Experimental Investigation and Numerical Modeling of Peak Shear Stress of Brick Masonry Mortar Joint under Compression

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Abstract: This paper presents a study on the shear load-displacement behavior of horizontal joints in unreinforced brick masonry subjected to constant compression. In general, under static shear loading masonry joints show a peak shear stress followed by a residual shear strength. To investigate these aspects in greater detail, triplet tests were conducted on masonry specimens using different types of mortar. The results found in this study and previous tests show that normal compressive stresses acting on the interface and the interface mortar strength affect the peak shear stress and the residual strength in a rather similar way. The cohesion and the internal friction angle, i.e., the two parameters required by the Mohr-Coulomb criterion, are then derived from a linear regression of the test results. The pre-peak and post-peak response of a masonry bed joint can best be represented by simple equations, and their shear stiffness depends on material properties and the magnitude of the normal compression. Computational modeling strategies are then presented considering the shear slip at the brick-mortar interface. The comparison of the model prediction with the results found in this study and previous tests shows the reliability of the proposed model for bed joint behavior. DOI: 10.1061/(ASCE)MT.1943-5533.0000958. © 2013 American Society of Civil Engineers.

Author keywords: Masonry; Triplet test; Shear strength; Numerical model.

Introduction

Shear failure is the dominant mode of failure observed in many masonry buildings subjected to lateral loading due to earthquakes, wind (in tall and slender structures), support settlements, or unsymmetrical vertical loading. Lateral loading can produce both diagonal cracking failures and shear failures of the horizontal joints. Joint resistance is of particular concern in the analysis of the load-bearing unreinforced masonry structures that are rather common among older buildings in many countries in the world. The shear generally acts in combination with compression caused by the self-weight and floor loads. Confinement by, for instance, structural frames to in-fill walls may also lead to shear compression.

The present state of knowledge concerning shear strength and shear load-displacement behavior of masonry is far less advanced than that concerning masonry behavior in compression, even though shear failure is an important, often governing mode of failure in many masonry buildings (Van Zijl 2004). This lack of understanding is reflected by the low values of shear resistance allowed by present U.S. building codes (ASCE 31-02). Information on the post-peak behavior and on the deformations associated with pre-peak and post-peak responses are also lacking. Only recently, the terms softening and dilatancy were introduced in the research community (Lourenço et al. 1998; Van Zijl 2004). Knowledge of such behavior is essential if adequate analytical models are to be developed to describe the in-plane behavior of masonry walls. Most of the research conducted to date regarding the masonry shear behavior has been limited to determining the peak shear stress and its affecting parameters.

A variety of experimental approaches (Fig. 1) has been adopted in the last two decades to determine the shear behavior of joints of unreinforced masonry. A widely used approach is the compressive loading of a prismatic masonry specimen that contains a single joint at an angle, , to the applied load as illustrated in Fig. 1(a) (Nuss et al. 1978; Hamid and Drysdale 1980). The nature of this force-controlled test makes it impossible to obtain data in the post-peak range, as the specimen collapses in an unstable manner after attaining its strength. Studies using this approach have, however, have provided valuable information concerning the factors (including mortar type) that influence the peak shear stress.

Van der Pluijm (1993) presents the most complete characterization of the masonry shear behavior for solid clay and calcium-silicate units. The test setup shown in Fig. 1(b) allows to apply a constant confining pressure upon shearing. The confining (compressive) stresses were applied at three different levels, namely, 0.1, 0.5, and 1.0 MPa. Thereby, the specimen edges could translate in the direction normal to the shearing deformation. The uplift or displacement normal to the shear joint, which is known as dilatancy, was also measured. Armaanidis (1998) measured a dilatation angle from 23.5° to 34.5° for limestone using a direct shear test. He proposed that the shear strength at the weak discontinuities of limestone be a combined effect of both the internal friction angle (φ) and the dilatancy angle (ψ) and, proposed the following expression:

\[ \tau_u = c + \sigma_n \tan(\phi + \psi) \]  

Hansen (1999), Gottfredsen (1997), and Chaimoon and Attard (2009) also used the same experimental technique in their study.

Many researchers (Yokel and Fattal 1975; Calvi et al. 1985; Gabor et al. 2006) have used the test configuration shown in Fig. 1(c) to study the shear strength of masonry subjected to diagonal compression. The concentrated diagonal load creates...
in-plane shear stress along the joints of the specimen. The distribution of normal and shear stresses along any given joint is strongly nonuniform, with the result that shear strength determined from this test represents an average value of progressive failure events, because of the stress redistribution during the failure process rather than reflecting a true material property. The post-peak behavior and deformations cannot be obtained realistically by means of this experimental configuration.

The triplet test configuration shown in Fig. 1(d) was adopted by Lourenço et al. (2004) as recommended by European Standard EN 1052-4 [European Committee for Standardization (CEN) 2000]. This test was conducted to verify the Mohr-Coulomb criterion with a cohesion value of the order of 1.4 MPa, and the initial friction coefficient \( \tan(\phi) \) of 1.03. Copeland and Saxer (1964) used the same specimen configuration to identify the parameters affecting the shear bond between brick and mortar.

Meli (1973) used the test configuration shown in Fig. 1(e) to investigate bond and friction of joints with different unit types. A linear variation of the shear strength with confining pressure was observed. Bond strength was found to vary with the mortar and unit types. Hamid and Drysdale (1980) also used the test configuration shown in Fig. 1(e) to study the shear response of both grouted and ungrouted concrete masonry. Their results showed that the coefficient of friction decreased with an increase in the confining stress and that grouted specimens yielded friction coefficients that were considerably higher than ungrouted specimens. Data concerning the deformation in the direction of the shear load showed that ungrouted masonry has a considerably higher initial shear stiffness in comparison to grouted concrete masonry. With the increase in normal stresses both the shear strength and the shear stiffness increase. The post-peak frictional response under shear loading that was applied in the same direction as the initial shear force was also determined.

Abdou et al. (2006), El-Sakhawy et al. (2002), and Atkinson et al. (1989) conducted direct shear tests on masonry couplets as shown in Fig. 1(f). They used a servo-controlled system to measure the shear load-displacement characteristics for different types of brick and mortar. Abdou et al. (2006) tested both hollow and solid bricks, and found that the shear stiffness of masonry with hollow bricks is higher than that of masonry with solid bricks, because of the mortar that entered inside the holes and acts as an abutment, thus giving more shear resistance than the solid brick. He also found that the ultimate shear strength and residual friction are independent of the brick types. Only one type of mortar was used (20 MPa), so no direct correlation between mortar grade and shear strength could be established. The joint failure could be well represented by the Mohr-Coulomb criterion when a shear load is applied together with compression. Atkinson et al. (1989) conducted a series of tests on various types of brick and mortar for both static and cyclic loading. Some of the results along with their test methods are given in Table 1 and Fig. 2.

Previous studies on joint shear behavior while providing insight into some of the parameters influencing the shear strength generally do not provide detailed information related to the constitutive behavior that would be required to set up analytical models for simulating the structural response under different loading conditions. Such a model will require definitions of (1) shear stiffness for initial loading states; (2) peak and residual stresses; and (3) the effect of materials properties and normal loads on shear strength/stiffness and dilatancy. As in the case of rock joints (Goodman 1976; Armaanidis 1998), dilatancy is the normal expansion or contraction upon shearing. A complex relationship exists among joint normal stiffness, normal displacement, and shear displacement (Saeb and Amadei 1988; Van Zijl 2004). Dilatancy can produce an increase in the normal load resulting in increased shear strength, when the normal boundary condition is displacement controlled. This applies, for example, to in-fill panels where the stiffness of the frame enclosing the panel affects its normal displacements. An evaluation of the dilatancy requires the measurement of both normal and shear displacements prior to and after the peak shear stress. It was also...
**Table 1. Various Experimental Results on Interface Shear Stress-Slip Test**

<table>
<thead>
<tr>
<th>Author’s name and test method</th>
<th>$f_{cm}$ (MPa)</th>
<th>$E_m$ (GPa)</th>
<th>$f_{cb}$ (MPa)</th>
<th>$\sigma_u$ (MPa)</th>
<th>$\tau_u$ (MPa)</th>
<th>$\tau_r$ (MPa)</th>
<th>$c$ (MPa)</th>
<th>$\phi_i$ (Deg)</th>
<th>$\phi_r$ (Deg)</th>
<th>$t_{mo}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hansen (1999) for solid clay bricks with couplet specimen [Fig. 1(b)]</td>
<td>3.8</td>
<td>2.8</td>
<td>32.0</td>
<td>0.1</td>
<td>0.7</td>
<td>0.08</td>
<td>0.68</td>
<td>23.9</td>
<td>40.1</td>
<td>12</td>
</tr>
<tr>
<td>Hansen (1999) for perforated clay bricks with couplet specimen [Fig. 1(b)]</td>
<td>3.8</td>
<td>2.8</td>
<td>46.0</td>
<td>0.1</td>
<td>0.69</td>
<td>0.12</td>
<td>0.68</td>
<td>45.0</td>
<td>45.8</td>
<td>12</td>
</tr>
<tr>
<td>Chaimoon (2007) for solid clay bricks with frog marks on couplet specimen [Fig. 1(b)]</td>
<td>7.3</td>
<td>6.2</td>
<td>11.1</td>
<td>0.2</td>
<td>0.70</td>
<td>0.28</td>
<td>0.68</td>
<td>31.0</td>
<td>53.7</td>
<td>12</td>
</tr>
<tr>
<td>Lourenço et al. (2004) for hollow clay bricks with triplet specimen [Fig. 1(d)]</td>
<td>30.3</td>
<td>22.2</td>
<td>31.8</td>
<td>0.2</td>
<td>1.5</td>
<td>0.26</td>
<td>1.39</td>
<td>37.6</td>
<td>32.5</td>
<td>25</td>
</tr>
<tr>
<td>Van der Pluijm (1993) for solid clay bricks with couplet specimen [Fig. 1(b)]</td>
<td>9.0</td>
<td>6.0</td>
<td>11.0</td>
<td>0.1</td>
<td>0.89</td>
<td>0.08</td>
<td>0.87</td>
<td>42.9</td>
<td>37.2</td>
<td>15</td>
</tr>
<tr>
<td>Abdou et al. (2006) for hollow clay bricks with couplet specimen [Fig. 1(f)]</td>
<td>20.0</td>
<td>14.7</td>
<td>24.0</td>
<td>0.36</td>
<td>1.62</td>
<td>0.82</td>
<td>1.50</td>
<td>23.9</td>
<td>41.7</td>
<td>10</td>
</tr>
</tbody>
</table>

Note: $f_{cm}$ = Uniaxial compressive strength of mortar and brick respectively; $E_m$ = Young’s modulus of mortar; $\sigma_u$ = Normal precompression; $\tau_u$ = Ultimate shear strength; $\tau_r$ = Residual shear stress; $c$ = Interface cohesion; $\phi_i$ = Initial friction angle; $\phi_r$ = Residual friction angle; $t_{mo}$ = Thickness of mortar.

Evidently, the effect of dilatancy under high compression is marginally small and can be neglected (Gabor et al. 2006). This paper examines the shear failure mode occurring in horizontal joints and the shear stress-slip behavior of unreinforced brick masonry under static loading. Nonstandard tests were conducted on four series of mortar specimens; triplet shear specimens [Fig. 1(d)] were used, in order to study the effect of mortar strength on joint shear behavior. At the outset, the paper describes the experimental apparatus and the sample preparation procedures. Then the experimental program is outlined. This is followed by a description of the experimental results. An analytical approach has been taken to correlate the shear strength with confining pressure and mortar strength. Finally, a macro mechanical model for shear stress slip is proposed with numerical examples. Finally, some exhaustive conclusions are drawn on the basis of the observed results.

### Experimental Program

The materials used in the preparation of the triplet shear test specimens included one type of wire-cut clay brick and four types of mortar with different proportions and strengths indicated as E, M, S, and N, respectively. The brick used here had an actual dimension of 250 × 120 × 70 mm. Bricks were immersed in water the day before the construction of the tested specimen assembly. They were then dried in normal laboratory conditions for at least 1 day prior to building the specimens to ensure saturation degree of 80%. The brick compressive strength was 17 MPa. The three types of ordinary mortar (M, S, N) used were prepared following the provisions of ASTM C270 (ASTM 2007a) for the construction of masonry walls. The type E mortar was prepared as high strength mortar with comparatively lower water-cement ratio. The reason for choosing these mortar types was to study the effect of mortar types and strength on the shear strength. The workability of the mortar was monitored using the flow test per ASTM C 1437 (ASTM 2007b). For each batch of mortar type used in the construction of the specimens 10 cylinders ($\varnothing = 50$ mm) were cast to determine the compressive and splitting strength of the mortar. Table 2 summarizes the results for the four types of mortar used in the present investigation.

### Specimen Preparation

The specimens were built with two full bricks, one 3/4 brick and 1/4 brick bonded together by a 10 mm-thick mortar joint as shown in Fig. 3(a). To ensure the correct dimension of the mortar joints, a timber block that was thicker than the bricks by 10 mm was placed over the first brick. More than the needed amount of mortar was then placed on the top face of the brick with a trowel. The one 3/4 brick (170 mm) was then placed in such a way that a small portion of it (60 mm) exceeded the bottom brick and rested on the timber block. Another quarter brick was then placed 30 mm apart from the 3/4 brick. The second brick was then placed on these two cut bricks, tapped with a wooden mallet, and leveled in two
Fig. 2. Shear-slip relationship of different experimental data from (a) Van der Pluijm (1993); (b) Hansen (1999); (c) Chaimoon (2007); for solid clay bricks and (d) Hansen (1999); (e) Lourenco et al. (2004); (f) Abdou et al. (2006) for perforated clay bricks

Table 2. Specification of Mortar Used in Triplet Shear Test Specimen

<table>
<thead>
<tr>
<th>Mortar type</th>
<th>Cement: sand (by volume)</th>
<th>Water/cement (by weight)</th>
<th>Compressive strength (MPa)</th>
<th>Splitting tensile strength (MPa)</th>
<th>Young’s modulus (GPa)</th>
<th>Poisson’s ratio ν</th>
<th>Flow (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T2:1</td>
<td>E</td>
<td>1:2.25</td>
<td>0.50</td>
<td>28.5</td>
<td>3.0</td>
<td>26.0</td>
<td>0.186</td>
</tr>
<tr>
<td>T2:2</td>
<td>E</td>
<td>1:2.25</td>
<td>0.50</td>
<td>28.5</td>
<td>3.0</td>
<td>26.0</td>
<td>0.186</td>
</tr>
<tr>
<td>T2:3</td>
<td>M</td>
<td>1:2.75</td>
<td>0.70</td>
<td>20.0</td>
<td>1.7</td>
<td>19.3</td>
<td>0.156</td>
</tr>
<tr>
<td>T2:4</td>
<td>S</td>
<td>1:3.5</td>
<td>0.86</td>
<td>12.5</td>
<td>1.5</td>
<td>15.7</td>
<td>0.200</td>
</tr>
<tr>
<td>T2:5</td>
<td>N</td>
<td>1:4.0</td>
<td>0.95</td>
<td>10.0</td>
<td>0.9</td>
<td>14.5</td>
<td>0.188</td>
</tr>
</tbody>
</table>
194 directions with a sprite level to create a 10 mm-thick mortar joint. The excess mortar, which squeezed to the sides, was removed with a trowel, and the sides of the mortar joint were flattened at the same level of bricks on all sides. The timber block was then removed, and the specimen was left in place for 5 days to allow the mortar to develop sufficient strength. During these 5 days, the specimens were covered with thin plastic sheeting for curing. After the 5 days of initial curing, the plastic sheet was removed. The specimens were then left for an additional 23 days to cure under ambient conditions in the laboratory before testing, which began 28 days from construction.

**Instrumentation and Test Setup**

Five specimens for each mortar type were built and cured for 28 days. Before testing, the length of the mortar joint was measured. Two steel plates were attached on both sides of the specimen and kept in position with four bolts. A uniform confining pressure was exerted on the specimen using a manually controlled hydraulic jack having a load gauge. When the expected level of pressure was reached, the specimen was ready for the shear test. Four linear displacement (LVDT) gauges were attached on the top of the 3/4 brick that will be load for shear [Fig. 3(b)] on opposite sides of joints to record the shear displacement. The specimen was designed in such a way that the applied load be transferred through the upper 3/4 brick as shear and the confining pressure be carried out by both the top and bottom bricks. The area resisting to shear and compression was calculated accordingly. Fig. 4 shows the loading and support arrangements used for testing of the specimens. Synthetic elastomers were used to ensure a uniform load distribution over the area and supports.

**Testing and Measurements**

For each type of mortar, five specimens were tested with a constant confining pressure of 0.25, 0.50, 1.0, 1.25, and 1.0 MPa, respectively, resulting in a total of 20 specimens. At the beginning, the required confining pressure is applied through the hydraulic jack. Then specimen was transferred under the actuator of a universal testing machine to apply the shear load as compression. The maximum loading capacity of the vertical actuator is 1,000 kN. The shear load was applied at a rate of 0.05 mm, and the corresponding shear displacement was measured by means of four LVDTs attached to two opposite sides of the specimen [Fig. 3(b)] and recorded through a data logger. The confining pressure was kept almost constant throughout the entire loading process. Fig. 5 gives a confining pressure as a function of time for each specimen tested; the plots show that there is little fluctuation of the confining pressure. This is due to the fact that when two rough surfaces of brick and mortar slide over each other dilatancy takes place, which causes an increase in volume, and thus pressure on the steel plate. This excess pressure somehow contributes to the overestimation of the shear strength at the interface, but for the sake of simplicity of the analysis, the dilatancy effect is neglected in the numerical modeling. A more detailed explanation of this exclusion is given in the subsequent paragraph.

**Test Results and Discussion**

It is quite obvious that the ultimate shear strength increases with increasing confining pressure normal to the shearing surface.
However, this is not the only governing factor that influences the shear strength of the brick-mortar interface. The other factors are (1) characteristics of bond between mortar and brick; (2) characteristics of brick and mortar; (3) coefficient of friction between the two sliding surfaces; and (4) the overall quality of the joint. Since during the fabrication of the specimens an overall uniformity was difficult to be attained, some inconsistencies are inevitable. Figs. 6(a and b) show the nominal shear stress as a function of the shear displacement for some of the tested specimens. According to the figures, it is easy to see that just before the peak shear stress the stiffness is very high with very little shear deformation. The interlocking between the grains of the brick and the mortar under confining pressure is the main reason for the high stiffness of the load-displacement relationship. There is a barely detectable hardening phase but just for a short range before the peak load. As the imposed shear displacement overcomes the interlock between brick and mortar, a phenomenon of volume increase (dilatancy) takes place and gives rise to a much higher strength than expected. In the present study, the effect of dilatancy was not considered due to the fact that at a confining pressure higher than 0.5 MPa this dilatancy becomes marginally small, the effect of internal friction dominates over dilatancy (Armanidis 1998), and therefore makes it possible to evaluate the shear strength by means of the Mohr-Coulomb criterion. Moreover, the high confining pressure restricts the upward dilatant deformation of the specimen and turns it into deformation of brick and mortar by squeezing them laterally at constant volume.

As previously mentioned, the interface behaves like a quasi-brittle material and exhibits a very small hardening branch that appears between the elastic limit and the peak stress. The post-peak damage and release of strain energy is quite evident as the stress drops gradually. After the initial damage, the shearing surface readjusts and relocates its position for new sliding resistance after losing the cohesive bond at the brick-mortar interface. This stage is called residual stress and depends mainly on the interface static friction and confining pressure; in the following, it will be indicated as residual shear strength. After reaching the residual shear strength, the relative movement between the two sliding surfaces turns into a rigid body movement with very little (or no) relative shear deformation. This stage can be considered as a complete failure stage, and the whole phenomenon can be indicated as dynamic friction, something that is beyond static equilibrium and static analysis.

In this study, two important parameters, confining pressure and mortar strength, were noticed as major factors contributing to the shear capacity at the brick-mortar interface. The increase in shear strength with increasing confining pressure for different mortar strengths (N, S, M, and E) can be seen in Fig. 7(a). The experimental parameters found from the triplet shear test are given in Table 3. It is quite evident from Fig. 7(a) that the shear strength does increase with increasing confining pressure in a rather nonlinear fashion for different mortar strengths, which is incompatible with Mohr-Coulomb criterion but consistent with the Mohr-Coulomb failure envelop or rupture line. For a specific material, the rupture line may be a curve as shown in Fig. 8.

The Fig. 7(a) also indicate that the strength of mortar have a significant role on the peak shear stress. The other two parameters, namely, cohesion ($c$) and internal friction angle ($\phi$), which are the inherent properties of the interface between brick and mortar, also vary with the mortar strength and confining pressure. The cohesion $c$ is independent of normal stress and only increases a little with increasing mortar strength whereas the friction angle $\phi$ is dependent on normal stress but cannot be verified independently at this present stage of knowledge.

The relationship between confining pressure and residual shear strength is shown in Fig. 7(b). Once the interface cohesion is lost, the ratio of residual shear strength to confining pressure (or residual friction coefficient) increases to an almost constant rate that is independent of the mortar strength.

The failure modes of the shear test specimens were predominantly interface failures. In all cases, the mortar separated from either the inner brick or the outer, or both. No substantial damages were seen on the brick surfaces; rather, some small mortar pieces appeared to remain attached to the brick surfaces (Fig. 9), and the whole phenomena can be indicated as dynamic friction, something that is beyond static equilibrium and static analysis.
something that indicates that if the brick strength is higher than the mortar strength, damage takes place within the mortar. During the increase of the shear load, a minor crack was observed, propagating almost halfway into the depth of the outer bricks at the peak shear load. This crack can be regarded as a flexural crack. Since the crack does not reach the interface, its effect on the average shear strength at the interface is not very significant.

**Shear Strength and Shear Stress-Slip Relationship**

The shear capacity of masonry joints with moderate confining pressure can be predicted by the Mohr-Coulomb criterion (Lourenço et al. 2004), which establishes a linear relationship between the...
Numerical Modeling

The failure analysis of masonry structures has been based on modeling techniques developed in modern concrete mechanics. For fully grouted reinforced masonry, where the influence of mortar joints is marginal, the smeared crack approach can be applied to the analysis of such masonry structures (Lotfi and Shing 1991). On the other hand, the behavior of unreinforced masonry may not be modeled accurately by the smeared crack approach as unreinforced concrete behavior cannot. Although intact brick units may be assumed homogeneous and isotropic, the presence of mortar joints makes unreinforced masonry composite both heterogeneous and anisotropic and shows distinctive directional properties at the time of load-reaction interaction. In the finite-element analysis of unreinforced masonry structures, the effect of mortar joints as the major source of weakness and material nonlinearity has been accounted for with different levels of refinement. In general, the approach towards its numerical representation can focus on the micromodeling of the individual components, viz. unit (e.g., brick, block) and mortar, or the macromodeling of masonry as a composite (Rots 1991). Depending on the level of accuracy and the simplicity desired, the modeling technique can be categorized into three possible ways (Fig. 10):

1. Detailed micromodeling: Units and mortar in the joints are represented by continuum elements whereas the unit-mortar interface is represented by interface elements.
2. Simplified micromodeling: Expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit-mortar interface is lumped into interface elements.
3. Macromodeling: Units, mortar, and the unit-mortar interface are smeared out in the continuum elements. In the macroanalysis, masonry is considered as a single material (also known as a homogenized material), which inherently includes the effect of mortar joints.

In the simplified micromodeling approach, masonry units are modeled with continuum elements, while mortar joints are modeled by means of interface elements. Each joint, consisting of mortar and the two unit-mortar interfaces, is lumped into an average interface while the units are expanded in order to keep the geometry unchanged. Masonry is thus considered as a set of elastic blocks bonded by potential fracture/slip lines at the joints. Accuracy is lost since Poisson’s effect of the mortar is not included. Early attempts with this approach were made by Arya and Hegemier (1978), Page (1978), and more recently, Rots (1991). Obviously, the approach with this level of refinement is computationally intensive for the analysis of large masonry structures, but it is certainly a valuable research tool and also a viable alternative to the costly and often time-consuming laboratory experiments. From a modeling point of view, the aforementioned approach is similar to the discrete element method, which was originally proposed by Cundall (1971) in the area of rock mechanics.

In this current study, the simplified micromodeling approach is adopted for simulating the behavior and failure mechanisms of masonry assemblages based on the behavior of the basic constituents. A simple but general model for shear cracking in the masonry interface is proposed. It is defined in terms of shear stress on the average plane of the crack and the corresponding relative shear displacement. In the following sections, the formulation of the interface model is explained, and the applicability of the interface model to mortar joints is validated by experimental results. The proposed model can be implemented directly as the constitutive law of the interface element in the context of discrete crack analysis.

Fig. 10. Modeling strategies for masonry structure: (a) masonry sample; (b) detailed micromodeling; (c) simplified micromodeling; (d) macromodeling
Shear-Compression Model

A simplified micromodeling approach is proposed here to model the masonry interface. The mortar thickness and the brick-mortar interfaces are lumped into zero-thickness 8-node interface elements while the dimensions of the brick unit are expanded to keep the geometry of a masonry structure unchanged and modeled as a 20-node solid element. At the level of the interface Gauss integration points for brick-mortar joints. The failure surface consists of a Mohr-Coulomb linear inelastic surface and a tension cutoff. The limit of the Mohr-Coulomb surface is assigned by adopting a compression cap line beyond the compressive strength of masonry and high enough to crush the mortar or bricks, whichever is weaker in strength. The local bond stress-slip, given by Eq. (3) (Fig. 12), only two parameters, interface fracture energy $G_f$ and interface initial shear stiffness $B$, are needed to define the bond stress-slip relationship. The interface initial shear stiffness $B$ is a function of mortar thickness $t_m$, Young’s modulus of brick and mortar $E_b$, $E_m$, normal precompression $\sigma_n$, and mortar compressive strength $f_{cm}$. In Eq. (4), $B$ reads as

$$B = 2.75 \frac{\log(E_b E_m t_m)}{\exp[4.039 \exp(-0.155 f_{cm}^{0.525})]}$$

The fracture energy parameter $G_f$ can be given by Eq. (5) once the peak shear stress $\tau_u$ and shear stiffness $B$ are known, which reads as

$$G_f = \tau_u / 0.5B$$

Here $\delta_u$ is the shear deformation corresponding to peak shear stress $\tau_u$. The post-peak regime of the bond stress-slip ($\tau-\delta$) relationship in Fig. 12 can be given by the following equation:

$$\tau = \tau_u \left( \frac{\delta}{\delta_u} \right)^n$$

where $n = \text{function of fracture energy } G_f$ and can be given by Eq. (8)

$$n = 1.01 \exp(-2.288G_f)$$

After the complete loss of cohesion and permanent deformation at the interface, only a fraction of shear stress can be transferred through the joint, which is residual shear stress $\tau_{res}$. Eq. (9) gives the magnitude of this residual shear stress, which depends on the level of available compression pressure and the tangent of the residual friction angle $\phi_{res}$. A maximum slip $\delta_{max}$ corresponds to the point on the curve where shear stress $\tau$ is equal to the residual shear stress $\tau_{res}$ in Eq. (7) (Fig. 12)

$$\tau_{res} = \sigma_n \tan \phi_{res}$$

Fig. 11. Failure surface of masonry joint with compression cap proposed by Chaimoon and Attard (2007)

Fig. 12. Shear stress-slip model for the interface

Fig. 13. Typical unloading-reloading behavior of brick-mortar interfaces
post-peak regime, the cohesion is completely lost and the overwhelming damage occurs on aggregate interlocking between brick and mortar grains. Once, the shear force due to aggregate interlocking is lost, the remaining shear resistance comes only from the interface friction in the order of residual stress where a state of complete damage prevails. Subsequent equations for unloading and reloading are postulated considering this phenomenological observation from the shear test on the brick-mortar assemblage.

\[
\text{Loading: } \delta > \delta_{\text{p max}}
\]

For pre-peak regime: \( \delta \leq \delta_u \)

\[
\tau = 2BG_f[\exp(-B\delta) - \exp(-2B\delta)]
\]  
(10)

For post-peak regime 1: \( \delta_u < \delta \leq \delta_{\text{max}} \)

\[
\tau = \tau_u \left( \frac{\delta - \delta_u}{\delta_{\text{max}} - \delta_u} \right)^n
\]  
(11)

**Fig. 14.** Comparison of simulation with experimental data from (a) and (b) Hansen (1999); (c) Van der Pluijm (1993); (d) Chaimoon (2007); (e) and (f) from this study

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For post-peak regime 2: $\delta_{\text{max}} < \delta$

$$\tau_{\text{res}} = \sigma_{\text{res}} \tan \phi_{\text{res}}$$ (12)

where $\delta$ = instantaneous deformation; $\delta_{p,\text{max}}$ = max slip during the loading history; $\delta_{\text{last}}$ = slip in the previous loading step; $\delta_u$ = slip for the peak shear strength; and $\delta_{\text{max}}$ = slip at which shear stress turn to residual.

Unloading: $\delta < \delta_{\text{last}}$

For pre-peak regime: $\delta \leq \delta_u$

$$\tau = \frac{1}{l_m} 2\alpha K_0 G_{\text{eff}} \delta_e$$ (13)

$$\delta_e = \delta - \delta_p$$ (14)

$$\delta_p = \left\{ \frac{\delta_{p,\text{max}}}{\delta_u} = \frac{20}{7} \left[ 1 - \exp \left( -0.35 \frac{\delta_{p,\text{max}}}{\delta_u} \right) \right] \right\} \delta_u$$ (15)

$$\alpha = K_0^2 + \left( \frac{\delta_{\text{last}}}{2K_0 G_{\text{eff}} (\delta_{\text{last}} - \delta_p)} - K_0^2 \right) \left( \frac{\delta - \delta_p}{\delta_{\text{last}} - \delta_p} \right)^2$$ (16)

$$K_0 = \exp \left[ -0.73 \frac{\delta_{p,\text{max}}}{\delta_u} (1 - \exp(-1.25 \frac{\delta_{p,\text{max}}}{\delta_u})) \right]$$ (17)

$$G_{\text{eff}} = G_i (1 + 0.784 \sigma_{e,\text{r}})$$ (18)

$$G_i = \frac{0.85 \exp[4.25 + 1.325 \ln(f_m^{\text{fmc}}) - 0.275 \ln(f_m^{\text{fmc}})^2]}{\exp(1/l_m)};$$

if $f_m^{\text{fmc}} < f_i^{\text{fbt}}$ (19)

$$G_i = \frac{0.85 \exp[4.5 + 1.25 \ln(f_i^{\text{fbt}}) - 0.125 \ln(f_i^{\text{fbt}})^2]}{\exp(1/l_m)};$$

if $f_m^{\text{fmc}} > f_i^{\text{fbt}}$ (20)

Here $K_0$ = fracture parameter; $\alpha$ = stress reduction factor; $\delta_p$ = plastic deformation; $\delta_e$ = elastic deformation; $G_i$ = initial shear modulus of the interface at zero pre-compression, and $G_{\text{eff}}$ = effective shear modulus at any level of precompression.

For post-peak regime 1 and 2: $\delta > \delta_u$

$$\tau = \frac{1}{l_m} G_{\text{unld}} (\delta - \delta_p)$$ (21)

$$G_{\text{unld}} = G_{\text{eff}} \times D_0$$ (22)

$$D_0 = \exp \left\{ -2.8 \frac{\delta_{p,\text{max}} - \delta_u}{\delta_{p,\text{max}}} \left[ 1 - \exp \left( -2.5 \frac{\delta_{p,\text{max}} - \delta_u}{\delta_{p,\text{max}}} \right) \right] \right\}$$ (23)

$$\delta_p = \left[ \frac{\delta_{p,\text{max}}}{G_{\text{eff}} D_0} \right]$$ (24)

Reloading: $\delta_{\text{last}} < \delta < \delta_{\text{max}}$

For pre-peak regime: $\delta \leq \delta_u$

$$\tau = \left[ \tau_{p,\text{max}} - (\tau_{p,\text{max}} - \tau_{\text{last}}) \frac{\delta_{p,\text{max}} - \delta_u}{\delta_{p,\text{max}} - \delta_{\text{last}}} \right]$$ (25)

$$\tau_{p,\text{max}} = \tau_{p,\text{max}}$$

Model Implementation and Validation

The proposed analytical model needs only the Young’s modulus of both brick and mortar $E_b$ and $E_m$, the thickness of the mortar $l_m$ and the overburden pressure $\sigma_c$. In most cases, experimental results given in Table 1 show good agreement (Figs. 14 and 15) with that of the analytical curves. In some cases, the pre-peak stiffness and the peak shear stress were simulated quite closely whereas in few cases the post-peak regimes show a little difference with the experimental one. This can partly be explained because of the experimental shear stress gives an average stress over the entire interface under investigation where a single softening constitutive law is not valid because of the variation of interface properties and cohesion over the sliding surface due to the lack of uniformity at the time of specimen fabricating involving human error, whereas in numerical models all of this variability is ignored and a single branch softening constitutive law is provided; hence, the softening behavior appears to be of the same nature for all cases. Moreover, in an experimental procedure the damage is gradual and the location where a complete state of interface damage prevails is rather difficult to locate; hence the exact point from where the residual stress is going to initiate is also difficult to determine. For numerical models, it is done when the softening shear stress meets the residual shear stress criterion, so the junction of these two is not smooth but an abrupt change in direction and slope that is evident in some experimental results also. In addition, the post-peak behavior during the testing procedure is so delicate and abrupt that it is quite impossible to obtain solicited data from the test unless one has very sophisticated and well controlled experimental facilities. In this experimental procedure the authors did not have that level of control over the post-peak regime and that is the reason for the straight line softening and inconsistent variation in the post-peak regime for most of the cases. This may also be true for other experimental results such as Lourenço et al. (2004) in Fig. 15. Nevertheless, the model curve can predict the ultimate shear strength and residual shear stress quite correctly, and this is the merit of this model over few others limitations. If an accurate estimation of the empirical parameter $B$ is ensured, the initial stiffness and shear displacement at peak shear stress will be very close to that of the experimental results.

If the modulus of mortar is not readily available, Eq. (26) is recommended by Euro Code 6 for normal weight concrete and can be used for mortar as well

$$E_m = 22,000 \left( \frac{f_c^{\text{fc}}}{10} \right)^{1/3}$$ (26)

The interface cohesion $c$ and the initial friction angle $\phi$ are the two inherent properties of mortar and brick, the materials used in this experiment. One is independent of stress and the other is a stress-dependent parameter. For numerical analysis of masonry wall for shear, any reasonable value of these two will produce consistent result. However, for normal strength mortar and brick a range from 0.15 to 0.25 for $c$ and 50 to 65° for $\phi$ will yield a good approximation of experimental results. For the residual friction angle $\tau_{\text{res}}$, a reasonable approximate value from 45 to 55° can be used safely for model implementation.

Conclusions

In this study, some results obtained on mortar joints in brick masonry under shear and compression, are compared with other similar results from the literature. First, various experimental results are compared on the basis of two major affecting parameters, namely,
confining pressure and mortar strength. From these comparisons the following conclusions can be drawn:

1. The shear capacity of the joints for both solid and hollow bricks will definitely rise with an increase in confining pressure acting normal to the joint. The resulting relationship shows a nonlinear tendency, something that is consistent with the Mohr-Coulomb failure envelope. Once cohesion is lost (or the shear stress reduces to the residual shear strength), the confining pressure plays a significant role also for the residual shear capacity.

2. The shear strength does increase with the increase of mortar strength, as the interface cohesion increases with the increase of mortar strength, but a definitive relationship cannot be established at this current stage of knowledge. Strong mortar with weak brick and weak mortar with strong brick will behave differently, and as a result, the shear strength will be very scattered in nature.

3. The shear stress-slip relationship can best be described as pre-peak and post-peak regime. In the pre-peak stage, the stiffness is somehow constant throughout the loading process, and shows little hardening phase near the peak shear stress. So, the pre-peak behavior can be said to be elastoplastic. In the post-peak regime, the damage is rather gradual and shear stress reaches to a constant value after the loss of interface cohesion; this stage is called residual shear strength.

4. Brick type, its surface roughness, and mortar texture definitely have an effect on interface friction. Until and unless some vigorous investigation is carried out to know what these effects are, an explicit correlation between friction coefficient and mortar strength cannot be established in this study. Merely a variation of friction coefficient can be shown with respect to mortar strengths and the strength of bricks.

5. The so-called dilatancy, which causes an upward displacement of the brick units upon sliding, has some marginal effect on the deformation of the brick and mortar themselves are large enough in comparison with the dilatant deformation.

6. The proposed model equations have uniqueness in their simplicity and can predict the peak shear stress as well as the initial stiffness and the softening behavior of the interface quite well for the experimental results examined in this paper. The necessary information for the proposed model is only the Young’s modulus of brick and mortar, the mortar thickness, and the normal compressive stress acting on the interface. Any reasonable value for $c$ and $\phi$ will produce good numerical approximation.

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References
