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IN-SITU TESTING OF THE SHEAR STRENGTH OF MASONRY JOINTS – PROPOSAL FOR A NEW SIMPLIFIED TESTING APPROACH AND COMPARISON TO EXISTING TECHNIQUES

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Abstract

For other than minor modifications or additions to existing masonry structures, an understanding of the in-situ mechanical properties of the masonry is required. Estimation of shear strength of the masonry bond is often made using the in-plane shove test as it only requires access from one side of the masonry being tested, and is therefore considered to be only mildly invasive. In-situ and laboratory simulations of the shove tests have however shown that a number of site-specific problems can arise. These include: preparation of the test area which can damage the mortar joints resulting in loss of cohesion, imprecise alignment of both the loading system and instrumentation leading to complications with data interpretation, and without an adequately long section of wall to be tested, flexural cracking and lateral displacement of large wall sections. In an attempt to address these issues, in this paper, an alternative in-situ test is proposed. This new '*pull test'* measures shear strength by extracting a single brick orthogonally in the wall's out-of-plane direction. A laboratory investigation is performed to compare the two types of in-situ test (shove and pull) using plain and frogged units, as well as lime and cement mortars, and both tests are further benchmarked using the standard laboratory couplet test. It is demonstrated that for regular (rectangular), plain units with weak mortar, the two in-situ tests produce strength measurements that are statistically equivalent, indicating that under this specific scenario the out-of-plane shear strength can be used as a proxy for the in-plane shear strength, and that the pull test could be used as an alternative to the shove test. In addition, the new pull test is shown to perform with demonstrated repeatability, requires minimal instrumentation, and is particularly relevant for design of anchorages under out-of-plane loading.

Keywords:

Unreinforced masonry; in-situ testing; shear strength of bed joint; shear couplet test; shove test

1. Introduction

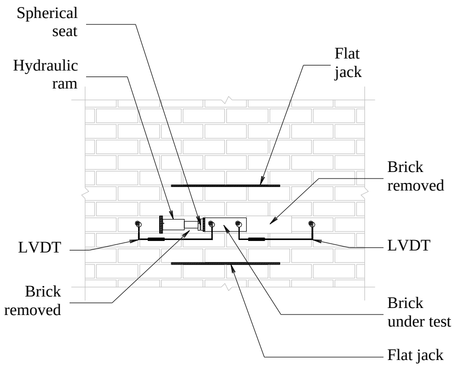
The assessment of existing unreinforced masonry (URM) structures against extreme loading such as earthquakes, and the design of corresponding retrofits requires knowledge of in-situ mechanical properties. These properties can then be applied in a variety of contexts including the evaluation of the global capacity of URM buildings, resistance against local out-of-plane collapse mechanisms, and the strength of connections between walls and other components of the building such as floor diaphragms, roof diaphragms, or remedial framing necessary to tie the building together. A material property of particular engineering importance is the shear strength of masonry joints, which governs various aspects of URM behaviour, including in-plane shear strength (e.g. Atkinson et al. 1988, Hendry et al. 2004, Incerti et al. 2016, Pela et al. 2017, Sherafati and Sohrabi 2017, Graziotti et al. 2018, Zhang et al. 2018) and out-of-plane flexural strength under horizontal bending, or two-way bending via torsional resistance along bed joints (e.g. Lawrence and Marshall 2000, Vaculik and Griffith 2017). It can also govern the strength of mechanical anchorage connections when loaded perpendicular to the face of the wall as a consequence of masonry unit extraction (e.g. Dizhur et al. 2016, Pisani 2016, Munoz et al. 2018, Giresini et al. 2020, Burton et al. 2021). Insufficient shear strength along the masonry joint is also regarded as one the most common causes of damage to historical masonry under seismic action (Capozucca and Sinha 2004).

The shear strength along the masonry joint is conventionally accepted to follow a Mohr-coulomb relationship comprising an initial shear strength at zero normal stress and a frictional component. For example, when anchoring into a single brick unit, the shear strength between the unit and the surrounding wall is developed through cohesion between the brick and the mortar around its perimeter, and through friction generated by vertical compressive stress along the bed joint. Similarly, in designing an anchor plate fixing to a wall, an understanding of cohesion and friction is essential to predict the capacity against punching shear failure (Pisani 2016).

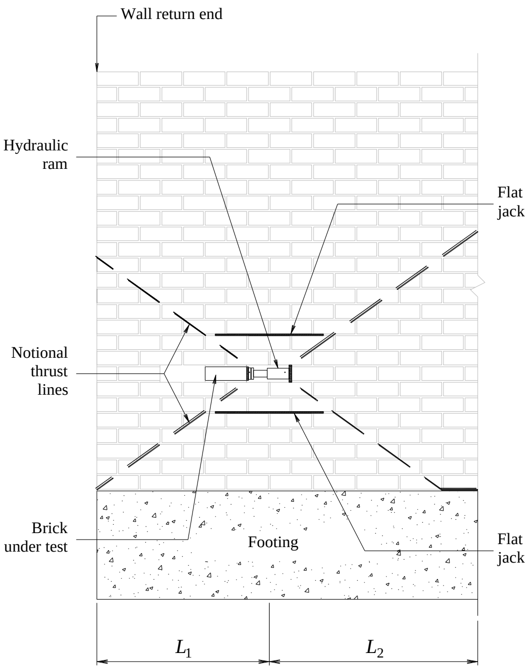
The experimental measurement of masonry shear strength in existing buildings requires either the extraction of undamaged sections of the masonry (including both brick and mortar elements) for subsequent testing in the laboratory, or the use of in-situ tests, which are generally less destructive. Armanasco and Foppoli (2020) observe that the "*Italian Guidelines for Reduction of Seismic Hazard for Cultural Heritage Buildings*" specifically advise that indirect non-destructive test (NDT) methods, for example sonic or ultrasonic, do not provide reliable quantitative estimates of mechanical parameters and as a result, either destructive test (DT) or minor destructive test (MDT) methods need to be employed.

One of the common MDTs is the in-situ bed joint shear test, commonly referred to as the *shove test*, shown diagrammatically in Fig. 1. The test involves removing the brick units adjacent to the test brick, and with a hydraulic ram displacing the test brick while measuring the applied in-plane horizontal load and displacement. The test was originally developed without a means for controlling vertical stresses (which must vary throughout the test as a result of dilation), but later modified by (Noland et al. 1988) to incorporate flat jacks above and below the test brick, and it is this modified procedure that has subsequently been standardised in ASTM C1531-16 (ASTM International 2016). A common criticism of the shove test is that it is difficult to implement in the field without causing damage to the surrounding masonry and therefore potentially influencing the properties that the test is trying to quantify (e.g. Lumantarna et al. 2014, Ferretti et al. 2019, Burton et al. 2020, Jafari et al. 2020, Segura 2020). A detailed review of the shove test and its challenges is presented in Section 2.

In a previous site-testing campaign by the authors focused on investigating strength of anchorage connections, the shove test was used to determine the bed joint shear of walls in situ, and in an attempt to address some of the complications of that test, an alternate test was developed and trialled (Burton et al. 2020). The revised test, referred to hereinafter as the *pull test*, involves extracting a single brick unit from the wall in the out-of-plane direction. The results of that campaign which included regular (rectangular cuboid) units, both plain and frogged, with a combination of weak lime and stronger cement mortars showed that both methods produced comparable measurements of ultimate shear strength under certain scenarios (see Fig. 2). Thus, given that the pull test is less intrusive and simpler to perform, it was hypothesised that it could be used as an attractive alternative to the conventional shove test for on-site application.



(a) configuration



(b) test location

Fig. 1 – Typical shove test arrangement

The purpose of the present study is to examine the efficacy of the pull test under controlled laboratory conditions using the same masonry and mortar typologies previously tested the field (Burton et al. 2020). By doing so, the study also seeks to examine whether the out-of-plane shear strength could be used as a proxy for the in-plane shear strength (and vice versa). A review of existing test methods is undertaken in Section 2. An overview of the pull test is provided in Section 3, followed by a laboratory test campaign in Section 4 for the purpose of benchmarking the pull test against the results of both the shove test and couplet test. The results are reported and discussed in detail in Section 5, and Section 6 concludes by summarising the main findings and recommendations.

2. Review of established test methods

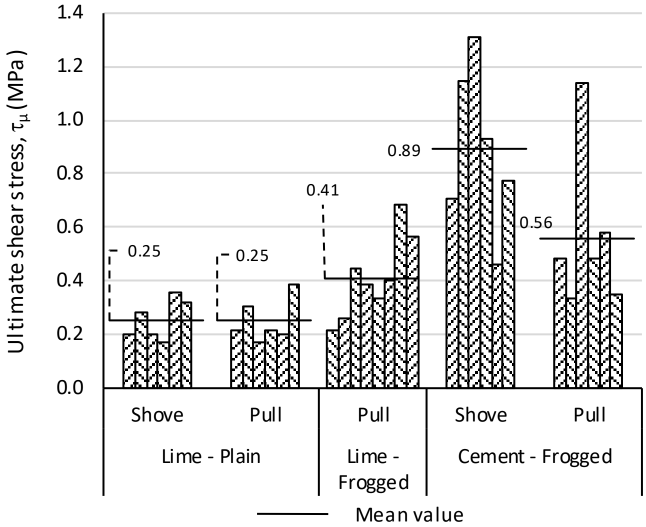


Fig. 2 – Results of in-situ shove and pull tests

Tests for quantifying the shear strength of the masonry bed joint in the field can be broadly categorised into those conducted in-situ within the wall, and those where segments of masonry are extracted and tested in the laboratory. The shove test is the most common in-situ approach, and is reviewed in Section 2.1. The couplet and triplet tests are the most reliable of the laboratory tests, and are reviewed in Section 2.2. Several other types of tests are covered in Section 2.3.

2.1. In-situ shove test

The shove test (Fig. 1a), introduced in Section 1, involves pushing a brick within the wall in the in-plane direction. It can be conducted in a number of variants that differ in how the horizontal load is applied: (a) removal of a single masonry unit at one side of the test brick and a perpend joint at the other side, and applying a monotonic load using a single hydraulic jack, (b) removal of two bricks (one either side of the test brick) allowing for load reversal using dual jacks, and (c) removal of only the perpends at each side of the test brick and loading the unit with a flat jack. Flat jacks to apply a vertical stress can also be included. As the test requires removal of only small portions of masonry (depending on the method adopted) it is generally considered to be only minor-destructive. The general test procedures of ASTM C1531-16 Method A (ASTM International 2016) which removes bricks either side of the test brick are summarised in Graziotti et al. (2018) and each of these procedural steps and their limitations are discussed below.

2.1.1. Selection of the brick to be tested

To minimise the influence of potentially incompletely filled (weak) perpends, the test brick should be located in stretcher bond (as is shown in Fig. 1). The test location in the wall must also be adequately distant from openings, the top of the wall, and external corners to ensure that adequate masonry is engaged to resist the thrust force generated by the loading jack. This introduces a potential challenge, in that it can be difficult to determine an adequate clearance a priori.

Fig. 1(b) shows schematically the elevation of a typical single-rise masonry wall, where the test brick is located eight courses above the footing and the hydraulic ram is centred four bricks from the free end. Upon loading, the resistance to the applied load is internally reacted along notional "thrust lines", generating shear stress along bed joints and horizontal tension in the bricks and perpends. There is not any specific code requirement for the vertical or the horizontal location of the test in the wall, but in relation to Fig. 1(b), if the test location is further up the wall than shown, the same scenario exists, and if it is closer to the free end of the wall than L_1 there is less resistance to the internal reaction. When the distance to a free end is greater as is shown by the dimension L_2, there is likely to be greater resistance unless there is (for example) a damp-proof membrane (DPM) at the wall/footing interface, which significantly reduces the shear capacity. This scenario was demonstrated in the in-situ tests reported in Burton et al. (2019), in which the wall slid on the footing (no DPM present) following the load release at failure (of the wall, not the test brick). This failure was reflected in a bending crack which propagated above the test location, stopping a few courses before the top of the wall as shown in Fig. 3(a) and sliding of the wall on the footing approximately 6mm as shown in Fig. 3(b).

(a) vertical bending crack (b) masonry "pushed" off footing

Fig. 3 – Wall cracking caused by the shove test in an existing building

A further selection criterion for the test location is that where flat jacks are to be used they must be installed parallel to each other, requiring joint alignment to be carefully checked.

2.1.2. Installing flat jacks

When using flat jacks, the bed joints two courses above and two courses below the test brick need to be "raked out" to allow insertion of the jacks. If the mortar is relatively soft, this work can be undertaken with minimal disturbance to the test area. However, with harder mortars, such as those containing some cement, significant damage can result. Furthermore, removal of the nominal three brick lengths of bed joint will remove all vertical stress on the joint. For the upper flat jack, this will allow dilation of the wall above the raked joint downwards into the flat jack slot as well as upward dilation of the five courses in the test area into the same joint, with similar (but reverse) actions at the lower flat jack slot. Whilst these displacements and subsequent negation of the displacement by application of load from the flat jacks are unlikely to impact measurements of the friction component in the test joint, it is possible that some of the adhesion (and consequently cohesion) between the brick and mortar will be lost.

2.1.3. Installing loading jack

To allow insertion of the loading jack and a clear space for the test brick to move, it is necessary to remove either: (a) one brick from each side of the test specimen, or (b) one brick from one side and a perpend joint from the other side. Similar to removing the joints for the flat jacks, the removal of either bricks or perpends creates a risk of damaging the joint(s) above and below the test brick, particularly if harder mortars are involved. It also alters the vertical stress state on and around the test brick, as discussed in the next section.

After the space is cleared, the loading jack along with a spherical seat, bearing plates and a load cell (if hydraulic pressure is not being used to determine jacking force) need to be installed. To avoid an eccentricity moment due to misalignment (Fig. 4), the jack needs to be aligned concentrically along the centreline of the tested unit, both in plan and elevation.

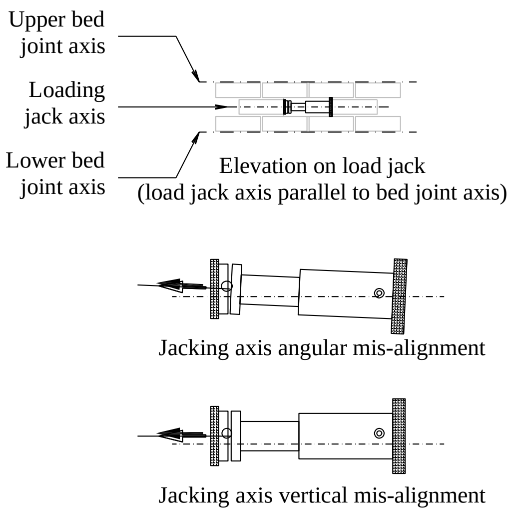


Fig. 4 – Required alignment of the loading jack in the shove test

2.1.4. Interpretation of results

Under Method A of ASTM C1531-16 (ASTM International 2016), the test brick is subjected to an initial vertical stress using flat jacks, and a horizontal jacking force is applied until bed joint cohesion is broken and the brick becomes displaced horizontally. With only frictional resistance remaining, the test is then repeated at different levels of normal stress (applied by flat jacks), relocating the horizontal jack to the opposite side as necessary to avoid running out of travel. This second phase of the test is used to estimate the coefficient of friction (μ) by means of a regression using the formula $\tau_{res} = \mu\sigma$, where τ_{res} is the residual frictional resistance and σ is the vertical normal stress. An alternative approach to obtain the friction coefficient is from the residual stress during the post-peak phase of the initial push, which is similar to the interpretation adopted in Method B of ASTM C1531-16. Finally, the initial bed joint strength at zero normal stress (τ_0) is calculated using the ultimate stress measured during the initial push (τ_u) and the friction coefficient estimated in the second phase, by projecting the shear stress back to the y-intercept of the τ-σ Mohr-Coulomb relationship:

$$\tau_u = \tau_0 + \mu\sigma \quad (1)$$

Being an in-situ test, one of the major limitations of the shove test is uncertainty in the amount vertical stress within the test brick. This is because firstly, the roughness along the joint surface results in dilatancy normal to the failure plane which generates additional vertical stress due to passive confinement by the surrounding masonry. And secondly, the removal of the adjacent bricks causes re-distribution of the vertical load path in the masonry in proximity to the test brick.

To account for these effects, ASTM C1531-16 suggests that a factor of 1.7 (calibrated by finite element modelling) should be applied to the flat-jack stress to obtain the average stress in the test brick. However, this value is applicable only to the test arrangement of Method A (removal of two bricks), and re-calibration of this factor would need to be undertaken if the specific arrangement is not achieved. Because of these issues and the argument that a zero force in the flat jacks does not necessarily imply zero vertical force across the test brick, Graziotti et al. (2018) suggest that the ASTM method is likely to overestimate τ_0. Simplified and refined approaches are then proposed by Graziotti et al. (2018) to better account for the stress concentration and non-zero vertical stresses from the in-situ overburden to estimate τ_0 more reliably.

2.2. Shear couplet and triplet test

The standard laboratory approach to shear testing of masonry joints is by the shear triplet or couplet test, as codified for example in EN 1052-3:2002 (British Standards Institution 2002). The triplet test involves a specimen of three bricks stacked vertically, and is performed by subjecting the specimen to vertical precompression, restraining the two end-bricks, and applying a horizontal force to the central brick until the joints above and below fail and the brick is displaced. The couplet arrangement is shown schematically in Fig. 5 and involves the testing of only a single bed joint. Both tests allow for the quantification of the initial shear

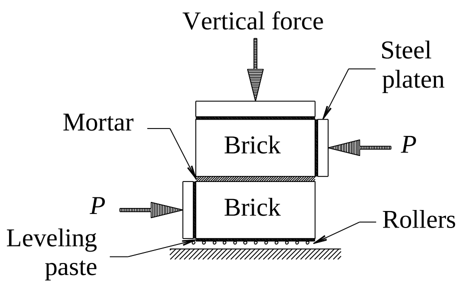


Fig. 5 – Shear couplet test, general arrangement

strength (τ_0) and the coefficient of friction (μ), and are regarded as the most reliable tests available for quantifying these mechanical properties, allowing for the highest degree of control over the loading arrangement and boundary conditions. Importantly, this includes the ability to impose a known vertical precompression that can be maintained at a constant level for the duration of the test.

Arguably, the couplet test offers several advantages over the triplet: the triplet introduces flexural normal stresses along the tested joints as a result of eccentricity between load and support, whereas the couplet test can minimise any normal stresses as long as the line of applied force is aligned along the bed joint. Furthermore, in the triplet test, one joint will usually fail before the other, which means that if the precompression load is insufficient, the two segments can rotate before the second joint can be failed and the frictional phase is reached. By contrast, the couplet may require a slightly more elaborate 'shear box' type arrangement in order to realise the aforementioned advantages (as used in current study) or the addition of extra rollers and supports as suggested by Popal and Lissel (2010).

Although these tests are considered the most reliable in terms of control over the test parameters, applying them to field masonry is often impractical due to the need to have access to a suitable lab test apparatus, and if not undertaken carefully, the extraction and transportation of the samples can damage them prior to testing. Also, both the triplet and couplet tests are moderately invasive as they require significant samples.

2.3. Other test methods

2.3.1. Brazilian test on core samples

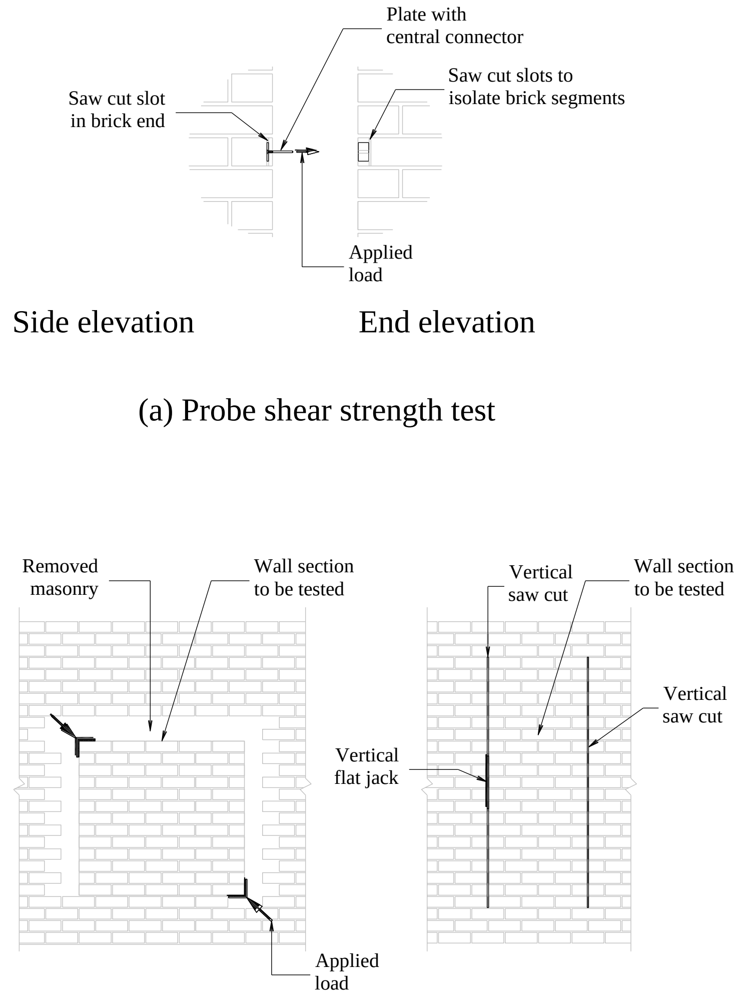
A relatively minimally invasive lab test that can be performed on field specimens is the Brazilian test on drilled cored masonry samples passing through the bed joint (e.g. Marastoni et al. 2016). The test can be undertaken under varied angles of inclination of the joint to the applied load, enabling for the characterisation of the mechanical properties of the mortar. Jafari et al. (2020) have further developed this test by using a displacement-controlled loading protocol with a slow sliding rate (0.05 μms^{-1}) to enable observation of the brittle fracture process. These modifications allow for insight into the post-peak softening in the non-linear sliding behaviour along the brick-mortar interface. However, similar to other test methods that involve extraction of samples to be removed back to the laboratory, the loss of vertical stress on the sample and the potential for damage during removal and transport need to be considered.

2.3.2. In-situ test by Pekmezci and Pekmezci

A novel in-situ technique for quantifying the local bond strength of the bed joint proposed by Pekmezci and Pekmezci (2021) is shown in Fig 6(a). The test involves cutting slots into a corner masonry unit to isolate a small segment of the brick, and pulling it out using a thin plate, thereby failing the remaining joint above and below in shear. The method is minimally invasive, and the reported results show good correlation with triplet tests. However, the method has a number of drawbacks: the test joint area is small and thus subject to localised variability in material properties; the arrangement introduces bending moment which is maximum at the tested joint interface; and the test is limited in its use to external corners where weathering and construction quality are likely to influence the localised material strength. Thus the shear stress capacity measured with this approach is likely to underestimate the actual capacity expected for the remainder of the wall.

2.3.3. In-situ shear tests on overall panels

While the preceding tests are intended for characterising the local properties at the joint interface, in-situ tests to determine overall shear strength of masonry panels under in-plane loading are also available, and thus are briefly discussed here for completeness. These include in-situ versions of the diagonal compression test (e.g. Dizhur and Ingham 2013) and



(a) Probe shear strength test

(b) Diagonal compression test

(c) Shear-compression test

Fig. 6 – Schematic layout of various in-situ tests

the shear-compression test (e.g. Armanasco and Foppoli (2020)). The diagonal compression test applies a load diagonally through a wall panel as shown schematically in Fig. 6(b). As can be seen from this figure, the test is highly destructive requiring significant amounts of the masonry to be removed to allow the test apparatus to be installed. The shear-compression test (Fig. 6c) requires a similar extent of intrusion in order for the wall to be isolated and for the horizontal shear load to then be applied. Each of these tests ultimately results in significant structural damage to the tested panel, requiring major repair. Understandably, when heritage or any other structures of value are involved, such destructive tests may be unsuitable. Finally, it is important to note that these large-scale tests are concerned with the global behaviour of overall wall panels. Therefore, the properties derived, such as the diagonal tensile strength of the masonry (f_t), are macro-properties embodying not just the mechanical behaviour of the material but also the geometric interlock of the masonry. Nonetheless, such tests can be vital for the reliable estimation of in-plane shear strength encapsulating the composite behaviour of the masonry, particularly in masonry with strong joints and weak units.

3. Overview of the pull test

The review of existing test methods in Section 2 has highlighted a range of issues, such as intrusiveness, potential damage to the specimens during test preparation, limitations regarding the test location, and difficulties with interpretation of the results. To address these limitations a new, alternative in-situ test—the *pull test* as shown in Fig. 7 is proposed.

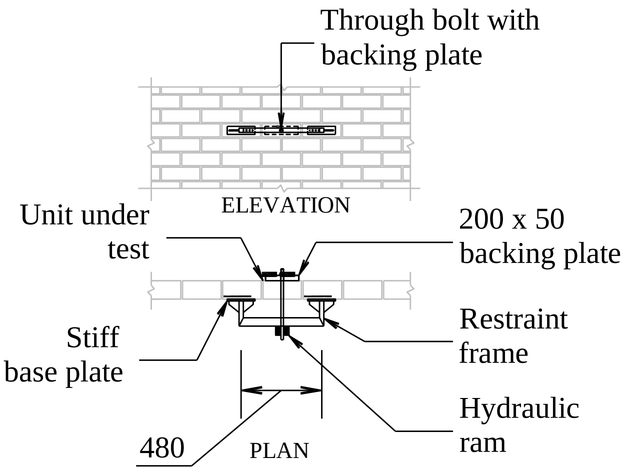


Fig. 7 – In-situ pull test general arrangement

The on-site setup of the test, which has been field-trialled by the authors as part of field investigations on vintage masonry in South Australia (Burton et al. 2020) does not require the removal of any portion of the wall, making it less invasive than other techniques.

The concept of the pull test is similar to the shove test, but unlike the shove test where the brick is loaded in the plane of the wall, the pull test loads the brick out-of-plane. With reference to Fig. 7, a hole is drilled through the test unit and a through bolt inserted, and a stiff backing plate is then mounted on the through bolt on the back of the wall. A restraint frame with stiffened base plates which reacts against horizontally adjacent bricks is installed, supported independently of the wall. A hydraulic cylinder is then fitted to the restraint frame which is then connected to the through bolt and the setup is then able to be loaded until and beyond failure of the mortar joint around the test unit. If the geometry of the restraint frame and the hydraulic cylinder have been sized appropriately, the test unit can continue to be loaded until it is removed completely from the wall.

3.1. General advantages and limitations

The backing plate loading arrangement (Fig. 7) means that the test requires access to the back face of the wall, making it most suited to single-wythe walls, or to cavity walls where it is possible to cut through the second wythe.

The pull test, as it has currently been trialled, addresses some of the general limitations associated with existing tests which have been discussed in Section 2, in that:

- The test is minimally invasive, particularly if testing is in a single-wythe wall with ready access to the back face.
- The test can be used on most types of masonry as long as a suitable bedding can be provided for the backing plate and the reaction points of the restraint frame. However

as with the other testing methods it cannot be performed on masonry units whose units are excessively irregular in shape, such as ‘rubble’ masonry.

- The test does not require raking-out of joints or perpends nor removal of masonry units or sections of wall, thus eliminating any redistribution of vertical load during setup and reducing the potential to cause damage to the joints during preparation.
- Since the test does not require removal of surrounding masonry, it is suitable for use with soft and hard mortars.
- Unlike the shove test, which generates in-plane thrust, the reaction to the applied load is self-contained within the test apparatus, thus minimising impact on the surrounding masonry, with the possible exception of joint expansion around the brick due to asperities along the failure plane. Consequently the test is less restricted by the test location.

Some limitations remain however, and should be understood before using the test.

- As with the shove test, it is difficult to reliably estimate the vertical stress acting across the test brick in-situ within the wall. For this reason, the ultimate shear stress measured by the test needs to be treated as an aggregate combination of cohesive and frictional components at the location under consideration. That is, in the context of the Coulomb relationship [Eq. (1)], the individual components τ_0 and $\mu\sigma$ cannot be easily separated, although the frictional term ($\mu\sigma$) could be estimated by undertaking the test at different heights along the wall, as long as a statistically sufficient number of (τ_u) data points, covering a spread of σ, can be obtained.
- Again, similarly to the shove test, even if the initial vertical stress in the undisturbed condition is known, once the brick commences sliding, the combination of dilatancy across the failed surface and confinement by the surrounding masonry causes an unknown increase to the vertical stress during the post-peak phase. This means that it is not possible to reliably estimate the frictional term ($\mu\sigma$) from the residual resistance.
- Pulling the brick in the out-of-plane direction creates a tendency for the adjacent bricks immediately above and below the test brick to rotate outwards, which could affect the vertical stress within the test unit, and distort dilation measurements if being monitored.

3.2. Selection of test location / brick

In its practical implementation, the test is not limited by the location within the wall. However, to ensure reliable results, there are certain issues that should be considered when selecting the test brick location:

As the pull test does not involve the removal of mortar joints around the test unit, the extraction of the brick following initial cohesion failure generates an in-plane bursting force due to a combination of dilatancy from irregularities along the failure surfaces (Andreotti et al. 2019) and confinement by the surrounding masonry. Consequently, in order to avoid a potential weakening effect, the test should not be conducted in close proximity to free edges or openings in the wall, unless those locations reflect the intended purpose of the test (e.g. design of anchors at similar locations).

As mentioned earlier, since the test provides an aggregate measurement of the cohesive and frictional components of shear strength, testing at various heights (different σ) could be conducted to attempt to separate these components. However, if doing so, the test locations should be selected to maintain other controllable parameters constant; for example, all test bricks should be equally distant from any free edges.

In existing masonry buildings, particularly vintage, brick units can often be variable with regard to their shape and quality. Thus in field application, the test brick should always be chosen with the view of providing representative results, for instance, it should appear whole, be similar in size to the other bricks, and not exhibit any obvious cracking.

3.3. Loading apparatus

To install the loading apparatus, a hole is drilled through the centre of the test brick, a threaded rod inserted through, and a back-plate secured onto the rod. The purpose of the back-plate is to apply a uniform bearing pressure and minimise any local bending of the brick. Thus the plate must be sufficiently thick and should have $L \times h$ dimensions close to but slightly less than the brick. In addition, it is recommended that the bearing plate on the back face should not be replaced with an adhesively attached plate on the front face, so as to avoid generating direct tensile stresses.

The loading apparatus used to pull on the threaded rod consists of a hydraulic ram fitted onto a reaction frame that in turn reacts horizontally against the wall. To minimise local out-of-plane bending of the wall, the frame should react as close as possible to the test brick without interfering with it. In the configuration used in the accompanying lab tests (shown in Fig. 7), the frame reacted against the adjacent bricks via stiff reaction plates to minimise local stress concentrations. Ideally, the loading apparatus (reaction frame and hydraulic ram) should be vertically supported independently of the wall. However, if the apparatus is fixed onto the wall itself for vertical support, then it should be sized as light and as close to the wall as possible, to minimise the eccentricity moment applied to the wall.

3.4. Test procedure and interpretation of results

In its simplest form, the test is undertaken by pulling the test brick until the initial shear failure of the joints around the perimeter of the brick following which, the brick is then extracted for at least 80% of its depth into the wall. This allows for the measurement of any secondary load peaks (which occur for example in frogged bricks) as well as for visual inspection of the joints after failure, for example to examine the influence of the perpends.

Often, the perpend joints are poorer quality compared to the bed joints, thus providing a lesser contribution to the overall joint capacity. However, if the perpends do provide some resistance, then ignoring it becomes un-conservative when inferring the shear stress capacity from the measured peak load and assuming only the bed joints to be contributing. Thus, if contribution of the perpends can't be clearly established, it should be assumed that they contribute the same stress capacity as the bed joints along their full area in order to obtain a lower-bound estimate of the ultimate shear stress. As such, the ultimate shear stress capacity of the bed joint (τ_u) should be calculated as

$$\tau_u = \frac{P_{\max}}{2Ld} \tag{2}$$

when contribution from the perpends is assessed as being negligible, or

$$\tau_u = \frac{P_{\max}}{2(L + h + 2t)d} \tag{3}$$

when perpend contribution is present, where $P_{\max}$ is the peak load, L is the length of the brick, d is the depth of the brick, h is the height of the brick and t is the thickness of the bed joint/perpend. In Eq. (3), it is assumed that the failure surface is central in the mortar joints and perpends surrounding the test unit, but judgement should be exercised as assuming a smaller failure perimeter (e.g. considering the perimeter of only the brick) is non-conservative. Thus, Eq. (2) should be used when there is clearly no contribution from the perpends and Eq. (3), with due consideration of the failure perimeter, should be used otherwise.

3.5. Results of field testing

Fig. 2 compares ultimate shear stress (τ_u) measured using the shove and pull tests at residential houses at three different in-situ test sites, as reported by Burton et al. (2020). The results are also summarised in Table 1 along with the results of the Student's t-test to establish statistical similarity between the measurement of the two tests. Disaggregation of the contributions from cohesion and friction was not attempted, as the tests were undertaken at roughly the same distance below the top of the wall, and thus had approximately constant vertical normal stress. Additionally, as the frictional contribution is expected in this instance to be at least an order of magnitude lower that the cohesion, insight into its influence could be clouded by the stochastic variability in the cohesion.

Table 1 – Results of in-situ brick shove tests and pull tests conducted on site (Burton et al, 2020). Statistical distributions are compared using the two-sample t-test with unequal variances.

	Ultimate shear stress τ_u (MPa)					
	Plain – Lime (PL)		Frogged – Lime (FL)		Frogged – Cement (FC)	
	Shove	Pull	Shove	Pull	Shove	Pull
Mean	0.25	0.25	-	0.41	0.89	0.56
Coef. of Variation	0.30	0.33	-	0.38	0.35	0.53
Count	6	6	-	8	6	6
	t-test for differences of two means					
Degrees of freedom	10		n/a		10	
t-statistic @ 95%	2.228		n/a		2.228	
t-score	0.142		n/a		1.88	
Null hypothesis (H_0)[1]	TRUE		n/a		TRUE	

1. Null hypothesis (H_0): mean of Test 1 (Shove) and the mean of Test 2 (Pull) come from the same distribution

The masonry at the first site comprised plain (unfrogged) bricks with lime mortar. The shove and pull tests produced almost identical strength measurements, both with a mean of 0.25 MPa and a similar coefficient of variation of ≈0.3. The t-test indeed confirms that the both tests produce measurements that can be considered to follow the same distribution. At the second site the masonry comprised frogged bricks with cement mortar. Here, the mean values produced by the shove and pull test were 0.89 and 0.56 MPa respectively. Despite the larger difference in means compared to the first site, the t-test still does not reject the null hypothesis that the two samples follow the same distribution at the 95% confidence limit. The third site had masonry built with frogged units with lime mortar. Only the pull test was undertaken at this site, as the purpose of the tests was investigation of the out-of-plane anchorage strength to which the pull test was considered more appropriate. The mean strength was 0.41 MPa, which was slightly lower than at the second site where cement mortar was used with the same brick type.

4. Laboratory test campaign - Methodology

Given the comparable values of ultimate shear stress measured by the shove and pull tests via on-site testing (Burton et al. 2020), a laboratory campaign was undertaken to evaluate the effectiveness of these in-situ tests under controlled conditions, while also benchmarking them against the shear couplet test. Complementary material testing was also undertaken. This section describes the test methodology, and Section 5 reports and discusses the results.

4.1. Test masonry

All three types of shear test were undertaken on purpose-built masonry intended to match the construction materials encountered during the on-site testing, including the following three combinations of brick type / mortar type:

1. Frogged bricks with lime mortar, representative of typical vintage masonry (denoted FL);
2. Frogged bricks with cement mortar, representative of vintage masonry with stronger mortar (denoted FC); and
3. Plain bricks with lime mortar, also emulating vintage masonry, but intended to eliminate complexities arising from the presence of frogs (denoted PL).

The frogged clay bricks in the FL and FC series (shown in Fig. 8) were 'sandstock' units (fabricated by moulding) with dimensions 230 × 110 × 76 mm ($L \times d \times h$). The plain clay bricks in the PL series were manufactured by extrusion but exhibiting similar surface roughness to the frogged units, and having dimensions 230 × 110 × 70 mm. All specimens,

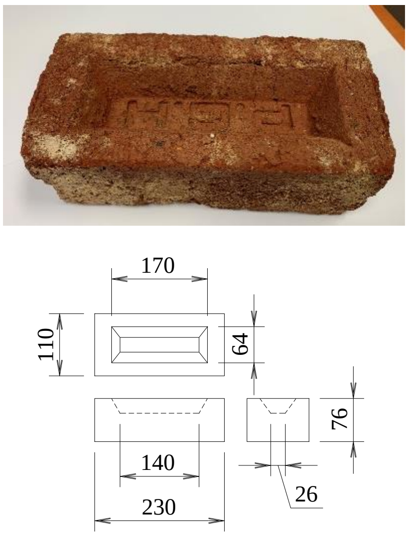


Fig. 8 – Frogged brick

including those used for material testing, were constructed on the same day by qualified brick layers. The lime mortar (FL and PL series) was batched as 0:1:3 (cement: lime: sand) and the cement mortar (FC series) as 2:1:9 (cement: lime: sand), with the constituents measured volumetrically. Water was added by the brick layers to achieve their desired mortar workability. The full list of constructed specimens is shown in Table 2. All specimens were left to cure for at least three months prior to testing.

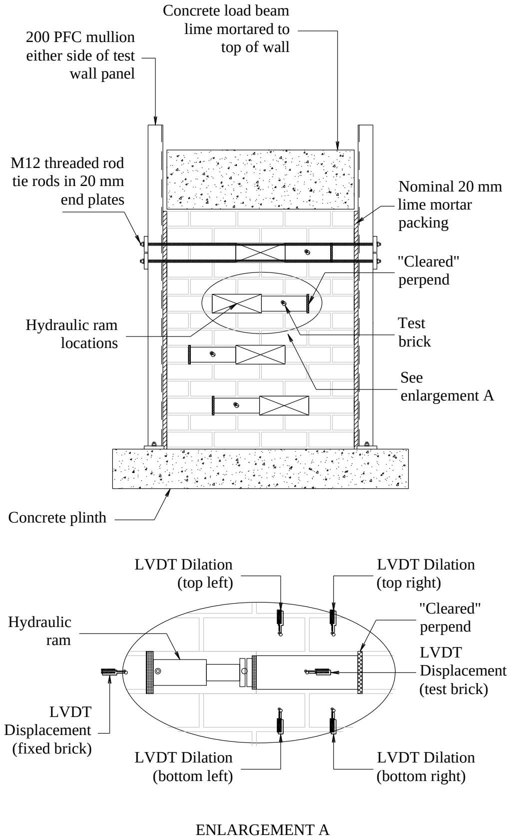
4.2. Shove tests

The shove tests were undertaken on laboratory-constructed wall panels, four bricks wide and 14 bricks high, mounted on a stiff concrete plinth with 200 PFC mullions on either side with lime mortar packing between the mullions and the wall panel, as shown in Fig. 9. Each wall panel provided a total of four test locations. A precompression load was applied at the top of the panel using a concrete loading beam that was mortared to the top course but allowed to move independently of the mullions. The total mass of the load beam was 126 kg, which, combined with the masonry above each test location produced a precompression stress (vertical stress) of approximately 0.017 MPa to each test unit (the tests were undertaken sequentially from top to bottom, relocating the load beam for each test). To minimise disturbance to the bed joints, flat jacks were not used, and the block-out for fitting the loading jack and also the perpends at the opposite end of the test brick were left as voids during the wall construction. Four M12 threaded tie rods (two each side of the wall segment) were attached in alignment with the brick being tested to provide lateral support to contain the thrust from the jacking load, and tightened to generate approximately 8 kN tension in each tie rod. The shear (jacking) load was applied using a 100 kN hydraulic ram at 2 kN/min, with a limiting displacement rate of 1 mm/min until the test was terminated.

Table 2 - Specimens used in the laboratory test campaign

Specimen type	Brick type	Mortar type	Number of specimens	Tests per specimen	Total tests
Shove test walls	Frogged	Lime	1	4	4
		Cement	1	4	4
	Plain	Lime	1	4	4
		Cement	-	-	-
Brick-pull test walls	Frogged	Lime	1	4	4
		Cement	1	4	4
	Plain	Lime	1	4	4
		Cement	-	-	-
Shear couplet tests	Frogged	Lime	6	1	6
		Cement	6	1	6
	Plain	Lime	6	1	6
		Cement	-	-	-
Prism compression tests	Frogged	Lime	3	1	3
		Cement	3	1	3
	Plain	Lime	5	1	5
		Cement	-	-	-
Bond wrench tests	Frogged	Lime	-	-	-
		Cement	-	-	-
	Plain	Lime	1	4	4
		Cement	-	-	-

Instrumentation comprised a 50 kN load cell attached to the hydraulic ram to measure shear load, four 10 mm linear variable differential transformers (LVDTs) to measure dilation on the courses directly above and below the test unit, one 25 mm LVDT to measure the lateral movement of the test brick , and one 10 mm LVDT to measure any displacement of the "fixed" brick. All LVDTs were supported from the concrete plinth, independently of the wall and mullions. The instrumentation locations are shown in the enlargement in Fig. 9(a).



(a) Schematic layout

(b) As constructed

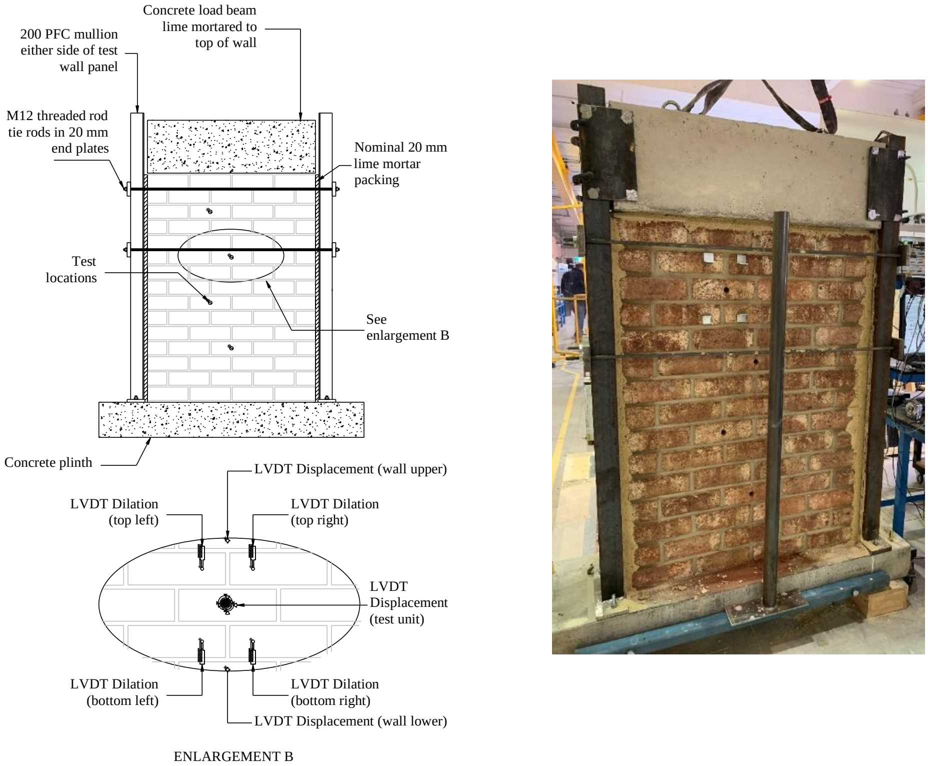
Fig. 9 – Shove test laboratory arrangement

After each test was conducted (at the uppermost test location), the concrete load beam, instrumentation and loading jack and load cell were removed. The lower pair of tie rods was then loosened and lowered to the line of the joint directly below the next test location, and re-tightened followed by the upper pair of tie rods. This process sought to limit any significant stress re-distribution with each iteration of the test. After the tie rods were correctly positioned

and tightened, the top three courses were carefully removed, the concrete load beam mortared back into place, instrumentation re-installed, and the test repeated at the next test location.

4.3. Pull tests

As with the shove tests, the pull tests were undertaken on wall panels four bricks wide but 15 bricks high to allow for slightly different placement of the tie rods. These walls were also constructed on stiff concrete plinths with 200 PFC mullions on either side and lime mortar packing between the mullions and the wall panel, as shown in Fig. 10. As with the shove test



(a) Schematic layout (b) As constructed

Fig. 10 – Pull test laboratory arrangement

setup, precompression load was applied at the top of the panel using a concrete loading beam that was mortared to the top course but allowed to move independently of the mullions either side. The same load beam as was used for the shove tests was used here, providing a precompression of approximately 0.017 MPa on the test brick. The tests were undertaken sequentially from top to bottom, relocating the load beam for each test. The holes to allow the through bolt and backing plate were drilled into the test bricks after the wall was constructed. Four M12 threaded tie rods (two each side of the wall segment) were used to resist any lateral thrust that may be generated during the brick pull operation, and as with the shove test, were tightened to produce approximately 8 kN tension.

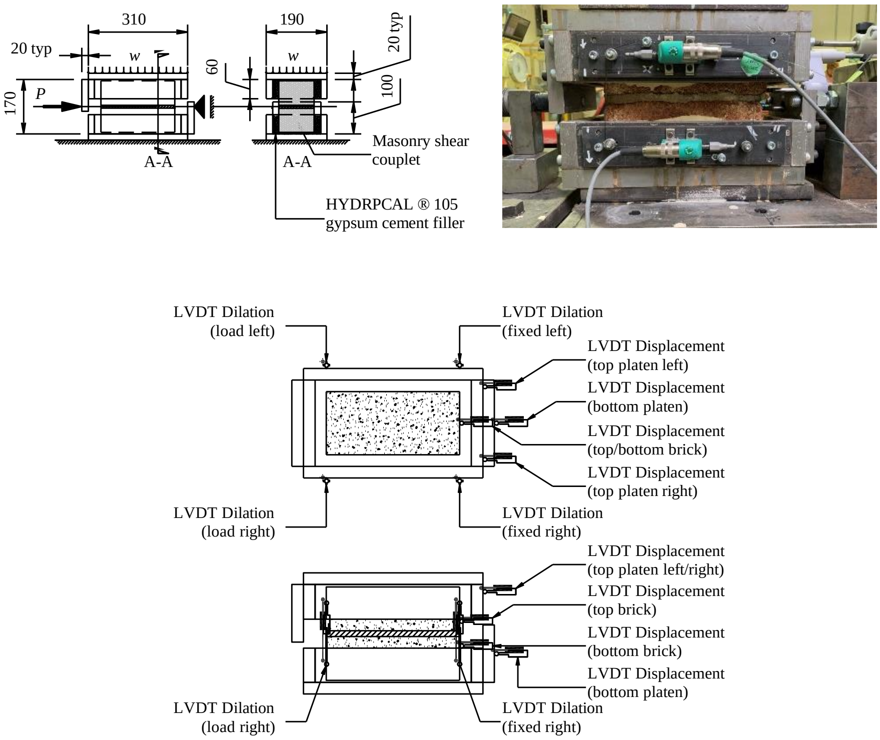
The brick loading arrangement comprised a 20mm thick backing-plate with a 16mm diameter threaded rod. The pull load was applied using a 100 kN hydraulic ram loading at 2 kN/min, with a limiting displacement rate of 1 mm/min until the test unit displaced 10mm and then at a maximum rate of 10 mm/min until the test was terminated.

Instrumentation consisted of a 50 kN load cell attached to the hydraulic ram to measure pull load, and four 10 mm LVDTs to measure dilation on the courses directly above and below the test unit. Two LVDTs were used to measure out-of-plane displacement of the wall one course above and one below the test location. A single 100 mm LVDT was used to measure the out-of-plane displacement of the test brick. All the LVDTs were supported on the concrete plinth via a stiff post which was independent of the wall and mullions as shown in Fig. 10(b), and the locations of the LVDTs are shown on enlargement B in Fig. 10(a).

Each wall panel provided four test bricks. In the same manner as for the shove test, after the test was conducted, the concrete load beam, instrumentation, loading jack, load cell and reaction frame were removed and the lower pair of tie rods loosened and lowered. After the tie rods were correctly positioned and tightened, the top three courses were carefully removed, the concrete load beam mortared back into place, instrumentation etc. re-installed and the test was repeated at the next location.

4.4. Shear couplet tests

Shear couplet tests were performed using a shear-box arrangement that applied the horizontal load along the line of the tested mortar joint. The specimens were mounted within rigid steel ‘boxes’ top and bottom (Fig. 11) and potted in gypsum cement filler to minimise local stress



(c) – Instrumentation

Fig. 11 – Shear couplet test arrangement

concentrations and potential difficulties associated with resisting eccentricity moments due to the shear-box arrangement (Montazerolghaem and Jaeger, 2014). The entire test assembly containing the couplet was mounted on a manually driven worm-drive apparatus allowing a near-constant load rate to be used, and avoiding “over-run” issues associated with a hydraulic system. Additional mass was added to the test apparatus such that the precompression on the couplet bed joint was approximately equal to the precompression on the bed joint at the base of the test unit for each of the pull and shove tests (approximately 0.018 MPa). As with the shove tests and the pull tests, the shear load was applied at a rate of 2 kN/min, with a further restriction of 1mm/min after failure until the test was terminated.

Instrumentation for these tests comprised two 10 mm LVDTs measuring longitudinal displacement of either side of the top platen, one each 10 mm LVDT measuring the central longitudinal displacement of the top brick, the bottom brick and the bottom platen. Four 10 mm LVDTs, one in each corner of the shear box arrangement, measured dilation (or contraction) in each corner of the test. A single 25 mm LVDT was used to measure overall machine displacement and a 50 kN load cell and the machine load cell were used to measure the applied load. The arrangement of the instrumentation is shown in Fig. 11(c).

4.5. Prism compression tests

Compression tests were performed on 5-brick-tall masonry prisms as shown in Fig. 12 to quantify the unconfined compressive strength of the masonry (f_{mc}) and the Young's modulus of elasticity (E_m). Instrumentation consisted of four 25 mm LVDTs, one in each corner, measuring vertical displacement, with the load measured by the testing machine. The specimens were placed on 6 mm thick plywood shims and were loaded from a fixed platen above onto a platen with spherical seat below to promote a uniform application of compressive stress under loading.

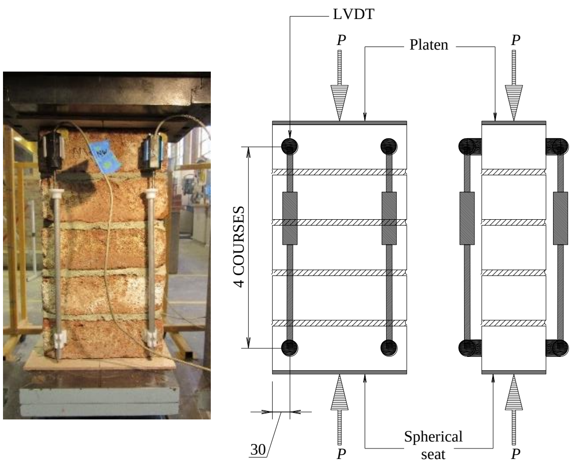


Fig. 12 – Compression test setup

4.6. Bond wrench tests

Bond wrench tests were undertaken to quantify the flexural tensile strength of the masonry (f_{mt}). The tests were only performed on the plain units/lime mortar masonry series (PL), as it was not considered of much value to undertake the test on frogged-unit masonry due to variability to which the brick frogs fill with mortar (bricks are generally laid with frogs down). These tests were performed on a 5-brick-tall stack in which the joints were tested from top to bottom, allowing four tests to be undertaken. The arrangement for the test was in accordance with procedure detailed in the Australian Standard *Masonry structures* (Standards Australia 2018).

5. Laboratory test campaign – Results and discussion

5.1. Material testing

Results of the prism compression tests including the unconfined compressive strength of the masonry (f_{mc}) and the Young's modulus of the masonry (E_m) are summarised in Table 3. The f_{mc} data is also shown graphically in Fig13(a). The weakening influence of the frog is apparent, as the PL specimens exhibited significantly higher compressive strength than the FL and FC

Table 3 – Results of laboratory tests

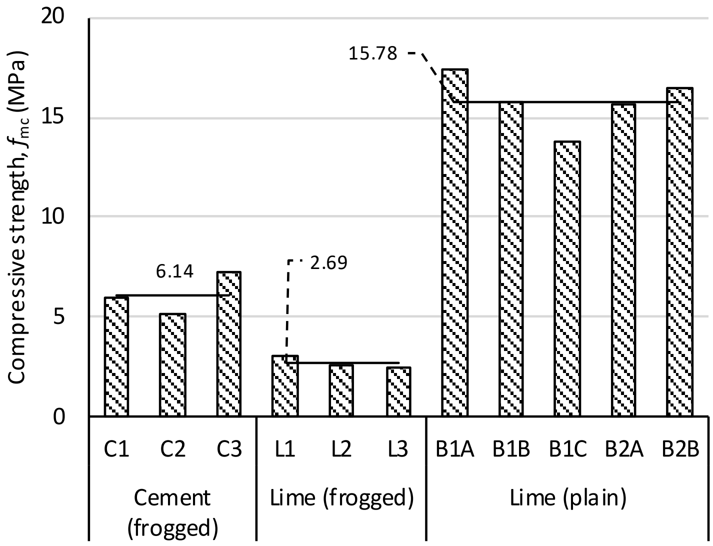
Test	Property [1]	Data points	Frogged Bricks / Cement Mortar (FC) Mean	CoV	Count	Frogged Bricks / Lime Mortar (FL) Mean	CoV	Count	Plain Bricks / Lime Mortar (PL) Mean	CoV	Count
Pull test	Ultimate shear stress, τ_u (MPa)	All	0.69	0.26	4	0.13	0.51	4	0.26	0.08	4
		Note 2	-	-	-	0.23	0.27	2	-	-	-
Shove test	Ultimate shear stress, τ_u (MPa)	All	0.39	0.26	4	0.16	0.08	4	0.23	0.08	4
	Residual shear stress, τ_{res} (MPa)	All	0.24	0.24	4	0.11	0.12	4	0.13	0.06	4
	Coefficient of friction μ [3]	All	14.2	0.24	4	6.60	0.12	4	7.72	0.06	4
Couplet test [4]	Ultimate shear stress, τ_u (MPa)	All	0.25	0.49	6	0.13	0.28	6	0.31	0.06	6
		Note 5	0.29	0.27	5	-	-	-	-	-	-
	Residual shear stress, τ_{res} (MPa)	All	0.025	0.25	6	0.023	0.11	6	0.018	0.12	6
	Coefficient of friction μ	All	1.38	0.25	6	1.26	0.11	6	1.02	0.12	6
Prism compression	Compressive strength, f_{mc} (MPa)	All	6.14	0.17	3	2.69	0.11	3	15.8	0.09	5
	Young's modulus, E_m (MPa)	All	3,490	0.12	3	726	0.56	3	1,250	0.09	5
Bond wrench	Flexural tensile strength, f_{mt} (MPa)	All	-	-	-	-	-	-	0.34	0.07	4

Notes

1. All stress capacities are calculated using the gross section (ignoring any frogs)
2. Excludes tests without lateral support of the test panel
3. Calculation of coefficient of friction assuming uniform vertical stress of 0.017 MPa acting on test unit
4. Vertical stress of 0.018 MPa applied to test specimens using gravity load
5. Excludes outlier result (Pull-FC-01)

specimens, with mean values of 15.8, 2.69 and 6.14 MPa respectively. This is a combination of the incomplete filling of the frog leading to a smaller bearing area, as well as generation of lateral bursting force by the mortar plug. This resulted in the typical failure pattern shown on Fig. 13(b)(i) compared to the plain unit failure in Fig. 13(b)(ii).

The results of the bond wrench test are also summarised in Table 3. This test was conducted only on PL specimens which was found to have a mean flexural tensile strength (f_{mt}) of 0.34 MPa with a coefficient of variation of 0.07 indicating a comparatively low variability for this property (e.g. refer to Griffith and Vaculik 2007).



(i) Frogged

(ii) Plain

(a) Unconfined compressive strength

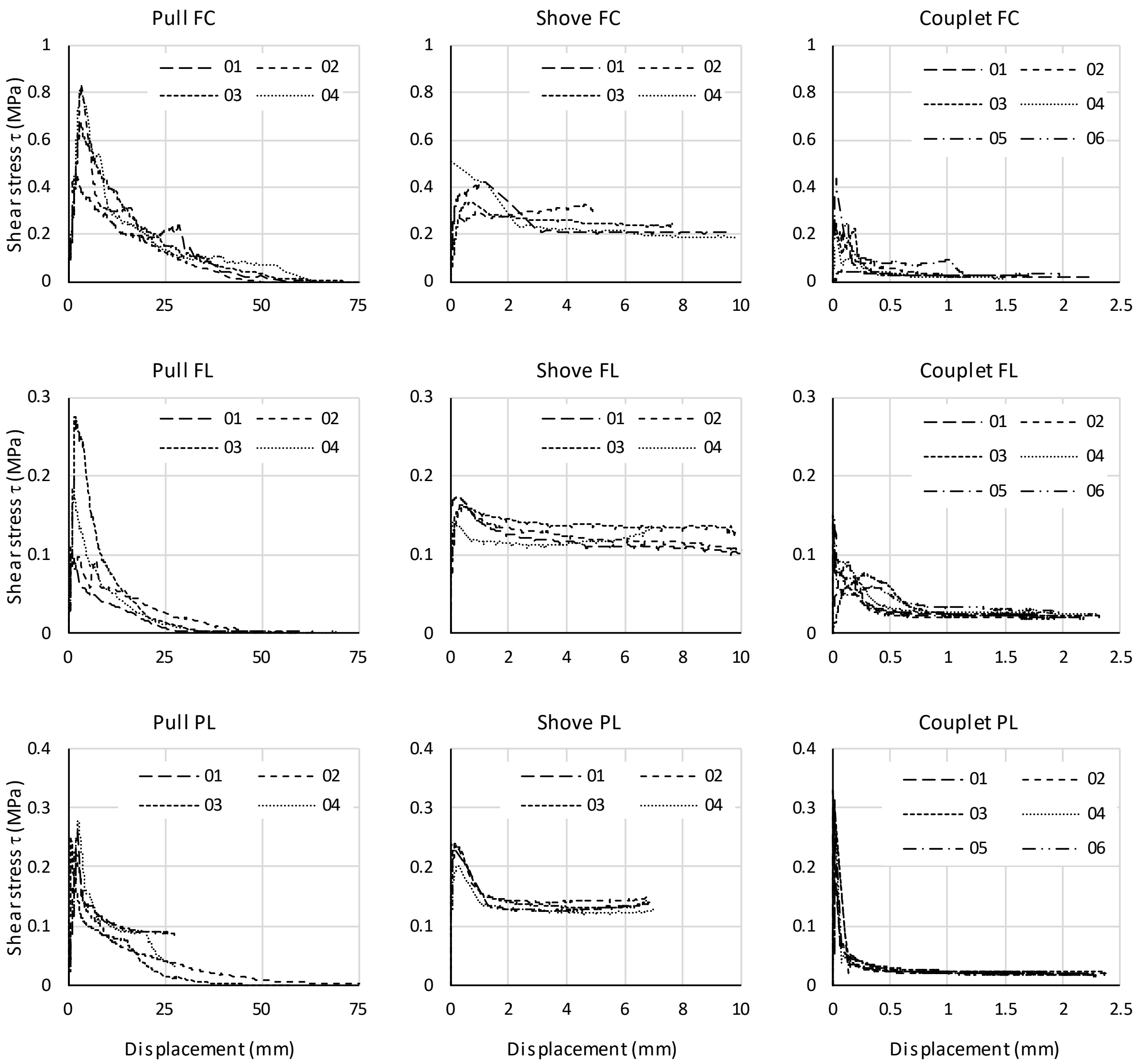
(b) Failure mode

Fig. 13 – Results of compression test (compressive stress calculated using gross section ignoring any frogs)

5.2. Pull test

The statistics for ultimate shear stress (τ_u) for these tests are presented in Table 3, and the shear stress vs displacement (τ–Δ) plots are shown in Fig. 14(a). Note that all stress including the stress plotted in Fig 14 was calculated using the gross area of the bed joints above and below, ignoring any reduction in overlap as the brick was pulled out. The first two pull tests for thefrogged bricks with lime mortar were conducted without lateral restraint (the tie rods were not used) and resulted in unrepresentatively low values of τ_u. Table 3 also shows the statistics for these tests with the first two data points removed which shows a mean ultimate shear stress of 0.69, 0.23 and 0.26 MPa for the FC, FL and PL series respectively based on Eq. (2) considering the contribution from perpends to be negligible in all tests.

Note that in the pull tests, the maximum applied displacement was considerably larger than in the shove and couplet tests. This was so that the failed surface including the perpends could be visually inspected.



(a) Pull test

(b) Shove test

(c) Shear couplet test

Fig. 14 – Shear stress (τ) vs shear displacement (Δ) response in the laboratory pull, shove and shear couplet tests (shear stress calculated using full gross area ignoring any frogs)

From the τ–Δ plots in Fig. 14(a), the following characteristics are observed to be common to all three masonry type series:

- The initial failure peak occurs at 2 to 3 mm displacement. This is a seemingly large displacement but it includes "take up" of the test equipment which cannot be reliably isolated and removed from the data set. The peak corresponds to the cohesion failure of the first and rapidly following, the second bed joint, after which the shear force resistance rapidly diminishes.
- Following the peak load, each test exhibited a continual drop in resisted shear force toward zero, which unlike the shove and couplet tests did not converge to a stable frictional plateau. This can be explained as follows: Unlike in the couplet test, where

the vertical force across the test unit remains constant, in the two in-situ tests the surrounding bricks in the wall provide an alternate path for the overburden axial load. The grinding of the asperities along the bed joint of the test brick in both the shove and pull test reduces the effective vertical stiffness of the test brick, thus causing more of the vertical load to be diverted to the surrounding bricks and reducing the normal stress across the bed joint contact area. This, in combination with by the progressive reduction in the contact area as the brick is pulled out leads to the continually descending frictional force resistance observed in the pull test.

With reference to the Pull-FC group and to a lesser extent the Pull-FL group, secondary stress peaks following the main peaks can be observed. It is surmised that these secondary peaks are caused by break-out of the side of the brick due to bearing against the mortar plug as shown in Fig. 15. In the Pull-FL series, only a single specimen exhibits a secondary peak, as due to the weaker lime mortar there was a lesser potential to fail the brick unit. The tests on the plain brick (PL) do not exhibit this secondary peak due to the absence of a frog.

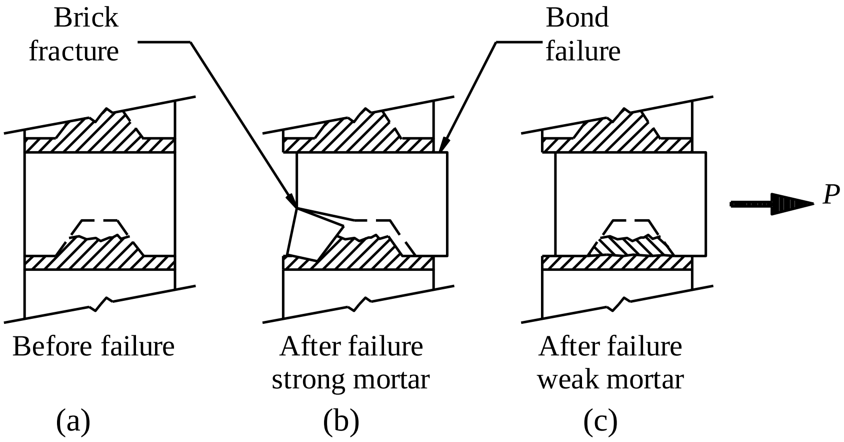


Fig. 15 – Local shear failure mechanisms in frogged units

Fig. 16 demonstrates the profile of vertical separation across the bed joints above and below the test brick for test Pull-PL-03, as a representative example of typical observed behaviour during a typical pull test. It is important to note that the measured expansion may not be ‘true’ unconfined dilation, as the vertical compressive load acting on the test unit is not maintained constant due to the passive confinement of the surrounding masonry. The plot demonstrates that the onset of significant dilation occurred only after the bond had been broken following the peak stress. Further, dilation gradually reduces with continued horizontal displacement, which can be explained by the progressive grinding and smoothing of the failure surface.

5.3. Shove test

Values of ultimate shear stress (τ_u) measured using the shove test are presented in Table 3, with the full τ–Δ data plotted in Fig. 14(b). The mean τ_u for the FC, FL and PL brick/mortar configurations was 0.39, 0.16 and 0.23 MPa respectively and the corresponding mean residual shear stress (τ_{res}) was 0.24, 0.11 and 0.13 MPa.

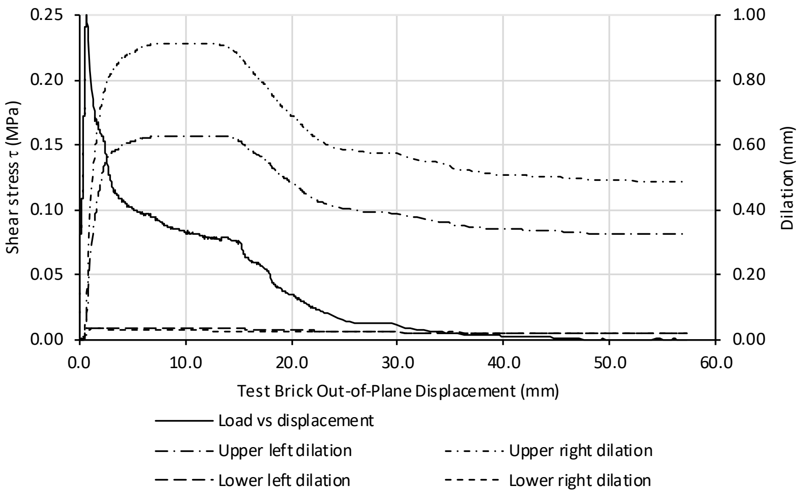


Fig. 16 – Typical response in a pull test in terms of shear stress (left vertical axis) and dilation (right vertical axis) versus shear displacement. Shown for specimen Pull-PL-03 (plain brick / lime mortar) (shear stress calculated using full gross area)

Calculating the coefficient of friction from the measured τ_{res} is complicated by the dilation across the failed bed joints interface altering the vertical stress through the test unit. If it is assumed that the precompression remains unaltered by dilation, mean values of the coefficient of friction (μ) of 14.2, 6.60 and 7.72 are obtained for the FC, FL and PL series respectively. These are unrealistically excessive by approximately an order of magnitude.

To obtain a more realistic estimate of the vertical force across the test unit during the post-peak phase, it has been assumed that the two brick courses above the test brick behave as a pin-supported beam exerting a point-reaction on the test brick. The details of the assumed configuration are presented in Appendix A. This leads to estimated μ of 0.38, 1.53 and 1.03 for the FC, FL and PL configurations respectively. These values are more realistic, but still demonstrate the difficulty/unreliability of estimating precompression following cohesion failure.

Common to each of the tests, as can be seen in Fig. 14(b) is the extended plateau of approximately constant residual shear stress following cohesion failure. This suggests that the failure mechanism is comparable between the different brick/mortar configurations. However, as can be seen from the preceding discussion, consistency in the results does not necessarily imply accuracy.

Unlike the dual stress peaks seen in the pull test, the influence of the frogs can be seen in the manifestation of an extended failure zone in most of the shove tests performed on the frogged-unit specimens (FC and FL), and is perhaps most apparent in test 01 of the FC series. Similar fracture of the frogged masonry units to that observed in the pull test also occurred in the shove tests as can be seen in Fig. 17, but as a result of the platen used to load the brick (loaded the full area of the end of the brick), the failure following brick fracture was more progressive in the latter. There is no evidence of this occurring with the plain-brick series (PL).

5.4. Shear couplet test

Values of ultimate shear stress (τ_u) measured using the couplet test are presented in Table 3, and the full τ–Δ data plotted in Fig. 14(c).

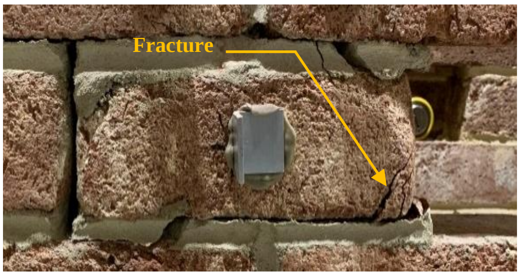


Fig. 17 – Typical localised fracture at the end of the brick observed in shove test on frogged units (bricks laid frog-down)

It is worthwhile to use this test to compare the three masonry series with regard to their ultimate shear stress (τ_u), as the couplet test provides the greatest control and consistency over the imposed boundary conditions and normal stress. Let us first compare the two frogged-unit series (FC and FL), which exhibited a mean strength of 0.29 MPa (cement mortar) and 0.13 MPa (lime mortar), respectively. The difference in strength can be explained by the cement mortar providing stronger interlock between the brick and mortar plug by the mechanisms shown in Fig 15, as well as stronger adhesion along the brick/mortar interface. Interestingly, comparing the two series with lime mortar shows that the series with plain units (PL) (0.31 MPa), outperformed the series with frogged units (FL) (0.13 MPa). This indicates that the benefit of having a full bonded area in the PL series outweighed the benefit of the mortar-plug interlock in the FL series but with a lesser bonded area. Examination of the failed surfaces also consistently showed the frogged-unit specimens (FL and FC) to fail along the bonded brick/mortar interface and the plain-unit specimens (PL) to fail through the mortar (Fig. 18), suggesting that the plain brick also provided stronger adhesion to mortar.

The mean residual shear stress of the FC, FL and PL series was 0.025, 0.023 and 0.018 MPa, respectively. These residual stress values are based on the gross plan area of the masonry unit, which given the constant vertical stress of 0.018 MPa, equates to a mean coefficient friction (μ) of 1.38, 1.26 and 1.02 for the three series, respectively.

Whilst the overall τ–Δ response measured via the couplet test is generally consistent with the pull and shove tests (at low Δ), the following notable features can be observed:

- Distinct secondary stress peaks occur in the frogged brick / cement mortar series (FC). Similarly to the other tests (shove and pull), this characteristic of the FC series can be explained by brick fracture following the initial cohesion failure caused by bearing ofthe mortar against the end of the brick (Fig.15b). Fig. 18(a)(i) shows a typical test in which the fractured portion of the masonry unit is clearly visible: in elevation (i) and after the test (ii).
- The secondary peaks are less pronounced in the FL tests as the lime mortar was generally weaker than the masonry unit, thus causing progressive shearing of the mortar plug rather than fracture of the brick (Fig. 15c). A typical failure in which the mortar plug remained within the frog is shown in Fig. 18(b).
- No such secondary peaks can be observed in the plain unit (PL) due to the absence of the frog. The resulting simple cohesion failure typical of the PL series can be seen in Fig. 18(c).

(i) – Elevation following failure

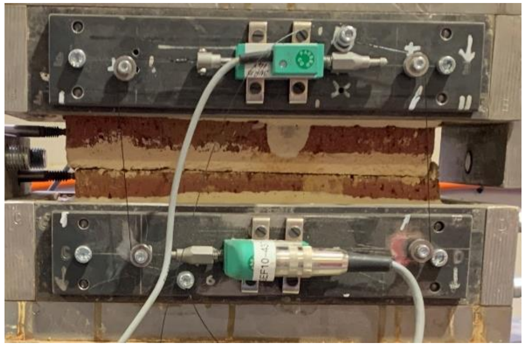
(ii) – Failed joint interface

(a) Frogged brick, cement mortar (FC)

(i) – Elevation following failure

(ii) – Failed joint interface

(b) Frogged brick, lime mortar (FL)



(i) – Elevation following failure

(ii) – Failed joint interface

(c) Plain brick, lime mortar (PL)

Fig. 18 – Typical joint failure observed in the shear couplet test

5.5. Comparison between pull, shove and couplet tests

Fig. 19 compares the data points and mean values of the ultimate stress τ_u for each of test types (shove, pull, couplet) within each of the masonry series (FC, FL, PL). Table 4 compares the pull and shove tests by benchmarking their respective means against the couplet, and

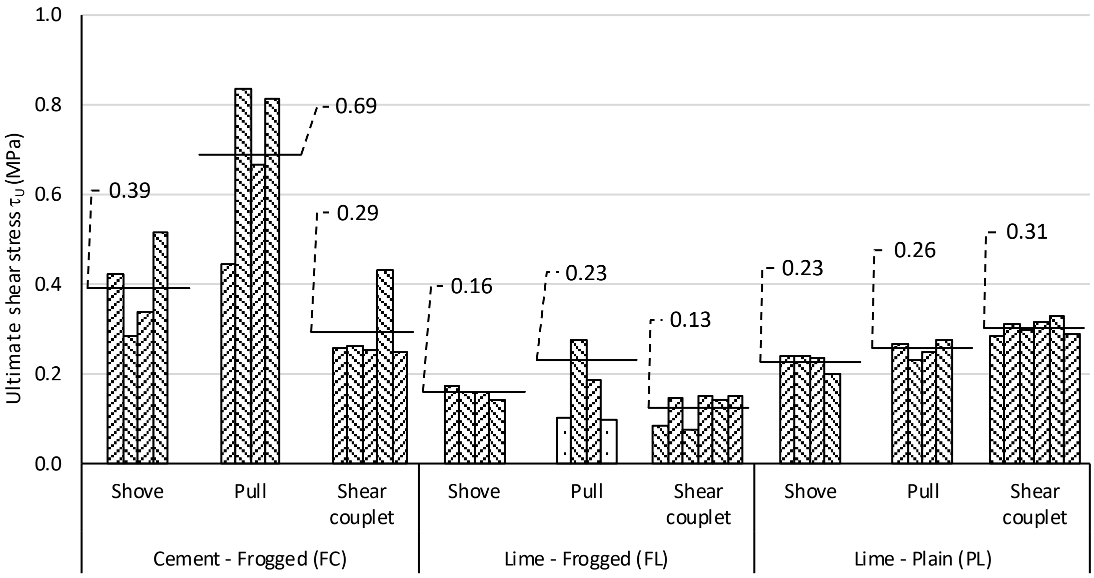


Fig. 19 – Ultimate shear stress measured using the shove, pull and couplet test, for the different masonry series (FL pull test outliers shown as not included in mean calculation)

summarises the results of the t-test conducted to establish whether the τ_u measurements from the different tests (shove, pull, couplet) could be considered statistically equivalent. The t-test examines the null hypothesis that the means of the sample pairs being compared (e.g. pull test vs couplet) are from the same distribution. Thus a *true* result indicates the samples being compared are statistically indistinguishable whereas *false* indicates they are statistically different at the 95% confidence limit.

5.5.1. Plain-unit series (PL)

Let us first compare the tests for the plain-unit series (PL), as this configuration avoids any complications due to the presence of the frogs. Among the different masonry types tested (PL, FC, FL) the PL series produces the most consistent results by each of the three tests. On average, the strength measured by the shove test is 75% of that from the couplet, and the pull test is 85% of the couplet (Table 4). However, despite the mean values of the three test types being relatively consistent (compared to the frogged-unit series), the t-test indicates that only the pull test and shove test are statistically equivalent, and that neither of these in-situ tests are equivalent to the couplet test. This is largely a consequence of the low scatter in data, regardless of the test method.

A possible explanation for why the in-situ tests (shove and pull) both produce slightly lower ultimate stress than the couplet for the plain-unit series is that the shear strength of a plain joint is dependent only on the cohesive and frictional components along the bedded interface [as per Eq. (1)] (unlike the frogged-unit series where the strength also relies on interlock between the

Table 4 – Comparison of the ultimate shear stress $\boldsymbol{\tau_u}$ (MPa) measured using different test methods. Statistical distributions are compared using the two-sample t-test with unequal variances.

	Frogged – Cement (FC)			Frogged – Lime (FL)			Plain – Lime (PL)		
	Shove	Pull	Couplet	Shove	Pull	Couplet	Shove	Pull	Couplet
Count	4	4	5	4	2	6	4	4	6
Mean ultimate shear stress τ_u (MPa) [1,2]	0.39	0.69	0.29	0.16	0.23	0.13	0.23	0.26	0.31
Percentage of shear couplet result (%)	135	236	100	126	181	100	75	84	100
Standard deviation	0.101	0.180	0.080	0.013	0.063	0.035	0.018	0.019	0.017
	Pull vs Shove	Pull vs couplet	Shove vs couplet	Pull vs Shove	Pull vs couplet	Shove vs couplet	Pull vs Shove	Pull vs couplet	Shove vs couplet
Calculated d_f	5	4	6	1	1	7	6	6	6
Two tailed t-score (95%)	2.571	2.776	2.447	12.706	12.706	2.365	2.447	2.447	2.447
Calculated t-statistic	2.915	4.293	1.594	1.568	2.356	2.088	2.094	2.546	6.538
Null hypothesis (H_0) [3]	FALSE	FALSE	TRUE	TRUE	TRUE	TRUE	TRUE	FALSE	FALSE

Notes

1. Mean shear couplet data factored by respective precompression stresses of shove and pull tests compared with shear couplet test
2. Ultimate shear stress calculated using the gross section (ignoring any frogs)
3. Null hypothesis (H_0): mean of Test 1 (e.g. Shove) and the mean of Test 2 (e.g. Pull) come from the same distribution

brick and mortar plug). Under initial shear deformation but prior to cohesive fracture, there is an initial tendency for the mortar joint to contract to maintain constant volume, a phenomenon observed in these tests and also in experiments by other researchers (e.g. Andreotti et al. 2018, Graziotti et al. 2018). This contraction is uninhibited in the couplet test; however, in the in-situ tests it is resisted by the confining effect of the surrounding masonry, thus inducing a tensile vertical stress component across the test unit. The resulting reduction in compressive normal stress across the test brick will act to diminish the frictional component in Eq. (1), thus weakening the joint.

As a measure of the sensitivity of the result, it can be demonstrated that, only a 2.5% increase to the ultimate shear stress data measured in the pull tests would produce statistical equivalence between the pull test and couplet according to the t-test. Similarly, a 12.5% strength increase in the shove test measurements would produce statistical equivalence among all three test types in the plain-unit series. The small number of tests undertaken is likely to have some further impact on the results, but this suggests that for plain-unit masonry with weak mortar, the pull test is an effective alternative to the shove test, and is slightly conservative when compared to the couplet test.

5.5.2. Frogged-unit series (FC and FL)

Comparing the ultimate shear stress measured by the different tests demonstrates the same overall trend with respect to both frogged-unit series, whereby pull test > shove test > couplet test. As can be seen in Table 4, the pull test measured ultimate strengths that were on average 135% and 80% larger than the couplet test for the FC and FL configurations respectively, while the shove test measurements were 35% and 25% larger than the couplet test. It is interesting but concerning that in the case of frogged units, both in-situ tests are shown to be unconservative compared to the couplet test, particularly as both in-situ tests were shown to be slightly conservative for plain units.

The primary cause of the measured shear strength disparity between the pull and couplet tests is thought to be compressive bearing due to the interlock between the mortar plug and inner side of the brick. Because the projected bearing area is substantially larger in the out-of-plane direction than in the in-plane direction, this effect leads to a disproportionate enhancement of strength in the pull test compared to the couplet and shove tests.

The higher strength measured in the shove test relative to the couplet test is more difficult to explain, as both were loaded in-plane and should therefore have experienced a comparable degree of interlock between the mortar plug and the brick. It is plausible, however, that in the shove test, the application of shear load caused wedging between the sloped interface between the mortar plug and inner side of the brick, which combined with confinement by the surrounding masonry panel generated additional vertical stress across the test unit, that in turn led to increased shear resistance. The increased strength in the shove test could also be partially due to the test unit carrying a slightly higher vertical normal stress due to the absence of the brick adjacent to the test brick within the panel (Fig. 9), and that this increase in vertical stress was sufficient to counteract any reduction in normal stress due to the initial contraction of the mortar joint (refer to discussion regarding the PL series).

6. Conclusions and recommendations

This paper has undertaken a laboratory study to investigate the effectiveness of a new in-situ approach for shear testing of masonry joints in which the joint is loaded out-of-plane, referred to here as the *pull test*. The appeal of the pull test is that it is simple to perform and minimally destructive to the surrounding masonry, and thus it could be used as a potential alternative to the conventional in-situ shove which loads the test brick in-plane. When applied to single-leaf masonry, the pull test requires only a single hole to be drilled through the test brick, and the damage caused by the test could be repaired by mortaring the brick back into the wall. Trials of the test in both laboratory and the field have also shown, unlike the shove test, the pull test does not generate any apparent damage to the surrounding wall. Thus the pull test is less restricted by the test location, and could be undertaken for example in the proximity to unsupported edges or in confined situations.

The pull test provides a measure of the ultimate shear strength along the masonry joint incorporating contributions from both cohesion and friction. Therefore, as with the shove test, one of its limitations is that it is difficult to disaggregate the results into these individual components, unless a large number of tests are performed at varying levels of known vertical stress. Nonetheless, the test is shown to perform with a comparable degree of repeatability to the shove test and laboratory couplet test.

The experimental testing undertaken in this study demonstrates that when applied to plain masonry units with weak (lime) mortar, the ultimate shear strength measurements obtained using the pull test are statistically equivalent with the shove test, and that are slightly conservative benchmarked relative to the couplet test (by about 15%). That is, for regular (rectangular) plain bricks combined with weaker mortars, the pull test has been shown to be interchangeable with the shove test, while producing a slightly conservative estimate of the actual joint capacity as per the couplet. This result also indicates that under these conditions, the out-of-plane shear strength could be considered a proxy for the in-plane shear strength of the bed joint (and vice versa). Further work is needed to establish if these conclusions remain valid if plain units are coupled with stronger (e.g. cement) mortars.

When applied to frogged units, the pull test produced higher strength measurements than both the shove and couplet tests. Considering frogged units with both lime and cement mortar, the strength measured by the pull test was on average about 110% higher than the couplet, whereas the shove test was on average about 30% higher than the couplet. This is surmised to be a result of the interaction between the mortar plug and the frog, which becomes more pronounced under out-of-plane loading compared to in-plane loading due to a larger interlocking bearing area. As such, the determination of whether the pull test is suitable or indeed unconservative in application to frogged units needs to be considered in terms of the intended purpose of the test. That is:

- if the purpose is to establish joint shear strength for design of out-of-plane anchorage, the pull test is suitable with frogged units; but
- if the purpose is to determine in-plane shear strength, then the pull test is unconservative for frogged units, and the shove test should be used instead.

Given these limitations, the pull test can be a suitable alternative to the shove test for determining the ultimate bed joint shear capacity of masonry. Its benefits are its simplicity of installation and less potential to cause damage than the shove test. In addition, the ability to use it near unsupported edges (vertical and horizontal) provides the opportunity to test the masonry in closer spatial proximity to where the measurements are required, for example in parapets and gable walls.

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Appendix A

Estimation of vertical stress on the test unit in the shove test

In the configuration adopted in the shove test (Fig. 9), a concrete loading beam was used to apply precompression to the test panel. In the undisturbed condition, this applies a uniform vertical stress at the location of the test unit, consisting of the weight of the concrete beam and the two masonry courses above the test unit, as shown in Fig. A1(a).

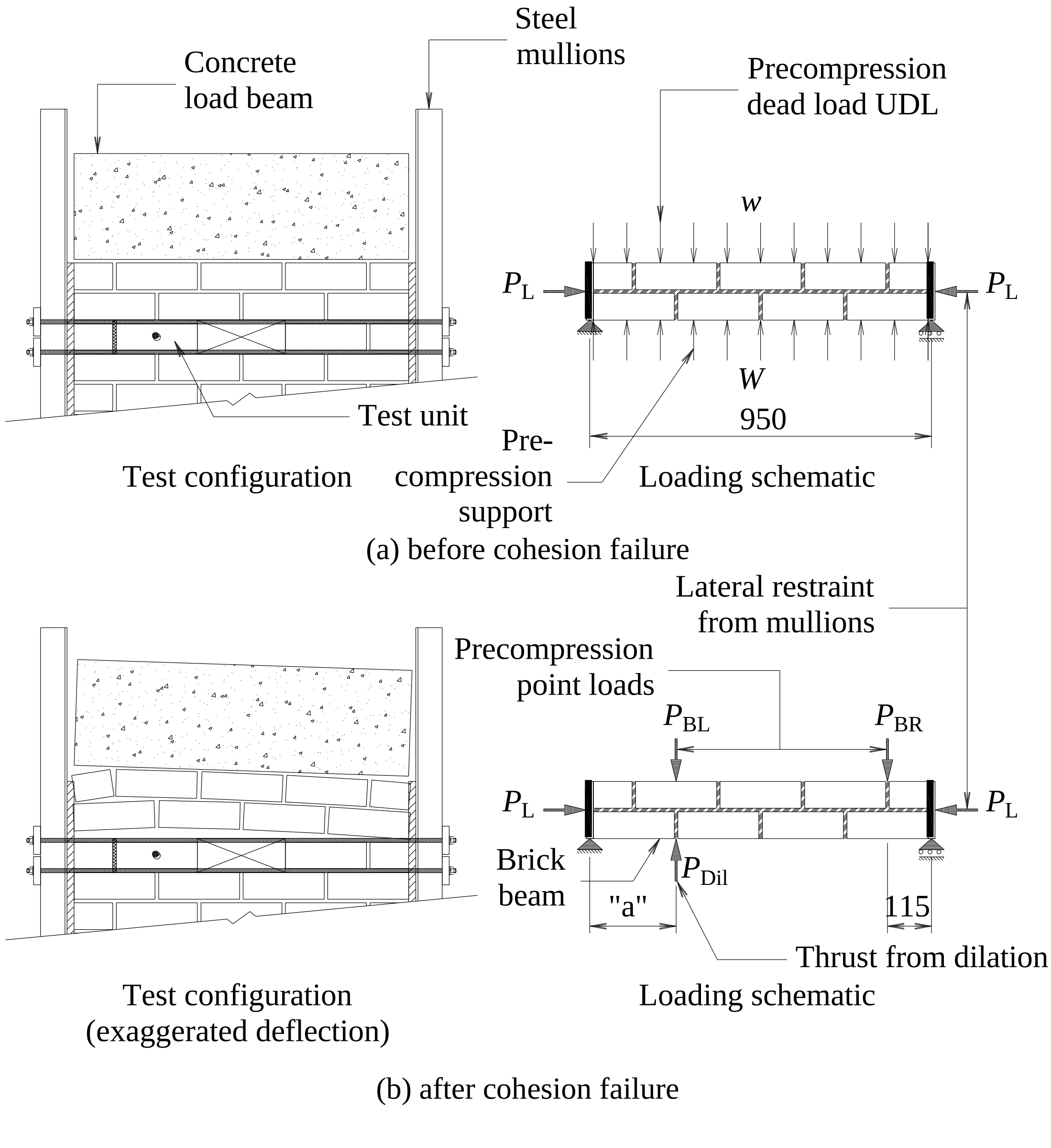

Fig. A1 – Shove test – assumed state of precompression before and after cohesion failure of the test unit, used to estimate coefficient of friction

Following cohesion failure (Fig. A1(b)), and the onset of sliding of the test brick, dilation (expansion) of the mortar joints surrounding the test unit occurs, causing the effective "brick beam" to deflect upwards, also lifting the concrete beam. The extent of the dilation and the consequent increase in vertical stress across the test brick needs to be determined to allow a useful estimation of the coefficient of friction.

Firstly, let us consider whether the idealised “brick beam” could be reasonably considered to separate from the lower portion of the masonry panel as a result of the dilation. If the deflection of the course immediately above the test unit is assumed to be linear, then the peak tensile stress in the confining masonry immediately adjacent to the test brick can be approximated as:

$$\sigma_t = E \cdot \varepsilon_t \qquad (A1)$$

where σ_t is the peak tensile stress, E = Young’s modulus of the masonry and ε_t is the peak strain (see Table A1). By estimating the average strain across the brick and joint combined as the mean dilation divided by the height of one brick plus one mortar joint, the tensile stress is predicted to be 64 MPa for the FC series, 4.0 MPa for the FL series and 9.0 MPa for the PL series. Such a tensile stress would greatly exceed the tensile capacity of the mortar joints (typically < 1 MPa), and so the assumption that the brick beam separates from the masonry panel below is considered justifiable.

Table A1 - Coefficient of friction estimated using the laboratory shove test, by accounting for increased vertical stress across test unit due to the combined effect of dilation and confinement by the surrounding masonry.

		Frogged Bricks / Cement Mortar			Frogged Bricks / Lime Mortar			Plain Bricks / Lime Mortar		
Statistic	Units	Mean	CoV	Count	Mean	CoV	Count	Mean	CoV	Count
Upper course dilation	mm	1.66	0.45	4	0.47	0.25	4	0.62	0.08	4
Test residual friction force	kN	12.2	0.24	4	5.68	0.12	4	6.64	0.06	4
Mean Young's modulus	MPa	3,490			725			1,250		
"Bending" load	kN	17.5	0.35	4	1.06	0.22	4	2.51	0.33	4
Nominal surcharge load	kN	0.84	0.13	4	0.84	0.13	4	0.84	0.13	4
Total pre-compression	kN	18.4	0.34	4	1.89	0.12	4	3.35	0.21	4
Coefficient of friction	–	0.38	0.53	4	1.53	0.22	4	1.03	0.23	4

This enables the use of statics to estimate the compression force across the test unit by treating the brick beam as a simply supported beam, which, combined with the assumptions of the brick beam having a uniform cross-section and a constant Young’s modulus, leads to the estimation of the coefficient of friction as 0.38, 1.53 and 1.03 for the FC, FL and PL series, respectively.

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