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FIELD TESTING OF VINTAGE MASONRY: MECHANICAL PROPERTIES AND ANCHORAGE STRENGTHS

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Abstract

Investigations of damage sustained following the 2010-2011 Canterbury earthquake sequence highlighted premature failure of anchorages in previously strengthened masonry. These failures suggest that a lack of understanding surrounding anchorage design is limiting the ability to seismically retrofit masonry structures. In order to provide experimental observations for use in developing and calibrating anchorage models a series of in-situ tests have been undertaken on three masonry structures to quantify in-situ mechanical properties and corresponding anchor pull-out loads and failure modes. Importantly, the outcomes of this field testing show poor correlation with the outcomes predicted by current design standards and approaches – that is, that the failure of anchorages by splitting of the masonry units rather than cone/wedge type failure or masonry unit extraction was the predominant observation. Additionally, due to difficulties with common test procedures, interpretation of standardised tests such as the “shove” test and the bond wrench test have been difficult. Alternative material tests and statistical distributions are proposed and a new anchorage failure model is suggested.

1. Introduction

Performance of unreinforced masonry (URM) buildings including historical structures and residential buildings in earthquakes is often poor. Reitherman and Perry (2009) reported that following strong earthquakes in the United States, five out of six URM buildings were so extensively damaged that potentially lethal amounts of brickwork fell and in the remainder of cases, the buildings either partially or completely collapsed. Additionally, seismic events in Newcastle in Australia (1989) and Christchurch in New Zealand (2011) caused considerable damage to unreinforced masonry structures and associated loss of life. Importantly, many of the structures that failed (particularly in New Zealand) had previously been strengthened (retrofitted) in accordance with design codes, and in many of those cases, the failure was at the connection between the strengthening elements and the masonry (Dizhur et al. 2016).

In a review of codified anchor design methods from Canada, Australia, New Zealand, Europe and the United states, McGinley (2006) identified that the American standard “Building Code Requirements for Masonry Structures” (American Concrete Institute and Masonry Standards Joint Committee 2013) provided design guidance, whilst the remaining codes used performance criteria. As a result, it is generally the responsibility of the anchor manufacturer to determine the capacities of their products.

Current manufacturer’s guidance for the strength design of tensile masonry anchors (e.g. Powers 2017, Hilti 2019, Ramset 2019) are the same as those required for the strength design of anchors in concrete. That is, designers are required to check anchor rod strength, anchor pull-out, brick breakout (cone/wedge) and brick pull-out. Minimum edge distances (and other dimensional restrictions) are then applied to the design to prevent splitting and spalling.

Similarly, failure models for post-installed anchors in masonry which have been developed from research have followed the same general approach as those developed for concrete. For example, plasticity theory has been used by Arifovic and Nielsen (2006) and Nielsen and Hoang (2016) for anchors in masonry by applying the analogy of concrete punching shear (cone) failure and their laboratory based experimental work has compared well to their theory.

Despite this good correlation to laboratory based experimental work, field observations by Moon et al. (2014) following the Christchurch earthquake sequence identified that failures of anchors in masonry did not reflect the ideal cone/wedge type of masonry breakout. Similar observations have also been reported by Dizhur et al. (2016) who conducted anchor pull out tests on in-situ masonry.

It is likely that the principal reason for the discrepancy between laboratory and field observations is that anchor failure modes in masonry are dependent upon the support conditions of the test specimen, the depth of embedment of the anchor and the condition of the test specimen and surrounding wall. It is difficult to adequately represent these and the consequent interaction of shear, bending, confinement, cohesion and friction within the wall in laboratory tests. Further, these conditions and interactions are difficult to match in damaged masonry. Compounding these difficulties is that in comparison to modern structures and materials, older masonry is generally comprised of weaker mortar and masonry units and is likely to be less homogeneous.

Older masonry is also more likely to require retrofitting for seismic strength as these older structures were not designed for seismic actions and are often in locations where collapse is likely to cause significant economic loss and potential loss of life. In-situ testing of existing (historical) masonry that is undamaged other than through normal “wear and tear” is able to provide proper representation of modern anchor failure in historical masonry and so field testing of undamaged older structures needs to be undertaken, and establishing this test data is the primary purpose of this paper. This was done through a series of tests conducted on three vintage masonry structures, each of which were single-rise cavity brick residential buildings with tiled roofs, constructed in the late 19th and early 20th century. In this context, vintage masonry is referring to masonry structures of approximately 100 years of age.

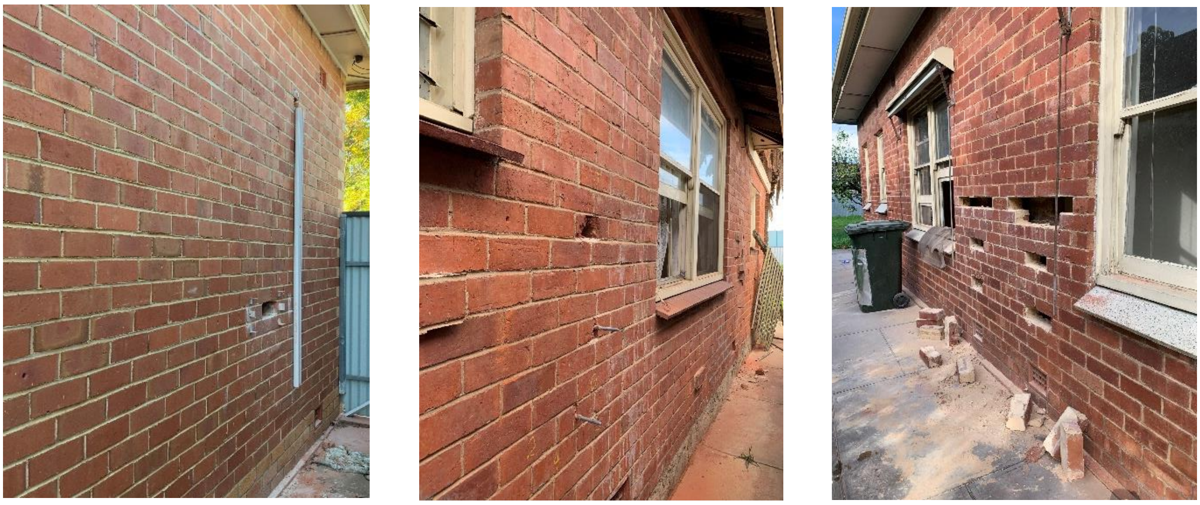
To fully characterise anchor behaviour it is necessary to understand the in-situ material properties of the mortar including the inherent variability that occurs in vintage masonry. Hence, in addition to reporting on anchor pull-out test results, this paper also reports on the results of in-situ tests to quantify associated material properties of the masonry through standard bond wrench and shove testing to quantify the flexural tensile strength and joint shear strength respectively.

From this testing it was identified that there are difficulties with these material tests, particularly with interpretation of results. The shove test can be difficult to interpret due to interactions between friction, cohesion and confinement (Graziotti et al. 2018) and to address this an alternative (brick pull-out test), which loads the brick out-of-plane, is trialed Due to the fragility of vintage mortar, the bond wrench test results in a grouping of experimental strength data at or near zero with increasing frequency (rising segment of the histogram) as the test results increase towards the mean. This is not well modelled with conventional distributions such as the normal, lognormal or Weibul statistical distributions and an alternative distribution which accommodates this grouping is suggested.

In the remainder of the paper, the methodology used for material tests for mortar shear and tensile strength and anchor pull out tests for determining anchor pull out capacity and failure mode are first presented. Findings of these tests, including comparison of expected cone type failure mode for chemical anchors (e.g. Collins et al. 1989, Cook et al. 1992, Fuchs et al. 1995, Arifovic and Nielsen 2006, Eligehausen et al. 2006, Nielsen and Hoang 2016, Lee and Gad 2017) versus observed failure modes for the anchor tests are then discussed in Section 3. Finally, suggestions for further research and the development of new test approaches are presented in Section 4.

2. Site investigations

To quantify the performance of modern chemical anchors in vintage masonry using realistic support conditions, a series of anchor pull-out tests and associated material tests (flexural tensile strength, mortar joint shear strength) were conducted on three houses located in Croydon Park, a suburb of Adelaide in South Australia. These properties (shown in Figure

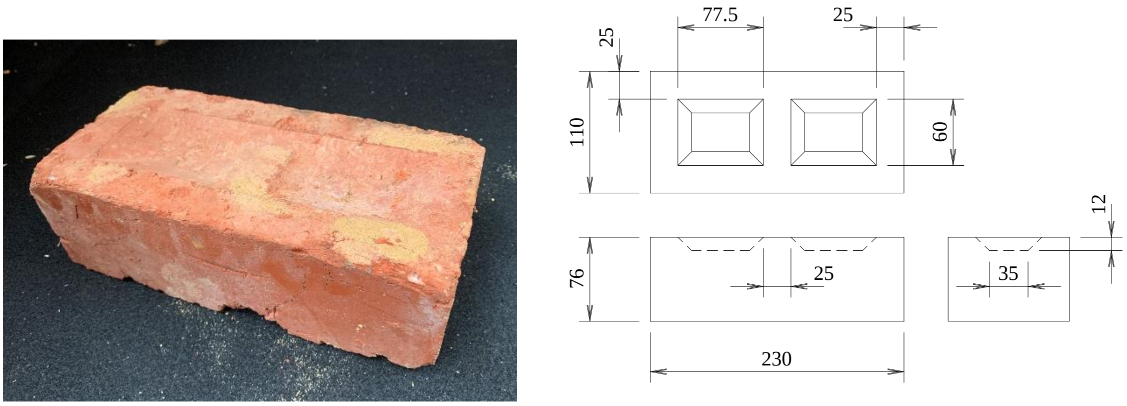


(a) Minerva Crescent (b) Pym Street (c) South Road

Figure 1 – Typical masonry at each site

1), are designated according to their locations (MC: Minerva Crescent, PS: Pym Street and SR: South Road) and are of double-clay-brick masonry construction using lime mortar. The sites were selected as they represented older, (vintage) masonry having been built using moulded (sandstock) bricks that were solid or frogged; these differing from more modern bricks which are commonly extruded, cored and fired at higher temperatures. The house at PS was constructed exclusively with solid (un-frogged) bricks, SR was from frogged bricks and MC exhibited a mixture of frogged and un-frogged. A typical frogged masonry unit (recovered from PS) with its nominal dimensions is shown at Figure 2.

Typical failure modes for adhesive anchors in masonry are (a) cone or wedge type failures, (b) the anchor pulling out of the hole through slippage or (c) the brick being extracted from the wall. A fourth failure mode involving splitting of the masonry also occurs when the anchor is close to an edge (Arifovic and Nielsen 2006). Failure of the steel anchor was not considered as anchor depths used were small (80 mm) and the ultimate capacity of the anchor rod large (over 33 kN).



(a) Typical "frogged" brick

(b) Typical dimensions

Figure 2 – Typical masonry unit (brick)

To conduct the anchor pull-out tests a rigid reaction frame shown in Figure 3, which is similar to that used by Dizhur et al. (2016) on New Zealand masonry was constructed. The tests were all undertaken using Hilti M12 galvanised Class 5.8 anchor rods with Hilti HIT-HY 170 injection mortar (a hybrid injection mortar) and all anchors were installed into solid clay masonry and sieves were not used. The reaction frame was designed with a span of 480 mm to ensure the test area was not influenced by the reactions imposed by the frame, and to minimise bending in the wall. The anchors were installed according to manufacturer's instructions (Hilti 2019) at least two hours and in most cases more than one day before testing along the lower half of the outer wythe of masonry where it was estimated that the normal stresses from the masonry and roof above ranged from 0.03 to 0.05 MPa. Only 12 mm anchors were tested because manufacturer's design tension resistance in solid masonry is independent of anchor diameter (Hilti 2019), and there was insufficient opportunity to incorporate anchorage diameter as a test variable within the sites available. To eliminate the impact of edge effects, the location of anchor tests relative to openings was greater than three masonry units horizontally and two units vertically. A map of the location of each pull-out test in relation to openings at each property can be found in the Supplementary Material and a summary of the number of tests at each location is provided in Table 1.

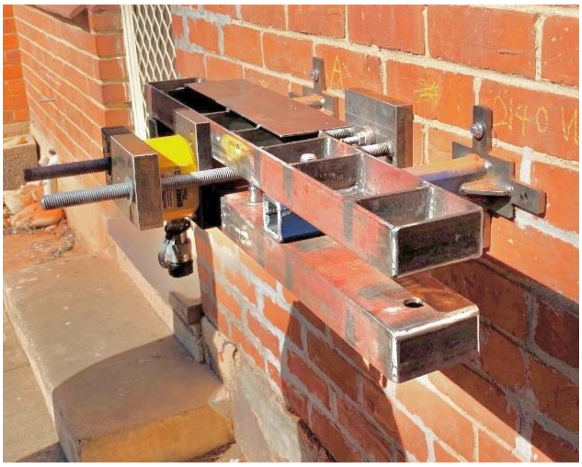
Table 1 - In-situ testing

Croydon Park - In-situ tests

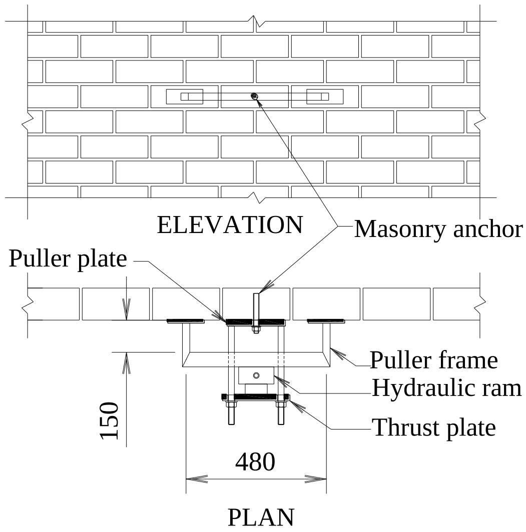
	MC	**PS**	**SR**	**Total**
Bond wrench test	1	4	18	23
Shove test[1]	6	6	-	12
Brick pull-out test	6	6	8	20
Anchor pull test	5	27	10	42

[1] *In-situ bed joint shear test*

For anchor testing, the anchor was loaded by reacting against the puller plate bolted to the anchor using a 250 kN low profile hydraulic ram reacting against the frame and the thrust plate bolted to the load transfer mechanism (see Figure 3). The applied load was measured using a pressure transducer located at the manual hydraulic pump and the slip of the anchor relative to the surrounding masonry was measured using a single 25 mm travel LVDT supported independently of the wall and reaction frame, measuring on the thrust plate. In this configuration, the test is displacement controlled.



(a) Photograph – before testing



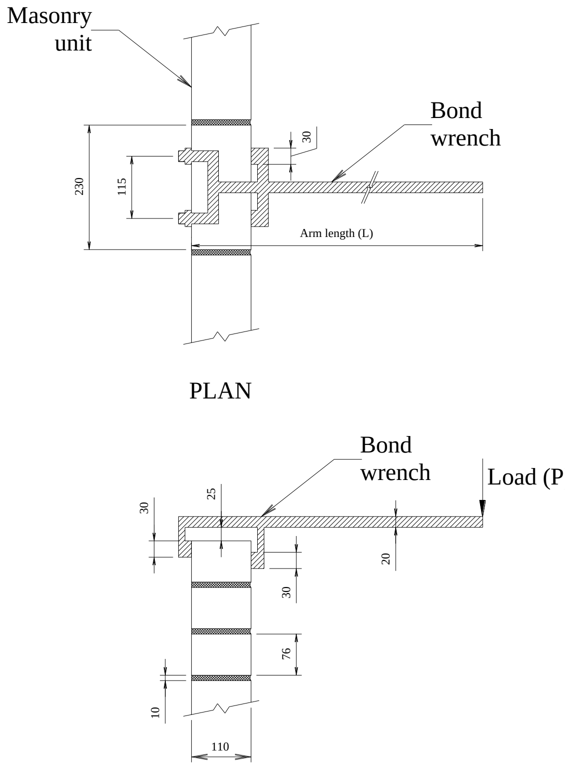
(b) Schematic layout

Figure 3 – Anchor pull out reaction frame

To quantify the associated mechanical properties of the masonry the bond tensile strength was measured through bond wrench tests conducted according to AS3700 Masonry structures (Standards Australia 2018) at locations at similar heights and on the same walls on which the anchor tests were conducted (see Supplementary Material for locations). Figure 4 shows a schematic configuration of the typical unit and also the two units that were used in this research. Both units are instrumented with strain gauges to allow direct determination of the applied load, and hence moment with Unit 1 using a direct readout of applied load. The length of the lever arm and weight of Unit 1 precluded its use on the weaker masonry as the self weight and consequently applied moment would fail the joints before additional load was applied. To overcome this difficulty, Unit 2, with a much shorter lever arm was used. These tests were specifically undertaken close to the anchor pull out tests to minimise variations in the masonry, but removed from them at least two bricks distant to minimise any latent damage there may be to the mortar joints from those tests.

Masonry bed joint (mortar) shear strength was initially measured using the simplified shove test according to AS 3826 *Strengthening existing buildings for earthquake* (Standards Australia 1998) which is based on the RILEM (International Union of Laboratories and Experts in Construction Materials, Systems and Structures) recommended test which is informed by ASTM C1531 (ASTM International 2003). The simplified test (without flat jacks above and below the test unit), as shown in Figure 5, has been adopted for this work as it is a less invasive procedure than the full test with flat jacks and whilst it is likely to be less accurate than the full test, it is more often used as an in-situ test which this work seeks to simulate. It is used to determine the combined shear capacity of the mortar joints directly above and below the masonry unit being tested.

In this simplified configuration, (shown in Figure 5) the "test unit" deflects elastically as load is applied from the hydraulic ram until the shear capacity is reached, either through failure of the mortar, failure of the bond between the mortar and the test unit, or a combination of the two. During this elastic phase and the subsequent plastic displacement until the test is terminated, friction and dilation (vertical displacement of the units above and below the test



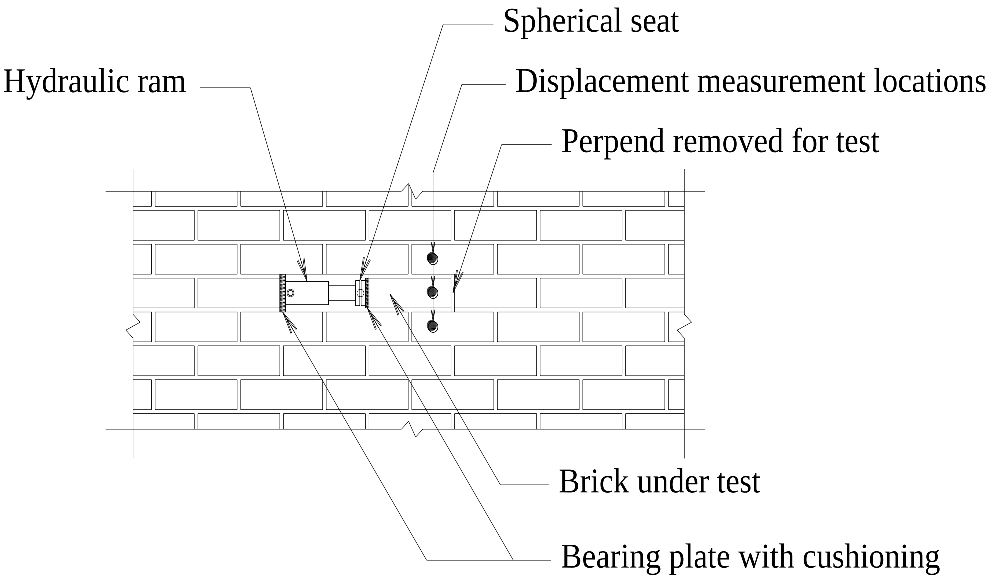
(a) – Schematic layout

(b) – Unit 1 with arm length 900 mm

(c) – Unit 2 with arm length 300 mm

Figure 4 – Bond wrench

unit) need to be considered. For all of the tests, (location shown in the Supplementary Material) the masonry unit was loaded by a 100 kN hydraulic ram with the applied load being measured using a pressure transducer located at the manual hydraulic pump. The lateral displacement and vertical dilatency were measured using 25 mm travel LVDTs supported independently from the wall. Similar to confining stresses for the anchor pull out tests, a vertical confining stress, of 0.03 to 0.05 MPa from the weight of wall and roof above the test locations is also applied to the brick under test.



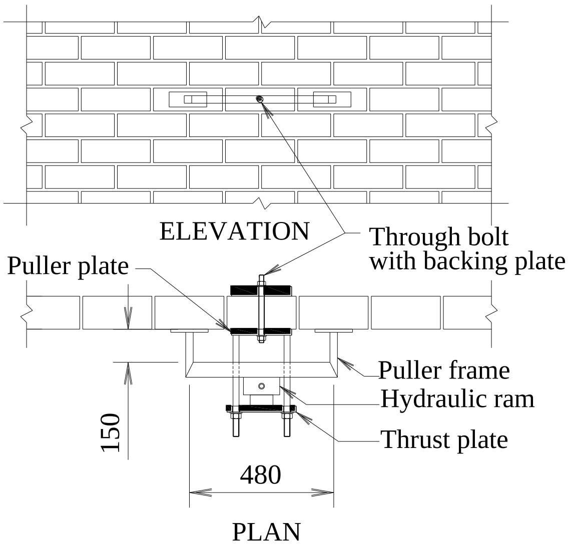
(a) Schematic layout

(b) Photograph after testing

Figure 5 – Simplified in-situ shove test

Experience gained from tests at MC and PS indicated that it was difficult to get meaningful shove test results when the wall was interrupted by free ends (doors, windows) or direction changes in walls (at corners) when these are only a short distances from the test location. Locations of doors and windows and direction changes of walls limited the extent of uninterrupted wall that was available at SR and consequently, the shove test was not undertaken at this site.

Given the well-established challenges in interpreting the results of shove tests (Graziotti et al. 2018), an alternative pull-out test to quantify the shear strength of the mortar is proposed and



(a) Schematic layout

(b) Photograph during test

Figure 6 – Brick pull-out test

was trialled here at each of the sites with the test locations shown in the Supplementary Material. For this test, a stiff reaction frame (shown in Figure 6); the same as that used for the anchor pull out tests was used, but the configuration incorporated a steel backing plate with dimensions 200 mm wide x 50 mm high, approximating but smaller than those of the masonry unit mounted on a 16 mm diameter through bolt onto the back of the wall, in lieu of the chemical anchor. This configuration allowed the backing plate to bear entirely on a single brick without overlap which was then reacted against the puller plate bolted to the through bolt using a 1000 kN hydraulic ram reacting against the frame and the thrust plate bolted to the load transfer mechanism with the applied load measured using a pressure transducer located at the manual hydraulic pump. The lateral movement of the brick relative to the surrounding masonry was measured using a single 100 mm travel LVDT supported independently of the wall and reaction frame, measuring on the thrust plate. Dilatancy was not measured in these tests. By using a reaction frame with close centres, it was possible to extract a single masonry unit "squarely" from the masonry, with a typical "extraction" demonstrating the efficacy of the method shown in Figure 6 (b). The tests which were undertaken involved extraction of a brick from the outer leaf of the wall which was a minimally invasive procedure externally, but an opening was required through the internal leaf to allow the backing plate to be attached.

3. Results

The data for each of the tests for each of the test sites is presented as independent data sets with corresponding statistics and also as aggregated data (across test sites) for each of the tests. Kolmogorov-Smirnoff (Lovric 2011) two sample tests suggests that other than for the bond wrench results, the data for each of the tests is not from the same continuous distribution (at the 5% confidence level) which suggests that aggregation of the data is not statistically valid. However, whilst the data may not necessarily be attributable to the same distributions, the aggregated results have been included as they assist in development of "typical" characteristic strengths that can be used in design. Sorrentino et al. (2016) recommend using the lognormal or normal distributions dependent upon the need to avoid negative values in calculations of 5th percentile and other predictive data, but note that neither of these distributions are a good fit to masonry data. Goodness of fit testing of each of the individual data sets as well as aggregated data for each test has been undertaken using the Kolmogorov-Smirnov test at the 5% confidence level with the null hypothesis that the site data is from a normal or lognormal distribution, and the results are shown in Table 2. As can be seen from Table 2, most of the hypotheses have been rejected, confirming Sorrentino's observation that neither of the two common distributions are valid.

Table 2 – Kolmogorov-Smirnov goodness of fit tests

Kolmogorov-Smirnov testing of goodness of fit to normal and lognormal distributions						
	Normal distribution			Lognormal distribution		
	KS statistic	Critical value	Accept/ Reject	KS statistic	Critical value	Accept/ Reject
Bond wrench						
MC	Not enough data			Not enough data		
PS	0.509	0.624	Accept	0.988	0.624	**Reject**
SR	0.500	0.309	**Reject**	0.954	0.309	**Reject**
Aggregated	0.500	0.281	**Reject**	0.954	0.281	**Reject**
Shove						
MC	0.690	0.519	**Reject**	0.382	0.519	Accept
PS	0.582	0.375	**Reject**	0.824	0.375	**Reject**
SR	Not enough data			Not enough data		
Aggregated	0.690	0.519	**Reject**	0.382	0.519	Accept
Brick pull-out						
MC	0.640	0.454	**Reject**	0.498	0.454	**Reject**
PS	0.578	0.454	**Reject**	0.811	0.454	**Reject**
SR	0.578	0.309	**Reject**	0.628	0.309	**Reject**
Aggregated	0.640	0.519	**Reject**	0.526	0.519	**Reject**
Anchor pull						
MC	1.000	0.391	**Reject**	0.981	0.391	**Reject**
PS	1.000	0.294	**Reject**	0.981	0.294	**Reject**
SR	1.000	0.203	**Reject**	0.986	0.203	**Reject**
Aggregated	1.000	0.409	**Reject**	0.994	0.409	**Reject**

3.1. Bond wrench test

The bond wrench test was undertaken at each site and the data and statistics for each site and for the aggregated data is shown in Table 3 and the data and aggregated statistics are shown graphically in Figure 7. A number of the bond wrench tests were abandoned as a result of either the bed joint failing during the removal of the adjacent bricks and perpends or during installation of the bond wrench tool after preparation of the test brick and these have been excluded from the data set and the analysis. In addition, the M1 (MC Test 1) result was excluded as it was more than 15 standard deviations from the mean and was considered an outlier.

Significantly, in the absence of test data, the local masonry design standard AS3700 – *Masonry Structures* (Standards Australia 2018) allows the designer to use a characteristic value of 0.2 MPa for the masonry flexural tensile strength. Other than the outlier test, the maximum determined strength was less than this, with the 5th percentile test results (which could be expected to align with the allowable value) all being less than zero (based on the normal distribution) and only slightly greater than zero assuming a lognormal distribution. Work on other sites in Adelaide (Derakhshan et al. 2017) with 60 tests on four different properties had a mean value of 0.16 MPa and a characteristic strength of 0.04 MPa. This again shows results less than those allowable in the local design standard. The authors consider that this evidence suggests that the code allowed value of 0.2 MPa is unconservative for vintage masonry.

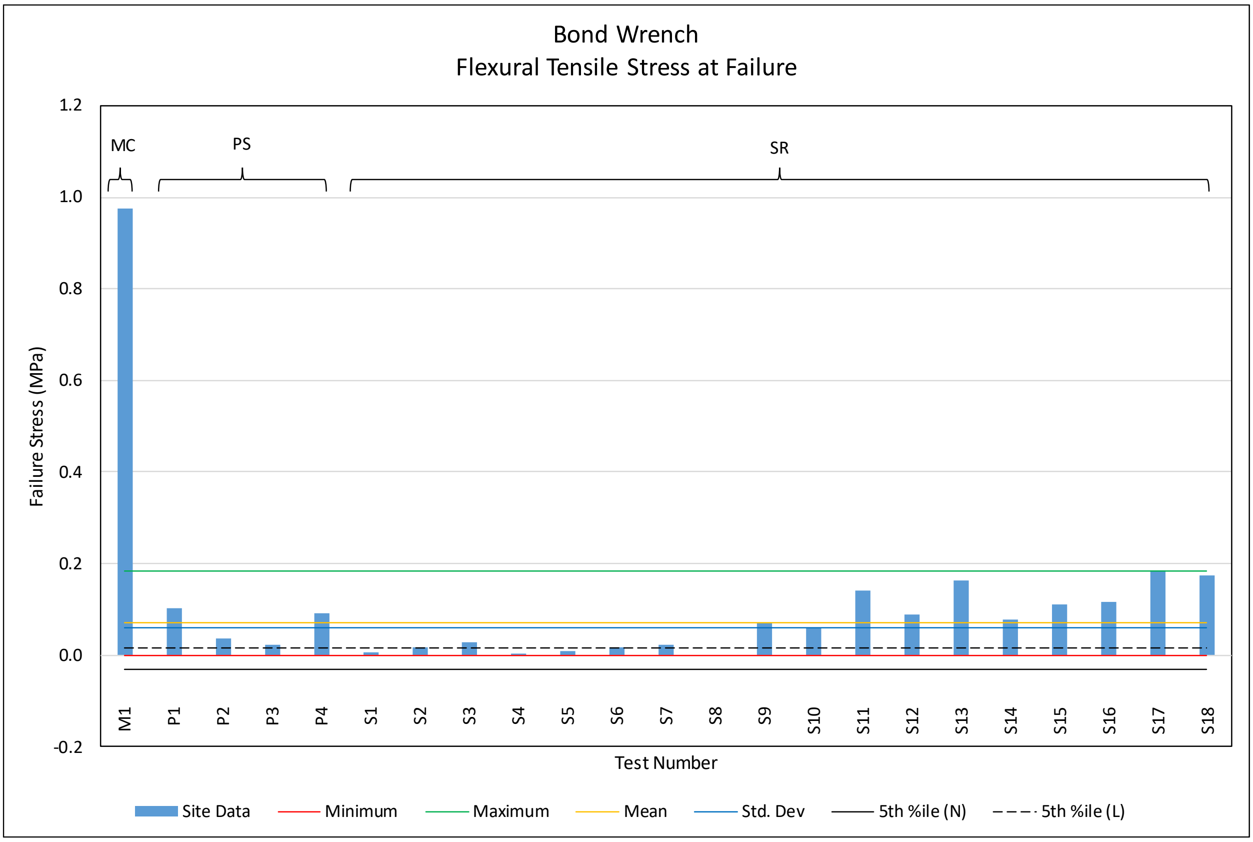


Figure 7 – Bond wrench test results

Frogged bricks are generally laid with the "frogs down" as this reduces the amount of mortar that is used in the bed joint contributing to more economical construction, but as this results in only the outer edges of the upper brick being in contact with the mortar, it can lead to reduction of joint strength. Also, diurnal temperature variations and more extreme temperature gradients from direct sun exposure may contribute to joint strength reduction as a result of thermally induced cyclic stresses. The bond wrench results for South Road (S1 to S18) shown in Figure 7 tend to show these phenomena and whilst not investigated further than general observation, they are worth noting. The tests S1 to S16 were undertaken on the north (sunny side) side of the house. Significant temperature differentials exist in summer, both diurnally and from external to internal faces (Alterman et al. 2017) leading to probable thermally induced movements between bricks and mortar. Of these, tests S1 to S8 were found to have been laid frogs down with incomplete bedding. When bricks are laid, typically the mortar is placed and then furrowed by the brick layer. When they are laid frogs down, as a result of the furrowed mortar and the frog on the lower side of the brick being laid, there is often no (or minimal) contact between the frog profile and the mortar bed, and so only the outer edges of the brick being laid are in contact with the brick below. Tests S9 to S16 had

been laid frogs up and the bedding was complete over the full face. Whilst the tests S9 to S16 were also lower in the wall and likely to have been laid by a different brick layer, the joint strength differences are clear. The two tests S17 and S18 were both done on the southern side of the house and this wall would not have been subjected to any direct sunlight since it was constructed, minimising thermal stresses. Noteworthy was that S17 was frog down and S18 frog up. Other possible sources of variability could include changing construction personnel and different times of the day the bricks were laid resulting in variations of mortar mix ratios, wetness of bricks to be laid, quality, consistency and orientation (frog up or down) and consistency of bedding. Also,
depending on the time of the day when the bricks are laid, there will be variations in temperature and humidity during construction and also the height of construction leading to differing confinement loads during curing, particularly overnight.

Table 3 – Bond wrench test data

Bed joint peak tensile strength (MPa)					
Test #	**MC**	**PS**	**SR**		
1	0.974	0.103	0.006		
2		0.037	0.016		
3		0.024	0.028		
4		0.091	0.002		
5			0.010		
6			0.018		
7			0.023		
8			0.000		
9			0.069		
10			0.060		
11			0.141		
12			0.088		
13			0.164		
14			0.077		
15			0.110		
16			0.115		
17			0.185		
18			0.175		
				Aggregated	**Aggregated (no M1)**
Count	1	4	18	23	22
Min	0.974	0.024	0.000	0.000	0.000
Max	0.974	0.103	0.185	0.974	0.185
Mean	0.974	0.064	0.071	0.109	0.070
St.Dev	n/a	0.039	0.064	0.197	0.059
CofV	n/a	0.616	0.891	1.803	0.846
5th % (N)[1]	n/a	-0.029	-0.039	-0.229	-0.032
5th % (L)[1]	n/a	0.014	0.014	0.007	0.015

[1] *based on T-distribution with n-1 degrees of freedom*

N – normal distribution

L – Lognormal distribution

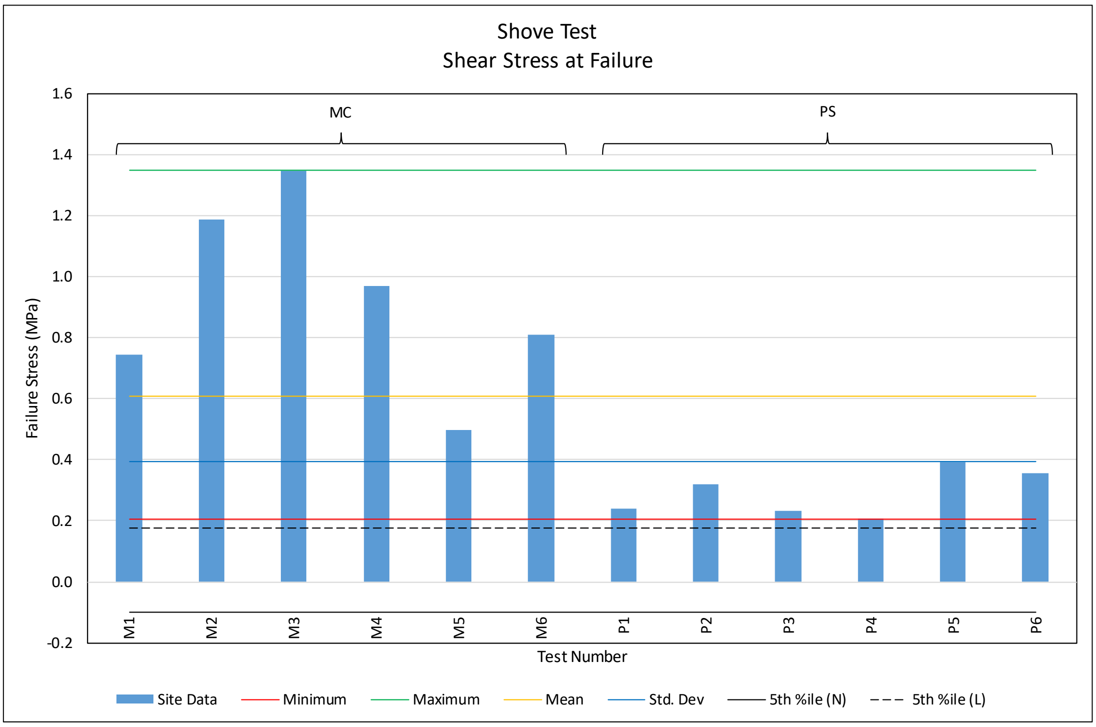


Figure 8 – Shove test results

The 5th percentile values for each of the tests has been calculated based on the alternative assumptions that the data follows either the normal or lognormal distribution. Use of these simple distributions facilitate simple calculation of statistics for the data, but neither of these distributions are a good fit to masonry data (Sorrentino et al. 2016). The normal distribution can lead to negative values for 5th percentiles when the test strength is low, the variability high, or combinations of the two, which has no valid meaning in reality. Use of the lognormal distribution eliminates the negative strength issue, but the cumulative density function has zero probability at zero strength which is also not reflected in site observations. The authors consider the probability density function of the normal distribution (to the right of zero) better reflects the true population distribution and suggest that more research needs to be undertaken in this area. Sorrentino et al. (2016) recommends using the lognormal distribution for low mean data and the normal distribution for higher strength tests but notes that neither distribution fits the data well. As an alternative to continuous distributions, the 5th percentile was also calculated by simply ranking the data but the inclusion of more than one of the abandoned (zero strength) tests resulted in a zero strength 5th percentile result rendering the method overly sensitive as a result of the small number of data points.

Given the extent of variability shown in the data and the number of unknowns associated with masonry construction, combined with issues calculating meaningful statistics, the authors consider that the attribution of 0.2 MPa characteristic tensile capacity given in the local code is not conservative and that a value of zero should be adopted for assessment of existing masonry structures until the time when a more consistent and repeatable result can be determined. The value of zero is adopted in the local code AS3826 - *Strengthening existing buildings for earthquake* (Standards Australia 1998), but this is a guideline rather than a mandatory code of practice.

3.2. Shove test

Figure 8 summarises the results for the shove tests undertaken at MC and PS while the data and distribution statistics are shown in Table 4. The results for the individual sites are not as variable as those for the bond wrench test but still display a considerable coefficient of variation, although the 5th percentile values for the individual properties accord to the extremes of values published for designers in AS3700. Assuming that the data is normally

Table 4 – Shove test data

Bed joint peak shear strength (MPa)				
Test #	**MC**	**PS**	**SR**	
1	0.746	0.238		
2	1.187	0.321		
3	1.349	0.234		
4	0.969	0.207		
5	0.496	0.394		
6	0.812	0.355		
				Aggregated
Count	6	6	-	12
Min	0.496	0.207	-	0.207
Max	1.349	0.394	-	1.349

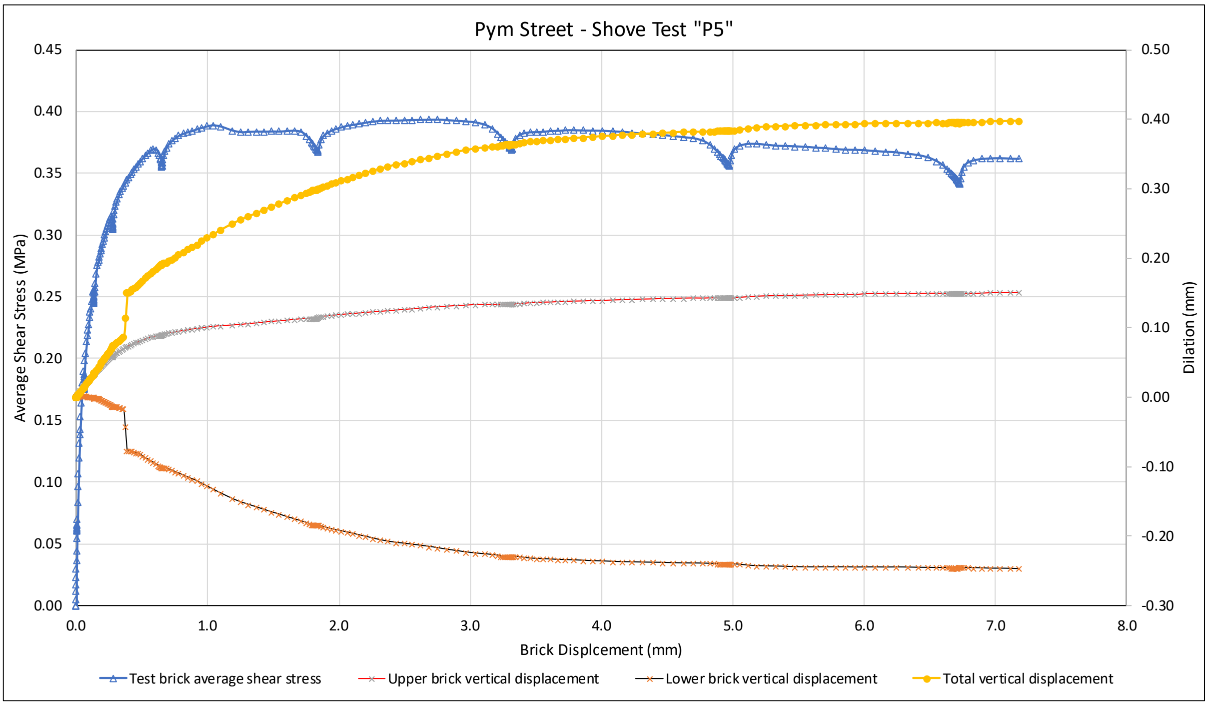
distributed
(Sorrentino et al. 2016), the F-test has been applied to the variances which indicates that they are not from the same distribution (at the 5% confidence level). In addition, the t-test suggests that the means are also not from the same distribution (at the 5% confidence level) which combined, suggests that there is significant difference between the two sites, making general assumptions about typical design values unreliable.

Mean	0.926	0.291	-	0.609
St.Dev	0.310	0.076	-	0.395
CofV	0.334	0.260	-	0.649
5th % (N)[1]	0.302	0.139	-	-0.101
5th % (L)[1]	0.456	0.168	-	0.176

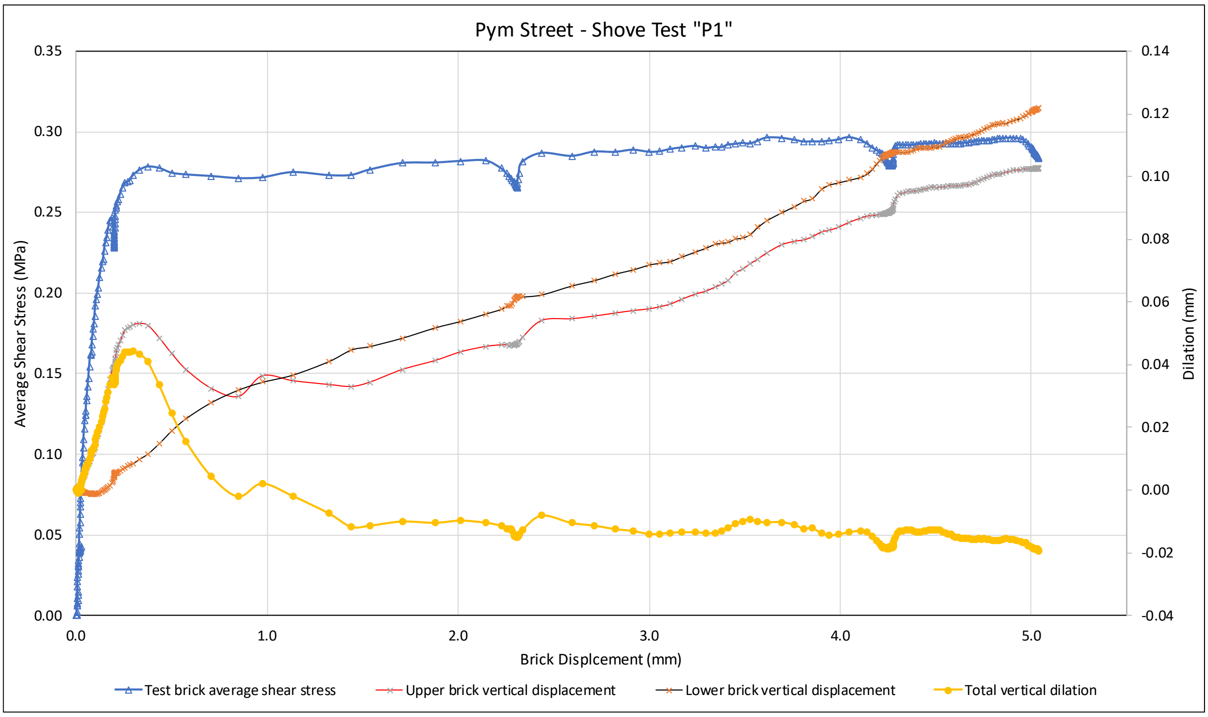
[1] *based on T-distribution with n-1 degrees of freedom*
N – normal distribution
L – Lognormal distribution

In addition to these issues, the test results can be difficult to interpret. Figure 9 (a and b) show the result of two discrete shove tests conducted on PS. The test depicted in Figure 9(a) (test P5) shows progressive dilatantcy and longitudinal displacement up to close to peak load where a step is noted in the downward displacement (likely linked to the initial failure of the lower bed joint) with an associated flattening of the upward displacement, which is then followed with progressive “opening up” (increasing overall dilation) of the masonry until the experiment was terminated. It is suggested that the increasing overall dilation is a result of top and bottom faces of the test brick not being parallel, forming a small wedge, but this was not measured. Note that the downward “dips” which show particularly in the plot of longitudinal displacement are associated with cycling of the manual hydraulic pump as load was measured with a pressure transducer rather than a load cell.

The test P5 in Figure 9(a) was conducted centrally in a long section of wall whereas test P1 (Figure 9b), performed with the same apparatus by the same operators was conducted closer to one end of a different wall. Initial loading of the test brick in Figure 9(b) shows an upward displacement of the upper brick, and a downward displacement of the lower brick, but following initial failure, a significant drop of the upper courses can be observed. At the same time, the lower courses are seen to be moving upwards and that the combined displacement is also upwards over the majority of the test.



(a) Test P5

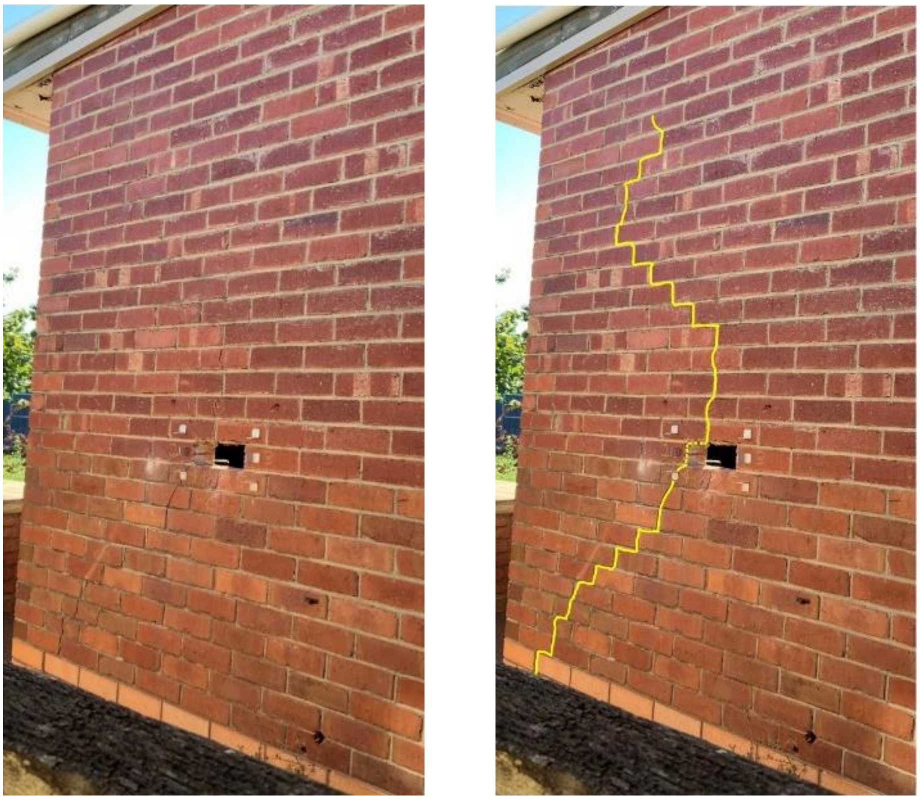


(b) Test P1

Figure 9 - Shove test dilatantcy – (PS)

Inspection of the wall following the test revealed no observable signs that the wall, or part of the wall had failed locally around the test site and there was insufficient instrumentation to measure global wall movements. It is surmised however that having the test ram angled slightly upwards (towards the test brick) during the test has resulted in a rotation of a segment of the wall following, but not necessarily coinciding with the peak failure stress indicating a possible further difficulty associated with this test, particularly in the field. Using flat jacks above and below the test location could assist in controlling and understanding vertical confinement stresses and deflection, but the potential for displacements of larger segments of the wall (other than the brick under test), possibly even beyond the bounds of the flat jacks will also lead to results that are difficult to interpret, irrespective of flat jack usage. It is postulated that the shove test can provide a reasonable estimate of peak joint strength, but it is likely to be an unreliable method for field determination of friction and cohesion.

In addition to the challenges described above, the shove test also has the capacity to be reasonably destructive dependent upon the wall geometry which has obvious implications, especially for heritage applications. Figure 10 shows the results of a shove test conducted at MC where the brick being tested has 3½ bricks between it and the left (unrestrained) end of the wall. The test brick was selected to be nominally equidistant in the wall with respect to



(a) – Wall crack (b) – Crack extent

Figure 10 – Shove test cracking – MC, test M3

the reaction brick and the expansion joint (out of picture) to the right and it is located in the 14th course above the footing. In this test, the test brick did not move relative to the bricks to the left of it, but rather failed the wall causing it to slide on the footing, cracking the wall over almost its full height, highlighted in Figure 10(b). This example underscores a further difficulty with the shove test when there is the possibility of there being differing mortar friction and brick tensile strengths, combined with inadequate wall length available to transfer those unbalanced forces into the footings. To overcome this some form of reaction frame at either end of the wall would be beneficial.

3.3. Brick pull-out test

Six each of these tests were conducted at MC and PS, and eight at SR with the data and statistics for the tests shown in Table 5 and Figure 11, and three typical stress vs displacement plots shown in Figure 12. This (Figure 12) shows that there is a well-defined

Table 5 – Brick pull-out test data

Bed joint peak tensile strength (MPa)					
Test #	**MC**	**PS**	**SR**		
1	0.513	0.243	0.374		
2	0.358	0.331	0.469		
3	1.163	0.197	0.552		
4	0.511	0.241	0.488		
5	0.605	0.228	0.404		
6	0.378	0.414	0.433		
7			0.731		
8			0.620		
				Aggregated	**Aggregated**

peak stress with an associated softening and that there is an on-going, albeit reducing capacity over a significant displacement and that the peak for the tests has occurred at a similar displacement suggesting repeatability. For all of the tests undertaken, the failure mode was extraction of a single masonry unit with no damage to the surrounding wall (see Figure 6b).

					(no M3)
Count	6	6	8	20	19
Min	0.358	0.197	0.374	0.197	0.197
Max	1.163	0.414	0.731	1.163	0.731
Mean	0.588	0.276	0.509	0.463	0.426
St.Dev	0.297	0.081	0.120	0.217	0.145
CofV	0.504	0.294	0.236	0.469	0.340
5th % (N)[1]	-0.009	0.113	0.282	0.088	0.175
5th % (L)[1]	0.201	0.148	0.319	0.194	0.227

[1] *based on T-distribution with n-1 degrees of freedom*

N – normal distribution

L – Lognormal distribution

Comparing the brick pull-out test to the shove test, Table 6 presents the ultimate shear stress statistics of both of these tests but with the M3 brick pull-out test (MC) excluded as it is considered to be a significant outlier as its value is more than five standard deviations from the mean. Based on observations and recommendations from Sorrentino et al. (2016) that higher strength data is best modelled by the normal distribution the F-test (based on this assumption of normality) has been applied which confirms that the variances of the shove and pull data for PS are from the same distribution at the 5% confidence level and t-testing indicates that the means are also from the same distribution (at the 5% confidence level). Also apparent when looking at the coefficient of variation is that the grouped results demonstrate a greater consistency for the pull out test than for the shove test.

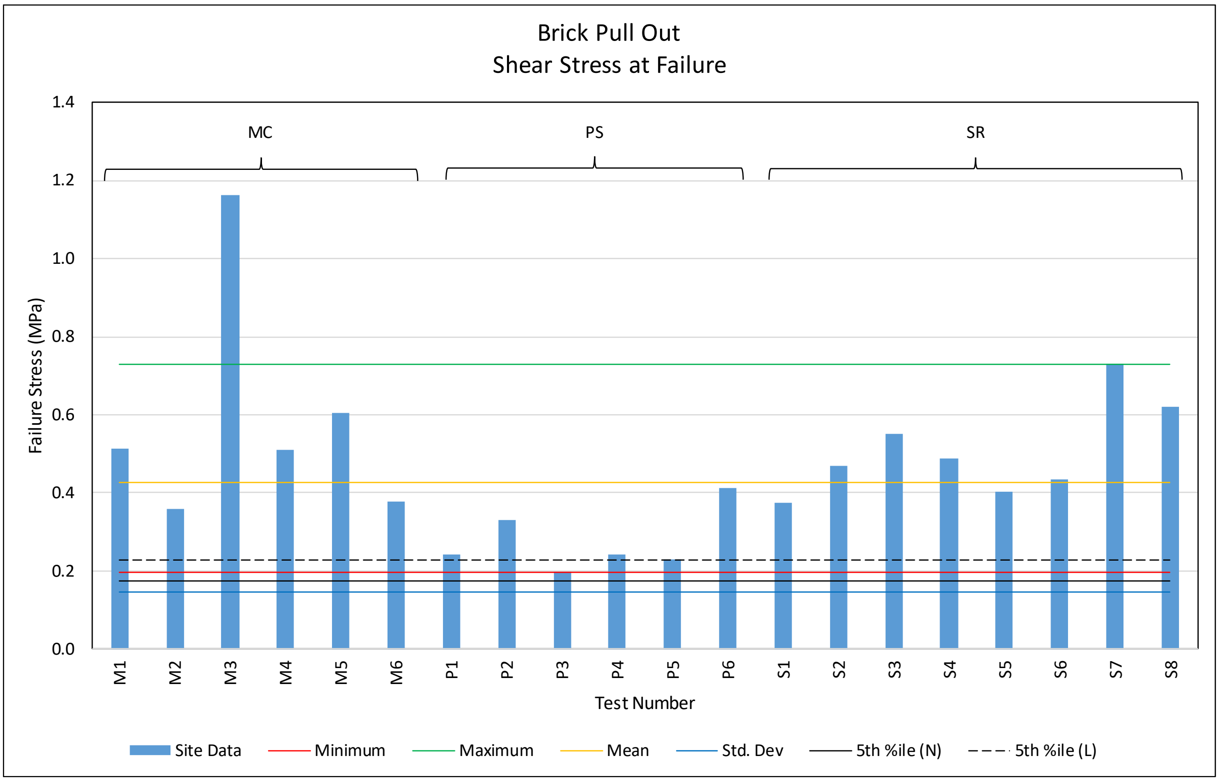


Figure 11 – Brick pull-out test results

The brick pull-out test has the ability to be less destructive than the shove test and there appears to be a better repeatability for the pull out test when compared to the shove test. Further, as the data can be considered as coming from the same distribution there appears to

Table 6 - Shove test vs. brick pull-out test

Bed joint peak tensile strength (MPa)

be a greater opportunity to aggregate the results to enable better generic masonry shear capacity values for

	MC		PS		SR		Aggregated (no M3)	
	Shove	*Pull*	*Shove*	*Pull*	*Shove*	*Pull*	*Shove*	*Pull*
Count	6	5	6	6	-	8	12	19
Min	0.496	0.358	0.207	0.197	-	0.374	0.207	0.197
Max	1.349	0.605	0.394	0.414	-	0.731	1.349	0.731
Mean	0.926	0.473	0.291	0.276	-	0.509	0.609	0.426
St.Dev	0.310	0.103	0.076	0.081	-	0.120	0.395	0.145
CofV	0.334	0.219	0.260	0.294	-	0.236	0.649	0.340
5th % (N)[1]	0.302	0.253	0.139	0.113	-	0.282	-0.101	0.175
5th % (L)[1]	0.456	0.292	0.168	0.148	-	0.319	0.176	0.227

[1] *based on T-distribution with n-1 degrees of freedom*
N – normal distribution
L – Lognormal distribution

use in retrofit design to be determined.

The authors recommend that further investigations are conducted on the pull out test compared to the shove test in controlled laboratory conditions to determine whether the brick pull-out test can be used as an alternative to the established shove test.

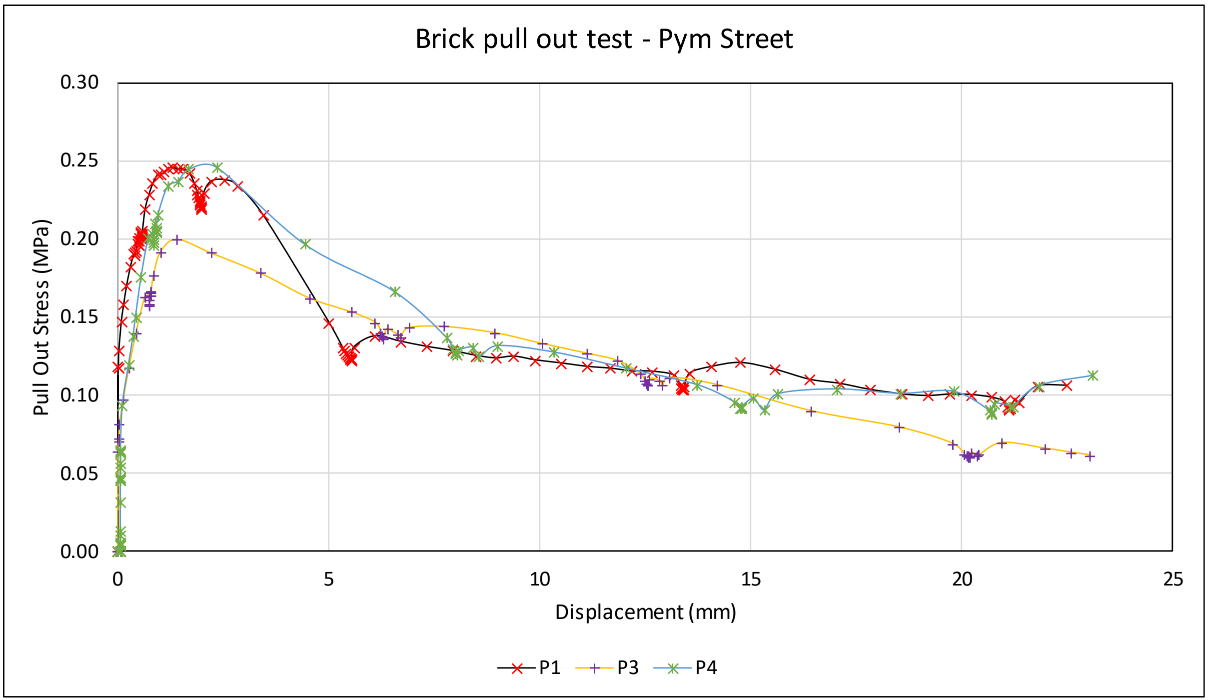


Figure 12 – Brick pull-out – stress vs displacement

3.4. Anchor pull out test

The data and statistics for the peak pull out capacity for the five anchor pull out tests performed at MC, 27 at PS and 10 at SR are shown in Table 7 and in Figure 13. Two

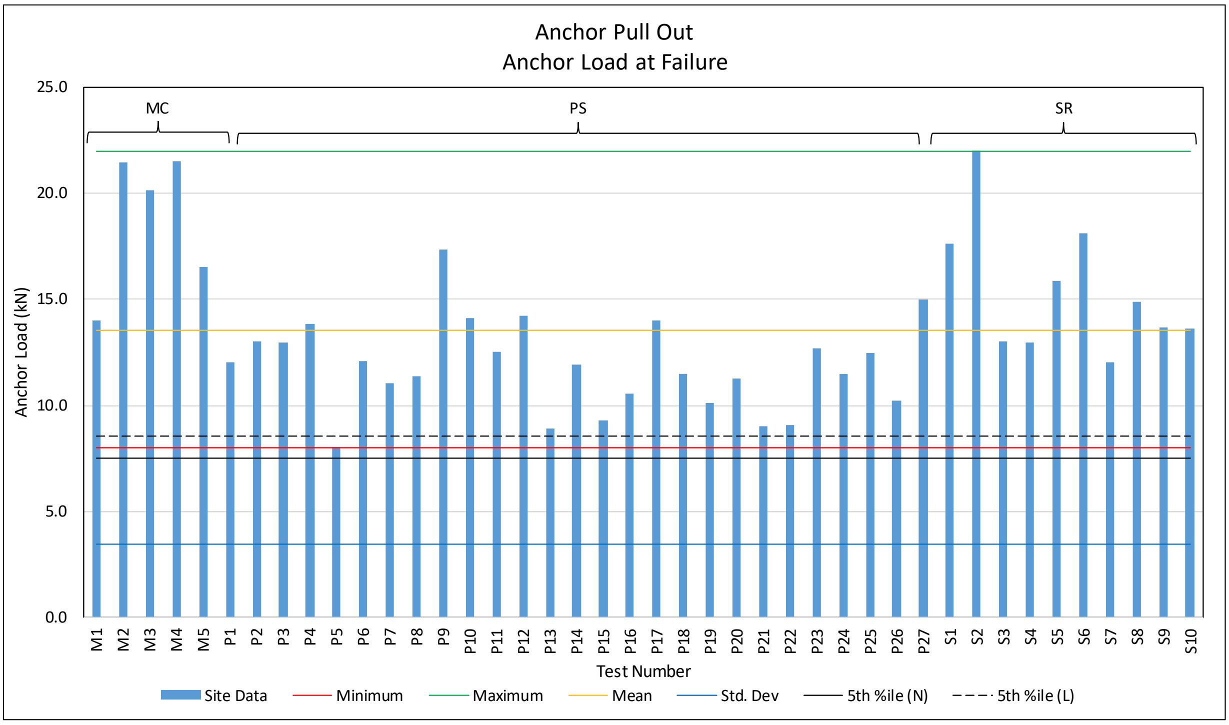


Figure 13 – Anchor pull out test results

valuable observations have been made following these tests. The first is that the anchor peak pull out capacity based on either the normal or lognormal distribution at the 5th percentile level is considerably higher than the characteristic capacity tabulated by the manufacturer (1.2 kN) for the anchors,

Table 7 – Anchor pull out test data

Peak Pull Out Capacity (POC) (kN)				
Test #	**MC**	**PS**	**SR**	
1	14.00	12.01	17.63	
2	21.47	13.04	21.99	
3	20.16	12.98	13.01	
4	21.50	13.82	12.96	
5	16.52	8.01	15.88	
6		12.06	18.12	
7		11.07	12.06	
8		11.38	14.90	
9		17.35	13.69	
10		14.13	13.62	
11		12.50		
12		14.20		
13		8.88		
14		11.92		
15		9.28		
16		10.54		
17		14.03		
18		11.49		
19		10.14		
20		11.28		
21		9.04		
22		9.07		
23		12.69		
24		11.48		
25		12.48		
26		10.24		
27		15.01		
				Aggregated
Count	5	27	10	42
Min	14.00	8.01	12.06	8.01
Max	21.50	17.35	21.99	21.99
Mean	18.73	11.86	15.39	13.52
St.Dev	3.33	2.13	3.08	3.47
CofV	11.62	8.22	9.75	7.66
5th % (N)[1]	11.62	8.22	9.75	7.66
5th % (L)[1]	12.65	8.61	10.50	8.55

[1] *based on T-distribution with n-1 degrees of freedom*
N – normal distribution
L – Lognormal distribution

adhesive and substrate being tested. Secondly, almost all failures were due to the brick splitting vertically at the anchor location with the anchor subsequently being extracted from the hole with the adhesive remaining attached to the anchor (see typical failures in Figure 14) In addition, whilst load carrying capacity reduced following initial failure, the anchors

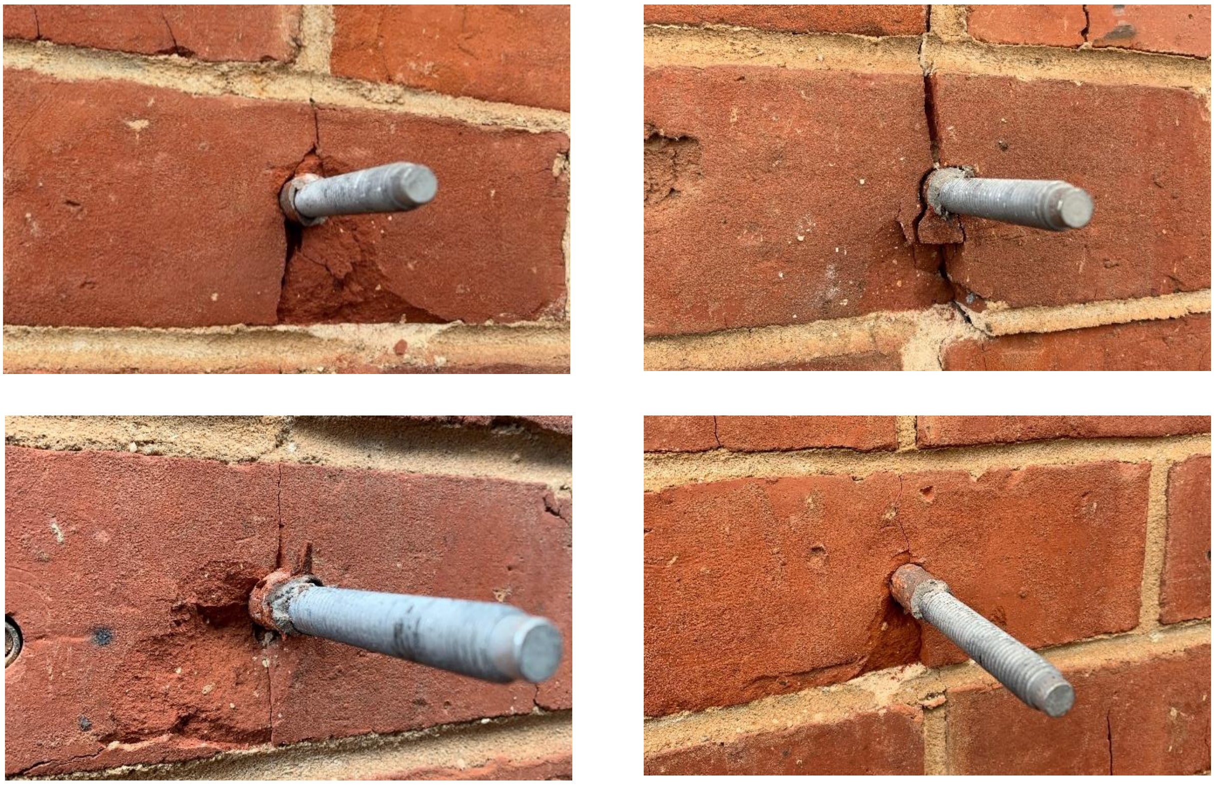


Figure 14 – Typical anchor pull out splitting failures

consistently demonstrated some degree of ductile behaviour by continuing to provide useful load capacity/resistance at displacements well in excess of the displacement at peak load as shown in Figure 15 and as can also be seen in Figure 15, the displacement to failure is relatively linear with respect to applied load, and similarly after failure as the load capacity reduces. Additional data showing anchor displacement at peak load capacity and anchor capacity at the maximum measured displacement for MC and PS are shown in Table 8. The maximum displacements shown in Table 8 were limited to the available travel of the hydraulic ram or the LVDT used to measure the displacement and so do not reflect complete anchor extraction. Note that there are no details for SR as displacement measurements were not taken at this site.

When considering failure modes of anchors in masonry, splitting of the brick is generally not considered with, for example ETAG 029 (European Organisation for Technical Approvals 2013) documenting “failure of the metal part ‘, “pull-out failure of the anchor”, “brick breakout failure”, “pull out of one brick” and “influence of joints” as the required proofs for resistance to tension loads. These same requirements carry across to the manufacturer’s documentation as generally, the products have been awarded an ETA, (Powers 2017, Hilti 2019, Ramset 2019) and are generally considered as the required tests by other authors (e.g. Arifovic and Nielsen 2006, Eligehausen et al. 2006, Nielsen and Hoang 2016). Laboratory test data from Arifovic and Nielsen (2006) shows the cone and anchor sliding failures, or combinations of the two as the predominant failure modes with a rapid reduction of load capacity following the initial failure. These modes of failure are quite different to the results that have been found here and the reasons for this are not entirely clear and warrant further experimental investigation under controlled laboratory conditions. The nature of the anchorage failures shown in Figure 14 are typical of tension failure at the outer face of the brick suggesting that bending and other

Table 8 – Anchor loads and displacements at peak load and maximum displacement

Pull Out Capacity at peak and maximum displacement [1]								
	MC				**PS**			
	Load and displacement at peak load capacity		Load and displacement at maximum measured displacement [2]		Load and displacement at peak load capacity		Load and displacement at maximum measured displacement [2]	
Test #	Load (kN)	Disp. (mm)	Load (kN)	Disp. (mm)	Load (kN)	Disp. (mm)	Load (kN)	Disp. (mm)
1	14.00	3.38	4.90	21.25	12.01	1.71	4.12	15.59
2	21.47	1.46	3.30	17.07	13.04	2.56	4.54	14.49
3	20.16	2.69	3.70	21.76	12.98	2.37	2.55	21.72
4	21.50	1.57	1.32	13.58	13.82	1.40	7.45	18.32
5	16.52	1.26	4.19	22.43	8.01	No data		
6					12.06	4.85	8.98	7.86
7					11.07	0.76	5.35	8.82
8					11.38	1.76	8.67	8.62
9					17.35	2.70	8.23	6.63
10					14.13	2.00	7.34	7.90
11					12.50	2.38	6.88	9.35
12					14.20	4.27	9.40	10.19
13					8.88	3.30	6.43	11.21
14					11.92	1.43	8.94	5.69
15					9.28	1.99	3.18	12.59
16					10.54	1.57	4.53	22.16
17					14.03	1.07	9.19	7.71
18					11.49	3.62	5.12	13.16
19					10.14	1.75	6.24	10.57
20					11.28	1.84	6.31	9.68
21					9.04	8.36	6.17	10.95
22					9.07	2.65	7.14	8.99
23					12.69	2.13	8.68	10.45
24					11.48	1.61	8.10	11.01
25					12.48	1.72	1.32	11.36
26					10.24	1.67	No data	
27					15.01	1.35	9.30	16.52

[1] Displacement data is not available for the anchor pull out tests at SR

[2] Maximum displacements are limited by the available travel of the hydraulic ram that was used to load the anchor and as such, the loads associated with maximum displacement are those loads that were measured at the end of travel

possibly lateral tension stresses are influencing the failure mode and investigations in the laboratory will assist in determining if this is a result of the masonry being Australian, or whether an alternative model for anchor failure in older masonry is required.

4. Conclusions and future research

Premature failure of anchorages in previously strengthened URM buildings has been observed following the 2010-2011 Canterbury earthquake sequence and this observation suggest that a lack of understanding surrounding anchorage design is limiting the ability to

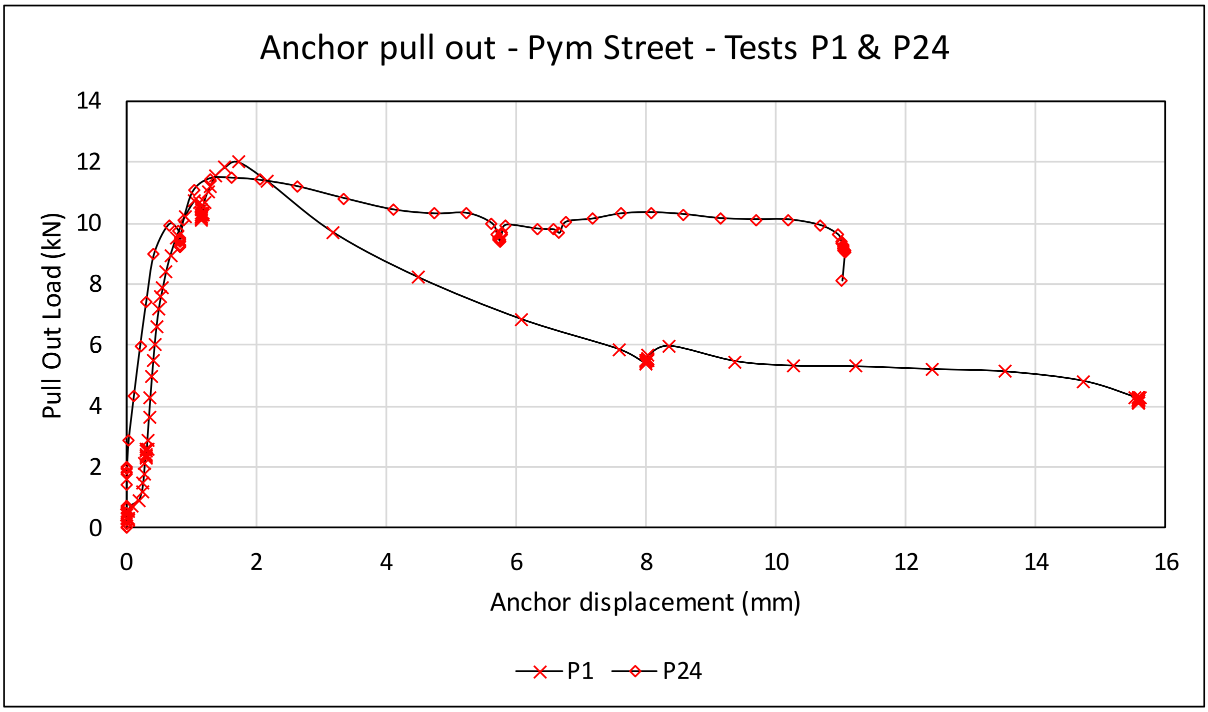


Figure 15 – Typical anchor load vs displacement - PS

seismically retrofit masonry structures. In-situ tests on three URM buildings have been undertaken to provide experimental data to develop and calibrate revised anchorage models for chemical anchors in vintage masonry. The outcomes of this field testing have shown poor correlation with published anchor capacities and failure mechanisms based on currently accepted design standards and analytical approaches with the predominant failure mode being brick splitting, and anchor pull out capacity remaining relatively constant over an extended extraction displacement. These discrepancies between site observations of anchor behaviour and generally accepted performance suggest that a new anchorage failure model is required. Also, due to complications encountered with common material test procedures, interpretation of standardised tests such as the "shove" test and the bond wrench test have proven to be difficult and alternative material tests and statistical distributions are proposed to overcome these difficulties. In particular, it is noted that:

- Anchor pull out capacity tests have resulted in a failure mechanism involving brick splitting with an extended softening region with some instances where the pull-out force after failure remained relatively constant over an extended anchor extraction. This failure mode does not compare well with the more commonly accepted failure models (typically involving cone types of failure) which have been developed from theories for anchor behaviour in concrete. A new model which predicts a splitting failure with an extended softening region needs to be developed to better understand the behaviour of chemical anchors in older vintage, historical (and by association) heritage masonry.

- In-situ testing for shear using the shove test is well documented and often used, but it can be damaging to the building fabric and the results can be difficult to interpret. An alternative test which loads the brick normal to the plane of the wall has been undertaken here. The benefit of this test is that it appears to demonstrate repeatability and provide results that are comparable to those from the shove test, but with less variability. Having less variability suggests that the test may provide a better method for estimating lower bound joint strengths that can be applied to design. A controlled laboratory investigation comparing the results of the brick pull-out test to those of the shove test should be undertaken with the aim of developing an alternative test for joint shear.

- The 5th percentile strengths based on the normal and lognormal distributions have been presented here as well as a simple calculation based on ranking the test data. For lower strength material tests, the normal distribution has generated 5th percentile results that are negative which clearly does not represent reality. Using the lognormal distribution prevents the generation of negative results, and skews the distribution "to the left" but it is based on a cumulative density function starting at zero with zero probability. The use of a simplified data ranking, particularly for the bond wrench test has shown considerable sensitivity to inclusion of zero strength (abandoned) tests which renders it minimally beneficial for development of design values. The authors consider the probability density function of the normal distribution (to the right of zero) better reflects the true population distribution provided that it is truncated at zero, but as this creates an unbalanced distribution for which the statistics are not valid, further work on determining a distribution that better fits the data is warranted.

- The bond wrench is a useful tool in laboratory experiments where test samples can be manufactured in the appropriate "ready to test" configuration but it provides limited scope for in-situ testing. In-situ bond wrench testing for this work resulted in a number of tests which were abandoned before any test load was applied, either because the mortar joint failed as a result of the removal of the adjacent bricks and perpends, or installation of the bond wrench tool after preparation of the test brick resulted in the mortar joint to be tested being broken. A reliable estimate of joint tensile capacity is a critical component of designing seismic retrofitting of masonry structures, but with the current poor reliability of the bond wrench test, a design value of zero for the tensile strength should be used and further research into alternative, less destructive joint tensile test methods needs to be undertaken so a more confident and useful design value can be established.

Acknowledgements

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