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Experimental Study on the Mechanical Properties of Straw-fibre Reinforced Adobe Masonry

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Abstract

Adobe has been used as an economic building material since ancient times and continues to be employed today, particularly in developing countries. Due to its sustainability credentials, interest in adobe as an alternative to traditional fired masonry has gathered pace. In order to develop guidance on the design of new and strengthening of existing adobe structures, a detailed understanding of the material's properties is required. Currently available studies on adobe material often concentrate on one particular property. Due to the differences in adobe make up, correlation of the existing results can be problematic. To address this, a detailed experimental study using one consistent, reproducible and realistic adobe mix is presented here. The material behaviour in compression (including strength gain with time), tension and shear is studied. For the first time, the interface between the adobe units and mud mortar is investigated in direct tension and in shear, including the effects of co-existing compression. Crucially, parameters such as tensile and shear fracture energy, often excluded from existing experimental work, are evaluated. The data provided herein will enable development of accurate numerical and analytical models to describe adobe masonry structural behaviour.

Keywords: Adobe masonry, material characterisation, bond strength, fracture energy

Introduction

Earthen materials such as adobe have been widely used in construction for thousands of years. Most commonly such materials are used to build houses and low-rise structures however more substantial mid-rise structures have been formed from adobe with notable examples including Djenné mosque in Mali (Minke, 2012) and historic multi-storey buildings in Shibam city, Yemen (Al-Masawa et al., 2018). Adobe is still prevalent in developing countries, it is estimated that 30% of the world's population live in earthen construction. In some developed countries, earthen construction was fairly common up until the 18th century industrial revolution (Galán-Marín et al., 2010). Nowadays, earthen construction is an area of growing interest, both for maintaining heritage structures and as a rediscovered eco-sustainability material (Quagliarini et al., 2015). Adobe can be a sustainable solution for construction and as such has been the subject of increased research interest in recent years. In addition to its inherent sustainability, adobe has other attractive properties such as its low cost, the ease of access and availability of the material and its excellent thermal and acoustic insulation characteristics. Despite the widespread use of adobe, there is a scarcity of detailed technical guidelines and national or international codes of practice to evaluate the mechanical properties of adobe materials and their application in structures. The limited codes and design guides which do exist such as the New Zealand code for earthen structures (New Zealand Standard, 1998), The Australian Earth Building Handbook (Walker, 2002) and the New Mexico Earthen Building Materials Code (Housing and Construction Rules and Laws, 2015) offer mainly qualitative guidance on adobe construction in general and some guidelines for simple compression testing.

A review of the existing experimental work (detailed in the following section) on adobe reveals a fairly wide scatter in results, this is partly due to the variation in materials and methodologies adopted and the lack of standardisation in the material constituents. Often more difficult to obtain parameters such as the unit to mortar tensile and shear fracture energies are not investigated, both of these are important in the development of accurate numerical models. Due to the aforementioned, relating the test results obtained by different researchers is problematic. Moreover, in the case of missing data, assumed values have to be adopted. All of these factors make it difficult to create a set of mechanical properties for adobe masonry which can be relied on for evaluating and assessing a given structure via numerical or theoretical analyses. The experimental work in this paper seeks to address this.

To understand and assess the structural performance of adobe, a systematic assessment of its constituent materials is required. Aside from presenting the suite of mechanical properties for one consistent and repeatable adobe mix, this study is also novel since it provides for the first time tensile behaviour of the adobe unit and mortar interface for mud mortars (mud mortars being of the same constituent as the adobe unit and are commonly used in the field). Detail on the shear and tensile fracture energies of the bond interface are needed for accurate numerical modelling (Abdulla et al., 2017) and to date no direct tensile strength data for the adobe unit to mortar interface is available for cases, where mud mortar is solely used to assemble adobe units. It is envisaged the results herein can be used by researchers and practitioners for numerical and theoretical analyses to provide structural solutions towards strengthening and improved resilience of adobe masonry structures (Abdulla et al., 2018).

This paper is structured as follows. First, a review of existing experimental studies on mechanical properties of adobe is presented. The experimental programme, details of specimen preparation and test methodology are then introduced. Initially the specification of the adobe mix is described including associated geotechnical characterisation for ease of repeatability; this mix is then used throughout the test series. The compression tests are then detailed comprising both cube and cylinder tests along with tests on adobe masonry assemblages. In the case of the cube tests, 3 different ages of samples are tested to ascertain the compression strength gain. The next set of tests centres on tensile strength and involve both direct and indirect methods. Additionally the strength of the adobe unit to mortar interface is tested in direct tension. Lastly, the shear properties of the adobe unit to mortar interface are studied, both with and without pre-compression. Results from the test series are then discussed and conclusions are given.

Review of Existing Experimental Studies

Existing studies on compressive, tensile and shear strength of adobe have been conducted by various researchers around the world.

The value of compressive strength of adobe samples reported in existing literature has a wide range, as low as 0.55 MPa (Yetgin et al., 2008) and as high as 5.5 MPa (Bouhicha et al., 2005). This reflects the differences in soil types, proportion of constituent materials in the adobe mixture and other factors such as production method, specimen shape, etc. In a study by Silveira et al. (2012), cylindrical adobe samples were collected from existing adobe

structures in Portugal, the mean compressive strength of the samples was 1.17 MPa. Quagliarini and Lenci (2010) conducted uniaxial compressive strength tests on 80 adobe units made from ten different soil mixtures. The average lower result was 2.14 MPa for the soil mixture containing less clay content. The average higher result was 2.88 MPa for the soil mixture containing more clay content. The coefficients of variation from samples made from the same mix ranged from 8.5% to 26%. In general the compression strength of adobe tends to increase with the clay content of the mix as summarised in Table 1. Similar findings on compressive strength of adobe samples are reported in (Achenza and Fenu, 2006; Adorni et al., 2013; Binici et al., 2005; Bouhicha et al., 2005; Figueiredo et al., 2013; Fratini et al., 2011; Galán-Marín et al., 2010; Illampas et al., 2014; Parisi et al., 2015; Wu et al., 2013; Yetgin et al., 2008).

The aforementioned studies focussed on adobe material specimens i.e. cubes, cylinders and adobe units. Some research has examined adobe masonry panels under compression. Wu et al. (2013) tested three masonry panels (dimensions 530 mm x 290 mm x 200 mm) using different earthen mortars, where the mortar to adobe unit compressive strength ratios were 0.84, 1.0 and 1.03. The compressive strength of the panels ranged from 0.88 to 0.98 MPa, the panel strength was proportional to the clay content of the mix. Miccoli et al. (2014) tested 500 mm x 500 mm x 110 mm panels made from adobe units and mud mortar (the constitutive materials of the mud mortar were similar to those of the adobe units, however the mud mortar clay and silt contents were lower) with mean compressive strengths of 5.21 and 3.32 MPa respectively. The results showed that the compressive strength value of the adobe masonry panels ranged from 2.7 to 3.8 MPa with a mean value of 3.28 MPa. In another study, Quagliarini et al. (2010) examined two 690 mm x 390 mm x 310 mm panels. The compressive strength values of the panels were reported as 0.77 and 1.2 MPa. Similar findings on compressive strength of adobe masonry are reported by others e.g. (Illampas et al., 2011; Martins and Varum, 2006; Venkatarama Reddy and Gupta, 2006; Vicente and Torrealva, 2014; Walker, 2004).

A limited amount of investigations have been conducted on tensile and shear properties. Adobe masonry is a jointed composite; it's tensile strength is dependant on the tensile strength of its constituent materials i.e. adobe units and mortar, as well as the tensile bond strength of adobe unit to mortar interfaces. The tensile strength of adobe units is most commonly obtained from cylinder splitting and three point bending tests or less commonly uniaxial direct tensile tests.

Adorni et al. (2013) conducted tensile splitting tests on seven cylinder samples obtained from an archaeological site in Turkmenistan, the results were between 0.12 and 0.32 MPa. In a study undertaken by Martins and Varum (2006), two sets of adobe specimens each comprising three cylinders were extracted from existing adobe structures in Portugal, the average result of the tensile splitting tests was 0.13 MPa. Similarly, Silveira et al. (2012) undertook tensile splitting tests on samples extracted from existing structures in Portugal, the mean tensile strength was reported as 0.19 MPa.

Some researchers have investigated the tensile strength of adobe samples based on three point bending tests. Caporale et al. (2014) obtained an average flexural strength of 0.56 MPa from 36 adobe samples. Liberatore et al. (2006) tested adobe units which were collected from buildings in Italy. The flexural strength results were reported as between 0.17 MPa and 0.4 MPa, the relative high scatter of the results were attributed to the influence of straw fibres in the soil mixture. In addition, three point bending test results of adobe blocks, collected from three different sites in Morocco, ranged between 0.18 and 0.35 MPa (Baglioni et al., 2010). Unlike the previous indirect tensile tests, only a few studies examined prismatic adobe samples in a uniaxial direct tensile test. Yetgin et al. (2008) examined 60 prismatic specimens made of fibrous i.e. straw fibre reinforced and non-fibrous soil mixtures, under direct tension. The results showed that the tensile strength of non-fibrous specimens ranged from 0.38 to 0.75 MPa, while a range from 0.15 to 0.27 MPa was seen in the fibrous specimens. In another study, Almeida (2012) tested 6 adobe specimens under uniaxial tensile load, the results revealed an average tensile strength of 0.081 MPa. Furthermore, Mesbah et al. (2004) tested compressed earth blocks under direct tension (block dimensions 100mm x 140mm x 295mm), the specimens were reinforced with sisal fibres. The tensile strength of the specimens ranged from 0.21 MPa to 0.23 MPa. Araki et al. (2016) also conducted direct tensile tests on compacted rammed earth specimens prepared from three different soil mixtures. The tensile strength of the specimens was between 0.075 MPa and 0.52 MPa depending on the soil mixture and age of specimens. The aforementioned tensile strength test results are summarised in Table 2.

The bond strength of unit to mortar interfaces at the bed and head joints usually governs the structural response and failure mechanism of adobe masonry structures as it is relatively weaker than the tensile strength of the adobe units. The common failure mechanisms in the joints are tensile cracking and/or sliding along bed and head joints. Therefore, the tensile strength and shear strength of the unit to mortar interfaces and their corresponding fracture

energies are important parameters to be included in the analysis of adobe structures. Very few studies exist on tensile strength of adobe unit-mortar interfaces. The tensile strength between units has indirectly been obtained through the wrench bond method for flexural bond strength. Varum et al. (2015) used this method to study the flexural bond strength of the interface between adobe units and mortar joints made with a mixture of soil and hydraulic lime. The results showed that the average bond strength was 0.061 MPa with a coefficient of variation of 30%. In a study by Almeida (2012), the tensile strength was directly obtained by testing adobe assemblages under uniaxial tensile load. The study comprised two adobe units (180 mm x 130 mm x 50 mm) assembled with a hydraulic lime mortar with an average thickness of 10 mm. The average tensile bond strength obtained from 5 samples was 0.01 MPa with a relatively high coefficient of variation of 88%. No further experimental data on the tensile bond strength was found in the literature to date. In addition, no studies on the tensile strength of adobe unit to mortar interfaces using mud mortar (the common mortar type in adobe construction), were reported.

The shear strength of unit to mortar interfaces of small adobe samples and shear strength of larger adobe panels have been obtained using triplet shear tests and diagonal compression tests, respectively. Miccoli et al. (2014) undertook triplet shear tests on 12 adobe specimens, (240 mm x 115 mm x 72 mm unit size) assembled using a mud mortar with the average thickness of 20 mm. The values of initial shear strength and shear strength at various levels of normal compressive stress were obtained. The initial shear strength obtained was around 0.02 MPa. Varum et al. (2015) tested 60 adobe specimens using the triplet shear test method, the specimens were built by using a hydraulic mortar. The results revealed an average initial shear strength of 0.05 MPa. Wu et al. (2011) tested three groups of specimens under shear stress using the triplet method. Three types of mud mortar were used, one for each group, incorporating different ratios of clay and sand as follows; 1.0:0.8, 1.0:1.0 and 1.0:1.2. The average initial shear strength for each group was 0.11, 0.10 and 0.086 MPa, respectively. The highest shear strength corresponded to specimens with a mud mortar made from the higher clay content.

A limited number of studies on shear strength of adobe have been made using the diagonal compression test method. Miccoli et al. (2014) undertook diagonal compression tests on 500 mm x 500 mm x 110 mm panels. The results showed that the shear strength ranged from 0.09 to 0.34 MPa. The upper value was achieved when the adobe units were wetted prior to laying them. Liberatore et al. (2006) also conducted *in-situ* diagonal compression tests on a

880 mm x 910 mm x 350 mm adobe panel chosen from a historical building, the resulting shear strength was 0.0206 MPa.

Post failure behaviour of unit to mortar interfaces i.e. tensile and shear fracture energies in normal and horizontal directions have only been reported by one author to date. Almeida (2012) determined the tension, mode I, fracture energy of adobe assemblages where a hydraulic mortar was used between two adobe units. The average tension fracture energy obtained from testing 5 samples was reported as 0.0045 N/mm. Apart from this study, no experimental data on tensile fracture energy is readily available in the literature. Similarly no data on tensile fracture energy of the unit to mortar interfaces using a mud mortar is readily available.

From the review of the existing experimental work, it can be noted that the results for discrete adobe samples and panels are scattered; different contributing factors are responsible for this. These factors are mainly the type of the soil used and the proportions of constituent materials (soil mixture). Further inconsistencies arise with the lack of technical and standard guidelines in adobe production, the amount of straw fibre in the soil matrix, the type of mortar used in the case of interface properties, the shape and size of samples, their drying process and testing procedure.

Given these inconsistencies, direct comparisons between the available results are not always possible. In addition, very limited studies have been carried out to identify the mechanical properties of adobe unit to mortar interfaces under tension and shear, i.e. tensile and shear strength of adobe unit to mortar interfaces and their corresponding fracture energies. Notably, no studies to date have examined tensile bond strength between adobe units and mud mortar (the most commonly adopted form of mortar in adobe masonry). The proceeding experimental programme aims to address these issues.

Experimental programme

Materials

The specimens in this study were made of a soil containing separate additions of clay, silt, sand and fine gravel in order to represent a traditional adobe mixture seen in the field (Jaquin and Augarde, 2012). The clay and silt materials were sourced from Spencroft Ceramics, Staffordshire, UK. The sand was angular and contained a small portion of fine gravel

(particles > 2 mm in diameter). The soil for the adobe mixture was prepared by mixing each of the constitute materials based on weight fractions. The weight fraction of the clay and silt was 50%, which was the same as the weight fraction of the sand and fine gravel. The final weight fractions of the soil are summarised in Table 3. The particle size distribution (Fig. 1) of the soil mix was obtained from the sieve analysis of the particles retained in a 75µm size sieve and from the hydrometer analysis of the particles passing through the 75µm size sieve, in accordance with Bs En 1377-2 (1990). The distribution of particles in the mixture was within the range of the soil types recommended by Jaquin and Augarde (2012) to produce adobe, so the adopted soil can be relied on as a traditional soil mix for the production of the adobe samples in this study. Atterberg limit tests were also conducted in accordance with Bs En 1377-2 (1990) to aid repeatability and to find a suitable water content to avoid reaching neither plastic state nor liquid state of the soil. The chosen water to dry soil ratio was set at 0.28, this is within the range recommended for adobe by Kouakou and Morel (2009) to achieve a workable mixture. Barley straw fibres were added to the soil/water mixture to reduce shrinkage cracking and provide some post-crack ductility to the adobe units. In order to reduce the risk of high deformation and creep, the weight fraction of the straw fibres was 0.5% of the soil matrix, as recommended by Yetgin et al. (2008) and Galán-Marín et al. (2010). Yetgin et al. (2008) states that a higher straw fibre content increases the risk of high deformation and creep of adobe due to the fibre's relatively low elastic modulus i.e. low stiffness and high elastic deformation of fibres as well as the relative weak connection between the fibres and the matrix. The length and diameter of straw fibres ranged from 25 to 80 mm and 0.5 to 1.2 mm, respectively. Fibres were added and mixed thoroughly to achieve an even dispersal within the mix. Table 3 gives more details of the mixture used to produce the adobe specimens.

Specimen preparation and test setup

Compressive strength test

The compressive strength of the adobe mixture was identified via cube samples; the strength gain of the cube samples was also studied. In addition, cylindrical samples were tested to understand the effects of geometry on samples made from the same adobe mixture. Finally, adobe assemblages were tested under axial compression to ascertain the compressive strength of adobe masonry.

Cubic specimens were cast in 100 mm x 100 mm x 100 mm moulds. Cylindrical specimens were prepared in moulds with a diameter of 100 mm and 200 mm height. The soil mixture was placed in the moulds in three layers; each layer was hand compacted to ensure the solidity of specimens with minimum voids. The samples were demoulded after 48 hours and then air-dried at room temperature $\sim 20^{\circ}\text{C}$, the average relative humidity in the laboratory during the curing and testing period was 39.5% with a coefficient of variation of 11.1%.

Due to the shrinkage effects the final average edge dimensions of the cubes were between 94 and 97mm, Table 3 gives details on the change in density and moisture content over 28 days for 30 representative adobe units from the test series. To achieve perfectly level faces at the top and bottom, cube specimens were capped with a thin layer (< 3 mm) of gypsum dental plaster. The tensile strength of dry gypsum dental plaster is between 3 and 4 MPa (Earnshaw and Smith, 1966); however the thickness of the applied dental plaster was maintained as thin as possible to avoid reinforcing the specimens. In total, 30 cubes were prepared. 12 cubes were tested at the age of 28 days, 12 at 14 days, and 12 at 56 days.

Similarly to the cubic specimens the average final diameter of the cylindrical specimens was 94 mm, with final heights from 186 to 192 mm. For the aforementioned reason, the top and bottom faces of the cylinders were also capped with a thin layer of dental plaster. A set of 6 cylindrical specimens was prepared and tested at the age of 28 days.

Three adobe masonry panels were assembled from adobe units (Dimensions: 204 mm x 140 mm x 46 mm) with a minimum age of 28 days. The adobe units were placed using a mud mortar with a thickness ranging from 10 to 15 mm. The mixture of all mud mortars in the experimental programme contained exactly the same contents, i.e. soil materials and straw fibres, as the mixture used to produce the adobe units (see Table 3). The surfaces of all units were moistened via a sponge prior to being laid in order to retard the drying process in the joints and prevent excessive shrinkage cracks. Consequently, a better bond between units and mortar could be achieved. The final average dimensions of the panels were 425 mm x 140 mm x 500 mm. Each panel consisted of 9 courses with 2 units laid in each course following a typical stretcher bond pattern. The geometry of the panels was adopted following Bs En 1052-1 (1999) for testing standard masonry panels. The panels were tested at 28 days after the build date to allow the mortar to gain strength. The top of the panels was also capped with a thin dental plaster layer so that they were level.

All compression tests were carried out under displacement control at a rate of 1mm/min. The tests were conducted using an Instron 4507 universal testing machine with a 200kN load cell. Compressive load was measured through a computer controlled 200kN load cell; with stresses calculated by dividing the load by the cross-sectional area. The axial compressive strains were obtained based on the relative displacement between the top and bottom platens of the testing machine. Note that attachment of strain gauges directly to the adobe specimens was not possible due to the friable nature of the specimens' surface.

When using measurements based on the relative displacement of the test machine, an initial load adjustment phase occurs before the platens of the machine are in direct contact with the specimen. In the stress vs. deformation plots presented in this study i.e. Figures 5, 7, 10, 11 and 12, this phase has been removed according to the following process. Firstly the load value at the end of the adjustment period is retained then the origin of the plot is extrapolated following the slope of the linear portion of the subsequent stress-deformation plot.

The adobe masonry panels were also tested under displacement control at a rate of 1mm/min as specified in Bs En 1052-1 (1999). The load was evenly distributed to the panels via a C-section steel beam, placed upside down on the top of the panels. The compressive strength was calculated by dividing the load by the loaded cross-section of the panels. Displacement measurements were obtained by means of four linear variable differential transformers (LVDT) attached to the panels (two on each side) as recommended in Bs En 1052-1 (1999), Fig. 2. The displacement allowance in each LVDT was 100 mm. The LVDTs and their angle-shaped steel support were positioned and fixed on the adobe panels by screws. The initial distances between the base of the LVDTs and their support were recorded. The strains were determined as a ratio of the displacement measurements from the LVDTs to their initial distances.

Tensile strength tests

The tensile strength of adobe dog bone specimens was obtained using the direct tensile test method. Indirect tensile strength testing of cylindrical specimens was conducted via tensile splitting tests while the flexural strength of adobe units was obtained via a three-point bending test.

The dog bone samples were prepared by placing the adobe mixture directly into steel moulds. The geometry and dimensions of the mould is shown in Fig. 3, the depth of the mould was 25

mm. The samples were demoulded after 24 hours and then air-dried at room temperature ~20°C for 27 days, before testing. The final average dimensions of the dog bone specimens were 72 mm long, and between 23 and 24 mm high and wide at their centre. In total, 16 specimens were tested under direct uniaxial tension. The specimens were placed between two C-shaped steel clips for the direct tensile tests. It was ensured that the specimens were in full contact with the steel clips before commencing. The tests were performed in displacement control at a rate of 0.5mm/min using a universal testing machine (Model: Zwick Roell Z010) with a load cell capacity of 10 kN. Stresses were determined by dividing the measured axial tensile load from the testing machine by the central cross-section area of the dog bone specimens. The displacements were directly obtained from the relative displacement readings from the platens of the testing machine.

The tensile splitting test specimens were prepared in the same way as for compression tests, however no dental plaster was applied. On average, the diameter of the cylinders was 94 mm, their height ranged from 186 to 192 mm. In total, 6 cylinders were tested at 28 days. The specimens were tested in accordance with Bs En 12390-6 (2009). The tests were carried out on an Instron 4507 universal testing machine with a load cell capacity of 10 kN. The load was applied at a rate of 1 mm/min under displacement control until complete failure of the cylinders. The tensile splitting stress was calculated following equation 1 (Bs En 12390-6, 2009). The vertical deformations and the stress vs. vertical deformation curves were obtained from the displacement measurements of the testing machine.

$$\text{Tensile splitting stress} = \frac{2P}{\pi L d} \quad \text{Eq. 1}$$

Where; P = Tensile splitting load (N)

L = Length of the specimens (mm)

d = Diameter of the cylinder specimens (mm)

Adobe units (46 mm high, 140 mm wide and 204 mm long) were tested in 3-point bending, Fig. 3. In total, 6 samples were tested at 28 days. The same machine and load rate as the tensile splitting tests was adopted. Adobe specimens were placed on two 20 mm diameter steel bars with the centre to centre span of 130 mm. The load was vertically imposed via another 20 diameter steel bar, which was positioned at the top centre of the specimens. The

flexural stresses were calculated based on the equation, Eq. 2, specified in Bs En 1015-11 (1999). The displacements were obtained from the relative displacement of the loading bar.

$$\text{Flexural stress} = 1.5 \frac{FL}{bd^2} \quad \text{Eq. 2}$$

Where; F = Flexural load (N)

L = Centre to centre span between the supporting bars (mm)

b = Width of the specimen (mm)

d = Depth of the specimen (mm)

Sandwich specimens i.e. two adobe units attached by means of a 10-15 mm thick mud mortar were tested next. The minimum age of the adopted adobe units was 28 days. The surfaces of the units were firstly cleaned and then wetted prior to being used in preparing the sandwich specimens. It was ensured that the bottom and top surfaces of the assemblages were level using a spirit level. The specimens were left to air-dry for 28 days before testing. The final average dimensions of the sandwich specimens were 204 mm x 140 mm x 104 mm. 10 specimens were tested at the age of 28 days. All the above specimens were kept and tested at room temperature ~ 20°C. To conduct the tests, the top and bottom faces of the adobe specimens were attached to two 10 mm thick steel plates, to allow the load transfer from the machine to the specimens. A 19mm diameter steel bar was attached to the centre of the plates to allow gripping by the machine, Fig. 4. One of the steel plates was firstly attached to the top surface of the adobe specimens, making sure the specimens were centred correctly, using high performance epoxy resin a few days prior to testing, the average thickness of the epoxy layer was around 1 mm. The epoxy resin was commercially known as ‘AT30 EL2 Epoxy Laminating Resin’ with a tensile strength ranging from 70 to 80 MPa. The strength of the epoxy was much higher than the expected tensile strength of adobe unit-mortar interfaces; this ensured failure in the adobe joint rather than in the adobe-steel plate interface. Once the epoxy set between the adobe and the top steel plate, the bottom steel plate was attached to the specimens. While the top steel plate plus the attached specimen was held by the top grip of the machine, the bottom plate was held by the lower grip and epoxy was spread on it. The bottom plate was then attached by slowly moving it towards the specimen until in full contact, testing commenced after the epoxy was allowed to set. The reason for attaching the bottom steel plate within the machine was to ensure the specimens were centred, thus

minimising the generation of bending moments due to eccentricity. Two LVDTs with maximum travel allowance of 10 mm were attached to the specimens (one on each side) via screws. The LVDTs were positioned in such a way so as to measure the vertical opening displacements at the joint. The load was applied at a rate of 0.05 mm/min under displacement control; the axial tensile loads were measured through a 10kN capacity load cell. Finally, tensile strength was calculated by dividing the axial tensile load by the area of the unit-mortar interface. The final displacements were considered as the average of the displacement readings from both LVDTs.

Shear strength tests

The shear bond strength, shear fracture energy and coefficient of friction of the joints in the adobe assemblages were examined via triplet shear tests. The specimens for the shear tests consisted of three adobe units of at least 28 days' age, joined together via 10-15 mm thick mud mortar, see Fig. 4. A thin layer of dental plaster (2-3 mm thick) was applied to the top and bottom surfaces of the specimens to ensure a level surface and that the applied force was parallel to the unit-mortar interfaces. The specimens were kept at room temperature $\sim 20^{\circ}\text{C}$ for 28 days prior to testing. 15 adobe specimens were tested in two phases, in the first phase 6 specimens were tested to obtain the initial shear strength and corresponding fracture energy of the unit-mortar interfaces. Shear tests were carried out according to Bs En 1052-3 (2002) for determination of the shear strength of standard masonry. Two LVDTs with maximum travel allowance of 10 mm were fixed to the specimens, one on each side to measure the vertical sliding in the joints.

In the second phase, 9 specimens were tested by imposing three different levels of pre-compression stress to investigate the enhancement effects on shear strength and to identify the coefficient of friction in the adobe unit to mortar interfaces. The pre-compression stress was set at 2.5%, 5% and 10% of the ultimate compressive strength of the masonry panels (see Table 5). The levels were adopted to ensure the compressive stresses remained below the ultimate strength during the tests, three specimens were tested with each level. The pre-compression was applied via a hydraulic jack horizontally positioned perpendicular to the joints of the specimens, Fig. 4. Pre-compression was uniformly distributed to the specimens via a 300mm x 170mm x 12 mm thick steel plate. Two steel plates were used at both ends to support the specimens and the jack; these were tightly held through four partially threaded steel bars. The mid-length of the bars was left un-threaded to allow free movements of the

middle steel plate used to distribute compressive loads. The compressive loads were measured via a 15kN capacity load cell placed between the jack and mid-plate. A spherical washer was also fixed between the load cell and the jack to ensure the loads were evenly distributed over the edge of the load cell, thus allowing for accurate readings. The pre-compression stress values were monitored during the tests and shown to be approximately constant throughout.

The specimens were tested in a universal testing machine (Model: RDP, T2000BS) with a load cell capacity of 200kN. The shear loads were applied parallel to the joints at a rate of 0.5 mm/min under displacement control up to failure. The outputs from all the instruments were recorded via a data logger (Model: 3530 ORION) with a recording rate of 1 second. Shear stresses were calculated via equation, Eq. 3, from Bs En 1052-3 (2002). The vertical slip in the joints was taken as the average from both LVDTs. The pre-compression stresses were determined as the ratio of pre-compression forces to cross sectional area of the mortar joints.

$$\text{Shear stress} = \frac{F}{2A} \quad \text{Eq. 3}$$

Where; F = Shear force (kN)

A = Cross sectional area of the mortar joints (mm)

Test Results

Compression tests

The mean compressive strength for 14, 28 and 56 day cubic specimens were 1.18, 1.45 and 1.59 MPa, respectively. The results from compression tests are summarised in Table 4. The compressive stress-strain curves for the 28 day case is shown in Fig. 5, 14 and 56 day curves follow a similar a pattern.

The results show a strength gain with age, the largest rate of gain occurred between 14 and 28 days, this is reflective of the relative slow drying of the adobe, in part due to the presence of the straw fibres. The non-linear correlations between the compressive strength and age of cubic specimens were also determined, see Fig. 6.

The mean compressive strength for the cylindrical specimens at 28 days' age was 1.11 MPa, with a coefficient of variation of 4.84%. Based on the mean values, the compressive strength

of the cylinders was 0.77 of the cube strength, see Fig. 7. As expected, this confirmed the effect of geometry on the apparent compressive strength and follows a similar pattern to concrete whereby the compressive strength of cylinders (with aspect ratio = 2) is around 0.8 that of cubes (Domone and Illston, 2002).

From Figures 4 and 7, a consistency between the compressive stress-strain behaviour of all cubic and cylindrical specimens was observed. The compressive stress-strain behaviour can be divided into three stages. The first stage is linear up to around 50% of peak compressive strength. In the second stage, non-linear behaviour in the adobe specimens was noted between 50% and peak compressive strength due to progressive cracking in a direction parallel to the loading direction, these cracks vertically or diagonally extended from the central regions towards the edges of the vertical faces of the specimens. In the last stage, following peak strength, gradual compression softening until failure occurred. The deformation capacity of the specimens was relatively high (no rapid drop in strength) compared to the post-peak softening behaviour of concrete, this is in part due to the presence of straw fibre in the soil matrix. In addition, the progressive reduction of the specimens' aspect ratio also potentially enhanced the influence of confinement effects from the platens' restraint. At this stage, the previously formed cracks widened and further cracks propagated through the vertical faces of the specimens. These cracks resulted in lateral expansion of the specimens. Consequently, the specimens disintegrated into several pieces. Initially, these pieces were held together by the action of straw fibres until the failure of the bond between fibres and soil. It was noted that no rupture of straw fibres occurred, which indicates the bond strength of the fibre/adobe interface is less than the tensile strength of the straw fibre itself.

The adobe masonry panel tests returned a mean compressive strength of 1.24 MPa as shown in Table 5. The behaviour of the panels under compression is elastic up to around 40% to 60% of the peak stresses, Fig. 8. Based on the strain measurements obtained from LVDT readings, the mean secant Young's modulus of the panels was 127 MPa, which was obtained based on the slope of the line joining the stress point at one-third of ultimate strength to the origin point of the stress-strain diagrams. Following this, non-linear behaviour was observed up to the peak compressive stresses. At the beginning of this stage, small cracks propagated at the head joints in the central regions of the panels, these widened and further cracks formed at the head joints located near the corners. Subsequently, some minor vertical and diagonal cracks were observed in the units, evenly distributed in the central regions, these cracks widened and extended towards the previously formed cracks at the head joints in the

central region. At this stage the average crack width was around 1 mm at ultimate compressive strength. These cracks resulted in slight deformation due to lateral expansion of the adobe panels at almost mid-height. At this stage, due to the deformation, one of the LVDTs detached from one of the panels, Comp-S1, so the readings for this particular panel stopped at peak stress. The post-peak behaviour was a gradual drop in compressive strength up to failure. The drop in strength was due to formation of severe cracks, the previously formed cracks at the central and corner regions vertically and diagonally extended towards each other, passing through the joints and adobe units. This behaviour was accompanied by spalling of the panel surface. The width of cracks ranged from 2 to 10 mm at the end of the tests. As a result adobe panels split into several pieces but did not fully collapse due to the bridging effects of the straw fibres, Fig. 9 (inset). It is notable that the coefficient of variation for the modulus of elasticity is relatively large, however this is consistent with that reported by other researchers e.g. 48% (Almeida, 2012), 25% (Miccoli et al., 2014) and 30% (Quagliarini et al., 2010). This may point towards differences in density and relative compaction of the units given the nature of their formation, which may be influential in the elastic stages of the compression test.

In general the cracks were evenly distributed within both joints and adobe units i.e. no significant difference between joint and adobe units behaviour occurred, this confirmed the relative homogeneity of these particular adobe panels.

Tension tests

The mean tensile strength from the dog bone specimens was 0.212 MPa. The mean tensile fracture energy was found to be 0.353 N/m. Herein, the term fracture energy is defined as the amount of energy consumed when the crack propagates in a unit area of adobe unit to mortar interface. This is calculated by integrating the post-peak area under the tensile stress-displacement curves using the same concept for measuring the fracture energy of concrete materials as recommended by Rilem Tc 50-Fmc (1985). The details of the tension test results are summarised in Table 6. The tensile stress-displacement behaviour for the dog bone specimens is shown in Fig. 10. The response of the dog bone samples was mainly linear up to the maximum tensile stress, beyond this, tensile cracks initiated at the central region of specimens. The cracks propagated across the specimens, gradually widened to around 3 mm and eventually split the specimens into two pieces at failure. The post-peak drop in the tensile stress was rather gradual, after crack initiation the straw fibres carried tensile stress in the

cracked region. Consequently, the tensile strength of specimens was lost due to gradual debonding between the straw fibres and the soil. No rupture of the straw fibres was observed.

In general, the behaviour and strength of adobe, in similarity to concrete, under direct tension noticeably varies due to irregularities in the aggregate/soil particle size and distribution, localised inconsistencies in density, uneven distribution of fibres etc. in the weakest region of the samples. For this reason, the coefficient of variation of the peak tensile stress in the dog bone specimens, 17.6%, was higher than that for compressive test results presented in Table 4. In a similar way, the variation in the fracture energy results was relatively high; this can be explained as a result of the random distribution of the straw fibres. Not only the amount of straw fibres, but their directions in the smallest cross-section region can significantly affect the post-peak behaviour. It should be noted that all 16 samples presented in this study exhibited failure by cracking in the critical (i.e. middle) section.

The mean peak stress from the tensile splitting tests was 0.15 MPa, see Fig. 11. The failure mechanism in all cylinders was consistent following a typical failure mode of tensile splitting tests of concrete i.e. fracture into two pieces. The behaviour of the cylindrical specimens was linear up to reaching around two thirds of the ultimate strength. Following this, stress hardening occurred until reaching ultimate strength. During this stage, minor vertical cracks parallel to the loading direction initiated in the central regions on both ends of the cylinders. Beyond this point, softening occurred due to widening and extension of vertical cracks from the central regions towards the top and bottom, the cracks were ~ 4 mm wide. With further vertical deformation, the cylindrical specimens split into two pieces due to the vertical crack extending along the length of specimens with straw fibres bridging until pull-out occurred.

The average flexural strength of the specimens from three-point bending tests was 0.71 MPa, see Fig. 12. The flexural stress versus corresponding mid-span vertical displacement at the top of the specimens was qualitatively similar to the direct tensile tests. At peak strength, a crack initiated approximately as a straight line passing through the centre of the bottom face of the specimens, as a result the flexural stress dropped rapidly up to around 4 mm vertical displacement. Following this, the straw fibres bridged between the two main pieces of the adobe specimens along the crack, see Fig. 13. From this point, the straw fibres provided a degree of ductility with a relative gradual decrease in the flexural strength up to complete failure, following pull-out of the fibres.

For the tensile strength of the adobe unit-mortar interfaces, a mean value of 0.041 MPa was obtained. The mean tensile fracture energy of all specimens was 0.007 N/m. Furthermore, the mean stiffness of the adobe mortar joints in the normal direction was determined as 2.41 N/mm³, which was calculated as the ratio between the peak tensile stress and corresponding displacement of the LDVTs at that point. A summary of the test results and the plot of direct tensile stresses versus opening of the mortar joints for all adobe specimens are given in Table 7 and Fig. 14, respectively. The response of the joints in all specimens was linear up to ultimate tensile strength; following this a sudden drop in the tensile strength occurred due to propagation of cracks in the mortar joints. The cracks mainly occurred in the unit-mortar interfaces as result of de-bonding between the mud mortar and units either in the top interface or bottom interface, Fig. 15. In two specimens (Tensile-S4 and Tensile-S6), the cracks followed a path transmitted from the top unit to mortar interface cutting diagonally through the approximate centre of the mud mortar to the bottom unit to mortar interface, Fig. 15.

Shear tests

The average initial shear strength (i.e. no pre-compression) and shear fracture energy from the 6 adobe specimens was 0.091 MPa and 0.089 N/mm, respectively. The summary of the shear test results is shown in Table 8. Fig. 16 shows the shear stress versus vertical slip in the joints for all initial shear tests. In all specimens, the shear stress rapidly increased up to reaching peak value when the mean vertical slip in the joints was 0.08 mm. In other words, the behaviour of the specimens was linear up to reaching the peak shear stress, the slope of the linear part was considered as the shear stiffness of the joints. The post-peak behaviour was brittle with a sudden drop in the shear stresses due to crack propagation along the joints. As the shear force increased, cracks in the joints progressively grew and the strength degraded up to failure. Failure was typically due to cracks propagating as a result of de-bonding between units and mortar, no cracks within the units and mortar themselves occurred. The crack path in the unit to mortar interfaces was either along the interface on one side, or transformed from one interface cutting through the mortar to the other interface, as was the case in two of the specimens, Fig. 17.

The confinement effect of normal pre-compression stress on shear strength was also obtained, see Table 8. The response of the shear specimens in terms of shear stress versus vertical slip in the joints have been detailed in Fig. 18. Unlike the linear behaviour of the initial shear tests, the shear stresses linearly increased up to reaching 70%-80% of the peak value. Beyond

this point, hardening behaviour in shear stresses occurred with formation of progressive cracks in the joint interfaces. The normal stresses in the joints slowed the propagation of initial cracks. This phenomenon provided some ductility with shear hardening up to reaching the peak shear stresses. At this point, the cracks fully developed along the joints and resulted in a gradual drop in shear stresses. Following this, the shear stresses were sustained only by the action of the normal stresses and the friction between the failed interfaces in the joints.

The major failure mechanism in all specimens in the shear tests was due to formation of cracks at the unit to mortar interfaces. During the shear hardening stages, the cracks progressively transferred from one interface through the mortar to the other interface in all specimens, Fig. 19. In the initial shear tests, rapid crack opening at one interface occurred in the majority of specimens due to absence of pre-compression stress. In specimens with relatively high pre-compression stresses i.e. 0.062 and 0.124 MPa, after formation of cracks at the unit to mortar interfaces, further evenly distributed shear cracks occurred within the mortar, Fig 16, as a result of continuous shear stresses imposed until the end of the tests.

The ultimate shear strength obtained from each specimen was plotted against their corresponding normal pre-compression stress. A linear best-fit line was drawn to derive a linear equation based on Mohr-Coulomb failure criterion as shown in Fig. 20. According to the derived equation, the initial shear strength was 0.098 MPa, which agreed relatively well with the mean result from the initial shear tests, 0.091 MPa (see Table 8). Furthermore, the coefficient of friction between the adobe units and mortar interfaces was 1.23.

Conclusions

A comprehensive experimental programme for a consistent, practical and repeatable type of adobe and corresponding adobe masonry assemblies has been presented. In addition to compression and tensile strengths of the materials, data on the tensile strength and fracture energy of mud mortar adobe assemblies has been produced for the first time. The results provide a detailed characterisation of the material which will contribute to improved analysis and understanding of adobe structures. Based on the test series examined, the principal conclusions of this study are as follows:

- The rate of compressive strength gain for cubic specimens increases from 14 to 28 days compared to 28 to 56 days, the strength achieved at 28 days was > 90% of the strength at 56 days. This confirmed the ability to achieve most of the possible strength of adobe

specimens at 28 days. For the particular mix adopted here an equation was proposed to predict strength gain from 14 to 56 days.

- The geometry of the specimens, in similarity to concrete, had an influence on compressive strength with cylinder strengths being 77% of cube strengths.
- The post-peak behaviour of adobe specimens under compression was fairly ductile. This behaviour was in part due to the presence of the straw fibres in the adobe mixture. The increase in the platen restraint due to reduction in the specimen's aspect ratio and resulting confinement effects also influenced this behaviour.
- The compressive strength of the adobe masonry panels and cubic specimens were relatively close, 1.24 and 1.45 MPa, respectively. This was due to the fact that both adobe units and mud mortar were made of the exactly same soil mixture i.e. their compressive strength was similar. Therefore, no significant distinction between the behaviour of adobe units and mortar joints in the masonry specimens occurred under compression. The cracks were generally evenly distributed within both joints and adobe units, which confirmed the fairly homogeneous behaviour of the adobe masonry panels, unlike standard masonry made from fired clay brick units.
- As expected, the tensile strength of the unit to mortar interfaces was lower than the tensile strength of adobe itself. This confirmed the dominance of unit to mortar interfaces in governing the behaviour of adobe masonry in situations other than direct compression.
- The post-peak behaviour of specimens in direct tension was controlled by the straw fibres. Due to the random distribution in the amount and direction of straw fibres in the weakest region of the dog bone specimens, the scatter in their fracture energy was relatively high.
- In the shear tests without pre-compression the peak shear stress rapidly dropped due to propagation of cracks along the unit to mortar interfaces. In specimens with pre-compression, the drop in shear strength was less rapid with occurrence of cracks within the mortar as well as in the interfaces.
- The normal pre-compression stresses linearly increased the shear strength. Based on the best-fit line between shear strength and normal stress, the coefficient of friction and initial shear strength were determined.

In similarity to many other researchers (Adorni et al., 2013; Illampas et al., 2014; Parisi et al., 2015; Quagliarini and Lenci, 2010; Silveira et al., 2012; Yetgin et al., 2008), the strains were

measured based on the test machine platen movements, this is due to the practical difficulties of attaching gauges to the adobe surface which is prone to spalling under load. In order to achieve a higher level of accuracy, alternative strain measurement e.g. optical methods including digital image correlation should be considered in future work.

In the field, by necessity, adobe units are usually dried by simple atmospheric exposure. This approach was adopted for unit manufacture in the present work. It is acknowledged that such an approach may lead to lack of uniformity in moisture content and density across the suite of units employed. Although the coefficient of variation of densities at 28 days was relatively low (3.18%) in this work, indicating reasonable uniformity, the issue could be further investigated by oven drying of units at moderate temperatures and the results compared with those from air dried units.

Data availability

Some or all data, models, or code generated or used during the study are available from the corresponding author by request. The available items are data series in excel for Figures 1, 5, 6, 7, 8, 10, 11, 12, 14, 16, 18 and 20.

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List of Tables

Table 1. Summary of existing results from compressive tests of various adobe specimens

Sample size (mm)	Mixture, % by weight		Straw fibre content, %	Compressive strength (MPa)	Coefficient of variation, %	Reference
	Clay and Silt	Sand and Gravel				
Cylinders, Height = 160 to 180 Diameter = 80 to 90		Not reported		1.17	39.4%	(Silveira et al., 2012)
230 x 150 x 130	28.2 77	71.5 23	0.3 by weight 0.0	2.14 2.88	8.5%-26%	(Quagliarini and Lenci, 2010)
50 x 50 x 50	78	22	20 – 40 by volume	1.75	11.8%	(Illampas et al., 2014)
	61	39	15 – 28 by volume	0.6	76.8%	
Cylinders, Height = Diameter = 50	78	22	20 – 40 by volume	1.41	20.9%	
	61	39	15 - 28 by volume	0.76	17.7%	
100 x 100 x 50	78	22	20 - 40 by volume	2.16	21.3%	
	61	39	15 - 28 by volume	0.52	23.7%	
200 x 90 x 50	49	49.5	0.5 by weight	1.70	11%-32%	(Wu et al., 2013)
	40	59.5	0.5 by weight	1.39		

Table 2. Summary of existing tensile tests of various adobe specimens

Test method	Sample size (mm)	Average of results (MPa)	Coefficient of Variation, %	Reference
Tensile splitting	Height: 160 to 180 Diameter: 80 to 90	0.19	55%	(Silveira et al., 2012)
	Height: 55 to 110 Diameter: 70 to 95	0.2	32%	(Adorni et al., 2013)
	Height: 170 Diameter: 85	0.13	52%	(Martins and Varum, 2006)
Three point bending	40 x 40 x 160	0.56	36%	(Caporale et al., 2014)
	155 x 165 x 380	0.17 to 0.4	58%	(Liberatore et al., 2006)
	Not given	0.18 to 0.35	31%	(Baglioni et al., 2010)
Direct tensile	20 x 40 x 160	0.38 to 0.75	28%	(Yetgin et al., 2008)
	110 x 130 x 180	0.15 to 0.27 0.081	23% 43%	

100 x 140 x 295	0.21 to 0.23	Not given	(Mesbah et al., 2004)
Height: 140	0.075 to 0.52	32%	(Araki et al., 2016)
Diameter: 50			

765

766 **Table 3.** Adobe mix constituents and physical properties

Soil contents		Determined in accordance with
Clay content	16.30%	
Silt content	33.7%	
Sand content	32.4%	(Bs En 1377-2, 1990)
Gravel content	17.6%	
Soil Classification	Sandy clay soil	(Bs 5930, 2015)
Atterberg limits		
Liquid limit	36.20%	
Plastic limit	19.70%	(Bs En 1377-2, 1990)
Plasticity Index	16.50%	
Straw fibres		
Length	25 - 70 mm	
Diameter	0.5 - 1.2 mm	
Weight fraction of the constituents		
Dry soil mix	77.50%	
Water	22% i.e. water to dry soil ratio = 0.28	
Straw fibre	0.5%	
Average density and moisture contents of adobe specimens (based on examination of 30 representative adobe specimens)		
Initial density of adobe mixture	1914 Kg/m ³	
Density at 28 days	1855 Kg/m ³ (Coefficient of variation = 3.18%)	
Moisture content at 28 days	20.75% (Coefficient of variation = 15.8%)	

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768

769 **Table 4.** Summary of compression test results

Results from Compression tests				
	Cubes			Cylinders
Age	14 days	28 days	56 days	28 days
Number of Specimens	12	12	6	6
Mean compressive strength (MPa)	1.18	1.45	1.59	1.11
Coefficient of Variation, %	12.34	11.53	9.16	4.84
Mean strain at ultimate compressive strength	0.042	0.038	0.042	0.013
Coefficient of Variation, %	38.28	20.67	19.76	12.96

770

771 **Table 5.** Summary of compression test results obtained from adobe masonry panels

Specimen Name	Compressive strength (MPa)	Strain at ultimate compressive strength (mm/mm)	Secant Young's modulus at 1/3 of ultimate compressive strength (MPa)
Comp-S1	1.17	0.047	59
Comp-S2	1.26	0.049	142

Comp-S3	1.27	0.041	179
Mean	1.24	0.046	127
Coefficient of Variation, %	4.44	9.63	48.61

772

773 **Table 6.** Summary of tension test results

Results from Tension tests			
Test Method	Direct tensile	Tensile splitting	Three point bending
Number of Specimens	16	6	6
Mean tensile strength (MPa)	0.212	0.15	0.71
Coefficient of Variation, %	17.6	5.74	16.92

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775 **Table 7.** Summary of results from direct tensile tests of adobe sandwich assemblages

Specimen name	Maximum Tensile strength (MPa)	Corresponding displacement at maximum Tensile strength (mm)	Joint stiffness in normal direction (N/mm ³)	Fracture energy (N/mm)
Tensile-S1	0.050	0.020	2.50	0.0056
Tensile-S2	0.048	0.021	2.32	0.0069
Tensile-S3	0.033	0.018	1.87	0.0066
Tensile-S4	0.044	0.019	2.36	0.0083
Tensile-S5	0.037	0.013	2.79	0.0070
Tensile-S6	0.038	0.014	2.69	0.0056
Tensile-S7	0.041	0.019	2.21	0.0061
Tensile-S8	0.038	0.014	2.67	0.0084
Tensile-S9	0.034	0.017	1.99	0.0076
Tensile-S10	0.048	0.017	2.73	0.0078
Mean tensile strength	0.041	0.017	2.41	0.0070
Coefficient of Variation, %	14.80	14.74	13.32	14.63

776 **Table 8.** Summary of shear test results of adobe specimens from triplet tests

Results from Shear tests			
Initial shear tests i.e. no normal pre-compression stress			
Number of Specimens	Mean initial shear strength (MPa)	Mean shear fracture energy, N/mm	Joint stiffness in shear direction, N/mm ³
6	0.091	0.089	1.2
Coefficient of Variation, %	7.6	23	21.3
Shear tests with normal pre-compression stress			
Number of Specimens	Normal pre-compression stress (MPa)	Ultimate shear strength (MPa)	Coefficient of Variation %
3	0.031 (2.5% of compressive strength)	0.137	10.75
3	0.062 (5% of compressive strength)	0.172	3
3	0.124 (10% of compressive strength)	0.251	5.45

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Fig. 1. Particle size distribution in the selected adobe soil mixture and recommended range

Fig. 2. Test set-up details for adobe masonry specimens tested under uniaxial compression

Fig. 3. Test configurations for tensile strength characterisations: (a) dimension of dog bone specimens; (b) direct tensile test; and (c) three-point bending test

Fig. 4. (a) details and test setup for direct tensile tests of adobe assemblages; (b) configuration of initial shear tests; and (c) configuration of triplet shear tests with application of normal pre-compression stresses

Fig. 5. Compressive stress-strain curves for 28 day aged cubes

Fig. 6. Polynomial relationship between compressive strength and age of adobe cubic specimens

Fig. 7. Compressive stress-strain curves for 28 day aged cylinders

Fig. 8. Compressive stress-strain curves for adobe masonry panels

Fig. 9. Crack pattern during the post-peak stage and cracking at failure (inset)

Fig. 10. Direct tensile test stress-displacement relationships

Fig. 11. Tensile stress-vertical displacement relationships from tensile splitting tests

Fig. 12. Flexural stress-deflection curves for adobe specimens tested under three point bending

(a)

(b)

Fig. 13. Typical failure mechanism of adobe specimens tested under three-point bending: (a) bridging action by straw fibres; and (b) de-bonding of the straw fibres

Fig. 14. Tensile stress-mortar joint opening responses of adobe assemblages from direct tensile tests

(a)

(b)

(c)

Fig. 15. Typical failure modes of adobe mortar joints under direct tension: (a) crack between unit and mortar at top interface; (b) crack between unit and mortar at bottom interface; and (c) crack paths combining top and bottom interfaces of units and mortar

Fig. 16. Response of adobe specimens from initial shear strength tests

Fig. 17. Typical crack patterns in adobe specimens from initial shear tests: (a) crack path along one interface only; and (b) crack path divided between both joints' interfaces

Fig. 18. Response of adobe shear specimens confined with different levels of normal pre-compression stresses

(a)

(b)

(c)

Fig. 19. Failure modes of adobe specimens confined with normal pre-compression stress and tested under shear: (a) crack path at unit to mortar interfaces only; (b) cracks at unit to mortar interfaces and mortar; and (c) crack at unit to mortar interfaces and excessive shear cracks in the mortar

Fig. 20. Correlation between peak shear stress and normal pre-compression stress

Figure Caption List:

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- **Fig. 4.** (a) details and test setup for direct tensile tests of adobe assemblages; (b) configuration of initial triplet shear tests; and (c) configuration of triplet shear tests with application of normal pre-compression stresses
- **Fig. 5.** Compressive stress-strain curves for 28 day aged cubes
- **Fig. 6.** Polynomial relationship between compressive strength and age of adobe cubic specimens
- **Fig. 7.** Compressive stress-strain curves for 28 day aged cylinders
- **Fig. 8.** Compressive stress-strain curves for adobe masonry panels
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