Possibilities of modeling masonry as a composite softening material: Interface modeling and anisotropic continuum modeling

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Abstract

Results of using recently developed material models for the analysis of masonry structures are shown. Both interface modeling, in which masonry components (units and joints) are represented, as continuum modeling, in which masonry is represented as a homogeneous continuum, are addressed. It is shown that the power of the models have reached those of more advanced research fields.

Introduction

New applications of masonry structures and the correction of pathologies observed in the current use of masonry have been hindered by the limited knowledge of the behavior of units, mortar and masonry as a composite material.

Models of analysis widely accepted by the masonry community are lacking and the possibilities of adopting tools from more advanced research areas, like the mechanics of concrete, rock or composite materials, are limited, in view of the particular characteristics of masonry. A detailed analysis must include units and the (joint) interface unit/mortar. It is applicable to masonry details. A simplified analysis does not distinguish the components but rather looks at the material as an anisotropic homogeneous continuum. It can be applicable to large structures.

This paper presents examples of applications using an interface model, fully described in Lourenço and Rots (1997), and a continuum model, fully described in Lourenço *et al.* (1997).

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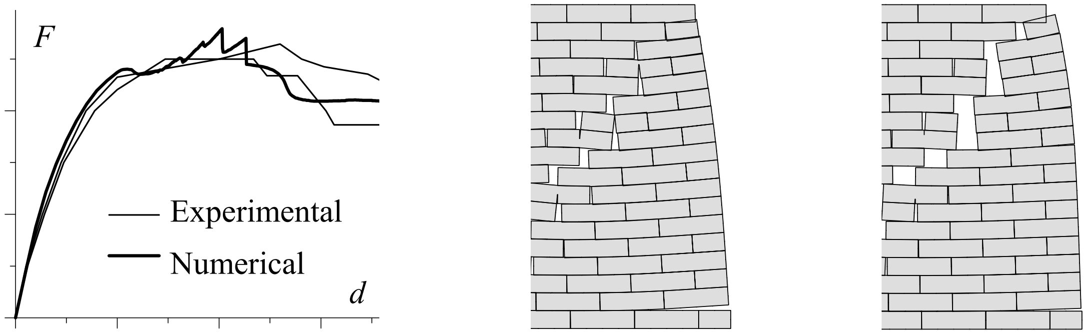
Possibilities of Interface Modeling

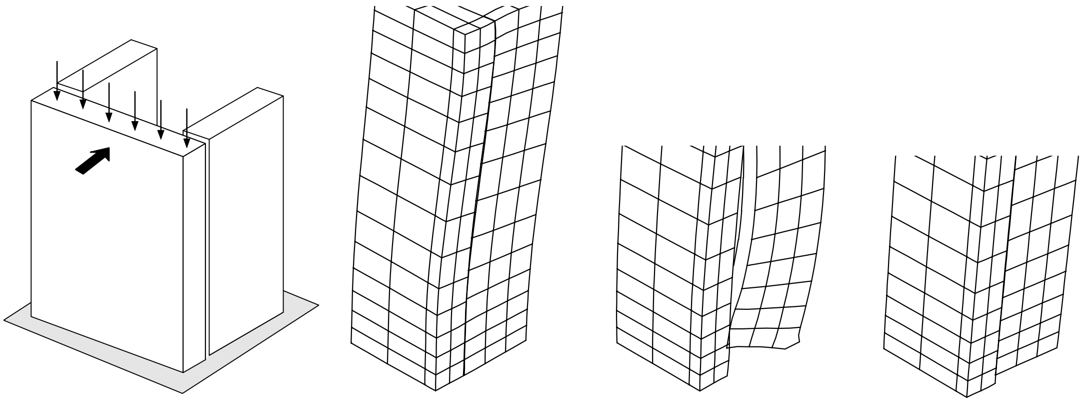
Figure 1 shows the results of modeling a shear wall with an initial vertical pre-compression pressure. The horizontal force *F* drives the wall to failure, keeping the top and bottom boundaries fully constrained, and produces a horizontal displacement *d* at top. Initially, two horizontal cracks develop at the top and bottom of the wall but, at failure, a diagonal stepped crack and crushing of the compressed toes are found. The experimental results can be found in Raijmakers and Vermeltfoort (1992) and a complete discussion of the numerical results has been given in Lourenço and Rots (1997).



(a) (b) (c)

Figure 1. Results for a shear wall: (a) force-displacement diagram; (b,c) deformed meshes at peak and ultimate load

Figure 2 shows the results of modeling a pier-wall connection subjected to wind load. Initially, a uniformly distributed vertical load is applied, before a horizontal load at the top of the wall drives it to failure. A detailed comparison between experimental and numerical results can be found in CUR (1994). In a first phase, the piers and wall are firmly glued, which yields a very stiff structure. At a



(a) (b) (c) (d)

Figure 2. Results for a pier-wall connection: (a) geometry and load; (b,c,d) incremental deformed meshes (only ½ of the wall is shown) before, during and after separation

certain stage, around peak load, separation between the two piers and the wall occurs. After this stage, a much lower load can be carried by the structure because piers and wall behave independently and slide over each other.

Figure 3 shows size effect results of a pier subjected to a point load. In the center of the pier a splitting crack arises which propagates in a catastrophic manner after peak load. The computed crack path is straight and vertical indicating that the crack jumps from head joint to head joint right through the unit.

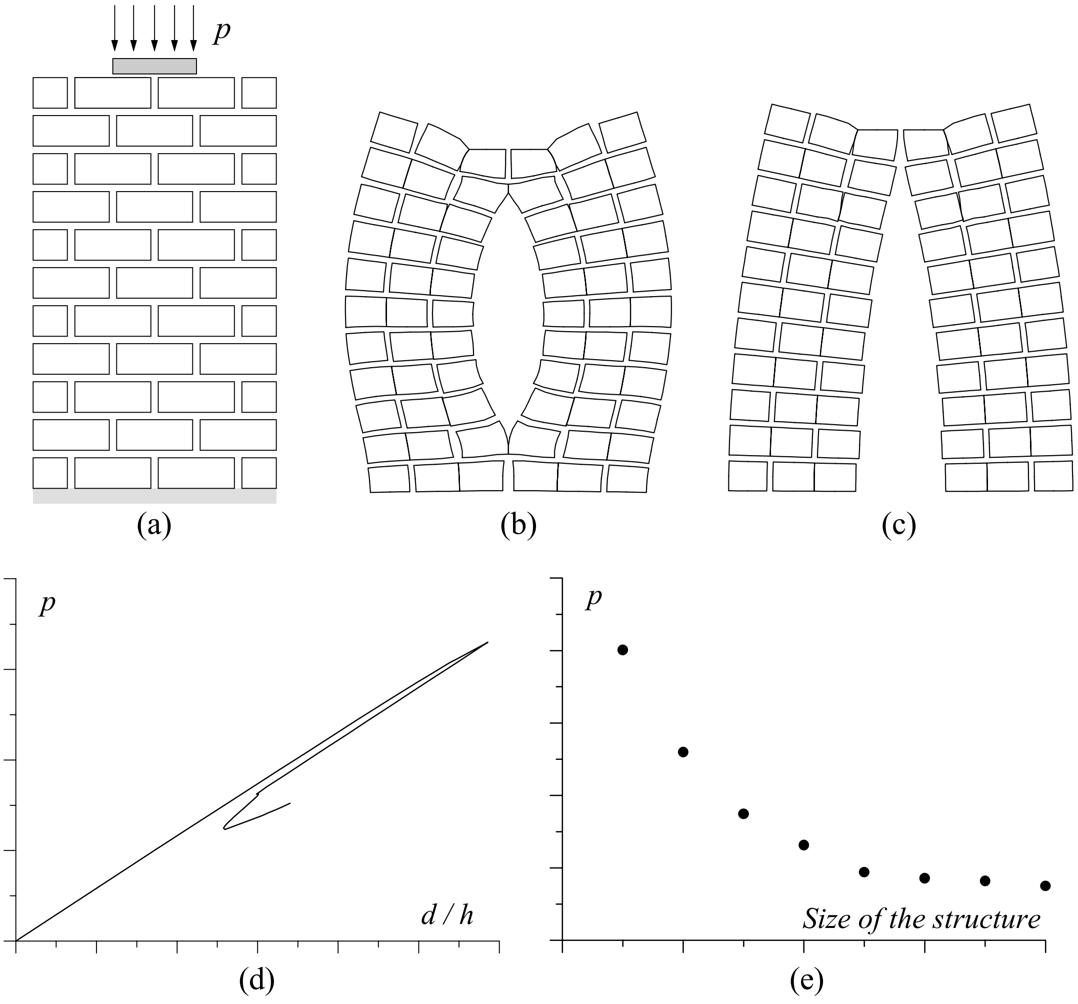


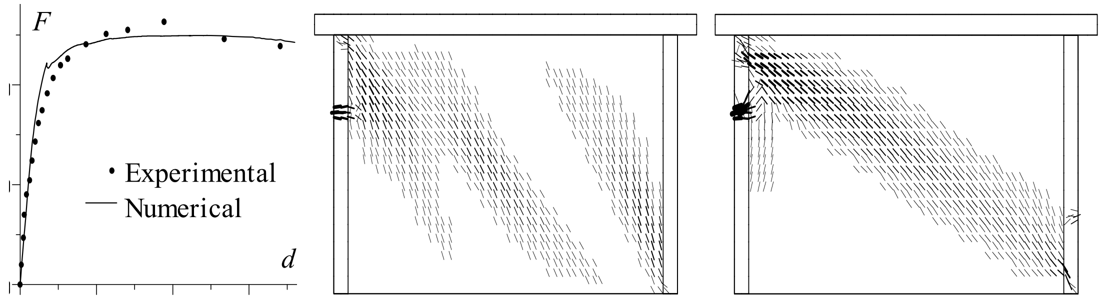
Figure 3. Results for a pier subjected to a point load: (a) geometry and load; (b,c) incremental deformed meshes at peak and ultimate load; (d) load-displacement diagram; (e) relation between nominal load (pressure *p*) and size of the structure

The rising portion of the diagram for the vertical pressure *p* vs. the non-dimensional displacement of the rigid plate *d*/*h*, where *d* is the vertical displacement of the plate and *h* is the specimen height, appears to be almost linear, indicating that the effect of cracking prior to reaching the maximum load is negligible. After reaching the maximum load, a very sudden decrease of both the load and the displacement occurs. This very sharp snap-back obtained at peak load is due to the sudden energy release in the straight crack that arises under the load. For similar peak configurations, the elastic energy stored in the structure would increase with the size. As the fracture energy of the material is a material property, the peak nominal load (or pressure) decreases with an increase in the size of the structure, see Bazant (1984). A complete size effect study of this problem can be found in Lourenço (1997b).

The examples above demonstrate the power of modern numerical tools to represent the complex interaction between masonry components (units and joints). The response of plane and three-dimensional structures controlled by the local behavior of masonry and difficult phenomena observed in the experiments, such as size effect, can be reproduced.

Anisotropic Continuum Modeling

Figure 4 shows the results of modeling another shear wall with an initial vertical pre-compression pressure. The horizontal force *F* drives the wall to failure and produces a horizontal displacement *d* at top. The wall is confined by two concrete slabs (top and bottom) and two masonry flanges (left and right). This confinement and the large size of the wall make it appropriate for continuum modeling. Initially, cracking occurs well distributed in the panel and finally concentrates in a single shear band from one corner of the panel to the other. The compressive stresses are well below the crushing strength of masonry, i.e. failure is dominated by tension. The experimental results can be found in Ganz and Thürlimann (1984) and a complete discussion of the numerical results has been given in Lourenço *et al.* (1997).



(b) (c)

Figure 4. Results for masonry shear wall: (a) load-displacement diagram; (b,c) predicted cracking pattern at peak and ultimate load

Figure 5 shows the results of modeling a panel with out-of-plane pressure. The panel is simply supported on two sides (left and right), fully clamped on one side (bottom) and free on the other (top). The central opening simulates a window and the panel was loaded with an air-bag with a uniformly distributed load. The predicted form of collapse includes diagonal cracks from each lower corner of the panel up to the opening, which were also observed in the experiments. This form of yield line collapse does not mean that yield line design is safe due to the quasi-brittle behavior of the material. The panel was tested by Chong *et al.* (1995) and a complete discussion of the numerical results has been given in Lourenço (1997a).

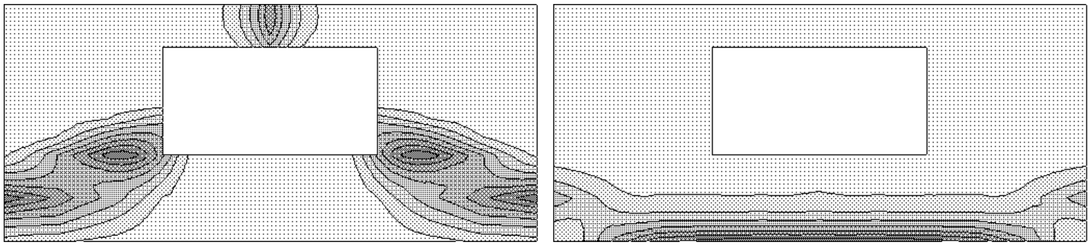

(a) (b)

Figure 5. Results for a panel subjected to out-of-plane loading: (a,b) predicted cracking pattern at bottom and top face of the panel

Figure 6 shows the results of a study concerning an investigation of the expected damage in old masonry buildings due to tunneling. A block of ten façades supported on wooden piles is adopted for this purpose. The applied loads include the self-weight of the structure, dead and live floor loads and the settlements of the piles induced by boring a tunnel parallel to the façade, from right to left. The settlements occur gradually with the construction of the tunnel, starting from the right end of the building. Once the loading process is completed a uniform settlement which amounts to 12.9 *mm* is obtained. The complete description of the finite element model, including geometry, material properties as well as loading and boundary conditions, is given in Hendriks e*t al.* (1995). Based on results of the analysis both the need for repair techniques or the need to reduce the settlement by a better tunneling process can be studied. A complete discussion of the behavior of the structure and criteria for damage acceptance can be found in Hendriks *et al.* (1995).

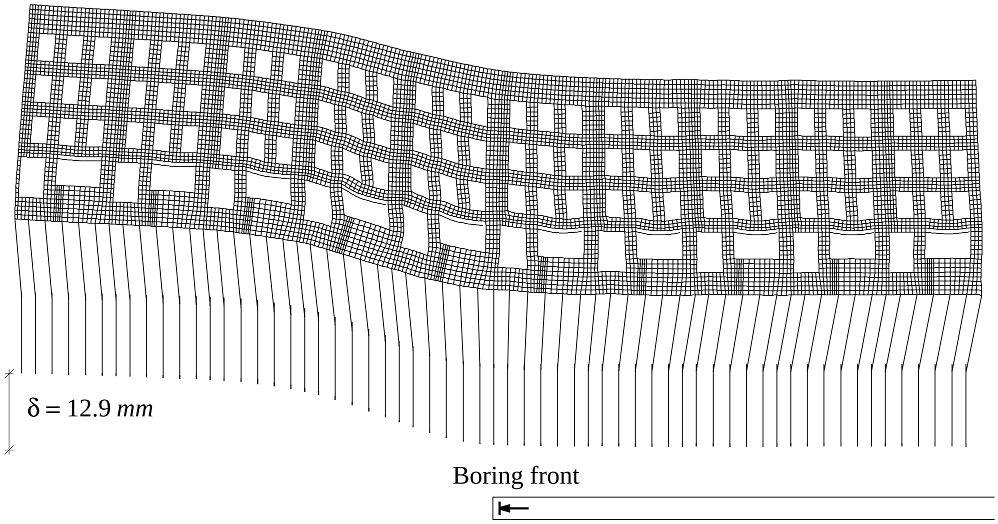


Figure 6. Results for settlement analysis due to tunneling: deformed mesh for boring front position at the middle of the structure

The examples above demonstrate the power of modern numerical tools to represent the composite behavior of masonry structures. The response of plane and shells structures that feature well distributed failure mechanisms can be reproduced.

Conclusions

Examples of applications using two modeling approaches for masonry structures have been given in the present paper: *micro-modeling*, in which joints are modeled with interface elements, and *macro-modeling*, in which a relation is established between average stresses and average strains. The power and level of sophistication of the models have been demonstrated by these results.

Acknowledgments

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