TORSION SHEAR TEST FOR MORTAR JOINTS IN MASONRY:
PRELIMINARY EXPERIMENTAL RESULTS

Mark J. Masia¹, Yan Han², Christie J. Player³, Marcio R.S. Correa⁴ and Adrian W. Page⁵

Abstract

The paper describes a new experimental testing procedure for characterising the shear behaviour of mortar joints under combined shear and compression loading. The test apparatus subjects a single joint specimen of annular circular cross section to normal compressive force combined with torsion. The choice of annular cross section results in predictable distributions of normal and shear stresses across the mortar joint, allowing shear behaviour at a material point to be characterised. The latter has been a widely reported shortcoming of existing mortar joint shear tests which all result, to varying degrees, in complex, non-uniform distributions of shear and normal stresses across the mortar joint. The paper describes the test methodology and details the results of a preliminary series of tests conducted using the apparatus.

Introduction

The shear behaviour of mortar joints in unreinforced brick and block masonry walls plays a crucial role in the load resisting systems of such walls under both in-plane and out-of-plane horizontal loads. It is essential that this behaviour be well understood to be able to predict the capacity of, and design, masonry walls.

¹ Senior Lecturer, Centre for Infrastructure Performance and Reliability, School of Engineering, The University of Newcastle, Callaghan, NSW 2308, Australia
² Postgraduate Student, Centre for Infrastructure Performance and Reliability, School of Engineering, The University of Newcastle, Callaghan, NSW 2308, Australia
³ Honours Student, Centre for Infrastructure Performance and Reliability, School of Engineering, The University of Newcastle, Callaghan, NSW 2308, Australia
⁴ Associate Professor, Department of Structural Engineering, EESC/USP, Brazil
⁵ Emeritus Professor, Centre for Infrastructure Performance and Reliability, School of Engineering, The University of Newcastle, Callaghan, NSW 2308, Australia
Commonly, when mortar bed joints are subjected to shear force, they will simultaneously be subjected to normal compressive force due to gravity loads. Riddington and Ghazali (1990) suggest that for relatively small levels of normal compressive stress ($< 2.0 \text{ N/mm}^2$) across the mortar bed joints, shear failure is initiated by joint slip and at higher levels of compressive stress shear failure is initiated by tensile failure within the mortar. The present study will focus on the former case (compressive stress $< 2.0 \text{ N/mm}^2$), for which the behaviour under shear loading can be approximated by a Coulomb relationship (Riddington and Ghazali 1990):

$$
\tau_u = \tau_o + \mu \sigma_n
$$

[1]

where $\tau_u$ is the shear strength ($\text{N/mm}^2$) of the mortar joint when subjected to a normal compressive stress of $\sigma_n$, $\tau_o$ is the bond shear strength or cohesion (that is, the shear strength for zero normal pre-compression) and $\mu$ is the coefficient of internal friction. For the prediction of wall strengths, $\tau_u$ is sought and $\sigma_n$ can be calculated from estimates of the gravity loads. Therefore, the values of $\tau_o$ and $\mu$ are required. These are properties of the particular masonry used, that is, they depend on the type of masonry unit (brick or block) and the mortar used to bind them, as well as other factors such as workmanship. In addition to the values of $\tau_o$ and $\mu$ further parameters are required to characterise the shear behaviour of mortar joints for analyses using finite element micro-modeling approaches. This additional information includes the shear stress versus shear displacement response at a point on the mortar joint both before and after joint cracking, and from this, the shear stiffness, the shear fracture energy (which increases with the level of pre-compressive stress), the post cracking coefficient of friction (which may be different from $\mu$) and the dilatancy (normal displacement during shearing) of the joints (Lourenço 1998). Further to this, mortar joint behaviour under cyclic shear loading is of interest for the study of the response of structures to earthquake effects (Atkinson et al. 1989).

Over the past several decades numerous experimental test methods have been developed to attempt to measure some or all of these parameters. The test methods typically use either a masonry couplet specimen (two units bonded together by mortar) (Van Der Pluijm 1993, Stöckl et al. 1990) or a triplet specimen (3 units) (European Standard 2002). The mortar joint(s) in the specimen are subjected to normal force, which is held constant during testing and then direct shear force is applied to the joint(s) until failure occurs. By using various levels of normal compression, the Coulomb relationship described by Equation [1] can be established and hence $\tau_o$ and $\mu$ determined. For these tests, the normal and shear stresses are assumed uniform over the tested mortar joints. Comprehensive reviews of the existing methods have been presented by various researchers (Stöckl et al. 1990, Jukes and Riddington 1997, Riddington et al. 1997). Using finite element analyses, these researchers have been able to show that the assumptions of uniform stress distributions are often far from true. In fact, the distributions of shear and normal stresses are complex and non-uniform. Joint failure usually initiates at a point at a shear stress higher than the average value calculated from the failure load. Strength values based on average stresses will therefore represent an underestimate of the true local joint shear strength. In some of the tests, tensile normal stresses develop across the mortar joint over some of its length leading to premature failure. In addition, Caballero Gonzalez and Schubert (1995) showed that different test methods can yield significantly different values of mortar joint shear strength for the same unit
mortar combination. The result is that while the tests are useful for comparisons between different combinations of brick and mortar, they do not accurately define the conditions under which shear failure occurs at a point in the material. In many cases they also fail to provide the additional parameters described above required for detailed finite element analyses of shear behaviour.

In an earlier study (Masia et al. 2006) the authors described a new experimental testing procedure for assessing the shear behaviour of mortar joints under combined shear and compression loading. The test apparatus subjects a single joint specimen of annular circular cross section to normal compressive force combined with torsion. The approach aims to address the shortcomings in existing mortar joint shear test methods and provide pre and post cracking data to allow the characterisation of shear behaviour at a material point. It is not expected that the proposed test will be suitable for routine testing due to its complexity and it is not suitable for perforated or frogged units. The latter limitation also applies to any existing shear test methods if data at a material point is required. However, it is hoped that it will allow accurate determination of the parameters required for finite element micro modelling of mortar joint behaviour under shear loading. The current paper describes the test apparatus and methodology and presents results from a preliminary experimental program.

Experimental Methodology

Lourenço (1998) defined the experimental data necessary for modelling masonry behaviour at the micro level: displacement controlled tests under tension, compression and shear actions, capable of capturing the entire load displacement response. The following section describes the torsion shear test methodology and details the parameters recorded in the test.

Test Apparatus

The apparatus is depicted schematically in Figure 1a and is shown in the test frame in Figure 1b. The masonry test specimens were circular (annular) cylindrical in shape and contained a single mortar bed joint (Figure 2). The apparatus was used to subject each specimen to compressive force normal to the bed joint (held constant) combined with increasing rotation (and hence torque) about the longitudinal axis of the specimen. The test procedure is discussed later.

Specimen Preparation

The apparatus was designed to test mortar joints in typical brick masonry in Australia; units 230 mm long x 76 mm high x 110 mm thick bonded with 10 mm thick mortar joints. Therefore, the overall length of the cylindrical specimens was 162 mm (76 + 10 + 76). The annular specimens reported herein had inner radius $r_i = 38$ mm and outer radius $r_o = 47.5$ mm.

For the preliminary testing reported in this paper, the specimens were fabricated by first coring annular circular cylinders from solid masonry units. The cylinders were cored vertically through the units from bedding face to bedding face and then bonded on the bedding faces
with a 10 mm thick mortar joint. After casting the joint the specimens were placed in a semi-circular steel channel to help align the upper and lower cored cylinders during curing. It is expected that it will be possible also to first cast a couplet specimen and then core the complete specimen through brick/joint/brick. This latter approach is preferred because it will allow the mortar joint to set and cure in a couplet environment prior to coring the annular specimen, thus resulting in a bond which is more representative of a mortar joint in a wall.

For the current test program, two series of tests were conducted. The specimens in Series 1 (8 specimens tested) were constructed using extruded solid clay units and 1:1:6 (Portland cement : lime : sand) mortar with a deliberate overdose of air entraining agent. This masonry was developed for a separate study where very low flexural bond strength was desired. The specimens in Series 2 (11 specimens tested) were constructed using the same brick type, but from a different production run, and 1:1:6 mortar with no further additives. Bond wrench results for each series were obtained in accordance with Australian Standard AS3700-2001 (Standards Australia 2001).

![Diagram of test apparatus]

(a) Figure 1. Test apparatus

![Test Specimen]

(a) Figure 2. Test Specimen

**Torsion Shear Test Procedure**

The apparatus was mounted within a Universal Testing Machine. Each single joint specimen was epoxy glued between two end plates. One end plate was fully fixed. The other end plate
was attached to a shaft which is free to rotate about, and displace along, the longitudinal axis of the specimen. The apparatus contains bearings to minimise any resistance to both of these degrees of freedom. The specimens were glued in situ into the apparatus to avoid any disturbance and/or residual stresses which may arise from gluing to the end plates first and then bolting into the apparatus.

Each specimen was first subjected to a compressive force normal to the mortar joint. This was applied using a hydraulic jack as shown in Figure 1. A load cell was positioned between the jack and the rotating shaft and the hydraulic pressure was automatically controlled during testing so that the compressive force remained constant at all stages (load control). It is assumed that the resulting normal stress was uniform over the cross section of the mortar joint. Dilation of the joint was not prevented.

Relative rotation (twisting) of the specimen was then introduced by displacing, at a constant rate, the cross head of the testing machine which contacted, at a point, a lever fixed to the rotating shaft of the apparatus (Figure 1b). The apparatus is very stiff and so controlling the machine cross head displacement is assumed to result in very close to a constant rate of increase of joint rotation. The rotation was increased until the mortar joint was cracked and completely softened as evidenced by an approximately constant (frictional sliding) value of the measured torque $T$. The displacement was applied very slowly resulting in quasi-static conditions.

Through all stages of loading the following information was continuously logged: normal compressive force $F_n$, machine cross head displacement and applied force $P$ (torque $T = P \times$ lever length), shearing displacement across the mortar joint (Figure 2a), from which relative rotation $\Phi$ was calculated, and the normal displacement (dilation) across the mortar joint.

**Derivation of Parameters Defining Shear Behaviour at a Material Point**

The procedure described above allows the generation of torque $T$ versus rotation $\Phi$ plots for various levels of normal compressive force. Characterisation of the material behaviour requires the shear stress $\tau$ versus shear displacement $\Delta$ response at a material point. Therefore, it is necessary to understand the relationships between the torque and shear stress and rotation and shear displacement at all stages of loading.

Under displacement controlled conditions the post peak shear behaviour at a material point has been observed to exhibit a softening response (Van Der Pluijm 1993) as idealised in Figure 3. When a shaft with a circular cross section (solid or annular) is subjected to torsion about its longitudinal axis, shear strain over the cross section varies linearly with the radius from the axis of the shaft. For the linear elastic stage of behaviour this results in a distribution of shear stress over the cross section which also varies linearly with the radius (Figure 4a) (Craig 2000). In Figure 4, $\tau$ is the shear stress (which at all points acts normal to the radial line passing through the point), $r$ is the radial distance from the axis of the shaft and $r_i$ and $r_o$ are the inner and outer radii respectively of an annular section. Under increasing rotation the material at the extreme outer fibres will reach its ultimate strength $\tau_u$ and begin to soften leading to the stress distribution depicted in Figure 4b. In the presence of normal
compressive stress the shear stress eventually softens to an approximately constant value equal to the residual coefficient of friction multiplied by the normal stress (Equation [2]).

\[ \tau_{\text{residual}} = \mu \sigma_n \]  \[2\]

The residual coefficient of friction \( \mu_r \) is not necessarily equal to the internal coefficient of friction \( \mu \). Once this frictional sliding stage has been established, the shear resistance is no longer a function of the shear displacement and the shear stress distribution will be approximately uniform with radius (Figure 4c).

**Figure 3.** Shear stress versus shear displacement at a material point

These various stages of material behaviour result in a complex relationship between shear stress and torque. However, if the thickness \( t = r_0 - r_i \) of an annular cross section is small compared to the mean radius \( r_m = (r_0 + r_i)/2 \) then it is usual to assume that the shear stress is approximately constant across the thickness \( t \) during twisting (Figure 4d). In other words the shear stress is assumed constant with radius and hence constant (uniform) over the complete cross section. This assumption is adopted for linear elastic and non-linear behaviour and greatly simplifies the relationship between shear stress and torque which, for all stages of behaviour, can be expressed as:

\[ r = \frac{3T}{2\pi(r_0^3 - r_i^3)} \quad (r \text{ approx. uniform for } t \ll r_m) \]  \[3\]

The relationship between relative rotation and shear displacement across the joint at any radius \( r \) is given by Equation [4]. For the current study, the mean radius \( r_m \) was used in Equation [4].

\[ \Delta = r \Phi \]  \[4\]

973
Equations [3] and [4] allow the shear stress versus shear displacement relationship at a material point to be derived directly from the experimentally recorded torque versus rotation response. Therefore, assuming that the normal stress due to any pre-compression force is also uniform over the cross section, the desired testing conditions of uniform normal and shear stress distributions can be achieved.

There clearly is a limit to how thin an annular masonry joint specimen can be produced in practice. It is therefore necessary to assess the value of \( r_m / t \) above which the assumption of uniform shear stress is reasonable. Using analytical comparisons and a detailed finite element study (Masia et al. 2006) the authors were able to show that the assumption of uniform shear and normal stresses is reasonable for peak and post-peak behaviour for annular specimens with \( r_s = 35 \text{ mm} \) and \( r_o = 50 \text{ mm} \) \( (r_m / t = 2.83) \). Prior to joint cracking, the latter study recommended the assumption of a linear distribution of shear stress. However, for stresses calculated at the mean radius \( r_m \) in an annular cross section, the uniform stress assumption results in close agreement with a linear shear assumption. Therefore, for the current experimental program \( (r_s = 36 \text{ mm}, r_o = 47.5 \text{ mm}, r_m / t = 3.63) \), Equations [3] and [4] were applied directly to derive plots of shear stress \( r \) versus shear displacement \( \Delta \) at a material point through all stages of loading.

The normal compressive stress \( \sigma_n \) was assumed uniform and equal to the normal force \( F_n \) divided by the cross sectional area \( \pi (r_o^2 - r_s^2) \). From the \( r \) versus \( \Delta \) plots the Elastic Shear Modulus \( G \) and ultimate shear strength \( r_u \) for various levels of normal stress \( \sigma_n \) were obtained. From this, the Coulomb relationship described by Equation [1] was established and hence \( r_o \) and \( \mu \) determined for each of the two test series. The residual shear stress \( r_{\text{residual}} \) was also plotted against normal stress \( \sigma_n \) for each test series allowing the residual coefficients of friction \( \mu_r \) to be determined from Equation [2]. The shear fracture energy \( G_f'' \) is defined by the shaded area in Figure 3 and is expected to vary with the level of pre-compression (Van Der Pluijim et al. 2000). For the current tests \( G_f'' \) was determined numerically for each specimen by summing the areas of thin slices beneath the experimentally derived \( r \) versus \( \Delta \) plots. For tests with zero normal compression joint softening occurred rapidly and could not be recorded using the current data logging system. Therefore, values of \( G_f'' \) are not reported for these cases.

Finally, the dilation behaviour of the joint during shearing is defined by the dilation angle \( \Psi \) of which the tangent is equal to the ratio between the normal and shear displacements occurring beyond the peak shear stress. Equation [5] defines the dilation angle which has been observed to reduce with increased normal stress and increased shearing displacement (Van Der Pluijim et al. 2000).

\[
\tan \Psi = \frac{\partial u_p}{\partial \Delta_p} \quad [5]
\]

where \( u_p \) and \( \Delta_p \) are respectively the plastic normal and plastic shear displacements occurring across the mortar joint after the peak shear stress \( r_u \) is reached. For the current tests, dilation was measured by positioning displacement potentiometers against the back face of the moving end plate of the testing apparatus. This approach relied on the assumption that, due to constant normal compression, the elastic compression of the brick would remain
approximately constant during testing, and the potentiometers would record only the normal displacement occurring across the mortar joint during shearing. Although the approach yielded qualitatively sensible results, large variability was observed and so the full results are not reported here. Future testing will require the normal displacement across the mortar joint to be measured directly at the joint itself.

Experimental Results and Discussion

Figures 5 and 6 show plots recorded and/or derived for a typical test. The inset in Figure 5 shows the loading branch of the response in more detail. The plots clearly show the various stages of behaviour idealised in Figure 3. For each specimen the parameters defining the shear behaviour were determined as described above and are summarized in Table 1.

Figure 5. Typical recorded torque versus rotation response

Figure 6. (a) Typical derived shear stress versus shear displacement response, (b) Typical recorded dilation behaviour.
### Test Series 1

Units: extruded solid clay, Mortar: 1:1:6 + overdose (8 x recommended) air entrainer  
Mean flexural bond strength = 0.14 MPa, COV 26%

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$\sigma_t$ (MPa)</th>
<th>$\tau_u$ (MPa)</th>
<th>$\tau_{residual}$ (MPa)</th>
<th>$G_f'$ (N/mm)</th>
<th>$G$ (MPa)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.66</td>
<td>NA</td>
<td>0.49</td>
<td>NA</td>
<td>NA</td>
<td>Joint cracked prior to test</td>
</tr>
<tr>
<td>2</td>
<td>0.68</td>
<td>1.03</td>
<td>0.40</td>
<td>0.45</td>
<td>1860</td>
<td>Joint</td>
</tr>
<tr>
<td>3</td>
<td>0.68</td>
<td>0.61</td>
<td>NA</td>
<td>NA</td>
<td>730</td>
<td>Joint crushing after peak $T$</td>
</tr>
<tr>
<td>4</td>
<td>0.34</td>
<td>0.49</td>
<td>0.29</td>
<td>0.11</td>
<td>1700</td>
<td>Joint</td>
</tr>
<tr>
<td>5</td>
<td>0.34</td>
<td>1.30</td>
<td>0.32</td>
<td>0.24</td>
<td>5500</td>
<td>Joint</td>
</tr>
<tr>
<td>6</td>
<td>0.14</td>
<td>0.34</td>
<td>0.13</td>
<td>0.06</td>
<td>1800</td>
<td>Joint</td>
</tr>
<tr>
<td>7</td>
<td>0.14</td>
<td>0.31</td>
<td>0.14</td>
<td>0.03</td>
<td>5860</td>
<td>Joint</td>
</tr>
<tr>
<td>8</td>
<td>0.33</td>
<td>0.60</td>
<td>0.31</td>
<td>0.05</td>
<td>350</td>
<td>Joint</td>
</tr>
</tbody>
</table>

Cohesion $\tau_c = 0.22$ MPa, Internal friction $\mu = 0.90$, Residual friction $\mu_r = 0.56$

### Test Series 2 (specimen 8 was unsuccessful)

Units: extruded solid clay, Mortar: 1:1:6  
Mean flexural bond strength = 1.74 MPa, COV 11%

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$\sigma_t$ (MPa)</th>
<th>$\tau_u$ (MPa)</th>
<th>$\tau_{residual}$ (MPa)</th>
<th>$G_f''$ (N/mm)</th>
<th>$G$ (MPa)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>1.28</td>
<td>0</td>
<td>-</td>
<td>1540</td>
<td>Joint</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0.72</td>
<td>0</td>
<td>-</td>
<td>1500</td>
<td>Joint</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>0.85</td>
<td>0</td>
<td>-</td>
<td>2470</td>
<td>Joint</td>
</tr>
<tr>
<td>4</td>
<td>0.69</td>
<td>1.46</td>
<td>0.55</td>
<td>0.60</td>
<td>1180</td>
<td>Joint</td>
</tr>
<tr>
<td>5</td>
<td>0.67</td>
<td>2.44</td>
<td>0.58</td>
<td>0.38</td>
<td>38860</td>
<td>Joint and brick</td>
</tr>
<tr>
<td>6</td>
<td>0.67</td>
<td>1.63</td>
<td>0.75</td>
<td>0.19</td>
<td>1360</td>
<td>Joint</td>
</tr>
<tr>
<td>7</td>
<td>1.33</td>
<td>1.59</td>
<td>1.21</td>
<td>0.07</td>
<td>6450</td>
<td>Joint and brick</td>
</tr>
<tr>
<td>8</td>
<td>1.34</td>
<td>2.04</td>
<td>0.70</td>
<td>0.92</td>
<td>48610</td>
<td>Joint and brick</td>
</tr>
<tr>
<td>9</td>
<td>1.34</td>
<td>2.05</td>
<td>1.38</td>
<td>0.25</td>
<td>5310</td>
<td>Joint</td>
</tr>
<tr>
<td>10</td>
<td>0.68</td>
<td>2.06</td>
<td>0.81</td>
<td>0.31</td>
<td>2140</td>
<td>Joint</td>
</tr>
</tbody>
</table>

Cohesion $\tau_c = 1.00$ MPa, Internal friction $\mu = 0.90$, Residual friction $\mu_r = 1.03$

### Table 1. Experimental results

The inset in Figure 5 indicates that the stiffness of the mortar joints typically began to reduce prior to reaching peak torque. However, cracking was usually not visible until peak torque was reached. The torque versus rotation response then softened until a plateau of residual frictional sliding was established. For the majority of specimens, failure was confined to the mortar joint with cracks developing along the brick mortar interfaces (Figure 2) and in several cases, diagonal cracks extending across the mortar joint were also observed (Figure 7a). For a small number of specimens cracking also occurred in the bricks on one or both sides of the joint (Figure 7b). Brick cracking was observed only for the Series 2 masonry which displayed very high flexural bond strength and cracking usually initiated at a location where the brick displayed fine cracks from manufacturing prior to testing. For one of the specimens, the normal compressive force suddenly increased during the post-peak stages of the test due to a hydraulic pump malfunction resulting in crushing of the joint. The failure mode for each specimen is recorded in Table 1. There were some instances of epoxy failure at the specimen ends due to insufficient time being allowed for the epoxy to gain strength. In these cases, it was possible to re-glue the specimen and resume testing once the epoxy had cured.
Figure 7. Failure modes (a) diagonal cracking across mortar joint, (b) brick cracking

Figures 8a and 9a show the ultimate $\tau_u$ and residual $\tau_{\text{residual}}$ shear stresses plotted against normal compressive stress $\sigma_n$ for the Series 1 and Series 2 masonry respectively. In Figure 8a, the unusually large value of $\tau_u$ for specimen 5 has been excluded. In Figure 9a all results in which brick failures were observed have been excluded. Cracking through the brick as shown in Figure 7b affects the distribution of shear stress such that Equation [3] can not be reliably used to estimate the shear stress from the torque. This comment also applies to the shear stress and fracture energy data presented in Table 1 for these specimens.

The linear regression lines fitted to the $\tau_u$ data show that even for the very small number of tests conducted, a sensible Coulomb relationship (Equation [1]) can be established for both the Series 1 and Series 2 data. From the relationship for each test series the cohesion $\tau_n$ and coefficient of internal friction $\mu$ values can be established as the $y$-axis intercept and slope respectively (Table 1). The regression lines fitted to the $\tau_{\text{residual}}$ data yield the residual coefficient of friction $\mu_i$ values (Table 1) as the slopes of the lines. Ideally the plot of $\tau_{\text{residual}}$ versus $\sigma_n$ should pass through the origin since once the joint is fully cracked there should be zero resistance at zero normal stress. The small non-zero values of $y$-axis intercept for each data series are thought to result from the very small number of specimens tested.

Figure 8. (a) Shear stresses and (b) Shear fracture energy versus normal compression for Series 1 data
Figures 8b and 9b show shear fracture energy $G^{f}$ plotted against normal compressive stress $\sigma_n$ for the Series 1 and Series 2 masonry respectively. For the Series 1 data, the expected increase in fracture energy with normal compressive stress is observed. However, there is considerable variability in the data and the negative y-axis intercept is not physically meaningful. For Series 2, no sensible relationship between the shear fracture energy and the normal stress can be established. It is clear that a greater number of tests will be required to better establish this relationship.

Lastly, the Elastic Shear Modulus $G$ was determined for each test by computing the slope of the loading branch of the $\tau$ versus $\Delta$ response using a secant drawn between 5% and 33% of $\tau_u$. Note that the shear displacement values were divided by the joint thickness (10 mm) to obtain values of shear strain. For the Series 2 data, the values of $G$ for a given normal compressive stress are quite consistent but for Series 1, the values show considerable scatter. It is the view of the authors that variations in the mortar joint thicknesses between specimens as well as insufficient accuracy in the measurement of the very small shear displacements occurring in the early stages of loading may be the cause for this scatter.

**Conclusion**

A new experimental testing procedure for characterising the shear behaviour of mortar joints under combined shear and compression loading was presented. The testing procedure was described and the results from a preliminary series of tests using two different brick and mortar combinations were presented. Despite the very small number of specimens tested, it was demonstrated that the procedure enables the shear stress versus shear displacement response at a material point to be fully described.
Further work will focus on: (i) more tests using a larger number of repeat specimens, (ii) refinement in the measurement of the dilation response, and (iii) comparison with the results of the triplet shear test specified by European Standard EN1052-3 (2002).

References


979