A detailed micro-modelling approach for the diagonal compression test of strengthened stone masonry walls

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Abstract. The aim to retrofit and preserve the monumental stone masonry buildings due to their historical and cultural relevance is accompanied by the necessity of understanding the behaviour of the unstrengthen structure, as well as its behaviour after the strengthening systems are applied. There is scarce information related to the mechanical properties of stone masonry buildings and even less regarding the assessment of these characteristics in numerical models. Therefore, simulating the force displacement variation and the stress-strain distribution of stone masonry loaded in diagonal compression is a challenging issue. This work contributes to this topic by developing two detailed micro non-linear 3D models. The first model was designed for an unreinforced masonry (URM) wall and the second one was developed for a strengthened URM wall. For this purpose, a commonly used seismic strengthening system, referred to as reinforced plastering mortar (RPM) or textile reinforced mortar (TRM) was applied on the wall. All the components of the TRM strengthening system and the interfaces between the system and the stone masonry wall were considered in the numerical model. The structural responses of the models were analysed and compared and the TRM system effectiveness in increasing the in-plane load resistance and ductility of stone masonry walls was highlighted.

1. Introduction
In Romania, like in many countries throughout the world, stone masonry structures were erected long before the 20th century and nowadays a large share of these buildings are part of the historical heritage of mankind. The low tensile strength and the brittle behaviour make stone masonry structures vulnerable to seismic actions. In addition, these structures were often subjected to environmental deterioration and various changes in loading. Due to these issues, a large share of these structures require strengthening interventions. In this respect, each strengthening strategy should be designed by taking into account both the behaviour of the existing structure as well as its behaviour after the proposed retrofitting interventions are made [1].

From a methodological point of view, the structural assessment techniques can be classified in two main groups: analytical methods and numerical approaches [1, 2]. The analytical methods are based on rigid body mechanics and they usually consist in evaluating a low number of variables. Even though these methods were proved to provide valuable and reliable outcomes for particular applications, they also present a significant drawback. More explicitly, regardless the type of the analytical method, the designer should be able to identify the specific collapse mechanism and select the analysis assumptions suitable for the particular analysed case. These critical aspects are difficult to be
addressed in case of stone masonry buildings, which are made of a heterogeneous material composed of stone units and joints (with or without mortar). The stone units may widely vary in terms of mineralogical particularities, sizes and shapes. Besides, the mortar layers can be made with many different materials (clay, lime, cement-based, etc.), usually in a non-uniform thickness throughout the element. Therefore, the assumptions on which the classical analytical methods were developed for the structural assessment of brick masonry structures cannot be applied to heterogenous stone masonry structures.

On the other hand, the advance in computational capabilities and methods has led to the development of a wide variety of numerical approaches which can account for the individual components of the stone masonry structures [3-6]. So far, the macro-numerical finite element (FE) models have been generally accepted for the numerical analysis of various masonry strengthening systems. These types of numerical models can provide reliable results at low computational cost. However, when the strengthening systems include composite elements oriented in different directions, the macro-numerical models may be unable to account for all the components of the stress-strain state. For example, in the case of textile reinforced mortar (TRM) strengthening systems, the stresses usually concentrate in narrow areas along the faces of the transversal connectors, elements which are not modelled in the macro-numerical FE analysis. In order to overcome this shortcoming, more recent modelling strategies were developed to enable identification and monitoring of all components of the stress-strain state of any type of strengthening systems of stone masonry structures. In the detailed micro-numerical approach, the stone units, the mortar joints and the constituents of the strengthening system are modelled separately as continuum elements and the interfaces between all the components are represented as discontinuous elements. Using either mathematical models to generate fictitious random polygonal shapes or image processing techniques, the irregular shapes of stones and mortar layers can be implemented in the numerical models. It can be noted that the micro-numerical models are designed to account for all components of the physical models, the corresponding materials and their characteristics, thus reflecting the realistic behaviour of the stone masonry structures strengthened with TRM.

In this study, a detailed micro non-linear 3D model has been developed to simulate the diagonal compression test of stone masonry walls strengthened with basalt fibre meshes embedded in a high-ductility lime-based mortar. For comparison purposes, a benchmark stone masonry wall was modelled without the TRM strengthening system. Various pictures of old stone masonry walls were taken and analysed, and the determined geometrical characteristics were reproduced in the numerical models. In this way, the latter account for the arbitrary stone arrangements in masonry walls and for the varying thicknesses of the mortar layers. The numerical simulations provided results in good agreement with the ones obtained by previous experimental tests, available in the literature.

2. Numerical modelling
The numerical study presented in this paper was carried out using the multi-purpose finite element software Ansys. For nonlinear analyses, the available numerical models from the software engineering data base allow to account for the influence of existing pre-damages, as well as for the distinct behaviour of materials in compression and tension. Beside these features, the software contains an extensive library of various constitutive laws, including linear/multi-linear, exponential and parabolic, which enable an exhaustive modelling of the softening behaviour of materials in compression and tension [7-9].

2.1. Model definition
To simulate the in-plane behaviour of the stone masonry walls strengthened with TRM under diagonal compression test, a micro non-linear 3D model was conceived. The latter accounts for the detailed masonry structure and for all components of the strengthening system. The stones and the mortar were modelled by individual units and the interfaces were modelled by contact/interface elements.
2.1.1. Modelling strategy. The geometrical characteristics of the micro-models were assessed so as to accurately represent the configurations of the experimental panels. For this purpose, several Romanian heritage stone masonry structures were analysed (figure 1). These monumental buildings were made with sedimentary limestone units and lime mortar. The stone units were extracted probably from Cotnari quarry, which had been the reservoir of these construction materials for the Eastern part of Romania, since the Medieval Ages.

![Figure 1. Stone Masonry monumental constructions a) Hadâmbu Monastery Jassy, b) The enclosure walls of Sihăstria Monastery, c) Detail A-A.](image)

In the second stage, the experimental panels (1200 x 1200 x 250 mm) were built, using the same types of stone units and mortar as in the analysed structures. The stone units were profiled to match the shapes identified in the pictures as close as possible. The masonry panels were built in the test position by laying the diagonal stone layers as shown in figure 2(b). During the construction and the handling stages, the panels were supported on their lower faces (figures 2(b), (c)). The stone masonry panels will be tested in diagonal compression and the outcomes of this experimental program will be presented in a subsequent paper.

![Figure 2. (a) Stone units, (b) Building process, (c) Experimental panel.](image)

During the execution of the experimental panels, several photos were taken from the same angle, before and after applying the lime plaster finish layer. These images were processed based on various parameters, including: pixels colours, pixels brightness values, grey levels and visible surface topography. Therefore, two distinct shape patterns were identified on the pictures, a continuous one
corresponding to the mortar layers (figure 3(b)) and a sequential one belonging to the stone units (figure 3(c)).

Figure 3. (a) Experimental panel, (b) Traces of mortar layers, (c) Traces of stone units.

The shape patterns were imported in a computer-aided design (CAD) software (AutoCAD), (figure 4(a), (b)) and merged together so as to obtain the complete image of the experimental walls (figure 4(c)). The overlapping layers were rectified and the final image was checked for any possible lines’ intersections. The total overlapping percentage before rectification was below 5%. Thus, it can be concluded that the shape patterns that had been previously determined were consistent with the configuration of the experimental panels.

Figure 4. 2D configurations of (a) mortar layers, (b) stone units, (c) stone masonry wall.

The 2D geometry was extruded with 250 mm along the Y-axis to obtain the final 3D shape of the panel (figure 5). Figure 5(a) and (b) illustrate the extruded geometries of both the mortar layers and the stone units. As it can be observed, the irregularity of the topographic profiles of the stone masonry constituents is represented in a realistic manner. The image processing techniques show two significant advantages when compared to the mathematical models designed to generate fictitious random polygonal shapes. First, this method accounts for both the geometries of stone and mortar layers, while the mathematical models generate polygonal shapes only for the stone units. Secondly, the stone units are represented close to their real shapes, while the mathematical models built random polyangular bodies which can easily concentrate stresses in the vicinity of their corners.

The 3D geometries were exported from AutoCAD to Ansys (figure 6). Two sets of bodies were assessed, a continuous one for the mortar layer and a sequential one composed of 60 parts for the stone units. The TRM components and the transversal connectors were modelled directly into the Ansys interface by using SHELL 63 finite element, since these types of components have both bending and membrane capabilities. Furthermore, bonded contact elements of type CONTA174 were defined at each interface level (stone-mortar, mortar-TRM and stone-TRM). These types of elements may
accurately simulate the interface delamination, since they were modelled to allow the contact separation for the stone units-TRM and mortar-TRM interfaces.

The interface levels that were mentioned above were defined by The Augmented Lagrange formulation. This type of contact formulation can be used only when the penetration between the contact and the target bodies is detected and controlled at a specific level [10]. The penetration points were assessed by applying integration point detection to the stone units and the mortar layers. This is
an iteration method that provides significantly more detection points when compared to the classical nodal detection method.

The mesh size was kept different for stone units, mortar layers, TRMs and transversal connectors, as per their geometrical configurations. For the stone units, the maximum mesh size was chosen as 4 mm, while for the mortar layers, TRM and transversal connectors, the mesh size was kept as 2 mm (figure 7). Thus, the mesh of the unstrengthen stone masonry wall consists in 24465 elements and 146635 nodes, while the mesh of the strengthened wall is made of 81599 elements and 456156 nodes, respectively.

![Figure 7. Mesh details of: a) mortar layers, b) stone units, c) unstrengthen stone masonry wall, d) TRM strengthened stone masonry wall.](image)

2.1.2. Material properties. Stone masonry is a heterogeneous material made with stone units and mortar. The mechanical models developed to characterise the shear structural behaviour of stone masonry structures falls under the category of no-tension material models. In general, any type of unreinforced masonry walls is considered to be incapable of withstanding significant tensile stresses and thus, they behave like a linear elastic material when subjected to compressive stresses. Nonetheless, depending on the mechanical and elastic characteristics of the masonry constituents, the mechanical model and, by default, the numerical one may shift from a linear-elastic towards an elastoplastic one.

The Young's modulus of real scale masonry wall specimens is usually distinct from the Young's modulus which is evaluated on prism test specimens. However, since this study is performed before the experimental program, the value of the Young’s modulus was selected as 14800 MPa, as determined on prism specimens in a similar research work [11]. Later, in the following stages, the numerical model will be updated using the experimental results.

A micro-numerical modelling approach based on interface FE requires two different stiffness levels, namely the stiffness corresponding to the stone units and the stiffness of the mortar layers (joints). The normal joints stiffness ($K_{n,\text{joint}}$) was calculated using the approach proposed by Lourenço (equation (1)) [12]. According to this formulation, the masonry wall is assimilated with a couple of springs acting in vertical direction, one symbolizing the stone units and the other symbolizing the joints.

$$K_{n,\text{joint}} = \frac{1}{h \left( \frac{1}{E_{\text{wall}}} + \frac{1}{E_{\text{stone}}} \right)}$$

(1)
Where:

\[ K_{n,joints} - \text{Normal joints stiffness}; \]

\[ h – \text{Height of the stone units (150 mm – average value)}; \]

\[ E_{\text{wall}} – \text{Young’s modulus of the wall}; \]

\[ E_{\text{stones}} – \text{Young’s modulus of the stones}. \]

The tangential stiffness of the joints \( (K_{s,joints}) \) was calculated according to The Theory of Elasticity, as follows:

\[ K_{s,joints} = \frac{K_{n,joints}}{2(1 + \nu)} \tag{2} \]

Where:

\[ K_{s,joints} - \text{Tangential joints stiffness}; \]

\[ \nu - \text{Poisson’s ratio equal to 0.2 for masonry}. \]

The inelastic parameters consist in 3 criterions that were assessed to the stone units-mortar interface. The inelastic criterions were as follows: Criterium 1 (tensile) - \( f_t \) (tensile strength) and \( G_{I}^{f} \) (fracture energy for Mode I); Criterium 2 (friction) - \( c \) (cohesion), \( \tan \phi \) (tangent of the friction angle), \( \tan \psi \) (tangent of the dilatancy angle) and \( G_{II}^{f} \) (Mode II fracture energy); Criterium 3 (compressive) - \( f_c \) (compressive strength) and \( G_{c}^{f} \) (compressive fracture energy). The inelastic properties of each criterion were selected as determined by Vasconcelos [13] observing the provisions given in [12]. The elastic and the inelastic properties of the stone masonry wall are given in tables 1 and 2.

| Table 1. The elastic properties of the stone masonry wall. |
|-----------------------------------------------------------|
| Young’s Modulus, \( E \) (MPa) | Poisson’s ratio, \( \nu \) | Normal joints stiffness, \( K_{n,joints} \) | Tangential joints stiffness, \( K_{s,joints} \) |
|---------------------------------|-----------------|-----------------|-----------------|
| 14800                           | 0.2             | 3.5             | 1.575           |

| Table 2. The elastic properties of the stone masonry wall. |
|-----------------------------------------------------------|
| Criterion 1 - Tension | Criterion 2 - Shear | Criterion 3 - Compression |
| \( f_t \) (MPa) | \( G_{I}^{f} \) (MPa) | \( c \) | \( \tan \phi \) | \( \tan \psi \) | \( G_{II}^{f} \) (MPa) | \( f_c \) (MPa) | \( G_{c}^{f} \) (MPa) |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| 0.05            | 0.01            | 0.1             | 0.4             | 0               | 0.1             | 6.1             | 9.0             |

The stone masonry strengthening system analysed in this study consists in a high ductility, lime-based, bi-component, mortar (Planitop HDM Restauro) [14] reinforced with a basalt fibre mesh (Mapegrid B 250) [15]. This TRM was applied on both faces of the stone masonry wall and the structural connections between the system and the supports were achieved by means of through-wall, basalt fibre cords (MapeWrap B FIOCCO) [16] impregnated with a bi-component, super-fluid epoxy resin (Mapei Epojet) [17]. Most of the necessary material properties were determined and provided by the manufacturer (tables 3-6).
Table 3. Planitop HDM Restauro mortar. Physical and mechanical properties [14].

| Properties                              | Unit                          |
|-----------------------------------------|-------------------------------|
| Density of wet mix                      | 1900 kg/m³                   |
| Adhesion to masonry                     | 0.8 MPa                       |
| Compressive strength                    | 15 MPa, determined at 28 days |
| Tensile strength                        | 0.15 MPa, determined at 28 days |
| Compressive modulus of elasticity       | 8000 MPa, determined at 28 days |

Table 4. Mapegrid B250 basalt fibre mesh. Physical and mechanical properties [15].

| Properties                              | Unit                          |
|-----------------------------------------|-------------------------------|
| Weight                                  | 250 g/m²                      |
| Mesh size                               | 6 x 6 mm                      |
| Density of fibres                       | 2750 kg/m³                    |
| Tensile strength per unit width         | 60000 N/m                     |
| Modulus of elasticity                   | 89000 MPa                     |
| Elongation at failure                   | 1.8 %                         |

Table 5. MapeWrap B FIOCCO. Physical and mechanical properties [16].

| Properties                              | Unit                          |
|-----------------------------------------|-------------------------------|
| Density                                 | 2670 kg/m³                    |
| Tensile strength                        | 3.101 MPa                     |
| Modulus of elasticity                   | 87 MPa                        |
| Elongation at failure                   | 3.15 %                        |

Table 6. Mapei Epojet. Physical and mechanical properties [17].

| Properties                              | Unit                          |
|-----------------------------------------|-------------------------------|
| Density (mixed)                         | 1140 kg/m³                    |
| Compressive strength                    | 95 MPa                        |
| Tensile strength                        | 44 MPa                        |
| Tensile modulus of elasticity           | 3400 MPa                      |
| Elongation at failure                   | 1 %                           |

3. Results and discussions

The nonlinear micro-models were analysed using a method based on iterative series of linear approximations [18]. This method is also referred to as the Newton-Raphson Approach and it is structured on three distinct levels, as follows: residual, convergence and equilibrium. In the first one, the differences between the external and the internal forces are evaluated and the results are recorded as “residuals”. In the convergence stage, various iterations are performed until the “residues” become acceptable (less than the criterion). When the “convergence” criterion is achieved, the solution is considered to be in “equilibrium”, within an acceptable tolerance.
The Newton-Raphson Approach was applied by gradually increasing the load until the convergence criterion was satisfied. In addition, several sub-steps were defined to further divide the ultimate load step. The final step that corresponds to the ultimate load (at the end of the elastic branch) was established for the stage when a sudden and abrupt change in displacement occurred (the convergence criterion was no longer satisfied).

The shear structural performance of the strengthened stone masonry wall was evaluated and compared with the results obtained for the unstrengthen module. Figures 8 and 9 illustrate the shear stresses distributions for both numerical micro-models.

![Figure 8. Shear stress distribution map for the unstrengthen stone masonry wall](image1)

![Figure 9. Shear stress distribution map for the stone masonry wall strengthened with TRM](image2)

As it can be observed, the shear resistance capacity of the TRM strengthened wall increased significantly. The ultimate load increases from 21 kN to 84 kN with respect to the unstrengthen stone masonry wall. Additionally, the maximum shear stresses determined for the wall strengthened with TRM are 3.5 times higher compared to the ones determined for the unstrengthen wall.

The transversal composite connectors that were considered for this strengthening system provided enough fastening and bonding capacity to avoid the interface debonding between the stone masonry support and the TRM system. Also, the fan angle (360°) and the radius (60 mm) ensured sufficient area for the distribution of the stresses and, therefore, no significant stress concentrations were observed.

During the numerical analyses it was found that for the unstrengthen stone masonry wall, cracks may develop at early stages of loading in a diagonal shear band throughout the mortar joints (figure 10). Upon increasing the loading, a second crack pattern developed towards the lateral sides of the wall. The TRM strengthened wall exhibit distinct crack distribution patterns (figure 11). In this case, the cracks appeared at later stages of loading and they spread more uniformly over the area of the wall. No cracks developed in the regions where the composite transversal connectors were located. This phenomenon is explained by the additional reinforcement provided by the end parts of the basalt cords which were spread in a 360° configuration.

The numerical micro-models led to acceptable results regarding the stiffness characteristics in the post-leak stage. However, the unstrengthen stone masonry wall exhibited a slightly more pronounced reduction in stiffness when compared to the strengthened wall. In particular, at a drop in force of 15 % on the softening branch, the total displacement was approximately the same for both models, while at a drop of 30 %, the displacements stabilized at 16.83 mm for the strengthened wall and 5.70 mm for the unstrengthen one (figures 12, 13).
4. Conclusions

Stone masonry is a heterogeneous material that exhibits significant variations in directional properties due to the irregular mortar joints acting as planes of weakness. Therefore, the structural response of stone masonry structures is difficult to evaluate based on classical FE approaches. Moreover, numerical models should account for a broad spectrum of influencing parameters, among which the most important are: stone units’ geometry, arrangements of head and bed mortar joints, arrangements of stone units, material properties, existing degradations, quality of workmanship and irregularity of mortar layers.

In this paper various modelling aspects related to a finite element analysis strategy designed to simulate the diagonal compression test of irregular stone masonry walls strengthened with TRM were analysed and discussed. The structural concept, the geometry and the materials of the strengthening system were selected according to the specifications of the manufacturer. For each type of component of the stone masonry wall and of the TRM system, distinct finite element models were assessed. The boundary conditions and the parameters of position and connectivity were modelled to accurately...
represent the characteristics of the experimental panels. Moreover, image processing techniques were applied so as to model the geometry features as close as possible to the real conditions.

A plasticity theory based on micro-modelling approach was employed to carry out the analysis. The numerically determined results confirmed the efficiency of shear strengthening of stone masonry walls using TRM. Furthermore, a good correspondence was found between the numerical outcomes and the experimental results that are available in current literature. Nonetheless, in order to draw reliable and broad conclusions, these results will be validated through the experiment already mentioned in the paper.

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