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DIAGONAL TENSION STRENGTH OF VINTAGE UNREINFORCED CLAY BRICK MASONRY WALL PANELS

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ABSTRACT

An experimental program was implemented where wall panels were obtained from two existing vintage unreinforced clay brick masonry (URM) buildings and subjected to a diagonal compression loading condition in order to induce a diagonal tension failure mode. The principal aims of the experimental program were to establish the diagonal tension strength of the vintage clay brick URM walls present in the two buildings and to establish a benchmark mortar mix suitable for use when manufacturing replica clay brick test assemblages that adequately represent the strength characteristics present in vintage URM buildings. It was concluded that the use of recycled clay bricks acquired from vintage URM buildings and a mortar mix of 1:2:9 (cement:lime:sand) appropriately replicates the material strength characteristics that are present in existing vintage clay brick URM buildings located in New Zealand.

1 INTRODUCTION

Recent earthquakes, such as the 2010 M7.1 Darfield (New Zealand) earthquake [1] and subsequent 2011 M6.3 Christchurch earthquake [2], have again highlighted the inadequate seismic performance of vintage unreinforced clay brick masonry (URM) construction [3-5]. Depending on the orientation of the building with respect to the earthquake epicentre, URM walls are subjected to a combination of lateral earthquake induced forces in the form of face (out-of-plane) loading and in-plane loading. Of the two loading conditions, the out-of-plane wall failure mechanism is generally considered to be the more critical deficiency in URM buildings [6], but once the out-of-plane failure mechanism is inhibited through the addition of adequate wall-diaphragm connections (and if necessary supplementary supporting structure) it is typically the response of the in-plane loaded walls that governs the global seismic performance of a URM building [7]. The in-plane wall failure mode depends upon wall aspect ratio, wall boundary conditions, the size and geometry of wall penetrations, the magnitude of axial compressive force and the wall constituent material properties [7].

The in-plane failure modes of URM walls may be categorised as either flexural or shear controlled [7]. The flexural mode of failure is characterised by rocking with associated masonry crushing, while the shear failure mode of URM wall components is identified by two main mechanisms: horizontal bed joint sliding and diagonal shear (tension) cracking [7]. Depending on the ratio of mortar strength to brick strength, diagonal tension cracking may result in diagonally inclined cracks through the brick units or may follow a diagonally inclined stepped pattern through the mortar bed and head joints. The latter is due to the formation of tensile horizontal cracks in the bed joints which can potentially become sliding planes when subjected to reverse earthquake loading. During earthquake shaking, sliding (see Figure 1a) and rocking failure modes may allow URM wall components to exhibit considerable displacement capacity, whereas diagonal tension cracking (see Figure 1b) is typically considered as a brittle failure mode where the initiation of visible diagonal cracking is associated with the peak lateral force capacity, followed by rapid strength degradation. Hence, the diagonal tension strength of URM wall components that are subjected to in-plane loads is a paramount strength parameter that is required for assessing the seismic performance of URM buildings, with invasive in-situ testing of existing URM wall components being a preferred method to establish these diagonal tension strength characteristics accurately. However, the feasibility of in-situ testing is limited due to the highly destructive nature, high cost, and prolonged preparation and testing duration constraints that are typically associated with this form of testing. Instead, 'indirect' approaches based on simplified theoretical models provided in building assessment codes and guidelines are typically used. One such simplified model that is commonly used by practicing structural engineers when assessing the diagonal tension strength of wall components in URM buildings is presented in ASCE – 41 [8]. This model allows prediction of the diagonal tension strength to be made based on wall constituent material properties and simplified in-situ testing procedures.

Following a detailed seismic assessment of a building having multiple URM walls, any wall elements found to have inadequate diagonal tension strength are required to be seismically improved. Due to the aforementioned constraints associated with in-situ testing, the effectiveness of various improvement techniques that enhance the performance of seismically deficient URM wall components is typically evaluated using replicated wall components in a laboratory-based setting. Such studies allow experimental evaluation of a large number of test specimens to be conducted in a cost effective manner and in a controlled environment. However, replicated wall components must have strength characteristics that are adequately representative of those present in existing vintage URM buildings in order for the laboratory study to have relevance.

Testing of existing stone rubble masonry wall components in diagonal tension was previously reported in the literature [9-15], however comparable research studies are unavailable for vintage clay brick URM buildings such as typically encountered in New World countries of North America and European colonies in the Southern Hemisphere. Details of historic URM construction that are typically encountered, particularly in New Zealand, were previously reported by Russell and Ingham [16].

To address the paucity of available data as detailed above, an experimental program was implemented where as-built (unretrofitted) wall panels obtained from two existing vintage URM buildings were tested in diagonal tension. The constituent materials used in the original construction of the two selected buildings are considered representative of the typical URM building stock present in New Zealand the typical brick compressive strength is typically much greater than the mortar compressive strength [17]. The two main aims of the reported experimental program were: (a) to establish the diagonal tensile strength of the two existing vintage clay brick URM buildings and validate the accuracy of the existing ASCE – 41 [8] diagonal tension strength model; and (b) establish a benchmark mortar mix suitable for use when manufacturing replica clay brick test assemblages that adequately represent the strength characteristics in existing vintage URM buildings.

2 PREVIOUS RESEARCH

ASTM E-519 [18] provides a standard test method for determining the diagonal tension strength of masonry assemblages. The test method involves the application of diagonal forces at the two opposite corners of a 1200 mm × 1200 mm masonry wall panel. The main focus of previous research that considered the in-situ diagonal tension strength of URM walls and adopted the above test method (or similar) was directed to the testing of existing vintage stone rubble masonry [9-15]. As reported by Brignola et al. [9] and others, the diagonal tension strength of vintage stone rubble masonry, which typically has a chaotic masonry pattern, is not representative of the diagonal tension strength of solid clay brick URM construction due to the latter's orderly bond pattern and regular use of header bricks to inter-connect the masonry leafs. Consequently, an experimental program was conducted to address this knowledge gap and to establish a benchmark for companion laboratory-based research [19-24] that considered the diagonal tension strength of vintage clay brick URM walls using recycled vintage clay bricks. Selected available experimental results from the aforementioned literature are provided in Table 1. The results are presented along with geometrical properties, material properties, and mortar mix ratio (cement:lime:sand) by volume for wall panels constructed in a laboratory-based setting. Note that the testing reported here was conducted prior to the reported companion studies, but that the release of test results was delayed due to the authors' prolonged involvement in the reconnaissance efforts following the 2010/2011 Canterbury earthquakes [4, 5, 25].

The correlation between the maximum measured experimental diagonal tension stress (maximum shear stress) and the mortar compressive strength for those wall panels tested in the companion studies [20, 21, 23] that closely resembled the masonry type present in the two selected buildings (regular solid clay bricks) is illustrated in Figure 2. As expected, the diagonal tension strength of wall panels generally increases with increasing mortar compressive strength. The trend, even though of low correlation, highlights the importance of establishing a benchmark mortar mix for replica wall components manufactured in a laboratory-based setting in order to provide accurate representation of strength characteristics found in existing vintage URM buildings.

3 EXPERIMENTAL PROGRAM

An experimental testing program was implemented to investigate the diagonal tension strength of existing New Zealand vintage clay brick URM buildings. The program involved the extraction of six wall panels from the B-East building (part of the Allen's Trade Complex, hereafter referred to as Building 1 and shown in Figure 3a) and the in-situ testing of two wall panels located in the Avon House building (hereafter referred to as Building 2, and shown in Figure 3b). Larger number of wall panels was not attainable due to practical and financial constraints. Wall panels from the two buildings were cut as close to 1200 mm × 1200 mm as practical and tested in diagonal tension to determine the diagonal tensile strength of masonry in the test buildings and to provide direct comparison to the laboratory-built wall panels tested in accordance with ASTM E-519 [18].

3.1 Building 1

Building 1 is a two storey building that was constructed in 1911 as part of the Allen's Trade Complex, located in Gisborne. The URM walls of the buildings consist of multi-leaf clay brick masonry constructed in common bond (aka American bond) with layers of stretchers and headers every 3-6 brick courses. The building was originally sandwiched between an existing timber framed building adjacent to the northern end gable (which shared a wall with Building 1) and a demolished building that previously was located adjacent to the southern end gable. Although the exact date of the adjacent building's demolition is uncertain, the removal of that building left the masonry of the southern end gable exposed to the environment for over a decade (see Figure 3a).

Building 1 suffered extensive structural damage during the M6.8 2007 Gisborne earthquake [26] to the extent that significant post-earthquake repairs were necessary and removal of the two gable ended walls was required. As a result, the building was designated as unstable and unsafe to occupy until reconstruction took place. Due to a limited time frame for building reconstruction, extraction of wall panels from an earthquake damaged three leaf URM south wall was adopted instead of in-situ testing.

The locations of the wall panels to be extracted from Buildings 1 were carefully selected to minimise the required length of masonry cutting, the cutting locations were clearly marked, and the vertical cuts were made using a concrete cutting chainsaw. In order to extract the wall panels, two through penetrations per panel were made to enable the use of lifting straps. To prevent the panels from cracking or breaking during lifting and/or transportation, each wall panel was vertically strapped with two heavy duty ratchet tie-downs. The applied axial force from the ratchet tie-downs minimised the risk of damage by securing the masonry together. Once the cutting and securing work was completed the wall panels were lifted out of the building and onto a truck using a medium sized crane, and were transported in an undamaged condition via a 6 hour journey to a testing facility. The wall panel preparation and extraction procedure is illustrated in Figure 4. The geometrical parameters and masonry bond pattern of the extracted wall panels (denoted as wall panels 1 to 6) from Building 1 are shown in Figure 5a, with wall panel dimensions summarised in Table 2.

3.2 Building 2

Building 2 (as shown in Figure 3b) was constructed in Wellington in 1884 using British architecture and construction techniques. The single storey building has walls constructed of unreinforced clay brick masonry, with a timber framed trussed roof overlaid with corrugated iron sheets. The internal, non-load bearing partition walls consist of single leaf URM walls constructed using running bond pattern (only stretcher courses), and the load bearing internal walls are composed of solid two leaf masonry (irregular bond pattern) with a plaster layer (weak approximately 20 mm thick lime-sand plaster reinforced with horse hair) on both sides of the wall. All walls consist of handmade clay brick masonry with lime-cement mortar. Thin metal strips measuring 25 mm × 1 mm were found enclosed within the mortar bed joints of the single leaf walls, that extended horizontally throughout the wall length at an irregular vertical spacing of greater than 800 mm. The building was scheduled to undergo seismic strengthening and substantial internal alterations that involved the removal of a number of internal URM walls, which allowed a section of a two leaf load-bearing wall (wall panel 7) and a section of non load-bearing single leaf partition wall (wall panel 8) to be tested in diagonal tension. Geometrical parameters and the masonry bond pattern of the tested wall panels from Building 2 are shown in Figures 5b and 5c, with wall panel dimensions summarised in Table 2.

4 MATERIAL CHARACTERISATION

The masonry used in the construction of Building 1 and Building 2 typically consisted of 75 mm × 220 mm × 105 mm solid red clay bricks with approximately 12 mm thick cement-lime mortar joints. For Building 1 the constituent materials were of good quality, with the mortar joints, when viewed from the interior side of the wall being less weathered and slightly darker in colour than when viewed from the external side of the wall, which was attributed to environmental exposure of the wall exterior. For Building 2 the constituent materials were of poor quality and consisted of soft red bricks (split easily when hit with a hammer) with soft lime-cement mortar. Close inspection of the two leaf walls in Building 2 revealed no mortar (a void) in the collar-joints between the two leafs. Furthermore, visual inspection and scratch tests of the lime-cement mortar indicated low strength properties [17].

4.1 Compression strength

Irregular mortar samples, single bricks and three brick high masonry prisms were extracted from the buildings and from wall panels following testing, and were assessed in the laboratory. Individual bricks extracted from the buildings for laboratory testing were subjected to the half brick compression test according to ASTM C 67 – 03a [27]. The compressive strength of the mortar was obtained through compression testing of irregular mortar samples in the laboratory. It was not possible to employ the mortar compression test recommended in ASTM C 109 – 08 [28] as 50 mm cube samples are typically unattainable from actual buildings, where mortar joints are only 15 to 20 mm thick. Instead, irregular samples were cut into approximately cubic shapes and tested in compression. As suggested by Lumantarna [17], a calibration factor was then applied to account for the mortar sample aspect ratio. Masonry prisms were tested in compression in accordance with ASTM 1314 [29]. Potentiometers were incorporated in the masonry prism testing setup to obtain the compression stress-strain response, and to calculate the Modulus of Elasticity, which was determined as the slope of the stress-strain curves between 0.05 and 0.70 times the maximum prism compressive strength [17]. Flexural bond strength was determined on site following ASTM 1072 – 00a [30]. Results from the aforementioned tests are summarised in Table 3.

4.2 Bed joint shear strength

The mortar bed joint shear strength (v_{to}) was determined on-site by conducting 8 and 6 in-situ bed joint shear tests as per ASTM C 1531 – 03 [31] in Building 1 and Building 2 respectively. The bed joint shear tests were conducted using a 12 tonne hydraulic actuator on wall sections located in the vicinity of where test panels were tested/extracted, with the magnitude of overburden estimated as the weight of the masonry above the test locations using a masonry density of 1700 kg/m^3 [17]. The average bed joint shear strength (v_{to}) results, adjusted for different levels of overburden as per ASCE-41 [8], are summarised in Table 4. For both buildings it was concluded that due to low increases in overburden the mortar bed joint shear strength did not markedly increase with an increasing level of axial precompression load and that the average shear strengths (v_{to}) were equal to 0.19 MPa and 0.16 MPa for Building 1 and Building 2 respectively.

4.3 Test setup

The wall panels from Building 1 and Building 2 were tested in diagonal tension in accordance with ASTM E519 – 07 [18]. The standard testing procedure involves rotation of the URM wall panel by 45° and vertical loading along one of the wall's diagonals. Due to wall panels 7 and 8 being tested in-situ and because of the low masonry bond strength of wall panels 1 to 6, the standard test method was modified such that the wall panel remained vertical in its original orientation and the loading mechanism was rotated as shown in Figure 6a. The setup used to test wall panel 7 (a similar setup was also used to test wall panel 8) is shown in Figure 6b. Loading was applied by a hydraulic actuator positioned between the load cell and the loading plate, which when loaded developed tension forces in the two steel rods positioned between the steel loading plates, consequently compressing the wall panel diagonally. The applied diagonal compression force was gradually increased until wall panel failure occurred. Load was applied at a uniform rate (approximately 0.5 kN/sec) using a hydraulic pump and a 300 kN hydraulic actuator coupled with a 150 kN load cell. Deformation of the wall panels was measured using two diagonally positioned potentiometers attached to the wall panel and oriented parallel and perpendicular to the direction of the applied loading.

The preparation of wall panel 7 and wall panel 8 involved cutting the sides of the wall panels vertically with a concrete cutting chainsaw, and removing the excess masonry above. The location of wall panel 7 was selected at the door opening to minimise the required length of masonry cutting, with the above masonry temporarily supported using a timber frame as illustrated in Figure 6b. Three brick courses from the bottom corner of the wall panel were removed to allow the steel bottom loading plate to be positioned. Wall panel 8 contained an original thin metal strip (25 mm x 1 mm) located in the middle of the mortar joint at a height of 9 brick courses above the floor level.

5 EXPERIMENTAL RESULTS

5.1 Crack Patterns

Load was applied along one of the wall panel diagonals until the shear stress decreased to approximately half of the peak shear stress or until 1% lateral drift was reached. In all wall panels, cracking occurred predominately through the mortar joints in a diagonal step pattern as shown in Figure 7. Post-test removal of the plaster layer for wall panels 7 and 8 revealed that cracking also occurred mainly through mortar bed joints for these panels, with a small number of cracks propagated through the brick units (see Figure 8). Wall panels 5 and 6 exhibited sliding along the horizontal mortar joint at a height of 3 courses above the loading plate, prior to the formation of a stepped diagonal crack. Wall panel 8 initially exhibited diagonally oriented cracking at the top corner of the wall panel opposite to the loading plate, with the crack extending towards the wall panel diagonal with increasing load. Wall panel 7 exhibited a sudden drop of load and a rapid propagation of diagonal cracking that was clearly visible on both exterior plaster surfaces. Wall panel 4 was damaged during positioning into the testing location, with a horizontal crack developed prior to testing at a location 6 courses above the bottom of the wall panel that extended through approximately half of the wall panel width. For all wall panels, sliding along the mortar bed joints was observed following the formation of diagonally extending cracks. Hence, the response of all tested wall panels was categorised as a diagonal tension failure followed by shear-sliding along the cracked diagonal step joints.

5.2 Shear stress-drift response

The shear stress (v) versus percent drift (δ) and diagonal force (P) versus percent drift response are shown in Figure 9, with the curves plotted to a maximum of 1% drift. The shear stress was calculated using the experimentally measured diagonal force, P using Eq. (1):

$$v = \frac{Pcos\alpha}{0.5t(H + L)} \qquad (1)$$

where t_m is wall panel thickness, H is wall panel height, L is the wall panel length, and α is the angle between the wall panel diagonal and the horizontal axis (see Figure 5a). Eq. (1) is based on an isotropic linearly elastic model (provided in ASTM E 519-07 [18], for $\alpha = 45^o$) and was adopted for data interpretation (rather than the alternative method suggested by Brignola et al. [9]) due to its wide use and because it enabled direct comparison to experimental test data available in the literature. Percent drift, δ, of the wall panels was calculated using Eq. (2):

$$\delta = \frac{\Delta S + \Delta B}{2g}(tan\alpha + cot\alpha) \qquad (2)$$

where ΔS is diagonal shortening along the axis of applied force, ΔB is diagonal elongation measured perpendicular to the axis of applied force, and g is the gauge length.

From the stress-strain plots shown in Figure 9 it was observed that the behaviour of the wall panels was highly nonlinear, even for low load levels. For all wall panels (with the exception of wall panel 3 and wall panel 7) the experimental curve was approximately linear prior to crack initiation, followed by a nonlinear portion of the curve up to the maximum strength. Similar behaviour was observed and reported by Borri et al. [15], Corradi et al. [32] and Brignola et al. [10] during the in-situ testing of vintage stone URM wall panels. With the exception of wall panel 7, all wall panels typically exhibited ductile behaviour following the peak load, with the ductility arising not from conventional yielding of steel reinforcing bars, but from the inelastic response and energy dissipation due to sliding along mortar joints. The diagonal tension strength of the wall panels from the two selected buildings ranged between 0.061 MPa and 0.100 MPa, (excluding wall panel 4) and corresponds to the higher end of the typical range (between 0.010 and 0.100 MPa) that is reported in literature [9] for vintage unreinforced masonry.

5.3 Stiffness

The shear modulus, *G*, is a measure of material stiffness and is calculated as the ratio of the shear stress to the shear strain. The shear modulus for the tested wall panels was determined as the secant modulus calculated between $0.05v_{max}$ and $0.70v_{max}$ of the shear stress – shear strain response curve, with the values presented in Table 5. Due to the abnormal shear stress – shear strain response of wall panel 8 that was attributed to the presence of a metal strip in the mortar joint, the shear modulus of that wall panel was not determined. The experimentally obtained shear modulus values varied significantly, with values ranging from 0.27 GPa to 5.70 GPa. Large variations in experimentally obtained shear modulus for vintage brick masonry were also observed by other researchers [19, 21, 23, 33].

The stiffness of a test wall panel is quantified by the Modulus of Elasticity, *E*, which is related to the shear modulus. For each wall panel a corresponding Modulus of Elasticity was calculated using Eq. (3).

$$E = 2G(1 + \nu) \tag{3}$$

where ν = Poisson's ratio (adopting ν = 0.25, as suggested by Harris [34] and Pande et al. [35] for unreinforced masonry). The Moduli of Elasticity determined for all test wall panels are summarised in Table 5. It is noted that it was previously highlighted by Bosiljkov et al. [33] that for walls constructed using weak lime mortars the deformation behaviour is highly anisotropic and that stiffness should be evaluated by testing specimens having the same size as the actual structural element under consideration.

5.4 Prediction of diagonal tension strength

As previously outlined, reliable prediction of the diagonal tension strength of URM walls subjected to in-plane lateral loads is an essential parameter used as an input when conducting earthquake assessment of vintage URM buildings. As reported by Russell (2010) and seen in Figure 2, a direct linear relationship between diagonal tension strength and mortar compressive strength is not attainable due to large scatter of the experimentally obtained results. The lateral strength of masonry responding in-plane is limited by diagonal tension stress and can be determined using ASCE-41 [8], which allows substitution of the diagonal tension strength instead using bed joint shear strength. Eq. (4), provided in ASCE-41 [8], can be used to determine the expected masonry shear strength (v_{pre}):

$$v_{pre} = \frac{0.75\left(0.75v_{te} + \frac{P_{CE}}{A_n}\right)}{1.5} \tag{4}$$

where v_{te} is the average bed joint shear strength as determined from a standard in-situ bed joint shear tests [36], P_{CE} is the expected gravity compressive force applied to a wall panel, and A_n is the cross-sectional area of the wall panel.

The diagonal tensile stress prediction model provided in Eq. (4) is based on a Mohr – Coulomb circle type model which in turn is based on a pure shear stress state assumption (Calderini et al. [37]). Eq. (4) also incorporates the initial work done by Mann and Müller [38] based on the assumptions that the bricks are much stiffer than the mortar bed joints, the mechanical properties of header bricks are negligible, and that cracks develop along the mortar bed joints only. The predicted maximum shear strength values (v_{pre}) calculated using Eq. (4) are presented in Table 6, with the calculations based on the values of mortar bed joint shear strength that are presented in Table 4. P_{CE} is calculated as half of the wall panel self-weight due to formation of a diagonal crack oriented along the diagonal of the wall panels. Also, due to the lack of seismic shear transfer across the collar-joint in a multi-leaf masonry wall, the shear resistance of the collar-joint was reduced by 0.75 (the ratio of the areas of the top and bottom bed joints to the sum of the areas of the bed and collar-joints for a typical clay brick) from the experimentally obtained bed joint shear strength values to account for the resistance provided across not only the bed joint shear planes, but also the collar-joint shear plane. The 0.75 factor was omitted for wall panels 7 and 8 due to the absence of mortar filled collar-joints.

From Table 6 it is evident that for wall panels 1 to 7 the diagonal tension strength prediction using ASCE-41 [8] is in good agreement with experimentally obtained results. The under-predicted diagonal tension strength for wall panel 8 is attributed to elevated experimental strength due to the presence of a metal strip located within the wall panel mortar joint.

It is noted that extrapolation of the experimental data based on diagonal tension tests to actual masonry walls is not necessarily accurate as the ratio of shear to normal stress is fixed at a constant ratio of 1.0 and the wall panel shear strength is inferred from the measured diagonal compressive force based on a theoretical distribution of shear and normal stress for a homogeneous and elastic element. However, the distribution of shear and normal stresses across a bed joint may not be as uniform for an actual wall as for a test wall panel, and the redistribution of stresses after the first crack development may not be represented by the theoretical stress distributions [8].

5.5 Vintage masonry compared to laboratory-built wall panels

The measured shear strength of wall panels obtained from two tested URM buildings and the measured shear strength of laboratory-built wall panels [20, 23, 39] that resembled the masonry type present in the tested buildings was compared. The comparison was made by expressing the average of the shear strength for the wall panels obtained from the two URM buildings (v_{field}) and the average of the shear strength for the laboratory-built wall panels (v_{lab}) as a ratio (v_{lab}/v_{field}) (see Figure 10). Results that were used for the comparison were: (1) wall panel tested by Russell [20] having a mortar mix of 0:1:3 (cement:lime:sand) and of (2) 2:1:9; (3) wall panels tested by Russell

[20] having a mortar mix of 1:2:9; wall panels tested by Lin et al. [23] (4) and Ismail et al. [39] (5) having a mortar mix of 1:2:9. As noted by Russell [20], laboratory-built wall panels (AP1-AP9) were tested immediately following a 28 days curing period with the exception of wall panels AP2 and AP4 having a 1:2:9 mortar mix, which were cured for over 5 months prior to testing. Consequently, the elevated shear strength of wall panels AP2 and AP4 tested by Russell [20] in comparison to wall panels AP6 to AP9 was attributed to a longer curing period and hence were excluded from comparison made in Figure 10**Error! Reference source not found.**.

The ratio between the shear strength measured for the wall panels obtained from the two URM buildings and the shear strength of wall panels having 1:2:9 mortar mix ranged between 0.97 and 1.54. The ratio between the shear strength of wall panels having 0:1:3 mortar mix and 2:1:9 mortar mix and the average measured shear strength for the wall panels obtained from the two buildings was 0.69 and 8.24 respectively. It is evident that the average shear strength of the wall panels obtained from the two buildings fell between the average shear strength of wall panels constructed using a 1:2:9 mortar mix (Figure 10). It was concluded that a benchmark mortar mix having a ratio of 1:2:9 is suitable for use when manufacturing replica clay brick test assemblages that represent within reasonable accuracy the shear strength characteristics in existing vintage URM buildings, particularly when laboratory-built replicas are tested approximately 28 days after construction is complete.

6 CONCLUSIONS

1. The diagonal tensile strength of vintage New Zealand unreinforced clay brick masonry was assessed via experimental testing of as-built (unretrofitted) wall panels from two vintage URM buildings. Despite the relatively small number of specimens, the test data supplemented the scarce database of in-situ results obtained from existing vintage clay brick URM buildings and was generated in order to assess the applicability of the predictive model available in ASCE - 41 [8]. Also, in-situ experimental results facilitated the establishing of a

benchmark for the diagonal tension characteristics of vintage clay brick masonry to be used in a laboratory setting in order to accurately simulate in-situ wall panels and to ensure adequate representation of the strength characteristics in existing buildings;

2. The experimental curves of shear stress plotted against drift are approximately linear prior to the initiation of masonry wall cracking, then follow a nonlinear curve up to maximum strength. This type of behaviour is similar to data reported in the international literature for vintage stone masonry panels tested in-situ;
3. It was established that the maximum average diagonal tension strength for Building 1 (i.e. wall panels 1 to 6) and Building 2 (i.e. wall panel 7 and wall panel 8) was 0.074 MPa and 0.096 MPa respectively;
4. The predictive equations provided in ASCE-41 [8] for estimating the diagonal tension strength of masonry based on test values obtained from standardised ASTM bed joint shear tests are within reasonable accuracy when compared to the experimental results obtained from diagonal compression tests;
5. The use of recycled clay bricks acquired from vintage URM buildings and a mortar mix having a 1:2:9 (cement:lime:sand) ratio provides a suitable approximation of the shear strength characteristics found in existing vintage clay brick URM buildings located in New Zealand when used to manufacture replica clay brick test assemblages, particularly if laboratory-built replicas are tested approximately 28 day after construction.

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(a) Sliding of a low aspect ratio pier

(b) Failure of piers in diagonal tension

Figure 1: Examples of sliding and diagonal tension failure following the February 2011 M6.3 Christchurch earthquake

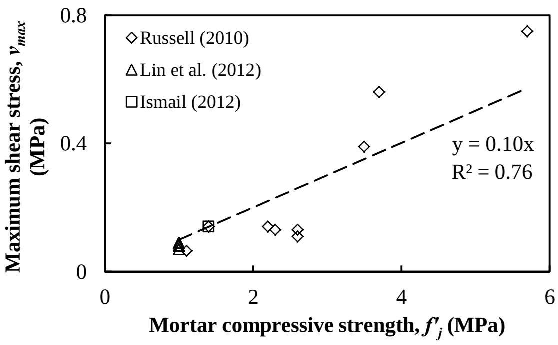


Figure 2: Maximum shear stress vs. mortar compressive strength plot

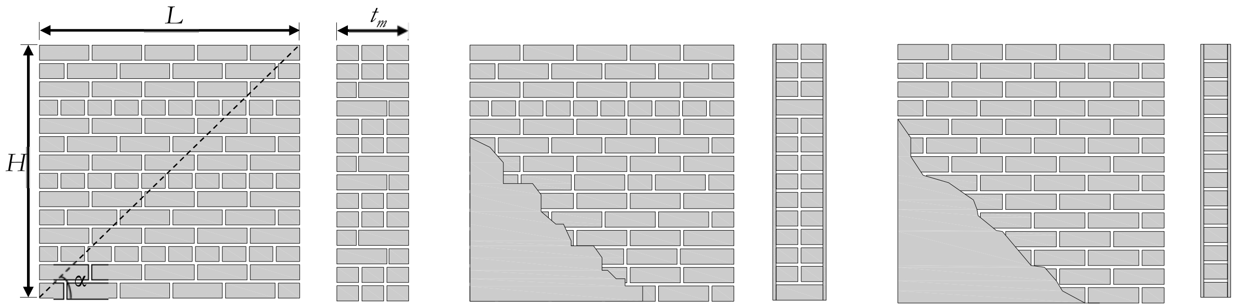
(a) B-East building, part of the Allen's Trade Complex (Building 1)

(b) Avon House building (Building 2)

Figure 3: Building elevations

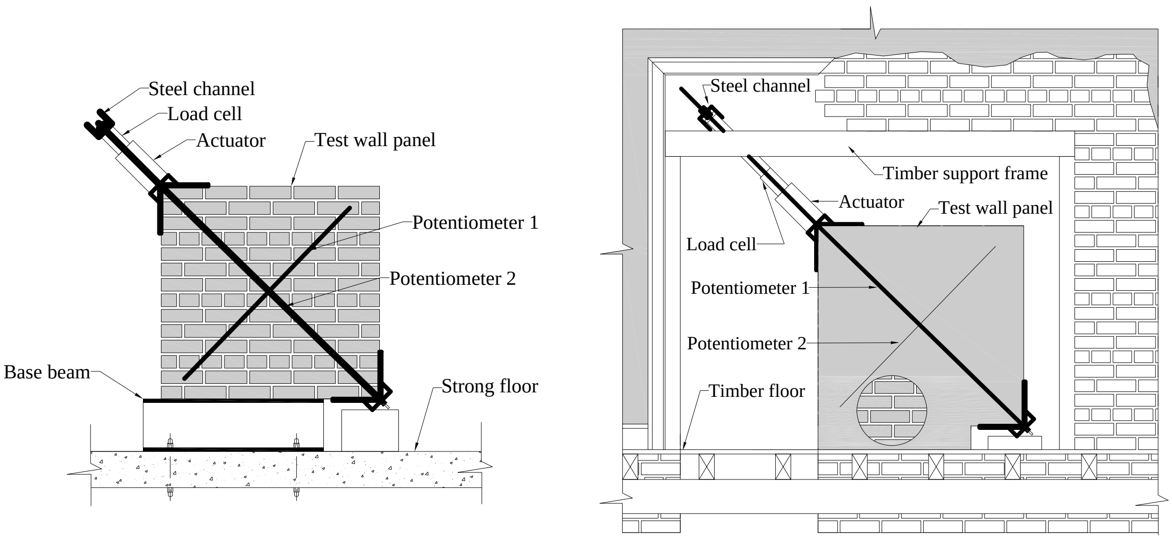
(a) Cutting through masonry
(b) Vertical straps
(c) Lifting wall panels using a medium sized crane
(d) Wrapping wall panels with protective plastic

Figure 4: Wall panel preparation and extraction from Building 1



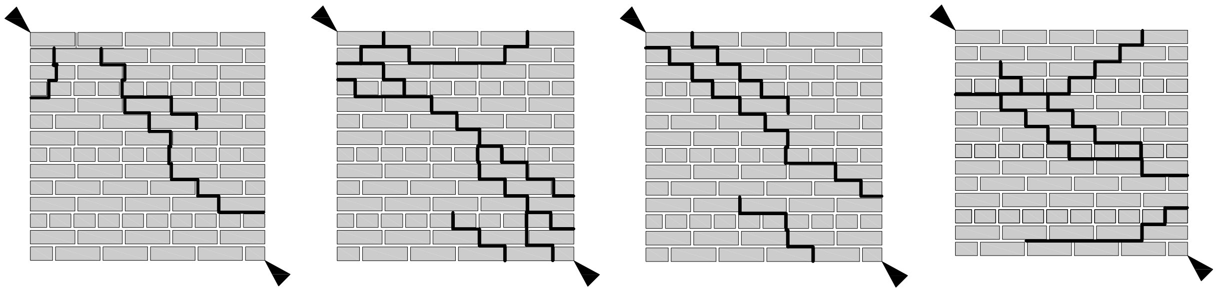
(a) Walls 1 to 6
(b) Wall 7
(c) Wall 8 (horizontal metal strip located 9 courses above base)

Figure 5: Wall panel parameters and masonry bond pattern

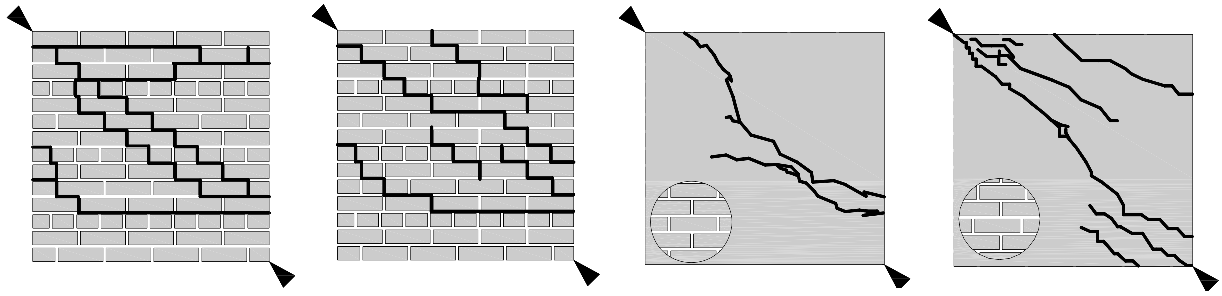


(a) Test setup for wall panels 1-6
(b) Test setup for wall panel 7

Figure 6. Typical in-plane diagonal tension test setup.



(a) Wall panel 1 (b) Wall panel 2 (c) Wall panel 3 (d) Wall panel 4



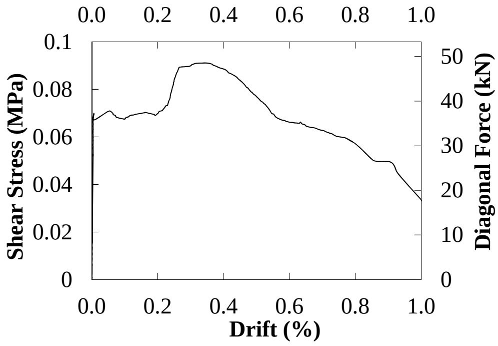
(e) Wall panel 5 (f) Wall panel 6 (g) Wall panel 7 (h) Wall panel 8

Figure 7: Observed crack patterns following testing

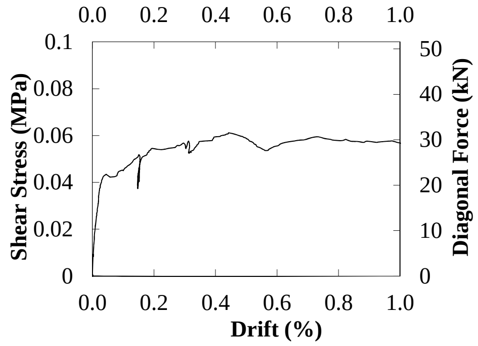
(a) Wall panel 7

(b) Wall panel 8
(weak plaster layer removed)

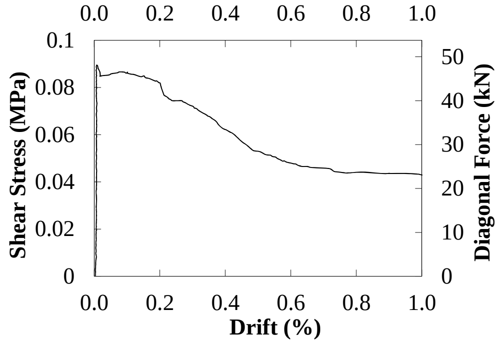
Figure 8: Photographs of tested wall panels



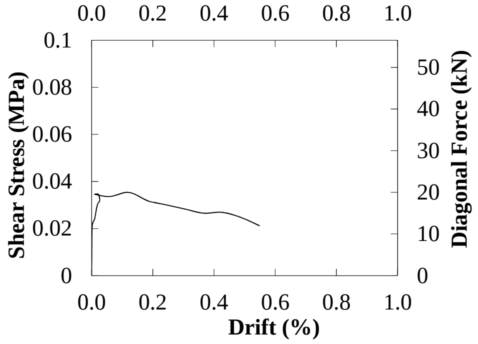
(a) Wall panel 1



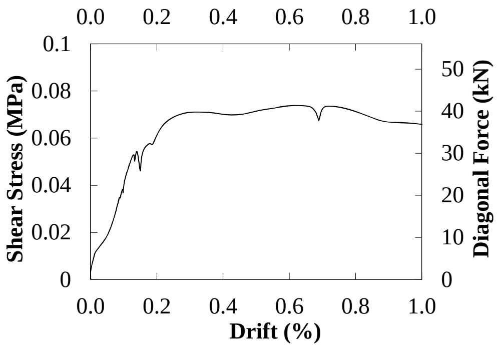
(b) Wall panel 2



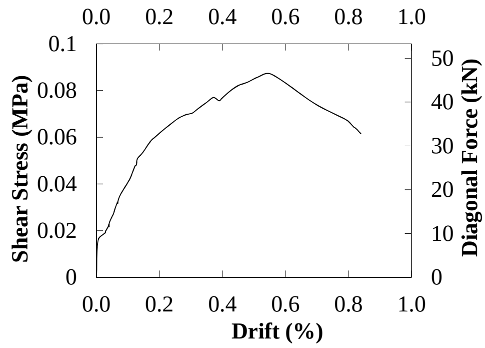
(c) Wall panel 3



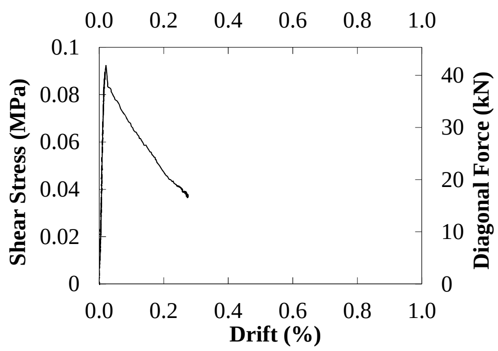
(d) Wall panel 4



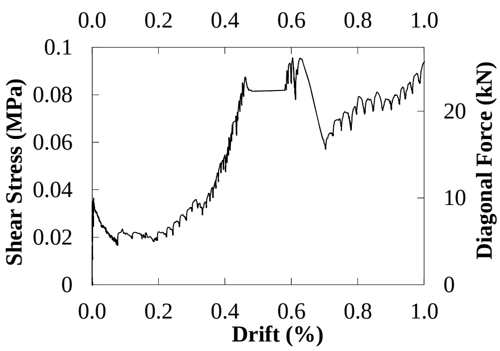
(e) Wall panel 5



(f) Wall panel 6



(g) Wall panel 7



(h) Wall panel 8

Figure 9: Shear stress – drift response

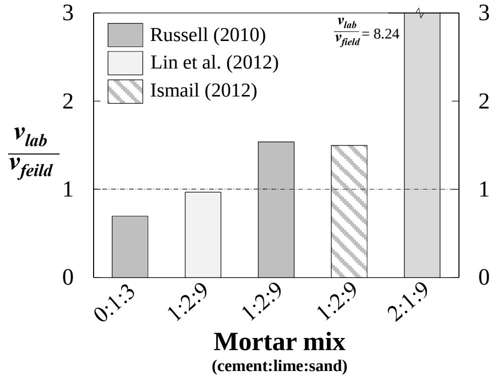


Figure 10: Ratio between shear strength of wall panels obtained from two existing buildings and shear strength obtained from testing laboratory-built wall panels having differing mortar mixes

Table 1: Previously conducted diagonal tension tests of wall panels

Reference	Wall panel properties						Type of test	Test results	
	Wall panel	t_m (mm)	Mortar mix	f_j (MPa)	f'_b (MPa)	f'_m (MPa)		v_{max} (MPa)	G (GPa)
[20] (SBM)	AP1	225	2:1:9	5.7	23.3	16.6	Lab	0.71	0.93[a]
	AP2		1:2:9	3.7	24.6	14.6		0.42	0.93[a]
	AP3		0:1:3	1.1	24.3	3.2		0.06	0.80[a]
	AP4		1:2:9	3.5	26.3	13.7		0.41	0.95[a]
	AP6		1:2:9	2.6	23.6	5.6		0.14	0.92[a]
	AP7		1:2:9	2.3	20.4	7.4		0.13	0.84[a]
	AP8		1:2:9	2.2	20.1	7.3		0.14	0.78[a]
	AP9		1:2:9	2.6	23.4	8.5		0.12	1.02[a]
[23] (SBM)	S0-2L-1	220	1:2:9	1.0	26.5	6.0	Lab	0.08	3.5
	S0-2L-2							0.09	4.5
	S0-3L-1	350						0.09	5.9
	S0-3L-2							0.09	0.4
	S0-4L-1	470						0.07	5.5
	S0-4L-2							0.08	0.5
[39] (SBM)	W2C-3	220	1:2:9	1.4	39.4	10.7	Lab	0.14	1.0
	ABI-01			1.4	16.4	5.9		0.15	0.9
	ABI-02			0.7	18.8	5.6		0.098	0.6
[21] (SBM)	W1C-1	110	1:1:6	-	-	32.1	Lab	0.84	2.8
	W1C-2	110	1:1:6			32.1		0.69	3.5
[40]	CSM (2)	480-670	-	-	-	-	In-situ	0.052[b]	0.059[b]
[11, 32]	CSM (6)	400-700	-	-	-	-	In-situ	0.06[b]	0.033[a,b]
	SBM	300						0.05	0.13[a]
[10]	RSM (4)	500-670	-	-	-	-	In-situ	0.077[b]	-
	RSM-H (5)	430-470						0.10[b]	
	CSM (5)	420-620						0.12[b]	
	SBM (3)	270-290						0.25[b]	
	HBM (2)	250						0.65[b]	

Note: ( ) – Number of wall panels; [a] – shear modulus determined using ASTM E 519 method; [b] – average of tested group; t_m – wall thickness; Mortar mix - cement:lime:sand ratio by volume; f_j – mortar compression strength; f'_b – brick compression strength; f'_m – masonry compression strength; v_{max} – maximum shear stress; G - shear modulus; RSM – rubble stone masonry; RSM-H – rubble stone masonry with horizontal courses; CSM – multiple-leaf roughly cut stone masonry; SBM – solid brick masonry; HBM – modern hollow brick masonry.

Table 2: Wall panel parameters

Building	Wall panel	Wall panel dimensions (mm)				Plaster thickness (mm)	Bond pattern	Test type
		H	L	t_m	α^o			
1	1	1150	1060	330 (3)	47.3	-	Common (aka American)	Lab
	2	1100	1030	330 (3)	46.9			
	3	1150	1150	330 (3)	45.0			
	4	1210	1160	330 (3)	46.2			
	5	1200	1210	330 (3)	44.8			
	6	1140	1220	330 (3)	43.1			
2	7	1170	1200	230* (2)	44.3	20	Irregular Running	In-situ
	8	1240	1210	110* (1)	45.7			

Note: 1 = Allen's Trade Complex; 2 = Avon House; H = wall panel height; L = wall panel length; t_m = wall panel thickness; α = angle in degrees between wall diagonal and horizontal; * - plaster thickness excluded; (#) = number of leafs (with outer and inner leafs being nominally identical).

Table 3: Building material properties

Building		f'_b (MPa)	f'_{bt} (MPa)	f'_j (MPa)	f'_m (MPa)	f_{fb} (MPa)	f'_p (MPa)	$E_m \times 10^3$ (MPa)
1	Value	19.40	1.87	5.82	9.60	n/a	-	2.45
	Sample size	9	9	9	6	n/a	-	6
	CoV	0.16	0.23	0.14	0.28	n/a	-	0.32
2	Value	8.80	1.19	1.32	3.20	0.04	1.40	0.44
	Sample size	7	5	7	5	9	9	5
	CoV	0.19	0.29	0.14	0.20	0.50	0.37	0.20

Note: (1) = Allen's Trade Complex; (2) = Avon House; n/a = data not available; f'_b = brick compressive strength; f'_{bt} = brick modulus of rupture; f'_j = mortar compressive strength; f'_m = masonry compressive strength; f_{fb} = masonry flexural bond strength; f'_p = plaster layer compressive strength; E_m = masonry Modulus of Elasticity.

Table 4: Bed joint shear strength

Building		v_{te} (MPa)
1	Value	0.19
	Sample size	6
	CoV	0.16
2	Value	0.16
	Sample size	8
	CoV	0.29

Note: (1) = Allen's Trade Complex; (2) = Avon House; v_{te} = average bed joint shear strength; CoV = coefficient of variation (CoV).

Table 5: Experimental results

Building	Wall panel	P_{max} (kN)	F_{max} (kN)	v_{max} (MPa)	$G \times 10^3$ (MPa)	$E \times 10^3$ (MPa)
1	1	49.0	33.2	0.091	0.37	0.93
	2	31.5	21.5	0.061	0.42	1.05
	3	48.0	34.0	0.090	2.17	2.53
	4	20.0	13.8	0.035	0.51	1.28
	5	45.8	32.5	0.082	0.31	0.78
	6	46.3	33.8	0.087	0.27	0.68
2	7	41.9	30.0	0.092	5.70	2.23
	8	28.5	19.9	0.100	-	-

Note: (1) = Allen's Trade Complex; (2) = Avon House; P_{max} = maximum applied diagonal force; F_{max} = maximum horizontal force; v_{max} = maximum shear stress; G = shear modulus; E = Modulus of Elasticity.

Table 6: Prediction of masonry diagonal tension strength

Wall panel	v_{exp} (MPa)	ASCE-41	
		v_{pre} (MPa)	$\frac{v_{exp}}{v_{pre}}$
1 - 6	0.074*	0.077	0.94
7	0.092	0.085	0.93
8	0.100	0.085	0.85

* - average of building 1 (wall panel 4 excluded); v_{exp} = maximum experimental diagonal tension strength of masonry; v_{pre} = predicted diagonal tension strength of masonry.