



EXPERIMENTAL AND NUMERICAL INVESTIGATIONS ON BEHAVIOUR OF URM
WALL SUBJECTED TO IN-PLANE LOADING

BY

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A thesis submitted as a fulfilment of the requirements for the degree of Master of Philosophy

The University of Adelaide School of Civil, Environmental & Mining Engineering

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TABLE OF CONTENTS

STATEMENT OF ORIGINALITY	1
DECLARATION.....	2
ACKNOWLEDGEMENTS.....	3
TABLE OF CONTENTS	4
ABSTRACT	6
Chapter 1. Introduction and General Overview	7
Chapter 2. Journal Paper 1- Assessment of diagonal tensile strength of URM panel subjected to diagonal compression loading using DIC analysis and numerical modelling	12
Chapter 3. Journal Paper 2- Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis.....	70
Chapter 4. Conclusions and Future Work.....	97
PRINCIPAL NOTATIONS	99
APPENDICES	104
A MATERIAL TESTS	104
A.1 Introduction.....	104
A.2 Compression test	109
A.2.1 Test method for mortar cube	109
A.2.2 Test method for unit	114
A.2.3 Test method for prism	117
A.3 Three-point bending test	122
A.4 Bond wrench test.....	126
A.5 Shear couplet test	130
B DIC APPLICATION	149
B.1 Introduction	149
B.2 Speckle pattern method	149
B.3 Error source and calibration processing	151
C DIAGONAL COMPRESSION TEST	153
C.1 Test arrangement	153
C.2 DIC application with DSLR camera.....	164
D BEHAVIOR OF MASONRY STRUCTURES BY NUMERICAL STUDY	184
D.1 Modelling strategies for masonry	184
D.2 Elastic element of expanded unit	186
D.3 Traction separation law	186
D.3.1 Linear elastic behaviour of stiffness.....	189
D.4 Failure modes of unreinforced masonry.....	190

D.5 Linear Drucker-Prager (DP) plasticity model	192
D.6 Extended finite element (XFEM)	194
E. FEM MODEL FOR MASONRY PRISM	196
E.1 Compression behaviour for prism	196
E.2 Discussion of FEM result	197
F. IDENTIFICATION OF DIAGONAL TENSILE STRENGTH OF MASONRY PANEL	201
F.1 Determination of diagonal tensile strength of masonry panel	201
F.2 Numerical analysis of diagonal compression test	202
F.2.1 Linear elastic analysis of diagonal compression test	203
F.2.2 Investigation of coefficient factor in DIC analysis	206
F.2.3 Nonlinear analysis for diagonal compression test	209
F.2.4 Influence of f_t/c on ultimate load, P_{ul} and failure pattern.....	214
REFERENCES.....	219

ABSTRACT

Predicting the strength of masonry panel is the primary concern to have enough resistance for severe an extreme seismic event. Hence, there are numerous design codes available for the estimation of the force capacity of masonry panel. What is yet unclear is that evaluation of the effectiveness of the design code to estimate the strength of wall and identification of critical role of parameters. Further investigation is required to assess the objectives and it is carried out by DIC analysis and numerical study. This paper proposes a new approach to determine the diagonal tensile strength of masonry panel by DIC analysis and offers the guideline to estimate diagonal tensile strength by numerical study. These findings contribute in more accurate ways of predicting the diagonal tensile strength and this assessment give better understanding of stress field of the masonry panel to engineers and researchers. Another outcome is that the estimated diagonal tensile strength plays a significant role of shear force capacity as proposed by Turnšek and Čačovič so the more significant findings to emerge from this study is that it offers to predict diagonal tensile, shear strength of masonry panel.

Chapter 1. Introduction and General Overview

Masonry wall is the oldest structure and most well-known construction material in the world. There are various benefits in using masonry for construction, one of which is for its flexibility. There are different types of masonry brick and mortar with each one having their own specification of strength effect on the damage propagation of masonry structure. Thus, there is a large variety of masonry structures, which can be adopted in multiple construction fields. Finite element method (FEM) plays a key tool for modelling and understanding the behaviour of masonry structure. A theoretical demonstration of this application of FEM was first carried out by Lorenzo.

There are two major modelling approaches introduced by Lorenzo. These are micro-modelling and macro-modelling for FEM of masonry structures. Each modelling strategy has their own benefits and disadvantages, so the determination of modelling strategies depends on the simplicity of joint interface and accuracy of results (Lorenzo 1996). The simplified micro modelling strategy is commonly used to investigate structural behaviour of masonry walls and buildings. Arya and Hegemier (1978) employed the simplified micro-modelling approach to obtain non-linear response prediction for concrete masonry assemblies. Lotfi and Shing (1978) also applied the simplified micro modelling approach to study fracture behaviour of joint interface. This study quantifies the structural behaviour of unreinforced masonry (URM) walls by using the simplified micro modelling approach to deliver effectiveness of computational cost in terms of time and intermediate complexity of interface.

Oliver et al 2004, Miglietta et al 2017, Lourenço et al 1997 had used 2D simplified micro modelling for investigating the structural behaviour of URM walls. Oliver et al (2004) studied structural behaviour of URM wall under in-plane cyclic loading in 2D model and Miglietta et al (2017) implemented FEM into URM wall under reversed cyclic in-planed loading. Both FEM models had showed a good correlation of force displacement response but neither of studies consider the compressive behaviour of masonry.

Lourenço and Rots et al (1997) introduced a multi-surface criterion for joint interface, which defines an interface based on three yield criterions, namely, a tension cut-off due to tensile stress, a Mohr-Coulomb failure envelop for shear failure and a cap model for compressive failure. Although the 2D analysis has good agreement with experimental studies, but it has severe limitations. The 2D analysis model can only apply to single leaf URM wall and require implementation of thickness along joint interface to capture out-of-plane behaviour.

There are several 3D analytical methods explored and studied by researchers. Mohammad et al (2015) studied partially grouted masonry assemblages and employed concrete damaged plasticity. Kurdo et al (2017) developed FEM modelling without employing any user subroutine for unreinforced masonry (URM)

wall under in-plane loading, out-of-plane loading with extended finite element method (XFEM) to capture unit crushing without pre-defined crack location. The aforementioned studies have dealt with damage criterion in FEM but the previous studies do not attempt to consider the impact of fracture energy under various normal stress along the joint interface neither experimental system and analysis. The principal objective of this project was to investigate the influence of fracture energy under various pre-compression and shear couplet test will be adopted for bond behaviour of clay brick and sandstone via 3D FEM numerical study.

Previous researchers have not provided comprehensive guidelines of experimental programs to clarify compressive behaviour of masonry, bond behaviour of joint interface, damage criterion of unit and relationship of fracture energy under various normal stress. Based on literature review, previous studies mainly dealt with plasticity model with experimental programs (Mohammad et al 2015, Nelson et al). Mohammad et al (2015) studied simplified micro modelling represented by Concrete Damaged Plasticity (CDP) model with compressive prism test and diagonal compression test. Nelson et al (2016) studied force displacement response of URM wall by comparing Drucker Prager(DP) model and CDP model. The previous study employed plasticity approach represented by either DP or CDP model. The proposed project will be undertaken with deep insight into compression test of prism for each clay brick, sandstone, limestone to capture compressive behaviour of masonry in numerical model.

Most studies in the field of masonry have only focused on tensile bond strength and shear-bond strength of joint interfaces and have little interest to fracture energy of joint interface and unit crack. Several studies related to joint bond behaviour reported by Van der plujim et al 1999, guinea et al 2000 and Boccao et al 1989. Van der plujim (1999) studied bond behaviour of joint interface by conducting direct tensile bond test and shear couplet test under different normal stress. This study has been set out to examine tensile bond behaviour of the joint interface and shear bond behaviour to address relationship between fracture energy and normal stress. Guinea et al 2000 and Boccao et al 1989 reported crack mouth opening displacement (CMOD) for unit by conducted 3-point bending test. The test involved fracture energy of unit crack mouth opening and size effect of unit. To date, little evidence has been found associated to fracture energy of unit crack growth. Kumar et al (2014) introduces plasticity approach based on user defined subroutine and proposed formula to estimate the fracture energy. The model has been verified but the proposed equation does not engage with other types of URM wall. The FEM of masonry structure of the past did not include influence of fracture energy for failure criterion on joint interface and unit. Therefore, more investigation should be involved to produce a comprehensive guideline for numerical study.

Literature has been published on DIC for masonry research to capture full-field measurement of crack propagation by Ramos et al (2015), Bolhassani et al (2015, 2016) and Nghiem et al. (2015). Ramos et al (2015) studied large scale of reinforced masonry wall under horizontal cyclic loading with 2D and 3D DIC experiments. It was the first attempt to use the 3D DIC for in-plane and out-of-plane. The author concludes the 3D DIC as a tool of structural health monitoring for future work. Bolhassani et al (2015, 2016) used DIC for grouted and ungrouted masonry assemblage by carrying out compressive prism and diagonal compression test. Nghiem et al (2015) evaluated the damage initiation on the masonry joint. The contribution of DIC analysis in masonry has received little attention in the past but it has become significantly important to investigate the crack propagation of URM wall and potentially it can be another tool to verify the FEM result by comparing strain profile.

Objectives and scope of research

Systematic and comprehensive studies will be conducted from focusing and evaluating basic material properties and structural behaviours to address the following research objectives from a comprehensive literature review:

1. *To evaluate diagonal tensile strength of URM wall based on ASTM, RILEM codes and proposed new equation by DIC analysis.* The diagonal tensile strength, f_t determined by ASTM or RILEM standard and these codes have different interpretations of stress field. The state of stress field carried out by elastic and inelastic analysis in numerical model and new equation of f_t proposed by DIC analysis. These include: FEM, XFEM for unit crack, 2D-DIC, influence of tensile strength of joint f_t , relationship of diagonal tensile strength of masonry, f_{t_DIC} and ratio of tensile strength and initial shear strength f_t/c .
2. *To understand crack growth of masonry panel under monotonic diagonal compression load.* The crack growth of masonry panel is identified in DIC analysis and it is presented in strain evolution profile. The virtual gauges in DIC analysis has been validated with LVDT and further strain evolution profile has also been introduced, which includes: DIC analysis, strain profile and 2D DIC in GOM correlate.
3. *To evaluate the shear force capacity of URM wall based on AS 3700 and proposed equation by Turnšek and Čačovič.* The parametric study to be undertaken to investigate influence of pre-

compression and aspect ratio in numerical model. The numerical study include: simplified micro approach, cohesive behaviour for joint interface, XFEM for unit crack, DP plasticity model and damage criterion.

4. *To understand error measurement subjected to 2D-DIC for large scale specimen and address calibration method.* The error source of DIC analysis introduced in terms of speckle pattern, quality of measurement device, image processing, tracking speckle algorithm and calibration plate. These include: various technique to apply speckle pattern, saturation of light, correct distortion in raw image, window and step size and user defined calibration plate.
5. *To better understand material properties of masonry, including masonry unit, mortar joint and joint interface.* The comprehensive experimental program delivered to define material properties masonry. The improvement of experimental set up and investigation of key parameters can be guideline for other researchers. The experiment include: compression test for unit, mortar cube, prism, three point bending test for unit, bond wrench test and shear couplet test.

Outline of the thesis

The thesis is organised into two manuscripts and one appendix. Each chapter includes the title of manuscript paper and introduction explaining research objectives and the innovation. The appendices consist with experimental program to define material properties of masonry and numerical study on masonry prism and panel.

Chapter 2. provide new assessment of diagonal tensile strength of masonry panel by DIC analysis. Traditionally, diagonal tensile strength of masonry panel is determined by ASTM, RILEM standard and each code has different interpretation of state of stress field. Thus, the state of stress field is investigated by numerical model and DIC analysis. One of the main obstacles is the variation of joint bond strength. As a result, virtual gauges attached in centre of masonry panels and used to estimate maximum principal stress. In order to achieve the objectives 1 and 2 the diagonal tensile strength approach in DIC analysis and demonstrated the relationship between flexural tensile strength of joint and diagonal tensile strength.

Chapter 3. Provide evaluation of the force capacity for masonry panel by numerical study. The mechanical properties of masonry obtained from experimental program and the result applied into unit, mortar joint and joint interface. Failure pattern of masonry panel is associated with the ultimate load. Hence it is important to simulate crack propagation with caution. For this purpose, the fracture energy of the joint interface is expressed as a linear fit so it allows to capture the more realistic trend of failure pattern. In order to achieve the objectives, 3 failure pattern of masonry panel have been identified by numerical model and force capacity calculated in conjunction with AS3700 and Turnšek and Čačovič.

Appendices. provide comprehensive guideline of experimental program and error sources of DIC include calibration method. Various type of masonry includes clay brick, sandstone, limestone and bluestone undertaken to study characteristic of mortar joint, joint interface and unit to achieve objective 4. Especially shear couplet test carried out in depth to identify modified Mohr's coulomb criterion by dry frictional coefficient and to discuss variation of cohesion. The objective 5 is delivered in terms of DIC algorithm and application of speckle pattern. The DIC analysis is not limited to variety of speckle pattern technique, image processing to correct distortion, determine optimized subset and step size and calibration plate for large scale specimen.

Chapter 2. Journal Paper 1- Assessment of diagonal tensile strength of URM panel subjected to diagonal compression loading using DIC analysis and numerical modelling

Introduction

This chapter contains the paper “Assessment of diagonal tensile strength of URM panel subjected to in-plane monotonic loading by DIC analysis and numerical study” which focuses on the assessment of diagonal tensile strength by DIC analysis and numerical model. The experimental program includes basic material tests for the masonry unit, mortar joint, joint interface and the diagonal compression test. Total of 4 diagonal compression tests have been carried out, consisting of 2 clay brick panels and 2 sandstone panels. Traditionally, the diagonal tensile strength of masonry panel have been determined by RILEM TC-76 and ASTM E519-02 standard. When incipient damage occurs in masonry panel, the codes become inaccurate and so, further investigation of stress field has been carried out by DIC analysis. The aim of this study is to propose a more accurate approach to determine the diagonal tensile strength and establishing the relationship between diagonal tensile strength and the ratio of tensile and shear strength. The diagonal tensile strength of masonry panel has crucial role in designing the shear force capacity of panel as proposed by Turnšek and Čačovič. Therefore, the paper offers new assessment in predicting the diagonal tensile and shear strength of masonry panel by DIC and FEM analysis.

List of manuscripts

Park, Y.K., Vaculik, J., Sheikh, A.H., Griffith, M.C., 2019. Assessment of diagonal tensile strength of URM panel subjected to in-plane monotonic loading by DIC analysis and numerical study. To be submitted to *Construction and Building Materials*.

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- the candidate's stated contribution to the publication is accurate (as detailed above);
- permission is granted for the candidate to include the publication in the thesis; and
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Assessment of diagonal tensile strength of URM panel subjected to in-plane monotonic loading by DIC analysis and numerical study

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ABSTRACT

This paper addresses the evaluation of diagonal tensile strength of masonry panels in experimental program and finite element method (FEM) analysis. The experimental program includes basic material tests for the masonry unit, mortar joint, joint interface and the diagonal compression test. A total of 4 diagonal compression tests work carried out on 2 clay brick panels and 2 sandstone panels. Traditionally the diagonal tensile strength of masonry panel is determined by RILEM TC-76 and ASTM E519-02 standard, but incipiently when damage occurs in masonry panel the codes shows inaccuracy so further investigation of stress field has been carried out by DIC analysis. The aim of this study is to propose a more accurate approach to determine the diagonal tensile strength and establishing the relationship between diagonal tensile strength and the ratio of tensile and shear strength. The diagonal tensile strength of masonry panel has crucial role in designing the shear force capacity of panel as proposed by Turnšek and Čačovič. Therefore, the paper offers new assessment to predict the diagonal tensile and shear strength of masonry panel by DIC and FEM analysis.

Keyword

Diagonal tensile strength; Finite element method; XFEM; 2D DIC; Strain evolution profile

1. INTRODUCTION

Identification of material properties of masonry is crucial in designing of masonry structures and to evaluate the performance of existing buildings. To date, many researchers[1-9] are focused on the diagonal compression test as it can be used to predict the shear capacity of masonry panel as proposed by Turnšek and Čačovič [19]. The majority of previous research programs were associated with unreinforced masonry (URM) wall and the diagonal tensile strength is determined by ASTM[11], RILEM[10] standard. However, the main

challenge faced by many researchers is that the great contribution of time and cost is involved in diagonal compression test set up. Furthermore, the secondary concern is whether the ASTM, RILEM codes is more accurate to determine diagonal tensile strength. Each code identified the different coefficient α and up to now, only Brignola et al[1] have investigated the diagonal tensile strength of masonry panel.

Brignola et al[1] worked on the evaluation of α by numerical study. The author adopted continuum homogenised model proposed by Calderini [12] and investigated the influence of interlocking ratio and the ratio of tensile to shear strength of masonry. It offers an important demonstration of how the α sensitivity affected by other parameters. However, the parametric study did not take into account for the variation of bond strength and the adopted constitutive model does not have bond strength in head joint. The crack propagation of masonry panel is mainly governed by the joint bond, so the random variation of bond strength plays a key role of diagonal tensile strength. To be able to capture the stress field of masonry panel the DIC approach is introduced by measuring the normalised maximum principal strain. This approach allow for a new coefficient α to be proposed and further parametric study conducted by inelastic analysis.

The fundamental experimental program was conducted to subsequent numerical model. The parametric study was conducted into two parts. One is that the sensitivity analysis of ultimate load and failure pattern by direct tensile strength of masonry. The other one is that investigating the influence of ratio of tensile, shear bond strength by diagonal tensile strength based on new DIC approach. These findings provide the better understanding of failure pattern in masonry panel and proposed more accurate coefficient. Therefore, this paper demonstrates the remarkable guideline to design diagonal tensile and shear strength of the masonry panel as studied by Turnšek and Čačovič [19].

2. DESCRIPTION OF EXPERIMENTAL PROGRAM

All masonry specimens were constructed by qualified masonry under in the laboratory. The clay bricks in this study were pressed sandstock bricks obtained from demolition of existing buildings. The bricks were all

nominally 230 x 110 x 70 mm although these dimensions varied slightly between individual bricks. The sandstone was obtained from a local quarry and was cut to 290 x 110 x 150 to build the test specimens.

The mortar mix proportion was lime : sand=1 : 3 and the same ratio was used for all the masonry couplets, prisms and walls. The thickness of the mortar joints were 10mm in all specimens. The current study focused on clay brick and sandstone.

2.1 Material testing

Material test for masonry conducted in this chapter. The tests included compression tests of mortar cubes, masonry units and prism, three-point bending tests for masonry units and bond wrench tests of couplet. A range of material tests on the masonry specimens for subsequent to use in numerical modelling

2.1.1 Compression test on mortar cube, unit and masonry prism

The general loading protocol was adopted from EN 14580 [17] and it was kept consistent for each type of compression tests. The protocol includes 3 cycles loadings prior to ultimate load as illustrated in Figure 1. The first cycle closed the gap between specimen and loading machine so the slopes of the second and third cycles were used to calculate elasticity of modulus, E . It was determined by a range of stress between 0.02 and 0.2 times σ_{ult} . The Young's modulus of elasticity was calculated based on Eq: (1) and the corresponding strain ε_u , ε_o taken from the average of the two gauges on opposite sides.

$$E = \frac{\Delta\sigma}{\Delta\varepsilon} = \frac{\sigma_u - \sigma_o}{\varepsilon_u - \varepsilon_o} \quad (1)$$

The compression test for mortar cube was conducted in accordance with EN 1015[18]. The compression test for masonry unit was based on EN 1926 [19]. The compression test for the prism was conducted in accordance with AS 3700 [21].

The compressive strength was calculated using Eq: (2) and different shape factor, k was used in each type of specimen. The k value was calculated based on the ratio of height and thickness, H/T hence the mortar($k=1$), unit($k_{claybrick}=0.85$, $k_{sandstone}=0.94$), masonry prism ($k_{claybrick}=0.92$, $k_{sandstone}=0.95$) were taken. The compressive strength of mortar cube presented in Table 1, masonry units in Table 2 and masonry prisms in Table 3.

$$f_c = k \frac{P_{max}}{A_d} \quad (2)$$

Where A_d is cross sectional area, P_{max} is the peak load and k shape factor

The stress versus strain response of masonry prism was required for numerical model. The post peak behaviour of the prism was back-filled based on idealized stress strain approach as studied by Kaushik [22] and it is indicated using the dash lines. The typical stress strain curves for these tests are presented in Figure 2.

2.1.2 Three-point bending test for masonry unit

Three-point bending test conducted based on ASTM C-67 [24] to determine the flexural tensile strength of unit, $f_{ut,flex}$ and fracture energy G_f^I . To date, very little research has been carried out on fracture energy of masonry unit, however a number of research carried out on concrete beam. The previous study associated with fracture energy of concrete beam reported by Guinea et al [25], Uday et al [26], Kozłowski et al [27], Tabatabaee et al [29] and it determined the on RILEM FMC-50 [23]. Hence, the current study adopted the RILEM FMC-50 [23] to determine fracture energy of masonry unit G_f^I .

The test arrangement is illustrated in Figure 3. The extensometer was mounted in the notch of masonry unit to measure the crack mouth opening displacement (CMOD). The test imposed displacement control with 0.6mm/min rate. The , $f_{ut,flex}$, G_f^I is determined by Eq. (3), (4) and the result is as presented in Table 2.

$$f_{ut, flex} = \frac{3P_{\max}}{2b(d-a)^2} \quad (3)$$

$$G_{f_u}^I = \frac{W_o + mg\delta}{b(d-a)} \quad (4)$$

Where W_o is the energy dissipated for fracture by external load represented by area of force deflection curve and m is mass of unit, g is gravitational acceleration, δ is final displacement after failure. The other notations include a , b and d are illustrated in figure 3 (a).

2.1.3 Bond wrench test on masonry couplet

The bond wrench test was conducted to investigate flexural tensile bond strength, $f_{mt, flex}$. The test arrangement is in accordance with AS 3700 [21]. Figure 4 shows the bond wrench bar and strain reading transfer into data acquisition system.

The bond had failed due to the applied moment so the $f_{mt, flex}$ can be determined by Eq: (5) and the result is presented in Table 3. The result shown the large CoV of $f_{mt, flex}$ and it represent the large variation of bond strength.

$$f_{mt, flex} = \left(\frac{M}{Z_d} \right) - \left(\frac{F}{A_d} \right) \quad (5)$$

Where, Z_d is the section of modulus, M is the applied bending moment at the centroid of the bed area, and F is the total compressive force.

2.1.4 Shear couplet test (cited in thesis)

Current study has taken shear parameters from the most recent study of shear couplet test reported by Park [30]. The detail test set up illustrated in Figure 5. These include dry friction coefficient, initial shear strength, fracture energy and shear stiffness. The test imposed the shear load, F to have 8mm of shear displacement and constant axial stress (σ_v (MPa)= 0.3, 0.6, 0.9) was applied.

Initial shear strength, namely cohesion c is determined by two approaches. One approach is taking the intercept c [Eq. (9)] from the linear form of ultimate shear stress. The other approach is taking the intercept c [Eq. 6] from the modified linear form of ultimate shear stress or calculate individual c based on Eq. (7). Second approach is introduced by enforcing to modify the slope as equivalent with dry friction coefficient, μ_d . The μ_d obtained from linear form of residual stress or obtained individual μ_d based on Eq. (8). Linear fit of shear stress of masonry couplet is represented in Figure 7. Surprisingly the value of the coefficient of determination, R^2 of linear form of ultimate shear stress and modified linear does not show significant different in both of clay brick and sandstone couplet. Variation of individual c presented in Figure 8 and CoV of cohesion for masonry couplet have 44%, 41% for clay brick and sandstone respectively. In this experiment, the variation of c was investigated by calculating the individual c and large scatter was observed in Figure 8.

$$\tau_u = \mu_d \sigma_v + c \quad (6)$$

$$c = \tau_u - \tau_{res} \quad (7)$$

$$\mu_d = \frac{\tau_{res}}{\sigma_v} \quad (8)$$

$$\tau_u = \mu \sigma_v + c \quad (9)$$

Fracture energy of joint interface G_f^{II} , represented as linear form as shown in Figure 9. The shear stiffness K_{ss} collected from the experiment and also calculated Eq. (10) as introduced by Pluijim [31]. All of the shear parameters are presented in Table 3.

$$K_{ss}, K_{tt} = \frac{G_u G_j}{H_j (G_u - G_j)} \quad (10)$$

Where G_u , G_j is shear modulus masonry unit and mortar joint.

3. DIAGONAL COMPRESSION TEST FOR MASONRY PANEL

3.1 Test specimens and loading arrangement

A total of 4 URM walls were prepared for diagonal compression test and each of the two walls made by clay brick and sandstone. The dimension of the URM walls are 1190mm by 1200mm and detail geometry of wall presented in Figure 10. 10mm of mortar layer place on the base so all of walls have 1200mm of height.

Noted that the sandstone wall made with 70mm height of unit on top coarse.

The test arrangement was adopted from ASTM and was implemented. Similar test set up was performed by Brignola et al [1], Miosevic et al [28], Corradi et al [4], Dizhur et al [15] and Mustafaraj et al [16]. The test consists of two loading shoes at diagonal corners of the panel which is connected with steel rod. The compressive loading is applied by hydraulic jack diagonally between the two of loading shoes though the steel rod. The monotonic loading applied until reach to ultimate load.

3.2 DIC APPLICATION

In recent years, there has been an increasing interest in used of DIC because of ability to give full-field measurements in masonry panel. For example, Ramos et al [33] conducted a large scale of reinforced masonry wall under horizontal cyclic loading with 2D and 3D-DIC experiments. Bolhassani et al [34-35] used the DIC for grouted and ungrouted masonry assemblage by carrying out compression test of prism, bed joint shear test and diagonal compression test. Nghiem et al [36] investigated a new approach of DIC technique to evaluate masonry damage. Hossain et al [37] has conducted shear compression test for semi-interlocking masonry subjected to in-plane cyclic loading with 2D DIC analysis. The study attempted to identify the crack opening along the joint by integrating aspect of crack growth.

The current study employed DIC analysis to monitor real time crack growth in strain map and it provides detail of crack propagation. It is the advanced feature of DIC analysis so that even the small crack opening which hard to observed by naked eye is able to present in strain map. To achieve good accuracy of DIC analysis, error sources discussed in this chapter. The error sources of DIC can be identified into two aspects. One is the external environment such as light saturation, image contrast, aperture of camera, quality of speckle

pattern and the other one is correlation algorithms such as window, step size. Many researchers used DIC analysis[34-38] however still no sufficient study conducted to correct error sources so current study delivered a deeper insight into lens correction by employing user-defined calibration plate and image correction.

3.2.1 Error sources of DIC application

The DIC algorithm does not simply track the speckle dot on the specimen surface but it tracks the normalised speckle in subset based on grey level. Thus, in the tracking process, the subset shifts until speckle pattern matches the image of the deformed surface and the match is calculated as the total difference in grey level. As a result, the role of speckle pattern is critical in DIC analysis. Especially the influence of speckle quality plays a critical role for larger scale URM wall [13], [38]. The speckle pattern can be made through several methods. It can be made by marker pen, stencil, paint gun or any other methods [38]. The key important factor for speckle patterns is the density of speckle, dot size and random variation. In this study, paint spray was used as the most suitable tool in terms of density, size of dot and random variation. The optimised dot size was 2-4mm and the pattern made by bold out spray nozzle.

The current study carried out 2D DIC with DSLR camera and the lens distortion fixed by calibration plate and image processing. The common issue for lens distortion is barrel distortion due to the wide angle lenses. In order to fix the distortion effect, calibration plate and image processing were adopted. The calibration plate with 35mm spacing of grid selected based on the recommendation of Correlates solution company. The captured image imported in adobe camera raw software to correct distortion effect and converted the imaged to TIFF file format prior to do DIC analysis. The converted images were processed in GOM correlate software. The GOM correlate software has a tool to evaluate the speckle pattern quality so the tool used to determine optimized size of subset and point distance. The typical optimised size of subset and point distance was determined as 15 pixel, 9 pixels.

3.2.2 DIC camera set up

Sony DSLR camera, namely DSC-RX10M3 with Zeiss vario sonnar T* 24-600mm F2.4-4 lens was used to capture the image. Two photography lights were used with 500mm by 700mm soft box studio to cover whole surface of masonry panel. In order to control light saturation, the distance between light, the panel and the angle of light stand adjusted to give enough contrast. As shown in Figure 11 (a), the object distance between camera was 2.8m and light distance is 2.4 m with 40° angle. To obtain fine level of detailed, the DSLR camera set up utilised an aperture of F/10, shutter speed 1/125 second and ISO-1600. The camera was positioned perpendicular to the centre of the masonry panel and the tripod was adjusted to face the centre point of the panel. The images recorded as ARW which raw file format of Sony camera with 20MP resolution and 1 frame per second was used during the test. The masonry panels coated was as white background with black speckle to emphasise contrast and to cover with the speckle evenly on the entire surface. The test arrangement was conducted as shown in Figure 11 (b).

3.3 Displacement measurement by LVDTs

The current study employed DIC analysis to perform full-field measurement but still LVDT was used to provide secondary form of global displacement measurement and to validate DIC analysis. The instrumentation of LVDT illustrated in Figure 12. The LVDT 1 mounted in loading shoes to monitored diagonal displacement along the loading shoes. The LVDT 2,3 mounted on masonry panel to record the diagonal compressive displacement of the wall and plotted the force displacement curve separately to evaluate eccentricity. The LVDT 4,5 mounted on masonry wall to monitored diagonal tensile displacement of the wall and used to calculate the shear modulus of panels. The LVDT 6,7 mounted for reference purpose and LVDT 8 mounted to monitor horizontal displacement of wall. Dental paste layer applied on the wall before install loading shoe to applied uniform distributed load.

3.4 TEST RESULTS

3.4.1 Force versus displacement response for masonry panel

The force versus displacement curves for clay brick wall 1,2 and sandstone wall 1,2 presented in Figure 13. The figure has acronyms stated as diagonal compression test 'DC', clay brick wall 'CBW', sandstone wall 'SSW'. The potential problem in adopted test arrangement is eccentricity. The diagonal load must transfer to rob in uniform state in front and back side. Comparing LVDT 2 and 3 could provide good evidence to evaluate eccentricity of applied diagonal load. Ideally if the loading applied even then the LVDT 2 and 3 should be identical so another graph was plotted in pre failure stage. Figure 13 plotted force versus diagonal displacement for masonry panels. The general trend of force displacement trend observed in the figure. The force has initial rising branch and it gradually softening as cracking initiated up to peak load. However beyond the peak load, the load reduce rapidly and constant residual load plotted due to the friction.

The loading eccentricity can be evaluate by gap of LVDT 2,3. Figure 13 (a), (b), (c) and (d) shown the gap between LVDT 2, 3 in early stage but still has same trend of response and the difference of LVDT 2, 3 was less than 0.05mm. Interestingly, Figure 13 (d) captured the unusual response for LVDT 3 and it assumed that the gap derived from untouched LVDT. Hence no displacement recorded until close to the gap but the LVDT 3 still has same response with LVDT 2.

During the test, another problem was observed that is the failure of LVDT bracket. For example, Figure 13 (d) shown that the LVDT 2 stopped reading after 4.5mm of diagonal compressive displacement. It is because the failure of LVDT bracket bond so the bond failed due to the large deflection and the LVDT is no longer attached on the wall. This is a common problem with large displacement. However, LVDTs were still able to capture the post behaviour of diagonal compression until bond has failed. Therefore, based on the good agreement of LVDT 2 and 3, the adopted diagonal compression test with loading shoe, successfully applied diagonal compressive load without eccentricity issue and some of LVDTs failed during the test but still capture the post behaviour.

3.4.2 Shear modulus of masonry panel

The shear modulus of masonry, G determined by ASTM as written in Eq. (14). The shearing strain, γ calculated based on Eq. (13). The horizontal extension, ΔH determined by corresponding displacement which collected from LVDT4,5 at peak load. Similarly, the vertical shortening, ΔV determined by LVDT 2,3. The vertical gauge length, g_v determined by gauge length of LVDT 2,3. The result presented in Table 4. One interesting finding is that the horizontal extension of sandstone panel is much greater than clay brick wall and sandstone panel have very low shear stiffness of masonry, G . However the G has similar result as reported by Milosevic et al [28]. The author performed diagonal compression test with rubble stone masonry panels based on ASTM standard. The author points out that the G is affected by the stone arrangement and this finding observed in current experiment. As shown in Table 4, good consistency of G for sandstone panels observed and it implied that regardless of the strength of stone masonry panels, the shear stiffness mainly governed by stone arrangement.

$$f_{t_ASTM} = \frac{1}{\sqrt{2}} \frac{P_{\max}}{A_{net}} \quad (11)$$

$$A_{net} = \frac{1}{2}(L + H)Tn \quad (12)$$

Where A_{net} is net area of masonry panel, L , H and T are length, height and thickness of the panel respectively, and n is percentage of gross area of the unit that is solid, it assumed as 1.

$$\gamma = \frac{(\Delta H + \Delta V)}{g_v} \quad (13)$$

$$G = \frac{f_{t_ASTM}}{\gamma} \quad (14)$$

3.4.3 Validation of DIC analysis

Prior to do DIC analysis, distortion effect fixed by calibration plate and image processing and the validation of the analysis conducted. The verification carried out by comparing LVDTs with virtual gauges. The virtual

LVDT gauges, namely DIC3, DIC5 and DIC8, were used for validation purposes. The virtual gages of DIC3, 5 parallel to LVDT 3, 5. The LVDTs mounted on backside namely LVDT 2, 4 also plotted with DIC result as represented in Figure 14. The sign of LVDT indicated the subtraction and extension. The plus sign indicated extension and minus sign subtraction. In pre failure stage, virtual gauge reading is affected by vibration but still has a good agreement with LVDT.

In summary, the responses in Figure 14 indicated that the virtual gauge of DIC has strong correlation with LVDT. In other words, the distortion effect fixed by calibration plate and image processing. There were some benefits to use virtual gauge compare to traditional LVDT. It was commonly observed the failure of LVDT brackets after large amount of displacement. For instance, Figure 14 (a) LVDT2, LVDT 5 (b) LVDT 2, LVDT 4, LVDT 5 (d) LVDT2 shown the detachment of LVDT due to the bond failure of LVDT bracket. However, the virtual gauges still were able to track the displacement regardless of large deformation.

3.4.4 Crack propagation by strain evolution map

DIC is able to perform full-field measurement so it was interesting to observe crack propagation of masonry panel subjected to diagonal compression load. The crack propagation represented by strain evolution on maximum principal strain map was shown in Figure 15. The figure illustrated strain contour map derived by 4 different stages in force displacement response. The force displacement curve plotted by mean of LVDT 2,3 and the 4 stages were identified by loading step. The stage 1,2,3 is taken from 50%, 80%, 100% of ultimate load and the stage 4 is post-peak of the panel. Majority of cracks propagated along the joint interface but still some unit failure found in clay brick panel, and the crack path marked in stage 4. The crack growth may open and close during the loading stage so to identify crack opening status, presented by section plot of principal strain, so the graph is plotted along section AB. Strange crack propagation found in Figure 15 (a) stage 3 stands out as having growth of minor crack which highlighted in orange circle. The crack was found in stage 3 but had disappeared in stage 4 implying that the minor crack had initiated but the crack in other joint had ultimately propagated. This physical phenomenon is an interesting feature of full-

field measurement, in terms of monitoring real time crack opening. Besides, it is key function to detect minor cracks which is hard to observe by naked eye due to small displacement. For example, in Figure 15 (a), (b) in stage 4 observed one discrete major crack visually but the minor crack can only be found in strain contour map. As shown in Figure 15 (c), (d) have very small displacement but the crack growth is still observed in strain contour map. To have a clear view of the crack path, major crack marked as red dash lines and minor crack marked as yellow dash line in Figure 15 in stage 4.

4. EVALUATION OF DIAGONAL TENSILE STRENGTH OF MASONRY PANEL

4.1 Interpretation of diagonal tensile strength of masonry panel

Diagonal tensile strength of masonry is determined by ASTM [10], RILEM standard [11]. These codes have different interpretations in determining f_t from experimental results. The main difference of these two codes is definition of stress field for masonry panel. The stress component on masonry panel subjected in diagonal compression load, illustrated in Figure 16 (a).

The ASTM standard interpreted state of stress field as a uniform shear stress. In uniform shear stress condition, shear stress σ_{xy} , maximum and minimum principal stress σ_I , σ_{II} are all equivalent. Thus, Mohr coulomb circle can be drawn as Figure 16 (b). The f_t obtained from maximum principal stress σ_I as written in Eq. (15).

The RILEM code considered the stress field is nonuniform as evaluated by Frocht et al [39] and discussed by Brignola et al [1]. Frocht et al evaluated the stress field by elastic homogeneous isotropic continuum approach and provided following stress components taken at the centre of panel that are subjected in pure diagonal compression load. The stress components presented in Eq. (15-20). According to RILEM code, f_t identified maximum principal stress at the centre of masonry walls. Mohr's circle is also presented in Figure 15 (b).

The limitation of RILEM standard is that the masonry is not homogenous material and stress field at the centre of the panel becomes variable once the damage is propagated in the panel. Therefore, another approach required to investigate stress field.

Turnšek and Čačovič [19] proposed the Eq.(20) to predict shear force capacity of masonry panel and the equation associated with diagonal tensile strength. Eq. (20) could be rearranged by f_t as written in Eq. (21). Therefore each codes define different interpretation of diagonal tensile strength and it is important to investigate the most accurate f_t .

Diagonal tensile stress according to ASTM standard

$$\sigma_{xy} = \sigma_{III} = \sigma_I = f_{t_ASTM} = \frac{1}{\sqrt{2}} \frac{P_{\max}}{A_{net}} \quad (15)$$

Stress components according to RILEM standard(=elastic solution evaluated by Frocht [39])

$$\sigma_x = \sigma_y = -0.5 \frac{P_{\max}}{A_{net}} \quad (16)$$

$$\sigma_{xy} = 1.1 \frac{P_{\max}}{A_{net}} \quad (17)$$

$$\sigma_{III} = -1.62 \frac{P_{\max}}{A_{net}} \quad (18)$$

$$\sigma_I = f_{t_RILEM} = 0.5 \frac{P_{\max}}{A_{net}} \quad (19)$$

$$V_{TC} = \frac{Lt}{b} f_t \sqrt{1 + \frac{\sigma_{v,mid}}{f_t}} \quad (20)$$

Where b is aspect ratio($1 \leq b \leq 1.5$), L , t each being length and thickness of the panel respectively, f_t is diagonal tensile strength of the panel, and $\sigma_{v,mid}$ is the mean vertical stress at the mid height of the panel.

$$f_{t_TC} = 0.437 \frac{P}{Lt} \quad (21)$$

4.2 INVESTIGATION OF STRESS FIELD AT THE CENTRE OF MASONRY PANEL BY DIC APPROACH

Diagonal tensile strength, f_t could be expressed in Eq. (22) and RILEM [10], ASTM codes [11] considered different coefficient, α . The α is constant parameter in elastic range but once damage arose at the centre of the panel, the α will be a variable. Thus, it is important to have undertaken analysis to investigate correct α . The previous investigation conducted by Brignola et al [1] and the author adopted the constitutive model proposed by Calderini et al [12]. Calerini et al [12] treated the masonry as homogenized material such as composite.

Brignola et al [1] revealed the influence of interlocking value, λ which as expressed in Eq. (23) and ratio of direct tensile strength of joint and cohesion, $f_{mt,dir}/c$ by parametric study. The α is not significantly affected by interlocking value until λ is greater than 2 but it has remarkably changed by $f_{mt,dir}/c$. The author concluded that the α should be larger than 0.5 for regular masonry panel and for irregular panel should be lower than 0.5. It would be advantageous to study the sensitivity of α by nonlinear analysis, however the limitations of adopted numerical model may imply incompleteness of α . The limitation of the constitutive model is that the masonry is a heterogenized material and bond strength of masonry has large variation.

As described in 2.1.3, 2.1.4 the variation of tensile, shear strength of joint are largely scattered, so each joint interface has different bond strength. In order to deal with variation of bond behaviour, another approach conducted by install virtual gauges in DIC analysis. The diagonal tensile strength, f_t is considered to be equivalent to maximum principal stress σ_I so it can be expressed as shown in Eq. (24). The stress can be written with strain and elasticity of young's modulus for masonry E_m and maximum principal strain ε_{I_VSG} can be measured by virtual gauge at the centre of the panel. As illustrated in Figure 17 (a),(b),(c),(d) multiple virtual strain gauges attached along one unit and one joint at the centre of the panel with 45 degree. The average of virtual strain taken as ε_{I_VSG} and used to calculate the normalised maximum principal stress.

Figure 17 (e) plotted the coefficient α versus load in diagonal compression test. At initiation of loading all 4 walls start at around $\alpha=0.48-0.52$. This is consistent with RILEM, which is based on linear elastic solution. But the force displacement response become progressively nonlinear and incipient damage occurs, α is increases. At the point of failure α for 4 walls were collected and have taken average value $\alpha=0.58$. The new assessment of diagonal tensile strength is expressed in Eq. (25). This indicates that the RILEM [10], Turnšek and Čačovič [19] approach is slightly conservative ($\alpha_{TC}=0.437$, $\alpha_{RILEM}=0.5$) and ASTM [11] approach ($\alpha=0.707$) is slightly unconservative.

$$\sigma_I = f_t = \alpha \frac{P_{\max}}{A_{net}} \quad (22)$$

$$\lambda = \frac{H_u + H_j}{L + H_j} = \frac{2}{\tan \rho} \quad (23)$$

Where, L is length of unit, H_j is height of joint, H_u is height of unit and ρ is angle of interlocking

$$\sigma_I = \varepsilon E = \varepsilon_{I_VSG} E_m \quad (24)$$

$$\sigma_I = f_{t_DIC} = 0.58 \frac{P_{\max}}{A_{net}} \quad (25)$$

5. NUMERICAL STUDY FOR LINEAR AND NONLINEAR ANALYSIS

A numerical study carried out in ABAQUS software package investigated state of stress field by linear, nonlinear analysis. The linear analysis conducted in macro scale and nonlinear analysis conducted in simplified micro scale as introduced by Lorenzo [32]. The numerical model simulated with C3D8 mesh element and the mesh size is 7x2x3 for a clay brick unit and 8x4x3 for a sandstone unit.

5.1 LINEAR ANALYSIS

The elastic analysis delivered to investigate stress field and compared to Frocht's elastic solution. The elastic material properties of masonry is presented in Table 5. Elasticity of young's modulus for brick unit, mortar

joint and prism have been obtained from compression test. The Poisson's ratio has been taken as 0.15 as common value.

The stress contour plot illustrated in Figure 18 and the state of stress component plotted in Figure 18. The figure shown the similar result compare to Frocht's [39] theoretical elastic solution. The stress components are written in Eq. (26-29).

$$\sigma_x = \sigma_y = -0.57 \frac{P_{\max}}{A_{\text{net}}} \quad (26)$$

$$\sigma_{xy} = -1.05 \frac{P_{\max}}{A_{\text{net}}} \quad (27)$$

$$\sigma_I = 0.49 \frac{P_{\max}}{A_{\text{net}}} \quad (28)$$

$$\sigma_{III} = -1.64 \frac{P_{\max}}{A_{\text{net}}} \quad (29)$$

5.2 NONLINEAR ANALYSIS

The stress components are not influenced by diagonal loads until damage has occurred at the centre of masonry panel. To consider damage initiation of masonry panel nonlinear analysis delivered in simplified micro scale. The simplified modelling approach considers thickness of mortar as finite thickness and then, expanded unit in horizontal, vertical directions. By expanding masonry unit, the original geometry of panel will be same but the elasticity of modulus will redefined by average of mortar joint and masonry unit. In this approach the Poisson effect of mortar neglected but as observed in DIC analysis, majority of crack propagated along a joint interface is still suitable for the current study.

5.2.1 Simplified micro scale- expanded unit

The elastic modulus of the expanded units must be adjusted and should have an equivalent elastic stiffness to the original masonry assemblage as shown in Figure 20. It is to be determined by taking elasticity of

modulus of original masonry unit and mortar and geometry of the masonry assemblage into account. The Young's modulus of expanded unit introduced by Pluijim [31] and modified by Lorenzo [32] as written in Eq. (30). It based on the assumption of a stack bond between masonry units and uniform stress distribution in masonry constituents.

$$E_{\text{exp}} = \frac{HE_u E_j}{H_u E_j + H_j E_u} \quad (30)$$

Where E_u , E_j , E_{exp} are Young's modulus of unit, mortar joint and expanded unit and H is total height of masonry assemblage

5.2.2 Failure modes of unreinforced masonry

Various cracking patterns are observed in masonry is subjected to ultimate loads. These failure modes of masonry under shear compression load reported by Mann and Muller [40]. Namely, the failure modes are friction failure, brick cracking and masonry compression failure. Lorenzo and Rots [41] reported the basic cases of failure mechanism for URM structures. The failure modes for joint interface and unit are illustrated in Figure 21, 22. Figure 21 illustrated the failure mode along the joint interface based on fracture mode. Namely, Figure 21 (a) crack opening, (b) in-plane sliding and (c) out of plane sliding are described based on traction separation law. The notation of fracture criterion described in Table 5. Figure 21 (a) correspond to bond wrench test so the maximum allowable nominal stress is taken as direct tensile strength of joint $t_n = f_{mt,dir}$ Figure 21 (b) corresponds to the shear couplet test so the maximum allowable nominal stress is taken as cohesion $t_s = c$. In the current study, the out of plane sliding has same direction with plane shear sliding so the behaviour of joint interface is derived by Benzeggagh-Kenane (BK) [42].

Failure mode for masonry unit illustrated in Figure 22 so namely, Figure 22 (a) unit direct tensile cracking, Figure 22 (b) unit diagonal tensile cracking is introduced. Figure 22 (a) corresponds to three-point bending test so flexural tensile strength, fracture energy collected and applied. Failure mechanism of unit diagonal

cracking in Figure 22 (b) is combined failure mechanism under shear and compression load. The combined failure mechanism is involved in unit and joint failure.

5.2.3 Traction separation law

The numerical model adopted cohesive behaviour in joint interface. The interaction of cohesive zone is a function of displacement on the layer of crack propagation. The cohesive zone has been studied by Dugdale [43] and developed by Barenblatt [44] for the fracture mode of brittle material. The surface based cohesive behaviour was suitable for the bonded interfaces with negligible thickness. As discussed in the chapter 5.2.1, the simplified micro model addresses the similar strategy into joint interface. The modelling strategy considers the mortar joint as a finite thickness and expands the brick element in vertical and horizontal directions. The contact of expanded units behaves very similar to glue, so the interaction surface is a bonded area.

In order to capture realistic trend of force displacement response, surface based cohesive model has been adopted and it is defined by damage criterion in ABAQUS software package [45]. As shown in Figure 23, the damage criterion is addressed with traction separation law and it corresponds to stress versus displacement response. Namely pre-failure stage called damage initiation and post failure called damage evolution. The damage initiation can be represented by either maximum stress or strain ratio or quadratic stress or strain ratio and to be able to capture realistic trend, current model selects quadratic stress criterion as written in Eq. (29). The Macaulay bracket is expressed in Eq. (31) to neglect the compressive stress in the masonry joint along the nominal direction.

$$\left\{ \frac{\langle t_n \rangle}{t_n^{\max}} \right\}^2 + \left\{ \frac{t_s}{t_s^{\max}} \right\}^2 + \left\{ \frac{t_t}{t_t^{\max}} \right\}^2 = 1 \quad (31)$$

As shown in Figure 23, the response of damage initiation, t is defined as linear behaviour, but the elastic stiffness, K degrades when the stress vector t reaches maximum allowable stress, $t^{\max}$. Thus, the corresponding crack propagation will respond based on the damage evolution. Eq. (32) is expressed the

traction by stiffness matrix K . D is the damage evolution variable which represent the degradation of K so $(1-D)$ is the reduction parameter due to the damage into stiffness. Initial value of the damage evolution variable is from 0 and increases up to 1 as shown in Figure 24. The damage evolution variable D is expressed in Eq. (33) as studied by Camanho and Davila [46] The effective displacement δ_{eff} was also studied by Camanho and Davila [46] as expressed in Eq. (34).

$$t = \begin{cases} (1-D)K\delta \leftarrow (t_{initial} \geq t^{\max}) \\ K\delta \leftarrow (t_{initial} < t^{\max}, \text{No damage into stiffness}) \end{cases} \quad (32)$$

$$D = \frac{\delta_{eff}^f (\delta_{eff}^{\max} - \delta_{eff}^o)}{\delta_{eff}^{\max} (\delta_{eff}^f - \delta_{eff}^o)} \quad (33)$$

$$\delta_{eff} = \sqrt{\langle \delta_n \rangle^2 + \delta_s^2 + \delta_t^2} \quad (34)$$

The damage evolution can be identified based on the fracture energy that dissipated during the separation process or based on maximum displacement. Due to the brittle characteristic of masonry, it is hard to capture the maximum displacement. Thus, the current study expresses damage evolution based on fracture energy. The geometry terms of total fracture energy in mixed mode, G_{TC} is the area of traction separation plot graph as shown in the Figure 23. It is assumed that the traction-separation response of masonry in mode 2 and mode 3 are in the same direction. Based on the assumption, the mixed mode fracture energy, G_{TC} can be calculated by BK law. The exponent η holds value of 2 for brittle behaviour of joint interface [42]. The critical mode fracture energy G_{TC} under mixed mode is written in Eq. (35).

$$G_{TC} = G_{IC} + (G_{IIC} - G_{IC}) \left\{ \frac{G_{II} + G_{III}}{G_I + G_{II} + G_{III}} \right\}^\eta \quad (35)$$

5.2.4 Linear Drucker-Prager (DP) plasticity model

The linear Drucker-Prager plasticity model was employed to define uniaxial compressive behaviour of masonry. The plasticity model allows masonry to have hardening and softening under compression. The yield criterion is defined by yield surface in meridional(p-t) plane and there is a possibility for it to be either linear, hyperbolic or exponential mode. The current study adopted the linear yield surface as shown in Figure 24. The yield criteria are given by Eq. (36-41).

The evolution of yield surface with deformation beyond elastic range was specified based on uniaxial compressive yield stress. The yield stress σ_c , plastic strain ε_c^{pl} of Drucker-Prager hardening defined based on compressive stress strain curve from compression test for masonry prism. The stress strain curve is represented in Figure 2. The angle of friction β , flow stress ratio K are assumed as 36 degree, 1 and dilation angle, ψ is taken as lower value reported by Mosalam K [47].

$$F = t - p(\tan \beta) - d = 0 \quad (36)$$

The equivalent pressure stress, P is given by:

$$p = -\frac{1}{3} \text{trace}(\sigma) \quad (37)$$

The deviator stress, t given by:

$$t = \frac{1}{2} q \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{q} \right)^3 \right] = q \quad (38)$$

Assumed K=1

The von mises stress, q given by:

$$q = \sqrt{\frac{3}{2} S : S} \quad (39)$$

The third stress invariant, r and deviatoric stress, s given by

$$r = \left(\frac{9}{2} S : S : S \right)^{\frac{1}{3}} \quad (40)$$

$$S = \sigma + PI$$

The intercept, d given by:

$$d = (1 - \frac{1}{3} \tan \beta) \sigma_c \quad (41)$$

5.2.5 Extended finite element (XFEM)

Previously, a number of researchers have demonstrated that unit stiffness along the crack propagates at the centre of unit. It was introduced by Lorenci [32] as simplified unit failure. However, what is still unclear about the location of actual unit crack and the crack growth. In order to capture crack growth of masonry unit, discontinuities are required for mesh element. The challenge is that the association of meshing crack surface. The mesh element needs to be continuously redefined as crack grows and the geometry of discontinuity have to be coordinated with mesh element. For this purpose, the current study carried out extended finite element method (XFEM).

The XFEM was firstly introduced by Belytschko and Black [48] but the concept of partition of unity is studied by Melenk and Babuska [49]. The partition of unity allows local enrichment functions to be incorporated into finite element. The XFEM displacement approximation in enriched finite element is expressed with displacement vector function u and is written as Eq. (42).

$$u = \sum_{I=1}^N N_I(x) \left[u_I + H(x) \alpha_I + \sum_{a=1}^4 F_a(x) b_I^\alpha \right] \quad (42)$$

where, $N_I(x)$ is nodal shape functions, u_I is the nodal displacement vector for continuous part of finite element solution and the $N_I(x) \cdot u_I$ applied to all nodes in the model. α_I is nodal enriched degree of freedom vector, $H(x)$ is associated discontinuous jump function along the crack surface and the $\alpha_I \cdot H(x)$ applied to nodes whose shape function support is cut by the crack interior. b_I^α is the nodal enriched degree of freedom vector, $F_a(x)$ is associated elastic asymptotic crack tip functions and the $F_a(x) \cdot b_I^\alpha$ applied to nodes whose shape function support is cut by the crack tip.

The current study employed XFEM to simulate damage initiation and evolution in masonry unit under tension, compression and shear. The crack growth allowed without predefined crack location and XFEM simulated based on cohesive segment method as studied by Remmers [50]. The cohesive segment method follows traction separation law as discussed in chapter 5.2.3 and the stiffnesses parameters are calculated based on elastic properties of enriched element in this case elasticity of masonry unit.

5.2.6 Material properties in Table- unit, joint interface

The material properties presented in Table 7, 8 and 9. Table 7 is represented summary of linear Drucker-Prager parameters. Table 8 is illustrated the material properties of masonry unit. The direct tensile strength of unit taken 1/6 of flexural tensile strength which obtained from three-point bending test as written in Eq. (43). The shear strength of unit, τ_{us} assumed 1.4 times direct tensile strength, $f_{ut,dir}$. The shear fracture energy $G_f^{II_u}$ taken 0.5 N mm/mm² as reported by Raijmakers [50].

$$f_{ut,dir} = \frac{1}{6} f_{ut,flex} \quad (43)$$

$$f_{mt,dir} = \frac{1}{3} f_{mt,flex} \quad (44)$$

Table 9 is presented properties of joint interface under tension, shear and compression. The flexural tensile strength of joint interface, $f_{mt,flex}$ determined by bond wrench test and taken 1/3 of $f_{mt,flex}$ for direct tensile strength of masonry, $f_{mt,dir}$ so it is determined by Eq. (44). The fracture energy G_f^I has a recommended value of 0.012 introduced by Angelillo et al [51]. The shear parameters of interface taken from Park et al [30]. One noted that as shown in Table 3, calculated shear stiffness[from Eq. (10)] does not associated with the shear stiffness obtained from shear couplet test. To do simulate more accurate damage initiation, normal stiffness K_{nn} calculated based on Eq. (47). The softening of damage evolution criterion is defined as linear form with

BK law. The fracture energy G_f^{II} is calculated and is adopted by linear regression line as described in Figure 9.

$$K_{nn} = \frac{t_n}{\delta_n} = \frac{E_u E_j}{H_j (E_u - E_j)} \quad (45)$$

$$G = \frac{E}{2(1 + \nu)} \quad (46)$$

$$K_{nn} = K_{ss} \times 2(1 + \nu) \quad (47)$$

5.2.7 Validation of FEM model

Numerical simulation was conducted based on the material properties of masonry as shown in Table 7, 8, 9. It is interesting to compare the failure pattern of DIC analysis and FEM model. The result for clay brick panels is presented in Figure 25 (a), (b) and (c). The deformed scale for FEM model adjusted as 20 to exaggerate failure pattern. The FEM model shown similar trends to DC-CBW2, so both results had shown diagonal crack. Noted that unit crack observed in DC-CBW2 and it highlighted in a yellow circle in Figure 25 (a). Surprisingly, the Figure 25 (c) XFEM also captured the same location of the unit crack. The status XFEM indicated partial crack unit element. FEM model for sandstone and DIC analysis was also compared. The numerical result has similar failure pattern with DC-SSW1 and both of the cracks have propagated through the diagonal direction as illustration in Figure 25 (d), (e). The XFEM was also applied to the sandstone but there was no crack observed in the experimental and numerical model.

Sensitivity analysis was conducted to investigate the influence of the direct tensile strength $f_{mt,dir}$ in ultimate load. In this purpose, the numerical model plotted $f_{mt,dir (max)}$, $f_{mt,dir (mean)}$ and $f_{mt,dir (min)}$ under constant cohesion. The force versus diagonal displacement response was plotted based on means of LVDT 2,3 and the corresponding diagonal displacement of numerical model also plotted in Figure 26. The diagonal tensile strength was determined based on ASTM, RILEM and new approach proposed by DIC analysis [Eq. (23)] and it presented in Table 10.

As shown in Figure 26 (a) DC-CBW2 has similar response compared to $f_{mt,dir} (mean)$. However, the ultimate load for DC_CBW1 is even lower than $f_{mt(min)}$ and it may have been due to the characteristic of masonry. In addition, the force and displacement responses for DC_SSW1, DC_SSW2 are within range of $f_{mt,dir(min)}$ and $f_{mt,dir(max)}$. The experiment of diagonal compression test imposed load control so the post behaviour remains unclear. The numerical analysis simulated by displacement, so the post behaviour of masonry panels captured. Surprisingly, no significant differences were been observed in Figure 26 (a), (b).

6. PARAMETRIC STUDY

A previous parametric study reported by Brignola et al[1] and the author adopted numerical model which was proposed by Calderini [12]. However, the proposed constitutive model does not take into account the bond strength of head joint. This may affect the failure pattern of masonry panel, so the current study carried out nonlinear analysis and the numerical model applied damage criterion on both of bed and head joint interface.

There were three different failure modes observed in the numerical model. Namely, diagonal crack, mixed failure and bed joint sliding were found and illustrated in Figure 27. As observed in Figure 27 (a) the crack propagated as diagonal crack and the ultimate load, P increase significantly up to $f_{mt,dir}/C = 1.4$. However, after failure mode changed as mixed failure, the P only slightly increase until the failure pattern changes into bed joint sliding. Same trend of influence $f_{mt,dir}/C$ observed in sandstone. The reason P does not increase in bed joint sliding is because only friction applied.

Based on the parametric study, ultimate load and failure pattern was mainly governed by $f_{mt,dir}/C$. Thus, the P can be taken to determine diagonal tensile strength f_t by proposed equation Eq. (24). The diagonal tensile strength, f_{t_DIC} plotted with f_t/C as illustrated in Figure 28. The diagonal tensile strength f_{t_DIC} increased considerably up to $f_{mt,dir}/C = 1.0$ and after that it has shown gradually increase until $f_{mt,dir}/C = 1.4$. Beyond the $f_{mt,dir}/C = 1.4$ the failure mode changes as mixed failure.

7. CONCLUSION

In this study, new approach with coefficient α introduced by investigating stress field to determine more accurate diagonal tensile strength of masonry panel. The state of stress field analysed by strain map in DIC. The fundamental experimental program includes basic material tests and diagonal compression tests carried out to subsequent FEM model. The parametric study conducted by FEM simulation in both linear and nonlinear analysis to predict diagonal tensile strength. Respectively, the following conclusions can be drawn:

1. Further investigation of stress field is carried out in DIC strain map by using virtual gauges. Multiple virtual gauges installed at the centre of masonry panels to measure normalised maximum principal strain and it is used to estimate maximum principal stress which correspond to diagonal tensile strength. This approach considers damage propagation at the centre of the panels prior to failure; hence it allows to propose more accurate equation to determine diagonal tensile strength.
2. The numerical model delivered for both linear and nonlinear analysis to evaluate ASTM, RILEM codes. The FEM simulation identified the relationship of failure pattern under different ratios of tensile and initial shear strength, $f_{mt,dir}/c$. Further parametric study is carried out to provide guideline to estimate diagonal tensile strength based on $f_{mt,dir}/c$. This guideline is valuable demonstration to predict the shear force capacity of masonry panel proposed by Turnšek and Čačovič [19].
3. The DIC analysis promotes new perspective of monitoring real time crack growth for masonry panel subjected to in-plane monotonic loading. The real time crack growth of the masonry panel is identified by strain evolution profile in DIC analysis. This paper found that the crack propagation is not only a gradual opening, but it can be a changing variable between opening and closing. Furthermore, various calibration methods, quality control of speckle pattern introduced to fix lens distortion.

4. The unit failure of masonry panel analysed by XFEM without predefined crack path and mechanical properties of masonry unit obtained from three-point bending test to determine the flexural tensile strength and the fracture energy of unit. The XFEM captured the correct unit location by comparing to the DIC strain map. The present study highlights that the XFEM has beneficial outcome to investigate actual crack location and growth of masonry unit without defined crack path.

REFERENCE

- [1] Anna Brignola, Sara Frumento, Sergio Lagomarsino & Stefano Podestà (2008) Identification of Shear Parameters of Masonry Panels Through the In-Situ Diagonal Compression Test, *International Journal of Architectural Heritage*, 3:1, 52-73, DOI: 10.1080/15583050802138634
- [2] J.A.P.P. Almeida, E.B. Pereira, J.A.O. Barros., Assessment of overlay masonry strengthening system under in-plane monotonic and cyclic loading using the diagonal tensile test *Constr. Build. Mater.*, 94 (2015), pp. 851-865, 10.1016/j.conbuildmat.2015.07.040
- [3] A. Borri, G. Castori, M. Corradi., Shear behavior of unreinforced and reinforced masonry panels subjected to in situ diagonal compression tests, *Constr. Build. Mater.*, 25 (12) (2011), pp. 4403-4414
- [4] Corradi, Marco & Borri, Antonio & Vignoli, Andrea. (2008). Experimental evaluation of the in-plane shear behaviour of masonry walls retrofitted using conventional and innovative methods. *Masonry International*. 21. 29-42.
- [5] H. Santa-Maria, G. Duarte, and A. Garib, Experimental Investigation of masonry panels externally strengthened with CFRPlaminates and fabric subjected to in-plane shear load, in: 13th World Conference on Earthquake Engineering, 2004, No. 1627, pp. 1–10.
- [6] S.B. Kadam, Y. Singh, B. Li Strengthening of unreinforced masonry using welded wire mesh and micro-concrete – behaviour under in-plane action *Constr. Build. Mater.*, 54 (2014), pp. 247-257
- [7] A. Dehghani, G. Fischer, F. Alahi, Strengthening masonry infill panels using engineered cementitious composites, *Mater. Struct.* (2013)
- [8] A. Gabor, A. Bennani, E. Jacquelin, F. Lebon, Modelling approaches of the in-plane shear behaviour of unreinforced and FRP strengthened masonry panels, *Compos. Struct.*, 74 (2006), pp. 277-288
- [9] B. Ozsayin, E. Yilmaz, M. Ispir, H. Ozkaynak, E. Yuksel, A. Ilki, Characteristics of CFRP retrofitted hollow brick infill walls of reinforced concrete frames, *Constr. Build. Mater.*, 25 (10) (2011), pp. 4017-4024

- [10] RILEM TC. 1994. 76-LUM. Diagonal tensile strength tests of small wall specimens, 1991. In RILEM, Recommendations for the Testing and Use of Constructions Materials. London: E& FN SPON, 488–489.
- [11] ASTM E519 / E519M-15, Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages, ASTM International, West Conshohocken, PA, 2015
- [12] Calderini C., and Lagomarsino S. 2008. Continuum model for in-plane anisotropic inelastic behaviour of masonry. *Journal of Structural Engineering ASCE* 134(2): 209–220.
- [13] Mojsilović, Nebojša & Salmanpour, Amir Hosein. (2016). Masonry walls subjected to in-plane cyclic loading: Application of digital image correlation for deformation field measurement. *International Journal of Masonry Research and Innovation*. 1. 165 - 187. 10.1504/IJMRI.2016.077473.
- [14] Abdulla, Kurdo & Cunningham, Lee & Gillie, Martin. (2017). Simulating masonry wall behaviour using a simplified micro-model approach. *Engineering Structures*, 151. 10.1016
- [15] Dizhur, Dmytro & Ingham, Jason. (2013). Diagonal tension strength of vintage unreinforced clay brick masonry wall panels. *Construction and Building Materials*. 43. 418–427. 10.1016/j.conbuildmat.2013.02.015.
- [16] Mustafaraj, Enea & Yardim, Yavuz. (2017). In-plane Shear Strengthening of Unreinforced Masonry Walls Using GFRP Jacketing. *Periodica Polytechnica Civil Engineering*. 62. 330-336. 10.3311/PPci.11311.
- [17] EN 14580(2005), Natural stone test methods. Determination of static elastic modulus, European Standard
- [18] EN 1015-11(1999), Methods of test for mortar for masonry. Determination of flexural and compressive strength of hardened mortar, European Standard
- [19] Turnšek V, Čačovič F. Some experimental results on the strength of brick masonry walls. In: *Proceedings of the 2nd international brick masonry conference, stoke-on-trent; 1970*. p. 149–56.

- [20] EN 772-1, Methods of test for masonry units Part 1: Determination of compressive strength, European Standard
- [21] AS3700-2018: Masonry Structures. Sydney: Standards Australia, 2018
- [22] Kaushik, Hemant & C. Rai, Durgesh & K. Jain, Sudhir & Asce, M. (2007). Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression. *Journal of Materials in Civil Engineering - J MATER CIVIL ENG.* 19. 10.1061/(ASCE)0899-1561(2007)19:9(728).
- [23] RILEM FMC-50: Determination of the fracture energy of mortar and concrete by means of three-point bend tests on notched beams. *Materials and Structures* 18(4) 1985; p. 287-290.
- [24] ASTM C67-03a, Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile, ASTM International, West Conshohocken, PA, 2003
- [25] Guinea, Gustavo & Hussein, Gamal & Elices, M & Planas, Jaime. (2000). Micromechanical modeling of brick-masonry fracture. *Cement and Concrete Research.* 30. 731-737. 10.1016/S0008-8846(00)00228-3.
- [26] Uday, Naik. (2017). Experimental Determination of Fracture Energy by RILEM Method. *The International Journal of Engineering and Science.* 06. 106-115. 10.9790/1813-060301106115.
- [27] Kozłowski, Marcin & Kadela, Marta & Kukiela, Alfred. (2015). Fracture Energy of Foamed Concrete Based on Three-Point Bending Test on Notched Beams. *Procedia Engineering.* 108. 349 - 354. 10.1016/j.proeng.2015.06.157.
- [28] Milosevic, Jelena & Gago, António & Lopes, Mário & Bento, Rita. (2013). Experimental assessment of shear strength parameters on rubble stone masonry specimens. *Construction and Building Materials.* 47. 1372-1380. 10.1016/j.conbuildmat.2013.06.036.
- [29] Tabatabaee, Hassan & Velasquez, Raul & Puchalski, Sebastian & Bahia, Hussain. (2012). Development of the Single-Edge Notched Beam (SENB) Test.

- [30] Park, Y.K. (2019). Experimental and numerical investigations on behaviour of URM wall subjected to in-plane loading.
- [31] Pluijm, van der, R. (1999). Out-of-plane bending of masonry : behaviour and strength Eindhoven: Technische, Universiteit Eindhoven DOI: 10.6100/IR528212
- [32] Lourenco, Paulo. (1996). Computational Strategy for Masonry Structures.
- [33] Ramos, T., et al. (2015). "2D and 3D Digital Image Correlation in Civil Engineering – Measurements in a Masonry Wall." *Procedia Engineering* 114: 215-222.
- [34] Bolhassani, Mohammad & A. Hamid, Ahmad & C.W. Lau, Alan & Moon, Franklin. (2015). Simplified micro modeling of partially grouted masonry assemblages. *Construction and Building Materials*. 83. 10.1016
- [35] Bolhassani, M., et al. (2016). Damage detection of concrete masonry structures by enhancing deformation measurement using DIC. *Nondestructive Characterization and Monitoring of Advanced Materials, Aerospace, and Civil Infrastructure 2016*.
- [36] Huu-Luyen Nghiem, Marwan Al Heib, Fabrice Emeriault, (2015). Method based on digital image correlation for damage assessment in masonry structures. *Engineering Structures*. Pages 1-15, <https://doi.org/10.1016/j.engstruct.2014.12.021>.
- [37] Hossain, Md Akhtar & Totoev, Y.Z. & Masia, Mark. (2018). Using digital image correlation (DIC) technique to investigate deformation of semi-interlocking masonry (SIM).
- [38] Salmanpour, Amir Hosein. (2017). Displacement Capacity of Structural Masonry.
- [39] Frocht M. M., and Arbor A. 1993. Recent advances in photoelasticity. *Transaction, ASME*, 55:135–153.

- [40] Mann, W., and Müller, H. (1982). "Failure of shear-stressed masonry- An enlarged theory, tests and application to shear walls." *Proceedings of the British Ceramic Society* 223-235.
- [41] Lourenco, P. B., and Rots, J. G. (1997). "Multisurface interface model for analysis of masonry structures." *Journal of Engineering Mechanics*, 123(7), 660-668.
- [42] Benzeggagh, M. L., and M. Kenane, "Measurement of Mixed-Mode Delamination Fracture Toughness of Unidirectional Glass/Epoxy Composites with Mixed-Mode Bending Apparatus," *Composites Science and Technology*, vol. 56, pp. 439–449, 1996
- [43] Dugdale DS. Yielding of steel sheets containing slits. *J Mech Phys Solids* 1960;8(2):100–4.
- [44] Barenblatt GI. The mathematical theory of equilibrium cracks in brittle fracture. *Adv Appl Mech* 1962;7(55–129):104.
- [45] Abaqus analysis user's manual, 6.13-3. RI, USA: Dassault Systems Providence 2013.
- [46] Camanho, P. P., and C. G. Davila, "Mixed-Mode Decohesion Finite Elements for the Simulation of Delamination in Composite Materials," NASA/TM-2002–211737, pp. 1–37, 2002.
- [47] Mosalam K, Glascoe L, Bernier J. Mechanical properties of unreinforced brick masonry section-1. Documented to US Department of Energy by Lawrence Livermore National Laboratory; 2009.
- [48] Belytschko, T., and T. Black, "Elastic Crack Growth in Finite Elements with Minimal Remeshing," *International Journal for Numerical Methods in Engineering*, vol. 45, pp. 601–620, 1999.
- [49] Melenk, J., and I. Babuska, "The Partition of Unity Finite Element Method: Basic Theory and Applications," *Computer Methods in Applied Mechanics and Engineering*, vol. 39, pp. 289–314, 1996.
- [50] Raijmakers T, Vermeltfoort AT. Deformation controlled tests in masonry shear walls; 1992.
- [51] Angelillo, Maurizio & Lourenco, Paulo & Milani, Gabriele. (2014). *Masonry behaviour and modelling. Mechanics of masonry structures*. Springer; 2014. P. 1-26, 10.1007/978-3-7091-1774-3_1.

List of figures

Figure 1: Loading protocol of compression test

Figure 2: Stress versus strain curve of masonry prisms

Figure 3: Three point bending test on the masonry unit

Figure 4: Bond wrench test set up for masonry couplet

Figure 5: Shear couplet test set up (Park [30]) (a) Schematic view, (b) Photo view (North side)

Figure 6: Key parameters of shear couplet test

Figure 7: Shear stress linear fit for masonry couplet (a) Clay brick, (b) Sandstone

Figure 8: Variation of cohesion for masonry couplet

Figure 9: Fracture energy linear fit of masonry couplet (a) Clay brick, (b) Sandstone

Figure 10: Geometry of masonry panels (a) Clay brick wall, (b) Sandstone wall

Figure 11: DIC camera set up (a) Schematic view, (b) Photographic view

Figure 12: LVDT instrumentation of diagonal compression test (a) Clay brick wall, (b) Sandstone wall

Figure 13: Force versus displacement response for masonry panels (a) DC-CBW1, (b) DC-CBW2, (c) DC-SSW1 and (d) DC-SSW2

Figure 14: LVDT vs DIC for masonry panels (a) DC-CBW1, (b) DC-CBW2, (c) DC-SSW1 and (d) DC-SSW2

Figure 15: Strain evolution for masonry panels (a) DC-CBW1, (b) DC-CBW2, (c) DC-SSW1 and (d) DC-SSW2

Figure 16: Identification of stress field and Mohr coulomb circle based on ASTM and RILEM standard

Figure 17: Normalized maximum principal stress by DIC virtual gauges for masonry (a) Virtual gauges in CBW1, (b) Virtual gauges in CBW2, (c) Virtual gauges in SSW1, (d) Virtual gauges in SSW2 and (e) Normalized maximum stress σ_I for masonry panels

Figure 18: Stress contour plot in elastic analysis (a) σ_x , (b) σ_y , (c) σ_{xy} , (d) σ_I , and (e) σ_{III}

Figure 19: State of stress components in centre of panel

Figure 20: Expanded unit in simplified micro scale

Figure 21: Bond failure along the joint interface (a) Tensile bond cracking (=Fracture mode 1), (b) In-plane shear sliding (=Fracture mode 2), and (c) Out of plane shear sliding (=Fracture mode 3)

Figure 22: Typical unit failure modes

Figure 23: Traction separation law along the joint interface

Figure 24: Yield criteria in p-t plane for Linear Drucker-Prager

Figure 25: Crack pattern of DIC and FEM result (a) Strain map for DC-CBW2, (b) Clay brick FEM σ_{III} , (c) Clay brick StatusXFEM, (d) Strain map for DC-SSW1 and (e) Sandstone FEM σ_{III}

Figure 26: Force vs displacement response for masonry panel (a) Clay brick wall and (b) Sandstone wall

Figure 27: Influence of $f_{mt,dir}/c$ and failure pattern for masonry walls (a) $f_{mt,dir}/c$ vs P_{ul} for clay brick wall, (b) $f_{mt,dir}/c$ vs P_{ul} for sandstone wall, (c) Diagonal crack, (d) Mixed failure and (e) Bed joint sliding

Figure 28: Diagonal tensile strength, $f_{t,DIC}$ vs $f_{mt,dir}/c$ for masonry panels

List of tables

Table 1: Mechanical properties of mortar

Table 2: Mechanical properties of masonry unit

Table 3: Mechanical properties of the masonry

Table 4: Shear modulus of masonry G , determined by ASTM standard

Table 5: Elastic properties of masonry

Table 6: Notations of fracture criterion

Table 7: Linear Drucker Prager parameters

Table 8: Material properties of masonry unit

Table 9: Material properties of joint interface

Table 10: Diagonal tensile strength, f_t for masonry panels (a) Clay brick wall and (b) Sandstone wall

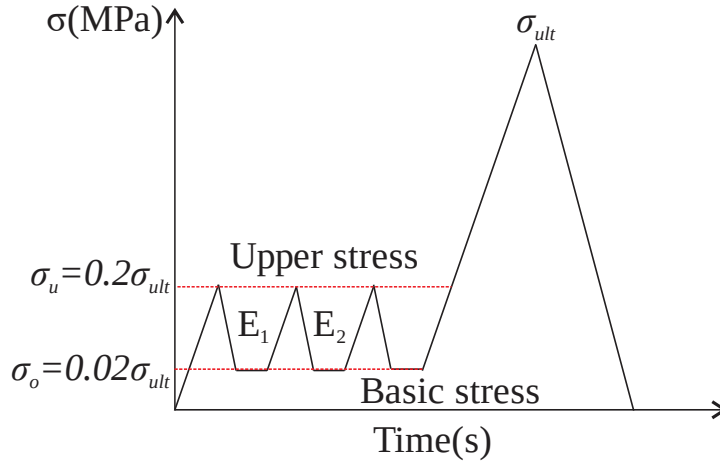


Figure 1: Loading protocol of compression test

Table 1: Mechanical properties of mortar

Property	Mortar cube		
	Mean	CoV	N
Compressive strength of the mortar, f_{jc} (MPa)	0.93	14.0	62
Young's modulus of the mortar, E_j (MPa)	436	7.3	25

Table 2: Mechanical properties of masonry unit

Property	Clay brick			Sandstone		
	Mean	CoV	N	Mean	CoV	N
Compressive strength of the unit, f_{uc} (MPa)	28.2	19.2	10	57.6	11.6	10
Young's modulus of the unit, E_u (MPa)	9,085	51.7	10	10,573	16.8	10
Flexural tensile strength, $f_{ut,flex}$ (MPa)	3.99	42.7	6	4.65	31.8	8
Fracture energy, G_f^I (N mm/mm ²)	0.32	61.2	6	0.25	17.0	8

Table 3: Mechanical properties of the masonry

Property	Clay brick			Sandstone		
	Mean	CoV	N	Mean	CoV	N
Compressive strength of the masonry, f_{mc} (MPa)	5.42	11.3	7	12.9	35.0	5
Young's modulus of the masonry, E_m (MPa)	2,272	35.7	7	3,206	20.2	5
Flexural tensile strength of the masonry, f_{mt} (MPa)	0.24	24.6	10	0.06	87.1	10
Initial shear strength, c (MPa), individual test [Eq: 7]	0.20	44.5	9	0.13	41.6	8
Initial shear strength, c (MPa), from linear fit [Eq: 6]	0.14	N/A	N/A	0.05	N/A	N/A
Dry friction coefficient, μ_d , individual test [Eq: 8]	0.67	31.7	9	0.68	4.31	10
Friction coefficient, μ , from linear fit [Eq: 9]	0.80	N/A	N/A	0.82	N/A	N/A
Shear stiffness, k_{ss} (MPa / mm)	3.2	46.8	8	5.04	40.7	8

Calculated shear stiffness, k_{ss} (MPa / mm) from [Eq: 10]	45.8	N/A	N/A	45.5	N/A	N/A
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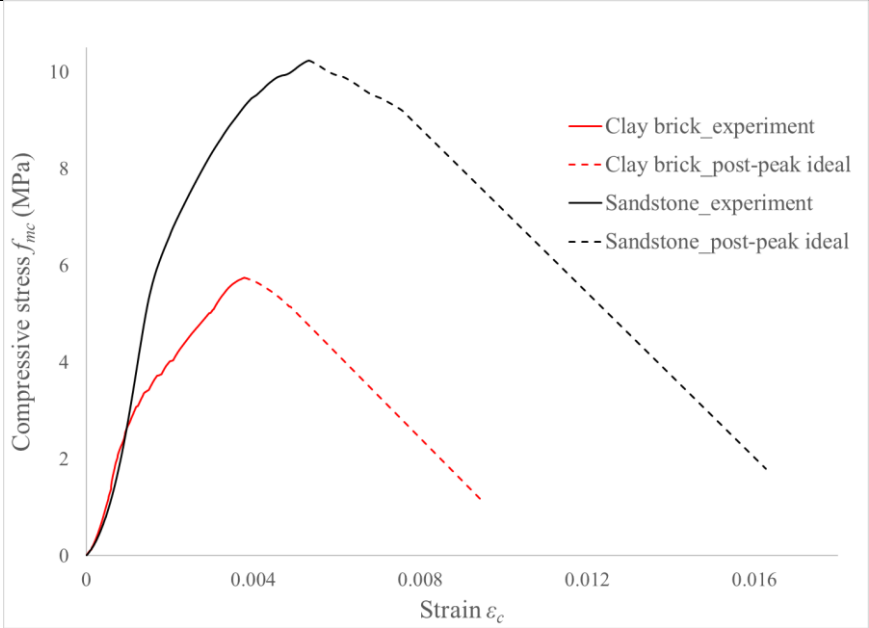


Figure 2: Stress versus strain curve of masonry prisms

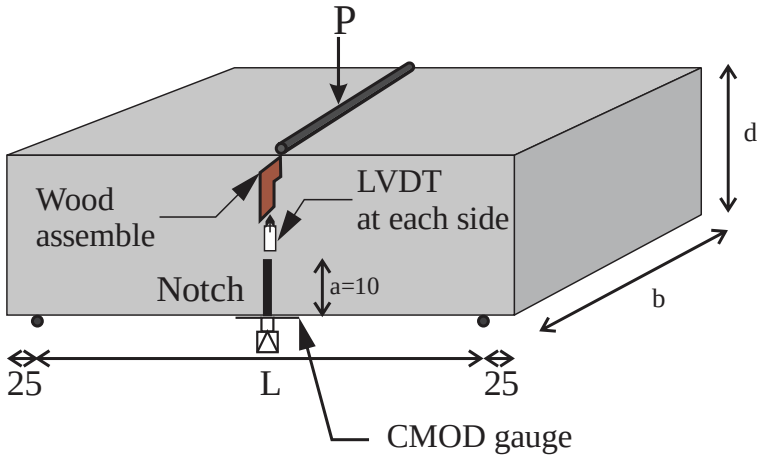
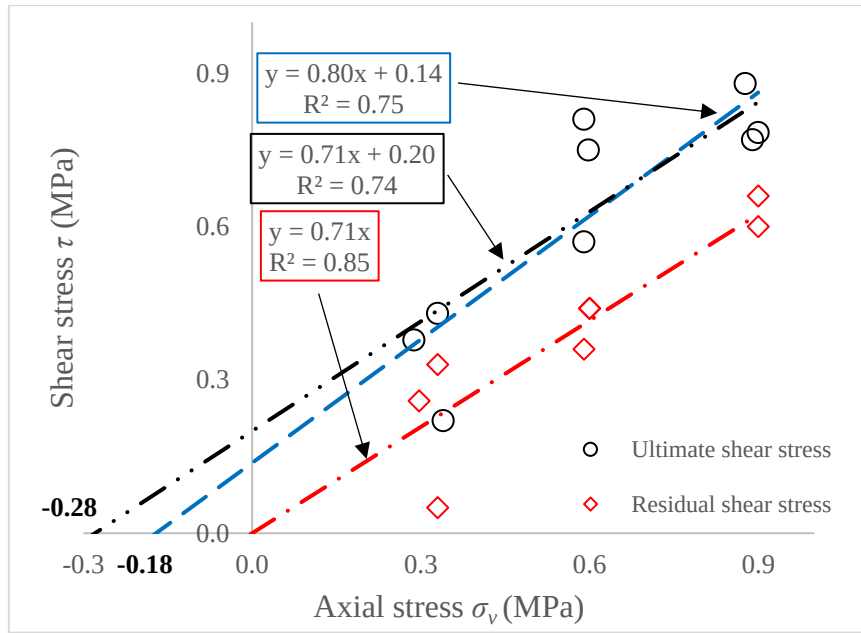


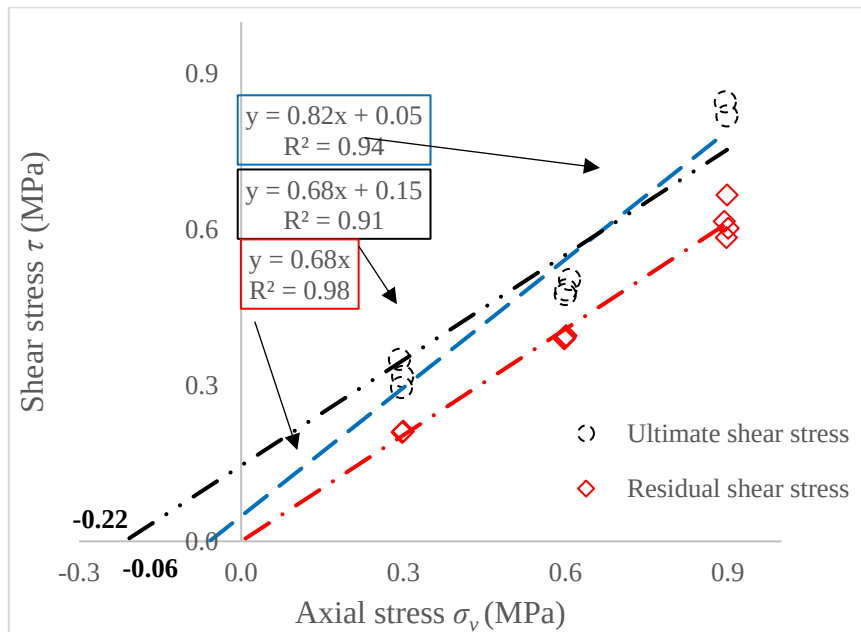
Figure 3: Three point bending test on the masonry unit



Figure 4: Bond wrench test set up for masonry couplet



(a) Clay brick



(b) Sandstone

Figure 7: Shear stress linear fit for masonry couplet

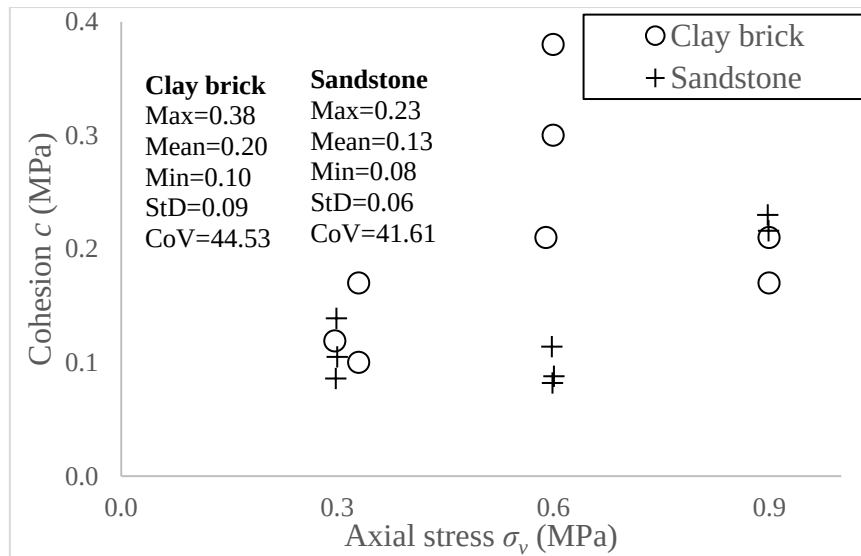
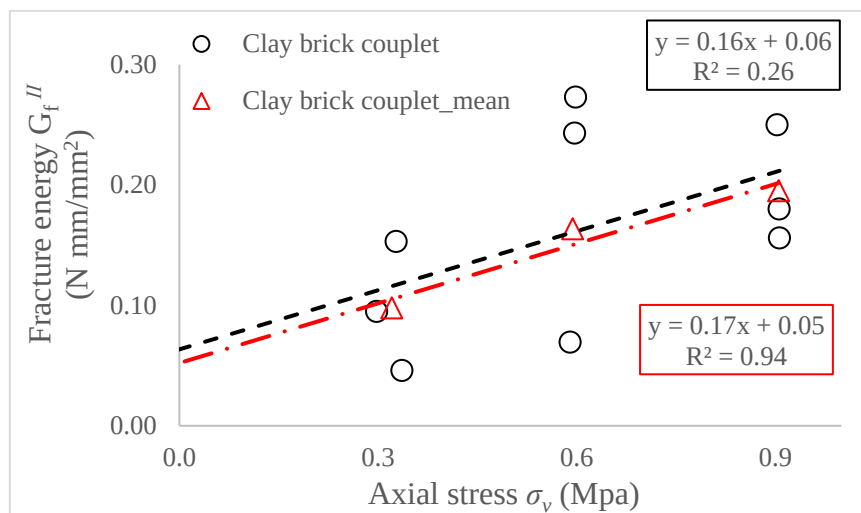
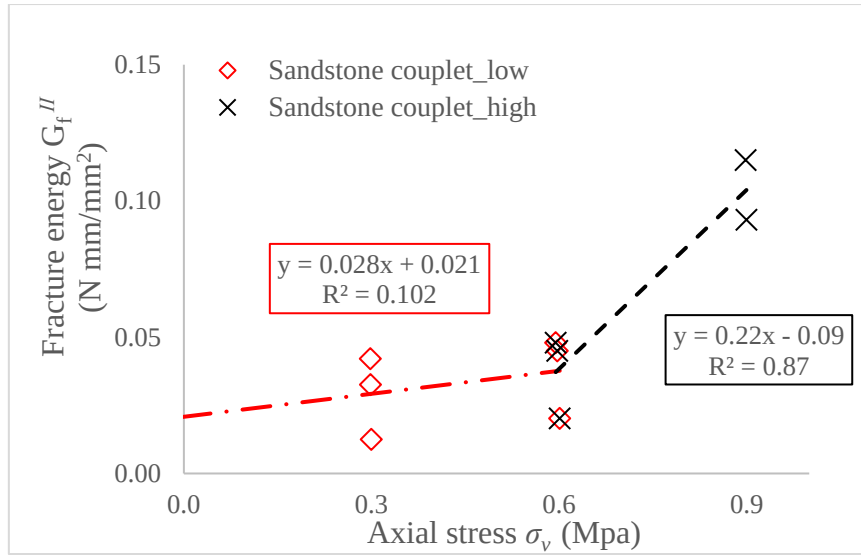


Figure 8: Variation of cohesion for masonry couplet



(a) Clay brick



(b) Sandstone

Figure 9: Fracture energy linear fit of masonry couplet

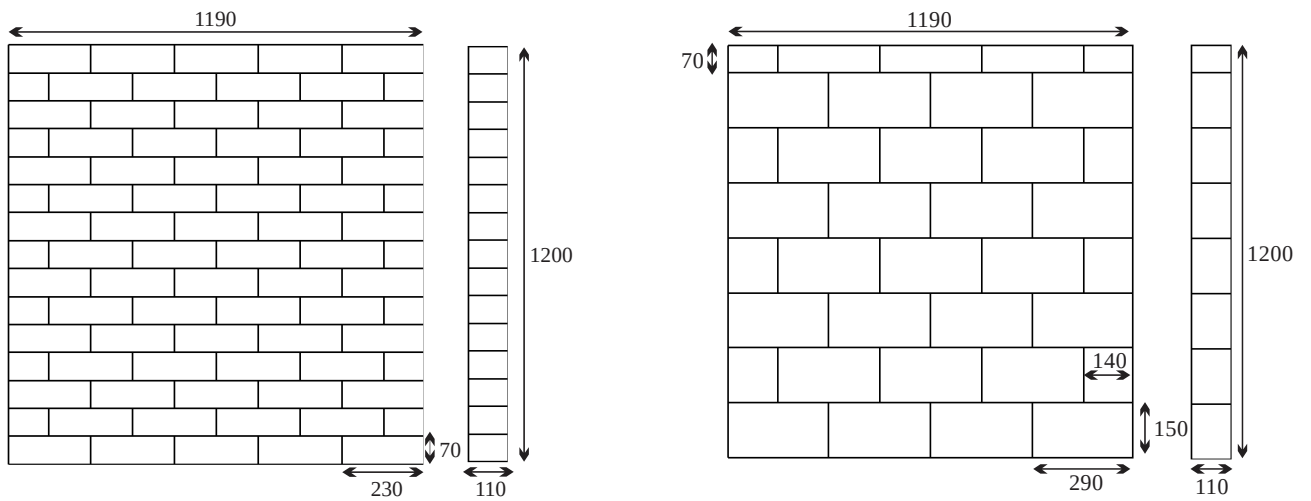
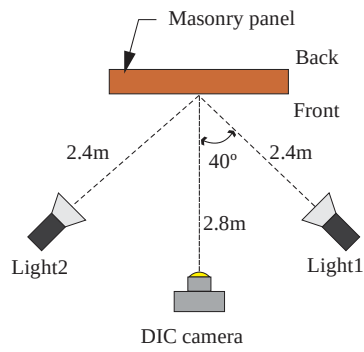


Figure 10: Geometry of masonry panels

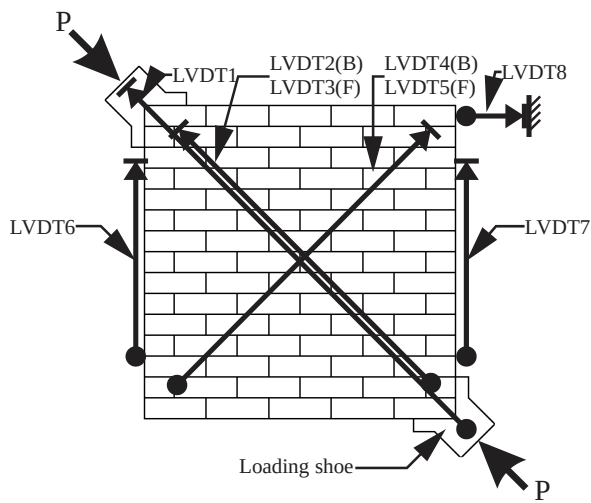


(a) Schematic view

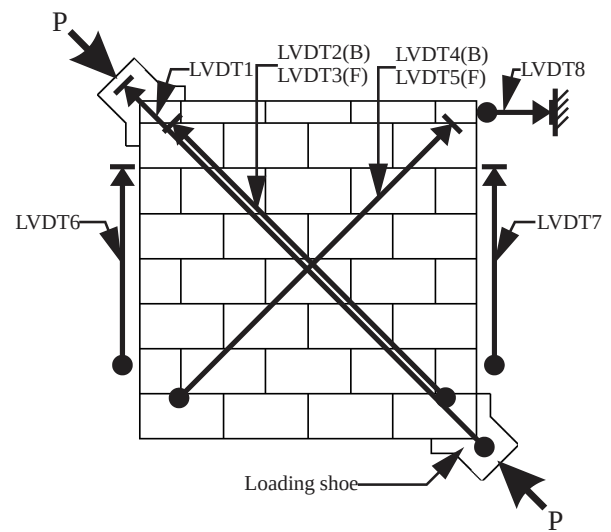


(b) Photographic view

Figure 11: DIC camera set up



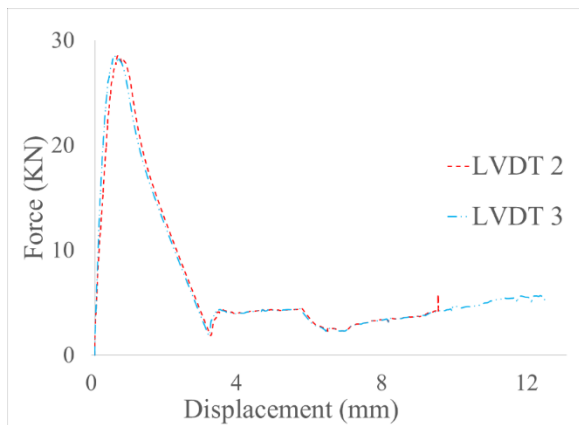
(a) Clay brick wall



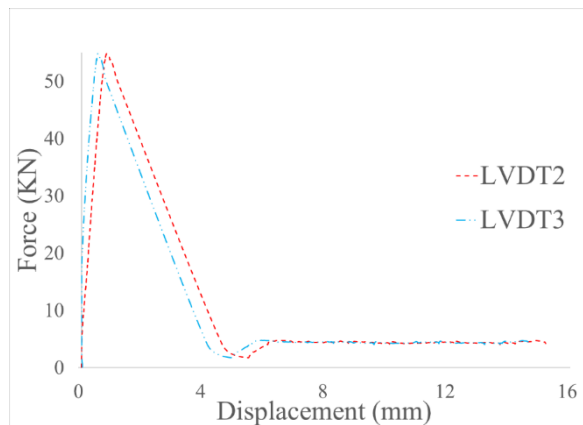
(b) Sandstone wall

Figure 12: LVDT instrumentation of diagonal compression test

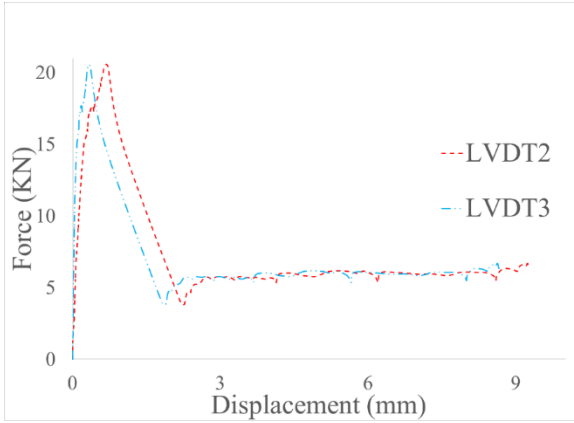
(F, B stand for front and back side)



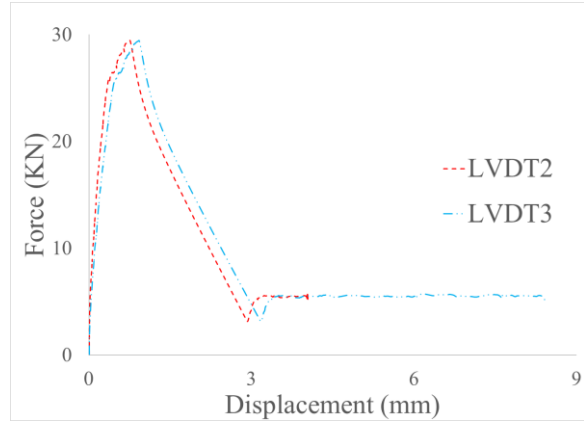
(a) DC-CBW1



(b) DC-CBW2



(c) DC-SSW1

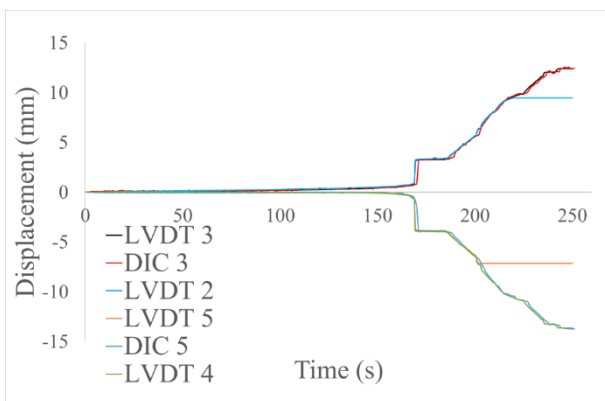


(d) DC-SSW2

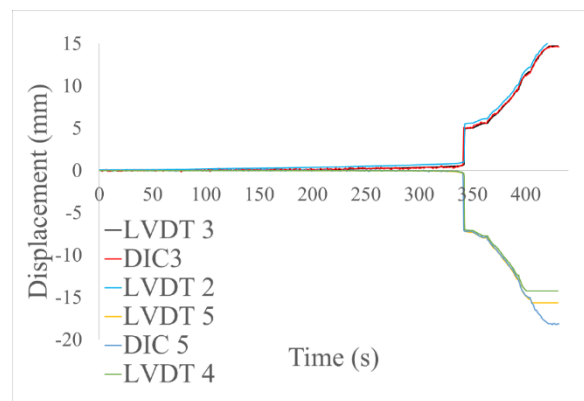
Figure 13: Force versus displacement response for masonry panels

Table 4: Shear modulus of masonry G , determined by ASTM standard

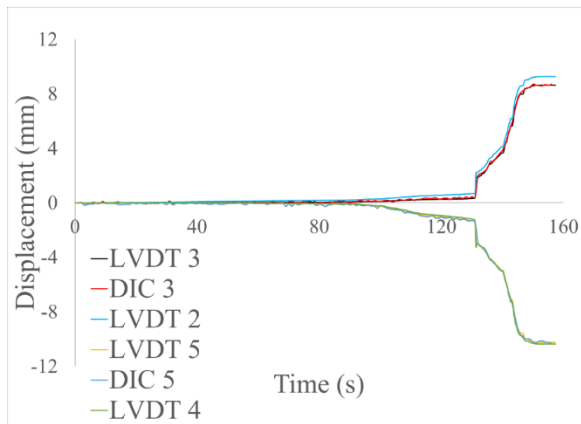
Masonry panel	Shear modulus of masonry, G (MPa)				f_{t_ASTM} (MPa)	G (MPa)
	LVDT 4&5 ΔH (mm)	LVDT 2&3 ΔV (mm)	LVDT 2&3 g_v (mm)	γ (%)		
DC-CBW1	0.21	0.58	1,360	0.058	0.15	264
DC-CBW2	0.14	0.67	1,360	0.059	0.29	500
DC-SSW1	1.23	0.50	1,340	0.13	0.11	85
DC-SSW2	1.84	0.84	1,340	0.20	0.16	78



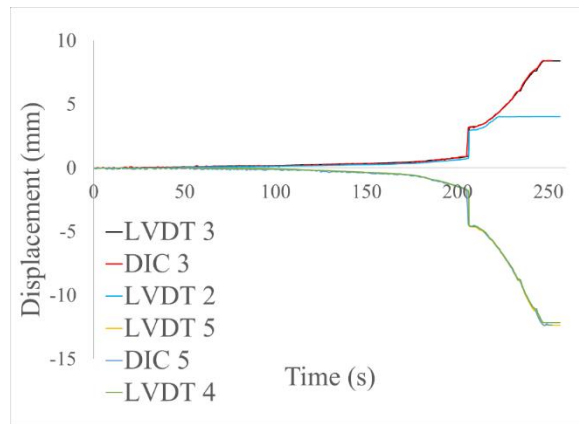
(a) DC-CBW1



(b) DC-CBW2

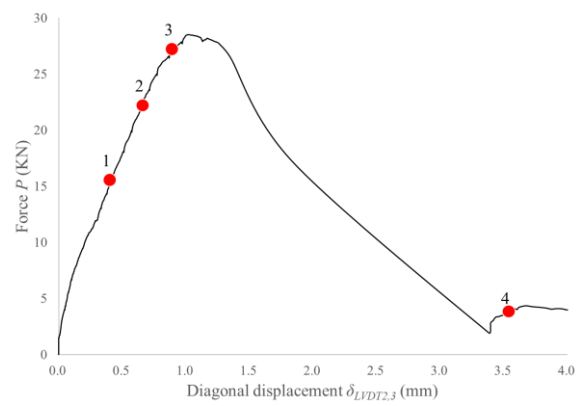


(c) DC-SSW1

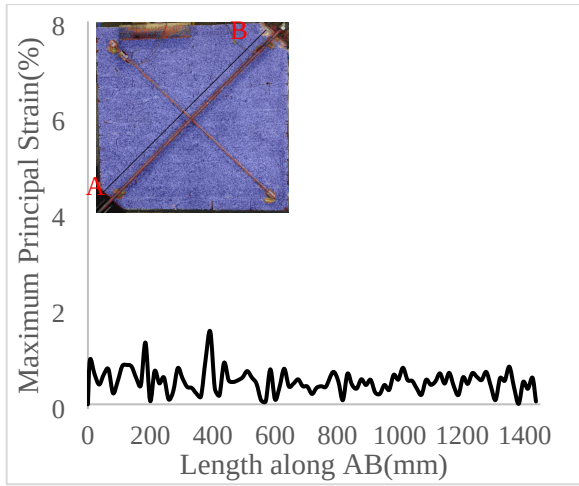


(d) DC-SSW2

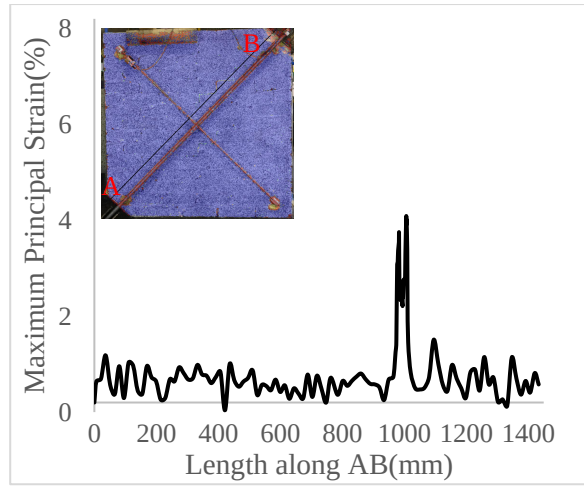
Figure 14: LVDT vs DIC for masonry panels



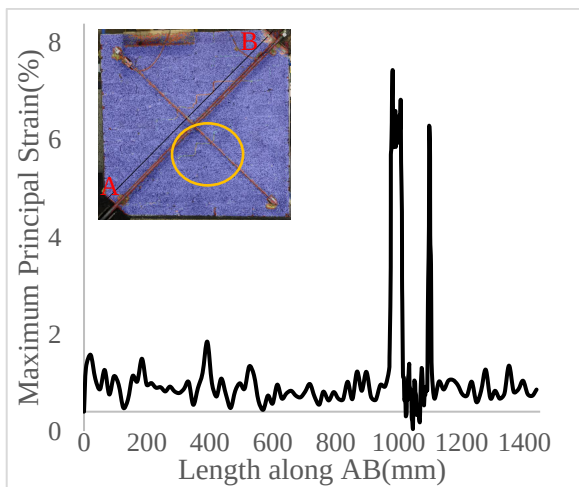
(a) DC-CBW1



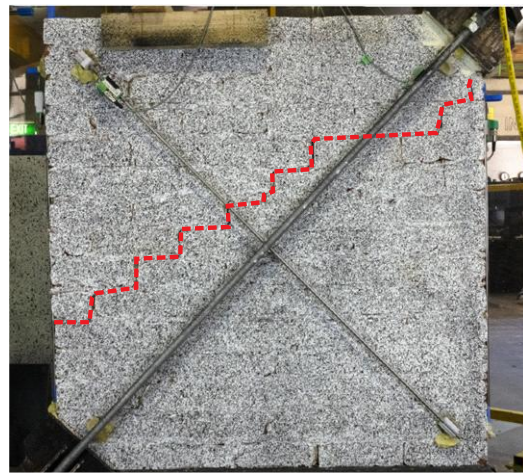
Stage 1



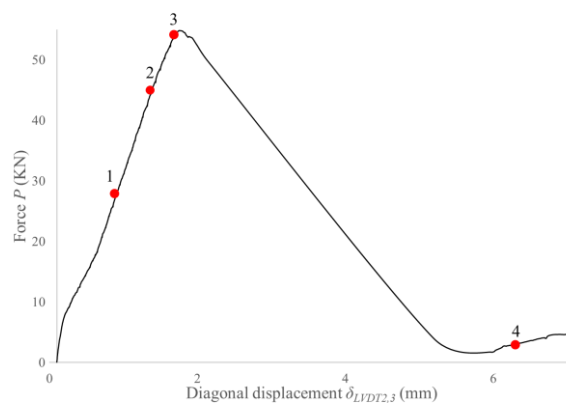
Stage 2



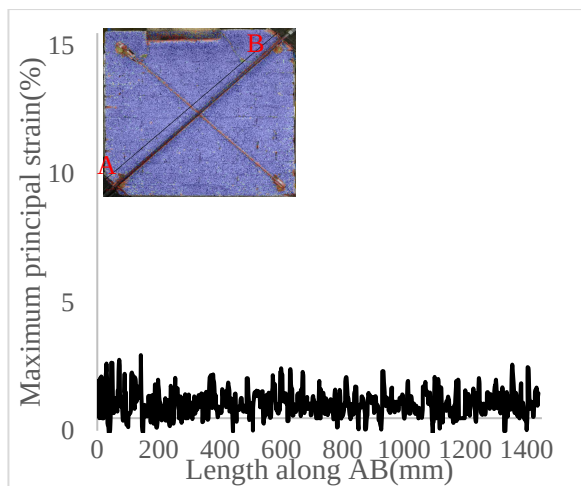
Stage 3



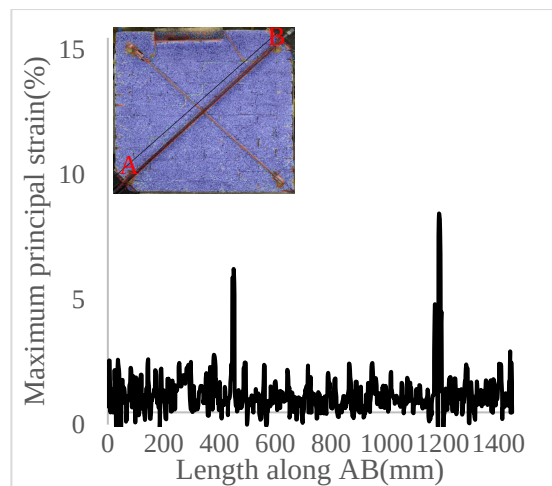
Stage 4



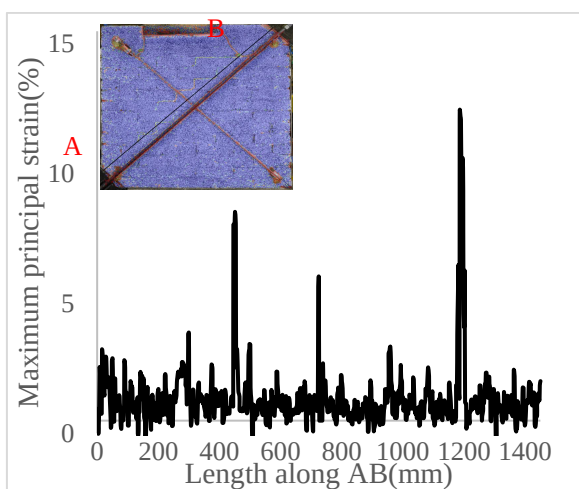
(b) DC-CBW2



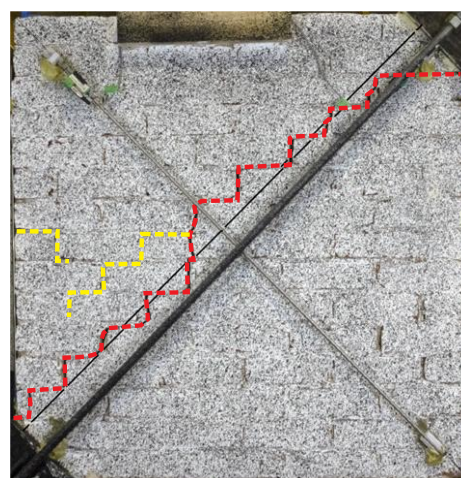
Stage 1



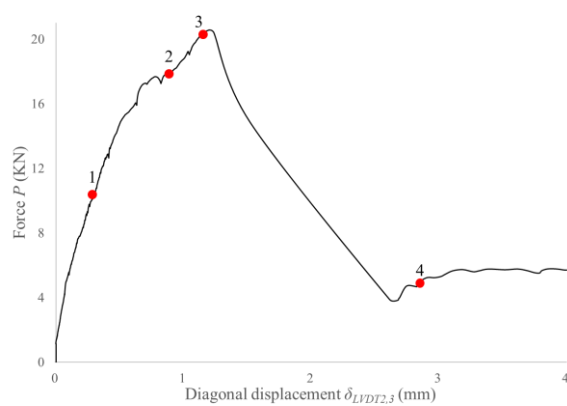
Stage 2



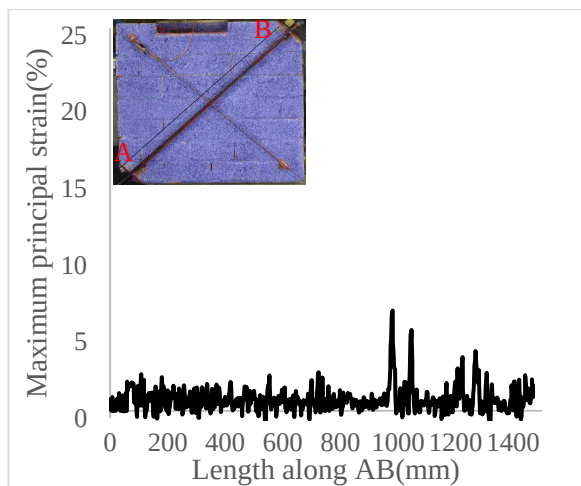
Stage 3



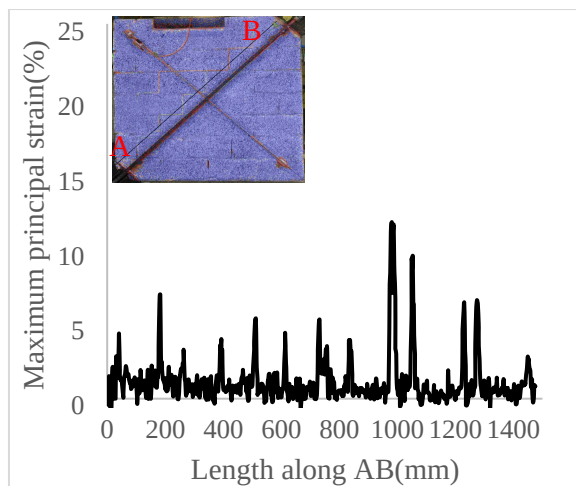
Stage 4



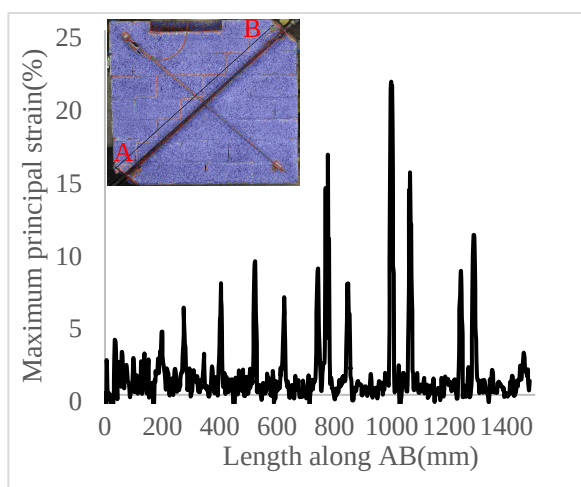
(c) DC-SSW1



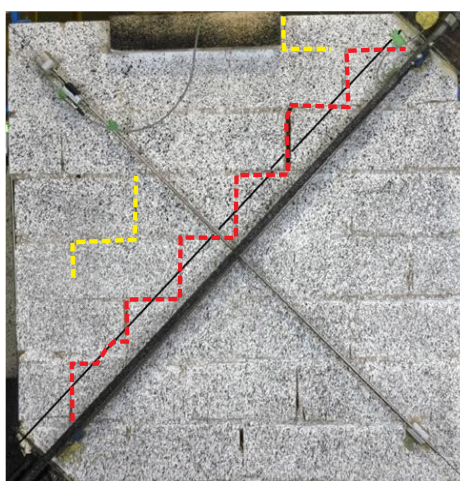
Stage 1



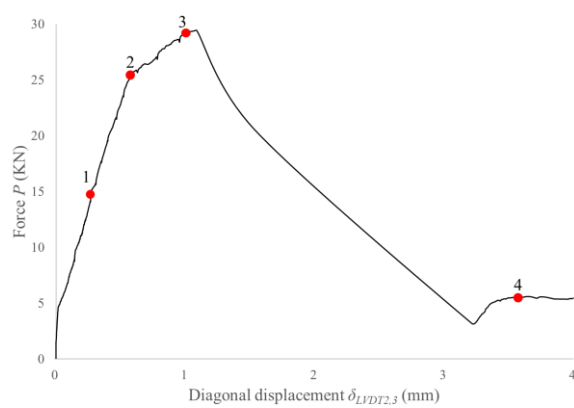
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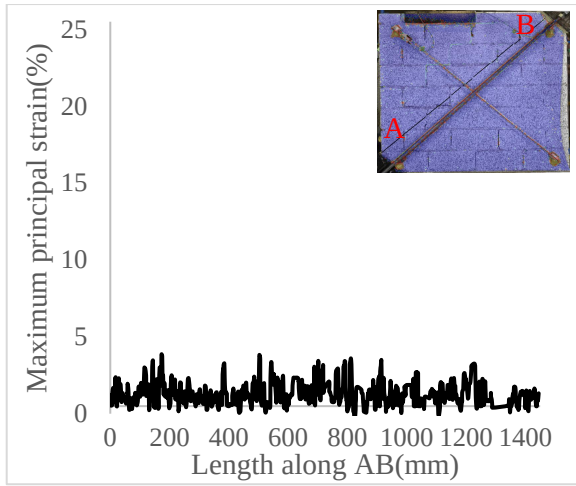
Stage 3



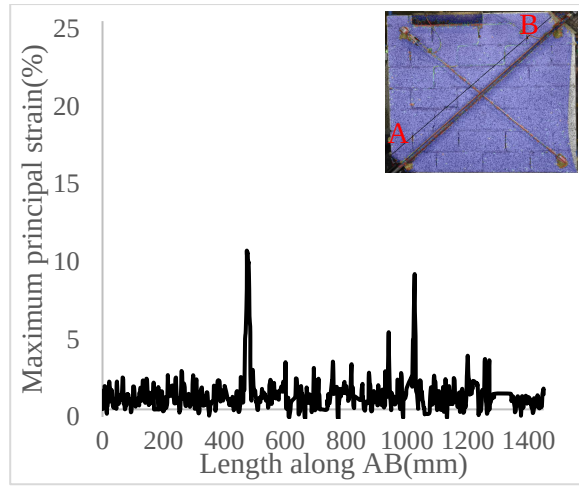
Stage 4



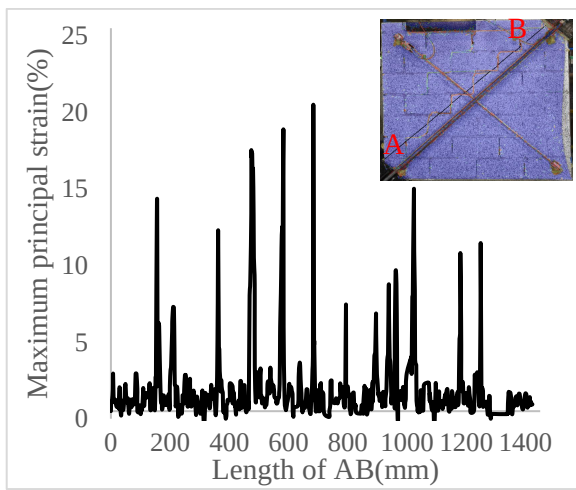
(d) DC-SSW2



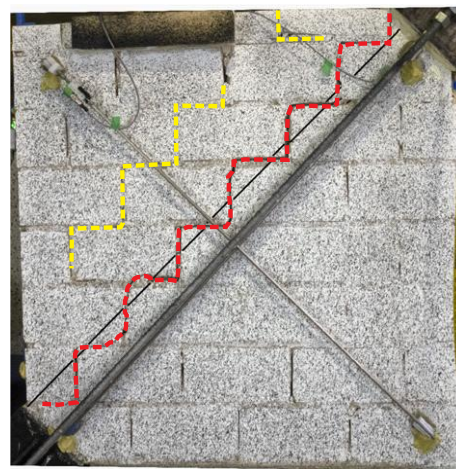
Stage 1



Stage 2

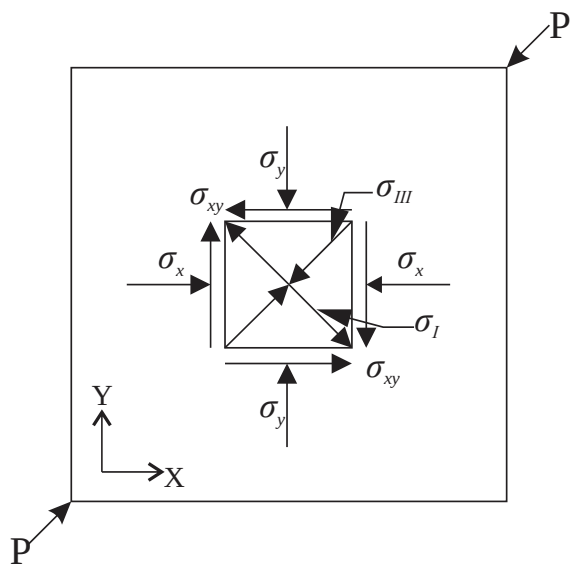


Stage 3

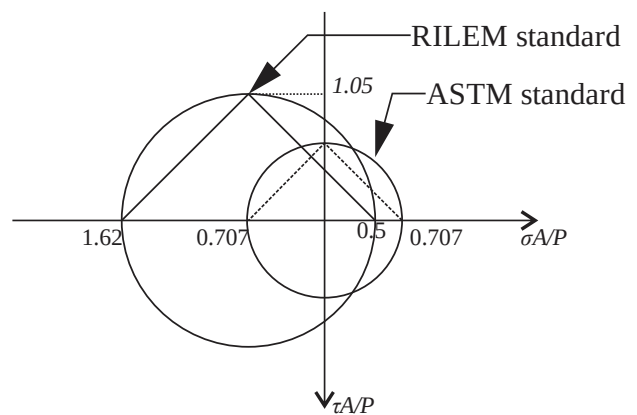


Stage 4

Figure 15: Strain evolution for masonry panels (Note that in stage 4, the highlighted red line represents major crack and the highlighted yellow line represents minor crack)

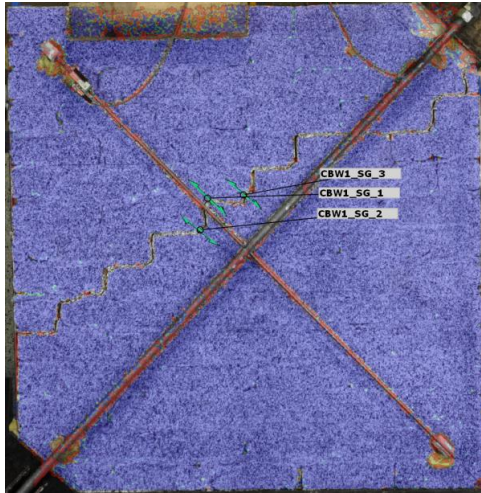


(a) Stress field

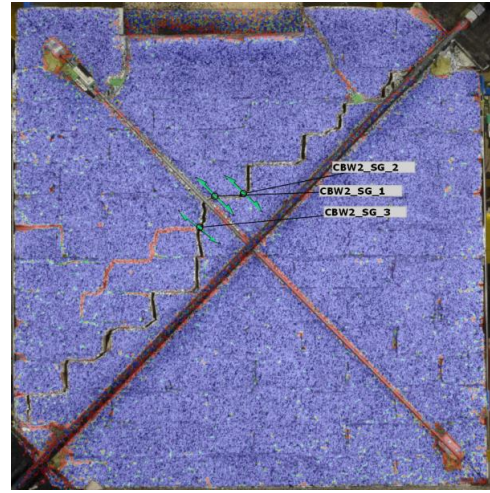


(b) Mohr coulomb circle by ASTM and RILEM

Figure 16: Identification of stress field and Mohr coulomb circle based on ASTM and RILEM standard



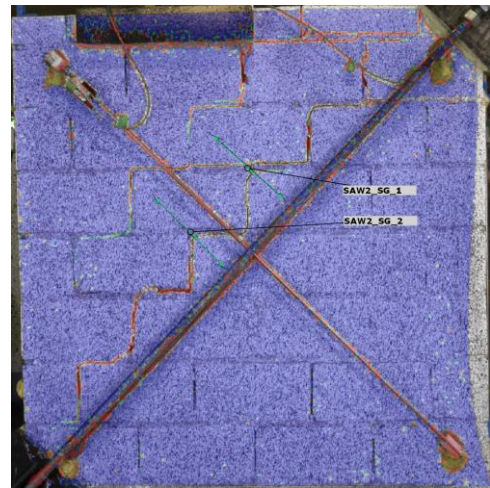
(a) Virtual gauges in CBW1



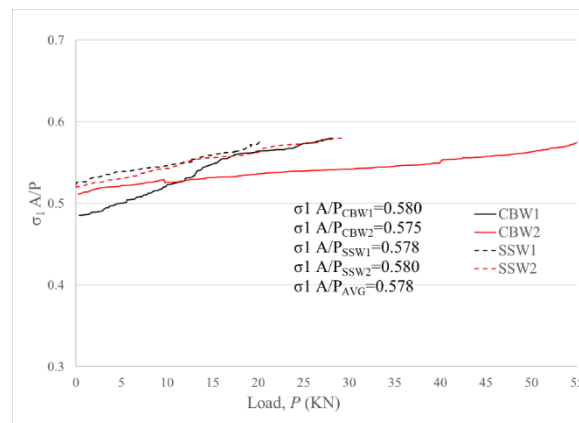
(b) Virtual gauges in CBW2



(c) Virtual gauges in SSW1



(d) Virtual gauges in SSW2

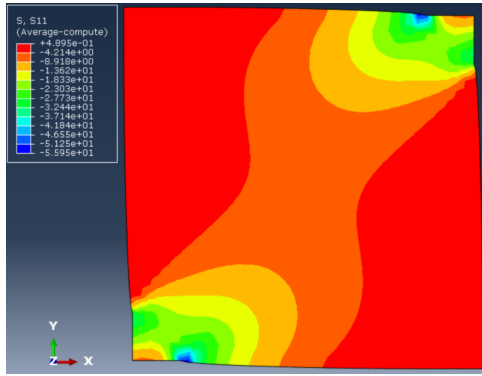


(e) Normalized maximum stress σ_I for masonry panels

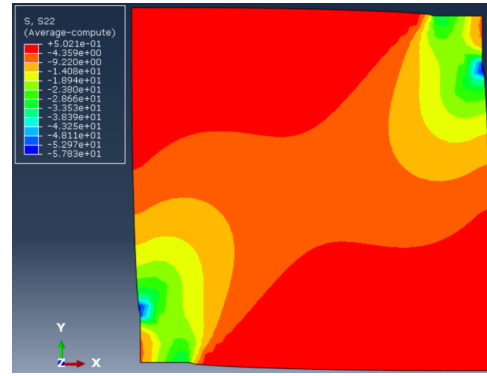
Figure 17: Normalized maximum principal stress by DIC virtual gauges for masonry

Elastic properties of masonry panel				
Masonry panel	Brick Unit	Mortar Joint	Masonry panel	
	E_u (MPa)	E_j (Mpa)	ν	E_m (Mpa)
Clay brick	9,085	436	0.15	2,272
Sandstone	10,573	436	0.15	3,206

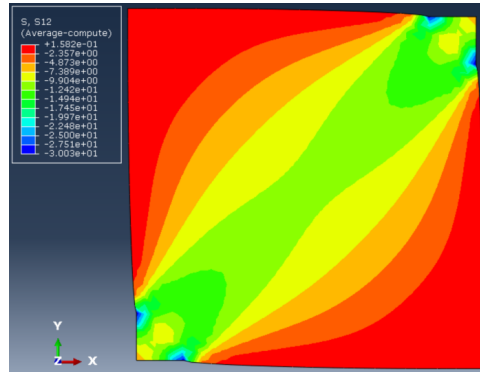
Table 5: Elastic properties of masonry



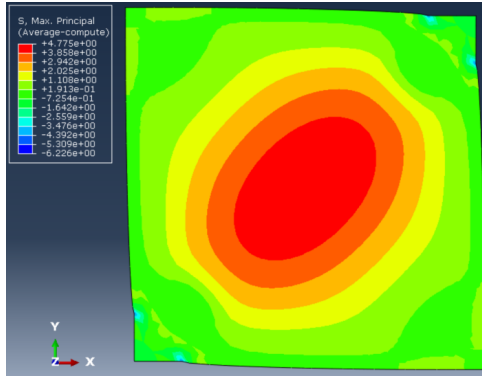
(a) σ_x



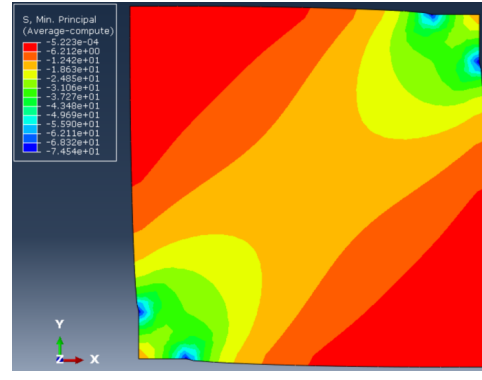
(b) σ_y



(c) σ_{xy}



(d) σ_I



(e) σ_{III}

Figure 18: Stress contour plot in elastic analysis

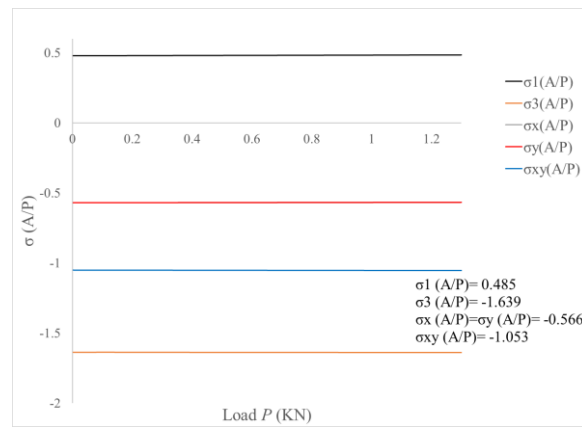


Figure 19: State of stress components in centre of panel

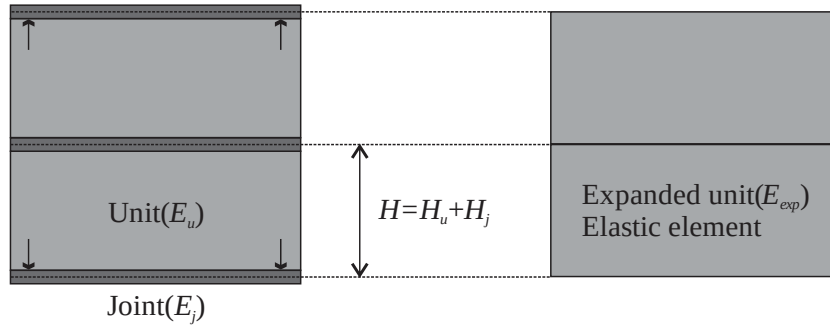
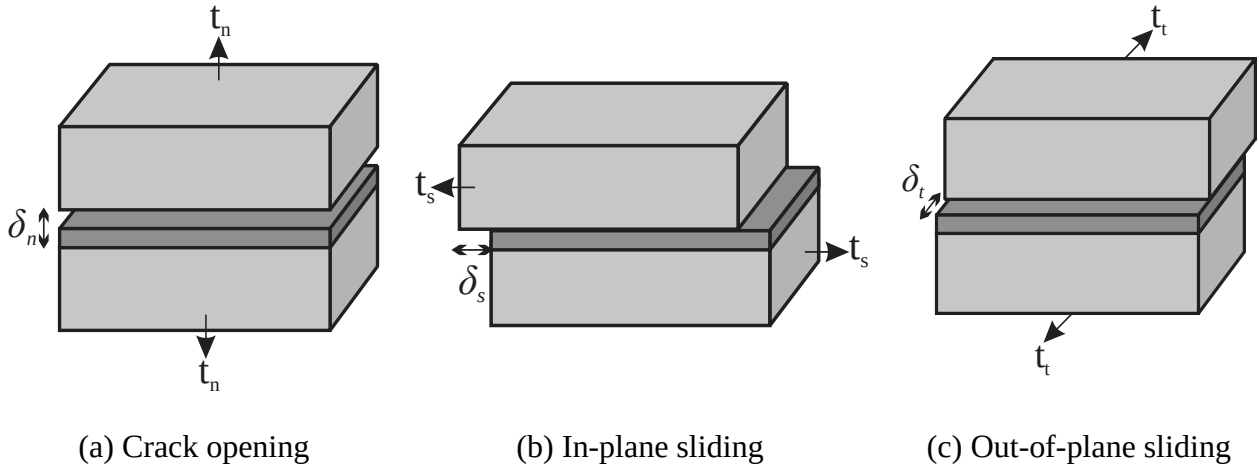


Figure 20: Expanded unit in simplified micro scale



(a) Tensile bond cracking (=Fracture mode 1)

(b) In-plane shear sliding (=Fracture mode 2)

(c) Out of plane shear sliding (=Fracture mode 3)

Figure 21: Bond failure along the joint interface

Stress = Traction	Displacement = Separation
t_n : Tensile stress along joint interface	δ_n : displacement of tensile bond cracking
t_s : In-plane shear stress along joint interface	δ_s : displacement of in-plane sliding
t_t : Out-of-plane shear stress along joint interface	δ_t : displacement of out-of-plane sliding

Table 6: Notation of fracture criterion

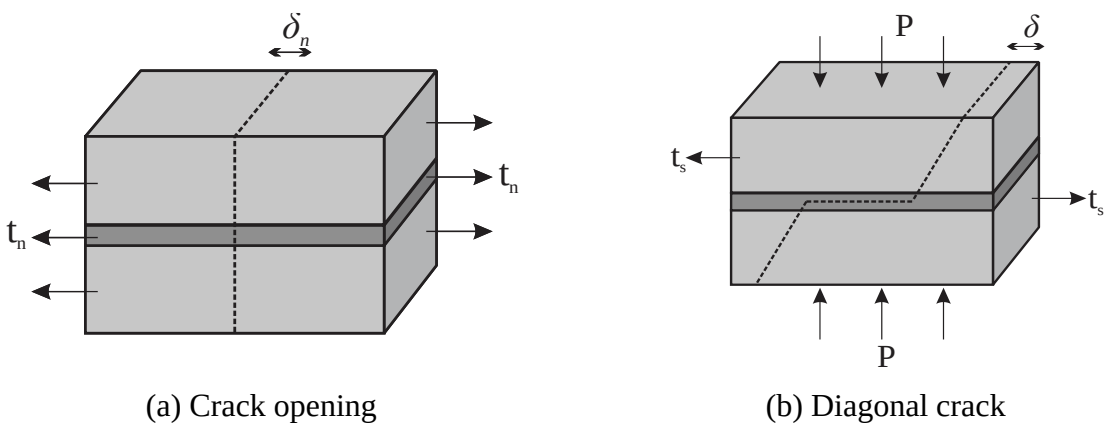


Figure 22: Masonry unit failure modes

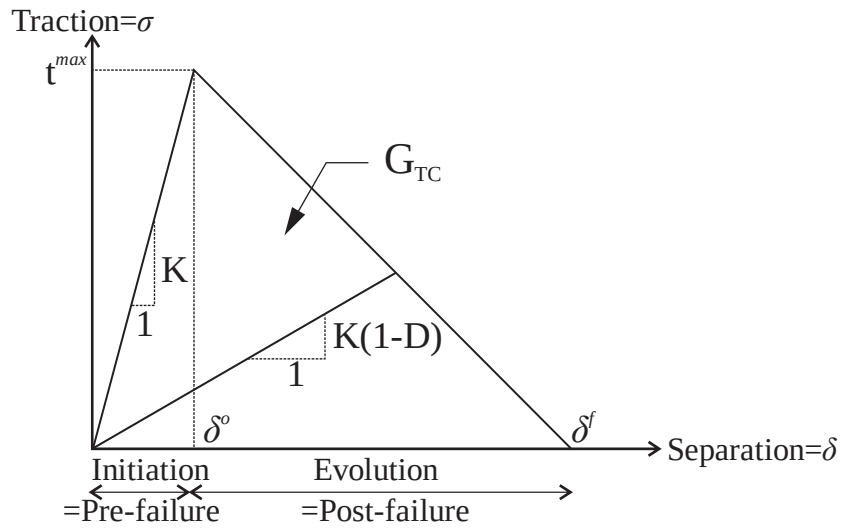


Figure 23: Traction separation law along the joint interface

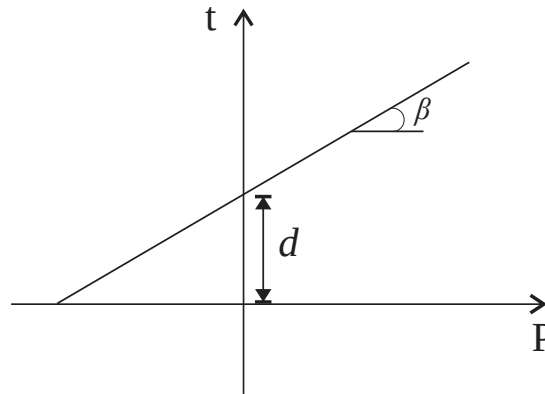


Figure 24: Yield criteria in p-t plane for Linear Drucker-Prager

Table 7: Linear Drucker Prager parameters

Linear Drucker-Prager parameters				
Material	Angle of friction, β	Flow stress ratio, K	Dilation angle, ψ	Compression yield stress, σ_c (MPa)
Clay brick	36	1	11.3	5.42
Sandstone				12.91

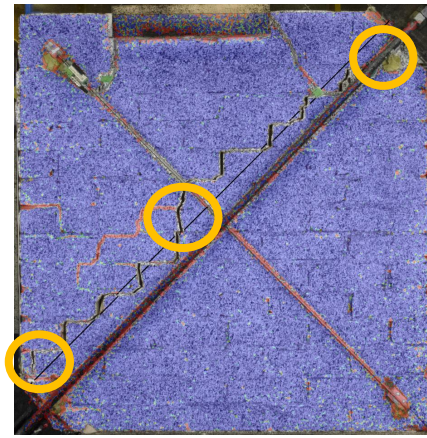
Table 8: Material properties of masonry unit

Material properties of masonry unit								
Masonry unit	Brick unit	Mortar joint	Expanded unit		Tension		Shear	
	E_u (MPa)	E_j (MPa)	ν	E_{exp} (Eq: 29) (MPa)	f_{ut} (MPa)	$G_f^{I_u}$ (N/mm)	τ_u (MPa)	$G_f^{II_u}$ (N/mm)
Clay brick	9,085	436	0.15	2,611	0.67	0.21	0.93	0.5

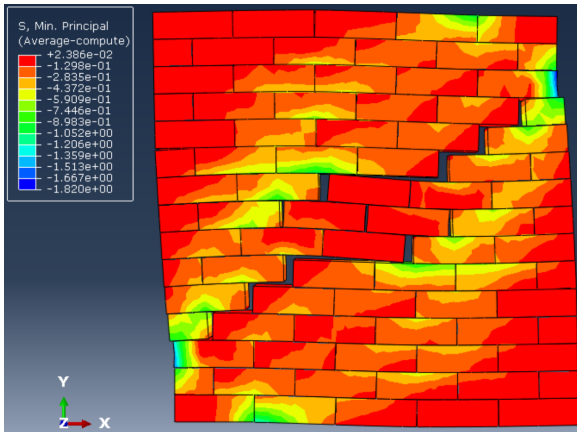
Sandstone	10,573	436	0.15	4,310	0.78	0.25	1.09	0.5
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Table 9: Material properties of joint interface

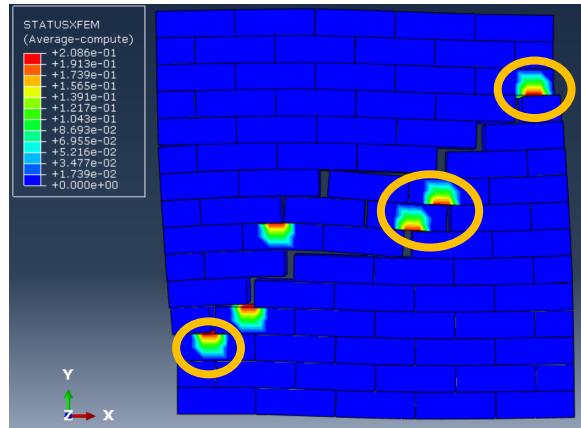
Material properties of masonry joint interface								
Masonry joint interface	Stiffness K (N/mm ³)			Tension		Shear		
	K_{nn}	K_{ss}	K_{tt}	t_n (Mpa)	G_f^I (N/mm)	c (MPa)	μ_d	G_f^{II} (N/mm)
Clay brick	7.37	3.2	3.2	0.24	0.012	0.2	0.71	0.052
Sandstone	11.59	5.04	5.04	0.06	0.012	0.15	0.68	0.021



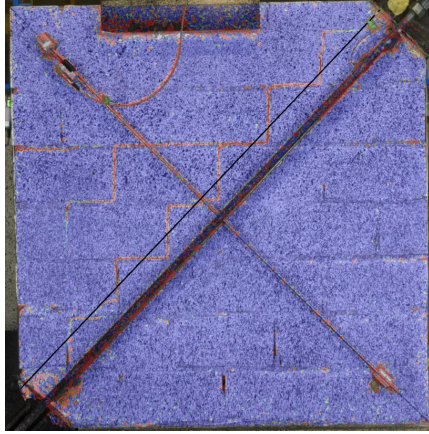
(a) Strain map for DC-CBW2



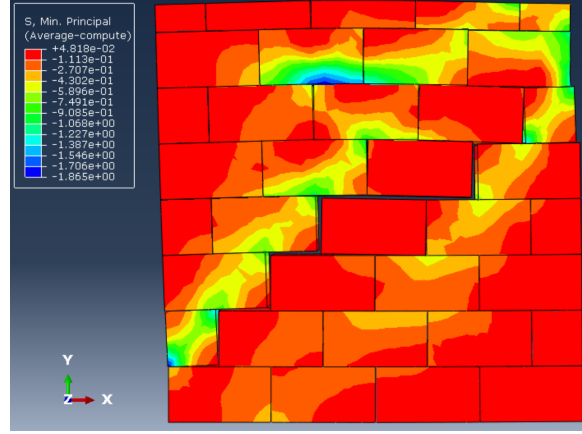
(b) Clay brick FEM σ_{III}



(c) Clay brick StatusXFEM

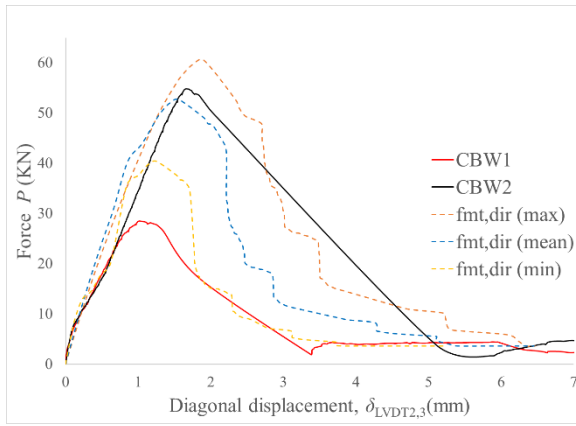


(d) Strain map for DC-SSW1

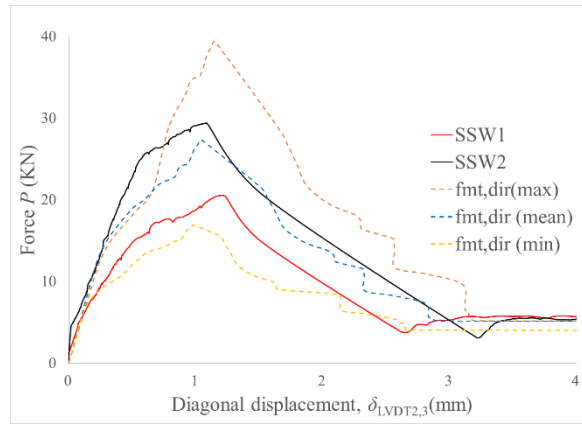


(e) Sandstone FEM σ_{III}

Figure 25: Crack propagation of DIC and FEM result



(a) Clay brick wall



(b) Sandstone wall

Figure 26: Force vs displacement response for masonry panel

Diagonal compression test and FEM result for clay brick wall					
	Experiment		FEM		
	DC_CBW1	DC_CBW2	$f_{mt,dir}$ (Max)	$f_{mt,dir}$ (Mean)	$f_{mt,dir}$ (Min)
$f_{mt,dir}$ (MPa)	-	-	0.11	0.08	0.05
P_{max} (KN)	28.52	54.84	60.52	53.67	40.39
f_{t_ASTM} (MPa)	0.153	0.295	0.326	0.289	0.217
f_{t_RILEM} (MPa)	0.108	0.209	0.230	0.204	0.154
f_{t_DIC} (MPa)	0.126	0.242	0.267	0.237	0.178

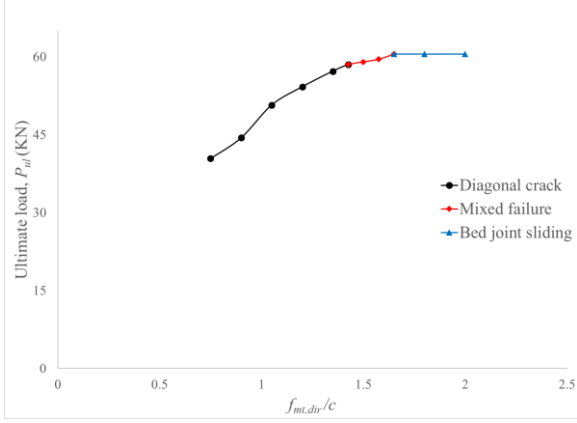
(a) Clay brick wall

Diagonal compression test and FEM result for sandstone wall					
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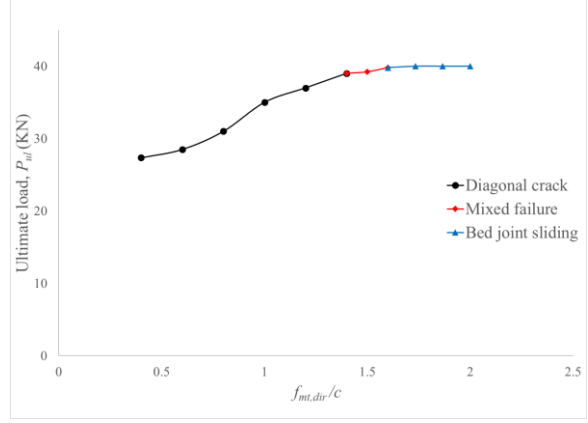
	Experiment		FEM		
	DC_SSW1	DC_SSW2	$f_{mt,dir}$ (Max)	$f_{mt,dir}$ (Mean)	$f_{mt,dir}$ (Min)
$f_{mt,dir}$ (MPa)	-	-	0.06	0.02	0.003
P_{max} (KN)	20.57	29.41	39.39	27.35	16.90
f_{t_ASTM} (MPa)	0.109	0.156	0.209	0.145	0.0899
f_{t_RILEM} (MPa)	0.0774	0.111	0.148	0.103	0.0636
f_{t_DIC} (MPa)	0.0897	0.128	0.172	0.119	0.0737

(b) Sandstone wall

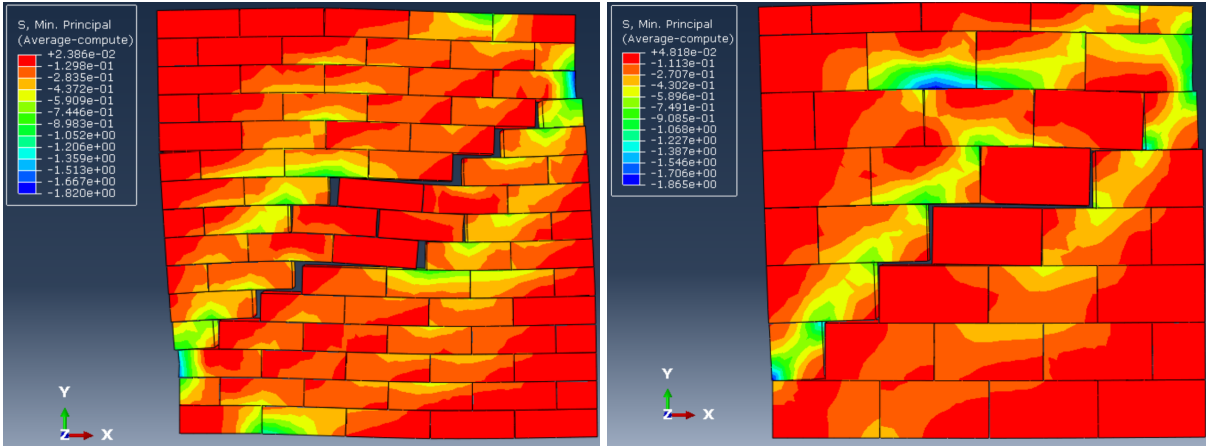
Table 10: Diagonal tensile strength, f_t for masonry panels



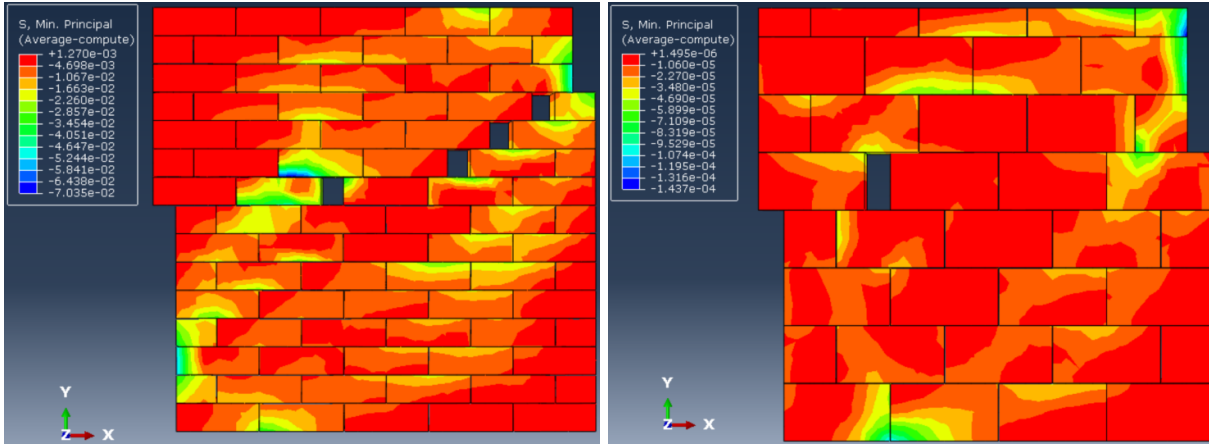
(a) $f_{mt,dir}/c$ vs P_{ul} for clay brick wall



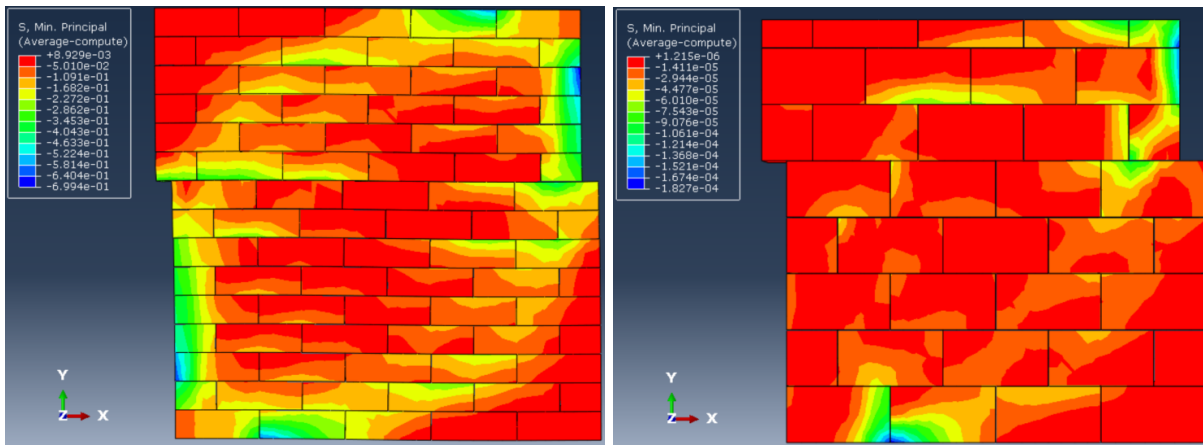
(b) $f_{mt,dir}/c$ vs P_{ul} for sandstone wall



(c) Diagonal crack



(d) Mixed failure



(e) Bed joint sliding

Figure 27 : Influence of $f_{mt,dir}/c$ and failure pattern for masonry walls

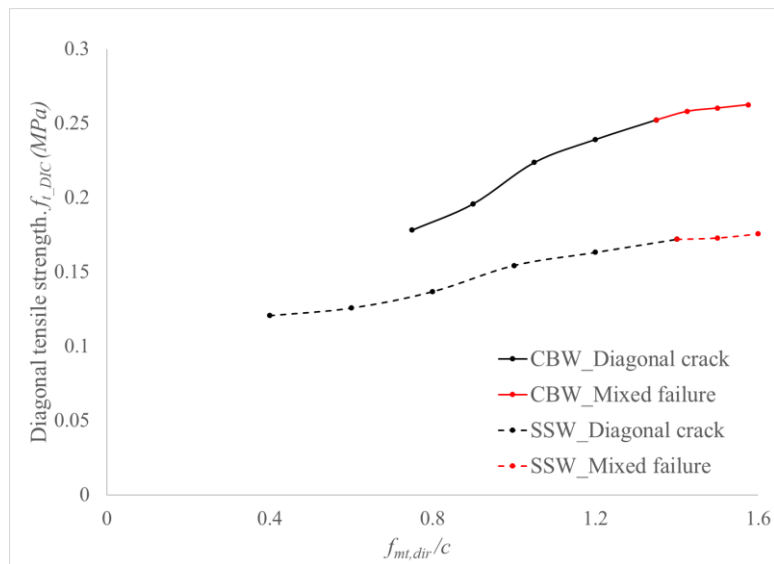


Figure 28: Diagonal tensile strength, $f_{L,DIC}$ vs $f_{mt,dir}/c$ for masonry panels

Chapter 3. Journal Paper 2- Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis

Introduction

This chapter contains the paper “Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis” which focuses on the prediction of force capacity for masonry panel by numerical study. The masonry has three failure modes namely flexural rocking crack, diagonal crack and shear sliding and force capacity of diagonal crack commonly estimated by proposed equation by Turnšek and Čačovič. However, it is still remained unclear how the proposed equation well capture the force capacity of the panel so many researchers conducted numerical study. Previous researchers have carried out the numerical model in DIANA or ABAQUS but has been mentioned by many that the analysis does not take account into effect of fracture energy under different pre-compression levels. Current study has adopted the fracture energy as a linear form so that the sensitivity analysis for failure modes have been performed. The numerical model deals with clay brick and sandstone masonry panel which is commonly used in Australia. The force capacity of the panel is evaluated based on AS 3700 and the equation proposed by Turnšek and Čačovič .

List of manuscripts

Park, Y.K., Vaculik, J., Sheikh, A.H., Griffith, M.C., 2019. Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis.

Statement of Authorship

Title of Paper	Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis
Publication Status	☐ Published ☐ Accepted for Publication ☐ Submitted for Publication ☒ Unpublished and Unsubmitted work written in manuscript style
Publication Details	Park, Y.K., Vaculik, J., Sheikh, A.H., Griffith, M.C., 2019. Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis.

Principal Author

Name of Principal Author (Candidate)	Young Kol Park		
Contribution to the Paper	Developed the test methodologies, performed analysis on all samples, interpreted data, and wrote manuscript.		
Overall percentage (%)	70		
Certification:	This paper reports on original research I conducted during the period of my Higher Degree by Research candidature and is not subject to any obligations or contractual agreements with a third party that would constrain its inclusion in this thesis. I am the primary author of this paper.		
Signature		Date	27/09/2019

Co-Author Contributions

By signing the Statement of Authorship, each author certifies that:

- the candidate's stated contribution to the publication is accurate (as detailed above);
- permission is granted for the candidate to include the publication in the thesis; and
- the sum of all co-author contributions is equal to 100% less the candidate's stated contribution.

Name of Co-Author	Professor Michael Griffiths		
Contribution to the Paper	Supervised development of work, helped in data interpretation and manuscript evaluation.		
Signature		Date	27/09/2019

Name of Co-Author	Associate Professor Abdul Hamid Sheikh		
Contribution to the Paper	Supervised development of work, helped in data interpretation and manuscript evaluation.		
Signature		Date	26/09/2019

Please cut and paste additional co-author panels here as required.

Evaluation of force capacity of URM wall subjected to in-plane loading by finite element analysis

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1. INTRODUCTION

Masonry wall is the oldest structure and most well-known construction material in the world. There are various benefits in using masonry for construction, one of which is for its flexibility. There are different types of masonry brick and mortar with each one having their own specification of strength effect on the damage propagation of masonry structure. Thus, there is a large variety of masonry structures, which can be adopted in multiple construction fields. Up to now, many studies have been carried out to predict force[3-5],[14], [22] or drift capacity[1-2] of unreinforced masonry(URM) wall by numerical modelling. It is important to predict the capability of URM wall to design panel appropriately so that it can resist to severe natural disaster such as earthquake or wind load. The main challenge in numerical approach is defining the interface of mortar joint . Generally, the crack propagates along the joint interface rather than unit or mortar so there has been many attempts carried out to numerical study of masonry[4-5], [7-8] but defining the parameter of fracture energy remains unclear. The fracture energy is not a constant parameter as studied by Pluijim [6]. The author provides deep insight of mechanical properties of joint interface by conducting direct tensile strength and shear couplet test. Based on the reported data, the relationship of fracture energy of joint interface with axial stress has linear form. The aim of current research project is to assess the force capacity of URM wall under different pre-compression levels and influence of aspect ratio. In order to achieve the objectives, ABAQUS software packages were employed to conduct finite element method(FEM) analysis.

2. Numerical modelling strategies for masonry panel

Previous research of FEM model has been established into two strategies in field of masonry. The two approaches are defined based on modelling scale of masonry assemblage. Namely, one is homogeneous approach and the other one is heterogeneous approach. The flow chart of approaches provided in Figure 1. The figure presented modelling strategies for masonry in two scales and techniques. It is noted that techniques to define joint interface is not limited cohesive behaviour, but current study focuses on cohesive behaviour.

The homogenous approach considers the masonry assemblage as isometric material, so that the unit-joint interfaces are smeared out in continuum element. Namely, macro modelling method saves significant amount of computational time. Thus, brick unit, mortar and joint interface are treated as composite material. The method is suitable for large scale of masonry structure to examine overall behaviour of masonry, but the output result may not be reliable since the masonry is a non-homogenous structure.

The main difference between heterogeneous approach and homogenous approach is whether considering joint interface as discontinuous elements or not. Since masonry failure is mainly governed by failure of joint interface, heterogeneous approach provides more realistic trend of failure mechanism. The heterogeneous modelling strategies can be defined into two strategies based on level of complexity. Namely, detailed micro modelling and simplified micro modelling introduced as described in Figure 2 and both strategies consider brick and mortar as a separate element. As shown in Figure 2, detailed micro strategy considers masonry unit, mortar and joint interface by discontinuous elements but simplified micro strategy, considers mortar joint in different way. The modelling approach considers thickness of mortar as finite thickness and then, expand unit in horizontal, vertical directions. The joint interface in simplified micro model lumped unit and joint interface so the stiffness is defined as average stiffness of masonry. The limitation of simplified micro approach is neglecting Poisson effect of mortar [9]. However, the geometry of walls are not changed since the masonry unit expanded in horizontal, vertical direction.

As shown in table 1, each of modelling technique has advantage and disadvantage in terms of complexity, accuracy and computational cost. The proposed modelling for this study employed the simplified micro modelling due to the simplicity of modelling and accuracy of the result and carried out surface based cohesive behaviour to identified joint interface. The Poisson's ratio of masonry taken 0.15 as previously used by Lorenzo [9], Bolhassani [7] and Kurdo [8]. The simplified micro modelling is not including the Poisson's effect of the mortar but still there is no significant differences with the other applications according to validation as reported by Lorenzo [9].

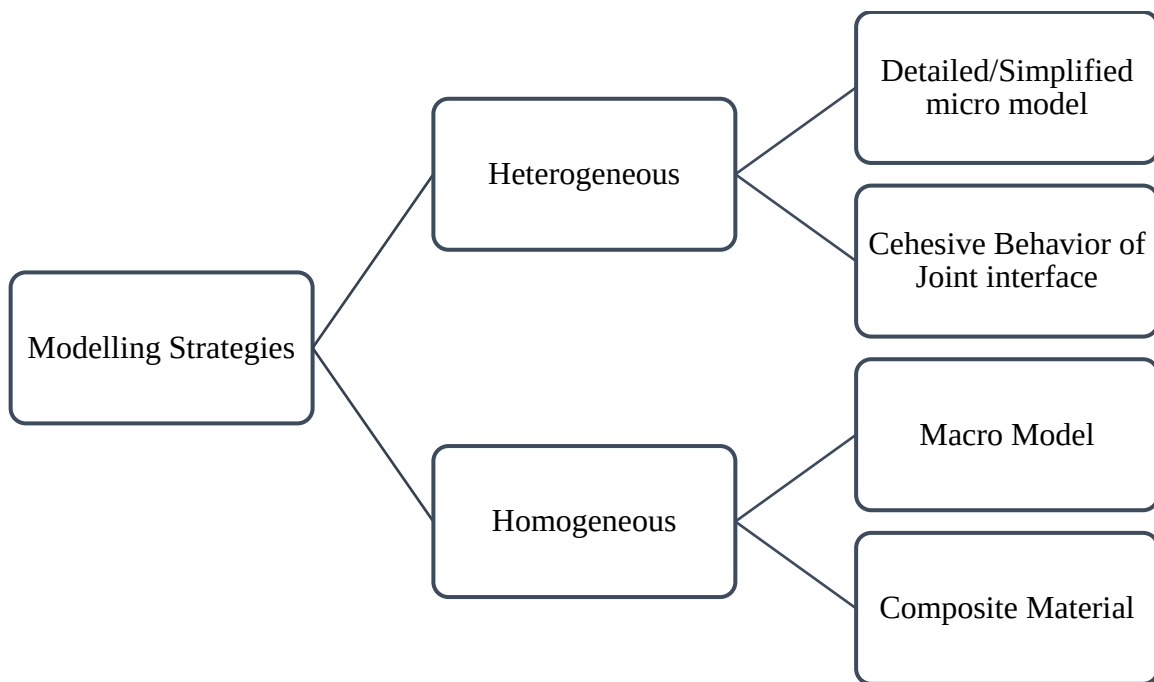


Figure 1: Flow chart of modelling strategies

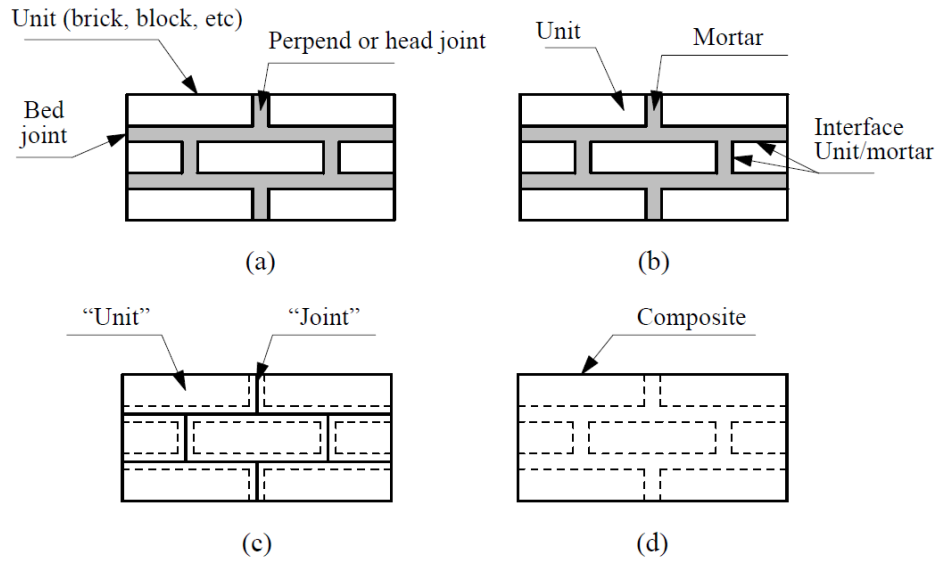


Figure 2: Masonry wall modelling approaches (Lourenco, Paulo. 1996)

Modelling Approach	Level of Complexity	Accuracy of Result
Detailed micro modelling	High	Accurate
Simplified micro modelling	Middle	Intermediate
Macro modelling	Low	Low

Table 1: Comparison of Modelling Approach

2.1 Elastic element of expanded unit

The elastic modulus of the expanded units must be adjusted and should have an equivalent elastic stiffness to the original masonry assemblage as shown in Figure 3. It is to be determined by taking elasticity of modulus of original masonry unit and mortar and geometry of the masonry assemblage into account. For this purpose, Eq: 1 is proposed based on the assumption of a stack bond between masonry units and uniform stress distribution in masonry constituents. Based on literature review, the proposed Eq: 1 is also corresponding to van der Plujim [6] and Kurdo et al [8].

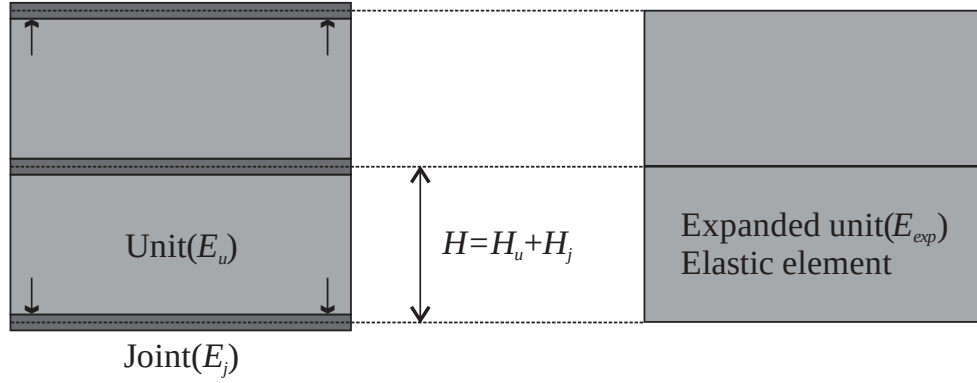


Figure 3: Expanded unit in simplified micro scale

$$E_{\text{exp}} = \frac{HE_uE_j}{H_uE_j + H_jE_u} \quad (1)$$

2.2 Traction separation law

One of the aims of this research is to investigate the post-peak behaviour of URM structure by FEM analysis. For this sake, simplified micro modelling approach was carried out and cohesive behaviour adopted in joint interface. The interaction of cohesive zone is function of displacement on the layer of crack propagation. The cohesive zone has been studied by Dugdale [10] and developed by Barenblatt [11] for the fracture mode of brittle material. The surface based cohesive behaviour was suitable for the bonded interfaces with negligible thickness. As discussed in the section 2, the simplified micro model addresses the similar strategy into joint interface. The modelling strategy considers the mortar joint as a finite thickness and expand the brick element in vertical and horizontal directions. The contact of expanded units behaves very similar to glue, so the interaction surface is a bonded area.

In order to capture pre-failure and post failure of joint interface, surface based cohesive model has been adopted and it is defined by damage criterion in ABAQUS [17]. As shown in Figure 4, the damage criterion is addressed with traction separation law and it corresponds to stress versus displacement response. Namely pre-failure stage called damage initiation and post failure called damage evolution. The damage initiation can be represented by either maximum stress or strain ratio or quadratic stress or stain ratio and to be able to

capture realistic trend, current model selects quadratic stress criterion as written in Eq: 2. The Macaulay bracket is expressed in the Eq 2 to neglect the compressive stress in the masonry joint along the nominal direction.

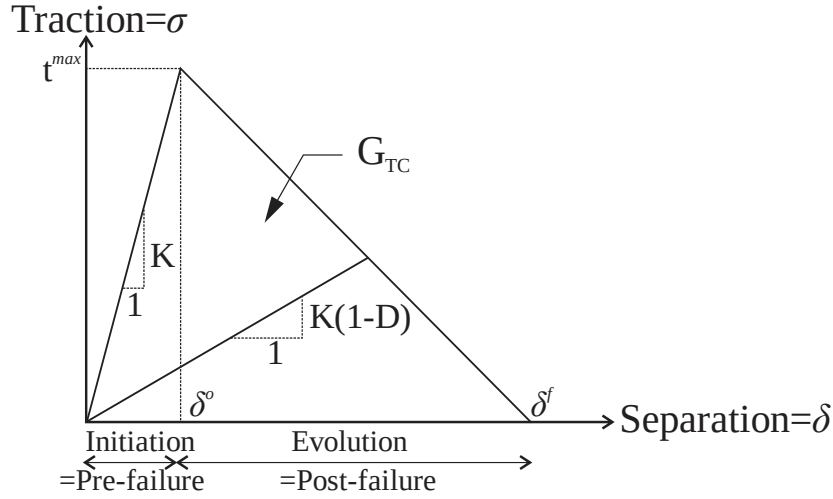


Figure 4: Traction separation law along the joint interface

$$\left\{ \frac{\langle t_n \rangle}{t_n^{\max}} \right\}^2 + \left\{ \frac{t_s}{t_s^{\max}} \right\}^2 + \left\{ \frac{t_t}{t_t^{\max}} \right\}^2 = 1 \quad (2)$$

As shown in Figure 4, the response of damage initiation, t is defined as linear behaviour, but the elastic stiffness, K will be degraded when the stress vector t reach to maximum allowable stress, $t^{\max}$. Thus, the corresponding crack propagation will respond based on the damage evolution. The Eq: 3 is expressed the traction by stiffness. The $(1-D)$ is the reduction factor due to the damage initiation beyond the maximum allowable stress component $t^{\max}$. D is the damage evolution variable which represent the degradation of stiffness matrix K . Initial value of the damage evolution variable is from 0 and increases up to 1 as shown in Figure 4. The damage evolution variable D is expressed as Eq 4 [12]. The $\delta^{\max}_{eff}$ denotes the maximum effective displacement during the loading history. The effective displacement δ_{eff} is expressed in Eq D.4 [12].

$$t = \begin{cases} (1-D)K\delta \leftarrow (t_{initial} \geq t^{\max}) \\ K\delta \leftarrow (t_{initial} < t^{\max}, \text{No damage into stiffness}) \end{cases} \quad (3)$$

$$D = \frac{\delta_{eff}^f (\delta_{eff}^{\max} - \delta_{eff}^o)}{\delta_{eff}^{\max} (\delta_{eff}^f - \delta_{eff}^o)} \quad (4)$$

$$\delta_{eff} = \sqrt{\langle \delta_n \rangle^2 + \delta_s^2 + \delta_t^2} \quad (5)$$

The damage evolution can be identified based on the fracture energy that dissipated during the separation process or based on maximum displacement. Due to the characteristic of brittle material, it is hard to capture the maximum displacement of masonry. Thus, the current study expresses damage evolution based on fracture energy. The geometry terms of total fracture energy in mixed mode, G_{TC} is the area of traction separation plot graph as shown in the Figure 4. It is assumed that the traction-separation response of masonry in mode 2 mode 3 are in the same direction. Based on the assumption, the mixed mode fracture energy, G_{TC} can be calculate by Benzeggagh-Kenane (BK) law [13]. The exponent η holds value of 2 for brittle behaviour of joint interface [13]. The critical mode fracture energy G_{TC} under mixed mode is written in Eq: 6.

$$G_{TC} = G_{IC} + (G_{IIC} - G_{IC}) \left\{ \frac{G_{II} + G_{III}}{G_I + G_{II} + G_{III}} \right\}^{\eta} \quad (6)$$

2.3 Linear elastic behaviour of stiffness

The stiffness vector K described in this section. The traction separation response of joint interface prior to yield is elastic behaviour and traction vector can be written as nominal stress as shown in Eq: 7. The traction vector can be rewritten by stiffness K and displacement δ . The rearrangement of traction vector is represented in Eq: 8. The displacement of joint interface, δ_n is equivalent to the total displacement of

masonry assemblage, δ_{total} minus displacement of expanded unit δ_u^j . Rearranging the nominal displacement of δ_n , it is as written in Eq: 9.

$$t = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} E_{nn} & E_{ns} & E_{nt} \\ E_{ns} & E_{ss} & E_{st} \\ E_{nt} & E_{st} & E_{tt} \end{bmatrix} \begin{Bmatrix} \varepsilon_n \\ \varepsilon_s \\ \varepsilon_t \end{Bmatrix} = E \varepsilon \quad (7)$$

$$t = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} K_{nn} & 0 & 0 \\ 0 & K_{ss} & 0 \\ 0 & 0 & K_{tt} \end{bmatrix} \begin{Bmatrix} \delta_n \\ \delta_s \\ \delta_t \end{Bmatrix} = K \delta \quad (8)$$

$$\delta_n = \delta_{total} - \delta_u^j$$

$$\delta_n = t_n \left(\frac{H_u}{E_u} + \frac{H_j}{E_j} \right) - \frac{t_n}{E_u^j} (H_u + H_j) \quad (9)$$

Height of mortar is very small compare to unit height. In other words, Young's modulus of mortar, E_j will not significantly affect to the expanded unit, so the Young's modulus of expanded unit, E_{exp} can be consider same as Young's modulus of original unit E_u . Cancel out and rearrange the Eq: D.5. Finally, Stiffness of K_{nn} , K_{ss} and K_{tt} can be written in Eq: 10, 11, 12. However, current study calculate the stiffness vector K by experimental approach. The most recent shear couplet test conducted by Park [23] and the author reveal that the Eq: 10, 11 is not accurate with experimental shear stiffness K_{ss} so author estimate the K_{nn} based on the Eq: 13 and current study follow same approach.

$$E_{exp} \approx E_u$$

Cancel out and rearrange.

$$\delta_n = \frac{t_n H_j}{E_j} - \frac{t_n H_j}{E_u}$$

$$\delta_n = t_n h_j \left(\frac{1}{E_j} - \frac{1}{E_u} \right)$$

$$\delta_n = t_n h_j \left(\frac{E_u - E_j}{E_u E_j} \right)$$

$$K_{nn} = \frac{t_n}{\delta_n} = \frac{E_u E_j}{H_j (E_u - E_j)} \quad (10)$$

$$K_{ss}, K_{tt} = \frac{G_u G_j}{H_j (G_u - G_j)} \quad (11)$$

$$G = \frac{E}{2(1 + \nu)} \quad (12)$$

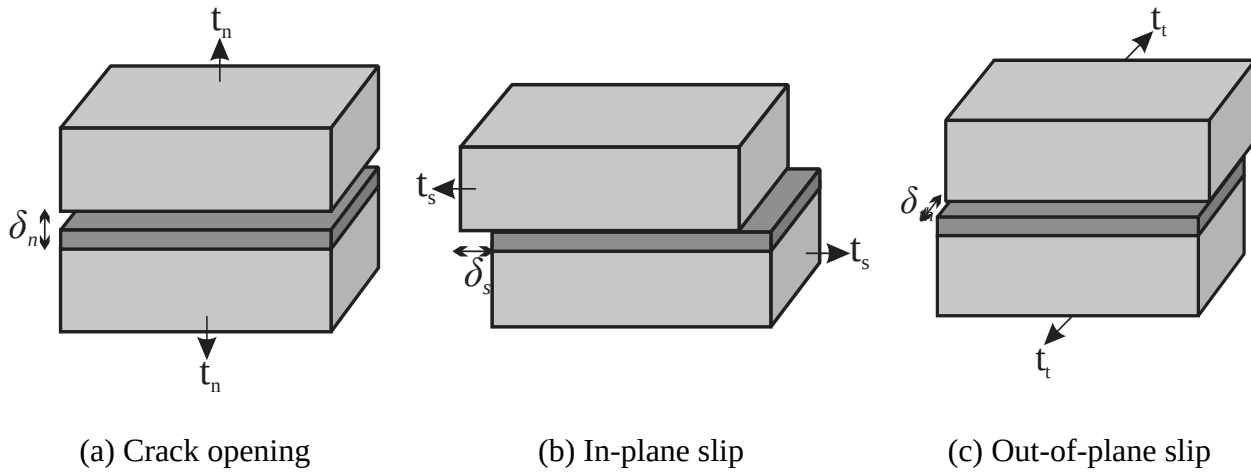
$$K_{nn} = K_{ss} \times 2(1 + \nu) \quad (13)$$

2.3 Failure modes of unreinforced masonry

Various cracking patterns are observed in masonry is subjected to ultimate loads. These failure modes of masonry under shear compression load reported by Mann and Muller [15]. Namely, the failure modes are friction failure, brick cracking and masonry compression failure.

Lorenzo and Rots [16] reported the basic cases of failure mechanism for URM structures. The failure modes for joint interface and unit are illustrated in Figure 5, 6. Figure 5 illustrated the failure mode along the joint interface based on fracture mode. Namely, Figure 5 (a) crack opening, (b) in-plane slip and (c) out of plane slip are described based on traction separation law. The principal of nation in Figure 5 is illustrated in table 2 and similarly Figure 6 followed same notations. In current study, it treats the out of plane slip is same direction as in plane shear slip and joint interface derived by BK law.

Failure mode for masonry unit illustrated in Figure 6 so namely, Figure 6 (a) unit direct tensile cracking , Figure 6 (b) unit diagonal tensile cracking is introduced. Failure mechanism of unit diagonal cracking in Figure 6 is combined failure mechanism under shear and compression load. The combined failure mechanism is involved in unit and joint failure.



(a) Tensile bond cracking (=Fracture mode 1)

(b) In-plane shear slip (=Fracture mode 2)

(c) Out of plane shear slip (=Fracture mode 3)

Figure 5: Bond failure along the joint interface

Stress = Traction	Displacement = Separation
t_n : Tensile stress along joint interface	δ_n : displacement of tensile bond cracking
t_s : In-plane shear stress along joint interface	δ_s : displacement of in-plane slip
t_t : Out-of-plane shear stress along joint interface	δ : displacement of out-of-plane slip

Table 2: Notations of fracture criterion

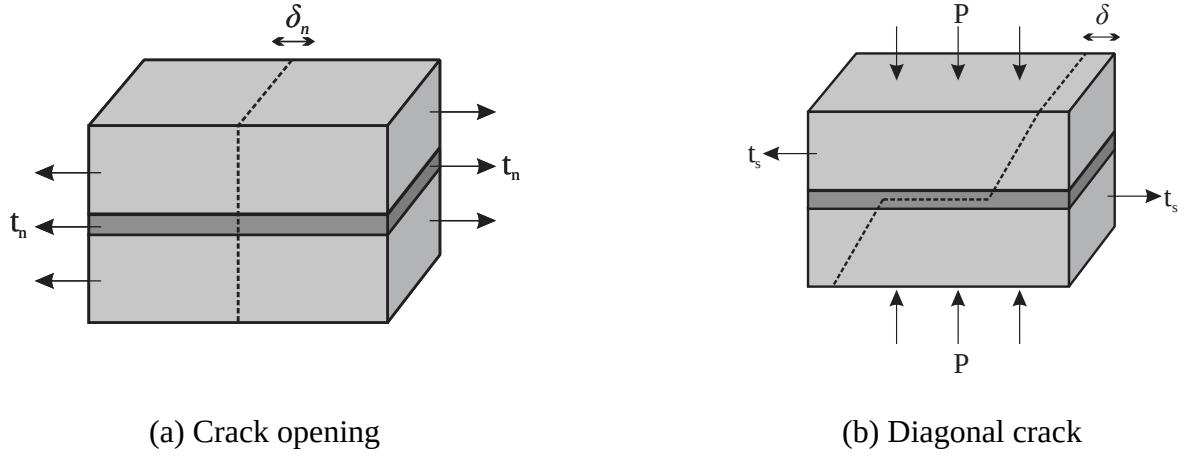


Figure 6: Typical unit failure mode

2.4 Linear Drucker-Prager (DP) plasticity model

The linear Drucker-Prager plasticity model was employed to define uniaxial compressive behaviour of masonry. The plasticity model allows masonry to have hardening and softening under compression. The yield criterion is defined by yield surface in meridional(P-t) plane and it could be linear, hyperbolic or exponential mode. The current study adopted the linear yield surface as shown in figure 7. The yield criterions are given by Eq: 14-19.

The evolution of yield surface with deformation beyond elastic range was specified based on uniaxial compressive yield stress. The yield stress σ_c , plastic strain ε_c^{pl} of Drucker-Prager hardening defined based on compressive stress strain curve from compression test for masonry prism.. The angle of friction β , flow stress ratio K are assumed as 36 degree, 1 and dilation angle, ψ is taken as lower value reported by Mosalam et al[18].

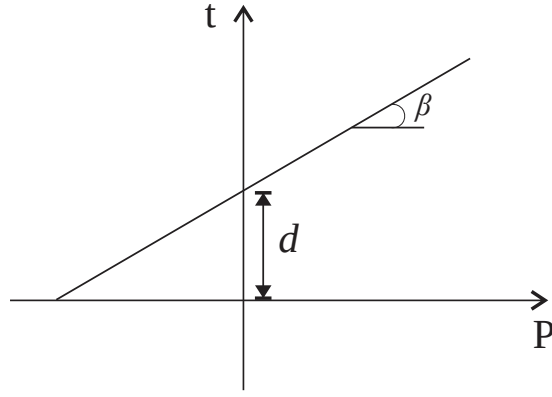


Figure 7: Yield criteria in P-t plane for Linear Drucker-Prager

$$F = t - p(\tan \beta) - d = 0 \quad (14)$$

The equivalent pressure stress, P is given by:

$$p = -\frac{1}{3} \text{trace}(\sigma) \quad (15)$$

The deviator stress, t given by:

$$t = \frac{1}{2} q \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{q} \right)^3 \right] = q \quad (16)$$

Assumed $K=1$

The von mises stress, q given by:

$$q = \sqrt{\frac{3}{2} S : S} \quad (17)$$

The third stress invariant, r and deviatoric stress, s given by

$$r = \left(\frac{9}{2} S : S : S \right)^{\frac{1}{3}} \quad (18)$$

$$S = \sigma + PI$$

The intercept, d given by:

$$d = \left(1 - \frac{1}{3} \tan \beta\right) \sigma_c \quad (19)$$

2.5 Extended finite element (XFEM)

Previously, a number of researchers have demonstrated that unit stiffness along the crack propagates in the centre of unit. It was introduced by Lorenzo [9] as simplified unit failure. However, what is still unclear about the location of actual unit crack and the crack growth. In order to capture crack growth of masonry unit, discontinuities are required for mesh element. The challenge is that the association of meshing crack surface. The mesh element needs to be continuously redefined as crack grows and the geometry of discontinuity have to be coordinated with mesh element. For this purpose, the current study carried out extended finite element method (XFEM).

The XFEM was firstly introduced by Belytschko [19] and Black but the concept of partition of unity is studied by Melenk et al [20]. The partition of unity is allowing local enrichment functions to be incorporated into finite element. The XFEM displacement approximation in enriched finite element is expressed with displacement vector function u and is written as Eq:20.

$$u = \sum_{I=1}^N N_I(x) \left[u_I + H(x) \alpha_I + \sum_{a=1}^4 F_a(x) b_I^a \right] \quad (20)$$

where, $N_I(x)$ is nodal shape functions, u_I is the nodal displacement vector for continuous part of finite element solution and the $N_I(x) \cdot u_I$ applied to all nodes in the model. a_I is nodal enriched degree of freedom vector, $H(x)$ is associated discontinuous jump function along the crack surface and the $a_I \cdot H(x)$ applied to

nodes whose shape function support is cut by the crack interior. b_I^α is the nodal enriched degree of freedom vector, $F_\alpha(x)$ is associated elastic asymptotic crack tip functions and the $F_\alpha(x) \cdot b_I^\alpha$ applied to nodes whose shape function support is cut by the crack tip.

In current study employed XFEM to simulate damage initiation and evolution in masonry unit under tension, compression and shear as illustrated in Figure 6. The crack growth allowed without predefined crack location and XFEM simulated based on cohesive segment method as studied by Remmers et al [21]. The cohesive segment method follows traction separation law as discussed in chapter 2.2 and the stiffnesses parameters are calculated based on elastic properties of enriched element.

3. FEM model for URM wall subjected to in-plane loading

The details on geometry of URM wall and numerical analysis are presented in Figure 8. As shown in Figure 9 (a), (b) loading beam was attached on top of the coarse of masonry panel and two axial loading plates were attached. The loading step consists of two steps where the pre-compression load was applied firstly, followed by applying the horizontal displacement. As the analysis result will be highly nonlinear, the mesh size should be carefully addressed for simulating shear compression tests. The current study focuses on the convergence study for mesh size. As shown in Figure 7, 8 different mesh size used to determine optimize mesh size and chose the 7x2x3 for clay brick panel and 8x4x3 for sandstone panel.

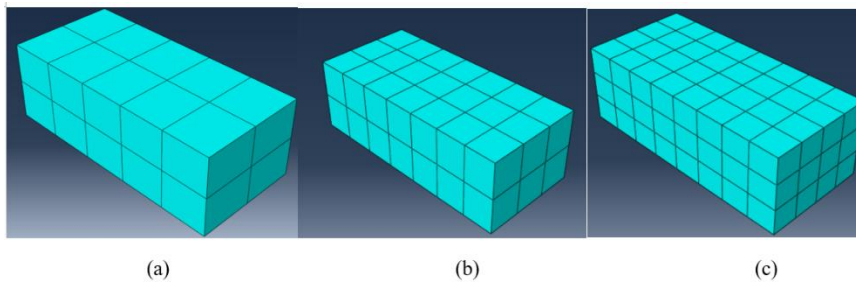


Figure 7: Mesh size for clay brick (a) 5x2x2 (b) 7x2x3 (c) 8x3x4

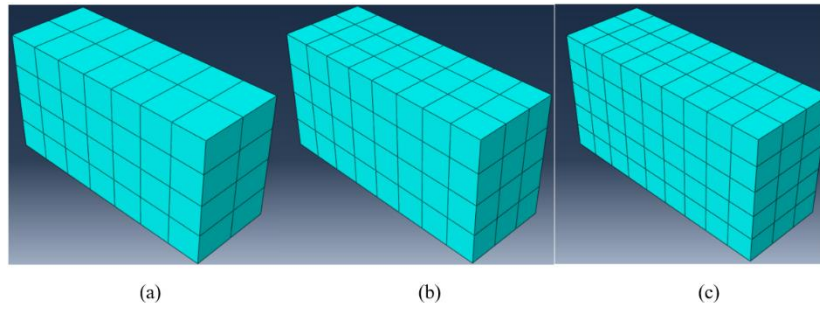
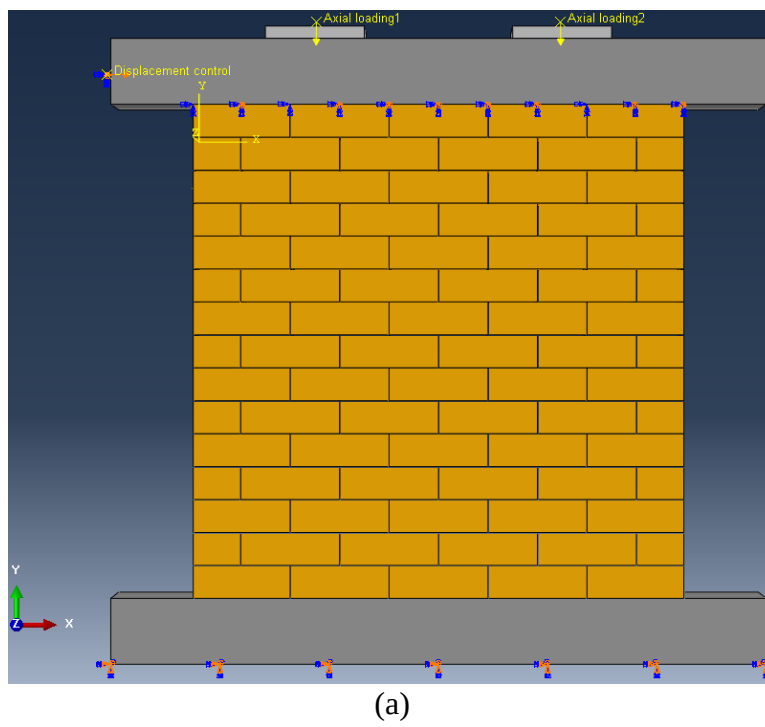


Figure 8: Mesh size for Sandstone : (a) 7x4x2 (b) 8x4x3 (c) 9x5x3



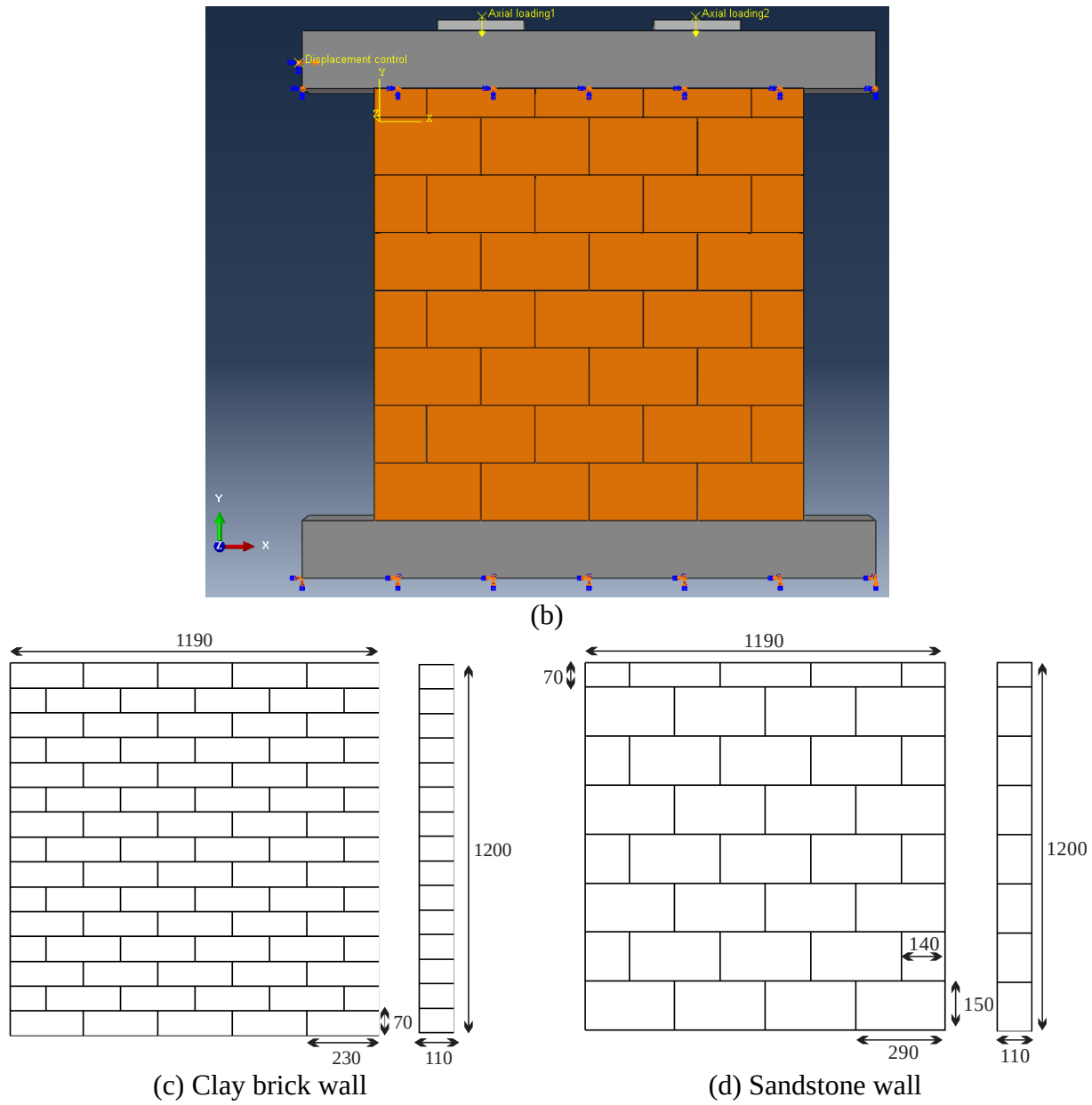


Figure 9: Simulation set up and detail geometry of masonry wall

3.1 Mechanical properties of masonry

Mechanical properties of unit, joint interface and masonry are taken from Park [23] and the material of unit presented in table 4. In the table 4, the direct tensile strength of unit taken 1/6 of flexural tensile strength which obtained from three point bending test. The shear strength, τ_u assumed 1.4 times direct tensile

strength. The shear fracture energy $G_{f_{II_u}}$ taken 0.5 N mm/mm^2 as reported by Raijmakers [24]. One noted that in order to capture realistic trend of unit failure, XFEM was applied and it captured the unit crack in flexural rocking failure mode.

Table 5 is presented properties of joint interface under tension, shear and compression. The flexural tensile strength of joint interface, f_t determined by bond wrench test and taken $1/3$ of f_t for direct tensile strength of joint. The fracture energy G_f^I taken recommended value 0.012 introduced by Angelillo [25]. The shear parameters of interface obtained from shear couplet test. The Mohr's coulomb failure criterion was modified with dry frictional coefficient, μ_d so initial shear strength, c was also modified. The shear stiffness, K_{ss} is calculated from shear couplet test and normal stiffness estimated based on K_{nn} based on Eq: 10, 11. The softening of damage evolution criterion is defined as linear form with BK law. The fracture energy G_f^{II} is calculated and is adopted by linear regression line as described in Figure 10 and table 6.

Linear Drucker-Prager parameters				
Material	Angle of friction, β	Flow stress ratio, K	Dilation angle, ψ	Compression yield stress, σ_c (MPa)
Clay brick	36	1	11.3	5.42
Sandstone				12.91

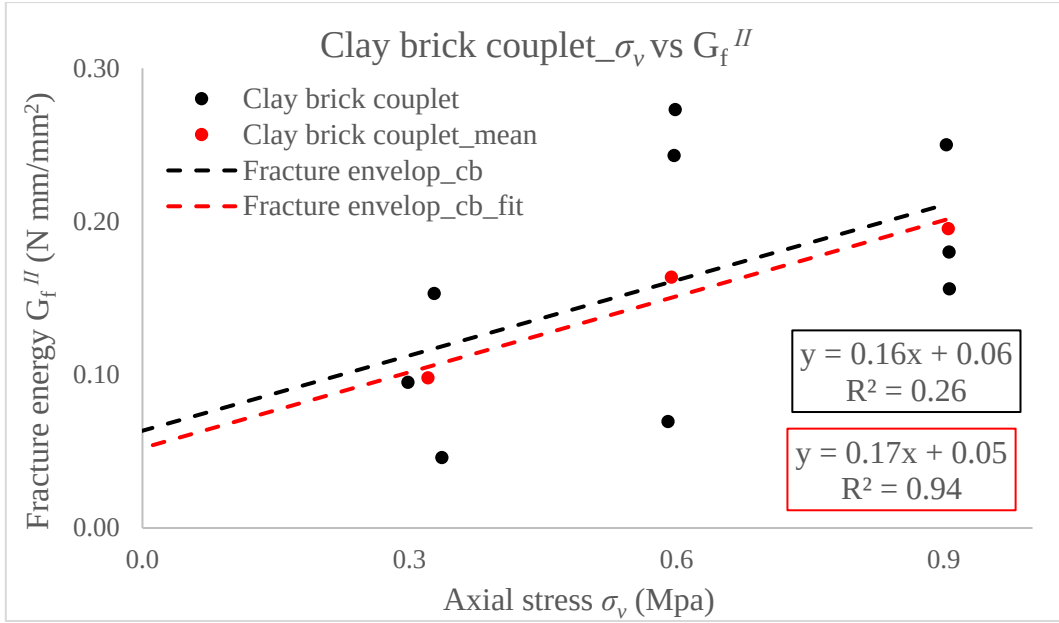
Table 3: Linear Drucker Prager parameters

Material properties of masonry unit								
Masonry unit	Brick unit	Mortar joint	Expanded unit		Tension		Shear	
	E_u (MPa)	E_j (MPa)	ν	E_{exp} (Eq: 1) (MPa)	f_{t_u} (MPa)	$G_{f_{I_u}}$ (N/mm)	$\tau_{_u}$ (MPa)	$G_{f_{II_u}}$ (N/mm)
Clay brick	9085	436	0.15	2611	0.67	0.21	0.93	0.5
Sandstone	10573	436	0.15	4310	0.775	0.25	1.09	0.5

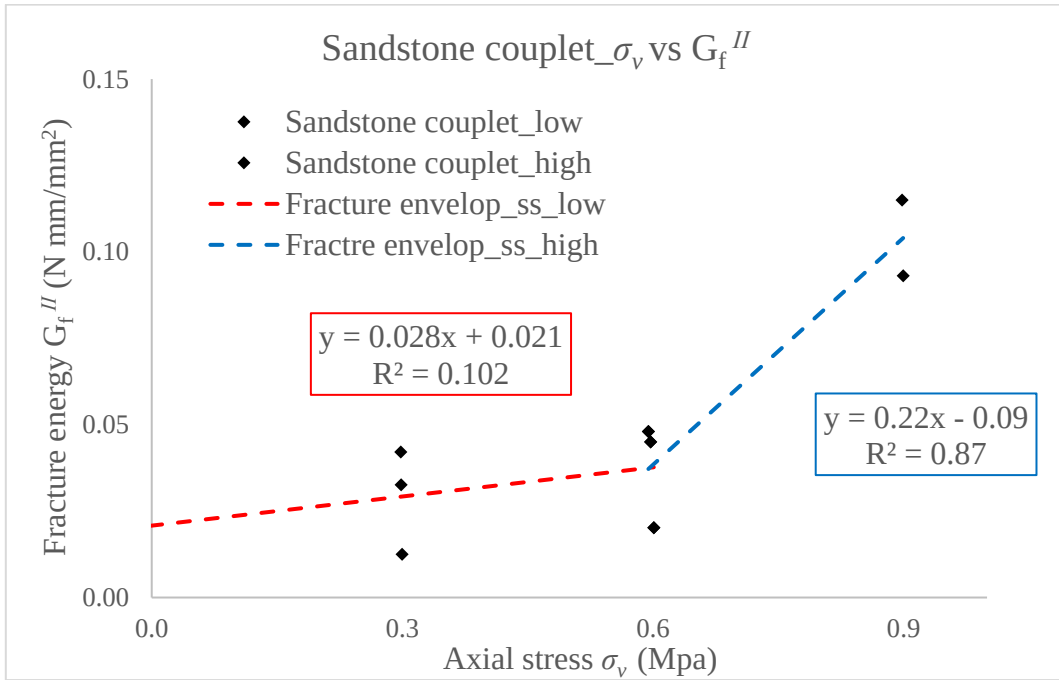
Table 4: Material properties of masonry unit

Material properties of masonry joint interface							
Masonry joint interface	Stiffness K (N/mm ³)			Tension		Shear	
	K_{nn} (Eq:13)	K_{ss}	K_{tt}	t_n (Mpa)	G_f^I (N/mm)	c (MPa)	μ_d
Clay brick	7.37	3.2	3.2	0.24	0.012	0.2	0.71
Sandstone	11.59	5.04	5.04	0.06	0.012	0.15	0.68

Table 5: Material properties of joint interface



(a) Clay brick couplet



(b) Sandstone couplet

Figure 10: Fracture energy linear fits of masonry couplet

Masonry	Fracture energy G_f'' linear fit	R^2
Clay brick couplet	$G_f'' = 0.17\sigma_v + 0.05$	0.94
Sandstone couplet	$G_f'' = \begin{cases} G_f'' = 0.28\sigma_v + 0.021(0 \leq \sigma_v \leq 0.6) \\ G_f'' = 0.22\sigma_v - 0.09(0.6 \leq \sigma_v \leq 0.9) \end{cases}$	$R^2 = 0.10$ $R^2 = 0.87$

Table 6: Fracture energy linear fit for masonry

3.2 Parametric study

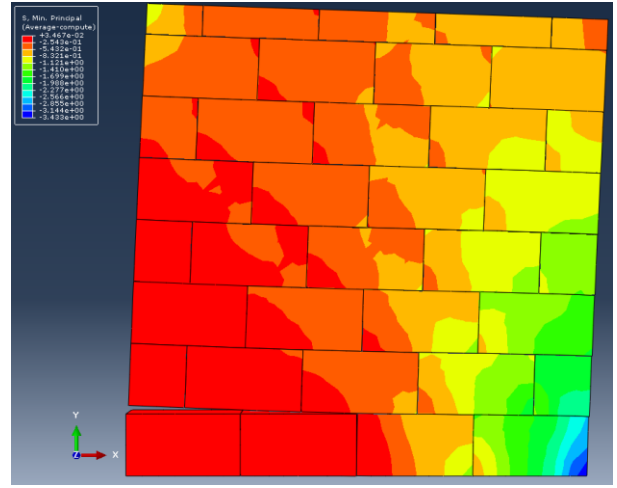
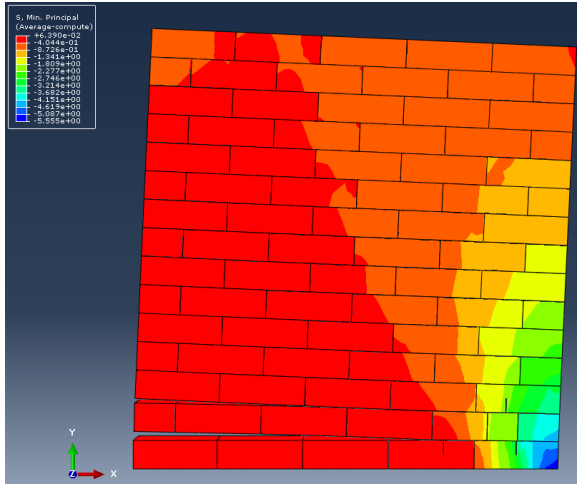
Current study simulates the URM wall under shear compression loading and parametric study carried out to investigate failure modes and lateral strength. The parametric study carried out in below:

- (1) Study the influence of various aspect ratio, $H/L = 1.0, 1.41$
- (2) Study the impact of fracture energy under various pre compression levels $\sigma_{pre}(\text{MPa}) = 0.3, 0.45, 0.6, 0.75, 0.9, 1.05$

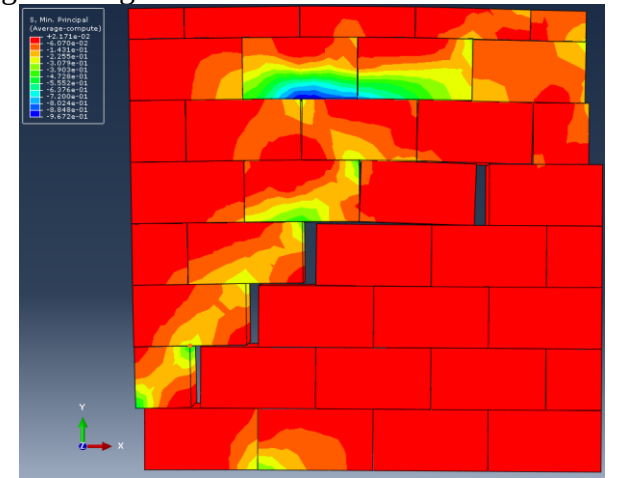
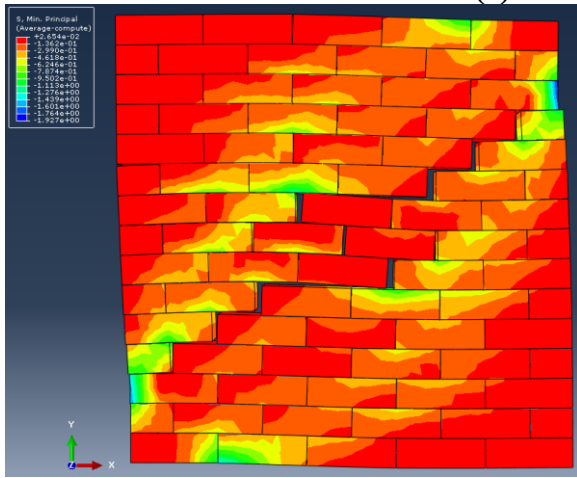
4. Result

Three failure modes namely, rocking crushing, diagonal cracking and shear sliding were observed as shown in Figure 10. The ultimate load obtained from numerical analysis was compared with estimated load. The design load is proposed based on the failure mode, so current study estimate flexural dominated failure and shear dominated failure. To be able to estimate shear force capacity of diagonal crack, the Turnšek and Čačovič [27] used and it expressed in Eq: 20 and the ultimate load for rocking failure estimated based on European code [26] as written in Eq: 21.

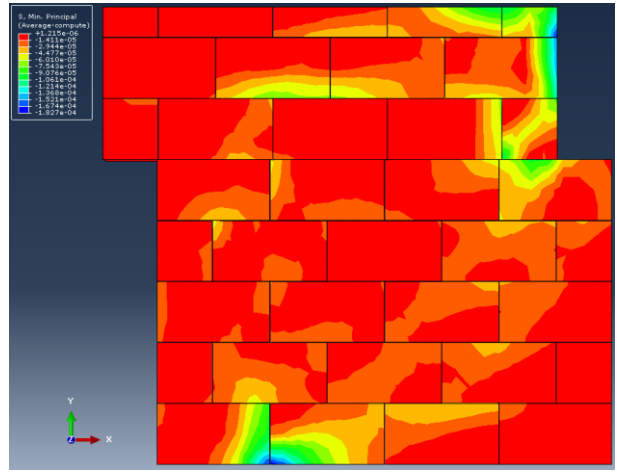
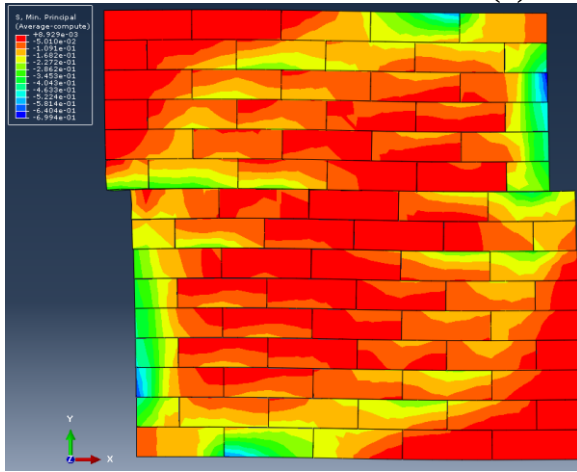
The result of numerical model represented in table 7 include ultimate load, failure mode, estimated load based on design standard. Surprisingly the FEM analysis provided low failure load compare to estimated value. It could be due to the characteristic of masonry or unknown effect. One noted that the failure mode of numerical model mostly governed by flexural failure and the potential reason could be the weak shear bond strength of joint respectively.



(a) Rocking crushing



(b) Diagonal crack



(C) Shear sliding

Figure 10: Failure pattern for masonry panel subjected in in-plane loading

$$V_{TC} = \frac{Lt}{b} f_t \sqrt{1 + \frac{\sigma_{v,mid}}{f_t}} \quad (20)$$

Where b is aspect ratio ($1 \leq b \leq 1.5$), L , t each being length and thickness of the panel respectively, f_t is diagonal tensile strength of the panel, and $\sigma_{v,mid}$ is the mean vertical stress at the mid height of the panel.

$$V_{fl} = \frac{L^2 t}{2H} \sigma_{v,tot} \left(1 - \frac{1}{\kappa} \frac{\sigma_{v,tot}}{f_c} \right) \quad (21)$$

Where $\sigma_{v,tot}$ is vertical stress at the base of panel, f_c is the compressive strength and $\frac{1}{\kappa}$ is the equal to 1.15

Numerical analysis for clay brick panel						
b	σ_{pre} (MPa)	$F_{numerical}$ (KN)	Failure mode	V_{TC} (KN)	V_{fl} (KN)	Difference (%)
1	0.3	28.4	Flexural rocking	-	36	-21.11
	0.45	30.7	Diagonal crack	44	-	-30.23
	0.6	35.3	Flexural rocking	-	68	-48.09
	0.75	45	Flexural rocking	-	68	-33.82
	0.9	48	Shear sliding	-	-	-
	1.05	54	Flexural rocking	-	106	-49.06
1.4	0.3	27.2	Flexural rocking	-	26	4.62
	0.45	28.5	Flexural rocking	-	38	-25.00
	0.6	32.5	Diagonal crack	35	-	-7.14
	0.75	40.6	Flexural rocking	-	58	-30.00
	0.9	45.3	Flexural rocking	-	68	-33.38
	1.05	46	Flexural rocking	-	76	-39.47

(a) Clay brick panel

Numerical analysis for sandstone panel						
b	σ_{pre} (MPa)	$F_{numerical}$ (KN)	Failure mode	V_{TC} (KN)	V_{fl} (KN)	Difference (%)
1	0.3	26.3	Flexural rocking	-	38	-30.79
	0.45	29.4	Flexural rocking	-	56	-47.50
	0.6	31.2	Flexural rocking	-	74	-57.84
	0.75	34.2	Diagonal crack	53	-	-35.47
	0.9	33.5	Shear sliding	-	-	-
	1.05	27.1	Flexural rocking	-	124	-78.15
1.4	0.3	38.2	Flexural rocking	-	27	41.48
	0.45	24.5	Flexural rocking	-	40	-38.75
	0.6	27.3	Shear sliding	-	-	-
	0.75	24.2	Flexural rocking	-	65	-62.77
	0.9	34.5	Flexural rocking	-	77	-55.19

	1.05	37.4	Flexural rocking	-	88	-57.50
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(b) Sandstone panel

Table 7: Numerical analysis for masonry panel

4. Future work

This paper focused on the estimation of force capacity for masonry panels by numerical simulation. The previous researcher also carried out the numerical study [3-5], [22] but they did not account for variability of fracture energy under different precompression level. The fracture energy is expressed as linear form and the mechanical properties of masonry is taken from Park[23]. The author conducted shear couplet tests under three axial stress $\sigma_v(\text{MPa}) = 0.3, 0.6, 0.9$ and represented fracture energy as linear form. The linear form for fracture energy may not suitable under high axial stress so it would be interesting to further study the fracture energy under high compression.

Future work will be carried out including validation of numerical model and investigation of other parameters such as drift capacity, elastic stiffness, force capacity, influence of fracture energy in failure mode.

REFERENCE

- [1] Salmanpour, Amir Hosein & Mojsilović, Nebojša & Schwartz, Joseph. (2015). Displacement capacity of contemporary unreinforced masonry walls: An experimental study. *Engineering Structures*. 89. 1-16.
10.1016/j.engstruct.2015.01.052.
- [2] Wilding, Bastian & Beyer, Katrin. (2017). Analytical and empirical models for predicting the drift capacity of modern unreinforced masonry walls. *Earthquake Engineering & Structural Dynamics*. 47.
10.1002/eqe.3053.
- [3] Wilding, Bastian & Beyer, Katrin. (2016). Force–displacement response of in-plane loaded unreinforced brick masonry walls: the Critical Diagonal Crack model. *Bulletin of Earthquake Engineering*. online.
10.1007/s10518-016-0049-7.
- [4] Haach, Vladimir & Vasconcelos, Graça & Lourenco, Paulo. (2011). Parametrical study of masonry walls subjected to in-plane loading through numerical modeling. *Engineering Structures - ENG STRUCT*. 33. 1377-1389. 10.1016/j.engstruct.2011.01.015.
- [5] Senthivel, R. & Lourenco, Paulo. (2009). Finite element modelling of deformation characteristics of historical stone masonry shear walls. *Engineering Structures*. 31. 1930-1943.
10.1016/j.engstruct.2009.02.046.
- [6] Pluijm, van der, R. (1999). Out-of-plane bending of masonry : behaviour and strength Eindhoven: Technische, Universiteit Eindhoven DOI: 10.6100/IR528212
- [7] Bolhassani, Mohammad & A. Hamid, Ahmad & C.W. Lau, Alan & Moon, Franklin. (2015). Simplified micro modeling of partially grouted masonry assemblages. *Construction and Building Materials*. 83.
10.1016
- [8] Abdulla, Kurdo & Cunningham, Lee & Gillie, Martin. (2017). Simulating masonry wall behaviour using a simplified micro-model approach. *Engineering Structures*, 151. 10.1016.

- [9] Lourenco, Paulo. (1996). Computational Strategy for Masonry Structures.
- [10] Dugdale DS. Yielding of steel sheets containing slits. J Mech Phys Solids 1960;8(2):100–4.
- [11] Barenblatt GI. The mathematical theory of equilibrium cracks in brittle fracture. Adv Appl Mech 1962;7(55–129):104.
- [12] Camanho, P. P., and C. G. Davila, “Mixed-Mode Decohesion Finite Elements for the Simulation of Delamination in Composite Materials,” NASA/TM-2002–211737, pp. 1–37, 2002.
- [13] Benzeggagh, M. L., and M. Kenane, “Measurement of Mixed-Mode Delamination Fracture Toughness of Unidirectional Glass/Epoxy Composites with Mixed-Mode Bending Apparatus,” Composites Science and Technology, vol. 56, pp. 439–449, 1996
- [14] Calderini, Chiara & Cattari, Serena & Lagomarsino, Sergio. (2010). The use of the diagonal compression test to identify the shear mechanical parameters of masonry. Construction and Building Materials - CONSTR BUILD MATER. 24. 677-685. 10.1016/j.conbuildmat.2009.11.001.
- [15] Mann, W., and Müller, H. (1982). "Failure of shear-stressed masonry- An enlarged theory, tests and application to shear walls." Proceedings of the British Ceramic Society 223-235.
- [16] Lourenco, P. B., and Rots, J. G. (1997). "Multisurface interface model for analysis of masonry structures." *Journal of Engineering Mechanics*, 123(7), 660-668.
- [17] Abaqus analysis user’s manual, 6.13-3. RI, USA: Dassault Systems Providence 2013.
- AS3700-2018: Masonry Structures. Sydney: Standards Australia, 2018
- [18] Mosalam K, Glascoe L, Bernier J. Mechanical properties of unreinforced brick masonry section-1. Documented to US Department of Energy by Lawrence Livermore National Laboratory; 2009.
- [19] Belytschko, T., and T. Black, “Elastic Crack Growth in Finite Elements with Minimal Remeshing,” International Journal for Numerical Methods in Engineering, vol. 45, pp. 601–620, 1999.

- [20] Melenk, J., and I. Babuska, “The Partition of Unity Finite Element Method: Basic Theory and Applications,” *Computer Methods in Applied Mechanics and Engineering*, vol. 39, pp. 289–314, 1996.
- [21] Remmers, J. J. C., R. de Borst, and A. Needleman, “The Simulation of Dynamic Crack Propagation using the Cohesive Segments Method,” *Journal of the Mechanics and Physics of Solids*, vol. 56, pp. 70–92, 2008.
- [22] Zucchini, A. & Lourenco, Paulo. (2009). A micro-mechanical homogenisation model for masonry: Application to shear walls. *International Journal of Solids and Structures - INT J SOLIDS STRUCT.* 46. 871-886. 10.1016/j.ijsolstr.2008.09.034.
- [23] Park, Young Kol. (2019). Experimental and numerical investigations on behaviour of URM wall subjected to in-plane loading.
- [24] Raijmakers T, Vermeltfoort AT. Deformation controlled tests in masonry shear walls; 1992.
- [25] Angelillo, Maurizio & Lourenco, Paulo & Milani, Gabriele. (2014). Masonry behaviour and modelling. *Mechanics of masonry structures*. Springer; 2014. P. 1-26, 10.1007/978-3-7091-1774-3_1.
- [26] Turnšek V, Čačovič F. Some experimental results on the strength of brick masonry walls. In: *Proceedings of the 2nd international brick masonry conference, stoke-on-trent*; 1970. p. 149–56.
- [27] Eurocode 8 (1996). : Design provisions for earthquake resistance of structures. London: British Standards Institution

Chapter 4. Conclusions and Future Work

Thesis outcomes

This thesis aims to investigate the behaviour of URM wall subjected to in-plane loading by DIC analysis and numerical study, which also involves an assessment of masonry methodologies evaluated. These tests include shear couplet test, diagonal compression test, bond wrench test and other fundamental masonry material test were conducted by ASTM, EN and RILEM standard. However, the previous methodologies still contain limitations to define masonry properties, so this current study aims to evaluate the previous methodologies or establish the new assessment procedure.

Having established that evaluation of diagonal tensile strength of masonry panel discussed in Chapter 2. Chapter 2 provides the guideline to predict diagonal tensile strength of masonry panel by DIC analysis. Interestingly the diagonal tensile strength of masonry has different interpretation of stress field in masonry panel. Traditionally diagonal tensile strength has been assessed by RILEM, ASTM standard. Yet, what remains unclear is that there was no attempt to investigate the stress field of masonry panel under in-plane loading. Therefore, current study investigates the stress field of masonry panel by DIC analysis and numerical model and it shows the RILEM's assumption is more suitable than ASTM standard. In order to demonstrate fundamental assessment, numerical study also employed inelastic range. Both DIC analysis and numerical study observed that similar failure pattern. Based on this relationship new assessment introduced to estimated diagonal tensile strength based on flexural tensile strength of masonry.

Having established that the shear force capacity of the masonry panel by numerical model and evaluated the AS 3700 and proposed equation studied by Turnšek and Čačovič. Chapter 3 investigates force capacity of masonry panel by simulate two different type of masonry such as clay brick and sandstone. These findings should make an important contribution to the field of masonry research. To be able to promote deeper insight of masonry research, scope of the future work is introduced below:

Scope of the future work

1. Investigate the post peak of masonry prism in compression test. Failure of masonry structure mainly governed by compression, so it is important to understand the compressive behaviour of masonry.

Previously a number of researchers performed compression test of prism imposed by load control but in order to obtain post-peak behaviour, the test should impose to displacement control. Hence, further study will be required, and the stress strain response will be compared to FEM analysis as presented in Appendices.

2. Evaluation of DIC analysis in rubble stone masonry without any speckle pattern. The purpose of applying speckle pattern is to give enough contrast but the DIC analysis is able to capture the natural texture.

However, there was no attempt to perform the DIC application without speckle pattern so this would be interesting to investigate. The rubble stone masonry has distinct texture so it should be able to capture the crack propagation in DIC analysis.

3. Investigate the influence of boundary condition, various aspect ratio for shear compression test. Chapter 3 mainly focuses on the shear, flexural rocking failure load but additional parameters need to be investigated. The interested parameters include; drift capacity, elastic stiffness of masonry panel and evaluation of the design codes of other countries namely European, German and Italian.

4. Analysing the effect of engagement in double leaf masonry panel by numerical model. Double leaf masonry panel is commonly used in Australia but there has been no attempts made to investigate the effect of engagement. The engagement may affect the ultimate load and/or failure pattern so further investigation is required. The potential outcome can be established through the use of design guideline for double masonry panel.

PRINCIPAL NOTATIONS

A	area	mm^2
W_o	area of force displacement graph	N mm
A_d	cross sectional area	mm^2
A_{net}	net area	mm^2
L	length	mm
H	height	mm
T	thickness	mm
E	Young's modulus of elasticity	MPa
E_u	elasticity of modulus of brick unit	MPa
E_j	elasticity of modulus of mortar joint (subscript: cb, ss, ls, bs)	MPa
E_{exp}	elasticity of modulus of expanded unit (subscript: cb, ss, ls)	MPa
E_m	elasticity of modulus of masonry (subscript: cb, ss, ls)	MPa
f_{mc}	compressive strength of masonry (subscript: cb, ss, ls)	MPa
f_j	compressive strength of mortar joint	MPa
f_{uc}	compressive strength of unit	MPa

	(subscript: cb, ss, ls)	
f_{mt}	flexural tensile strength of mortar joint	MPa
	(subscript: cb, ss, ls, bs)	
f_t	direct tensile strength of masonry	MPa
	(subscript: cb, ss, u)	
G_f	Fracture energy	N mm/mm ²
	(subscript: cb, ss, ls, bs)	
G_{TC}	total fracture energy in mixed mode	N mm/mm ²
G_f^I	fracture energy mode 1	N mm/mm ²
G_f^{II}	fracture energy mode 2	N mm/mm ²
G_f^{III}	fracture energy mode 3	N mm/mm ²
G_u	shear modulus of unit	MPa
G_j	shear modulus of mortar joint	MPa
H	total height	mm
H_j	joint height	mm
H_u	unit height	mm
t_u	thickness of unit	mm
K	stiffness	N/mm ³
K_{nn}	nominal stiffness	N/mm ³
	(subscript: cb, ss, ls, bs)	
K_{ss}	in plane shear stiffness	N/mm ³
	(subscript: cb, ss, ls, bs)	
K_{tt}	out of plane shear stiffness	N/mm ³
	(subscript: cb, ss, ls, bs)	

ν	Poisson's ratio	-
F	normalised force	N
P	pressure load	N
δ	normalised displacement	mm
δ_n	displacement of tensile bond cracking	mm
δ_s	displacement of in-plane slip	mm
δ_t	displacement of out-of-plane slip	mm
t_n	Tensile stress along joint interface	MPa
t_s	In-plane shear stress along joint interface	MPa
t_t	Out-of-plane shear stress along joint interface	MPa
Δ	displacement	mm
Z_d	section of modulus	mm ³
W	weight	kg
M	moment	N mm
m	mass	N
θ	friction angle	°
σ	normal stress	MPa
σ_v	axial stress	MPa
μ	wet frictional coefficient (subscript: cb, ss)	-
μ_d	dry frictional coefficient (subscript: cb, ss)	-
μ_c	cohesive frictional coefficient	-
τ	shear stress	MPa

	(subscript: cb, ss)	
τ_{res}	residual shear strength	MPa
	(subscript: cb, ss)	
c	cohesion	MPa
	(subscript: cb, ss)	
k_a	aspect ratio of prism	-
k_b	shape factor	-
N_{spe}	Number of specimen	-
$2D$	two dimension (X, Y)	-
$3D$	three dimension (X, Y, Z)	-
ε	Strain	-

Subscript

cb	clay brick
ss	sandstone
ls	limestone
bs	bluestone
$q1$	25 th percentile
$q2$	50 th of percentile
$q3$	75 th of percentile
max	maximum
min	minimum
$median$	median value
$mean$	mean value

ABBREVIATIONS

CMOD	crack mouth opening displacement
LVDT	linear variable displacement transducer
SG	strain gauge
DIC	digital image correlation
BK law	Benzeggagh-Kenane law
CDP	concrete damaged plasticity
DP	Drucker Prager
DIC	digital image correlation
CoV	coefficient of variation
StD	standard deviation
URM	unreinforced masonry
VSG	virtual strain gauge
CBW	clay brick wall
SSW	sandstone wall
DC	diagonal compression test
SCT	shear couplet test
FEM	finite element method
XFEM	extended finite element method

APPENDICES

A MATERIAL TESTS

A.1 Introduction

The post behaviour of URM panel highly depend on the properties of joint interface and unit, however since masonry structures have unique properties, it is hard to define by a few tests. For this sake, a fundamental experiment program was carried out. All the material test results are herein.

The main interested parameters were:

Elasticity of modulus of brick unit mortar joint and masonry; E_u, E_j, E_m

Unconfined compressive strength of masonry; f_{mc}, f_j, f_u

Flexural tensile strength of masonry; f_{mt}, f_{mt_u}

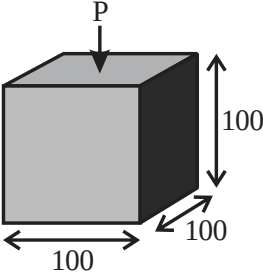
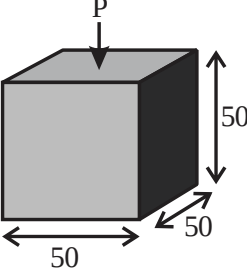
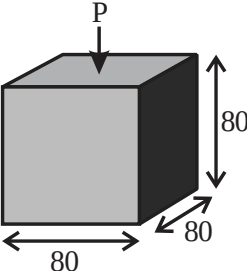
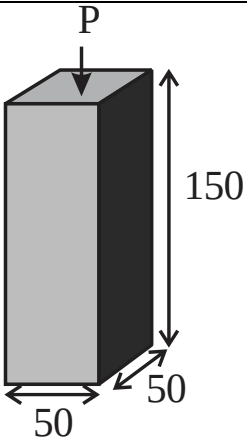
Shear bond strength of masonry; τ, c

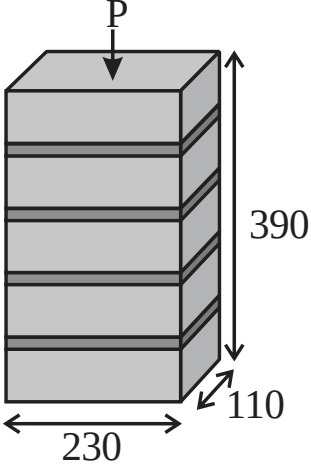
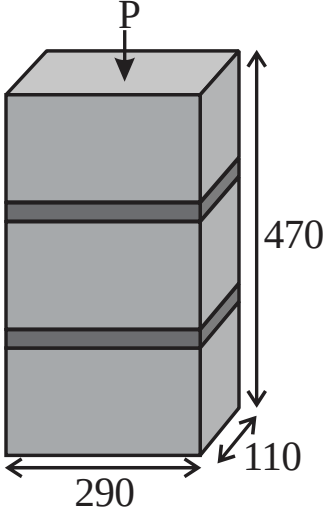
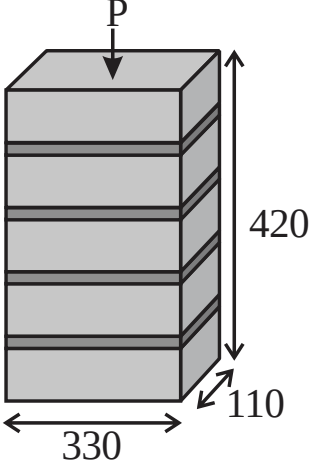
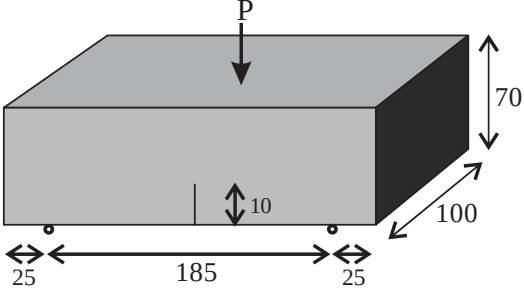
Frictional coefficient; μ, μ_{dry}

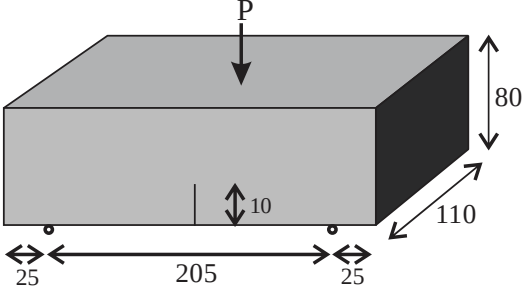
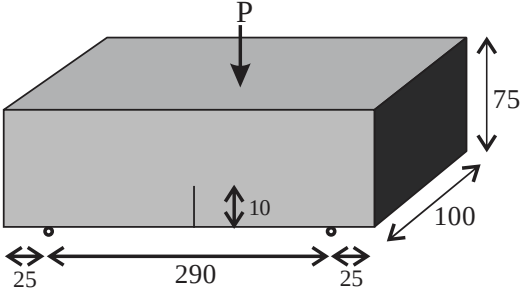
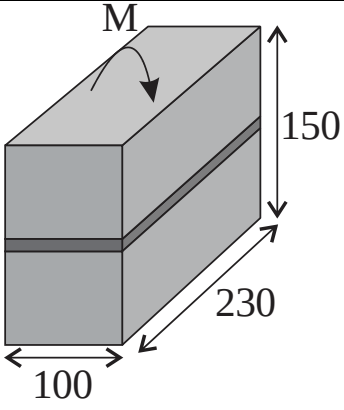
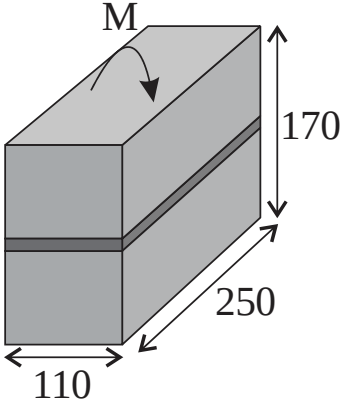
Fracture energy of masonry; G_f^{II}

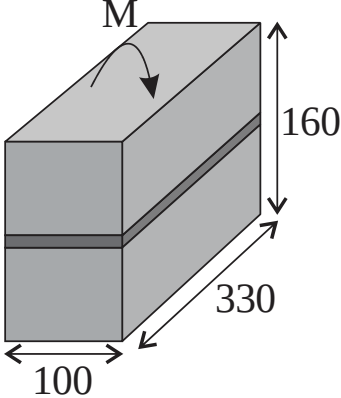
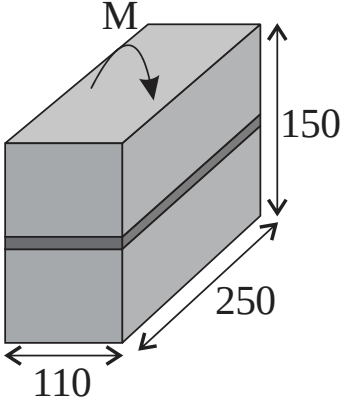
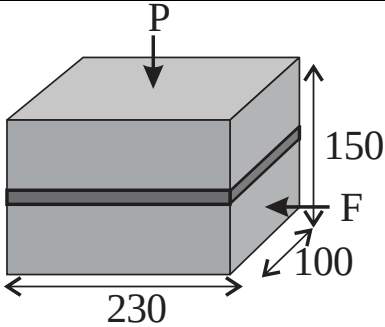
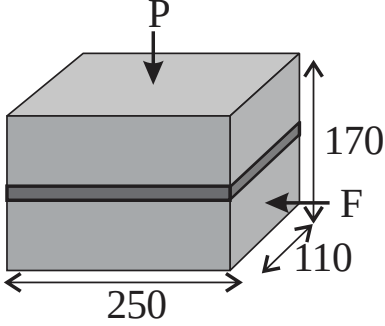
Masonry structures have been studied by many researchers using cement based mortar but there is very little published data on other type of mortar so current experimental program built all of specimen with pure lime mortar. The mortar joint has same ratio of sand and lime, but it has two different moisture content. The mixture ratio in the mortar is Lime: sand: water=1:3:1.5(high moisture content) and 1:3:1(low moisture content). The experimental program includes geometry of specimen, conducted material test, a number of specimens, interested parameters and detail dimension of clay brick frog are illustrated in the Table A.1. One noted that all of masonry specimens have 10mm thickness of mortar joint.

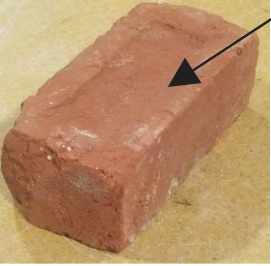
Table A.1: Experimental program

Dimension of specimen (mm)	Test	N_{spe}	Key parameter
 A 3D diagram of a cube specimen with dimensions 100 mm (width), 100 mm (depth), and 100 mm (height). A downward arrow labeled 'P' indicates the compression load applied to the top face.	Compression test for mortar cube	41(High moisture) 21(Low moisture)	f_j, E_j
 A 3D diagram of a cube specimen with dimensions 50 mm (width), 50 mm (depth), and 50 mm (height). A downward arrow labeled 'P' indicates the compression load applied to the top face.	Compression test for clay brick	10	f_{uc_cb}
 A 3D diagram of a cube specimen with dimensions 80 mm (width), 80 mm (depth), and 80 mm (height). A downward arrow labeled 'P' indicates the compression load applied to the top face.	Compression test for sandstone, limestone	10(Sandstone) 10(Limestone)	f_{uc_ss}, f_{uc_ls}
 A 3D diagram of a rectangular prism specimen with dimensions 50 mm (width), 50 mm (depth), and 150 mm (height). A downward arrow labeled 'P' indicates the compression load applied to the top face.	Elasticity of modulus test for clay brick, sandstone and Limestone	6(Clay brick) 5(Sandstone) 4(Limestone)	$E_{u_cb}, E_{u_ss}, E_{u_ls}$

	Compression test for clay brick prism	7	f_{mc}, E_{m_cb}
	Compression test for sandstone prism	5	f_{mc}, E_{m_ss}
	Compression test for Limestone prism	5	f_{mc}, E_{m_ls}
	Three-point bending test for clay brick	6	f_{mt_ucb}

	Three-point bending test for sandstone	8	f_{mt_uss}
	Three-point bending test for limestone	9	f_{mt_uls}
	Bond wrench test for clay brick	9	f_{mt_cb}
	Bond wrench test for sandstone	9	f_{mt_ss}

	Bond wrench test for limestone	9	f_{mt_ls}
	Bond wrench test for bluestone	9	f_{mt_bs}
	Shear couplet test for clay brick	9	$\mu_{cb}, \mu_{dry_cb},$ $C_{cb}, \tau_{cb},$ $G_{f_{II_cb}}^H, K_{SS_cb}$
	Shear couplet test for sandstone	8	$\mu_{ss}, \mu_{dry_ss},$ $C_{ss}, \tau_{ss},$ $G_{f_{II_ss}}^H, K_{SS_ss}$

	All of clay brick units	Detail dimension of frog
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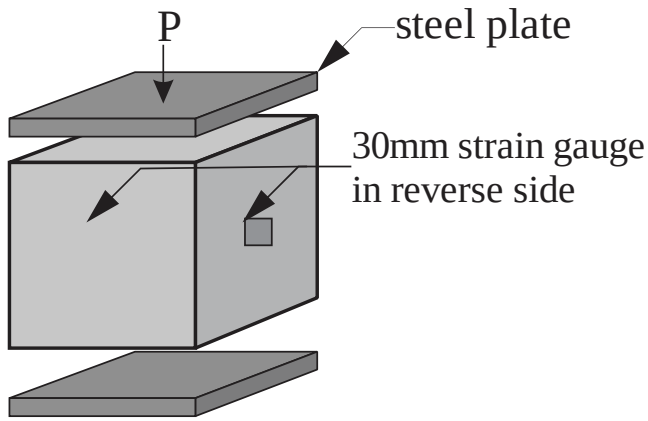
A.2 Compression test

The compression test for masonry specimen carried out to determine material properties and the test conducted according to European or Australian Standard. The compressive strength of mortar cube determined by Eq. A.1.

$$f_j = \frac{P_{\max}}{A_d} \quad (\text{Eq: A.1})$$

A.2.1 Test method for mortar cube

A primary concern of compression test of mortar cube focuses on the compressive strength but until now, far too little attention has been paid to elasticity of modulus; E_j hence, compression test of mortar cube has been implemented. Two 30mm strain gauges attached on opposite sides as shown in figure A.1 and the cyclic load protocol employed. The test standard conducted according to EN 1015.



(a) Schematic view

(b) Image view

Figure A.1: Test set up for compression test of mortar cube

The total 62 number of mortar cubes casted with 28 days of curing and 41 specimens represented for high moisture content and 21 specimens represented for low moisture content. Table A.2 shown the mix proportion of mortar joint. . The obtained test result is provided in figure A.2. Associated results have shown good consistency except batch #32. Surprisingly in high moisture content, compressive strength of batch #32 was relatively higher than other batches. The unknown effect on batch #32 required more investigation.

The result also shown as quartile box plot in figure A.3 to compare different moisture content. As reported in figure A.3, mean value of compressive strength in low moisture content was higher than high moisture content. In other words, the moisture content may have had potential effect to compressive strength of mortar cube and it may applied to the batch #32. The corresponding mean value of f_j (1.13MPa) from batch #32 was close to Q3 value of f_{jc} (1.11MPa) in low moisture content. Thus, it is assumed that the unknown effect on batch#32 may occurred due to the low moisture when it was casted.

Mortar joint mixture	
Mortar type	Cement : Lime : sand : water
High moisture	0 : 1 : 3 : 1.5
Low moisture	0 : 1 : 3 : 1

Table A.2: Mix proportion of mortar joint

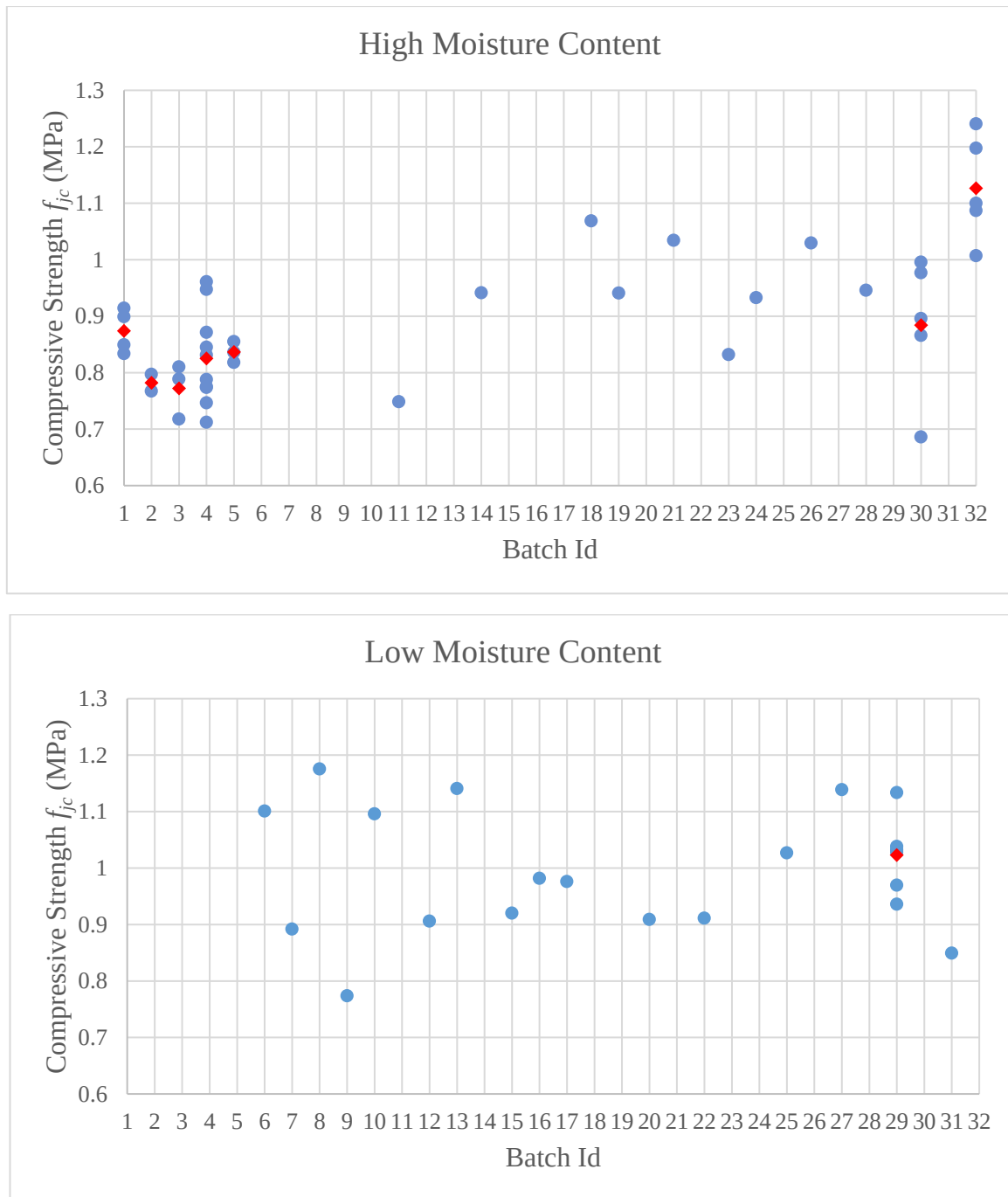
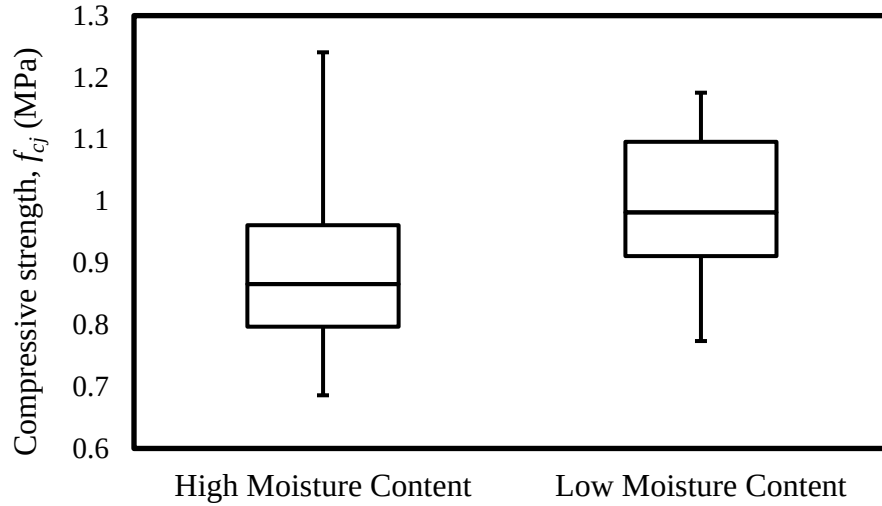


Figure A.2: Compressive strength of mortar f_{jc} cube (MPa) in bath Id
 (Note that each compressive strength of mortar cube shown as blow dot  and average compressive strength of mortar cube shown as red diamond )



Compressive strength f_{cj} (MPa)							
Mortar cube	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
High Moisture content	41	1.24	0.87	0.89	0.69	0.13	14.28
Low Moisture content	21	1.18	0.98	1.00	0.77	0.11	10.58

Figure A.3: Comparison of measured f_{cj} (MPa) between high moisture content and low Moisture content

II

All of compression tests were imposed load control and the loading protocol adopted to calculate elasticity of young's modulus. The loading protocol [EN 14580] had been addressed with 3 cycle loadings prior to ultimate load as illustrated in figure A.4 and same protocol applied to the other compression test including mortar cube, unit and prism. The first cycle closed the gap between specimen and loading machine so the slopes of second and third cycle loads calculate elasticity of modulus. The upper stress in figure A.4 targets 20% of expected ultimate stress and basic force is aimed to 2% of expected ultimate stress. The elasticity of young's modulus calculated based on Eq: A.2 and corresponding strain ϵ_u , ϵ_o taken from the mean of two gauges for eccentricity purpose. The elasticity of young's modulus also taken by mean value of E_1 and E_2 . The multiple mortar cubes mounted strain gauges and it represent the result in figure A.5 based on moisture content to explore effect of moisture content. The mean value of elasticity of modulus were 430MPa and 425MPa and the quartile box plot had not shown any significant difference. Therefore, in the pure lime mortar, the moisture content mainly contributed to compressive strength but the effect into elasticity of young's modulus was negligible.

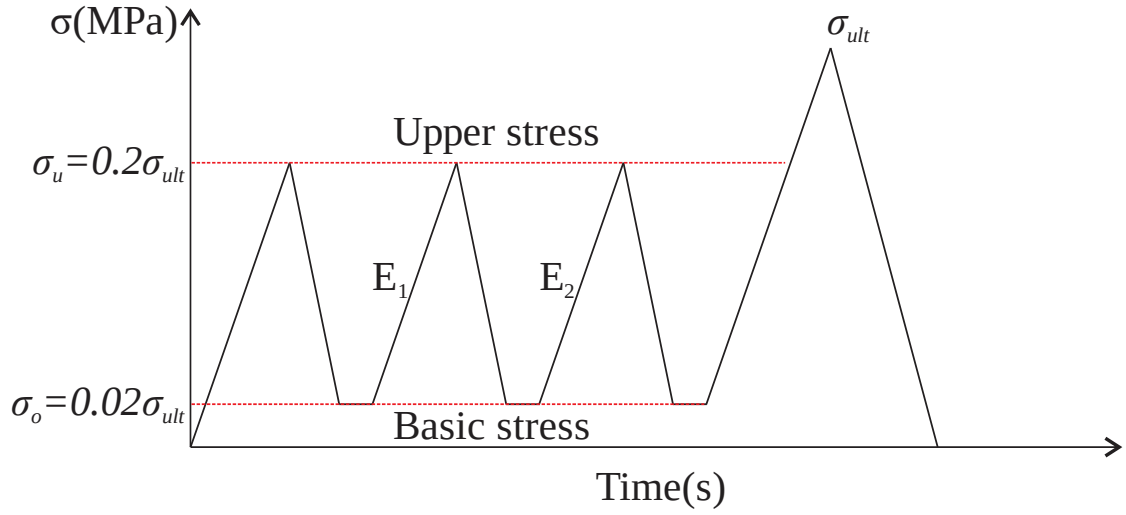
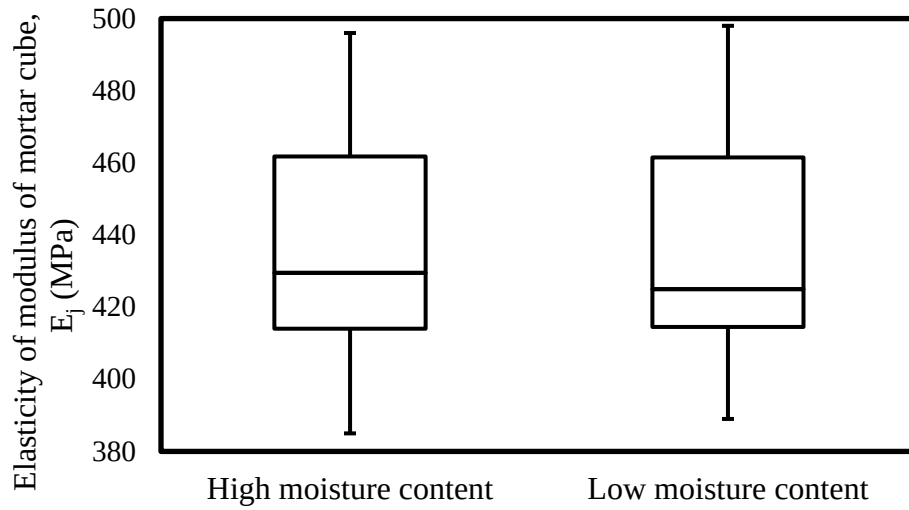


Figure A.4: Loading protocol of compression test

$$E = \frac{\Delta\sigma}{\Delta\varepsilon} = \frac{\sigma_u - \sigma_o}{\varepsilon_u - \varepsilon_o} \quad (\text{Eq: A.2})$$



Elasticity of Modulus E_j (Mpa)							
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
High moisture content	14	496	430	436	385	31.80	7.29
Low moisture content	11	498	425	437	389	37.10	8.49

Figure A.5: Elasticity of young's modulus of mortar cube E_j (MPa)

A.2.2 Test method for unit

Two different compression tests of unit were conducted separately to obtain compressive strength and elasticity of young's modulus. The compressive strength of unit, f_{uc} was determined in accordance with EN1926 and EN772 respectively. Each of 10 clay brick, sandstone and limestone specimens had been prepared and geometry of the specimen is as described in the table A.1.

$$f_{uc} = k_b \frac{P_{\max}}{A_d} \quad (\text{Eq: A.3})$$

Where k_b is the shape factor and it calculated based on aspect ratio of specimen as followed by EN772. The shape factor clay brick was 0.85, sandstone and limestone were 0.94 used. The figure A.6 illustrated test set up of clay brick unit and both top and bottom side has dental paste layer to apply uniform distributed load. Variations in compressive strength of the unit presented in figure A.7. The mean value of the clay brick unit, f_{uc_cb} (28.2MPa) is about 2.6 times higher than the limestone unit, f_{uc_ls} (10.68MPa) and sandstone unit, f_{uc_ss} (57.57MPa) is about 2 times higher than f_{uc_cb} (28.2MPa).

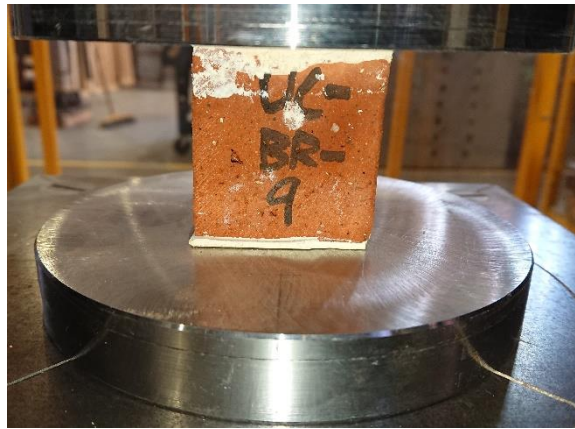
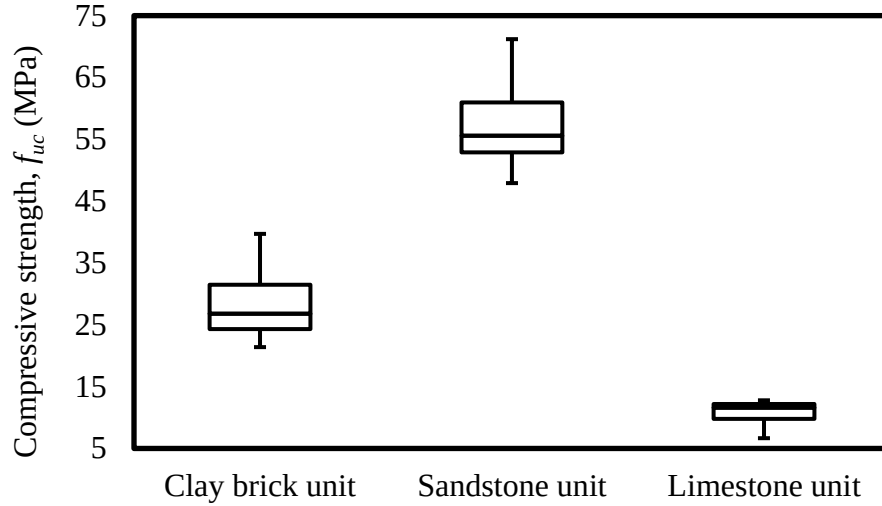


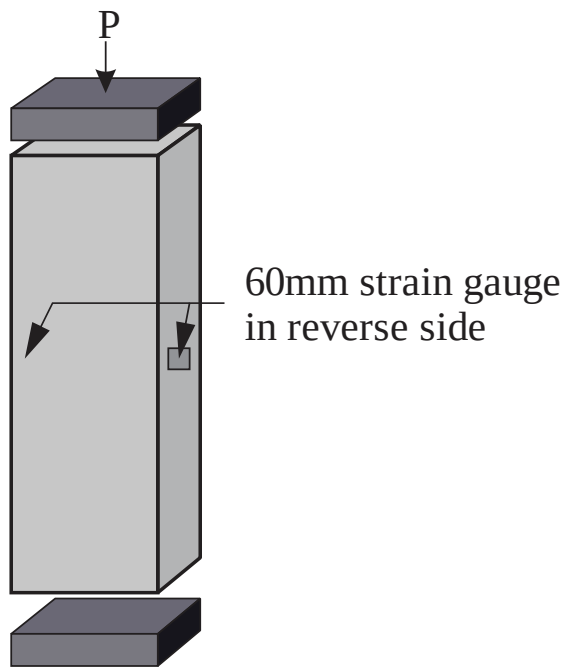
Figure A.6: Compression test set up for compressive strength of unit, f_{uc}



Compressive strength f_{uc} (Mpa)							
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
Clay brick unit	10	39.71	26.80	28.20	21.39	5.41	19.18
Sandstone unit	10	71.20	55.57	57.57	47.92	6.69	11.63
Limestone unit	10	12.79	11.60	10.68	6.66	2.05	19.20

Figure A.7: Compressive strength of masonry unit, f_{uc} (MPa)

In order to determine the elasticity modulus, two 60mm strain gauges were used as shown in figure A.8 and spherical load cell on base for eccentricity of loading. It is noted that only two cyclic loadings applied so only second cycle load was considered to calculate young's modulus. Figure A.9 shows the calculated young's modulus and its coefficient of variation (CoV) is highest for the clay brick units. The clay brick's quartile box plot shows big gap between max value and Q3(75th of percentile). It could be the random variation of masonry or could be an effect of frog. The mean value of clay brick, sandstone and limestone are 9085MPa, 10574MPa, 7094MPa. Generally, there was no significant difference of mean elasticity of young's modulus, E_u observed between type of masonry unit.

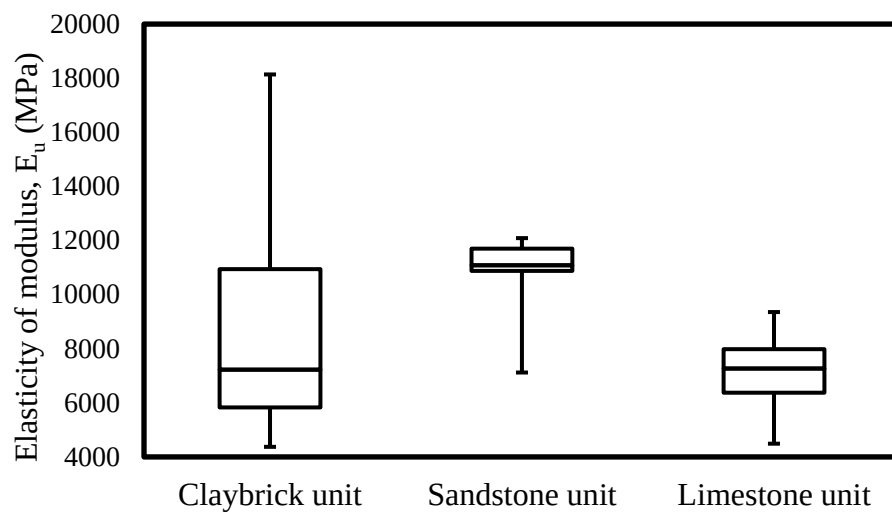


(a) Schematic view



(b) Image view

Figure A.8: Compression test set up for elasticity of modulus of unit, E_u



Elasticity of Modulus, E_u (Mpa)							
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
Clay brick unit	6	18138	7226	9085	4373	4696	51.69
Sandstone unit	5	12088	11082	10573	7121	1780	16.83
Limestone unit	4	9355	7266	7094	4489	2008	28.30

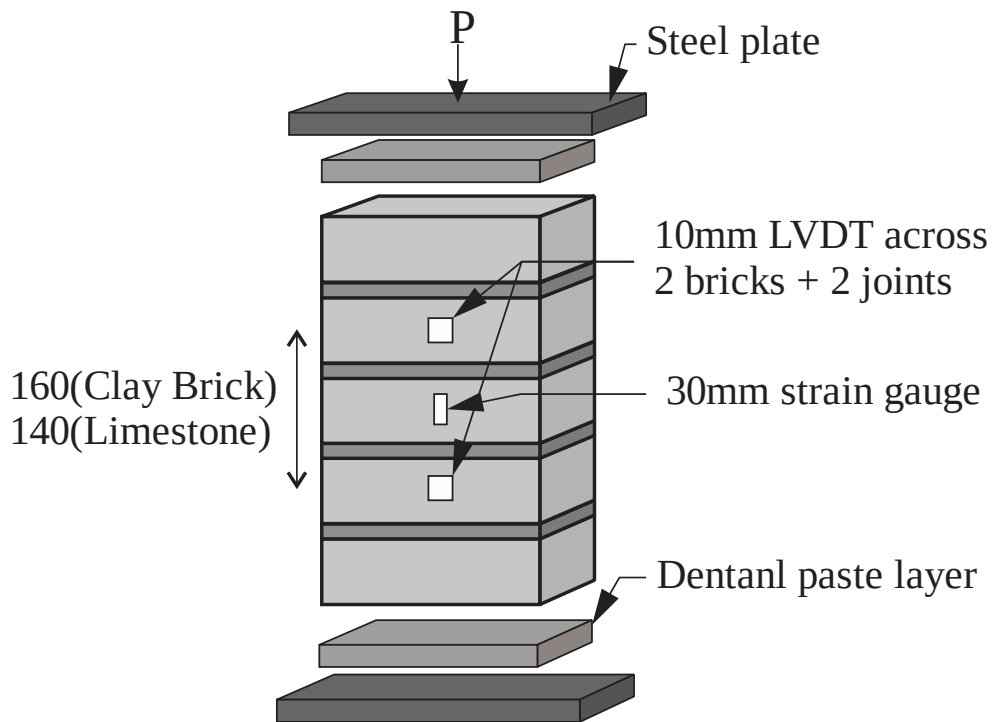
Figure A.9: Elasticity of modulus of masonry, E_u (MPa)

A.2.3 Test method for prism

The prism test was conducted to determine masonry properties including compressive strength f_{mc} , elasticity of Young's modulus E_m , compressive behaviour for numerical study. Each type of prisms have been built from identical mortar batch. Prior to perform prism test, approximation of failure load was important to select right loading machine and determine approximate range of upper and base force. The case study to determine the compressive strength of masonry reported by Thaickavil, N N & Thomas(2017) and current study determined the expected f_{mc} based on the proposed equation(Eq:A.4) which studied by Kumavat. Mean value of compressive strength of unit f_{uc} and f_j used as prescribed in section A.2.1 and A.2.2

$$f_{mc} = 0.69 f_{uc}^{0.6} f_j^{0.35} \quad (\text{Eq: A.4})$$

The compression test set up for masonry prism provided in figure A.10 and each prism attach two of 30mm strain gauges and two of 10mm LVDTs on front and back side. Height of clay brick unit had large scatter but the distance of LVDT point was constant 160mm and similarly LVDT point distances were constant for limestone and sandstone prism. Dental paste layer applied on the top and bottom of surface for uniform distributed load and spherical load cell on the base to avoid eccentricity issue.



Schematic view (clay brick & Limestone)

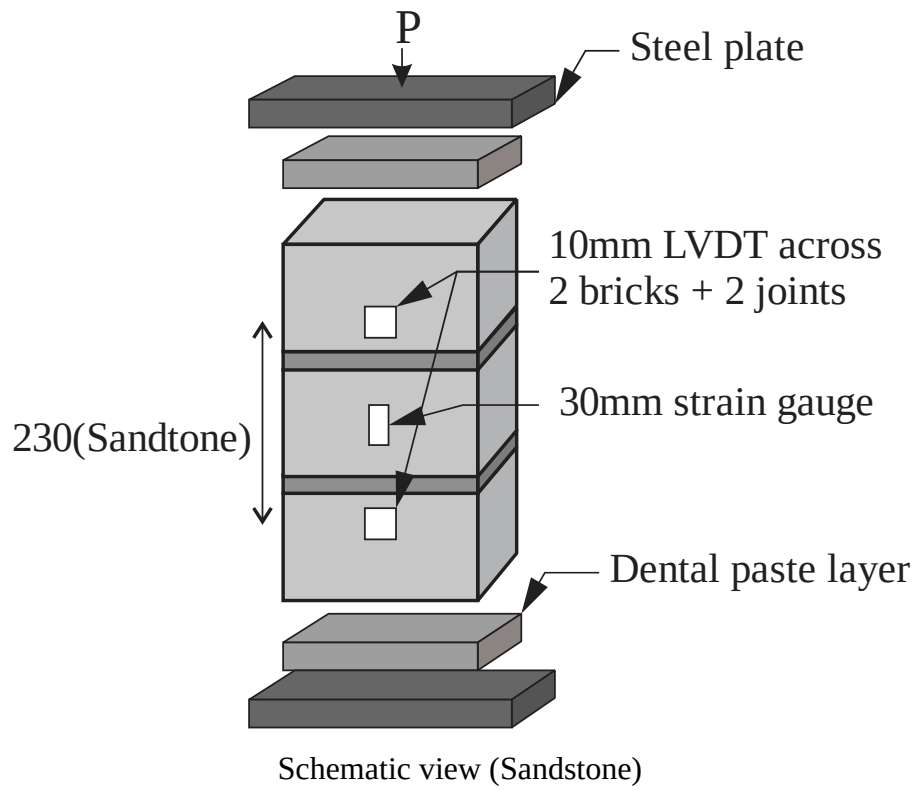


Image view

(a) Clay brick prism

(b) Sandstone prism

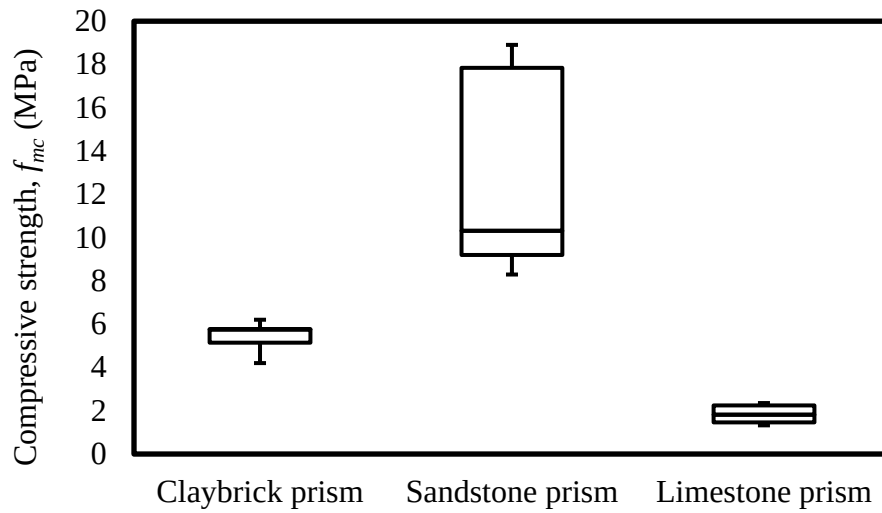
(c) Limestone prism

Figure A.10: LVDT and strain gauge instrumentation for masonry prism

The unconfined compressive strength of masonry prism was calculated accordance with AS 3700 as Eq: A.5.

$$f_{mc} = k_a \left(\frac{P_{\max}}{A_d} \right) \quad (\text{Eq: A.5})$$

where k_a is the aspect factor based on the code and calculate based on height to thickness ratio. The shape factor was taken as 0.92 for the clay brick prism, 0.95 for the sandstone prism and 0.9 for the limestone prism. Figure A.11 shows the confined strength of masonry prisms and compared difference between the estimated strength and mean of confined strength. The compressive strength of clay brick prism, sandstone prism were below estimation, but limestone prism was above the estimated value. The differences were - 8.77%, -68.73% and 33.91%. It is noted that estimated strength of sandstone was 68.73% lower than mean value but it was still within capacity of loading machine.



Compressive strength of masonry prism, f_{mc} (Mpa)									
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)	Estimated f_{mc}	Difference (%)
Clay brick prism	7	6.21	5.76	5.42	4.20	0.61	11.29	4.99	-8.77
Sandstone prism	5	18.90	10.32	12.91	8.29	4.52	35.01	7.65	-68.73
Limestone prism	5	2.36	1.82	1.84	1.32	0.41	22.24	2.79	33.91

Figure A.11: Compressive strength of masonry prism, f_{mc}

Failure of masonry is mainly governed by compressive stress rather than tensile stress, so it is important to define compressive behaviour of masonry prisms. The compressive behaviour captured by stress versus strain curve from the prism compression test and used in numerical model. Especially plasticity range of stress strain curve is the critical parameter for numerical study. However, since the compression test imposes

load control, as soon as the applied load reaches failure load, it will suddenly drop down, so the gauge reading is not true strain.

In order to understand the post behaviour of compression test, adopted idealized stress strain curve method which studied by Kaushik. The figure A.12 is illustrated that the idealized compressive stress versus strain relationship into two different stages. The two stages are parabolic variation and linear variation. Namely parabolic variation could represent with mean value of LVDT reading up to peak load, f_{mc} and mirrored the stress versus strain curve up to $0.9f_{mc}$. The post behaviour for linear variation was applied as shown in figure A.12. The figure A.13 presented the idealized stress strain curve for each masonry prism. In figure A.13 the solid line represented for parabolic variation and dash line represents linear variation for each type of unit.

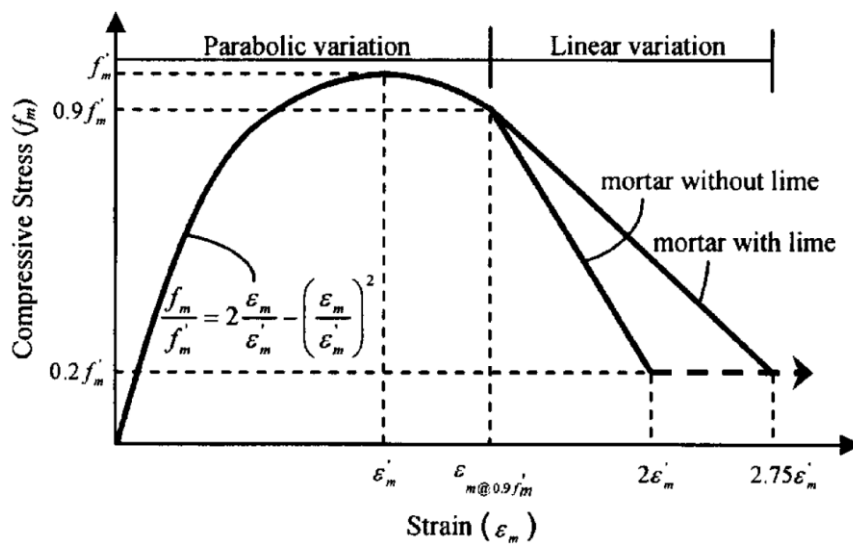


Figure A.12: Idealized compressive stress-strain relationship for masonry (Kaushik et al 2007)

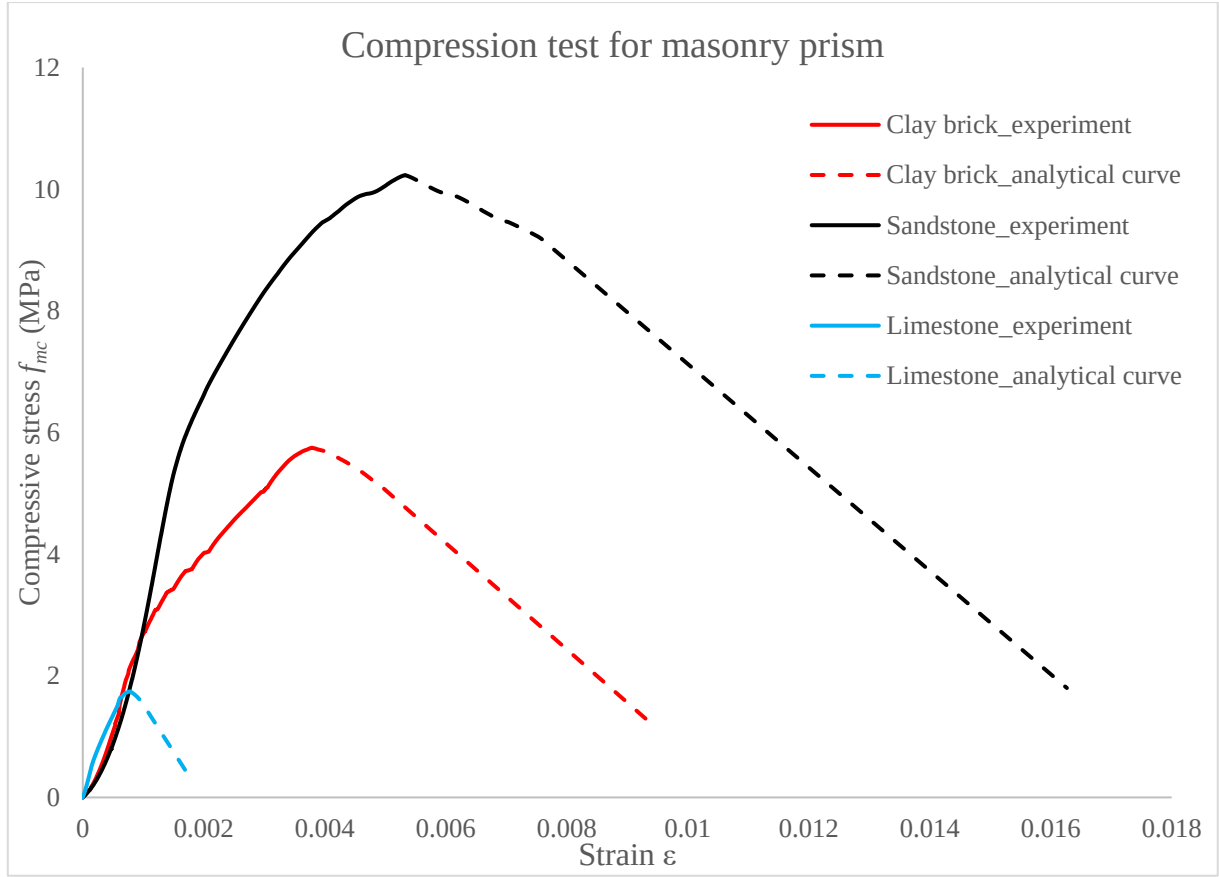


Figure A.13: Compressive stress versus strain curve of masonry prism

The young's modulus of masonry E_m could be defined as slope of stress strain curve or calculated based on the Lorenzo's proposed equation (Eq: A.6). Lorenzo introduced Eq: A.6 for simplified micro approach. Eq: A.6 where H total height of unit and joint, H_u height of unit and H_j height of joint, E_{exp} young's modulus of expanded unit and E_m young's modulus of masonry.

There is an interest within the difference between calculated E_m proposed by Eq: A.6 and slope of stress strain curve from experiments. Based on the stress limit state studied by Ewing and Kowalsky, $0.75f_{mc}$ used as control point for elasticity range. The figure A.14 represented difference between calculated E_m and experimental E_m . The mean elasticity of modulus of unit and joint were collected from Section A.2.1 and A.2 and showed a difference of result was under 35% for all masonry prisms. Thus, Eq: A.6 is well represented for elasticity of modulus of masonry. Another way to use of this equation will be an estimation of E_j by subtracting E_m from the equation.

$$E_{exp} = E_m = \frac{HE_uE_j}{H_uE_j + H_jE_u} \quad (\text{Eq: A.6})$$

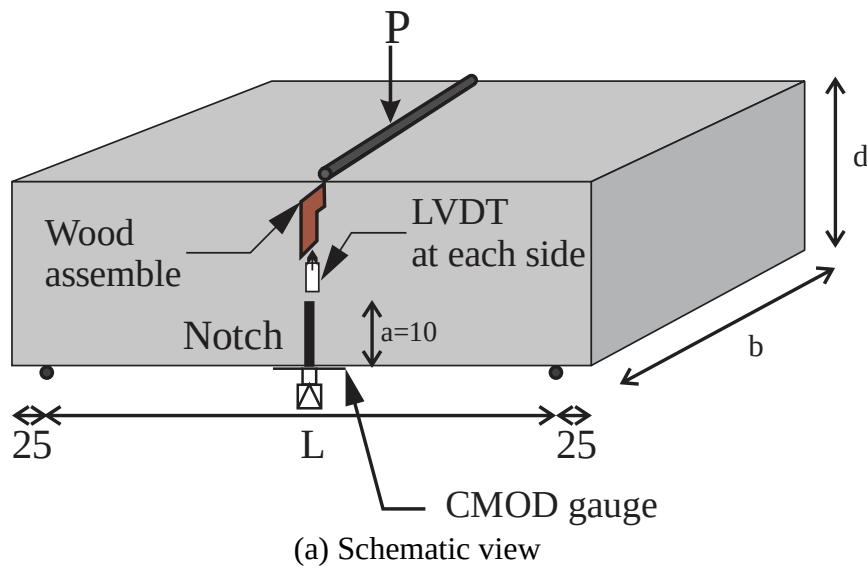
Elasticity of Modulus, E_m (Mpa)					
Material	E_{u_avg}	E_{j_avg}	$E_{_calculated}$	$E_{_experimental}$	Difference (%)
Clay brick prism	9085	436	2611	2272	13.0
Sandstone prism	10573	436	4310	3206	25.6
Limestone prism	7094	436	2230	2979	-33.6

Figure A.14: Elasticity of young's modulus of masonry, E_m

A.3 Three-point bending test

The three-point bending test was performed to investigate fracture energy, G_f^I and flexural tensile strength, f_{mt_u} . The three-point bending test for masonry unit adopted from the RILEM FMC-50 was used to determine G_f^I and f_{mt_u} in accordance with RILEM TC 187-SOC.

The main improvement of the test set up was measurement of loading point deflection. The test implemented bonded wood assemble such that it is treated as rigid body and install LVDT as shown in figure A.15. The loading point deflection is measured by deducting machine displacement from LVDT displacement. Detailed geometry of specimens are illustrated in table A.1 with the gap between the edge of a brick unit and rollers were constant 25mm. The 5mm gauge length of extensometer mounted in notch to measure crack mouth opening displacement (CMOD). The test-imposed displacement control and the displacement rate was 0.6mm/min. The depth of notch was constant 10mm and thickness of notch was 3.2mm.



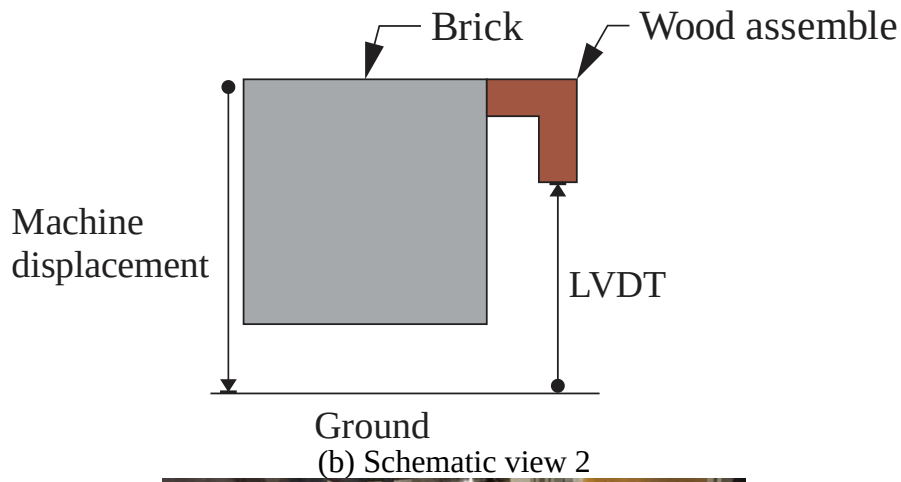
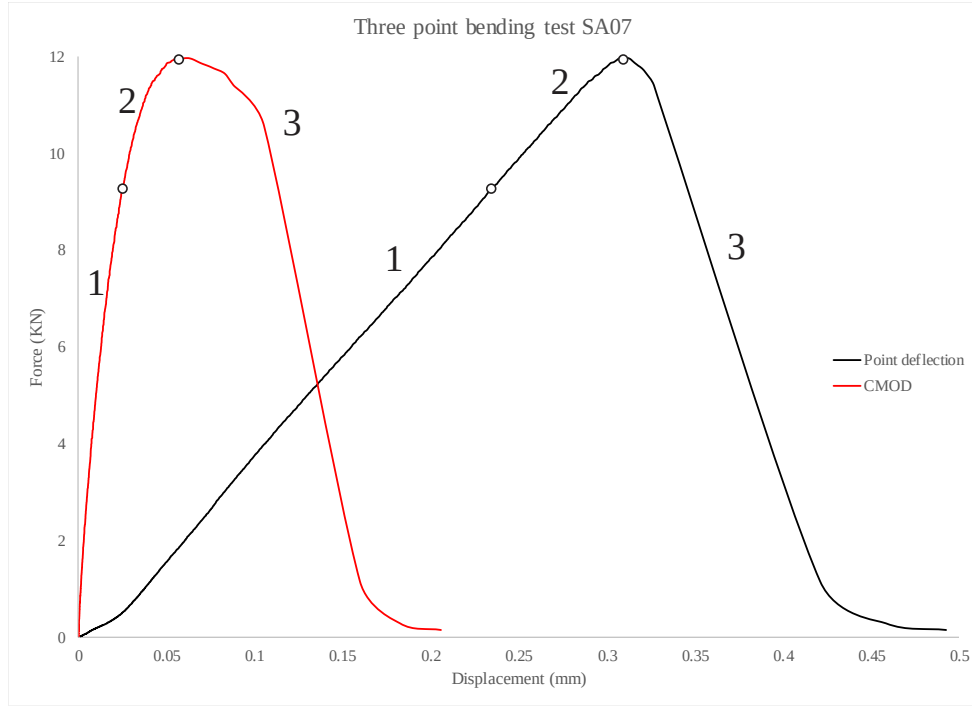


Figure A.15: Test set up for three-point bending test

The main interested parameters were calculated as shown in Eq: A.7 and A.8. Where W_0 is energy dissipated for fracture by external load represented by area of force deflection curve and m is mass of unit, g is gravitational acceleration, δ is final displacement after failure. The other notations include a , b and d are illustrated in figure A.15 (a).

Typical force displacement response of CMOD and loading point deflection represented in figure A.16 (a). The crack propagation of unit comprises in three stages. In the first stage, deflection increases linear while CMOD is opened but does not show any crack. In second stage, crack growth in notch is noticeable also visual propagation of crack mouth opening observed. In third stage, the unit failed, and rapid crack growth happened. The typical failure surface of unit indicated in figure A.16 (b).



(a) Force versus displacement for CMOD and loading point deflection



(b) Typical failure surface of masonry unit

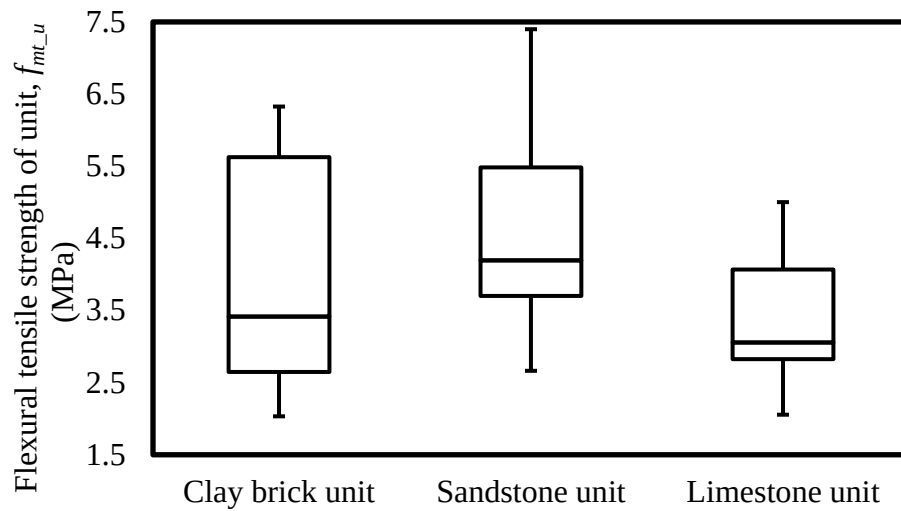
Figure A.16: Three-point bending test force displacement plot

$$f_{mt_u} = \frac{3P_{\max}}{2b(d-a)^2} \quad (\text{Eq: A.7})$$

$$G_f^I_u = \frac{W_o + mg\delta}{b(d-a)} \quad (\text{Eq: A.8})$$

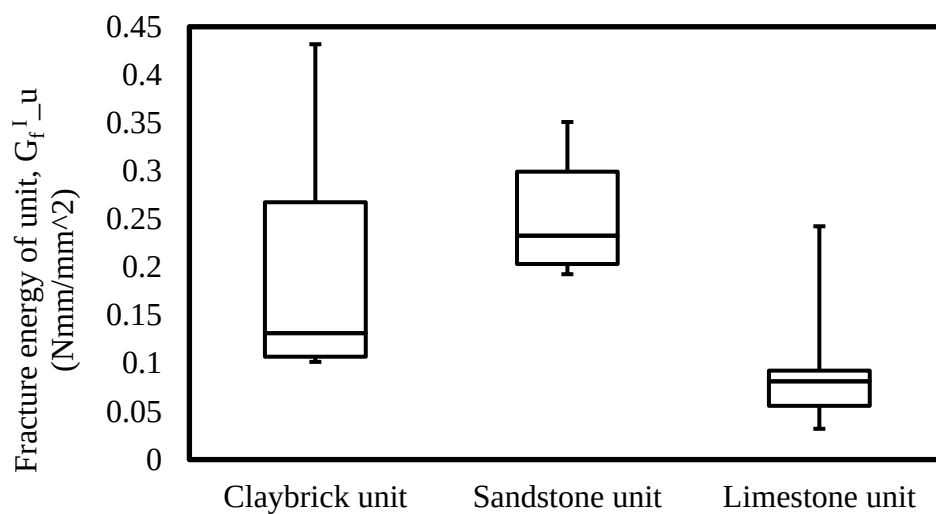
Figure A.17 shows flexural tensile strength of unit, f_{mt_u} and figure A.18 shows fracture energy for mode 1, $G_f^I_u$ quartile box plot graph. The trend of quartile box plot for f_{mt_u} has no significant difference between masonry unit. The coefficient of variation, CoV is similar between sandstone and limestone but CoV for clay brick unit is 42.67%. It is observed that random variation of clay brick is higher than stone unit.

Fracture energy mode 1, $G_f^I_u$ is calculated based on area of force loading point deflection curve and represented in figure A.18. The CoV of clay brick unit, limestone unit was higher than sandstone due to the characteristic of brittle material properties. In addition, the failure surface of clay brick and limestone were not consistent so the energy dissipation may variable as well.



Flexural Tensile strength of unit, f_{mt_u} (Mpa)							
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
Clay brick unit	6	6.33	3.42	3.99	2.03	1.70	42.67
Sandstone unit	8	7.40	4.19	4.65	2.66	1.48	31.78
Limestone unit	9	5.00	3.06	3.45	2.06	0.89	25.92

Figure A.17: Flexural tensile strength of unit f_{mt_u} (MPa)



Fracture energy of unit, $G_f^I{}_u$ (Nmm/mm ²)							
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
Clay brick unit	6	0.43	0.13	0.21	0.10	0.13	61.24
Sandstone unit	8	0.35	0.23	0.25	0.19	0.06	16.95
Limestone unit	9	0.09	0.08	0.09	0.03	0.06	64.04

Figure A.18: Fracture energy of unit from three-point bending test, $G_f^I{}_u$ (Nmm/mm²)

A.4 Bond wrench test

The bond wrench test conducted to investigate flexural tensile bond strength, f_{mt} in variety type of masonry. The test specimens were prepared 10 each for clay brick couplet, sandstone couplet, limestone couplet and bluestone couplet and the test arrangement accordance with AS 3700. Each of couplet has different bond strength and some of weak bonds have risk to fail prior to apply load due to the heavy self-weight of bond wrench bar. In order to avoid the issue, the test comprised two approaches by prepared two different size of bond wrench bars. The figure A.19 is shown the two size of bond wrench bar and data acquisition system. The small bond wrench bar transfers the calibrated strain reading into data acquisition system in computer with sample rate 20 samples per second and the big bond wrench bar has its own calibrated strain gauge, so it displays the maximum load.



Strain gauge reading transfer
into data acquisition system
in computer

(a) Small bond wrench bar



(b) Big bond wrench bar

Figure A.19: Two different size of bond wrench bars

The load is applied slowly by hand until the bond failure occurs. The f_{mt} determined accordance with AS 3700 as written in Eq: A.9, A.11 and A.12. The self-weight of bond wrench bars were measured before test and self-weight of brick, m_3 measured separately after test finished and the other notations are as described in figure A.20 and these were measured for each specimen. Direct tensile strength of unit estimated by 1/3 of flexural tensile strength as recommend by Milani. The proposed equation written in Eq: A.10.

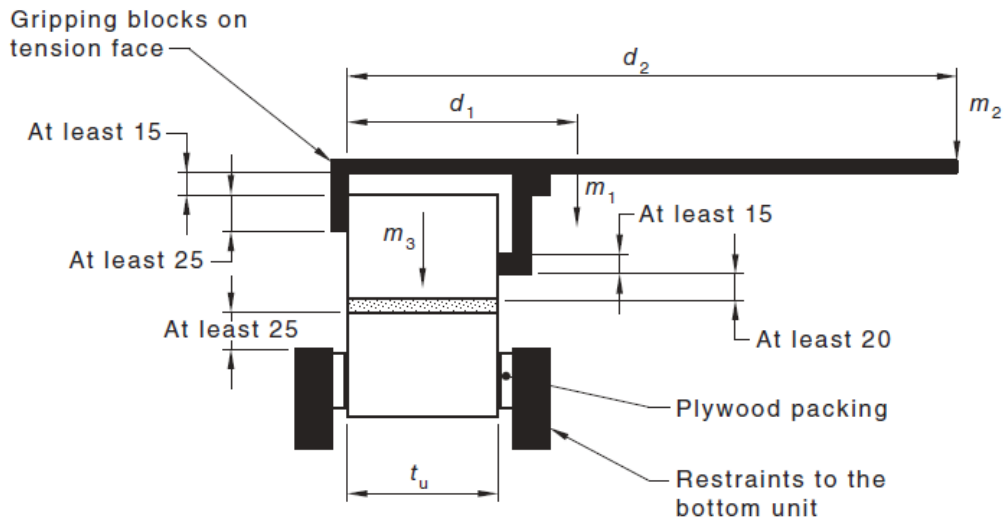


Figure A.20: Bond wrench bar elevation view (AS 3700)

$$f_{mt} = \left(\frac{M}{Z_d} \right) - \left(\frac{F}{A_d} \right) \quad (\text{Eq: A.9})$$

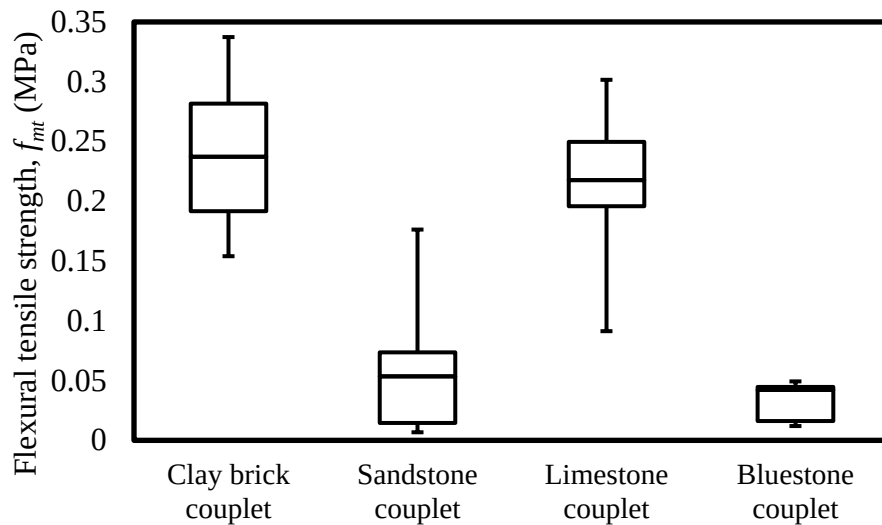
$$f_t = \frac{1}{3} f_{mt} \quad (\text{Eq:A.10})$$

$$M = 9.81m_2 \left(d_2 - \frac{t_u}{2} \right) + 9.81m_1 \left(d_1 - \frac{t_u}{2} \right) \quad (\text{Eq: A.11})$$

$$F = 9.81 \left(\sum_{i=1}^3 m_i \right) \quad (\text{Eq: A.12})$$

The test result shown in figure A.21 and the result collected from 10 specimens except bluestone couplet. One of bluestone couplet bond failed prior to the test and hence, only 9 data remained usable. The clay brick couplet, limestone couplet observed similar trends for f_{mt} and the sandstone and bluestone found minor difference only in max f_{mt} . There was notable variation found from sandstone couplet due to high CoV which is 87.14%. The failure surface of each specimen presented in figure A.22 but there was no major difference failure surface in sandstone. The unknown bond behaviour of sandstone may come from workability.

Majority of bond failure surfaces are either top or bottom surface for sandstone and bluestone couplets. On the other hand, the failure surface of limestone couplet is mainly propagated through joint as indicated in figure A.22 (c). It assumed that limestone absorb the moisture rapidly and become harden so the bond between joint and unit is stronger than joint strength. The clay brick couplet has frog, so it made unique failure surface as shown in figure A.22 (a).



Flexural tensile strength of joint f_{mt} (Mpa)							
Material	N _{spe}	Max	Median	Mean	Min	StD	CoV(%)
Clay brick couplet	10	0.34	0.24	0.24	0.15	0.06	24.60
Sandstone couplet	10	0.18	0.05	0.06	0.01	0.05	87.14
Limestone couplet	10	0.30	0.22	0.21	0.09	0.06	27.29
Bluestone couplet	9*	0.05	0.04	0.03	0.01	0.01	44.56

**One specimen already failed

Figure A.21: Flexural tensile strength of joint f_{mt} (Mpa)



(a) Failure surface of clay brick couplets



(b) Failure surface of sandstone couplets



(c) Failure surface of limestone couplets



(d) Failure surface of bluestone couplets

Figure A.22: Failure surface of couplets after bond wrench test

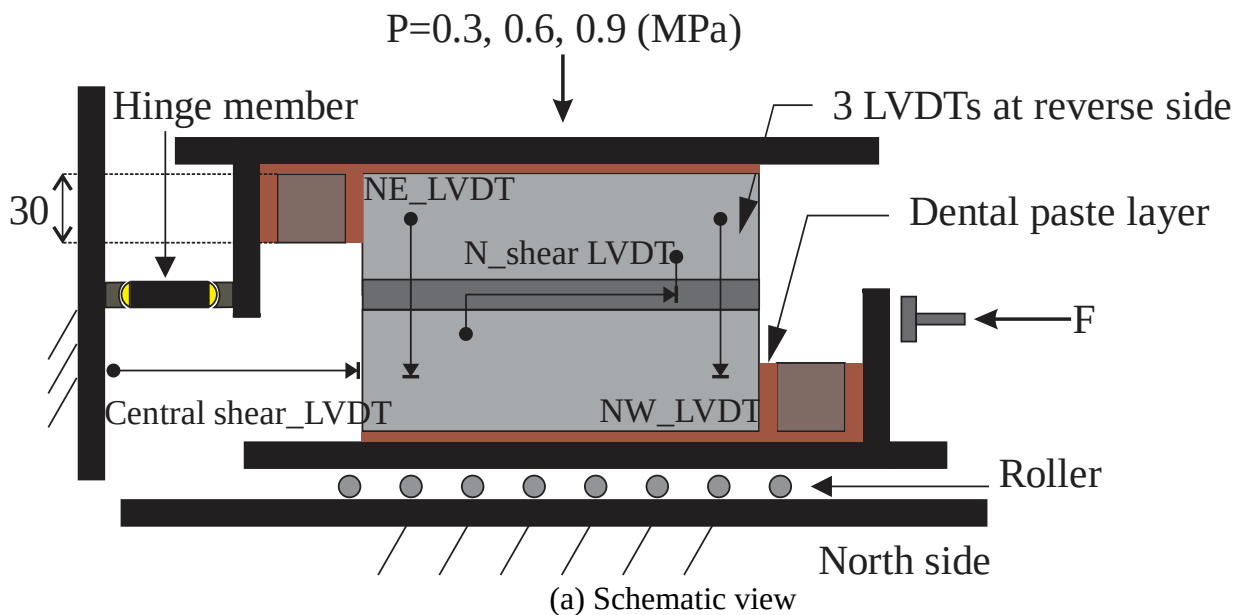
A.5 Shear couplet test

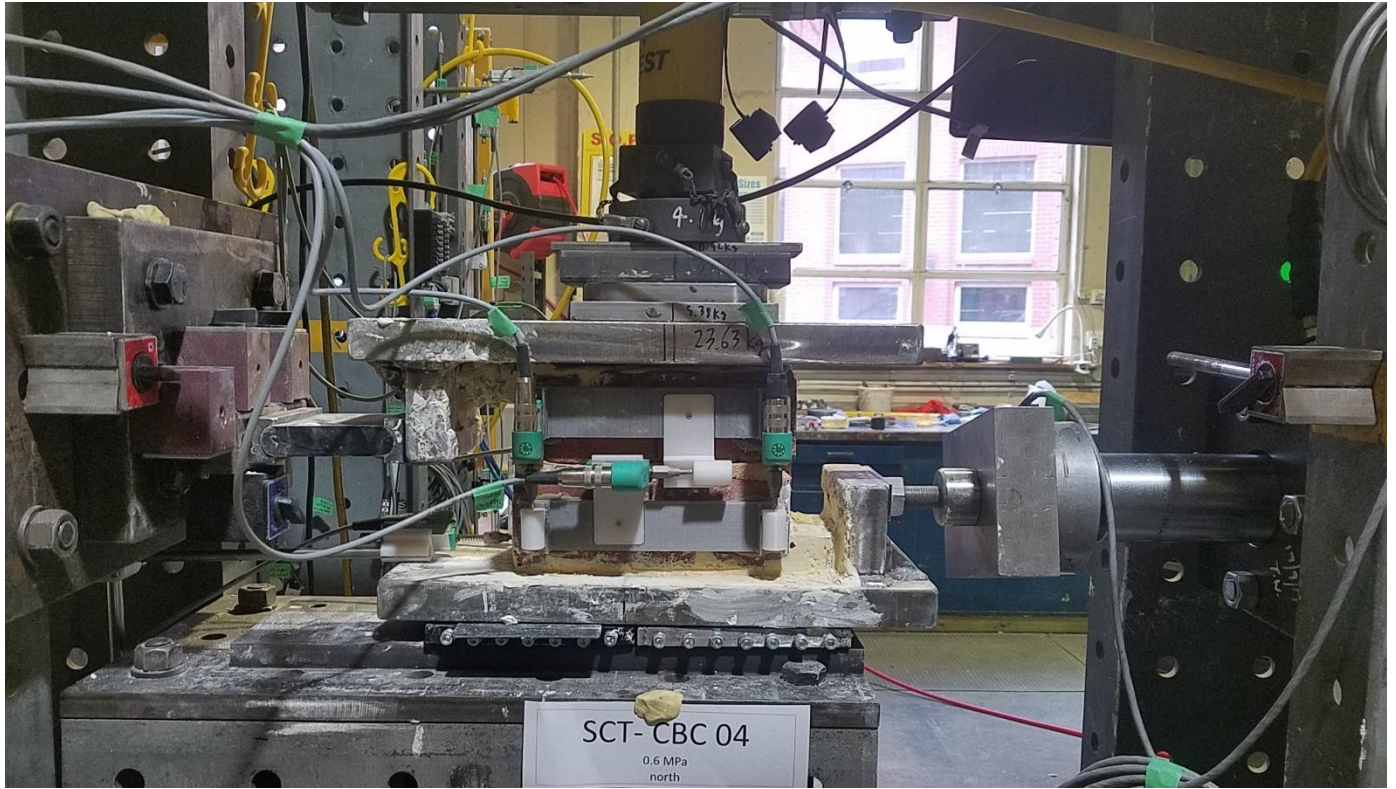
The shear couplet test was conducted to determine shear strength τ_u , residual shear strength τ_{res} , cohesion $c(=\tau_o)$, fracture energy mode 2 G_f^{II} and shear stiffness K_{ss} . The shear couplet test mainly studied by Van Der Pluijm and current study adopted and modified as shown in figure A.23. Three different levels of axial stress specified as 0.3, 0.6 and 0.9 MPa and the hydraulic ramp used included self-weight from the steel packers. The 3D printed LVDT bracket measured normal displacement and shear displacement in both sides. The test aimed to have 8mm of shear displacement so namely central shear LVDT mounted to monitor the relative shear displacement. The test imposed load control with 2KN/min until the force reached failure load after pre-failure stage, the test turned into displacement control by controlled with central shear LVDT and the displacement rate was 4mm/min.

The challenges of the test was maintaining eccentricity of the axial and shear load, while minimizing the rotation effect. The shear load must be applied through the centre of the joint and also the axial load ramp has to be positioned at the centre of the joint otherwise entire couplet will be twisted. To avoid the issue, all of the steel packers, L-plate and specimen have to align with the shear loading point. Each specimen had top and bottom L plates marked at the centre line and aligned with a shear loading cell. In addition, post processing the result for the shear displacement of joint the mean of two shear LVDTs to correct minor twisting effect. The axial hydraulic ram is in a fixed position, so rotation effect is inevitable. As shown in figure A.23 (a), the current study placed roller below the bottom L plate, so after bottom L plate has large displacement, the moment effect will be increase. The moment affects crack propagation and entire couplet

rotated. For instance, two of test specimens namely CBC 05, CBC 08 were unsuccessful due to the rotation. In order to minimize the moment effect, the axial ramp was repositioned from centre of specimen to offset 8mm in east direction.

The LVDT bracket attached in the south and north side and one central shear LVDT installed so in total seven 10mm LVDTs were used. The LVDT reading transfer into DAQ system and sampling rate was 10 data/second. The dental paste layer applied between couplet and L plate to apply uniform load. Two of 30mm steel block attached with dental paste layer to transfer load directly to couplet as shown in figure A.23 (a).





(b) Photo view (North side)
Figure A.23: Shear couplet test set up

Researchers suggest that failure envelope with Mohr coulomb failure criterion based on cohesive frictional coefficient. However, what is not yet clear is the overall representation of cohesion, c_c defined with cohesive frictional coefficient μ_c . As masonry has solitary properties in each specimen, the cohesion may not represent the whole masonry couplet. The new approach of failure envelopes is proposed to investigate cohesion. In the pre failure stage of shear couplet test mainly governed by fracture and post behaviour is governed by friction. Thus, the proposed failure envelope defined with dry frictional coefficient μ_d and modified cohesion c_d specified as gap of ultimate shear strength and residual shear strength.

The figure A.24 illustrated shear stress versus shear displacement response and the key parameters were indicated. The ultimate shear bond strength of the joint can be written in Eq: A.13 and where c represents cohesion, which is equivalent with initial shear strength, dry frictional coefficient μ_d , axial stress, σ_v and τ_u ultimate shear stress. The test aimed to collect nine meaningful responses under three levels of axial stress, so eleven couplets were used for each clay brick and sandstone.

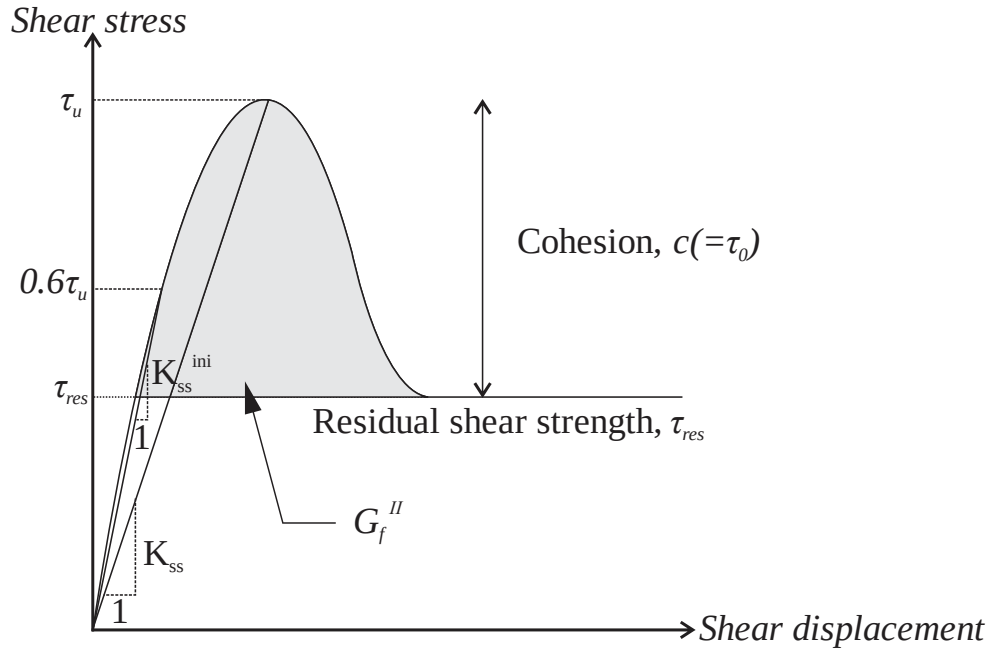


Figure A.24: Shear stress versus shear displacement response

$$\tau_u = \sigma_v \mu_d + C \quad (\text{Eq: A.13})$$

The shear stress versus shear displacement response for clay brick couplet and sandstone couplet were represented in figure A.25, A.26 and noted that only 8 data plotted for sandstone couplet due to the broken specimens. In addition, the failure surface of clay brick and sandstone couplets were indicated in figure A.27, A.28. In the figure A.26 surprisingly sandstone couplet shown almost identical stress displacement response under same axial stress. It is assumed that the reason for good consistency is derived from well positioned of steel members and identical bonded area. All of failure surfaces of sandstone couplets are either top interface or bottom surface as shown in figure A.28. Noted that specimen SSC07 shows a crumbled mortar joint but it still has a clear failure surface at the top interface.

On the other hand, clay brick shown large scatter of shear response and failure surface. The inconsistency may be due to frog in clay brick unit. As illustrated in figure A.27. the mortar joint was not completely filled the frog, so the bonded area was variable in each specimen. It contributed the inconsistency of shear response and failure surface. Relatively shear response under mid, high axial stress have better agreement compare to the shear response under low confining pressure. One noted that the shear response for CBC02 shown distinct trend. The shear stress rapidly decreases, and the residual shear strength is too low compare to CBC 01, CBC03. The residual shear strength is caused by friction so theoretically the strength should be identical. The unknown effect in CBC02 may be due to workability.

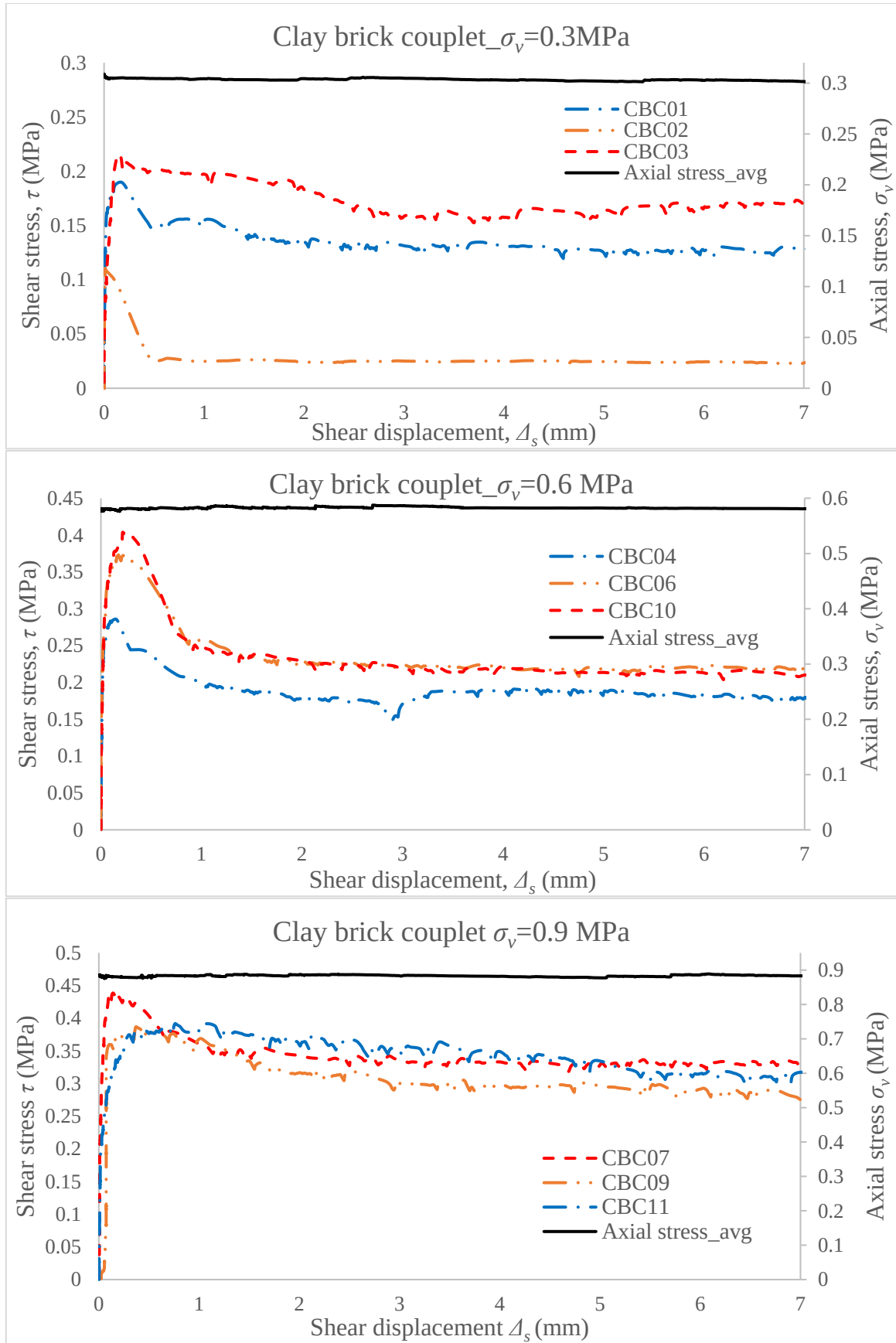


Figure A. 25: Shear stress versus shear displacement for clay brick couplet

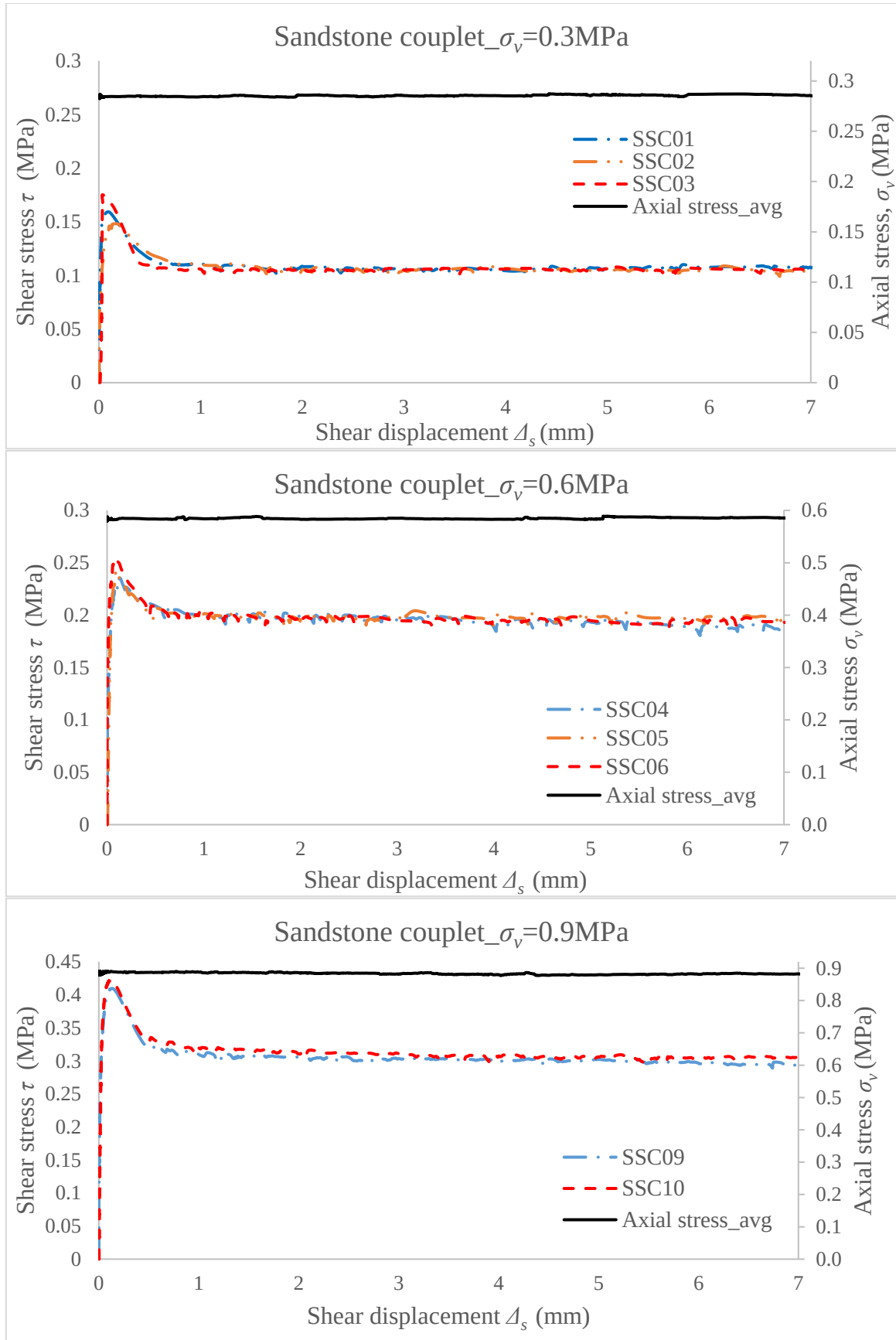


Figure A. 26: Shear stress versus shear displacement for sandstone couplet



SCT-CBC 01

0.3 MPa
north



SCT-CBC 02

0.3 MPa
south



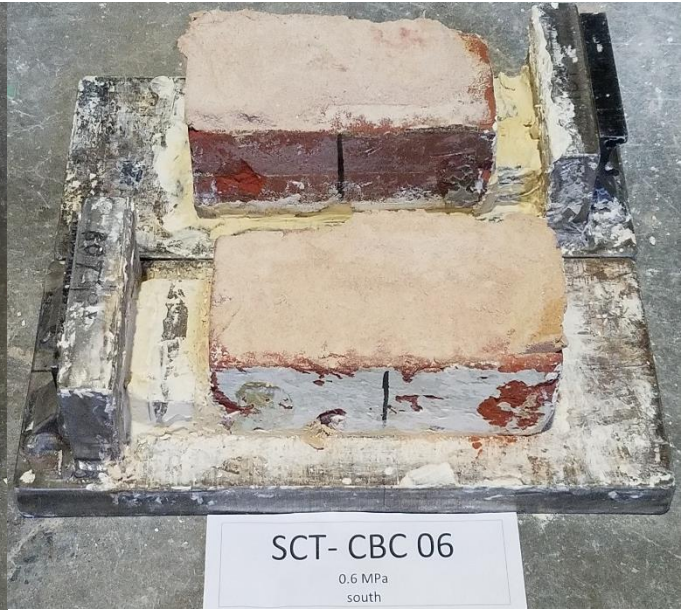
SCT- CBC 03

0.3 MPa
south



SCT- CBC 04

0.6 MPa
south



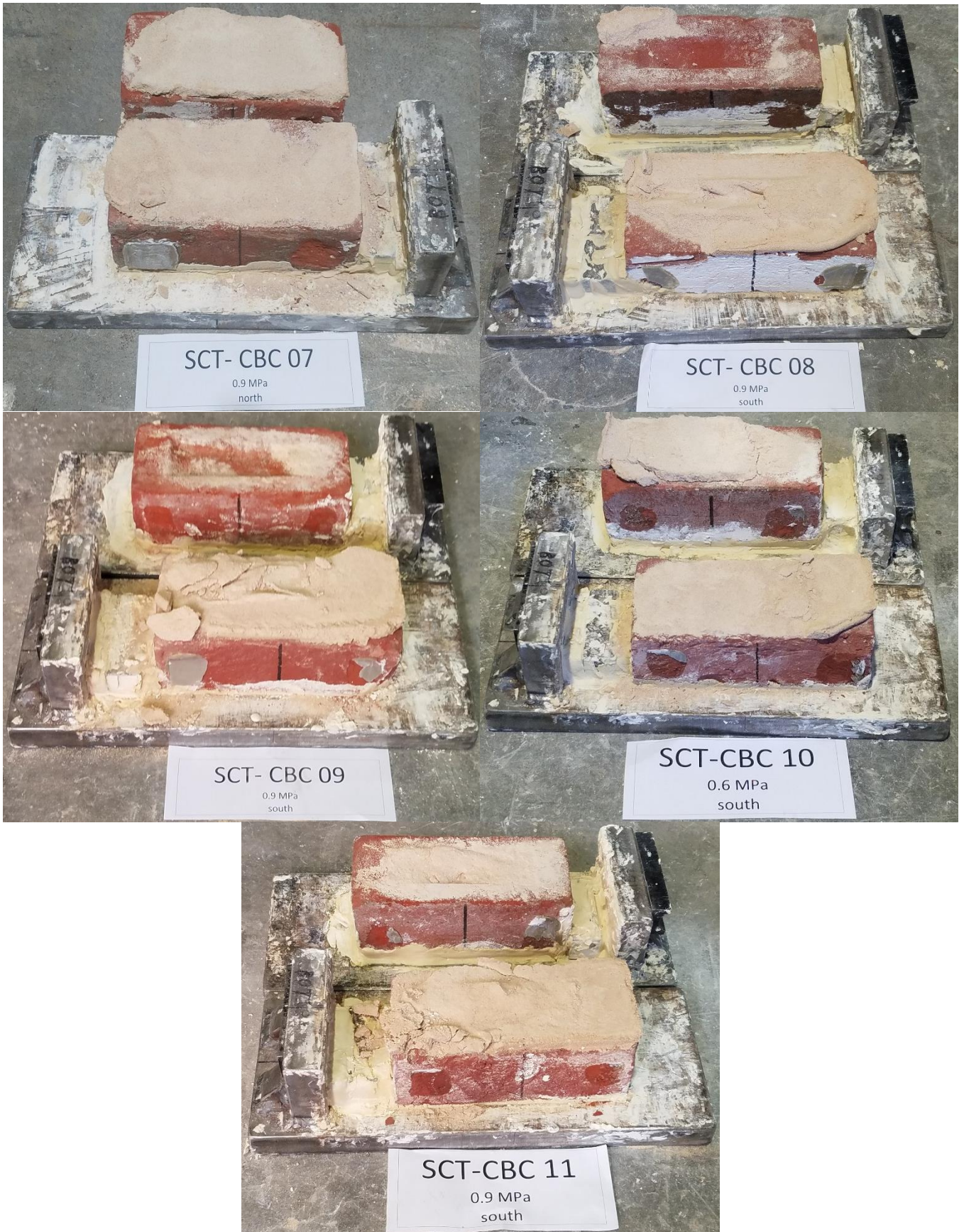


Figure A. 27: Failure surface of clay brick couplet after shear couplet test
(The name label indicated the specimen number and applied axial stress)



SCT-SSC 01

0.3 MPa
south



SCT- SSC 02

0.3 MPa
south



SCT- SSC 03

0.3 MPa
south



SCT- SSC 04

0.6 MPa
south



SCT- SSC 05

0.6 MPa
south



SCT- SSC 06

0.6 MPa
south

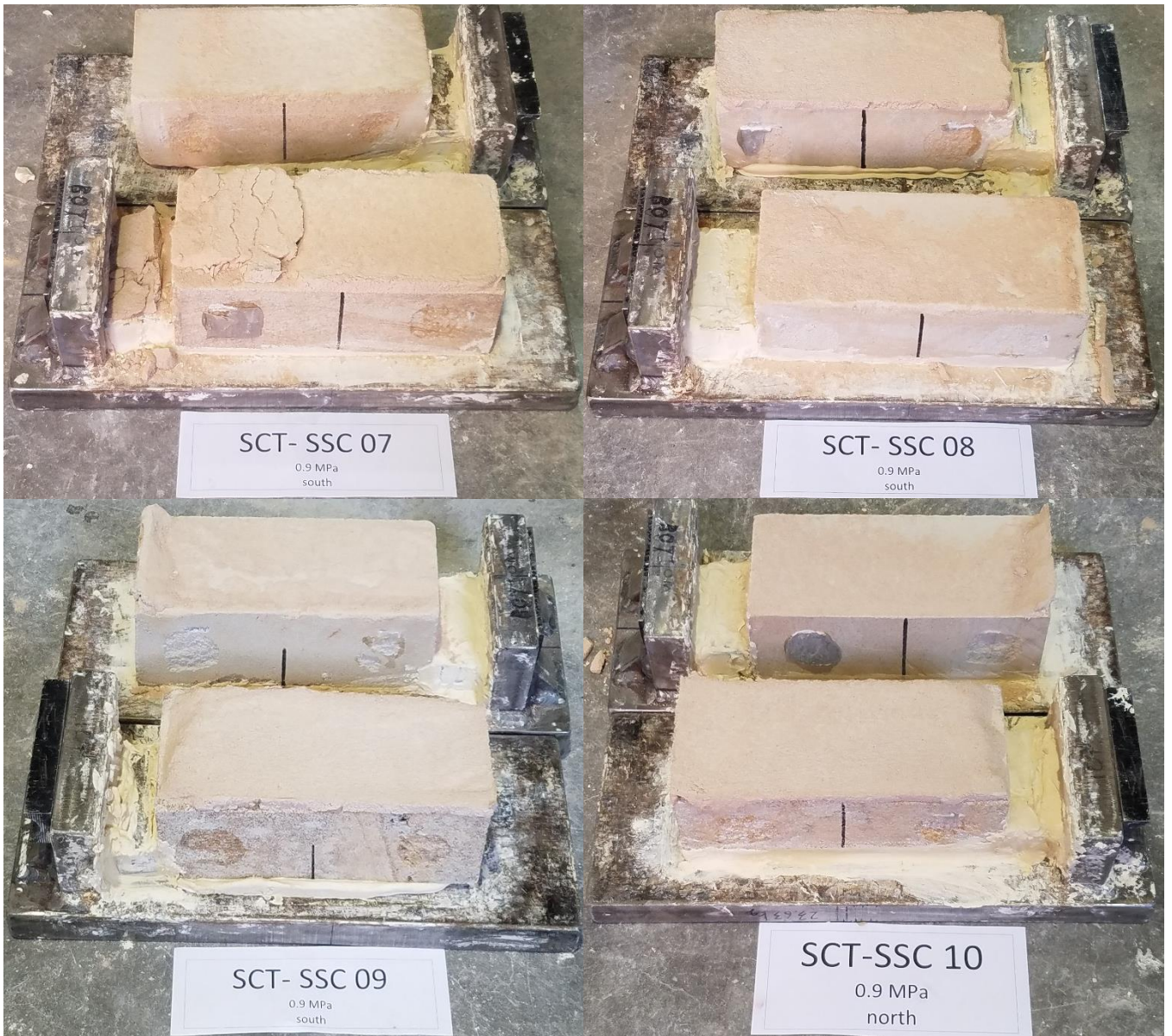


Figure A. 28: Failure surface of sandstone couplet after shear couplet test
(The name label indicated the specimen number and applied axial stress)

To be able to get the initial shear strength c , dry frictional coefficient μ_d , the regression line plotted under three level of axial stress in figure A.29, A.30. The residual shear strength, τ_{res} taken as mean of corresponding shear stress at 2mm, 6mm of shear displacement. The strength used to calculate the fracture energy mode 2, G_f^{II} .

To be able to estimate pure dry frictional coefficient, regression line with zero intercept plotted by residual shear strength versus axial stress and taken slope of regression line. Modified failure envelopes and Mohr coulomb failure envelopes and plotted in figure A.31 (a), (b). The modified failure envelopes presented in

figure A.30 and it found that there was no significant difference in R^2 respectively, but the cohesion and tensile bond strength show notable difference.

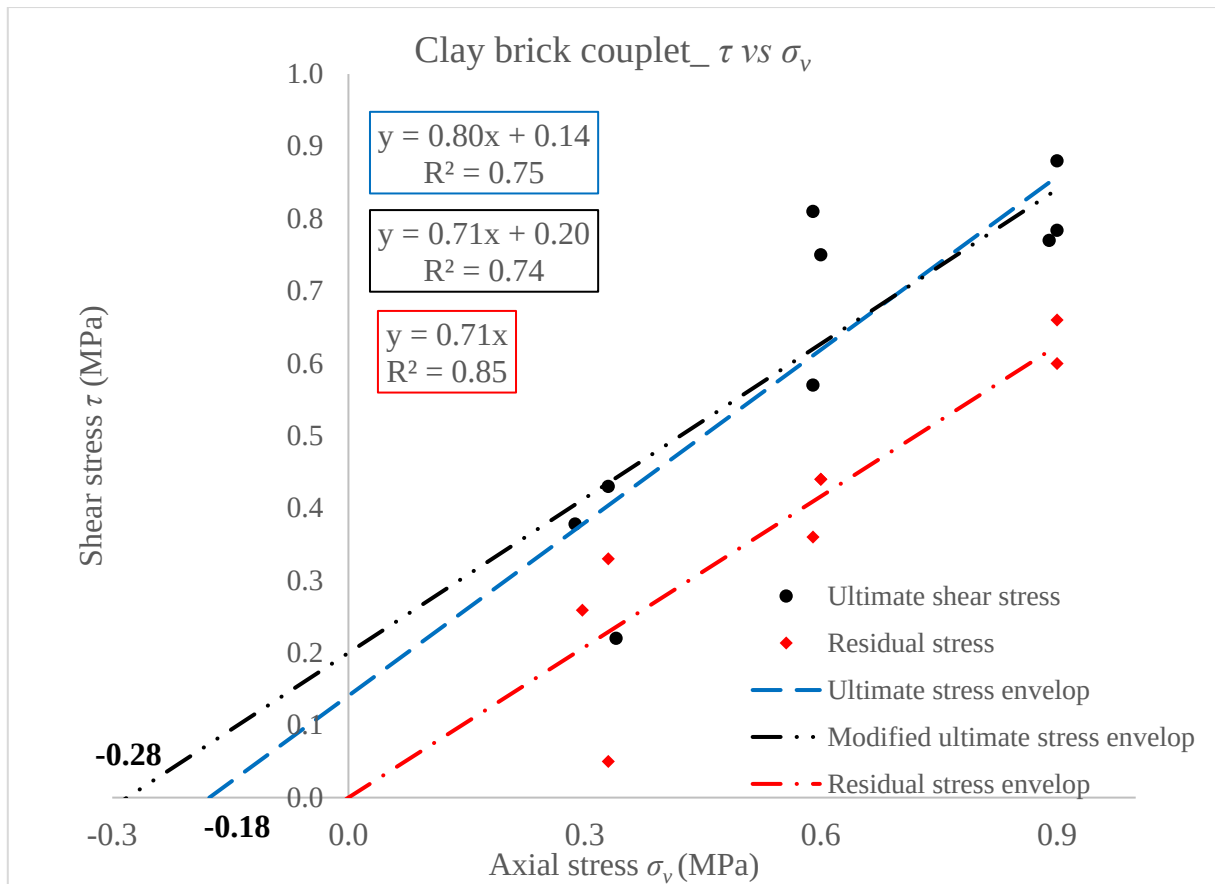
In Mohr's coulomb failure criterion defined the intercept of X-axis as direct tensile strength of joint. Comparing the flexural tensile strength and direct tensile strength from Mohr's coulomb failure criterion shows an interesting result. As indicated in figure A.21 mean of f_{mt} for clay brick and sandstone are 0.24 MPa and 0.06 MPa. Interestingly, for the clay brick, the mean of f_{mt} was in a range of two intercepts by failure envelopes. The intercept of two failure envelope were 0.28, 0.18 so the mean of f_{mt} was about middle point. The failure envelope for sandstone observed different trend. The mean f_{mt} for sandstone was 0.06MPa and the intercept of two failure envelop were 0.22 and 0.06. Thus, the mean of f_{mt} is closer to the intercept of Mohr coulomb failure envelop. It is noted that the CoV of f_{mt} was 87.14% so the large scatter of f_{mt} could be the reason why it had different trend. The cohesion shows a different trend in clay brick and sandstone. The cohesions for clay bricks were 0.2, 0.14 however cohesions for sandstones were 0.15 and 0.05. The cohesion can be also calculated individually based on Eq: A.14. To investigate variation of calculated cohesion, figure A.30 presented. Mean value of cohesion for clay brick and sandstone are 0.2 MPa and 0.13MPa and these are close to the cohesion defined with modified failure envelop.

$$C = \tau_u - \tau_{res} \quad (\text{Eq: A.14})$$

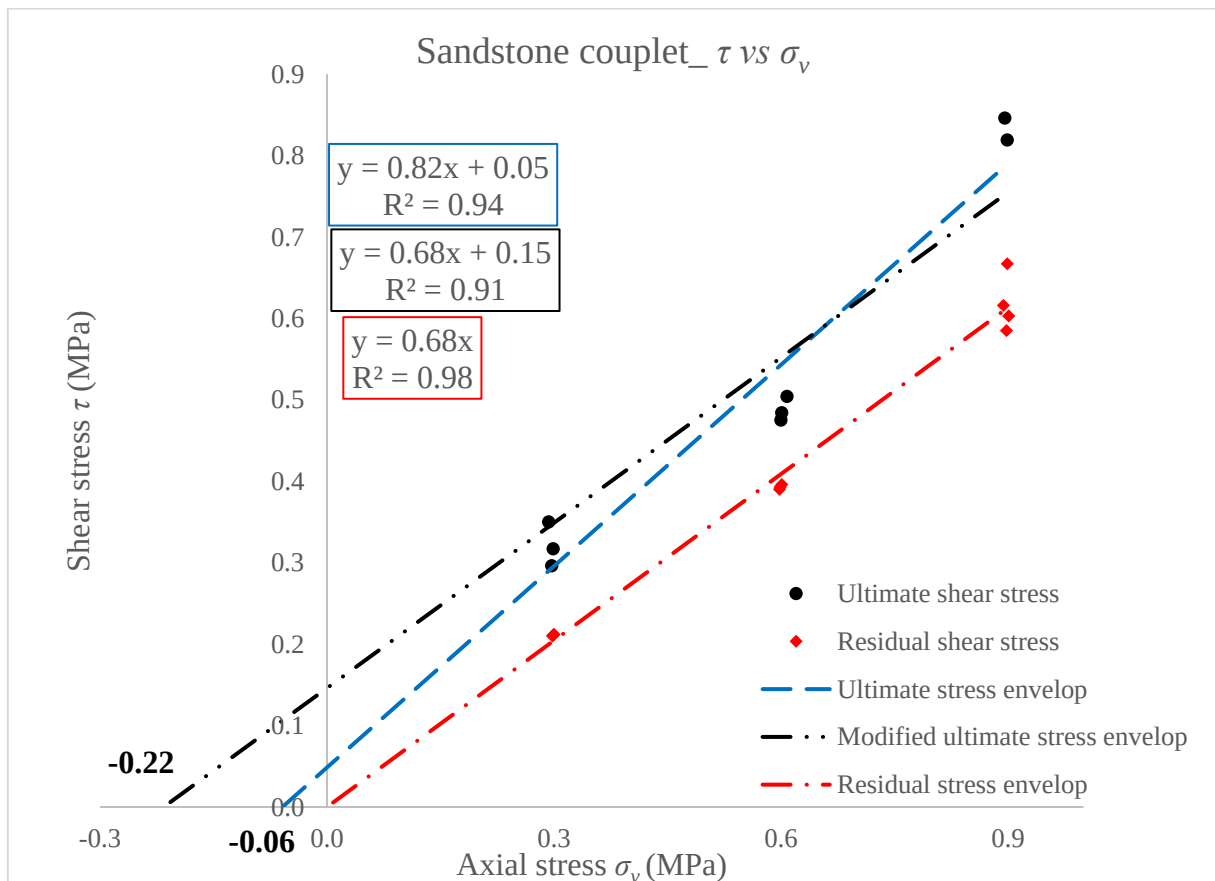
$$\mu_d = \frac{\tau_{res}}{\sigma_v} \quad (\text{Eq: A.15})$$

The dry frictional coefficient can be calculated individually as written in Eq: A.15. The calculated μ_d presented in figure A.31 to investigate coefficient of variation, CoV. The CoV of sandstone couplet is 4.31% and clay brick couplet is 31.69%. It found that clay brick has large scatter compare to sandstone couplet. Especially, in the figure A.31 under 0.3MPa, one point shown strange result and that is taken from CBC02. As observed in figure A. 25, the unknown factor may affect dry frictional coefficient. The mean of calculated dry frictional coefficient is still close to slope of regression line by residual shear stress.

Therefore, the proposed failure envelop does not show significant difference of R^2 for clay brick, sandstone but show notable difference of cohesion for sandstone. The difference of cohesion for sandstone may be derived from workability of masonry or other unknown effect due to moment effect. There is still uncertainty; however, the proposed failure envelops gave another way to estimate direct tensile strength of masonry. Further study can be focused on other type of masonry couplet to see how the proposed failure envelop represent overall cohesion.



(a) Clay brick couplet



(b) Sandstone couplet

Figure A. 29: Shear bond strength of masonry couplet failure envelopes

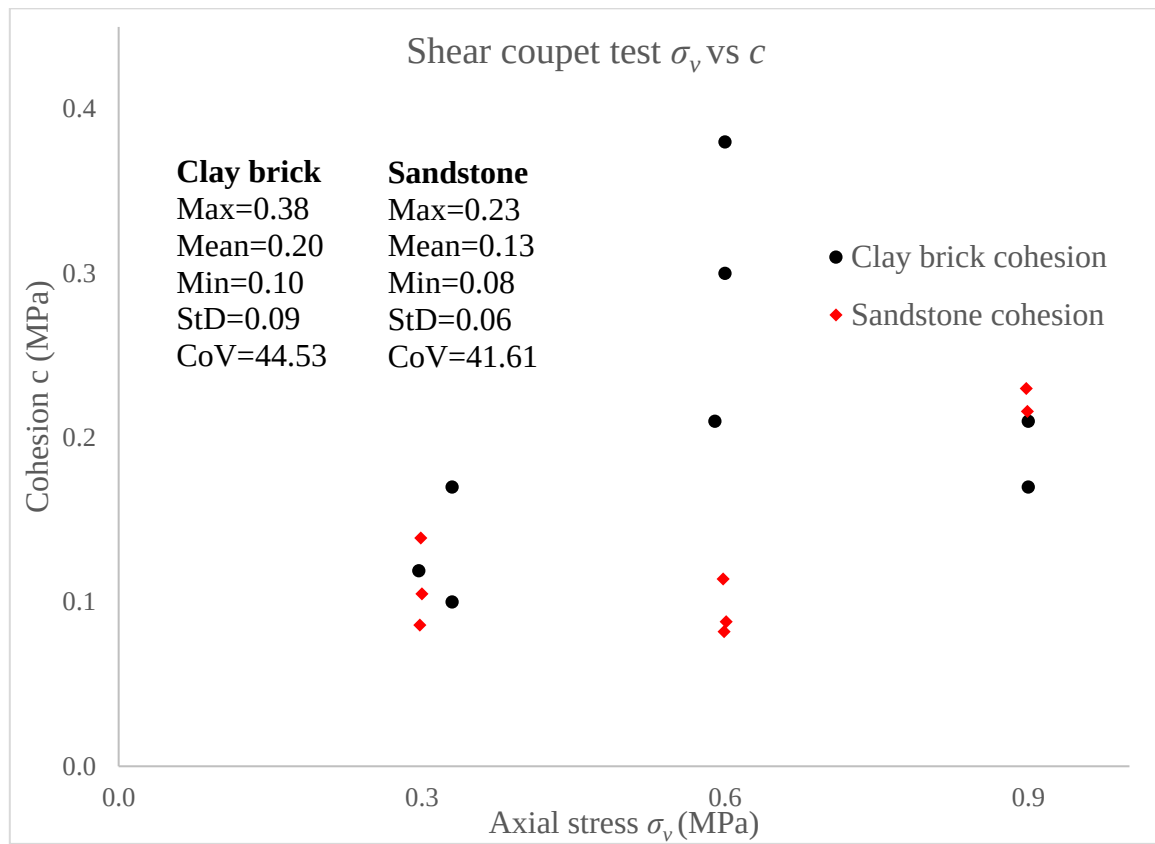


Figure A.30: Calculated cohesion c from individual test

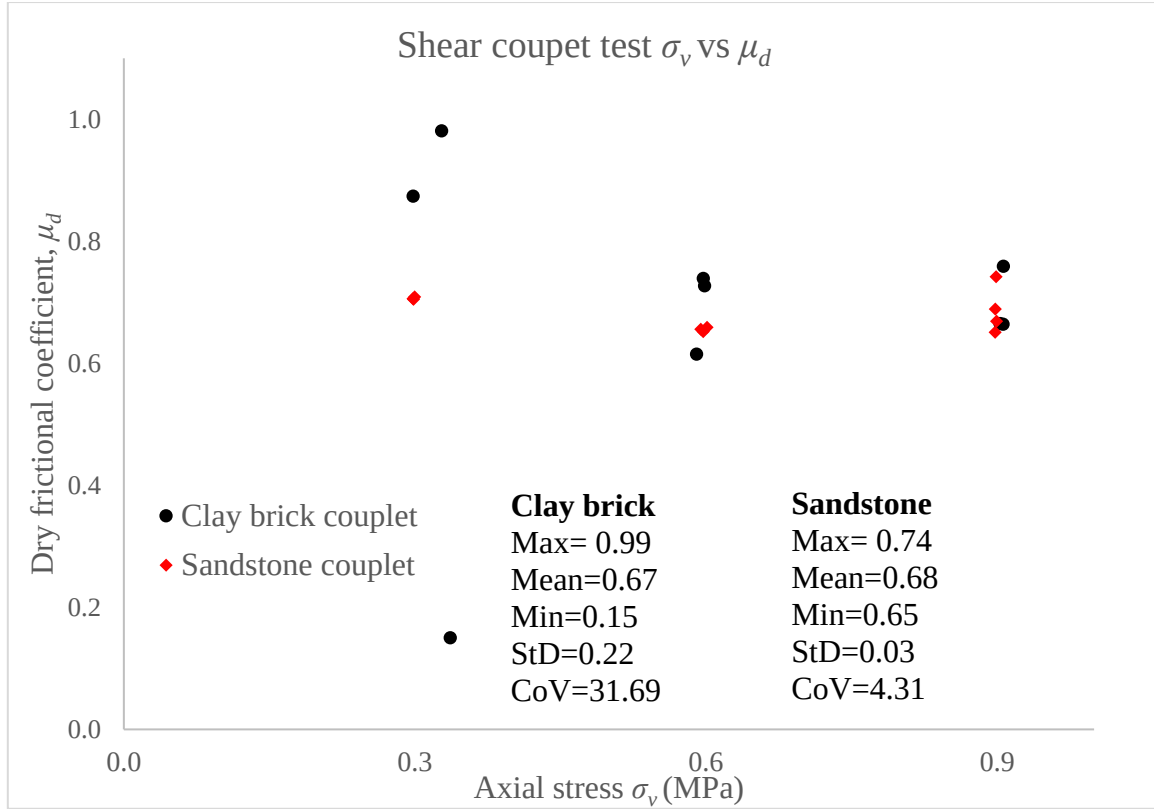
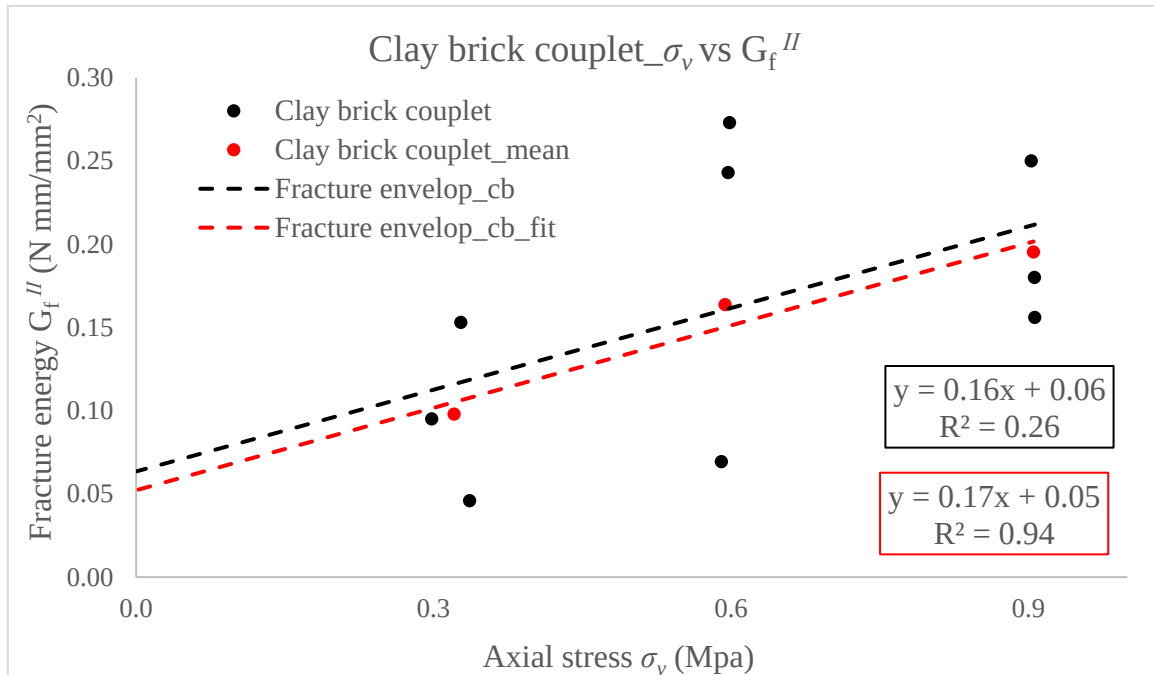


Figure A.31: Dry frictional coefficient of masonry couplet

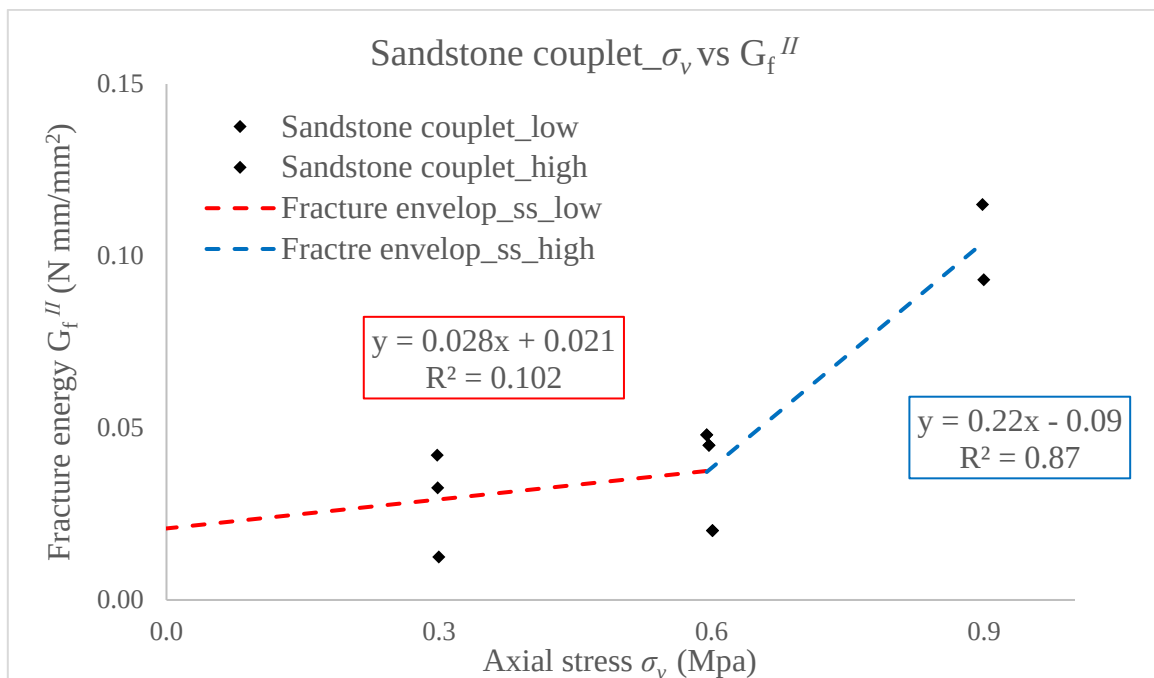
The fracture energy mode 2, G_f^{II} calculated based on area of stress versus displacement curve. It is plotted with regression line by the fracture energy versus axial stress as indicated in figure A. 32 (a), (b). In figure A.32 (a) has large scatter and it may be caused by two factors. One is external force may not fully transfer to joint mortar. For example, failure surface in CBC04 indicated unit crack as shown in figure A. 27 and it mean that the partial fracture energy used in unit failure. Another factor is the frog may affect to failure mechanism, so the failure surfaces of clay brick were variable. These factors could explain why CBC04 has lower G_f^{II} compare to CBC06 and CBC10 as shown in table A.2. Due to the frog in clay brick, the G_f^{II} has large scatter and linear fitted line does not represent for overall result. To be able to capture the better representative regression line, taken mean of G_f^{II} in each axial stress level and plotted another regression line. The two regression lines does not show prominent difference. The new regression line is more likely offset of original line.

In figure A.32 (b), shown good consistency in each axial stress level but it has remarkably low G_f^{II} at 0.6MPa. To examine the low G_f^{II} at 0.6MPa, failure surface and stress displacement response investigated. The failure surface for SSC 4,5 and 6 in figure A.28 were no difference revealed. The stress versus displacement response in figure A. 26 is also observed but no noticeable difference was found. It could be G_f^{II} governed by axial stress over 0.6MPa so in this term, bilinear method have been applied due to the low G_f^{II}

at 0.6MPa. It is noted that only two sandstone couplet results under 0.9MPa collected due to the failure specimen.



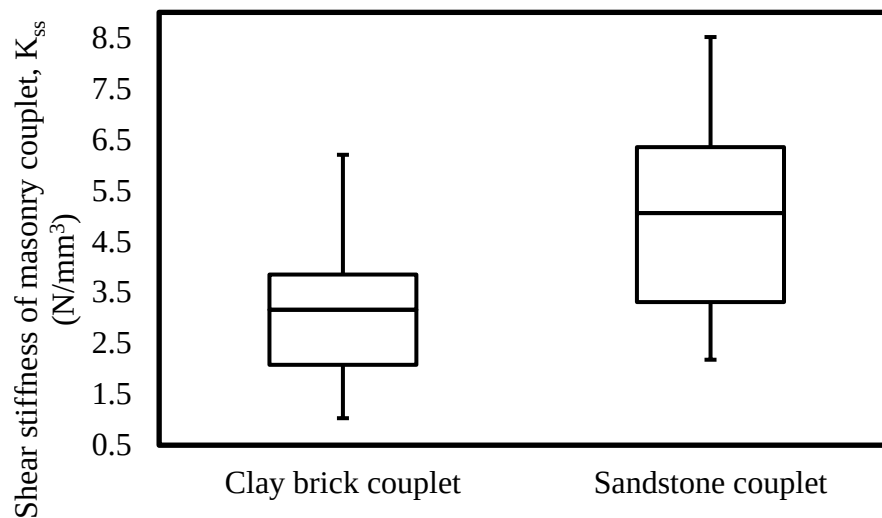
(a) Clay brick couplet



(b) Sandstone couplet

Figure A.32: Fracture energy envelope of masonry couplet

Shear stiffness, K_{ss} calculated as corresponding linear slope of stress versus displacement response at ultimate shear stress are represented in figure A.33. It is noted that CBC02 is not excluded due to the unexpected response. The mean of K_{ss} (N/mm^3) for clay brick and sandstone were 3.2 and 5.04. In addition, the CoV of K_{ss} for clay brick and sandstone were 46.67%, 40.06% and the variation were similar. The current study defines K_{ss} from the shear couplet test, but it can also be calculated based on spring stiffness as written in Eq: A.16, 17 and 18. Previous studies of stiffness have only been undertaken using Eq: 16, 17 but the estimated stiffness may not account for actual stiffness of joint interface. Thus, current study compares calculated shear stiffness and experimental shear stiffness. It is assumed that the Poisson's ratio is 0.15. As shown in figure A. 35, the calculated shear stiffness for clay brick and sandstone were 45.80 and 45.48. This is significant difference compare to experimental shear stiffness. The experiment shear stiffness is much lower than calculated spring stiffness so current study has taken experimental K_{ss} . Besides, the normal stiffness, K_{nn} also estimated is based on experimental K_{ss} as written in Eq: A.18. The key parameters of shear couplet tests were presented in figure A. 34.



Shear stiffness of joint interface, K_{ss} (N/mm^3)								
Material	N_{spe}	Max	Median	Mean	Calculated K_{ss}	Min	StD	CoV(%)
Clay brick couplet	8*	6.20	3.16	3.20	45.80	1.03	1.50	46.77
Sandstone couplet	8	8.52	5.06	5.04	45.48	2.18	2.02	40.06

* CBC02 was not collected due to unusual response

Figure A.33: Shear stiffness of masonry couplet

$$K_{nn} = \frac{t_n}{\delta_n} = \frac{E_u E_j}{H_j(E_u - E_j)} \quad (\text{Eq: A.16})$$

$$K_{ss}, K_{tt} = \frac{G_u G_j}{H_j(G_u - G_j)} \quad (\text{Eq: A.17})$$

$$G = \frac{E}{2(1 + \nu)} \quad (\text{Eq: A.18})$$

$$K_{nn} = K_{ss} * 2(1 + \nu) \quad (\text{Eq: A.19})$$

Shear bond strength of joint, τ (Mpa)							
Material	N _{spe}	μ_{dry}	c (MPa)	c_avg (Mpa)	R ²	K _{ss_avg} (N/mm ³)	Kn (N/mm ³)
Clay brick couplet	9	0.71	0.20	0.20	0.75	3.20	7.37
Sandstone couplet	8	0.68	0.15	0.13	0.91	5.04	11.59

Figure A. 34: Shear bond strength of masonry joint, τ

The table A.2 is the summary of shear couplet test result. The specimen ‘CBC’ was represented clay brick couplet and ‘SSC’ was represented sandstone couplet. The initial shear stiffness K_{ss} is taken corresponding linear slope of stress versus displacement response at $0.6\tau_u$ and other parameters were taken as described in previous paragraph. The graphical description of key parameters in stress versus displacement curve shown in figure A. 24.

Note that the CBC05 and CBC08 results were not collected due to the rotation effect. The SSC07, SSC08 were completed but there was no peak load observed so only K_{ss}^{ini} , τ_{res} and μ_{dry} have been collected and SSC11 failed during lifting of specimen.

Specimen	at $0.6\tau_u$	Ultimate strength (MPa)			Residual strength (MPa)			Fracture energy (N mm/mm ²)		Cohesion (MPa)
	K_{ss}^{ini} (N/mm ³)	τ_u	at σ_v	K_{ss} (N/mm ³)	τ_{res}	at σ_v	$\mu_{dry} = \tau_{res}/\sigma_v$	G_f^{II}	at σ_v	$c = \tau_u - \tau_{res}$
CBC 01	26	0.38	0.29	2.00	0.26	0.30	0.87	0.10	0.30	0.12
CBC 02	38.6	0.22	0.34	18.38	0.05	0.33	0.15	0.049	0.33	0.17
CBC 03	6.33	0.43	0.33	2.86	0.33	0.33	0.98	0.15	0.33	0.10
CBC 04	30.2	0.57	0.59	3.71	0.36	0.59	0.62	0.07	0.59	0.21
CBC 05*	-	-	-	-	-	-	-	-	-	-
CBC 06	28.6	0.75	0.60	4.27	0.44	0.60	0.74	0.24	0.60	0.30
CBC 10	19.5	0.81	0.59	3.46	0.44	0.60	0.73	0.27	0.60	0.38
CBC 07	22.1	0.88	0.90	6.20	0.66	0.90	0.74	0.18	0.90	0.21
CBC 08*	-	-	-	-	-	-	-	-	-	-
CBC 09	16.2	0.77	0.89	2.11	0.60	0.90	0.67	0.25	0.90	0.17
CBC 11	11.7	0.78	0.90	1.03	0.68	0.90	0.76	0.16	0.90	0.10

*Test response was not complete due to rotation

(a) Clay brick couplet

Specimen	at $0.6\tau_u$	Ultimate strength (MPa)			Residual strength (MPa)			Fracture energy (N mm/mm ²)		Cohesion (MPa)
	K_{ss}^{ini} (N/mm ³)	τ_u	at σ_v	K_{ss} (N/mm ³)	τ_{res}	at σ_v	$\mu_{dry} = \tau_{res}/\sigma_v$	G_f^{II}	at σ_v	$c = \tau_u - \tau_{res}$
SSC 01	20.5	0.32	0.30	2.90	0.21	0.30	0.71	0.013	0.30	0.11
SSC 02	8.45	0.30	0.30	2.18	0.21	0.30	0.70	0.042	0.30	0.09
SSC 03	31.1	0.35	0.29	8.52	0.21	0.30	0.71	0.33	0.30	0.14
SSC 04	17.5	0.48	0.60	3.45	0.39	0.60	0.66	0.048	0.60	0.08
SSC 05	14.1	0.48	0.60	5.30	0.40	0.60	0.66	0.20	0.60	0.09
SSC 06	48.3	0.50	0.61	4.82	0.39	0.60	0.65	0.045	0.60	0.11
SSC 07**	11.8	-	-	-	0.67	0.90	0.74	-	-	-
SSC 08**	5.74	-	-	-	0.59	0.90	0.65	-	-	-
SSC 09	38.6	0.82	0.90	6.14	0.60	0.90	0.67	0.093	0.90	0.22
SSC 10	31.7	0.85	0.90	7.01	0.62	0.89	0.69	0.12	0.90	0.23
SSC11**	-	-	-	-	-	-	-	-	-	-

**Specimen already broken

(b) Sandstone couplet

Table A.3: Key parameters from shear couplet test

B DIC APPLICATION

B.1 Introduction

Recently, there has been renewed interest in DIC for full-field measurements. It has been noted that DIC is the ideal tool to capture crack propagation of in many fields. A number of studies including, Ramos et al (2015), Bolhassani et al (2015, 2016) and Nghiem et al. (2015). Ramos et al (2015) have studied and reported a large scale of reinforced masonry wall under horizontal cyclic loading with 2D and 3D-DIC experiments. It was the first attempt to use the 3D DIC for out-of-plane and DIC result shown good agreement with LVDT. The author has emphasised the importance of the 3D DIC as a new approach of structural health monitoring. Bolhassani et al (2015, 2016) used the DIC for grouted and ungrouted masonry assemblage by carrying out compression test of prism, bed joint shear test and diagonal compression test. The experiments associated with traditional LVDT and validate the DIC result by comparing the LVDT travel. The result of DIC shown good agreement with LVDT and it also monitored failure pattern by maximum principal strain contour. Nghiem et al (2015) explored new approach of DIC technique to evaluate masonry damage. The study attempted to identify the crack opening along the joint by integrating aspect of crack width.

Even though the DIC has good agreement with LVDT and captured the crack propagation however, a number of unanswered questions remain about error source of DIC. The remain unclear error sources discussed in chapter B.2, B.3 for further improvement of DIC application. The error sources of DIC can be identified into two aspects. One is the experimental environment such as light saturation, image contrast, aperture of camera and the other one is correlation algorithms such as window size, speckle pattern and step. Many researchers used DIC analysis[Ramos et al, Bolhassani et al, Nghiem et al] however still no sufficient study conducted to correct error sources so current study delivered a deeper insight into lens correction by employing user-defined calibration plate and image correction.

B.2 Speckle pattern method

As an optical tool, the DIC analysis is able to capture the displacement of virtual gauge and specify failure pattern with principal strain contour.

The DIC algorithm does not simply track the speckle dot on specimen surface but it tracks the normalized speckle in subset based on grey level. Thus, in the tracking process, the subset shifts until speckle pattern matches to the image of deformed surface and the match is calculated as the total difference in grey level.

If the size of the pattern is too large, the subsets can be just pure black or white field which is undesired to perform tracking. If the size of pattern is too small, it will have aliasing effect in DIC analysis. Thus, the role of speckle pattern is critical in DIC analysis. In order to achieve comprehensive idea of proper speckle pattern, the current study involved the influence of speckle quality for large scale of URM wall.

The speckle pattern can be made through several methods as shown in figure B.1. It can be made by marker pen, stencil, paint gun or any other methods. The key important thing for speckle pattern is that the density of speckle, dot size and random variation. Each speckle pattern application has its own advantages and disadvantages for speckle quality.

The primary concern about density of speckle is to have enough contrast on interested area. The tracking algorithm may lose the tracking under low contrast in capture image. For example, the speckle generated by marker pen can be advantage of large speckle size, but it may lose the information due to the low density. On the other hand, paint gun gives better density, but it may too small size of dot. The self-adhesive stencil can be strong method in terms of speckle size and density, but the random variation is still remained as weak point relatively. The size of dot is engaged with the object distance between camera to specimen. Especially, the current research deals with large scale of URM wall so the object distance is more than 2.5m.

In order to capture the large interested area with long object distance, determination of proper speckle pattern method was essential. The author generated the speckle patterns made by marker pen, paint spray and brush and evaluated with GOM correlated software package. Based on pattern quality tool in GOM correlate, author determined the paint spray is the most suitable tool in terms of density, size of dot and random variation. The optimized dot size is 2-4mm and the pattern made by bold out spray nozzle as shown in figure B.2.

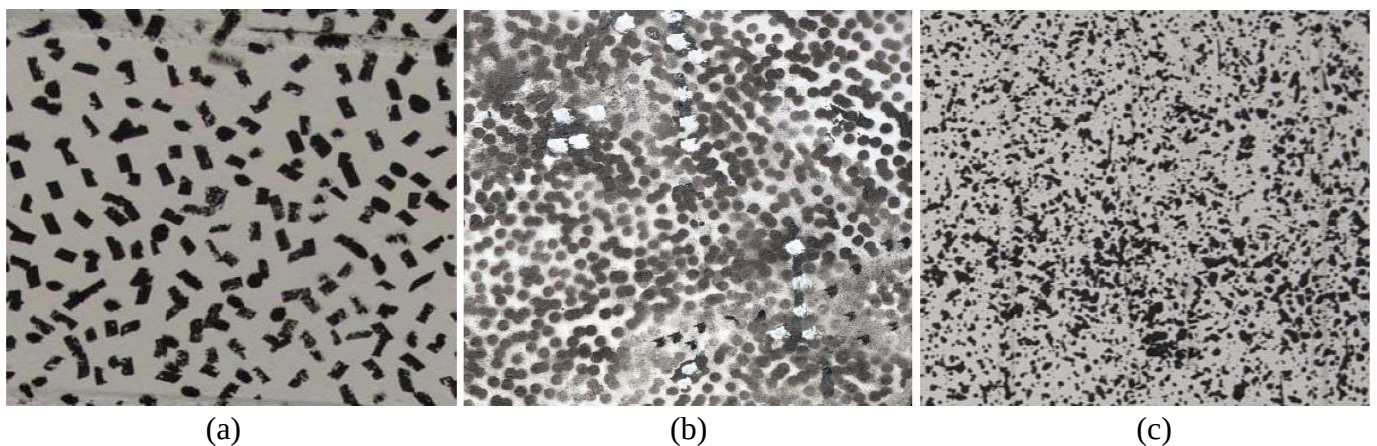


Figure B.1: Random pattern: (a) marker pen, (b) stencil (c) paint gun. (Salmanpour et al. 2013)

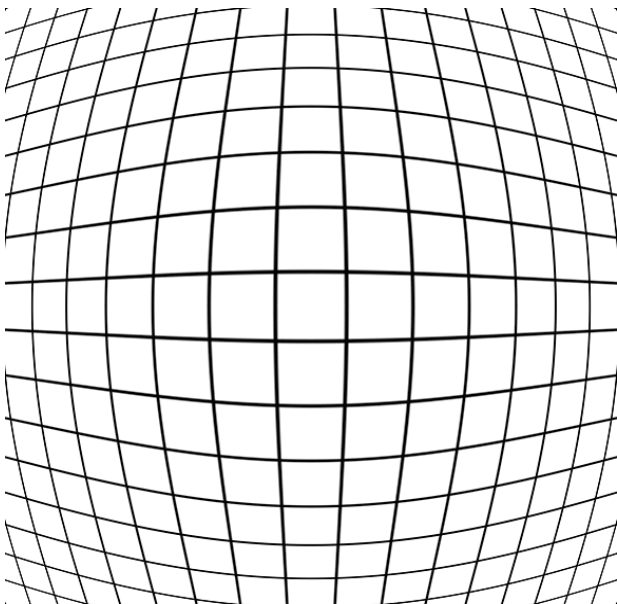


Figure B.2: Bold out paint spray nozzle

B.3 Error source and calibration processing

The current study carried out 2D DIC with DSLR camera and the lens distortion fixed by calibration plate and image processing. The common issue for lens distortion is barrel distortion due to the wide angle of lenses. The figure B.3 (a), (b) are shown the example of barrel distortion with exaggerated URM wall image. The distortion will cause error measurement of virtual gauge and principal strain contour graph. In order to fix the distortion effect, calibration plate and image processing was adopted. The calibration plate with 35mm spacing of grid selected based on the recommendation of Correlates solution company. Several photos are taken in different angles to correct lens distortion as shown in figure B.4.

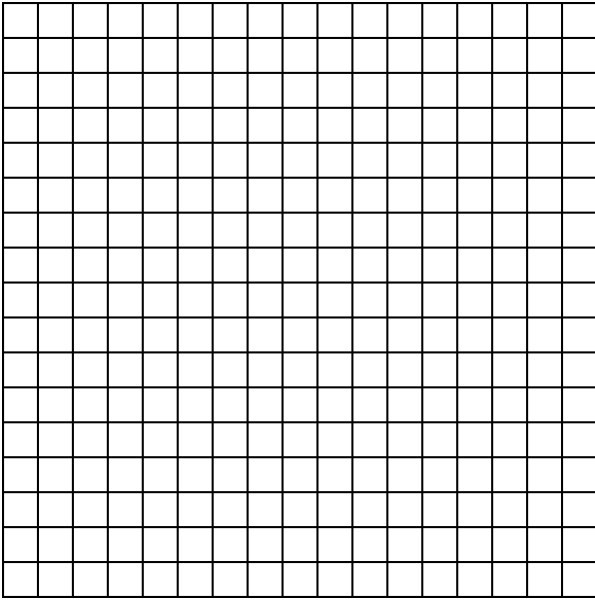
A number of photographers faced with the distortion problem especially for landscape photo, so they introduced image processing method. The image processing is the correct image based on lens profile and current study used Adobe camera raw software package. The images captured in raw file format and correct the distortion based on lens profile and convert the file format as TIFF. It is not recommended to use JPEG format due to the image information loss, so all images converted as TIFF format.



(a) Barrel distortion



(b) distorted masonry panel image



(c) fixed barrel distortion



(d) Calibrated masonry panel image

Figure B.3: Comparison of distorted image and calibrated image

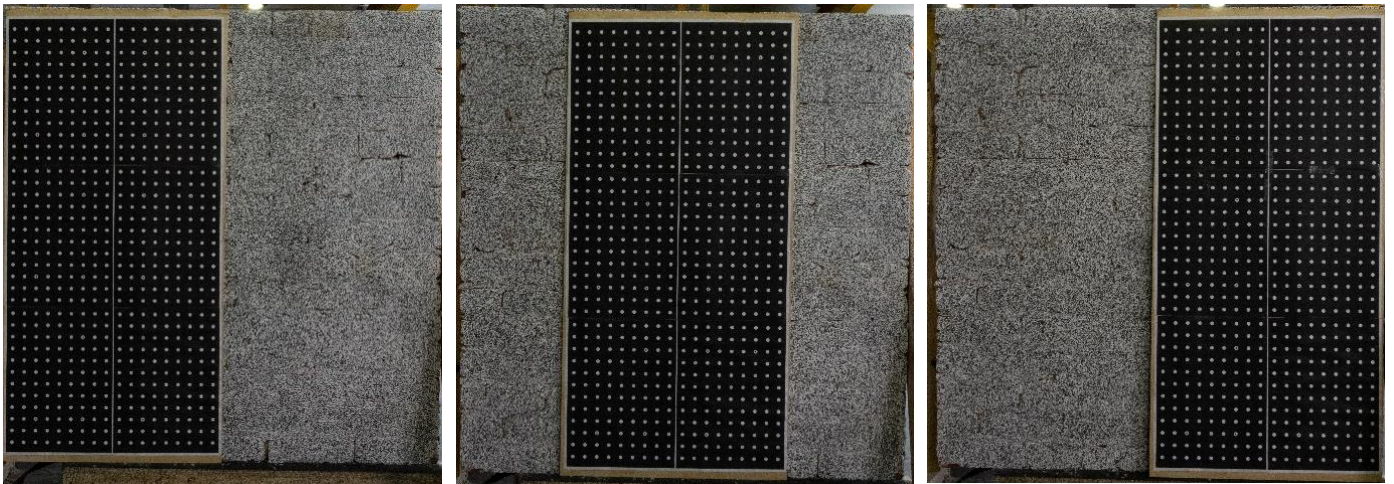


Figure B.4: Calibration image with wood board calibration plate

C DIAGONAL COMPRESSION TEST

C.1 Test arrangement

The diagonal compression test was carried out to determine diagonal tensile strength of masonry. The diagonal tensile strength, f_t of masonry panel regulated by ASTM standard, RILEM standard. It is important to understand the different identification of f_t in each standard, so the proposed equations presented in Eq: E.2 and E.3. Total 4 of URM walls have been prepared for diagonal compression test and each two walls made by clay brick and sandstone. The dimension of URM walls are 1190mm by 1200mm and detail geometry of wall presented in figure C.1. 10mm of mortar layer place on the base so all of walls has 1200mm of height. The sandstone wall made with 70mm height of unit on top coarse as shown in figure C.1.

$$A_{net} = \frac{1}{2}(L + H)Tn \quad (\text{Eq: C.1})$$

$$f_{t_Rilem} = 0.5 \frac{P}{A_{net}} \quad (\text{Eq: C.2})$$

$$f_{t_ASTM} = \frac{1}{\sqrt{2}} \frac{P}{A_{net}} \quad (\text{Eq: C.3})$$

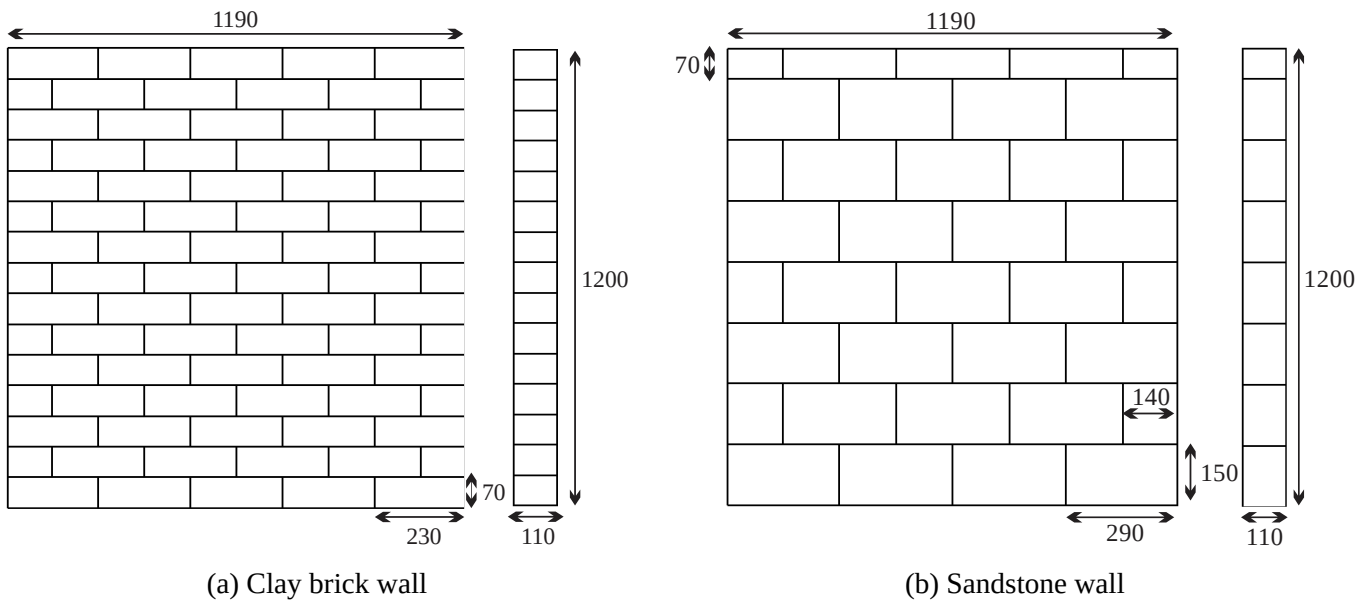
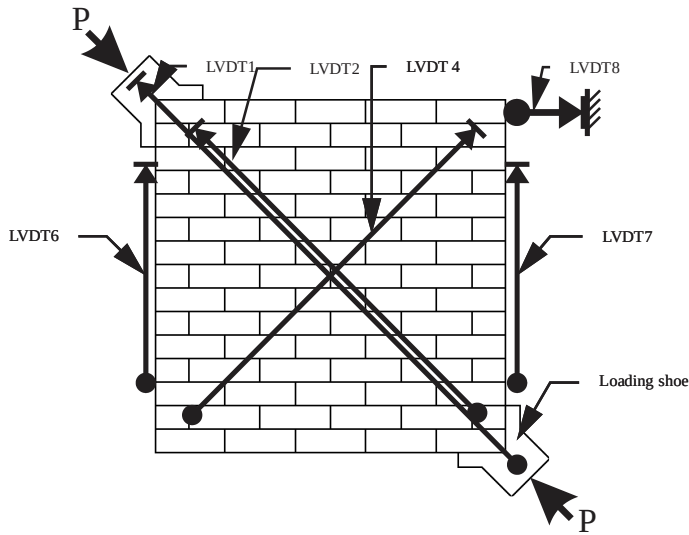


Figure C.1: Geometry of masonry panels

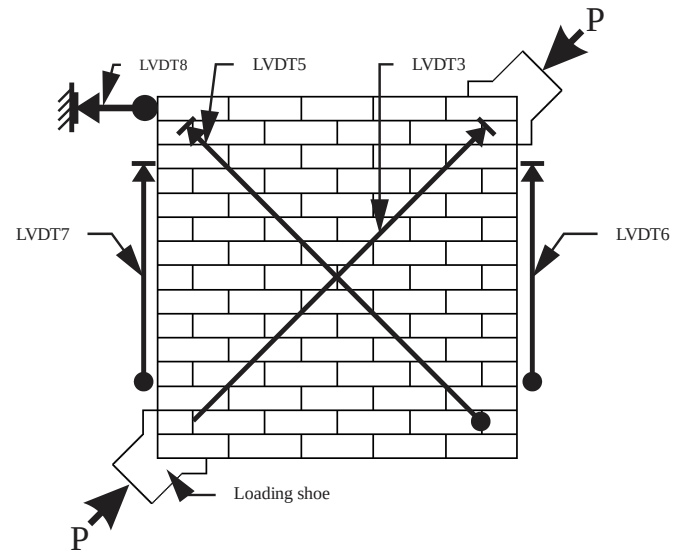
(Note that all of thickness of mortar joint is 10mm and layer of joint applied on the base)

The test imposed force control and the key interested parameter is diagonal tensile strength f_t and shear modulus of masonry G . The test arrangement adopted from ASTM E519-02 and implemented as shown in figure C.2. The test set up consists of two loading shoes diagonal corners of the panel which connected with steel rod. The compressive loading applied diagonally between the two of loading shoes along the steel rod. In order to capture ultimate load, loading rate addressed as 10KN/min. The test conducted until crack opening is visually observed so at least it has 10mm of diagonal deflection for all tests.

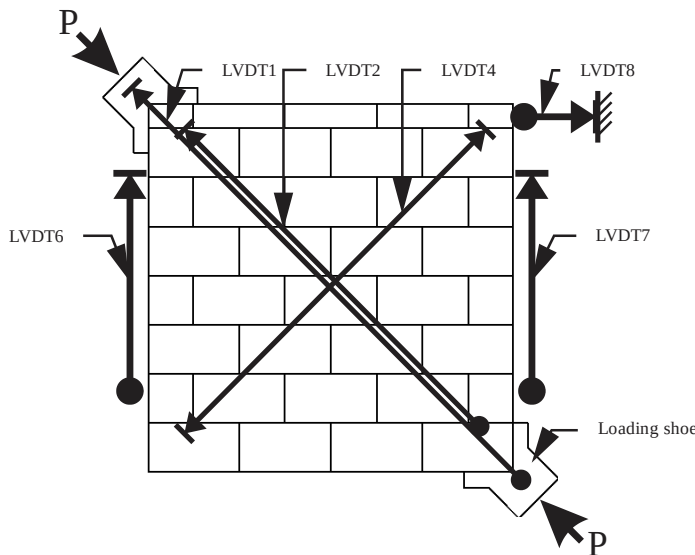
The instrumentation of LVDT illustrated in figure C.2. The LVDT 1 mounted in loading shoes to monitored diagonal displacement along the loading shoes. The LVDT 2,3 mounted on masonry panel to record the diagonal compressive displacement of the wall and plotted by force displacement curve to check twisting. The LVDT 4,5 mounted on masonry wall to monitored diagonal tensile displacement of wall and plotted by force displacement response to see twisting. The LVDT 6,7 mounted for reference purpose and LVDT 8 mounted to monitor horizontal displacement of wall. Dental paste layer applied between loading shoes and the wall to applied uniform distributed load.



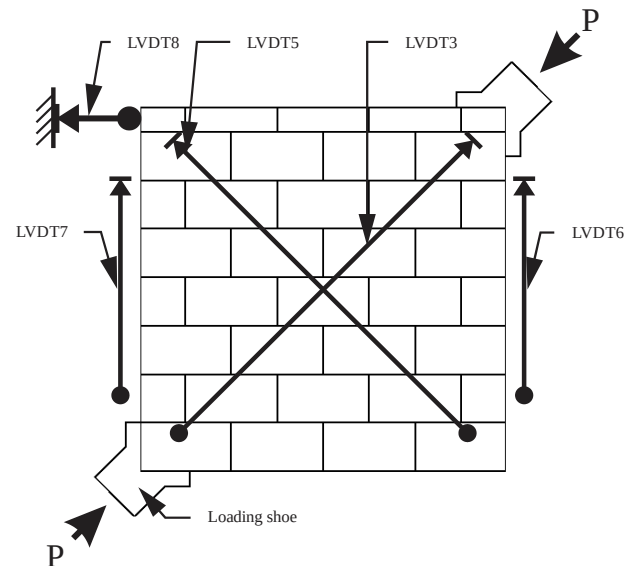
(a) Back side of clay brick wall



(b) Front side of clay brick wall



(c) Back side of sandstone wall



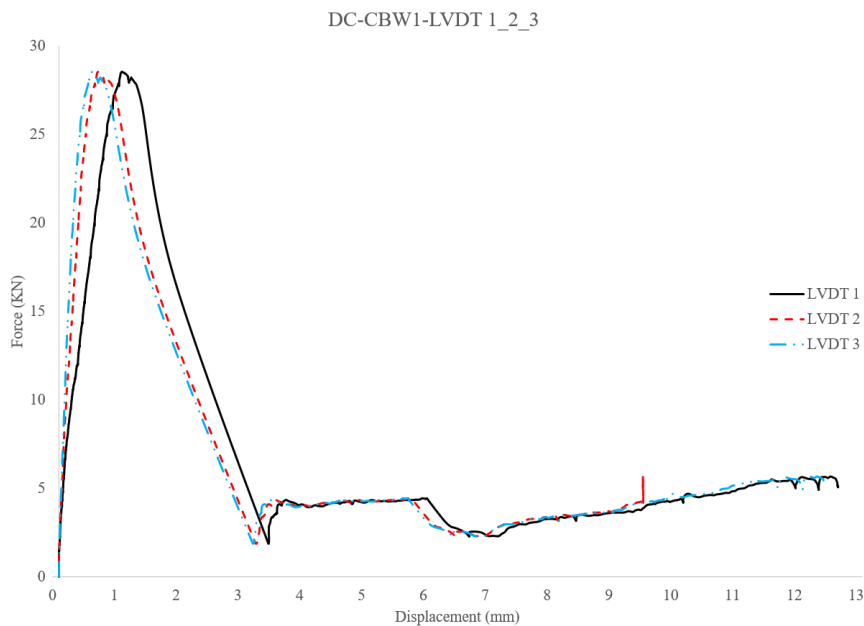
(d) Front side of sandstone wall

Figure C.2: LVDT instrumentation of diagonal compression test

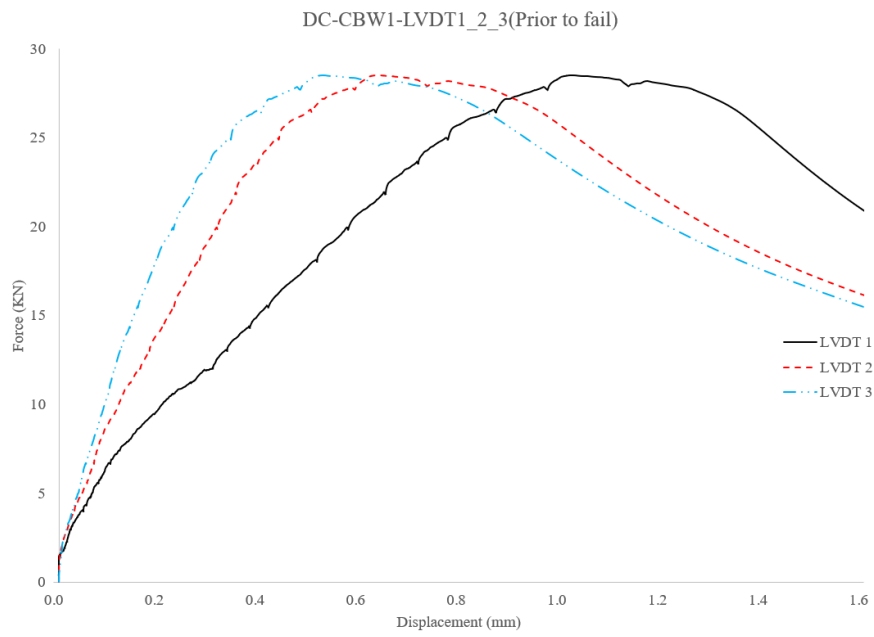
The force versus displacement curves for clay brick wall 1,2 and sandstone wall 1,2 presented in figure C.3, C.4. Figure C.3, C.4 label the test as diagonal compression test ‘DC’, clay brick wall ‘CBW’, sandstone wall ‘SSW’. The figure C.3 (a), (d) plotted force versus diagonal compressive displacement and the LVDT 1 plotted as reference LVDT. In figure C.1 (a) shown that the LVDT 2 stop reading after 9.5mm of diagonal compressive displacement. It is because of failure of LVDT bracket bond so the bond failed due to the large deflection and the LVDT is no longer attach on the wall. This is a common observation during the test. For example, LVDT 2 in figure C.3 (a), LVDT5 in figure C.3 (c), LVDT 4 in figure C.3 (f), LVDT 2 in figure C.4 (d) shown the fall of LVDTs. The LVDTs were still able to capture the post behaviour of diagonal compression until bond fail.

Comparing LVDT 2 and 3 could be a good evidence to evaluate eccentricity of applied diagonal load. Ideally if the loading applied evenly then the LVDT 2 and 3 should be identical so author plotted another graph in pre failure stage. Figure C.3 (b) shown gap between LVDT 2, 3 in early stage but still has same trend of response and the difference of LVDT 2, 3 was less than 0.05mm. Figure C.3 (e) captured the unusual response for LVDT 3 and it assumed that it is the gap between LVDT bracket and LVDT. The LVDT is not touching the bracket so the displacement recorded after close the gap. There was no prominent difference observed in figure C.4 (b), (e). Therefore, based on the good agreement of LVDT 2 and 3, the adopted diagonal compression test with loading shoe, successfully carried out by applied diagonal compressive load without eccentricity issue.

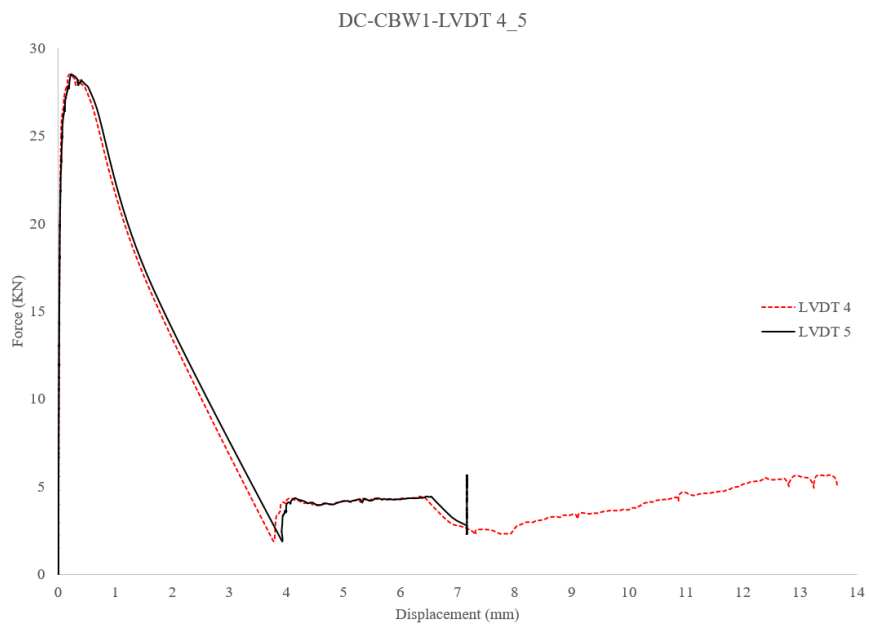
The LVDT 4, 5 were plotted to see twisting in tensile direction and all of test shown good agreement except figure C.4 (c). The unusual response in figure C.4 (c) may cause by the partial failure of the bond in LVDT 4, 5 so the LVDT bracket is still attached but became loose.



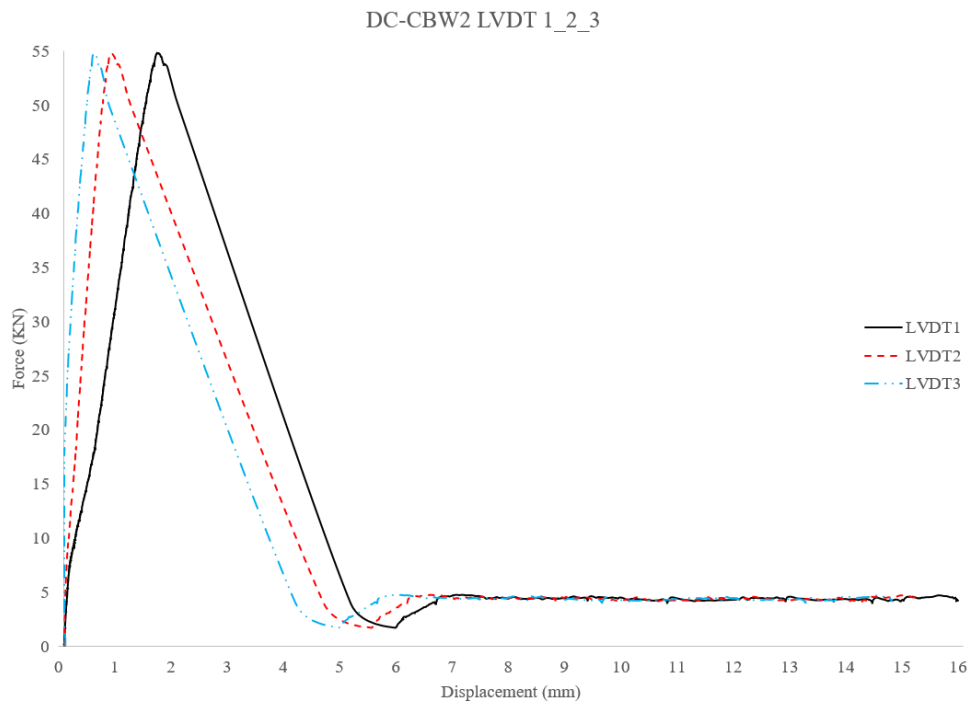
(a) DC-CBW1 LVDT 1_2_3



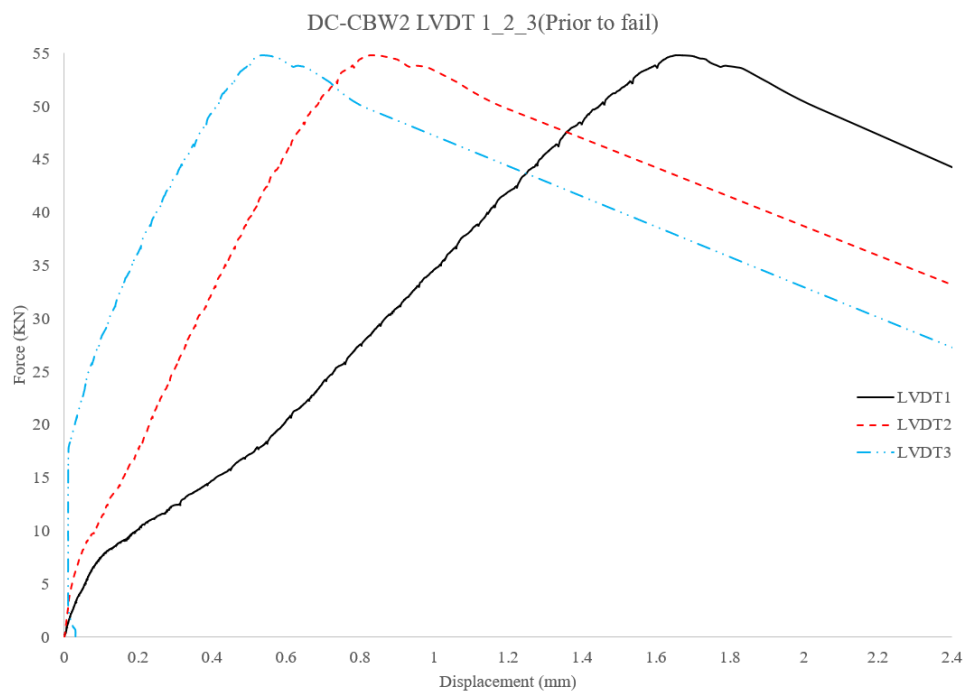
(b) DC-CBW1 LVDT 1_2_3 Prior to fail



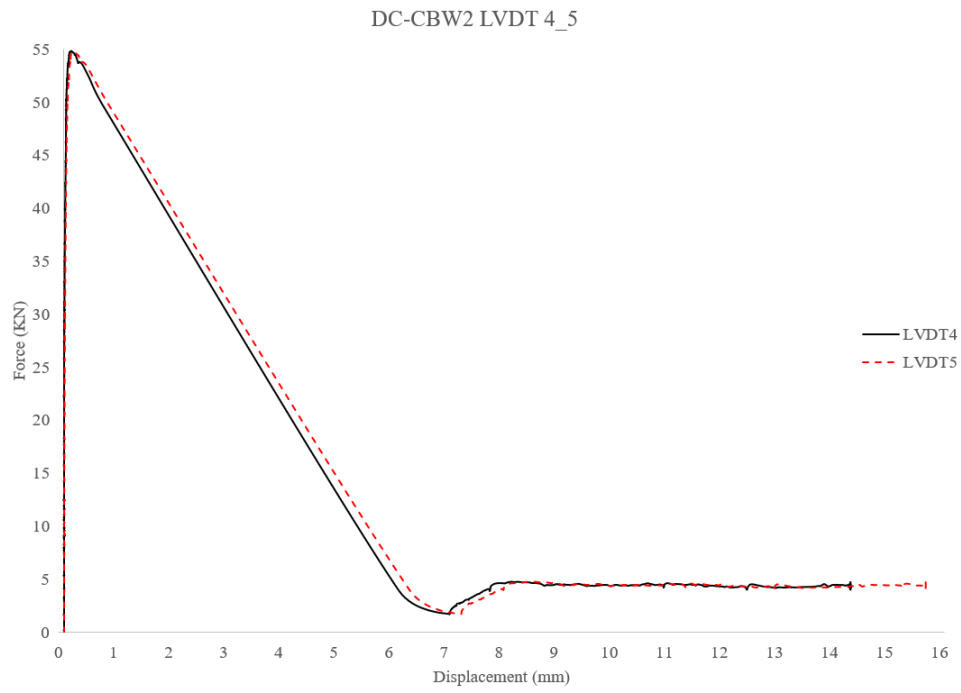
(c) DC-CBW1 LVDT 4_5



(d) DC-CBW2 LVDT 1_2_3

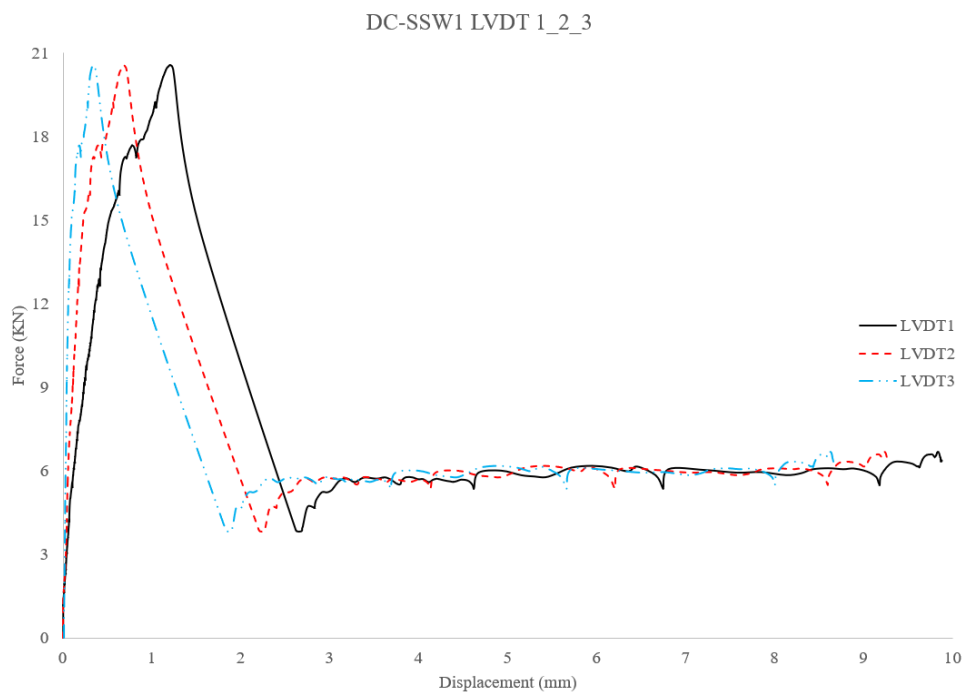


(e) DC-CBW2 LVDT 1_2_3 Prior to fail

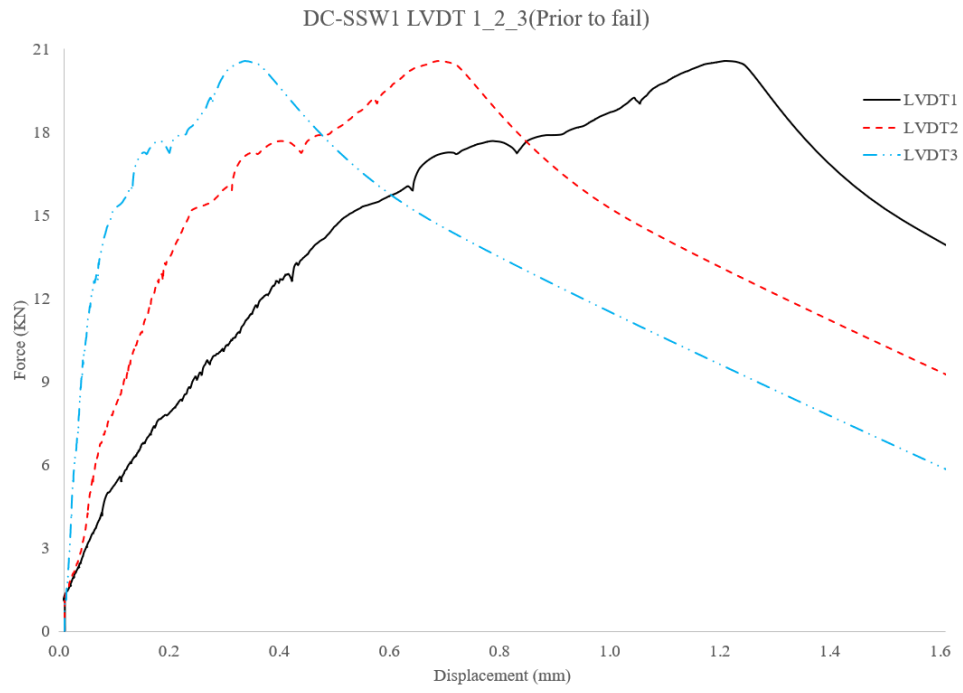


(f) DC-CBW2 LVDT 4_5

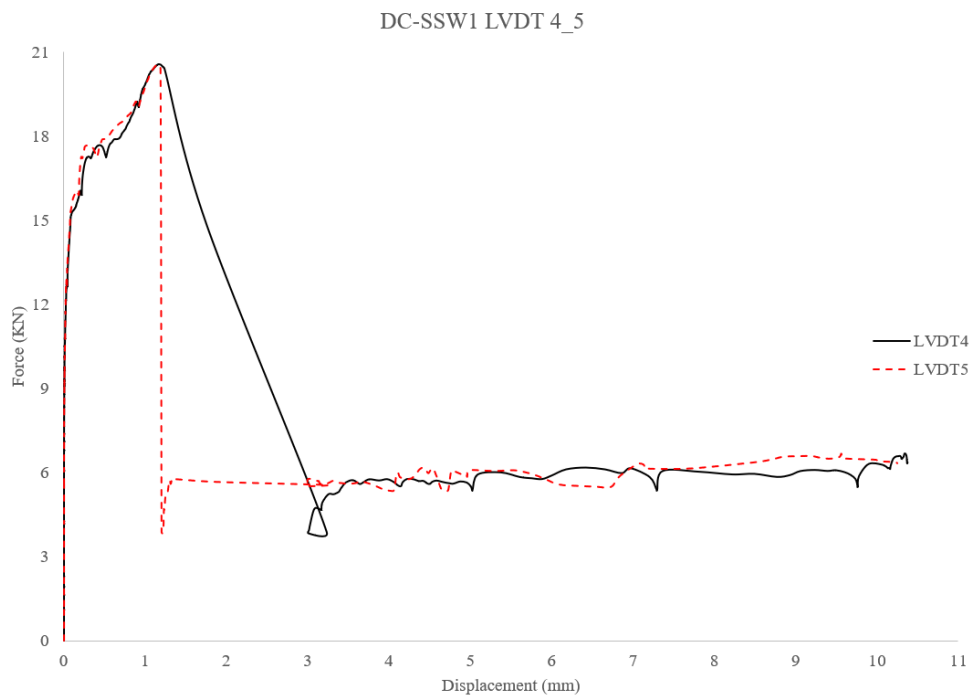
Figure C.3: F vs Δ for clay brick wall 1,2



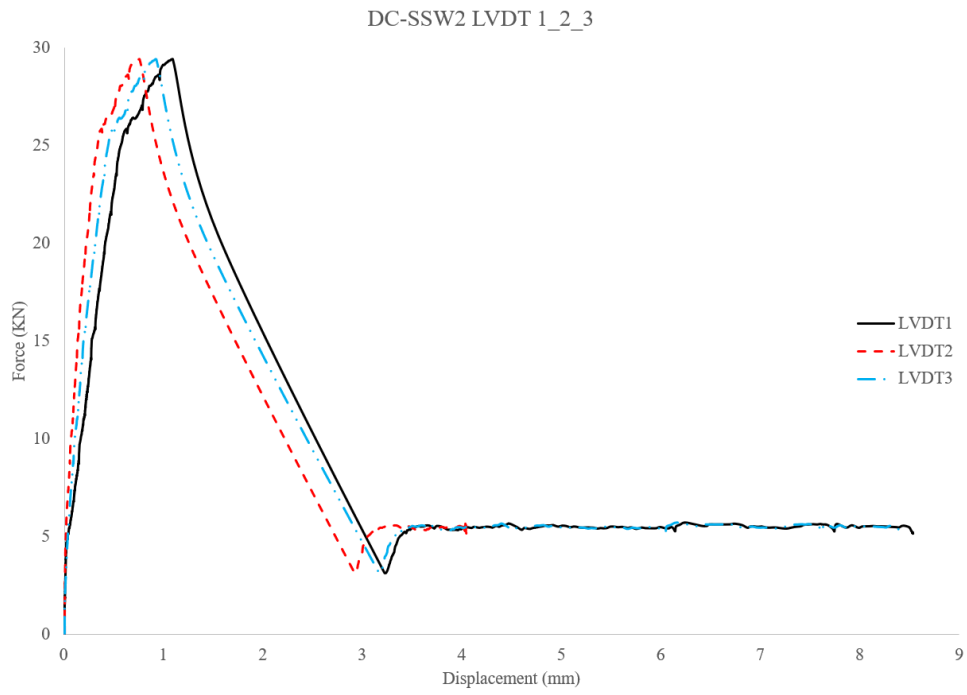
(a) DC-SSW1 LVDT 1_2_3



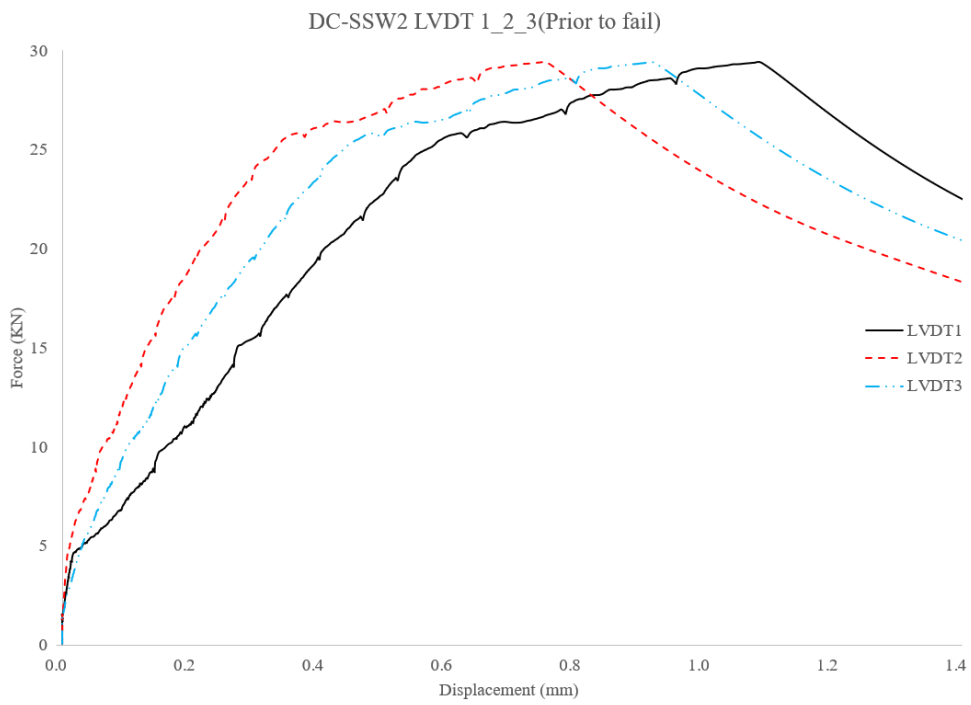
(b) DC-SSW1 LVDT 1_2_3 Prior to fail



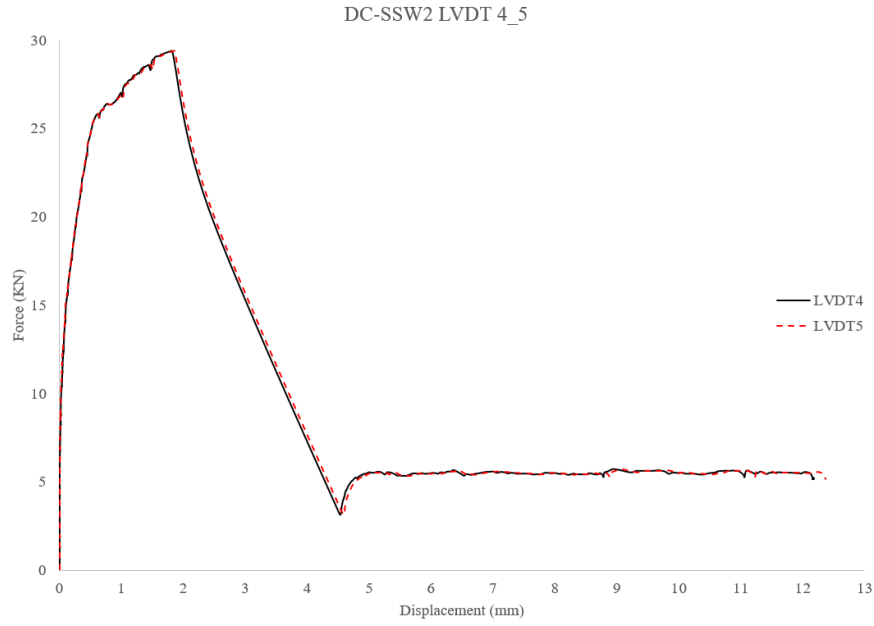
(c) DC-SSW1 LVDT 4_5



(d) DC-SSW2 LVDT 1_2_3



(e) DC-SSW2 LVDT 1_2_3 Prior to fail



(f) DC-SSW2 LVDT 4_5

Figure C.4: F vs Δ for sandstone walls

The diagonal tensile strength of masonry, f_t determined ASTM, RILEM standard and table C.1 represented result. It is noted that during the construction of CBW 1, oil pump busted by accident, so the oil went through mortar joint. The figure C.5 is shown CBW1 after diagonal compression test to see the failure surface of mortar joint. Figure C.5 (b) shown the failure surface of joint and visually observed the oil content and figure C.5 (c), (d), (e) and (f) shown the oil content which absorbed through mortar joint but only small area indicated oil content. The inconsistency of ultimate load for clay brick wall may be due to characteristic of masonry. The f_t (MPa) of CBW 1,2 were 0.15, 0.3 based on ASTM standard and determined as 0.11, 0.21 based on RILEM standard. As explained, the ultimate load for CBW1 was about 50% of CBW2. Similarly, f_t (MPa) of SSW 1,2 were calculated based on ASTM, RILEM standard and represented in Table C.1.

The shear modulus of G determined based on ASTM standard as written in Eq: C4 and C5. Where γ , shearing strain, ΔH horizontal extension, ΔV vertical shortening and g_v gauge length along the loading shoe. The ΔH , ΔV and g_v taken mean of corresponding LVDT and the result presented in table C.2 the G for sandstone walls have shown good consistency but the stiffnesses are much lower than clay brick wall. Especially, the horizontal extension of sandstone panel is much greater than clay brick wall. A possible explanation for this may be due to the characteristic of sandstone. The sandstone does not absorb as much moisture from the mortar and it could be related to the bond of the joint.



(a)



(b)



(c)



(d)



(e)



(f)

Figure C.5: CBW1 after diagonal compression test

Diagonal tensile strength of masonry, f_t (MPa)						
Specimen	L(mm)	H(mm)	T(mm)	P_{max} (KN)	f_{t_ASTM} (MPa)	f_{t_RILEM} (MPa)
DC-CBW1	1190	1200	110	28.52	0.15	0.11
DC-CBW2	1190	1200	110	54.84	0.30	0.21
DC-SSW1	1195	1210	110	20.60	0.11	0.078
DC-SSW2	1200	1230	110	29.41	0.16	0.11

Table C.1: Diagonal tensile strength of masonry f_t based on ASTM and RILEM standard

$$\gamma = \frac{(\Delta H + \Delta V)}{g_v} \quad (\text{Eq: C.4})$$

$$G = \frac{f_{t_ASTM}}{\gamma} \quad (\text{Eq: C.5})$$

Shear modulus of masonry, G (MPa)						
Masonry panel	LVDT 4&5	LVDT 2&3	LVDT 2&3	γ (%)	f_{t_ASTM} (MPa)	G (MPa)
	ΔH (mm)	ΔV (mm)	g_v (mm)			
DC-CBW1	0.21	0.58	1360	0.058	0.15	264
DC-CBW2	0.14	0.67	1360	0.059	0.29	500
DC-SSW1	1.23	0.50	1340	0.13	0.11	85
DC-SSW2	1.84	0.84	1340	0.20	0.16	78

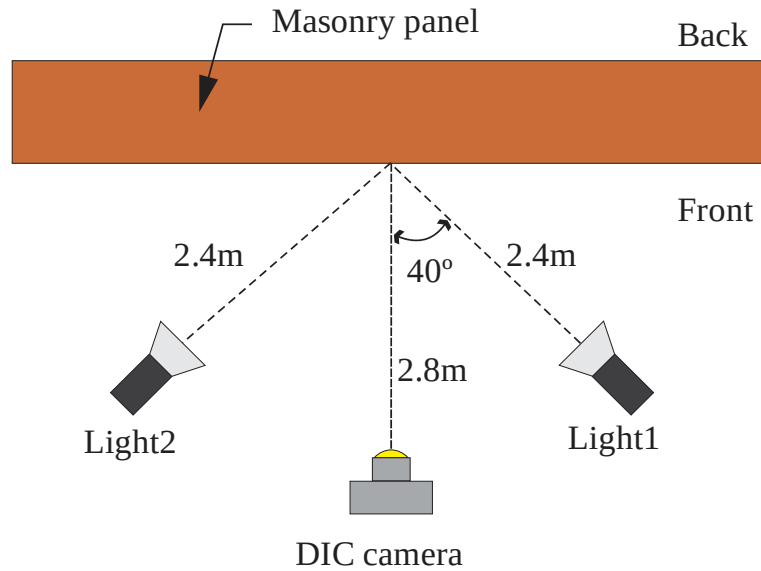
Table C.2: Shear modulus of masonry G , determined by ASTM standard

C.2 DIC application with DSLR camera

The 2D DIC employed to capture crack propagation of entire wall surface. Prior to perform visual representation for each panel, the virtual gauges were compared with LVDT to validate the DIC analysis. The DIC camera positioned only one side so LVDT 3, 5 and 8 were used to verified result. Sony DSLR camera, namely DSC-RX10M3 with Zeiss vario sonnar T* 24-600mm F2.4-4 lens was used to capture image. Two of photography lights were used with 500mm by 700mm soft box studio to cover whole surface of masonry panel. In order to control light saturation, the distance between light and the panel and angle of light stand carefully address. As shown in figure C.6 (a), the object distance between camera was 2.8m and light distance is 2.4 m with 40° angle. To obtained fine level of detailed, the DSLR camera set up followed with aperture set as F/10, shutter speed 1/125 second and ISO-1600. The camera positioned perpendicular to

centre of masonry panel and also adjusted tripod to face centre point of the panel. The images recorded as ARW which raw file format of Sony camera with 20MP and 1 frame per second was used during the test.

The application of speckle pattern was critical factor in DIC analysis. The current experiment campaign selected paint spray with bold out nozzle to have increased dot size. The variation of size of dot was 3-5mm. The masonry panels coated as white background with black speckle to emphasised contrast and cover the speckle evenly for whole surface. The test arrangement conducted as shown in figure C.6 (b).



(a) Schematic view



(b) Photographic view

Figure C.6: DIC camera set up

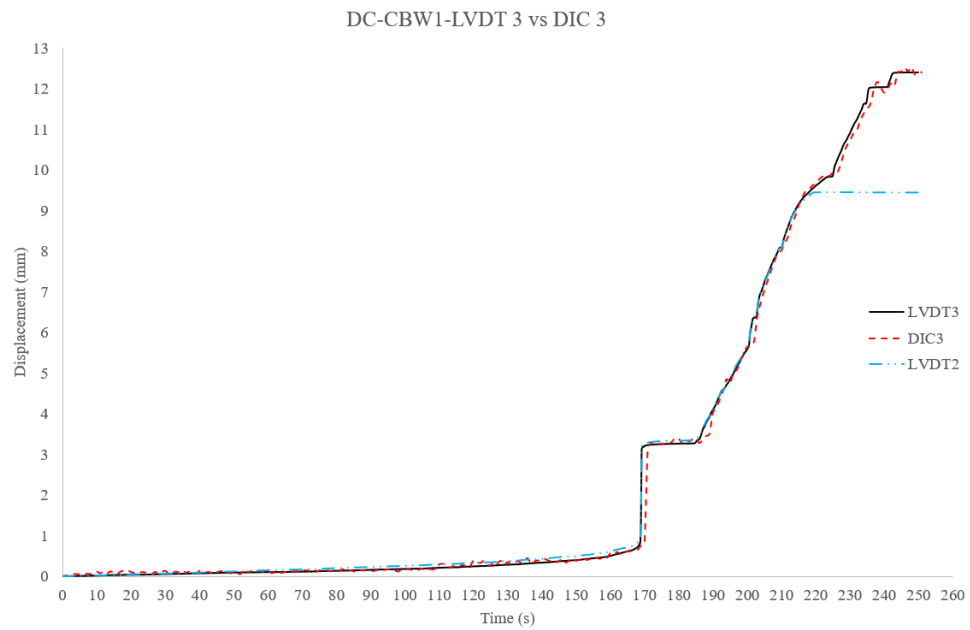
Each test took reference image with calibration plate prior to do test as described in figure B.4. The capture image imported in adobe camera raw software to correct distortion effect and converted TIFF file format. The converted images were processed for DIC analysis in GOM correlate software. The GOM correlate software has a tool to evaluate the speckle pattern quality so the tool used to determine optimized size of subset and point distance. The typical optimized size of subset and point distance determined as 15 pixel, 9 pixels.

Virtual LVDT gauges which namely DIC3, DIC5 and DIC8 were used for validation purpose. Figure C.7 shows the example of virtual gauges under principal strain map. The virtual gages of DIC3, 5 parallel to LVDT 3, 5 and DIC 8 defined as point on the left top corner. The LVDT includes LVDT 2, 4 and DIC result plotted as displacement versus time and represented in figure C.8, C.9. The sign of LVDT indicated the compression and extension. The plus sign indicated extension and minus sign indicated compression. In pre failure stage, virtual gauge reading affect vibration but still has good agreement with LVDT in entire loading step.

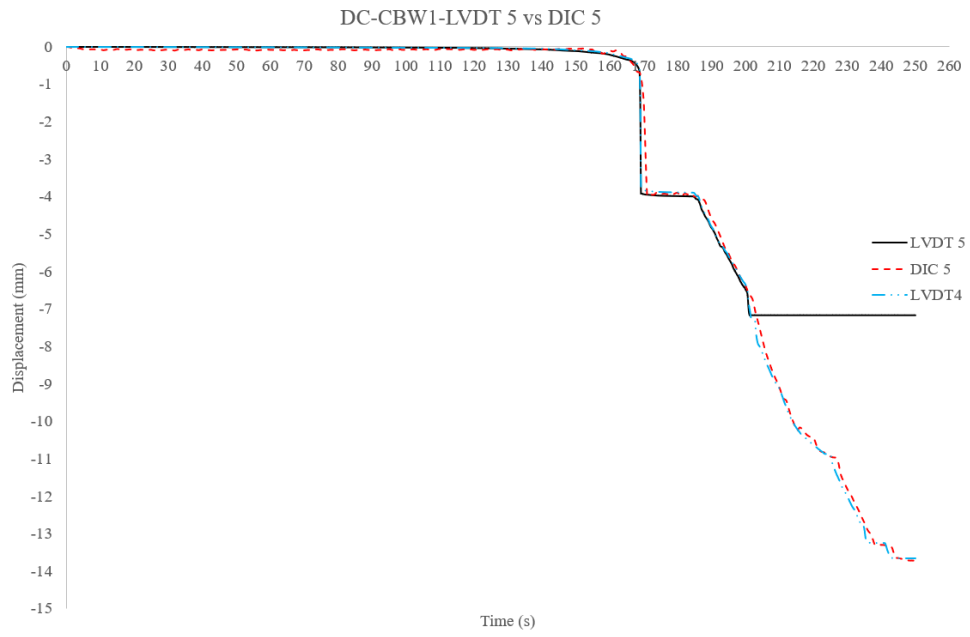
In summary, the responses in figure C.8, C.9 indicated that the virtual gauge of DIC has strong correlation with LVDT. In other words, the distortion effected fixed by calibration plate and image processing. There were some benefits to use virtual gauge compare to traditional LVDT. It was commonly observed the failure of LVDT brackets after large amount of displacement. For instance, LVDT2 in figure C.8 (a), LVDT 5 in figure C.8 (b), LVDT 9 in figure C.8 (c), LVDT 4, 5 in figure C.8 (e), LVDT 9 in figure C.9 (e), LVDT 2 in figure C.9 (c) and LVDT 8 in figure C.9 (f) shown failure of LVDT bracket bond. However, the virtual gauges of DIC still were able to track the displacement regardless of large deformation. In future, DIC can be the much more advanced measuring tool compared to traditional LVDT to obtain displacement, crack opening and crack propagation in many disciplines.



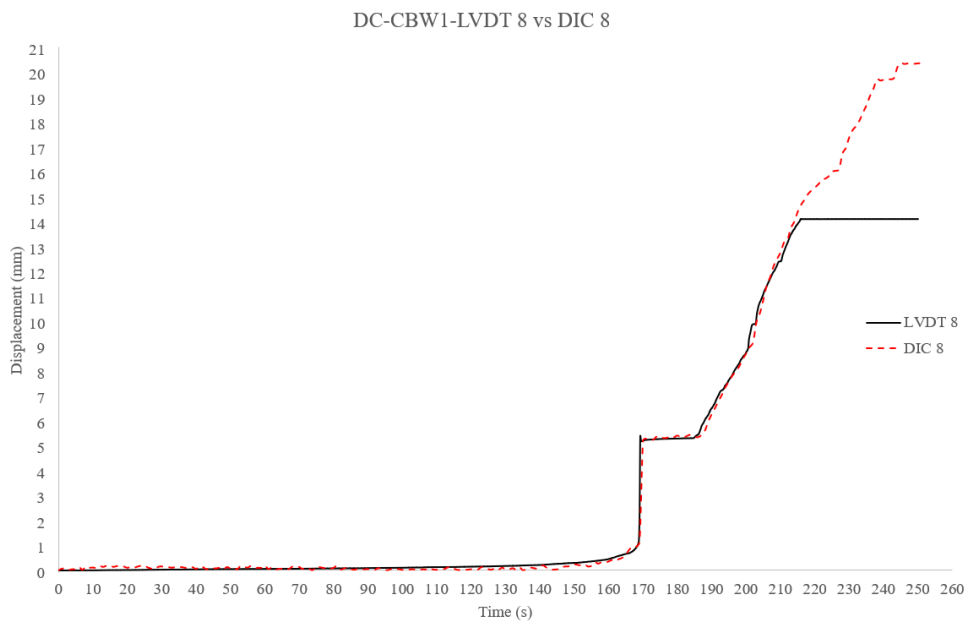
Figure C.7: Virtual gauges on masonry panels



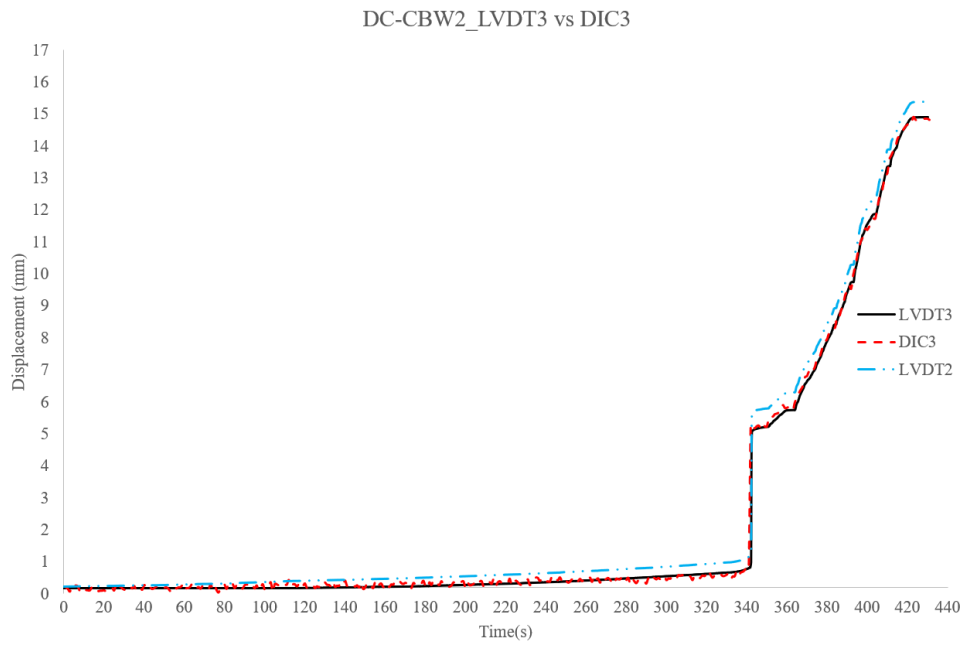
(a) DC-CBW 1 experimental LDVT 3 versus virtual LVDT 3



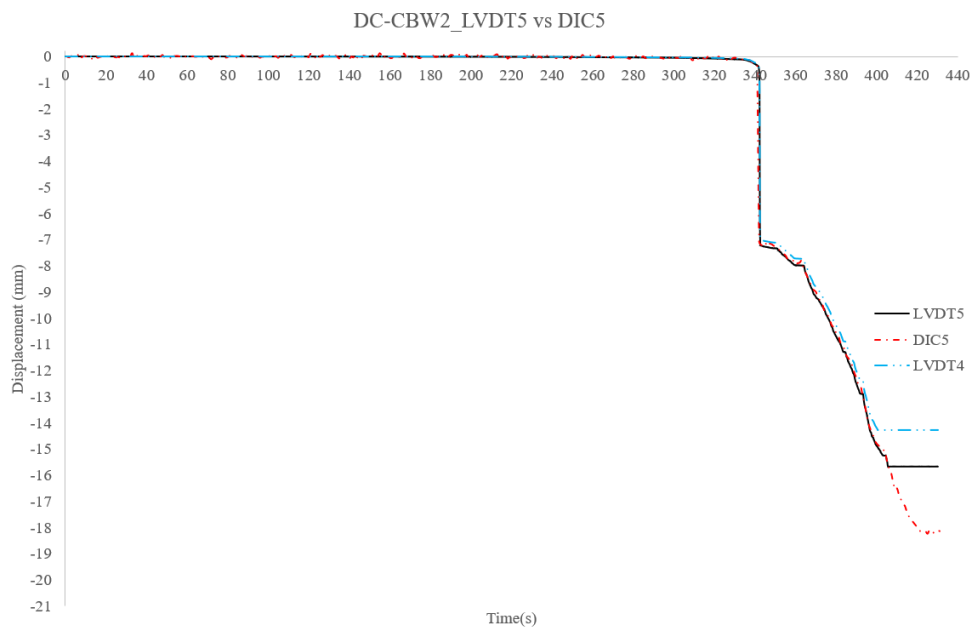
(b) DC-CBW 1 experimental LDVT 5 versus virtual LVDT 5



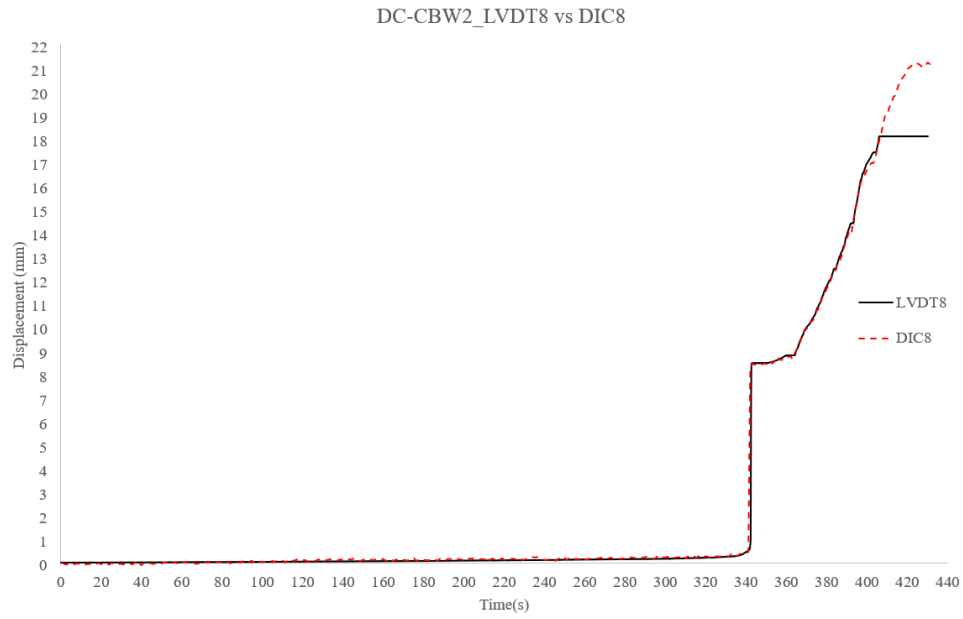
(c) DC-CBW 1 experimental LDVT 8 versus virtual LVDT 8



(d) DC-CBW 2 experimental LDVT 3 versus virtual LVDT 3

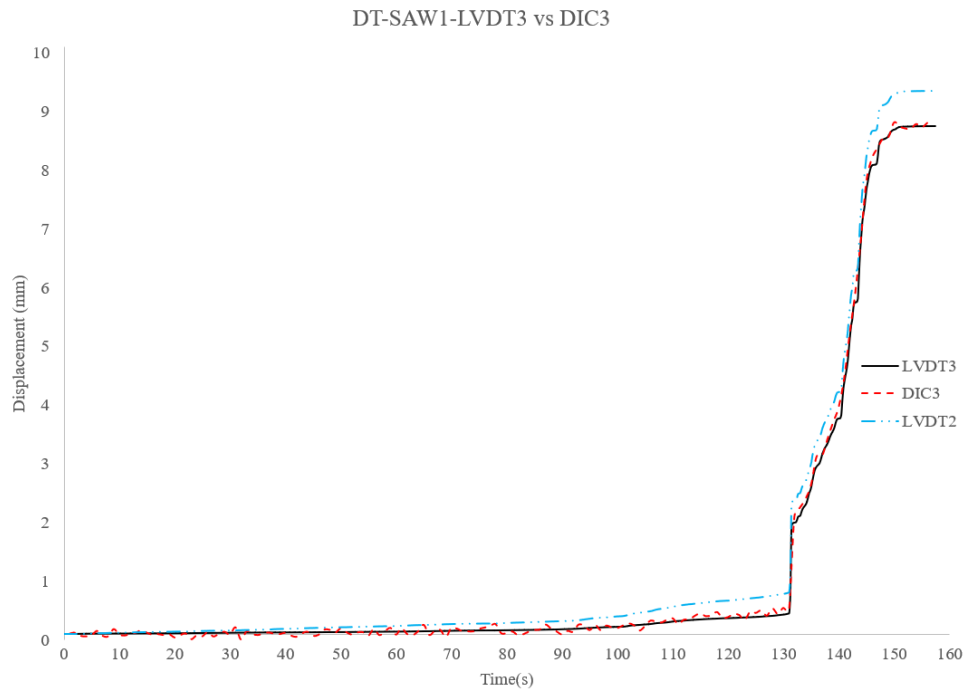


(e) DC-CBW 2 experimental LDVT 5 versus virtual LVDT 5

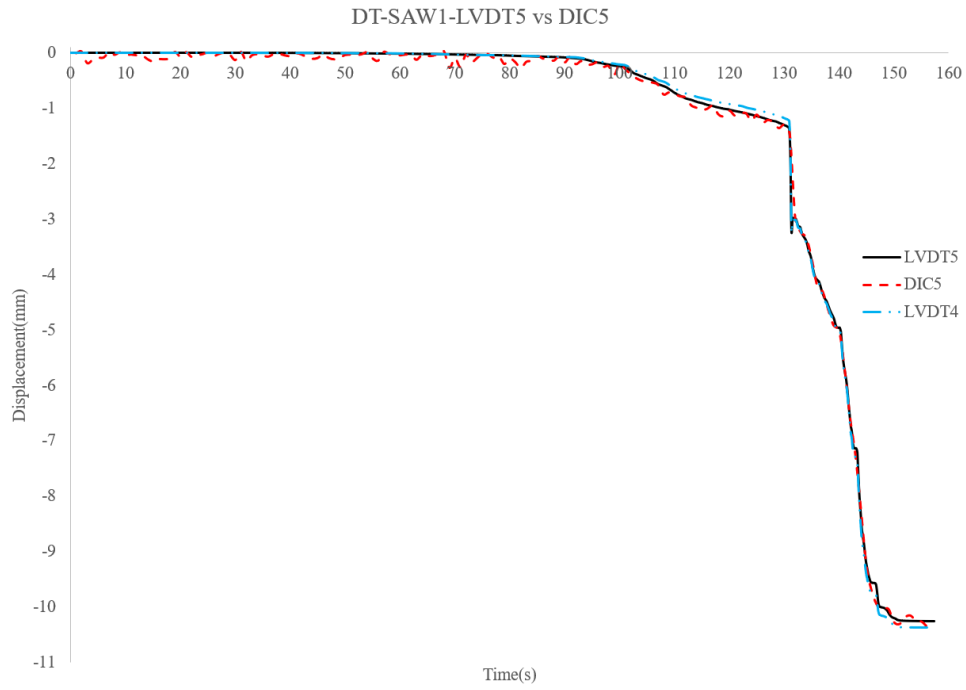


(f)DC-CBW 2 experimental LDVT 5 versus virtual LVDT 5

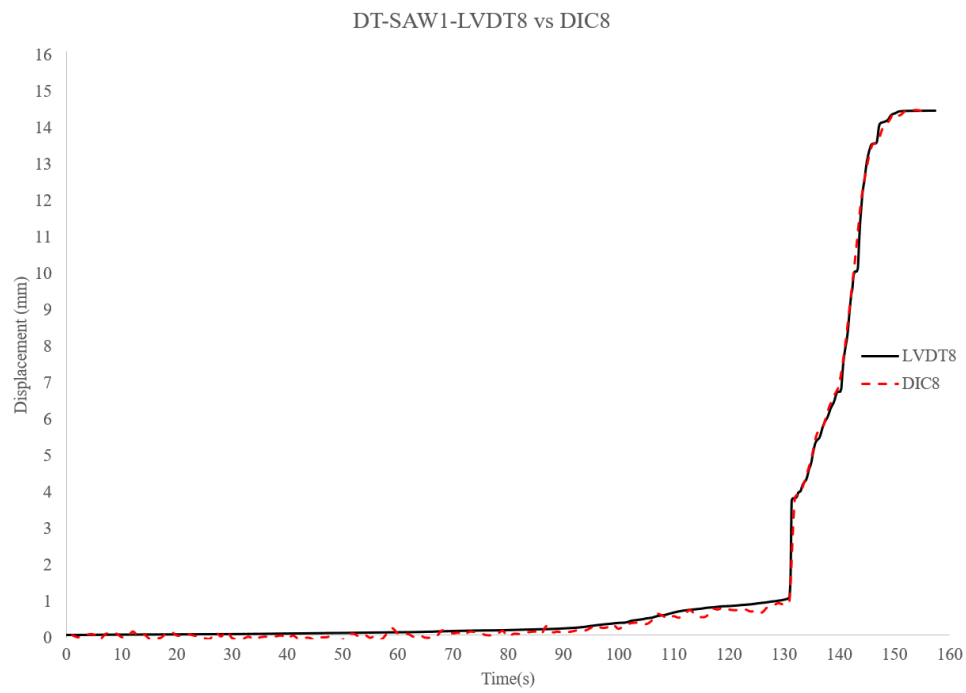
Figure C.8: DIC vs LVDT for CBW1 and 2



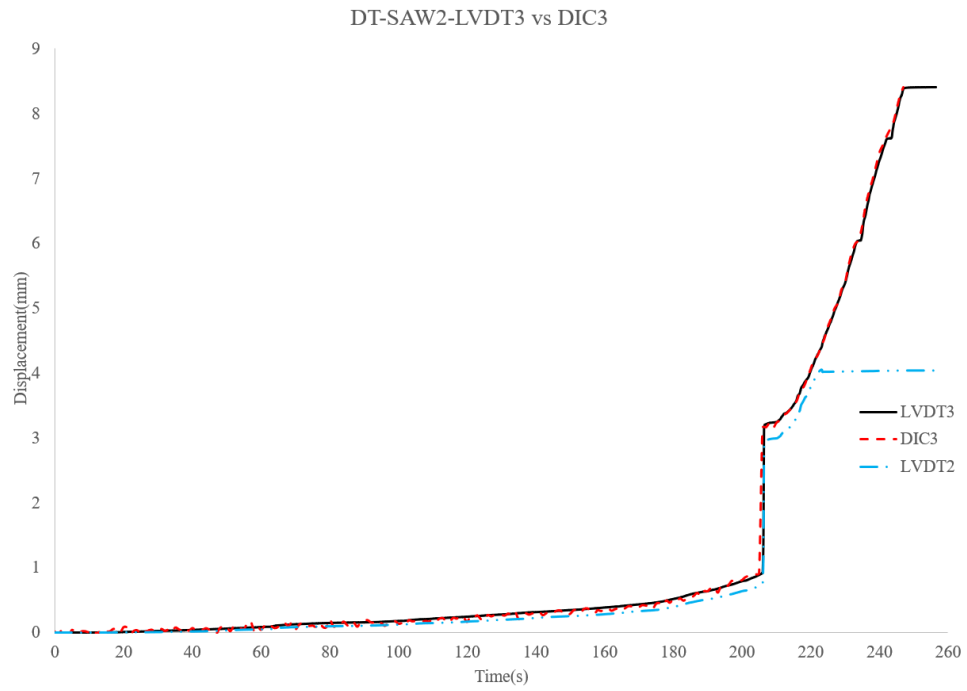
(a) DC-SSW 1 experimental LDVT 3 versus virtual LVDT 3



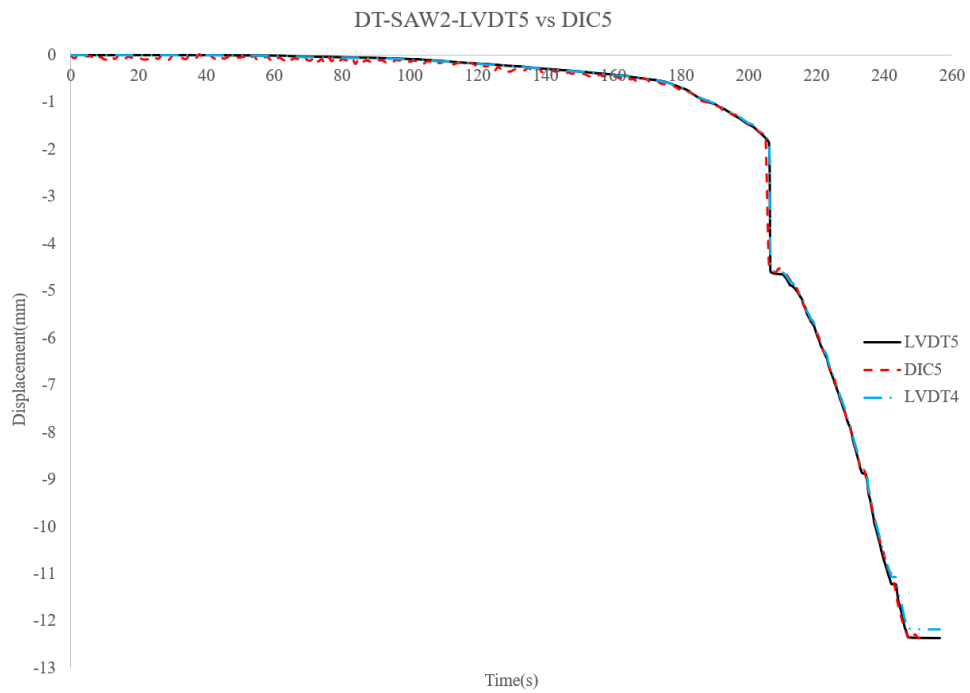
(b) DC-SSW 1 experimental LDVT 5 versus virtual LVDT 5



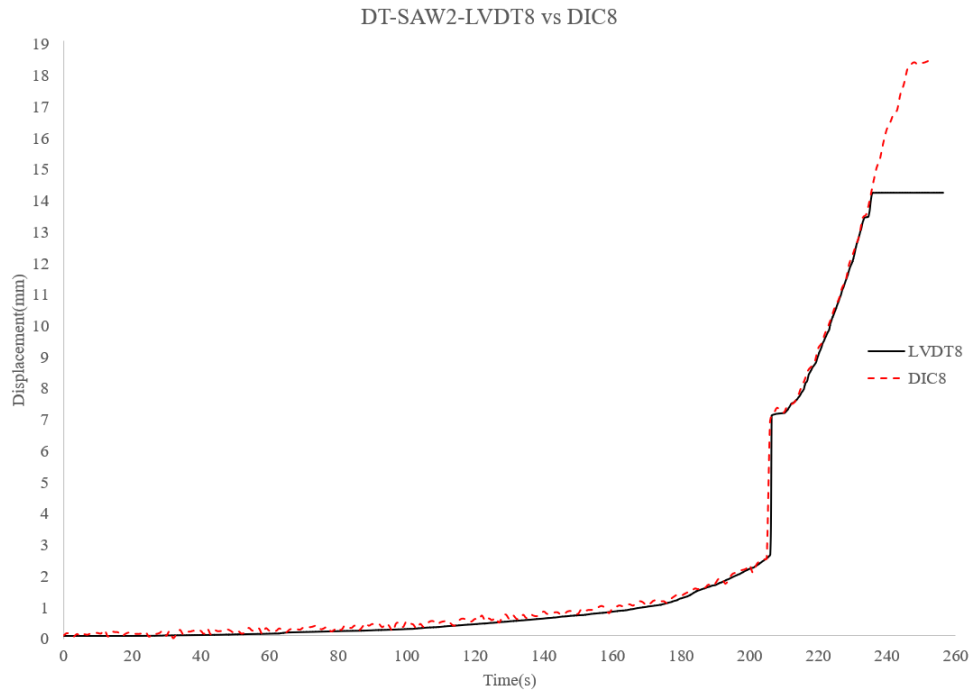
(c) DC-SSW 1 experimental LDVT 8 versus virtual LVDT 8



(d) DC-SSW 2 experimental LDVT 3 versus virtual LVDT 3



(e) DC-SSW 2 experimental LDVT 5 versus virtual LVDT 5



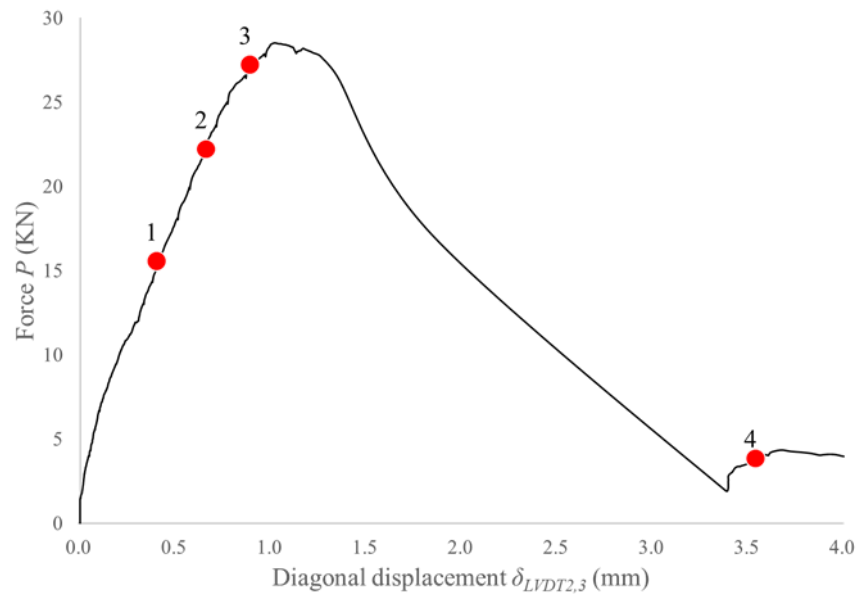
(f) DC-SSW 2 experimental LDVT 8 versus virtual LVDT 8

Figure C.9: DIC vs LVDT for SSW1 and 2

The DIC analysis verified by comparing LVDT and virtual gauge and the distortion effect fixed by calibration plate and image processing. The DIC analysis can also examined install more virtual gauge on masonry panel and current study demonstrated full-field measurement for masonry panel to observe crack opening. The crack propagation represented with principal strain map as shown in figure C. 10, C.11. The figures illustrated strain contour map derived by 4 different stages in force displacement response. The force displacement curve plotted by mean of LVDT 2,3 and the 4 stages were identified by applied load. The stage 1 is about 50% of ultimate load, stage 2 80% of ultimate load, stage 3 ultimate load and stage 4 is after failure of panel. Majority of cracks propagated along the joint interface but still some unit failure found in clay brick panel, so the crack path marked in stage 4.

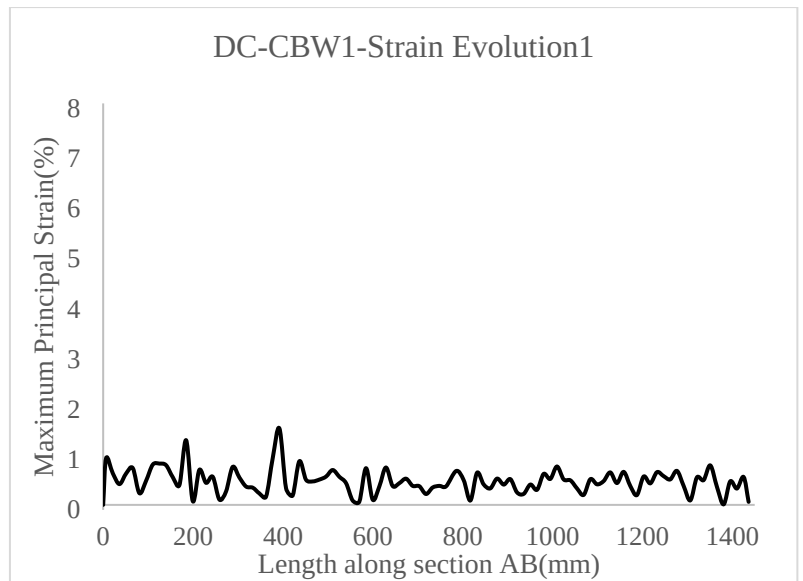
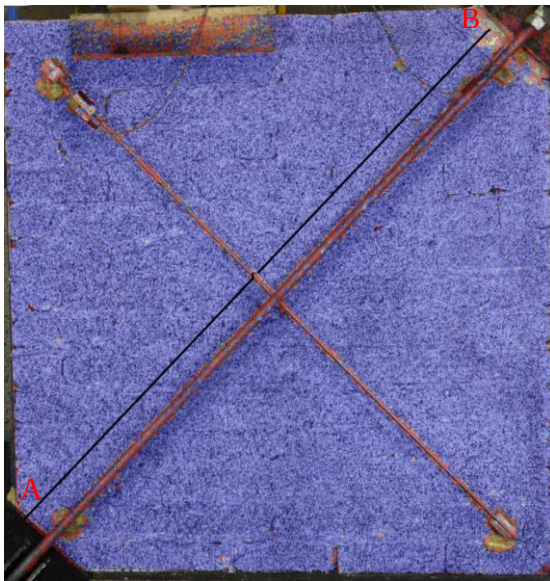
The crack growth may open and close during the loading stage so to identify crack opening status, principal strain map field has been presented, and graph is plotted along section AB. What stands out in the figure C.10 (a) stage 3 is growth of minor crack which highlighted in orange circle. The crack found in stage 3 but disappeared in stage 4 so it means that the minor crack opened but closed at ultimate load step. It is interesting feature of full-field measurement, in terms of monitoring real time of crack opening. Besides, it is key function to detect minor cracks which hard to observe by eye due to small displacement. For example, in figure C.10 (b), figure C.11 (a), (b) in stage 4 observed one discrete major crack visually but the minor crack can only be found in strain contour map. As shown in figure C.11 (a), (b) have very small displacement but

the crack growth is still observed in strain contour map. To have clear view of crack path, major crack marked as red dash line and minor crack marked as yellow dash line in figure C.10, C.11.

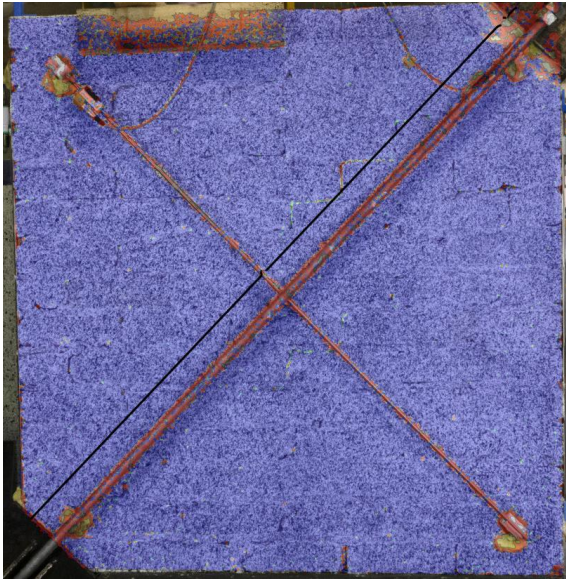


(a) DC-CBW1 strain evolution in 4 stages

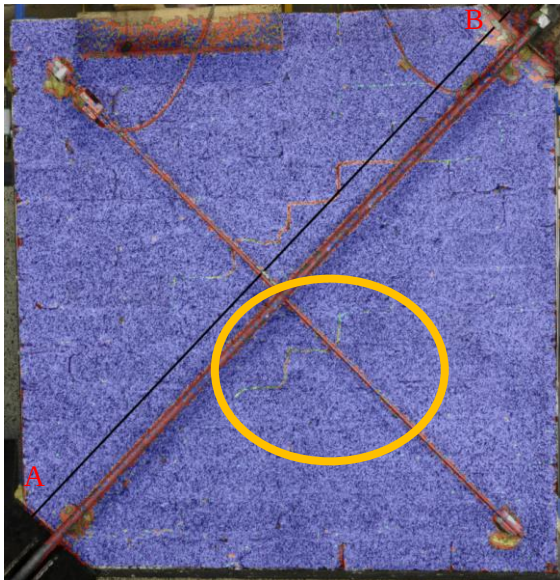
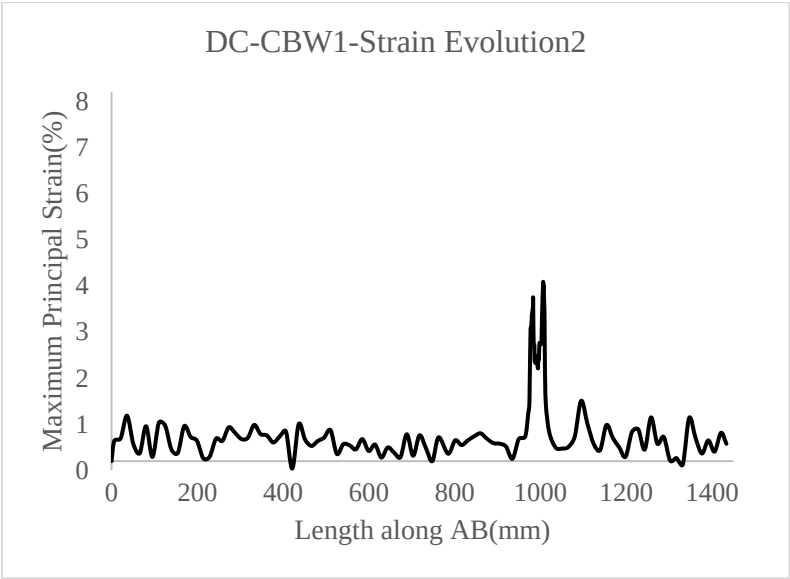
Stage 1



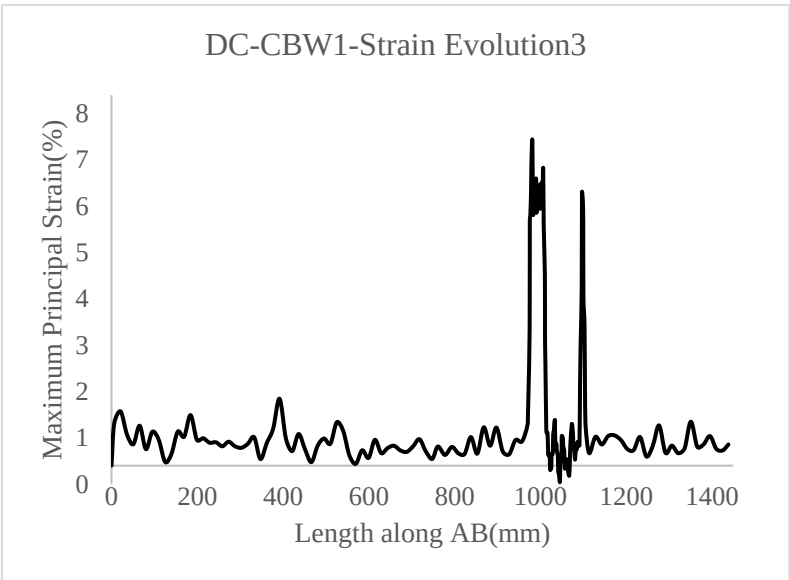
Stage 2

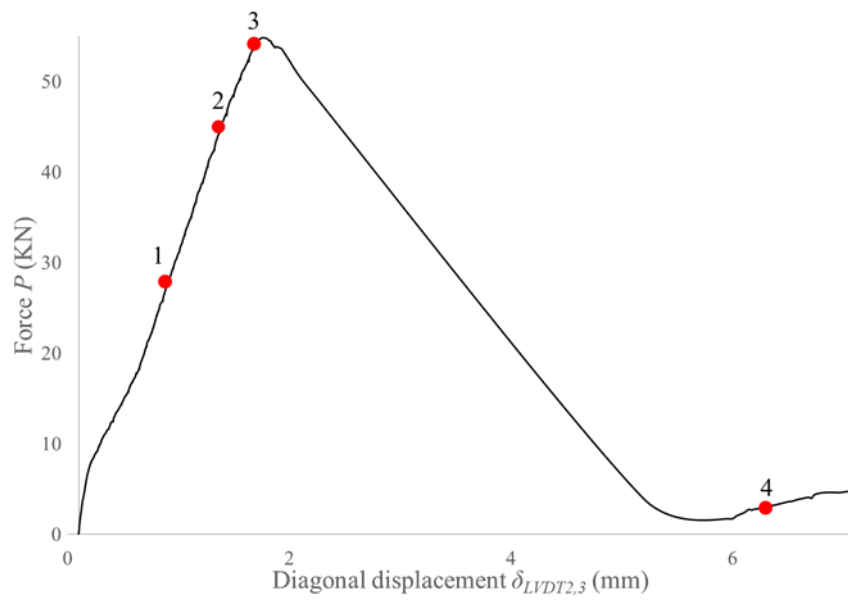
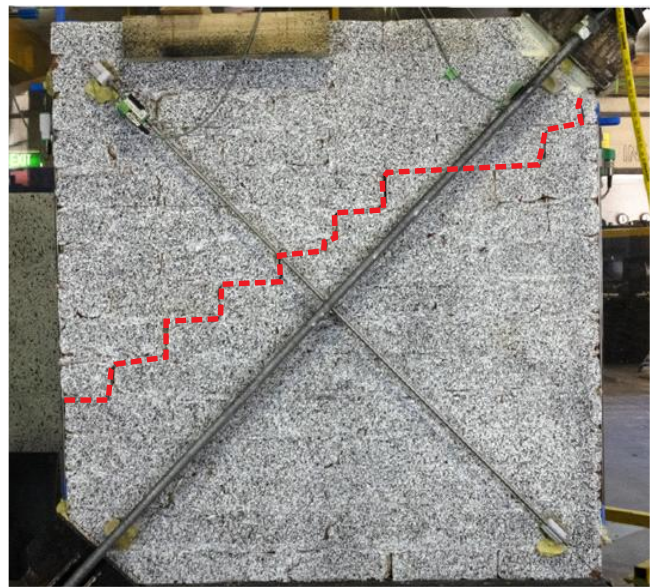
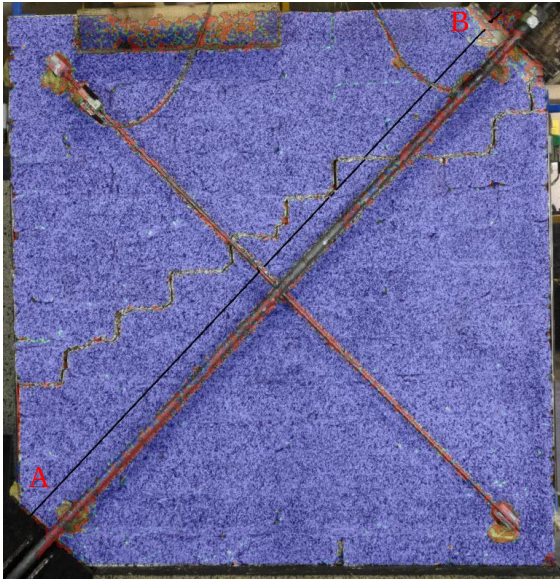


Stage 3



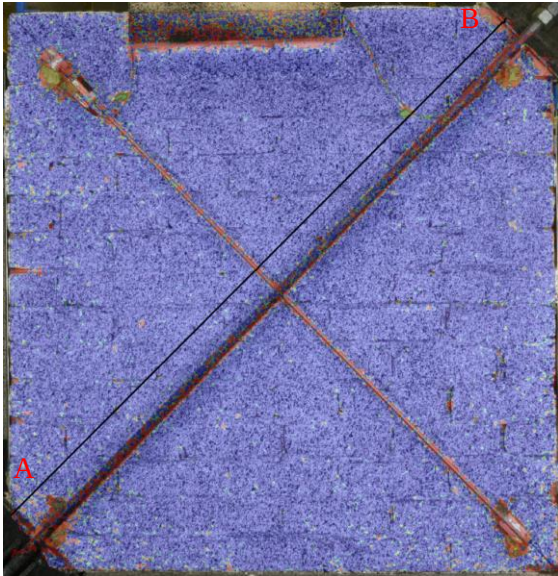
Stage 4



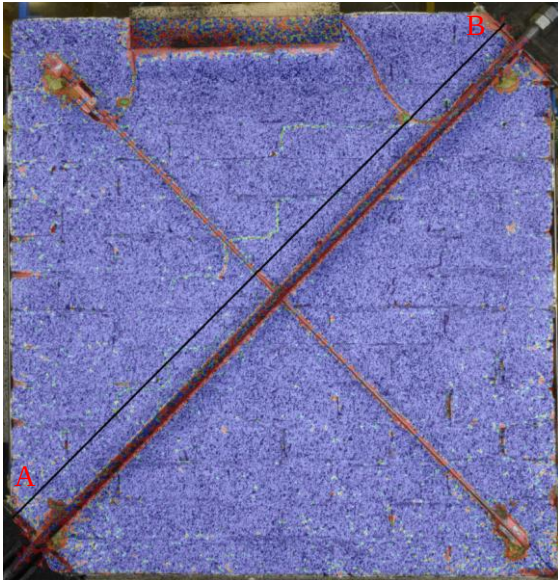
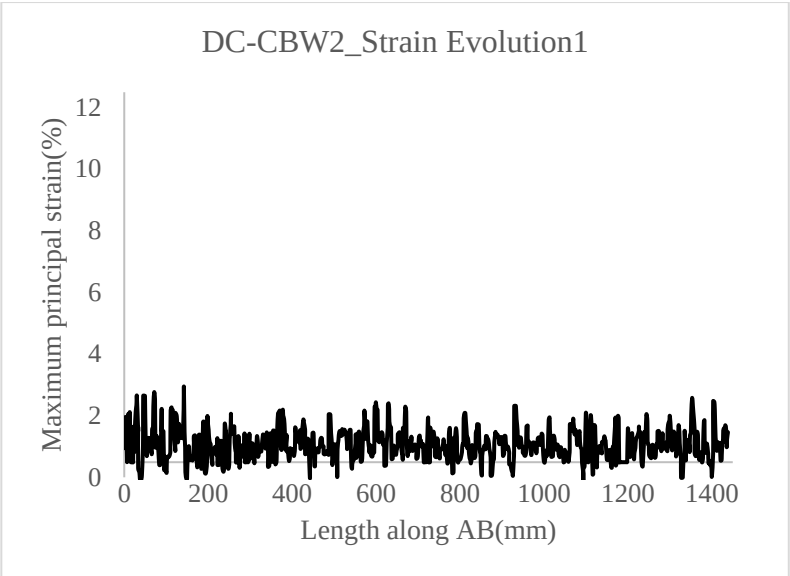


(b) DC-CBW2 strain evolution in 4 stages

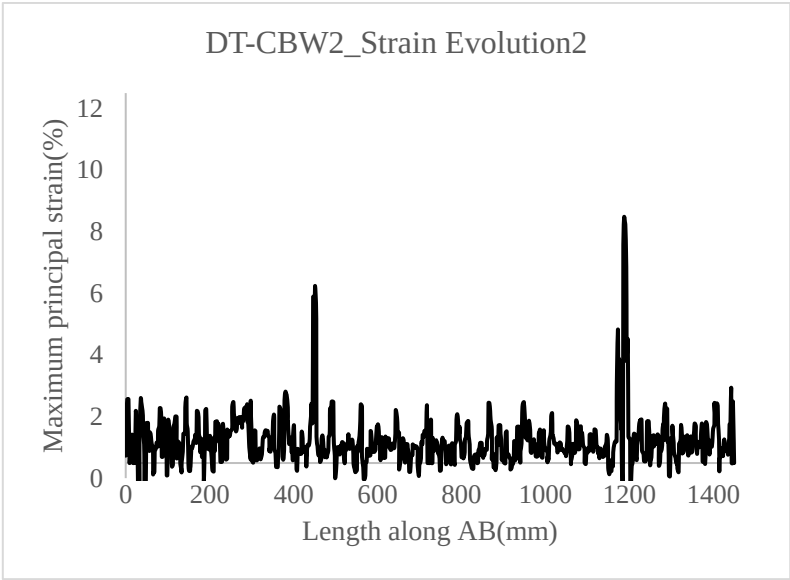
Stage 1

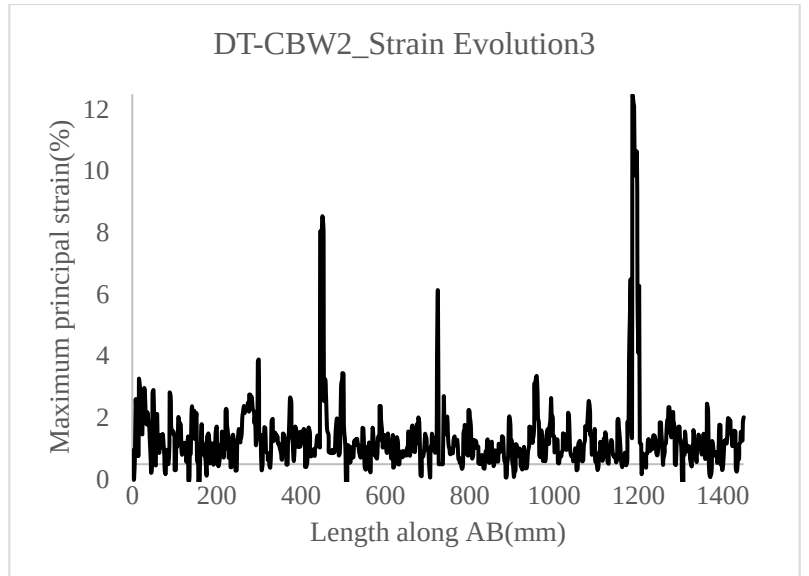
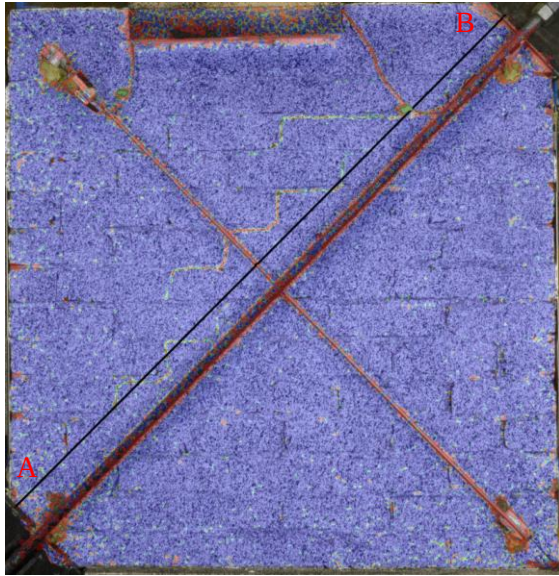


Stage 2



Stage 3





Stage 4

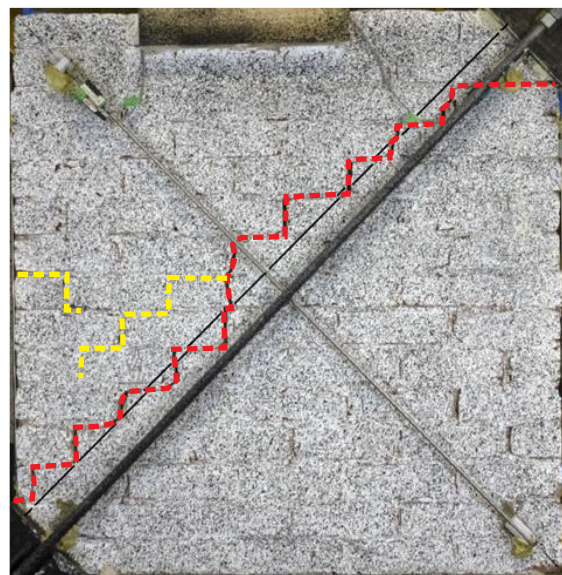
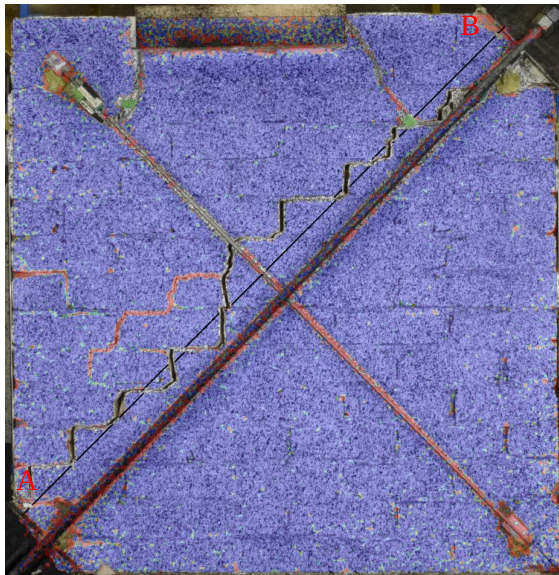
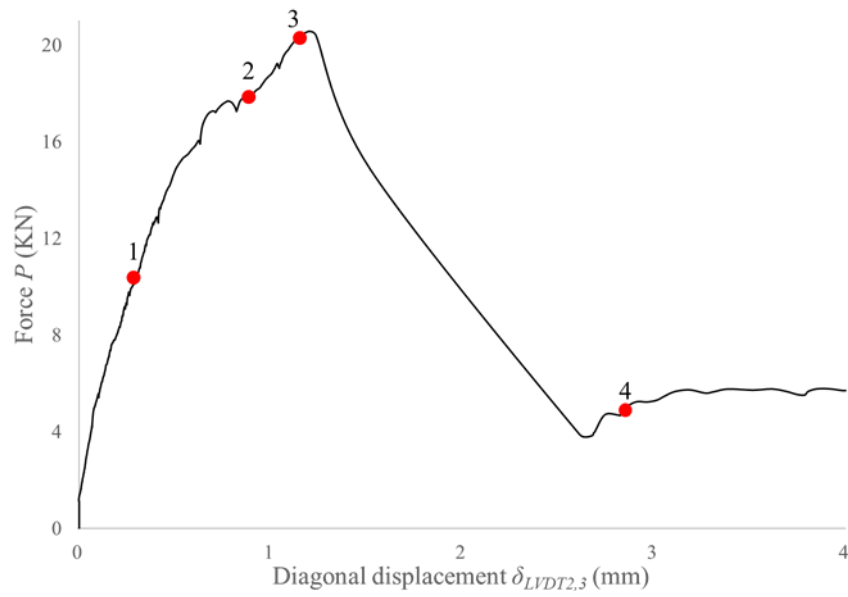


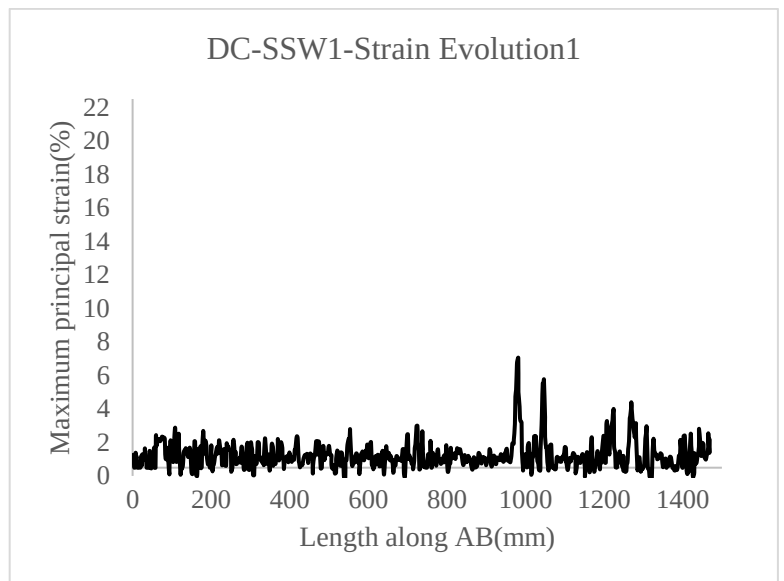
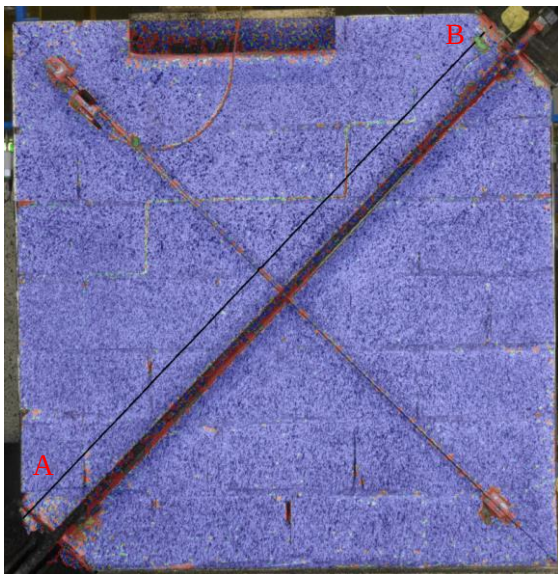
Figure C.10: Strain evolution for clay brick panels

(Note that in stage 4, the highlighted red line represents major crack and the highlighted yellow line represents minor crack)

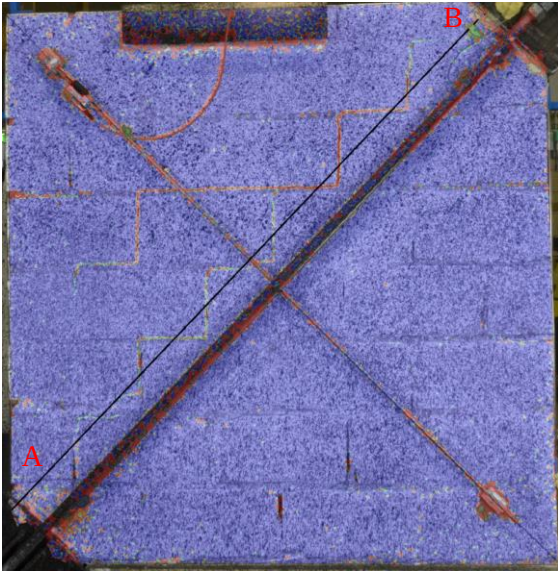


(a) DC-SSW1 strain evolution in 4 stages

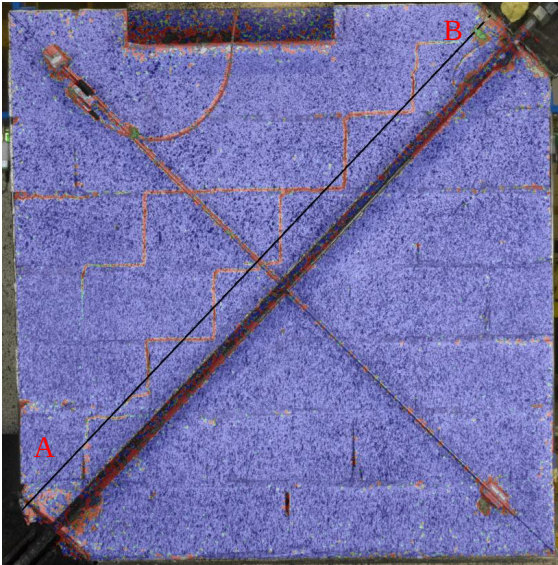
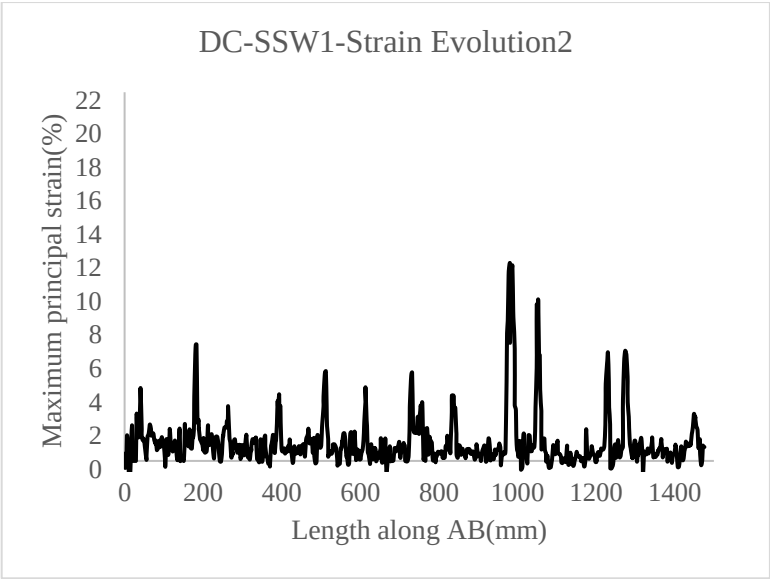
Stage 1



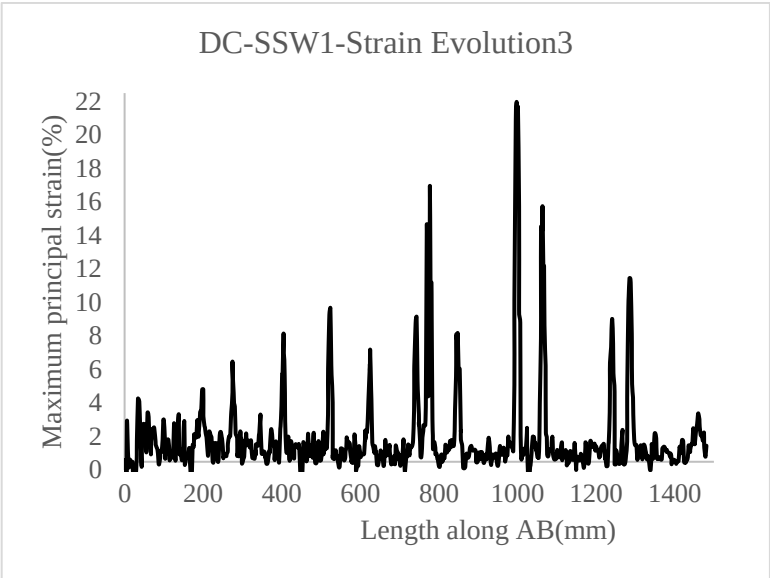
Stage 2

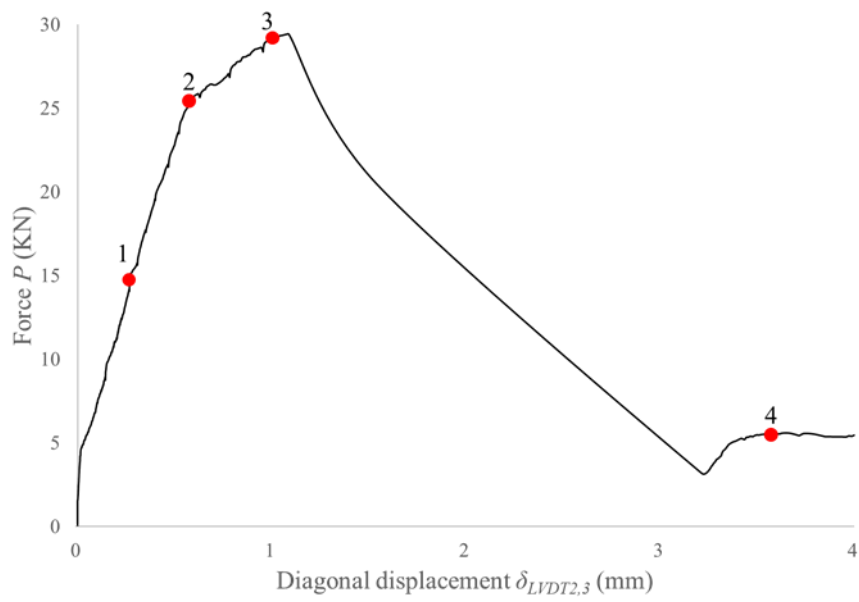
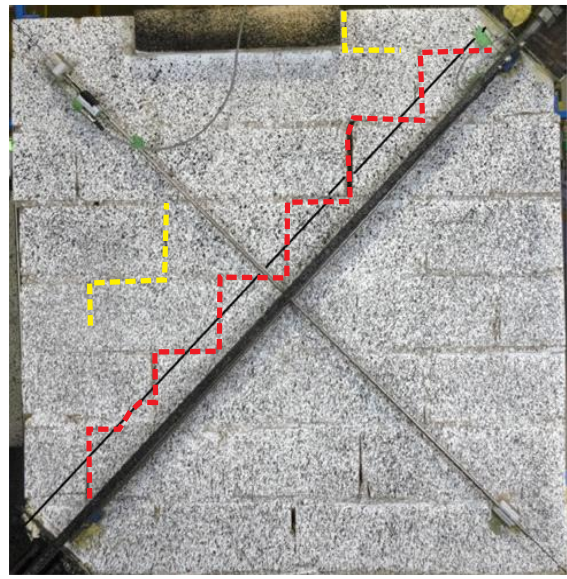
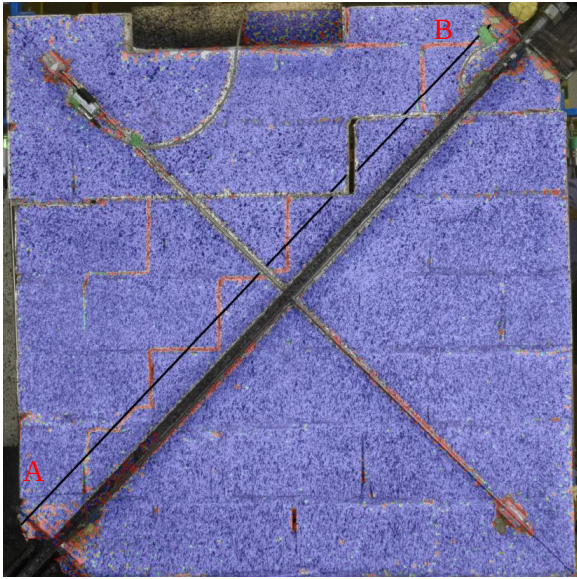


Stage 3



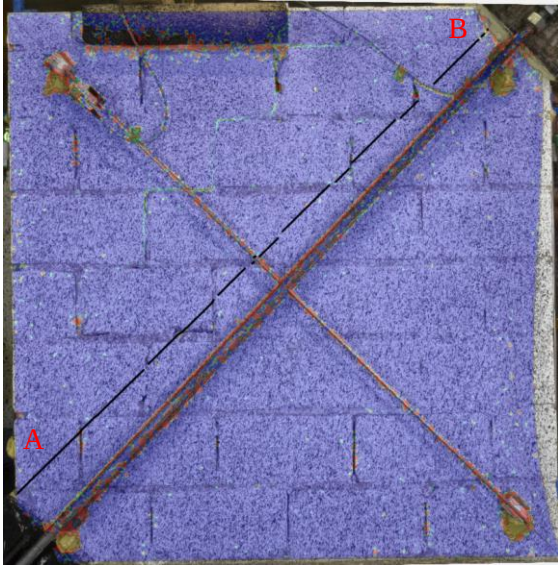
Stage 4



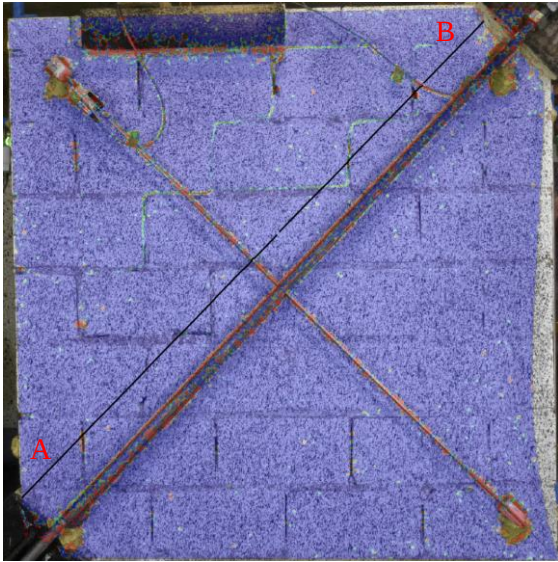


(b) DC-SSW2 strain evolution in 4 stages

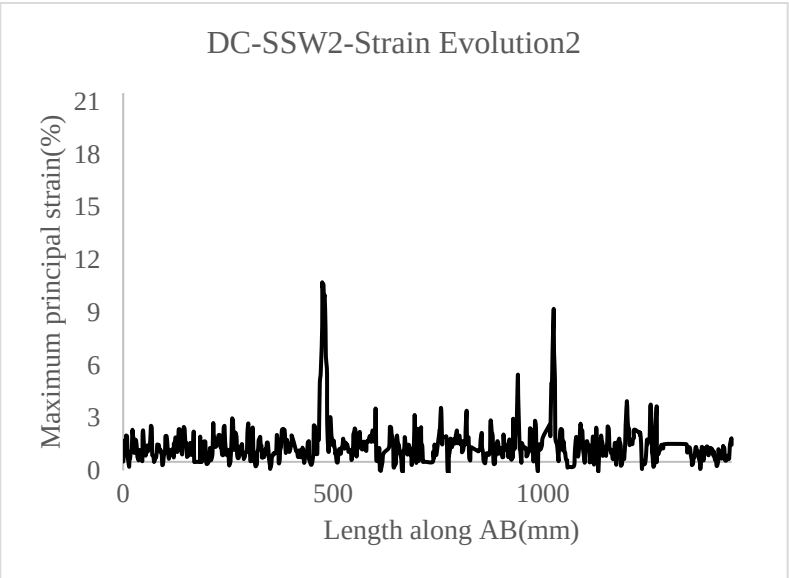
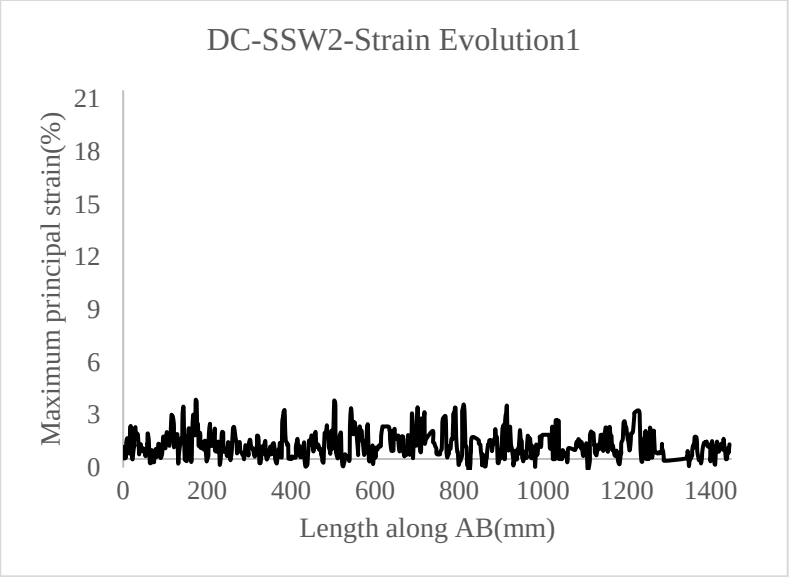
Stage 1

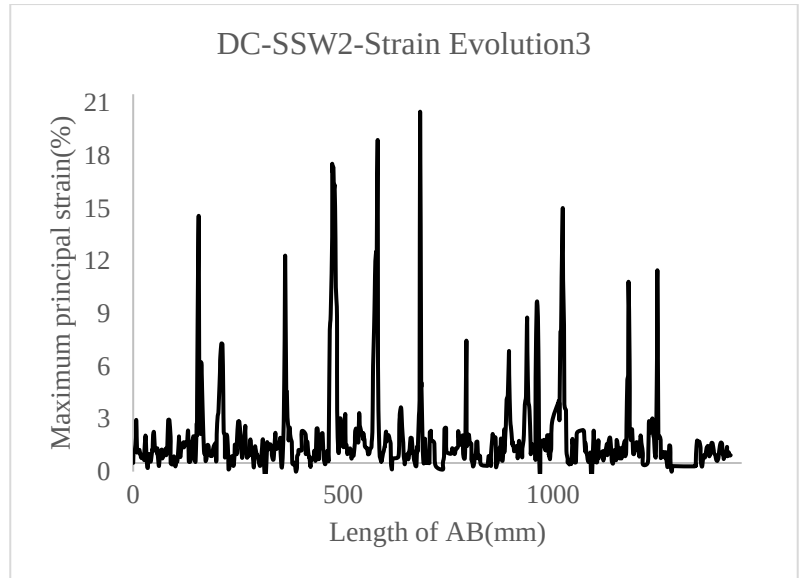
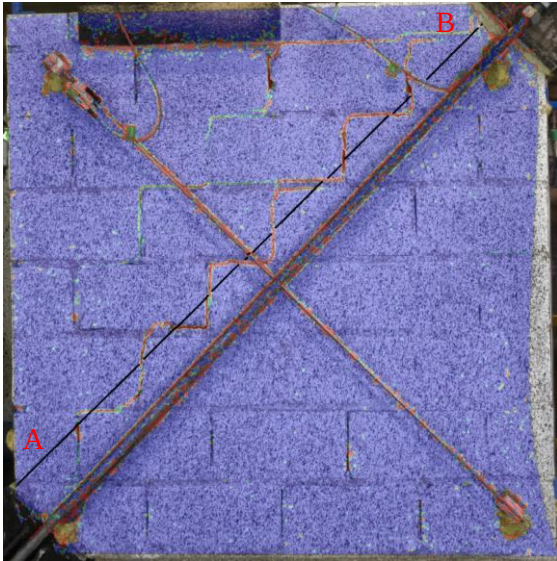


Stage 2



Stage 3





Stage 4

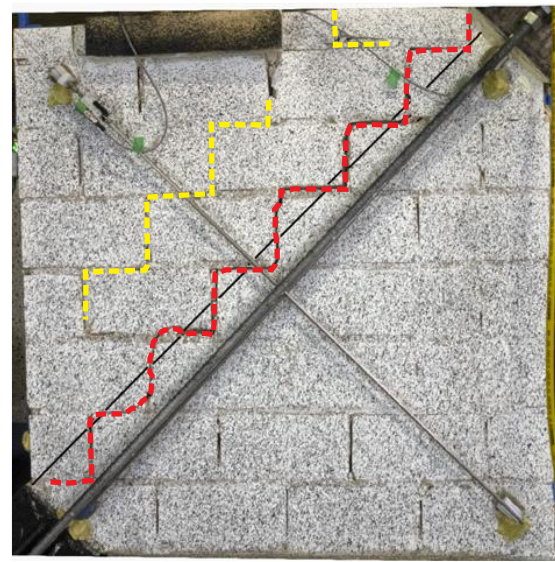
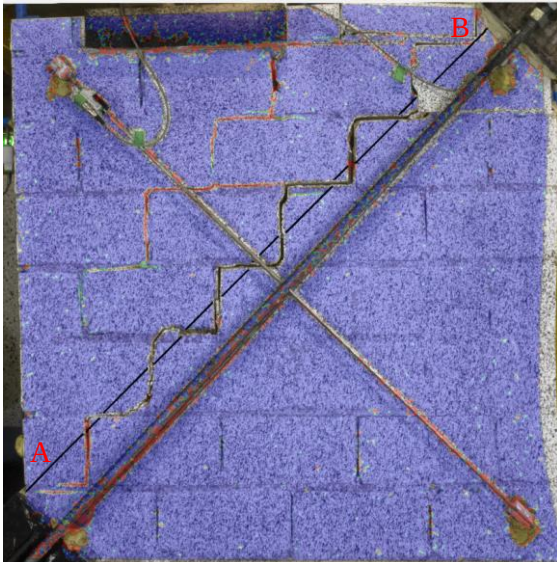


Figure C.11: Strain evolution for sandstone panels

(Note that in stage 4, the highlighted red line represents major crack and the highlighted yellow line represents minor crack)

D BEHAVIOR OF MASONRY STRUCTURES BY NUMERICAL STUDY

D.1 Modelling strategies for masonry

Previous research of finite element method (FEM) model has been established into two strategies in field of masonry. The two approaches are defined based on modelling scale of masonry assemblage. Namely, one is homogeneous approach and the other one is heterogeneous approach (Bolhassani, Hamid et al. 2015). The flow chart relating to this is provided in figure D.1. The figure presented modelling strategies for masonry in two scales and techniques. It is noted that techniques to define joint interface is not limited cohesive behaviour, but current study focuses on cohesive behaviour.

The homogenous approach considers the masonry assemblage as isometric material, so that the unit-joint interfaces are smeared out in continuum element. Namely, macro modelling method saves significant amount of computational time. Thus, brick unit, mortar and joint interface are treat as composite material. The method suitable for large scale of masonry structure to examine overall behaviour of masonry, but the output result may not be reliable since the masonry is a non-homogenous structure.

The main difference between heterogeneous approach and homogenous approach is whether considering joint interface as discontinuous elements or not. Since masonry failure mainly governed by failure of joint interface, heterogenous approach provide more realistic trend of failure mechanism. The heterogenous modelling strategies can be defined into two strategies based on level of complexity. Namely, detailed micro modelling and simplified micro modelling introduced as described in figure D.2 and both strategies consider brick and mortar as a separate element. As shown in figure D.2, detailed micro strategy considers masonry unit, mortar and joint interface by discontinuous elements but simplified micro strategy, considers mortar joint in different way. The modelling approach considers thickness of mortar as finite thickness and then, expand unit in horizontal, vertical directions. The joint interface in simplified micro model lumped unit and joint interface so the stiffness is defined as average stiffness of masonry and it written in Eq: A.16, A.17. The limitation of simplified micro model is neglecting Poisson effect of mortar (Lourenco, Paulo. 1996). However, the geometry of walls are not changed since the masonry unit expanded in horizontal, vertical direction.

As shown in table D.1, each of modelling technique has advantage and disadvantage in terms of complexity, accuracy and computational cost. The proposed modelling for this study employed the simplified micro modelling due to the simplicity of modelling and accuracy of the result and carried out surface based cohesive behaviour to identified joint interface. The Poisson's ratio of masonry taken 0.15 as previously

used by Lorenzo, Bolhassani and Kurdo. The simplified micro modelling is not including the Poisson's effect of the mortar but still there is no significant differences with the other applications according to validation as reported by Lorenzo (1996).

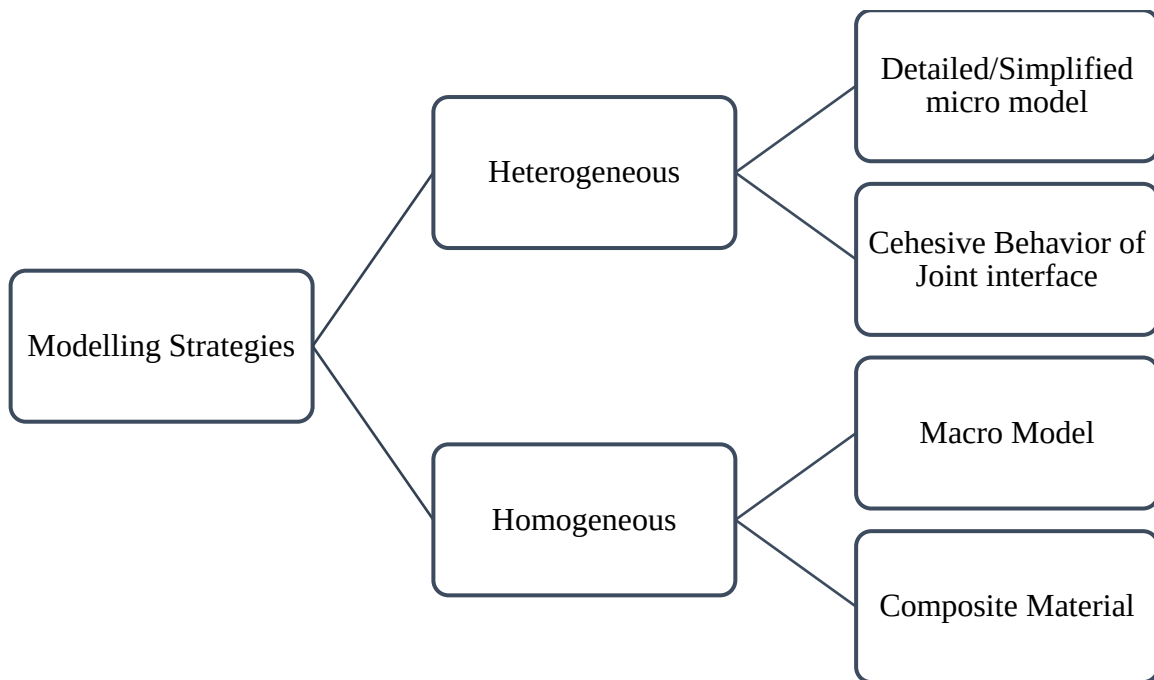


Figure D.1: Flow chart of modelling strategies

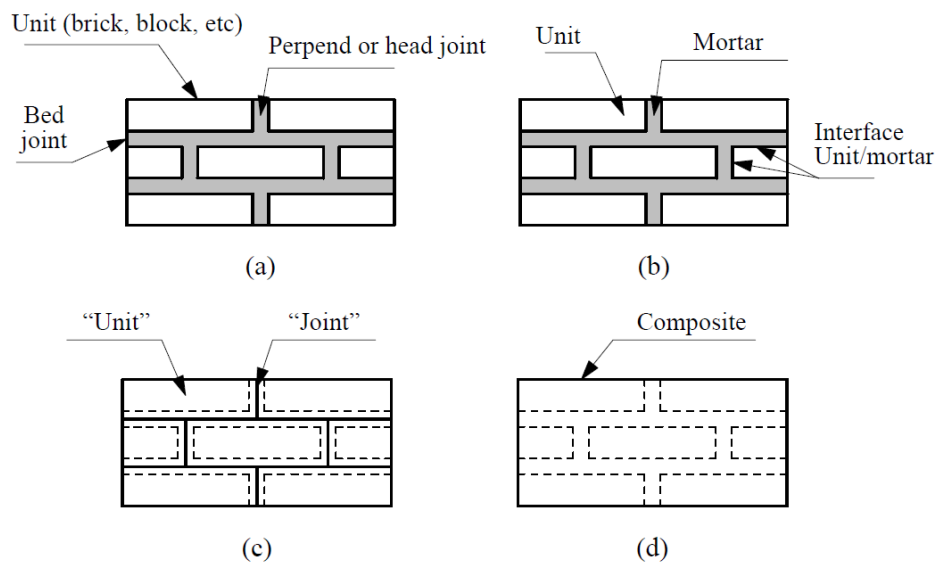


Figure D.2: Masonry wall modelling approaches (Lourenco, Paulo. 1996)

Modelling Approach	Level of Complexity	Accuracy of Result
Detailed micro modelling	High	Accurate
Simplified micro modelling	Middle	Intermediate
Macro modelling	Low	Low

Table D.1: Comparison of Modelling Approach

D.2 Elastic element of expanded unit

The elastic modulus of the expanded units must be adjusted and should have an equivalent elastic stiffness to the original masonry assemblage as shown in figure D.3. It is to be determined by taking elasticity of modulus of original masonry unit and mortar and geometry of the masonry assemblage into account. For this purpose, Eq (1) is proposed based on the assumption of a stack bond between masonry units and uniform stress distribution in masonry constituents. Based on literature review, the proposed Eq D.1 is also corresponding to van der plujim (1999) and Kurdo et al (2017).

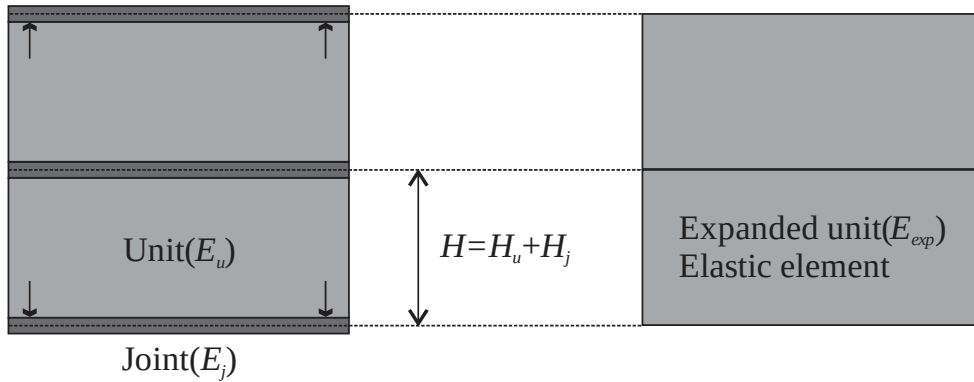


Figure D.3: Expanded unit in simplified micro scale

$$E_{\text{exp}} = \frac{HE_uE_j}{H_uE_j + H_jE_u} \quad (\text{Eq: D.1})$$

D.3 Traction separation law

An objective of this thesis was to investigate post behaviour of URM structure by FEM study. For this sake, simplified micro modelling approach was carried out and cohesive behaviour adopted in joint interface. The interaction of cohesive zone is function of displacement on the layer of crack propagation. The cohesive zone has been studied by Dugdale (1960) and developed by Barenblatt (1962) for the fracture mode of

brittle material. The surface based cohesive behaviour was suitable for the bonded interfaces with negligible thickness. As discussed in the section D.1, the simplified micro model addresses the similar strategy into joint interface. The modelling strategy considers the mortar joint as a finite thickness and expand the brick element in vertical and horizontal directions. The contact of expanded units behaves very similar to glue, so the interaction surface is a bonded area.

In order to capture pre-failure and post failure of joint interface, surface based cohesive model has been adopted and it is defined by damage criterion in ABAQUS. As shown in figure D.4, the damage criterion is addressed with traction separation law and it corresponds to stress versus displacement response. Namely pre-failure stage called damage initiation and post failure called damage evolution. The damage initiation can be represented by either maximum stress or strain ratio or quadratic stress or stain ratio and to be able to capture realistic trend, current model selects quadratic stress criterion as written in Eq: D.2. The Macaulay bracket is expressed in the Eq D.2 to neglect the compressive stress in the masonry joint along the nominal direction.

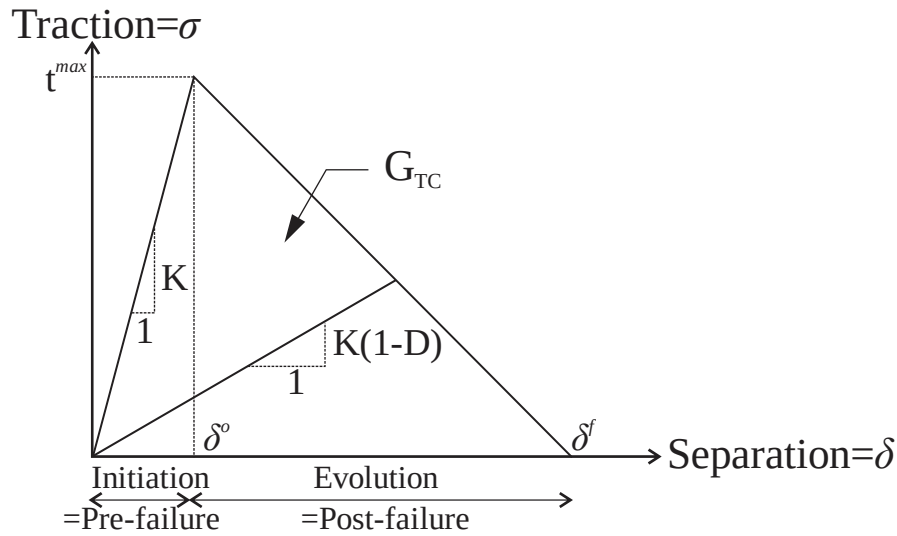


Figure D.4: Traction separation law along the joint interface

$$\left\{ \frac{\langle t_n \rangle}{t_n^{\max}} \right\}^2 + \left\{ \frac{t_s}{t_s^{\max}} \right\}^2 + \left\{ \frac{t_t}{t_t^{\max}} \right\}^2 = 1 \quad (\text{Eq: D.2})$$

As shown in figure D.4, the response of damage initiation, t is defined as linear behaviour, but the elastic stiffness, K will be degraded when the stress vector t reach to maximum allowable stress, t^{max} . Thus, the corresponding crack propagation will respond based on the damage evolution. The figure D.5 is expressed the traction by stiffness. The $(1-D)$ is the reduction factor due to the damage initiation beyond the maximum allowable stress component t^{max} . D is the damage evolution variable which represent the degradation of stiffness matrix K . Initial value of the damage evolution variable is from 0 and increases up to 1 as shown in figure D.4. The damage evolution variable D is expressed as Eq D.3 (Camanho and Davila 2002). The δ_{eff}^{max} denotes the maximum effective displacement during the loading history. The effective displacement δ_{eff} is expressed in Eq D.4 (Camanho and Davila 2002).

$$t = \left\{ \begin{array}{l} (1-D)K\delta \leftarrow (t_{initial} \geq t^{max}) \\ K\delta \leftarrow (t_{initial} < t^{max}, \text{No damage into stiffness}) \end{array} \right\}$$

Figure D.5: Traction, t expressed with stiffness K

$$D = \frac{\delta_{eff}^f (\delta_{eff}^{max} - \delta_{eff}^o)}{\delta_{eff}^{max} (\delta_{eff}^f - \delta_{eff}^o)} \quad (\text{Eq: D.3})$$

$$\delta_{eff} = \sqrt{\langle \delta_n \rangle^2 + \delta_s^2 + \delta_t^2} \quad (\text{Eq: D.4})$$

The damage evolution can be identified based on the fracture energy that dissipated during the separation process or based on maximum displacement. Due to the characteristic of brittle material, it is hard to capture the maximum displacement of masonry. Thus, the current study expresses damage evolution based on fracture energy. The geometry terms of total fracture energy in mixed mode, G_{TC} is the area of traction separation plot graph as shown in the figure D.4. It is assumed that the traction-separation response of masonry in mode 2 mode 3 are in the same direction. Based on the assumption, the mixed mode fracture energy, G_{TC} can be calculate by Benzeggagh-Kenane (BK) law. The exponent η holds value of 2 for brittle behaviour of joint interface (Benzeggagh, M. L et al. 1996). The critical mode fracture energy G_{TC} under mixed mode is written in Eq: D.6.

$$G_{TC} = G_{IC} + (G_{IIC} - G_{IC}) \left\{ \frac{G_{II} + G_{III}}{G_I + G_{II} + G_{III}} \right\}^\eta \quad (\text{Eq: D.6})$$

D.3.1 Linear elastic behaviour of stiffness

The traction separation response of joint interface prior to yield is elastic behaviour and traction vector can be written as nominal stress as shown in Eq: D.3. The traction vector can be rewritten by stiffness K and displacement δ . The rearrangement of traction vector is represented in Eq: D.4. The displacement of joint interface, δ_n is equivalent to the total displacement of masonry assemblage, δ_{total} minus displacement of expanded unit δ_u^j . Rearranging the nominal displacement of δ_n , it is as written in Eq: D.5.

$$t = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} E_{nn} & E_{ns} & E_{nt} \\ E_{ns} & E_{ss} & E_{st} \\ E_{nt} & E_{st} & E_{tt} \end{bmatrix} \begin{Bmatrix} \epsilon_n \\ \epsilon_s \\ \epsilon_t \end{Bmatrix} = E \epsilon \quad (\text{Eq: D.3})$$

$$t = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} K_{nn} & 0 & 0 \\ 0 & K_{ss} & 0 \\ 0 & 0 & K_{tt} \end{bmatrix} \begin{Bmatrix} \delta_n \\ \delta_s \\ \delta_t \end{Bmatrix} = K \delta \quad (\text{Eq: D.4})$$

$$\delta_n = \delta_{total} - \delta_u^j$$

$$\delta_n = t_n \left(\frac{H_u}{E_u} + \frac{H_j}{E_j} \right) - \frac{t_n}{E_u^j} (H_u + H_j) \quad (\text{Eq: D.5})$$

Height of mortar is very small compare to unit height. In other words, Young's modulus of mortar, E_j will not significantly affect to the expanded unit, so the Young's modulus of expanded unit, E_{exp} can be consider same as Young's modulus of original unit E_u . Cancel out and rearrange the Eq: D.5. Finally, Stiffness of K_{nn} , K_{ss} and K_{tt} can be written in Eq: D.6, D.7. The elastic stiffness K vector obtained from shear couplet test as discussed in A.5.

$$E_{exp} \approx E_u$$

Cancel out and rearrange.

$$\delta_n = \frac{t_n H_j}{E_j} - \frac{t_n H_j}{E_u}$$

$$\delta_n = t_n h_j \left(\frac{1}{E_j} - \frac{1}{E_u} \right)$$

$$\delta_n = t_n h_j \left(\frac{E_u - E_j}{E_u E_j} \right)$$

$$K_{nn} = \frac{t_n}{\delta_n} = \frac{E_u E_j}{H_j (E_u - E_j)} \quad (\text{Eq: D.6})$$

$$K_{ss}, K_{tt} = \frac{G_u G_j}{H_j (G_u - G_j)} \quad (\text{Eq: D.7})$$

$$G = \frac{E}{2(1 + \nu)} \quad (\text{Eq: D.8})$$

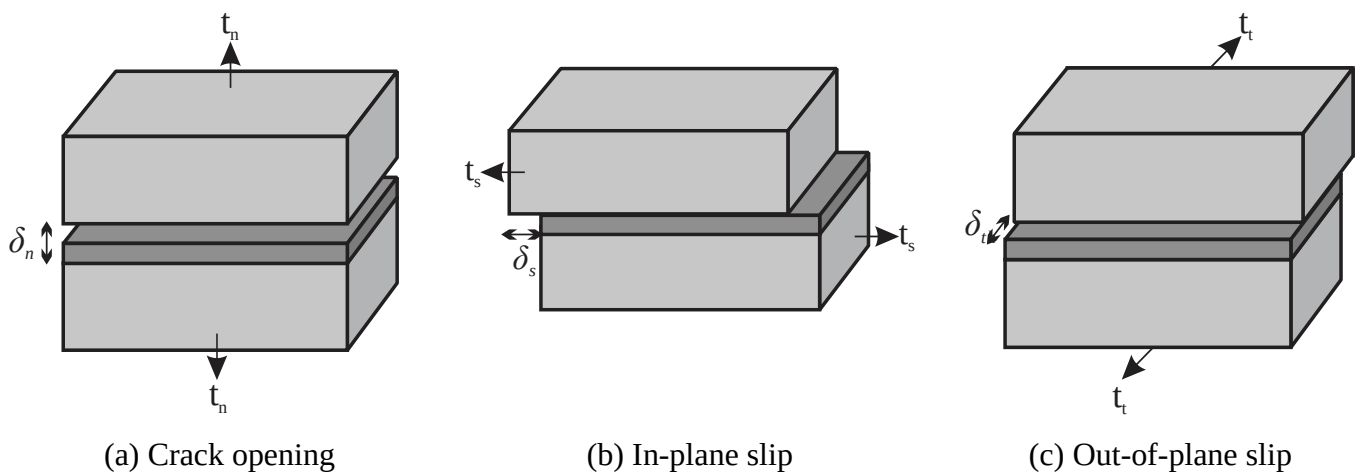
D.4 Failure modes of unreinforced masonry

Various cracking patterns are observed in masonry is subjected to ultimate loads. These failure modes of masonry under shear compression load reported by Mann and Muller. Namely, the failure modes are friction failure, brick cracking and masonry compression failure.

Lorenco and Rots (1997) reported the basic cases of failure mechanism for URM structures. The failure modes for joint interface and unit are illustrated in figure D.6, D.7. Figure D.6 illustrated the failure mode along the joint interface based on fracture mode. Namely, figure D.6 (a) crack opening, (b) in-plane slip and (c) out of plane slip are described based on traction separation law. The principal of nation in figure D.6 is illustrated in table D.2 and similarly figure D.7 followed same notations. The figure D.6 (a) corresponding to direct tensile test and the current test arrangement adopted for bond wrench test. However, the maximum allowable stress t_n has been determined by AS 3700. The figure D.6 (b) corresponds to the shear couplet test in accordance with the experimental campaign and key parameters determined and discussed in chapter A.

5. In current study, it treats the out of plane slip is same direction as in plane shear slip and joint interface derived by BK law.

Failure mode for masonry unit illustrated in figure D. 7 so namely, figure D.7 (a) unit direct tensile cracking , figure D.7 (b) unit diagonal tensile cracking is introduced. Figure D.7 (a) corresponds to three point bending test and key parameters are discussed in chapter A. 3. Failure mechanism of unit diagonal cracking in figure D.7 (b) is combined failure mechanism under shear and compression load. The combined failure mechanism is involved in unit and joint failure.



(a) Tensile bond cracking (Fracture mode 1)

(b) In-plane shear slip (Fracture mode 2)

(c) Out of plane shear slip (Fracture mode 3)

Figure D.6: Bond failure along the joint interface

Stress = Traction	Displacement = Separation
t_n : Tensile stress along joint interface	δ_n : displacement of tensile bond cracking
t_s : In-plane shear stress along joint interface	δ_s : displacement of in-plane slip
t_t : Out-of-plane shear stress along joint interface	δ_t : displacement of out-of-plane slip

Table D.2: Principal of fracture criterion

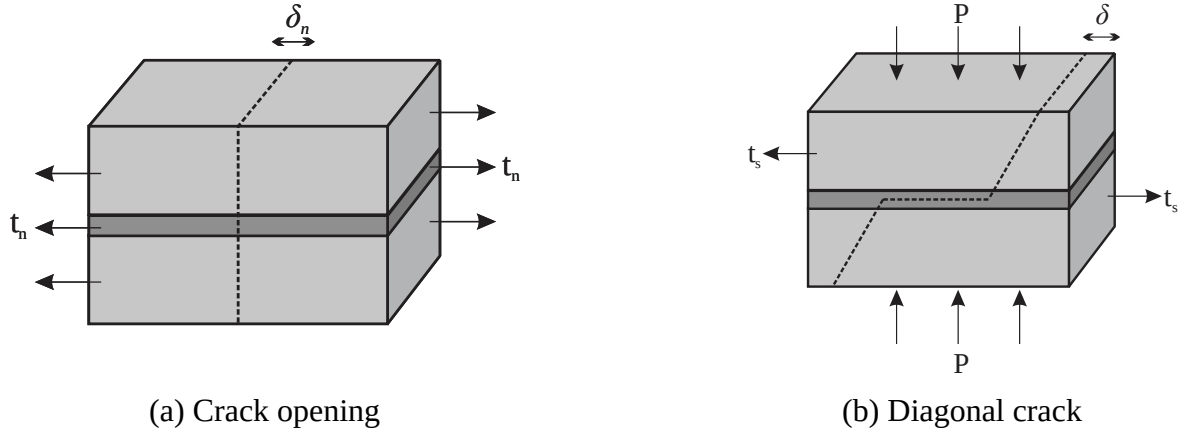


Figure D.7: Typical unit failure mode

D.5 Linear Drucker-Prager (DP) plasticity model

The linear Drucker-Prager plasticity model was employed to define uniaxial compressive behaviour of masonry. The plasticity model allows masonry to have hardening and softening under compression. The yield criterion is defined by yield surface in meridional(P - t) plane and it could be linear, hyperbolic or exponential mode. The current study adopted the linear yield surface as shown in figure D.8. The yield criterions are given by Eq: D.9 to D.14.

The evolution of yield surface with deformation beyond elastic range was specified based on uniaxial compressive yield stress. The yield stress σ_c , plastic strain ε_c^{pl} of Drucker-Prager hardening defined based on compressive stress strain curve from compression test for masonry prism. The stress strain curve is represented in figure A.13. The angle of friction β , flow stress ratio K are assumed as 36 degree, 1 and dilation angle, ψ is taken as lower value reported by Mosalam K. These Drucker-Prager parameters are represented in table D.3

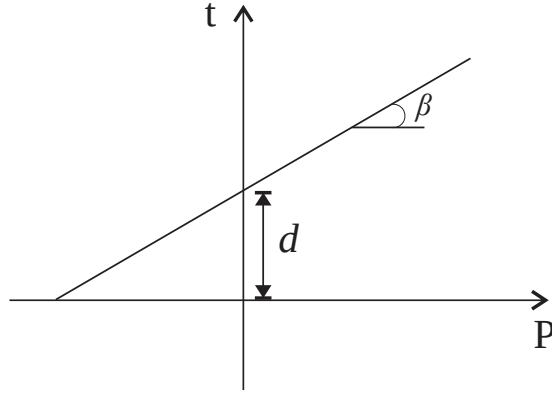


Figure D.8: Yield criteria in P-t plane for Linear Drucker-Prager

$$F = t - p(\tan \beta) - d = 0 \quad (\text{Eq: D.9})$$

The equivalent pressure stress, P is given by:

$$p = -\frac{1}{3} \text{trace}(\sigma) \quad (\text{Eq:D.10})$$

The deviator stress, t given by:

$$t = \frac{1}{2} q \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{q} \right)^3 \right] = q \quad (\text{Eq: D.11})$$

Assumed $K=1$

The von mises stress, q given by:

$$q = \sqrt{\frac{3}{2} S : S} \quad (\text{Eq:D.12})$$

The third stress invariant, r and deviatoric stress, s given by

$$r = \left(\frac{9}{2} S : S : S \right)^{\frac{1}{3}} \quad (\text{Eq:D.13})$$

$$S = \sigma + PI$$

The intercept, d given by:

$$d = \left(1 - \frac{1}{3} \tan \beta \right) \sigma_c \quad (\text{Eq: D.14})$$

Linear Drucker-Prager parameters				
Material	Angle of friction, β	Flow stress ratio, K	Dilation angle, ψ	Compression yield stress, σ_c (MPa)
Clay brick	36	1	11.3	5.42
Sandstone				12.91

Table D.3: Linear Drucker Prager parameters

D.6 Extended finite element (XFEM)

Previously, a number of researchers have demonstrated unit stiffness along the centre of unit such that the crack growth propagated in centre of unit. It is introduced by Lorenzo(1997) to simplified unit failure. However, what is not yet clear is the location of actual unit crack and the crack growth. In order to capture crack growth of masonry unit, discontinuities are required for mesh element. The challenge is that the association of meshing crack surface. The mesh element needs to be keep redefined as crack growth and geometry of discontinuity have to coordinate with mesh element. For this purpose, the current study carried out extended finite element method (XFEM).

The XFEM was firstly introduced by Belytschko and Black but the concept of partition of unity is studied by Melenk and Babuska. The partition of unity is allowing local enrichment functions to be incorporated into finite element. The XFEM displacement approximation in enriched finite element is expressed with displacement vector function u and is written as Ea:D.15.

$$u = \sum_{I=1}^N N_I(x) \left[u_I + H(x)\alpha_I + \sum_{a=1}^4 F_a(x)b_I^\alpha \right] \quad (\text{Eq:D.15})$$

where, $N_I(x)$ is nodal shape functions , u_I is the nodal displacement vector for continuous part of finite element solution and the $N_I(x) \cdot u_I$ applied to all nodes in the model. α_I is nodal enriched degree of freedom vector, $H(x)$ is associated discontinuous jump function along the crack surface and the $\alpha_I \cdot H(x)$ applied to nodes whose shape function support is cut by the crack interior. b_I^α is the nodal enriched degree of freedom vector, $F_a(x)$ is associated elastic asymptotic crack tip functions and the $F_a(x) \cdot b_I^\alpha$ applied to nodes whose shape function support is cut by the crack tip.

In current study employed XFEM to simulate damage initiation and evolution in masonry unit under tension, compression and shear as illustrated in figure D.7. The crack growth allowed without predefined crack location and XFEM simulated based on cohesive segment method as studied by Remmers. The cohesive segment method follows traction separation law as discussed in chapter D.3 and the stiffnesses parameters are calculated based on elastic properties of enriched element.

D.7 Linear mesh element (C3D8)

It has been suggested that many types of mesh elements is required to complete sound structural system (Attard and Tin-Loi, Lapidus). Attard and Tin-Loi (1999) developed basic triangle mesh element formed by nine constant strain. Lapidus introduced basic linear brick element with 9 nodes as shown in figure D.9. The nodes of elements are mainly used to calculate the displacement or degrees of freedom.

The proposed model selects the linear element C3D8 for the brick element in order to save computation cost and to achieve mis convergence issue. The name of mesh element C3D8 implies the meaning of element. The C3D8 implied that, C continuum element, 3D three-dimensional, 8 the number of contact nodes. The linear brick element C3D8 has been commonly used by many researchers especially under complex state of load. Alli and Maan employed C3D8 mesh element for masonry room under dynamic loading. The FEM model simulated in three different scale which detail micro, simplified micro and macro and shown good agreement with experiment result. Kurdo studied FEM model for masonry under in-plane and out of plane loading with C3D8 mesh element. The force displacement response of FEM models validated with experimental results and sensitivity study conducted to determine optimized mesh size. Mohammad Bolhassani reported FEM model for compression test of prism, bed joint shear test and diagonal compression test. The FEM models applied C3D8 to simulate compression test for prism and diagonal compression test.

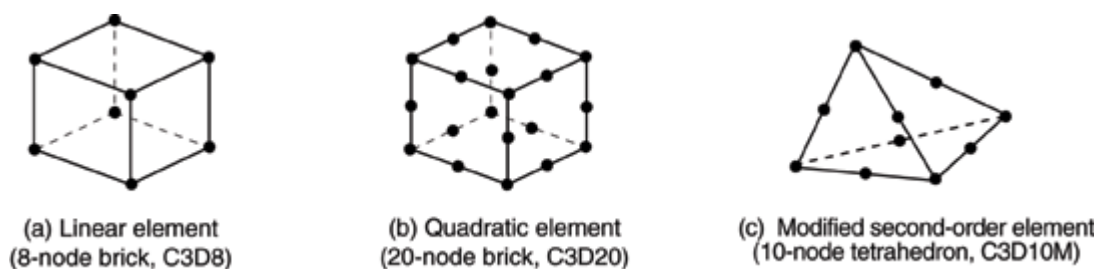


Figure D.9 : Typical mesh elements (ABAQUS user manual 2013)

E. FEM MODEL FOR MASONRY PRISM

E.1 Compression behaviour for prism

The current experiment campaign carried out compression test for prism by load control so post behaviour of masonry is remain unclear. In order to investigate post behaviour of masonry prism under uniaxial compression, FEM simulation conducted with displacement control. The FEM model defined masonry prism in simplified micro scale so thickness of joint considered finite thickness and joint interface defined by cohesive behaviour. In addition, softening of evolution criterion defined as linear form with BK law.

The table E.1 is represented material properties of masonry unit. Elasticity of young's modulus unit, joint measured from compression test and Poisson's ratio taken 0.15 as common value. The direct tensile strength, f_t is estimated 1/6 of f_{mt} which determined from three point bending test. The shear strength, τ_{us} assumed 1.4 times direct tensile strength, f_t as written in Eq: E.1, E.2. The shear fracture energy $G_f^{II}_{-u}$ taken 0.5 N mm/mm^2 as reported by Raijmakers. The crack propagation of unit is mainly governed by tensile stress so damage evolution for shear stress will be less sensitive. Previous studies of unit failure have only been undertaken using brick interface through center of unit so the crack path already predefined. It has been introduced by lorencio but it still does not cover realistic trend of unit failure. Thus, the current study carried out failure of unit by XFEM without predefined crack path and it allows to have crack propagation of unit more sensitively.

The table E.2 is represented nonlinear properties of joint interface with tension, shear and compression. The stiffness K measured from shear couplet test and adopted to estimate normal stiffness K_{nn} . The flexural tensile strength of t_n determined by bond wrench test and fracture energy, G_f^I followed recommended value 0.012 introduced by Angelillo. The Mohr's coulomb failure criterion modified with dry frictional coefficient so initial shear strength of joint interface had been modified. The fracture energy G_f^{II} calculated and adopted by linear regression line as described in figure A.32.

Masonry unit	Brick unit	Mortar joint	Expanded unit			Tension		Shear	
	E_u (MPa)	E_j (MPa)	ν	$E_{exp_calculated}$ (MPa)	E_{exp_test} (Mpa)	f_{t_u} (MPa)	$G_f^I_{-u}$ (N/mm)	τ_{-u} (MPa)	$G_f^{II}_{-u}$ (N/mm)
Clay brick	9085	436	0.15	2611	2272	0.67	0.21	0.93	0.5
Sandstone	10573	436	0.15	4310	3206	0.775	0.25	1.09	0.5

Table E.1 Elastic properties of masonry unit

$$f_t = \frac{1}{6} f_{mt} \quad (\text{Eq: E.1})$$

$$\tau_{us} = 1.4 f_t \quad (\text{Eq: E.2})$$

Masonry joint interface	Stiffness K (N/mm ³)			Tension		Shear			Compression
	K_{nn}	K_{ss}	K_{tt}	t_n (Mpa)	G_f^I (N/mm)	c (MPa)	μ_{dry}	G_f^{II} (N/mm)	σ_c (Mpa)
Clay brick	7.37	3.2	3.2	0.24	0.012	0.2	0.71	0.20	5.42
Sandstone	11.59	5.04	5.04	0.06	0.012	0.15	0.68	0.11	12.91

Table E.2 : Inelastic properties of joint interface

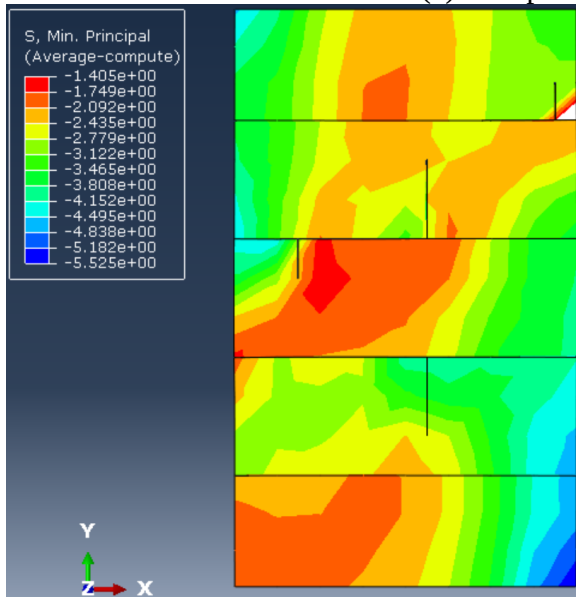
E.2 Discussion of FEM result

Figure D.10 represented failure pattern of clay brick prism and FEM result. Figure D.10 (a), (b) are presented to compare crack propagation and it shown similar crack propagation. The principal stress concentrated at center of unit and propagated to upper right direction. The stress distribution is well match with crack pattern of clay brick prism. Figure D.10 (c) indicated the crack opening status of elements. The opening status represented as value between 0(0%) to 1(100%) and it is indicated partial crack of element. One noted that crack propagation in figure D.10 (c) does not show any partial failure in center of unit. The unknown effect may due to the shear slip along joint interface.

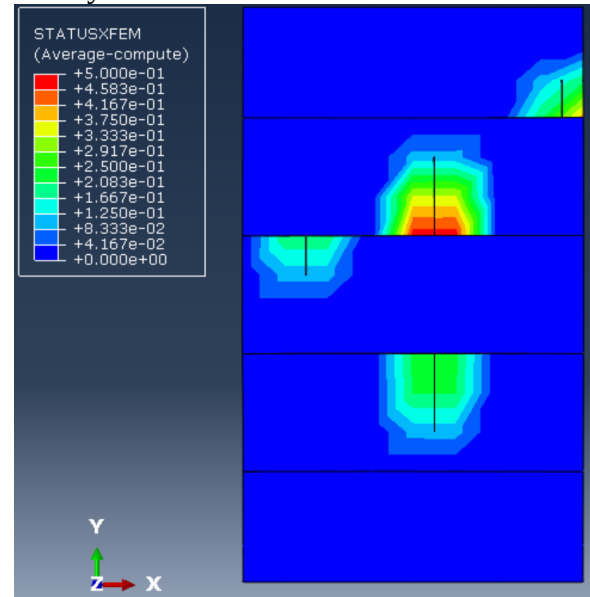
In order to investigate the compressive behaviour of masonry prism, stress strain response has been plotted in figure D.11 (a) and compressive strengths, f'_c are compared in table D. The response has good agreement with experiment up to ultimate load and the stress tend to drop off due to the unit failure. It is worth to point out that the response obtained from numerical model is more reliable compare to idealized stress strain curve studied by Kaushik. Besides, the compressive strength, f'_c in FEM overestimated about 1% compare to experiment response.



(a) Compression test for clay brick



(b) FEM σ_{III}



(c) StatusXFEM

Figure D.10: FEM model for compression test of clay brick prism

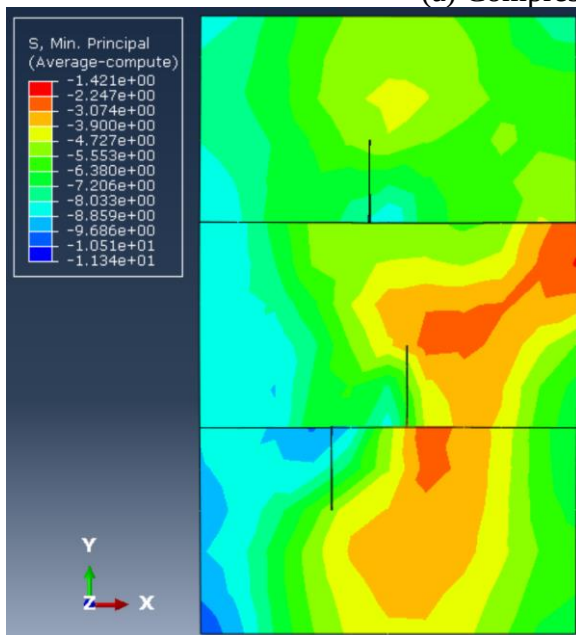
Figure D.11 represented failure pattern of sandstone prism in experimental campaign and FEM model. The principal stress contour plot is illustrated in figure D.11 (b) and stress distribution is similar to failure pattern of prism in figure D.11 (a). Regarding crack pattern, it is mainly observed in center unit in vertical direction and figure D. 11 (c) has similar trend.

To validate numerical model stress strain curve presented in figure D. 12. Unlike stress strain response for clay brick, ultimate load of sandstone prism is underestimated by about 2.65% and the response completely

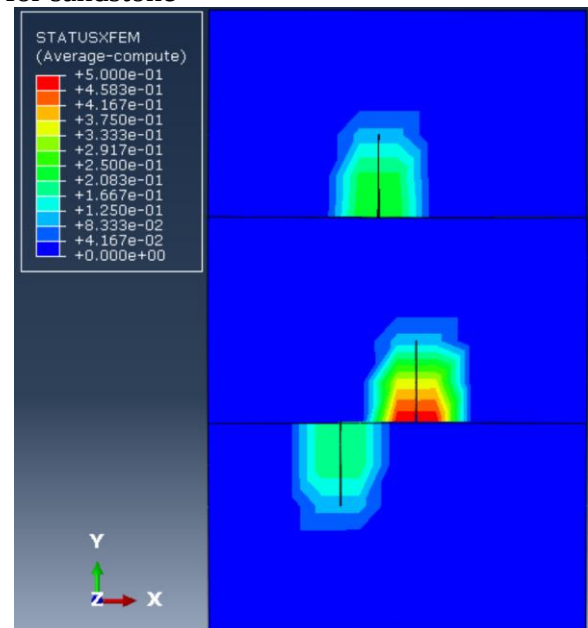
drops off after reaching the ultimate load. It is distinct response compared to idealized stress response. For future work, it would be interesting to conduct compression test of prism by displacement control and compare to numerical model.



(a) Compression test for sandstone

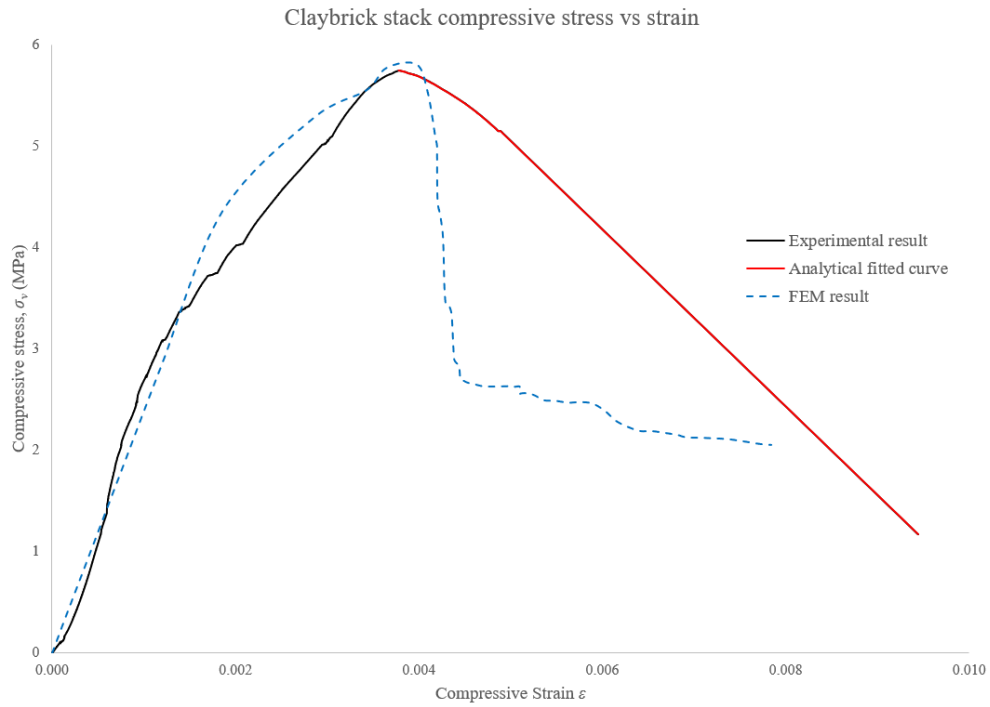


(b) FEM σ_{III}

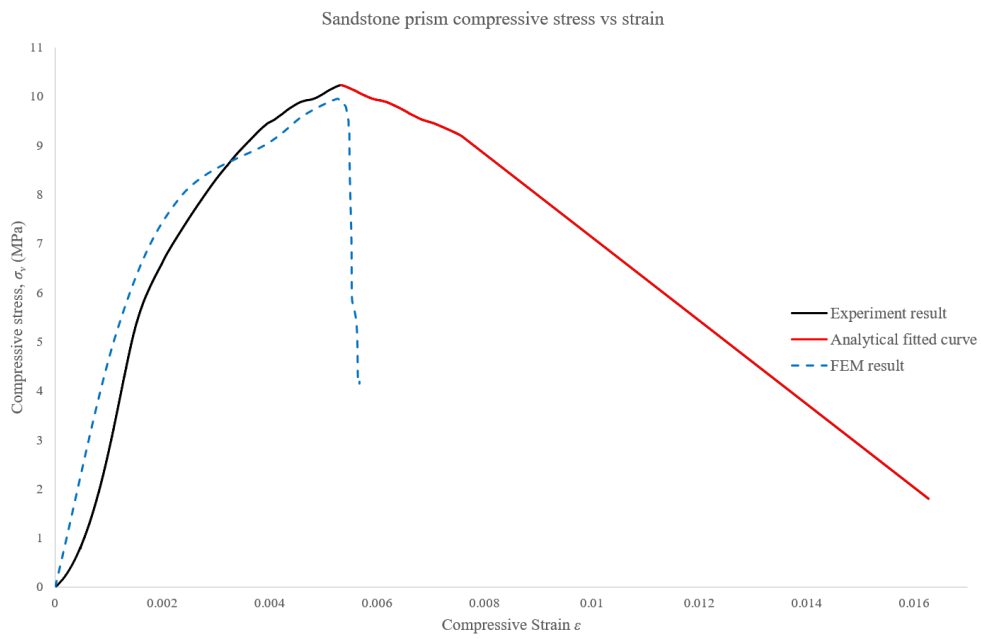


(c) StatusXFEM

Figure D.11: FEM model for compression test of sandstone prism



(a) Clay brick prism



(b) Sandstone prism

Figure D.12: Compressive stress vs strain response for masonry prism

Compressive strength of masonry, f'_c (MPa)			
Masonry prism	f_{c_test} (MPa)	f_{c_FEM} (MPa)	difference (%)
Clay brick	5.75	5.80	0.98%
Sandstone	10.23	9.96	-2.65%

Table E.3: Comparison of f'_c in FEM and experiment

F. IDENTIFICATION OF DIAGONAL TENSILE STRENGTH OF MASONRY PANEL

F.1 Determination of diagonal tensile strength of masonry panel

Diagonal tensile strength of masonry determined by ASRM, RILEM standard. These codes have different interpretations in determining f_t in experiment result. The main difference of two codes is definition of stress field in masonry panel under diagonal compression load. The stress component on masonry panel subjected in diagonal compression load, is illustrated in figure F.1 (a).

The ASTM standard defined state of stress as a uniform shear stress. In uniform shear stress condition, shear stress σ_{xy} , maximum and minimum principal stress σ_I , σ_{II} are all equivalent. Thus, Mohr coulomb circle can be drawn as figure F.1 (b). The f_t obtained from maximum principal stress σ_I as written in Eq: F.1.

The RILEM code considered the stress field is ununiform and it evaluated by Frocht. Frocht considering masonry panel as elastic homogeneous isotropic continuum element and provided following stress components at centre of panel that are subjected in pure diagonal compression load. The stress components presented in Eq : F.1-F.6. According to RILEM code, f_t identified maximum principal stress in centre of masonry wall. The Mohr's circle is also presented in figure F.1 (b).

Diagonal tensile stress according to ASTM standard

$$\sigma_{xy} = \sigma_{III} = \sigma_I = f_{t_ASTM} = \frac{1}{\sqrt{2}} \frac{P}{A} \quad (\text{Eq: F.1})$$

Stress components according to RILEM standard(=elastic solution evaluated by Frocht)

$$\sigma_x = \sigma_y = -0.5 \frac{P}{A} \quad (\text{Eq: F.2})$$

$$\sigma_{xy} = 1.1 \frac{P}{A} \quad (\text{Eq: F.3})$$

$$\sigma_{III} = -1.62 \frac{P}{A} \quad (\text{Eq: F.4})$$

$$\sigma_I = f_{t_RILEM} = 0.5 \frac{P}{A} \quad (\text{Eq: F.5})$$

$$A_{net} = \frac{1}{2}(L + H)Tn \quad (\text{Eq:F.6})$$

Where A_{net} is net area of masonry panel, L, H and T each being length, height and thickness of the panel respectively, and n is percentage of gross area of the unit that is solid, it assumed as 1

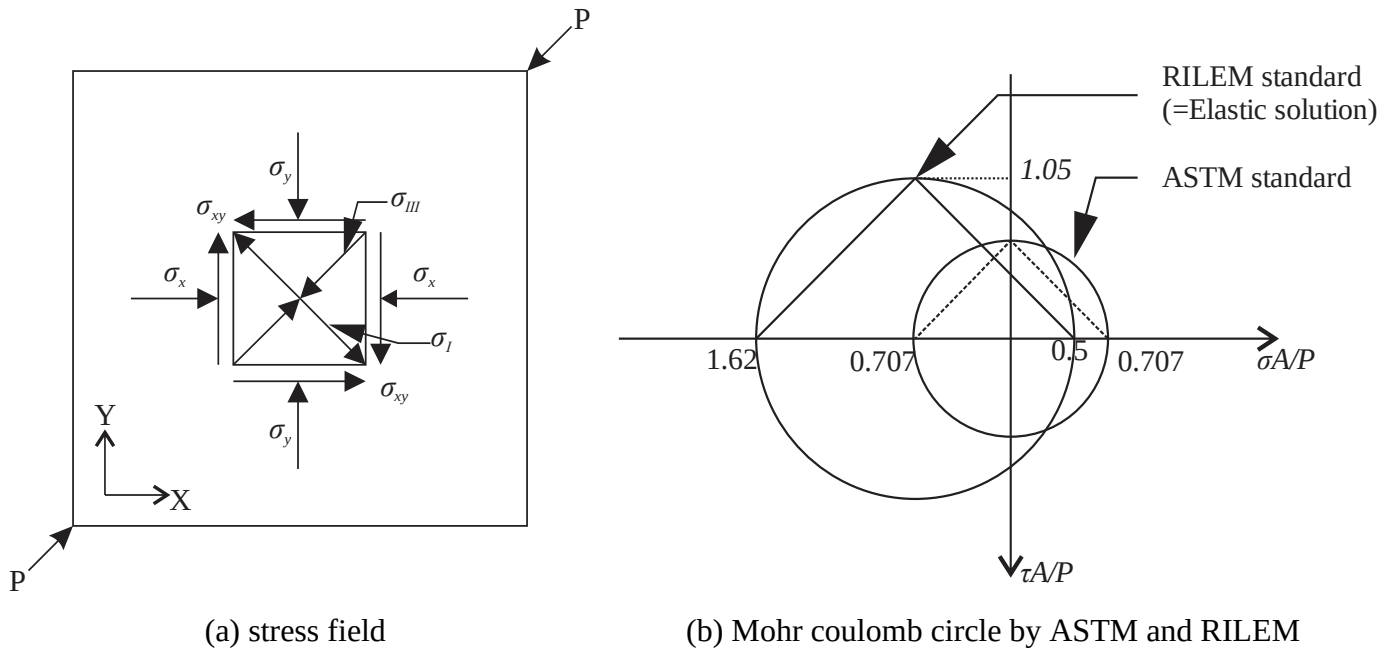
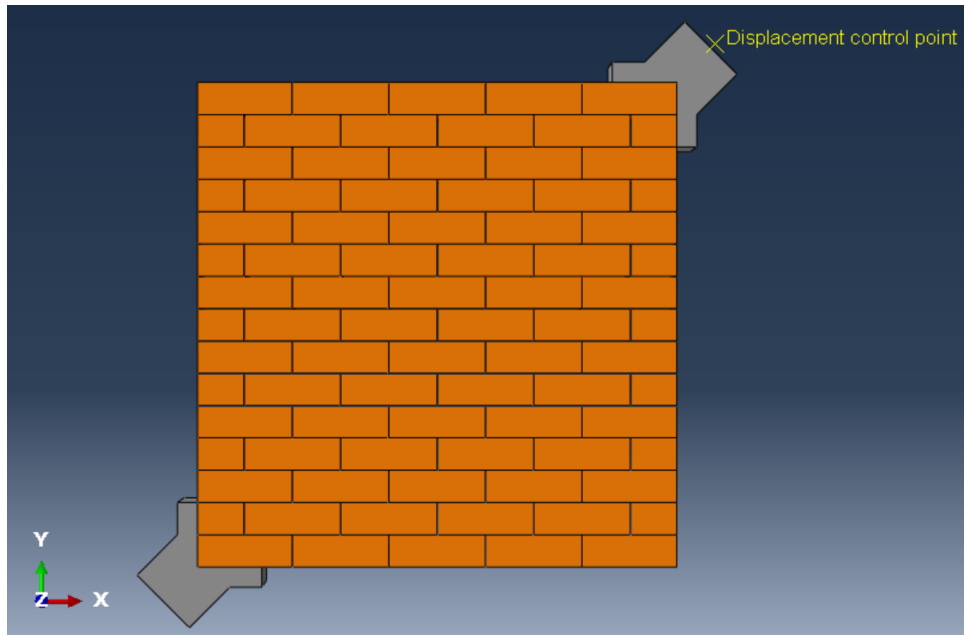


Figure F.1: Identification of stress field and Mohr coulomb circle based on ASTM and RILEM standard

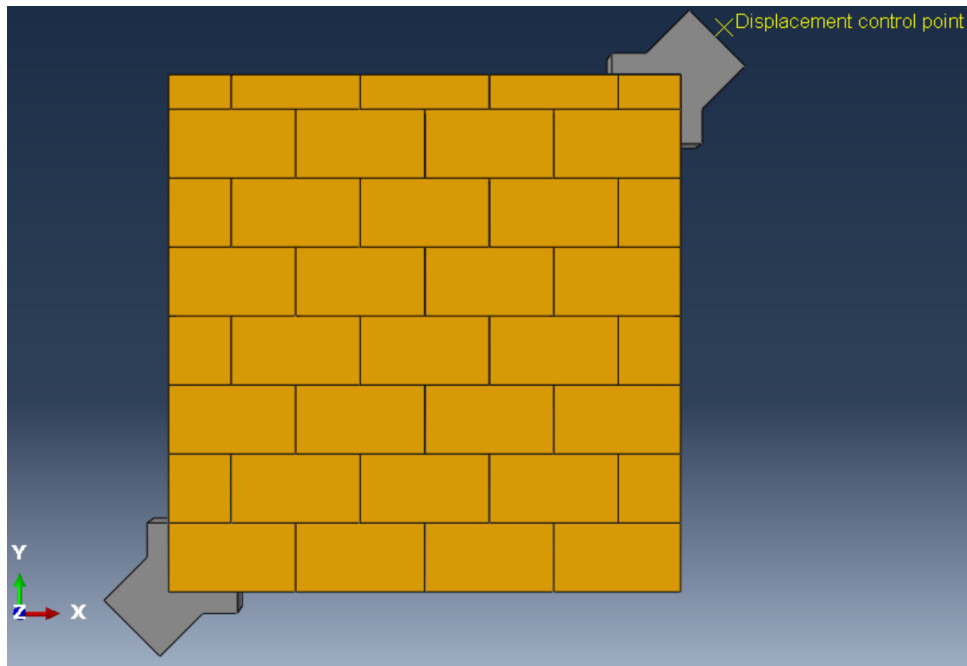
F.2 Numerical analysis of diagonal compression test

The numerical study has been carried out to analyse stress field and the simulation delivered with ABAQUS software package. The numerical model for clay brick panel and sandstone panel presented in figure F.2. The walls modelled with same geometry of panel including the loading shoe. The constitutive model simulated with assigned reference point and imposed displacement control along diagonal in 45 degree angle. The displacement control has been enforced by 7mm for clay brick panel and 5mm for sandstone panel to capture post behaviour.

The numerical model has been simulated in two modelling strategies One is elastic analysis in macro scale and the other one is inelastic analysis in simplified micro scale. The material properties of unit, joint and the joint interfaces are as described in chapter A.



(a) Clay brick panel



(b) Sandstone panel

Figure F.2: Diagonal compression test in FEM

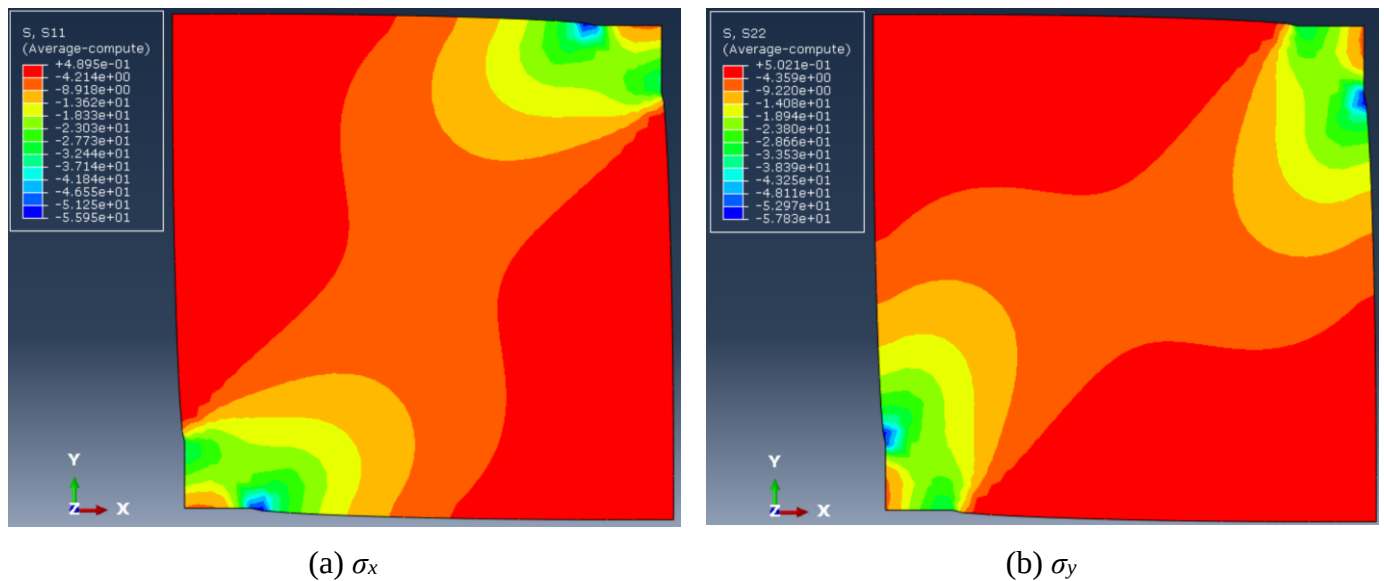
F.2.1 Linear elastic analysis of diagonal compression test

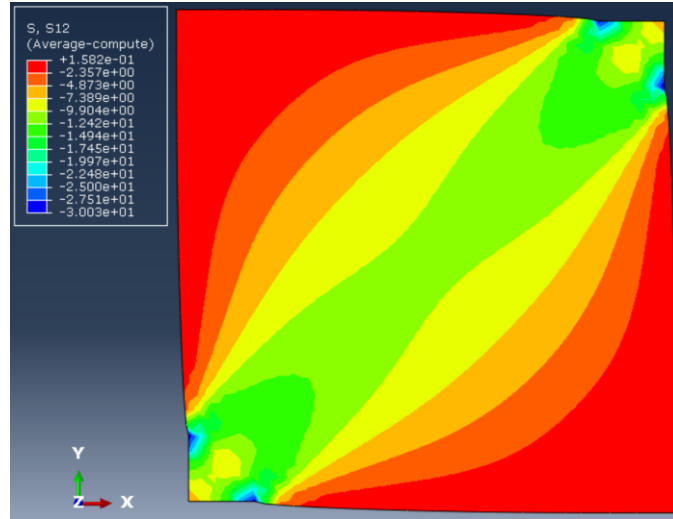
Elastic material properties of masonry is presented in table F.1. Elasticity of young's modulus for brick unit and mortar joint have been obtained from compression test. Young's modulus of expanded unit, E_{exp} is presented by calculated value from Eq: A.6 and estimated based on stress strain response. The Possion's ration has been taken 0.15 as common value.

The stress contour plot illustrated in figure F.3 and these are similar to RILEM's theoretical elastic solution. Stress concentrated in the contact of loading shoe. Yet, the stress in the centre of the panel is not affected. The stress components are written in Eq: F.7-F.10. Investigation has been carried out to evaluate state of stress subjected in monotonic diagonal loading. Response of stress component subjected in monotonic diagonal loading was taken into account to investigate the influence. Figure F.4 illustrates the state of stress and in elasticity range there no differences were observed.

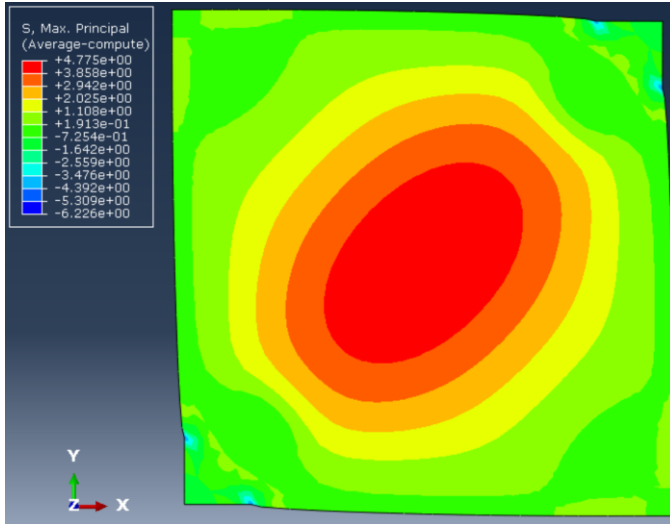
Elastic properties of masonry panel					
Material	Brick unit	Mortar Joint	Expanded unit		
	E_u (MPa)	E_j (Mpa)	ν	$E_{exp_calculated}$ (Mpa)	E_{exp_test} (Mpa)
Clay brick wall	9085	436	0.15	2611	2272
Sandstone wall	10573	436	0.15	4310	3206

Table F.1: Elastic properties of masonry

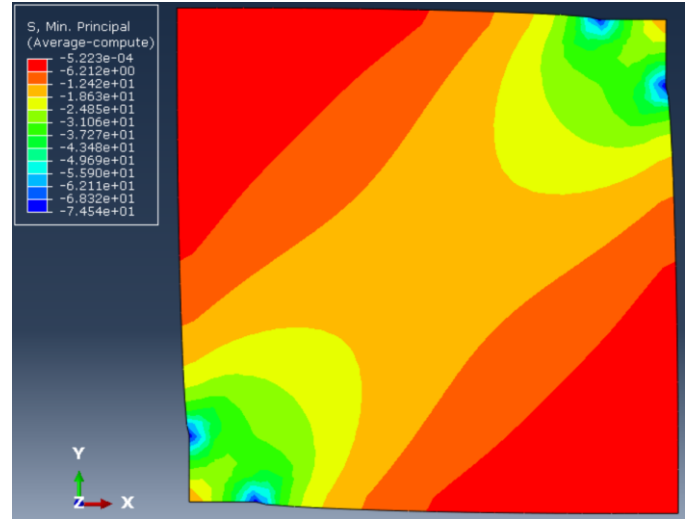




(c) σ_{xy}



(d) σ_I



(e) σ_{III}

Figure F.3: Stress contour plot from elastic analysis

$$\sigma_x = \sigma_y = -0.5 \frac{P}{A} \quad (\text{Eq: F.7})$$

$$\sigma_{xy} = -1.05 \frac{P}{A} \quad (\text{Eq: F.8})$$

$$\sigma_I = 0.5 \frac{P}{A} \quad (\text{Eq: F.9})$$

$$\sigma_{III} = -1.64 \frac{P}{A} \quad (\text{Eq: F.10})$$

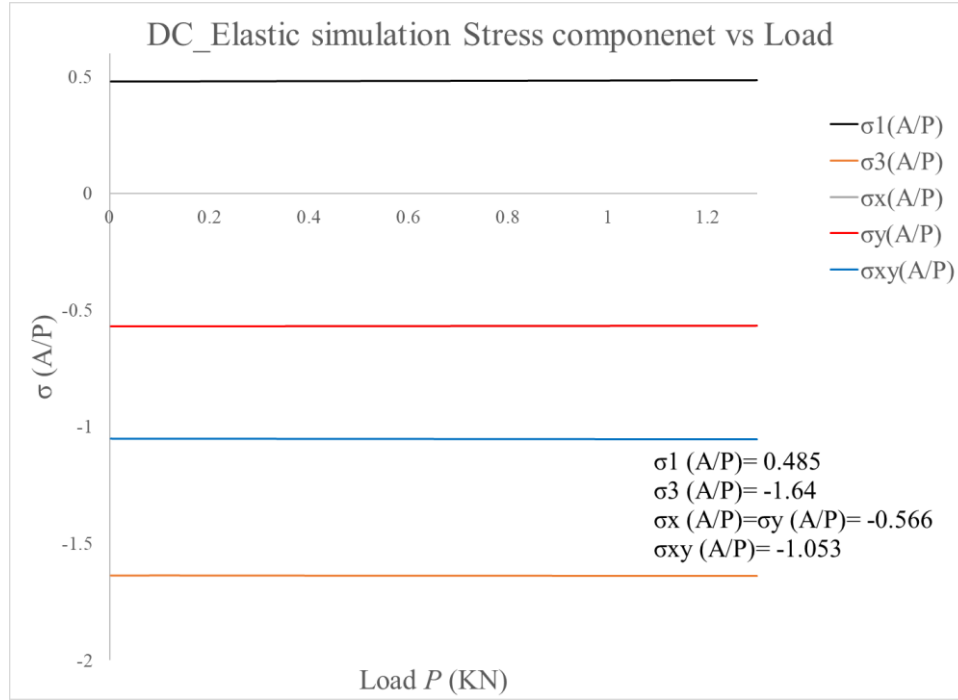


Figure F.4: Stress components in loading step

F.2.2 Investigation of coefficient factor in DIC analysis

Diagonal tensile strength, f_t expressed in Eq: F.11 and RILEM, ASTM considered different coefficient factor, α . The coefficient factor is constant parameter in elasticity range but once it has damage in centre of the panel, the α will be variable. Thus, it is important to have undertaken nonlinear analysis for diagonal compression test to investigate α . The previous investigation conducted by Brignola and the author revealed the influence of interlocking value, λ which as expressed in Eq: F.12 and ratio of tensile strength of joint and cohesion, f_t/c . The coefficient factor is not much affected by interlocking value until λ is greater than 2 but it has significantly changed by f_t/c . The author concluded that the α should be larger than 0.5 for regular masonry panel and for irregular panel should be lower than 0.5. It is great study to demonstrate coefficient factor by nonlinear analysis. However the limitation of this application is that masonry is not homogenized material and it has very large scatter of bond strength along the joint interface.

As described in chapter A.4, A.5 the variation of tensile strength of joint and cohesion are largely scattered, so each joint interface has different bond strength. In order to deal with variation of mortar bond strength, another approach conducted by install virtual gauges in DIC analysis. The diagonal tensile strength, f_t is considered to be equivalent to maximum principal stress σ_1 so it can be expressed as shown in Eq: F.11, F.13. The stress can be written with strain and elasticity of young's modulus E and maximum principal strain ϵ_{L_VSG} can be measured by virtual gauge in centre of panel along 45 degree angle. As illustrated in

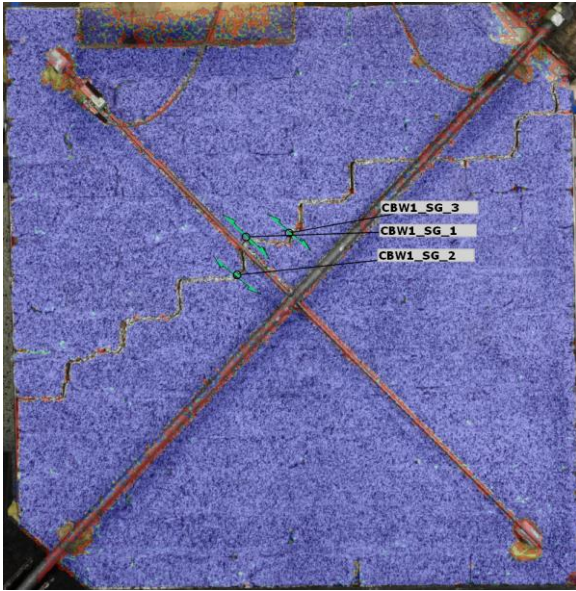
figure F.5 (a),(b),(c),(d) multiple virtual strain gauges attached along one unit and one joint in diagonal angle. The average of virtual strain taken as ε_{I_VSG} and used to calculate the normalised maximum principal stress. Figure F.5 (e) plotted the coefficient factor α versus load in diagonal compression test. As shown the figure, the α is higher than 0.5 but lower than 0.6. The α identified by virtual gauge and it is 0.58 so the diagonal tensile strength can be written as shown by Eq: F.14. It is similar to coefficient compare to RILEM standard but the α is still larger than 0.5. The reason is that since damage propagate in centre of panel, the α is increasing until the panel has completely failed.

$$\sigma_I = f_t = \alpha \frac{P}{A} \quad (\text{Eq: F.11})$$

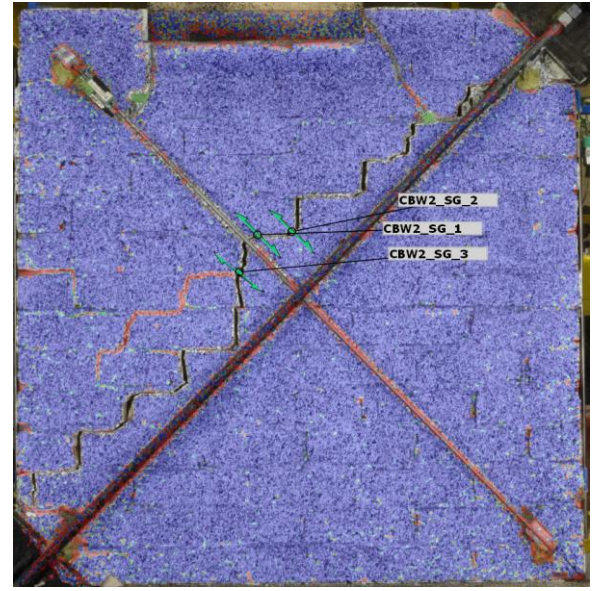
$$\lambda = \frac{H_u + H_j}{L + H_j} = \frac{2}{\tan \rho} \quad (\text{Eq: F.12})$$

Where, L is length of unit, H_j is height of joint, H_u is height of unit and ρ is angle of interlocking

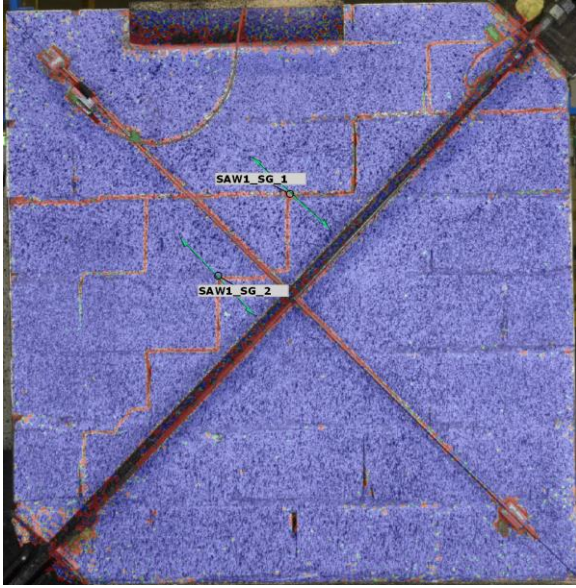
$$\sigma_I = \varepsilon E = \varepsilon_{I_VSG} E_m \quad (\text{Eq: F.13})$$



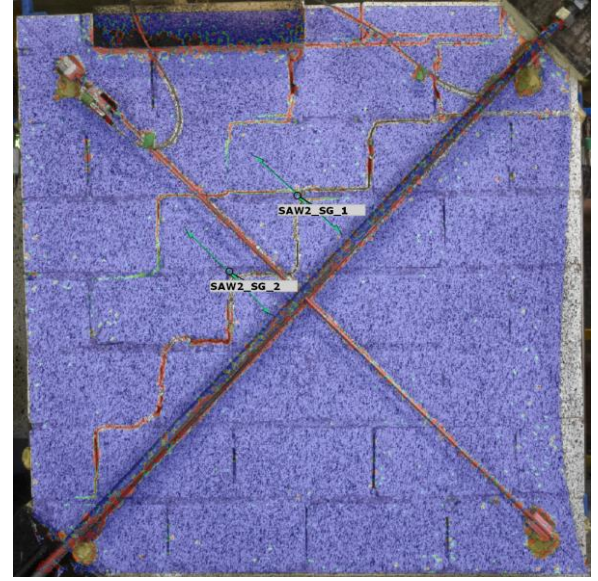
(a) Virtual gauges in CBW1



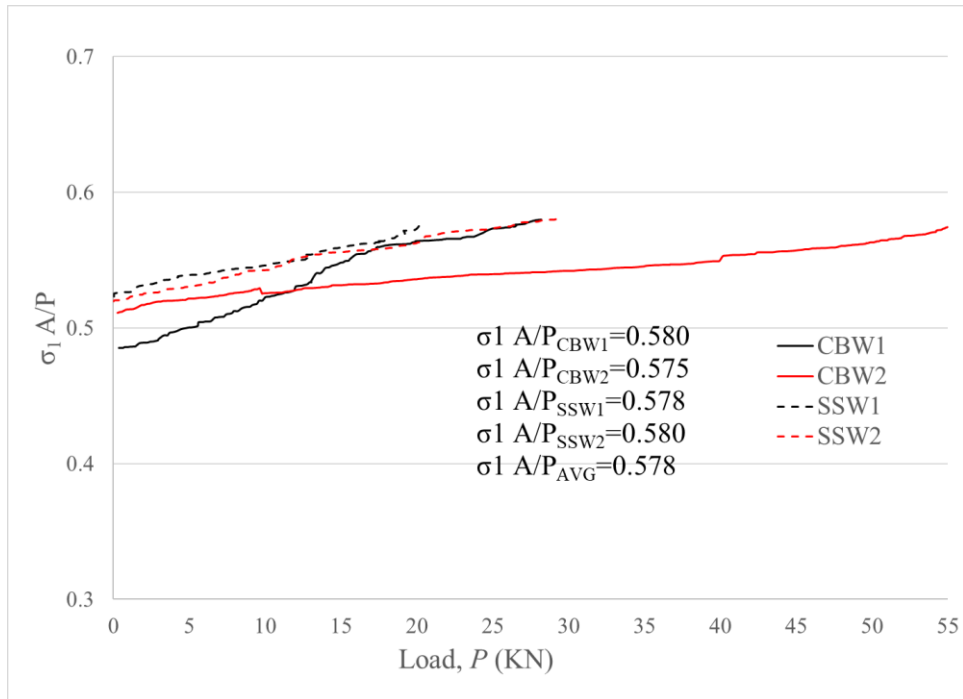
(b) Virtual gauges in CBW2



(c) Virtual gauges in SSW1



(d) Virtual gauges in SSW2



(e) Normalized maximum stress σ_1 for masonry panels

Figure F.5: Normalized maximum principal stress with DIC virtual gauge for masonry

$$f_{t_DIC} = 0.58\sigma_1 \frac{A}{P} \quad (\text{Eq: F.14})$$

F.2.3 Nonlinear analysis for diagonal compression test

As described in chapter C, majority of crack is propagated through joint interface and it was interested to evaluate diagonal tensile strength of masonry panel, f_t by FEM model. To offer an accurate evaluation, inelastic analysis in simplified micro scale conducted. The aim of inelastic analysis is to demonstrate relationship between diagonal tensile strength of panel and direct tensile strength of joint interface.

The material properties presented in table F.2. In the table F.2, the direct tensile strength of unit taken 1/6 of flexural tensile strength which obtained from three point bending test. The shear strength, τ_{-u} assumed 1.4 times direct tensile strength, f_t as written in Eq: E.1, E.2. The shear fracture energy $G_f^{II}_{-u}$ taken 0.5 N mm/mm² as reported by Raijmakers. One noted that in order to capture realistic trend of unit failure, XFEM was applied and it successfully captured the unit crack in clay brick panel.

Table F.3 is presented properties of joint interface under tension, shear and compression. The flexural tensile strength of joint interface, f_t determined by bond wrench test and taken 1/3 of f_t for direct tensile strength of joint. The fracture energy G_f^I taken recommended value 0.012 introduced by Angelillo. The shear parameters of interface obtained from shear couplet test. The Mohr's coulomb failure criterion was modified with dry frictional coefficient, μ_d so initial shear strength, c was also modified. The shear stiffness, K_{ss} is calculated from shear couplet test and normal stiffness estimated based on K_{nn} based on Eq: A.19. The softening of damage evolution criterion is defined as linear form with BK law. The fracture energy G_f^{II} is calculated and is adopted by linear regression line as described in figure A.32.

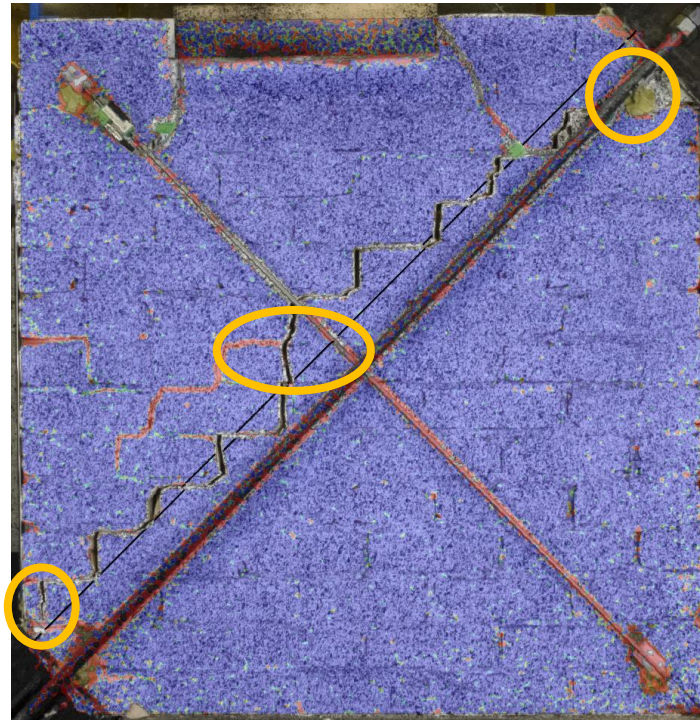
Masonry	Brick unit	Mortar joint	Expanded unit			Tension		Shear	
	E_u (MPa)	E_j (MPa)	ν	$E_{exp_calculated}$ (MPa)	E_{exp_test} (Mpa)	f_{t_u} (MPa)	$G_f^I_{-u}$ (N/mm)	τ_{-u} (MPa)	$G_f^{II}_{-u}$ (N/mm)
Clay brick wall	9085	436	0.15	2611	2272	0.67	0.21	0.93	0.5
Sandstone wall	10573	436	0.15	4310	3206	0.78	0.25	1.09	0.5

Table F.2: Material properties of masonry unit

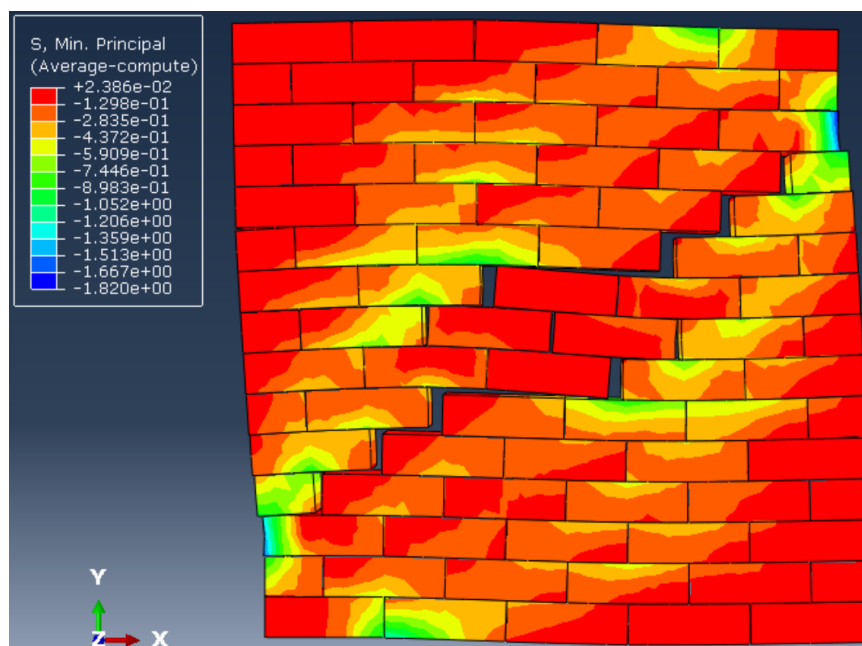
Material	Stiffness K (N/mm ³)			Tension		Shear			Compression
	K_{nn}	K_{ss}	K_{tt}	t_n (Mpa)	G_f^I (N/mm)	c (MPa)	μ_d	G_f^{II} (N/mm)	σ_c (MPa)
Clay brick wall	7.37	3.2	3.2	0.24	0.012	0.2	0.71	0.05	5.42
Sandstone wall	11.59	5.04	5.04	0.06	0.012	0.15	0.68	0.021	12.91

Table F.3: Material properties of masonry joint interface

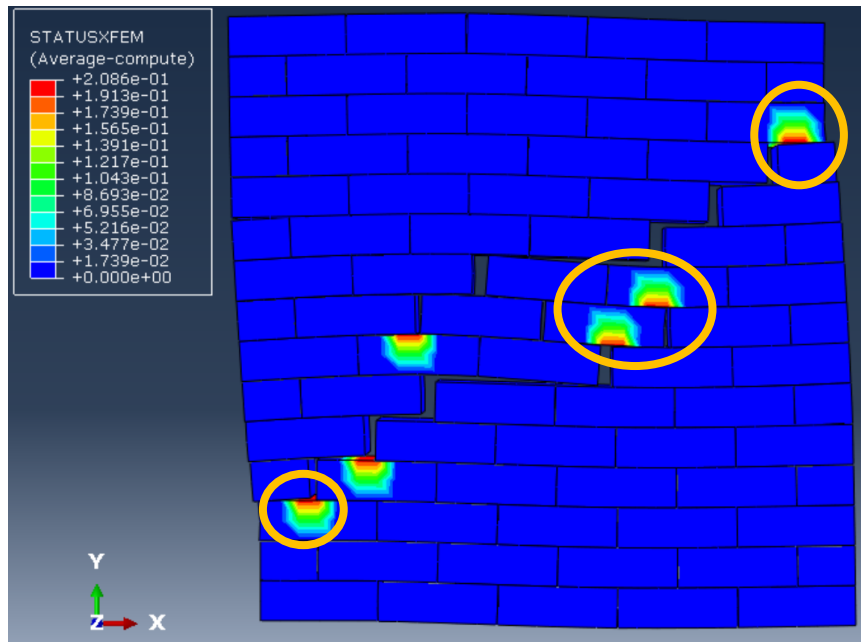
Based on the material properties of masonry shown in table F.2 and F.3 simulation was conducted. It is interested to compare failure pattern of DIC analysis and FEM model. The result for clay brick panels is presented in figure F.6. The deformed scale for FEM model adjusted as 20 to exaggerate failure pattern. The FEM model shown similar trend with DC-CBW2 so both results shown diagonal crack. One noted that unit crack observed in DC-CBW2 and it highlighted in yellow circle in figure F.6 (a). Surprisingly, the XFEM also captured same location of unit crack. Figure F.6 (c) shown the statusXFEM and it illustrated the partial crack unit element.



(a)DC-CBW2



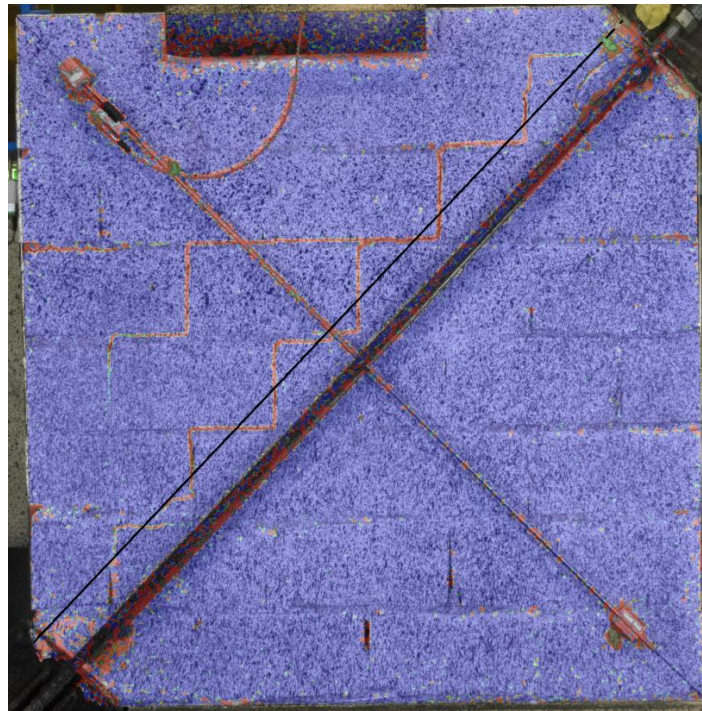
(b)Clay brick FEM σ_{III}



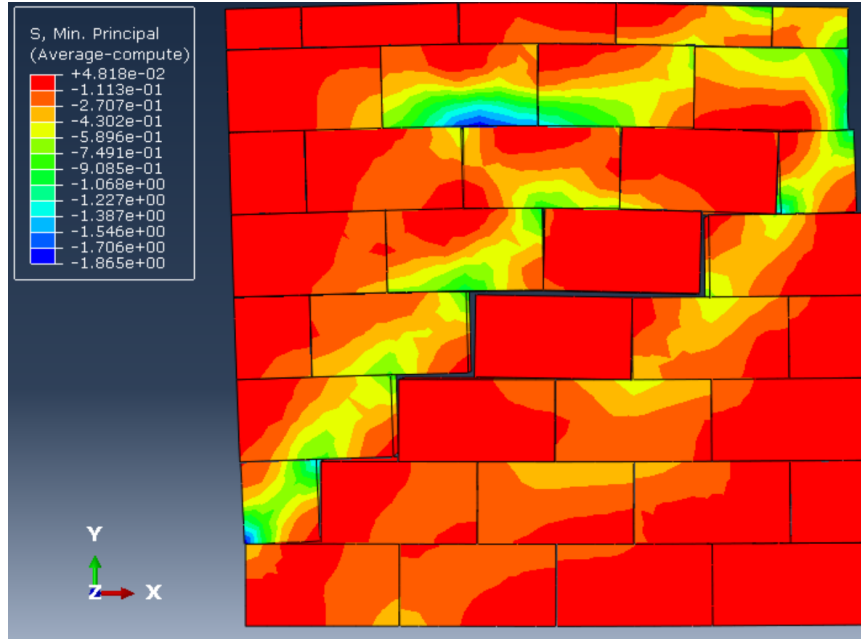
(c)Clay brick FEM StatusXFEM

Figure F.6: Crack pattern of FEM and experiment result for clay brick panel

FEM model for sandstone and DIC analysis also compared in figure F.7. The numerical result has similar failure pattern with DC-SSW1 and both of cracks are propagated through diagonal direction. The sandstone also applied XFEM but there was no crack observed in experiment and numerical model.



(a)DC-SSW1



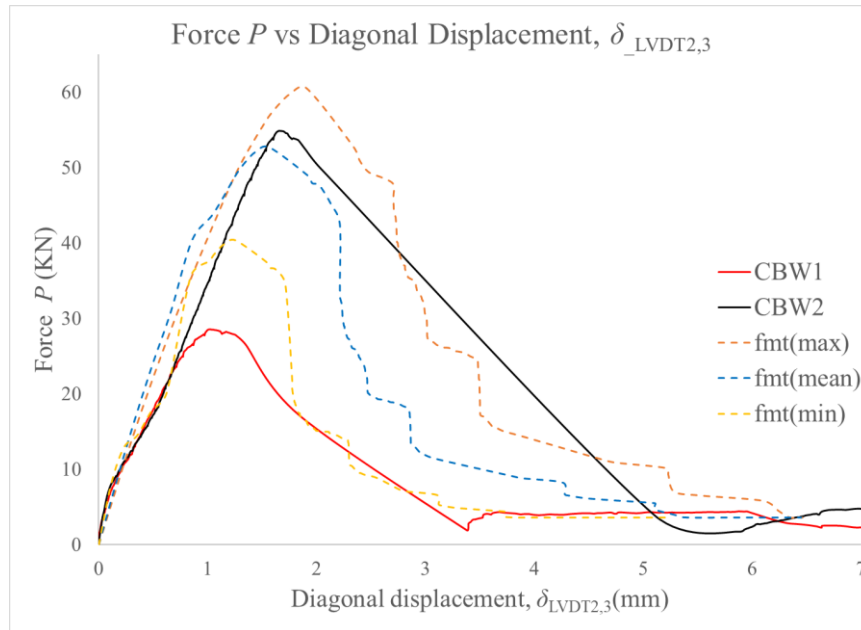
(b) Sandstone FEM σ_{III}

Figure F.7: Crack pattern of DIC and FEM result

The diagonal tensile strength, f_t is can be determined by ASTM, RILEM standard and proposed equation by DIC analysis as expressed in Eq: F.14. More research is required to better understand the influence tensile strength of joint mortar, f_{mt} so force displacement response plotted in figure F.8, F.9. Due to the large variation of f_{mt} which was obtained from bond wrench test, the numerical model plotted $f_{mt(max)}$, $f_{mt(mean)}$ and $f_{mt(min)}$ under constant cohesion.

The key interested parameters include diagonal tensile strength addressed by ASTM, RILEM code and DIC analysis and force displacement response presented in figure F.8, F.9. The response was plotted based on average of LVDT 2,3 which is diagonal compressive displacement and the response of numerical mode plotted from reference point which on the masonry panel. As shown in figure F.8 DC_CBW2 has similar response compared to $f_{mt(mean)}$. In other words, the mean of tensile strength of joint mortar is well represented behaviour of clay brick panel under diagonal compression. However, the ultimate load for DC_CBW1 is even lower than $f_{mt(min)}$ and it may be affected by oil content along joint interface as shown in figure C.5. The experiment of diagonal compression test imposed load control so the post behaviour is not true force displacement response. Surprisingly, no significant differences have been observed in figure F.8 (a). Similarly, FEM result for sandstone panel was also plotted in figure F.9. Both of force and displacement responses for DC_SSW1, DC_SSW2 are within the range of $f_{t(min)}$ and $f_{t(max)}$. In addition, the responses of experimental and numerical model did not observe notable difference. Therefore, based on the findings, to

be able to establish the actual post behaviour of diagonal compression test, further work will be required by conducting diagonal compression test under displacement control.

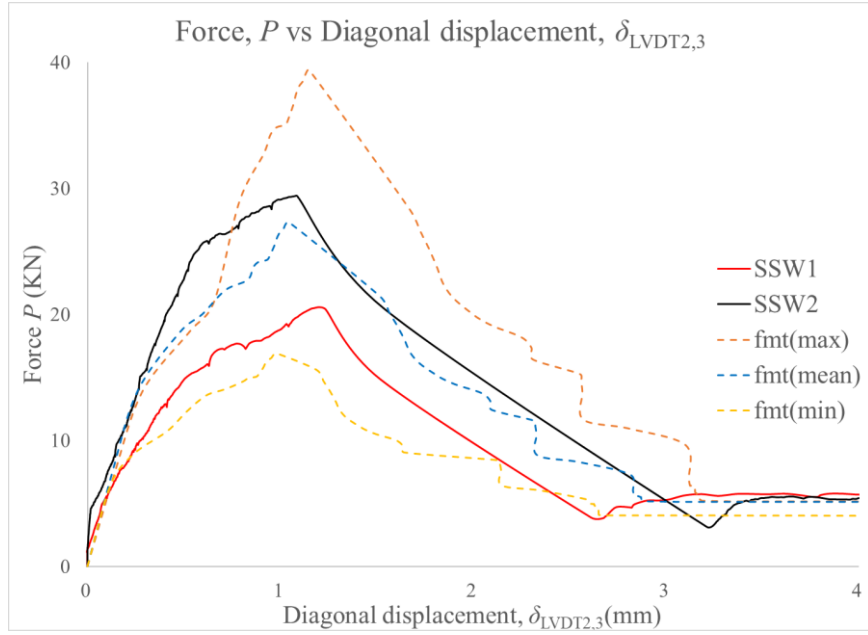


(a) Force vs displacement response for clay brick wall

Diagonal compression test and FEM result for clay brick wall					
	Experiment		FEM		
	DC_CBW1	DC_CBW2	$f_{mt}(\text{Max})$	$f_{mt}(\text{Mean})$	$f_{mt}(\text{Min})$
P_{max} (KN)	28.52	54.84	60.52	53.67	40.39
f_{t_ASTM} (MPa)	0.15	0.29	0.32	0.29	0.21
f_{t_RILEM} (MPa)	0.11	0.21	0.23	0.20	0.15
f_{t_DIC} (MPa)	0.12	0.24	0.26	0.23	0.18

(b) Diagonal tensile strength, f_t for clay brick panel

Figure F.8: Ultimate load and diagonal tensile strength of clay brick wall



(a) Force vs displacement response for sandstone wall

Diagonal compression test and FEM result for sandstone wall					
	Experiment		FEM		
	DC_SSW1	DC_SSW2	$f_{mt}(\text{Max})$	$f_{mt}(\text{Mean})$	$f_{mt}(\text{Min})$
P_{max} (KN)	20.57	29.41	39.39	27.35	16.90
f_{t_ASTM} (MPa)	0.11	0.16	0.21	0.15	0.091
f_{t_RILEM} (MPa)	0.078	0.11	0.15	0.10	0.064
f_{t_DIC} (MPa)	0.090	0.13	0.17	0.12	0.074

(b) Diagonal tensile strength, f_t for sandstone panel

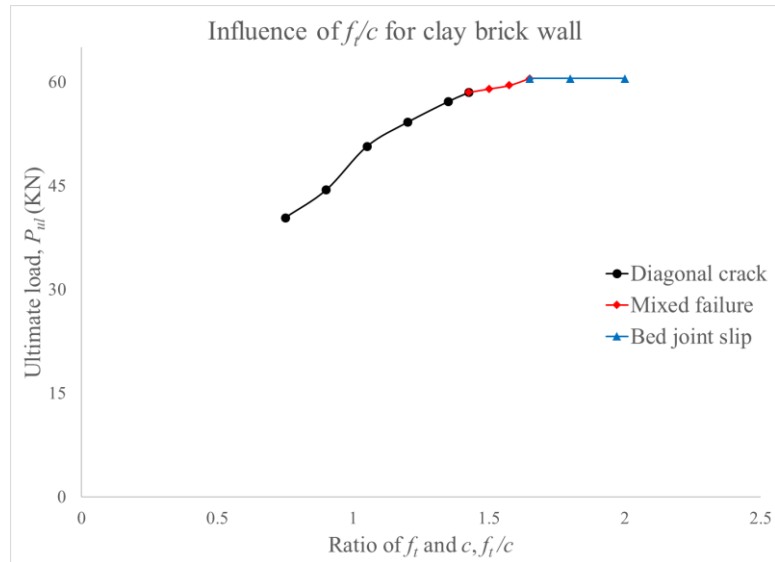
Table F.5: Ultimate load and diagonal tensile strength of sandstone wall

F.2.4 Influence of f_i/c on ultimate load, P_{ul} and failure pattern

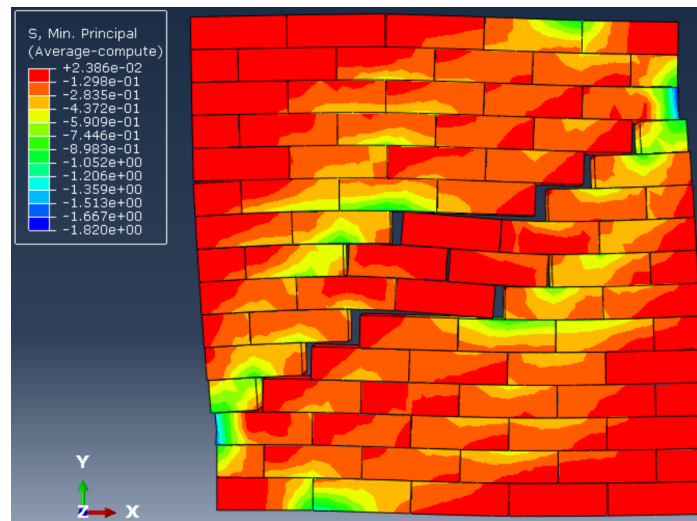
The parametric study conducted to investigate influence of f_i/c in inelastic analysis. Previously the influence of f_i/c which is associated with coefficient of α in Eq. F.14 is studied by Brignola and the author adopted non elastic analysis proposed by Calderini. On the other hand, the proposed constitutive model does not take into account the bond strength of head joint. This may affect the failure pattern in numerical model, so the current study of nonlinear analysis has been undertaken with bed and head joint interface by damage criterion.

There were three different failure modes observed in numerical model. Namely, diagonal crack, mixed failure and bed joint slip were found and illustrated in figure F.10, F.12. In particular, the ultimate load, P increase significantly up to $f_i/c=1.4$ and after that it become slightly increase until the failure pattern changes

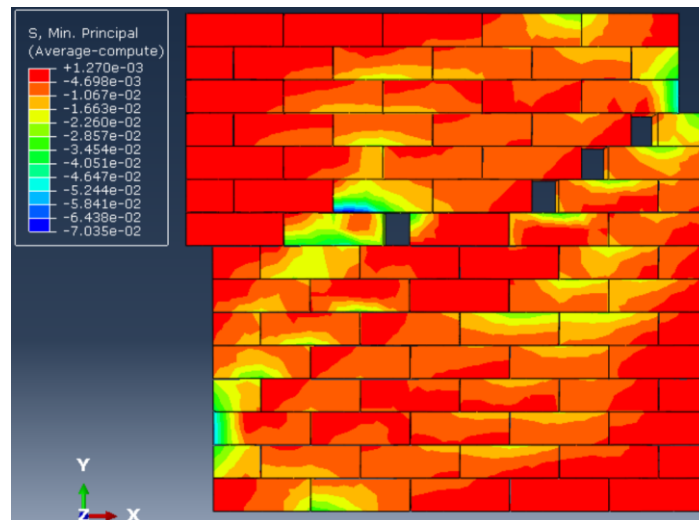
into bed joint slip. Same trend of influence f_t/c observed for clay brick and sandstone. It is noted that the reason P does not increase in bed joint slip failure is because the failure mainly governs by cohesion.



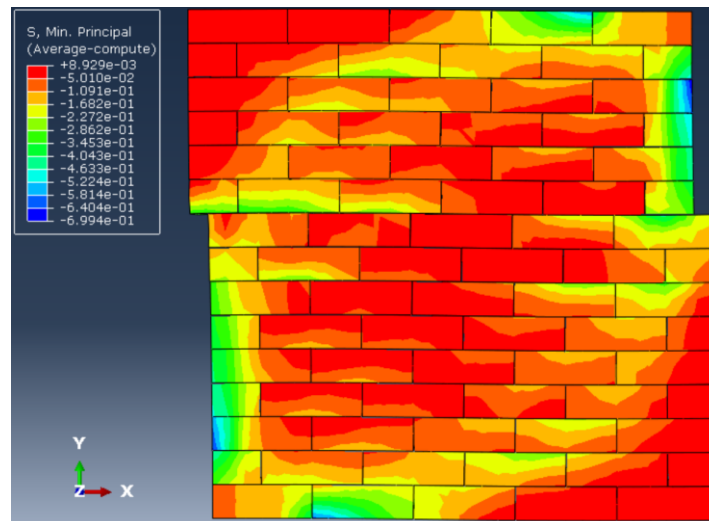
(a) f_t/c vs P_{ul} in different failure modes



(a) Diagonal crack

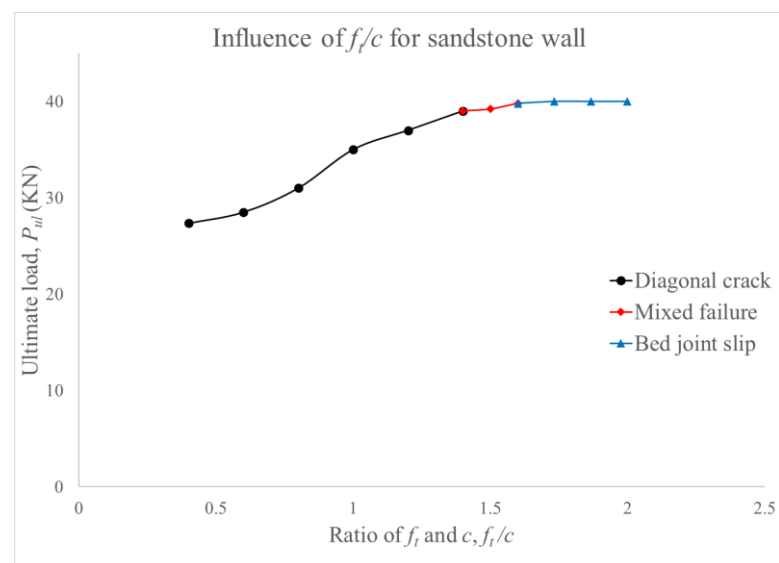


(b)Mixed failure

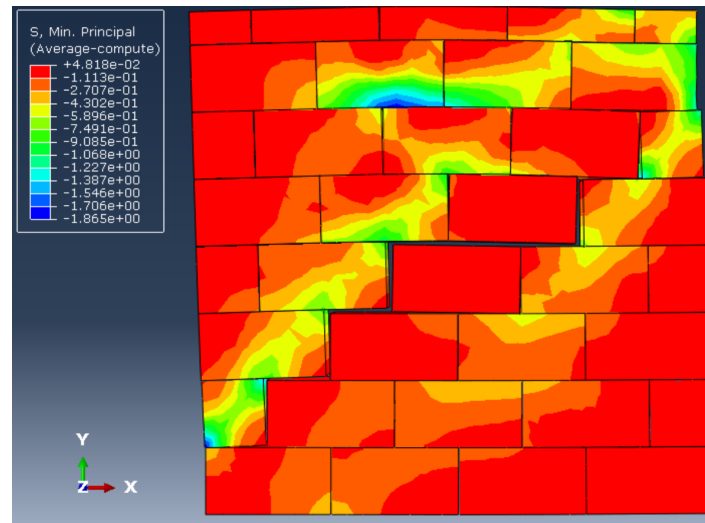


(c)Bed joint slip

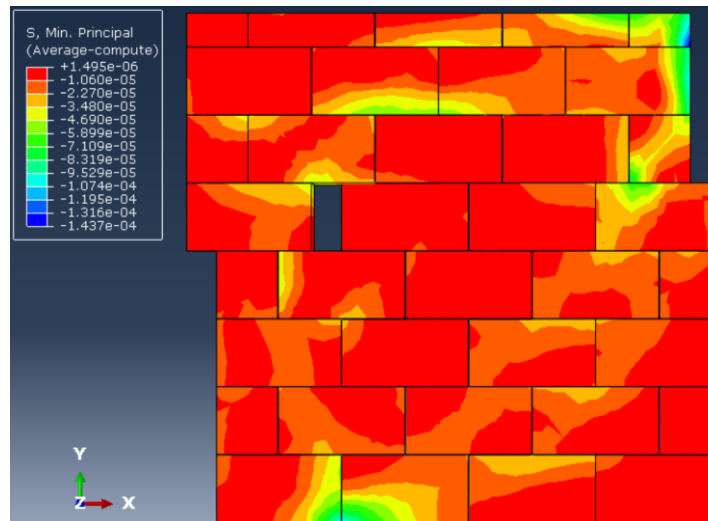
Figure F.10 : Influence of f_t/c and failure pattern for clay brick wall



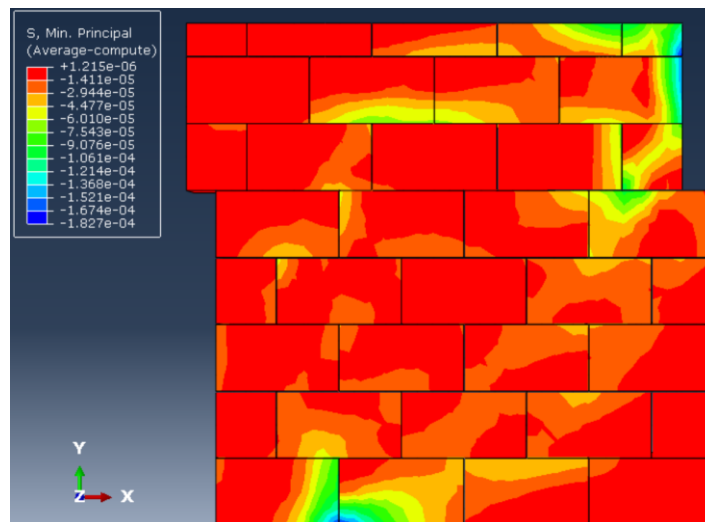
(a) f_t/c vs P_{ul} in different failure modes



(a) Diagonal crack



(b) Mixed failure



(c) Bed joint slip

Figure F.11 : Influence of f_t/c and failure pattern for sandstone wall

Based on the parametric study, ultimate load and failure pattern was mainly governed by f_t/c . Furthermore, the P can be taken to determine diagonal tensile strength f_t by proposed equation Eq: F.14. Normalized diagonal tensile strength, f_{t_DIC} plotted with f_t/c as illustrated in figure F.12.

The f_t/c was calculated out based on the range of flexural tensile strength obtained from bond wrench test so the range of f_t/c for clay brick and sandstone are different, nevertheless, it has similar response. The diagonal tensile strength f_{t_DIC} increased considerably up to $f_t/c=1.0$ and after that it has shown slight increase until $f_t/c=1.4$. Beyond the $f_t/c=1.4$ the failure mode changes as mixed failure. Therefore, current research paper derived coefficient factor α by DIC analysis to include variation of bond strength and found relationship between f_{t_DIC} and f_t/c . For future work, it would be interested to investigate irregular masonry panel such as rubble wall.

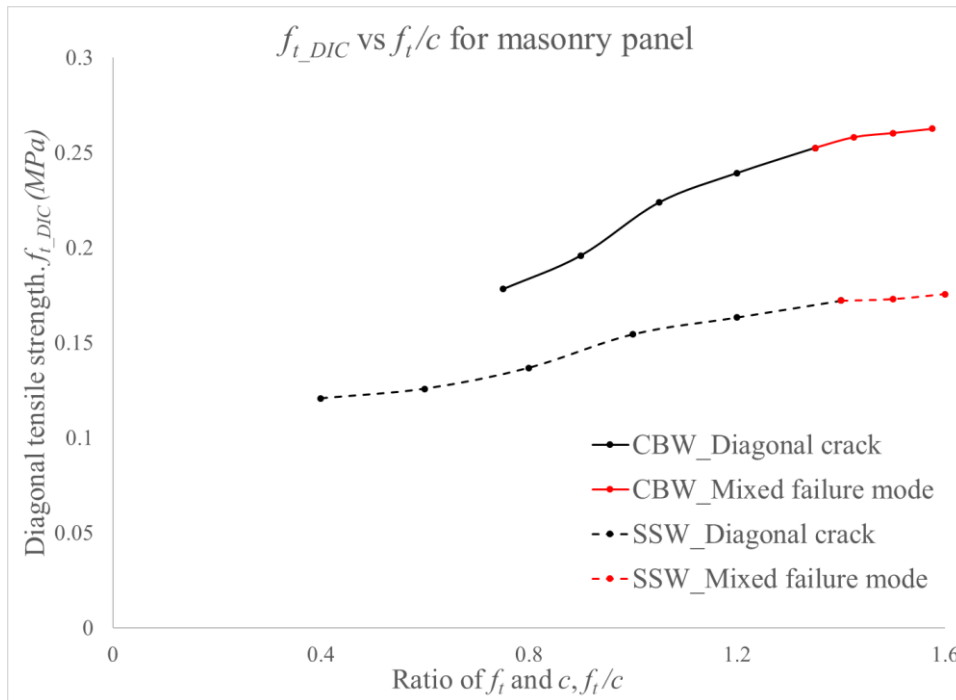


Figure F.12: Diagonal tensile strength, f_{t_DIC} vs f_t/c for masonry panel

REFERENCES

- Abaqus analysis user's manual, 6.13-3. RI, USA: Dassault Systems Providence 2013.
- Abdulla, Kurdo & Cunningham, Lee & Gillie, Martin. (2017). Simulating masonry wall behaviour using a simplified micro-model approach. *Engineering Structures*, 151. 10.1016
- Ali. Laftah. Abbas and Maan. H. Saeed., Representation of The Masonry Walls Techniques By Using FEM. *Aust. J. Basic & Appl. Sci.*, 11(13): 39-48, 2017
- Angelillo, Maurizio & Lourenco, Paulo & Milani, Gabriele. (2014). Masonry behaviour and modelling. *Mechanics of masonry structures*. Springer; 2014. P. 1-26, 10.1007/978-3-7091-1774-3_1.
- Anna Brignola, Sara Frumento, Sergio Lagomarsino & Stefano Podestà (2008) Identification of Shear Parameters of Masonry Panels Through the In-Situ Diagonal Compression Test, *International Journal of Architectural Heritage*, 3:1, 52-73, DOI: 10.1080/15583050802138634
- AS 3700-(2018) Masonry structures, Australian standard
- ASTM C1006-07(2013), Standard Test Method for Splitting Tensile Strength of Masonry Units, ASTM International, West Conshohocken, PA, 2013
- ASTM C67-03a, Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile, ASTM International, West Conshohocken, PA, 2003
- ASTM E519 / E519M-15, Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages, ASTM International, West Conshohocken, PA, 2015
- ASTM C1314-16, Standard Test Method for Compressive Strength of Masonry Prisms, ASTM International, West Conshohocken, PA, 2016
- Attard, M. M., Chaimoon, K., and Nappi, A. (2004). "Fracture of unreinforced masonry walls under shear." *Proceedings CDROM of the 6th World Congress on Computational Mechanics (WCCM VI) in conjunction with the 2nd Asian Pacific Congress on Computational Mechanics (APCOM'04)*, Beijing, China, 4341-4346.
- Aref, Amjad & Dolatshahi, Kiarash. (2013). A three-dimensional cyclic meso-scale numerical procedure for simulation of unreinforced masonry structures. *Computers & Structures*. 120. 9–23. 10.1016/j.compstruc.2013.01.012.
- Barenblatt GI. The mathematical theory of equilibrium cracks in brittle fracture. *Adv Appl Mech* 1962;7(55–129):104

Bazant ZP and Pfeiffer PA (1986). Shear Fracture Tests of Concrete. *Materials and Structures*. 19. 111-121. 10.1007/BF02481755.

Belytschko, T., and T. Black, "Elastic Crack Growth in Finite Elements with Minimal Remeshing," *International Journal for Numerical Methods in Engineering*, vol. 45, pp. 601–620, 1999.

Benzeggagh, M. L., and M. Kenane, "Measurement of Mixed-Mode Delamination Fracture Toughness of Unidirectional Glass/Epoxy Composites with Mixed-Mode Bending Apparatus," *Composites Science and Technology*, vol. 56, pp. 439–449, 1996

Bocca, Pietro & Carpinteri, Alberto & Valente, S. (1989). Fracture mechanics of brick masonry: size effects and snap-back analysis. *Materials and Structures*. 22. 364-373. 10.1007/BF02472507.

Bolhassani, Mohammad & A. Hamid, Ahmad & C.W. Lau, Alan & Moon, Franklin. (2015). Simplified micro modeling of partially grouted masonry assemblages. *Construction and Building Materials*. 83. 10.1016

Bolhassani, M., et al. (2016). Damage detection of concrete masonry structures by enhancing deformation measurement using DIC. *Nondestructive Characterization and Monitoring of Advanced Materials, Aerospace, and Civil Infrastructure 2016*.

Calderini C., and Lagomarsino S. 2008. Continuum model for in-plane anisotropic inelastic behaviour of masonry. *Journal of Structural Engineering ASCE* 134(2): 209–220.

Camanho, P. P., and C. G. Davila, "Mixed-Mode Decohesion Finite Elements for the Simulation of Delamination in Composite Materials," NASA/TM-2002–211737, pp. 1–37, 2002.

Dugdale DS. Yielding of steel sheets containing slits. *J Mech Phys Solids* 1960;8(2):100–4.

EN 1926 : 2006, Natural stone test methods. Determination of uniaxial compressive strength, British Standards Institution

EN 14580(2005), Natural stone test methods. Determination of static elastic modulus, European Standard

EN 1015-11(1999), Methods of test for mortar for masonry. Determination of flexural and compressive strength of hardened mortar, European Standard

EN 772-1, Methods of test for masonry units Part 1: Determination of compressive strength, European Standard

Ewing, B. D., and Kowalsky, M. J. (2004). "Compressive behavior of unconfined and confined clay brick masonry." *J. Struct. Eng.*, 130(4), 650–661.

- Frocht M. M., and Arbor A. 1931. Recent advanced in photoelasticity. Transaction, ASME, 55:135–153.
- Ghorbani, Rahim & Matta, Fabio & Sutton, Michael. (2015). Full-Field Deformation Measurement and Crack Mapping on Confined Masonry Walls Using Digital Image Correlation. *Experimental Mechanics*. 55. 227-243. 10.1007/s11340-014-9906-y.
- Guinea, Gustavo & Hussein, G & Elices, M & Planas, Jaime. (2000). Micromechanical modeling of brick-masonry fracture. *Cement and Concrete Research*. 30. 731-737. 10.1016/S0008-8846(00)00228-3.
- Howlader, Milon & J Masia, M & Griffith, M & W Jordan, J. (2018). Design of in-plane unreinforced masonry wall testing program and preliminary finite element analysis (FEA). Australian masonry conference.
- Huu-Luyen Nghiem, Marwan Al Heib, Fabrice Emeriault, (2015). Method based on digital image correlation for damage assessment in masonry structures. *Engineering Structures*. Pages 1-15, <https://doi.org/10.1016/j.engstruct.2014.12.021>.
- Kaushik, Hemant & C. Rai, Durgesh & K. Jain, Sudhir & Asce, M. (2007). Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression. *Journal of Materials in Civil Engineering - J MATER CIVIL ENG*. 19. 10.1061/(ASCE)0899-1561(2007)19:9(728).
- Kumar, Nitin & Rajagopal, Amirtham & Pandey, Manoj. (2014). Plasticity based approach for failure modelling of unreinforced masonry. *Engineering Structures*. 80. 40–52. 10.1016/j.engstruct.2014.08.021.
- Lapidus, L. and Pinder, G.F., *Numerical solution of partial differential equations in science and engineering*. John Wiley & Sons, New York (1982).
- Lourenco, Paulo. (1996). Computational Strategy for Masonry Structures.
- Lourenco, P. B., and Rots, J. G. (1997). "Multisurface interface model for analysis of masonry structures." *Journal of Engineering Mechanics*, 123(7), 660-668.
- Mann, W., and Müller, H. (1982). "Failure of shear-stressed masonry- An enlarged theory, tests and application to shear walls." *Proceedings of the British Ceramic Society* 223-235.
- Melenk, J., and I. Babuska, "The Partition of Unity Finite Element Method: Basic Theory and Applications," *Computer Methods in Applied Mechanics and Engineering*, vol. 39, pp. 289–314, 1996.
- Milani G, Lourenço P, Tralli A. Homogenization approach for the limit analysis of out-of-plane loaded masonry walls. *J Struct Eng* 2006;132(10):1650–63.

- Mosalam K, Glascoe L, Bernier J. Mechanical properties of unreinforced brick masonry section-1. Documented to US Department of Energy by Lawrence Livermore National Laboratory; 2009.
- Pan, Bing & Qian, Kemao & Xie, Huimin & Asundi, Anand. (2009). TOPICAL REVIEW: Two-dimensional digital image correlation for in-plane displacement and strain measurement: a review. *Measurement Science & Technology - MEAS SCI TECHNOL*. 20. 10.1088/0957-0233/20/6/062001.
- Pluijm, van der, R. (1999). Out-of-plane bending of masonry: behaviour and strength Eindhoven: Technische Universiteit Eindhoven DOI: 10.6100/IR528212
- Ramos, T., et al. (2015). "2D and 3D Digital Image Correlation in Civil Engineering – Measurements in a Masonry Wall." *Procedia Engineering* 114: 215-222.
- Raijmakers T, Vermeltfoort AT. Deformation controlled tests in masonry shear walls; 1992.
- Remmers, J. J. C., R. de Borst, and A. Needleman, "The Simulation of Dynamic Crack Propagation using the Cohesive Segments Method," *Journal of the Mechanics and Physics of Solids*, vol. 56, pp. 70–92, 2008.
- RILEM FMC-50: Determination of the fracture energy of mortar and concrete by means of threepoint bend tests on notched beams. *Materials and Structures* 18(4) 1985; p. 287-290.
- RILEM TC. 76-LUM. Diagonal tensile strength tests of small wall specimens, 1991. In RILEM, *Recommendations for the Testing and Use of Constructions Materials*. London: E&FN SPON, 488–489
- RILEM TC 187-SOC: Experimental Determination of the Stress-Crack Opening Curve for Concrete in Tension. Final report of RILEM Technical Committee TC 187-SOC; 2007.
- Salmanpour, A. H. and N. Mojsilovic (2013). Application of Digital Image Correlation for strain measurements of large masonry walls. The 5th Asia Pacific congress on computational mechanics Singapore 11-14th December, 2013
- Xiao, X & Shrive, N.G 2004, 'Mixed mode fracture of brick', in 13th International Brick and Block Masonry Conference Amsterdam, July 4-7, 2004
- H.R. Kumavat, An experimental investigation of mechanical properties in clay brick masonry by partial replacement of fine aggregate with clay brick waste, *J. Inst. Eng. India Ser. A* 97 (3) (2016) 199–204
- Thaickavil, N N & Thomas, Job. (2017). Behaviour and strength assessment of masonry prisms. *Case Studies in Construction Materials*. 8. 10.1016/j.cscm.2017.12.007.
- Frocht M. M., and Arbor A. 19ur31. Recent advanced in photoelasticity. *Transaction, ASME*, 55:135–153.