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TORSION SHEAR TEST FOR MORTAR JOINTS IN MASONRY: SPECIMEN PREPARATION AND EXPERIMENTAL RESULTS

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ABSTRACT

The paper describes an 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 approximately uniform distributions of normal and shear stresses across the mortar joint, allowing shear behaviour at a material point to be characterised. The paper describes the test methodology and results from five series of tests conducted using the apparatus. The paper also elaborates on the difficult issue of specimen preparation.

KEYWORDS: masonry, mortar joint, shear behaviour, test method

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.

Commonly, when mortar bed joints are subjected to shear force, they will simultaneously be subjected to normal compressive force due to gravity loads. [1] suggest that for relatively small levels of normal compressive stress (< 2.0 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 N/mm^2), for which the behaviour under shear loading can be approximated by a Coulomb relationship (Equation 1) [1]:

$$\tau_u = \tau_o + \mu\sigma_n \tag{1}$$

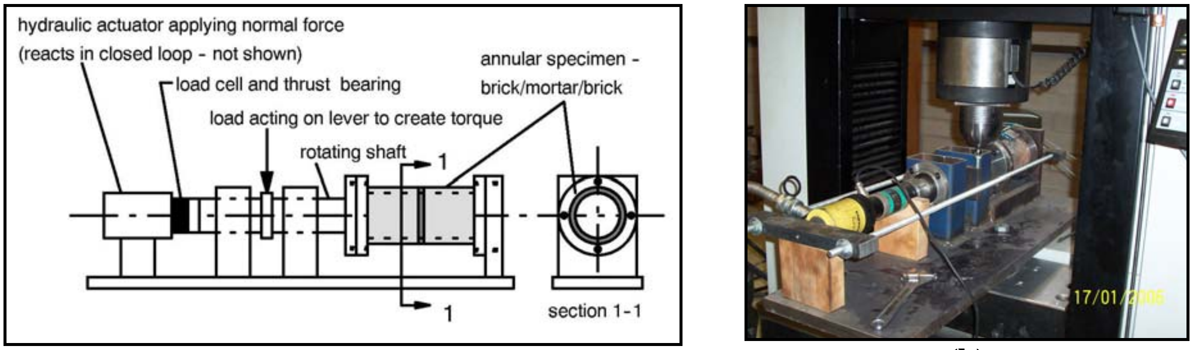
where τ_u is the shear strength (N/mm^2) of the mortar joint when subjected to a normal compressive stress of σ_n, τ_o is the bond shear strength or cohesion (that is, the shear strength for zero normal pre-compression) and μ is the coefficient of internal friction. For the prediction of wall strengths, τ_u is sought and σ_n can be calculated from estimates of the gravity loads. Therefore, the values of τ_o and μ are required. These are properties of the particular masonry used. In addition to the values of τ_o and μ further parameters are required to characterise the shear behaviour of mortar joints for analyses using finite element micro-modelling 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, the post cracking coefficient of friction (which may be different from μ) and the dilatancy (normal displacement during shearing) of the joints [2].

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) ([3], [4]) or a triplet specimen (3 units) [5]. 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 τ_o and μ 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 ([4], [6], [7]). 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. 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 [8] 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. 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 methodology and results from five series of tests conducted using the apparatus. [9] report testing apparatus almost identical to that proposed herein. The latter research, conducted simultaneously but completely independently from that reported below, further reinforces the validity of the proposed approach.

EXPERIMENTAL METHODOLOGY

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.



(a) **(b)**

Figure 1: Test apparatus

(a) **(b)**

Figure 2: Test specimen

The apparatus was mounted within a universal testing machine. Each single joint specimen was epoxy glued in situ 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. Each specimen was first subjected to a compressive force normal to the mortar joint using a hydraulic jack (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). 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.

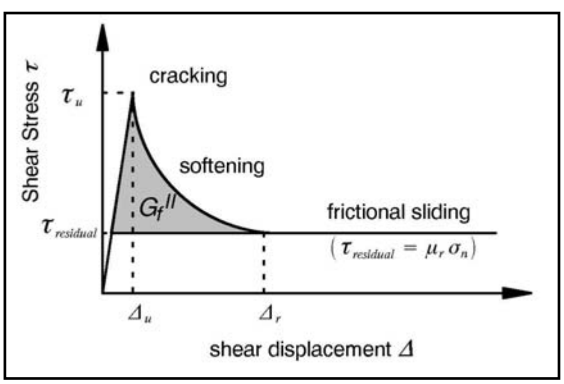
Through all stages of loading the following information was continuously logged: normal compressive force F_n, force P applied by the universal testing machine (torque $T = P$ x lever length), shearing displacement across the mortar joint (Figure 2a), from which relative rotation Φ was calculated, and the normal displacement (dilation) across the mortar joint. This allowed the generation of torque T versus rotation Φ plots for various levels of normal compressive force. Characterisation of the material behaviour requires the shear stress τ versus shear displacement Δ 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.

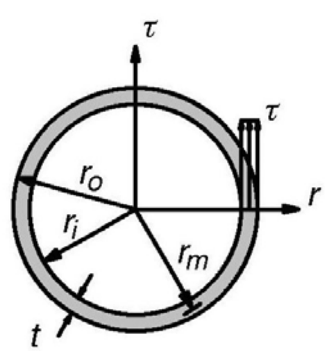
Under displacement controlled conditions the post peak shear behaviour at a material point has been observed to exhibit a softening response [3] as idealised in Figure 3a. In the presence of normal compressive stress the shear stress eventually softens to an approximately constant value equal to the residual coefficient of friction multiplied by the normal stress (Figure 3a and Equation 2).

$$\tau_{residual} = \mu_r \sigma_n \tag{2}$$

These various stages of material behaviour result in a complex relationship between shear stress and torque. However, if the thickness $t = r_o - r_i$ of an annular cross section is small compared to the mean radius $r_m = (r_o + r_i)/2$ then it is usual to assume that the shear stress is approximately constant across the thickness t during twisting (Figure 3b). In Figure 3b, τ 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. 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:

$$\tau = 3T / (2\pi(r_o^3 - r_i^3)) \qquad (\tau \text{ approx. uniform for } t << r_m) \tag{3}$$





(a) **(b)**

Figure 3: a) Shear stress versus shear displacement at a material point; b) Theoretical distribution of shear stress across "thin" annular sections subjected to torsion

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.

$$\Delta = r\,\Phi \tag{4}$$

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 compressive stress σ_n is also uniform over the cross section and equal to the normal force F_n divided by the cross sectional area $\pi(r_o^2 - r_i^2)$, the desired testing conditions of uniform normal and shear stress distributions can be achieved.

Using analytical comparisons and a detailed finite element study [8] the authors were able to show that the assumption of uniform shear and normal stresses is reasonable for all stages of behaviour for annular specimens with r_i = 35 mm and r_o = 50 mm (r_m / t = 2.83). Therefore, for the current experimental program (r_i = 36 mm, r_o = 47.5 mm, r_m / t = 3.63), Equations 3 and 4 were applied directly to derive plots of shear stress τ versus shear displacement Δ at a material point through all stages of loading.

From the τ versus Δ plots the ultimate shear strength τ_u for various levels of normal stress σ_n were obtained. From this, the Coulomb relationship described by Equation 1 was established by linear regression of the test data and hence τ_o and μ determined for each of the test series. The residual shear stress $\tau_{residual}$ was also plotted against normal stress σ_n for each test series allowing the residual coefficients of friction μ_r to be determined from linear regression using Equation 2.

Although not reported in this paper due to space limitations, several other parameters were determined for each test. The shear fracture energy G_f^{II}, defined by the shaded area in Figure 3a was determined. The dilation of the joints during shearing was measured by positioning displacement potentiometers against the back face of the moving end plate of the testing apparatus. Lastly, the Elastic Shear Modulus G was determined for each test by computing the slope of the loading branch of the τ versus Δ response using a secant drawn between 5% and 33% of τ_u. Note that the shear displacement values were divided by the joint thickness to obtain values of shear strain. Results for these three parameters for Series 1 and Series 2 masonry are reported in [10].

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 = 36 mm and outer radius r_o = 47.5 mm.

The preparation of specimens is arguably the most challenging aspect of the proposed torsion shear test. One possible approach is to first cast a couplet specimen and then core the complete specimen through brick/joint/brick. This approach has the advantage that 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 representative of a mortar joint in a wall. The disadvantage is that the relatively weak mortar joint could be damaged during coring resulting in the joint failing prior to testing, or even worse, remaining unbroken but being damaged to an unknown extent hence potentially affecting the subsequent test results. Another approach is to first core annular circular cylinders from solid bricks and then bond them together. This latter approach requires construction of a mortar joint with a very thin bedded area resulting in shrinkage edge effects unrepresentative of joints in a masonry wall. The current research investigated a range of specimen preparation techniques as outlined below.

The results of five series of tests are reported in this paper. The specimens in Series 1 were constructed using extruded solid clay units and 1:1:6 (Portland cement : lime : sand – by volume) 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 to 5 were constructed using the same brick type, but from a different production run, and 1:1:6 mortar with no further additives. Each series was constructed using a different batch of mortar. The flexural tensile bond strength f_t for each series was obtained using bond wrench tests in accordance with Australian Standard AS3700-2001 [11]. The specimen construction techniques were as follows:

<u>Series 1 and 2:</u> 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 taking considerable care to align the upper and lower cylinders prior to air curing in the laboratory.

<u>Series 3:</u> Masonry couplets were first prepared by stack bonding two units together with a 10 mm thick mortar joint. The annular specimens were then cored from the couplet. To do this, the couplet was held in place while a solid diameter 95 mm cylinder was cored from the centre of the couplet. The solid cylinder was then pre-compressed by placing a timber template across its top which was held down using steel tie rods on either side of the specimen (Figure 4a). A hole in the centre of the template allowed the inner (72 mm diameter) coring drill to pass through and also acted to centre and guide the drill. This approach resulted in some pre-compression acting on the annular cross section mortar joint as the coring drill passed through the joint.

<u>Series 4:</u> In Series 4, the authors wished to quantify the effect, if any, of coring through the mortar joint during specimen construction. Therefore, six specimens were constructed using the same approach as for Series 1 and 2. A further six specimens were constructed by first casting couplets and then coring the specimens through brick/joint/brick as for Series 3 (note that the same mortar batch was used for all 12 specimens in Series 4). However, unlike Series 3, the inner diameter 72 mm core was drilled first, followed by the outer diameter 95 mm core. To attempt to prevent damage to the mortar joint during coring, pre-compression was applied across the complete couplet while drilling the inner core and then an Aluminium "plug' style template, held down by a steel tie rod through the centre of the specimen was used to pre-compress the joint and guide the diameter 95 mm drill for the outer core (Figure 4b).

<u>Series 5:</u> Annular cylinders were first cored from solid bricks as for Series 1 and 2. However, when casting the thin annular mortar joints, an innovative technique was developed to ensure full joints and zero moisture gradients at the outer surface (using a Perspex tube) and inner surface (using a sand filled plastic bag) of the joint during curing (Figures 4c and 4d). By doing this, the authors believe that the two seemingly conflicting objectives can be achieved; namely joints which represent those in a masonry wall and are not damaged by coring.

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

Figure 4: Specimen preparation a) Series 3; b) Series 4; c) and d) Series 5

EXPERIMENTAL RESULTS AND DISCUSSION

Figure 5 shows the shear stress versus shear displacement response derived for a typical test and the key experimental results are summarised in Table 1. Note that torque data was not recorded for the Series 4 specimens due to a load cell error. The inset in Figure 5a shows the loading branch of the response in more detail. The plots clearly show the various stages of behaviour idealised in Figure 3a.

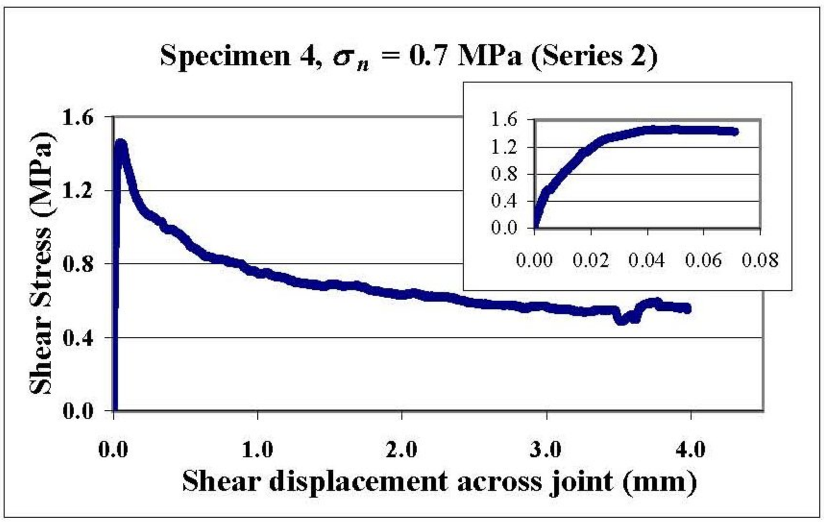


Figure 5: Typical shear stress versus shear displacement response

The inset in Figure 5a indicates that the stiffness of the mortar joints typically began to reduce prior to reaching peak torque (and hence peak stress τ_u). However, cracking was usually not visible until peak torque was reached. For most 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 6a). For several specimens cracking also occurred in the bricks on one or both sides of the joint (Figure 6b). Brick cracking was more prevalent for the test series for which the masonry displayed very high flexural bond strength (Table 1). Cracking usually initiated at a location where the brick displayed fine cracks prior to testing; the cracks presumably the result of the brick manufacturing process. In particular, the Series 5 masonry combined very high bond strength with quite friable bricks which contained numerous pre-existing cracks. The failure modes for each series are summarised in Table 1.

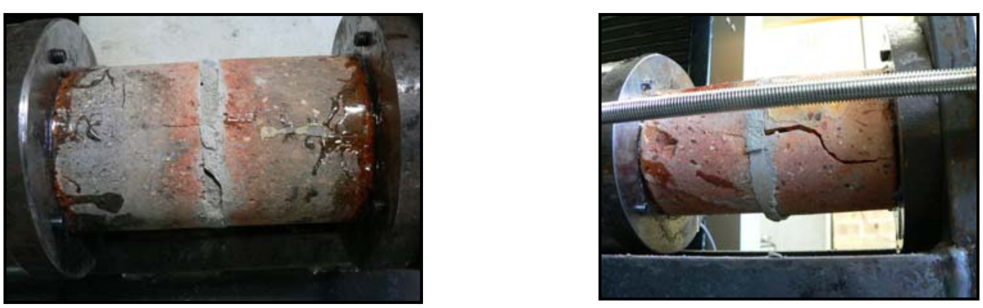
A number of specimens which were cored through brick/joint/brick failed prior to testing, indicating that despite efforts being made to pre-compress the mortar joint during coring, the torque and vibration applied by the coring drill was sufficient to damage the joint in many instances. For Series 4 in particular, every specimen constructed by coring through the mortar joint failed during the coring process. Furthermore, the comparatively low value for cohesion (τ_o) for the Series 3 masonry indicates that the mortar joints were likely damaged during the coring process. It is the opinion of the authors that specimen preparation techniques which involve coring through the mortar joint should be avoided.

Table 1: Summary of Experimental Results

Series	Specimen construction	f_t (COV) [a] MPa (%)	τ_o MPa	μ	μ_r	No. of specimens: failure modes	Least squares fit to data
1	Core then bond	0.14 (26)	0.22	0.90	0.56	8 spec: 1 prior to test / 7 joint	$\tau_u = 0.90\sigma_n + 0.22$, $R^2 = 0.71$ $\tau_{res} = 0.56\sigma_n + 0.09$, $R^2 = 0.90$
2	Core then bond	1.74 (11)	1.02	0.83	0.85	13 spec: 8 joint / 5 brick + joint	$\tau_u = 0.83\sigma_n + 1.02$, $R^2 = 0.76$ $\tau_{res} = 0.85\sigma_n + 0.04$, $R^2 = 0.88$
3	Core joint	1.08 (37)	0.41	1.29	0.95	12 spec: 8 joint / 4 brick + joint	$\tau_u = 1.29\sigma_n + 0.41$, $R^2 = 0.52$ $\tau_{res} = 0.95\sigma_n + 0.02$, $R^2 = 0.95$
4a	Core then bond	No torque data recorded due to load cell error				6 spec: 4 joint / 1 brick / 1 brick + joint	
4b	Core joint	-	-	-	-	6 spec: 6 prior to test	
5	Core then bond using moisture barrier	1.71 (21)	2.06	**0.23**	1.20	12 spec: 7 joint / 5 brick + joint	$\tau_u = 0.23\sigma_n + 2.06$, **$R^2 = 0.01$** $\tau_{res} = 1.20\sigma_n + 0.01$, $R^2 = 0.98$

[a] f_t = flexural tensile bond strength from bond wrench tests (Standards Australia 2001)

Comparing Series 2 and Series 5 (for which flexural bond strengths are similar) the much higher cohesion for Series 5 indicates that the technique of providing a zero moisture gradient for the thin annular mortar joint during curing increases its strength significantly. This technique, which is designed to simulate the moisture environment during curing within a masonry wall is that most preferred by the authors.

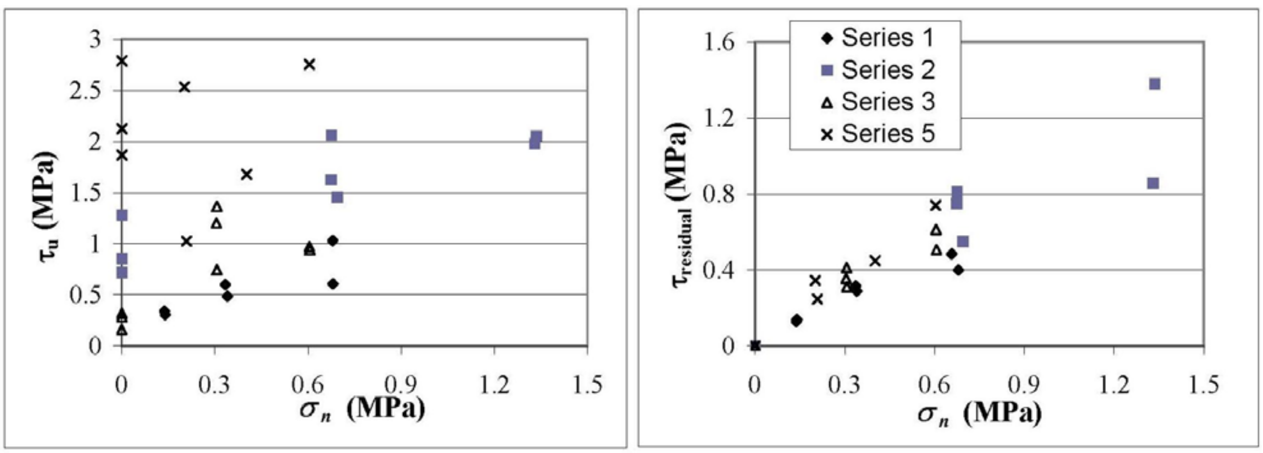


(a) **(b)**

Figure 6: Failure modes a) diagonal cracking across mortar joint; b) brick cracking

Figures 7a and 7b show the ultimate τ_u and residual $\tau_{residual}$ shear stresses respectively, plotted against normal compressive stress σ_n for all series of tests, excluding Series 4. All results in which brick failures were observed have been excluded. Cracking through the brick as shown in Figure 6b affects the distribution of shear stress such that Equation 3 can not be used post cracking to estimate the shear stress from the torque. As brick cracking usually coincided with peak torque and hence peak shear stress, it could be argued that Equation 3 could still be used to estimate the peak stress for each test. For the Series 5 data in particular, the specimens displaying brick cracking typically failed at higher peak stress than those for which failure was confined to the joint. This implies that the joints were at least as strong as the stress required to initiate brick cracking. If the brick cracking data is included in the calculation of the Mohr Coulomb Equation for Series 5, a much larger internal friction coefficient ($\mu = 2.11$) is obtained than that reported in Table 1. However, as brick cracking could be initiated at pre-existing defects within the bricks, the resulting data does not accurately reflect joint strength and has been excluded here.

Linear regression lines fitted to the τ_u data (Table 1) show that sensible Coulomb relationships (Equation 1) can be established for Series 1, 2 and 3 data. For Series 5, after excluding brick cracking data, so few data remain that the linear fit can not be established with any confidence. The regression lines fitted to the $\tau_{residual}$ data show strong correlation for all series of data.



(a) **(b)**

Figure 7: a) Peak shear stress and b) Residual shear stress versus normal compressive stress (brick cracking data removed)

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 five series of tests using two different brick and mortar combinations and a range of specimen construction techniques were presented. The key findings are:

1. The procedure enables the shear stress versus shear displacement response at a material point to be fully described. It therefore allows the material parameters defining the pre and post peak shear behaviour to be obtained.
2. Specimen preparation is challenging. The authors recommend that specimen construction techniques which involve coring through the mortar joint be avoided due to the risk of damaging the joint prior to testing. The preferred construction technique (used for Series 5) involves first

coring the units and then bonding the annular brick cylinders together using moisture barriers to simulate the curing conditions present in a masonry wall.

3. The incidence of brick cracking failures increased with the bond strength of the masonry. While it may be possible to use the test method to obtain the peak shear strength, post peak behaviour can not be obtained for specimens that fail by brick cracking.

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REFERENCES

1. Riddington, J.R. and Ghazali, M.Z. (1990) "Hypothesis for Shear Failure in Masonry Joints" Proceedings of the Institution of Civil Engineers, Part 2, 1990, pp. 89-102.
2. Lourenço, P.B. (1998) "Experimental and Numerical Issues in the Modelling of the Mechanical Behaviour of Masonry" Structural Analysis of Historical Constructions II, Barcelona, 1998.
3. Van Der Pluijm, R. (1993) "Shear Behaviour of Bed Joints" Sixth North American Masonry Conference, Philadelphia, Pennsylvania, pp. 125-136.
4. Stöckl, S., Hofmann, P. and Mainz, J. (1990) "A Comparative Finite Element Evaluation of Mortar Joint Shear Tests" J. Brit. Mas. Soc., Masonry International, 3(3), 1990, pp. 101-104.
5. European Standard 2002: European Standard EN1052-3 Methods of Test for Masonry - Part 3: Determination of Initial Shear Strength, July, 2002.
6. Jukes, P. and Riddington, J.R. (1997) "A Review of Masonry Joint Shear Strength Test Methods" J. Brit. Mas. Soc., Masonry International, 11(2), 1997, pp. 37-43.
7. Riddington, J.R., Fong, K.H. and Jukes, P. (1997) "Numerical Study of Failure Initiation in Different Joint Shear Tests" J. Brit. Mas. Soc., Masonry International, 11(2), 1997, pp. 44-50.
8. Masia, M.J., Han, Y. and Correa, M.R.S. (2006) "Torsion Testing for the Characterisation of the Shear Behaviour of Mortar Joints in Masonry" J. Brit. Mas. Soc., Masonry International, 19(2), 2006, pp. 77-88.
9. Schmidt, U., Hannawald, J. and Bramesbuber, W. (2008) "Theoretical and practical research on the flexural strength of masonry" 14th International Brick and Block Masonry Conference, Sydney, Australia, on CD-ROM.
10. Masia, M.J., Han, Y., Player, C.J., Correa, M.R.S. and Page, A.W. (2007) "Torsion shear test for mortar joints in masonry: preliminary experimental results" Tenth North American Masonry Conference, St Louis, Missouri, pp. 968-979.
11. Standards Australia 2001: Australian Standard AS3700-2001 Masonry Structures: Appendix D – Method of Test for Flexural Strength, Standards Australia Int. Ltd, Sydney, Australia, 2001.