



MASONRY SHEAR STRENGTH UNDER IMPACT LOADING

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ABSTRACT

Various test methods to evaluate the shear strength of the brick/mortar interface of masonry have been developed over the past few decades. From this work it has been shown that a Mohr-Coulomb type relationship can be used to approximately represent the increase in shear strength under increased levels of pre-compression. This work has principally considered the quasi-static performance of masonry; the shear strength of masonry under impact loading has not been considered until recently.

The performance, assessment and design of unreinforced masonry parapets when subjected to vehicle impacts has been investigated over the last few years. During the course of these studies the shear behaviour of the brick/mortar interface under impact loading was considered. It was found that the dynamic shear strength of triplet specimens was higher than that measured quasi-statically.

This paper describes the test method used to determine the dynamic shear strength of masonry. The test method and apparatus are critically appraised. Results from a series of triplet tests are also presented. From these results an evaluation of the shear strength enhancement over a range of stress rates is made. Consideration is also given to the effect of the support conditions on the behaviour of the triplet specimens.

These studies should lead to an improved understanding of the relationship between the strength increase and the applied stress rate. The results should then allow an improvement to be made to the numerical analysis, which is currently being used as a predictive tool for assessing the performance of masonry parapet walls when subjected to vehicle impact. The general-purpose explicit finite element software LS-DYNA3D is being used for this purpose.

Key words: masonry, impact, triplet, shear, parapets

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INTRODUCTION

Ghazali and Riddington (1988) presented a simple method for assessing the shear strength of the brick/mortar interface by using a triplet test specimen tested without the complication of adding a precompression force. From this work it was shown that the Mohr-Coulomb failure envelope could be established by conducting tests with zero precompression and by then measuring the coefficient of friction at the joint.

Molyneux (1994) found that for finite element models to accurately predict the impact and post impact behaviour of the masonry parapet walls, enhanced values of shear and tensile strength had to be used at the interfaces within the finite element model. In order to investigate the reason for enhanced shear values being required in the finite elements, triplets were tested in a drop hammer rig at Liverpool University. This allowed triplets with the same loading arrangement as those tested quasi-statically to be tested at stress rates similar to those likely to be encountered in a wall subjected to a vehicle impact. Molyneux (1994) showed that the shear strength obtained under these conditions was strain rate sensitive and strengths of up to three times the quasi-static values could be obtained when the duration of loading was suitably short.

This work resulted in the development of finite element modelling strategies using LS-DYNA3D. For the masonry walls a discrete modelling strategy was adopted whereby individual masonry units were modelled separately. These units were then connected together using specialised contact surface definitions (Halquist *et al*, 1985). The finite element models were validated by full-scale vehicle impact tests. These results were then used to help with the preparation of County Surveyors Guidance Note (1995) and BS 6779 pt 4 (1999) which provide guidance on the assessment and design of masonry bridge parapets. The above work was completed in collaboration with work conducted at the University of Teesside, Gilbert *et al* (1998).

Although the apparent strain rate sensitivity provided some insight into the behaviour of masonry under impact loading, the actual variation of strength with rate of loading was not considered in detail. Beattie (1996) investigated this using the drop hammer rig developed by Molyneux. The results of this work indicated that there was a noticeable increase in shear strength at stress rates over approximately 4.7kN/mm^2 per second (see Figure 1). However, the reasons for this were never fully explored. This paper examines this phenomenon in more detail by evaluating the test method used and by presenting additional results.

TEST METHOD

Apparatus

The drop hammer rig used for the triplet impact tests is shown in Figure 2. This was developed at Liverpool University to test brick triplets dynamically. The rig consists of a 23kg, 50mm diameter steel bar with a hemispherical lower end. The bar can be lifted to a height of 2m using a winch and dropped using an electro-magnetic release

mechanism. Load cells positioned under the outer bricks of a triplet specimen are used to measure the reaction forces. These are connected to a PC via amplifiers and a data acquisition board. The data acquisition card (Amplicon PC226) is capable of storing data at rates in excess of 100kHz. Retrieval of the data from the PC is carried out using 'log and plot' software (developed at the University of Teesside).

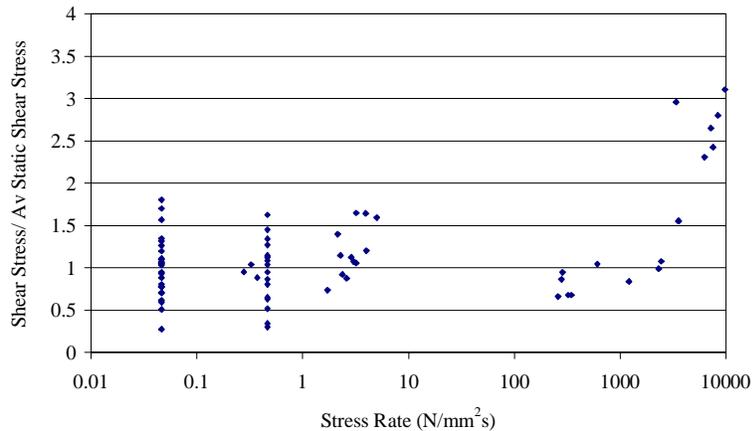


Figure 1 – Results from work by Beattie (1996)

VALIDATION OF TEST RIG

Before this test rig was used for the current test programme the dynamic response of the rig was studied. As the apparatus was supported on a reinforced concrete slab it was important to be certain that the dynamic response of the slab or the stress waves reflected from interfaces were not significantly modifying the force readings.

A number of tests using accelerometers and load cells were completed. A summary of the results from these tests is presented diagrammatically in Figure 3. This shows the time taken for a stress wave to travel through the rig (from a datum of $t = 0$ at the top of the impactor). The results correspond well with the predicted behaviour based on stress wave propagation theory. Furthermore it was also concluded that although the duration of the tests are sufficient to allow the slab to respond before the test is complete, this response does not affect the load cell readings.

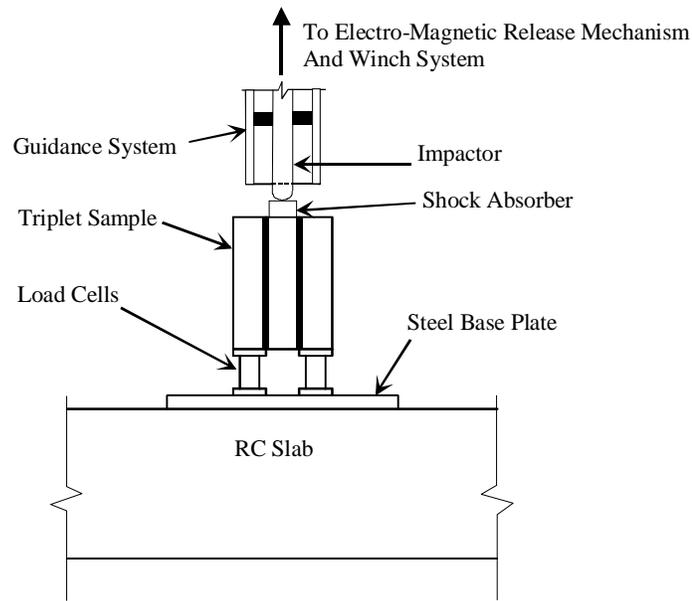


Figure 2 – Drop hammer rig (Not to Scale)

REVIEW OF SUPPORT CONDITIONS

Once the test rig had been validated it was necessary to understand the influence of the support conditions on the results obtained to ensure these were correctly interpreted.

Ghazali and Riddington (1988), and Riddington and Ghazali (1990) considered various support conditions used in triplet tests, each of which varied the amount of bending applied to a sample. Although none of the loading conditions were identical to those in the work by Molyneaux and later by Beattie, Bouzeghoub *et al* (1995) did assess a similar support type in their study. The six support conditions considered by Bouzeghoub *et al* are shown in Figure 4.

Support condition type F is similar to that used in the work by Molyneaux (1994). Bouzeghoub *et al* showed results using this loading arrangement were highly sensitive to the tensile bond strength of the joint. This finding therefore raised the question as to whether the apparent increase in strength measured in the work by Molyneaux was due to an increase in shear strength, tensile strength or a combination of the two. To address this issue a series of tests were prepared. Initially 15 triplet tests were conducted using a Dennison Universal Testing Machine. Eight were tested using support type F and produced a mean shear stress at failure of 0.46 N/mm^2 . The remaining seven samples were tested using support type A and produced a mean shear stress at failure of 0.6 N/mm^2 .

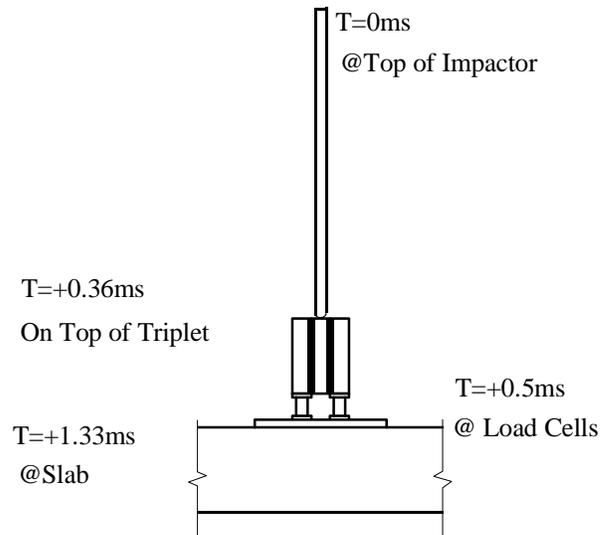


Figure 3 – Stress wave travel through drop hammer rig (Not to Scale)

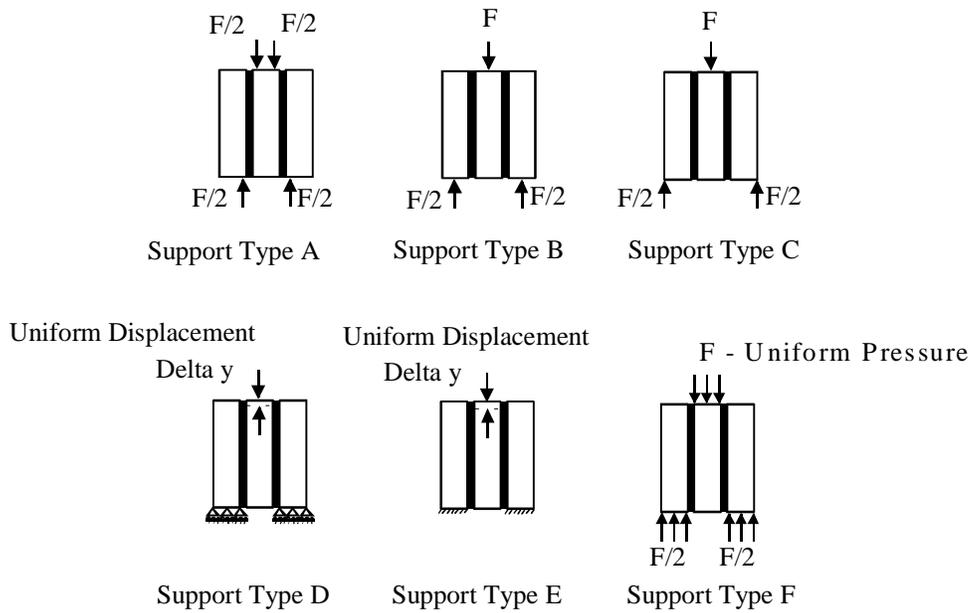


Figure 4 – Loading arrangements considered by Bouzeghoub et al (1995)

CRACK SPEED

The test samples described previously were also instrumented to allow the rate at which a crack propagates through a triplet to be monitored. This was achieved by the use of high speed logging equipment and conductive paint (shown in Figure 5).

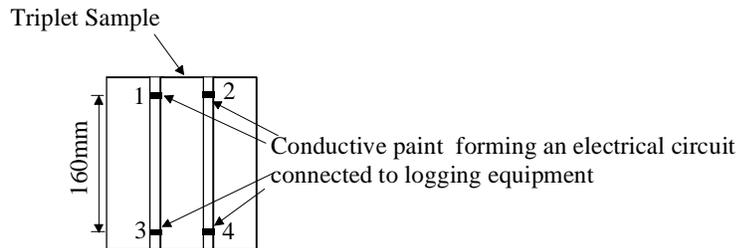


Figure 5 – Set up to determine crack speeds

These tests gave a clear indication of the failure mode. It was apparent that the type F loading conditions were initiated by tensile failure between the brick/mortar interface at the bottom of the joint (see Figure 6a), whereas shear initiated failure with the type A loading condition (Figure 6b). This appeared to confirm that the increase in strength previously observed was not due to an increase in shear strength alone.



Figure 6 – Test specimens showing failure mode: (a) failure initiated by tensile stresses; (b) failure initiated by shear stresses

MODELLING MORTAR-BRICK INTERFACE BEHAVIOUR

Work by Molyneaux *et al* (1995) using the explicit finite element code LS-DYNA3D represented the joints in masonry using a specialized contact surface formulation. In this approach, nodes on the surface of a given masonry unit remain tied to the surface of an adjacent unit whilst failure criterion (1) is satisfied, after which the surfaces are free to separate or slide with friction.

$$\left[\frac{tension}{tension_{limit}} \right]^2 + \left[\frac{shear}{shear_{limit}} \right]^2 \leq 1 \quad (1)$$

Using the original interface formulation, the shear limit was assumed not to be affected by the presence of normal compression. The approach was used reasonably successfully to model brickwork and stone walls undergoing large deformations following vehicle impacts. However, to achieve good correlation between full-scale test data and the numerical results, enhanced (above normal quasi-static) values of the limiting stresses had to be used. The requirement that enhanced values be used was perceived to be partly due to strain rate enhancement (also observed in similar materials such as concrete by Zielinski and Reinhardt, 1982) and the brittle failure mode assumed in the LS-DYNA3D interface type used. In the finite element model, when an initially tied node fails the force on adjacent nodes increases dramatically at the next time-step, causing these to then fail in turn. The crack speed is thus dependent on the time-step. In reality cracks in such materials do not form in an entirely brittle manner but have a post peak softening response as energy is dissipated. In the model more than one tied node at or near the crack tip could be on a post-peak softening curve at any one time.

Rots and Lourenco (1993) developed a failure criterion that included post peak softening based on fracture energy data from a series of physical tests. Gilbert *et al* (1997), Molyneaux and Gilbert (1997), and Gilbert *et al* (1998) developed a model which has similarities to that proposed by Rots. For this model an exponential softening relationship is adopted (Figure 7). This is used to calculate a softening parameter k :

$$k = k^I + k^{II} - 1 \quad (2)$$

This allows a smooth transition between the initial failure surface and the residual failure surface (Figure 8). The initial failure surface comprises the failure criteria of (1) combined with the Mohr-Coulomb failure criterion in the presence of compressive direct stresses. This approach does however require data to be available on the post peak stress vs. displacement response to enable the mode I and mode II fracture energies to be determined. However, reliable fracture energy data is difficult to obtain and is the subject of current test work by the authors.

Considering Figure 8, if the stresses calculated during the analysis are found to lie outside the initial failure surface they are scaled down using a scale factor f . This factor is then recalculated for each subsequent timestep by taking into consideration the magnitudes of the current plastic tensile and shear deformations (u_t^p , and u_s^p). The areas under the two softening curves used to scale down the stresses are the mode I (tensile) and mode II (shear) fracture energies (G_f^I and G_f^{II} respectively). The ultimate mode I and mode II displacements, u_f^I and u_f^{II} , may be defined as the point on the assumed exponential softening relationship where the interface stress has reached 0.1% of the failure stress.

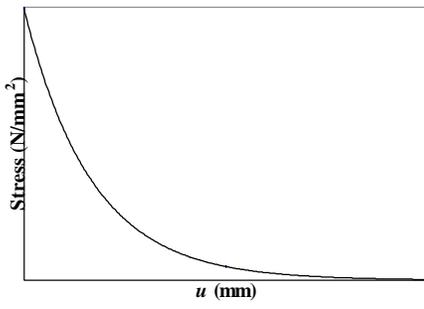


Figure 7 – Exponential softening curve

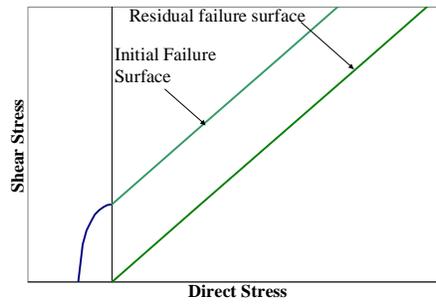


Figure 8 – Failure surface

Therefore, given the exponential form of Figure 7:

$$\sigma = \sigma_f e^{(-bu_f)} \quad (3)$$

The fracture energy is thus given as:

$$G_f = \int_{\sigma=0.001\sigma_f}^{\sigma=\sigma_f} \frac{\ln\left(\frac{\sigma}{\sigma_f}\right)}{-b} d\sigma \quad (4)$$

Solving this gives:

$$u_f^I = \frac{-\ln(0.001)G_f^I}{\sigma_f} \quad (5)$$

$$u_f^{II} = \frac{-\ln(0.001)G_f^{II}}{c} \quad (6)$$

Where c is the cohesion (or 'shear bond strength') and k^I and k^{II} referred to in (2) are defined as:

$$k^I = e^{\left\{ \ln(0.001) \frac{u_f^I}{u_f^I} \right\}} \quad (7)$$

$$k^{II} = e^{\left\{ \ln(0.001) \frac{u_f^{II}}{u_f^{II}} \right\}} \quad (8)$$

The scaling factor f is then defined as:

$$1 \leq f = \sqrt{\left(\frac{\sigma^{trial}}{k\sigma_f}\right)^2 + \left(\frac{\{\tau_{trial} - \sigma^{trial} \tan \phi\}}{kc}\right)^2} \quad (9)$$

Where ϕ is the angle of friction of the masonry joint and where σ^t_{trial} , σ^c_{trial} and τ_{trial} are respectively the trial tensile normal, compressive normal and shear stresses at the joint.

Stresses throughout the softening phase are scaled down as shown in (10), (11) and (12):

$$\sigma^t = \frac{\sigma^t_{trial}}{f} \quad (10)$$

$$\sigma^c = \sigma^c_{trial} \quad (11)$$

$$\tau = \frac{\{\tau_{trial} - \sigma^c_{trial} \tan \phi\}}{f} + \sigma^c_{trial} \tan \phi \quad (12)$$

When $u_p > u_f$ the interface reverts to contact with friction.

ANALYSIS OF RESULTS

This section examines the strain rate behaviour observed in triplet tests in view of the effects of post-failure softening. The results discussed here have been obtained from tests performed over a range of stress rates and under support conditions type F (see Section 3.3). Work is continuing on measurement of post failure softening and on testing specimens using support type A.

The triplet tests described in section 3.4 showed that a crack initiates at the bottom of the test sample and travels upwards. The average crack time (measured over 160mm) was found to be 0.092 ± 0.036 ms giving a crack speed of 1716m/s. The time taken for the crack to propagate through the full depth of the triplet sample is therefore 0.124 ± 0.048 ms. These measurements were consistent with earlier measurements from dynamic triplet tests (Beattie, 1996).

Figure 9 shows the results of triplet tests conducted at stress rates from quasi-static up to 10kN/mm^2 per second. The dynamic tests were conducted as described in Section 3.1. The range of stress rates was achieved by varying both the form of the crushable target (load attenuator) on the centre brick and the drop height. Due to the non-linearity of the force-time histories of these dynamic tests, the load rate is taken as the average gradient from 0.124ms before failure up to the point of failure. The figure shows three continuous lines that have been calculated assuming that the inertial effects are negligible and using the measured crack propagation speed as follows. The calculation is based on the premise that the failure initiates at a particular stress (a quasi-static value from the y-axis) and that while the crack is propagating the load is continuing to rise as the brick

joints are still transferring load. The calculated dynamic failure is taken to be the quasi-static value plus the loading rate (from the x -axis) times the time taken for the crack to propagate across the specimen. The three lines are based on the average, highest and lowest experimentally recorded quasi-static shear strength values. It is evident that the lines envelope most experimental data points, even at the higher loading rates.

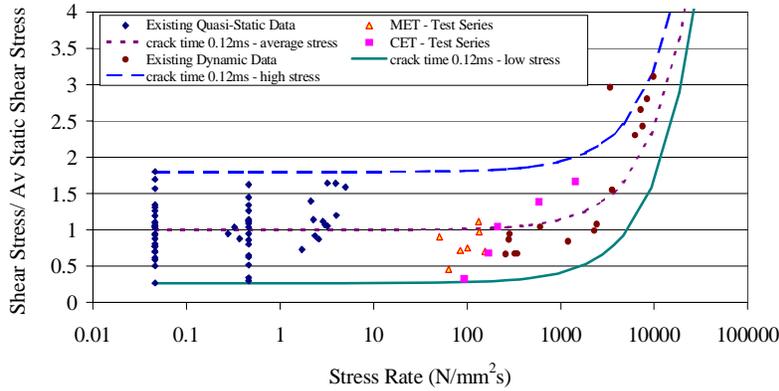


Figure 9 – Triplet results (type F supports)

For a given series of tests, employing the same drop height and crushable medium, the recorded pre-peak force-time history was found to be quite repeatable. For example, taking tests conducted at a drop height of 2m and with a particular target, (CET test series) the force-time histories obtained from the tests are shown in Figure 10. The figure also shows a line of best fit based on an exponential form where:

$$F = 0.002 e^{1076 \times t}$$

For this set of data a prediction of the range of likely failure loads can be calculated by taking the stress rate at failure (from the equation above for a given quasi-static failure load) and then adding the “dynamic enhancement” due to the finite time of crack propagation. Figure 11 shows the results of such a calculation. The results indicate reasonably close correlation between the predicted values and the test results – suggesting that the “dynamic enhancement” may be attributed to the finite crack propagation velocity.

CONCLUSIONS

1. Masonry, when subjected to high stress rates does exhibit enhanced bond strength due to the finite crack propagation velocity. However, the interaction between shear and tension actions has not yet been established.
2. When using triplet tests to obtain shear data on masonry the results are sensitive to the support conditions used.

- Crack propagation velocity data may be a useful parameter for use in objective finite element modelling for masonry subjected to dynamic loading.

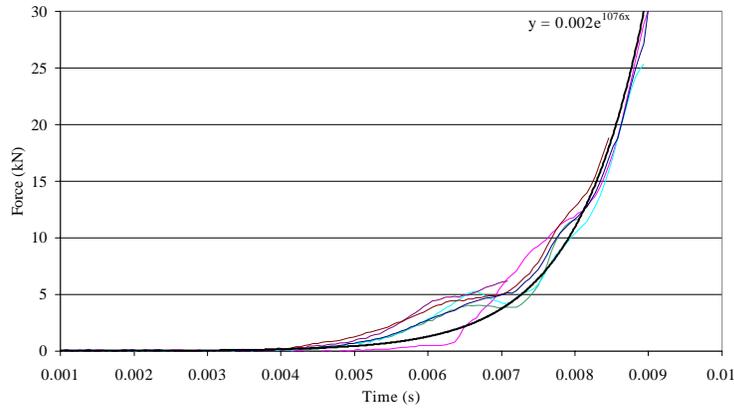


Figure 10 – Force-time histories of CET test series

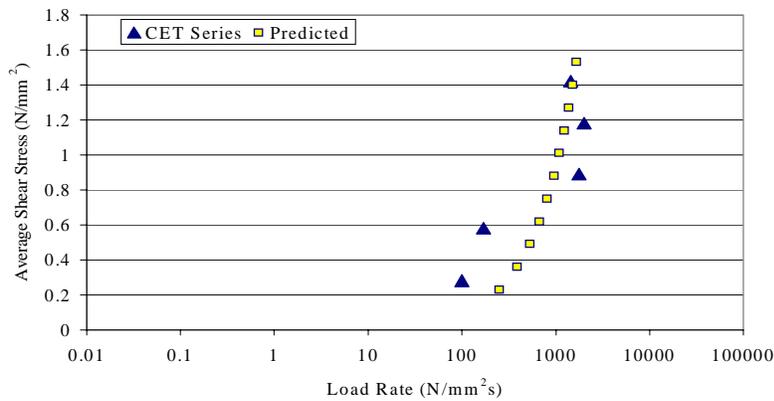


Figure 11 – Predicted Values of CET Test Series

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