BEHAVIOR OF MORTAR UNDER MULTIAXIAL STRESS

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Abstract

An understanding of mortar behavior in a multi-axial stress state is important and necessary to determine its influence on the strength, deformation, and failure mode of masonry. When masonry is subjected to vertical loads, mechanical interactions between blocks and mortar at the joints induce lateral tension and compression stresses. This phenomenon may affect the capacity and the failure mode of masonry structures. An experimental and analytical research program, therefore, was designed with the objective to expand the knowledge on the behavior of mortar subjected to triaxial stresses. This article presents the results of that experiment program and an analysis of the mechanical behavior of mortar samples under triaxial tests. In addition to compressive strength values, elastic modulus and Poisson’s ratio values were obtained and are presented. Meaningful differences were observed between the triaxial and uniaxial tests and these differences are also discussed. In addition, the tests results and observed mortar behavior under different levels of confinement are compared to those made by other researchers.

Keywords: confinement, stress-strain diagram, elastic modulus, Poisson’s ratio, masonry

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Introduction

When masonry is submitted to vertical loads, a number of joint phenomena occur between the units and the mortar inducing the compression as well as lateral tension in the system. Experimental tests of blocks under compression show a stress-strain behavior that is essentially linear. However, the mortar under the same conditions of stress and strain has a nonlinear behavior. Such behavior must be considered in any numerical simulation to predict the capacity of masonry strength. The understanding of such nonlinear behavior is important to correctly define the failure criterion of the assembly caused by the crushing of the mortar and it is essential for accurate constitutive modeling of masonry.

Cheema and Klingner (1986) simulate masonry prisms using numerical models to predict the stress level and failure mode of non-grouted stack-bonded prism. The authors correlated the stiffness ratio between mortar and block \( \frac{E_{\text{mortar}}}{E_{\text{block}}} \) under triaxial stress state to the influence coefficient of stress. The goals of that research were to determine the failure mode of masonry and the component that reached its failure stress level first: the block reaching its tensile capacity or the mortar reaching its crushing capacity. To describe the mechanical properties of mortar under triaxial stress states, the authors used a linear failure envelope for concrete under triaxial stress state. The angular coefficient adopted was 4.1, which was very high for typical mortars; i.e., mortars with compressive strength between 2 to 8 MPa. The authors established two curves to describe the failure mechanism of masonry prism. The first curve increased values with increasing \( \frac{E_{\text{mortar}}}{E_{\text{block}}} \) ratio, while the second curve decreased in value with increasing \( \frac{E_{\text{mortar}}}{E_{\text{block}}} \) ratio. The intersection point of the two curves indicated the change in failure mode of the masonry from tensile splitting of the block to crushing of the mortar. The model developed by Cheema and Klingner (1986), however, did not consider experimental tests on mortars done by other researchers; i.e., those by Khoo (1972) and Atkinson (1985), which showed a modification on mechanical proprieties of mortars under a triaxial stress state condition.

Barbosa et al. (2007) presented the Poisson behavior of mortar for four different confinement stresses and the main conclusion was that the configuration of the stress/strength ratio and Poisson’s ratio curve for the higher confining levels were affected by lateral confining stress. Barbosa et al. (2010) presented the results of a combined experimental program and numerical modeling program to evaluate the behavior of ungrouted hollow concrete blocks prisms under uniaxial compression. A continuous model with a smeared approach using a) plane stress, b) plane strain, and c) a three-dimensional condition were considered. Good agreement between experimental and numerical results was obtained for the peak load and the failure mode using the three-dimensional model. However, less agreement was obtained for plane stress and plane strain models.

The nonlinear behavior of masonry was shown by Knutson (1993) through several stress-strain diagrams, reproduced herein in Figure 1, for different combinations of mortar and brick. Knutson (1993) concluded that the stress-strain relationship could be approximated through Equations 1 and 2 where \( \sigma \) is the normal stress, \( \varepsilon \) is the normal strain, \( f_{\text{mas}} \) is the masonry compressive strength and \( E_o \) is the tangent modulus of elasticity.
The study by Knutson (1993) showed that the stress-strain diagrams were significantly different as a result of unit type and mortar, and depended on the stress-strength ratio.

\[
\begin{align*}
\text{if } \sigma/f_{\text{cmas}} & \leq 0.75 \\
\varepsilon & = -f_{\text{cmas}} \frac{\sigma}{E_0} \ln \left(1 - \frac{\sigma}{f_{\text{cmas}}}\right) \tag{1} \\
\text{if } \sigma/f_{\text{cmas}} & > 0.75 \\
\varepsilon & = -4. \frac{f_{\text{cmas}}}{E_0} \left(0.403 - \frac{\sigma}{f_{\text{cmas}}}\right) \tag{2}
\end{align*}
\]

Figure 1. Compressive stress-strain diagram of masonry.

The work presented herein expands the analytical work of Cheema and Klingner (1986) and that of Knutson (1993) as well as the experimental work conducted by Khoo (1972) and Atkinson (1985). The tests conducted evaluated the failure mechanism of masonry prisms with compressive strength of mortar between 2 and 8 MPa, which fails mainly by crushing.

**Experimental tests on compressive strength of masonry prism**

Figure 2 presents the stress (MPa) and horizontal and vertical strain (mm/m) curves for two 3-unit high prisms under compression: one assembled with a strong mortar of 19.8 MPa (Figure 2(a)) and one assembled with a weak mortar of 7.2 MPa (Figure 2(b)). The blocks have the same net area compressive strength of 23.1 MPa. The negative and positive value is for lateral and axial strain, respectively. It is possible to observe that for the prism with strong mortar, when the stress-strength ratio reaches about 60%, there is a sudden increase in lateral strain, which was caused by a vertical crack that developed and propagated through the block. For the prism with weak mortar, there is a gradual rather than a sudden increase in lateral strains. The failure of the prism was due to localized crushing and crumbling of mortar. The crushing was located in the upper mortar joint of the prism at a stress of approximately 50% of the compressive strength. After the crumbling of the mortar, localized spalling was also observed, and vertical cracks began to propagate towards the top and bottom faces of the prism.
From the results, it was possible to conclude that the mortar governed the failure mode of the prisms. For the strong mortar prism, the mortar compressed axially and due to Poisson’s effect expanded laterally. As it expanded laterally, it induced tensile stresses on the block. Since it was a strong mortar, it’s compressive as well as its shear strengths were greater than the tensile strength of the block; thus, the block cracked vertically, and the prism failed. For the weak mortar prism, the failure mechanism was as follows: the mortar compressed and expanded laterally as for the case of a prism with strong mortar. Since it was a weak mortar, however, its compressive strength was lower than that of the block and crushing of the mortar occurred causing localized spalling of the face of the block. As loading increases, vertical random cracks propagated from the mortar to the blocks. Researchers believe that much of this phenomenon happens because of the changes on mechanical properties of the mortar under triaxial stress.

**Triaxial studies of Khoo (1972)**

Khoo (1972) was one of the first researchers who studied the failure criterion of brickwork under multiaxial stress conditions. To describe the failure mode of masonry, Khoo (1972) carried out biaxial test on bricks (tensile and compression) and triaxial compression test on mortar specimens. Two types of mortar mixes (1:0.25:3 and 1:1:6) with water/cement ratios of 0.64 and 1.29 respectively were used with cement, lime and sand proportions designated by volume. Mortar specimens were casted in cylinders 3.8 cm in diameter by 10.2 cm in height. Table 1 presents the triaxial mortar failure envelopes developed by Khoo (1972) for the mortars. The best models to represent the failure envelope of the confined mortars were linear models. For the strong mortar, the angular coefficient was 3.4 and for the weak mortar it was 2.3.

**Table 1. Envelope failure of the mortar obtained by Khoo (1972).**

<table>
<thead>
<tr>
<th>Mortar type</th>
<th>Failure envelope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:0.25:3</td>
<td>$f_m^* = f_m + 3.4 \sigma_3$</td>
</tr>
<tr>
<td>1:1:6</td>
<td>$f_m^* = f_m + 2.3 \sigma_3$</td>
</tr>
</tbody>
</table>

As shown in Figure 3 for the 1:0.25:3 mortar, the tangent modulus of elasticity (or initial elasticity modulus) magnitude increased slightly with increasing lateral stress while for the 1:1:6 mortar it decreased with increasing lateral stress. The results also show that Poisson’s ratio
decreased with increasing lateral pressure: for the mortar type 1:0.25:3 the decrease was linear while for mortar type 1:1:6 the decrease was non-linear.

Figure 3. Tangent modulus of elasticity and Poisson’s ratio under lateral stress – Khoo (1972).

**Triaxial studies of Atkinson et al. (1985)**

Atkinson et al. (1985) performed triaxial tests in four types of mortar with six different levels of confined pressure. Table 2 summarizes the physical and mechanical characteristics of the different mortars and the failure envelope of the samples under triaxial compression. Failure envelopes were also linear with angular coefficients similar to those obtained by Khoo (1972). The behavior of the two sets of specimens was nonlinear, as shown in Figure 4, with a clear distinction between brittle and ductile behavior with increasing confinement. For mortar type 1:0.5:4.5, the stress-strain curves suggest brittle behavior for stress levels of 0.2, 0.7 and 1.7 N/mm² and ductile behavior for stress levels of 3.45, 6.90 and 10.90 N/mm². For mortar type 1:1:6, the stress-strain curves suggest brittle behavior for stress levels of 0.2 and 0.7 N/mm², and ductile behavior for stress level of 1.7 N/mm².

**Table 2. Mechanical characteristics of mortar studied by Atkinson et al. (1985).**

<table>
<thead>
<tr>
<th>Type</th>
<th>Uniaxial Compressive strength (N/mm²)</th>
<th>Water / cement</th>
<th>Lateral stress (N/mm²)</th>
<th>Failure envelope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1/4:3</td>
<td>32.6</td>
<td>0.55</td>
<td>0.21; 0.69; 1.72; 3.44; 6.88; 10.31</td>
<td>$f_m^* = f_m + 5 \sigma_3$</td>
</tr>
<tr>
<td>1:1/2:4.5</td>
<td>26.4</td>
<td>0.85</td>
<td>0.21; 0.69; 1.72; 3.44; 6.88; 10.31</td>
<td>$f_m^* = f_m + 3 \sigma_3$</td>
</tr>
<tr>
<td>1:1:6</td>
<td>13.7</td>
<td>1.19</td>
<td>0.21; 0.69; 1.72; 3.44; 6.88; 10.31</td>
<td>$f_m^* = f_m + 2 \sigma_3$</td>
</tr>
<tr>
<td>1:2:9</td>
<td>3.4</td>
<td>1.96</td>
<td>0.21; 0.69; 1.72; 3.44; 6.88; 10.31</td>
<td>$f_m^* = f_m + 2 \sigma_3$</td>
</tr>
</tbody>
</table>

Besides the common brittle and ductile behaviors, Atkinson et al. (1985) suggested using bilinear behavior, characterized by a continuous increase in strain without a significant increase in stress. The authors classified the behavior of mortar types 1:1:6 and 1:2:9 as being bilinear for high confining pressures. The bilinear behavior may be a result of the collapse of the internal structure of the mortar with rearrangement of the grains. This rearrangement may modify the stable configuration of the materials (cement and sand) causing a progressive failure of the system.
Figure 4. Stress and axial and lateral strain diagram for confined mortar - Atkinson et al. (1985).

Figure 5 presents the tangent modulus of elasticity and Poisson’s ratio of mortars 1:0.5:4.5 and 1:1:6 under triaxial compression. Similar to the results obtained by Khoo (1972), both quantities vary with increasing confinement.

Mohamad (1998) conducted a series of triaxial compression tests on mortar types 1:0.25:3, 1:0.5:4.5, 1:1:6 and 1:2:9 and obtained the failure envelopes presented in Table 3. Failure envelopes were linear with angular coefficients similar to those obtained by other researchers.

Table 3. Envelope failure of the mortar.

<table>
<thead>
<tr>
<th>Mortar – Type</th>
<th>Failure envelope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:0.25:3</td>
<td>( f_m^* = f_m + 4 \sigma_3 )</td>
</tr>
<tr>
<td>1:0.5:4.5</td>
<td>( f_m^* = f_m + 3.6 \sigma_3 )</td>
</tr>
<tr>
<td>1:1:6</td>
<td>( f_m^* = f_m + 2.6 \sigma_3 )</td>
</tr>
<tr>
<td>1:2:9</td>
<td>( f_m^* = f_m + 2.5 \sigma_3 )</td>
</tr>
</tbody>
</table>
From the experimental results, Mohamad (1998) concluded that there is an increase in ultimate strain of the mortar with increasing confinement. Figure 6 presents the Mohr-Coulomb failure envelope of the strong and weak mortar under triaxial compression, while Figure 7 presents the variation of the cohesion with mortar compressive strength. The friction angle appears to be independent of the mortar compressive strength, and the cohesion can be represented reasonably well by a linear function of the mortar compressive strength.

![Mohr-Coulomb failure envelope](image1)

Figure 6. Mohr-Coulomb envelope of mortar.

![Cohesion and compressive strength](image2)

Figure 7. Cohesion and the compressive strength of mortar.

Figure 8 depicted the elastic modulus of the mortars. For mortar type 1:0.25:3 and 1:0.5:4.5, an increase in the tangent modulus of elasticity was observed with the increase in lateral pressure while for mortars types 1:1:6 and 1:2:9, there was a slight decrease in the tangent modulus of elasticity with the increase in lateral pressure.
Table 4 presents the Poisson’s ratio for the mortars at a stress-strength ratio of 30% (initial stress level) and near failure (ultimate stress level). In general, there is a decrease in Poisson’s ratio with increasing lateral stress for all mortars.

Table 4. Poisson ratio of confined mortar.

<table>
<thead>
<tr>
<th>Type</th>
<th>Lateral stress (N/mm²)</th>
<th>Poisson ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial stress level</td>
</tr>
<tr>
<td>1:0.25:3</td>
<td>0, 0.5, 1</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>0.10</td>
</tr>
<tr>
<td>1:0.5:4.5</td>
<td>0</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>0.09</td>
</tr>
<tr>
<td>1:1:6</td>
<td>0</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.07</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.02</td>
</tr>
<tr>
<td>1:2:9</td>
<td>0</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.05</td>
</tr>
</tbody>
</table>
Studies developed by Hayen (2004)

Triaxial tests have also been conducted by Hayen et al. (2004) using specimens from putty lime mortar, hydraulic lime mortar, and lime/cement mortar with compressive all three types with a strength of approximately 1.85 N/mm². The relationship between lateral and vertical stresses (k) changed throughout the tests with k values of 0; 0.05; 0.10; 0.15; 0.25; 0.5; 0.75 and 1. Hayen et al. (2004) assessed the influence of the variables on the pore structure of the mortars by measuring total pore volume using vacuum submersion, mercury intrusion and scanning electron microscopy. The multiaxial stress analysis of Hayen et al. (2004) led to the conclusion that the shear failure mechanism was the main cause of the volumetric strain under triaxial loads for k < 0.25. It was observed an initial decrease in the specimen volume (probably due to the internal collapse of the defects) and followed by an increase volume, where shear bands develop; the collapse of the samples occurs along diagonal shear bands. For k ≥ 0.25, the failure mechanism is rather distinct being characterized by a linear decrease in volume; thus, pore collapse occurs when k ≥ 0.25. The pore collapse phenomenon is a microstructural observation, which indicates no change of the volume of the sample.

A proposed model for Poisson´s ratio of confined mortar

Ottosen (1979) is one of the first researchers that understood the failure process of brittle materials such as concrete and tried to represent this mechanism by the correlation between stress/strength ratio and the Poisson’s behavior. For masonry, however, the failure mechanism is quite different because masonry is an anisotropic material composed of mortar and concrete or clay blocks bonded together by adhesion at the interfaces. Many researchers have tried to study the failure mechanism of masonry by testing blocks, prisms, wallets, and walls. Some of them have developed models to predict the masonry strength by combinations of the compressive strength of the materials. This is a difficult task because there are several variables affecting the masonry failure mode including the tensile strength level of the block and/or the internal voids on the mortar that could induce crushing of the mortar. Herein, the model developed is a modification of the Ottosen’s model and represents only the second failure mode of masonry walls. No researcher so far has represented the failure mechanism of masonry by considering that the triaxial stress state of mortar modifies the internal porosity of the mortar that could produce a decrease in the material’s volume and a “pore” collapse of the mortar.

The use of a constant value for Poisson’s ratio during loading is common, although it does not represent the changes in volume due to confined stress, as observed by Khoo (1972), Atkinson et al. (1985) and Mohamad (1998). The failure mechanisms of masonry under compression depends on the mortar type. For weak mortars, the failure mechanism starts by the initiation and propagation of cracks in the mortar due to the high porosity and different void sizes. There is probably a decrease in volume caused by the collapse of defects and voids, after which Poisson’s ratio increases significantly until failure. The failure mode is slightly different for strong mortars. With the increase on lateral stress, there was not a modification on the Poisson ratio until a stress/strength level of 0.8 was reached. After this, the Poisson ratio increased until failure occurred.
The following constitutive model for representing the observed changes in Poisson’s ratio is proposed. This model is a modification of Ottosen’s (1979) model for high strength concrete. The model gives the values for Poisson’s ratio (x-axis) as a function of the stress/strength ratio, \( \beta \) (y-axis). In this model, Poisson’s ratio remains constant until reaching a threshold value, \( \beta_1 \). After reaching this threshold value, Poisson’s ratio increases until failure. At failure, the stress/strength ratio (\( \beta \)) reaches unity, as shown in Figure 9.


\[
\vartheta^a = \vartheta^a_i \quad \text{if, } \beta \leq \beta_1 \tag{3}
\]

\[
\vartheta^a = \vartheta^a_f - (\vartheta^a_f - \vartheta^a_i) \cdot \sqrt{1 - \left(\frac{\beta - \beta_1}{1 - \beta_1}\right)} \quad \text{if, } \beta > \beta_1 \tag{4}
\]

Due to the relatively large range in mortar qualities, a modification of Ottosen's model is being proposed. This modification is shown in Figure 10. Two cases are proposed to represent the failure mechanism and change of mortar behavior. For mortars of case “a” Poisson’s ratio decreases until reaching \( \beta_1 \) and then gently increases until collapse occurs and shear failure mechanism develops. For mortars of case “b”, Poisson’s ratio decreases like for case “a” until reaching \( \beta_1 \) and then increases suddenly due to pore collapse of voids, cohesive loss between the grains and the closing of cracks. Equations [5] and [6] represent the change in Poisson’s ratio behavior for case “a”, while Equations [7] and [8] represent the behavior for case “b”, as shown in Figure 10.

\[
\vartheta^a = (\vartheta^a_i) \cdot e^{-\beta} \quad \text{if, } \beta \leq \beta_1 \tag{5}
\]

\[
\vartheta^a = \vartheta^a_f - (\vartheta^a_f - \vartheta^a_i) \cdot \sqrt{1 - \left(\frac{\beta - \beta_1}{1 - \beta_1}\right)} \quad \text{if, } \beta > \beta_1 \tag{6}
\]

\[
\vartheta^a = (\vartheta^a_i) \cdot e^{-\beta} \quad \text{if, } \beta \leq \beta_1 \tag{7}
\]

\[
\vartheta^a = (\vartheta^a_i) \cdot e^{\beta} \quad \text{if, } \beta > \beta_1 \tag{8}
\]
Figure 10. Ottosen model modification for case “a” and “b”.

An example of Poisson’s ratio as a function of β using Equations [7] and [8] for a mortar under a 7 MPa confinement and failure mode of the assembly is presented in Figure 11. When β reaches 0.8, an abrupt increase in Poisson's ratio occurs, causing a volume change (Branch 1); the cohesive loss of mortar arises suddenly, increasing tensile stress in masonry unit. Branch 2 is characterized by a slow cohesive and adhesion loss between block and mortar.

Figure 11. a. Example of measured Poisson’s ratio versus stress/strength and the failure mechanism.

The model proposed does not take into account the Poisson’s ratio of the block. In general, masonry prism strength is not sensitive for variations in Poisson’s ratio of the block nor to variation in Poisson’s ratio of the mortar at values lower than the Poisson’s ratio of the block. Vyas and Reddy (2010) explain that when the Poisson’s ratio of the mortar “crosses” the actual value of Poisson’s ratio of the block, the prism strength starts decreasing because the intensity of lateral tensile stresses in the block increases due to an increased Poisson’s ratio of mortar. Currently, experimental tests on mortar under triaxial stress conditions are being conducted to understand its failure mechanism better. In addition, the influence of Poisson’s ratio of the block on the strength and failure mechanism of masonry is being studied. The proposed model is also being refined with the ultimate goal of a more reliable model, capable of predicting the pore collapse of mortar and the strength of masonry.
Conclusions

The main conclusions from the work presented herein are:

The failure envelope of confined mortar is linear and can be expressed by a linear relationship.

For weak mortars, the elastic modulus decreases with an increase in confining stress, while for strong mortars the elastic modulus increases with an increase in confining stress.

There is a decrease in Poisson’s ratio of mortar under triaxial compression. This reduction is apparently exponential for weak mortars and linear for strong mortars.

A model to represent the relationship between Poisson’s ratio and stress-strength ratio is proposed.

This model represents the failure mode of mortar and is the first step toward an acknowledgment of the failure mechanism of stack bonded prisms under compression.

Acknowledge

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