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Pull out capacity of chemical and mechanical anchors in clay masonry under quasi-static, cyclic and impact loading

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Abstract

Following the earthquakes in Christchurch New Zealand in 2010-11, post earthquake investigations revealed that in some instances, retrofitted seismic strengthening of un-reinforced masonry structures did not perform well. In particular, in a significant number of situations, it was the connection of the strengthening to the masonry that failed, or performed well below the expected capacity. This included failure of chemical and mechanical anchors as well as through bolt connections with backing plates. Failures included extraction of single or groups of masonry units (shear failure of mortar and/or bricks), wedge and cone failures of individual masonry units and failure of the bond/friction between anchors and the masonry unit.

This paper looks particularly at the pull out capacity of chemical anchors determined by in-situ testing. The testing has been conducted in masonry construction of three separate premises where the masonry is solid (not cored or hollow) and either plain or frogged bricks with lime mortar. Other tests particularly for determining material properties that have been undertaken at the same sites are discussed with some commentary regarding the difficulties encountered and implications regarding the results obtained.

Finally, an alternative test to the in-plane shear test for determining mortar shear capacity is proposed.

Keywords: masonry, chemical anchors, in-situ testing, pull out capacity

1 Introduction

Moon, Dizhur et al. (2014) report that the principle out of plane failure mode of seismically retrofitted parapet walls as a result of the Christchurch New Zealand 2010–2011 Canterbury Earthquake Sequence was through anchorage failure (the anchor pulling out of the masonry) or punching shear failure (either failure of single masonry units where the anchors did not pass through the wall, or groups of masonry units where through bolting with a backing plate had been used in the retrofit).

Furthermore, in 2016, Dizhur, Schultz et al. (Dizhur, Schultz et al. 2016) reported that observations of clay brick un-reinforced masonry (URM) buildings made during the initial reconnaissance and the subsequent damage surveys following the 2010/2011 Canterbury, New Zealand earthquakes showed that the masonry anchors appear to have failed prematurely.

There are a number of difficulties associated with investigation of anchors used for seismic retrofitting under seismic loading. Firstly, there is the need for a retrofit to have been undertaken and that building stock to still exist. In New Zealand, URM construction was banned following the 1931 Hawkes Bay earthquake. The available URM building stock is therefore limited in New Zealand to structures built before this time. In Australia, seismic events have been recorded since white settlement with the first report made by Governor Phillip in 1788 (Woodside and McCue 2016). However, publication of a design code of practice for engineers did not occur until the release of AS 2121 in 1979, but there was not a requirement to adopt the documented principles and loadings until it was incorporated in the Building Code of Australia (as a new Standard AS 1170.4) in 1995 (Woodside and McCue 2016). Exceptions to this were that 1) the Western Australian government required buildings in the Meckering area to adopt earthquake design to AS 2121 as a direct response to the earthquake in that area in 1968, and 2) the South Australian Building Act incorporated AS 2121 in 1982 (Woodside and McCue 2016). Similar interest in seismic events, but with reticence for adoption of design requirements has occurred to varying degrees in other parts of the world, including the United Kingdom, the United States of America, Japan, and Europe (Eiby 1975, Dhakal 2011, Gobesz and Kegyes 2013, Beavers, Hall et al. 2014, Ishiyama 2016).

Secondly, "grandfathering" clauses, generally adopted in most jurisdictions, that do not require adoption of updated or improved design requirements in existing structures unless major changes to that structure are proposed, have also hampered seismic understanding. So whilst we now generally have well understood and codified methodologies for seismic design, it is still only a requirement for new or significantly modified construction to use those methodologies, and thirdly, a suitably large seismic event needs to have occurred where a URM building or buildings have been seismically retrofitted and that following that event, the performance of the retrofit has been assessed.

In Australia, we are fortunate to have a low frequency of significant seismic activity. But the corollary of this is that whilst the percentage of URM building stock which has been seismically retrofitted is increasing, there is little knowledge of how well that augmentation has performed. And whilst we do experience a low frequency of significant seismic activity, the consequences are none-the-less significant. This is evidenced by insurance industry reporting that the risk to the economy of even a moderate 'design earthquake' in any Australian capital city is billions of dollars (Blong 1993).

Research currently being undertaken at the University of Adelaide aims to develop a cost-effective technique to mitigate the seismic risk posed by the many URM structures in Australia and elsewhere with a particular emphasis on parapets and walls that loom over the footpaths of every Australian city. This paper presents some early results from in-situ investigations undertaken on three URM buildings along the South Road corridor in Adelaide this year.

2 In-situ Investigations

As part of the State and Federal government's North-South Corridor development, many residential, commercial and industrial properties have had to be compulsorily acquired. This has allowed the University in conjunction with the Department of Planning, Transport and Infrastructure (DPTI) access to these properties before they were demolished for conducting in-situ experiments on existing masonry buildings. In particular, older construction comprising solid masonry units (not cored or hollow) with lime mortar were selected as these types of

Figure 1 - Croydon Park house locations

structures best represented the historical (and heritage) construction throughout Adelaide and South Australia.

Given the age of the developments in the area (between Pym Street and Regency Roads at Croydon Park), it was expected that there would be many suitable buildings for investigations. However, following detailed inspection of the (almost 50) properties ear-marked for demolition, less than half met the initial selection criterion. Additionally, the DPTI demolition programme was very tight. We finally settled on three properties with locations as shown on Figure 1 [Base map Google Maps (Google Maps 2019)]. The demolition programme for these three properties matched well with our own timing allowing us the maximum possible time for the in-situ testing.

Details of the tests undertaken and results obtained are discussed later in this paper. The tests undertaken and when at each of the test sites are noted below:

2.1 Pym Street

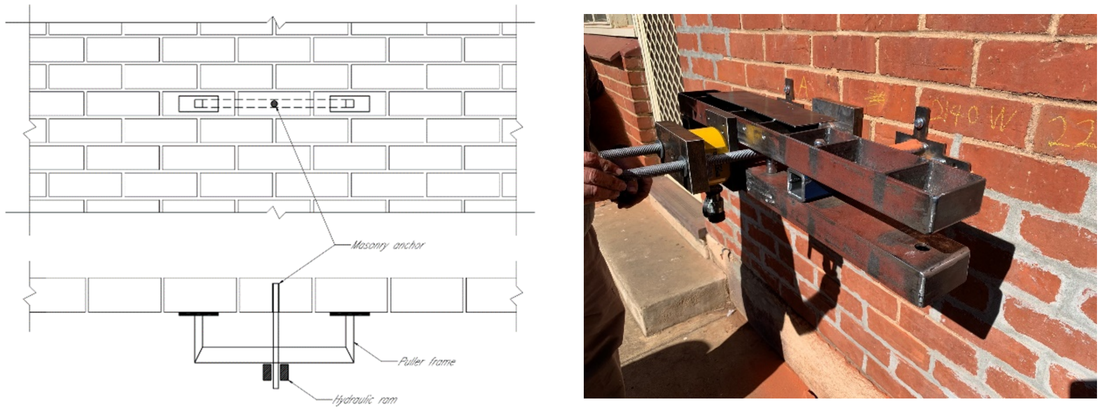
This site was visited on three separate occasions being the 3rd and 30th of May and the 13th of June, all this year. The tests undertaken were anchor pull out capacity (POC), in-plane shear ("shove"), brick pull out, bond wrench and punching shear ("pattress plate"). In this paper the term "pattress plate" is used to distinguish the stiffened oval plate that has been used for the punching shear tests from other plates such as "rose" or simple "anchor" plates.

2.2 Minerva Crescent

This site was visited on three separate occasions being the 8th, 13th and 14th of May, all this year. The tests undertaken were anchor pull out capacity (POC), in-plane shear ("shove"), brick pull out and bond wrench. The punching shear test was not undertaken at this site as testing at the Pym Street site demonstrated that our "pulling rig" was too short to obtain meaningful results but by the time a new rig was constructed, this house had been demolished.

2.3 South Road

This site was also visited on three separate occasions being the 1st, 2nd and the 3rd of July, all this year. The tests undertaken were anchor pull out capacity (POC), brick pull out, bond wrench and punching shear ("pattress plate"). The in-plane shear test was not undertaken at this site as testing at both Pym and Minerva demonstrated a number of pitfalls with the test. These are discussed later.



(a) Schematic layout (b) Test apparatus

Figure 2 - Anchor pull out test

3 In-situ Tests

3.1 Anchor "pull out" – this "quasi-static" test has been used extensively in work undertaken by Dizhur, Schultz et al. (2016) on New Zealand masonry. However, there is little information available for Australian masonry construction. Figure 2 shows a schematic layout of the testing apparatus and a photograph of the apparatus as used. It applies tension to the anchor by pressure applied through the hydraulic cylinder. For the in-situ testing only one size (M12) of chemical anchor has been used as the documented tensile capacity of anchors in masonry (Hilti HIT-HY 170) is not generally dependent upon diameter.

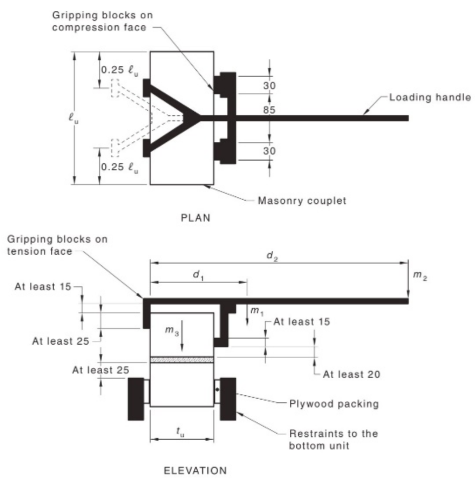


Figure 3 - Schematic diagram of the bond wrench

3.2 Bond wrench – this "static" test is fully described in AS 3700 (Masonry structures) (Standards Australia 2018). The aim of the test is to provide a simple method to determine in-situ tensile (in bending) strength of mortar. Figure 3, taken from AS 3700 shows the general arrangement of the test. Note that this arrangement is shown for testing of a couplet of bricks with a single mortar joint. With the in-situ test, the lower brick is adequately held in the wall.

3.3 Brick "pull out" – this "quasi static" test is intended to extract a single masonry unit "squarely" from the masonry. A heavy steel plate with dimensions approximating those of the masonry unit is mounted on a through bolt onto the back of the wall and hydraulically pulled out of the wall. The testing rig is similar to that of the anchor pulling frame shown in Figure 2 with the actual test rig shown in Figure 4. This test is to determine the shear capacity of the brick/mortar interface, eliminating interaction with the masonry units. The close spacing of the base plates of the puller frame minimises any bending effects. It is being trialled as an alternative to the in-plane shear test.

3.4 In-plane shear – or "shove" test. This "quasi-static" test is fully described in AS 3826 (Strengthening existing buildings for earthquake) (Standards Australia 1998). The test, shown in Figure 5 (from AS 3826) is used to determine the combined shear capacity of the mortar joints directly above and below the masonry unit being tested.

Figure 4 - Brick pull out test

In this configuration, the "test unit" is loaded by the hydraulic ram. As the load is increased, the test unit will deflect elastically 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. Friction and dilation (upward movement of the units above and downward movement of the units below) also need to be considered. A number of difficulties were encountered undertaking these tests which are discussed later in "Results".

3.5 Punching shear – this "quasi-static" test is similar to the anchor pull out test and the brick pull out test except that a "large" pattress plate covering a number of bricks is mounted

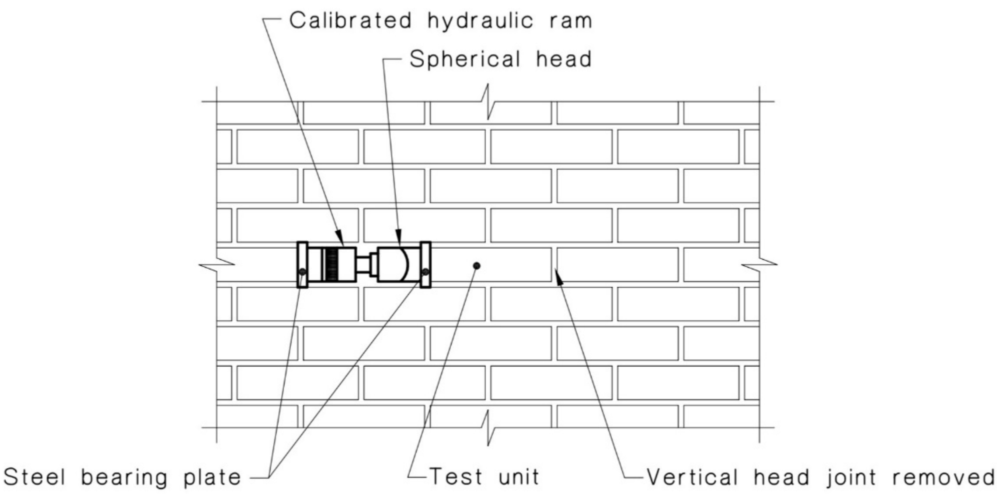


Figure 5 - Simplified in-situ bed joint shear test

on a through bolt on the inside face of the outer leaf of the wall. It is then pulled through the wall to determine the "punching shear" capacity of a group of masonry units. A "pulling frame" with larger span (1500mm) than for the anchor and brick pull out tests is used to mount the hydraulic cylinder, and the frame is oriented vertically. The test rig used is shown in Figure 6.

(a) Test rig after wall failure

(b) Pattress plate on inside face of outer leaf

Figure 6 - Punching shear test

4 Results

This paper is predominantly concerned with the results obtained from the anchor pull out tests conducted on the three houses in Croydon Park, but there is also some discussion of the results from the other tests undertaken and some of the difficulties encountered doing those tests and implications for further tests.

4.1 Anchor pull out

The results for the anchor pull out tests undertaken at the three locations are shown in Figure 7. The anchors used were Hilti HIT-V-5.8 M12 anchor rod 150mm long used with Hilti HIT-HY 170 standard hybrid mortar. These were installed into solid clay masonry notionally central in the unit with an effective embedment depth of 80mm. The holes were drilled using a 14mm diameter solid masonry drill bit with a Hilti TE 6-A22 cordless rotary hammer with a single impact energy of 2.5J and a hammer frequency of 5100 impacts per minute. Enough mortar was injected to allow the hole to be overfilled with the threaded anchor fully inserted with excess mortar being extruded. Different curing times were used with the shortest being one hour between installation and testing and the longest 14 days. The in-situ installation and testing of anchors was undertaken in autumn and winter with typical average daytime maximum temperatures between 15^{o} and 20^{o}C. Temperature of the masonry was not recorded but would have been generally between these temperature ranges. The Hilti design manual (Hilti 2019) recommends a maximum tightening torque of 6Nm for the M12 HIT-V anchors. A torque wrench was not used for the installation but the anchors were only “nipped” up to facilitate installation of the testing rig, not exceeding this torque.

The anchors were installed at varying heights in the masonry to allow for differing “normal” stress and consequently frictional resistance between the mortar and masonry. However, there were no instances of the brick pulling out in any of the tests that were conducted, removing this complication.

Based on the Hilti design manual for pull-out failure of the anchor, brick breakout failure or local brick failure at characteristic edge distances for a single anchor, the design resistance for these anchors is 1.2kN which is at face value significantly less that the results obtained. This design resistance was used as these modes of failure were the only failure modes observed. The design manual takes into account a 50 year design life however and (likely) manufacturers risk aversion. If we allow the following design considerations as given in AS 3700 – Masonry Structures (Standards Australia 2018), AS 1170.0 - Structural design actions: General principles (Standards Australia 2002), AS 1170.4 - Structural design actions: Earthquake actions in Australia (Standards Australia 2007) and the National Construction Code of Australia (Australian Building Codes Board 2011) for an earthquake load in a retrofitted structure that meets the requirements of AS 3826 - Strengthening existing buildings for earthquake (Standards Australia 1998) with:

- Importance level 2
- Earthquake design category III
- Threshold load 2/3 × AS 1170.4
- Capacity reduction factor 0.65

and using the lowest strength encountered in the test data (8kN), the nominal design tensile capacity of the anchor is:

$$N^* = \frac{8 \times 0.65}{\left(\frac{2}{3}\right)} = 7.8kN \qquad (1)$$

which is significantly larger than the published design resistance. If we look at a more demanding scenario of a live load in a typical masonry structure that doesn't meet the requirements of AS 3826, then:

- Earthquake factor 0.30 – (live load office floor)
- Capacity reduction factor 0.60
- Live load factor 1.50

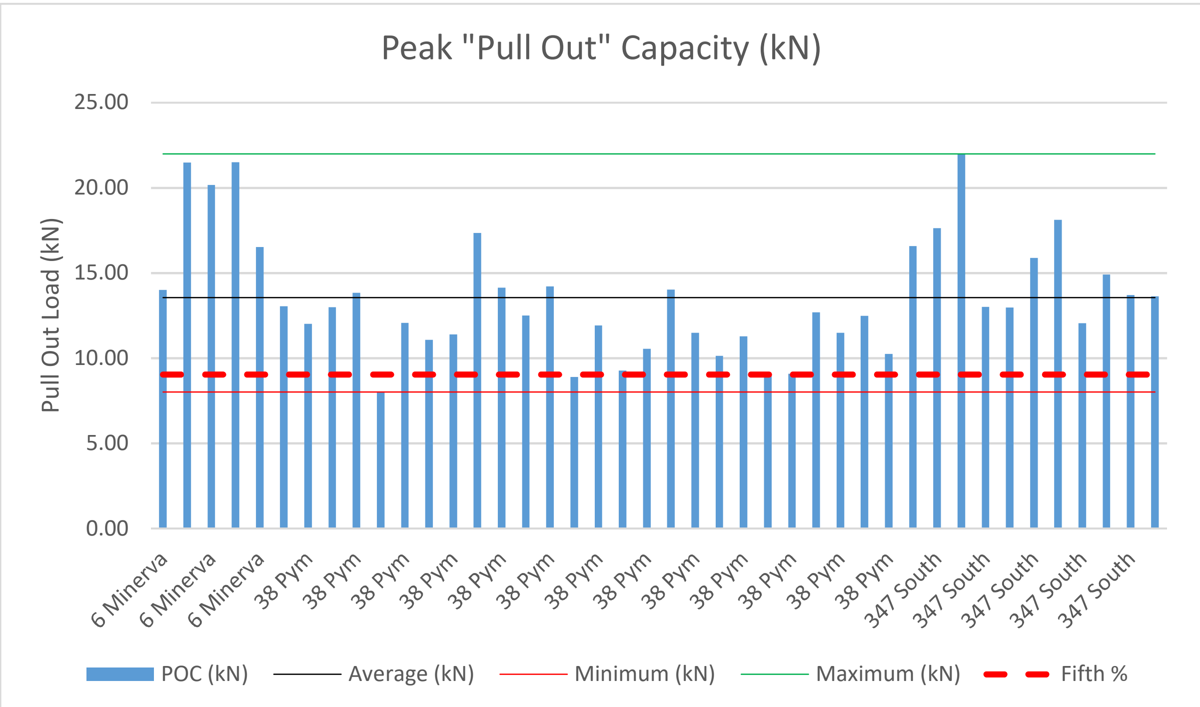


Figure 7 - Chemical anchor pull out capacity

and again using 8kN, the nominal design tensile capacity of the anchor is:

$$N^* = \frac{8 \times 0.30 \times 0.65}{1.5} = 1.0kN \quad (2)$$

or 1.6 kN if the capacity reduction factor is excluded which does compare well with the design resistance from the Hilti design manual as the design resistance has already factored the capacity reduction factor into the analysis. If the fifth percentile of strength of the recorded data is used (9kN) then the results, especially for earthquake retrofitting are even more conservative, except it should be noted that Hilti specifically exclude the use of their chemical anchors in seismic loading scenarios. The calculation of the fifth percentile from ETAG 029 (European Organisation for Technical Approvals 2013) tends towards a normal distribution when there are many test points, and towards a log-normal distribution when there are few. The pull out capacity values for those options as well as using the direct data points are shown below.

	Pym	Minerva	South	Combined
Mean (kN)	12.00	18.73	15.39	13.65
Standard dev. (kN)	2.03	3.33	3.08	3.40
Coefficient of Variation (%)	17	18	20	25
5th Percentile [raw] (kN)	9.05	14.51	12.46	9.07
5th Percentile [log-normal] (kN)	5.98	7.91	6.23	4.67
5th Percentile [ETAG 029] (kN)	7.67	7.39	7.49	6.82
5th Percentile [normal] (kN)	8.66	13.25	10.33	8.06

If the fifth percentile ETAG value is used (for the combined capacity), and the capacity reduction factor is excluded (as above), then the tensile capacity becomes 1.4kN, still comparing well with the Hilti design manual.

4.2 Bond wrench

The results for these tests were quite varied and generally unreliable. A number of the tests undertaken had a result of zero with others spread over a considerable range, from a minimum of 0.001 MPa to a maximum of 0.97 MPa. The fifth percentile flexural tensile strength was 0.002 MPa. It is worth noting that the flexural characteristic strength for unreinforced clay masonry is given is AS 3700 as 0.2 MPa, which is designed to represent the fifth percentile value.

There are a number of difficulties we found with the test that has contributed to the spread of data. Assuming a standard stretcher bond construction, the two bricks above the test specimen must be removed from the wall and the perpend joints either side of the test specimen must also be removed down to the level of the upper face of the course below. This requires significant effort and the vibrations from the tools used is likely to have damaged many of the mortar joints, accounting for the some of the “zero” results.

Further contributing factors to the variability are the quality of workmanship, the mortar mix that was used on the day, the wetness of that mix and the dampness of the bricks when laid, the temperature of the day, whether the brick (if frogged) was laid frog up or down etc.

The only anecdotal conclusion that we could draw from the information gathered to date is that when the bricks were laid “frogs up” the joint was generally stronger likely to be a result of more complete bedding.

4.3 In-plane shear

The results from these tests showed greater consistency than those from the bond wrench tests. This can be particularly attributed to a less invasive demolition process. The brick being tested remains under the existing confining (normal) stress which generally holds it against damage, thus providing a good test specimen. The difficulties that occurred with this test have come from interpretation of the results.

The test arrangement shown in Figure 5 is quite idealised. Unless the hydraulic ram is mounted exactly parallel to the bed joints, the load is not axial. This has implications for understanding the dilation of the courses above and below the unit being tested and thus the normal stresses, and consequently the friction component of the calculated shear capacity. Slightly more complex test setups are also suggested using flat jacks above and below the brick being tested. This has some ability to provide a better understanding of the friction component but the setup is considerably more invasive with a greatly increased potential for damaging the joints under test rendering the results unreliable.

A further complication of this test is that unless there is a considerable extent of wall either side of the test location, and in the same plane as the test specimen, there is limited ability for the wall to resist the thrust being applied. This has resulted in entire wedges of masonry (failure planes following mortar joints and perpends at about 35° above and below the line of thrust) being thrust out of the end of the wall (at a corner) and more interestingly on one occasion, the failure plane travelled downwards to the footing whence the wall slid on the damp proof membrane and subsequently developed a bending drack propagating notionally vertically above the test location, stopping a few courses before the top of the wall.

In the few instances where the test did manage to shear the courses above and below the brick being tested and there were no overall wall failures, the results remained difficult to interpret. In particular, it was expected that the applied axial load would continue to rise to a failure peak and then reduce to a "constant" value as a result of friction. We were not able to obtain this result with any instances of the tests resulting in the load continuing to increase with increased displacement, albeit at a lower rate than before slipping occurred. It is likely that if a larger displacement was possible (removal of the adjacent brick rather than only the perpend) then a better estimate of the friction component could be made, provided there is not excessive damage caused to the mortar joints through grinding as the brick is pushed laterally.

Due to the abovementioned difficulties encountered with these tests, we abandoned them early in the in-situ testing programme. The shear stress results (MPa) obtained at maximum displacement from those tests were:

	4A Minerva	6 Minerva	38 Pym
Minimum	0.54	0.33	0.14
Maximum	1.30	0.89	0.26
Average	0.92	0.61	0.19
Fifth percentile	0.58	0.37	0.14

Note that:

- 4A Minerva was a modern house with cement mortar and cored modern bricks and was used as a comparison only
- Although dilation was measured, no attempt has been used to account for normal stress and consequently friction effects

4.4 Brick pull out

The brick pull out test involves breaking out enough wall of the inner leaf to fit a plate approximating the elevation size of the brick being tested (we have used 200x50) to allow it to

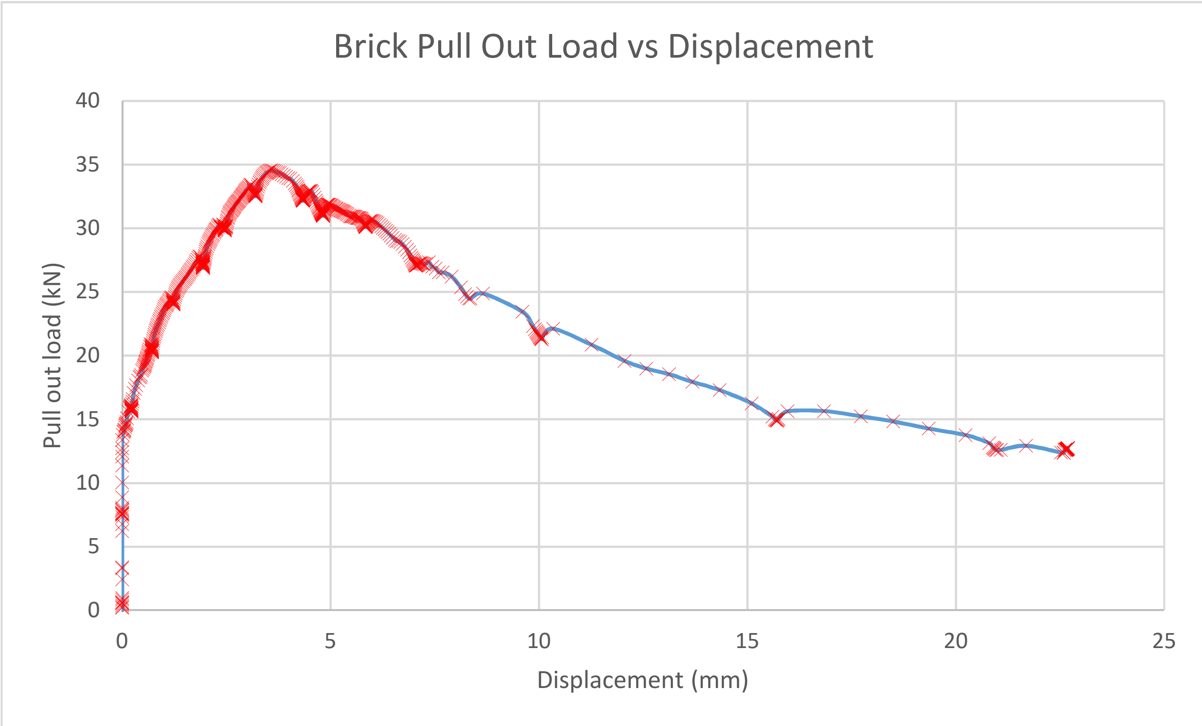


Figure 8 - Brick pull out load vs displacement

bear against the inner face of the outer leaf of masonry. It is subsequently pulled via a through bolt. This test is clearly more invasive (to the inner wall leaf) than the in-plane shear test but it is minimally damaging to the outer leaf which is important from a test result perspective as well as (if applicable) a heritage perspective. If the inner leaf is rendered and/or painted, it is a relatively easy repair. Experience to date is that there is minimal damage caused to the outer leaf, other than the brick unit that was drilled to fit the through bolt, and that that brick is extracted from the wall.

The results obtained to date show a clear peak load/stress with a subsequent tapering off as the brick is extracted and the shear/friction surface area reduces. Once accurate laboratory testing has been undertaken, we expect to be able to use the results to make an estimate of the friction component. A typical load displacement plot is shown at Figure 8. Looking at the statistics obtained from the in-plane shear test and those from this test, we believe there is some possibility that this test can become an alternative to the in-plane test in situations where the in-plane test is not suitable. The peak shear stress results (MPa) obtained from the brick pull out tests are shown below.

	6 Minerva	38 Pym	347 South
Minimum	0.36	0.20	0.37
Maximum	1.20	0.41	0.73
Average	0.59	0.28	0.51
Fifth percentile	0.36	0.21	0.38

Figure 9 - Punching shear test in progress

Note that no correction has been made to the above results to account for friction so that they are able to better compare to the in-plane tests.

4.5 Punching shear

The punching shear tests are similar to the brick pull out tests except that a larger "pattress" plate has been used which covers more than a single masonry unit allowing an interaction between the shear capacity of the masonry unit/mortar interface as well as allowing for damage to the wall as a whole resulting from one way and two way bending. The test rig that has been used is shown at Figure 6.

One of the difficulties that we have experienced in this test is that there is not necessarily adequate bending strength in the masonry to achieve a punching shear failure. Figure 9 shows one of the tests on the South Road house where the wall deflection was close to 80mm out of plane with two way bending cracking evident, but no shear failure.

At this time, interpretation and analysis of the results is not yet complete.

5 Further work

Additional in-situ testing will be undertaken when further premises become available. Laboratory testing is commencing shortly where greater accuracy in setting up experiments and measurements are possible. Those experiments will use solid (frogged) old and new masonry units and will look at the same tests that have been conducted in the field to allow data comparison and extension to be undertaken. It will also include:

Anchors

- different anchor types (chemical, mechanical)
- different anchor diameters
- different anchor manufacturers
- good and poor installation quality
- cyclical loading
- impact loading

Test methods *(in manufactured wall units)*
- in-plane shear
- brick pull out

Material properties
- mortar compressive and shear strengths
- masonry unit compressive, shear and tensile strengths

Shake table *(Large scale tests on the University shake table with manufactured walls)*
- un-strengthened walls
- strengthened (retrofitted) walls

6 Conclusions

This paper has particularly set out to look at the pull out capacity of chemical anchors in older masonry structures (solid masonry units with lime mortar) using installation methods typical of the industry, to establish a base line for that capacity in Australian masonry construction. The results found to date indicate that in a quasi-static loading scenario, the pull out capacities are comparable to the published design capacities from the manufacturers, which also accords with other observers, eg. Dizhur, Schultz et al. (2016). As such, this does not suggest that anchors would fail prematurely assuming that they were designed correctly and installed in accordance with the manufacturer's instructions initially.

Particularly with regards to these types of anchors, additional work involving variable construction quality, cyclical and impact loading will be undertaken to better understand why premature failures have been observed during seismic events.

Other tests involving whole and multiple masonry units in in-situ walls have been discussed with some preliminary observations also being made. This has included the use of an alternative in-situ test for determining mortar shear strength of existing masonry.

7 Acknowledgements

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