A NOVEL DISCONTINUUM FINITE ELEMENT MODELLING APPROACH FOR THE STRUCTURAL EVALUATION OF MASONRY STRUCTURES

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ABSTRACT

This paper defines a novel modelling approach for the evaluation of the structural behavior of new and existing masonry elements subjected to lateral and vertical loads. This approach has the objective to provide a computational tool that allows to model the non-linear behavior of masonry structures with a reduced numerical cumbersome, without jeopardizing the accuracy of the obtained results.

The proposed model is a typical D-FEM (Discontinuum Finite Element Model) characterized by the fact that, differently from the most common modelling methodologies, deformable blocks, covering more than a unitary portion of masonry, are separated by interface elements that are properly arranged along pre-established potential cracks surfaces. To this aim, the “Combined Cracking-Shearing-Crushing” model, proposed by Lourenço for FEM analyses of simplified micro-models, is used in order to simulate fracture, frictional slip and crushing along the interfaces, but the original formulations are properly modified in order to account for the assumptions related to the definition of the failure surfaces.

In a first stage some experimental tests provided by literature, used for calibration purposes, are described. These concerns five brick walls characterized by different geometric features, masonry layouts and failure modes. Then, the proposed model is introduced for the above panels: the coefficients that have to be selected for the “Combined Cracking-Shearing-Crushing” formulations are properly modified by a trial and error procedure and analyses simulating the above experimental tests are run and re-run until the achievement of a satisfying fitting between experimental and numerical results.

The obtained outcomes allow to envisage that suitable close-form equations can be picked out in the next future by applying proper fitting techniques, which have to be set up on the basis of a parametric analysis that the authors are already carrying out.

Keywords: Seismic Vulnerability, Masonry Structures, Discontinuum Finite Element Model, Combined Cracking-Shearing-Crushing model.

1. INTRODUCTION

One of the most challenging issues that concerns the non-linear analyses of masonry structures is the model that has to be used in order to simulate the real behavior of elements. Several approaches were proposed in the past and two classes of methods are nowadays well recognizable in literature: Macro and Micro Modelling based methods.

Macro Models are based on homogenization techniques (Massart et al. 2007) that lead to consider masonry as an unicum and to consider mortar joints effects on masonry elements implicitly (Asteris et al. 2016). In this model, the non-linear behavior is represented by the features of an equivalent material that responds to specific inelastic behaviors and failure criteria determined on the basis of properly calibrated procedures. The Macro Modelling approach is usually preferred when large scale masonry structures are analyzed, but fails when stress and strain analysis of masonry have to be picked out at a

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local level. Moreover, Macro Model based methods generally do not allow to capture all the potential failure modes of masonry elements, due to the fact that the main weaknesses, usually concentrated in the mortar joints, are not explicitly considered.

A diametrical modelling strategy, introduced for the first time by Page (1979), is the Micro Modelling approach, which accounts for all the parts forming the masonry walls, namely units (bricks, blocks, etc), mortar bed/perpend joints and unit-to-mortar interfaces. Two ways can be generally followed, those are the so called Detailed and Simplified Micro-Modelling. The first -more refined, but also more computationally demanding- is based on a three phase material in which units and mortar joints are modeled by means continuum elements whereas the interface is given by zero-thickness discontinuum elements (Lourenço et al. 1997). The latter, that is of paramount interest for the study reported in this paper and that is also known as D-FEM (Discontinuum Finite Element Model), considers masonry as a two-phase material in which units are modelled as expanded bricks having the original sizes plus the real joint thickness, whereas the mortar and its features are included in the zero-thickness mortar joints (Asteris et al. 2003, Sutcliffe et al. 2001). Obviously, the main goal in implementing micro modelling techniques is to accurately account for the contribution that each component of the masonry is able to confer to the performance of a masonry element under different types of loading conditions. Nevertheless, although the simplification carried out, the accuracy of this type of approach is often frustrated by the numerical cumbersome of the related analyses, in particular when these are carried out on articulated structures in the non-linear field.

A compromise solution between the approaches described above is given by Discrete Element Models (DEM) which consider the masonry elements as the result of the assemblage of blocks. These can be rigid or deformable and interact each others on specific contact points ruled by specific law able to simulate fracture, frictional slip and crushing. Initially launched for geotechnical applications (Cundall 1988, Cundall 1989), several studies carried out in the recent past proved the effectiveness of this approach in representing the real behavior of masonry elements even when large displacements are attained and collapse conditions are closed (Giordano et al. 2002), due to the fact that the contact law existing between blocks are updated for each step of analysis. A particular formulation of the DEM approach is given by the Distinct Element approach, based on the finite difference principles, in which blocks, characterized by an arbitrary geometry, are internally meshed through finite elements and the interaction between them is concentrated in a set of point contacts with no attempt to obtain a continuous stress distribution through the contact surface (Asteris et al. 2016).

In the research field described above, the current paper introduces a new modelling approach for unreinforced masonry elements subjected to axial and shear loading. It considers the tools and the modelling techniques that characterize the D-FEM methodology, but, taking inspiration from the DEM approach, the masonry element is considered as a set of deformable linear blocks that are separated by interfaces arranged along the panel surfaces where sliding, crushing and separation phenomena are most likely to occur. The interface elements are characterized by the “Combined Cracking-Shearing-Crushing” model proposed by Lourenço (1996), which, however, is properly modified in order to account for the simplifications done with concern to the failure position. To find the modification factors to be used, a calibration procedure is carried out on the basis of some experimental tests retrieved by literature.

Because the reduced non linearity sources, with respect to the most common D-FEM approaches, the here proposed model allows a minor computational cost without, nevertheless, hiding the possible failure mechanisms.

The paper is organized as follows: (i) presentation of the experimental specimens considered by literature for calibration purpose; (ii) description of the D-FEM approach proposed in this paper; (iii) comparison between experimental and numerical results downstream a proper calibration process.

2. THE CONSIDERED EXPERIMENTAL TESTS

2.1. Basis
The proposed modelling approach presents several simplifications with respect to the more common and more precise D-FE models formulated on the basis of the “Combined Cracking-Shearing-Crushing” for the joint interface. For this reason, some modifications factors have to be properly calibrated. These should be able to provide a final formulation able to give back the same failure modes and strength observed by the experimental tests.

2.2. The experimental tests

Five experimental tests developed in the past by other researchers have been considered. The first four come from the experimental campaign carried out by Raijmakers and Vermeltfoort (1992), Vermeltfoort and Raijmakers (1993). Single leaf masonry panels characterized by a width of 990 mm and a height of 1000 mm, made of a masonry layout with eighteen layers of clay bricks (bricks sizes: 210x52x100 mm³) separated by 10 mm thick mortar joints, were subjected to three different vertical loads (30 kN for two panels, 120 kN and 210 kN for the others) and pushed laterally, with a displacement control procedure, by lateral forces until the achievement of collapse. These loads were transmitted on the top of the walls through a rigid steel beam. The mechanical features of clay blocks are: Elastic modulus \( E = 16700 \text{ N/mm}^2 \), Poisson ratio \( \nu = 0.15 \), Density \( \rho = 1900 \text{ kg/m}^3 \), Compression strength \( \sigma_c = 12 \text{ MPa} \), Tensile strength \( \sigma_t = 2 \text{ MPa} \), Tension and Compression Fracture energy of \( G_{Ic} = 0.08 \text{ N/mm} \) and \( G_{IIc} = 6 \text{ N/mm} \), respectively.

Joints are made of a 1:2:9 cement: lime:sand mortar, characterized by a Compression strength \( \sigma_c = 11 \text{ MPa} \), Tensile strength \( \sigma_t = 0.2 \text{ MPa} \), Tension, Shear and Compression Fracture energy of \( G_{Ic} = 0.016 \text{ N/mm} \), \( G_{IIc} = 0.125 \text{ N/mm} \) and \( G_{IIlc} = 6 \text{ N/mm} \), respectively. Moreover, a cohesion of \( c = 0.28 \text{ N/mm}^2 \) and a friction angle of \( \alpha = 36.87^\circ \) has been revealed.

In Figure 1 the failure mechanisms of the four tested panels are depicted. Those were mainly due to diagonal shear cracks, which were evidently expected due to the aspect ratio (almost 1:1).

![Figure 1](image1.png)

Figure 1. Failure modes of the four masonry panels tested by Vermeltfoort and Rajmakers (1993)

The last masonry panel considered in this paper for calibration purpose was tested by Magenes and Calvi (1997). It is characterized by a width of 1000 mm and a height of 2000 mm, with twenty-seven layers of bricks. For the units, 242,5x60x120 mm³ bricks were assumed, with the following mechanical features: \( E = 16700 \text{ N/mm}^2 \), \( \nu = 0.15 \), \( \rho = 1750 \text{ kg/m}^3 \), \( \sigma_c = 6.2 \text{ MPa} \), \( \sigma_t = 1.1 \text{ MPa} \), \( G_{Ic} = 0.09 \text{ N/mm} \), \( G_{IIc} = 0.125 \text{ N/mm} \) and \( G_{IIlc} = 30 \text{ N/mm} \). These units were separated by 10 mm thick mortar layers with, \( \sigma_c = 10 \text{ N/mm}^2 \), \( \sigma_t = 0.04 \text{ MPa} \), \( G_{Ic} = 0.15 \text{ N/mm} \), \( G_{IIc} = 0.085 \text{ N/mm} \), \( G_{IIlc} = 50 \text{ N/mm} \), \( c = 0.056 \text{ MPa} \) and \( \alpha = 24.3^\circ \).

On the top, the wall were subjected to a vertical load of 60 kN and pushed laterally, according to a displacement control procedure, up to the collapse.

In Figure 2 the tested panel is depicted.

The slender wall tested by Magenes and Calvi collapsed due to axial force and bending, presenting at the base section horizontal cracks on the tensile side and vertical cracks on the compression one.
3. THE PROPOSED MODELLING APPROACH

3.1. General

The basic idea of the proposed modelling approach is that a wall can be divided in a very limited number of modules characterized by the same mechanical features. If these modules are able to reproduce the potential cracks under different loading conditions, then all the modules together will be able to simulate the non-linear behaviour of the whole masonry element formed by them. On the other hand, this is the idea that implicitly characterizes all those methods that, in the last decades, were proposed for masonry with periodical layouts, but, for those, the considered modules were so small (usually two units plus a mortar joint) that applications to articulated structures were seriously jeopardized by numerical issues.

3.2. The module

In order to overcome issues describe above, the here proposed methodology is based on an elementary module that represents a wider portion of masonry, it has been assessed that a square of 250 mm side length is sufficient, but a sensitivity study must be carried out to confirm this statement. This module is treated through a D-FEM approach, according to Figure 3a. In detail four elastic prismatic blocks, two with trapezoidal and two with pentagonal base, are detected and characterized by a linear behaviour having the same elastic modulus and Poisson ratio of the units. The four elastic blocks are separated by zero-thickness interface elements characterized by the “Combined Cracking-Shearing-Crushing” introduced by Lourenco, which will be better described in the next Section. The assemblage of the modules described above allows to obtain the final model of a given wall, as shown in Figure 3b for the slender panel, quoted previously, tested by Magenes and Calvi. It is to be underlined that in the assemblage process the interface elements described above are also spread along the free boundaries of each elastic unit, so to model the interaction between two adjacent modules, but an internal rigid constraint along the vertical surfaces is imposed in order to prevent possible slips in that direction.
It is clearly understandable that the proposed modelling approach does not allow to detect the precise cracks that can arise on a masonry panel, due to the fact that possible slips, crushing or tears are rigidly constrained to the layout of the interface elements. However, all the possible failure mechanisms of a masonry panel under different loading conditions can be represented, as shown schematically in Table 1, with possible cracks patterns that, moreover, can be very closed to the one that are usually observable in reality.

For this reason, one can envisaged that if a reliable model is adopted for the interface elements, the modification that have to be implemented to the fundamental parameters, in order to account for the applied simplifications, should not be excessive.

3.3. The Combined Cracking-Shearing-Crushing model (Lourenco 1996)

The proposed D-FE model is based on the hypothesis that the inelastic features of mortar joints can be spread along a zero-thickness unit-to-unit interface ruled by the “Combined Cracking-Shearing-Crushing” multi-surface yield model, which is basically determined by the three different criteria listed below:

- “tension cut-off criterion”, which rules the tensile behavior along the interfaces through two parameters, namely the tensile strength of mortar joints ($\sigma_t$) and the tensile fracture energy ($G_{It}$) related to the post peak response of the mortar.

- “coulomb friction criterion”, which defines the shear frictional behavior along the interface as a function of the cohesion $c$ and the angle of internal friction $\phi$, the last being defined by an initial ($\phi_0$) and the residual ($\phi_r$) angles of internal friction, which rules the initial and the residual shear strengths, respectively. Moreover, the post peak response is determined by the shear energy of fracture $G_{IIc}$.

- “compressive cap criterion”, which defines the crushing behavior along the interface as a function of the parameter $C_s$, a parameter controlling the shear stress contribution to failure, and the maximum compression strength $\sigma_c$ that is assumed to evolve according a proper strain hardening hypothesis.

Moreover, two elastic normal and tangential stiffness, $k_n$ e $k_t$, are defined in order to establish the linear behaviour of the interface.

4. VALIDATION OF THE PROPOSED MODEL

4.1. Implementation of the proposed model in a software environment

For each of the tested panels described in Section 2, the here proposed D-FE model has been
Table 1. Collapse Mechanisms for Masonry

<table>
<thead>
<tr>
<th>Masonry Type</th>
<th>Collapse Type</th>
<th>Mechanism of Global Collapse</th>
<th>Global Collapse for D-FEM Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak Connections</td>
<td>Sliding Joints</td>
<td>![Sliding Joints Diagram]</td>
<td>![Global Collapse Diagram]</td>
</tr>
<tr>
<td>Weak Bricks</td>
<td>Axial-Flexure</td>
<td>![Axial-Flexure Diagram]</td>
<td>![Global Collapse Diagram]</td>
</tr>
<tr>
<td>Weak Bricks</td>
<td>Diagonal Units Cracking</td>
<td>![Diagonal Units Cracking Diagram]</td>
<td>![Global Collapse Diagram]</td>
</tr>
<tr>
<td>Mixed</td>
<td>Diagonal Units Cracking and Sliding Joints</td>
<td>![Diagonal Units Cracking and Sliding Joints Diagram]</td>
<td>![Global Collapse Diagram]</td>
</tr>
<tr>
<td>Mixed</td>
<td>Compression</td>
<td>![Compression Diagram]</td>
<td>![Global Collapse Diagram]</td>
</tr>
</tbody>
</table>

implemented in the MIDAS-FEA non linear software (CSPFEA 2016), which allows to consider the Cracking-Shearing-Crushing model along interface elements.
For the elastic units, triangular six node shell elements, with three Gauss integration points, have been assumed, whereas two-nodes/zero thickness elements with a general formulation have been adopted for the interface elements.
The panels have been fixed to the ground, whereas the vertical displacements of the top beams have been fully restrained.
In Figure 4 the geometrical model of the considered panels is shown.
A two steps nonlinear static analysis has been carried out. In the first step, vertical loads have been statically imposed. Then (second step), an increasing controlled lateral displacement has been applied to the top beam.
The Newton-Raphson method has been used for convergence purpose, with an energetic control criterion for the first step and a displacement control criterion for the latter.

4.2. Fundamental parameter estimation by trial and error procedure

In order to reproduce the experimental tests, the fundamental parameter of the “Combined Cracking-Shearing-Crushing” formulations were properly selected by a trial and error procedure consisting in running and re-running the analyses with new value until the achievement of a satisfying fitting between experimental and numerical results. The initial values of the above parameters are those ones selected according to Loureno indications (Loureno 1996), which proved to be pretty apt to simulate the behavior of masonry panels in terms of detailed micro-modelling approach. These are reported in Table 2, where the analogous values obtained downstream the trial and error procedure are also given.
4.3. Experimental vs. Numerical Results

The comparison between the tests results of Vermeltfoort and Raijmakers and the numerical outcomes obtained by assuming the values given in Table 2 is shown in Figure 5a. As it is possible to observe, the experimental behavior is reproduced in a very satisfying way and the panels shear strength is assessed with good approximation, also in the post peak phase. The sudden loss of resistance after the peak response is due to localized failures in the mortar joints, as it is shown in Figure 5b, where the panels states for two different inelastic lateral displacements are shown. Also, in Figure 6, the same comparison is shown for the panel tested by Magenes and Calvi. Also in this case, the proposed D-FEM model, calibrated according to the parameters of Table 2 works in a very satisfying manner.

Table 2. Combined Cracking-Shearing-Crushing parameters according to Lourenco indications (Detailed Micro Model) and according to the here proposed model based on a trial and error procedure (Assumed D-FE Model)

<table>
<thead>
<tr>
<th>Test</th>
<th>Model</th>
<th>$k_n$</th>
<th>$k_i$</th>
<th>C</th>
<th>$\phi$</th>
<th>$\sigma$</th>
<th>$G_{I}'$</th>
<th>$G_{II}'$</th>
<th>$\alpha_c$</th>
<th>$C_s$</th>
<th>$G_{III}'$</th>
<th>$k_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vermeltfoort and Raijmakers (p=30 kN)</td>
<td>Lourenco Model</td>
<td>82</td>
<td>36</td>
<td>0.28</td>
<td>36.87</td>
<td>0.2</td>
<td>0.016</td>
<td>0.125</td>
<td>11</td>
<td>9</td>
<td>6</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Assumed D-FE Model</td>
<td>41</td>
<td>18</td>
<td>0.364</td>
<td>30.8</td>
<td>0.26</td>
<td>0.6</td>
<td>0.125</td>
<td>11</td>
<td>9</td>
<td>120</td>
<td>0.09</td>
</tr>
<tr>
<td>Vermeltfoort and Raijmakers (p=120 kN)</td>
<td>Lourenco Model</td>
<td>110</td>
<td>50</td>
<td>0.224</td>
<td>36.87</td>
<td>0.16</td>
<td>0.012</td>
<td>0.05</td>
<td>11.5</td>
<td>9</td>
<td>6</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Assumed D-FE Model</td>
<td>61.1</td>
<td>27.8</td>
<td>0.434</td>
<td>25</td>
<td>0.31</td>
<td>0.6</td>
<td>0.125</td>
<td>11.5</td>
<td>9</td>
<td>120</td>
<td>0.09</td>
</tr>
<tr>
<td>Vermeltfoort and Raijmakers (p=210 kN)</td>
<td>Lourenco Model</td>
<td>82</td>
<td>36</td>
<td>0.224</td>
<td>36.87</td>
<td>0.16</td>
<td>0.012</td>
<td>0.05</td>
<td>11.5</td>
<td>9</td>
<td>6</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Assumed D-FE Model</td>
<td>41</td>
<td>18</td>
<td>0.434</td>
<td>25</td>
<td>0.31</td>
<td>0.6</td>
<td>0.125</td>
<td>11.5</td>
<td>9</td>
<td>120</td>
<td>0.09</td>
</tr>
<tr>
<td>Magenes and Calvi (p=60 kN)</td>
<td>Lourenco Model</td>
<td>30.13</td>
<td>13.1</td>
<td>0.056</td>
<td>24.3</td>
<td>0.04</td>
<td>0.15</td>
<td>0.085</td>
<td>10</td>
<td>9</td>
<td>50</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Assumed D-FE Model</td>
<td>16.74</td>
<td>7.28</td>
<td>0.14</td>
<td>15</td>
<td>0.1</td>
<td>0.6</td>
<td>0.125</td>
<td>10</td>
<td>9</td>
<td>120</td>
<td>0.09</td>
</tr>
</tbody>
</table>
CONCLUSION

This paper has introduced a novel modelling approach for masonry panels under axial forces and shear.
In particular, a new type of Discontinuum Finite Element Model (D-FEM), with elastic blocks separated by pre-established potential cracks surfaces characterized by the “Combined Cracking-Shearing-Crushing” model proposed by Lourenco, has been presented.
The model has been used in order to simulate five full scale tests on masonry panels characterized by different geometries, axial forces and types of failure. The comparison between numerical and experimental results has proved the reliability of the proposed modelling approach once that the fundamental parameters of the Lourenco model are properly changed.
The main conclusions of the proposed study are listed in the following:

- The proposed modelling approach seems to be robust enough to be used for masonry elements, as it is able to capture the main mechanical features –stiffness, strength and ductility- and the failure modes observed by tests;
The Model is convenient for numerical non linear analysis of complex buildings, as it is not heavy from the computational point of view. From this point of view, it must to be underline that some preliminary study that we carried out following alternative approaches put in evidence that our model is able to cut the time of analyses of 90% with respect to the more common D-FEM currently proposed in literature.

The obtained outcomes allow to envisage that suitable close-form equations for the modified parameters of the Lourenco model can be picked out in the next future by applying proper fitting techniques, which have to be set up on the basis of a parametric analysis that the authors are already carrying out.

6. ACKNOWLEDGEMENT

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7. REFERENCES


