15th Canadian Masonry Symposium
Ottawa, Canada
June 2-5, 2025

A Laboratory Investigation of the Variability of the Coefficient of Friction in Unreinforced Clay Masonry

Lewis J. Gooch[i], Mark J. Masia[ii], Mark G. Stewart[iii], and Michele Spadari[iv]

ABSTRACT

The frictional resistance along a shear plane contributes significantly to the load carrying capacity of masonry structures, particularly those loaded under in-plane lateral forces. This resistance is typically defined simply in terms of a shear-friction coefficient. This property ranges significantly in the literature, with values observed from as low as 0.25 to greater than 1.0 for unreinforced clay-brick masonry. Despite this degree of uncertainty, few studies consider the variability of the shear-friction resistance. This limitation is particularly relevant to structural reliability-based analyses, where the result is highly dependent upon assumed variability of strength defining material parameters. This manuscript presents the findings of a laboratory investigation of the statistics of this shear-friction behaviour through the application of repeat testing of nominally identical specimens. Five distinct clay-brick masonry unit types (three extruded, perforated units, and two pressed, solid units) were utilised, with an Australian standard 1:1:6 (cement: lime: sand, by volume) mortar mix. Masonry couplets were constructed from these materials and were tested in shear under a constant vertical pre-compression. The data produced from these tests allowed for the determination of suitable statistical models to be developed to describe the variability of the shear-friction coefficient.

KEYWORDS

clay brick masonry, material characterisation, shear-friction, unreinforced, variability

[i] Research Associate, The University of Newcastle, Callaghan, Australia, & University of Technology Sydney, Ultimo, Australia, lewis.gooch@newcastle.edu.au
[ii] Professor, The University of Newcastle, Callaghan, Australia, mark.masia@newcastle.edu.au
[iii] Distinguished Professor, University of Technology Sydney, Ultimo, Australia, mark.stewart@uts.edu.au
[iv] Senior Technical Officer, The University of Newcastle, Callaghan, Australia, michele.spadari@newcastle.edu.au

INTRODUCTION

The frictional resistance of mortar joints contributes significantly to the in-plane shear capacity of unreinforced masonry (URM) structures. This resistance is typically characterised by a friction coefficient, μ_f, that relates the compressive normal force acting on a mortar joint to a lateral shear resistance. The property μ_f is often considered in the predictive models of the ultimate in-plane shear capacity of URM, such as those presented by [1-6]. However, although this property has been observed to vary significantly, ranging in magnitude from 0.25 to 1.20 [7-9], few studies consider the statistical properties of μ_f. This limitation may result in high and non-conservative values of a nominal friction coefficient being adopted in design standards (cf. [3-6], where the nominal value of μ_f ranges from 0.25 to 0.45); i.e.: a higher friction coefficient will result in a higher design capacity. However, if the mean true (or test) friction coefficient is low (or sufficiently variable) the design resistance may exceed the true resistance of a URM structure. Furthermore, a failure to consider an accurate probabilistic model of μ_f when performing a structural reliability analysis may result in a non-conservative estimate of the risk associated with URM subject to lateral loading. It should also be noted that the values of μ_f specified in [3-6] have not been determined as a characteristic value (i.e.: as the 0.05-fractile of a statistical model), but rather have been determined as a conservative lower-bound fit of a Mohr-Coulomb model to existing URM shear wall resistance data [10] or from a conversion from existing allowable stress design [6].

The potential non-conservatism associated with the current deterministic models for μ_f are not as significant for the design of new URM shear walls subject to simplified monotonic loading conditions, as the presence of a shear bond component of resistance (a property that is reasonably well characterised in the literature and typically contributes >50% of the lateral load resistance) reduces the impact of any non-conservatism associated with μ_f. However, for existing URM structures or those subject to cyclic in-plane loading conditions – such as under wind and earthquake loading conditions – the shear failure surface at the ultimate limit state will typically consist of cracked mortar joints. In this situation, the frictional resistance is the only contributor to the lateral load resistance. This is evident in the recent assessment of the model accuracy of the predictive equations of cyclically loaded URM shear wall resistance by [11] that found that the standardised predictions of shear sliding resistance in [3-6] were too high and non-conservative. While the study by [11] suggests that the observed non-conservatism in shear-based failure predictions primarily stems from failures to account for the presence of mortar joint cracking, his non-conservatism may be mitigated by the adoption of a lower nominal friction coefficient, that reflects the statistical properties of this material parameter. As such, understanding μ_f in probabilistic terms is essential part of accurately characterising the structural reliability of URM subject to cyclic lateral loading.

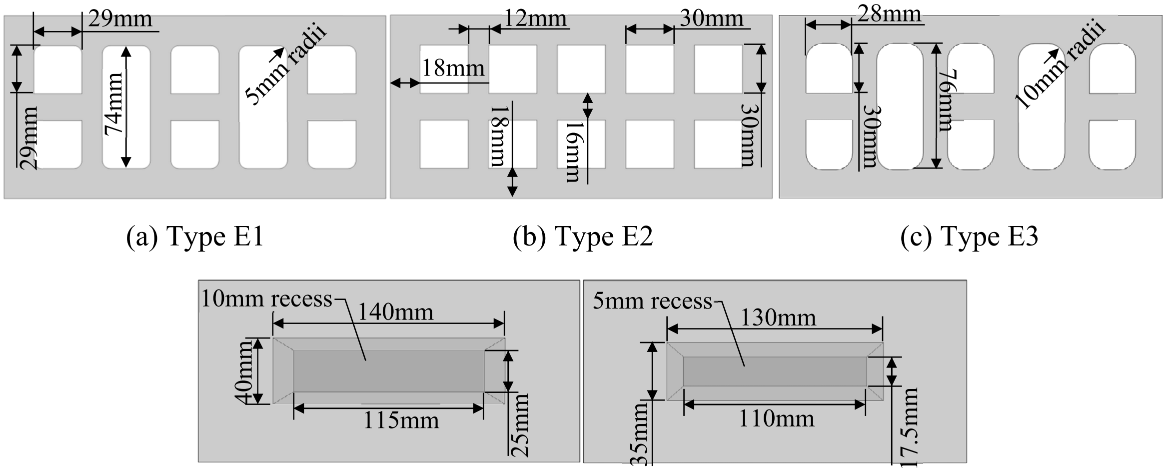
The current study, therefore, presents the results of an on-going investigation to define the friction coefficient of the interface between mortar and clay-brick masonry in probabilistic terms. This has been performed through the experimental testing of a number of distinct masonry couplet types, including both extruded (perforated) and pressed (frogged) masonry units.

METHODOLOGY

Masonry Specimens

Due to the expected influence of the unit perforations/frogs via mortar plug interlocking and bonded area, shear tests were performed on five different masonry unit types – three extruded and two pressed units: denoted by E and P, respectively. The perforation patterns and frog geometries of these units are presented in Figure 1. Furthermore, each unit type maintained the same overall dimensions of 230 mm × 110 mm × 76 mm (length × width × height), consistent with standard Australian brick sizes.

The standardised 1:1:6 (cement: lime: sand, by volume) mortar mix (or approximately 1:0.5:7.6, by mass) was used for all masonry specimens tested in this study. Additionally, all specimens were aged beyond 28-days prior to testing.



(a) Type E1 (b) Type E2 (c) Type E3

(e) Type P1 (f) Type P2

Figure 1: Masonry Unit Geometries.

Testing Set-ups

The Australian standards for masonry do not specify a standardised method for testing URM under shear. Furthermore, the triplet method provisioned by [12] was deemed unsuitable to the current study, as the failure of a single mortar joint was required in order to accurate quantify the friction coefficient of the unit-mortar interface. Initially, the testing method devised by [13] was adopted. In this arrangement, shear loads were applied concentrically with the mortar joint through the use of "L-shaped" brackets that could be gripped by a universal testing machine (see Figure 2(a)). Steel plates were first affixed to the couplet via a high strength epoxy. The brackets could then be bolted to these plates before inserting the test specimen into the universal testing machine. Shear forces were applied to the couplet specimen at a rate of 1 mm/minute, while a constant normal force was applied using a hydraulic jack and load cell. Both the applied shear and normal forces were recorded during testing, allowing for the average friction resistance across the mortar joint to be estimated.

However, several limitations were observed during this initial pilot phase. The primary issue encountered was that, despite the use of a high strength epoxy adhesive, failure between the steel plates and masonry units was common and would result in no useable data being obtained from a test. Additionally, this epoxy was required to cure for a minimum 24-hour period (and up to 72-hours), significantly reducing the efficiency of testing due to a finite number of suitable steel plates. To address these limitations, the final testing arrangement shown in Figure 2(b) was developed.

This testing arrangement was developed based on the testing method presented by [14] and considering the discussions by [15]. In this testing arrangement, the couplet was placed into a shear box and packed with steel plates and plywood such that the bottom brick was in contact with the moveable shear box component of the testing arrangement, while the top brick was contacting the rigidly restrained box. Shear loading was then applied at a rate of 1.14 mm/minute, while constant normal forces were again applied using a hydraulic jack. A spherical seat was also utilised when packing the bottom brick in order to ensure that full contact to the head of the brick was maintained in the event that the two bricks were misaligned.

The applied displacement to the bottom brick introduced a shear force into the unit-mortar interface, and into the top brick. This force was recorded as the top brick is rigidly restrained against horizontal displacement by the rigid box shown in Figure 2. This box subsequently transferred the applied shear force into a load cell on the opposite side of the specimen, to which it is rigidly connected, via tension loading.

While this testing apparatus addressed the issues encountered in the initial pilot study, a limitation of this arrangement is the non-uniformity of normal stresses along the mortar joint due to the offset between the applied resultant shear force and the opposing lateral restraint. However, it is noted by [15] that this testing arrangement performs well in terms of both shear stress distribution and average normal stress.

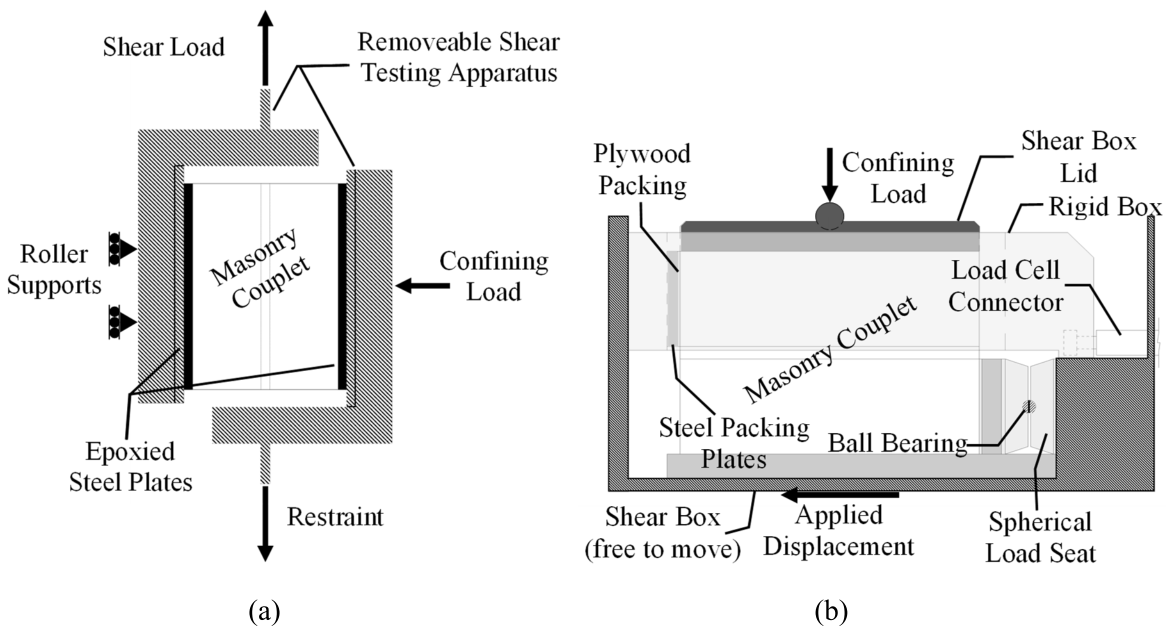


Figure 2: Experimental Testing Arrangements.
(a) Initial Pilot Arrangement, and (b) Final Test Set-up.

EXPERIMENTAL RESULTS

As this investigation is on-going, the results presented in the subsequent sections should be considered as preliminary. Further testing of each masonry type, as well as of solid units (without perforations or a frog), are underway at The University of Newcastle, Australia.

As the shear bond strength of the examined masonry couplets was not known, the frictional component of the peak shear resistance cannot be readily determined. As such, the friction coefficient quantified in this study was that determined once a stable residual shear load was observed experimentally. An example of this derivation of the friction coefficient for each test is shown in Figure 3.

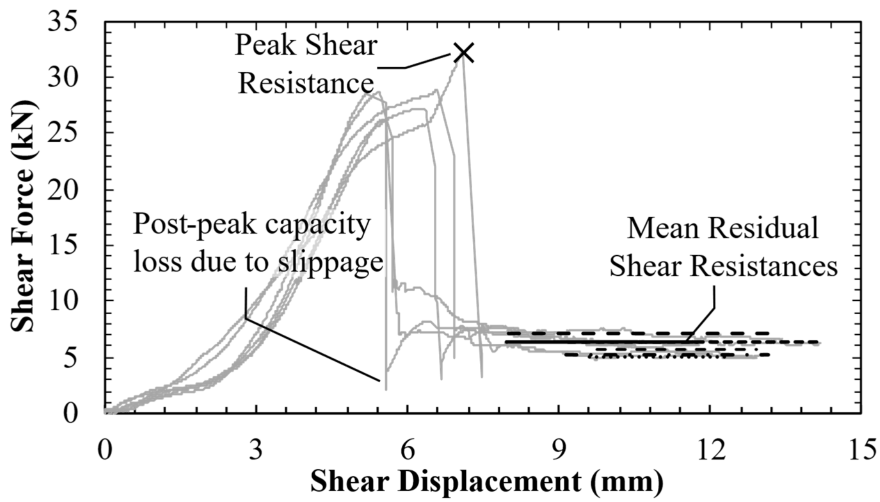


Figure 3: Example derivation of residual shear resistance.

Furthermore, the findings of [8] indicate that this residual friction coefficient is often not significantly distinct from that relevant to the peak shear capacity, and as such it may be expected that a probabilistic model of the residual friction coefficient would be accurate for the prediction of the peak friction coefficient as well. The residual friction coefficients, as well as the corresponding applied normal forces, P_n, for each specimen type are summarised in Table 1.

Table 1: Results of shear couplet tests.

Specimen Type	Sample Size	Mean Normal Force, P_n (kN)	Mean Residual Friction Coefficient, μ_f
E1	7	5.55 [0.06]	1.15 [0.19]
	7	15.18 [<0.01]	0.82 [0.08]
	7	25.30 [<0.01]	0.60 [0.12]
E2	7	5.15 [0.02]	1.12 [0.02]
	7	15.16 [<0.01]	0.98 [0.07]
	6	25.28 [<0.01]	0.79 [0.07]
E3	10	5.28 [0.04]	1.11 [0.09]
	9	15.13 [<0.01]	0.95 [0.05]
	9	25.30 [<0.01]	0.80 [0.13]
P1	6	5.49 [0.08]	1.07 [0.14]
	6	15.11 [<0.01]	0.95 [0.15]
	5	25.29 [<0.01]	0.88 [0.11]
P2	7	5.46 [0.07]	1.08 [0.13]
	6	15.19 [<0.01]	1.01 [0.10]
	5	25.33 [<0.01]	0.95 [0.08]

Note: coefficients of variation (COVs) are shown in [ ].

Normal Stress Dependence

As is evident in the results shown in Table 1, the friction coefficient of each brick type was observed to be significantly dependent upon the amount of applied normal force. The determined residual friction coefficients for each shear test have been plotted against the applied normal stresses in Figure 4. Here it may be seen that the friction coefficient decreases at a rate between 0.4 MPa^{-1} and 0.7 MPa^{-1} for extruded masonry, and approximately 0.15 MPa^{-1} for pressed.

This observed relationship is highly relevant for the design and structural reliability of URM. While the mean values of μ_f presented in Table 1 are notably greater than the standardised nominal values specified by [3-6], even at higher pre-compressions, the nominal values may no longer be suitable to satisfy structural reliability requirements (such as those presented by [16, 17]), as the observed variability of friction coefficient, as well as a lower mean value, may significantly reduce the lower limit of structural resistance relative to applied loads [18]. This is particularly relevant for cracked or cyclically loaded URM walls subject to shear-based where the pre-compression is typically larger, and the frictional resistance governs the ultimate limit state capacity.

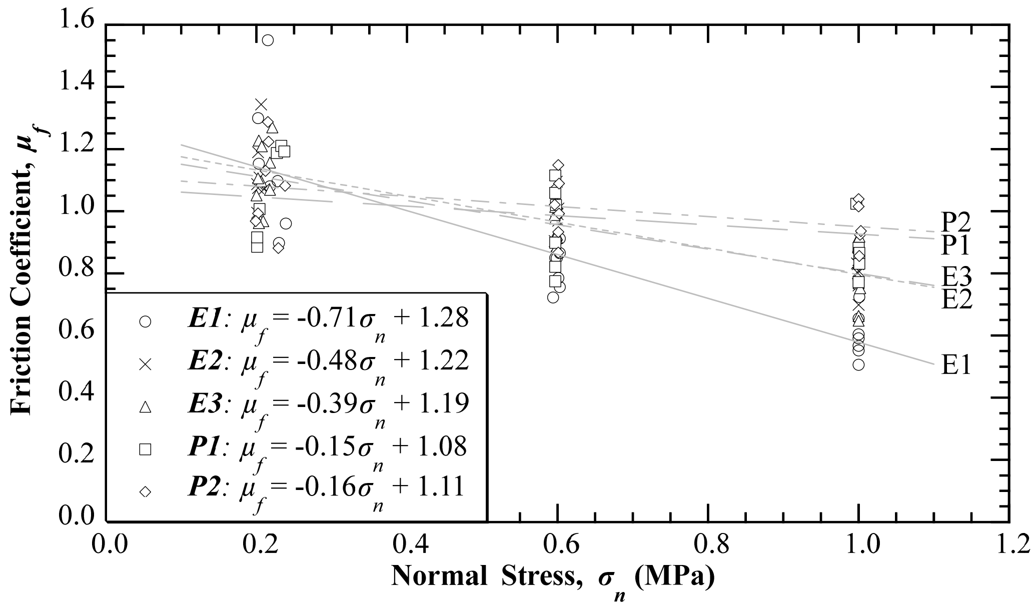
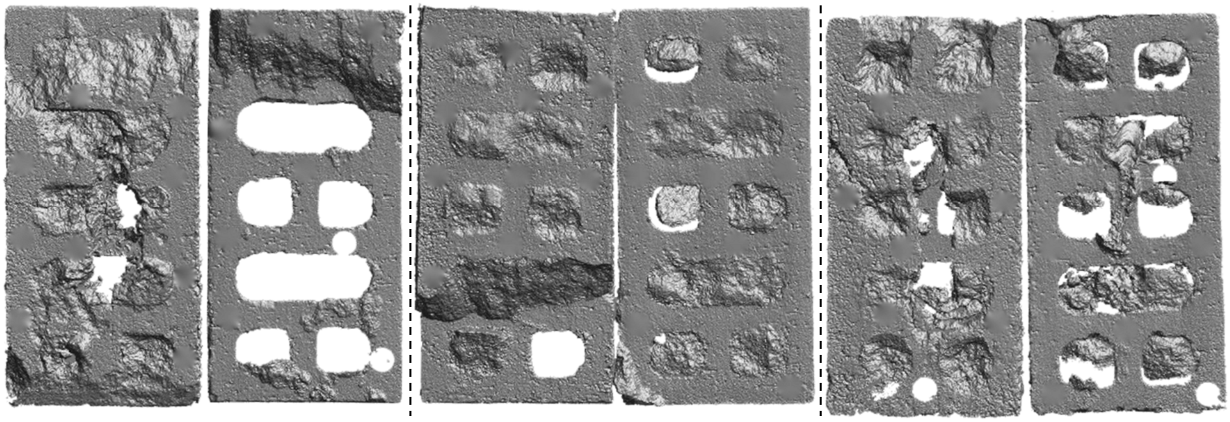


Figure 4: Observed relationship between friction coefficient and applied normal stress.

It is currently hypothesised that this dependence of friction coefficient on normal stress is due to the influence of dilatancy, i.e.: shearing that occurs along an microfacet (inclined at the dilatancy angle) that increases the observed angle of shearing [19]. At a relatively small applied normal force (such as the 0.2 MPa equivalent gross stress applied during testing), it would be expected that the sliding zone of the masonry couplets could articulate over these facets more readily than at high levels of pre-compression. The additional restraint against vertical displacement imposed during the 0.6 MPa and 1.0 MPa equivalent gross normal stress tests would result in an increased degree of crushing of these microfacets as the couplets cannot displace vertically to the same extent, ultimately degrading the surface roughness, and decreasing the dilatancy angle and observed angle of shearing.

In order to verify this hypothesis, 3D laser scans of the final failure surfaces of specimens tested at 0.2 MPa, 0.6 MPa and 1.0 MPa gross normal stress are underway. An example of these scans is shown in Figure 5. These scans allow for the roughness of each failure surface to be quantified and should indicate if any additional smoothing has occurred in specimens loaded under a higher normal force. The interpretation of these scans will be discussed in future studies.



(a) (b) (b)

Figure 5: 3D scan of shear failure surfaces for Type E3 couplets subject to gross normal stresses of (a) 0.2 MPa, (b) 0.6 MPa, and (c) 1.0 MPa.

Variability of Friction Coefficient

A key objective of this investigation is to define the variability of the friction coefficient of clay-brick masonry mortar joints, and to define this property in probabilistic terms. As may be seen in Table 1, the COV of μ_f has been observed to range from 0.02 to 0.19 for extruded masonry and 0.08 to 0.15 for pressed. While these results indicate that the friction coefficient has a relatively low variability in comparison to properties such as flexural tensile or shear bond strengths [20, 21], it is comparably variable to the compression strength of masonry (reported between 0.10 and 0.15 [4, 22, 23]). As such, this variability is significant in applications such as a structural reliability analysis of URM shear resistance.

The underlying cause of this variability may be attributable to a number of sources. Workmanship is likely to be a key source of the observed variability in the frictional resistance of URM, as it is with most masonry material properties. Specifically, inconsistent bedding and furrowing, disturbance to the bed joint after laying, mortar batch quality, etc. Another source of variability may be the distribution of aggregate within the mortar. While the same aggregate was used for all specimens (a washed beach sand with a fine to medium particle size; 100% passing at 2.36 mm), and all mortar was batched at the same volumetric proportions (1:1:6, cement: sand: lime), the water content of each mortar batch was not as closely controlled, nor were the batch mix times and mortar workability recorded. As such, it is likely that there was considerable spatial variability in the distribution of the relatively rough aggregate within each mortar joint. Finally, the presence of brick perforations in the extruded units (and, to a lesser extent, the frogs in the pressed units), likely contributed to the variability in the recorded friction resistances. As with improper furrowing, brick perforation may introduce voids in the sliding plane, resulting in uncontrolled variations in the shear resistance of a couplet.

The preliminary values of COV are presented in Figure 6. Here it may be observed that the pressed brick couplets exhibited a large COV at pre-compressions less than 1.0 MPa, however, the values of μ_f across all levels of normal stress (refer to Figure 4) are more consistent among the pressed brick couplets, resulting in the lower overall COV of 0.14; in comparison to the value of 0.22 for extruded brick couplets. Furthermore, a suitable probabilistic density function (PDF) that may be utilised to describe the statistics of μ_f is to be determined upon the completion of the outlined experimental testing program.

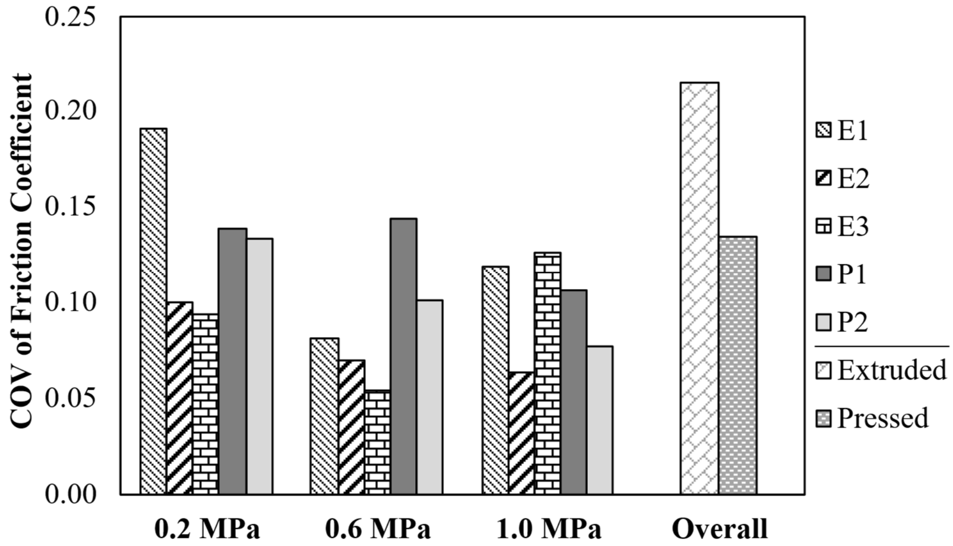


Figure 6: COVs of Friction Coefficient.

CONCLUSIONS

This paper presents an on-going experimental investigation of the variability of the friction coefficient, μ_f, of clay-brick masonry mortar joints. The preliminary results of this study indicate that the friction coefficient is comparably variable to compression strength of masonry, with a coefficient of variation ranging from 0.02 to 0.19. This suggests that probabilistic assessments (such as stochastic numerical models) and structural reliability analyses of unreinforced masonry shear walls should consider the μ_f as a random variable.

In addition to the results related to variability, it was found that the friction coefficient was highly dependent upon the amount of applied normal force. At 0.2 MPa gross equivalent normal stress, a value of μ_f ranging from 1.07 to 1.15 was determined. As the normal stress was increased to 0.6 MPa and 1.0 MPa, μ_f was found to decrease significantly. For extruded masonry bricks, μ_f decreased at a mean rate between 0.4 MPa^{-1} and 0.7 MPa^{-1}, while for pressed masonry, this rate was approximately 0.15 MPa^{-1}. This result is significant to the design of masonry shear walls as it indicates that the design value of μ_f depends not only on its mean and coefficient of variation, but also on the boundary conditions (pre-compression) of masonry structure under consideration.

Further experimental testing is on-going to verify the conclusions presented in this paper, as well as to define probabilistic models (in the form of probability density functions) to describe μ_f. These derived statistics may then be applied to future stochastic models of masonry, as well as to structural reliability analyses.

ACKNOWLEDGEMENTS

This research was funded by the Australian Research Council under Discovery Project DP220102758. The authors gratefully acknowledge the contributions of The University of Newcastle laboratory technical staff for their assistance on this project. The support provided by Think Brick Australia in providing brick units is gratefully recognised.

References

[1] Turnšek, V. and Čačovič, F. *Some experimental results on the strength of brick masonry walls*. in *Proceedings of the 2nd International Brick Masonry Conference*. 1970. Stoke-on-Trent.

[2] Magenes, G. and Calvi, G.M., *In-plane seismic response of brick masonry walls.* Earthquake Engineering and Structural Dynamics, 1997. **26**: p. 1091-1112.

[3] CSA, *S304-14: Design of masonry structures*. 2014, Canadian Standards Association: Mississauga.

[4] SAI, *AS3700: Masonry Structures*. 2018, Standards Australia: Sydney.

[5] CEN, *EN1996-1-1: Eurocode 6: Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures*. 2022, European Committee for Standardization: Brussels.

[6] TMS, *TMS 402/602-22 Building Code Requirements and Specification for Masonry Structures*. 2022, The Masonry Society: Colorado.

[7] Manzouri, T., Schuller, M.P., Shing, P.B., and Amadei, B., *Repair and Retrofit of Unreinforced Masonry Structures.* Earthquake Spectra, 1996. **12**(4): p. 903-922.

[8] Masia, M.J., Petersen, R.B., Han, Y., and Corrêa, M.R.S., *Torsion Shear Test for Mortar Joints in Masonry: Experimental Evaluation.* Masonry International, 2010. **23**(3): p. 91-102.

[9] Salmanpour, A.H., Mojsilović, N., and Schwartz, J., *Displacement capacity of contemporary unreinforced masonry walls: An experimental study.* Engineering Structures, 2015. **89**: p. 1-16.

[10] Page, A.W., *Personal communication with Emeritus Professor Adrian W. Page, Chair of Standards Australia Committee BD-004, Masonry Structures*. 2024.

[11] Gooch, L.J., Stewart, M.G., and Masia, M.J., *Model Accuracy for the Prediction of Unreinforced Masonry Shear Wall Resistance.* Bulletin of Earthquake Engineering, 2025.

[12] CEN, *EN1052-3: Methods of test for masonry. Part 3: Determination of initial shear strength.* 2002, European committee of standardization: Brussels.

[13] Van der Pluijm, R. *Material properties of masonry and its components under tension and shear*. in *Proceedings of the 6th Canadian Masonry Symposium*. 1992. Saskatoon.

[14] Stöckl, S. and Hofmann, P., *Tests on the shear-bond behaviour in the bed-joints of masonry.* Masonry International, 1986. **9**.

[15] Stöckl, S., Hofmann, P., and Mainz, J., *A comparative finite element evaluation of mortar joint shear tests.* Masonry International, 1990. **3**(3): p. 101-104.

[16] JCSS, *Probabilistic Model Code. Part 1: Basis of Design*. 2001, Joint Committee on Structural Safety.

[17] SAI, *AS5104: General principles on reliability for structures*. 2017, Standards Australia: Sydney.

[18] Melchers, R.E. and Beck, A.T., *Structural Reliability Analysis and Prediction*. 3rd ed. 2018, Hoboken: John Wiley and Sons Ltd.

[19] Bolton, M.D., *The strength and dilatancy of sands.* Géotechnique, 1986. **36**(1): p. 65-78.

[20] Lawrence, S.J., *Behaviour of brick masonry walls under lateral loading*, in *School of Civil Engineering*. 1983, University of New South Wales: Australia.

[21] McNeilly, T., Scrivener, J., Lawrence, S.J., and Zsembery, S., *A Site Survey of Masonry Bond Strength.* Australian Civil/Structural Engineering Transactions, 1996. **CE38**(No. 2, 3 & 4): p. 103-109.

[22] Al-Harthy, A.S. and Frangopol, D.M. *Reliability analysis of masonry walls*. in *Proceedings of the 7th Specialty Conference on Probabilistic Mechanics and Structural Reliability*. 1996. Worcester, MA: American Society of Civil Engineers.

[23] Stewart, M.G. and Lawrence, S.J., *Model Error, Structural Reliability and Partial Safety Factors for Structural Masonry in Compression.* Masonry International, 2007. **20**(3): p. 107-116.