Computational implementation of constitutive models of mortar joints for the analysis of masonry panels in ANSYS

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Abstract. The modeling of a masonry panel may be a complex task due to the anisotropic behavior of the composite material. Deformation and failure properties depend on both the stress orientation and the joint direction, since the mortar-unit interfaces act as weak surfaces. The simplified micro-modeling approach is one of the several strategies that have been proposed for the numerical analysis of masonry panels in the frame of the Finite Element Method. This approach makes use of zero-thickness interface elements to represent mortar joints and simulate their sliding and opening behavior with appropriate constitutive models. This work analyzes the behavior of structural masonry panels by using the simplified micro-modeling approach with the application of the Finite Element Method software ANSYS. Two plasticity constitutive models for the mortar joints could be investigated: a Mohr Coulomb envelop with tension cut-off and compressive cap with hardening, and a Mohr Coulomb envelop with tension cut-off and bilinear damage. The corresponding algorithms were implemented into an ANSYS user material subroutine by employing an implicit scheme using the return mapping algorithm. Numerical results were compared to some experimental results available in literature.

Keywords: masonry; finite element method; interface elements.

1 Introduction

Numerical modeling of historical and contemporary masonry buildings can positively contribute in understanding the structural behavior and provide a guidance for design and retrofit studies (Giordano et al. [1], Theodossopoulos et al. [2], Lourenço [3]). However, modeling masonry structures may be challenging due the complex material behavior; masonry is a heterogeneous and anisotropic quasi-brittle material, with high compressive strength but low tensile strength, with nonlinear joint behavior and several rupture mechanisms (Lourenço [4]).

Two main approaches using the Finite Element Method are usually applied to model masonry panels: macro-modeling and micro-modeling (Lourenço et al. [5], Lourenço [6]). The micro-modeling approach treats units and mortar joints separately and explicitly represents the unit-mortar interfaces. In the detailed micro-modeling, both units and mortar are discretized into solid elements while only the unit-mortar interface is discretized into interface elements. In the simplified version, the unit boundaries are extended so that interface elements represent unit extensions, mortar and unit-mortar interface (Fig 1-a). As sliding and separation between units and mortar are taken into account, the micro-modeling approach provides better estimates of the local response of the masonry structure. Moreover, the use of specific constitutive models for mortar joints is imperative to capture the real masonry behavior near collapse.

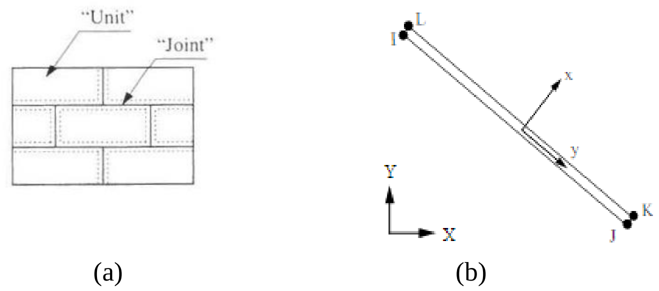


Figure 1. FE modeling of masonry panels: (a) the simplified micro-modeling approach (adapted from Lourenço [1]); (b) Interface element INTER202 (adapted from ANSYS [7]).

This work uses de FE software ANSYS® with the simplified micro-modelling approach for the plane stress analysis of masonry panels. Horizontal joints were modeled with zero thickness interface elements INTER202, as depicted in Fig. 1-b. In this element, strain components ε_N and ε_S are measured as the relative displacements between top (face LK) and base (face IJ) in the normal and tangential directions, respectively. A decoupled constitutive elastic matrix $\boldsymbol{D}^E$ relates the stresses and elastic strains acting on the interface plane as

$$\begin{Bmatrix} \sigma \\ \tau \end{Bmatrix} = \boldsymbol{D}^E \begin{Bmatrix} \varepsilon_N^E \\ \varepsilon_N^E \end{Bmatrix}, \;\; \boldsymbol{D}^E = \begin{bmatrix} K_N & 0 \\ 0 & K_S \end{bmatrix}, \tag{1}$$

where σ is the normal stress (x-direction), τ is the tangential stress (y-direction), K_N is the normal stiffness coefficient and K_S is the tangential stiffness coefficient.

In ANSYS®, interface elements may be used with a cohesive damage law, either bilinear or exponential, which alone shows inability to reproduce compressive hardening and frictional behavior. For this reason, this work implements more appropriate constitutive models by means of the ANSYS® user subroutine userCZM in FORTRAN language.

The subroutine userCZM is called for each iteration of the global nonlinear problem at each Gauss point. The input data are stress and strain values of the previous iteration ($\boldsymbol{\sigma}_i$ and $\boldsymbol{\varepsilon}_i$) and the current strain increment ($\Delta\boldsymbol{\varepsilon}$). As a result, the subroutine returns stress values of the current iteration ($\boldsymbol{\sigma}_{i+1}$) as well as the consistent Jacobian matrix ($\frac{d\sigma_{i+1}}{d\varepsilon_{i+1}^E}$). The programmed algorithms were tested for several return-mapping cases and validation examples.

2 Implementation of constitutive models for mortar joints into ANSYS

2.1 Mohr Coulomb yield envelop with tension cut-off and compressive cap with hardening

This constitutive model, first proposed by Lourenço [8], is comprised by three yield surfaces (see Fig. 2-a) given as

$$f_1 = |\tau| + \sigma \, tg\phi - c\,, \tag{2}$$

$$f_2 = \sigma - \sigma_T\,, \tag{3}$$

$$f_3(\kappa) = (\sigma - \sigma_M(\kappa))^2 + \tau^2 - r^2(\kappa)\,, \tag{4}$$

in which f_1 express a Mohr-Coulomb envelop, f_2 defines a cut-off surface in traction, and f_3 defines a circular cap in compression with center and radius

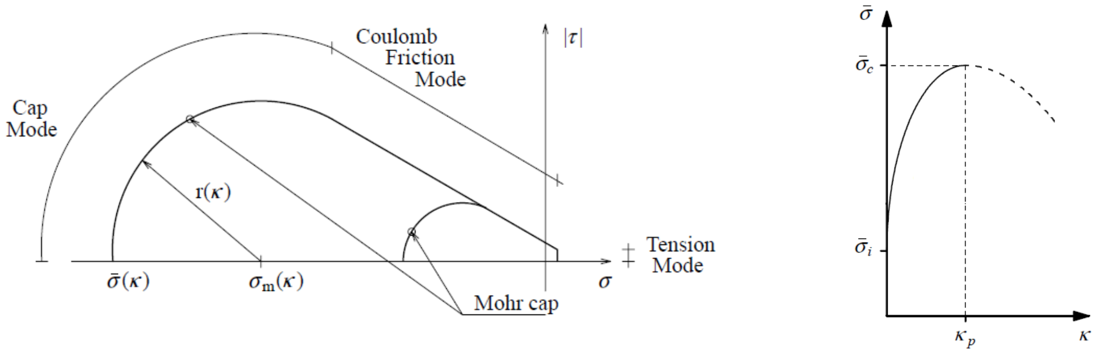
$$\sigma_M(\kappa) = \frac{-\bar{\sigma}(\kappa) + c\cos\phi}{1 + \sin\phi}, \tag{5}$$

$$r(\kappa) = \frac{\bar{\sigma}(\kappa)\sin\phi + c\cos\phi}{1 + \sin\phi}. \tag{6}$$

In the above expressions, the property parameters are cohesion (c), friction angle (ϕ), tensile strength (σ_T), and compressive strength ($\bar{\sigma}$). Hardening occurs only in the compressive cap, controlled by a hardening parameter (κ) that affects the compressive strength in a parabolic relation (van der Pluijm and Vermeltfoort [9])

$$\bar{\sigma} = \bar{\sigma}_i + (\bar{\sigma}_c - \bar{\sigma}_i)\sqrt{\frac{2\kappa}{\kappa_p} - \frac{\kappa^2}{{\kappa_p}^2}}, \tag{7}$$

where σ_c is the maximum compressive strength reached for a hardening κ_P, and the initial compressive strength is taken as a third of $\bar{\sigma}_c$ (see Fig. 2-b).



(a) (b)

Figure 2. Constitutive model for unit-mortar interface: (a) Mohr-Coulomb yield envelop with cut-off in traction and a circular cap in compression (reproduced from Lourenço [8]); (b) evolution of the hardening parameter.

The constitutive model was formulated in the frame of the classic plasticity theory, with associative flow rule and strain-hardening law. The discrete problem was integrated by applying the implicit return-mapping algorithm, which uses the prediction-correction scheme. For the case of an active compressive cap, the return-mapping algorithm results in a nonlinear equation with the hardening increment as variable, so the Newton-Raphson method was employed. For the sake of brevity, expressions were omitted here – the reader may refer to Lourenço [6] and Silva [10] for further details.

2.2 Mohr Coulomb yield envelop with tension cut-off and bilinear damage

Damage models provide a simple way to represent stiffness degradation after cracking initiates. Although a bilinear damage law is available for interface elements in ANSYS [7], a new subroutine was implemented to include the Mohr Coulomb yield envelop (described by Eq. 1) in the constitutive model.

The normal and tangential damage modes, and consequently their parameters, were considered independent from each other, as well as the traction and compressive behavior. For each fracture mode, a damage parameter D_k reduces the corresponding stiffness coefficient K_k after the strain reaches a specified limit ε_{k_0}. In the case of a bilinear law,

$$D_k = \max\{0, \min\{1, \bar{D}_k\}\} \quad \text{where} \quad \bar{D}_k = 1 - \frac{\eta_k}{\eta_k - 1}\frac{\varepsilon_{k_0}}{|\varepsilon_k|} + \frac{1}{\eta_k - 1} \quad \text{and} \quad \eta_k = \frac{\varepsilon_{k_R}}{\varepsilon_{k_0}}, \tag{8}$$

being $\varepsilon_{k_R}$the strain for which K_k becomes null (Asfano and Crisfield [11] apud ANSYS[7]). An explicit direct integration scheme was employed to calculate the evolution of the damage parameters D_k. The Mohr-Coulomb yield envelop was formulated similarly to the previous section.

3 Examples

3.1 Example 1: Masonry panel subjected to compressive load and settlement

A reduced-scale model comprised by a masonry panel of ceramic units (Fig. 3) supported by a concrete beam was experimentally tested by Holanda Jr. [12]. First, a compressive load of 1.553 MPa was applied on the top of the panel. After that, a settlement was prescribed to the central beam support in such a way that the corresponding reaction force became null. However, experimental results showed that all beam supports had experienced dislocations during the test; so prescribed values were applied in the numerical model accordingly (see Table 1).

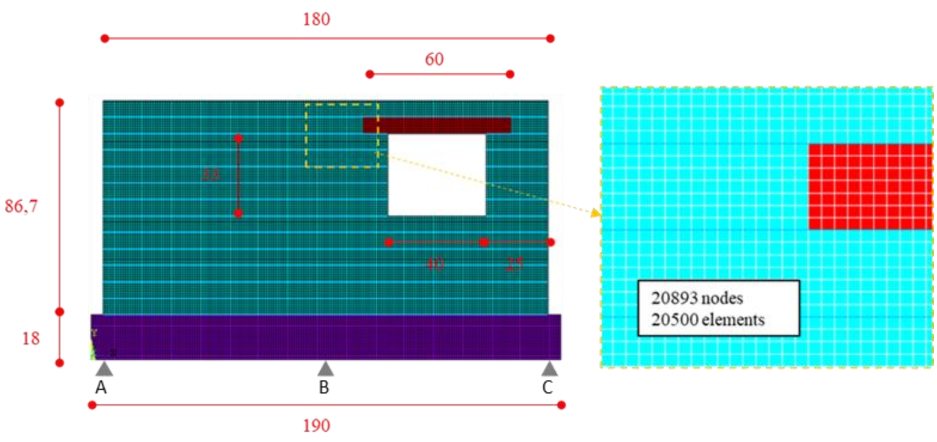


Figure 3. Example 1: Geometric configuration and FE mesh (adapted from Andrade [13], dimensions in mm).

Table 1. Example 1: Displacement values measured at the beam supports after each load stage in the experimental test.

Beam support	After the application of the compressive load	After the settlement of the beam support B
A	1.08 mm	1.27 mm
B	1.31 mm	3.02 mm
C	0.84 mm	0.84 mm

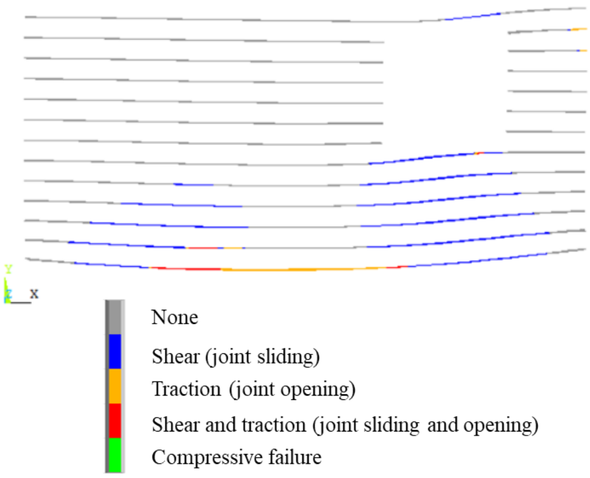
The masonry panel was discretized into linear plane elements and zero-thickness interface elements with a mesh size of 10 mm (Fig. 3). However, mesh generation using interface elements is not a straightforward task – several issues emerged at joint intersections that could not be properly handled in ANSYS®. For this reason, head mortar joints have been disregarded.

Ceramic units (mass density = 1300 kg/m^3) and concrete elements (mass density = 2500 kg/m^3) were assumed as homogeneous and isotropic linear materials. The Mohr-Coulomb elastoplastic constitutive model of Section 2 was assigned to interface bed joints. Deformability and strength parameters of units and mortar were obtained from Holanda Jr. [12] (Table 2). For this masonry panel, mortar has shown stiffer than ceramic units so a continuum equivalent technique could not be applied for the evaluation of stiffness coefficients. The normal and tangential stiffness coefficients were calibrated numerically to match the panel failure pattern and null the force reaction of support B. Numerical results show good agreement with the experiment by Holanda Jr. [12]. As can be seen in Fig. 4, the bed joint failure mechanisms that contributed to the cracking pattern could be identified.

Table 2. Example 1: Deformability and strength material parameters.

Material	Deformability parameters	Strength parameters
Ceramic (units)	E = 10554 MPa ν = 0.1	
Mortar (joints)	E = 15270 MPa ν = 0.2 K_N = 55 MPa/mm K_S = 23 MPa/mm	σ_{cR} = 15.81 MPa σ_{tR} = 0 MPa c = 0.6 MPa ϕ = 36.9º
Concrete (beam)	E = 18620 MPa ν = 0.2	
Concrete (lintel)	E = 27060 MPa ν = 0.2	

E =Young's modulus
ν = Poisson ratio
K_N = Normal stiffness coeff.
K_S = Tangential stiffness coeff.
σ_{cR} = Compressive strength
σ_{tR} = Tensile strength
c = Cohesion
ϕ = Friction angle



(a) (b)

Figure 4. Example 1: (a) Cracking pattern obtained from the experiment (from Holanda Jr. [12], p. 94) (b) Deformed panel (scale=25) and active joint failure mechanisms obtained from numerical results.

3.2 Example 2: Masonry panel subjected to compressive and horizontal load

Mata [14] experimentally tested a masonry panel of concrete units (thickness=140 mm) supported by a concrete slab (Fig. 5). First, a compressive load of 150 kN was applied on the top of a sufficiently rigid steel beam, which in turn transmitted the load evenly to the top of the panel. In this step, no damage to the structure was observed. After that, a horizontal load was applied up to the failure of the panel at 52,09 kN.

The masonry panel was discretized into linear plane elements and zero-thickness interface elements with a mesh size of approximately 16.5 mm (Fig. 5). Head mortar joints were once again disregarded. The bilinear damage model from Section 2.2 was assigned to the bed joints. Deformability and strength parameters of units and mortar were obtained from Mata [14] (Table 3).

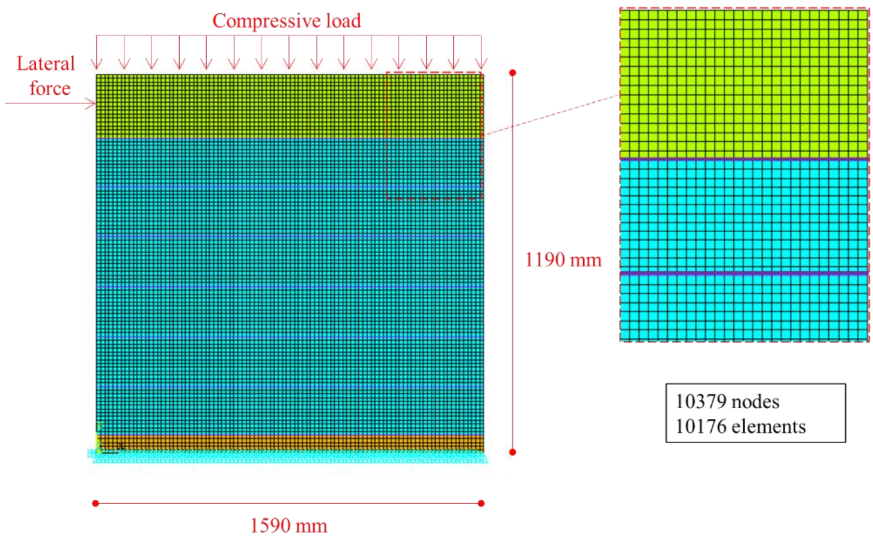


Figure 5. Example 2: Geometric configuration and FE mesh (dimensions in mm).

Table 3. Example 2: Deformability and strength material parameters.

Material	Deformability parameters	Strength parameters
Concrete units	E = 7586 MPa ν = 0.37 ρ = 1300 kg/m³	
Mortar (joints)	E = 4860 MPa ν = 0.09 K_N = 58.49 MPa/mm K_S = 161.82 MPa/mm G_c = 11.64 MPa.mm G_t = 0.005 MPa.mm	σ_{cR} = 15.81 MPa σ_{tR} = 0.093 MPa c = 0.2086 MPa ϕ = 31.467°
Concrete (beam)	E = 20000 MPa ν = 0.2 ρ = 2500 kg/m³	
Steel (beam)	E = 210000 MPa ν = 0.3 ρ = 7850 kg/m³	

E =Young's modulus
ν = Poisson ratio
K_N = Normal stiffness coeff.
K_S = Tangential stiffness coeff.
σ_{cR} = Compressive strength
σ_{tR} = Tensile strength
c = Cohesion
ϕ = Friction angle
G_c = Compressive fracture energy
G_t = Tensile fracture energy

The major failure mechanism observed in the experiment was the opening of the bed joint between the panel and the concrete base (Fig. 6-a). The numerical model was able to correctly identify the cracking pattern, as shown in Fig. 6-b which presents the evolution of the tension damage variable D_k for the bed joints.

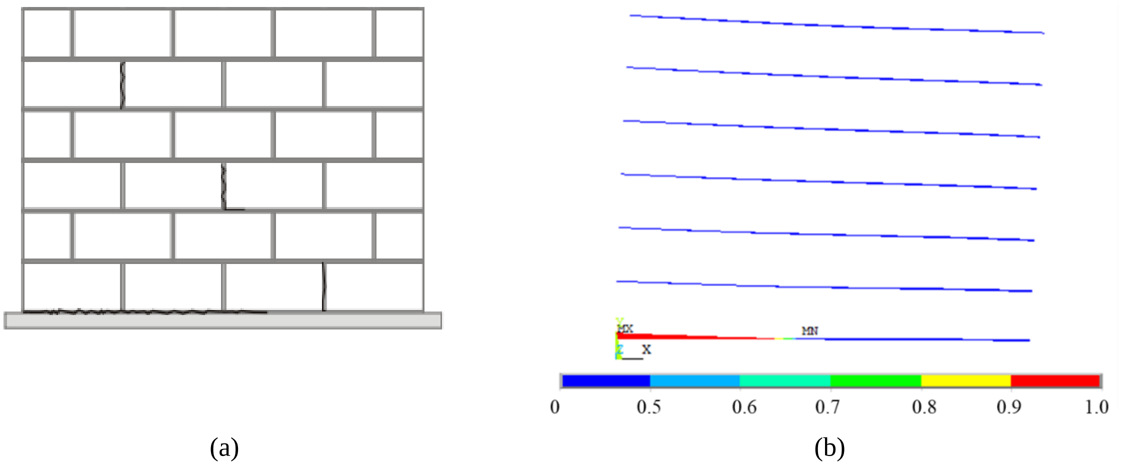


Figure 6. Example 2: (a) Cracking pattern obtained from the experiment (from Mata [14], p. 124) (b) Deformed panel (scale=25) and traction damage parameter obtained from numerical results.

4 Conclusions

This work analyzed the behavior of structural masonry panels by using the simplified micro-modeling approach with the application of the Finite Element Method software ANSYS. Two plasticity constitutive models for mortar joints could be investigated: a Mohr Coulomb envelop with tension cut-off and compressive cap with hardening, and a Mohr Coulomb envelop with tension cut-off and bilinear damage.

Numerical results of the presented examples made evident the importance of modelling properly the joint behavior of masonry structures, in special the shear and traction failure mechanisms. It should be noticed, however, that the success in realistically represent the behavior of a masonry structure is highly dependent on the input data quality. Values for the normal and shear stiffness coefficients as well as for the other input parameters required by the joint constitutive model should be assessed through laboratory tests. In this sense, a bilinear damage model should be preferred over the compressive cap since it may provide good results with fewer tests.

Using the Finite Element software ANSYS® with the user subroutine userCZM is a viable solution for the micro-modeling of masonry structures. However, issues may arise in mesh generation when using interface elements.

Acknowledgements. The authors gratefully acknowledge the support from Carlos Chagas Filho Research Support Foundation of Rio de Janeiro State (FAPERJ).

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