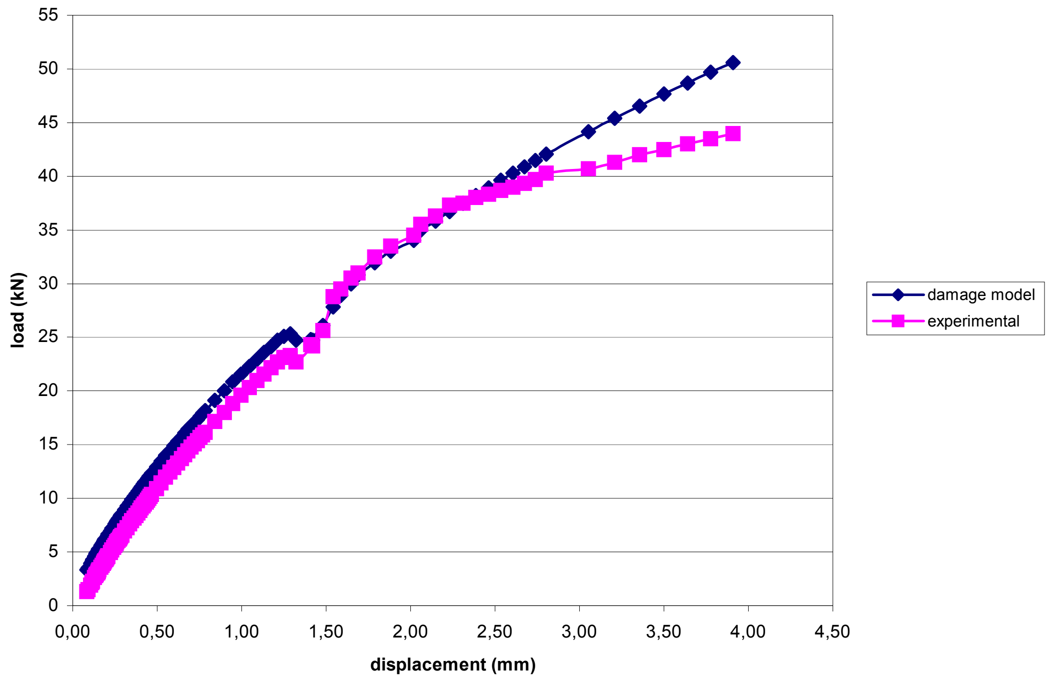

Figure 2 Load- displacement diagram for the masonry panel

Undamaged elastic modulus	E = 2500 MPa
Shear modulus	G_{xy} =900 MPa
Poisson's ratio	ν_{xy} = 0,10

Table 1 Elastic material properties of masonry

Uniaxial elastic limit in compression	f_{c0} = 1,80 MPa
Uniaxial initial compressive strength	f_c = 3,00 MPa
Uniaxial initial tensile strength	f_t = 0,2 MPa
Shear strength	f_τ = 0,4 Mpa
Fracture Energy	G_f = 0,4 N/mm
A parameter	A_c = 0,3
B parameter	B_c = 1,0

Table 2 Material properties of masonry used in the damage model

An incremental solution strategy has been proposed, based on a formulation in terms of displacements, stresses, damage and load parameters, and on a path-following iterative scheme which solves, in the incremental step, all the equations of the problem. It allowed a reliable recovery of the whole wall behaviour, even in the presence of strongly unstable damage growth.

The damage contour for principal stress is shown in Figure 3. The failure mechanism is characterized by the formation, growth and propagation of inclined damage bands, as it typically occurs in structures subjected to horizontal forces. It can be noted that the mechanical response of wall subjected to shear loading is characterized by:

- an initial elastic response;
- a first steep softening branch due to the damage propagation concentrated where the maximum tensile strains occur;
- a hardening phase during which the plastic evolution process becomes more significant than the damage one;
- a softening branch due to the formation and growth of the damage band.

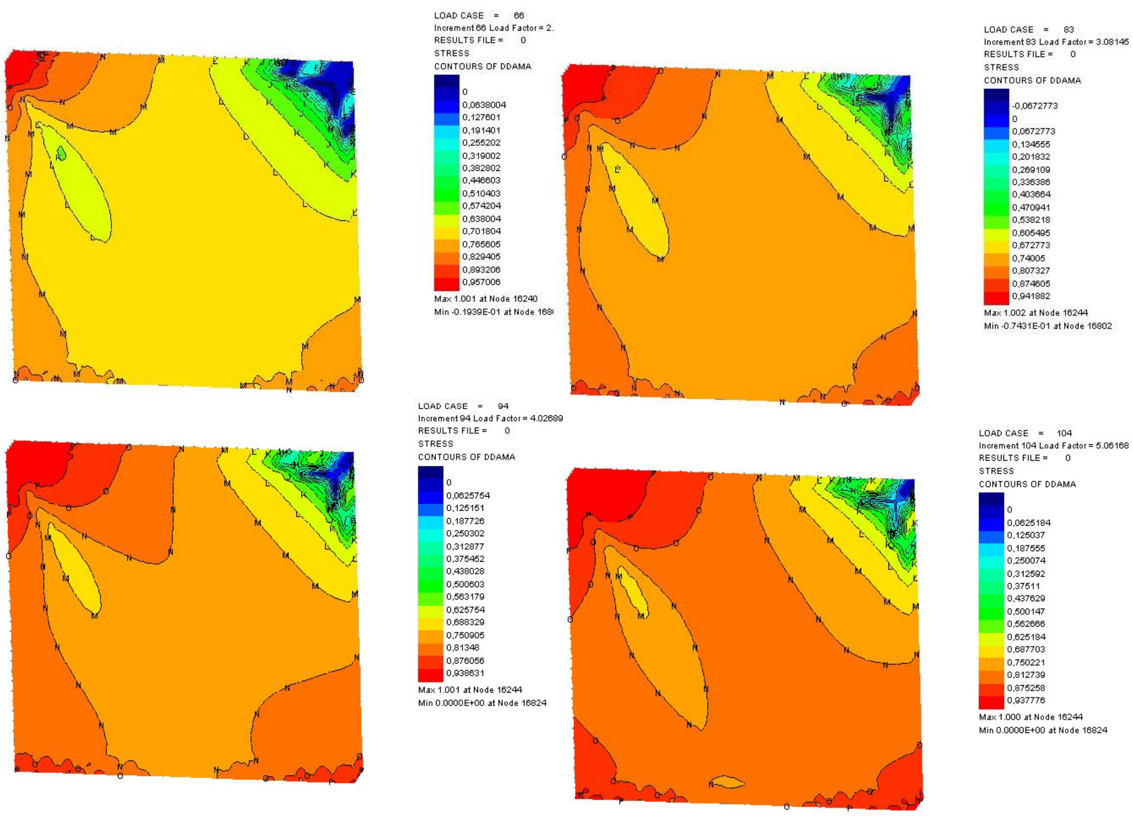

Figure 3 Damage contours for principal stress (for different factor of shear loading: a) Load Factor = 2; b) Load Factor = 3,08; c) Load factor = 4,02; d) Load Factor = 5,06)

4 CONCLUSIONS

Proposed damage model, able to represent the behaviour of homogenizated masonry structures, has been formulated and applied to the analysis of masonry shear panels. The comparison with experimental results have confirmed its capability of well reproducing, qualitatively and quantitatively, the complex behaviour of the masonry structures.

One of the advantages of a such a model is the independence of the analysis with respect to cracking directions which can be simply identified a posteriori once the non-linear solution is obtained. This allows to overcome the problems associated with most elastic-plastic-brittle smeared cracking models. Moreover valuable features of this model are:

- the relatively few number of the required material parameters;
- its easy implementation into existing finite element codes.

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