MICRO VS. MACRO MODELS FOR PREDICTING BUILDING DAMAGE UNDERGROUND MOVEMENTS

Linh Truong Hong* and Debra F. Laefer **

Abstract:
For over 30 years various micro and macro models have been used for analysing masonry, but no strong consensus within the structural engineering community exists as to usage. Selection remains driven by field scenarios, cost restrictions, and level of result detail needed. This paper contributes to this discussion by comparing micro and macro models of buildings subjected to excavation-induced ground movements. A smeared crack model is used to represent cracking in bricks and mortar joints, and the brick structure is modelled as an isotropic continuum. In the macro model, a homogeneous procedure employed is an alternative approach for determining mechanical properties of a basic cell. Results are compared to large-scale modelling work. Respective advantages and disadvantages are shown.

Key words: micro modeling; macro modeling; homogenization technique; smeared crack model; large-scale model test; building damage

Introduction
Prior to World War II, the predominant urban building material in Europe and North America was brick. These structures reflect the fabric and architectural heritage of many of those cities. In a combination of heightened sensibilities about both sustainability and cultural preservation, protection from subsurface construction works is gaining importance. Unfortunately, their low tensile and shear capacities make them especially vulnerable to settlement. Accurate prediction under such circumstances is vital to damage minimization.

Since the early 1980’s, significant advances have been made in the analyzing of unreinforced masonry (Page 1978; Drysdale 1982; Lofti and Shing 1991; Pietruszczak and Niu 1992; Lourenço 1996a; Anthoine 1997; Anthoine 1997; de Buhan and de Felice 1997; Pluim 1997; Zucchini and Lourenco 2002; Vermeltfoort 2005; Wu and Hao 2005; Zucchini and Lourenco 2007). In parallel, a large number of experiments have been conducted to obtain material characteristics of masonry structures, as well as tests of components subjected to different loading conditions (Drysdale 1982; Binda, Fontana et al. 1988; Vermeltfoort 2005). These contributions provided critical insights into failure mechanism developments and failure interface shapes for various stress states, such as compression-compression and compression-tension, as well as tension-tension. With development of numerical methods and computation power, various strategies and failure criteria have been developed to model realistic behavior. However, describing the complex behavior of the bricks/mortar joint interface in a global context remains a challenge. During adjacent excavation, nearby buildings often settle. Empirically-based, damage criteria has been widely adopted (Burland and Wroth 1974; Boscarding and Cording 1989; Burd 1995; Boone 1996). However, these
mainly focus on the generation of a trough profile and selection of single mechanical parameters (e.g. Young’s modulus and shear modulus) to describe the structure’s overall performance. There is also an assumption that the building can be represented as a simple deep beam, with associated simplification of mode shapes. Arguably several important topics are not considered such as soil-structure interaction, localized stiffness variations, and the load transfer mechanisms provided by diaphragms and roofs. This more inclusive input set is likely to produce improved results.

This paper investigated adaptive modeling strategies, through micro modeling and macro modeling to provide parameters for damage prediction for buildings subjected to excavation-induced movements. Damage levels were assessed based on criteria in existing literature and current codes. A smeared crack model was applied, and the onset of cracking of was predicted by a failure criterion expressed by the William-Warnke failure criterion. Furthermore, the Drucker-Prager yield criterion with an associated flow rule was used to model non-linear properties of masonry components. Through this study, differences between the micro and macro model were established. Laboratory experiments provided displacement data against which to benchmark the performance of the various models within ANSYS®.

**Background**

Masonry is an anisotropic composite material, so modeling depends upon unit and joint size, and modeling can be performed with different detail levels (Lourenço 1996a; J. Lopez 1999) as shown in fig. 1. In detailed micro modeling, bricks and mortar are represented by continuous elements and are modeled separately, whereas the interface behavior between brick and mortar is shown by discontinuous elements. In this model, all component characteristics can be considered, thus more fully reflecting real component behavior.

In simplified micro modeling, expanded units are represented by continuous elements and mortar joints and the interface by discontinuous elements. With this model, mortar joints are ignored and replaced by interface elements, whose characteristics are based on interface behavior. In macro modeling, bricks, mortar joints, and interface are globally represented by a single element. Mechanical properties of homogenous elements must represent the overall structure. Data is obtained experimentally or through a homogenization technique (Rivieccio 2005; Wu and Hao 2005; Zucchini and Lourenco 2007).

![Masonry modeling strategies](image)
Large scale model test

A high vulnerability, prototype building subjected to excavation-induced ground movements was chosen for large-scale, model testing (Laefer 2001). A two-story unreinforced masonry building to represent a small to medium, residential or commercial structure from the early 20th century was constructed at 1/10th scale (largest scale that could be accommodated in the testing chamber). The model was made of individually laid bricks in a lime putty mortar (Table 1). The structure had a single wythe and six windows per story. The excavation system was a sheetpile wall with tie-backs (fig. 2). Kinematic, as and geometric, similarity was upheld (Table 1 and fig. 2); see building details on fig. 3.

Table 1. Summary of dimensional characteristics of prototype and scale models

<table>
<thead>
<tr>
<th>Aspects</th>
<th>Prototype</th>
<th>Scale model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation depth (m)</td>
<td>12.192</td>
<td>1.219</td>
</tr>
<tr>
<td>Lot width (m)</td>
<td>7.620</td>
<td>0.762</td>
</tr>
<tr>
<td>Lot depth (m)</td>
<td>24.384</td>
<td>2.438</td>
</tr>
<tr>
<td>Building width (m)</td>
<td>6.096</td>
<td>0.610</td>
</tr>
<tr>
<td>Building depth (m)</td>
<td>18.300</td>
<td>1.830</td>
</tr>
</tbody>
</table>

*Note: numbers in bracket were used for Pi5W

Figure 2. Schematic of testing arrangement

The smallest commercially available extruded bricks (1/4th scale) were chosen to replicate the prototype bricks. The units were shipped unfired from Belden Brick Co. to the University of Illinois at Urbana-Champaign, where they were dried and fired to generate strengths only 1/10th of compressive strength of prototype bricks to comply with scaling requirements. Similarly, the mortar was designed to replicate a type N mortar (1:1:6 cement:lime:sand) mixture. The lintels 1.58mm thick were made from oak. The mechanical properties of components of the large-scale brick structure model adopted are in Table 2.
Figure 3. Details of a masonry structure for physical model tests

Table 2. Mechanical properties of the masonry components

<table>
<thead>
<tr>
<th>Aspects</th>
<th>Brick</th>
<th>Mortar</th>
<th>Lintels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensions (mm)</td>
<td>57.86x15.24x29.75</td>
<td>3.21, 2.25</td>
<td>1.58</td>
</tr>
<tr>
<td>Mass density (Kg/m³)</td>
<td>1.7839</td>
<td>1.5579</td>
<td>674</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>19.66</td>
<td>1.310</td>
<td>N/A</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>5.00</td>
<td>0.152</td>
<td>N/A</td>
</tr>
<tr>
<td>Elastic modulus (MPa)</td>
<td>81.28</td>
<td>27.58</td>
<td>11,700</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
<td>0.30</td>
<td>0.25</td>
</tr>
</tbody>
</table>

* Note: (a) head joint thickness; (b) bed joint thickness

The scale-model tests Pit4 and Pit5 were analyzed. Each test included two identical, unreinforced buildings named West and East building subjected to different loads. Data was used from buildings Pit4E and Pit5W, which respectively had shallow footings under light loads and deep footings subjected to heavy loads (Table 3). Displacements were recorded at multiple stages. Three points were of special interest: i) just before tensioning of the first tied-back anchors – corresponding to a depth of 0.305m (representing 25% of the excavation's final depth) in Test 4 and to a depth of 0.178m (representing 14.6% of the excavation’s final depth) in Test 5; ii) immediately before tensioning of the third tied-back anchors at 0.914m (75%) in the Test 4 and 0.787m (64.6%) in the Test 5, respectively, and iii) at design grade corresponding to 1.22m (100%) in Test 4 and 1.09m (89.4%) in test 5.

Table 3. Summarized vertical loads on the scale model test

<table>
<thead>
<tr>
<th>Aspects</th>
<th>Pit4E</th>
<th>Pit5W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Self-weight building (kg)</td>
<td>45.36</td>
<td>45.36</td>
</tr>
<tr>
<td>Top building (N)</td>
<td>889.64</td>
<td>1779.29</td>
</tr>
<tr>
<td>Top windows (N)</td>
<td>222.41</td>
<td>222.41</td>
</tr>
<tr>
<td>Bottom window (N)</td>
<td>222.41</td>
<td>222.41</td>
</tr>
<tr>
<td>Footings (N)</td>
<td>222.41</td>
<td>222.41</td>
</tr>
<tr>
<td>Total external loads (N)</td>
<td>1668.7</td>
<td>2579.95</td>
</tr>
</tbody>
</table>

Numerical analyses

Non-linear analysis of the brick buildings was conducted using ANSYS®, V11.0. The three-dimensional (3D) element, Solid65 was used to model the brick structure, which was de-
fined by 8 nodes each with 3 degrees of freedom, isotropic behavior, and 2x2x2 integration points. The element can cracking in tension and crushing in compression. The smeared crack model was used to model crack patterns of continuous elements. Cracking and crushing in solid65 was evaluated with five parameters as part of the William-Warnke failure criterion, which was initially developed for concrete but has been applied to a wide range of brittle materials (William and Warnke 1975). The failure criterion was defined by means of ultimate tensile strength ($f_t$) and ultimate compressive strength ($f_c$), in which the stress space of the failure surface must be considered as $0 \leq \theta \leq 60^\circ$ (William and Warnke 1975) and three additional parameters: ultimate biaxial compressive strength ($f_{cb}$), ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state ($f_1$), and ultimate compressive strength for a state of uniaxial compression superimposed on hydrostatic stress state ($f_2$). These were defined by default values in the program: $f_{cb} = 1.2f_c$, $f_1 = 1.45f_c$ and $f_2 = 1.725f_c$ (ANSYS). A crack at the integration points of each element was represented through the modification of the stress-strain relationship. The crack occurs at the principal weakness plane, when the principal stress is lower than the tensile strength of each material component. For consideration of subsequent loads that induce sliding across a crack plane, the shear transfer coefficients $\beta$ are introduced: $\beta_t$ for open cracks and $\beta_c$ for re-closed cracks. Values of 0.2 and 0.8 for $\beta_t$ and $\beta_c$, respectively were chosen and the failure criterion for multi-axial stress state equation 1 (ANSYS)

$$\frac{F}{f_c} - S \geq 0$$  \hspace{1cm} (1)$$

where $F$ is a function of the principal stress state, and $S$ is a failure surface expressed in terms of principal stress and five input parameters $f_t$, $f_c$, $f_{cb}$, $f_1$ and $f_2$ uniaxial crushing strength. If equation 1 is satisfied, the material will crush or crack.

The Drucker-Prager yield criterion (Equation 2) with its associated flow rule and no hardening rule (referring to an elastic-perfectly plastic behavior) was implemented for performing plastic behavior.

$$f = \alpha I_1 + \sqrt{J_2} - k$$  \hspace{1cm} (2)$$

where $I_1$ and $J_2$ are the first invariant stress and the second deviatoric stress invariant, and $\alpha$ and $k$ are defined as equations 3 and 4 respectively

$$\alpha = \frac{2\sin \phi_f}{\sqrt{3(3 - \sin \phi_f)}}$$  \hspace{1cm} (3)$$

$$k = \frac{6\cos \phi_f}{\sqrt{3(3 - \sin \phi_f)}}$$  \hspace{1cm} (4)$$

where $\phi_f$ and $\phi_c$ are, respectively, the friction angle and internal cohesion.

Additionally, to describe the Drucker-Prager yield criterion in ANSYS\textsuperscript{®}, the friction and dilatancy angles. Values of 35‘ and 5° for bricks, mortar joints as well as homogenous elements were used. According to Zucchini and Lourenco (2007), the internal cohesion can be determined through the uniaxial experimental yield stress in compression, $\sigma_c$. The Drucker-Prager yield criterion (Equation 2) with its associated flow rule and no hardening rule (referring to an elastic-perfectly plastic behavior) was implemented for performing plastic behavior.

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$$k = \frac{6\cos \phi_f}{\sqrt{3(3 - \sin \phi_f)}}$$  \hspace{1cm} (4)$$

where $\phi_f$ and $\phi_c$ are, respectively, the friction angle and internal cohesion.
Furthermore, as failure through other elements was not expected, solid45 was used to model lintels and footings, and both target170 and conta173 were employed as interface elements between the lintels and structure, and between the structure and footings. Contact between lintel and mortar joints was assumed to allow the lintels to slide freely, whereas a Coulomb friction model was used at the wall/footing interface, with values of the friction coefficient and internal cohesion respectively 35° and 20.7kPa. A non-linear solution was undertaken via a full Newton-Raphson interactive solution algorithm, with displacement convergence. A converged solution was obtained once displacement changes (checked as the square root sum of the squares) was less than or equal to 0.05 (ANSYS).

Description of detailed micro modeling

Initially, the detailed micro modeling strategy was used. The mechanical properties (Table 2) were calibrated to the component laboratory tests. The micro numerical model included 19816 nodes and 10162 elements (fig. 4a). The structure was subjected to self-weight, external vertical loads, and excavation-induced displacements. Boundary conditions were assumed fixed at the bottom of the footings. Applied loads and self-weight for the numerical model were identical to the physical model test by means of load steps. Self-weight was defined based on gravitational acceleration and density and volume of each material. Displacements were constrained at load application points, and displacement values at the nodes were determined by linear interpolation from discrete test readings.

![a) FE mesh of micro model](image)a) FE mesh of micro model  ![b) FE mesh of macro model](image)b) FE mesh of macro model

Figure 4. Finite element model in ANSYS® simulated the large scale-model test – PiT5W

Description of macro modeling model

The macro modeling process was the same as that for the micro model. Homogenized properties were determined through a Representative Volume Element (RVE), which is defined by average stress and strain of these components from numerical analysis, for which experimental mechanical properties are used as the initial inputs. To obtain overall mechanical properties of the basic cell, each loading condition was applied to the basic cell associated with the x, y, and z directions. These averaged values were defined as

\[
\sigma_{ij} = \frac{1}{V} \int_{V} \sigma_{ij} dV
\]  

(6)
\[ \overline{e}_{ij} = \frac{1}{V} \int_{V} e_{ij} dV \]  

(7)

where \( V \) is the volume of the basic cell, and \( \sigma_{ij} \) and \( e_{ij} \) are the element’s stress and strain.

The basic cell includes bricks and mortar joints (bed and head) (fig. 5a and Table 2). As the 3D basic cell’s geometry was symmetric, analysis was performed on 1/8th of it. The brick-mortar interface was assumed to be perfectly bonded. The mesh (see fig. 5b) had 6699 nodes and 5600 elements. The solid65 element was employed for modeling both bricks and mortar joints. For performing non-linear behavior of the basic cell, the Drucker-Prager yield criterion was used to simulate plasticity behavior in compressive, and the Rankine model for determining tensile cracking (Zucchini and Lourenco 2002).

Through simulation of the stress-strain relationship of the basic cell, the equivalent elastic modulus and Poisson’s ratio can be derived from uniaxial tensile and compressive loading model via previously pioneered definitions (e.g. Rivieccio 2005; Wu and Hao 2005):

\[
\overline{E}_{xx} = \frac{\overline{\sigma}_{xx}}{\overline{e}_{xx}}; \quad \overline{v}_{xy} = \frac{\overline{e}_{xy}}{\overline{e}_{xx}}; \quad \overline{v}_{xz} = \frac{\overline{e}_{xz}}{\overline{e}_{xx}}
\]

\[
\overline{E}_{yy} = \frac{\overline{\sigma}_{yy}}{\overline{e}_{yy}}; \quad \overline{v}_{yx} = \frac{\overline{e}_{yx}}{\overline{e}_{yy}}; \quad \overline{v}_{yz} = \frac{\overline{e}_{yz}}{\overline{e}_{yy}}
\]

\[
\overline{E}_{zz} = \frac{\overline{\sigma}_{zz}}{\overline{e}_{zz}}; \quad \overline{v}_{zx} = \frac{\overline{e}_{zx}}{\overline{e}_{zz}}; \quad \overline{v}_{zy} = \frac{\overline{e}_{zy}}{\overline{e}_{zz}}
\]

(8)

(9)

(10)

where \( \overline{\sigma}_{xx}, \overline{\sigma}_{yy}, \) and \( \overline{\sigma}_{zz} \) are averaged tensile or compressive stress along X, Y, Z direction under uniaxial tensile or compressive load, and \( \overline{e}_{xx}, \overline{e}_{yy} \) and are, respectively, averaged strain of the basic cell along X, Y, Z direction similarly subjected. The equivalent elastic mechanical properties of the basic cell are compared to the analytical solutions (Table 4). Only small differences appear. Thus the elastic modulus \( E_{yy} \) (59.41MPa), compressive strength \( f_{cy} \) (4.28MPa), tensile strength \( f_{tx} \) (0.131MPa), as well as Poisson’s ratio are adopted for macro elements, which behaved as isotropic. The macro model (Fig. 4b) for Pit5W consisted 6018 nodes and 3133 elements.
Table 4. Comparison between analytical solution and the FEM results

<table>
<thead>
<tr>
<th>Aspects</th>
<th>Micro mechanical model (Zucchini and Lourenco 2002)</th>
<th>This study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus $E_{xx}$ (MPa)</td>
<td>70.34</td>
<td>75.78</td>
</tr>
<tr>
<td>Elastic modulus $E_{yy}$ (MPa)</td>
<td>72.15</td>
<td>59.41</td>
</tr>
<tr>
<td>Ultimate compressive strength $f_{cx}$ (MPa)</td>
<td>1.31</td>
<td></td>
</tr>
<tr>
<td>Ultimate compressive strength $f_{cy}$ (MPa)</td>
<td>4.280</td>
<td></td>
</tr>
<tr>
<td>Ultimate tensile strength $f_{tx}$ (MPa)</td>
<td>0.131</td>
<td></td>
</tr>
<tr>
<td>Ultimate tensile strength $f_{ty}$ (MPa)</td>
<td>0.138</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio $v_{xy}$</td>
<td>0.253</td>
<td>0.242</td>
</tr>
</tbody>
</table>

**Results and discussion**

Building movements at each construction stage in the physical test were imposed at the bottom of the building. In all cases, physical test results closely reflected the numerical solutions. As, the focus of this paper was to investigate divergences between the micro and macro models, the experimental displacements may be considered as benchmarks.

**Figure 6.** Displacements of the top East building of test 4 at the design excavation stage

**Figure 7.** Displacements of the top West building of test 5 at the design excavation stage

Numerical analyses were conducted for the 3 aforementioned excavation stages of both physical tests. Most results show extremely good agreement between numerical solution and the physical test. Only the last stage is shown in Fig. 6 and Fig. 7 corresponding to Pit4E and Pit5W due to space limitations.
Generally, the micro model vertical displacements agreed more closely to the physical test results than the macro model’s, although Pit5W at the façade the absolute difference between the vertical displacements from the macro model and the physical test was about 1.31% lower than with the micro model which was around 5.63%. The vertical displacements from the physical test, micro model, and macro model respectively -4.928mm, -4.650, and -4.863mm. Similar to Pit5W, in the Pit4E test at the façade, vertical displacement from the macro model was -5.698mm, -5.686mm from the micro model, which 4.54% and 4.74% different from the physical test respectively. (-5.969mm). However, the minimum vertical displacement from the micro model better agrees with the large scale model test. Pit4E and the Pit5W were -0.119mm about 6.3% and 0.214mm about 21.39% of the physical test respectively, while the macro model for Pit4E and the Pit5W was -0.049mm (61.42% of the experimental) and -0.078mm (63.44% of the experimental) respectively. 

Fig. 6 and Fig. 7 show close agreement between the horizontal displacements at the top of the building (Pit4E and Pit5W) with the micro model. The highest absolute difference was 15.22% for Pit4E at the right bay 2, and the smallest difference 0.71% at the left bay 1. Most are within 10%. There is good agreement between the macro model and the experiments in the horizontal movement at the top of bay 1 in Pit4E and bay 2 of Pit5W. However, most other areas have substantially poorer correlation. For example in Pit4E in bay 3, these vary 18.87%-28.79% or in Pit5W 24.12%-31.18%. Generally, the macro model over predicts horizontal displacements.

Another consideration is computational costs including CPU time, storage, competence, human cost, and computer hardware. All models were run on a Precision Workstation T5400 Intel(R) Pentium (R) Xeon (8CPU) 2GHz with 8190Mb RAM. The micro model required more than 6 times the CPU time (1496.9s vs 228.28s CPU) than the macro model for Pit4E for the design grade respectively and 517.44s and 433.94s for the micro and the macro model because of the higher loads applied in Pit4E. The micro model also required much more memory for data storage. For Pit4E model 240Mb for the data file and 1.56Gb for the results, and 2.4Gb for all data. The respective numbers for the macro model are only 16.3Mb, 0.163Mb, and 0.292Gb. The micro model requires approximately ten times the resources of the macro model.

Conclusions

The micro model generated marginally better results than the macro but required approximately ten times the resources. The macro model was less successful in predicting horizontal movements due to the anisotropy of the masonry. The comparison shows that the macro model can provide a general response of building subjected to excavation-induced ground movements, but fails to reliably report the horizontal movements, which are arguably the more critical for unreinforced masonry structures, as they tend to represent the tensile forces which control cracking.

Acknowledgements

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References

ANSYS® Academic Research, Release 11.0, Help System, Theory Ref.s, ANSYS, Inc.


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