DESIGN OF EXPERIMENTAL PROGRAM AND PRELIMINARY FINITE ELEMENT ANALYSES OF UNREINFORCED MASONRY SHEAR WALL

Lewis J Gooch[1], Mark J Masia[2] and Mark G Stewart[3]

1. *PhD Student, Centre for Infrastructure Performance and Reliability, The University of Newcastle, University Drive, Callaghan, NSW, Australia, lewis.gooch@uon.edu.au*
2. *Professor, Centre for Infrastructure Performance and Reliability, The University of Newcastle, University Drive, Callaghan, NSW, Australia, mark.masia@newcastle.edu.au*
3. *Professor and Director, Centre for Infrastructure Performance and Reliability, The University of Newcastle, University Drive, Callaghan, NSW, Australia, mark.stewart@newcastle.edu.au*

Abstract

The use of masonry in historical and contemporary construction is common throughout Australia in a wide range of both structural and non-structural applications. However, despite this, relatively few studies have been undertaken to examine and to quantify the reliability associated with masonry structures and in particular: unreinforced masonry (URM) structures. As a result, our current limit state methods of masonry design have not been suitably calibrated, and the risk of failure of new and historical masonry structures is unknown. Current limit state design requirements are typically based upon the probability distributions of applied loads and structural resistances which are utilised to estimate the probability of failure. The ongoing research associated with this current study intends to quantify the probability of failure of URM shear walls subjected to the unfavourable load actions imposed under wind and earthquake events. To gain a detailed understanding of the behaviour of URM shear walls, a series of experimental tests will be performed on a range of structural wall configurations intended to target several distinct failure modes that URM shear walls are susceptible to. The paper details the design of the experimental testing program through the use of preliminary stochastic and deterministic finite element modelling.

Keywords: experimental program design, shear walls, FEA, unreinforced masonry

1 Introduction

Unreinforced masonry (URM) is among the most used construction material, particularly in residential structures, in Australia. This popularity is due in part to the simplicity of construction, the durability and insulative qualities of the material and the aesthetic appeal of masonry structures. Despite these benefits, URM structures maintain a high risk of damage and collapse when subjected to adverse loading conditions. Due to masonry's low tensile strength and high mass and stiffness, lateral loads induced under seismic actions are particularly detrimental to the stability of URM structures. This susceptibility to collapse was manifest during the 1989 Newcastle earthquake, during which masonry structures suffered severe and extensive damage despite the moderate severity of the seismic event. The design and construction of stiff shear walls is a common means of resisting the laterally aligned forces imposed during seismic and wind events.

A building's shear walls are typically designed to resist both the vertical gravity loads imposed by the floors of a structure, as well as any lateral loading applied through in-plane shear actions. Due to the essential nature of these elements, their behaviour and reliability are of critical importance in reducing the risk of collapse of URM buildings. Recent studies of the reliability of URM structures have focused on the effects of spatially variable material properties, supplemented with the application of detailed computational methods of estimating a structure's load resistance (Stewart & Lawrence, 2002; Lawrence, 2009; Müller, et al., 2017). Expanding upon this research, the current study intends to produce an experimental program through which a greater understanding of the load-displacement response of URM shear walls may be achieved, as well as providing a detailed baseline that will facilitate the calibration of subsequent numerical analyses.

This research will focus on the behaviour of contemporary URM structures and as such will be performed in the context of our current masonry design standards (Standards Australia, 2018). As such, the use of outdated mortar compositions, as well as the potential for the degradation of masonry units and mortar joints has not been considered. Through the use of preliminary finite elements analyses (FEAs), laboratory specimens intended to capture the various failure mechanisms of URM shear walls have been designed. Furthermore, while the use of a greater range of structural configurations would certainly aid in the calibration of future studies, the focused set of configurations presented in the following section allows for a greater number of replicates to be used to enable the estimation of meaningful statistics of the mean and variance of test results – this may be termed Monte-Carlo laboratory testing. This ensures that a greater level of accuracy may be achieved in examining the behaviour of each distinctive configuration.

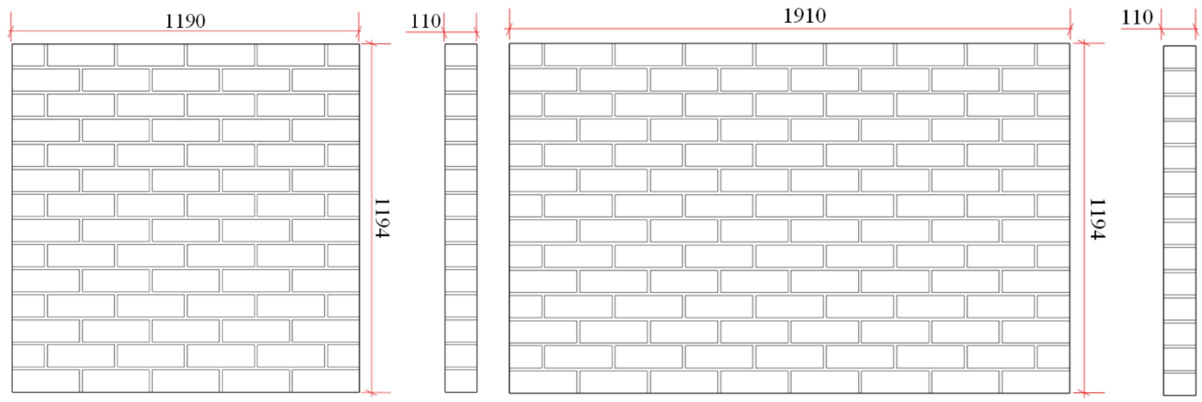
2 Wall Geometries, Materials and Load Configuration

The experimental program outlined in the current study was designed such that the examined structures would tend towards distinct failure mechanisms. In the case of URM shear walls, there is a wide range of unique failure modes making the design of an experimental program somewhat challenging. To simplify this process, these various failure mechanisms were divided into three more general modes that could be readily targeted by adjusting the structure's aspect ratio, boundary conditions and applied pre-compression. These modes were: (i) flexural/rocking failure, (ii) bed joint shear sliding, and (iii) diagonal tensile cracking. Generally, URM shear walls with greater height-to-length ratios, lower levels of pre-compression, and little to no restraint against in-plane rotation (cantilevered walls), tend to experience a flexural failure. Conversely, those structures with low height-to-length ratios subjected to higher levels of pre-compression and fixed against in-plane rotation are more susceptible to a diagonal tensile failure. Finally, a shear sliding failure may occur as an intermediate failure mechanism between to these two more extreme modes.

2.1 Specimen Geometry

Three sets of twelve, single wythe, half-storey URM wall specimens will be constructed in the University of Newcastle (UON) laboratory. Each of these sets will include ten specimens to subjected to a cyclic in-plane, pseudo-static loading, and two specimens to be deconstructed sequentially at the minimum and maximum curing times of the loaded walls to capture the spatial variability in material properties induced from variations in specimen age. The first set of walls will maintain an aspect ratio of approximately 1.0, as noted in Figure 1(a). This slender wall geometry will tend to favour a flexural failure mechanism, as noted above. The second and third sets of walls will adopt an aspect ratio of approximately 0.625, as noted in Figure 1(b), increasing the likelihood of a shear sliding or diagonal tensile failure mode. The failure mode of these sets will be distinguished through differing boundary conditions and degrees of pre-compression.

While the use of multiple wythes of masonry is common (Russell, 2010), particularly in load bearing masonry structures, the current study has adopted single leaf structures for all examined wall configurations. This was implemented in order to reduce the expense and time require to construct each specimen as well as to mitigate overloading of the horizontally aligned hydraulic jacks use to apply the cyclic loading. Furthermore, due to the plane stress nature of in-plane shear loading, a reduction in wall thickness should have no effect on the response of each structure other than a proportional increase in failure load.



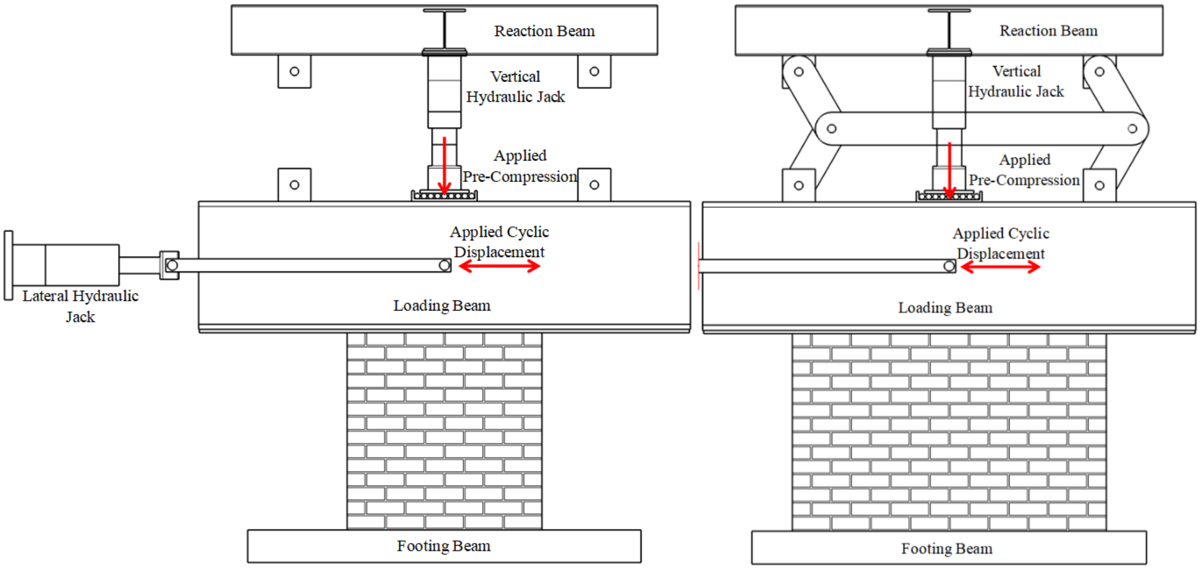
(a) Aspect ratio = 1.0 (b) Aspect ratio = 0.625

Figure 1: Proposed geometry of URM wall specimens (all dimensions are in mm).

2.2 Boundary Conditions

As with the geometries of the proposed sets of URM wall specimens, the boundary conditions may be adjusted to better target the specific failure modes of interest to this study. Each of the three sets of walls will be constructed upon a reinforced concrete footing beam to facilitate the movement of specimens into the testing apparatus. While all wall specimens will be loaded with a fixed pre-compression, only the first set of walls will be released against in-plane rotation, i.e., cantilevered. The second and third sets of walls shall be restrained against in-plane rotation, simulating a double fixed or coupled wall. This restraint significantly reduces the likelihood of flexural cracking, and thus favours the shear modes targeted by the second and third sets. Furthermore, this shift in failure mechanism is achieved without significantly increasing the required pre-compression, mitigating the chance of overloading the hydraulic jacks.

In order to achieve this fixity, the steel truss system initially proposed by Bosiljkov et al., (2008), and shown in Figure 2, will be utilised. The use of a steel pantograph ensures that any vertical movement that occurs at one end of the loading beam is matched at the other end, effectively eliminating any in-plane rotation.

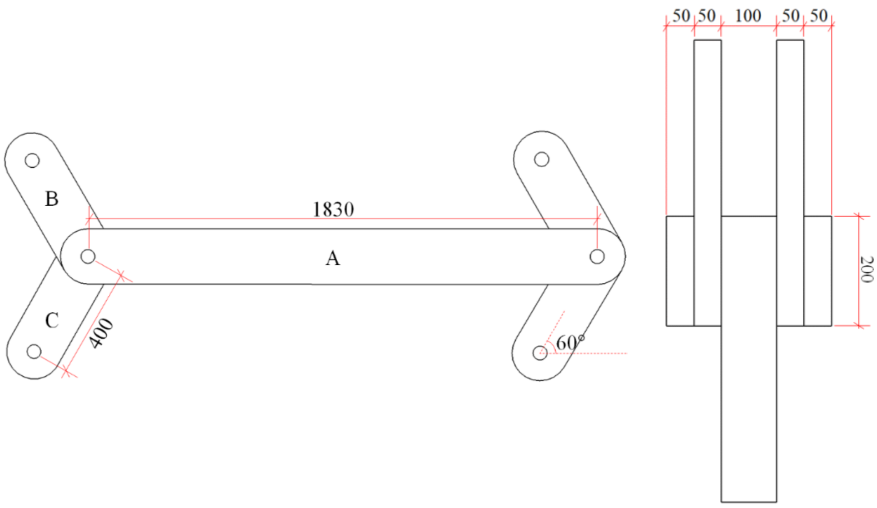


(a) (b)

Figure 2: Proposed restraint systems: (a) rotationally released (b) fixed against in-plane rotation, after Bosiljkov et al., (2008).

The dimensions of the truss arrangement proposed for the current study is shown in Figure 3. Each of the three types of truss element: A, B, and C, are 200 mm in depth, with the varying widths shown in Figure 3(b). The use of paired 50 mm central and upper linkages facilitates a large enough gap between the truss members to permit the vertical jack to contact the loading beam, as well as ensuring that the boundary conditions are symmetrical about the axis of the wall.

Finally, a shear stop is to be installed at the bottom course during the testing of the third and final set. The horizontal displacement restraint will serve to reduce the specimens' tendency toward shear sliding at the interface between the masonry wall and the footing beam. While this failure mode may still occur at any of the courses above the first, the stop will act to somewhat mitigate a mixed sliding and diagonal tensile failure. As the configurations of the second and third sets are quite similar, the presence of this additional restraint may be necessary to prevent this mixed mode failure. The details of this restraint are still to be determined but will likely take the form of a steel member affixed to the floor in-line with the axis of each wall specimen.



(a) (b)

Figure 3: Dimensions of steel truss arrangement (all dimensions are in mm).

2.3 Construction Materials

The materials chosen for the construction of each of the wall specimens were selected to be consistent with those commonly found in URM buildings in Australia. Furthermore, while some control over the mechanical properties of the chosen material may be exerted through the use of weaker or stronger masonry units and higher or lower classes of mortar, the more specific adjustments to the behaviour afforded by changes to the geometry or boundary conditions cannot be achieved through these general modifications. As such, the current study will adopt similar masonry units and mortar composition throughout all three sets of tests.

The masonry units utilised in the proposed experimental program will be of fired, extruded clay bricks of the common available dimensions: 230mm × 110 mm × 76 mm. Fired clay bricks of these dimensions are Australia's most widely used external wall cladding material (Inglis & Downton, 2013), with cored, extruded units being more common than the pressed or moulded varieties.

Similar to the prevalently used clay units, a common mortar mix is proposed throughout the construction of all wall specimens. The M3 class mortar mix, 1:1:6 (GP cement: lime: sand, by volume) is one of the most commonly used mortar mixes in residential construction in Australia. This mortar is of an intermediate strength and, with an appropriate water content, is of good workability. Preliminary testing of this mortar mix across a variety of units types place the characteristic flexural tensile strength between 0.1 MPa and 0.7 MPa. Given the inherently high variability of mortar strength, this range of strengths is not inconsistent with the recommended 0.2 MPa presented by AS 3700 (2018).

2.4 Load Configuration

The pre-compression load, as with the specimen geometry and boundary conditions, plays an important part in determining the failure mechanism of URM shear walls. The various pre-compression loads selected in this study are based on both the utility of achieving these modes of interest, but also in simulating realistic axial stresses experienced in masonry buildings. A wall located in the higher levels of a structure will be subjected to lesser compressive stresses than one at the ground floor. As such, a pre-compressive stress of 0.5 MPa will be utilised in the first and second testing sets. This level of stress, determined through trial-and-error FEAs, is representative of a wall with two or three storeys above. A greater stress of 1.0 MPa will be adopted in the third set in order to achieve a diagonal tensile failure mode. While this level of pre-compression is somewhat high, perhaps representative of a URM structure of five or more storeys, a greater level of axial compression is a key characteristic of a diagonal tensile failure.

Lateral loading of the specimens will be applied as a quasi-static, cyclic, displacement-based loading. Reversing cycles of displacements with increasing amplitudes and displacement rates are proposed, similar to that adopted by Howlader et al., (2018). Each displacement amplitude shall repeat three times each in the positive and negative direction prior to an increase in the amplitude of displacements. These sinusoidal waves shall continue up to a peak in-plane displacement of ±24 mm (or ±2.0% drift). The proposed cyclic loading history is consistent with the recommendations of Tomaževič (1999) and is presented in Table 1 and Figure 4, below.

Table 1: Proposed displacements and displacement rates.

Displacement (mm)	Drift (%)	Rate (mm/s)	Displacement (mm)	Drift (%)	Rate (mm/s)
0	0	0	5	0.42	0.080
0.25	0.02	0.004	6	0.50	0.096
0.5	0.04	0.008	7	0.59	0.112
1	0.08	0.016	8	0.67	0.128
1.5	0.13	0.024	10	0.84	0.160
2	0.17	0.032	12	1.01	0.192
2.5	0.21	0.040	16	1.34	0.256
3	0.25	0.048	20	1.68	0.320
4	0.34	0.064	24	2.01	0.384

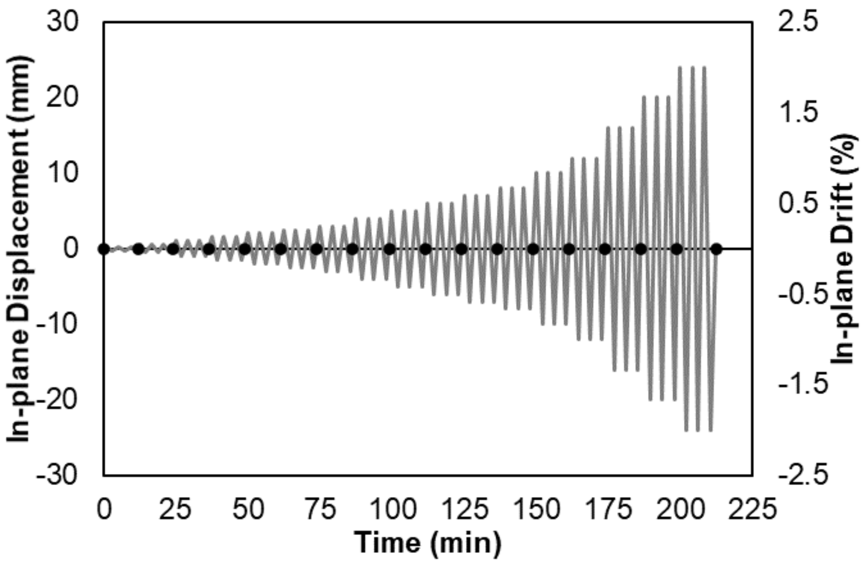


Figure 4: Proposed quasi-static time-displacement history.

As presented in Figure 2, pre-compression loading shall be applied at the top of the loading beam. This will allow for the locally applied load to distribute along the length of the specimen, resulting in a largely uniform compressive stress equal in magnitude to that intended for each test. Lateral, cyclic loading shall be applied through a steel loading arm to the centre of the loading beam. This central point shall be used as a displacement control point in order to monitor and adjust the magnitude of each quasi-static cycle. The location of this lateral displacement is representative of the movement of a supported floor undergoing movement during a seismic event.

While the application of a quasi-static loading scheme is less representative of real seismic loading, the utilisation of quasi-static loading allows for the accurate measurement of many of the key performance parameters of real structures subjected to seismic loads (Howlader, et al., 2018). The use of this loading scheme facilitates an accurate recording of the load-displacement behaviour of each structure that may be readily compared to a numerical simulation. Furthermore, the slower and more stable loading method allows for the application of DIC imagery, allowing for detailed strain fields and crack patterns to be determined for each of the examined structures.

3 Preliminary Numerical Modelling

Preliminary numerical modelling of the proposed wall specimens has been performed using the finite element modelling package DIANA 10.3 (DIANA FEA, 2019). A simplified micro-modelling approach, as outlined in Lourenço (1996), has been adopted as previous applications of this modelling method produced a balance between the accuracy and computational expense of each simulation (Howlader, et al., 2020; Gooch, et al., 2021). The continuum of masonry units was modelled using triangular, six noded, plane stress elements, CT12M (DIANA FEA, 2019), with a membrane thickness of 110 mm. As shown in Figure 5, mortar joints were not explicitly modelled, rather the interface between units and mortar were analysed through the use of quadratic, one-dimensional (CL12I), combined cracking-shearing-crushing elements (DIANA FEA, 2019). Furthermore, similar elements utilising a discrete cracking model were utilised in order to represent the local failure of masonry units. The material models adopted in these elements comprise the portions of the applied models capable of capturing the non-linear response of the examined structures.

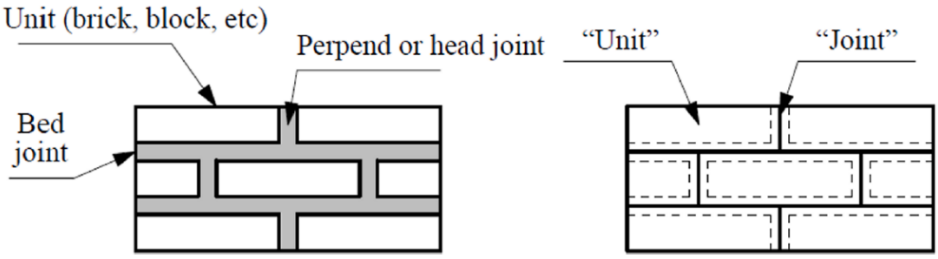


Figure 5: Modelling method utilised in the simplified micro-modelling approach (Lourenço, 1996).

3.1 FEA Results

Calibration of the material models of each simulation was achieved through the adoption of mean material parameters consistent with the recommendations of AS 3700 (2018), as well as the various literary sources outlined in Gooch et al., (2021). Each simulation was performed by first applying the pre-compression load at the top of the loading beam in a single load step, then maintained throughout the simulation. Next, in-plane displacements were gradually applied with step sizes of up to 0.1 mm until the maximum drift of 24 mm or collapse of the structure was achieved.

These FEA simulations were performed for the following three sets of in order to verify the intended failure mechanisms and target peak in-plane shear capacities:

1. In-plane rotational restraint is released, aspect ratio of 1.0, and applied pre-compression of 0.5 MPa.
2. In-plane rotational restraint fixed, aspect ratio of 0.625, and applied pre-compression of 0.5 MPa.
3. In-plane rotational restraint fixed, aspect ratio of 0.625, and applied pre-compression of 1.0 MPa.

The load-displacement behaviour of each of these simulations is presented in Figure 6.

Significant post-peak softening was observed in both the second and third simulations, as expected of a shear sliding and diagonal tensile failure mode. The first simulation experienced very little reduction in the strength after a peak was achieved, this is characteristic of a rocking failure which is subject more to the geometry and pre-compression of the structure rather than the non-linear material properties (Gooch, et al., 2021).

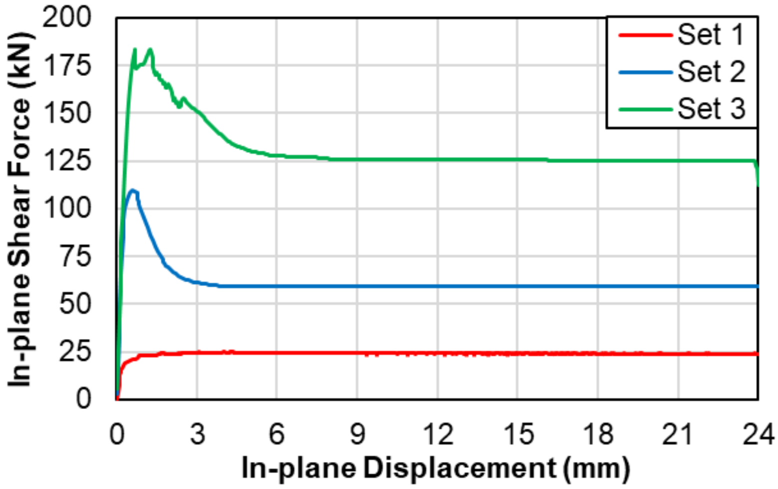
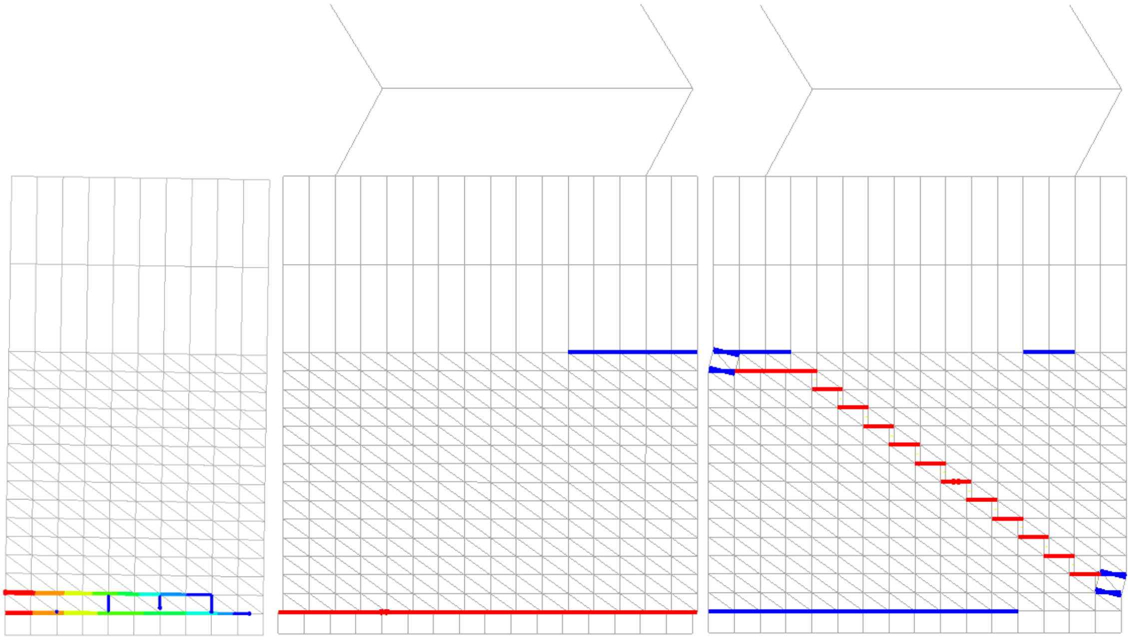


Figure 6: Load displacement response of the proposed URM wall specimens.

In addition to the load-displacement behaviour of each simulation, the deformed shape and predicted cracking patterns were analysed as these provide a clear indication of the expected failure mechanism of each wall specimen, as may be seen in Figure 7. As expected, the heel uplift observed in Set 1 is characteristic of flexural/rocking failures. Furthermore, the cracking of multiple courses at the base of structure is common among flexural failures as tensile stresses are redistributed as the structure beings to rock. The horizontal sliding at the base of specimen observed in Set 2 is the simplest form of a shear sliding failure, with peak strength governed by the Mohr-Coulomb failure criterion specified in DIANA 10.3 (2019). Finally, Set 3's results show a clear stepped failure through both bed and perpend joints. While this is a common form of a diagonal tensile failure in structures with low mortar strength relative to the strength of the masonry units, a combination of failures through both mortar joints and masonry units may also be observed in URM shear walls.



(a) Set 1 (b) Set 2 (c) Set 3

Figure 7: Predicted crack patterns for proposed wall specimens.

Additionally, the peak load resistance of each simulation has been compared to a code predicted value from the current Australian masonry standard (Standards Australia, 2018) and from the New Zealand technical guidelines for the seismic assessment of URM buildings

(NZSEE, 2017), as shown in Table 2. As an anticipated research outcome of this study is to produce more accurate methods of estimating the failure load of URM shear walls, as well as to evaluate the reliability of current capacity reduction factors, comparison to current design methodologies is an essential step in future application of this research.

Table 2: Comparison of standard and FEA peak load resistance predictions.

Specimen	FEA (kN)	AS3700:2018 (kN)	NZSEE:2017 (kN)
Set 1	25.1	20.1 (-20%)	34.1 (+36%)
Set 2	109.4	136.7 (+25%)	169.7 (+55%)
Set 3	183.6	168.3 (-8%)	287.3 (+56%)

It may be observed here that, particularly in the case of NZSEE (2017), the code predictions typically overestimate the failure load when compared to a numerical prediction. This non-conservatism is somewhat offset by the application of the capacity reduction factors and the use of characteristic values for material strengths, neither of which have been utilised in the above results.

4 Conclusions

This paper proposes an experimental research program for the cyclic, in-plane shear loading of URM walls. These results will form an accurate baseline from which subsequent applications of FE modelling may be compared. This will facilitate the analysis of a wide range of URM configurations without the need for excessive and expensive full-scale laboratory testing.

5 Acknowledgements

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