FLEXURAL STRENGTH OF UNREINFORCED LATTICE MASONRY WALLS SUBJECTED TO LATERAL OUT-OF-PLANE LOADING

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ABSTRACT
Lattice masonry is a form of construction in which the mortar perpend joints are left unfilled and the masonry units are spaced along the courses to leave gaps between adjacent units. When designing such walls to resist out-of-plane lateral loading due to wind and/or earthquake actions, the Australian Standard AS3700: Masonry Structures provisions for one way vertical bending can be applied by using a section modulus based on the net bedded area. However, the provisions for horizontal bending require that the masonry be constructed with all perpends completely filled and therefore lattice masonry falls outside the scope of AS3700 for horizontal bending. The paper describes a preliminary study to assess the behaviour of unreinforced lattice masonry walls subjected to lateral out-of-plane pressure loading. Six lattice masonry panels, 14 courses (1194 mm) high x 4 units with 90 mm gaps (1190 mm) long, were constructed using extruded clay bricks (230 mm long x 110 mm wide x 76 mm high) and 1:1:6 (cement : lime : sand) mortar. Three panels were tested in one way vertical bending and three were tested in one way horizontal bending. The load versus deformation behaviour and the observed failure modes are reported. The AS3700 provisions for solid masonry were used to predict the panel strengths and an assessment of the suitability of the provisions for the design of lattice masonry is made.

KEYWORDS: *flexural strength, lattice masonry, out-of-plane loading*

INTRODUCTION
Lattice masonry (also known as hit and miss brickwork) is a form of construction in which the mortar perpend joints are left unfilled and the masonry units are spaced along the courses to leave

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gaps between adjacent units. This creates a wall which has a lattice type appearance, allowing the passage of light and air through the wall. A range of examples is shown in Figure 1.

Figure 1: Examples of Lattice Masonry Construction, also known as "Hit and Miss Brickwork" (photographs supplied by Think Brick Australia)

Walls of this form can be used to enclose areas where complete weather proofing is not required, but a level of privacy and limited protection from wind and rain is desired. Regardless of whether this masonry is used in a loadbearing or nonloadbearing application, it must still be capable of spanning between supports to meet minimum robustness requirements and to resist out-of-plane lateral loading due to wind and/or earthquake actions, and hence it requires structural design. Australian Standard AS3700: Masonry Structures [1] provisions for one way vertical bending of unreinforced masonry can be applied in the case of lattice masonry by using a section modulus based on the net bedded area. However, the AS3700 provisions for horizontal bending require that the masonry be constructed with all perpends completely filled and therefore lattice masonry falls outside the scope of AS3700 for horizontal bending. By extension, in their present form, the AS3700 provisions for two way bending action, which rely on the horizontal bending capacity, also cannot be applied for lattice masonry. Numerous recent enquiries to Think Brick Australia from architects and structural engineers wishing to specify and design using lattice masonry prompted the current study which is designed to better understand the behaviour of unreinforced lattice masonry walls subjected to lateral out-of-plane pressure loading. A review of overseas codes and relevant literature revealed a lack of existing information on the design of this form of construction, thus further justifying the need for research in this area.

EXPERIMENTAL PROGRAM

Six unreinforced clay brick lattice masonry panels were constructed using one type of extruded clay brick (230 mm long x 110 mm thick x 76 mm high) with 10 mm thick bed joints using a 1:1:6 cement:lime:sand (by volume) mortar. The panels were constructed by an experienced mason using consistent mortar throughout. The dimensions chosen for the panels are shown in Figure 2. The choice of gap (90 mm) and unit overlap (70 mm) shown in Figure 2 was based on a compromise between aesthetic value and maintaining sufficient unit overlap, and hence strength, to allow the specimens to be handled in the laboratory. The single leaf panels were 110 mm thick. The lateral modulus of rupture of the masonry units was tested in accordance with AS/NZS4456.15 [2] and found to be 2.90 MPa (mean) with a COV of 0.16. For each batch of mortar mixed during the panel construction, two x 6 unit high (10 mortar joints) stack bonded piers were constructed and the flexural tensile strength of the masonry was determined by bond wrench testing in accordance with AS3700 [1]. The bond wrench tests were performed on the same date of testing of the corresponding panel constructed using that mortar batch. The panels and bond wrench piers were cured under ambient conditions in the laboratory for a minimum of 28 days prior to testing.

The panels were simply supported in a one way bending configuration and subjected to out-of-plane pressure loading using an airbag system as shown in Figure 3. Three of the panels were orientated to produce one way vertical bending (bending stresses acting normal to the mortar bed joints, Figure 3b), and three of the panels were orientated to produce one way horizontal bending (bending stresses acting parallel to the mortar bed joints, Figure 3c). All panels were tested in a vertical spanning orientation (simply supported along the top and bottom edges with centre to centre span of 1104 mm, Figure 3a). In the case of the horizontal bending specimens, this required first rotating the specimens through 90° as shown in Figure 3c. This allowed the use of the same apparatus as used for vertical bending tests and avoided the problems associated with providing a frictionless base support. The panels were supported on stiff water filled rubber tubes to absorb any specimen irregularities (Figure 3). The air bag pressure was applied monotonically at a constant rate until failure was observed. The displacements of each panel at mid-height and at the supports were continuously recorded using potentiometers throughout each test.

For later use in the presentation and discussion of results, the specimen naming convention is as follows: the first letter denotes the bonding pattern, L for lattice bond; and the second letter denotes the bending direction, V for one way vertical bending or H for one way horizontal bending. Three replicates of each specimen type were constructed leading to the inclusion of a number (1, 2 or 3) in the naming convention.

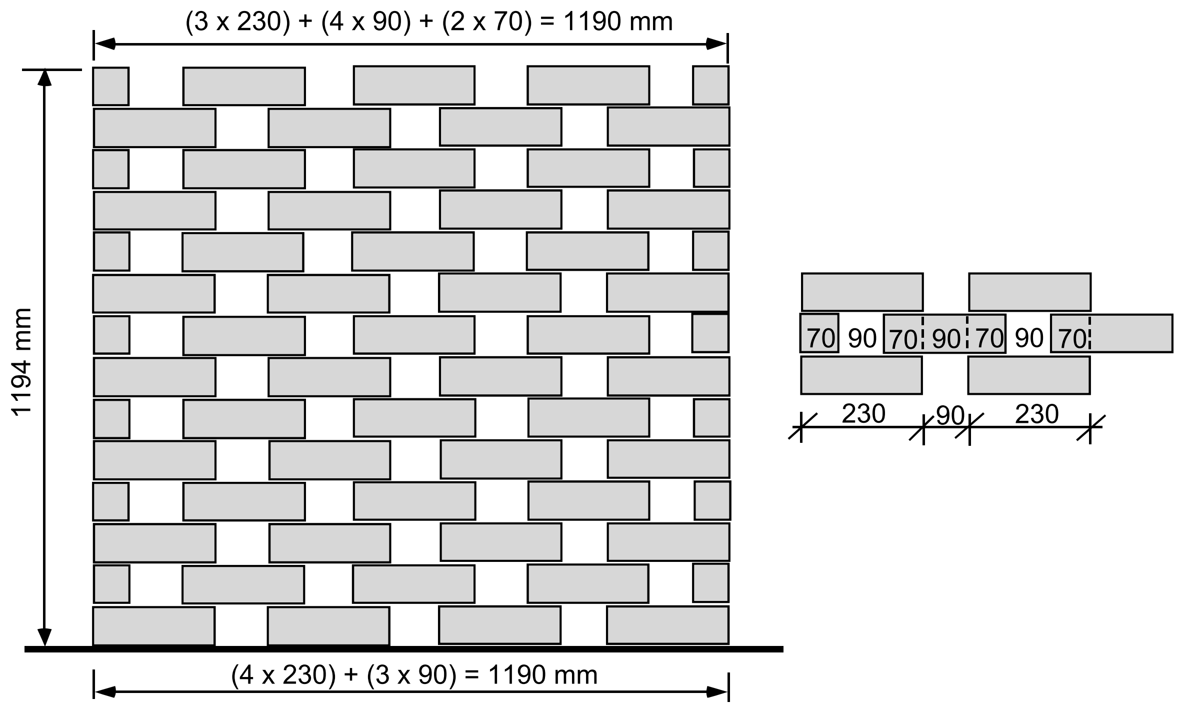
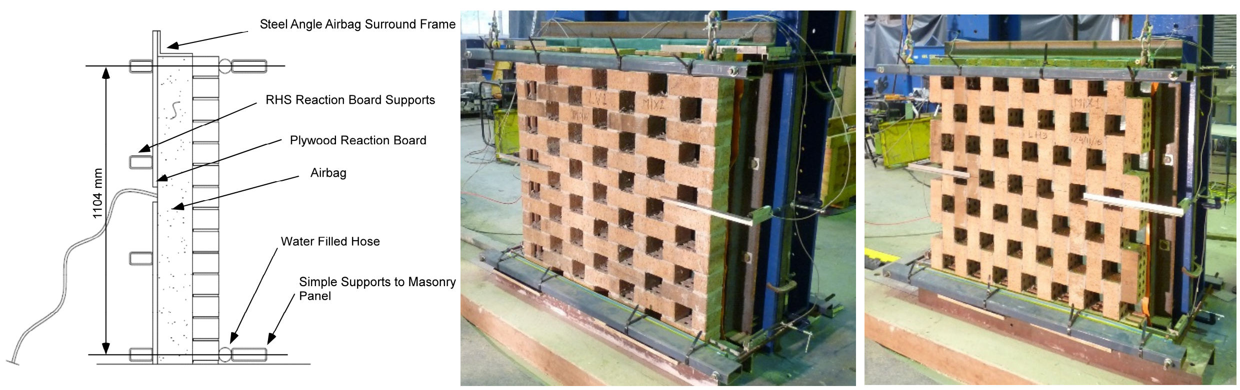


Figure 2: Lattice Masonry Panel Dimensions



(a) **(b)** **(c)**

Figure 3: Test Setup: (a) Schematic elevation, (b) vertical bending, (c) horizontal bending

STRENGTH PREDICTIONS

The primary motivation for the current study is to provide guidance to designers wishing to specify and design lattice masonry walls. Therefore, the AS3700 [1] provisions for solid masonry were used to predict the panel strengths and an assessment of the suitability of the provisions for the design of lattice masonry was made. For the specimens subjected to one way vertical bending, the vertical bending moment capacity M_{cv} was predicted using Equation 1:

$$M_{cv} = \Phi f'_{mt} Z_d + f_d Z_d \tag{1}$$

Where Φ = the capacity reduction factor; f'_{mt} = characteristic flexural tensile bond strength of the masonry, f_d = compressive stress acting on the bed joints; and Z_d = section modulus of the bedded area. In applying this equation, Z_d was calculated based on the net bedded area (after deducting the 90 mm gaps along each bedded course), and f_d was taken as the compressive stress due to panel self weight at the panel mid-height as this is the expected region of failure under the adopted test arrangement. For each of the specimens subjected to one way horizontal bending, the horizontal bending moment capacity M_{ch} was predicted as the lesser of the values determined using Equations 2 and 3:

$$M_{ch} = 2.0\Phi k_p \left(\sqrt{f'_{mt}}\right)\left(1+\frac{f_d}{f'_{mt}}\right) Z_d \tag{2}$$

$$M_{ch} = \Phi \left(0.44 f'_{ut} Z_u + 0.56 f'_{mt} Z_p\right) \tag{3}$$

Where Φ and f'_{mt} are as defined above for vertical bending; k_p = perpend spacing factor (see below); f_d = compressive stress acting on the bed joints (zero in the case of panels rotated 90°); f'_{ut} = characteristic lateral modulus of rupture of the masonry units: Z_d, Z_u, and Z_p = section moduli of the bedded area, masonry units, and perpend joints respectively. Equation (2) represents a stepped failure along the mortar joints while Equation (3) represents a line failure extending alternately through masonry units and perpend joints (if present). In the case of lattice masonry, there are no perpend joints, so Z_p reduces to zero. The perpend spacing factor k_p represents the degree of overlap of masonry units in adjacent courses and is taken as the lesser of the ratios of (unit overlap : unit width), (unit overlap : unit height) and 1. For the lattice masonry geometry shown in Figure 2, k_p = 0.64.

Once the bending moment capacity was calculated for each panel using Equations 1 to 3 as appropriate, the lateral out-of-plane pressure (denoted W in the next section) required to induce the moment capacity at panel mid-span was calculated and taken as the predicted panel strength in kPa. Despite the wall being perforated, the gross area of contact was used when calculating the moment induced by the airbag pressure. This approach was used because during load application, the airbag "parachutes" across the holes in the masonry, thus transferring the pressure incident on the holes to the masonry surrounding the holes. That is, during the test no pressure escapes through the holes as would occur, for example, with wind pressure acting on a real wall.

The strength predictions were made using measured mean, rather than characteristic, values for material strengths and a capacity reduction factor of 1 to assess the accuracy of the code strength based equations in predicting the experimentally observed strengths. A second set of design based calculations were also performed using code default values for characteristic strengths (f'_{mt} = 0.2 MPa and f'_{ut} = 0.8 MPa) and the code specified values for capacity reduction factor (Φ = 0.6 for both vertical and horizontal bending) to assess whether the code would produce conservative designs for panels loaded under similar conditions to those tested.

RESULTS AND DISCUSSION

The experimental results and AS3700 strength predictions are summarized in Tables 1 and 2. The recorded plots of load versus mid-height displacement for all panels are shown in Figure 4. The displacements reported in Figure 4 are the net displacements after subtracting any movement occurring at the water filled hose supports. The inset in Figure 4 shows an expanded view of the small displacement region of the observed behaviour.

Table 1: Material Properties for Lattice Masonry Panels

Test ID	Unit Lateral Modulus of Rupture[1] f_{ut} (MPa)		Masonry Flexural Tensile Strength[2] f_{mt} (MPa)		
	Mean	COV	Mean	COV	Mortar Batch
LV1	2.90	0.16	0.82	0.24	Mix 2
LV2	2.90	0.16	0.82	0.24	Mix 2
LV3	2.90	0.16	0.49	0.14	Mix 3
LH1	2.90	0.16	0.75	0.34	Mix 1
LH2	2.90	0.16	0.49	0.14	Mix 3
LH3	2.90	0.16	0.75	0.34	Mix 1

[1] Determined in accordance with AS/NZS4456.15 [2]
[2] Determined in accordance with AS3700 [1]

The panel specimens subjected to one way vertical bending (LV) displayed linear elastic response with no damage observed prior to a crack developing suddenly at peak pressure. The failure line in all three panels extended completely along a single bed joint close to the panel mid-height (region of maximum bending moment under the imposed loading). In the case of Specimens LV1 and LV3, the crack occurred in the bed joint one course below the panel mid-height (Figure 5a). For Specimen LV2 the crack occurred in the bed joint two courses below mid-height. Post cracking, the pressure resisted by the panels dropped suddenly but the panels did not collapse. Rather, they were able to continue to resist small post peak pressures (Figure 4) by virtue of the restoring moment generated due to the self-weight of the two “halves” of the panel acting as rigid blocks rotating about the contacting edges.

For the LH specimens, the failure mode was also non-ductile, with no damage being observed prior to sudden failure at peak pressure (Figure 4). In the case of Specimens LH1 and LH3, the panels collapsed completely at peak pressure making it difficult to observe the failure surface. Inspection of video footage and panel debris revealed that the failure surface for Specimen LH1 extended partly through the mortar joints (stepped failure) and partly by flexural rupturing of the units (line failure). For Specimen LH3, the failure surface extended predominantly through the units (line failure) with some stepped failure as shown in Figure 5b. Specimen LH2 displayed a stepped failure mode in which the panel did not collapse, but rather the failure surface extended solely through the mortar joints in a torsional shearing mode (Figure 6a). Although the cracking was well distributed throughout the panel, most of the post peak rotation of the panel occurred along a line of bed joints just below panel mid-height as shown in Figure 6b.

Table 2: Observed and Predicted Lattice Panel Strengths

Test ID	Wall panel capacity (kPa)					Observed failure mode
	w_{exp}[1]	w_{pred}[2,4]	w_{AS3700}[3]	w_{pred} / w_{exp}	w_{AS3700} / w_{exp}	
LV1	1.94	5.21	0.85	2.69	0.44	flexural failure along bed joint one course below mid-height
LV2	2.69	5.21	0.85	1.94	0.32	flexural failure along bed joint two courses below mid-height
LV3	3.32	3.16	0.85	0.95	0.26	flexural failure along bed joint one course below mid-height
Mean (COV)	**2.65 (0.26)**			**1.86 (0.47)**	**0.34 (0.28)**	
LH1	13.32	6.91	2.14	0.52	0.16	Sudden complete collapse via combined step and line failure
LH2	10.50	5.58	2.14	0.53	0.20	Stepped failure through joints
LH3	17.25	6.91	2.14	0.40	0.12	Sudden complete collapse via combined step and line failure
Mean (COV)	**13.7 (0.25)**			**0.48 (0.15)**	**0.16 (0.25)**	

[1] Experimentally observed wall panel capacity (peak pressure resisted).
[2] Predicted capacity based on mean material strengths (data from bond wrench, modulus of rupture) and capacity reduction factor $\Phi = 1$
[3] Design capacity to AS 3700 [1]: using code default characteristic strengths and code capacity reduction factor $\Phi = 0.6$
[4] Using Equation (3) only for LH specimens results in $w_{pred} = 16.9$ kPa

As shown in Table 2, the LH specimens were able to sustain considerably higher pressures prior to failure (average peak pressure of 13.7 kPa) than the LV specimens (average peak pressure of 2.65 kPa). Failure of the LH specimens engages the flexural strength of the units and / or the torsional shearing strength of the bed joints as a result of the unit overlap in the bonding pattern, whereas failure of the LV specimens is limited only by the relatively low flexural strength of the mortar bed joints. This difference in strengths (also observed in fully bonded masonry) highlights the need for reliable methods to predict the horizontal bending capacity of lattice masonry walls in order for designers to best capitalise on the considerably higher strength when spanning in the horizontal direction.

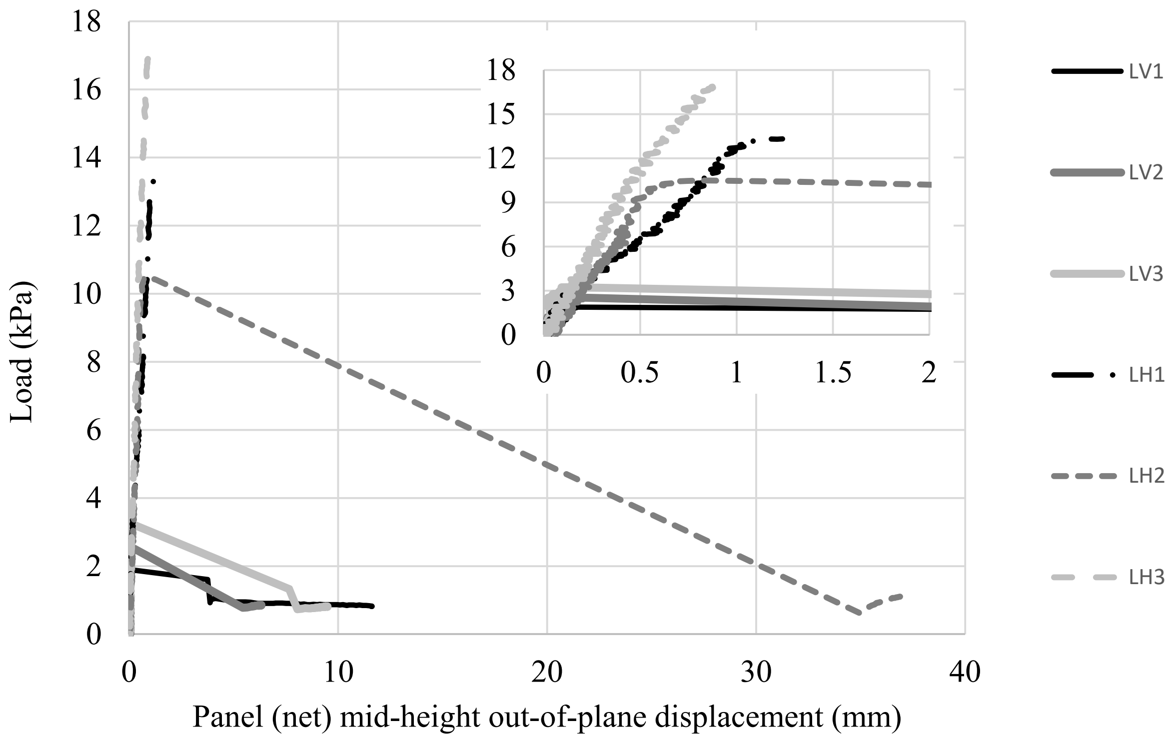
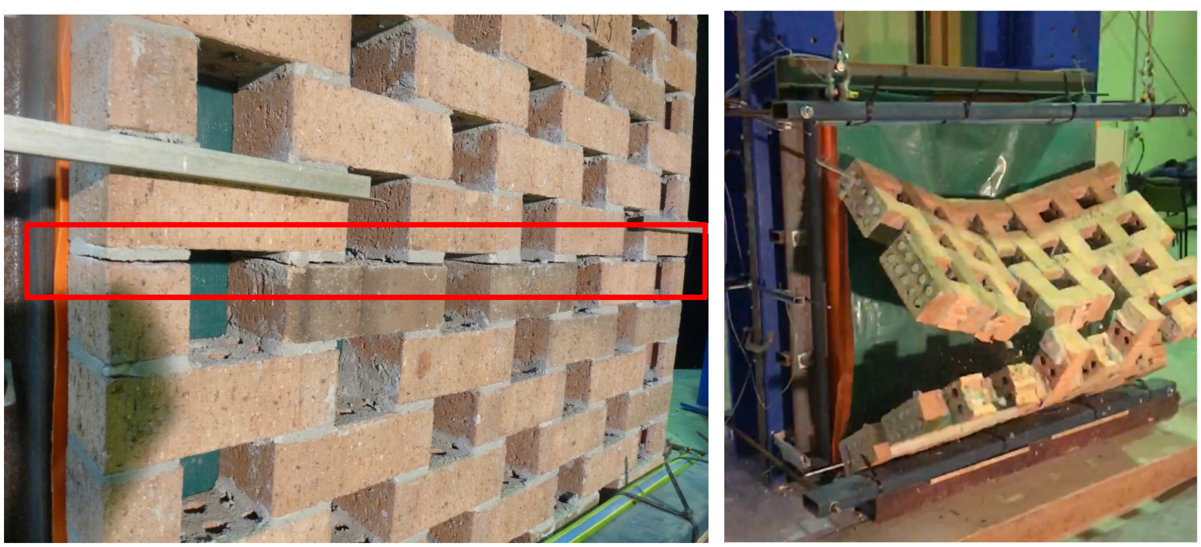


Figure 4: Load versus Displacement for all Specimens



(a) **(b)**

Figure 5: Observed Failure Modes for Specimens LV3 and LH3

(a) **(b)**

Figure 6: Observed Failure Mode for Specimen LH2

Results of AS3700 Strength Predictions

The panel strength predictions are summarised in Table 2. For the one way vertical bending (LV) specimens it was observed that the strength predictions based on the mean masonry flexural tensile strength over-predicted the experimentally observed panel strengths by 86% on average. This is not unexpected, as such brittle behaviour tends to be initiated by a weaker than average bond somewhere along the bedded length and cracking then propagates along the bed joint at an average stress lower than the mean material strength. This hypothesis is supported by Specimen LV3 for which the COV of masonry flexural tensile strength is much lower (Table 1) and the predicted panel strength is much closer to the experimentally observed strength than for Specimens LV1 and LV2.

The design strengths calculated for the LV specimens using code default values for characteristic strengths and capacity reduction factor are consistently below the experimentally observed strengths (mean ratio of predicted to observed strength of 0.34) showing that the AS3700 provisions are conservative for the specimens tested in this study.

For the one way horizontal bending (LH) specimens, the strength predictions based on mean material properties under-predicted the experimentally observed panel strengths by between 47 and 60%. In all cases, the predicted failure mode was stepped failure through the mortar joints (that is, Equation 2 returned a smaller value than Equation 3). However, as noted in Table 2, the panels exhibited mixed failure modes, with only LH2 displaying exclusively joint failures, while LH1 and LH3 included both joint and unit ruptures in their failure surfaces. This tends to indicate that the AS3700 provisions may underestimate the torsional shearing strength associated with stepped failure through the joints. Clearly, further testing is needed to confirm this hypothesis.

It should be noted that the AS3700 provisions for horizontal bending specifically require that the masonry be constructed with all perpend joints completely filled and therefore lattice masonry falls outside the scope of AS3700 for horizontal bending. The presence of filled perpend joints in a stepped failure mode is expected to significantly impact the behaviour compared to the lattice masonry tested in the current study. Resistance would be provided by the perpend joints during horizontal bending and the presence of filled perpends would potentially shift the centre of rotation for torsional shearing on the bed joints compared to lattice masonry. It was expected by the authors that the absence of perpend joints in the tested panels would result in Equation 2 over-predicting, rather than under-predicting the experimentally observed strengths. The differences in behaviour between fully bonded and lattice masonry, and hence the suitability (or otherwise) of Equation 2 as a model for stepped failure in lattice masonry is the subject of further studies by the authors.

However, for line failure through the masonry units, the use of $Z_p = 0$ in Equation 3 results in a rational model for the lattice masonry panels tested in this study. The predicted strength using Equation 3 for the LH specimens is w_{pred} = 16.9 kPa which agrees very well with the observed strength of Specimen LH3 which failed predominantly via a line failure mode.

Design strengths for the LH specimens were very conservative, returning a ratio of predicted to observed strength of just 0.16, on average. Again, stepped failure was predicted to govern.

CONCLUSION

Six unreinforced lattice masonry panels were subjected to lateral out-of-plane pressure loading to assess the flexural strength and failure behaviour. Three panels were tested in one way vertical bending and three were tested in one way horizontal bending. The AS3700 provisions for solid masonry were used to predict the panel strengths and an assessment of the suitability of the provisions for the design of lattice masonry is made.

Panels subjected to one way vertical bending failed in a non-ductile mode via bed joint cracking which occurred suddenly at the peak load along a single course close to panel mid-height. Strength predictions determined using the provisions of AS3700 [1] using mean material strengths and capacity reduction factor of one, over-predicted the experimentally observed strengths. This is thought to relate to the influence of variability in the flexural tensile bond strength leading to crack initiation at an average stress lower than the mean bond strength. Design strengths calculated for the LV specimens using code default values for characteristic strengths and capacity reduction factor are consistently below the experimentally observed strengths showing that the AS3700 provisions are conservative for the specimens tested in this study.

Panels subjected to one way horizontal bending also displayed non-ductile failure modes with no observable damage prior to a sudden failure surface developing at peak load. The failure surface varied between specimens from stepped failure exclusively through the mortar joints in LH2, to a combination of stepped and line (fracture of the units) failure in LH1 and LH3.

For the one way horizontal bending (LH) specimens, the strength predictions based on mean material properties under-predicted the experimentally observed panel strengths and the predicted failure mode was stepped failure through the mortar joints. Strength prediction based solely on the model for line failure (Equation 3), showed closer agreement with the experimentally observed strengths. This tends to indicate that the AS3700 provisions may underestimate the torsional shearing strength associated with stepped failure through the joints (Equation 2). Design strengths for the LH specimens were very conservative, again with stepped failure predicted to govern. The differences in horizontal bending behaviour between fully bonded and lattice masonry, and hence the suitability (or otherwise) of Equation 2 as a model for stepped failure in lattice masonry is the subject of further study by the authors.

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