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DEFORMATION AND FAILURE MODE OF MASONRY

Fernando S. Fonseca¹, Gihad Mohamad², Humberto R. Roman³, A.T. Vermeltfoort⁴ and Eduardo Rizzatti⁵

Abstract

Masonry is widely used in many parts of the world and one of the challenges engineers face is how to increase the performance of the masonry wall element. There are many possible ways to increase the wall performance including the use of better materials and the designing and enforcing of a high standard quality control and quality assurance program. These possibilities, however, will most likely increase the cost of construction and eliminate the masonry as a viable construction system. Another possible way to increase the wall performance is to better understand the current system and try to modify current conservative code requirements. Although there is an initial research cost, such possibility appears to be more economically feasible in the end. The research presented herein focused on understanding the mechanical properties of the basic block-mortar set, which is responsible for the wall performance and failure. The main goal was assessing the failure mode and deformation capability of walls, through an extensive experimental program. It was concluded that one of the causes of the nonlinearity of the masonry is an increase in lateral deformation with increasing loading, which was due to extensive cracking of the mortar, a progressive increase in the Poisson’s ratio of the system, and vertical cracks that occurred in the interface block-head mortar joint. The data show that these phenomena happened when the stress on the wall reach approximately 60% of the ultimate strength.

Keywords: structural masonry, masonry deformation, out-of-plane deformation, masonry wall, failure mode

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Introduction

Khoo and Hendry (1975), Hamid and Drysdale (1979a, 1979b), Atkinson and Noland (1983), Atkinson et al. (1985a, 1985b) and Cheema and Klingner (1986) indicate that the development of tension on the concrete masonry unit and that mortar crushing are the two main factors responsible for masonry failure. The development of tension on the block depends on the relative stiffness between the mortar and the unit while the crushing of the mortar depends on the relative strength between the mortar and the unit. The objective of this article is to report on a study conducted to determine the in-plane biaxial deformation and the failure mode of concrete masonry. The article describes the testing, which was conducted on prisms and wallettes, and the results. Two groups of prisms, all three units high, were tested: one group with three full units and one group with two full units and the half units; the half units were used in the second course. The wallettes, all of them 1.0 meter high, were also subdivided into two groups: one group having wallettes that were 1.0 meter wide and the other having wallettes that were 0.8 meter wide.

Prisms Description

Prisms were hollow and three units high, and were constructed as recommended by BS-5628-1 (1992). The first prism group was constructed with full units and the second group was constructed with two full units and two half units in the middle course. The purpose of the second group was to investigate the influence of a head joint in the response of the prisms.

Prisms with only full units had only bed joints and were constructed with three types of mortar: type I (1:0.25:3), type II (1:0.5:4.5), and type III (1:1:6). The water/cement ratio of the mortars varied slightly due to workability requirements. Four prisms were constructed with type I mortar and with type II mortar but seven prisms were constructed with type III mortar because of the expected higher variability of the results. Prisms with two full units and two half units had one head joint in the middle course and were constructed with only two types of mortar: type I and type II. Four prisms were constructed with type I mortar and 6 prisms were constructed with type II mortar. Prisms were assembled using a steel guide to insure that the bed joints had a final thickness of 10 ± 2 mm. Capping of the prisms was done using a cement paste as required by ABNT–NBR 12118 (2011a) instead of the high strength sulfur or gypsum as suggested by ASTM C617/C617M (2011b); the cement paste is used due to health and environment concerns. As required, the paste was made using a high strength cement in order to obtain a strength equal to or greater than that of the units; the cement paste had a final average thickness of 2 mm.

The tests were conducted in a 2000 kN servo-controlled press as shown in Figure 1; testing was displacement controlled with a constant velocity of 0.002 mm/sec. The deformation of the prisms was measured by 4 LVDTs as shown in Figure 2.

The results herein presented and discussed are the unit strength ($f_{bloc}$), mortar strength ($f_{arg}$), prisms strength ($f_{prism}$), average compressive strength ($f_{p,ave}$), efficiency ratio ($f_{p,ave}/f_{bloc}$) and component strength ratio ($f_{arg}/f_{bloc}$). Axial deformation and in-plane lateral deformation are also presented and discussed. Furthermore, the mode of failure of the prisms, which was determined visually from the distribution of cracks, is also discussed.
Prisms Results

Prisms with only Full Units

Compressive strength \( (f_{\text{prism}}) \), average compressive strength \( (f_{p,\text{ave}}) \), and coefficient of variation for the prisms with full units are summarized in Table 1. Average compressive strength of the units \( (f_{\text{bloc}}) \) and mortar \( (f_{\text{arg}}) \) as well as the efficiency ratio and the component strength ratio are also presented. The change in \( f_{p,\text{ave}} \) was relatively small with the change in \( f_{\text{arg}} \). The average strains, obtained from the deformation measured by LVDTs 1a and 1b, are summarized in Table 2 for different level of applied stress \( (\sigma) \); the ratio between applied stress and \( f_{p,\text{ave}} \) is also presented. The values shown prior to the ultimate applied stresses correspond to those of the appearance of the first crack. The data show that there is a slightly increase in average compression deformation of the prisms with a decrease in mortar strength.

### TABLE 1. Strength Results

<table>
<thead>
<tr>
<th>Mortar type</th>
<th>( f_{\text{bloc}} ) (MPa)</th>
<th>( f_{\text{arg}} ) (MPa)</th>
<th>( f_{\text{prism}} ) (MPa)</th>
<th>( f_{p,\text{ave}} ) (MPa)</th>
<th>C.V (%)</th>
<th>Efficiency ratio</th>
<th>Component strength ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>23.1</td>
<td>19.8</td>
<td>15.9</td>
<td>15.1</td>
<td>15.2</td>
<td>7.2</td>
<td>0.86</td>
</tr>
<tr>
<td>II</td>
<td>0.31</td>
<td>7.2</td>
<td>15.1</td>
<td>15.0</td>
<td>14.3</td>
<td>0.62</td>
<td>0.31</td>
</tr>
<tr>
<td>III</td>
<td>0.33</td>
<td>9.55</td>
<td>14.2</td>
<td>15.6</td>
<td>14.3</td>
<td>0.61</td>
<td>0.31</td>
</tr>
</tbody>
</table>

### TABLE 2. Strains Results

<table>
<thead>
<tr>
<th>Mortar type</th>
<th>( f_{p,\text{ave}} ) (MPa)</th>
<th>( \sigma ) (MPa)</th>
<th>( \sigma/f_{p,\text{ave}} )</th>
<th>Strain ( \times 10^{-4} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>15.2</td>
<td>4.75</td>
<td>0.31</td>
<td>2.24</td>
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<tr>
<td>II</td>
<td>14.30</td>
<td>6.38</td>
<td>0.42</td>
<td>3.22</td>
</tr>
<tr>
<td>III</td>
<td>14.20</td>
<td>7.94</td>
<td>0.52</td>
<td>4.27</td>
</tr>
</tbody>
</table>

Figure 3 shows typical stress-strain diagrams for prisms assembled with all three types of mortar. For prisms assembled with type I mortar, lateral deformations were linear up to a ratio of applied stress to \( f_{p,\text{ave}} \) of approximately 0.7. After which there was a sudden increase in
lateral deformations causing, sometimes, a relaxation in the axial deformations. At failure, the ratio between lateral and axial deformations was approximately 5.0. For prisms assembled with type II mortar, the first vertical crack developed at a ratio of applied stress to $f_{p,ave}$ of approximately 0.8. The increase in lateral deformations was progressive rather than sudden as for prisms assembled with type I mortar. For prisms assembled with type III mortar, an increase in lateral deformations was observed at a ratio of applied stress to $f_{p,ave}$ of approximately 0.6. This increase, however, was not caused by the development of vertical cracks but rather by localized crushing of mortar.

Figure 3. Stress-Strain Diagrams

Figure 4 shows the front and back faces of 3 prisms assembled with the three types of mortar after testing. The measured deformations suggest that the mode of failure of the prisms were different. For prisms assembled with type I mortar, cracks were typically through the entire thickness of the prisms, vertical, and scattered; no mortar crushing were observed. At ultimate load, prisms typically broke in two pieces along one of the major vertical cracks. Prisms assembled with type II mortar failed due to a combination of vertical cracks and localized crushing of the mortar. Prisms assembled with type III mortar experienced crushing of the mortar as shown in Figure 5. Typically crushing started at a ratio of applied stress to $f_{p,ave}$ of approximately 0.5 (region a), followed by spalling and/or through the thickness splitting of the face shell (region b), and development of vertical cracks (region c).

Figure 4. Prisms after Testing
Prisms with Full and Half Units

Compressive strength ($f_{\text{prism}}$), average compressive strength ($f_{p,\text{ave}}$), and coefficient of variation for the prisms with full units are summarized in Table 3.

### Table 3. Strength Results

<table>
<thead>
<tr>
<th>Mortar type</th>
<th>I</th>
<th>II</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{\text{bloc}}$ (MPa)</td>
<td>23.10</td>
<td></td>
</tr>
<tr>
<td>$f_{\text{arg}}$ (MPa)</td>
<td>18.20</td>
<td>8.50</td>
</tr>
<tr>
<td>$f_{\text{prism}}$ (MPa)</td>
<td>10.20</td>
<td>7.00</td>
</tr>
<tr>
<td></td>
<td>10.90</td>
<td>9.20</td>
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<td></td>
<td>12.30</td>
<td>9.30</td>
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<tr>
<td></td>
<td>9.50</td>
<td>9.60</td>
</tr>
<tr>
<td></td>
<td>9.90</td>
<td>9.90</td>
</tr>
<tr>
<td></td>
<td>6.50</td>
<td>6.50</td>
</tr>
<tr>
<td>$f_{p,\text{ave}}$ (MPa)</td>
<td>10.70</td>
<td>8.60</td>
</tr>
<tr>
<td>C.V (%)</td>
<td>11.10</td>
<td>16.90</td>
</tr>
<tr>
<td>Efficiency ratio</td>
<td>0.46</td>
<td>0.37</td>
</tr>
<tr>
<td>Component strength ratio</td>
<td>0.79</td>
<td>0.37</td>
</tr>
</tbody>
</table>

Average compressive strength of the units ($f_{\text{bloc}}$) and mortar ($f_{\text{arg}}$) as well as the efficiency ratio and the component strength ratio are also presented. Average compressive strength of prisms with full and half units was lower than that of prisms with only full units; the reduction was 42 and 66 percent for type I and II mortars, respectively. The introduction of the head joint also caused an increase in the coefficient of variation, which is partially explained by the larger variation of the bond strength between the units and the mortar at that location as compared to the variation of the tensile strength of the unit.

Similar to the results for prisms with only full units, the change in $f_{p,\text{ave}}$ was relatively small with the change in $f_{\text{arg}}$. An increase in mortar strength of approximately 215% resulted only in an increase in average compression resistance of approximately 24%. The efficiency of prisms with full and half units was also lower than that of prisms with only full units: 43 and 70% for mortar I and II, respectively.

Figure 6 shows typical stress-strain diagrams for the prisms assembled with both types of mortar.
Figure 6. Typical Axial and Lateral Stress-Strain Diagram

For prisms assembled with type I mortar, the lateral deformation is linear until approximately an applied stress of 6 MPa, when vertical cracks at the head joint started to develop and at the same time vertical cracks started to propagate through the top and/or bottom full units. Prisms assembled with type II mortar experienced a non-linear response at a much lower applied stress. The lower strength mortar had most likely also lower bond strength, which probably caused the vertical cracks along the head joint to develop earlier. However, the lower strength mortar was able to deform more both axially and laterally along the bed delaying the propagation of the vertical cracks through the top and/or bottom units.

Figure 7 shows the front and back faces of two prisms assembled with the two types of mortar after testing. Cracking, for both cases, typically started at the interface of the mortar and unit at the head joint and propagated through the top and bottom full units. Localized crushing of the mortar was also observed causing scattered and erratic cracking throughout the prisms. Localized mortar crushing also caused spalling of the block face to occur. At ultimate load, some of the prisms assembled with type II mortar broke in two pieces along the middle vertical cracks.

(a) Type I  
(b) Type II

Figure 7. Prisms after Testing

Preliminary Wallette

One wallette, 1.0 m by 1.0 m, was constructed and tested with the objective to determine the appropriate dimensions of the specimen and consequently layout of the blocks for the
remaining wallettes tests. Axial and lateral deformations were monitored and the mode of failure determined.

The wallette was assembled with type II (1:0.5:4.5) mortar with a water-cement ratio of 1.07 and had both top and bottom surfaces “capped” with a Portland cement paste that had a final average thickness of 2 mm.

Figure 8 show the overall testing setup. The test was displacement controlled with a constant velocity of 0.0015 mm/s. The deformation of the wallette was monitored with 10 LVDTs as shown in Figure 9. Three LVDTs measured the vertical deformation across 2 bed joints (group 1) and two LVDTs measured the vertical deformation across 4 bed joints (group 2); two LVDTs measured the horizontal deformation along the top and bottom courses (group 3). Two LVDTs measured the deformation across a head joint in the second and fourth courses (group 4) while 1 LVDT measured the horizontal deformations of 2 head joints combined (Group 5).

![Figure 8. Testing Equipment](image)

![Figure 9. LVDTs Setup](image)

The average compressive strength of the blocks and mortar were 23.1 MPa and 8.3 MPa, respectively, and the compressive strength of the wallette was approximately 11.0 MPa. The wallette-block strength (efficiency) ratio was 0.47. When compared with the efficiency of prisms with full units, there is a reduction of approximately 25 percent while when compared with the efficiency of prisms with half and full units, there is a gain of approximately 27 percent. The mortar-block strength ratio was 0.36, which is similar, as expected, to that calculated for the prisms.

Figure 10 shows the axial and lateral strains, which were calculated from the LVDT measurements.

Strains at the top and bottom courses are smaller than the strains at other courses because of the restraint caused by the top and bottom loading plates. Vertical cracks started to develop in the middle part of the wallette at a stress-strength ratio of approximately 0.6, as indicated by the discontinuity in the lateral strains in group 5 at that stress-strain level. The cracks started along the head joint and propagated through the blocks and/or along the bed joint at a stress of approximately 8 MPa. Axial strains became distinct at a stress-strength level of approximately 0.3, which happened probably due to cracking along the bed joints.
Figure 10. Axial and Lateral Stress-Strain Diagram

Figure 11 shows the front side of the wallette at the end of the test. Four types of cracks were observed. Type 1 cracks were caused by crushing of the mortar along the bed joint and consequent opening of the head joint; typically they developed at a stress-strength ratio of approximately 0.60. Type 2 cracks were due to tension failure of the block, were propagation of type 1 cracks, and typically occurred at a stress-strength ratio between 0.80 and 0.90. Type 3 cracks ran along the bed joint, were due to crushing of the mortar, and typically occurred simultaneously with type 2 cracks. Type 4 cracks were also along the bed joints and were due to bond breaking between the units and mortar. Type 4 cracks were observed on both faces of the wallette, thus eliminating the possibility of out-of-plane bending of the wallette. It is plausible that the blocks experienced out-of-plane expansion causing a slight bulging of the face shells of the wallette; such out-of-plane bulging would cause a slight separation between the face shell and bed joint mortar.

Figure 11. Wallette at the End of Testing

Typically, cracking started in the middle part of the wallette and propagated vertically along the head joint but failure was due to crushing of the bed joint followed by development and propagation of vertical cracks through the blocks. The capping material may have influenced the cracking pattern, the types of cracks, and even the capacity of the wallette. To obtain true uniaxial compression, the capping and wallette (as well as prisms) must have equal Poisson's ratios and there must be a frictionless platen-wallette interface. Interlocking at the capping-wallette interface restricts strain deformation of the material with the higher rate of expansion.
The differing rate of lateral strain between the two materials induces stresses at the wallette ends producing a tri-axial stress state. When the lateral strain of the wallette is greater than the lateral strain of the capping, the ends become confined under compressive stresses. If the capping strain is greater than the wallette strain, the ends are subjected to lateral tensile stresses. Although it is believed that the influence of the capping material used herein is small, further research is needed to determine and quantify such an influence. It became apparent from this testing (a) the importance of biaxial symmetry of the wallette, (b) block-bed joint behaved like a unit despite the different compressive strength of the individual materials, and (c) the weak point is the head joint.

**Wallettes**

Six wallettes, 0.8 m wide and 1.0 m high were constructed and tested with the purpose to determine their compressive strength, axial and lateral deformation capacity, and mode of failure. The materials, construction, testing setup were exactly as those used for the preliminary wallette. The only physical difference between this set of wallettes and the preliminary wallette was their width. The testing setup and procedures were also the same except that testing was conducted with a constant load platen displacement of 0.003 mm/s. The deformation of the wallettes was also monitored slightly differently as shown in Figure 12.

![Figure 12. LVDTs Setup](image)

Table 4 summarizes the results of the testing. The coefficient of variation is approximately 12%. The characteristic resistance of the wallettes, \( f_{pk} \), was obtained for a confidence level of 95%. Essentially, there is no difference between the results for the wallettes and those for the preliminary wallette. Figure 13 shows the axial and lateral strains for wallette P1. The lateral strains at the middle of the wallette were almost twice as those at the top and bottom and were linear until a stress of approximately 8 MPa, after which a significant increase in lateral strains is observed, especially at location 2. The different lateral strain slopes indicate that the crack at the head joint in the middle of the wall is opening faster than those in the top and bottom head joints. Axial strains at location 4 and 5 were essentially the same until a stress of approximately 4 MPa, after which the strain at location 4 increased progressively more than that at location 5. Most likely, the vertical crack that started in the middle head joint, decreased the stiffness of the wallette around the middle of the wallette.
The response of wallette P3 was very similar to that of Wallette P1. The differences were the compressive strength, as shown in Table 4; the lateral strains, which were linear until a stress of approximately 6 instead of 8 MPa; and the axial strains, which were linear until a stress of approximately 2 instead of 4 MPa. The deformation of wallette P4 was also similar to that of wallette P1. The differences were the compressive strength and the axial strains, which were essentially the same until a stress of approximately 6 instead of 4 MPa. In addition, the axial strains at location 5 were linear up to near failure. The response of wallette P5 was very similar to that of wallette P4 with the following differences: the compressive strength and the axial strains, which were essentially the same until a stress of approximately 2 instead of 6 MPa. The response of wallette P6 was similar to that of wallette P3 except that the lateral strains at location 1 experienced a significant increase at approximately 4 MPa due to the development of a vertical crack at the head joint at that location. Another small difference were the axial strains, which were essentially the same until a stress of approximately 2 instead of 6 MPa. The response of wallette P7 was similar to that of wallette P1. The differences were the compressive strength; the lateral strains, which were linear until a stress of approximately 4 instead of 8 MPa; and the axial strains, which were linear until a stress of approximately 2 instead of 4 MPa. Figure 14 shows the wallets after testing. Typically, cracks started at the head joint in the middle of the wallette and propagated through the blocks toward the top and
bottom surfaces of the wallets; failure was due to these vertical cracks. The vertical cracks initiated at a stress-strength ratio of approximately 60%. In some locations, crushing of the mortar was observed. It appears that some wallets rotated about a longitudinal axis through the mid height of the wallette causing, in certain points, bond detachment of the bed joint mortar in one side and crushing of the mortar and unit on the opposite side. There were scattered cracks at some of the bed joints and at the head joints along the top and bottom of some of the wallets. Wallettes P1 and P7 experienced large cracks at the edges causing some of the units to break off. Also, a large portion of one of the faces in the middle of wallette P1 broke off suddenly. The failure of wallets P1 and P7 was brittle while that of the other wallets was ductile.

Figure 14. Wallettes after Testing

Conclusions

Specimens should be biaxially symmetric and capped because surface irregularities cause cracks in the head joints of the top and bottom courses. The capping material, however, influences the cracking pattern, the types of cracks, and even the capacity of the specimens. The capping, therefore, should have a Poisson’s ratio as close as that of the specimen.

Lateral strains in the middle of the wallets were 3 to 4 times greater than those at the top and bottom. Thus, top and bottom loading plates restrained the lateral deformation of the specimen; in other words, there was friction between the loading plates and the specimens. Such restraining may cause either vertical or horizontal scattered cracks to develop at the top and bottom surfaces.

The onset of masonry nonlinearity corresponds to an increase of lateral deformations due to extensive cracking and possibly a gradual increase in Poisson’s ratio, which needs to be further investigated. Vertical cracks typically start in the head joints in the middle of the specimens at a stress-strength ratio of 0.6, for this type of masonry.

Crushing of the mortar is possible either along both faces of the specimen or along just one of the faces; if it happens, it may cause a small rotation of the specimen along a longitudinal axis. The rotation may then cause further crushing of the mortar and localized face shell spalling along the same face and bond failure along the opposite face.

Strong mortar typically caused the prisms and wallets to have a brittle failure while weak mortar typically caused the prisms and wallets to have a ductile failure.
The compressive strength of prisms with full units was significantly different from that of prisms with half and full units and that of walllettes. However, the compressive strength of prisms with half and full units was somewhat similar to that of the walllettes. Thus, prisms with half and full units, and consequently a head joint in the middle, better represent, in terms of compressive strength values, the response of the masonry.

For type II mortar, the prisms with full units had an efficiency of 0.62 while the prisms with half and full units had an efficiency of 0.37; the efficiency of the walllettes was 0.47. The reduction in efficiency for the walllettes and prisms with half and full units was significant. Such reduction was most likely due to the fact that the webs in the walllettes and prisms with half and full units did not lineup vertically as they did in the prisms with full units.

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References


