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School of Mathematics, Computer Science & Engineering

Improved Interface Bond Relationships of FRP-
Strengthened Masonry Structures through Experimental
and Numerical Simulations

PhD Thesis for the degree
of Doctor of Philosophy in Structural Engineering

by

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London December 2022

Abstract

Masonry walls are vulnerable to extreme loading and fibre-reinforced polymer (FRP) composites are a good solution for the reinforcement of un-reinforced masonry (URM) walls exposed to in-plane or out-of-plane loads. Collapse of walls in masonry structures is one of the main reasons for material damage and people's injury due to earthquakes. Thus, the investigation of effective and inexpensive methods for strengthening masonry walls is crucial. FRP composites have many benefits, such as a high strength-to-weight ratio, and ease of application in terms of workforce and time. The behaviour of masonry walls can be improved by retrofitting with fibre reinforced polymeric (FRP). Reinforcement of masonry walls with FRP, can increase strength and ductility and thus reduce the risk of failure under extreme loading. However, the reinforcement of cracked masonry walls and the interface and bond behaviour between brick and mortar, especially in the case of retrofitting with FRP, has not been carefully studied.

The main aim of this research is to investigate the bond behaviour of different carbon fibre-reinforced polymer (CFRP) configurations on masonry structures when subjected to compressive and shear loads. Also, the experimental shear tests on cracked triplet masonry strengthened with different configurations of CFRP were carried out which had not been studied before. The experimental work was carried out with seven different CFRP configurations and layers on triplet masonry and each test was repeated four times. In addition, finite element modelling by the commercial software Ansys and verification against results from experimental tests were carried out.

The present study has shown the strength of masonry triplets with CFRP under compression loads was increased up to 3 times and under shear loads, all types of reinforced triplets were at least 4 to 6 times stronger than triplets without CFRP.

Comparing one and two layers of CFRP wrap or CFRP strips on triplet masonry showed that there is no significant bond improvement between one layer and two layers. Depending on CFRP configurations, two layers make triplets 5% to 15% stronger, which is not sufficient. The cracked triplet samples repaired with CFRP and tested under shear loading showed significant improvement in bond strength and the load capacity of triplet masonry increased by 3 to 5 times in comparison with samples without initial damage.

Finally, based on test results to predict the shear strength of masonry triplets with and without CFRP new formulas for bond behaviour were developed. In addition, for an accurate finite element modeling (FE), these formulas were added as the new user-defined functions (UDF), in the commercial software Ansys. The outcome of this thesis improves the current knowledge on the use of CFRP to reinforce masonry walls with brick and mortar which will contribute to the understanding of the effect of CFRP on masonry structures. Conclusions are provided and future research needs in the area of masonry strengthening with CFRP are outlined.

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List of Abbreviation

FRP	Fibre-reinforced polymer
URM	Unreinforced masonry
URTM	Unreinforced triplet masonry
CFRP	Carbon fibre reinforced polymers
GFRP	Glass fibre reinforced polymers
BFRP	Basalt fibre reinforced polymers
UDL	Uniformly distributed load
UDF	User defined function
AFRP	Aramid fibre reinforced polymer
Tb	The in-plane shear strength at the failure surface
To	The shear strength at zero normal stress
μ	Coefficient of friction between units and mortar
σ_y	Normal stress
σ_b	Normal bond stress
f_{vko}	Shear strength under zero compressive strength
σ_d	Design compressive stress perpendicular to the shear
f_v	Shear strength for URM walls
RC	Reinforced concrete coat
NSM	Near-surface-mounted

RCM	Reinforced concrete masonry
f_k	Characteristic compressive strength of masonry [N/mm ²]
K	Constant, which is a function of the type of masonry units and mortar
f_b	Normalised average compressive strength of units [N/mm ²]
f_m	Average compressive strength of mortar [N/mm ²]
E	Modulus of elasticity of masonry [N/mm ²]
KE	Constant equal to 1000 according to UK National Annex to EC 6
f_k	Characteristic compressive strength of masonry [N/mm ²]
t_m	Thicknesses of mortar
t_b	Thicknesses of brick
E_{mr}	Elastic modulus of mortar
E_b	Elastic modulus of brick
ν_m	The Poisson's ratio of mortar
ν_b	The Poisson's ratio of brick
E_b	Elastic modulus of brick
t_{hb}	Thickness of brick units
t_{hm}	Thickness of mortar joints
f_m	Compressive strength of masonry walls
E_m	Elastic modulus of masonry sample

Chapter 1

Introduction

1.1. Background

Most current buildings are made up of masonry and a big number of investigations conducted on masonry confirmed the workability of this material for many applications. Most of the recent studies have focused on improving the safety of new buildings constructed with masonry. Unreinforced masonry (URM) buildings are vulnerable to lateral loads, these lateral loads are transferred to the foundation through load-bearing walls. These buildings may be subjected to in-plane or out-of-plane loads. (Ehsani and Saadatmanesh, 1997). URM walls may crack, tear and collapse under the out-of-plane, in-plane, and vertical loads caused by earthquakes, as shown in Figures 1.1A, B, C, and D.



Figures 1.1 A and B: Sarpole Zahab 2017, Kermanshah earthquake, IRAN: by Alireza Vasigh Ansari



Figure 1.1 C and D: IRAN earthquake 2017- Sarpole Zahab, 2017 Kermanshah earthquake, IRAN, by Farzad Menati (2017)

According to (Ehsani and Saadatmanesh, 1997) Masonry buildings consist of several structural element types and these elements designed to resist vertical (axial) loads. However, they are often subjected to in-plane or out-of-plane loads resulting from lateral loads such as earthquakes. Many of these buildings unreinforced and mortar joints have very low shear capacity. Under earthquake loads the shear capacity of mortar is exceeded, resulting in the failure of the walls. Thus, effective reinforcement methods are required to increase the resistance of masonry walls. Sometimes, insufficiencies in design and calculations, use of cheap materials, and construction problems may be found unsatisfactory and need improvements. To make stronger masonry walls, sometimes, complete or partial demolition and reconstruction are not possible. In such cases, Fibre Reinforced Polymer (FRP) composites are excellent solutions for the reinforcement of URM walls subjected to overstress loading. The reinforcement methods based on traditional materials, such as steel and concrete often add extra load to the structures and make them riskier. Also, they are quite expensive (Ehsani and Saadatmanesh, 1997). The reinforced concrete (RC) jacket technique for strengthening masonry structures consists of the application of jackets on one or both sides of masonry walls. The steel reinforced brick masonry consists of brick masonry which has Steel reinforcement or mild steel iron mesh or bars embedded in the mortar.

The interface behavior of brick-mortar in masonry composites has been investigated by Mohammadipour, (2014). In his experimental program to evaluate the behavior of the interface under shear loading, a Digital Image Correlation, DIC, system was employed to measure the surface deformation and failure modes of unreinforced masonry composites during the test. Figure 1.2 illustrates the deformation field of a triplet specimen captured by the DIC system. The red colour presents strain concentration on the inner interface. This indicates a vertical crack through its height and it is due to tensile debonding or shear delamination or a combination of them.

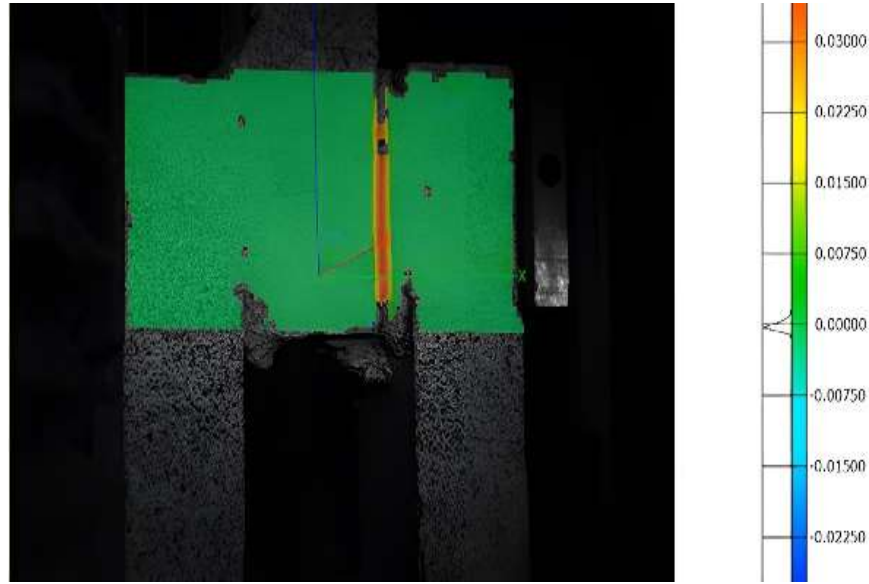


Figure 1.2: The vertical crack through its height, Mohammadi pour, (2014).

There are many masonry structures constructed from stone, brick, or their combination, and these buildings were usually constructed from brick units and lime/cement or cement mortar. Behavior composite material such as CFRP depends on the mechanical properties of the components, the workmanship, and the details of connections to other structural elements.

Many researchers studied the behavior of FRP-strengthened triplet masonry in terms of strength and load-displacement relationship with one or two FRP configurations.

The idea is to improve the shear and compression resistance of the structure by increasing its strength and ductility. Also, repair cracked masonry walls with CFRP which has not been studied before.

In this research, strength, and bond stress-slip relationships of unreinforced triplet masonry (URTM) with eight CFRP configurations and with more than one layer and the interface behavior between brick and mortar were studied.

1.2. Aims and objectives

The objective of this research is to experimentally and analytically investigate the effectiveness of CFRP on current masonry structures, evaluate the compressive and shear capacity of CFRP-strengthened triplets and compare URM and FRP externally reinforced masonry triplets. The experimental program has been developed as a small-scale experiment. This research improves essential data for engineers who are involved in the reinforcement or repair of existing masonry structures. The finite element (FE) models are developed and validation of these models in comparison with experimental tests is conducted.

1.2.1. The aim and objectives:

- 1 - Study the effect of CFRP on the bond's behaviour between brick and mortar in masonry structures that has not been studied before.
- 2 - Explore the interaction between seven different configurations of FRP on triplet masonry to increase the capacity and ductility of masonry walls when subjected to compressive and shear loads.
- 3 - Develop an analytical model based on experimental tests to compute shear and compression capacity of retrofitted masonry triplets and compare its results with the experimental database.
- 4 - Investigate the interaction between different CFRP configurations to strengthen cracked triplet masonry (CTM).
- 5- Increase the capacity and ductility of the cracked triplet masonry (CTM) when subjected to shear loading and experimentally and analytically investigate the effectiveness of CFRP on existing cracked masonry structures.

1.2.2. Research significance

While masonry is one of the oldest construction materials, its behavior has not been investigated as extensively as other construction materials. An alternative solution rather than demolish-rebuild culture for the masonry is to use FRP composites to reinforce masonry structures.

Based on the experimental and analytical outcomes, this study will contribute to the efforts of the worldwide research community by providing a deeper understanding of the behavior of CFRP, in particular its ability to increase the shear and compression capacity of the masonry walls. This study generates the required data for engineers who are involved in the field of repairing and strengthening of existing masonry structures damaged by extreme loading.

1. 3. Methodology

Firstly, the literature review was carried out aiming to better understand the behaviour of the masonry walls and to gather the data associated with the past and recent experimental and numerical procedures for unreinforced and strengthened masonry walls.

Secondly, experimental tests were carried out after the determination of the needed parameters. Brick units, mortar, and CFRP were selected as the main materials to be tested. The experimental work was divided into four parts: (1) mechanical identification and evaluation of the mechanical properties of brick masonry and its components; (2) evaluation of the behaviour of unreinforced and strengthened triplet masonry subjected to compression and shear loads; (3) experimental tests on reinforced cracked triplet masonry subjected to shear loads; (4) evaluation of the behaviour of strengthened triplet masonry with more than one layers CFRP under shear loads.

Thirdly, the validation and comparison of the available design methods and analytical models with the obtained experimental results were carried out. The goal was to test and suggest a design model for reinforcing masonry walls with CFRP composites.

1.4. Outline of the thesis

The thesis is divided into eight chapters:

Chapter 1: Presents abstract, the background and introduction to the topic, defines research problem, states the aim and objectives, the method of investigation used in the research project, and outlines.

Chapter 2: The literature review refers to the experience of damage to masonry houses during the earthquakes in Turkey, Iran, and Chile and reviews the previously published studies in the field of masonry structures. The fundamental characteristics of masonry as structural material and its elements and a short review of the types of masonry, types of CFRP composites, types of failures in masonry, and the design codes are also shown.

Chapter 3: This chapter describes the material properties used in the experimental work, mortar, brick and triplet masonry. Also, presents an experimental study on unreinforced and reinforced masonry triplets with one and two layers CFRP under compression load.

Chapter 4: Chapter 4 presents an experimental study on the shear behaviour of strengthened triplet masonry under shear loading to find the difference between one-layer and two-layers CFRP, experimental study on the shear behaviour of strengthened triplet masonry with a different configuration of CFRP and CFRP net. In addition, the new equations were developed to predict the maximum shear strength of reinforced triplet masonry and results were compared with experimental tests.

Chapter 5: Experimental shear tests on cracked triplet masonry strengthened with a different configuration of CFRP and comparison of stress, load and sliding of cracked triplets is presented in chapter 5.

Chapter 6: Presents finite element modelling for compression tests, shear loading tests and shear loading tests on cracked triplet masonry by Ansys and verification against results from experimental tests with different CFRP applications in this research. Also, the newly developed user-defined functions (UDF) for all CFRP configurations are shown.

Chapter 7: Finite element modelling for one layer of CFRP wrap and strips on two sides of the masonry wall is presented in chapter 8. This chapter shows Ansys with new UDF can be used to calculate the shear strength of bigger walls.

Chapter 8: In the final chapter a short summary and the main conclusions from each chapter are presented, followed by recommendations for further research.

Figure 1.3 illustrates this research flowchart.

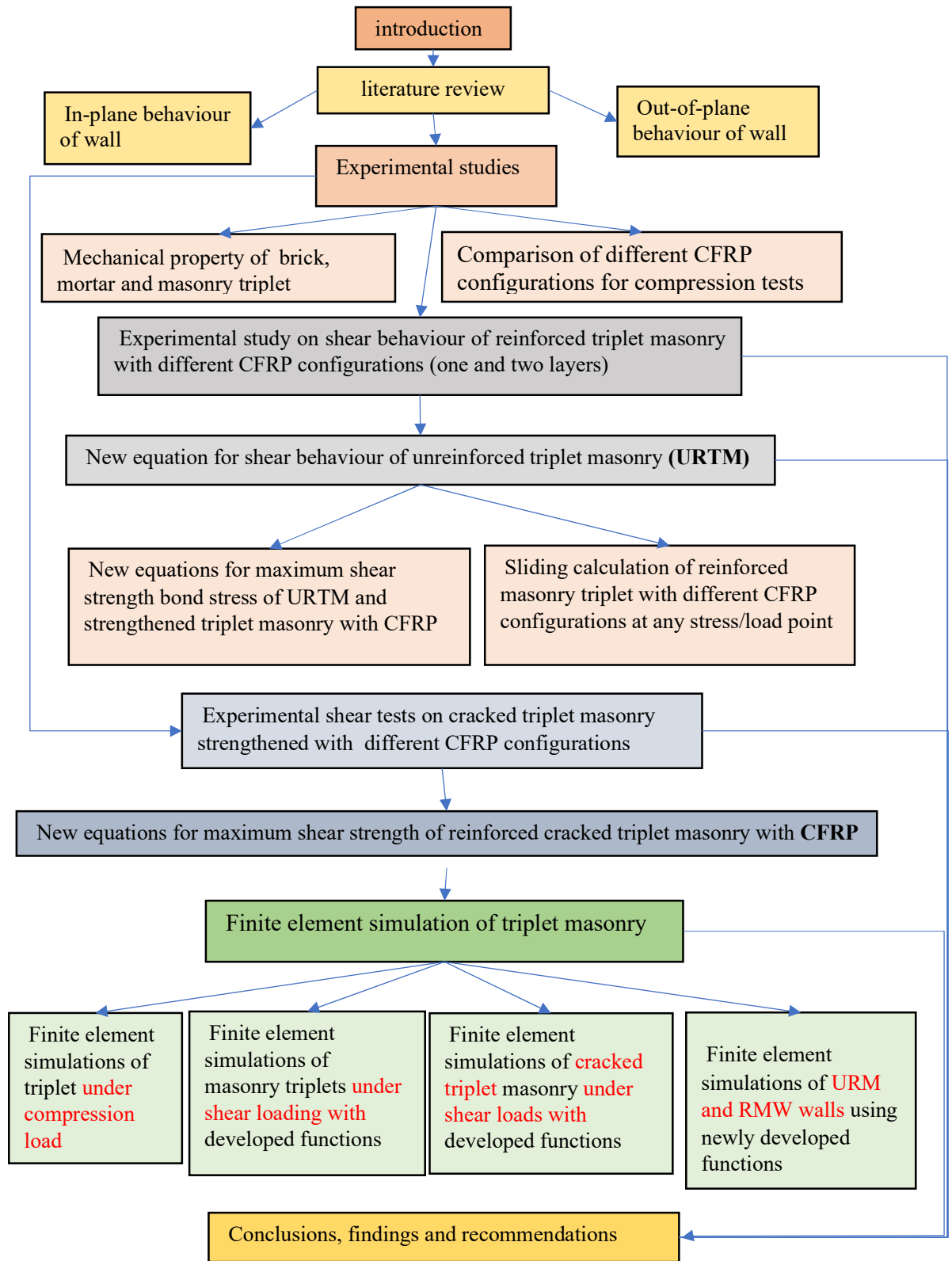


Figure 1.3: Research flowchart

Chapter 2

Literature review

2.1. introduction

Masonry is a complex structure with bricks or stones and mortar as the jointing material with excellent properties in terms of strength and price. The main advantage of masonry structures is that it is possible to use the same materials to perform a variety of applications (Hendry et al., 2004). Masonry structures are reasonably inexpensive and easy to build. URM walls are normally used as internal dividers or external walls. Depending on the design, these walls can resist lateral and/or gravity loads. Due to weak anchorage to concrete members, or the absence of anchorage, URM walls may crack, and collapse under the effects of out-of-plane and in-plane loads created by seismic forces, as shown in Figure 2.1. (Tumialan & Nanni, 2001).



Figure 2.1. The collapse of URM Walls, Turkey, 1999. (Tumialan & Nanni, 2001).

Fibre-reinforced polymer (FRP) offers answers for the reinforcement of URM walls exposed to over-stresses caused by seismic activity loads (Tumialan and Nanni, 2001). The failures of unreinforced masonry (URM) walls are one of the main causes of material damage and loss of human life due to seismic events (Secondin, 2003). Thus, the progress of effective and inexpensive retrofitting techniques for masonry members is crucial. Masonry constructions are simple and complex building materials in terms of mechanical properties. One of the most important elements in masonry construction are walls, which transfer horizontal and vertical loads such as wind and lateral forces. The

behavior of masonry walls can be improved by retrofitting with fiber-reinforced polymeric (FRP).

Reinforcing masonry walls with fiber Reinforced Polymers (FRP) has been considered to increase strength and ductility and thus reduce the risk of failure under dangerous loading conditions. Fiber-reinforced polymer (FRP) composites may offer technically and economically practical solutions (Tumialan and Nanni, 2001).

2.2. Masonry as a structure material

Due to the geometries and material properties of brickwork such as form, sizes, slenderness ratio, strengths and modulus of elasticity of materials, and water absorption, masonry, is considered an anisotropic composite material. The excellence of masonry buildings depends on the quality of the materials used. Therefore, all materials in masonry constructions must follow the standards. The main masonry components are block, brick, and mortar (Hendry et al, 1996).

2.3. Bricks

A brick is defined as a building materials unit with sizes (mm) not exceeding $337.5 \times 225 \times 112.5$ (L $\times$ w $\times$ t). Any unit with a dimension that goes beyond any one of those detailed above is called a block. Blocks and bricks are made of clay, calcium silicate, or concrete. These should follow related national standards, for example in the United Kingdom to BS 3921 (clay units), BS 187 (calcium silicate), and BS 6073: Part 1 (concrete units). In these standards, two classes of bricks are recognized, specifically common and facing; BS 3921 identifies a third category, engineering:

- Common bricks are good for general structure work.
- Facing bricks are used for external and internal walls.
- Engineering bricks are solid with compressive strength.

The frog type of London brick (Figure 2.2). The coordinate size and work size of bricks were according to BS 5628-1 (Table 2-1).



Figure 2.2. London brick (Hanson, 2010)

Coordinate size			Work size		
Length	Width	Height	Length	Width	Height
228	112.5	75	215	102.5	65

Table 2.1. Coordinate and work sizes for bricks (BS 5628-1)

2.4. Resistance of masonry walls

2.4.1. Compressive strength

The controlling factor of brick is compressive strength. Bricks with various strengths are available depending on architectural necessities. For low-rise buildings, bricks of 5.2 N/mm² should be sufficient. For damp-proof courses, low-absorption engineering bricks are usually required. For reinforced and pre-stressed brickwork, it is highly unlikely that brick strength lower than 20 N/mm² will be used in the UK. Table 2.2 shows the classification of bricks allowing the compressive strength (Hendry et al, 1996).

<i>Designation</i>	<i>Class</i>	<i>Average compressive strength not less than (N/mm²)</i>	<i>Average absorption (5 h boiling) not greater than (% by weight)</i>
Engineering	A	70	4.5
	B	50	7.0
Loadbearing brick		5–100	no specific requirement
Damp-proof course 1		5	4.5
Damp-proof course 2		5	7.0

Table 2.2 Classification of clay bricks according to compressive strength and absorption (Hendry et al, 1996).

2.4.2. Frost resistance

The resistance of bricks to frost is very variable and depends on the degree of exposure to driving rain and temperature. Engineering bricks with high compressive strength and low absorption are expected to be frost resistant. However, some bricks of low strength and high absorption may be resistant to frost compared to low-absorption and high-strength bricks. (Hendry et al, 1996).

2.4.3. Fire resistance

Clay bricks are subjected to very much higher temperatures during firing than they are likely to be exposed to in a building fire. As a result, they possess excellent fire resistance properties. Calcium silicate bricks have similar fire resistance properties to clay bricks. The type of loading, configuration of loading, and workmanship are other factors that affect the strength of masonry components.

2.5. Dimensional changes

(a) Thermal movement

All building materials expand or contract with the rise and fall of temperature.

(b) Moisture movement

One of the common causes of cracking and decay of building materials is moisture movement, which may be wholly or partly reversible or, in some circumstances, irreversible. The designer should be aware of the magnitude of this movement. Clay bricks being taken from the kiln expand owing to the absorption of water from the atmosphere. The magnitude of this expansion depends on the type of brick and its firing temperature and is wholly irreversible. A large part of this irreversible movement takes place within a few days, as shown in Fig. 2.3, and the rest takes place over a period of about six months. Because of this moisture movement, bricks coming fresh from the kiln should never be delivered straight to the site. Generally, the accepted time lag is a fortnight. Subsequent moisture movement is unlikely to exceed 0.02%. (Hendry et al., 2004).

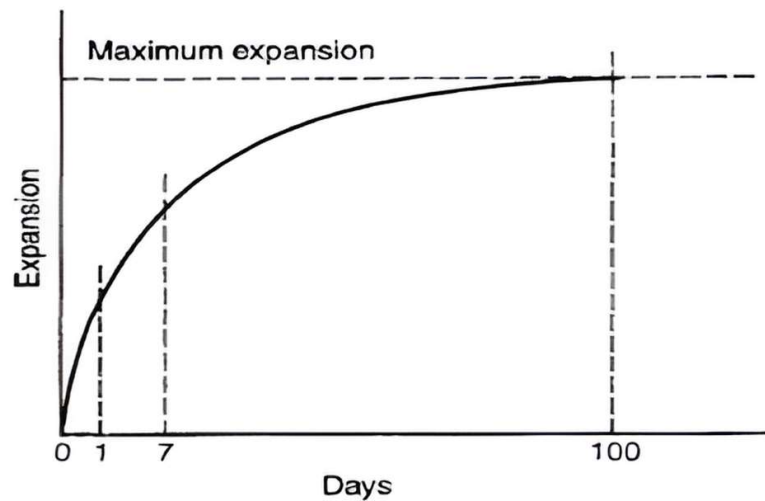


Fig. 2.3: Expansion of kiln-fresh bricks due to absorption of moisture from atmosphere.

2.6. Mortar

The mortar for load-bearing brickwork must be a cement: lime: sand mix in one of the descriptions shown in Table 2.3. For low-strength bricks, a weaker mortar, 1:2:9 mix by volume, may be suitable. For reinforced masonry, mortar should not be weaker than grade (ii).

Mortar designation	Types of mortar (proportion by volume)			Mean compressive strength at 28 days (N/mm ²)	
	Cement:lime:sand	Masonry cement:sand	Cement:sand with plasticizer	Preliminary (laboratory) test	Site tests
(i)	1:0 to 1:3	—	—	16.0	11.0
(ii)	1:1/4 to 1:4	1:2 1/2 to 3 1/2	1:3 to 4	6.5	4.5
(iii)	1:1.5 to 6	1:4 to 5	1:5 to 6	3.6	2.5
(iv)	1:2.8 to 9	1:5 1/2 to 6 1/2	1:7 to 8	1.5	1.0

↑ Increasing strength
 ↑ Increasing ability to accommodate movement, e.g. due to settlement, temperature and moisture changes
 ↓

Direction of change in properties is shown by the arrows

Increasing resistance to frost attack during construction
 Improvement in bond and consequent resistance to rain penetration

Table 2.3: Requirements for mortar (BS 5628)

According to UK National Annex (NA to BS EN 1996-1-1:2005), four types of mortar can be used for masonry walls. (Table 2.4)

Compressive strength class ^{A)}	Prescribed mortars (proportion of materials by volume) (see Note)				Mortar designation
	Cement ^{B)} : lime : sand with or without air entrainment	Cement ^{B)} : sand with or without air entrainment	Masonry cement ^{C)} : sand	Masonry cement ^{D)} : sand	
M12	1 : 0 to ¼ : 3	1 : 3	Not suitable	Not suitable	(i)
M6	1 : ½ : 4 to 4½	1 : 3 to 4	1 : 2½ to 3½	1 : 3	(ii)
M4	1 : 1 : 5 to 6	1 : 5 to 6	1 : 4 to 5	1 : 3½ to 4	(iii)
M2	1 : 2 : 8 to 9	1 : 7 to 8	1 : 5½ to 6½	1 : 4½	(iv)

Table 2.4. Different types of mortars (Table 2 of UK National Annex to EC6).

According to EC6 Elastic modulus of masonry is: $E = 1000$. $fk = 1000 \times 3.7 = 3700$ (N/mm²)

2.7. Cement, Sand and Water

2.7.1. Portland cement

According to (Hendry et al, 1996), normal Portland cement must follow a standard such as BS 12. Rapid-hardening cement may be used instead of ordinary Portland cement where higher early strength is required; otherwise, its properties are similar. Sulphate-resistant cement should be used in situations where the brickwork is expected to remain wet for prolonged periods or where it is susceptible to sulphate attack, e.g., in brickwork in contact with sulphate-bearing soil.

2.7.2. Masonry cement

This is a mixture of approximately 75% ordinary Portland cement, an inert mineral filler, and an air-entraining agent. The mineral filler is used to reduce the cement content, and the air-entraining agent is added to improve the workability. Mortar made from masonry cement will have lower strength compared to a normal cement mortar of a similar mix. The other properties of the mortar made from masonry cement are intermediate between cement: lime: sand mortar and plasticized cement, sand, mortar.

2.7.3. Sand

The sand for mortar must be clean, sharp, and free from salt and organic contamination. Most natural sand contains a small quantity of silt or clay. A small quantity of silt improves workability. Loam or clay is moisture-sensitive and in large quantities causes shrinkage of mortar. Marine and estuarine sand should not be used unless washed completely to remove magnesium and sodium chloride salts which are deliquescent and attract moisture. Specifications of sand used for mortar, such as BS 1200, prescribe grading limits for the particle size distribution. The limits given in BS 1200 are as shown in Fig. 2.4, which identifies two types of sand: sand type S and sand type G. Both types of sand will produce satisfactory mortars. However, the grading of sand type G, which falls between the lower limits of sand S and sand G, may require slightly more cement for a particular grade of mortar to satisfy the strength requirement envisaged in BS 5628, refer to Table 2.3 (Hendry et al, 1996).

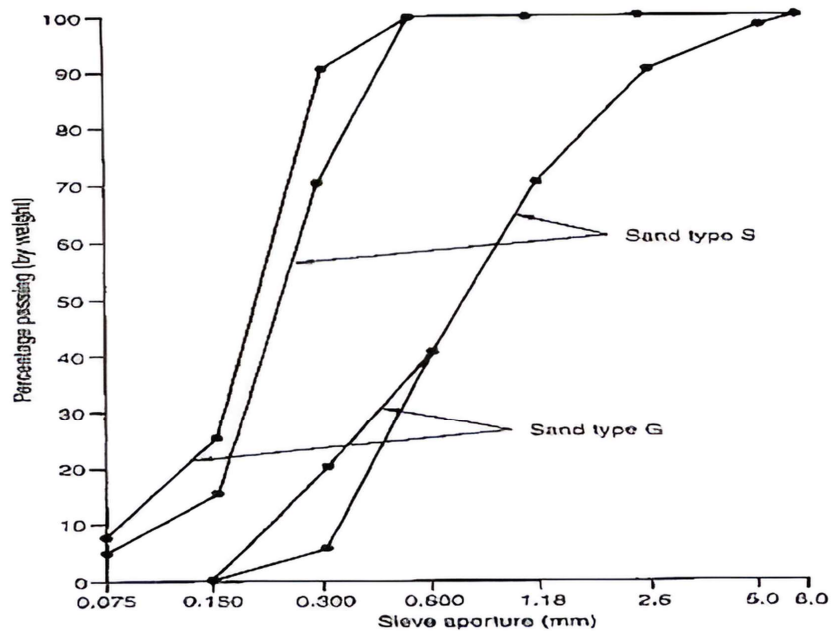


Fig. 2.4: Grading limits of mortar sand (BS 1200).

2.7.7. Water

Mixing water for mortar should be clean and free from contaminants either dissolved or in suspension. Ordinary drinking water will be suitable (Hendry et al, 1996).

2.8. The behaviour of masonry walls under lateral load

In masonry structures, the main elements are walls with other elements like the roof, floors, beams and piers transferring the loads to the foundations. Masonry walls are regularly subjected to in-plane and out-of-plane loads. As the name indicates, in-plane loads act along the central plane of the wall, whereas out-of-plane loads take action in the perpendicular direction of the wall plane. Masonry should be designed to resist both types of loads. In addition to the lateral loads, masonry walls are essentially loaded in the vertical direction from self-weight, dead and live loads. Therefore, they are subjected to a complex stress state. Typical failure mechanisms associated with unreinforced masonry are shown in Figures 2.5 to 2.8 (Churilov, 2012).

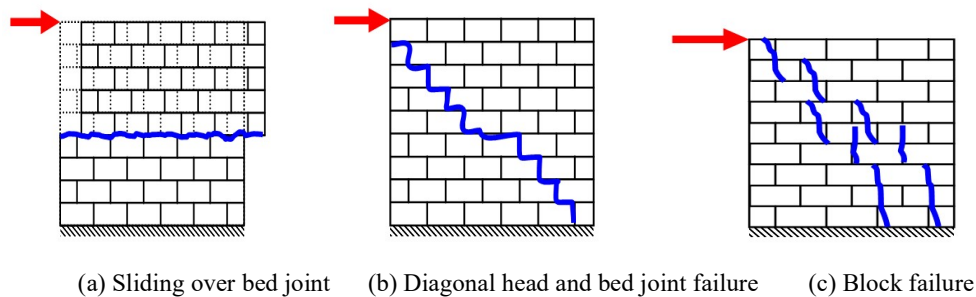


Figure 2.5: Shear failure modes

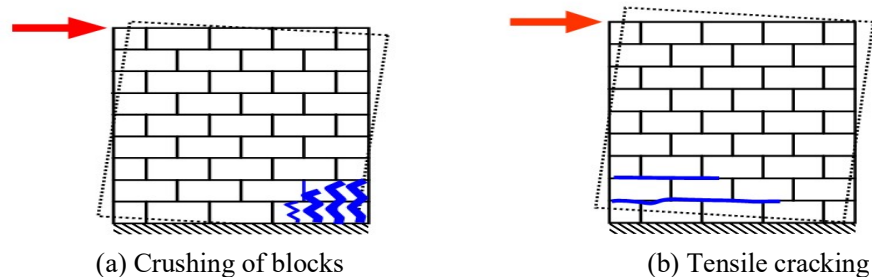


Figure 2.6: Compression and tension failure modes (Churilov, 2012).

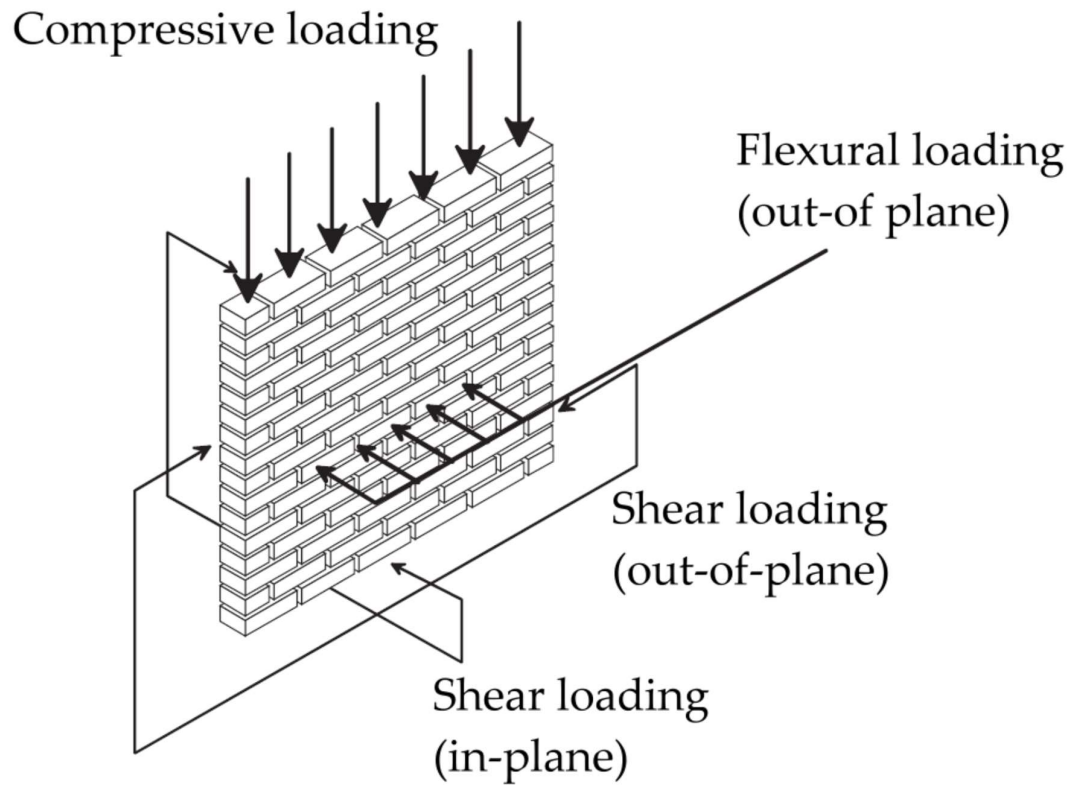


Figure 2.7: Loading conditions expected on a wall (Churilov, 2012).

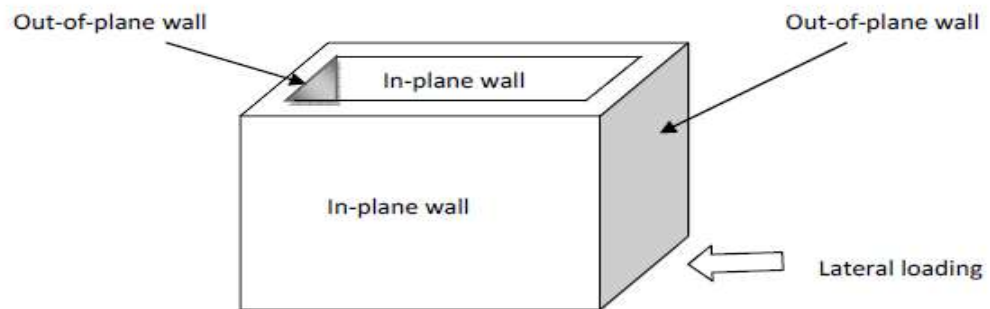


Figure 2.8. The behaviour of Masonry walls against lateral loading (Churilov, 2012).

The failure in masonry walls depends on the direction loads. There are two possible scenarios:

2.9. In-plane behaviour

Simplified calculation method for shear walls according to Euro code 6, part 3 (2006).

Verification of shear resistance of walls

Under the ultimate limit state, it shall be verified that:

$$V_{Ed} \leq V_{Rd}$$

where :

V_{Ed} is the design shear load on the wall,

V_{Rd} is the design shear resistance of the wall.

2.10. Design shear resistance

The design shear resistance V_{Rd} of a four-sided section may be determined as follows:

$$V_{Rd} = c_v \left[\frac{l}{2} - e_{Ed} \right] t f_{vdo} + 0,4 \frac{N_{Ed}}{\gamma_M} \leq 3 \left[\frac{l}{2} - e_{Ed} \right] t f_{vdu} \quad (2.1)$$

Where:

c_v is 3 for masonry with filled perpend joints or

1,5 for masonry with unfilled perpend joints;

l is the length of the wall in the bending direction;

e_{Ed} is the eccentricity of the compressive load in the cross-section being considered

$$e_{Ed} = \frac{M_{Ed}}{N_{Ed}} \quad (2.2)$$

not taken less than $l / 6$

M_{Ed} is the design value of the moment in the cross-section being considered;

N_{Ed} is the design value of the compressive load in the cross-section being considered;

t is the thickness of the wall;

f_{vdo} is the design value of the initial shear strength equal to f_{vko} , according to 3.4, divided by γ_M ;

f_{vdu} is the design value of the limit to the shear strength according to 3.6.2(3) and 3.6.2(4) of EN 1996-1-1:2005.

According to Ehsani et al., 1997, Churilov (2012) and Tumialan (2001) the friction failure, tensile failure, compression failure and shear failure occur as a result of an in-plane loads. (Figures 2.9 to 2.12).

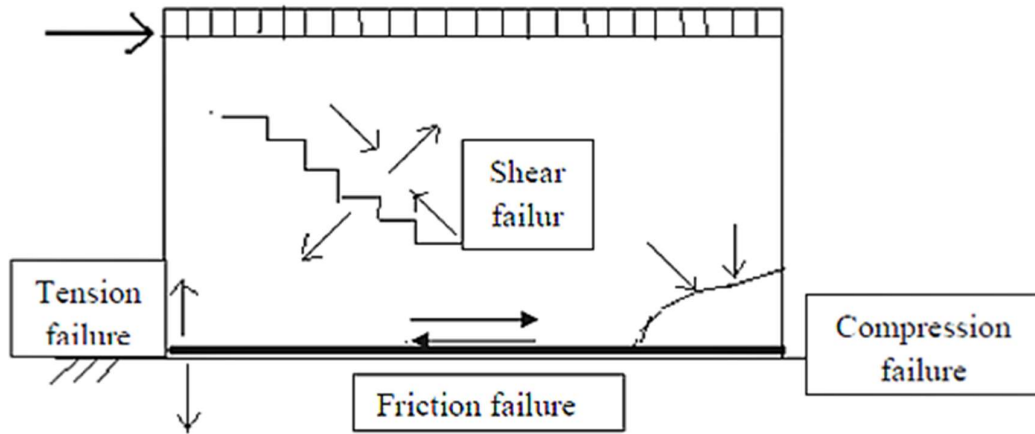


Figure 2.9: Failure modes under in-plane loading (Zilch et al., 2001 cited by Arash Syari)

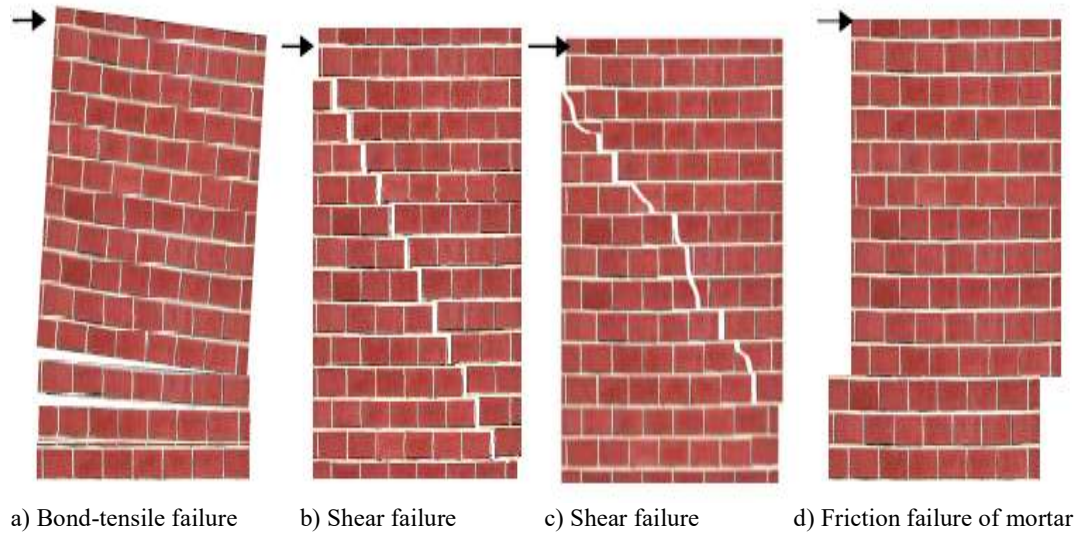


Figure 2.10: Failure modes under in-plane loading (Emami et al., 2008, cited by Syari, 2010)



(a) Out-of-plane failure



(b) In-plane failure

Figure 2.11: In-plane failure earthquake in Izmit, Turkey in 1999 (Casareto et al, 2002)



Figure 2.12: In-plane shear failure of poorly confined masonry walls: a) the 2010 Maule, Chile earthquake (M. Astroza), and b) the 2001 El Salvador earthquake (EERI, 2001). (Roberto Meli et al, 2011)

2.11. Characteristic shear strength of masonry

Euro code 6, 2005, EN 1996-1-1:2005), recommended equation (2.3) to calculate the shear strength of URM walls.

$$f_{vk} = f_{vko} + 0,4\sigma_d \quad (2.3)$$

but not greater than $0,065 f_b$ or f_{vlt}

where:

f_{vko} is the characteristic initial shear strength, under zero compressive stress;

f_{vlt} is a limit to the value of f_{vk} ;

σ_d is the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance;

f_b is the normalised compressive strength of the masonry units, for the direction of application of the load on the test specimens being perpendicular to the bed face.

To find the shear resistance of unreinforced masonry walls, Euro code 6 [2005] defines the shear strength of URM walls according to the friction theory shown in Equation (2.4).

$$f_{tk} = f_{vk0} + \mu c \cdot \sigma_d \quad (2.4)$$

f_{tk} , is the characteristic shear strength of masonry

f_{vk0} , is initial shear strength under zero compressive stress

c , is friction coefficient, describing the contribution of the compressive stresses

d , is designed compressive stress perpendicular to the shear in the member

The code suggests the determination of f_{vk0} by experimental tests according to EN 1052-3 [2002] and EN 1052-4 [2000]. The friction coefficient is considered in the code as $c = 0.4$. This approach corresponds to the Mohr-Coulomb analogy with initial shear strength considered as the cohesion of masonry and friction coefficient. The upper limitation of the characteristic shear strength of masonry, given with Eq (2.5) accounts for a possible failure of the units by shear instead of the mortar joints.

$$f_{vk} = 0.065 f_b \quad (2.5)$$

where f_b is the normalised compressive strength of the masonry unit.

When shear failure mechanisms are considered, Euro code 6 [2005] suggests calculation of the shear resistance of reinforced masonry wall by summing up the contributions of the plain masonry and the horizontal reinforcement only. This simplification is given with Eq 2.6 and 2.7.

$$H_{str,s} = H_{r1} + H_{r2} \quad (2.6)$$

$$H_{sh} = \frac{f_{vk}}{\gamma_M} t l + 0.9 A_{sh} \frac{f_{yh}}{\gamma_s} \leq (2 \text{ MPa}) t l \quad (2.7)$$

Where,

$H_{str,s}$	shear resistance of reinforced masonry wall
f_{vk}	characteristic shear strength of masonry
f_{yh}	yield strength of horizontal reinforcement
A_{sh}	area of horizontal reinforcement
γ_M	partial safety factor for masonry
γ_s	partial safety factor for steel

The distinction in the length of the wall has been made when calculating reinforced masonry walls. In this case, the total length of the wall is taken for the calculation of the contribution of the plain masonry. For simplification, verification of reinforced masonry elements subjected to bending loading should be performed with rectangular stress distribution. The ultimate bending moment can be derived from Eq 2.8.

$$M_{Ru} = A_{sv} F_{yv} Z \leq 0.4 F_{yv} b w d^2 \quad (2.8)$$

Where,

M_{Ru}	ultimate bending moment
f_k	characteristic compressive strength of masonry
A_{sv}	area of vertical reinforcement
F_{yv}	yield strength of vertical reinforcement
z	lever arm between the compressive and tensile force
$b w$	width of the wall (thickness of the wall)
d	effective depth of the section (length of the wall)
L	length of a wall
$d = \sigma_v L / 0.38 f_k$	

2.12. Out-of-plane behaviour

Wind or earthquake loads cause bending and shear stresses in the wall. This may product cracking and probable failure by overturning. Due to an increase in spectral accelerations up the structure height, the out-of-plane seismic effects are more significant at higher floor levels, as shown in Figure 2.2.9 a. In the area affected by the 2010 Maule, Chile earthquake, wall cracking due to out-of-plane seismic effects was observed at the top floor level, as shown in Figure 2.13.b (Meli et al, 2011).

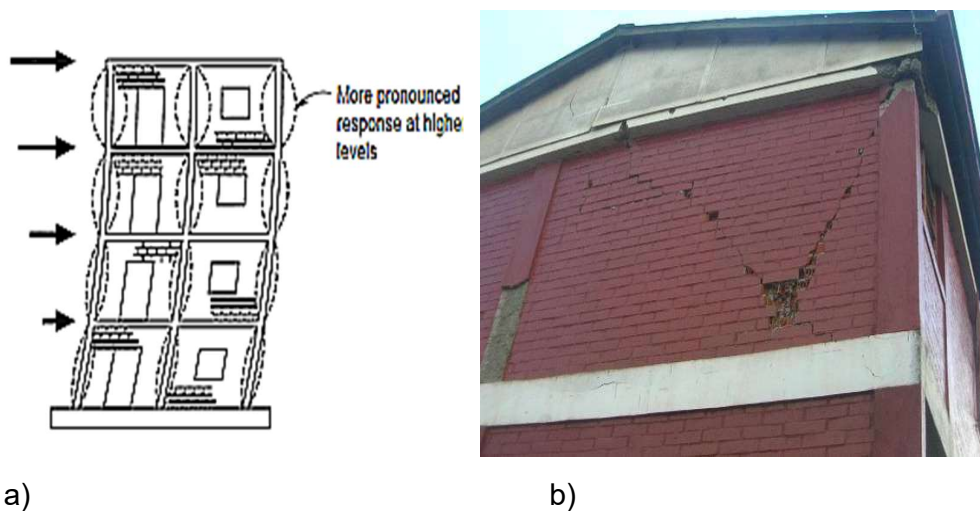
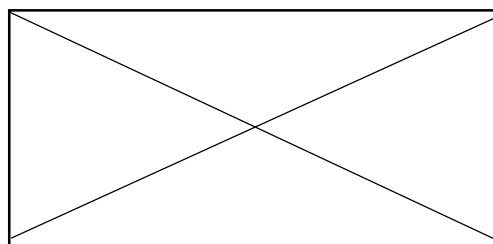
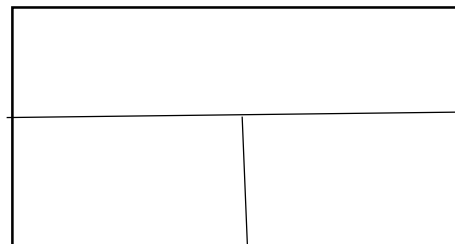


Figure 2.13. The seismic response masonry walls, (a) (Tomazevic, 1999), and (b) damage at the top floor after the 2010 Maule, Chile earthquake (M. Astroza) Cited by Roberto Meli et al, 2011).

Figure (2.14A and 2.14b) shows the cracks in the wall for out-of-plane behaviour of walls in Iran's 2021 earthquake.



Two-directional moments in the plane of the wall



One-directional moment in the plane of the wall



Figure 2.14. Cracks in the wall for out-of-plane and in-plane behaviour (Iran, kokkiloeh, 2021 by Fathi)

2.13. Traditional methods for strengthening the masonry

According to Sayari, (2011) several methods can be applied to reinforce the masonry walls. Some of these methods consist of removing a part of masonry walls and replacing it with an alternative high-strength material such as concrete or steel. This technique is useful and practical, but it needs surface preparation, adding significant mass to the wall and it requires a long time for application. Furthermore, there are some other methods for strengthening the existing masonry walls that consist of external jacketing to one or both sides of the walls. For example, reinforced concrete coat (RC), steel coat, mesh-reinforced mortar coat, sprayed concrete and some other type of coating can be applied externally. These techniques increase the mass of the walls and structures and it takes a long time to apply.

The traditional techniques in some cases are not optimum (for example due to increased dimensions, increased weight, and disruption of occupancy) and not applicable, in the case of building facades with arches and columns that must not be changed by using RC jackets due the architectural reasons.

2.14. Definition of FRP

Rahman and Ueda (2016) conducted in-plane shear strength of the reinforced walls is significantly improved by using the FRP and although the CFRP increases the shear capacity of a masonry wall.

Fibre Reinforced Polymer (FRP) is a relatively new composite material manufactured from fibres and resins that have proven efficient and economical for the repair of weak structures, Figure 2.15.



Figure 2.15. Common FRPs Used in Structural Engineering (Derias and Hacha, 2007)

Glass and carbon connected by a matrix are strong composite materials. The fibres provide stiffness and strength, while the matrix transfers the load between fibres and makes the composites stable. The results of experimental tests showed an increase in the bending capacity due to the out-of-plane loads and in-plane shear loads (Derias and Hacha, 2007)

2.15. FRP Composites

Fibre-reinforced polymers (FRP) are composite materials, made of high resistance fibbers saturated with polymeric resins. The mixing result is a material with tensile properties between fibbers and resin's (Figure 2.16).

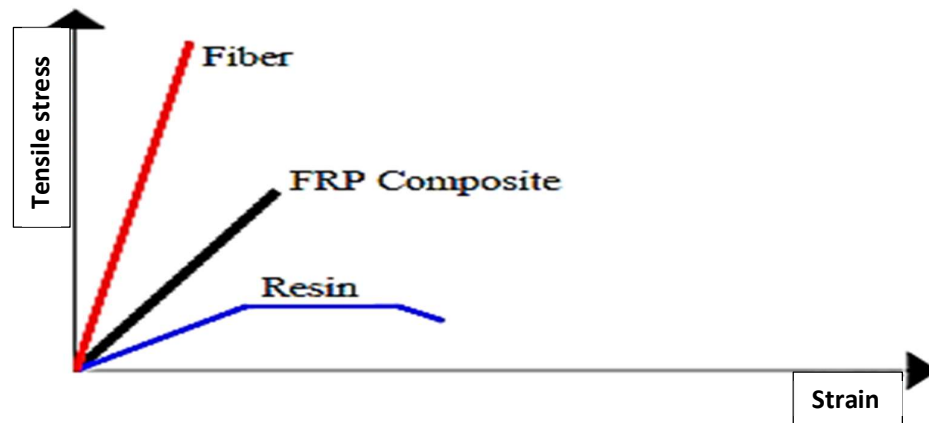


Fig. 2.16. Comparison among fiber's, resins and composite's tensile properties (Casareto et al, 2002).

According to (Casareto et al, 2002), they are characterised by brilliant tensile strength in the direction of the fibbers and by negligible strength in the direction transverse to the fibers. FRP composites do not exhibit yielding, but instead are elastic up to failure and they are also characterized by the relatively low modulus of elasticity in tension and low compressive properties.

The general advantages of FRP reinforcement compared to steel are:

- Durability in aggressive environments
- High strength-to-density ratio
- Magnetic and electric neutrality
- Low specific weight
- Low axial coefficient of thermal expansion

FRP materials are commercialized in different forms: rods, tendons, laminates, and three-dimensional modules. FRP reinforcement comes in the shape of rods of circular cross-sections, strips of rectangular cross-sections, strands, and laminates, which allow different types of applications. Figure 2.17, shows different kinds of FRP (Casareto et al, 2002).



Fig. 2.17. Different kinds of FRP (Casareto et al, 2002).

2.15.1. Carbon fibers:

Fibre is produced by heating organic materials having a considerable amount of carbon (93.95%), such as rayon, polyacrylonitrile (PAN), or pitch (a black residue from the distillation of petroleum) in an inactive environment. This fibre is the toughest, hardest, and most durable, they are more expensive than glass fibres but offer an excellent combination of strength, low weight, high modulus and fatigue properties. (Casareto et al, 2002).

2.15.2. Aramid fibers:

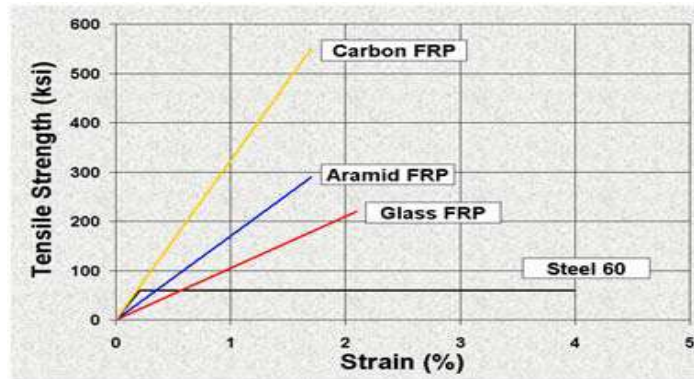
This kind of fibres offers excellent impact resistance, and good electric and temperature insulating properties and they are also, resistant to organic solvents, fuels, and lubricants. They have a medium modulus and a very low density as compared to glass and carbon.

2.15.3. Glass fibers:

E-Glass fibres are considered the main reinforcement for polymer matrix composites, due to their high electrical insulating properties and low vulnerability to moisture.

S-Glass, with higher strength, and heat resistance. Glass products are cheap, but they weigh more than carbon. Glass has been the main fibre for many civil engineering applications because of an economical balance of cost and specific strength properties. Comparison based on fibre area only among sheets made of carbon (CFRP), aramid

(AFRP), glass (GFRP) and reinforcing steel in terms of stress-strain relationship is illustrated in figure 2.18. (Casareto et al, 2002).



Note: 1 ksi = 6.89 MPa

Fig. 2.18. Comparison among AFRP, CFRP, GFRP and Steel (Casareto et al, 2002).

2.15.4. Epoxy

Epoxy resins have brilliant physical properties as well as very good bond characteristics and can be framed with different materials or mixed with other epoxy resins for better performance.

2.16. The benefits of FRP composites:

1. High strength to weight ratio,
2. High stiffness to weight ratio,
3. Resistance against corrosion,
4. Lightweight,
5. Easy application,
6. Speed of construction (can be installed in a short period),
7. Savings in erection costs,
8. Low life cycle cost,
9. Limited access requirements,
10. Possibility of removal,
11. High fatigue resistance,
12. Potentially high durability,

13. Availability of composites in required dimensions,
14. Reducing the necessity of joins.

The drawbacks of FRP composites:

1. Linear–elastic behaviour up to failure,
2. Low resistance against high temperature,
3. Poor ductility and a brittle mode of failure under long thermal-fatigue and loading,
4. Peeling,
5. Sensitive to moisture,

2.17. Experimental studies

Experimental studies by many researchers such as Ehsani et al. (1996) investigated the flexural behavior of URM walls strengthened with GFRP sheets.

Ataur and Tamon (2016), Tumialan and Nanni (2001), Alsayed (2006), and Sayari (2010) conducted out-of-plane and in-plane tests on FRP strengthened masonry walls. They conducted that either the FRP or CFRP increases the shear capacity and ductility of the masonry wall.

Berna and Erkan (2017) studied the mechanical behavior of masonry walls before and after rehabilitation by using glass-polypropylene fiber and expanded glass granular-based plaster. The shear capacity of triplet samples without applying glass-polypropylene fiber under shear load was 15 kN (Figure 2.19 and 2.20). As can be seen in Figure 2.19, due to lack of ductility, the shear strength of the brick units suddenly decreased and brittle fractures occurred rapidly under stress. These damages are abrupt. The failure surface of the specimen shows little degradation before the fracture occurs.

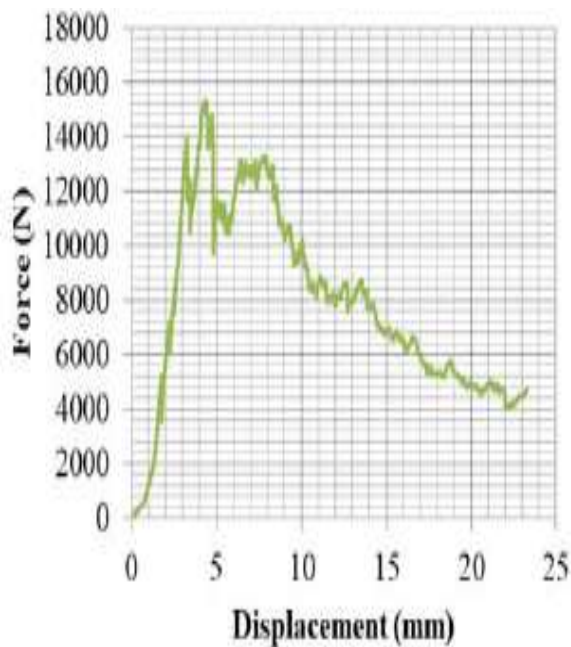


Fig 2.19: Force-displacement diagram for URT



Fig. 2.20: Failure of URT (Berna and Erkan, 2017)

In the second stage of the experimental study, the effect of the applied seismic textile and the plaster made of expanded glass granular was applied to both surfaces of the triplet, and the shear capacity of mortar joints was investigated. The thickness of the plaster on each surface was considered as 1 cm. The maximum shear capacity for reinforced triplets at both sides was 36KN (Fig. 2.21). The failure mode of all plastered specimens was due to vertical cracks in the direction of loading, which followed the interface between the bricks and joint mortar (Fig. 2.22).

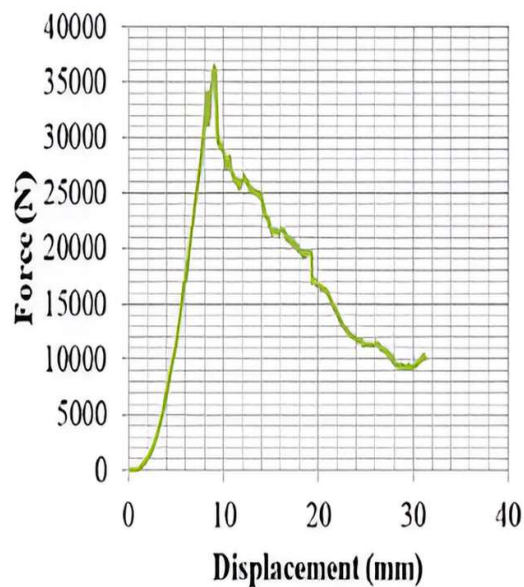


Fig 2.21: Reinforced triplet diagram



Fig 2.22: Reinforced triplet (Berna and Erkan, 2017)

They concluded the earthquake performance of masonry structures can be improved by 168% by simple strengthening techniques, such as seismic fabric with expanded glass granular-based plaster.

Pavan and Nanjunda (2015), investigated the shear behavior of FRP-glued masonry composed of masonry units having young's modulus lower than mortar. They focused on characterising the interfacial behavior of brick–mortar bed joints of masonry assemblages composed of solid burnt clay bricks and cement–sand mortar (Emu/Em ratio less than one), strengthened with FRP composites. In their study, two types of masonry triplets were employed to study the interfacial behavior of brick–mortar joints

of the masonry. They are (1) triplet masonry assemblage for sliding shear strength; and (2) off-axis masonry prisms to understand the interaction between shear and normal stress acting on the bed joint of masonry. The thickness of mortar joint was maintained at about 10 mm in all the prisms. Figs. 2.23 (a–d) shows the control masonry triplets T-Co and FRP-strengthened triplets T-G360-3-4, T-G600- Full, and T-C200-3-2.54.

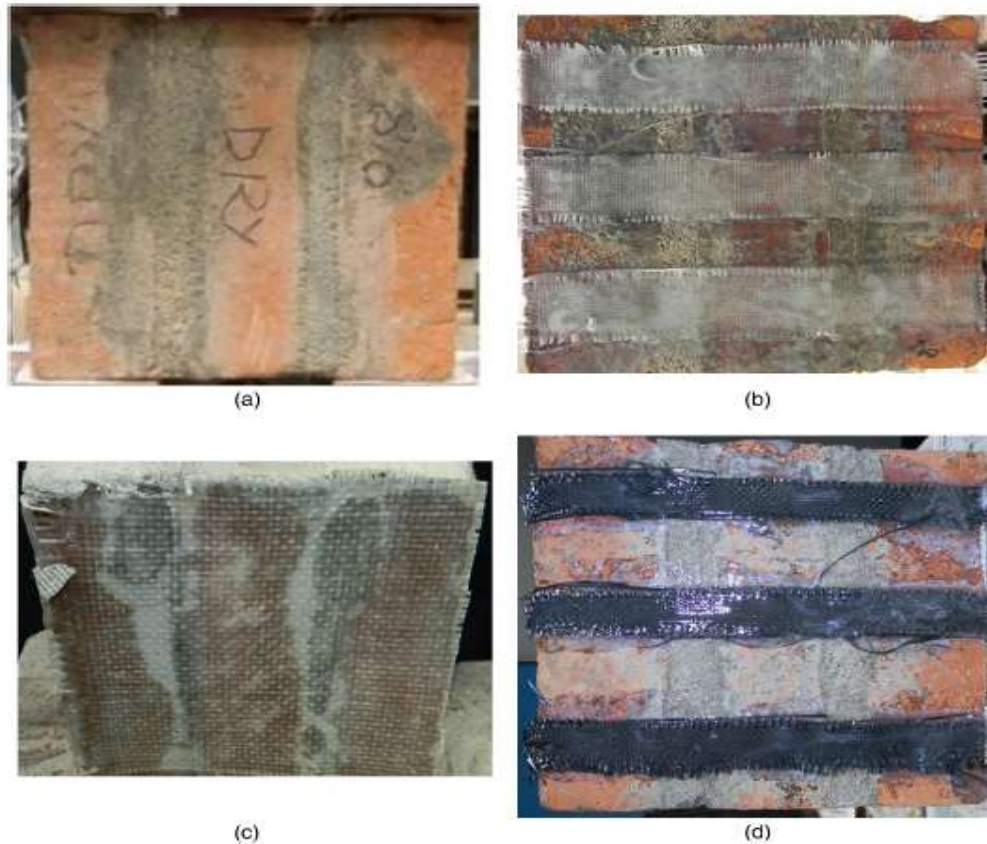


Fig. 2.23. Masonry triplets: (a) T-Co; (b) T-G360-3-4; (c) T-G600-full; (d) T-C200-3-2. 54..
Pavan and Nanjunda (2015)

The test results showed that control triplets without FRP strengthening (T-Co) resisted a maximum load of 15.7 kN. The average ultimate displacement of control triplet specimens was 1.0 mm. Figure 2.24 indicates that T-Co triplets increase in shear stiffness in the initial part of the curve. The different grades of GFRP fabric employed in this study were 200, 360, and 600 g/m². GFRP 360 and GFRP 600 were glued to masonry triplets in full and strip configuration, while GFRP 200 was glued in strip configuration only.

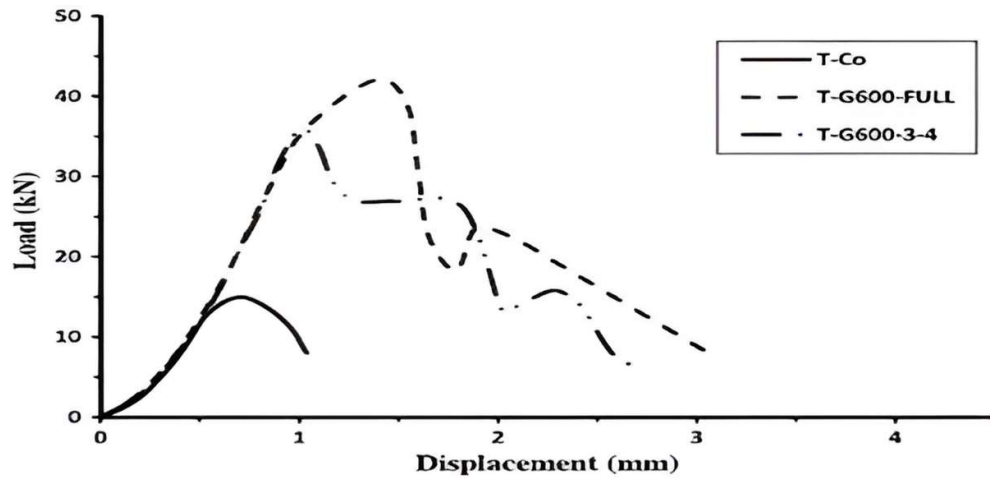


Fig. 2.24. Load versus displacement response of masonry triplets attached with GFRP of grade 600 g/m²
Pavan and Nanjunda (2015)

The shear behavior of the previously-mentioned masonry triplets glued with GFRP composites exhibited improved performance in comparison to control masonry triplets. The load-displacement responses of masonry triplet specimens are given in Figs. 2.25 to 2.28. Triplets glued with GFRP in full configuration exhibited higher gain in shear strength in comparison to triplets glued with GFRP in strip configuration. T-G600-Full and T-G360-Full specimens experienced nearly equal gains in shear strength.

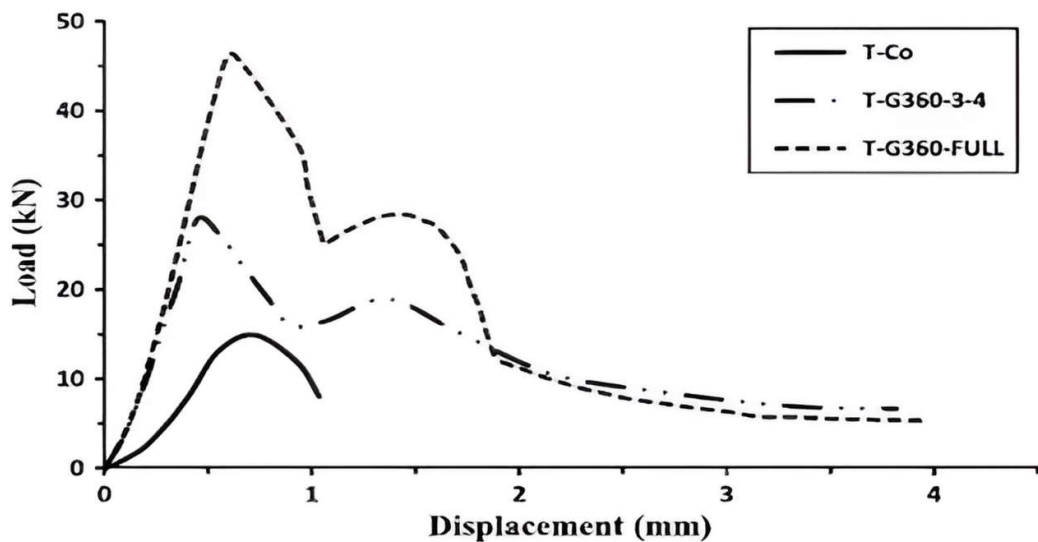


Fig. 2.25. Load versus displacement response of masonry triplets attached with GFRP of grade 360 g/m².
Pavan and Nanjunda (2015)

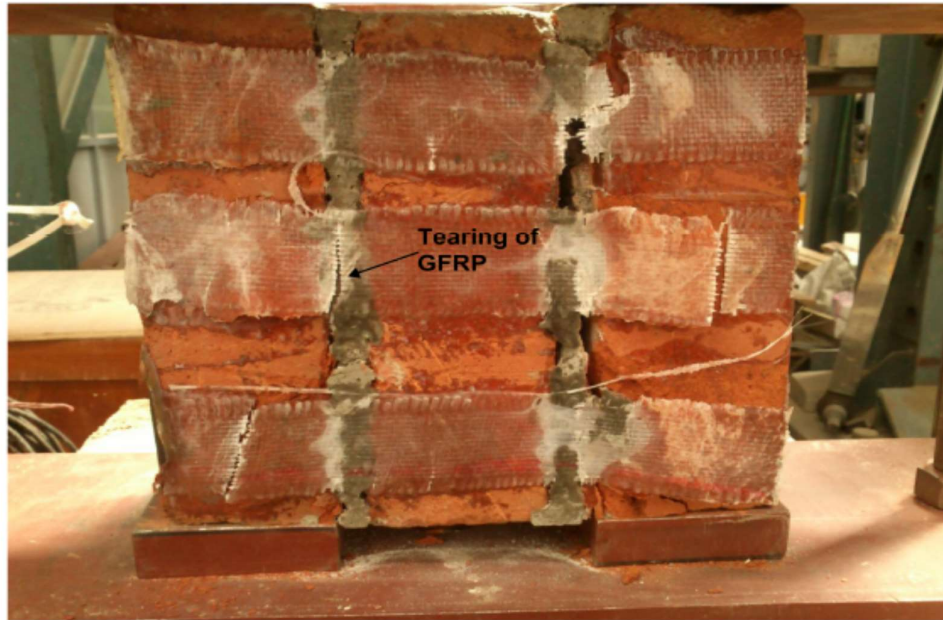


Fig. 2.26. Load versus displacement response of masonry triplets attached with GFRP of grade 360 g/m², Pavan and Nanjunda (2015)

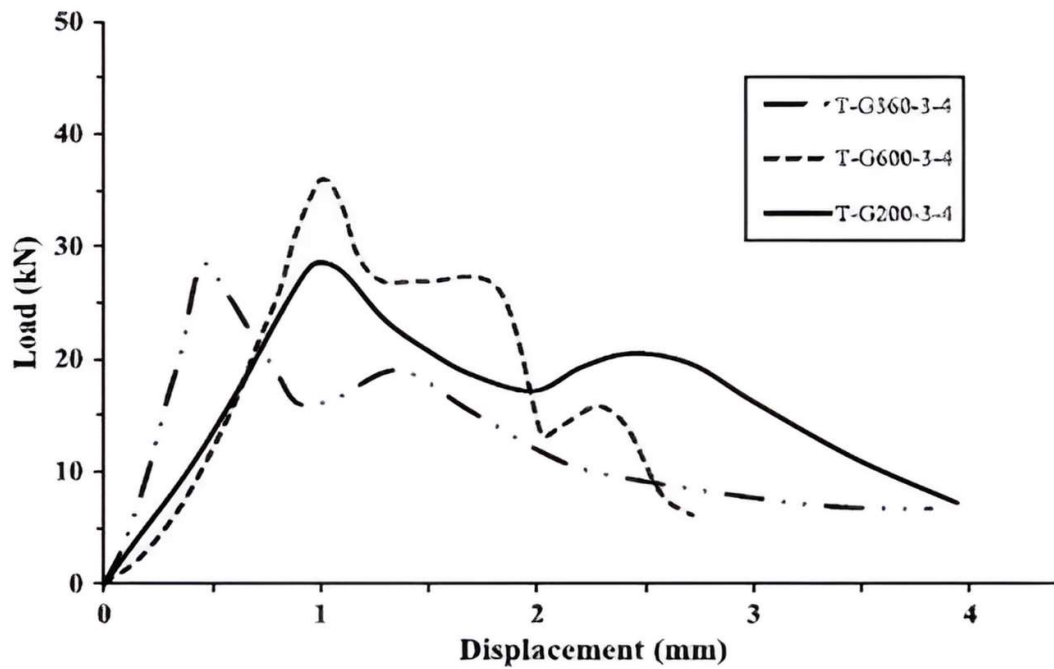


Fig. 2.27. Load versus displacement response of T-G200-3-4, T-G360-3-4, and T-G600-3-4 specimens. Pavan and Nanjunda (2015)



Fig. 2.28. Failure patterns of triplet specimens attached with GFRP composites: (a) debonding failure of T-G360-full triplets; (b) debonding failure of T-G600-3-4 triplets; (c) tensile rupture failure of GFRP 200 in T-G200-3-4 triplets. Pavan and Nanjunda (2015)

Fig. 2.29 shows the load versus displacement response of masonry triplets glued with CFRP composites. CFRP strips were glued to masonry triplets in two configurations: T-C200-2-4 (two strips each of width 4 cm) and T-C200-3-2.54 (three strips each of width 2.54 cm).

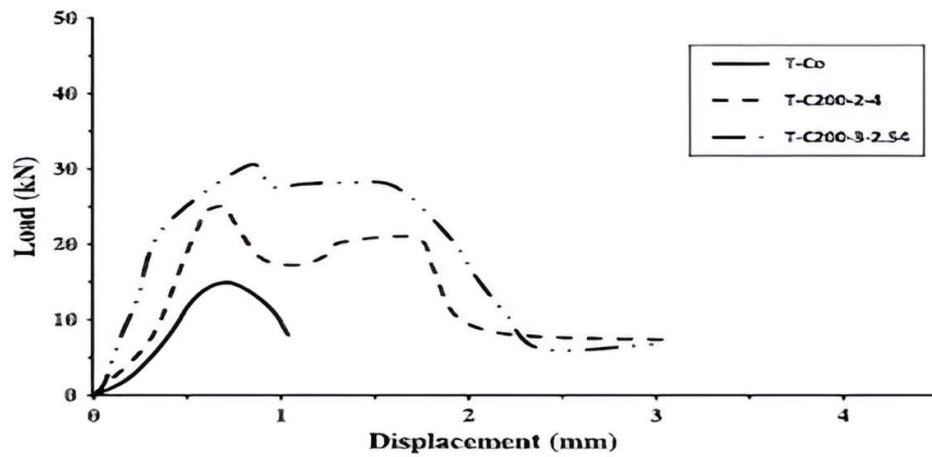


Fig. 2.29. Load versus displacement response of masonry triplets attached with CFRP of grade 200 g/m²



Fig. 2.30. Tearing and debonding of CFRP laminate observed in T-C200-3-2.54 triplets. Pavan and Nanjunda (2015)

Their research showed that the shear strength of triplets strengthened with the same amount in terms of area reinforced with GFRP but with different grades was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity possessed by different grades of GFRP. However, the gain in shear strength increased with an increasing amount of GFRP glued. The percentage increase in shear strength for T-G360-FULL and T-G600- FULL triplets were about 185 and 168%,

respectively, whereas the percentage increase in shear strength for T-G360-3-4 and T-G600-3-4 was 80 and 127%, respectively. The percentage increase in shear strength of T-C200-3-2.54 and T-C200-2-4 triplets was 112 and 70%, respectively.

The influence of the grade of FRP was observed on deformational characteristics of masonry triplets such as ultimate displacement and the initial slope of the load-displacement curve. Ultimate displacement and the initial slope of the load-displacement response of triplets glued with higher grades of GFRP were lower than triplets glued with lower grades of GFRP. The percentage increase in the ultimate displacement of T-G360-FULL and T-G600-FULL triplets was 296 and 208%, respectively, whereas the increase for T-G360-3-4, T-G600-3-4, T-G200-3-4, and T-C200-3-2.5 triplets was 282, 171, 294, and 199%, respectively. The area under the load-displacement curves for masonry triplets glued with FRP was much larger in comparison to control triplet specimens, indicating an enhanced ability to absorb energy. Also, Pavan and Nanjunda, (2015) developed an equation for triplet specimens fully glued with glass-fiber reinforced polymer (GFRP). According to their study, shear strength of FRP-strengthened masonry can be expressed as the sum of the resistance provided by brick–mortar joint and FRP as

$$V_{mc} = V_{m0} + V_{frp} \quad (2.9)$$

where V_{m0} = shear resistance of the unreinforced brick–mortar bed joint; and V_{frp} = resistance offered by FRP composites. V_{m0} can be determined based on equations available in design guidelines such as IS: 1905 (BIS 2002a). In their study, V_{m0} was obtained from the shear tests on control triplet specimens. The tensile force in the FRP strip is given by

$$T_f = E_f \times \epsilon_f \times t_f \times b_f \quad (2.10)$$

where E_f = modulus of elasticity of FRP; ϵ_f = tensile strain in FRP; t_f = thickness of FRP; and b_f = width of FRP strip. Hence, the contribution of the FRP strip towards shear resistance was expressed as

$$V_{frp} = E_f \times \epsilon_f \times t_f \times b_f \times \mu_f \quad (2.11)$$

where μ_f = coefficient of friction between brick-and-mortar joint. Tensile strain in FRP composites ϵ_f can be expressed as

$$\epsilon_f = k \times \epsilon_{frp} \quad (2.12)$$

where k = constant; and ϵ_{frp} = tensile strain limit of FRP composite. Hence V_{frp} is given by

$$V_{frp} = k \times E_f \times t_f \times b_f \times \mu_f \times \epsilon_{frp} \quad (2.13)$$

$$V_{frp} = k \times \mu_f \times T_{frp} \quad (2.14)$$

$$V_{frp} = \beta \times T_{frp} \quad (2.15)$$

where $\beta = k \times \mu_f$ and $T_{frp} = \sigma_{frp} \times t_f \times b_f$. Tensile strength of FRP composites is denoted as σ_{frp} .

Ehsani et al (1997) investigated clay brick specimens reinforced with FRP, the experimental study involved the construction and testing of samples with FRP overlays. Three different fabric densities were used and the fiber orientation, as well as the fabric length, were varied to observe their effect on the developed strength. Two modes of failure were observed: (1) shear failure along the bed joint; and (2) delamination of fabric at the middle brick region or fabric edges. The type of failure was influenced by the fabric's strength. Shear failure and bond failure were observed. The strength and stiffness of the specimens were highly influenced by the fiber orientation. Changing the fiber orientation from 90 to 45 degree led to a slight increase in the ultimate load. In addition, the fabric length was very important in the strength development. This indicates that in order to develop the full strength of the material, a sufficient fabric length is required. The test results proved indisputably that FRP overlays can be very effective in strengthening the bed joints of URM walls.

Rahman and Ueda (2016) investigated the in-plane shear performance of externally strengthened masonry walls using two types of fiber-reinforced polymer (FRP) sheets, namely, carbon FRP (CFRP) and polyethylene terephthalate-FRP (PET-FRP) sheets. Among these two, PET-FRP has a low tensile strength but possesses a higher fracturing strain than CFRP. In their study, PET-FRP and CFRP have been used as two distinct strengthening materials. The strength of CFRP is higher than PETFRP, whereas PET-FRP possesses a relatively higher fracturing strain than CFRP. The purpose of this study was to show the difference in the behavior of masonry shear walls strengthened by these two different FRPs and the superiority of one FRP over the other. They conducted that the in-plane shear strength of the strengthened walls is considerably improved by using either of the FRPs but the ductility is compromised when CFRP is used as strengthening material. The experimental study demonstrated that PET-FRP has a better seismic performance than CFRP, as it shows a better ductile behavior than CFRP.

Although the elastic stiffness of URM walls was largely improved by the use of FRP, externally bonded over the surface of the walls but it was observed that stiffness value

beyond some specific range does not increase the in-plane shear strength of masonry and it will only increase the cost of strengthening works.

The influence of the plaster between wall and FRP composites on the out-of-plane behavior of masonry walls was studied by Tumialan et al. (2003, 2000). In his researches large-scale (2400mm×2400mm) masonry walls were tested under out-of-plane loading. For some of the walls GFRP laminates were attached to the surface of the walls directly and for the rest of them, GFRP laminates were attached to the plaster over the surface of the walls. For the FRP laminates directly attached to the walls, the strengthened system had better performance with increased strength in comparison to the walls with FRP laminates attached to the plaster. For the walls with FRP composites directly applied to the masonry surface, the load capacity of the walls improved by about 40% in comparison to the control wall (without FRP). In addition, the research indicated that the application of plaster between FRP and masonry walls reduces the load-bearing capacity by about 17% in comparison to the case without plaster between wall and FRP materials.

Arash Sayari (2010) conducted out-of-plane and in-plane shear strength of the strengthened walls is considerably improved by using either of the FRP and CFRP increases the shear capacity and ductility of a masonry wall. El Malyh et al (2020) studied the effect of CFRP wrap ratios on the strength, stiffness, and other mechanical characteristics of URM walls. An experimental program was pursued to investigate the in-plane behavior of the standardised specimens following ASTM E-519. The test included testing clay brick panels of 1200*1200*100mm fixed at the ends of their diagonal by two steel shoes, the diagonal compression forces were applied via the steel shoe located at the top corner. All the masonry walls had the same dimension, and they were strengthened by different configurations of unidirectional CFRP wrap. The variables considered in their study were the ratio, number of layers, and dispositions. They investigated the influence of the orientation and the ratios of the reinforcement on the behavior of masonry walls. The test results under diagonal compression loading indicated that each retrofitting system had special behavior related to many factors such as the position and the orientation of CFRP wrap that influence the ability of the specimen to resist diagonal compression loading. In all cases, the retrofitting system

improved the shear strength, the energy dissipation, and the deformations of masonry panels before failure.

Under diagonal compression loadings, cracks propagated diagonally in the URM wall, from the top corner, parallel to the loading direction, and across the wall width. While increasing compressive stress, the diagonal tension, and the shear stress increased too, then diagonal shear cracks in bed and head joints followed by a few cracks in the bricks were observed. However, when the specimen reached its maximum load-bearing capacity of 9,3KN, a brittle failure produced by a shear sliding along the mortar joints occurred a detachment of a significant part of the wall figure 2.31.

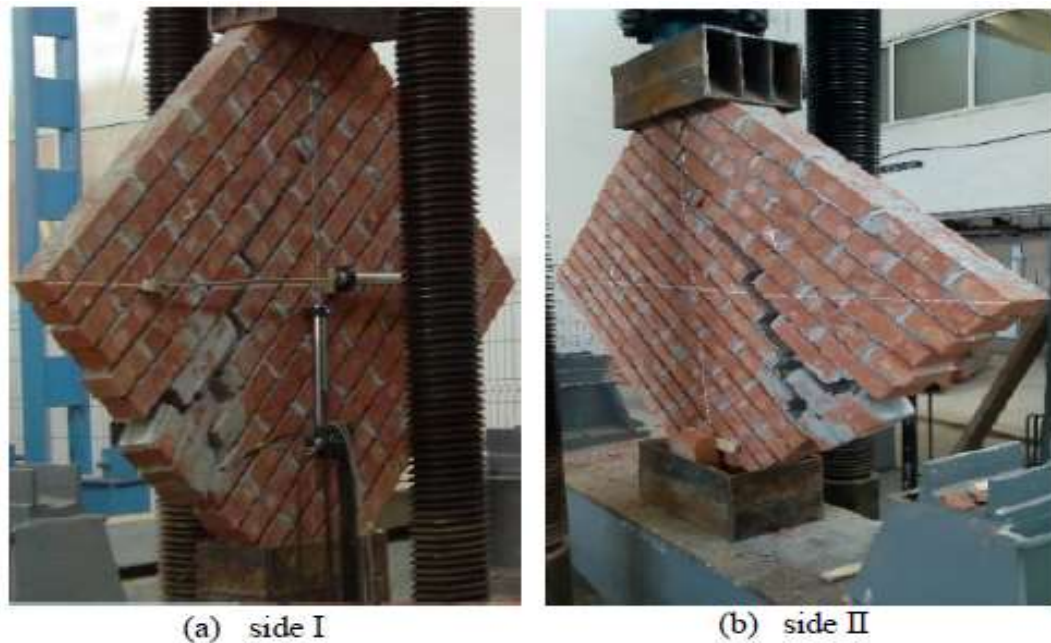


Figure 2.31: Failure mode of unreinforced masonry wall (El Malyh et al. 2020).

The failure of the second specimen with one-layer CFRP started gradually by continuous diagonal shear cracking that propagates along the brick surface and across the mortar, accompanied by delamination in several points on the diagonal CFRP wrap and finished by the detachment of a considerable part of the wall and the shear slippage of a layer of masonry substrate at one edge as shown in figure 2.32. However, the use of CFRP wrap enhanced the strength and the stiffness of the wall. The peak load reached 23KN, which presents an increase by a factor of 2.5 when compared to URM.



Figure 2.32: Failure mode of masonry panel with on layer CFRP (El Malyh et al. 2020).

The wall with three parallel diagonals CFRP reached 37.8 KN, cracks initiated simultaneously from opposite directions parallel to the vertical compression loading, although due to the interface between masonry wall and CFRP wrap, the shear force transmitted to the diagonal CFRP wrap, which was cracked too. An increase in shear strength, deformation, energy dissipation and carrying capacity of masonry wall was observed. Also, the failure of the specimen occurred in a high compression load started by successive cracks in masonry followed by delamination in the strengthening system (figure 2.33).



Figure 2.33: Delamination and shear cracking in wall with three parallel diagonals CFRP (El Malyh et al. 2020).

According to their study, the retrofitting system restrains the URM wall from shear failure and shear sliding; it enhanced their strength up to 319%, and all CFRP configurations, improved considerably the load-bearing capacity of the specimens from 2.5 to 4.2 times. In addition, the FRPs retrofitting systems are effective in increasing the in-plane strength and the deformability of URM walls, and the compressive strength of the strengthened walls had experimentally observed to be 147.31% to 319.35% higher than URM walls.

Luccioni and Rougier (2011) studied the mechanical behavior under in-plane compression and diagonal compression of clay brick panels reinforced or repaired with carbon fiber-reinforced polymer laminates. The specimens were tested under uniaxial compression normal and parallel to the bed joints and diagonal compression. For comparison, some of the specimens were tested up to failure without reinforcement, others were reinforced with CFRP and tested up to failure, and others were tested until a predefined degree of damage, then they were unloaded and repaired with woven carbon fabric, laminated and bonded on-site and, finally, they have tested again up to failure. Three different possible retrofitting layouts presented in Fig.2.34 were studied: total reinforcement with fibers laid out normally to load direction, diagonal strips bonded orthogonally to the compressive diagonal, and strips parallel to the bed joint. One layer of CFRP was used in each face of the specimens in all cases.

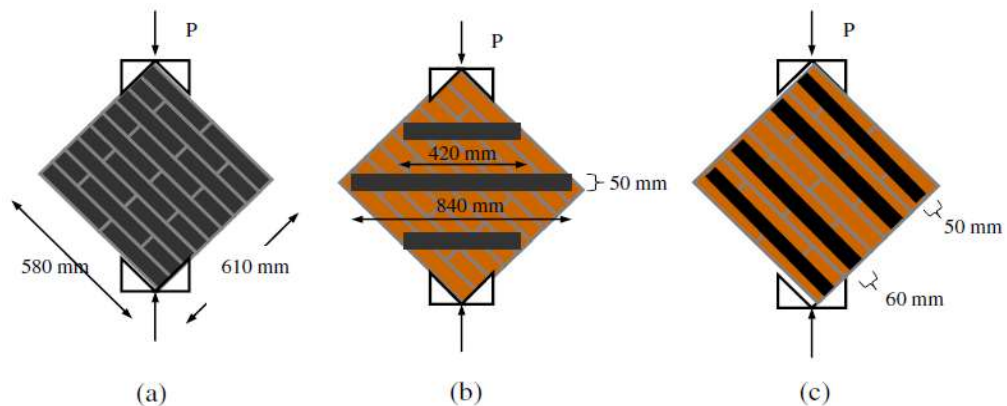


Fig. 2.34: CFRP layout and failure patterns of retrofitted panels under diagonal compression. (a) Completely retrofitted. (b) Retrofitted with diagonal bands. (c) Retrofitted with bands parallel to mortar joints (Luccioni and Rougier, 2011).

Different failure modes shown in Fig 2.35 were obtained for the different retrofitting schemes. In the case of total reinforcement, the integrity of the specimen was preserved without evidence of cracks near to failure load. Failure was produced by the brick crushing near the upper support. Specimen retrofitted with diagonal strips failed due to the detachment of the superficial layers of the bricks. This failure is initiated near the upper support and propagates to the lower support. The behavior was less brittle than for unreinforced specimens and the predicted increase in ultimate strength was obtained. A brittle failure initiated in the support and followed by the sliding of the central mortar joint is observed in the case of the specimen retrofitted with strips parallel to the bed joints.

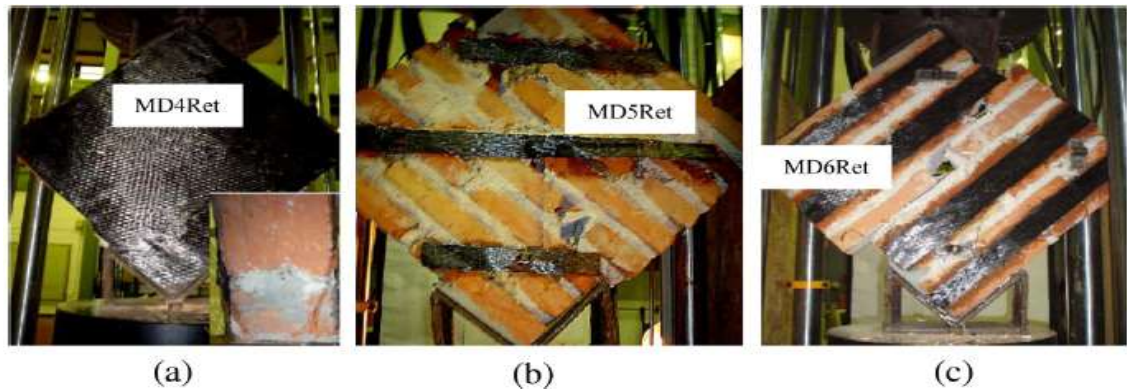


Fig.2.35: Failure patterns of retrofitted panels under diagonal compression. (a) Completely retrofitted. (b) Retrofitted with diagonal bands. (c) Retrofitted with bands parallel to mortar joints (Luccioni and Rougier, 2011).

Luccioni and Rougier (2011) indicated that CFRP does not significantly increase strength under compression normal to bed joints, it improves ductility and failure mode. The increase in deformation capacity can reach 240% of the original value for total retrofitting. Depending on the dimensions and orientation, retrofitting with CFRP increases ductility under in-plane shear, prevents the joints from sliding, and increases the ultimate strength. Regarding the increase in load capacity and ductility, retrofitting with diagonal CFRP bands is effective and relatively economic when compared with total reinforcement. This fact is important when full-sized walls must be retrofitted. Moreover, an optimum length of the reinforcing bands could be obtained in order to balance ductility and strength increase with the amount of CFRP.

Babatunde (2017) investigated Glass, Aramid, and Carbon fiber reinforced polymer (GFRP, AFRP, and CFRP) to reinforce masonry walls. The experimental tests showed that for in-plane loading externally bonded FRP system increases lateral resistance and

improves lateral displacement and for out-of-plane loading improves flexural strength, improves ductility, and increases strain capacity and stability. Near-surface mounted FRP system improves out-of-plane bending and lateral resistance. FRP strip strengthening technique for in-plane loading improves strength, ductility and improves lateral load resistance up to 4 times of unreinforced wall and for out-of-plane loading minimises displacement.

Luccioni and Rougier (2010) assessed the shear behavior of brick–mortar joints strengthened or repaired with CFRP composite. The study showed that the repair of masonry with CFRP composites not only helped in recovering the shear strength but also increased it. On-the-surface bonded fibers (FRP) is a reinforcement method that has been shown that can be used to repair or reinforce concrete and masonry buildings. Masonry walls reinforced with FRP can have a big increase in strength (Hernan and Pablo, 2011). The shape of externally bonded FRP has a significant impact on the effectiveness of strengthening. If FRP reinforcement is applied only across the shear cracks, even if there are flexural cracks, the initial lateral stiffness of the repaired wall may equal the stiffness of the flexural cracked section (Santa-Maria and Alcaino 2011).

According to Hamrat et al, (2019) the repair and strengthening of damaged or undamaged structural members with externally bonded fiber-reinforced polymer (CFRP) sheets has become one of the important challenges confronting civil engineers and researchers. CFRP composite systems when bonded to various damaged or undamaged structural members can significantly improve their mechanical behavior in serviceability conditions, at failure mechanism, and reduce crack propagation by increasing both the stiffness and the load capacity resistance.

2.18. Conclusions

CFRP improves the behavior of masonry, increasing ductility, ultimate strength, and stiffness in some cases. Although retrofitting with CFRP does not significantly increase strength under compression normal to bed joints, it improves ductility and failure mode. Depending on the dimensions and orientation, retrofitting with CFRP increases ductility under in-plane shear, prevents the joints from sliding, and increases the ultimate strength. According to experimental studies on strengthening masonry walls with FRP composites, the following results can be drawn:

1. The single-sided reinforced masonry walls with seismic textile and relevant plaster improve shear resistance by 68% and deformability capacity by 19%. However, two-sided reinforcing provided remarkable additional strength by up to 168% and increases ductility by 46%.
2. Shear strength of triplets strengthened with the same amount in terms of area reinforced with GFRP but with different grades was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity possessed by different grades of GFRP. However, the gain in shear strength increased with an increasing amount of GFRP glued.
3. The compressive strength of the strengthened clay brick walls is 147.31% to 319.35% higher than the URM wall
4. The FRP reinforced system restrains the URM wall from shear failure and shear sliding; it enhanced their strength by up to 319%.
4. All the used configurations, improved considerably the load-bearing capacity of the specimens from 2.5 to 4.2 times.
5. The FRPs retrofitting systems are effective in increasing the in-plane strength and the deformability of URM walls.

6. Regarding the increase in load capacity and ductility, retrofitting with diagonal CFRP bands is effective and relatively economic when compared with total reinforcement. This fact is important when full-sized walls must be retrofitted.
7. The stiffness and ultimate load capacity of specimens repaired with CFRP bands increased significantly.

Chapter 3

**Mechanical properties of brick,
mortar, triplet masonry and
reinforced triplets**

3.1. Introduction

Unreinforced masonry buildings are usually used for their simplicity, durability, and economic sustainability. Various types of masonry structures such as stone masonry, brick masonry, and concrete block masonry are commonly used in many countries for construction. In Iran, brick masonry is widely used for construction and based on the availability and cost, different types of brick and mortar have been adopted for masonry structures. Generally, masonry resists compressive stresses very well but has low tensile and shear strength. This leads to very early cracking and it is the main reason why unreinforced masonry is not allowed to be used as the main structural material in seismic zones. Any experimental, numerical or analytical analysis can be accurate only if the mechanical properties of masonry and its components are known. Reliable results can be produced with numerical tools if appropriate and sufficient material properties are established.

Masonry is a mixed composite structure with brick and mortar, mortar can be lime or a mixture of cement, lime, sand and water. Masonry properties are different from one structure to the next depending on the type of brick units and mortar. Variability of masonry properties includes the dimension of units, mortar joint width and head joints, arrangement of brick units and workmanship. Therefore, the properties of mortar, brick units and masonry triplets should be examined to understand the behaviour of masonry structures.

The prediction of the structural capacity of unreinforced masonry buildings under combined compression and shear loading requires an accurate characterisation of the masonry strength and general response under shear stresses. The experimental investigations of shear strength parameters typically rely on shear walls, or on standard triplets (Segura et, al 2021). According to standards BS EN 1052-3 (2007), the triplet shear test allows the determination of the in-plane shear strength of the horizontal bed joint in masonry. The triplet test has been mainly applied in both research and industrial fields and is used to characterise many different materials (Pelà, et, al. 2017 and Cavalheiro & Cavalheiro, 2000). According to Sarhosis and Sheng (20014) the allowable pre-compression level has already been investigated by various researchers;

experimental evidence has pointed out that this parameter is another important factor that influences the masonry elements' in-plane shear strength and deformability.

According to Hernoune, et al, (2022), In order to extend the life of masonry structures, strengthening or repairing by employing new techniques have been developed. Many of these strengthening techniques, including the use of fiber-reinforced polymer and (FRP) composites, have aided in reinforcing masonry structures. These composites are manufactured in different features depending on the fiber material type, such as carbon fiber (CFRP), glass (GFRP), and aramid (AFRP). FRP strips offer the possibility of application by gluing on externally bonded of element using the near-surface mounted (NSM) technique.

This chapter presents an experimental programme designed to establish the necessary material properties for masonry components. Characterisation of masonry components, their physical properties and behaviour under compressive, tests. Masonry components under investigation represent the most common types of bricks and mortar used in the country. Also, the materials for triplets strengthening by application of CFRP are tested and the results are presented in this section.

3.2. Mechanical properties of brick, mortar and triplets

- **Test set up for measurements of elastic modulus and compressive strength**

Based on the literature review, in the first stage, an experimental test program was established, aiming to identify the mechanical properties of masonry and its components. The second stage was a compression test on unreinforced and CFRP-reinforced triplets.

The mechanical properties including elastic modulus (Young's modulus) of the brick, mortar, and triplets are the main factors that affect the performance of masonry structures. In this research, experiment tests were designed to measure the elastic modulus of masonry and mortar samples. In addition, because of the direct relation between compressive strength and the elastic modulus, the compressive strength of these samples was measured. To measure the elastic modulus and compressive strength of mortar, the cylinder specimens were constructed. The diameter of the cylinders was

150mm and the height was equal to 300mm (Figures 3.1a and 3.1b). Also, thirty-five triplet masonries were constructed in the laboratory with mortar M4 and solid bricks. The thickness of the mortar between the brick's layer was 10mm. (Figures 3.2A and 3.2B)

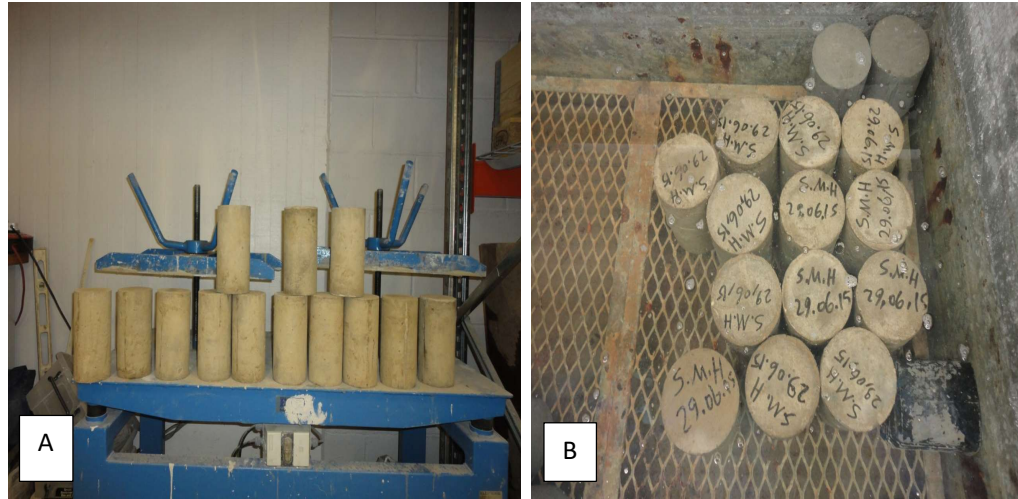


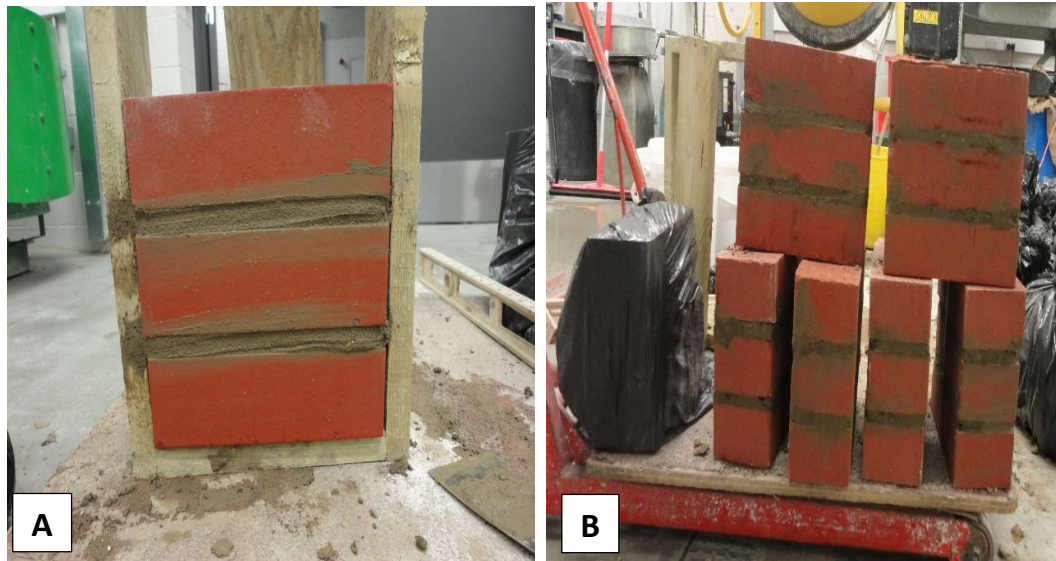
Figure 3.1: Mortar samples

The mortar M4 with Portland cement was used in this study according to the UK National Annex (NA to BS EN 1996-1-1:2005) Table 3-1.

Compressive strength class ^{A)}	Prescribed mortars (proportion of materials by volume) (see Note)				Mortar designation
	Cement ^{B)} : lime : sand with or without air entrainment	Cement ^{B)} : sand with or without air entrainment	Masonry cement ^{C)} : sand	Masonry cement ^{D)} : sand	
M12	1 : 0 to ¼ : 3	1 : 3	Not suitable	Not suitable	(i)
M6	1 : ½ : 4 to 4½	1 : 3 to 4	1 : 2½ to 3½	1 : 3	(ii)
M4	1 : 1 : 5 to 6	1 : 5 to 6	1 : 4 to 5	1 : 3½ to 4	(iii)
M2	1 : 2 : 8 to 9	1 : 7 to 8	1 : 5½ to 6½	1 : 4½	(iv)

Table 3.1. Different types of mortars (Table 2 of UK National Annex to EC6)

Mixing and casting of the mortar were performed following the BS EN 998-2 (2003). Sand and cement were mixed with the proper amount of water (the water-cement ratio was 1:3). All samples were checked to have a horizontal surface using a bubble level. After the construction of each sample, it was left for more than 28 days for curing and developing its design capacity.



Figures 3.2A and B: Construction of Triplet masonry.

3.3. Experimental investigations

The material properties of brickwork components are important to understand the behavior of CFRP composite reinforced masonry through experimental study and later this data is needed to develop accurate finite element models. The strength and elastic modulus of bricks, mortar, and triplet masonry were calculated through testing. Tests were carried out in controlled laboratory conditions, 30 days after building the mortar cylinders and triplets. Axial compression test was conducted with an average load application rate of 0.6 kN/s according to the compressing test machine manual and each test lasted 15 to 25 minutes.

3.4. Bricks tests

The bricks were tested under compression to find the modulus of elasticity and compressive strength. Compressive strength is often considered as the most important mechanical characteristic of construction materials but its measurement has always been a subject of controversy. Regardless of the type of material tested (concrete, stone, fired bricks, or earth bricks), the experimental measurement of the compressive strength depends strongly on the dimensions of the specimens. For cement-based materials (concrete or mortar), this problem is easily solved by standardizing the size of the test

specimens. For example, the European standard for concrete (EN 206-1) authorizes measurement of the compressive strength of concrete on cubes or on cylinders with an aspect ratio equal to two (European Standard (2004) EN 206-). For masonry blocks, the British Standard considers the effects of the geometry of the blocks in the determination of the compressive strength of masonry (Standard British (1992) BS 5628-1).

Six bricks were tested with an average size of 220 mm $\times$ 104 mm $\times$ 65 mm. The specimens were tested under load control in the universal testing machine. Based on compressive strength, stress and strain, the average modulus of elasticity of the bricks was calculated.

To measure the elastic modulus of brick samples according to the testing machine manual, a compressive load with the rate of 0.6 kN/sec was applied to the sample (Figures 3.3 A and B). The distributed load was applied on the surface of the sample and after loading, the deformation was measured by a mechanical strain gauge in the machine.

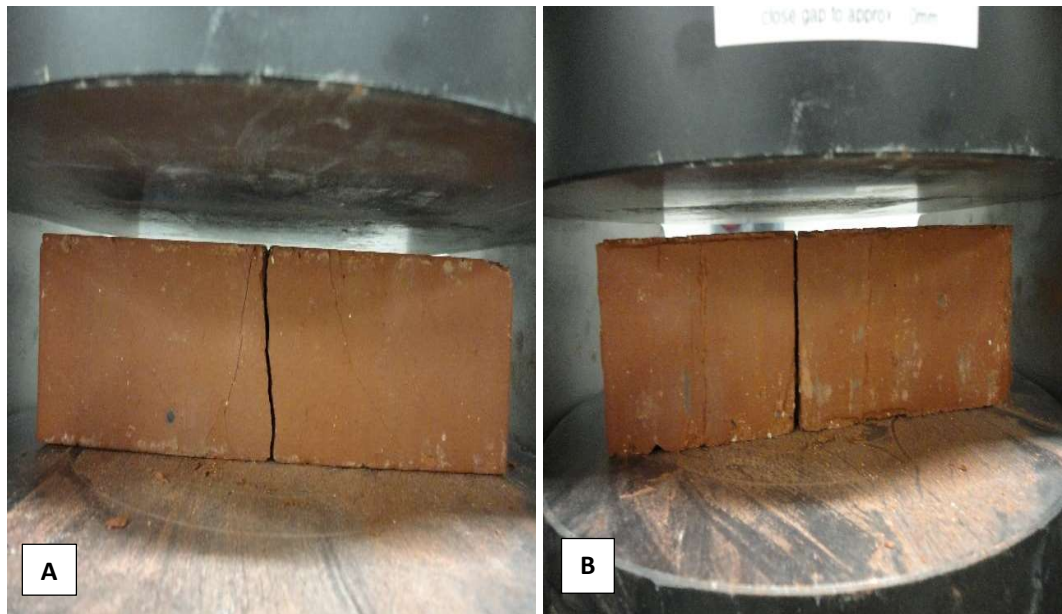


Figure 3.3: Compression tests on bricks.

3.5. Mortar tests

As stated by Haach [2009], mortar is one of the components of masonry that is responsible for uniform stress distribution, correction of irregularities of units, and accommodation of thermal deformations and shrinkage. It plays a major role in the final behavior of masonry in terms of stresses and strains. Its participation in the compressive and shear strength of masonry is appreciated by many codes. In this research, cement mortar was used in all subsequent tests, and thus, primarily physical and mechanical properties were tested.

Twelve mortar cylinders were constructed with mortar M4 (1: ordinary Portland cement, 4: natural river sand and the water-cement ratio was determined to be 1.3.). To mix and prepare all mortar specimens regular tap water was used and mixed with an electric mixer. All specimens were checked with a bubble level to have a horizontal surface. After the construction of samples, they were left for more than 28 days to allow curing and to achieve their maximum design capacity.

Mortar specimens were first cleaned from any loose material leftover from the previous test, to measure the elastic modulus of mortar samples, three strain gauges were attached to each sample, and an axial compressive load with the rate of 0.6 kN/sec was applied to each sample according to testing machine manual (Figures 3.4A) and all mortar specimens failed immediately after reaching the ultimate failure load. The failure mode of mortar specimens is presented in (Figure 3.4B).

The vertical load was applied without shocks and the load was increased uniformly until failure of the specimens and the maximum compressive strength was automatically recorded with the testing system set-up. All mortar specimens showed similar failure modes expressed by crushing the specimen over the whole height at the end of loading, the deformation was measured using a mechanical strain gauge attached to the sample.

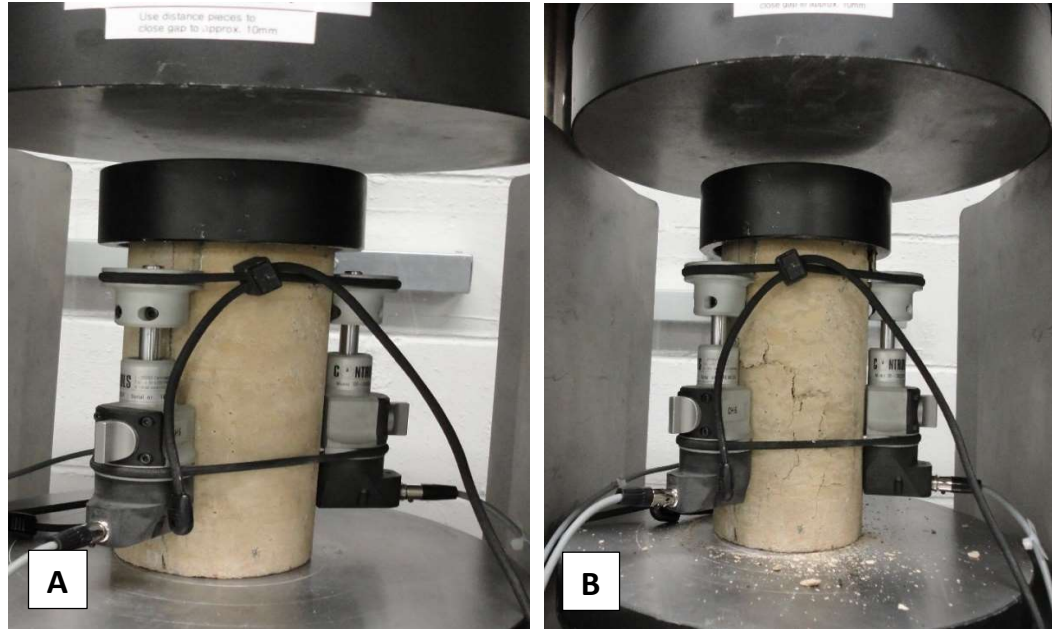
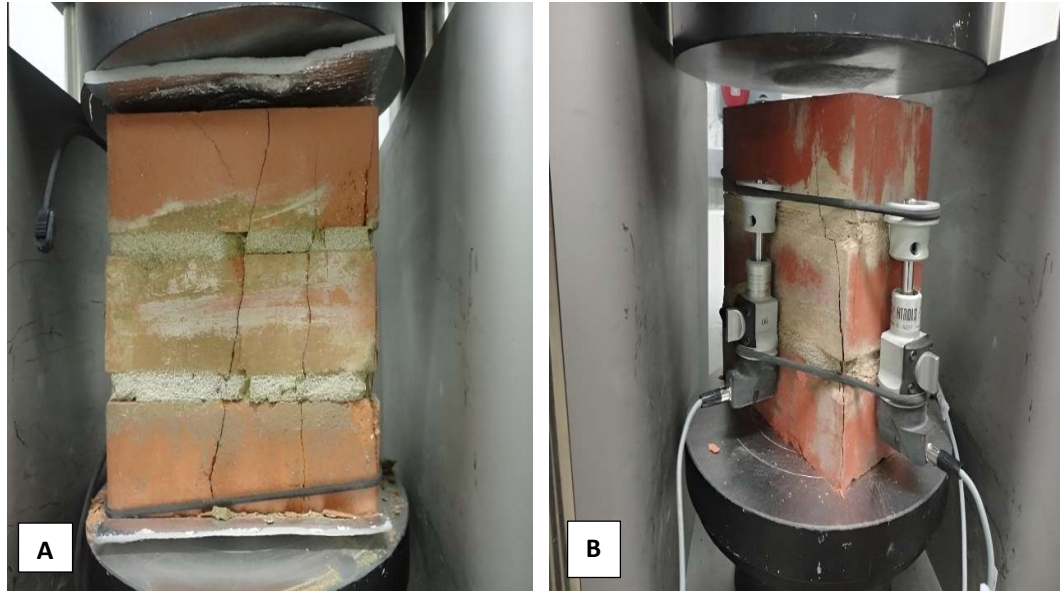


Figure 3.4: Compression test on mortar, before and after failure.

3.6. Triplet masonry tests

Overall, three full-scale specimens were constructed with a nominal dimension of 215mm×104 mm ×65 mm (height, length, thickness,). The masonry triplets were built with solid clay bricks and the thickness of mortar for each layer of the triplets was approximately 10mm. Due to the high-water absorption capacity of bricks, the bricks were soaked before building the specimens, so that the rapid absorption of mortar water, which leads to degradation of mortar and thus degradation in joint (bonding) between bricks and mortars, can be prevented. When the brick wall construction was finished, for curing, the walls were wetted two times a day for three days.

The horizontal surface of all specimens was checked with bubble level. The axial compressive load with the rate of 0.6 kN/sec was applied to each sample and after loading, the deformation was measured using three mechanical strain gauges attached to the sample. This test was repeated Six times (Figures 3.5A and 3.5B).



Figures 3.5: Compression test on triplet masonry

3.7. Measurement of the elastic modulus of samples

The modulus of elasticity (Young's modulus) E is a material property, that describes its stiffness and is therefore one of the most important properties of solid materials. Mechanical deformation puts energy into a material. The energy is stored elastically or dissipated plastically. The way a material stores this energy is summarized in stress-strain curves. Stress is defined as force per unit area and strain as extension or contraction per unit length. When a material deforms elastically, the amount of deformation likewise depends on the size of the material, but the strain for a given stress is always the same and the two are related by Hooke's Law (stress is directly proportional to strain):

$$\sigma = E \cdot \varepsilon$$

where σ is stress [MPa]

E is modulus of elasticity [MPa]

ε is strain [unitless, or %]

From the Hooke's law the modulus of elasticity is defined as the ratio of the stress to the strain:

$$E = \sigma / \varepsilon \quad [\text{MPa}]$$

$$\sigma = E / \varepsilon$$

Strain is defined as the change of the length divided by the original (initial) length.

$$\varepsilon = \Delta L / L_0 = (L_1 - L_0) / L_0 \quad [\text{unitless or \%}]$$

where ΔL is change of the length [m]

L_1 is length after extension [m]

L_0 is original (initial) length [m]

Stress: $\sigma = P/A$, N/mm²

Strain: $\varepsilon = \Delta L/L$, mm/mm

Elastic modulus: $E = (P/A)/(\Delta L/L)$, N/mm²

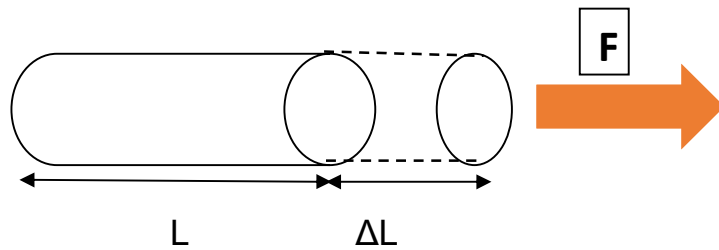


Figure 3.6: Determination of the strain

3.8. Measurement of the elastic modulus of triplet masonry samples

The test results for measurement of the elastic modulus for triplet masonry samples are summarized in table 3.2. Elastic modulus is presented as ratio of stress to strain.

The average value of the elastic modulus at initial stages of loading is taken as data for the modelling. The elastic modulus for each sample is an average of stress/strain ratios in 13 steps of loading.

	load (kN)	Displacement: mm	Stress, $\sigma = P/A$, N/mm ²	Strain, $\epsilon = \Delta L/L$	$E = (P/A)/(\Delta L/L)$, N/mm ²
1	0	0	0	0	0
2	10.247	0.00914	0.462723	9.14E-05	5062.614
3	20.287	0.0181	0.916098	0.000181	5061.317
4	30.327	0.0277	1.369474	0.000277	4943.949
5	40.3	0.03585	1.819824	0.000359	5076.217
6	50.211	0.0446	2.267374	0.000446	5083.798
7	60.185	0.05502	2.717769	0.00055	4939.602
8	70.225	0.06482	3.171145	0.000648	4892.232
9	80.363	0.070928	3.628946	0.000709	5116.379
10	90.272	0.080906	4.076406	0.000809	5038.446
11	100.279	0.09039	4.528291	0.000904	5009.725
12	110.3	0.098824	4.980808	0.000988	5040.08
13	120.392	0.109304	5.436532	0.001093	4973.772
14	130.334	0.118594	5.885482	0.001186	4962.715
Elastic modulus of masonry triplets					5015.65 N/mm ²

Table 3.2: The average elastic modulus of 4 triplet masonry samples.

The stress-strain curve, stress-displacement curve and load-displacement curve for masonry triplets are presented in figures 3.7 to 3.9.

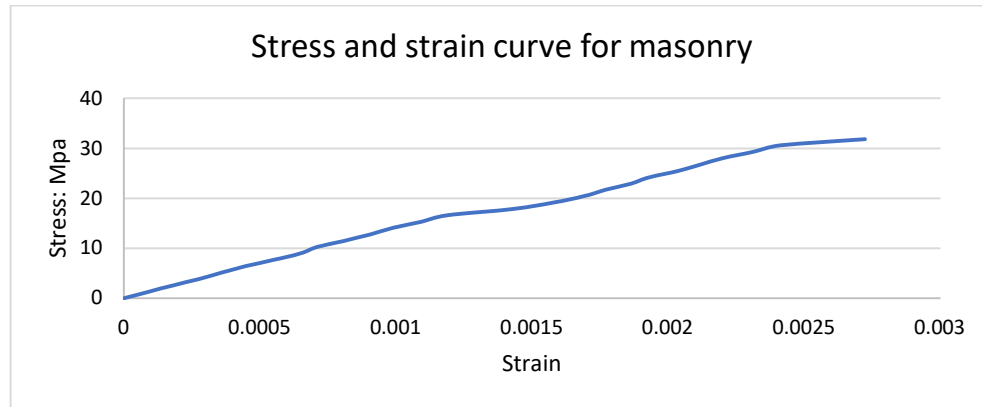


Figure 3.7: Stress and strain curve for masonry triplets.

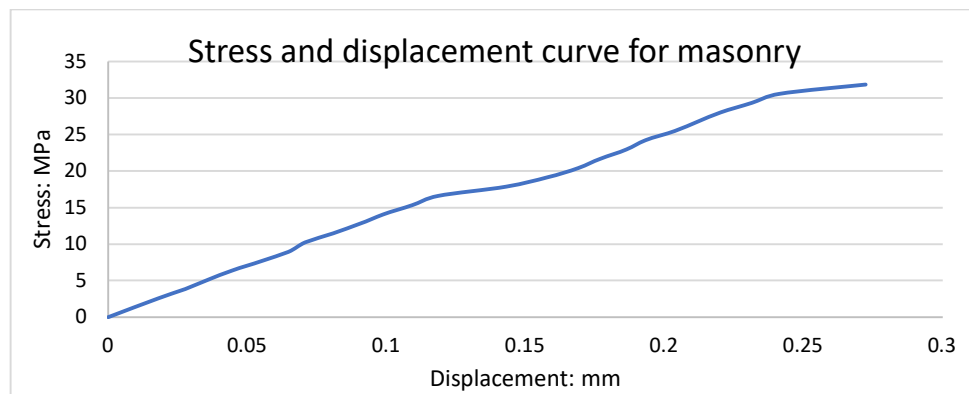


Figure 3.8: Stress and displacement curve for masonry triplets.

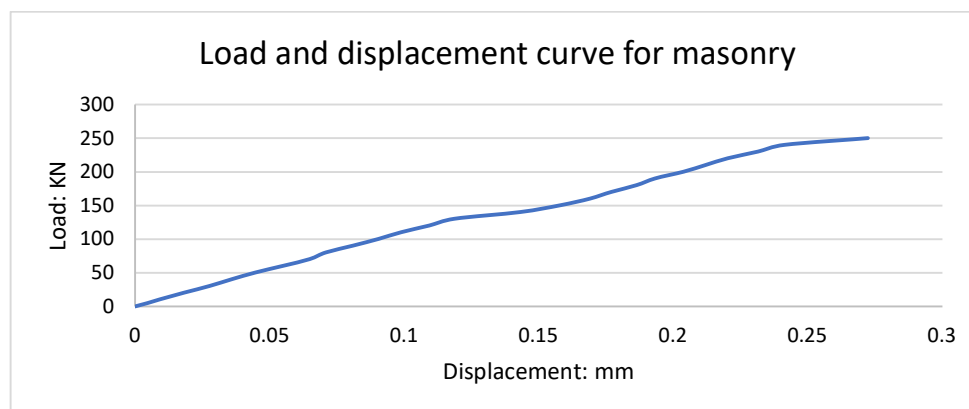


Figure 3.9: Load and displacement curve for masonry triplets.

3.9. Measurement of the elastic modulus of London solid brick samples

The test results for the measurement of the elastic modulus for London solid brick samples are summarized in table 3.3. Elastic modulus is presented as the ratio of stress to strain. The average value of the elastic modulus at the initial stages of loading is taken as data for the modeling. The elastic modulus for each sample is an average of stress/strain ratios in 13 steps of loading (the average of 6 samples).

	load (kN)	Displacement: mm	Stress, $\sigma = P/A$, N/mm ²	Strain, $\epsilon = \Delta L/L$	E = (P/A)/($\Delta L/L$), N/mm ²
1	0	0	0	0	0
2	10.576	0.0047	0.477579589	7.23E-05	6604.824104
3	20.616	0.0094	0.930955069	0.000145	6437.455263
4	30.525	0.013775	1.378414992	0.000212	6504.317567
5	40.369	0.018025	1.822939716	0.000277	6573.707712
6	50.572	0.02325	2.283675773	0.000358	6384.469904
7	60.612	0.0275	2.737051253	0.000423	6469.393871
8	70.456	0.03125	3.181575977	0.000481	6617.678031
9	80.529	0.0363	3.636441635	0.000558	6511.534608
10	90.47	0.040375	4.085346579	0.000621	6577.028549
11	100.444	0.044575	4.535741702	0.000686	6614.093341
12	110.518	0.0503	4.990652517	0.000774	6449.153353
13	120.557	0.054475	5.44398284	0.000838	6495.803297
14	130.532	0.05875	5.89442312	0.000904	6521.48941
The elastic modulus of London solid brick (average of 6 samples)					6520 .073 N/mm ²

Table 3.3: The elastic modulus of London solid brick (average of 6 samples)

The stress-strain curve, stress-displacement curve and load-displacement curve for London solid bricks are presented in figures 3.1 to 3.12.

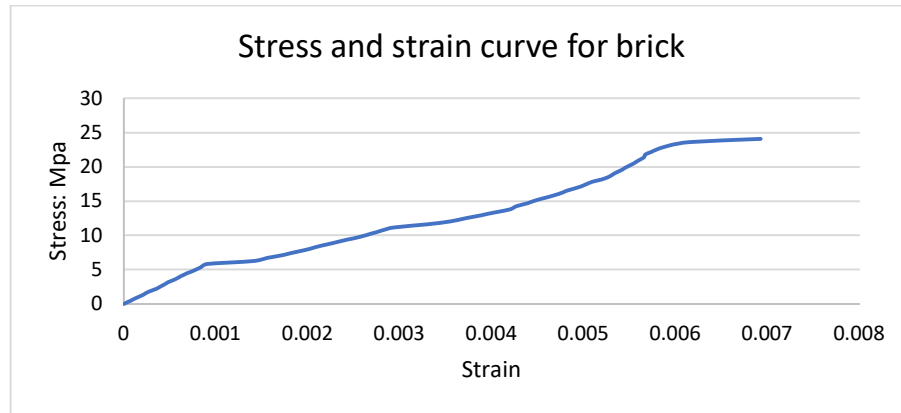


Figure 3.10: Stress and strain curve for London solid brick (average of 6 samples)

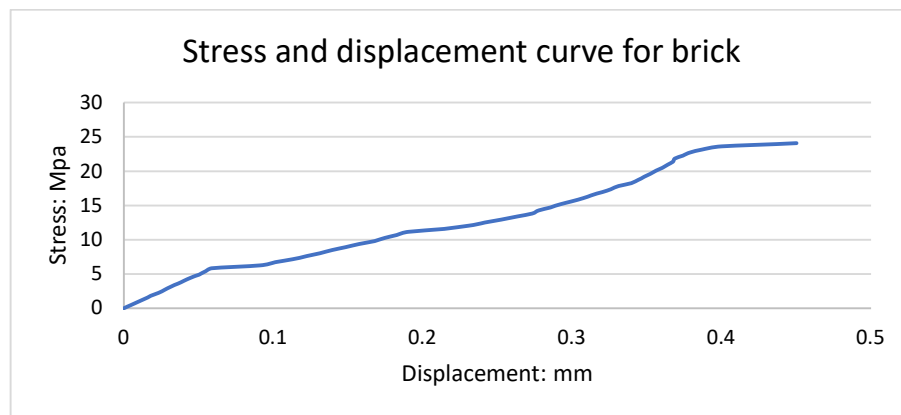


Figure 3.11: Stress and displacement curve for London solid brick (average of 6 samples)

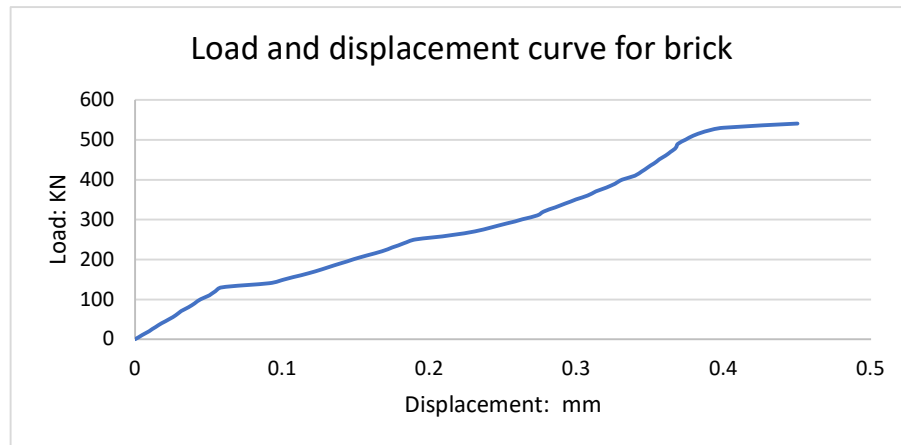


Figure 3.12: Load and displacement curve for London solid brick (average of 6 samples)

3.10. Measurement of the elastic modulus of mortar M4 samples

The test results for the measurement of the elastic modulus for mortar M4 samples are summarized in table 3.14. Elastic modulus is presented as a ratio of stress to strain.

The average value of the elastic modulus at the initial stages of loading is taken as. The elastic modulus for each sample is an average of stress/strain ratios in 13 steps of loading (the average of 6 samples).

	load (kN)	Displacement: mm	Stress, $\sigma = P/A$, N/mm ²	Strain, $\epsilon = \Delta L/L$	$E = (P/A)/(\Delta L/L)$, N/mm ²
1	0	0	0	0	0
2	10.182	0.014683	0.576166	0.000147	3923.943
3	20.124	0.02825	1.138751	0.000283	4030.975
4	30.196	0.0438	1.708692	0.000438	3901.123
5	40.105	0.055517	2.269409	0.000555	4087.798
6	50.08	0.071002	2.833861	0.00071	3991.26
7	60.054	0.084787	3.398257	0.000848	4008.009
8	70.094	0.099353	3.966388	0.000994	3992.204
9	80.101	0.113167	4.532651	0.001132	4005.288
10	90.043	0.128167	5.095235	0.001282	3975.476
11	100.148	0.137	5.667044	0.00137	4136.528
12	110.058	0.160127	6.227818	0.001601	3889.307
13	120.097	0.168781	6.795892	0.001688	4026.463
14	130.17	0.179595	7.36589	0.001796	4101.389
The elastic modulus of mortar M4 (average of 6 samples)					4005.366 N/mm ²

Table 3.13: The elastic modulus of mortar M4 (average of 6 samples).

The stress-strain curve, stress-displacement curve and load-displacement curve for mortar M4 are presented in figures 3.13 to 3.15.

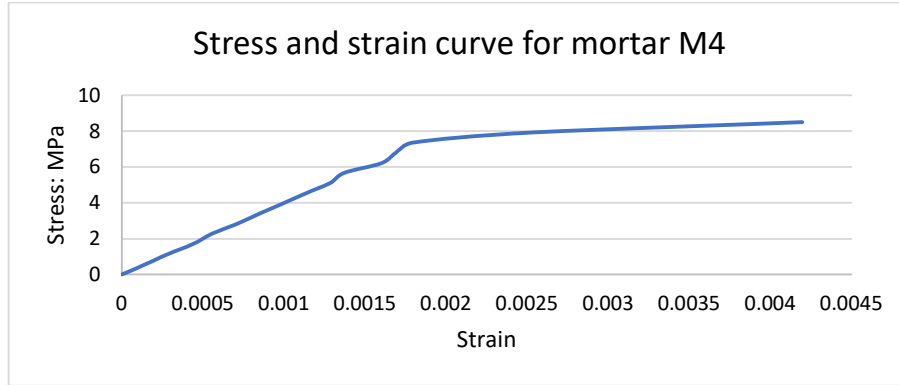


Figure 3.13: Load and displacement curve for mortar M4 samples (average of 6 samples).

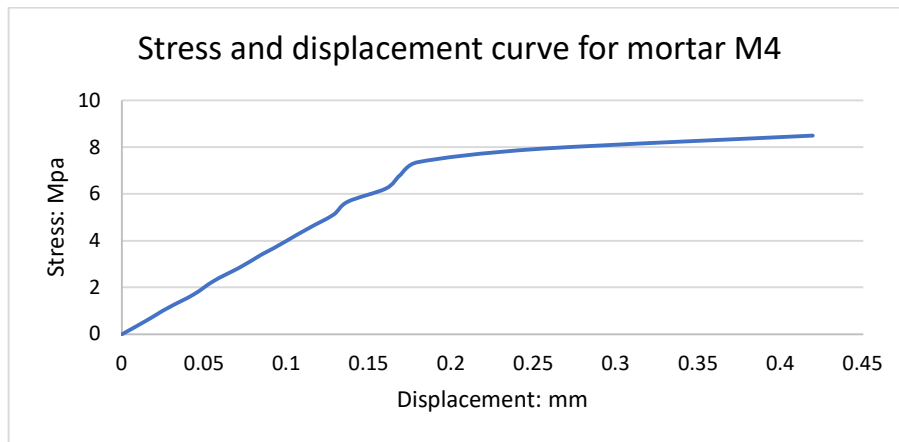


Figure 3.14: Load and displacement curve for mortar M4 samples (average of 6 samples).

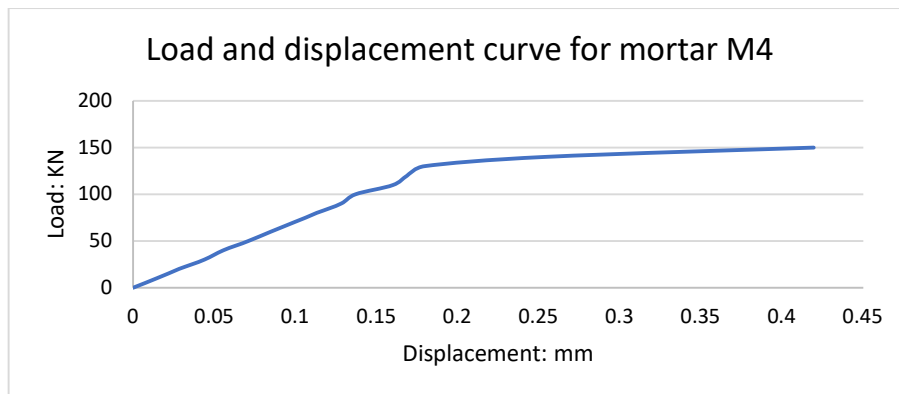


Figure 3.15: Load and displacement curve for mortar M4 samples (average of 6 samples).

Figures 3.16 and 3.17 show the comparison of elastic modulus of brick, mortar M4 and triplet masonry and comparison of maximum stress, load and displacement at failure point of brick, mortar M4 and triplet masonry.

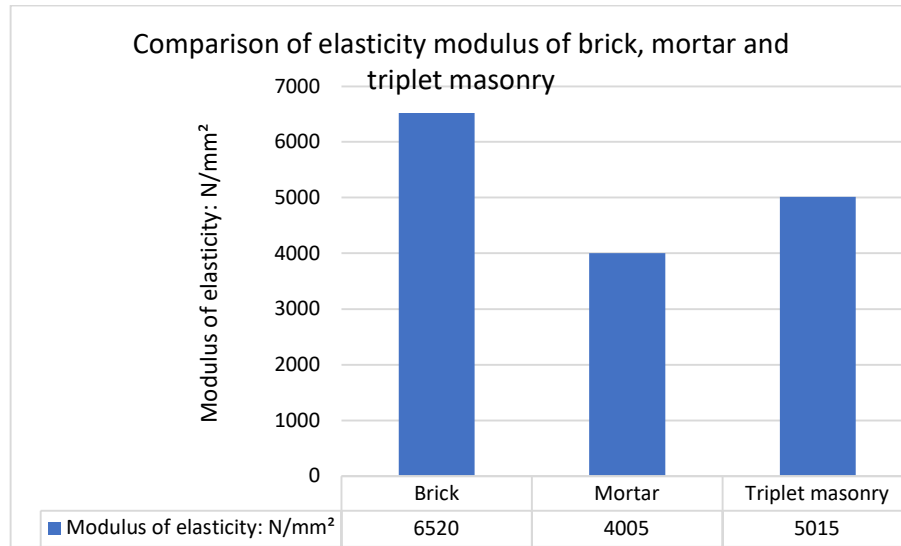


Figure 3.16: Comparison of elastic modulus for mortar, brick and triplet masonry samples

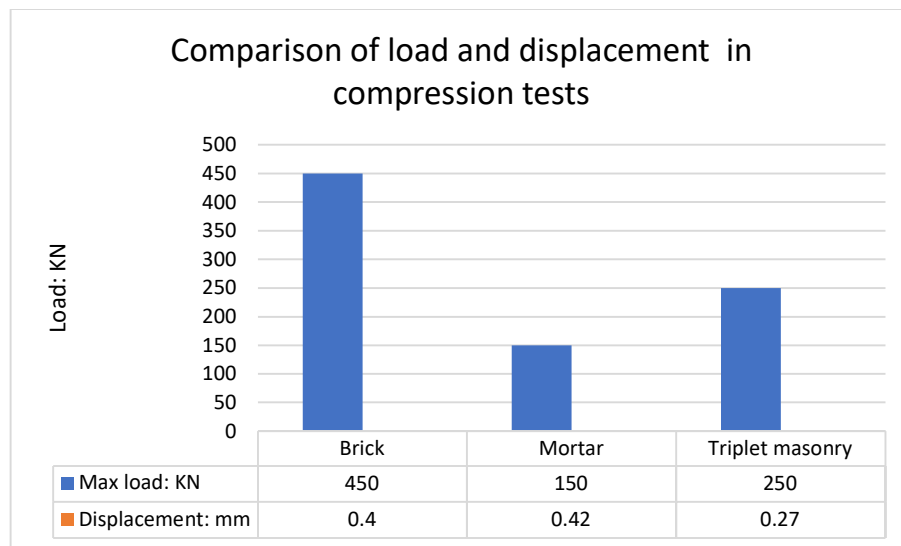


Figure 3.17: Maximum load and displacement at failure point of brick, mortar M4 and triplet masonry.

The test results of the load and displacement, stress and displacement and stress-strain for triplet masonry, mortar M4, bricks and comparison curves are presented in figure 3.18 to 3.20.

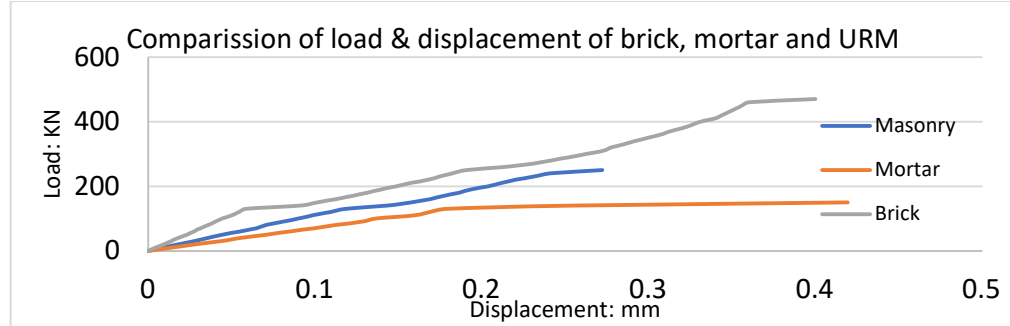


Fig 3.18: Comparison of load and displacement curve for mortar, brick and URM samples

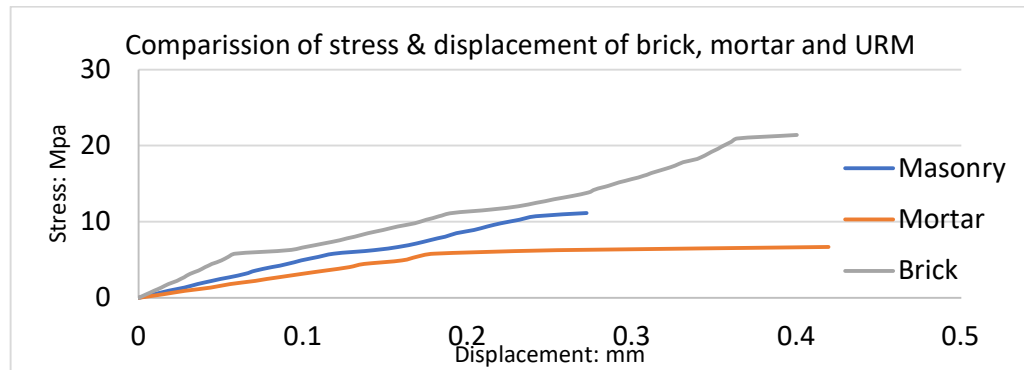


Fig 3.19: Comparison of Stress and displacement curve for mortar, brick and URM samples

The test results of the Stress-Strain curve for Triplet masonry, Mortar, bricks and comparison curves are presented in Figure 3.10.

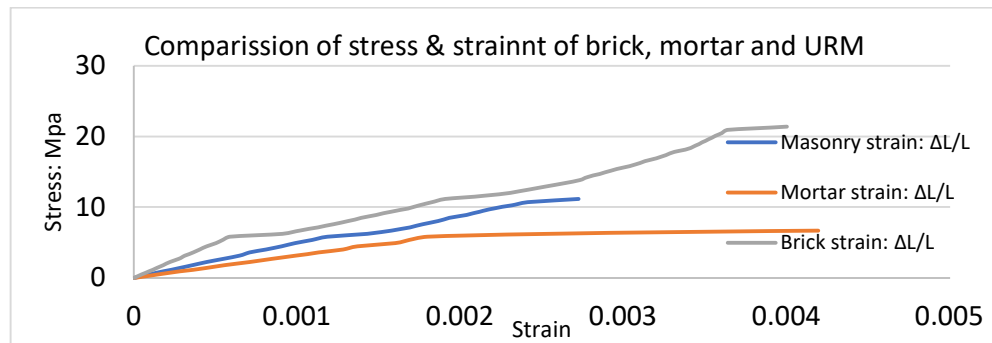


Fig 3.20: Comparison of stress-strain curve for mortar, brick and URM samples

3.11. Experimental study on the compression behaviour of strengthened triplet masonry

3.11.1. General setup and preparation for experimental tests

Tests were carried out in laboratory conditions, 30 days after building the triplets. An axial compression test was conducted with an average load application rate of 0.6 KN/s. The whole test lasted 15 to 25 min. The main results are presented by the force-displacement curve. Following the literature review, an additional experimental investigation was planned for an appropriate assessment of the behavior of Triplet masonry under compression strengthened with FRP. The dimension of triplet masonry is with dimensions 215mm×215mm×103mm (height × length × thickness) as shown in Figure 3.21. The ultimate load was accepted at the point of the triplet's collapse.

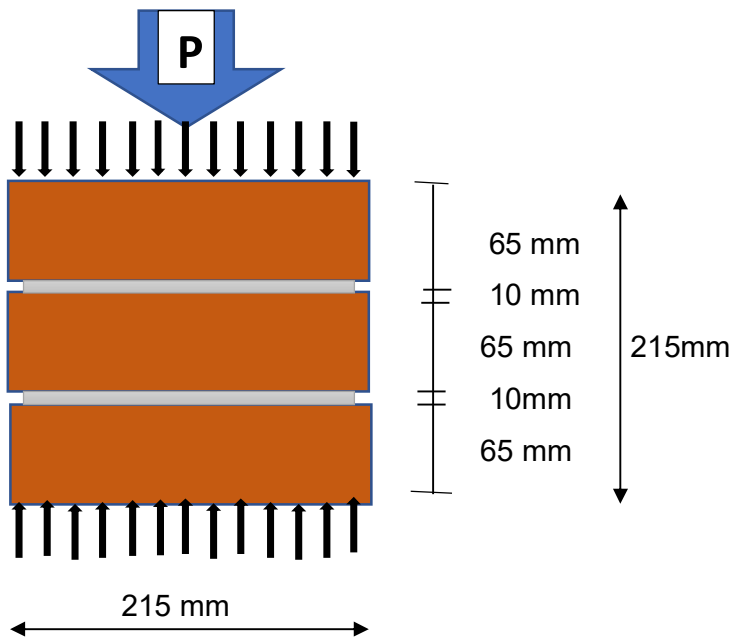


Figure 3.21: Three solid bricks and two mortar layers with 10mm thickness.

The dimensions of tested triplets in this research were designed according to the dimensions of the test machine. However, the purpose of this study is the comparison of the compression behavior of different configurations of FRP-strengthened masonry. For this part of the experiment study, thirty-five specimens were constructed with London solid clay bricks and mortar M4 (Figure 3.22A and B). The brick units used to build the samples were solid London bricks with dimensions 215mm×103mm×65mm.



Figure 3.22: The brick units used to build the triplet samples

The bricks were in the water for 30 minutes, this pre-wetting helped to reduce the suction of water in mortar matrix by bricks during the strengthening process. Then bricks layered with mortar M4 (1: ordinary Portland cement, 4: natural river sand, and the water-cement ratio was 1.3). The thickness of bed joints between brick rows was kept at about 10mm. Each sample was constructed inside a wooden frame. To construct samples exactly in one vertical surface, a wooden frame was designed and each sample was constructed inside the wooden frame (Figure 3.24A). After the construction of each triplet masonry, it was cured for at least 28 days to allow for the development of its maximum load capacity. During the curing period, masonry walls were located in the structural laboratory with an environmental temperature of about 20°C.

3.12. Preparation of adhesive mix and attaching FRP composites to the samples

The carbon, fibre-reinforced polymer composites (CFRP) with the resin epoxy matrix were attached to the triplets. (Figure 3.23 and 3.24). The use of these composites is according to the technical report of the factory in this research. The characteristics of FRP and resin are presented in tables 3.4 and 3.5.



Figure 3.23: FRP



Figure 3.24: Resin

Technical data		
	Test Type 78	Type 70-75
Sp. Gr.	1.05 g/cm ³	1.2 g/cm ³
Pot life a 20°C (500g mass)	50 min.	30 min.
Harding (start) at 20°C	7 hrs	6/7 hrs
Harding (complete) at 20°C	7 days	7 days
Resistance to compressive stress	46 MPa	82 MPa
Resistance flexural/tensile stress	40 MPa	45 MPa
Adhesion to concrete	> 2.5 MPa	> 2.5 MPa
Minimum temperature for use	>= 10°C	>= 10°C

Table 3.4: The characteristics Resin



Figure 3.25: Strengthening with one layer of CFRP sheets on both sides



Figure 3.26: Strengthening with two layers of CFRP sheets on both sides

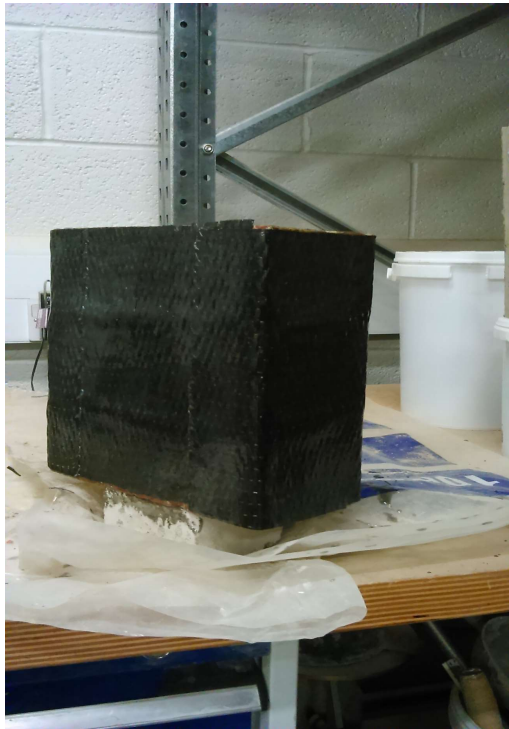


Figure 3.27: Strengthening with one layer of CFRP wrapping on four sides



Figure 3.28: Strengthening with two-layer of CFRP wrapping on four sides



Figure 3.29: one layer of CFRP strips on both sides Figure 3.30: two layers of CFRP strips on both sides

Immediately after the application of the adhesive mix, the CFRP composites were applied to the surface of the samples and pressed using a rubber roller to remove as much air between the composites and the surface of the triplet masonry as possible.

3.13. Experimental results for compression test on the triplet masonry

The experimental study on the triplet masonry produced results for the compression behavior of strengthened masonry under uniform pressure in terms of displacement, and maximum strength with and without FRP. As mentioned before, the displacement was measured by using 3 gauges. Thirty-five triplet masonries were constructed in the structural laboratory to study the compression and In-plane behavior of different configurations of FRP-strengthened masonry. Each test was repeated four times.

FRP composites were attached to masonry triplets. Before applying CFRP, to remove minor imperfections like small chunks of mortar to obtain a plane surface, the specimens were scrubbed with sandpaper. Epoxy resin was mixed with a hardener according to the manufacturer's manual. A coat of the prepared glue was applied on the

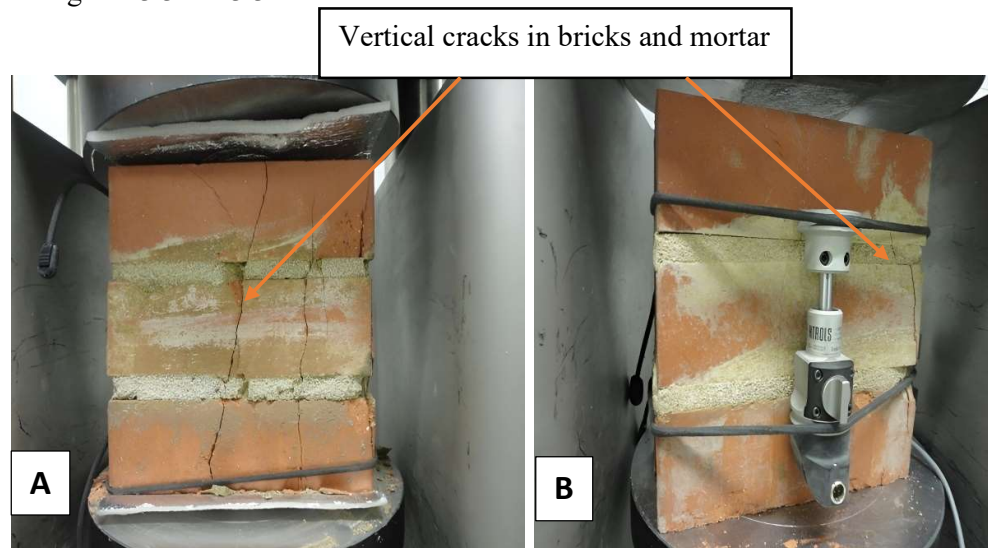
prism's surface followed by applying CFRP to the masonry triplets by wrapping it around the prism or sides.

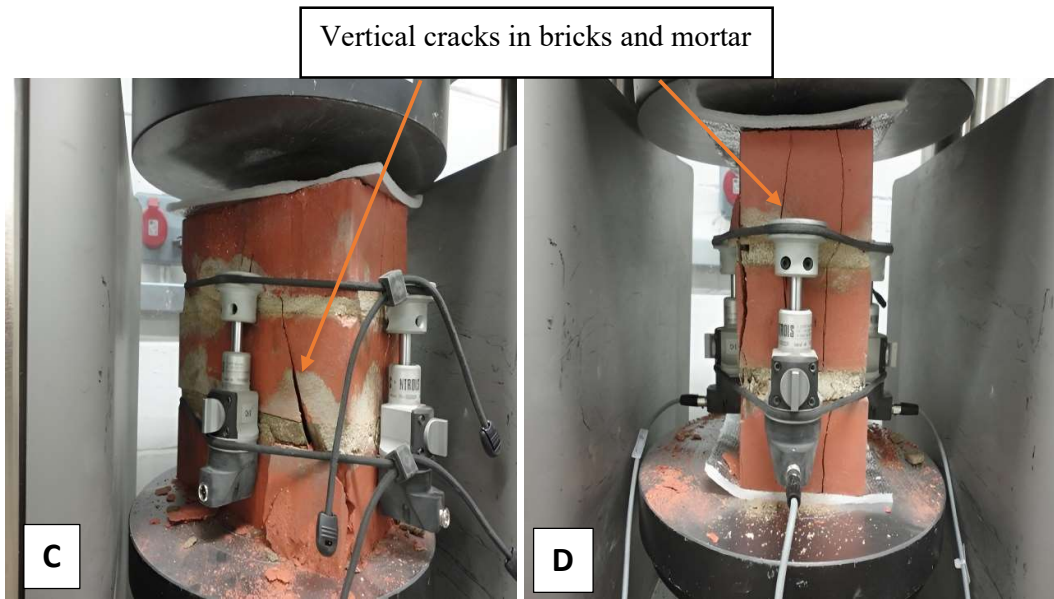
3.14. Compression test on triplet masonry without CFRP

(URTM)

Four triplet samples were constructed without FRP (URM) as the control sample. No cracking was observed in the triplet up to the pressure of 200 kN. Under increasing compression loading the first signs of distress to the triplet were the crushing of the mortar and vertical cracking of the bricks. The mortar joints started to crumble when the load was around 200kN. When the pressure exceeded 200 kN, some small vertical cracks appeared in the middle and corners of the samples. Small cracks started to appear in some brick units when the load was over 200kN. Vertical splitting cracks developing on four faces of the walls gradually led to the failure of triplets. This result is due to the difference in movement properties of the two materials. Under stress, the softer mortar tends to spread more than the stiffer and stronger bricks.

Figures (3.31A to 3.31D) show the obtained failure modes after testing the compressive strength. All triplets exhibited similar failure modes and brittle failure. The masonry triplets initially showed strengthening behavior, followed by elastic behavior until the appearance of the first cracks. Vertical cracks occurred first, followed by cracks in the unit mortar interface. After reaching the compressive strength of the bricks, extensive cracking and crushing of the units and the mortar was obtained. The results can be seen in Figures 3.32 to 3.34.





Figures 3.31A to 3.31D: The obtained failure modes after testing the compressive strength.

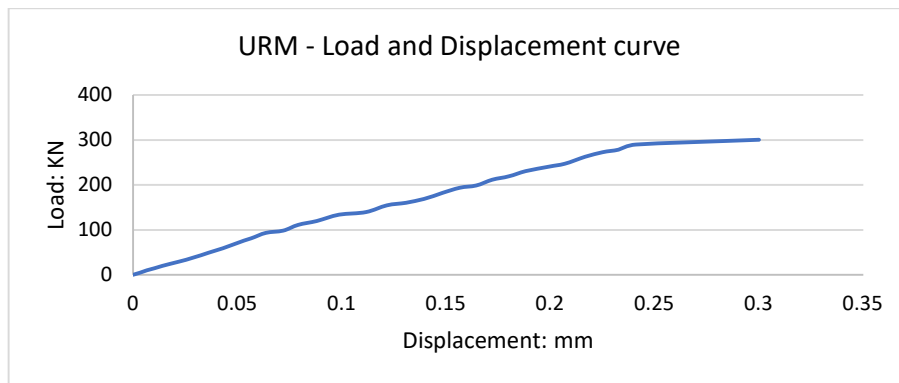


Figure 3.32: URM - Load and Displacement curve

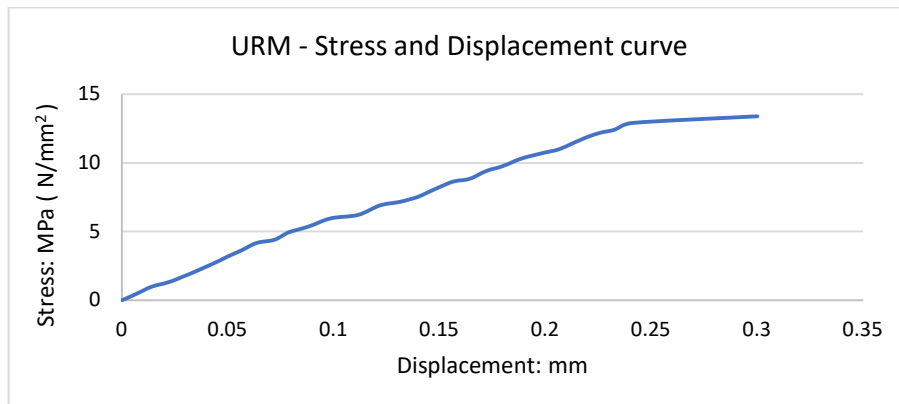


Figure 3.33: URM - Stress and Displacement curve

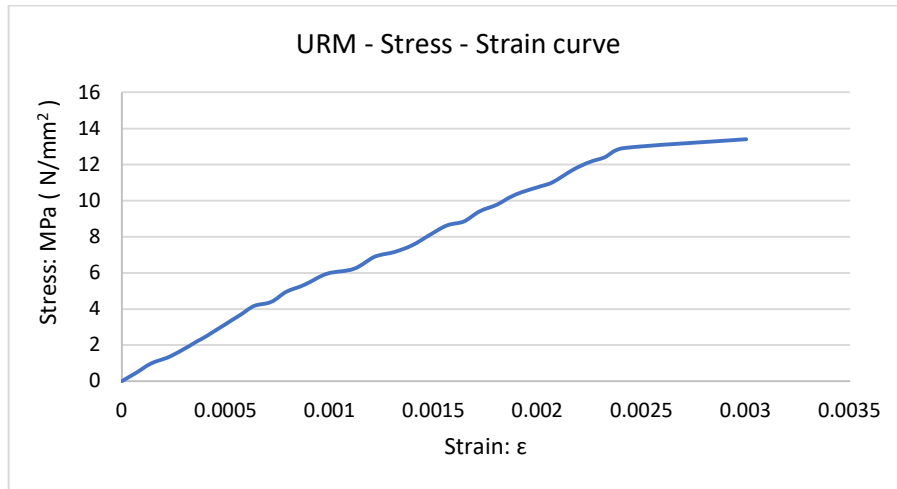


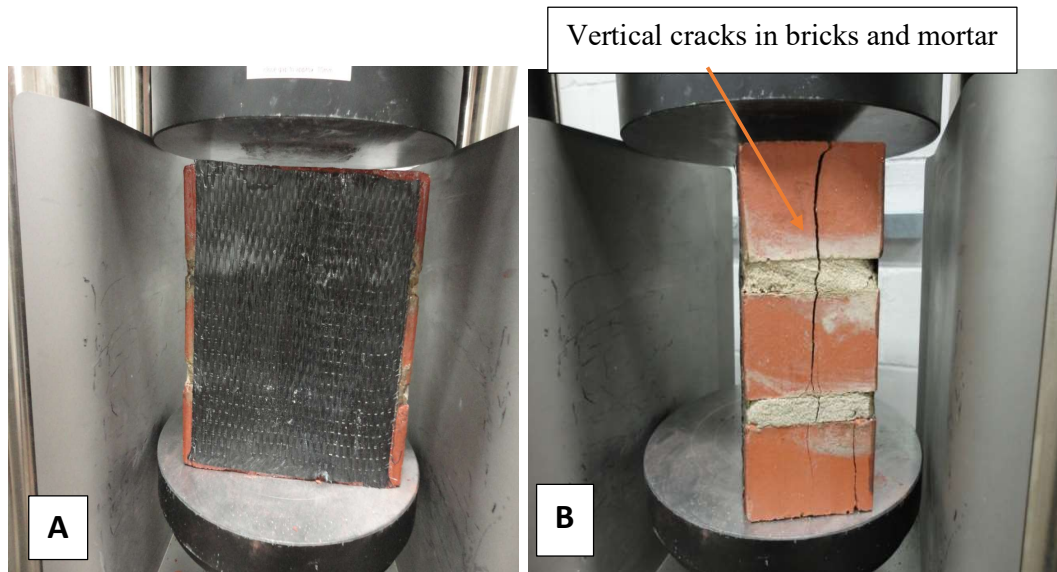
Figure 3.34: URM triplet's stress and strain curve

3.15. Compression test for triplet masonry with one-layer CFRP on two sides

Strengthening with one layer of CFRP sheet on both sides of triplet masonry. The mode of failure was observed to be ductile and cracks appeared in the middle and corner of the sample (Figure 3.35A and 3.35B).

In this test, the triplet masonry with one layer of CFRP sheet on both sides was 200% stronger than URM samples. Figures 3.36 to 3.38 show the results and total reinforcement significantly increases the stiffness and the strength of the triplet.

Debonding is the most common failure and one of the unwanted failure mechanisms in FRP-strengthened masonry structures, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.



Figures 3.35A and B: The obtained failure modes of triplet with CFRP on both sides.

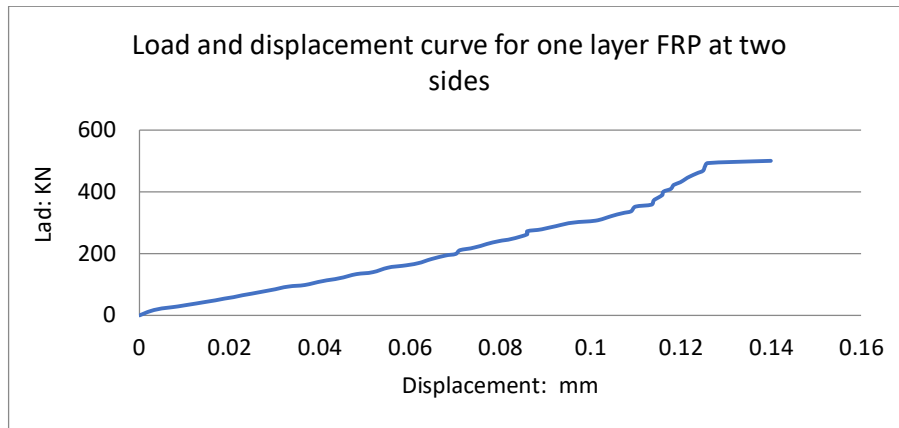


Figure 3.36: Load and Displacement curve for one-layer FRP at two sides

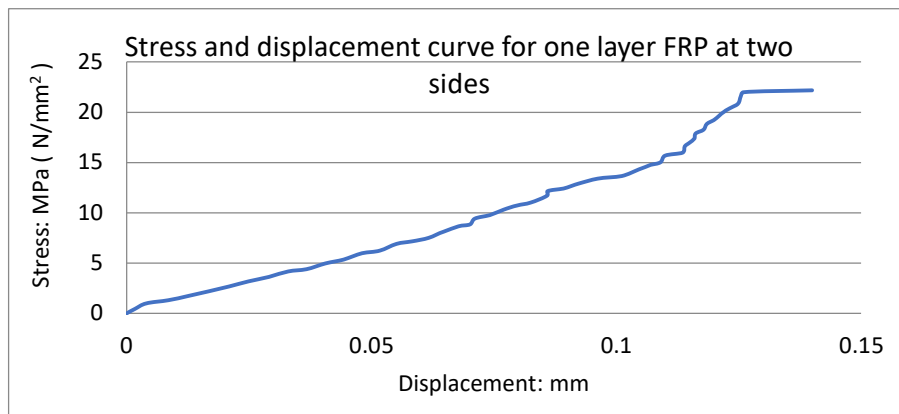


Figure 3.37: Stress and Displacement curve for one-layer FRP at two sides

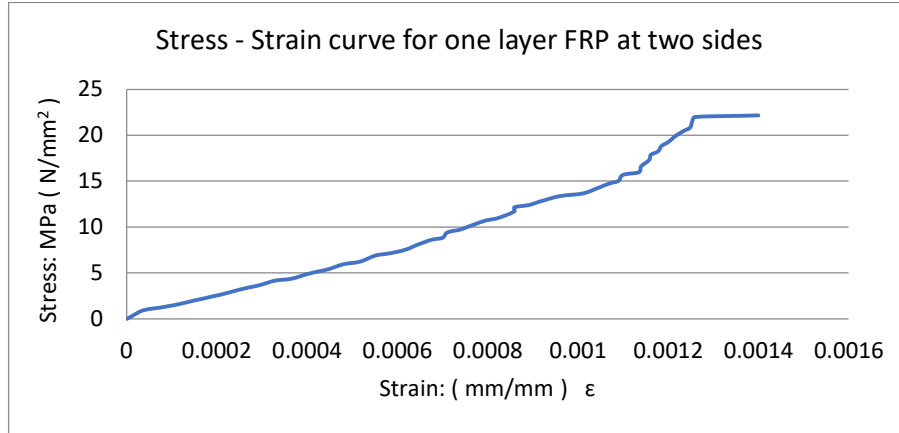


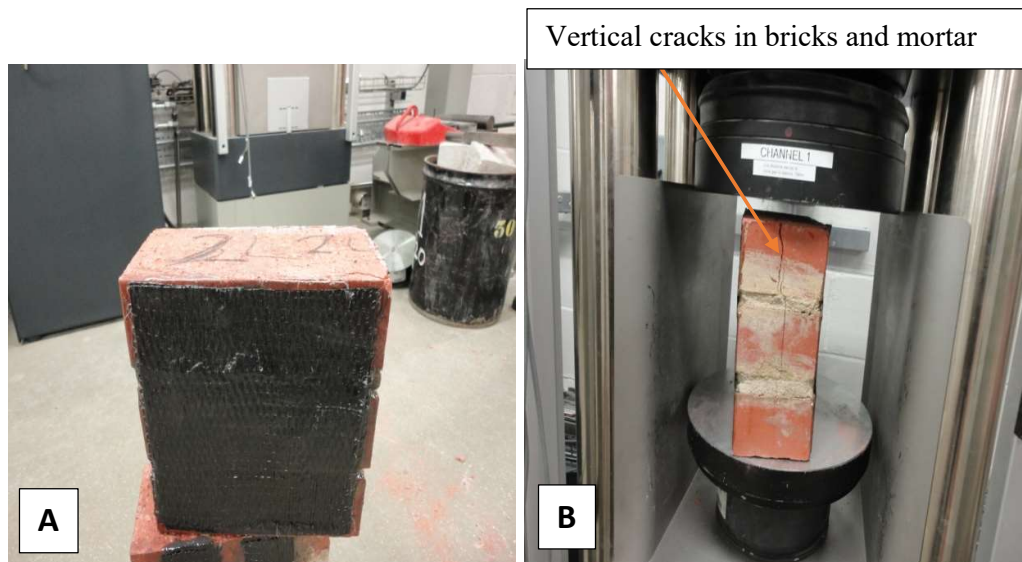
Figure 3.38: Stress-Strain curve for one-layer FRP at two sides

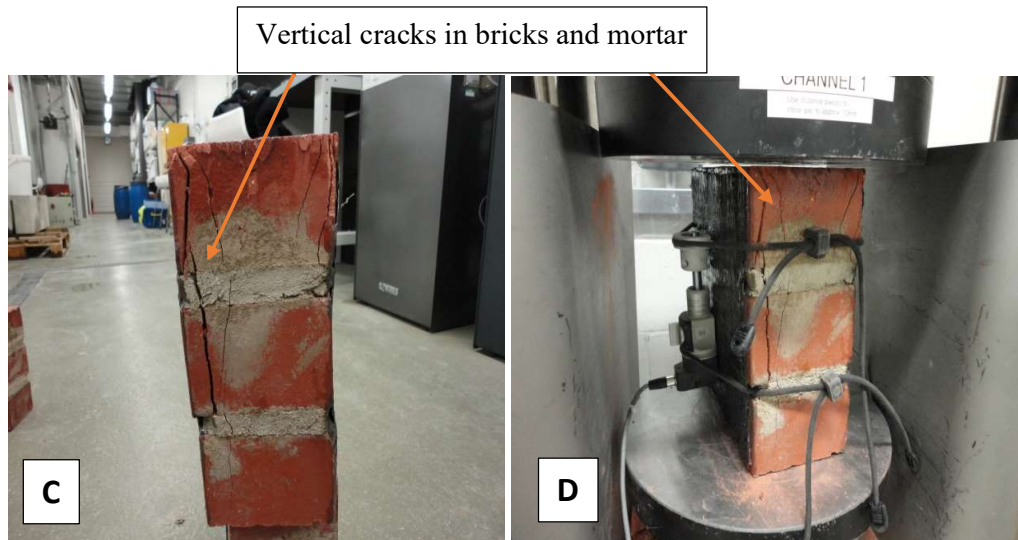
3.16. Compression test for triplet masonry with 2-layers

CFRP on 2 sides

Strengthening with two layers of CFRP sheet on both sides of triplet masonry. The mode of failure was ductile, cracks appeared in the middle of the sample, and they were smaller than one-layer CFRP (Figures 3.39A to 3.39D).

It is not a significant difference between one layer and two layers of CFRP on the triplet masonry sample. In this test, the triplet masonry with two layers of CFRP sheet on both sides was 200% stronger than URM samples as illustrated in Figures 3.40 to 3.42.





Figures 3.39A to 3.39D: The obtained failure modes of triplet with 2 layers CFRP on both sides.

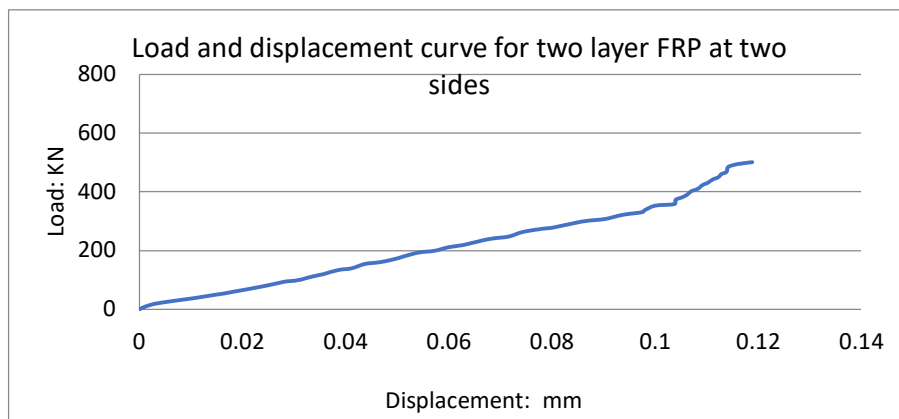


Figure 3.40: Load and displacement curve for two-layer FRP at two sides

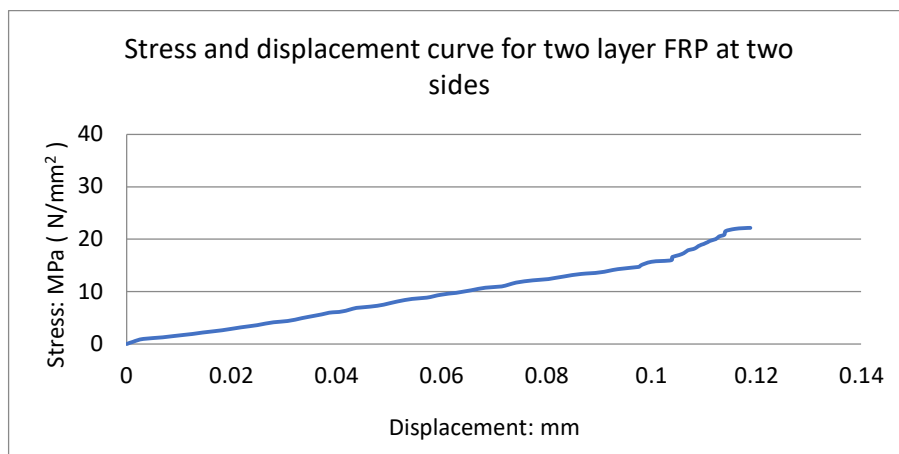


Figure 3.41: Stress and displacement curve for two-layer FRP at two sides

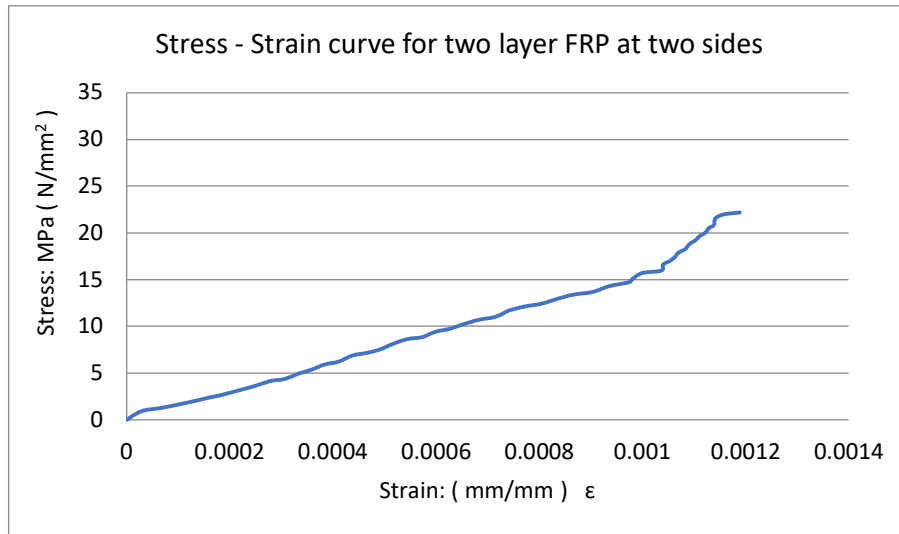
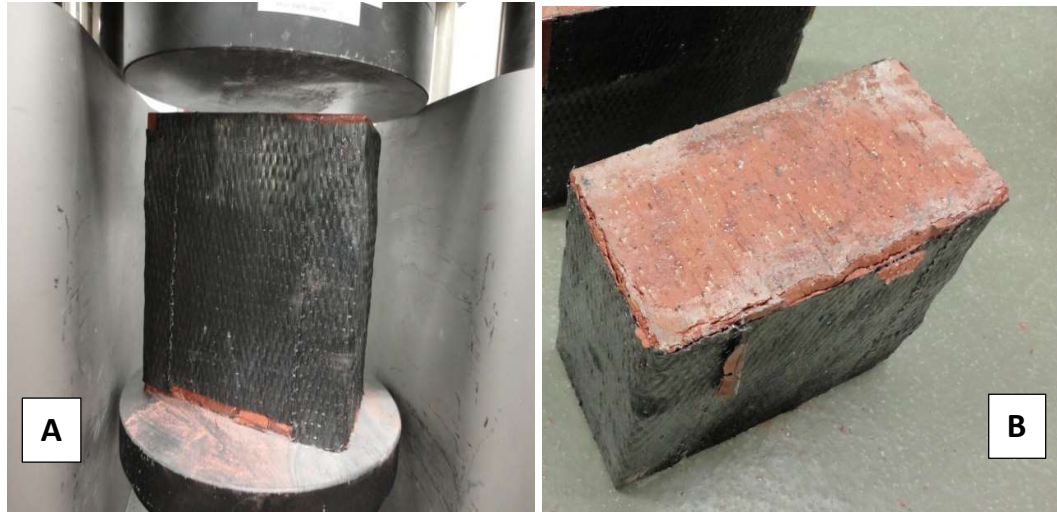


Figure 3.42: Stress-strain curve for two-layer FRP at two sides

3.17. Compression test for triplet masonry with one-layer CFRP on four sides

Strengthening with one layer of CFRP wrapping on four sides of triplet masonry. The triplets displayed the propagation of small cracks in the edge and corner of the sample (Figures 3.43A and 3.43B). Delamination also occurred just before failure and the mode of failure was observed to be ductile. The total reinforcement significantly increases the stiffness and strength of the triplet. The triplet masonry with one layer of CFRP wrapping on four sides was 250% stronger than the URM sample. Figures 3.44, 3.45 and 3.46 show the results. This result shows the columns in masonry buildings can be reinforced without putting extra load on structures.

CFRP debonding was the main reason for triplet failure in FRP-strengthened masonry structures, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.



Figures 3.43A and 3.43B: The obtained failure modes of triplet with 1 layers CFRP on four sides.

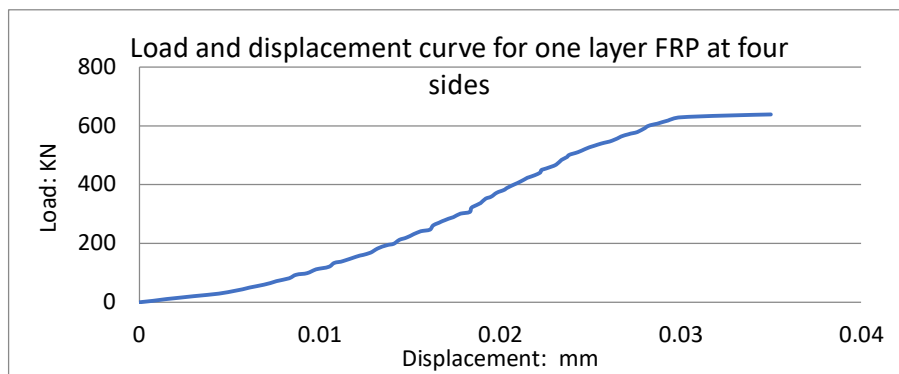


Figure 3.44: Load and displacement curve for one-layer FRP at four sides

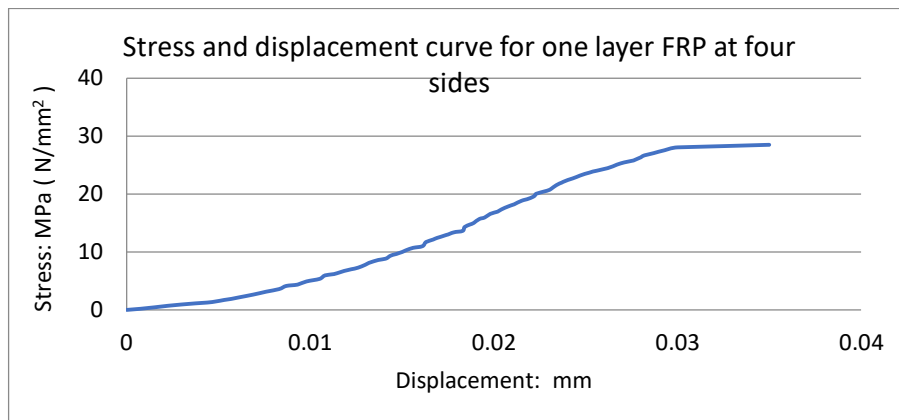


Figure 3.45: Load and displacement curve for one-layer FRP at four sides

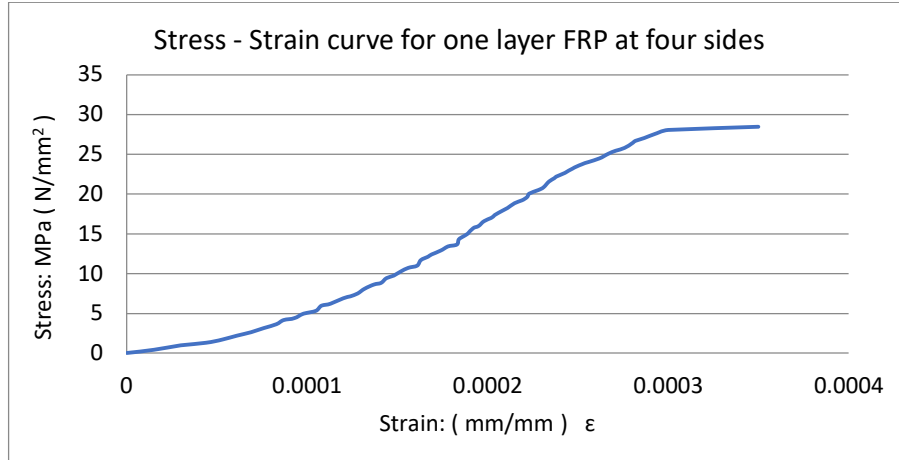
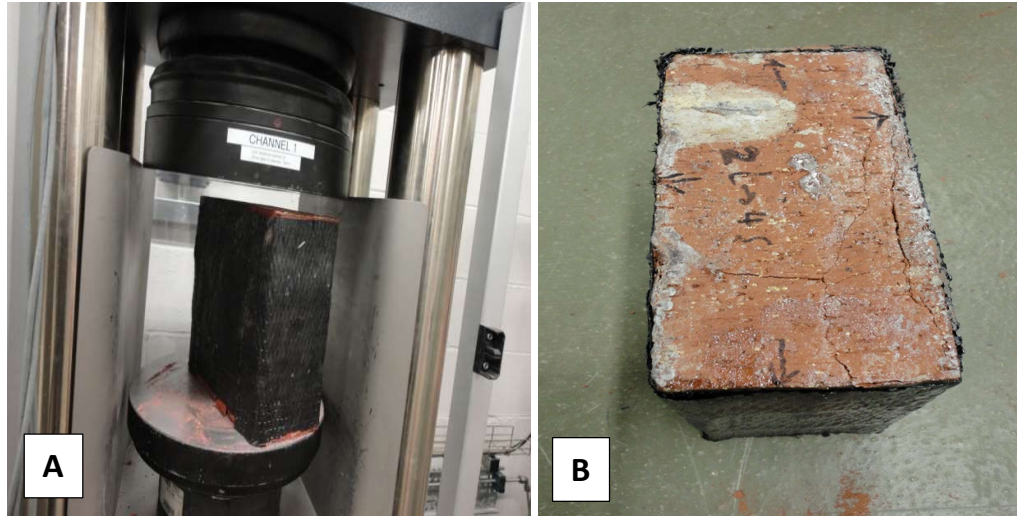


Figure 3.46: Stress-strain curve for one-layer FRP at four sides

3.18. Compression test for triplet masonry with two-layer CFRP on four sides

Reinforcement with two layers of CFRP wrapping on four sides of triplet masonry. This triplet displayed the propagation of small cracks in the edge and corner of the sample (Figures 3.47A and 3.47B). Delamination also occurred just before failure and the mode of failure was observed to be ductile.

In this test, the triplet masonry with two layers of CFRP wrapping on four sides was 300% stronger than the URM sample. Also, according to this experiment, it is not a significant difference between one layer and two layers of CFRP on four sides of triplet masonry. Figures 3.48, 3.49 and 3.50 illustrate the results. CFRP debonding was the main reason for triplet failure in FRP-strengthened masonry structures, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.



Figures 3.47A and 3.47B: The obtained failure modes of triplet with two layers CFRP on four sides.

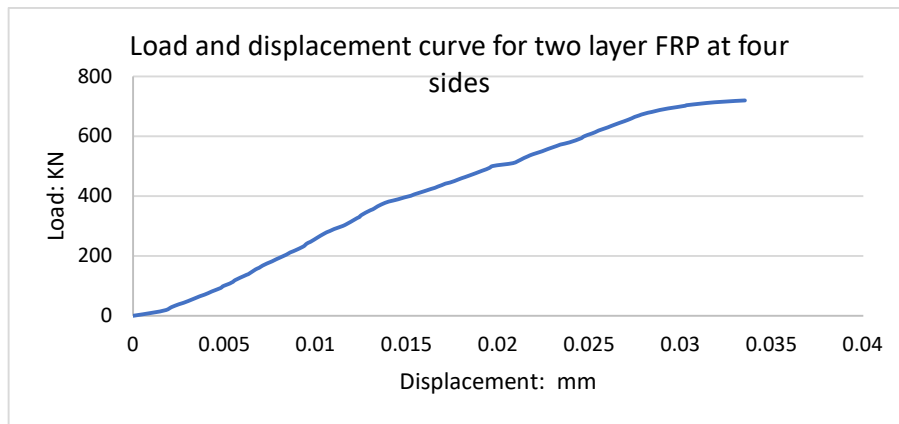


Figure 3.48: Comparison of Max load and displacement

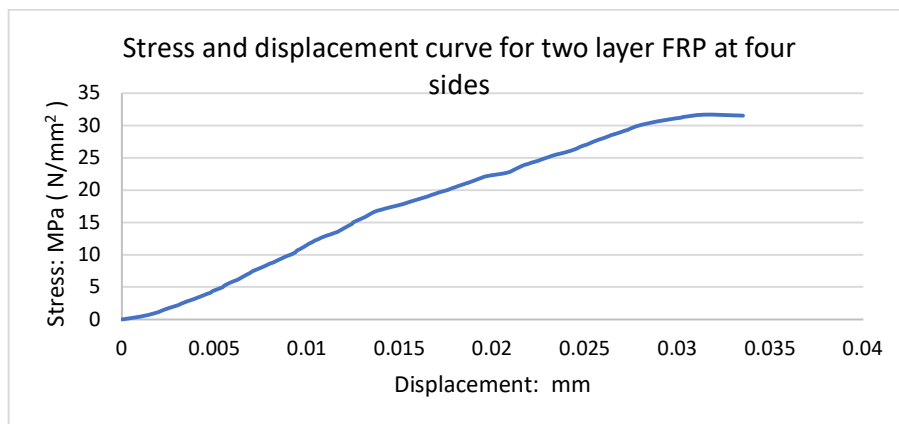


Figure 3.49: Comparison of Max stress and displacement

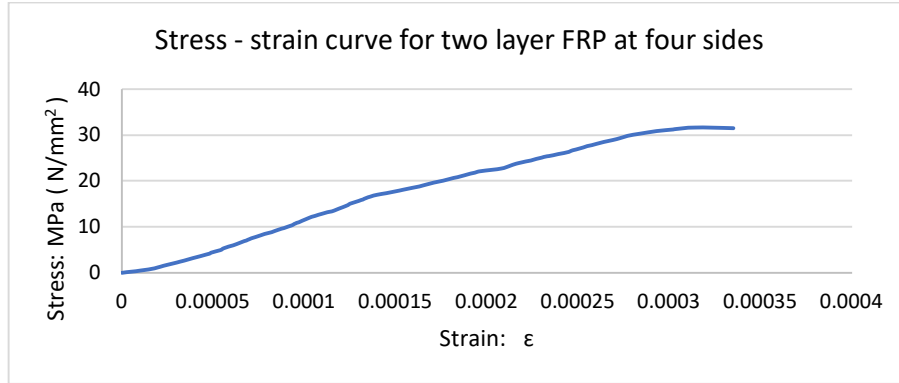
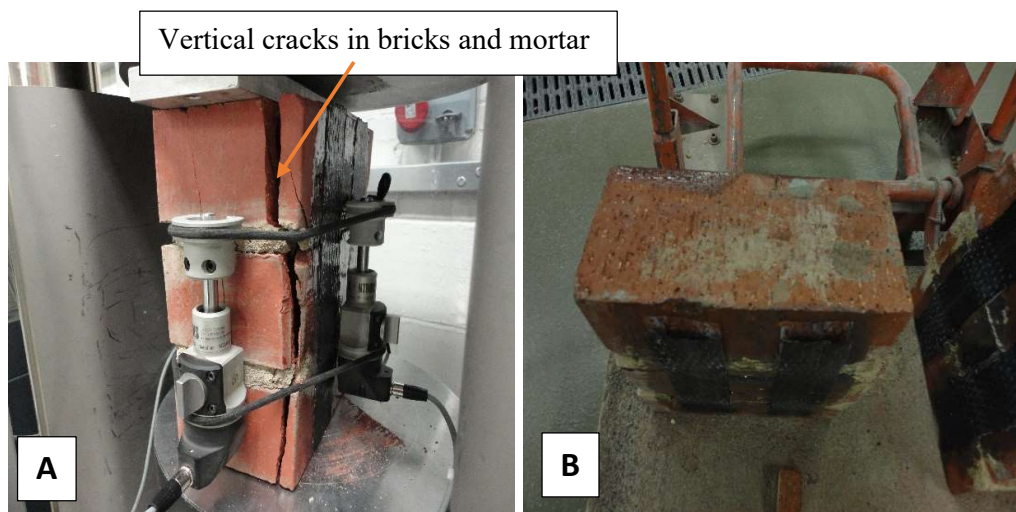


Figure 3.50: Stress-strain curve for two-layer FRP at four sides

3.19. Compression test for triplet masonry with one-layer CFRP strip on two sides

Strengthening triplets with one layer of CFRP strips on both sides of triplet masonry. The mode of failure was observed to be ductile and cracks appeared in the middle and corner of the samples (Figure 3.51A and 3.51B). The triplet masonry with one layer of CFRP strips on both sides was 200% stronger than URM samples. Figures 3.52, 3.53 and 3.54 shows the results, and total reinforcement significantly increases the stiffness and the strength of the triplet. CFRP debonding was the main reason for triplet failure in FRP-strengthened masonry structures, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.



Figures 3.51A and 3.51B: The obtained failure modes of triplet with one layers CFRP strips on two sides.

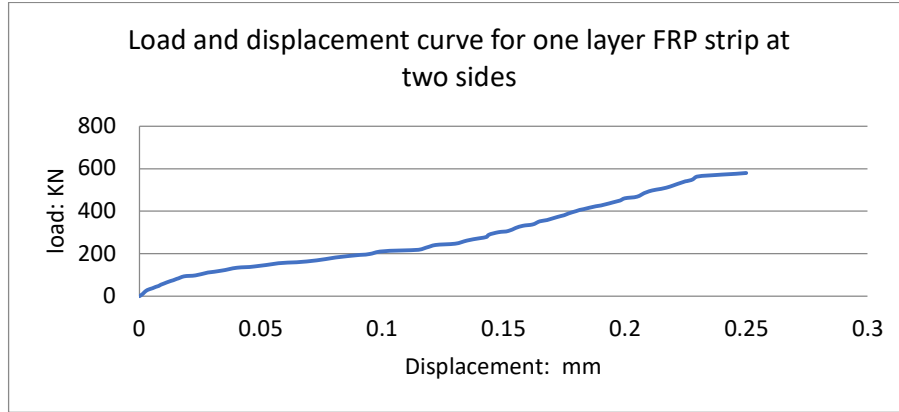


Figure 3.52: Load and displacement curve for one-layer FRP strip on two sides

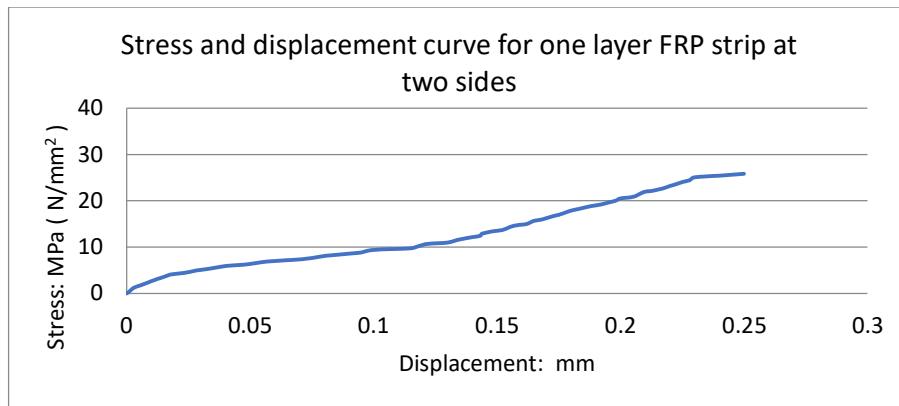


Figure 3.53: Stress and displacement curve for one-layer FRP strip on two sides

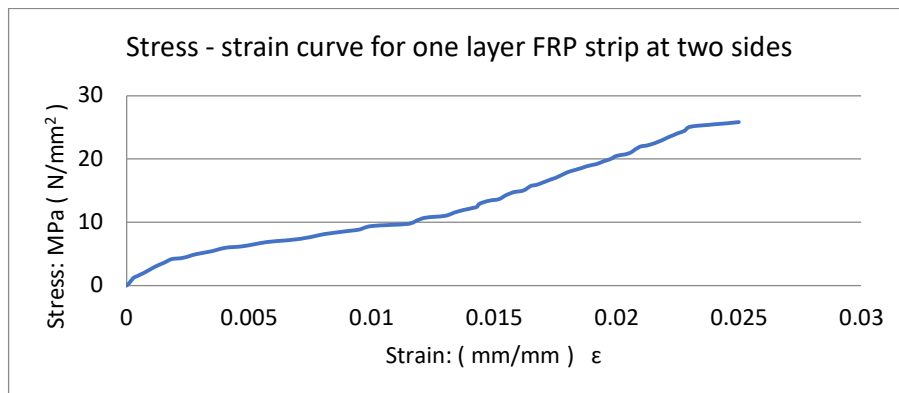


Figure 3.54: Stress-strain curve for one-layer FRP strips on two sides

3.20. Compression test for triplet masonry with two-layer CFRP strip on two sides

Strengthening with two layers of CFRP strips on both sides of triplet masonry. The mode of failure was observed to be ductile (Figure 3.55).

In this test, the triplet masonry with two layers of CFRP strips on both sides was 200% stronger than URM samples. Figures 3.56, 3.57 and 3.58 show the results and total reinforcement significantly increases the stiffness and the strength of the triplet.

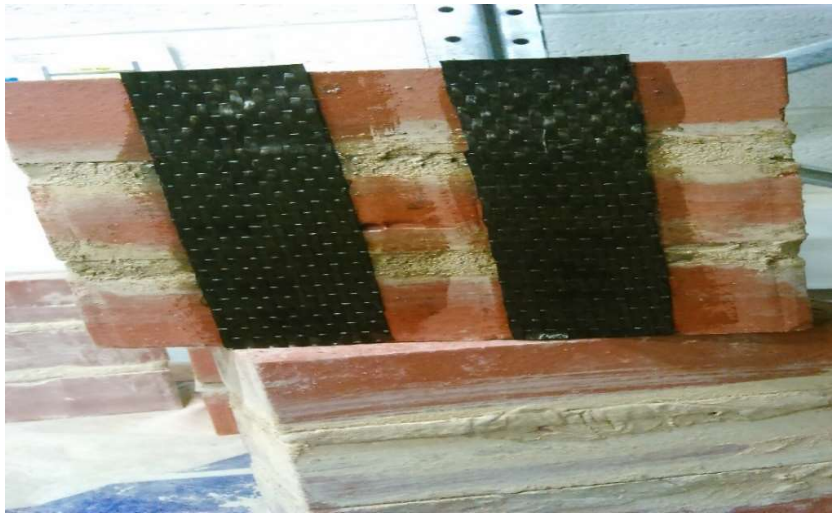


Figure 3.55: Two layers of CFRP strips on both sides of triplet

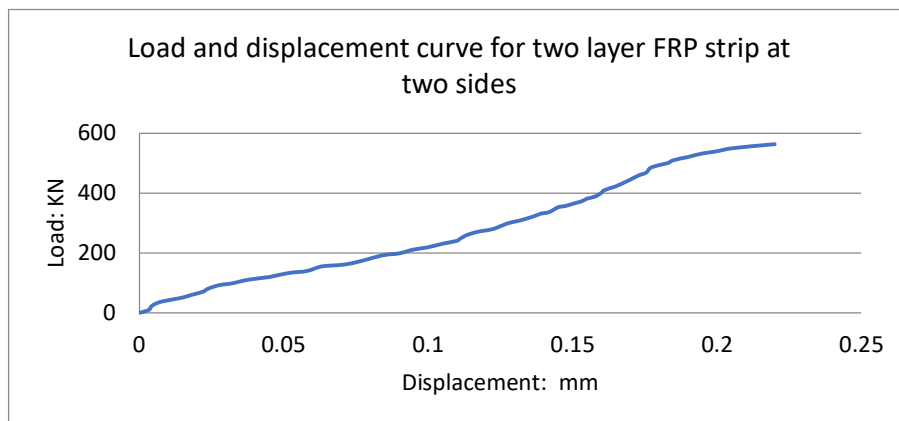


Figure 3.56: Load and Displacement curve for two layers FRP strip at two sides

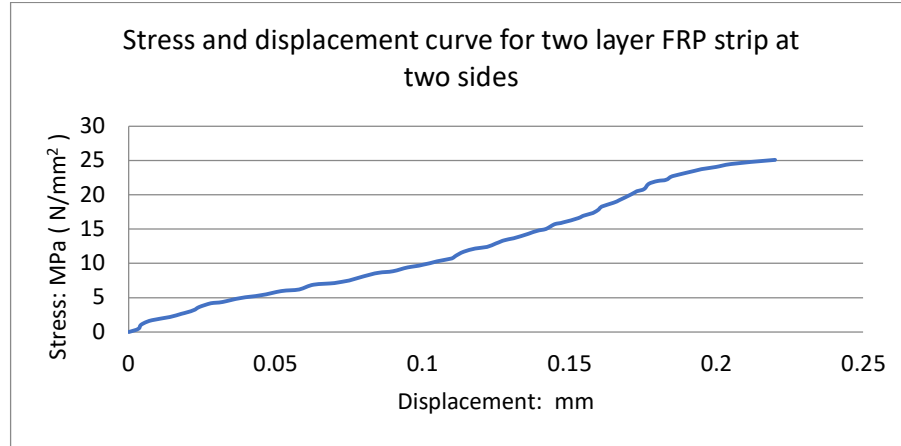


Figure 3.57: Stress and Displacement curve for two layers FRP strip at two sides

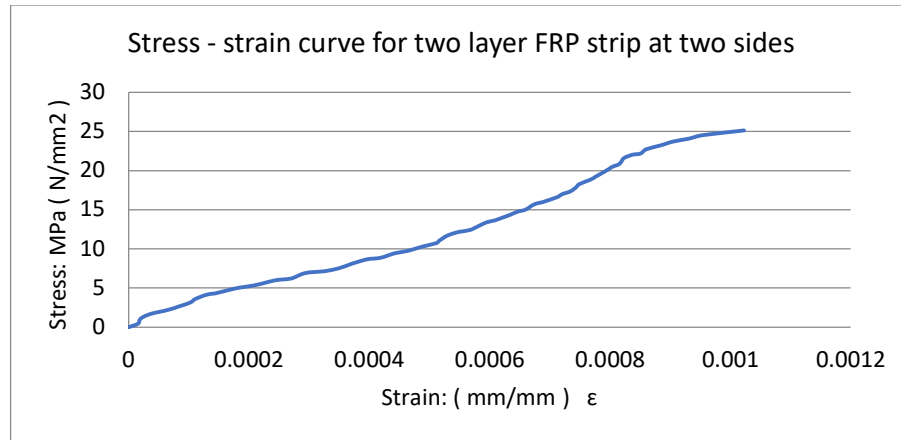


Figure 3.58: Stress-strain curve for two layers FRP strip on two sides

3.21. Comparison of different configurations of CFRP and mechanism of failure for compression tests

Comparison of displacement in compression tests for un-reinforced masonry triplets, reinforced with one and two-layer CFRP strips and one and two layers CFRP sheets on two and four sides of the masonry prism shown in table 3.6.

Figures 3.59 to 3.61 illustrate the comparison of compression tests with different configurations of CFRP on triplet masonry samples. It was observed that the CFRP improved the triplet's bond performance by changing the failure from brittle failure to

flexural failure. This change caused an increase in the capability of deformation of approximately 100% by preventing a brittle failure mode.

In the strengthened triplets, the failure is produced by the loss of bonding between the CFRP and the masonry units. Comparing the recorded crack widths in the front and back sides, the crack growth in the un-strengthened triplets increased at a higher rate than in the strengthened samples. The crack produced by debonding of the masonry units from the mortar in the backside, travelled through the triplet thickness until debonding of the epoxy from the masonry units as shown in photos. At this point, the triplet fails because the tensile stresses are no longer transferred to the CFRP.

Comparing one layer and two layers of CFRP or CFRP strips on triplet masonry showed no significant bond improvement between one layer and two layers. and the reason is, the layers stick together and act as one layer. Depending on CFRP configurations, two layers make triplets 4% to 10% stronger, which is not sufficient to apply two layers.

Debonding is the most common failure and one of the unwanted failure mechanisms in FRP-strengthened masonry structures, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.

Type of triplet masonry	MAX load: KN	Displacement: μm	Stress: MPa (N/mm^2)	Type of failure
URM	300	300	13.5	Ductile
2 layers on 2 sides	500	120	22	Ductile
1 layer on 2 sides	500	127	22	Ductile
1 layer on 4 side	640	32	28.5	Ductile
2 layers on 4 side	720	34	31.5	Ductile

Table: 3.6: Comparison of maximum load and displacement

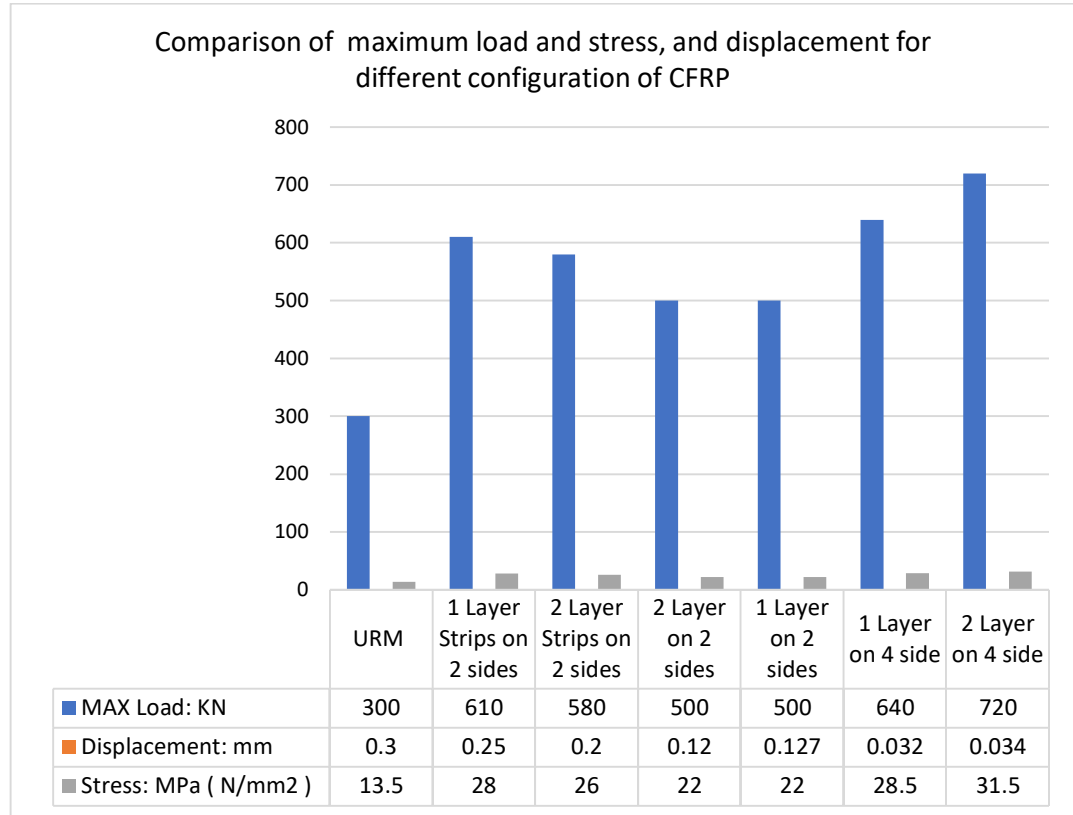


Figure 3.59: The comparison of maximum load, stress and displacement

According to experimental compression tests in this research, there is no significant improvement between one-layer and two layers CFRP applications on triplets. The reason is the layers stick together and act as one layer. As can be seen in figure 3.60, one-layer CFRP sheets on four sides are too close to two layers of CFRP on four sides and one-layer CFRP sheet on two sides is close to two layers of CFRP sheets on two sides too. The displacement of two layers of CFRP strips after 300KN is less than a one-layer CFRP strip.

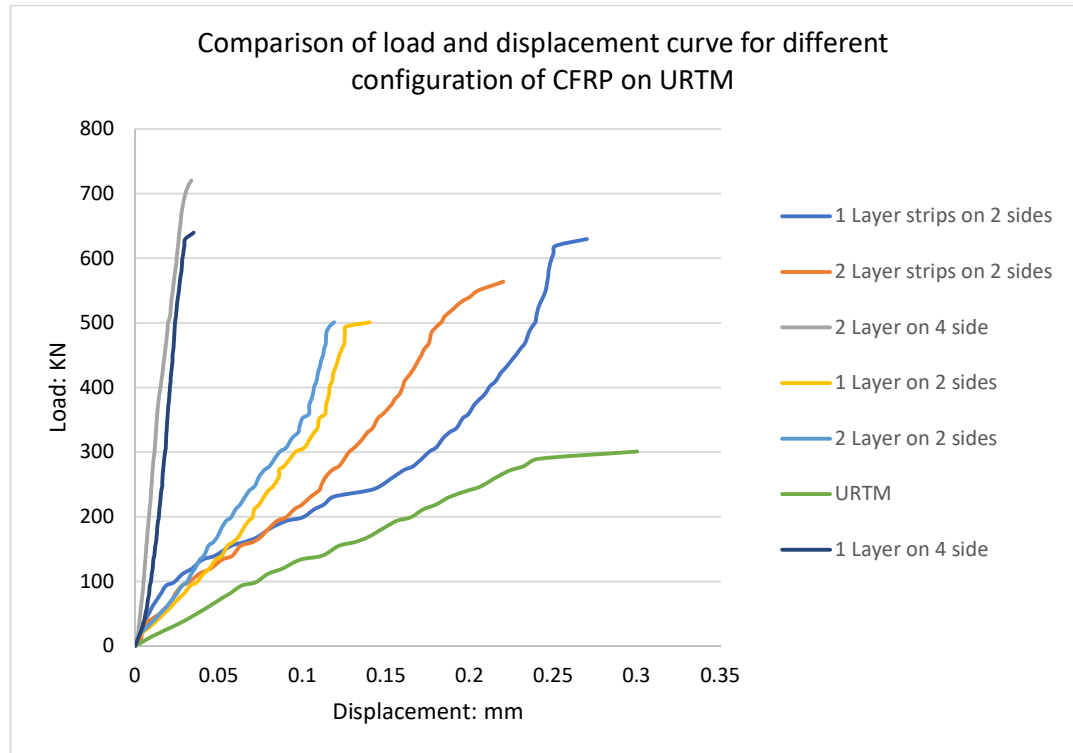


Figure 3.60: Comparison of maximum load and displacement of different CFRP configurations

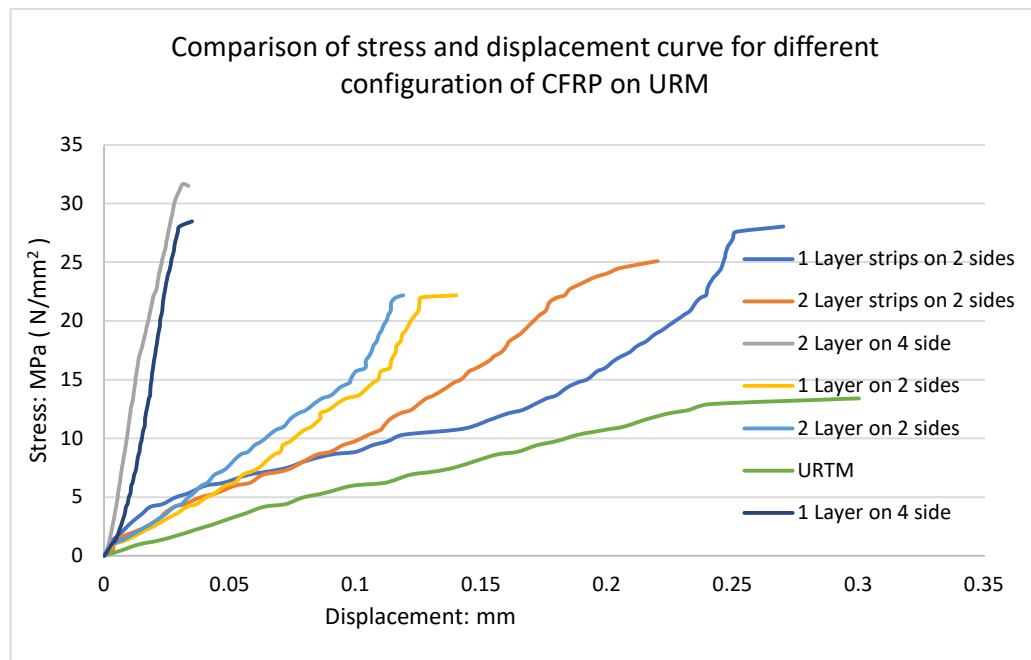


Figure 3.61: Comparison of stress and displacement of different CFRP configurations

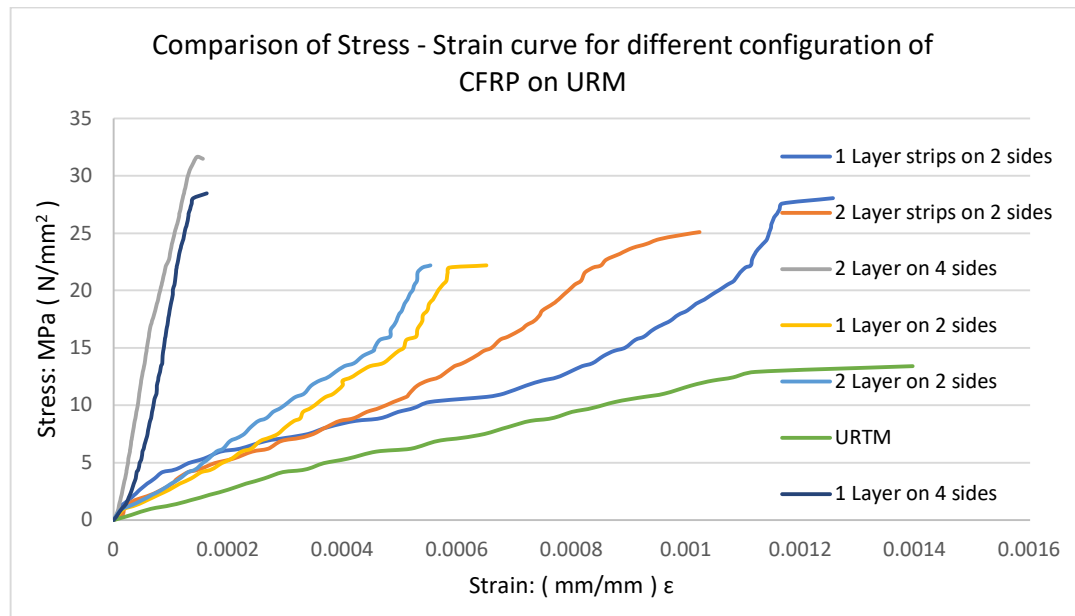


Figure 3.62: Comparison of stress and strain of different CFRP configurations

3.22: Conclusion

Masonry triplets under compression load

The following conclusions can be drawn from the Triplets strengthened by FRP:

1. Debonding is the most common failure and one of the unwanted failure mechanisms in FRP-strengthened masonry structures, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure.
2. The experimental tests showed that the masonry triplets under compression for Triplets with one layer of CFRP sheet on both sides, one layer of CFRP strips on both sides, two layers of CFRP strips on both sides, and two layers of CFRP sheet on both sides were 200% stronger than URM samples. The triplet masonry with one a layer of CFRP wrapping on four sides was 250% and triplets with two layers of CFRP wrapping on four sides was 300% stronger than the URM samples.
3. Most effective strengthening in terms of ultimate load capacity is for triplet sample with FRP reinforcement on four sides and the highest ductility in terms of maximum deflection at ultimate load is observed for all samples.
4. The increase of the stiffness of URM triplets with CFRP sheets and strips are significant for the first layer and not so effective for the additional layers on the top of the first layer.
5. Comparing one layer and two layers of CFRP or CFRP strips on triplet masonry showed It is no significant bond improvement between one layer and two layers. The layers stick together and act as one layer. Depending on CFRP configurations, two layers make triplets 5% to 15% stronger, which is not sufficient to apply two or three layers.

Chapter 4

**Experimental study on shear strength
of triplet masonry with one and two
layers of CFRP**

4.1. Introduction

Masonry structures have been used for millennia due to their ease of construction, simplicity, low costs, and durability. Masonry can be commonly defined as a composite material, made of natural or artificial elements assembled in various shapes and bonded together with or without mortar (Bui et al., 2017). It is well known that unreinforced masonry (URM) buildings are vulnerable during an earthquake (Coburn and Spence 2002). In order to extend the service life of such structures, it is essential to repair or retrofit them using suitable techniques. ElGawady et al. (2004) have reviewed the conventional techniques for retrofitting URM, stating clearly the advantages, disadvantages, and efficiency of each of these techniques.

In the past three decades, several studies have been carried out exploring the use of FRP composites to repair or strengthen reinforced concrete and masonry structures (Saadatmanesh 1997). Retrofitting of masonry using FRP composites offers several advantages such as a high strength-to-weight ratio, ease of application in terms of labour and time, reduced curing period, and minimal interference with the existing architecture of the structure. ElGawady and Lestuzzi

(2004) and Tumialan et al. (2009) have reviewed the techniques of retrofitting masonry using FRP composites highlighting the practical applications and their limitations.

According to (Ferretti et al., 2019). from the mechanical point of view, masonry is a heterogeneous, nonlinear, and anisotropic material, consisting of masonry units, mortar, and masonry unit-to-mortar interfaces. Also, masonry is characterised by its very low tensile strength, substantial strength in compression, and weak performance in bending and shear. Indeed, it is well known that masonry buildings are weak against horizontal actions. Tensile and shear bond strengths are quite low and highly variable. Thus, masonry walls fail mostly through masonry unit-to-mortar interfaces, known as planes of weakness. Pavan and Nanjunda (2015) conducted that the main disadvantages of retrofitting masonry with FRP are the high cost, impermeability to moisture, and brittle failure modes such as tensile rupture and FRP debonding. FRP composite attains their tensile strength at very low values of strain and results in a brittle failure. Furthermore, debonding is another brittle failure mode to be considered. Among the different modes of failure, sliding shear mode represents horizontal shearing through the bed joints of

masonry infill. This failure mode is generally associated with strong frames and infills with weak mortar joints. Hence, the shear strength and deformation ability of the brick–mortar bed joint in masonry is critical for the in-plane shear behavior of masonry-infilled frames. Masonry with externally-bonded FRP composites can enhance the shear strength and shear deformation ability of brick–mortar joints. Lourenco et al. (2004) assessed the shear behavior of stack-bonded masonry by conducting shear tests on masonry panels under normal precompression. Lourenco et al. (2004) determined cohesion and tangent of the friction angle of stack-bonded masonry to be 1.39 N/mm² and 1.03, respectively, and concluded that the results obtained for stack-bonded masonry are similar to the values obtained for masonry with discontinuous vertical joints. Mojsilovic (2012) adopted masonry triplets in his study on the shear strength of masonry with a damp-proof course membrane embedded in mortar joints. Alecci et al. (2013) conducted a study for the assessment of the shear strength of masonry by both diagonal compression tests on masonry panels and masonry triplet tests and concluded that shear testing of masonry triplets is more suitable and provides reliable results, apart from being simple and convenient. Similar to the research investigations conducted on the shear response

of the brick–mortar bed joint of unreinforced masonry, studies investigating the effectiveness of FRP composites in improving the shear response of brick–mortar bed joints have been conducted. Hamid et al. (2005) studied the shear behavior of FRP-glued masonry composed of hollow concrete blocks. The study demonstrated the effectiveness of FRP in preventing the brittle failure modes exhibited by URM and improving the shear strength and deformation ability of masonry. Ehsani et al. (1997) investigated the influence of FRP fabric density, orientation, and length in enhancing the shear strength and stiffness of masonry. FRP overlays were bonded to masonry triplets that contained pieces of plywood sandwiched between the bricks. Luccioni and Rougier (2010) assessed the shear behavior of brick–mortar joints strengthened or repaired with CFRP composite. The study showed that the repair of masonry with CFRP composites not only helped in recovering the shear strength but also increased it. Roca and Araiza (2010) studied the effect of normal pre-compression on the shear behavior of brick–mortar joints under monotonic and cyclic loading. Roca and Araiza

(2010) characterized the contribution of FRP composite towards shear strengthening in terms of two mechanisms, namely, (1) mobilization of shear due to friction that is enhanced by FRP strengthening, and (2) shear strength of FRP composite.

Mortars play a significant role in the masonry's performance and the main purpose of mortar in masonry is to bond masonry units into an assemblage. Mortars should typically be weaker than the masonry units so that any cracks will occur in the mortar joints where they can more easily be repaired. The compressive strength of mortar depends largely on mortar and cement type and the water-cement ratio. Bond strength is generally the most important factor but also flexural strength because it measures the ability of a mortar to resist cracking (Papanicolaou et al., 2011).

According to Pavan and Nanjunda (2015), a critical component for the success of strengthening structures by externally reinforcing them with FRP composites depends on the bond mechanism between FRP and the substrate material. The transfer of forces between the two systems is mainly through shear stresses in the adhesive. Inadequate bond strength results in premature failure due to delamination and peel-off of the FRP from the substrate. As a composite material, masonry can present different failure modes, depending on the geometry, material, and load factors. Generally, it can fail in compression, tension, bending, or shear. This chapter focuses on the shear failure of triplet masonry.

The interface shear performance of masonry is important in developing analytical methods for masonry wall in-plane shear response. In shear walls, the failure occurs at the brick–mortar interface, resulting in separation of the two materials. The frictional resistance would continue to transmit shear stress through the separated elements. The bond between unit and mortar is the weakest link in masonry structures. Lourenco (2008) conducted, at the unit–mortar interface, two separate phenomena can occur: one associated with tensile failure (mode I) and the other with shear failure (mode II).

According to Euro code 6 [2005], the shear strength of masonry is obtained from the characteristic initial shear strength of masonry and a portion of the vertical stress, see equation 4.1, for all joints filled and equation 4.2, for perpend joints unfilled. The initial shear strength of masonry is suggested to be determined from tests, following the recommendations given in EN 1052-3 [2002] or EN 1052-4 [2000].

$$f_{vk} = f_{vko} + 0.4 \sigma_d \quad (4.1)$$

$$f_{vk} = 0.5 f_{vko} + 0.4 \sigma_d \quad (4.2)$$

but not greater than $0.065 f_b$ or f_{vlt}

where:

f_{vko} is the initial shear strength, under zero compressive stress and f_{vlt} is a limit to the value of f_{vk} .

σ_d is the design compressive stress perpendicular to the shear in the member

f_b is the normalised compressive strength of the masonry units,

4.2. Test set-up and test procedure

This chapter includes two parts, the first part is an experimental study on triplets reinforced with one layer of CFRP, and the second part is experimental tests on triplets with two layers of CFRP with different CFRP configurations and a comparison between one layer and two layers of CFRP applications. Also, new equations were developed to predict the shear stress and displacement. This study intends to provide information related to the effect of parameters such as application type, extra layers, and amount of CFRP composite on the improvement in shear strength and shear deformational capacity of brick–mortar joints of the masonry.

Masonry triplets were glued with CFRP composites in two main configurations, namely, strip and full face. The notation L stand for layer and S is a side, each CFRP strip was 5 cm wide on both or four faces of the test specimen. CFRP overlap for samples that had four faces covered fully or strips, was 5 cm.

According to EN 1052-3 [2002] recommendation, in this study, the initial shear strength of masonry was obtained from small masonry specimens tested to destruction.

Specimens were tested with dimensions 220 x 103 x 215 mm made from solid clay bricks and mortar M4. All specimens had three brick units and a 10mm thick joint between them. The specimens were cured in the laboratory within controlled environment conditions. Each specimen was tested at the age of 40 days. Shear stresses in the unit-mortar interface were introduced by gradually increasing the shear forces at the top of the middle brick until the failure.

FRP composites were bonded to masonry prisms and before FRP application the surface of masonry specimens was scrubbed with sandpaper to remove minor imperfections like small chunks of mortar in order to obtain a plane surface. Later, the surface was cleaned to remove loose matter. Two-component epoxy glue was used to bond FRP fabric to the masonry substrate. Epoxy resin was mixed with hardener and was applied on the prepared prism surface followed by FRP application to the triplet masonry. During the wrapping of the masonry prism with FRP, an overlap length of 50 mm was provided. Another coat of epoxy glue was applied onto the surface of FRP and left for curing for a minimum period of 4 days [minimum curing period specified by the manufacturer was one day].

Masonry triplets are adopted in the present study to evaluate the shear strength along the brick–mortar bed joints of the masonry. As discussed earlier in the “Introduction” section, for masonry panels that act as an infill, the stress normal to the bed joint along which sliding shear failure occurs will be small. Furthermore, the primary objective of the current investigation is to assess the enhancement in shear strength and shear deformational capacity of brick–mortar joints due to externally-bonded FRP composites. Hence, the triplet masonry assemblage in the present study is subjected to loading parallel to the bed joint. To achieve the mortar joint interfaces, the middle brick was loaded at the top and the side bricks were supported at the bottom without side preloads according to (EN 1052 – 3) as shown in Figure 4.1. The test setup is similar to the one adopted by Istegun and Celebi (2017) and EN 1052 - 3. The prisms were tested under compressive loading in a hydraulic universal testing machine with a 70 MPa pressure capacity. The load was applied at the rate of 1.0 KN/s according to the test machine’s manual and the sliding was measured by three strain gages that was attached to the triplet masonry. Each test was repeated three times.

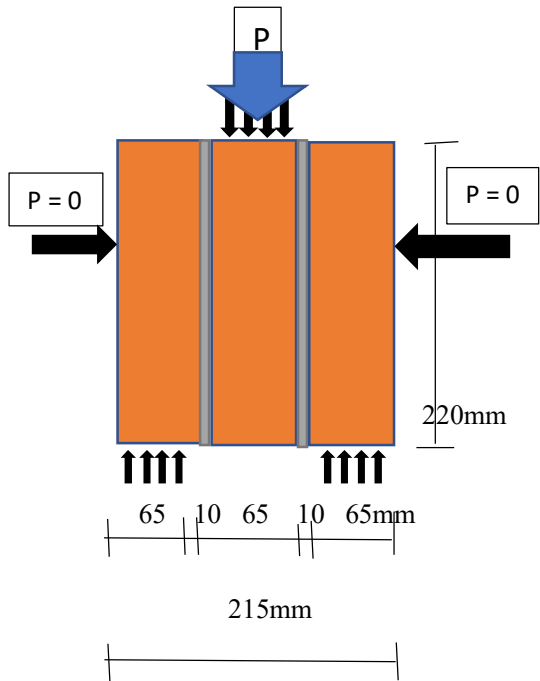


Figure 4.1: Loading the middle brick at the top and supporting the side bricks at the bottom without sides preloads according to (EN 1052 – 3), $P = 0$.

4.3. Experimental study on shear behaviour of reinforced triplet masonry with different CFRP configurations

Composite material, masonry can present different failure modes, depending on the geometry, material, and load factors. According to the literature review, additional experimental studies were carried out to investigate the behavior of triplet masonry under shear loading with a different configuration of CFRP sheet and CFRP net.

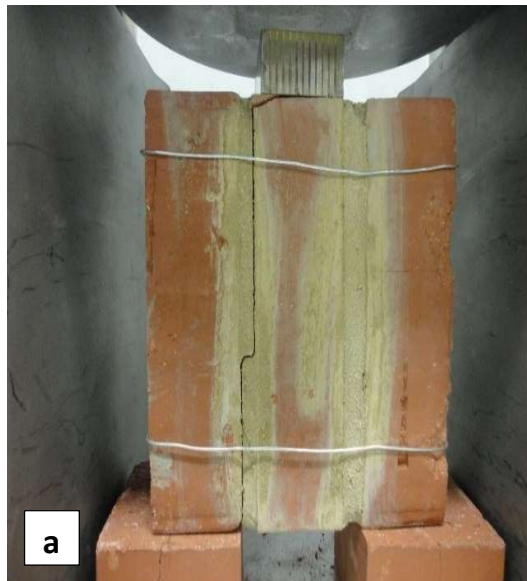
In the un-strengthened samples, the failure was brittle. In this triplet, some material comes loose after the ultimate load. At the final state, all strengthened triplets did not have material loos and wider cracks were mostly observed in the un-strengthened triplets. In this stage, the experimental tests were carried out with One-layer CFRP.

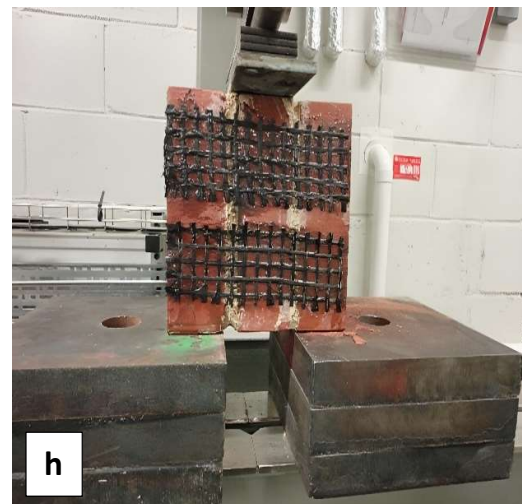
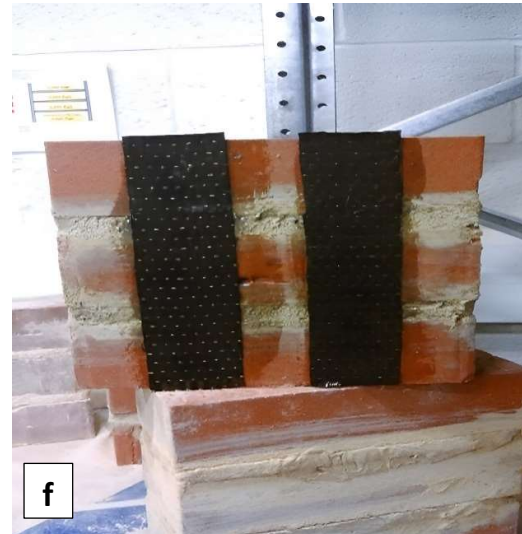
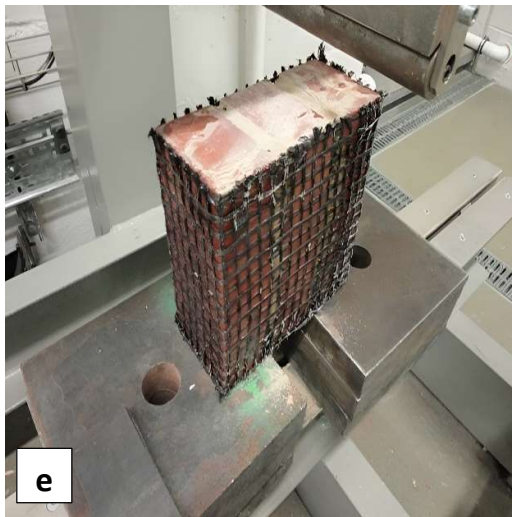
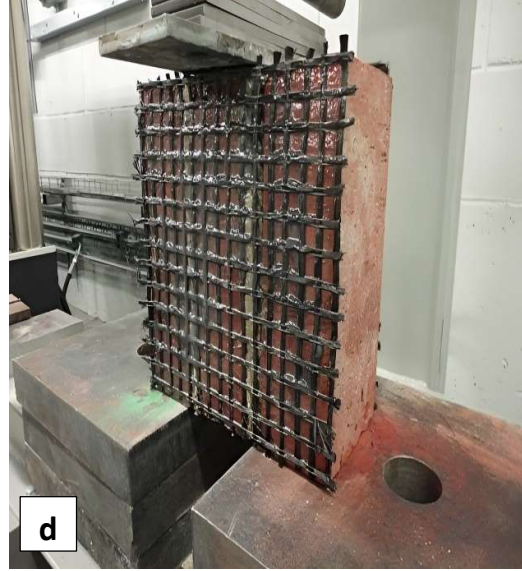
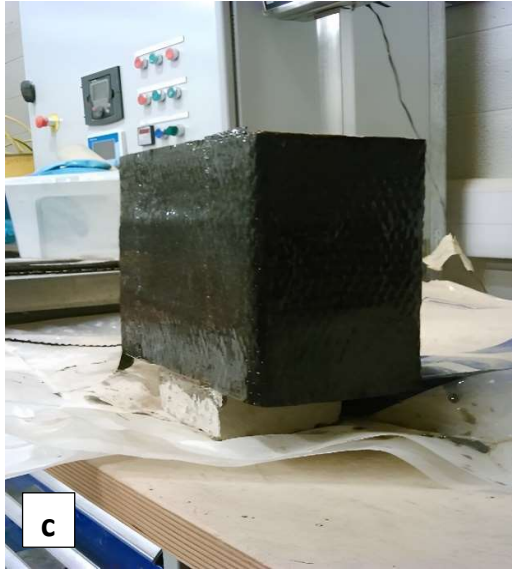
The triplet masonry in the present study is subjected to the loading parallel to the bed joint. This was achieved by loading the middle brick at the top and supporting the side bricks at the bottom according to EN 1052-3. The prisms were tested under compressive

loading in a hydraulic universal testing machine with a 70 MPa pressure capacity. The load was applied at the rate of 1.0 KN/s according to the test machine's manual and the sliding was measured by three strain gages that was attached to the triplet masonry. Each test was repeated three times.

4.3.1. Different configurations for strengthening Triplet masonry used in this section are as follows:

- 1 -Triplet masonry without CFRP (URTM) as control samples. (Figure 4.2a).
- 2 - Strengthening with CFRP sheets on both sides of triplet (Figure 4.2b).
- 3 - Strengthening with CFRP sheets on four sides of triplet (Figure 4.2c).
- 4 - Strengthening with CFRP net on both sides of triplet (Figure 4.2d).
- 5- Strengthening with CFRP net on four sides of triplet (Figure 4.2e).
- 6 - Strengthening with CFRP strips on both sides of triplet (Figure 4.2f).
- 7 - Strengthening with CFRP strips on four sides of triplet (Figure 4.2g).
- 8 - Strengthening with CFRP net strips on both sides of triplet (Figure 4.2h).
- 9 - Strengthening with CFRP net strips on four sides of triplet (Figure 4.2i).







Figures 4.2: The control specimen and eight different CFRP applications on triplet masonry.

4.4. Triplet masonry test without CFRP (URTM) as control samples and failure mode

Four specimens were constructed without FRP (URTM) as a control sample. The samples suddenly failed at 10KN, 10KN, 12, and 13KN. The mode of failure under in-plane shear loading for all samples was sudden and brittle. Specimens failed by the development of cracks along the brick–mortar joint interface followed by complete collapse as shown in (Figures 4.3a to 4.3d). Shear fracture (cracking) in the mortar joints was either confined to one of the interfaces (Figure 4.3b and 4.3c) or in a number of tests stepped diagonally from the inner interface to the outer interface (when viewed in direction of the load) as shown in Figure 4.3d (the top interface of the specimen on the right cracked horizontally). As expected, low shear strength of masonry specimens was obtained and all triplets failed in the brick-mortar interface by shear sliding. This failure is one of the valid failures established by EN 1052-3. Figures 4.4a to 4.4c show an increase in shear stiffness in the initial part of the curve. This phenomenon can be attributed to the effect of dilatation and the average ultimate displacement of control triplet specimens was 2.0 mm. The maximum load and ultimate displacement of control triplets of the present study and Mojsilovic (2012) are comparable.

The failure modes were mainly caused by mortar layer failures. During the increase of the shear load, a diagonal crack was observed in the mortar layer and propagated along the mortar, therefore, leading to the specimen being separated into two distinct bodies from either the inner brick or the outer, or both. No substantial damages were seen on

the brick sides. This may be attributed to the lack of frictional resistance due to the absence of compressive stress normal to the bed joint during the shearing stage. A similar failure mode was likewise reported by Prakash (2007) and Bompa et al, (2020). It is observed that the shear triplet presented an almost linear shear stress–strain relationship before cracking, followed by a drop of shear strength once cracking had occurred in the mortar layer, followed by a slight increase in shear strength and deformation with the application of further loading before rupture. This happened when cracks appeared in the mortar layer and interface of the triplet. All triplet specimens failed similarly and shear failure occurred mainly in mortar joints. The specimens demonstrated failure in the joint with relative movement of the middle brick.

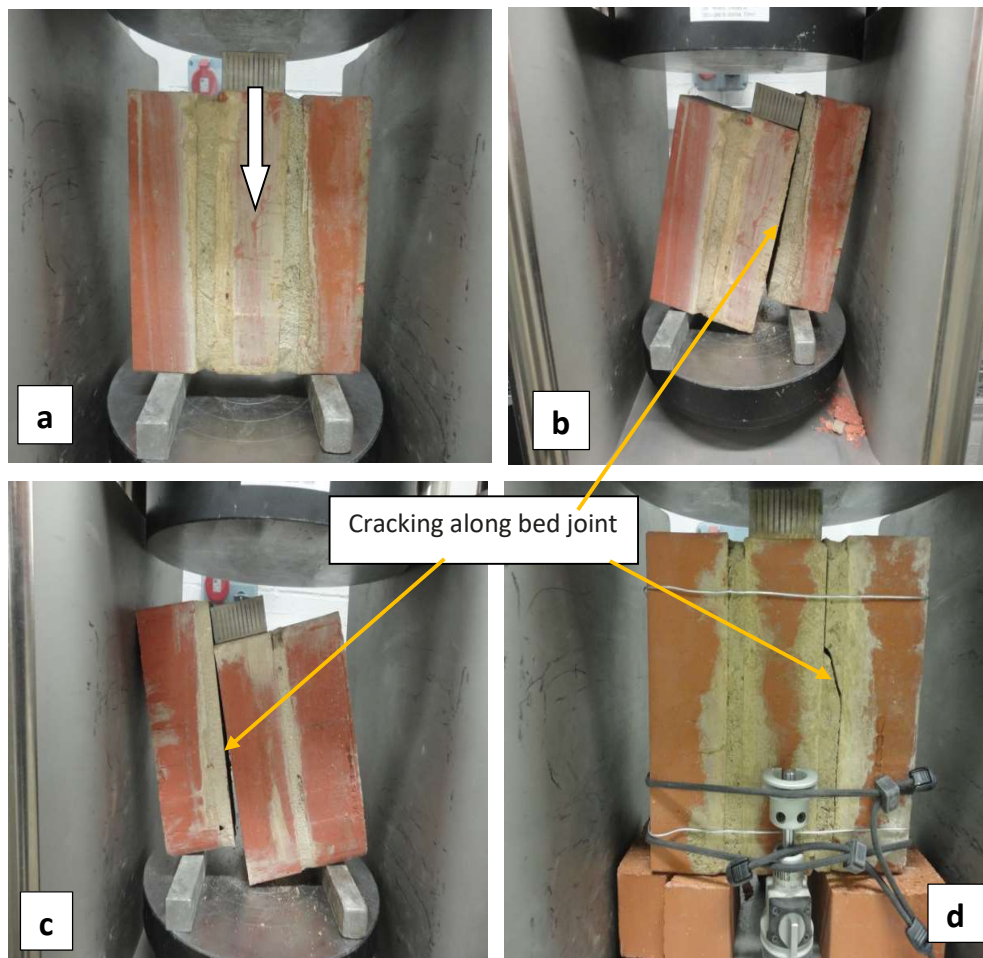


Figure 4.3a to 4.3d: The triplet masonry failure: (b and c) sliding a long bed joint, (c) diagonal shear crack at mortar on the right side,

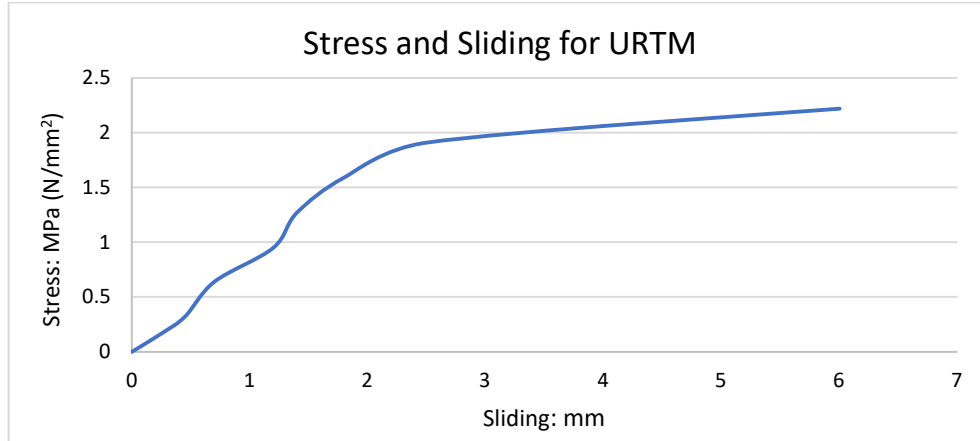


Figure 4.4a: URM stress and slip curve

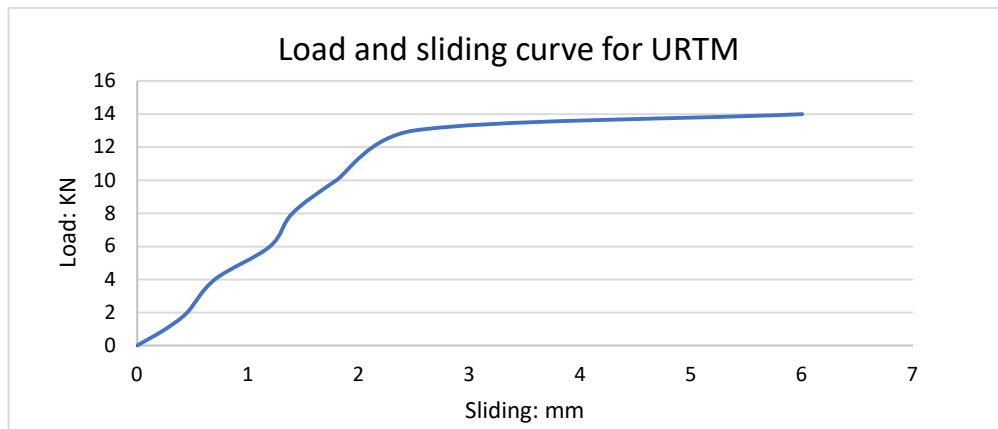


Figure 4.4b: URM load and displacement curve

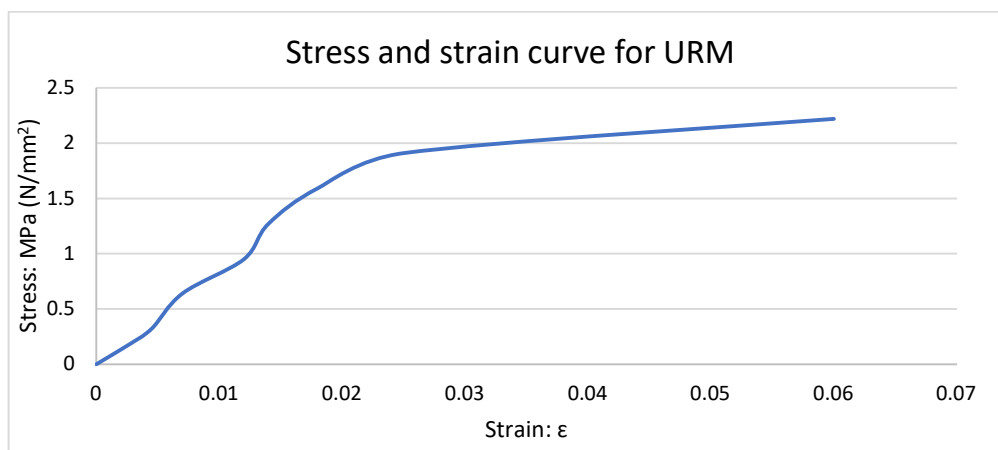
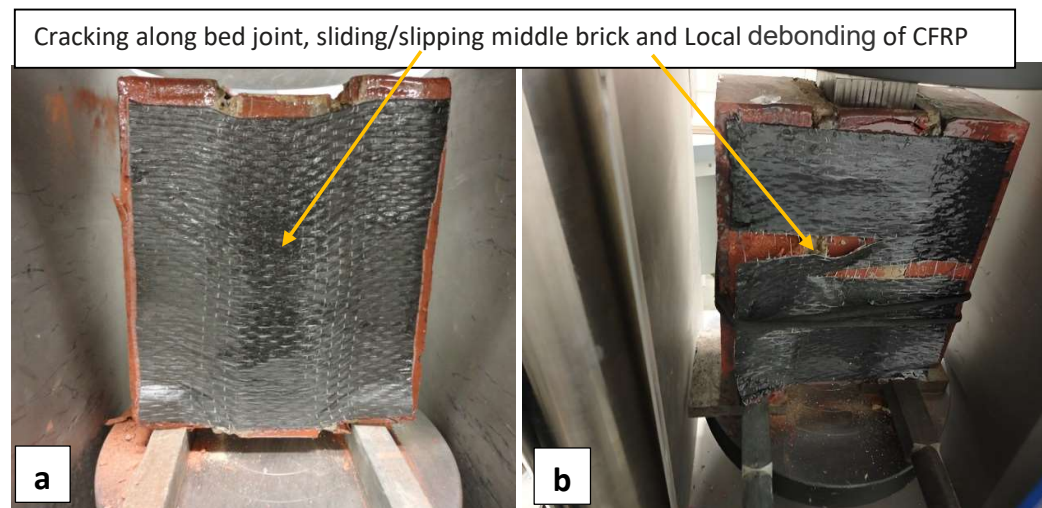


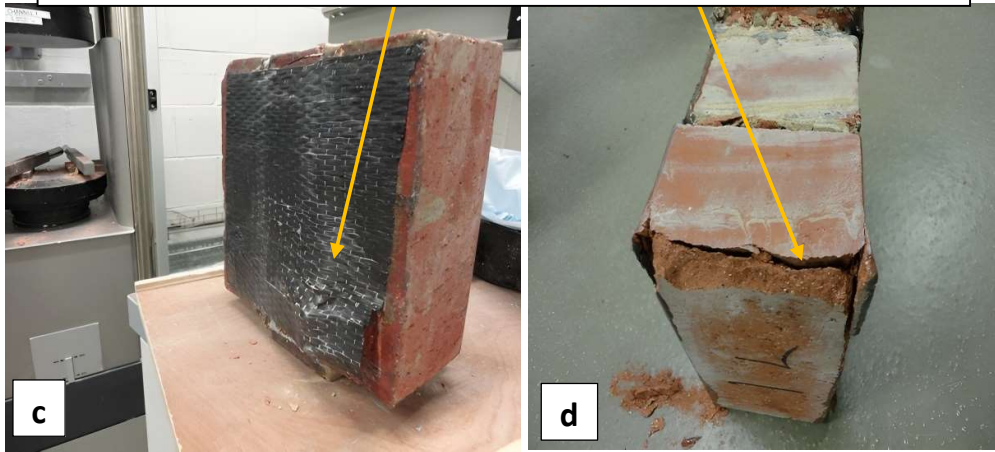
Figure 4.4c: Stress and strain curve for URM

4.5. Test results for triplet masonry with one layer of CFRP on both sides

Three samples were constructed and strengthened with one layer of CFRP sheet on both sides of triplet masonry. The mode of failure was ductile and the ultimate load was about 88kN. Debonding also occurred just before failure, the FRP ruptured at mid-span, followed by shearing of the sheets at the bottom. Debonding occurs when stresses transferred by the CFRP composite exceed the tensile strength of the substrate material. This causes the CFRP to peel off from the brick. The shear failure in the unit-mortar interface (Figures 4.5a to 4.5d). Figures 4.6a to 4.6c show the load-displacement curve, stress-displacement curve, and stress-strain curve. In this test, the triplet masonry with one layer of CFRP sheets on both sides was 7 times stronger than URM samples. Triplets glued with CFRP in full configuration exhibited higher gain in shear strength in comparison to triplets glued with CFRP in strip configuration. The premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints. Debonding of CFRP from triplet is the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure. It is observed that the shear triplet presented an almost linear shear stress-strain relationship before cracking of mortar, followed by a drop of shear strength.



Cracking along bed joint, sliding of middle brick and local debonding of CFRP



Figures 4.5a to 4.5d: Failure for triplet masonry with one layer of CFRP on both sides

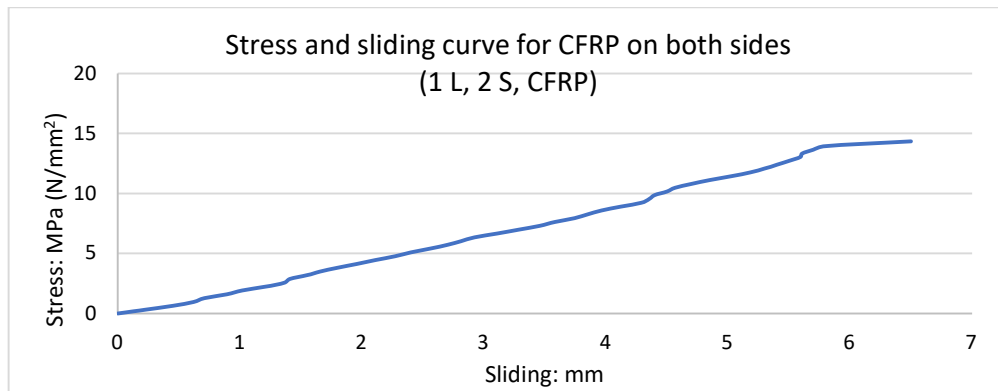


Figure 4.6a: Stress and sliding curve for CFRP on both sides

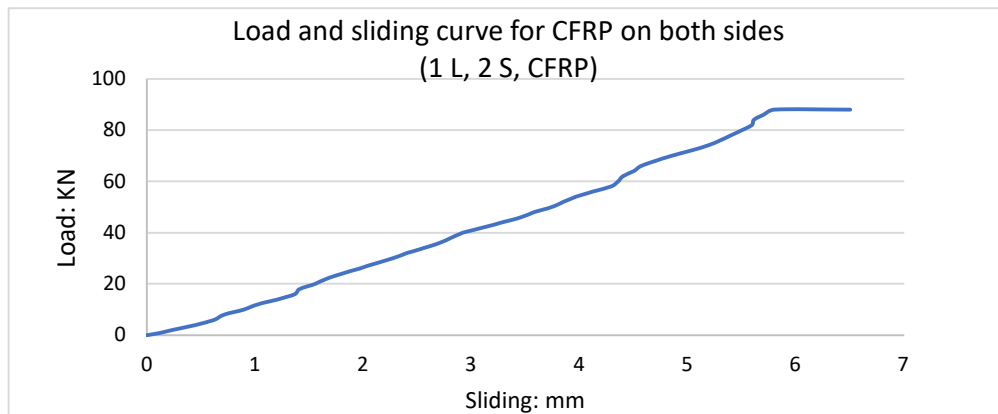


Figure 4.6b: Load and sliding curve for CFRP on both sides

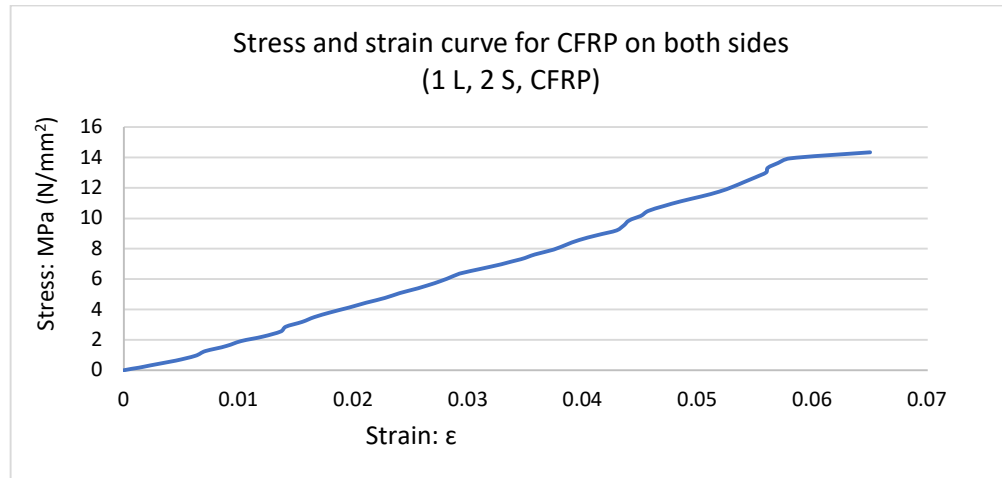


Figure 4.6c: Stress and strain curve for CFRP on both sides

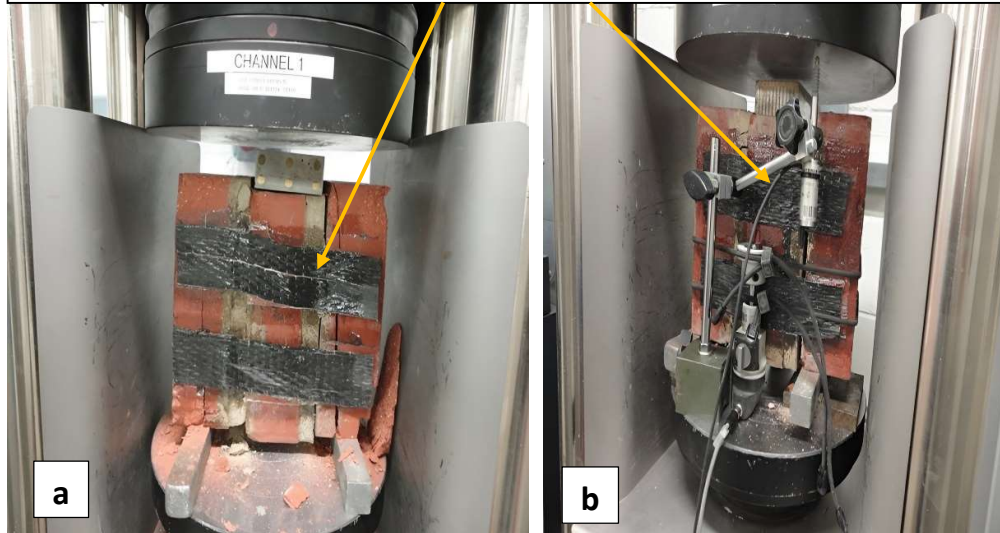
4.6. Test results for triplet masonry with one layer of two stripes on both sides

These samples were reinforced with one layer of CFRP, a 50mm wide strip on both sides of triplets. The mode of failure was ductile and the ultimate load was about 57kN. These triplets also displayed the propagation of cracks, as did the previous test. De-lamination also occurred before failure and shear failure in the unit-mortar interface shown in (Figures 4.7a to 4.7d). Figures 4.8a to 4.8c show the load-displacement curve, stress-displacement curve, and stress-strain curve. In this test, the triplet masonry with one layer of CFRP strips on both sides was 4 times stronger than URM samples. The presence of the CFRP improved the performance by changing the brittle failure to flexural failure.

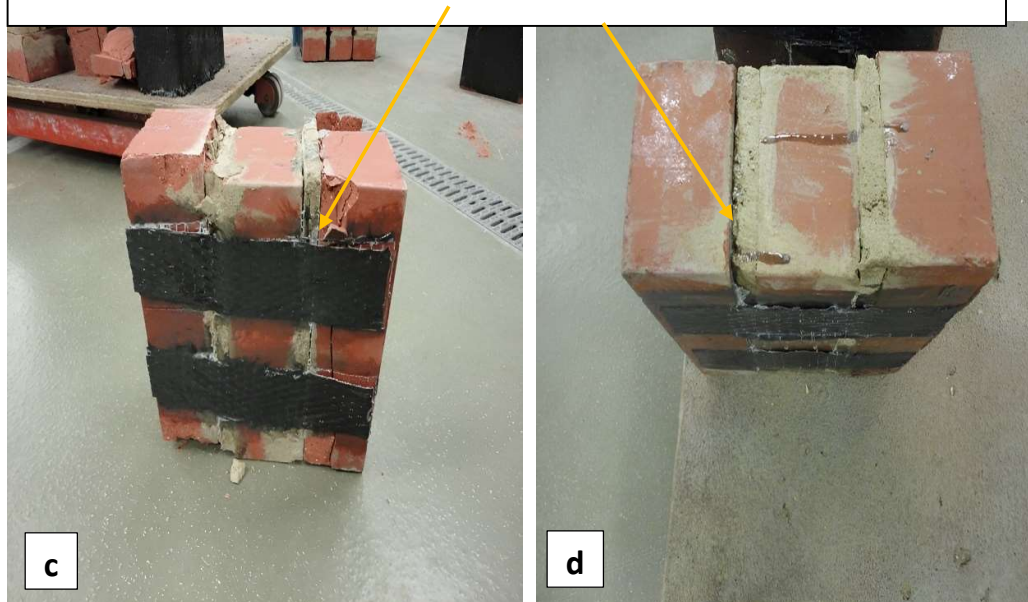
The premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints. The strips provided a ductile flexural behavior to the reinforced masonry triplets. This study aimed to increase the system ductility, generate uniform crack distribution, and increase the load-carrying capacity of the masonry. In the strengthened triplets, the failure occurred by the loss of bonding between the CFRP and the masonry units. The crack is produced by the de-bonding of the masonry units from the mortar until de-bonding of the epoxy from the masonry units. At this point, the triplet fails because the tensile stresses are no longer transferred to the CFRP. It is observed that the

shear triplet presented an almost linear shear stress–strain relationship before cracking of mortar, followed by a drop of shear strength.

Cracking along bed joint, sliding/slipping middle brick and Local debonding of CFRP



Cracking along bed joint, Sliding/slipping middle brick and Local debonding of CFRP



Figures 4.7a to 4.7d: Failure for triplet masonry with one layer of two stripes on both sides

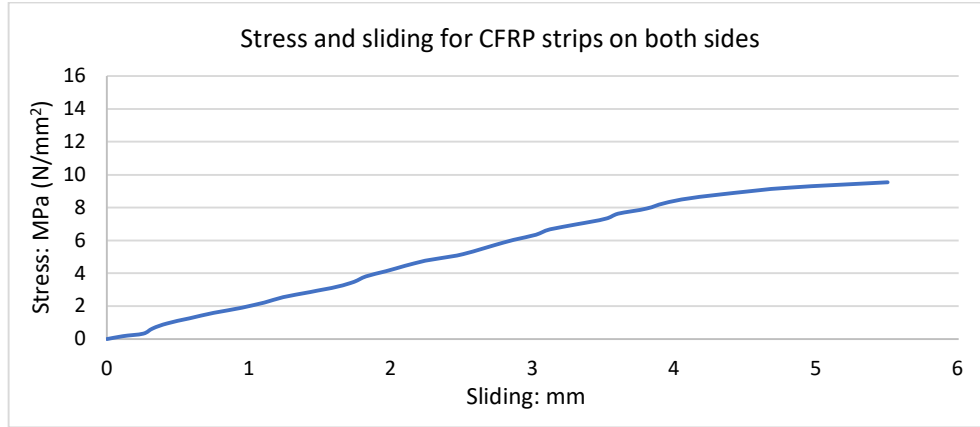


Figure 4.8a: Stress and sliding for CFRP strips on both sides

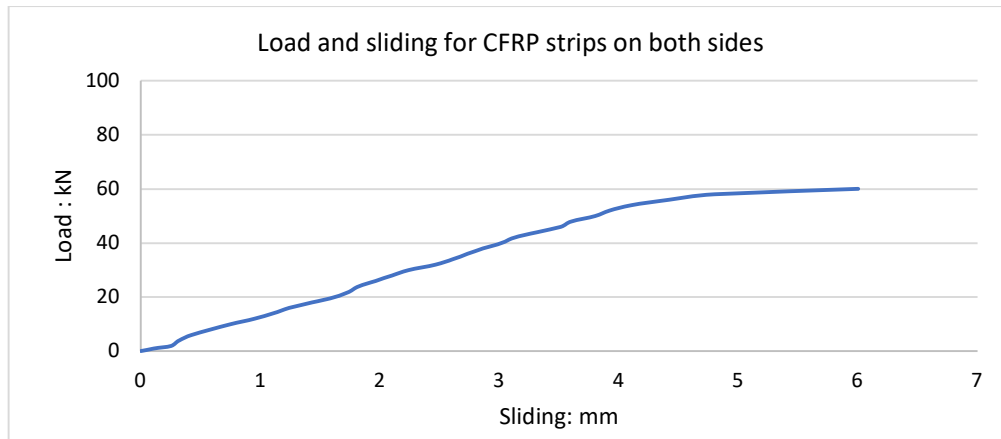


Figure 4.8b: Load and sliding for CFRP strips on both sides

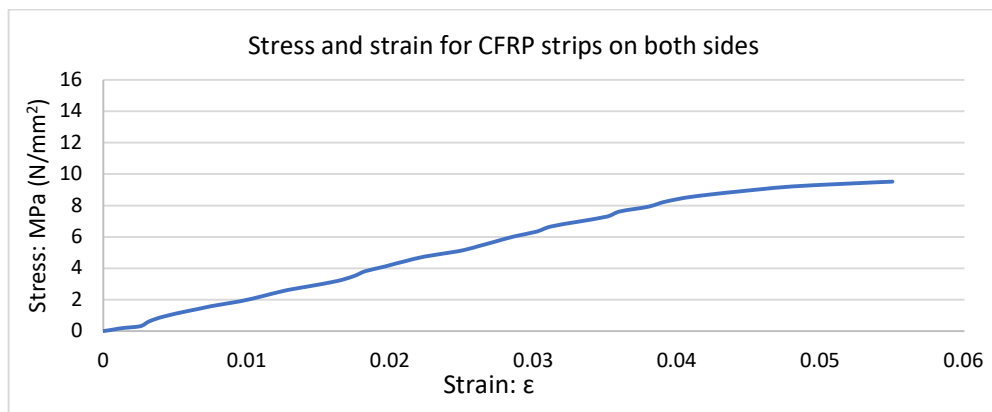


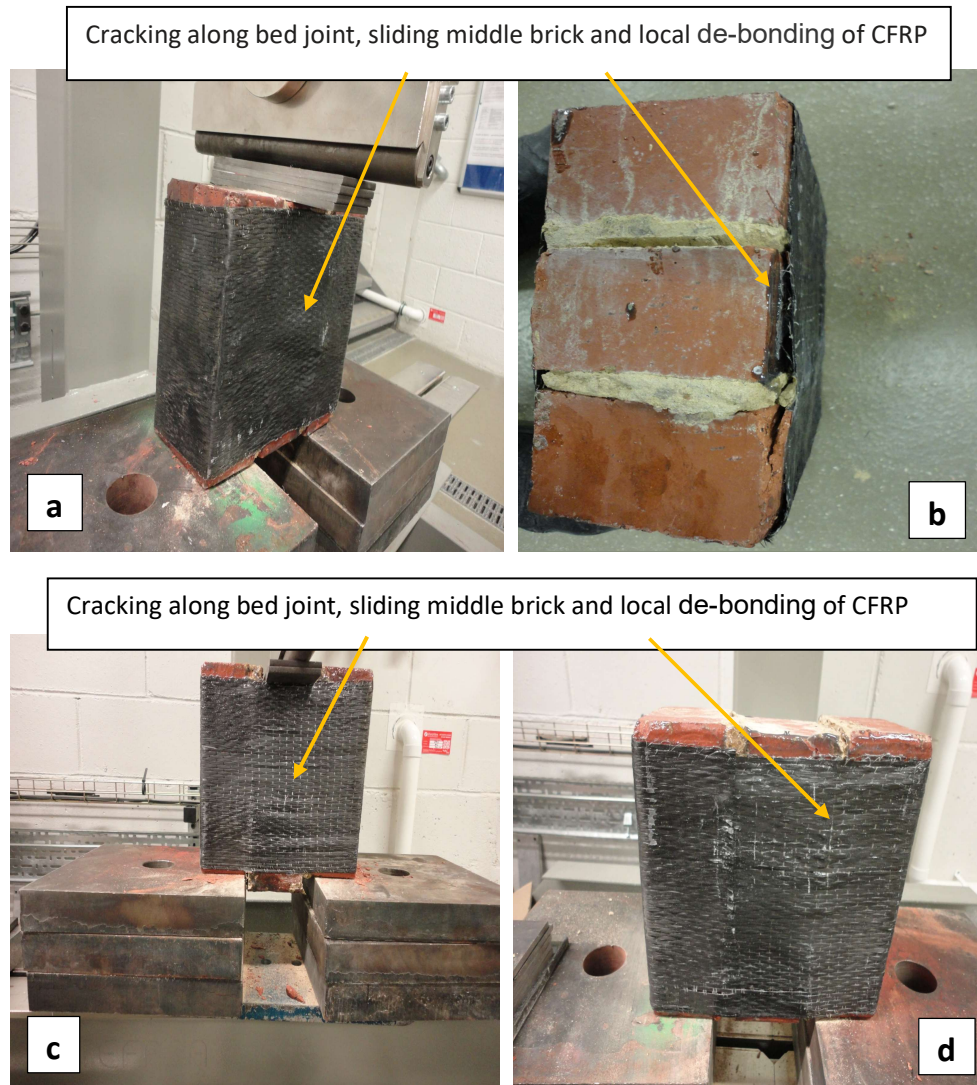
Figure 4.8c: Stress and strain for CFRP strips on both sides

4.7. Strengthening with one layer of CFRP sheets on four sides of triplet masonry

Samples were strengthened with one layer of CFRP on four sides of the triplet. The mode of failure was observed to be ductile and the ultimate load was about 88kN. These samples also displayed the propagation of cracks as did the previous test. De-lamination also occurred before failure. The shear failure in the unit-mortar interface is shown in (Figures 4.9a to 4.9d). Figures 4.10a to 4.10c show the load-displacement curve, stress-displacement curve, and stress-strain curve. In this test, the triplet masonry with one layer of CFRP on four sides was 6 times stronger than URM samples. The maximum load capacity of CFRP sheets on four sides and two sides is nearly the same, but reinforced triplets at four sides showed less displacement. This result shows the columns in old masonry buildings can be reinforced without putting extra load on structures.

The failure in the specimen reinforced with CFRP on four sides, as shown in the figures 4.9a to 4.9d, occurred through the development of sliding on the brick-mortar interface and starting of shear cracks in the mortar layer. The premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints.

Debonding of CFRP from triplet is the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure. It is observed that the shear triplet presented an almost linear shear stress-strain relationship before cracking of mortar, followed by a drop of shear strength



Figures 4.9a to 4.9d: Strengthening with one layer of CFRP sheets on four sides of triplet

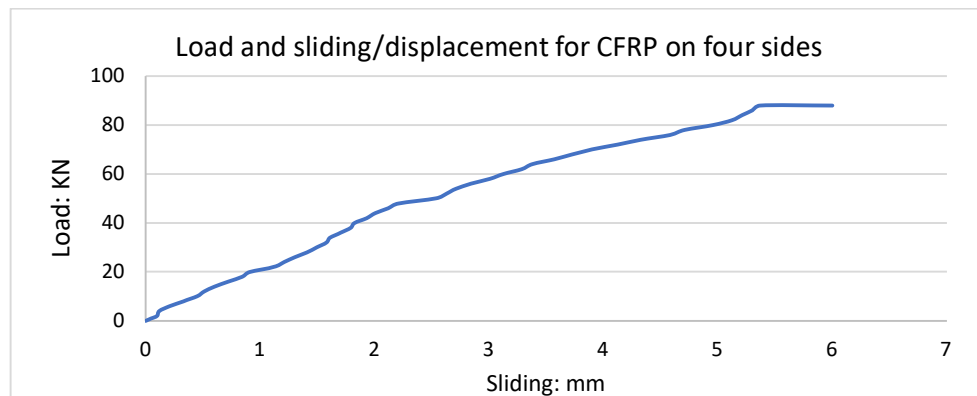


Figure 4.10a: Load and sliding/displacement for CFRP on four sides

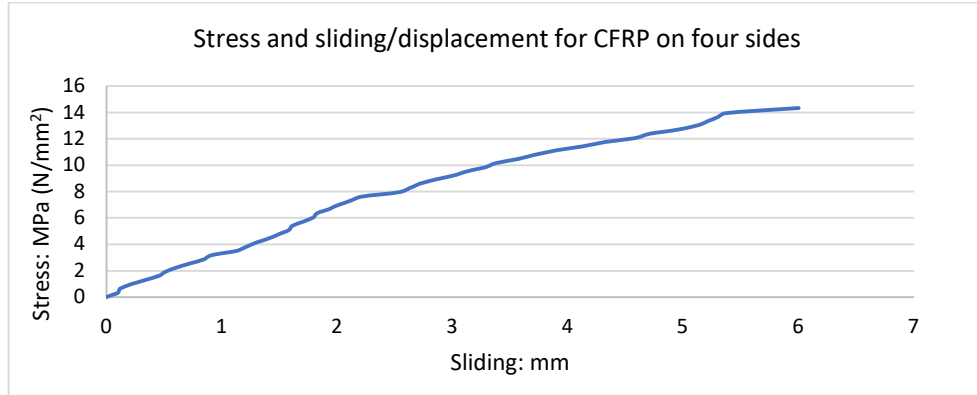


Figure 4.10b: Stress and sliding/displacement for CFRP on four sides

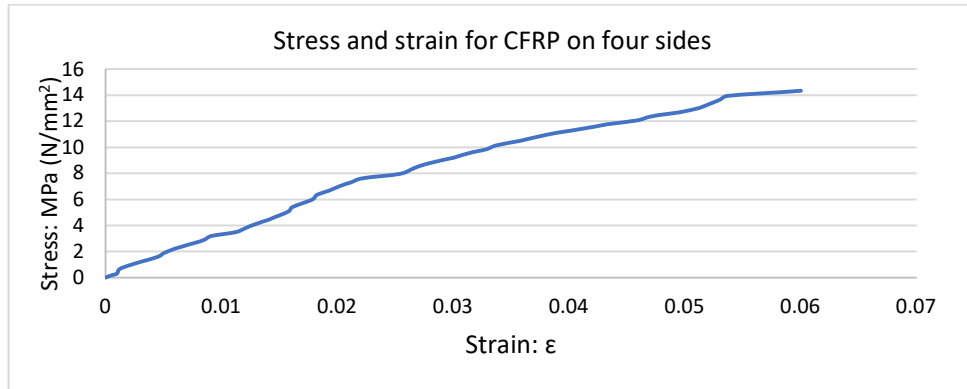


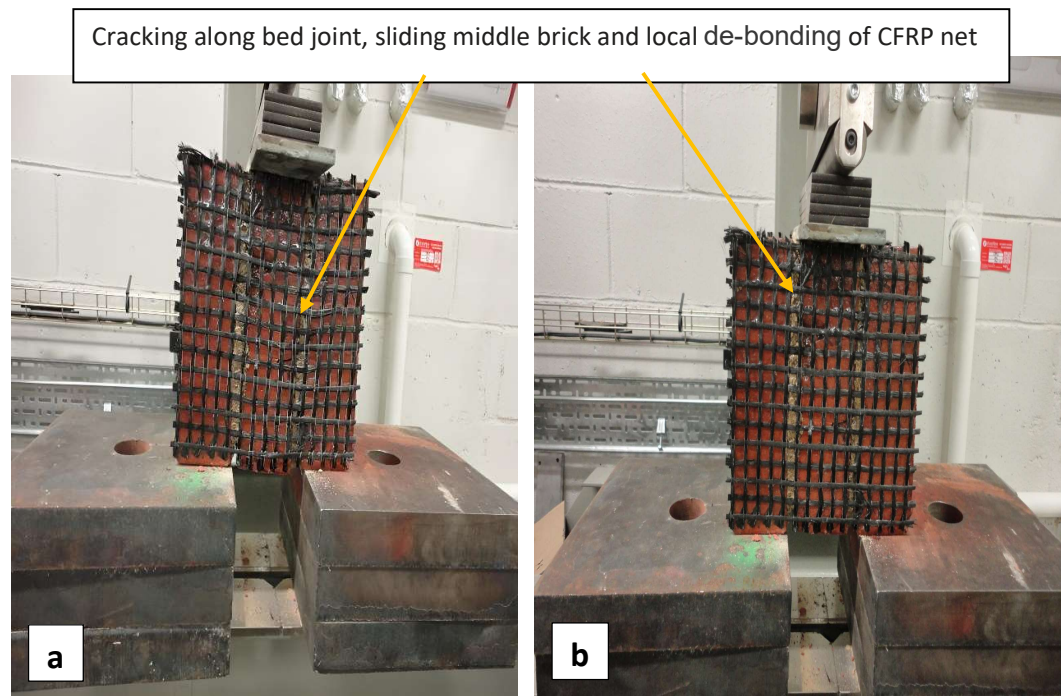
Figure 4.10c: Stress and strain for CFRP on four sides

4.8. Strengthening with one layer of CFRP net on both sides of triplet masonry

These Samples were strengthened with one layer of CFRP net on both sides of the triplets. The mode of failure was observed to be ductile and the ultimate load was about 71kN. These triplets also displayed the propagation of cracks as did the previous test. De-lamination before failure and the shear failure in the unit-mortar interface also occurred as shown in (Figures 4.11a and 4.11b). Figures 4.12a to 4.12c show the load-displacement curve, stress-displacement curve and stress-strain curve. As observed the initial slope of the load-displacement curves, for a CFRP net on four sides and CFRP on two sides of triplet specimens were nearly the same. In this test, the triplet masonry with one layer of CFRP net on both sides was 6 times stronger than URM samples and the

ultimate load capacity of CFRP net glued on four sides and two sides was nearly the same. The premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints.

The failure in the specimen reinforced with CFRP net on two sides, as shown in the figures 4.11a and 4.11b, occurred through the development of sliding on the brick–mortar interface and starting of shear cracks in the mortar layer. Debonding of CFRP from triplet is the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure. It is observed that the shear triplet presented an almost linear shear stress–strain relationship before cracking of mortar, followed by a drop of shear strength.



Figures 4.11a and 4.11b: Strengthening with one layer of CFRP net on both sides of triplet

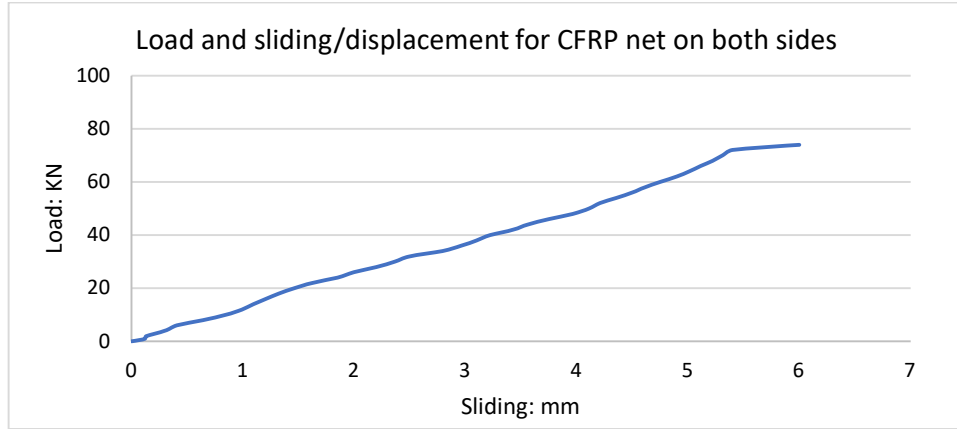


Figure 4.12a: Load and sliding/displacement for CFRP net on both sides

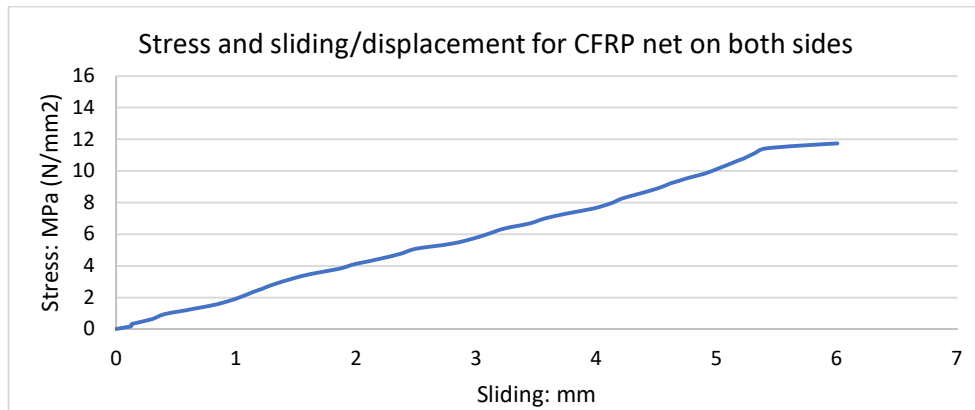


Figure 4.12b: Stress and sliding/displacement for CFRP net on both sides

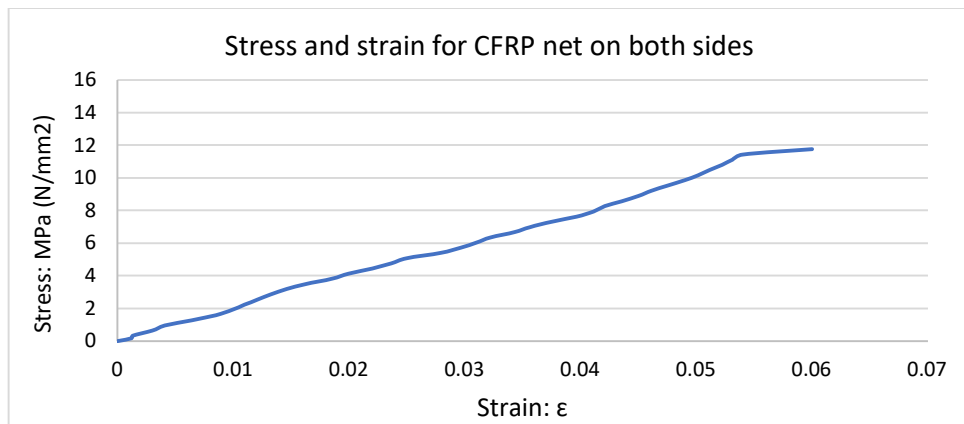


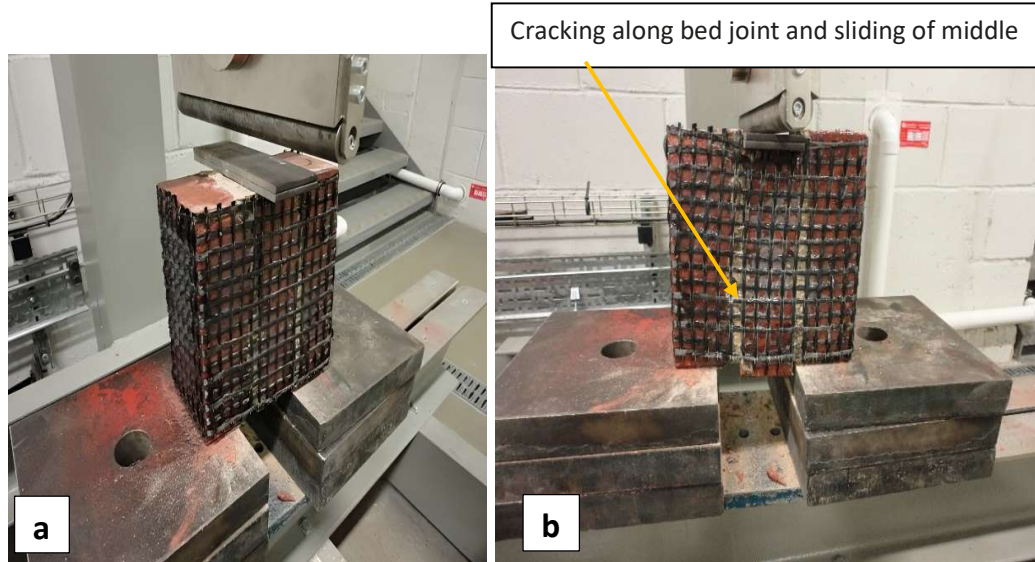
Figure 4.12c: Stress and strain for CFRP net on both sides

4.9. Strengthening with one layer of CFRP net on four sides of triplet masonry

These samples were strengthened with one layer of CFRP net on four sides of triplet masonry. The mode of failure was observed to be ductile and the ultimate load was about 78kN. Debonding occurs when stresses transferred by the CFRP composite exceed the tensile strength of the substrate material. This causes the CFRP net to peel off and cause damage to the triplet specimen.

The shear failure in the unit-mortar interface is shown in (Figures 4.13a and 4.13b). Figures 4.14a to 4.14c show the load-displacement curve, stress-displacement curve, and stress-strain curve. In this test, the triplet masonry with one layer of CFRP net on four sides was 6 times stronger than URM samples. As observed the initial slope of the load-displacement curves, for a CFRP net on four sides and CFRP on two sides of triplet specimens were nearly the same.

The failure in the specimen reinforced with CFRP net on four sides, as shown in the figures 4.13a and 4.13b, occurred through the development of sliding on the brick-mortar interface and starting of shear cracks in the mortar layer. Debonding of CFRP from triplet is the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure. It is observed that the shear triplet presented an almost linear shear stress-strain relationship before cracking of mortar, followed by a drop of shear strength. Also, the premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints.



Figures 4.13a and 4.13b: One layer of CFRP net on four sides of triplet masonry before and after failure

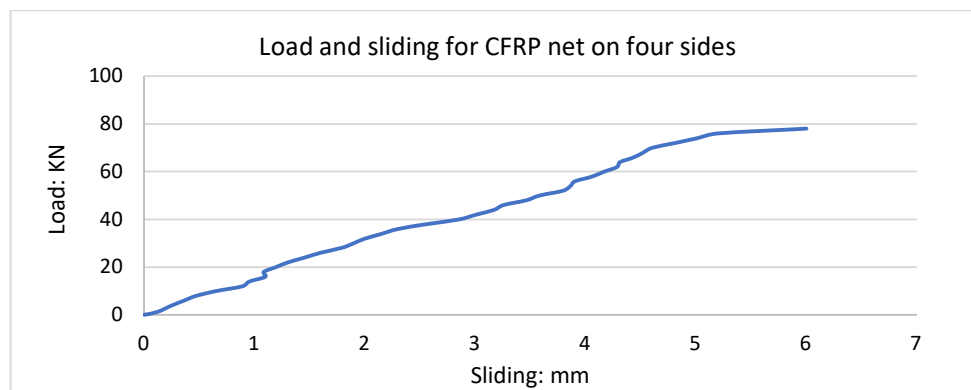


Figure 4.14a: Load and sliding/displacement for CFRP net on four sides

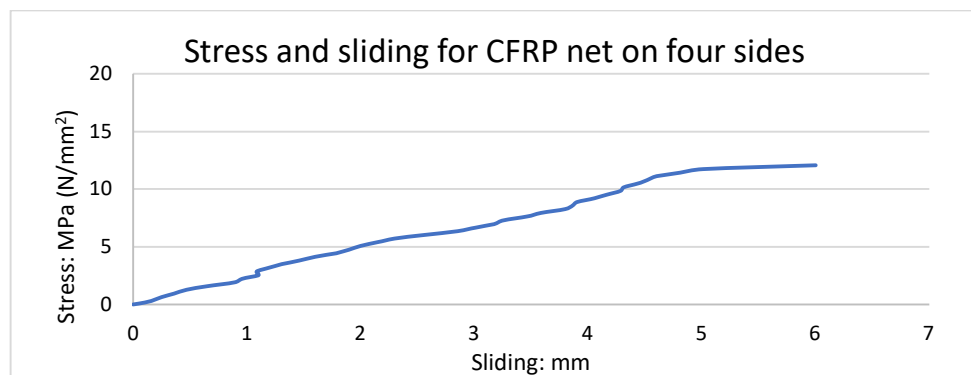


Figure 4.14b: Stress and sliding/displacement for CFRP net on four sides

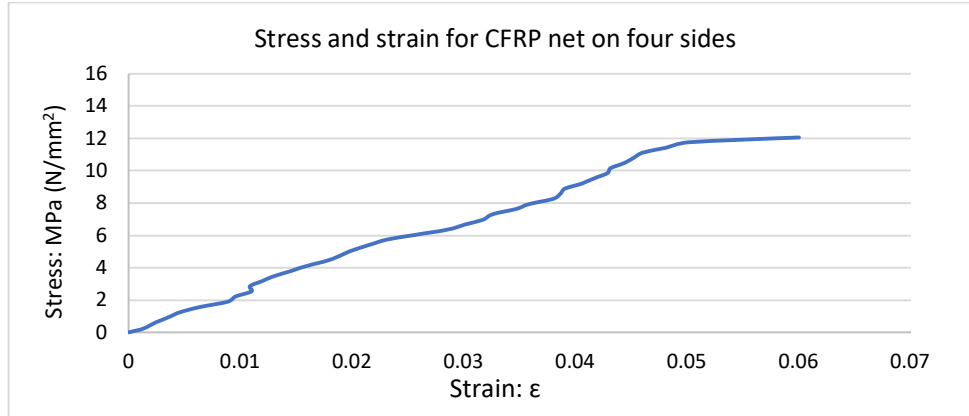


Figure 4.14c: Stress and strain for CFRP net on four sides

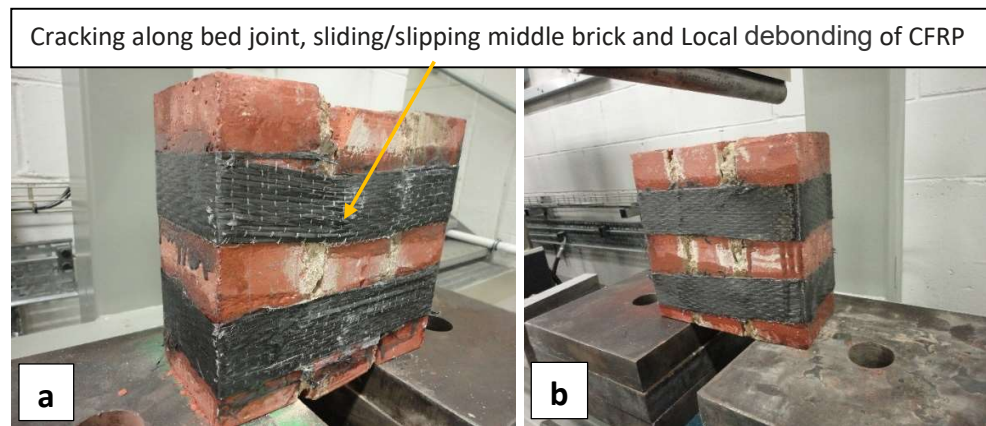
4.10. Strengthening with one layer of CFRP strips on four sides of triplet masonry

These samples were strengthened with one layer of CFRP strip 50mm wide on four sides of triplet masonry. The mode of failure was ductile and the ultimate load was about 57kN, which is slightly stronger than CFRP strips on two sides of the triplet. The propagation of cracks and de-lamination also occurred before failure.

Figures 4.15a and 4.15b present the shear failure in the unit-mortar interface and Figures 4.16a to 4.16c show the load-displacement curve, Stress-Displacement curve and stress-strain curve. Triplet masonry with one layer of CFRP strips on four sides was 5 times stronger than URM samples and it is a reliable option to reinforce the columns in old masonry buildings.

The failure in the specimen reinforced with CFRP strips on two sides, as shown in the figures 4.15a and 4.15b, occurred through the development of sliding on the brick-mortar interface and starting of shear cracks in the mortar layer. Debonding of CFRP from triplet is the reason for failure of CFRP-strengthened triplet, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure. The premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints.

It is observed that the shear triplet presented an almost linear shear stress–strain relationship before cracking of mortar, followed by a drop of shear strength.



Figures 4.15 and 4.15b: Strengthening with one layer of CFRP strips on four sides of triplet masonry.

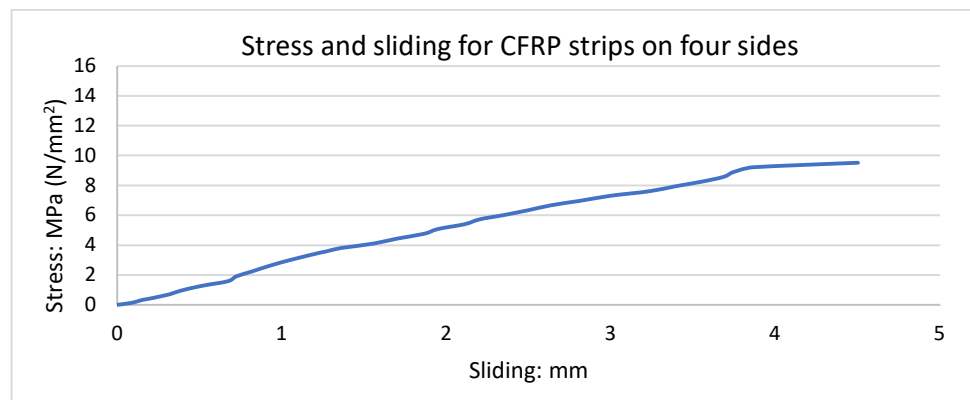


Figure 4.16a: Stress and sliding for CFRP strips on four sides

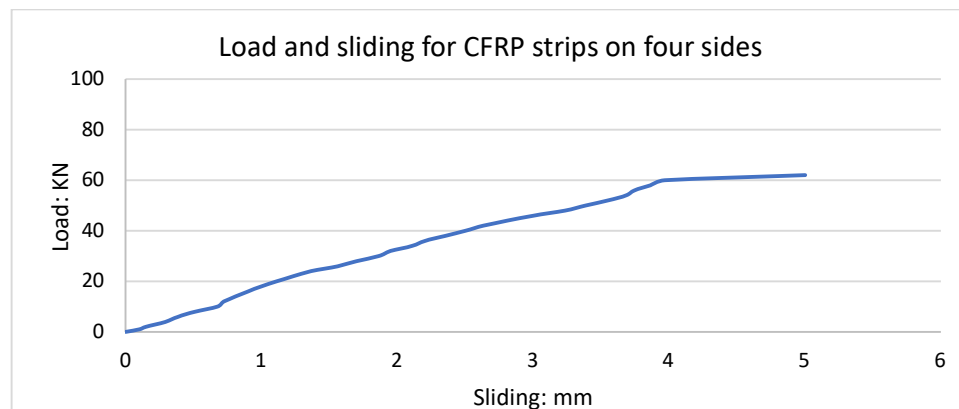


Figure 4.16b: Load and sliding for CFRP strips on four sides

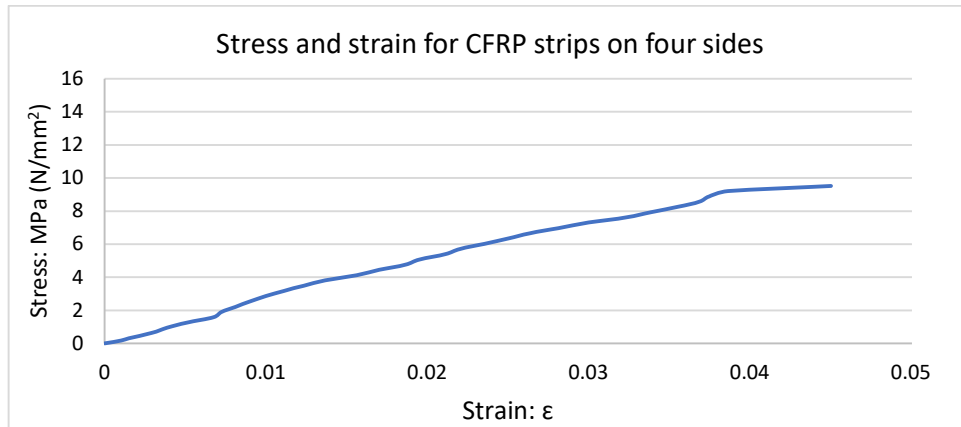


Figure 4.16c: Stress and strain for CFRP strips on four sides

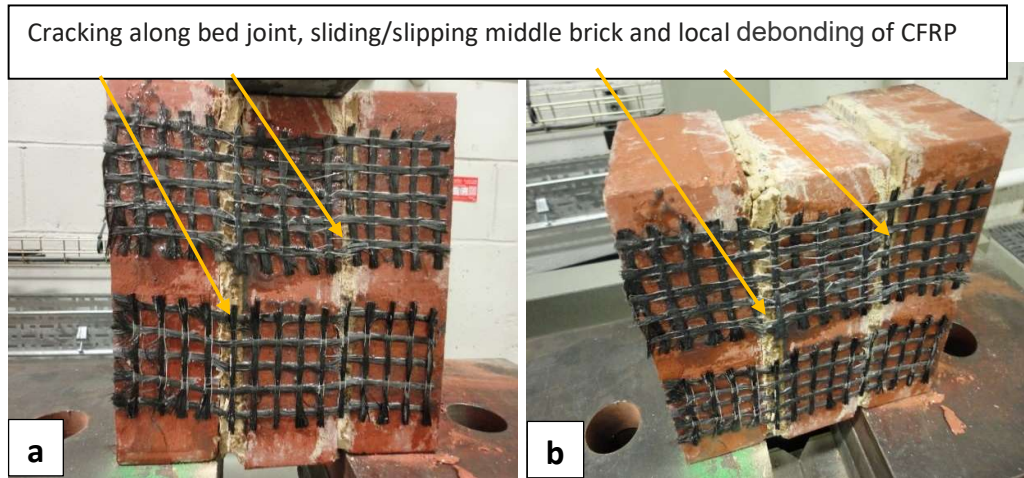
4.11. Strengthening with one layer of CFRP net strips on both sides of triplet masonry

These samples were strengthened with one layer of two CFRP net strips 50mm wide strip on both sides of triplets. The mode of failure was observed to be ductile and the ultimate load was about 52kN. The triplets also displayed the propagation of cracks as did the previous test and de-lamination also occurred before failure.

Figures 4.17 a and 4.17b show the shear failure in the unit-mortar interface and Figures 4.18a to 4.18c show the load-displacement curve, stress-displacement curve and stress-strain curve.

In this test, the triplet masonry with one layer of CFRP net strips on both sides was 4 times stronger than URM samples. The failure in the specimen reinforced with CFRP net strips on two sides, as shown in the figures 4.17a and 4.17b, occurred through the development of sliding on the brick-mortar interface and starting of shear cracks in the mortar layer. Debonding of CFRP from triplet is the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.

The premature shear failure of triplet masonry specimen can be eliminated by CFRP net reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints.



Figures 4.17a and 4.17b: Strengthening triplets with one layer of CFRP net strips on both sides

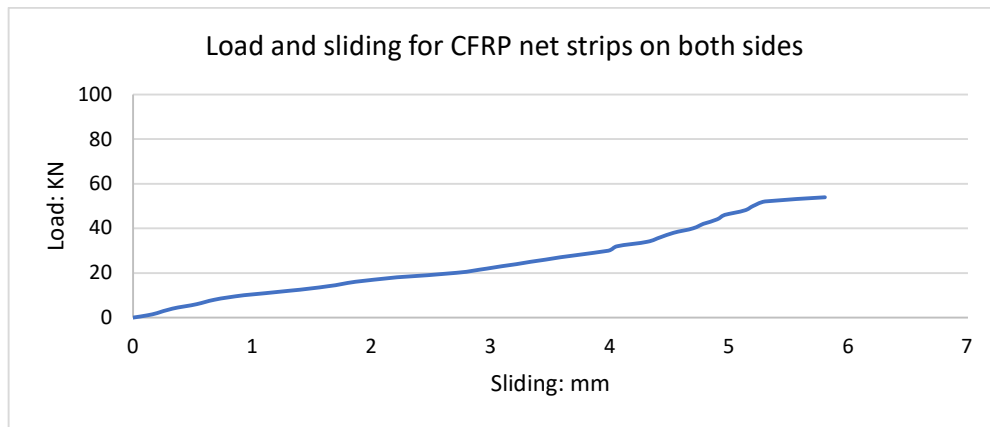


Figure 4.18a: Load and sliding for CFRP net strips on both sides

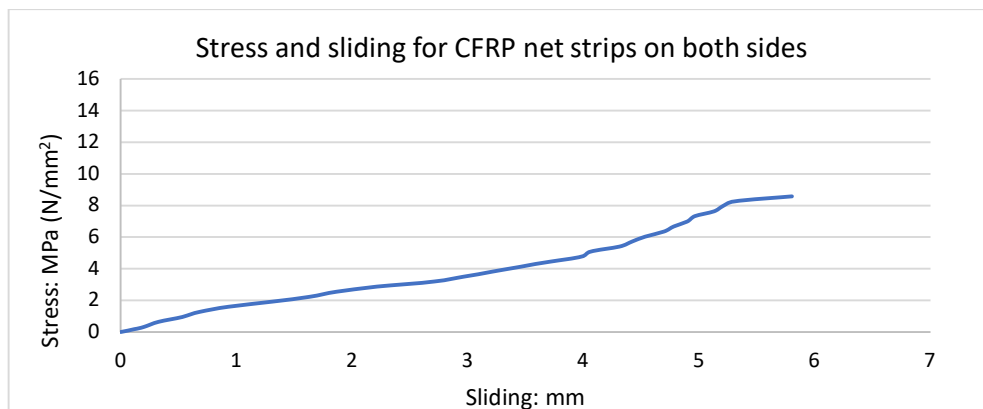


Figure 4.18b: Stress and sliding for CFRP net strips on both sides

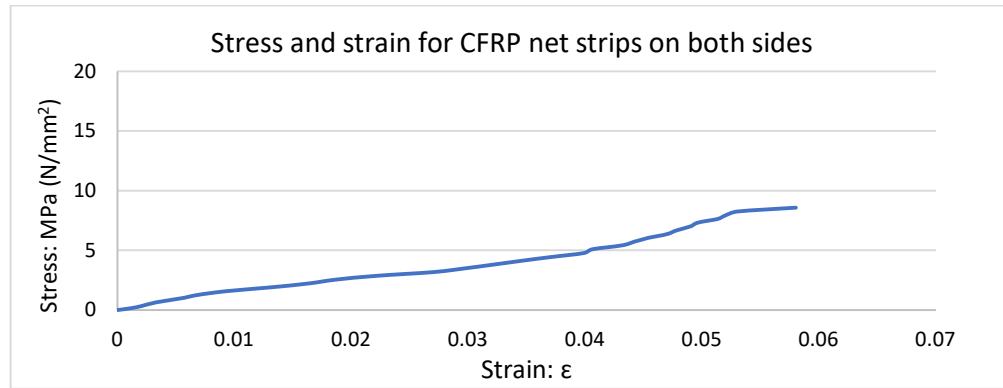
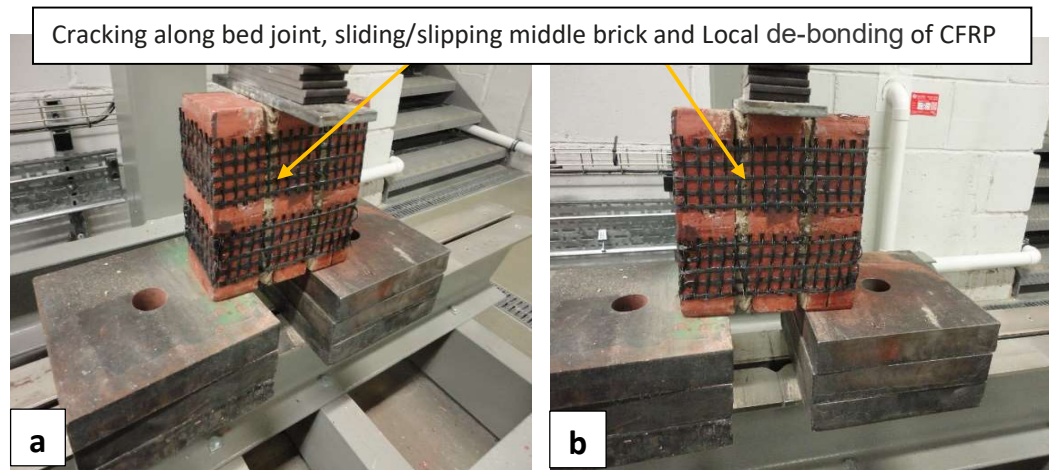


Figure 4.18c: Stress and strain for CFRP net strips on both sides

4.12. Strengthening with one layer of CFRP net strips on four sides of triplet masonry

The samples were strengthened with one layer of two CFRP net strips 50mm wide strip on four sides of triplets. The mode of failure was ductile and the ultimate load was about 60kN. These triplets displayed the propagation of cracks and de-lamination also occurred before failure. The shear failure in the unit-mortar interface and brick is shown in (Figures 4.19a and 4.19b). Figures 4.20a to 4.20c show the load-sliding, stress-sliding and stress-strain curves. It is observed that the shear triplet presented an almost linear shear stress–strain relationship before cracking of mortar, followed by a drop of shear strength.

In this test, the triplet masonry with one layer of CFRP net strips on four sides was 5 times stronger than URM samples. This result shows the failure in the specimen reinforced with CFRP net on four sides, as shown in the figures 4.19a and 4.19b, occurred through the development of sliding on the brick–mortar interface and starting of shear cracks in the mortar layer. Debonding of CFRP from triplet is the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. In contrast with URM walls, reinforced triplets were stable after failure. Also, the premature shear failure of triplet masonry specimen can be eliminated by CFRP reinforcement, which also prevented the micro-cracks from extending through the mortar layer. This shows that CFRP reinforcement helped to support the tensile force at the bed joints.



Figures 4.19a and 4.19b: Strengthening with one layer of CFRP net strips on four sides of triplets

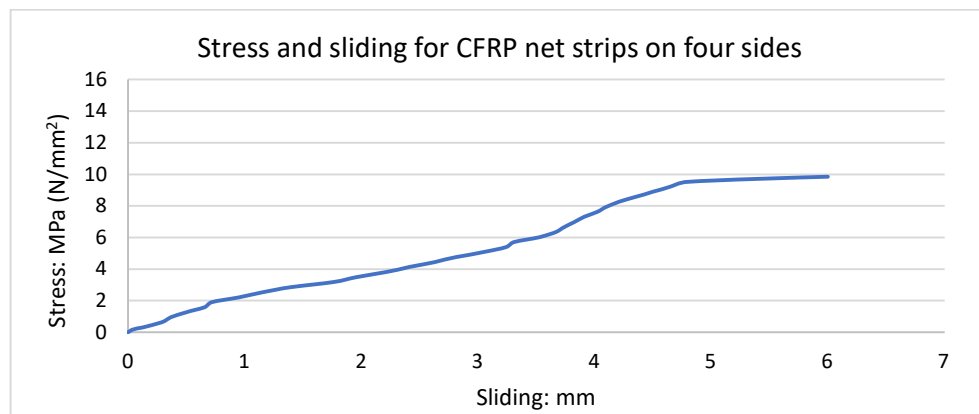


Figure 4.20a: Stress and sliding for CFRP net strips on four sides

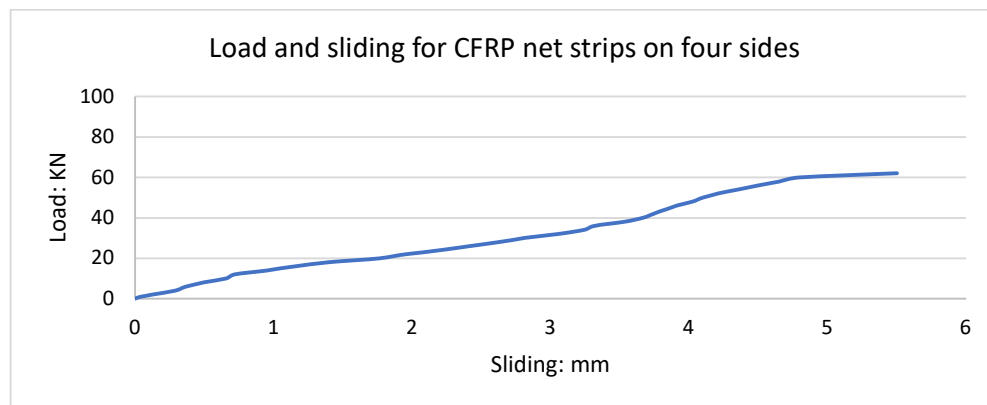


Figure 4.20b: Load and sliding for CFRP net strips on four sides

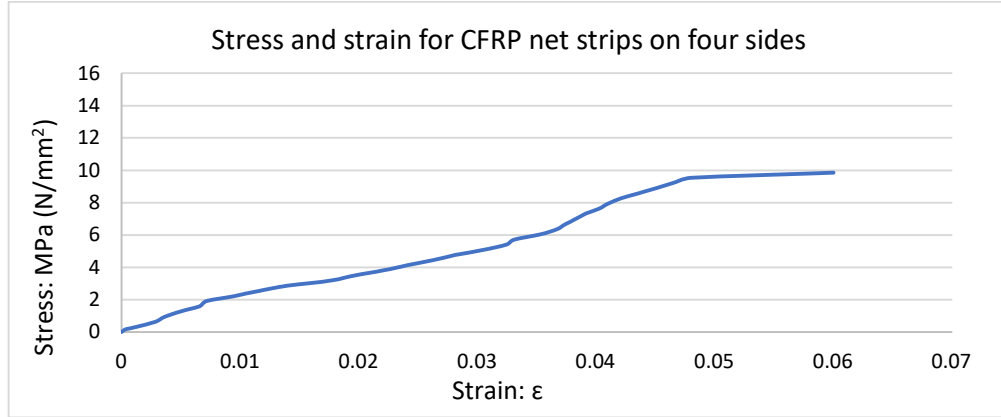


Figure 4.20c: Stress and strain for CFRP net strips on four sides

4.13. New equation for shear behaviour of unreinforced triplet masonry (URTM)

The results of the experimental program are used to evaluate the shear behavior of the investigated masonry. According to EN 1052-3:2002, the normal and shear stresses can be supposed as uniformly distributed over the two bed joints during the triplet test. The maximum value of the experimental shear stress $\tau_{\max}$ on the bed joints can be calculated as follows:

$$\tau_{\max} = \frac{F_{\max}}{2A} \quad (4.3)$$

Where $\tau_{\max}$ is the maximum value of the experimental shear stress and $F_{\max}$ is the maximum

shear force at failure and A is the total area of the bed joint equal to the cross-section of the unit. Based on experimental tests in this research equation 3 for solid London bricks and mortar M4 is not accurate and cannot be used. Therefore, the shear strength of triplet masonry with solid brick and mortar M4 can be calculated as Equation 4.4.

$$\tau_{\max} = \frac{0.70 \times F_{\max}}{A \times \rho} \quad (4.4)$$

Where $\rho = 0.20$ for mortar M4 and A is the total area of the bed joint equal to the cross-section of the unit and $F_{\max}$ is the maximum shear force at failure, also in this new equation τ (the shear strength of triplet masonry) can be calculated at any shear force point. Table 4.1 and Figure 4.21 shows the comparison of the experimental shear tests and the results from equation 4.4 The 0.7 is a coefficient calculated based on the experimental test.

Test	Maximum Load: KN	$\tau_{\max} = \frac{F_{\max}}{2A}$ (Mpa)	$\tau_{\max} = \frac{0.70 \times F_{\max}}{A \times \rho}$ (Mpa)	Experimental shear tests (Mpa)
1	4	0.087	0.612	0.64
2	6	0.13	0.918	0.95
3	8	0.18	1.23	1.27
4	10	0.22	1.53	1.59
5	12	0.26	1.84	1.88
6	14	0.31	2.1	2.15

Table 4.1: Comparison of the experimental shear tests the results from Equations 4 and EN 1052-3:2002.

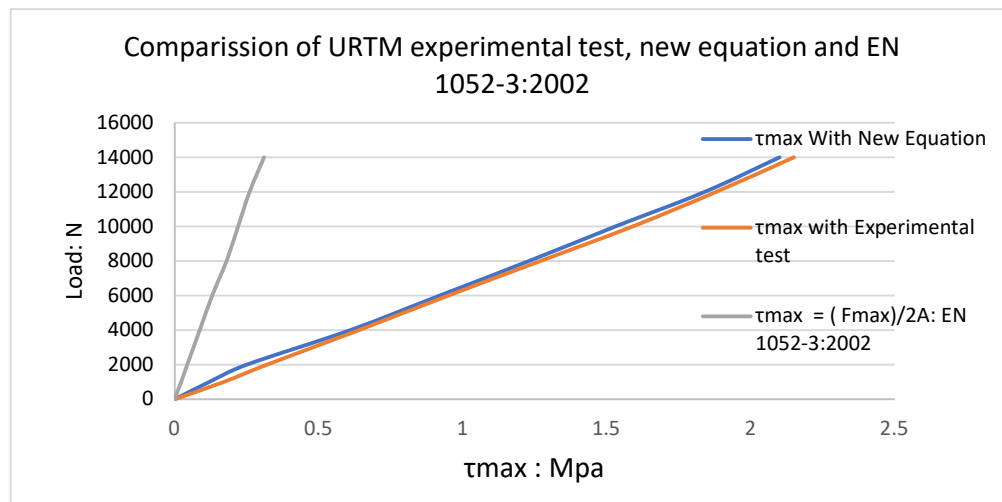


Figure 4.21: Comparison between results from EN 1052-3:2002, experimental test and new equation (URTM).

4.14. Experimental study on shear behaviour of masonry triplet with two-layers CFRP and comparisons with one-layer

The experimental tests contained 60 masonry triplets and they were tested under in-plane shear loading with CFRP and CFRP net composites in seventeen configurations:

- 1 - One-layer CFRP on two sides of triplet masonry
- 2 - One-layer CFRP on four sides of triplet masonry
- 3 - Two-layer CFRP on two sides of triplet masonry
- 4 - Two-layer CFRP on four sides of triplet masonry
- 5 - One-layer CFRP strips on two sides of triplet masonry
- 6 - One-layer CFRP strips on four sides of triplet masonry
- 7 - Two-layer CFRP strips on two sides of triplet masonry
- 8 - Two-layer CFRP strips on four sides of triplet masonry
- 9 - One-layer CFRP net on two sides of triplet masonry
- 10 - One-layer CFRP net on four sides of triplet masonry
- 11 - Two-layer CFRP net on two sides of triplet masonry
- 12 - Two-layer CFRP net on four sides of triplet masonry
- 13 - One-layer CFRP net strips on two sides of triplet masonry
- 14 - One-layer CFRP net strips on four sides of triplet masonry
- 15 - Two-layer CFRP net strips on two sides of triplet masonry
- 16 - Two-layer CFRP net strips on four sides of triplet masonry
- 17 - URTM (un-reinforced triplet masonry)

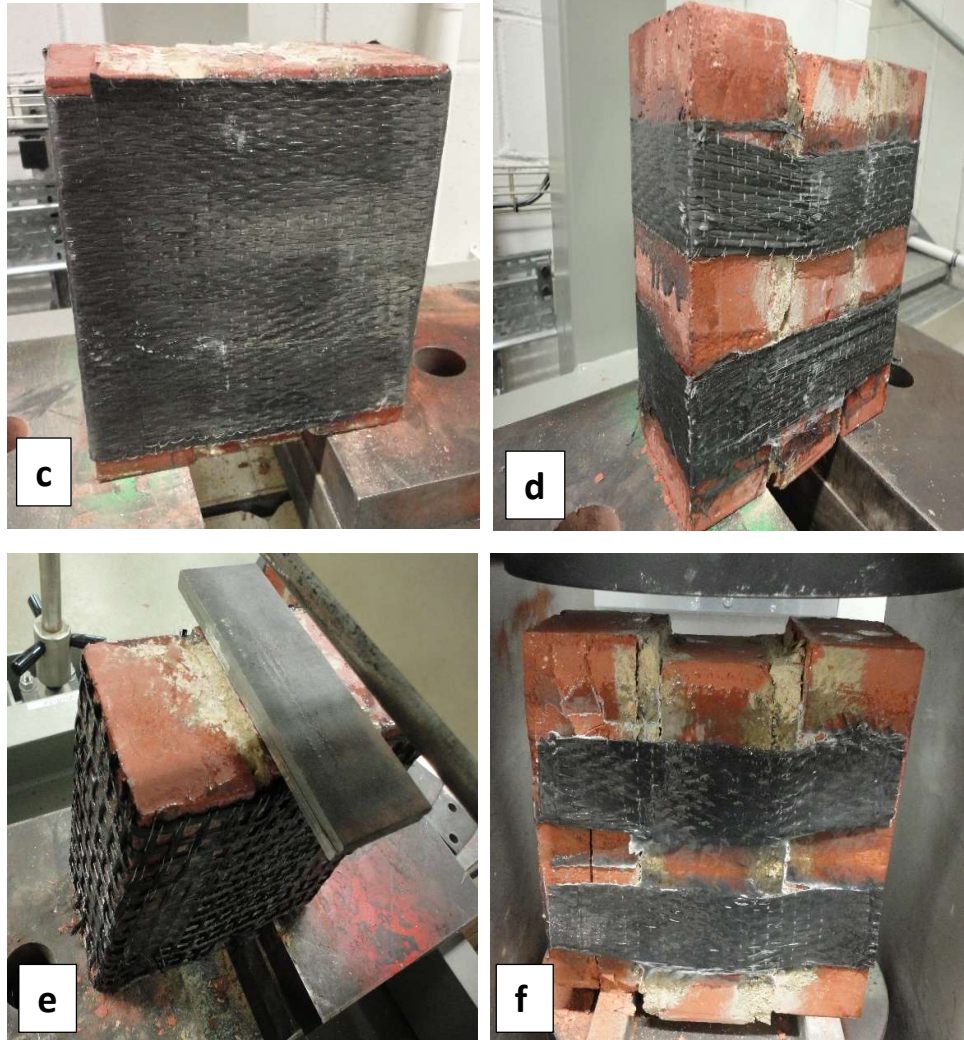
The experimental tests were carried out with a Hydraulic compression machine. According to experimental tests sliding continued after failure and it was assumed that the line continues horizontally. The test setup for testing the shear triplet specimens was a steel plate at the top and bottom of the triplets and the shear loading was applied at the top of the middle brick. The shear resistance of URTM increased without deformation up to a load of 10 kN and increased linearly from 10 kN to 13 kN up to failure. The crack started along the bed joint after deformation of 0.2 mm and the triplets failed suddenly. This showed that the triplets without reinforcement failed through the splitting of the brick and bed joint, which is an effect of the weak bond between mortar

and brick. The CFRP retrofitting increased the ultimate load of shear triplets. As the results are shown in Figure 4.23 to Figure 4.40, the CFRP reinforcement increases the ultimate load resistance and deformational capacity. Comparing one layer and two layers of CFRP or CFRP strips on triplet masonry showed It is no significant bond improvement between one layer and two layers. It may the layers stick together and act as one layer. Depending on CFRP configurations, two layers make triplets 5% to 15% stronger, which is not sufficient to apply two or three layers. A comparison of reinforced triplets with one layer and two layers are shown in figures 4.41 to 4.56.

The ultimate load increased significantly with CFRP material. Therefore, the increase in strength is quite significant and the flexure failure followed by the sliding of the central brick and mortar joint is observed. The load-displacement curves for the specimens retrofitted with different CFRP and the comparison with the unreinforced triplets are presented in Figure 4.57. A comparison of ultimate load, type of collapse, stress, and displacement for different configurations of FRP on triplet masonry is shown in Table 4.2. (In all Figures L is the layer and S represents a side of triplets).

Two layers of CFRP configurations on maspnry triplets shown in Figures 4.22 a to 4.22f.





Figures 4.22 a to 4.22f: Two layers of CFRP configurations on maspnry triplets.

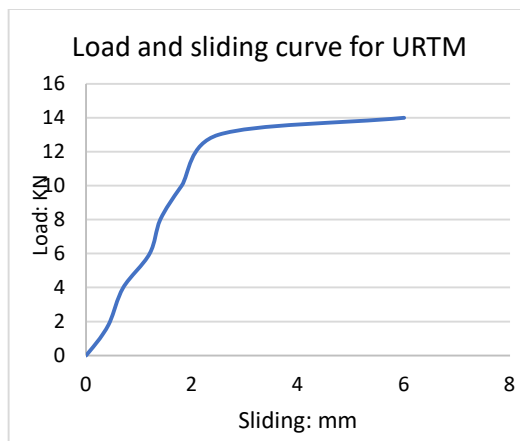


Figure 4.23: Load and sliding for URTM

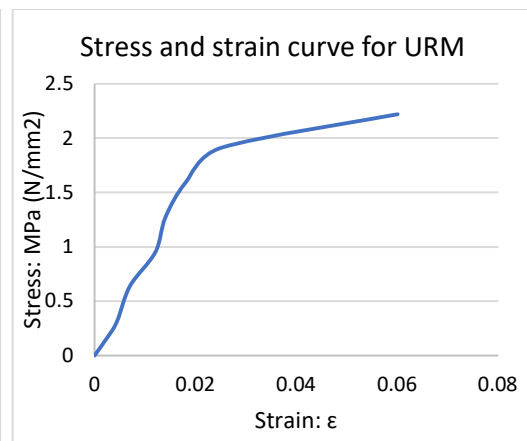


Figure 4.24: Stress and strain curve for URM

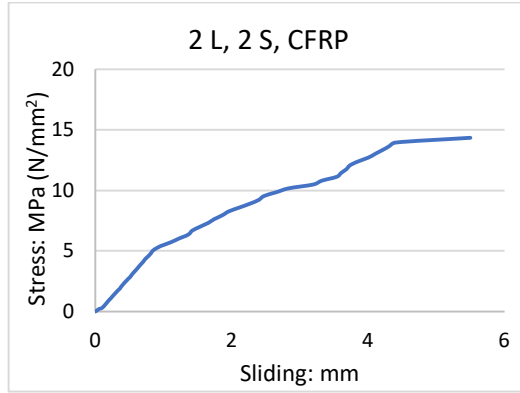


Figure 4.25: Stress and sliding for 2 L, 2 S, CFRP

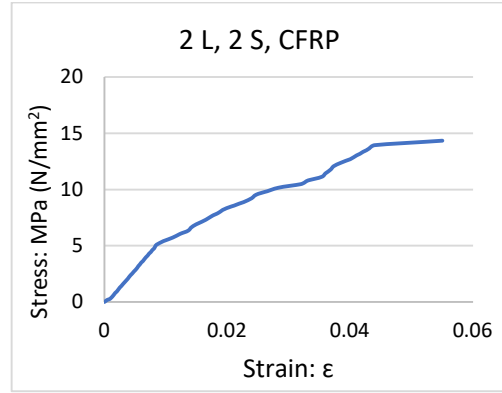


Figure 4.26: Stress and strain for 2 L, 2 S, CFRP

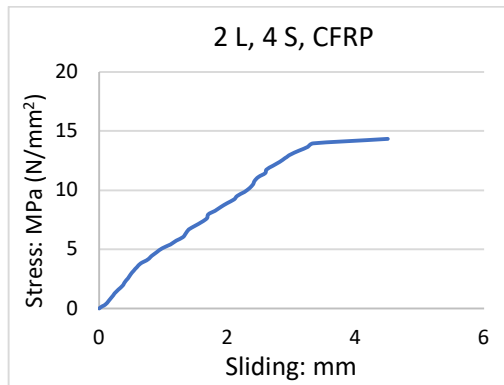


Figure 4.27: Stress and sliding for 2 L, 4 S, CFRP

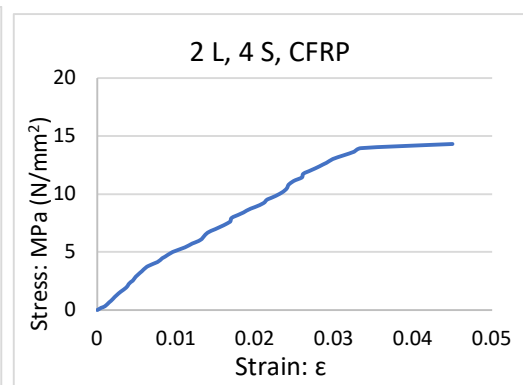


Figure 4.28: Stress and strain for 2 L, 4 S, CFRP

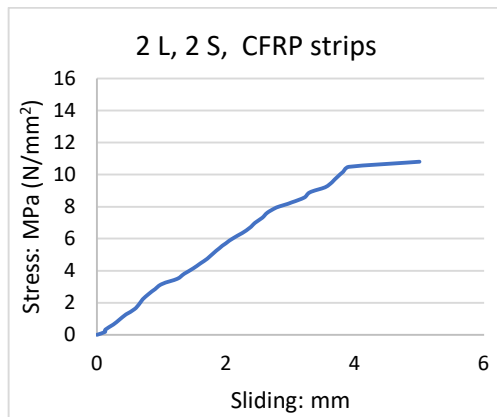


Figure 4.29: Stress and sliding for 2 L, 2 S,
CFRP strips

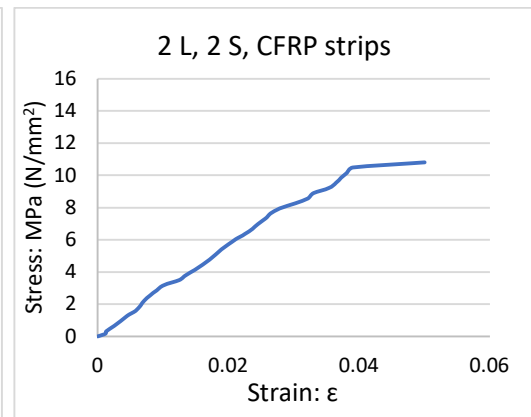


Figure 4.30: Stress and strain for 2 L, 2 S,
CFRP strips

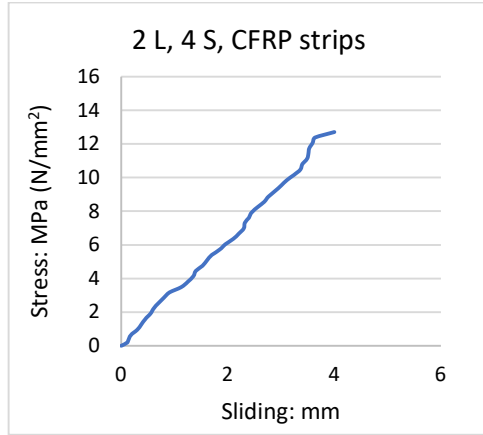


Figure 4.31: Stress and sliding for 2 L, 4 S,
CFRP strips

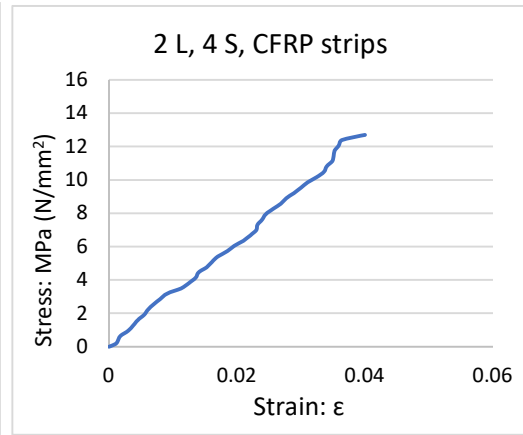


Figure 4.32: Stress and strain for 2 L, 4 S,
CFRP strips

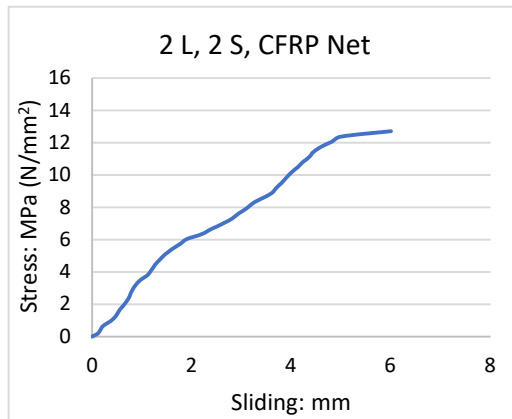


Figure 4.33: Stress and sliding for 2 L, 2 S,
CFRP net

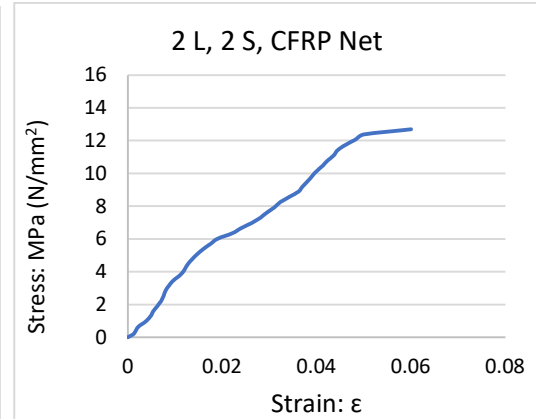


Figure 4.34: Stress and strain for 2 L, 2 S,
CFRP net

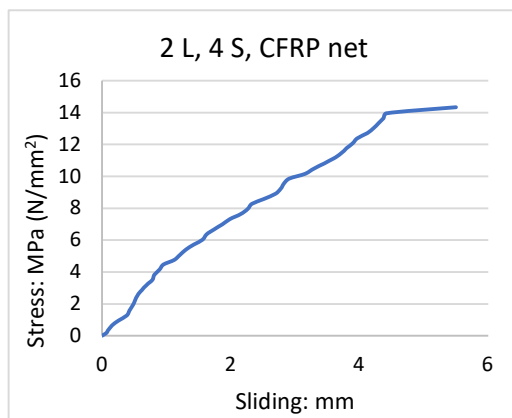


Figure 4.35: Stress and sliding for 2 L, 4 S,
CFRP net

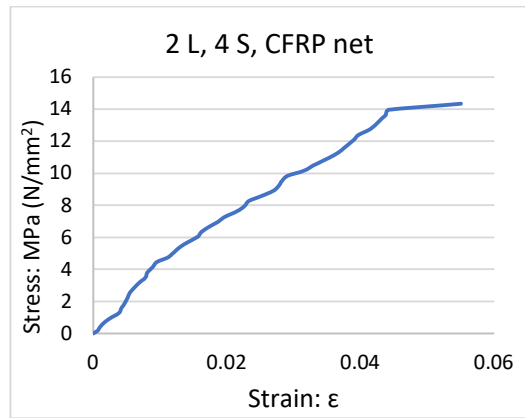


Figure 4.36: Stress and strain for 2 L, 4 S,
CFRP net

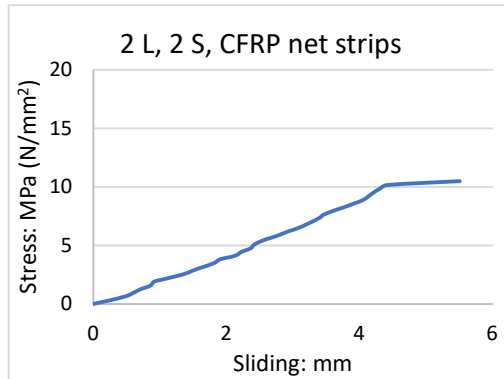


Figure 4.37: Stress and sliding for 2 L, 2 S, CFRP net strips

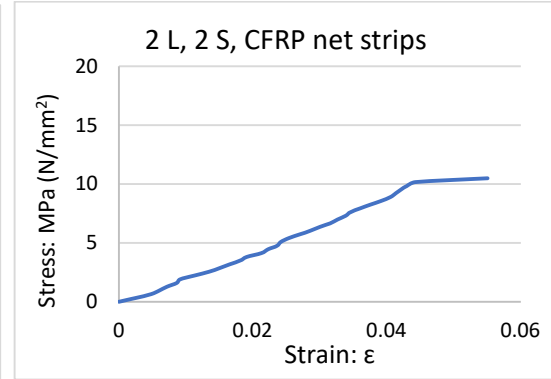


Figure 4.38: Stress and strain for 2 L, 2 S, CFRP net strips

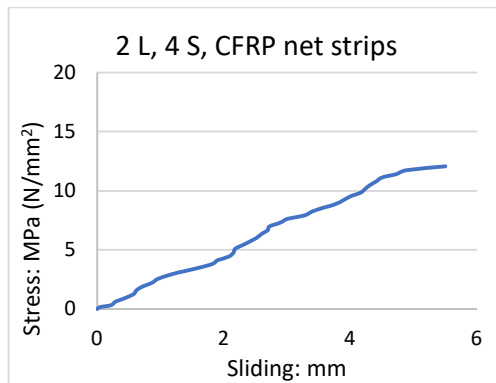


Figure 4.39: Stress and sliding for 2 L, 4 S, CFRP net strips

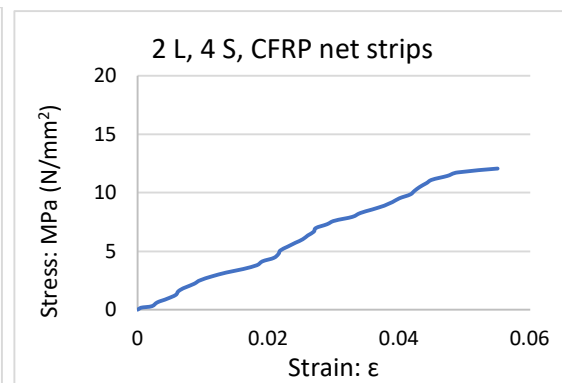


Figure 4.40: Stress and strain for 2 L, 4 S, CFRP net strips

4.15. Comparison of one layer and two layers of CFRP on triplet masonry with different configurations

To investigate the effectiveness of extra layers of FRP composite on masonry triplets, the experimental tests were set up with retrofitting triplets with two layers of CFRP with eight different configurations. Comparing one layer and two layers of CFRP or CFRP strips on triplet masonry showed It is no significant bond improvement between one layer and two layers. It proved that layers stick together and act as one layer. During the increase of the shear load on reinforced triplets with CFRP sheets or CFRP strips, first cracks were observed in the mortar layer and propagated along the mortar followed by CFRP debonding. Debonding is the most common failure mechanism in FRP-

strengthened masonry structures, this failure mostly occurs when the bond between the reinforcement composite and masonry interface is lost, also in contrast with URM walls, reinforced triplets were stable after failure. It is observed that the shear triplet presented a nearly linear shear stress and sliding relationship before cracking of mortar, followed by a drop of shear strength once cracking had occurred in the mortar layer, followed by a slight increase in shear strength and deformation with the application of further loading before CFRP rupture.

Depending on CFRP configurations, two layers make triplets 5% to 15% stronger, which is not sufficient to apply two or three layers. Triplets that were covered on two sides by CFRP were 20% to 40% stronger than strips and CFRP strips are 5% to 10% stronger than CFRP net strips. Comparison of reinforced triplets with one layer and two layers is shown in figures 4.41 to 4.56.

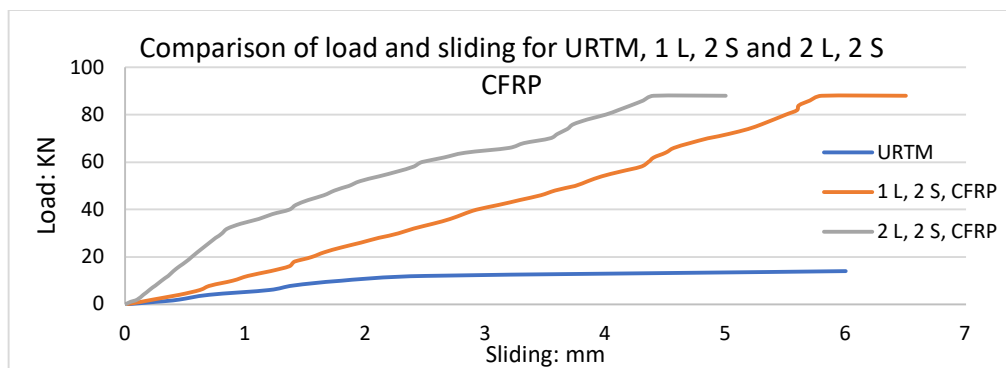


Figure 4.41: Comparison of load and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP

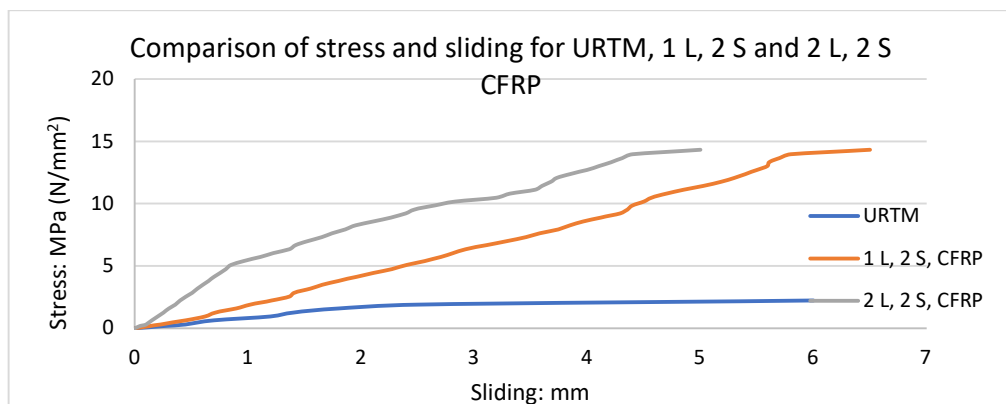


Figure 4.42: Comparison of stress and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP

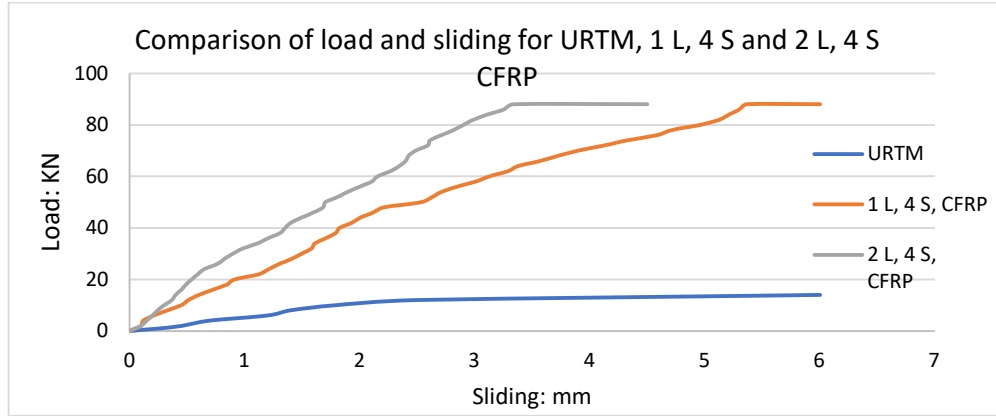


Figure 4.43: Comparison of load and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP

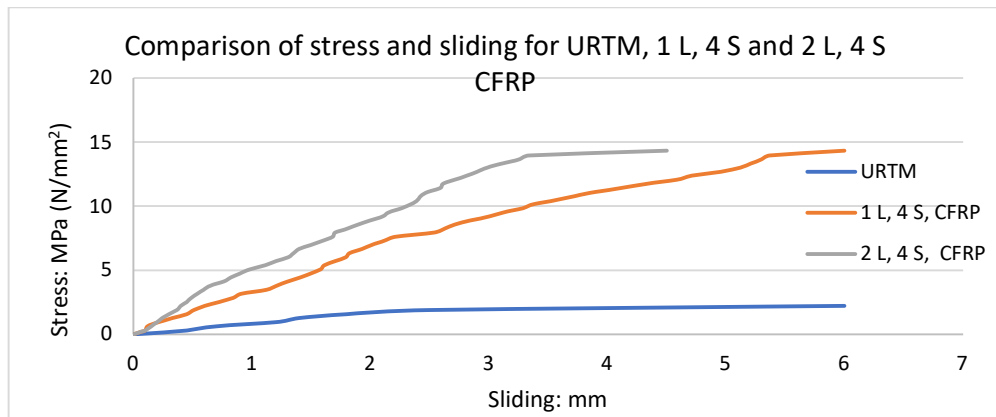


Figure 4.44: Comparison of stress and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP

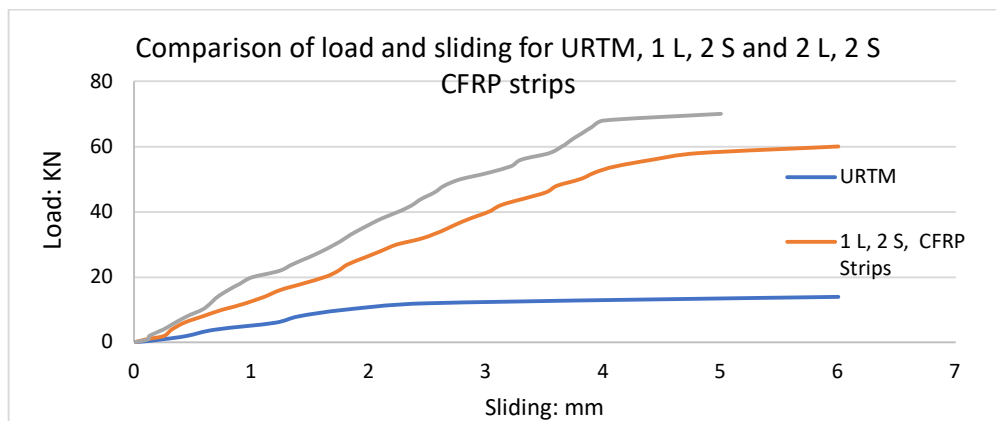


Figure 4.45: Comparison of load and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP strips

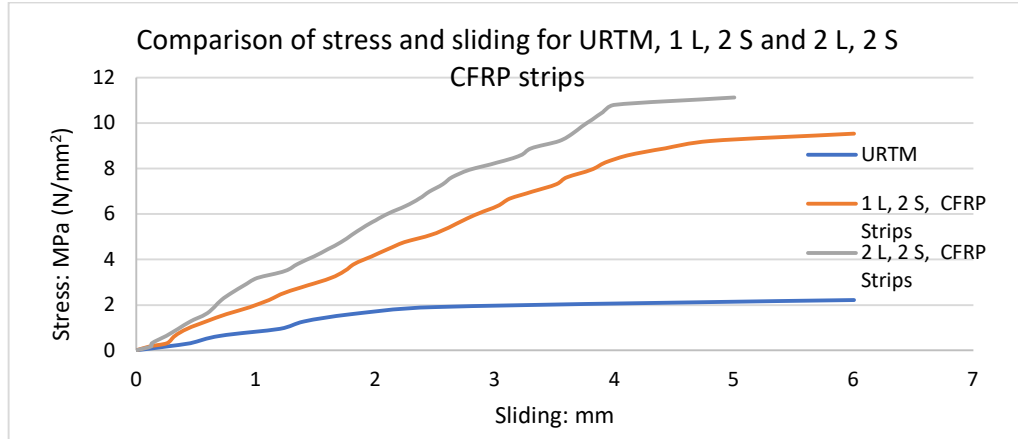


Figure 4.46: Comparison of stress and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP strips

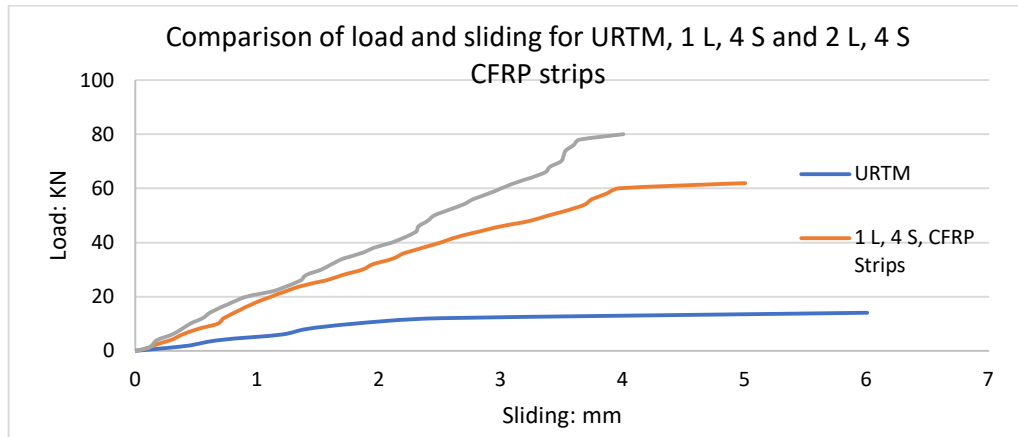


Figure 4.47: Comparison of load and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP strips

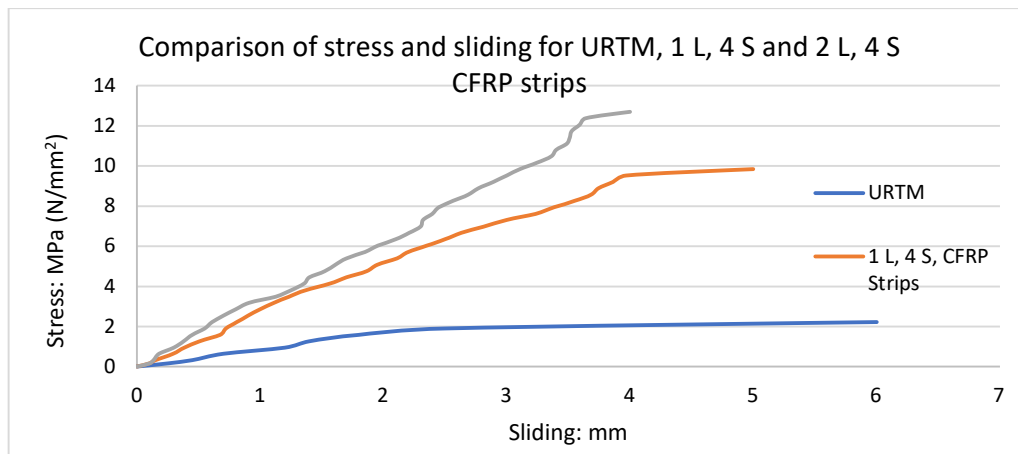


Figure 4.48: Comparison of stress and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP strips

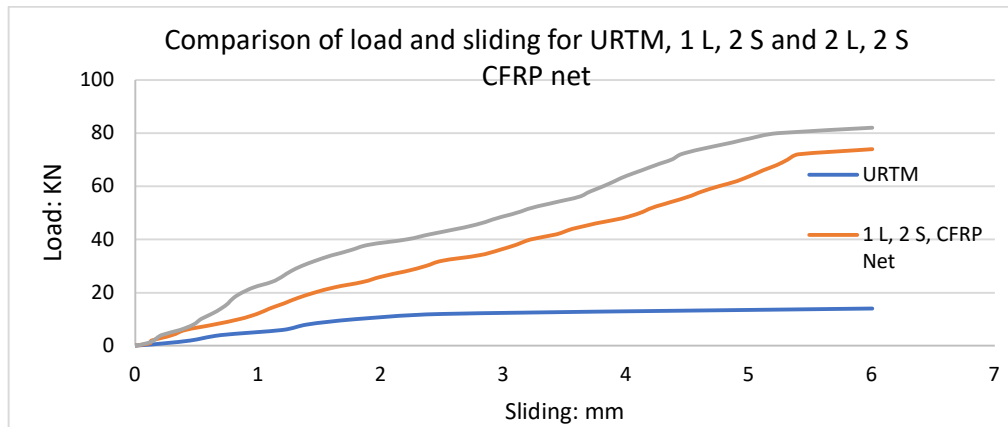


Figure 4.49: Comparison of load and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP net

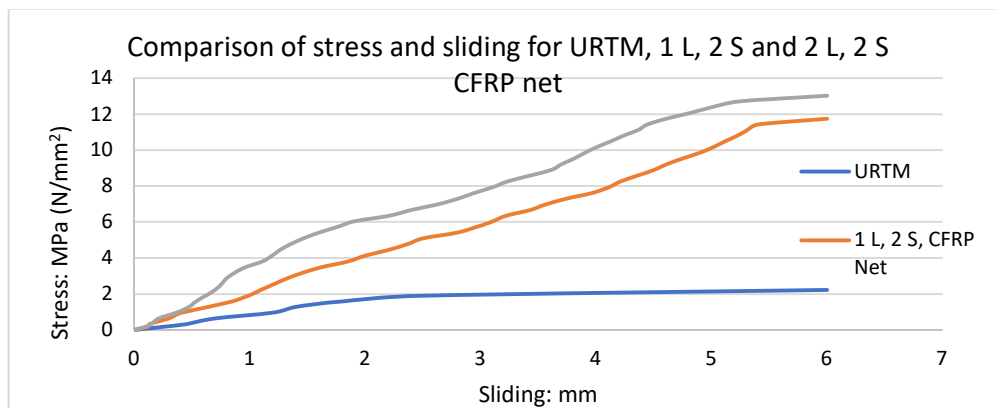


Figure 4.50: Comparison of stress and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP net

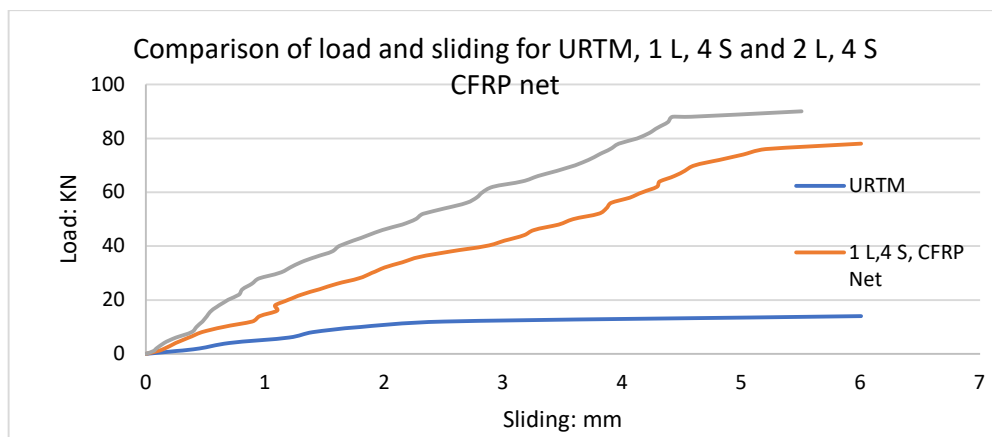


Figure 4.51: Comparison of load and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP net

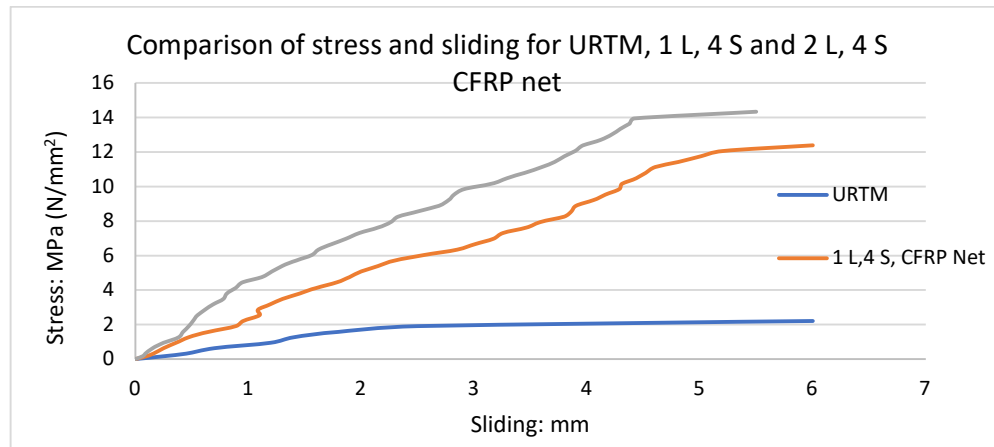


Figure 4.52: Comparison of stress and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP net

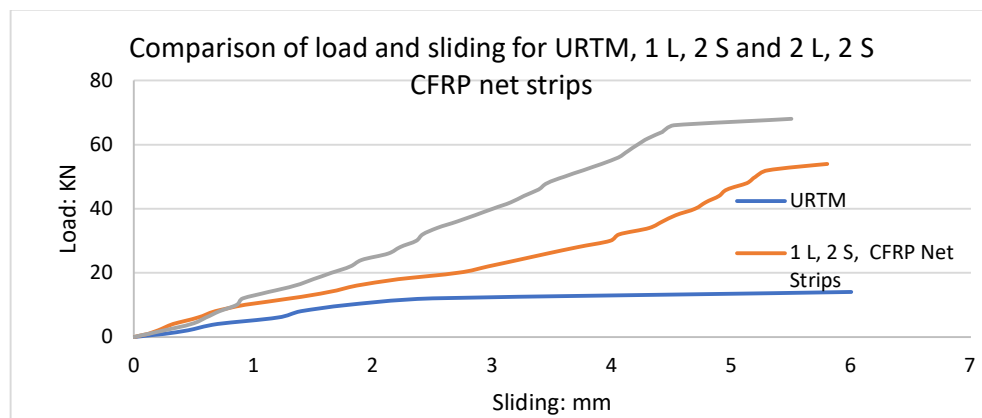


Figure 4.53: Comparison of load and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP net strips

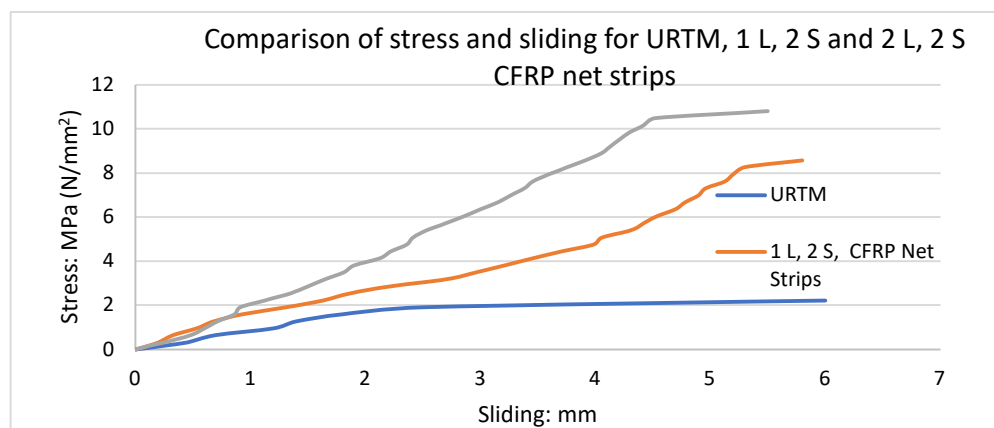


Figure 4.54: Comparison of stress and sliding for URTM, 1 L, 2 S and 2 L, 2 S CFRP net strips

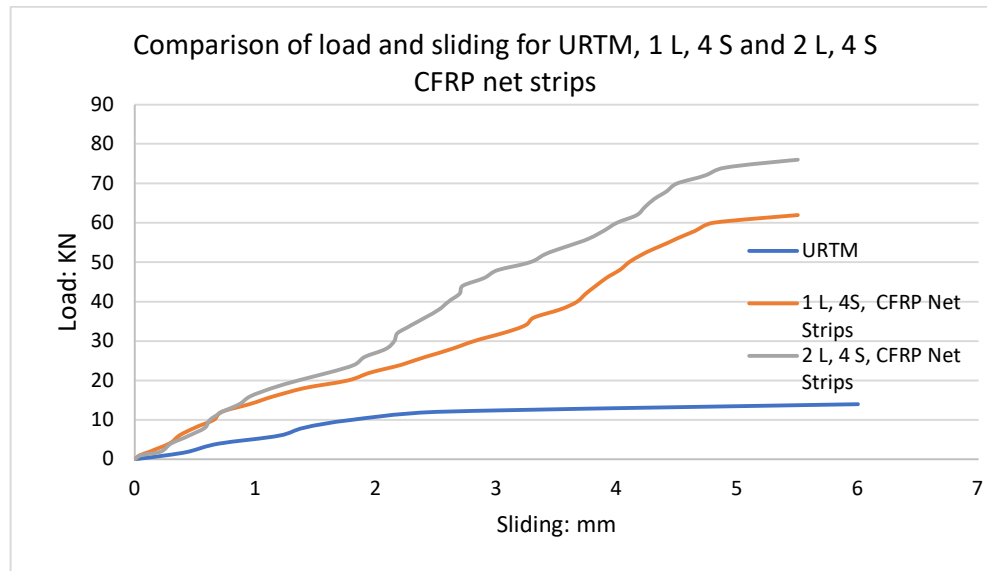


Figure 4.55: Comparison of load and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP net strips

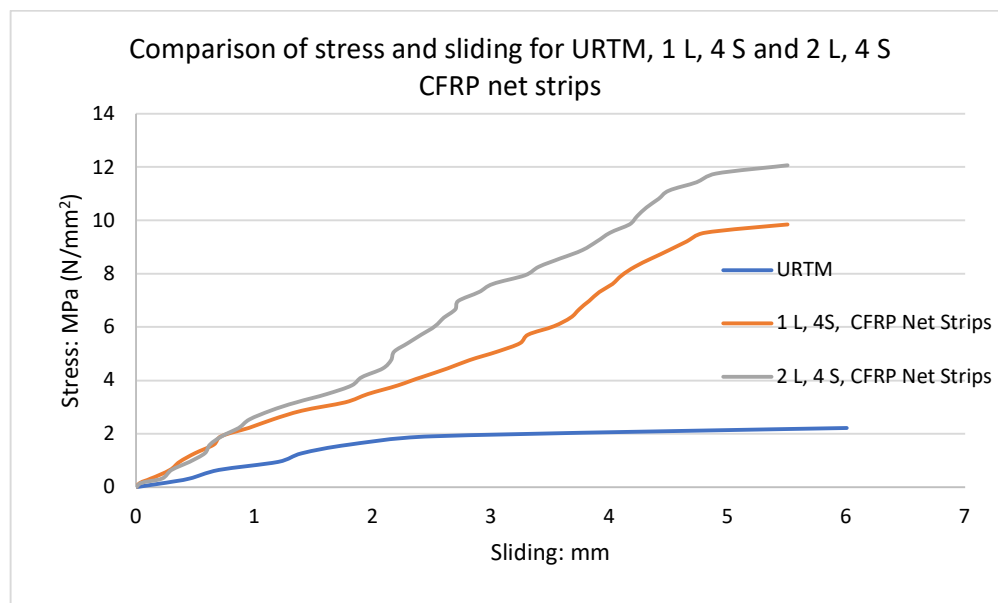


Figure 4.56: Comparison of stress and sliding for URTM, 1 L, 4 S and 2 L, 4 S CFRP net strips

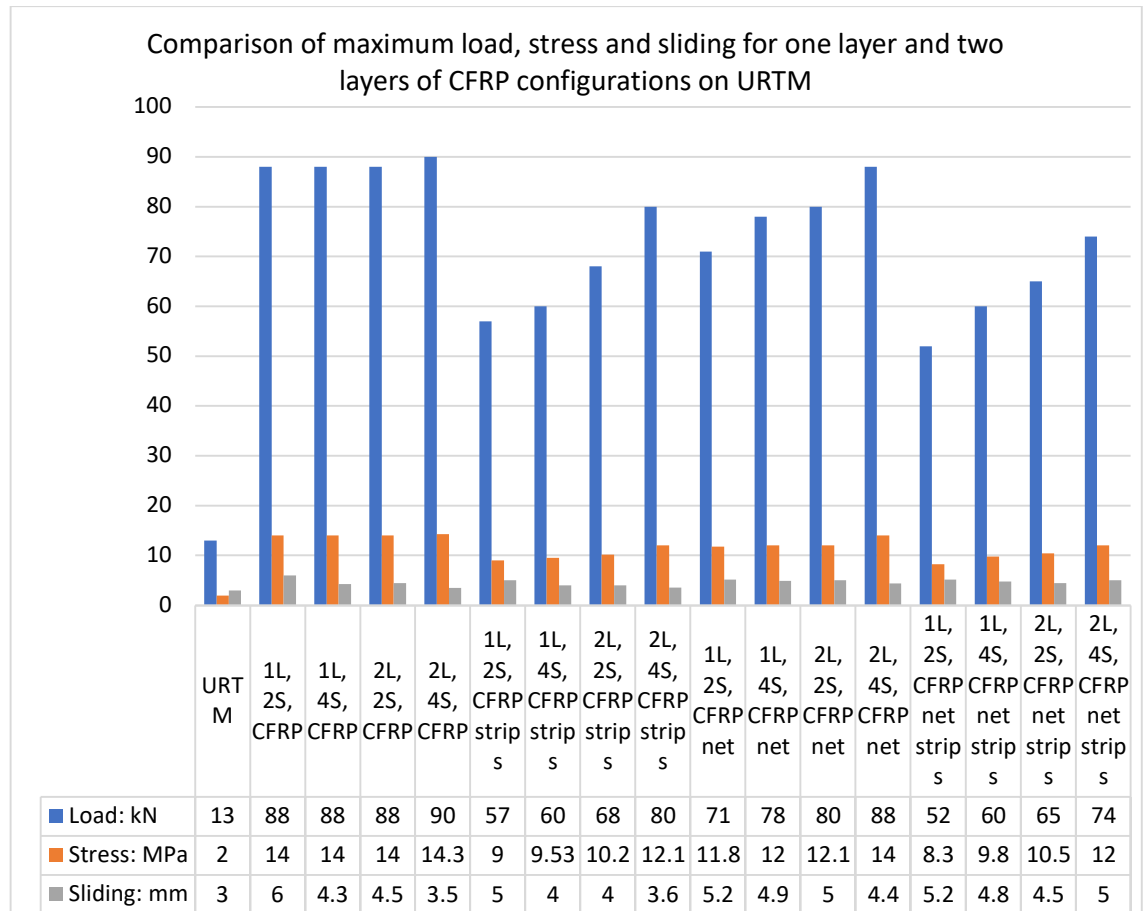


Figure 4.57: Comparison of maximum load, stress and sliding for one layer and two layers of CFRP configurations on URTM

Table 4.2 shows a comparison of ultimate load, type of collapse, stress and displacement for different CFRP configurations on triplet masonry. (L represent layer and S stand for side)

CFRP configurations on triplet masonry	MAX load: KN	Displacement: mm	Stress: MPa / (N/mm ²)	Type of failure
URTM (un-reinforced triplet masonry)	12	3	2	Brittle
1L, 2S, CFRP	88	6	14	Ductile
1L, 4S, CFRP	88	4.30	14	Ductile
2L, 2S, CFRP	88	4.50	14	Ductile
2L, 4S, CFRP	90	3.50	14.33	Ductile
1L, 2S, CFRP strips	55	5	9	Ductile
1L, 4S, CFRP strips	60	4	9.53	Ductile
2L, 2S, CFRP strips	68	4	10.20	Ductile
2L, 4S, CFRP strips	80	3.60	12.07	Ductile
1L, 2S, CFRP net	71	5.20	11.75	Ductile
1L, 4S, CFRP net	78	4.90	12	Ductile
2L, 2S, CFRP net	80	5	12.10	Ductile
2L, 4S, CFRP net	88	4.40	14	Ductile
1L, 2S, CFRP net strips	54	5.20	8.30	Ductile
1L, 4S, CFRP net strips	60	4.80	9.80	Ductile
2L, 2S, CFRP net strips	64	4.50	10.48	Ductile
2L, 4S, CFRP net strips	76	5	12	Ductile

Table 4.2: Comparison of maximum load and displacement and failure type.

4.16. New equations for maximum shear strength of triplet masonry with CFRP

The modulus of elasticity of FRP, the bond between FRP, masonry properties, and FRP configurations, make a prediction of maximum shear strength difficult. Pavan and Nanjunda (2015), investigated two types of masonry assemblages to study the interfacial behavior of brick–mortar joints of masonry; (1) triplet masonry assemblage for sliding shear strength; and (2) off-axis masonry prisms to understand the interaction between shear and normal stress acting on the bed joint of masonry. They developed an equation for triplet specimens fully glued with glass-fiber reinforced polymer (GFRP).

The maximum shear strength of CFRP-strengthened masonry for all CFRP configurations strips or full faces reinforced at one, two or four sides, can be calculated as the sum of the resistance provided by brick–mortar joint and CFRP as:

$$S_m = S_{m0} + S_{frp} \quad (4.5)$$

where:

S_m = Shear strength of CFRP reinforced masonry

S_{m0} = shear resistance URTM

S_{frp} = resistance obtained by FRP composites

S_{m0} is available in design guidelines such as IS: 1905 (BIS 2002a). In this study, S_{m0} was obtained from the shear tests carried out on control triplet specimens (Equation 4.4).

$$S_{m0} = \tau_{max}$$

The expressions that calculate S_{frp} need to be developed. The sliding of the middle brick during the loading will produce tensile stress in FRP composites and the compressive stress on the bed joint keeps increasing with increases in sliding of the middle brick until the specimen reaches failure either due to de-bonding or due to rupture of the FRP composite. Thus, the maximum shear strength of CFRP could be expressed as Equation (4.6).

$$S_{frp} = \sigma_{frp} \times A_f / A_t \times K \times \mu_f \quad (4.6)$$

where:

σ_{frp} , is the tensile strength of the CFRP which, is given by the factory.

The average tensile strength of the CFRP and CFRP net is 3500 Mpa.

S_{frp} : is the maximum shear strength of masonry reinforced with CFRP

A_t is the area of triplet masonry

A_f is the area of triplet masonry covered by CFRP

$k=1$, for one layer of CFRP on two sides

$k=0.7$, for one layer of CFRP on four sides

μ_f is the coefficient for CFRP

$\mu_f = 0.007$ for CFRP

Table 4.3 shows the comparison of the maximum shear strength bond of different configurations of experimental Triplet masonry strengthened with CFRP and the results from equations 4.5 and 4.6:

Test	CFRP configurations on triplet masonry	A_f/A_t	$S_{frp} = \sigma_{frp} \times A_f/A_t \times K \times \mu_f$ $S_m = S_{m0} + S_{frp}$ (MPa)	Experimental shear tests (MPa)	Error %
1	One-layer CFRP on two sides	0.51	14.10	14.33	2
2	One-layer CFRP on four sides	0.75	14.45	14.33	1
3	One-layer CFRP strips on two sides	0.23	7.3	9.53	25
4	One-layer CFRP strips on four sides	0.35	10.1	9.53	6
5	One-layer CFRP net on two sides	0.51	14	11.75	16
6	One-layer CFRP net on four sides	0.75	14.45	12.06	17
7	One-layer CFRP net strips on two sides	0.23	7.3	8.57	17
8	One-layer CFRP net strips on four sides	0.35	7.6	9.57	20
	$S_{m0} = \tau_{max} = 1.59$				

Table.4.3: Triplet masonry strengthened with CFRP and the results from Equations 5 and 6.

4.17. New equations for bond stress of URTM and strengthened triplet masonry with CFRP

In this study and based on the database of experimental results, if the sliding/slipping distance is known, the bond stress for all CFRP configurations of triplet masonry can be calculated with newly developed equations in table 4.4. On the other hand, bond stress can be calculated at any sliding point. In addition, the failure point is predictable. To find the reliable equation, a quadratic formula with a trial-and-error coefficient was used to develop equations and validated against experimental tests. The bond stresses predicted by equations are compared with the experimental tests and shown in figures 4.58 to 4.66. The bond stress and sliding curves are in good agreement with the experimental results and the accuracy of the expression developed for predicting the bond stress strength of masonry bonded with FRP composites was found to be satisfactory. In all figures, L stands for the CFRP layer, and S represents the side of the triplet.

By using the new equations, it is possible to predict correct stress at any sliding distance for solid London brick and mortar M4 reinforced with eight different CFRP configurations and unreinforced triplet masonry. The equations in this study are a step forward in theoretical study and will directly benefit engineers to predict the correct wall resistance in case of shear loading.

	CFRP configurations on triplet masonry (one layer)	Equation number	Bond stress $\sigma_b = f(S)$
1	URTM	4.7	$[1/(0.15*S)] * \sigma_b^2 - (10-S) * \sigma_b + (S) = 0$
2	CFRP on two sides	4.8	$[1/(0.28*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
3	CFRP on four sides	4.9	$[1/(0.28*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
4	CFRP strips on two sides	4.10	$[1/(0.20*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
5	CFRP strips on four sides	4.11	$[1/(0.22*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
6	CFRP net on two sides	4.12	$[1/(0.24*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
7	CFRP net on four sides	4.13	$[1/(0.25*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
8	CFRP net strips on two sides	4.14	$[1/(0.15*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
9	CFRP net strips on four sides	4.15	$[1/(0.21*S)] * \sigma_b^2 - (15-S) * \sigma_b + (S) = 0$
S represents sliding (mm) and σ_b is bond stress (MPa) for any sliding			

Table 4.4: The newly developed formula for predicting bond stress

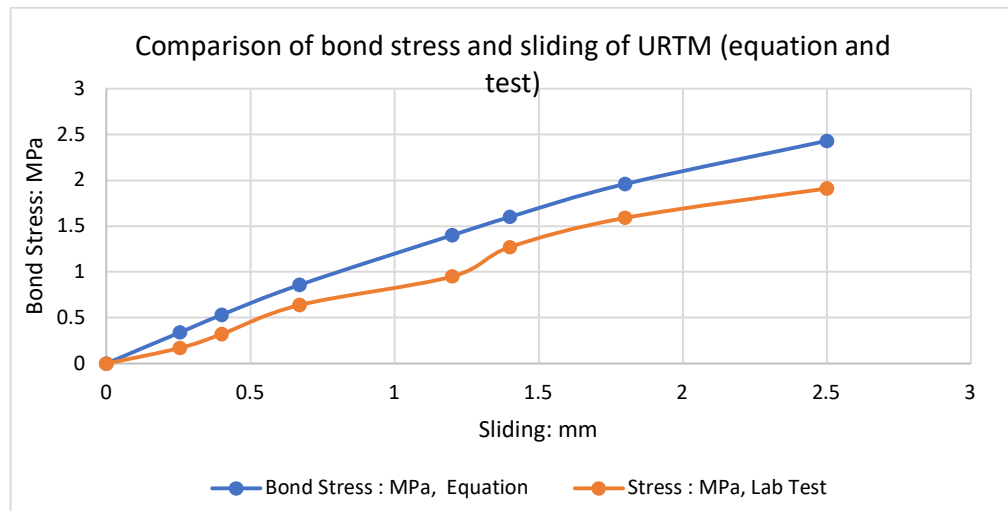


Figure 4.58: Comparison of bond stress and sliding from experimental test and equation

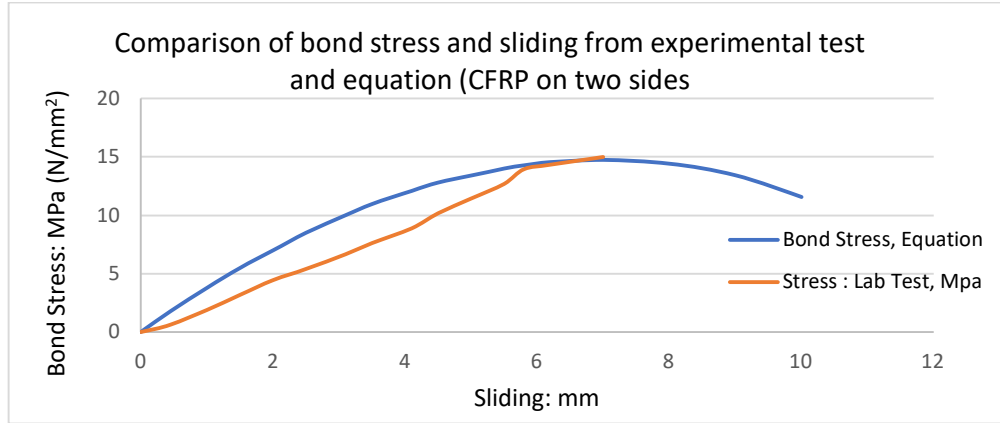


Figure 4.59: Comparison of bond stress and sliding from experimental test and equation
(CFRP on two sides)

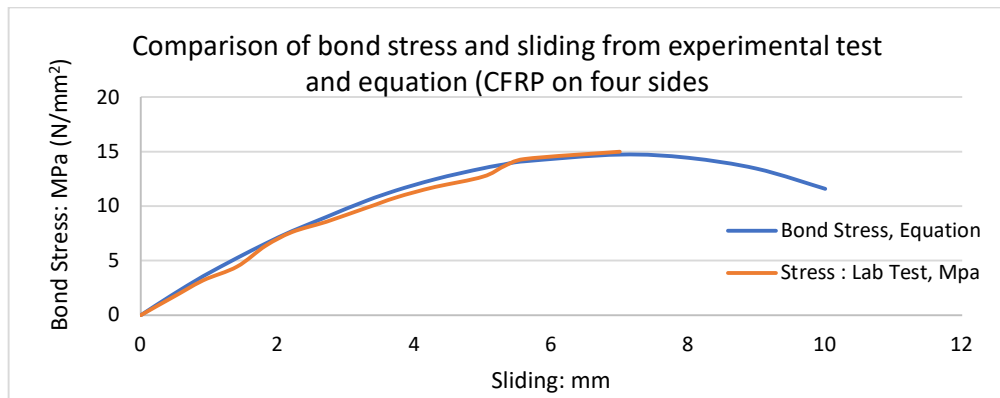


Figure 4.60: Comparison of bond stress and sliding from experimental test and equation
(CFRP on four sides)

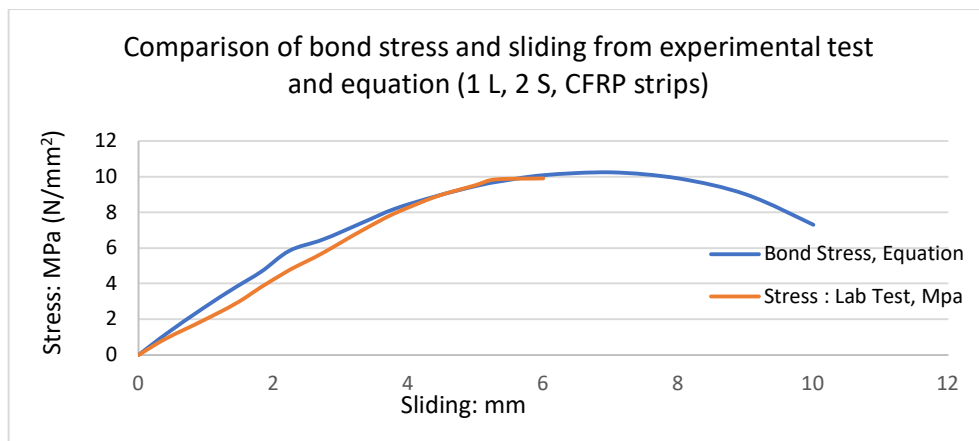


Figure 4.61: Comparison of bond stress and sliding from experimental test and equation
(1 L, 2 S, CFRP strips)

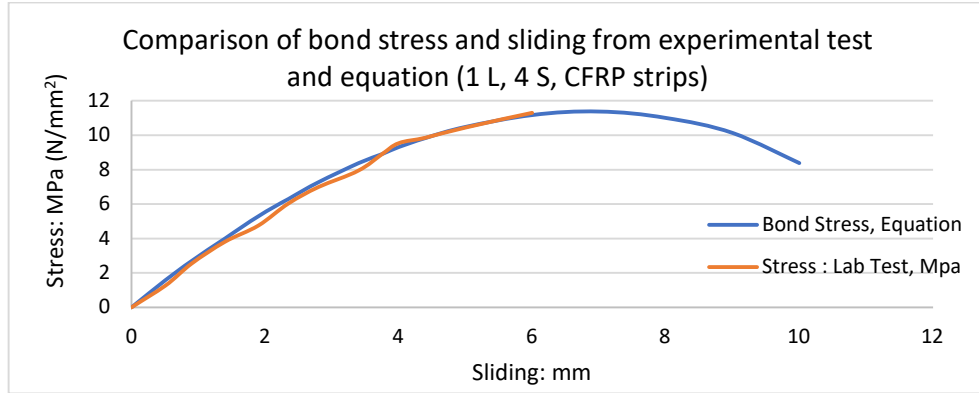


Figure 4.62: Comparison of bond stress and sliding from experimental test and equation (1 L, 4 S, CFRP strips)

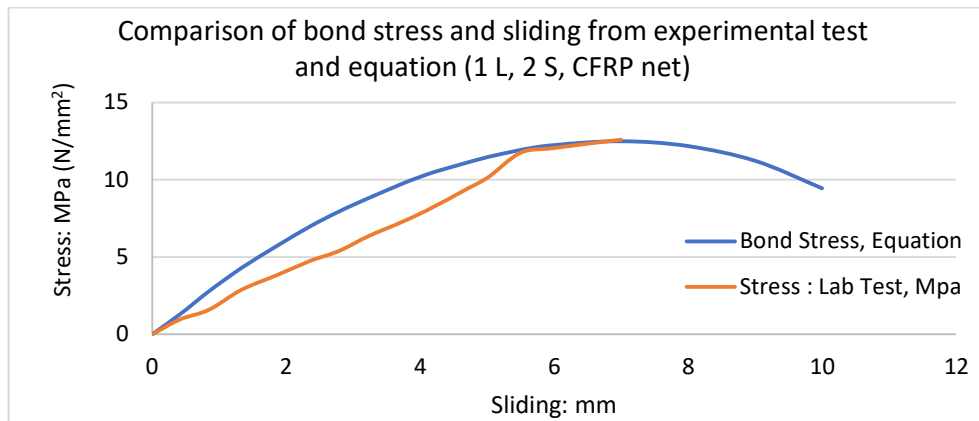


Figure 4.63: Comparison of bond stress and sliding from experimental test and equation (1 L, 2 S, CFRP net)

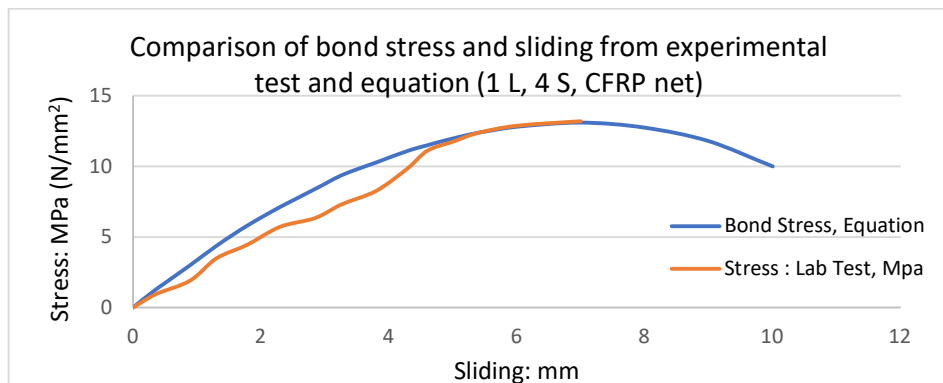


Figure 4.64: Comparison of bond stress and sliding from experimental test and equation (1 L, 4 S, CFRP net)

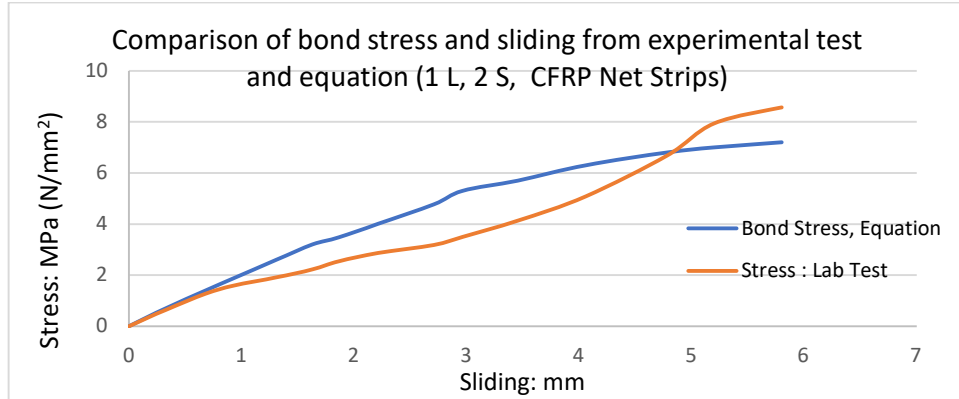


Figure 4.65: Comparison of bond stress and sliding from experimental test and equation
(1 L, 2 S, CFRP net strips)

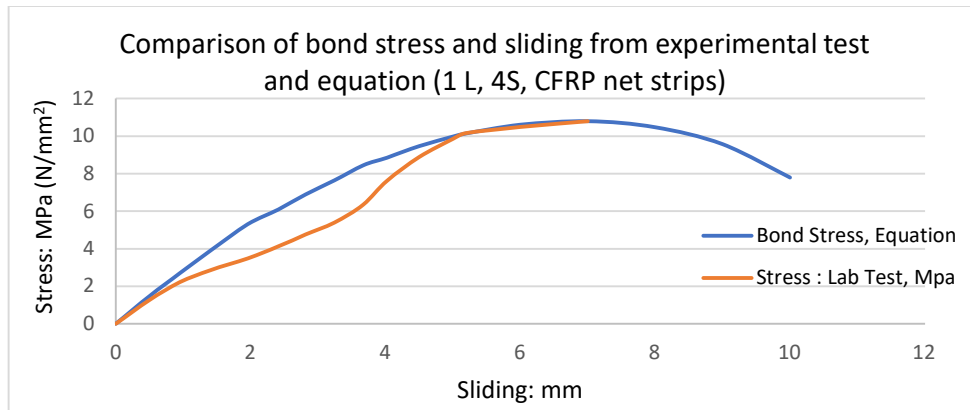


Figure 4.66: Comparison of bond stress and sliding from experimental test and equation
(1 L, 4S, CFRP net strips)

4.18. Sliding calculation of reinforced masonry triplet with different CFRP configurations at any stress/load point

In this section and based on the database of experimental results, if the bond stress is known, the sliding for all CFRP configurations of triplet masonry can be calculated with developed equations in table 4.5. The sliding was predicted and compared with the experimental values as shown in figures 4.67 to 4.76. In all figures, L stands for the CFRP layer, and S represents the side of the triplet. The bond stress sliding curves are in good agreement with the test data from the experimental results and the accuracy of the expression developed for predicting of sliding of masonry bonded with CFRP

composites was found to be satisfactory. To find the reliable equation, a quadratic formula with a trial-and-error coefficient was used to develop equations and validated against experimental data.

By using the new equations, it is possible to predict correct sliding at any load or stress point for solid London brick and mortar M4 reinforced with eight different CFRP configurations and unreinforced triplet masonry. The equations in this study are a step forward in theoretical study and will directly benefit engineers to predict correct wall displacement in case of shear loading.

	CFRP configurations on triplet masonry (one layer)	Equation number	Equation $S = f(\sigma_b)$
1	URTM	4.16	$[1/(0.50*\sigma_b)] * S^2 - (5 - \sigma_b) * S + (\sigma_b) = 0$
2	CFRP on two sides	4.17	$[1/(0.030*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
3	CFRP on four sides	4.18	$[1/(0.020*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
4	CFRP strips on two sides	4.19	$[1/(0.025*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
5	CFRP strips on four sides	4.20	$[1/(0.025*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
6	CFRP net on two sides	4.21	$[1/(0.030*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
7	CFRP net on four sides	4.22	$[1/(0.025*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
8	CFRP net strips on two sides	4.23	$[1/(0.032*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
9	CFRP net strips on four sides	4.24	$[1/(0.028*\sigma_b)] * S^2 - (30 - \sigma_b) * S + (\sigma_b) = 0$
S represents sliding (mm) and σ_b is bond stress (MPa)			

Table 4.5: The newly developed formula to calculate sliding at any stress point

Where: σ_s is the shear stress (MPa) and S is slipping/sliding (mm)

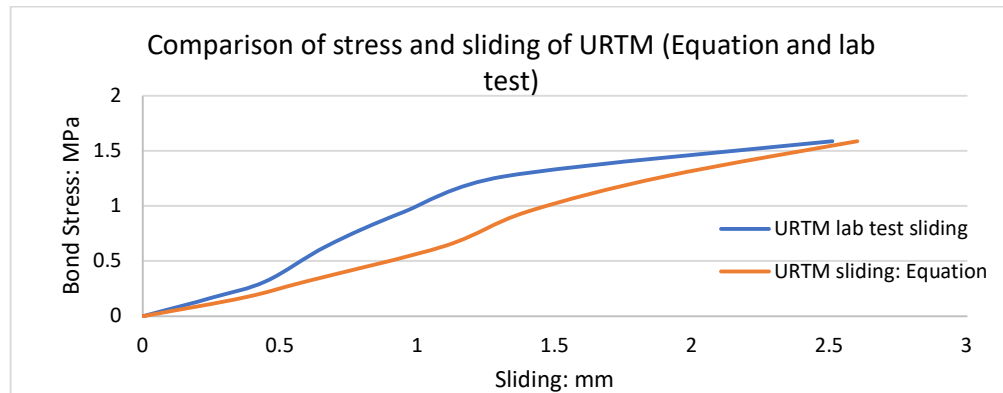


Figure 4.67: Comparison of stress and sliding of URTM (equation and test)

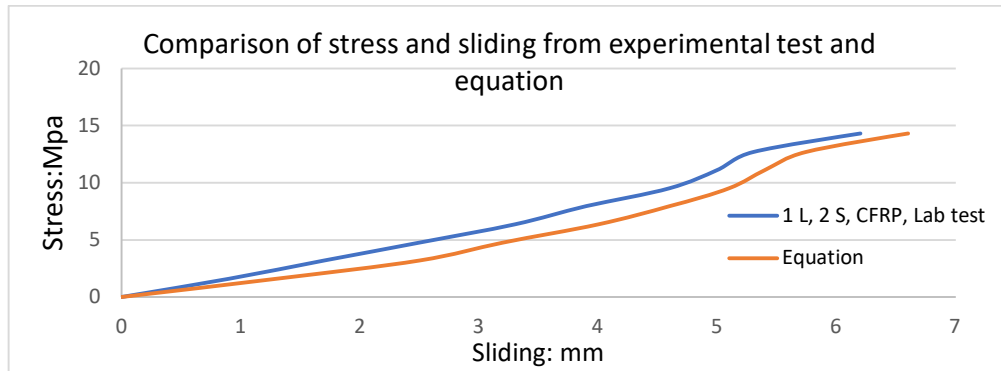


Figure 4.68: Comparison of stress and sliding from experimental test and equation

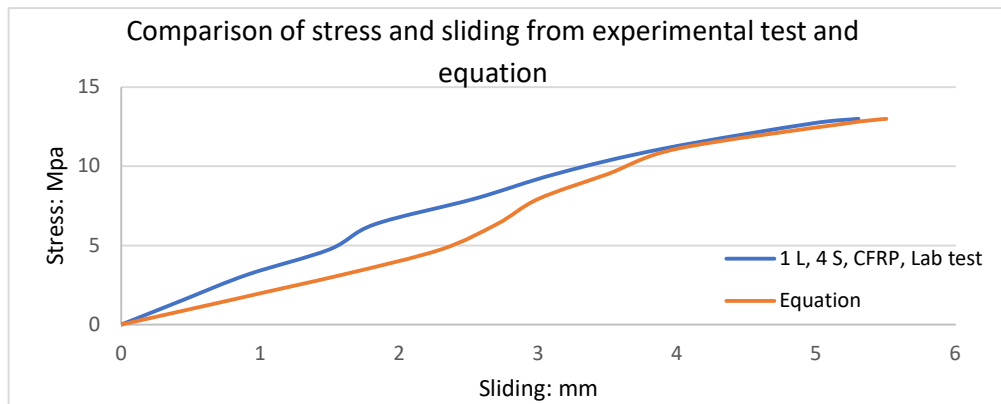


Figure 4.69: Comparison of stress and sliding from experimental test and equation

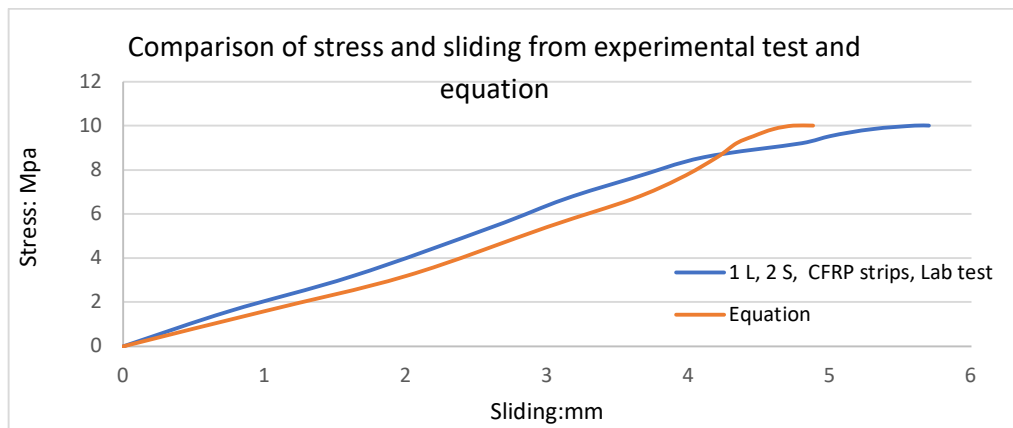


Figure 4.70: Comparison of stress and sliding from experimental test and equation

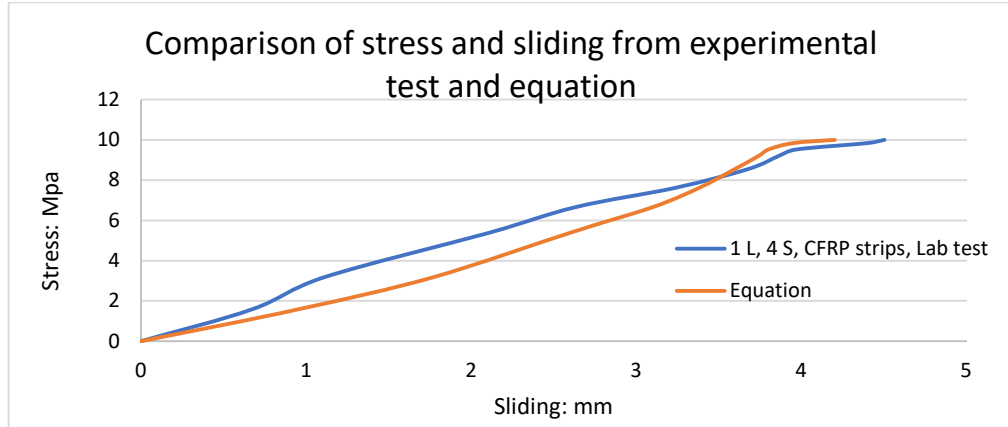


Figure 4.71: Comparison of stress and sliding from experimental test and equation

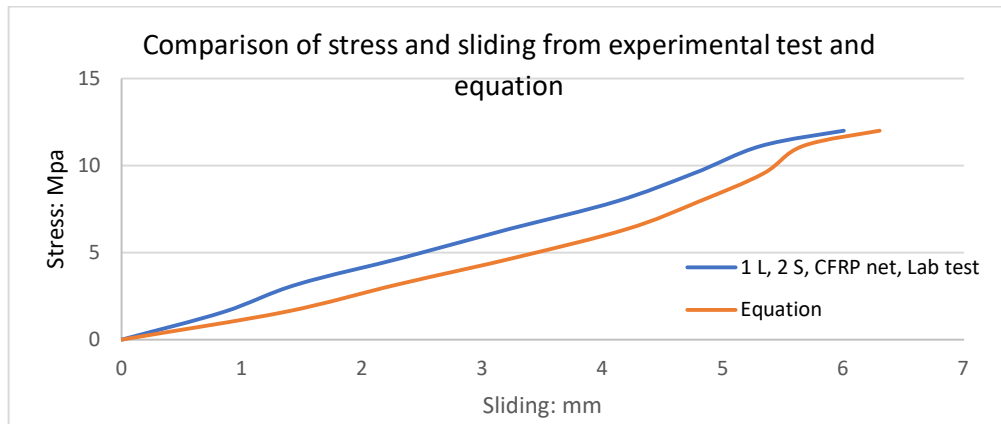


Figure 4.72: Comparison of stress and sliding from experimental test and equation

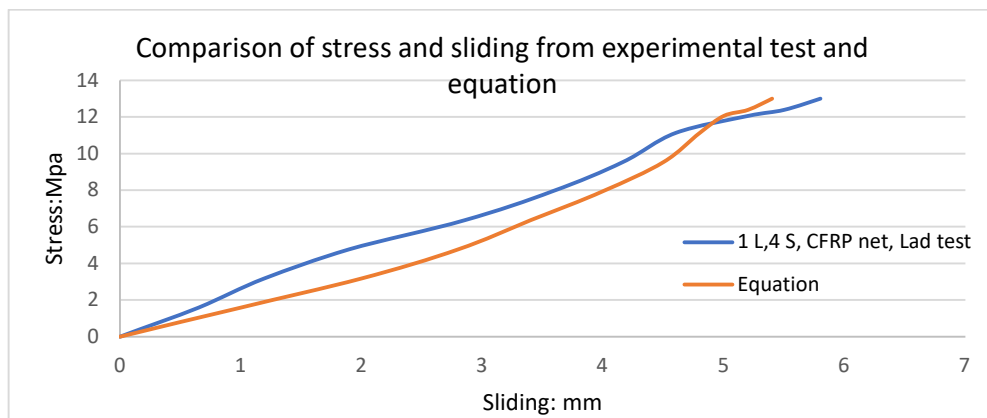


Figure 4.73: Comparison of stress and sliding from experimental test and Equation

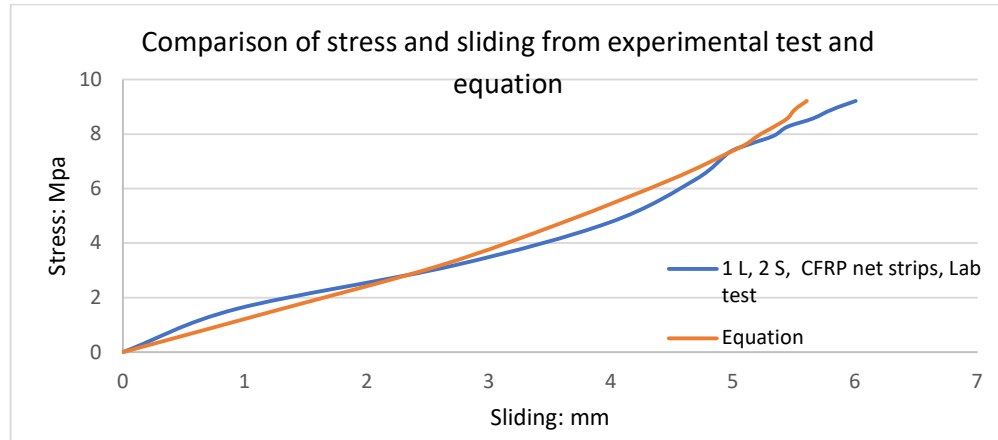


Figure 4.74: Comparison of stress and sliding from experimental test and equation

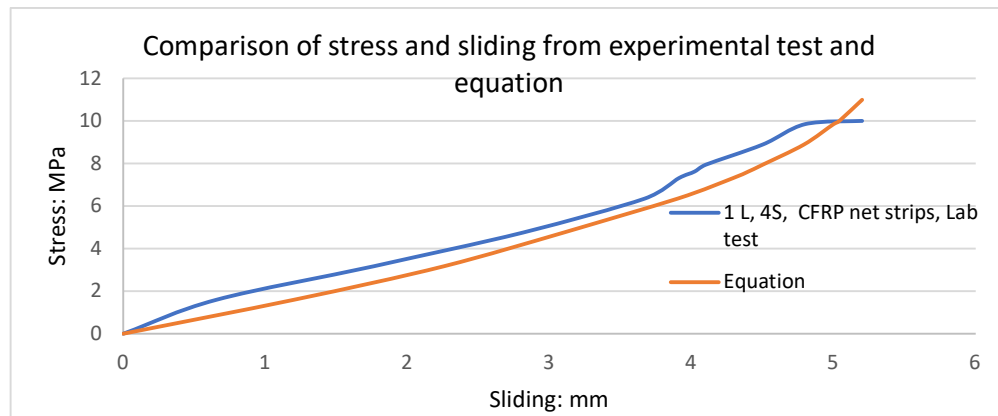


Figure 4.75: Comparison of stress and sliding from experimental test and equation

Table 4.6 illustrates the results of sliding from equations and lab tests at failure points and errors for all CFRP configurations. This table shows acceptable errors and a good agreement between newly developed equations and lab tests.

	CFRP configurations on triplet masonry (one layer)	Bond stress at failure point (MPa)	Lab test sliding (mm)	Equation sliding (mm)	Error (%)
1	URTM	1.59	2.5	2.6	3.8
2	CFRP on two sides	14.33	6.2	6.6	6
3	CFRP on four sides	13	5.3	5.3	0
4	CFRP strips on two sides	10	5.5	4.88	11
5	CFRP strips on four sides	10	4.5	4.2	6.6
6	CFRP net on two sides	11.12	6	6.3	4.8
7	CFRP net on four sides	13	5.8	5.5	5.1
8	CFRP net strips on two sides	9.21	6	5.6	6.66
9	CFRP net strips on four sides	10	5.2	5.04	3

Table 4.6: Comparison of the newly developed formula and lab tests at failure point and error

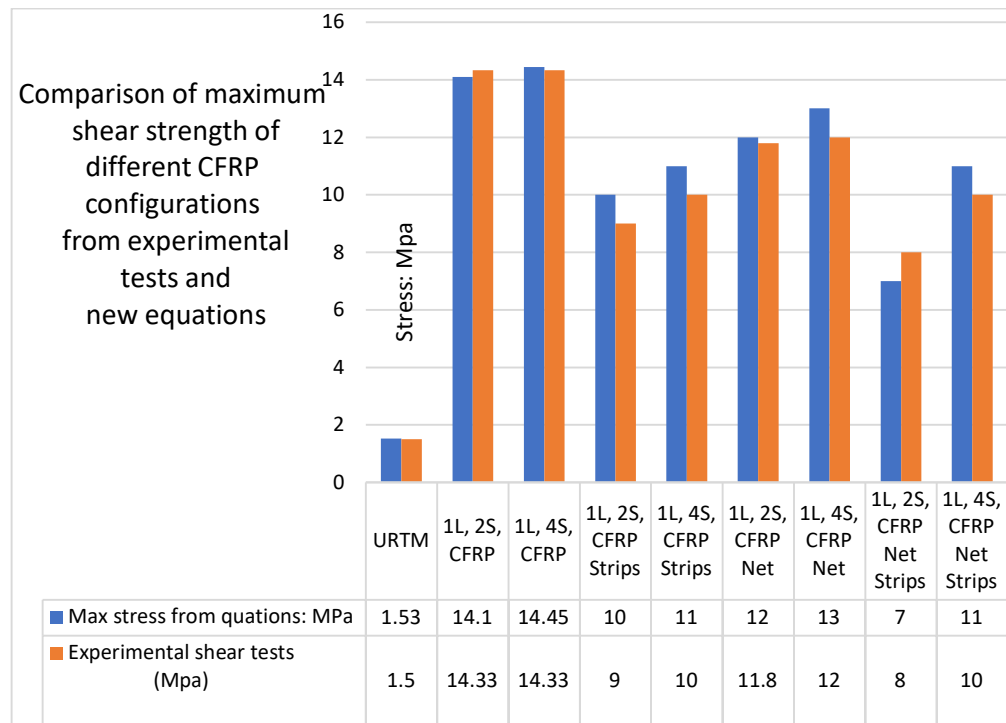


Figure: 4.76: Comparison of maximum shear strength of different CFRP configurations on triplet masonry from experimental tests and new equations

4.19. Conclusions for this chapter:

The following conclusions can be drawn from the walls strengthened by FRP:

1. All types of reinforced triplets were four to seven times stronger than triplets without CFRP and shear failure of brickwork started with cracking and sliding through the mortar joints. The failure occurred along with the interface between the brick-and-mortar joint.
2. Highest ductility in terms of maximum deflection at ultimate load is observed for all samples. Also, according to this experiment, it is no significant difference between one layer and two layers of CFRP on triplet masonry.
3. The increase of the stiffness of URM triplets with CFRP sheets and strips are significant for the first layer of reinforcement and not so effective for the additional layers on the top of the first layer.
4. The comparison of the load-displacement graphs from the theoretical modeling with the results from the experimental tests were not accurate for London solid brick and Mortar M4, thus the new equations for finite element modeling were developed.
5. The interfacial shear stress concentrations between the brick-and-mortar surfaces are reduced due to retrofitting of shear triplets with CFRP composites.
6. Shear strength of triplets reinforced with the same amount in terms of area reinforced with CFRP and CFRP net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity
7. Debonding failure pattern was observed in triplet specimens glued with CFRP composites in full and strip configuration and in contrast with URM triplets, strengthened samples were stable after failure.

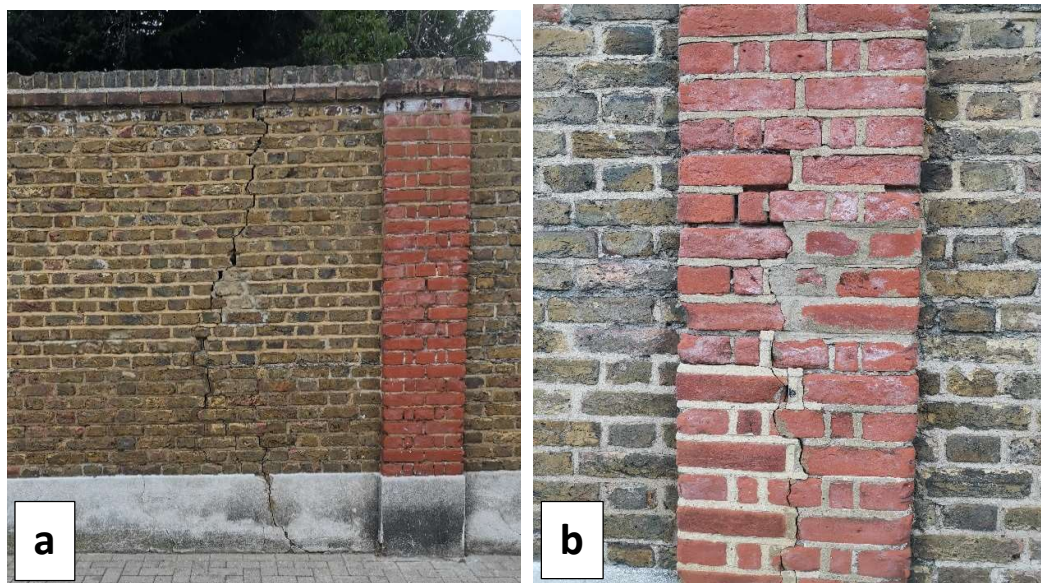
Chapter 5

**Experimental shear tests on cracked
triplet masonry strengthened with
different CFRP configurations**

5. 1. Introduction

Widespread damage in a huge number of current masonry buildings during seismic activity has shown the need for structural repair techniques for masonry structures. On-the-surface bonded fibers (FRP) is a reinforcement method that has been shown that can be used to repair or reinforce concrete and masonry buildings. Masonry walls reinforced with FRP can have a big increase in strength (Hernan and Pablo, 2011).

Currently, masonry structures are still used in many countries and brickwork is commonly the structure in load-bearing masonry buildings. In European countries, masonry is mainly used for infill walls in buildings with reinforced concrete frames. Transference of loading can occur due to building structure/wall interactions. These interactions, as well as other effects like dimensional changes of the walls, due to moisture movements or foundation settlement, can lead to many different failures in masonry walls in serviceability states, such as cracking (Figure5.1).



Figures 5.1a and 5.1b: Example of masonry wall with cracking

Also, the main in-plane failure type of unreinforced masonry walls under seismic loads can be sliding failure. This type of failure is due to vertical loads and poor mortar quality. Cracks can generate sliding and extend to the length of the wall leading to the displacement of the upper part of the wall from the bottom part of it (Bibiana ana

Viviana, 2010). Mahmood and Ingham (2011) conducted a series of experimental tests on URM walls, they illustrated the effectiveness of the FRP retrofitting system in improving the shear strength of unreinforced masonry with a factor of 3.25.

Valluzzi et al (2020) tested a series of unreinforced brick masonry panels strengthened by different materials (GFRP and CFRP) of different configurations. All tested specimens were subjected to a diagonal compression test, the results showed that the application of FRP at one side of the wall only produces a significant out-of-plane deformation, corners, and the failure mode of the out-of-plane displacement exaggerated by the little restraint at the top and bottom of the wall. Hernoune et al (2020) conducted experimental and numerical studies of brick masonry walls reinforced with (CFRP) composite subjected to a shear load. His study showed the increase in the proportion of sand in the mortar (From 3 to 5) led to an increase in the shear strength and the ductility of the masonry panels, especially in the case of reinforcement.

Mahini., (2014), conducted the retrofitting CFRP can be used to control the opening hinges in the extrados of the masonry vaults, which are subjected to lateral loads, and can also be used to upgrade the ultimate, load carrying capacity and the ductility of the structures.

Ghobadi et al (2018) carried out an experimental study on repaired cracked masonry walls with crimped wire mesh, Figure 5.2. According to their study, the initial stiffness of 2470%, ultimate strength, and dissipated energy would be considerably increased by using infill. This upgraded performance was possible because the infill is confined by the surrounding steel frame, which in turn is strengthened by the infill because of the imposed shear-resistant behavior. The experimental study showed that the proposed repair method could restore the structural performance of damaged infill in a stable manner under cyclic reversal loading. Indeed, the stiffness, initial strength, and ultimate strength of the damaged infill were completely restored compared with the intact infill. Both the infill and repaired infill specimens tolerated 240 kN, and 255 kN ultimate lateral load in the positive direction of loading.

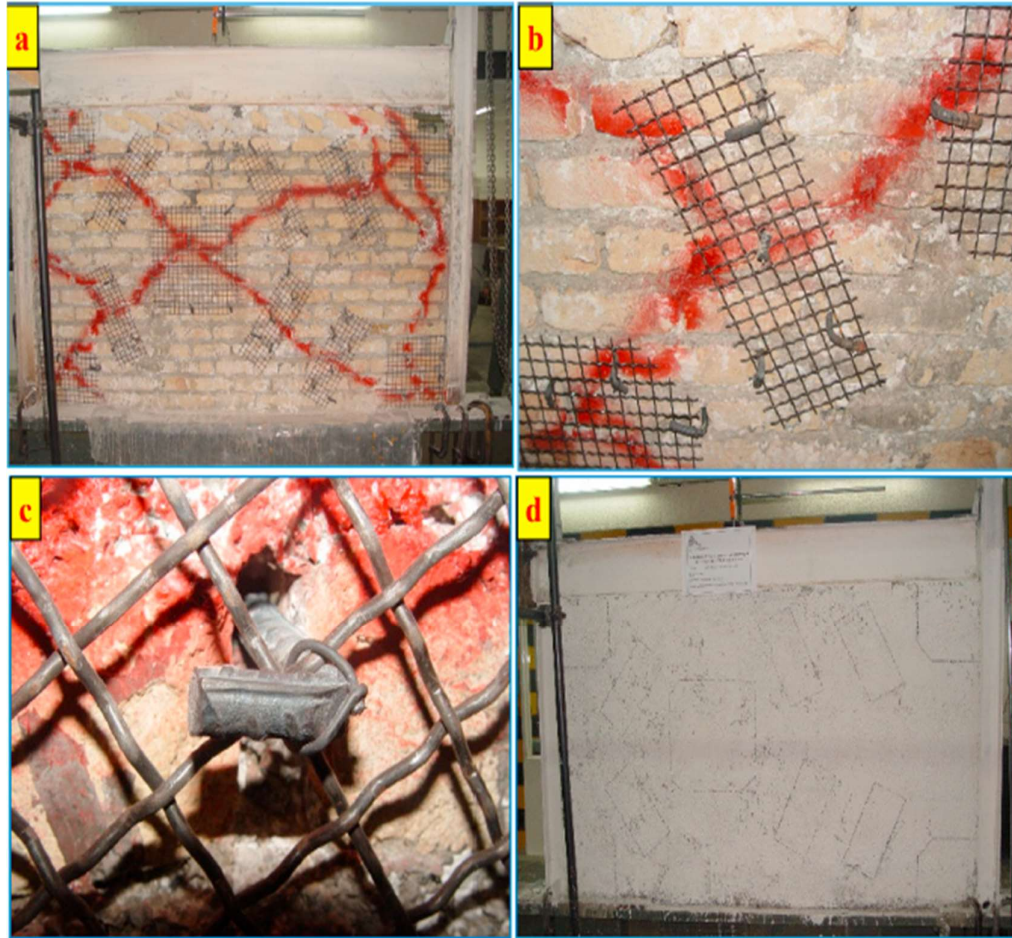


Figure 5.2: (a) Installation of crimped wire mesh on cracked wall, (b) anchored wire mesh with hooked nails, (c) fastening nail and wire mesh with thin wire knot, (d) final step of cement plaster coating on the wall with stitched crack, Ghobadi et al (2018).

Corradi et al, (2020) investigated repairing brickwork masonry walls using smooth titanium rods Figures 5.3a and 5.3b. Rods were embedded into the horizontal joints using an epoxy paste or a cement mortar. A double-sided repair was considered. According to their study, the premature debonding of the rods occurred in panels repaired with a cement mortar, which had lower strength compared to the panels repaired using an epoxy paste; the shear strength of panels repaired with a cement mortar was only 47.6% of control specimens. For panels repaired with epoxy paste, the average shear strength was 74.1%. The experimental study on titanium-repaired panels has shown that shear failures do not necessarily occur in the repaired area, and as such,

the design of such members should be based on different criteria from those for wall panel strengthening. Walls panels repaired with titanium rods and epoxy resins failed due to the formation of a diagonal crack outside the repaired area of the panel.



Figures 5.3a and 5.3b: A repaired wall panel using titanium rods for crack stitching, Corradi et al (2020).

The shape of externally bonded FRP has a significant impact on the effectiveness of strengthening. If FRP reinforcement is applied only across the shear cracks, even if there are flexural cracks, the initial lateral stiffness of the repaired wall may equal the stiffness of the flexural cracked section (Santa-Maria and Alcaino 2011).

The repair and strengthening of damaged or undamaged structural members with externally bonded fiber-reinforced polymer (CFRP) sheets has become one of the important challenges confronting civil engineers and researchers. CFRP composite systems when bonded to various damaged or undamaged structural members can significantly improve their mechanical behavior in serviceability conditions, at failure mechanism, and reduce crack propagation by increasing both the stiffness and the load capacity resistance, Hamrat et al, 2019.

According to (Tumialan 2001) greater than a 200% increase in compressive resistance of masonry columns can be expected if the anchorage of FRP is reliable. Gu (2012), investigated the strengthening of a scaled five-story masonry structure built with concrete perforated bricks and the structure was initially tested on a shake table, from which its displacement, acceleration responses, and cracking were recorded. The

partially damaged structure was subjected to rapid strengthening (in less than 3 days) using basaltic fiber-reinforced polymer (BFRP). The strengthening was carefully designed, and special anchorage measures were used on the first floor of the structure to ensure the strengthening effect of the externally bonded BFRP stripped sheets. The strengthened structure was then retested on the shake table to quantify the strengthening effect. By comparing the performance of the masonry structure during the two shake table tests, it was found that the externally bonded BFRP sheets worked well with the masonry structure, and the seismic performance of the masonry structure was improved significantly. The proposed strengthening method led to an increase in the resistance, deformability, and integrity of the structure. A favorable change in the crack mode and an increase in energy dissipation of the cracked structure were observed. This method can be an efficient alternative to current methods for rapidly strengthening masonry structures cracked in earthquakes.

The effectiveness of FRP strengthening is partly dependent on the properties of the substrate. As reported by Hamed and Rabinovitch (2010), solid autoclaved aerated concrete (AAC) block walls can be successfully strengthened by gluing FRP to them, whereas hollow concrete block walls strengthened by gluing FRP suffer premature shear failure, which diminishes the increase in load and deformation capacities (Hamed and Rabinovitch 2010).

Grando et al, (2003) and Tumialan (2003), performed experiments by applying diagonal compression to concrete masonry walls retrofitted with externally-bonded glass FRP (GFRP) laminates or GFRP NSM bars in the mortar joints. Results showed a remarkable enhancement in shear capacity. Yu et al, (2007) tested under diagonal compression concrete block walls strengthened externally with different amounts of GFRP grid-reinforced polyurea. The results again showed considerable improvements in shear capacity. According to the American concrete institute (ACI) (2010) and Al-Salloum et al, (2012), the popularity of FRP stems from its lightweight, simplicity of application, and availability in different forms such as pre-cured laminates, sheets, grids, and bars.

Babaeidarabad et al, (2014) investigated the in-plane behavior of un-reinforced concrete masonry walls externally strengthened with a fabric-reinforced cementitious matrix (FRCM) system. In the experimental program, nine un-reinforced concrete masonry

walls strengthened on both sides with two different FRCM schemes (one and four reinforcement fabrics) were tested. The test results proved the technical viability of FRCM strengthening of un-reinforced concrete masonry walls when subjected to diagonal compression. Depending on the amount of FRCM, retrofitted concrete masonry wall shear capacity increased between 1.95 and 2.36 times that of the control specimen and FRCM strengthening is effective in increasing the stiffness of URM walls.

In order to extend the life of old and cracked structures, strengthening or repairing by implementing new techniques have been developed. Many of these strengthening techniques including the use of fiber-reinforced polymer (FRP) composites were used to reinforce masonry structures. These composites are manufactured in different features depending on the fiber material type like carbon fiber (CFRP), glass (GFRP), and aramid (AFRP). FRP strips offer the possibility of application by gluing on the outside surface (externally bonded) or inserting inside the groove of the element by the near-surface mounted (NSM) technique. The use of FRP composites embedded into polymeric matrices such as (polyester, epoxy, and mortar mix) can be the perfect solution for retrofitting the structure walls in a seismic area. Experimental studies evaluate the effect of different variables, for example, the effect of retrofitting configuration of FRP composite, type of FRP composites, and the type of masonry components, through small-scale testing such as triplets test, Wallette, and by full-scale testing.

The objective of this study was to investigate the strength and stiffness of masonry walls repaired with different layouts of FRP by comparing them to similar undamaged walls strengthened with the same retrofit schemes. The effect of CFRP with different configurations on damaged or cracked walls has not been studied properly. This research addresses a method to reinforce cracked masonry walls to acceptable levels.

This chapter presents the experimental study of brick masonry triplets reinforced with carbon fiber-reinforced polymer (CFRP) composite subjected to a shear load. The results showed that the application of the CFRP composite on unreinforced masonry (URM) has a great influence on strength of masonry triplets as well as changing of failure modes and ductility.

5.2. Research objectives of cracked triplet's study:

- 1 - Study the interaction between different CFRP configurations to reinforce cracked triplet masonry (CTM),
- 2- Repairing and increasing the capacity and ductility of the (CTM) when subjected to shear loading,
- 3 - experimentally and analytically investigate the effectiveness of CFRP to retrofit existing cracked masonry structures,
- 4 - Develop an analytical model of retrofitted masonry walls and compare the results with the experimental results.

5.3. Experimental details of proposed repair method

The experimental program has been developed as a small-scale experiment. Theoretical finite element (FE) models are developed and verification of these models in comparison with experimental results is conducted. This data can provide a reference for the correction of formulas for the design and assessment of masonry retrofitted with composite materials.

Twenty-seven masonry triplets were built with three brick layers and mortar M4, which lead to a shear collapse mode by sliding of the medium layer. They were initially damaged by loading them up to their maximum strength. Then they were repaired using eight configurations of externally bonded carbon fiber and tested under shear loading up to failure. The cracks were cleaned but not repaired before bonding the CFRP. It was observed that the maximum strength, deformation capacity, and failure mode of the repaired samples were similar to undamaged triplets that were strengthened with the same reinforcing schemes. Figures 5.4 to 5.29 shows the results and comparison of different CFRP configurations illustrated in Figures 5.30 and 5.31.

The repaired triplets were tested in two stages: during the first stage, the triplets were damaged by pre-loading them up to a deformation level (2 mm crack) and debonding brick and mortar.

The experimental tests on reinforced cracked triplet masonry showed that the unreinforced triplets failed at the early stage of loading and the displacement of the

middle brick was about 20mm. Because of pre-loading and cracks in triplets, some of them already failed before loading. The maximum load just before failure for samples with CFRP sheets on both sides was about 50KN which shows CFRP reinforcement at both sides makes the triplet 4 times stronger than samples without initial damage. The maximum load for the CFRP net on four sides of the cracked triplet was about 50KN that shows reinforcement with the CFRP net on four sides makes the triplet 4 times stronger and has no significant difference between the CFRP net and the CFRP sheet.

The maximum load just before failure for the CFRP net on two sides was about 40KN which indicates the CFRP net on two sides makes the triplet 3 times stronger. The failure point for the CFRP strips on two sides was about 38KN that illustrates reinforcement with CFRP strips on two makes the triplet 3 times stronger. The maximum load just before failure for the CFRP strips on two sides of the cracked triplet was about 38KN and 3 times stronger than samples without initial damage and CFRP strips on two sides are slightly weaker than CFRP strips on four sides. The test for CFRP net strips on four sides makes cracked triplet 4 times stronger and the maximum load just before failure was about 45KN. CFRP net strips on four sides and CFRP net strips on two sides are the same in case of maximum load at the failure point. The failure point for CFRP net strips on two sides of the samples was about 45KN which makes triplet 4 times stronger and CFRP net strips on four sides and CFRP net strips on two sides are the same in case of maximum load at the failure point. The CFRP sheets on four sides resist up to 50KN triplet was 4 times stronger and no significant difference between CFRP application on four sides and two sides.

The main reason for the failure of CFRP configurations on all triplets was sliding middle brick along the bed joint and delamination and debonding of CFRP from bricks. The reinforced triplet masonry with CFRP were 3 to 4 times stronger than samples without initial damage.

5.4. Cracked URTM under shear loading

The measured responses of the unreinforced triplets in the loading stage are shown in Figures 5.4a and 5.4b. The result shown triplets failed at an early stage of loading and the displacement of the middle brick was about 20mm. Because of pre-loading and cracks in triplets, some of them already failed before loading.

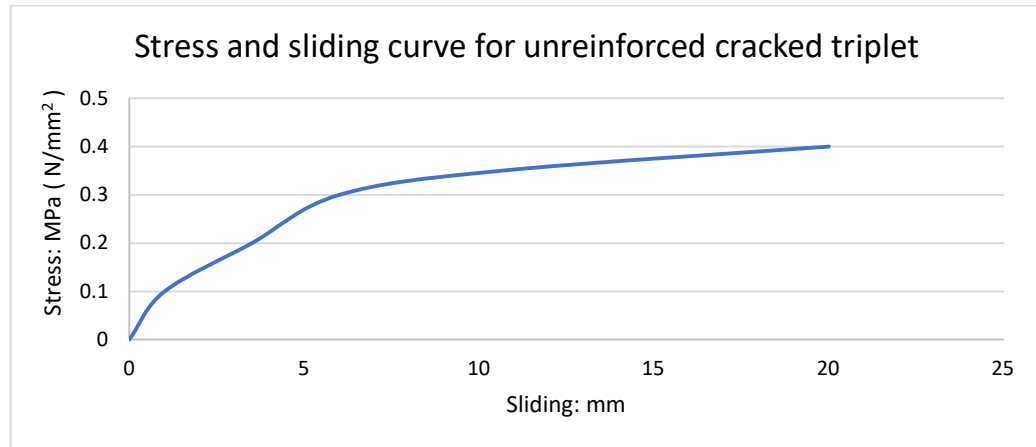


Figure 5.4a: Stress and sliding curve for unreinforced cracked triplet

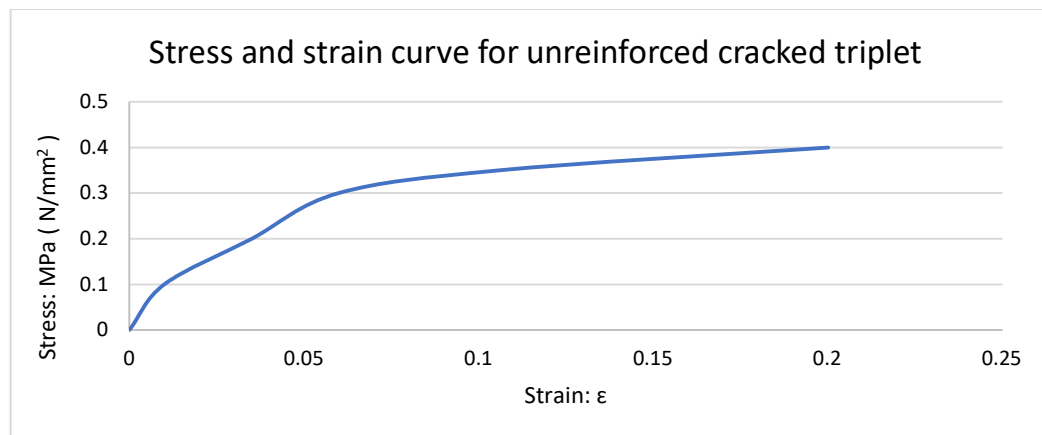


Figure 5.4b: Stress and strain curve for unreinforced cracked triplet

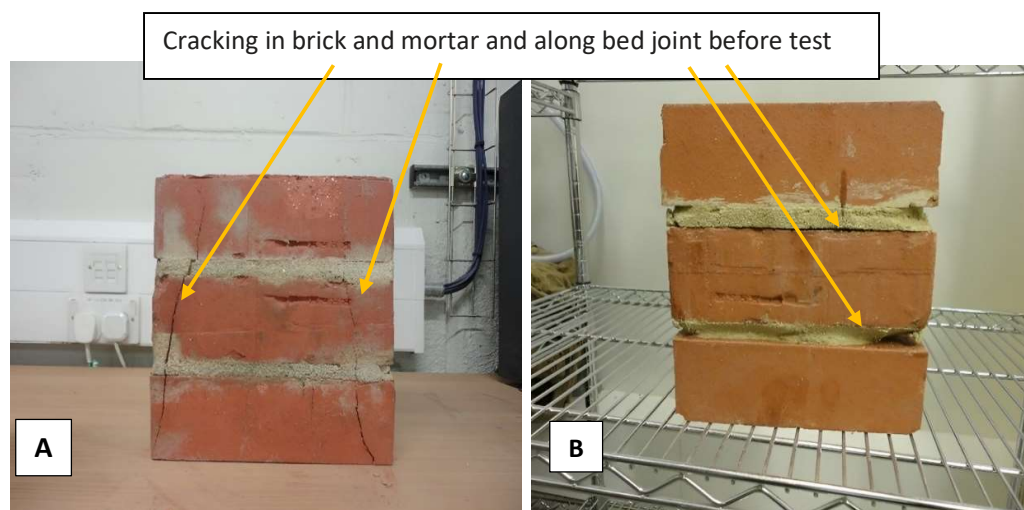


Figure 5.5: Cracked triplet masonry without CFRP application

5.5. Cracked triplet with 2 sides CFRP sheets

The results for the CFRP sheets on both sides of the cracked triplet are shown in figures 5.6 and 5.7. The maximum load just before failure was about 50KN which shows CFRP reinforcement at both sides makes the triplet 4 times stronger than samples without initial damage. The main reason for the failure of the triplet was sliding middle brick along the bed joint, debonding and rupture of CFRP from bricks as can be seen in figure 5.8.

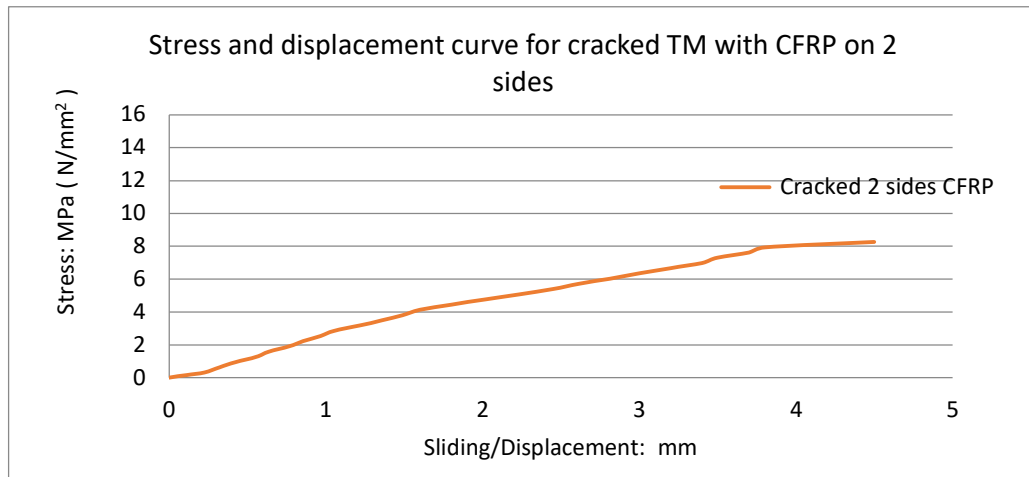


Figure 5.6: Stress & sliding/displacement of cracked triplet with 2 sides CFRP sheets

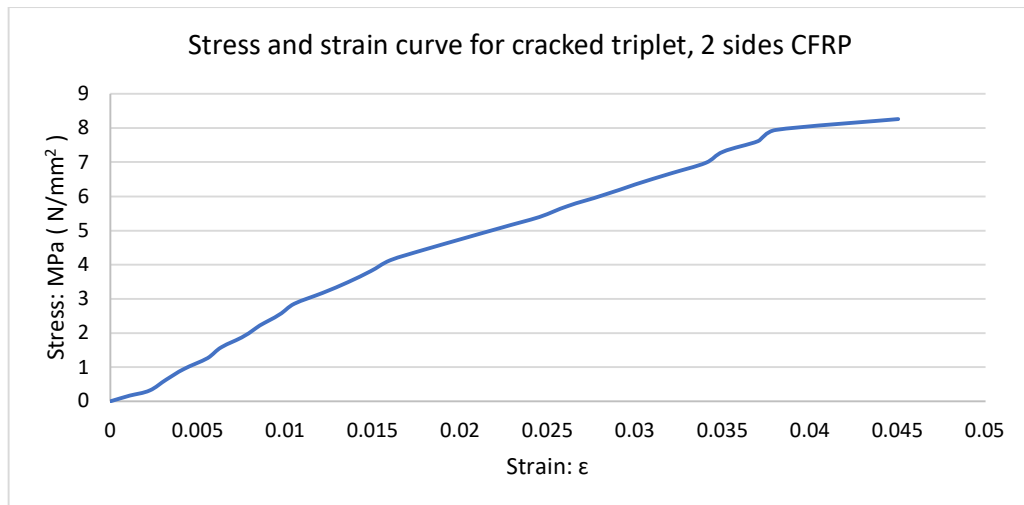


Figure 5.7: Stress and strain of cracked triplet with 2 sides CFRP sheets

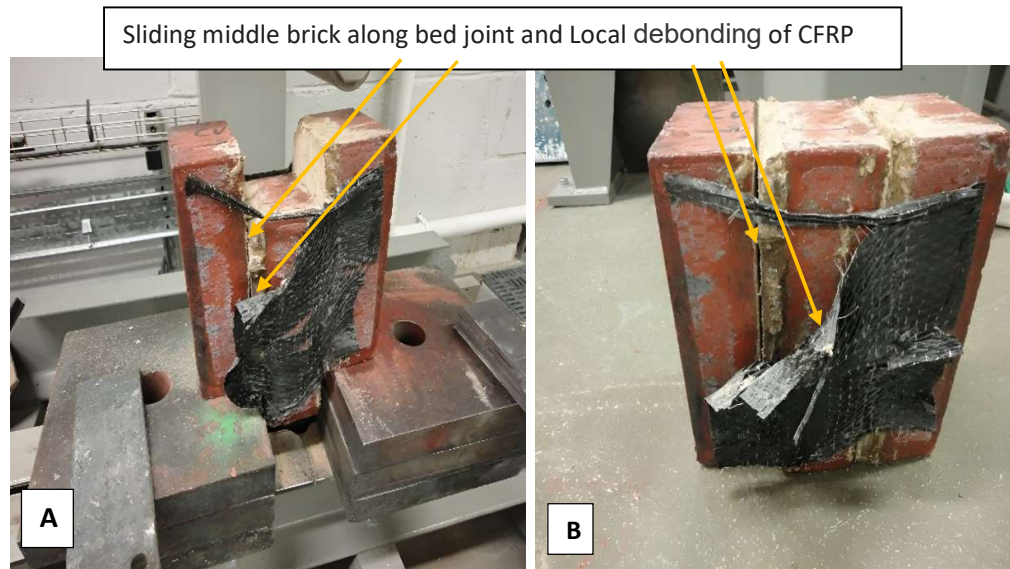


Figure 5.8: Cracked triplet masonry with two sides CFRP

5.6. Cracked triplet with 4 sides CFRP net

The results for the CFRP net on four sides of the cracked triplet are shown in figures 5.9 and 5.10. The maximum load just before failure was about 50KN which shows reinforcement with CFRP net on four sides makes triplet 4 times stronger than samples without initial damage and no significant difference between CFRP net and CFRP sheet. The main reason for the failure of the triplet was the sliding middle brick along the bed joint, debonding and rupture of CFRP net from bricks as can be seen in figure 5.11.

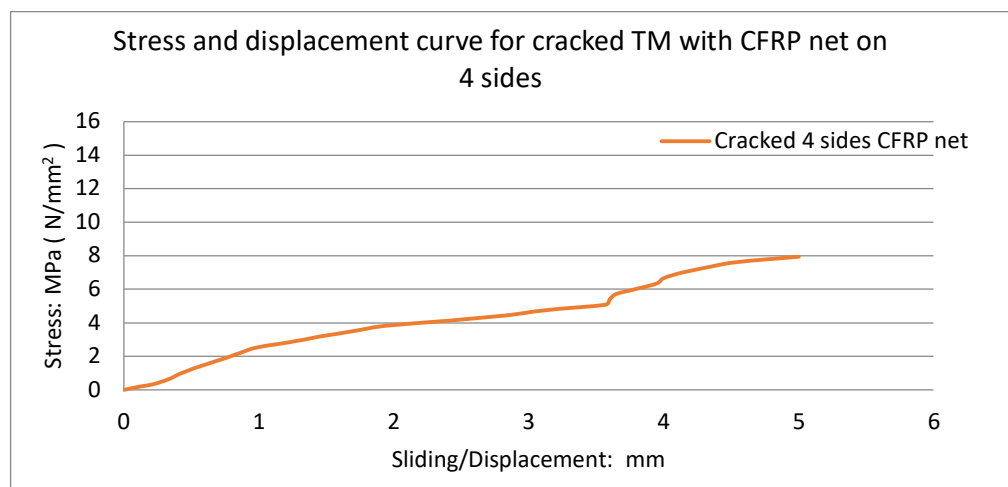


Figure 5.9: Stress & sliding/displacement of cracked triplet with 4 sides FRP net

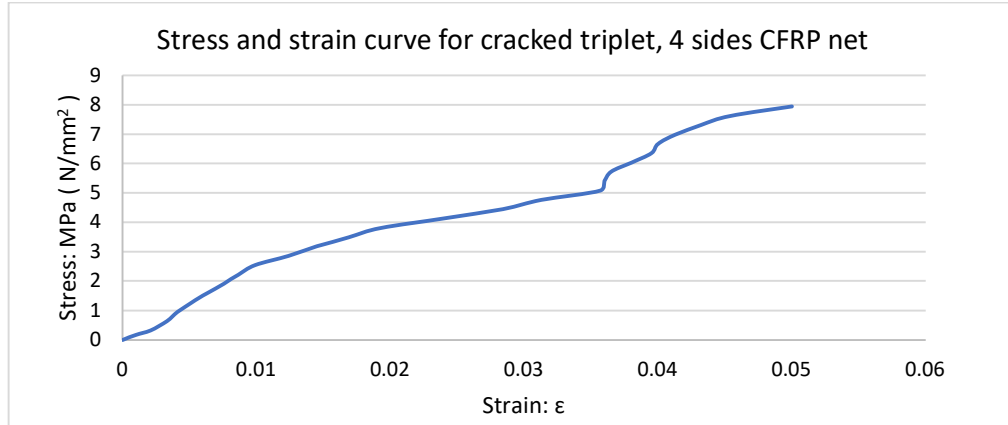


Figure 5.10: Stress and strain of cracked triplet with 4 sides FRP net

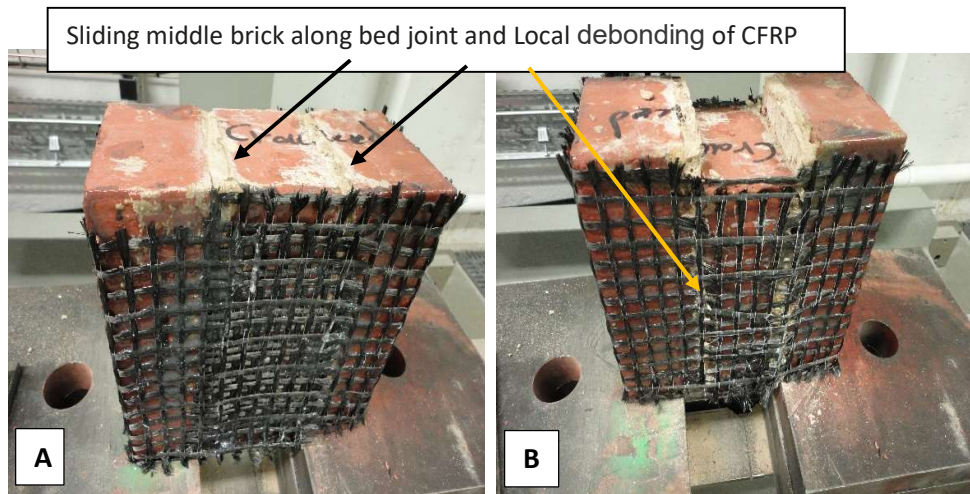


Figure 5.11: Cracked triplet masonry with four sides CFRP net

5.7. Cracked triplet with 2 sides CFRP net

The results for the CFRP net on two sides of cracked triplet shown in figures 5.12 and 5.13. The maximum load just before failure was about 40KN that shows reinforcement with CFRP net on two sides makes triplet 3 times stronger than samples without initial damage. The main reason for failure of triplet was sliding middle brick along bed joint, rupture and debonding of CFRP net from bricks as can be seen in figure 5.14.

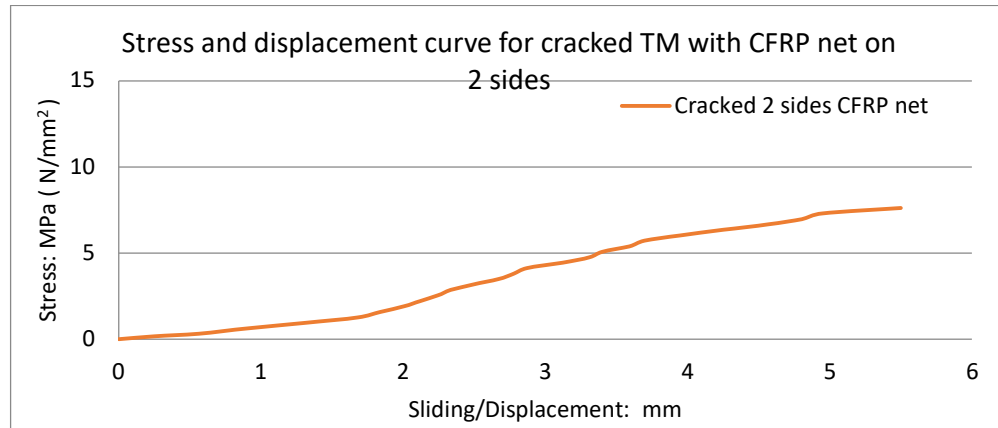


Figure 5.12: Stress & sliding/displacement of cracked triplet with 2 sides FRP net

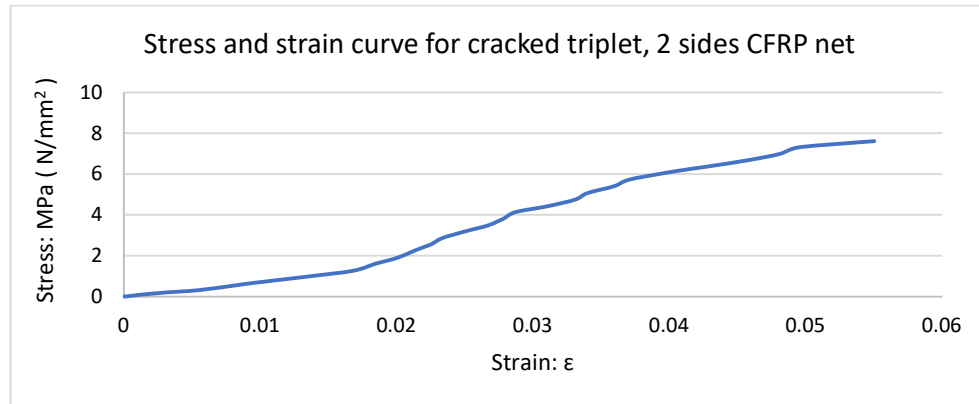


Figure 5.13: Stress and strain of cracked triplet with 2 sides FRP net

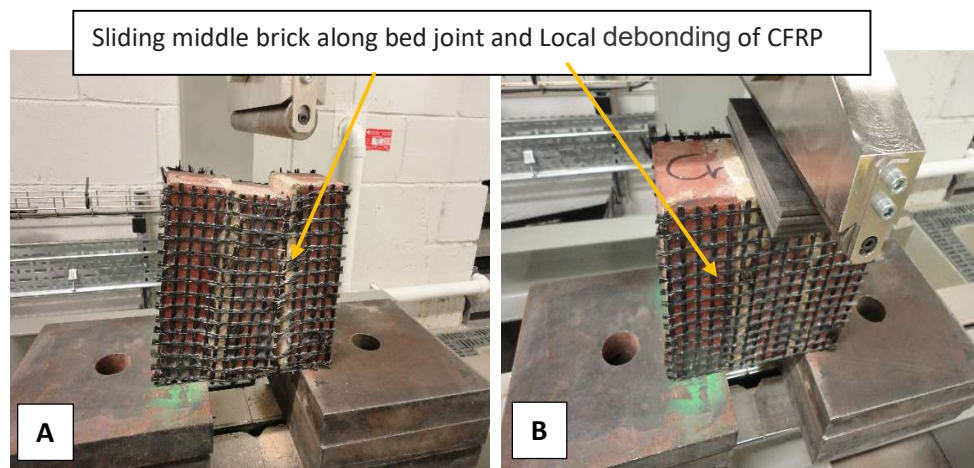


Figure 5.14: Cracked triplet masonry with two sides CFRP net

5.8. Cracked triplet with 4 sides CFRP strips

The results for the CFRP strips on two sides of the cracked triplet are shown in figures 5.15 and 5.16. The maximum load just before failure was about 38KN which shows reinforcement with CFRP strips on two makes the triplet 3 times stronger than samples without initial damage and CFRP strips on four sides are slightly stronger than CFRP strips on two sides. The main reason for the failure of the triplet was the sliding middle brick along the bed joint, rupture and debonding of CFRP strips from bricks as can be seen in figure 5.17.

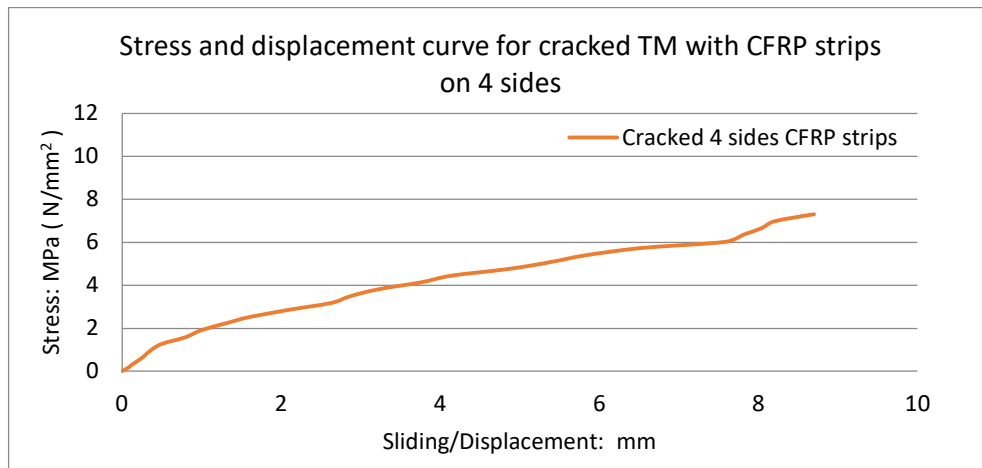


Figure 5.15: Stress & displacement/sliding of cracked triplet with 4 sides FRP strips

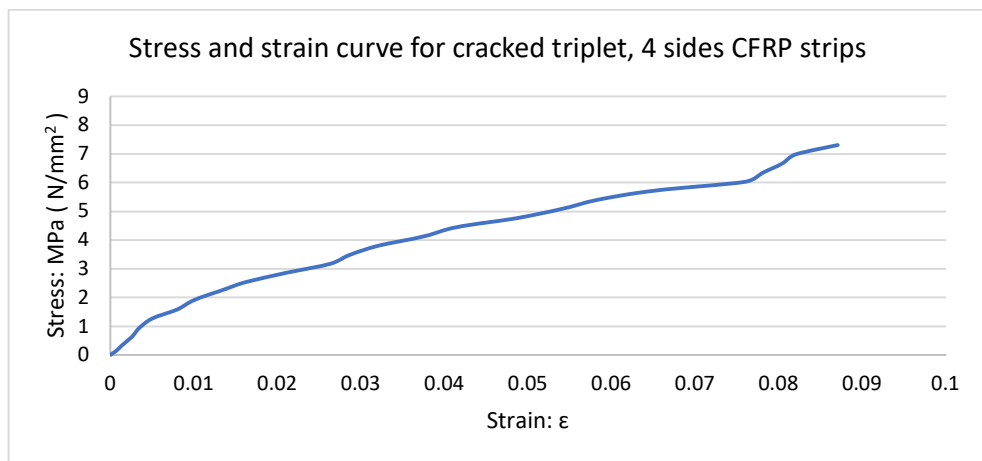


Figure 5.16: Stress and strain of cracked triplet with 2 sides FRP net

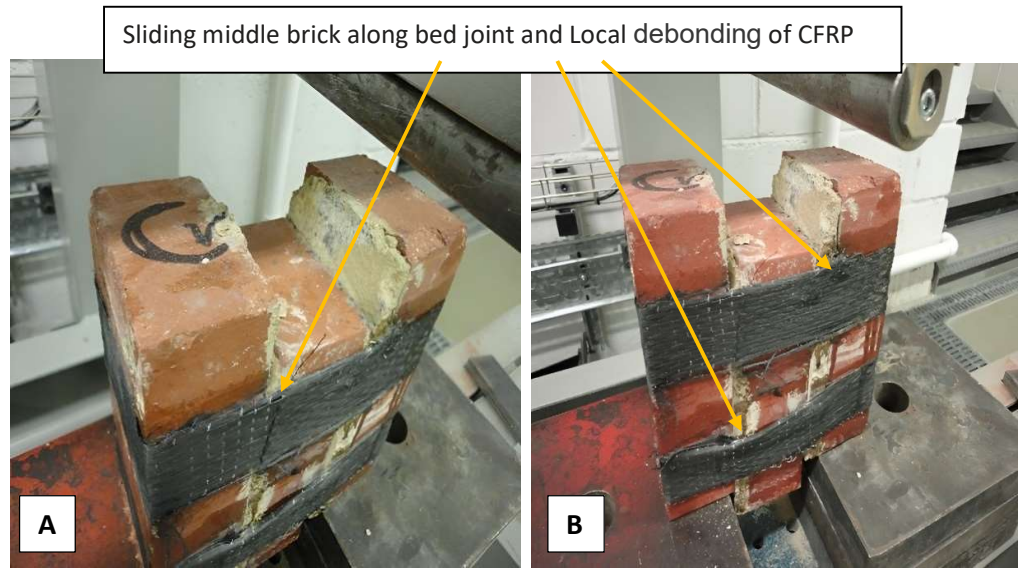


Figure 5.17: Cracked triplet masonry with four sides CFRP strips

5.9. Cracked triplet with 2 sides CFRP strips

The results for the CFRP strips on two sides of the cracked triplet are shown in figures 5.18 and 5.19. The maximum load just before failure was about 38KN which shows reinforcement with CFRP strips on two makes the triplet 3 times stronger than samples without initial damage and CFRP strips on two sides are slightly weaker than CFRP strips on four sides. The main reason for the failure of the triplet was the sliding middle brick along the bed joint and the delamination and debonding of CFRP strips from bricks as can be seen in figure 5.20.

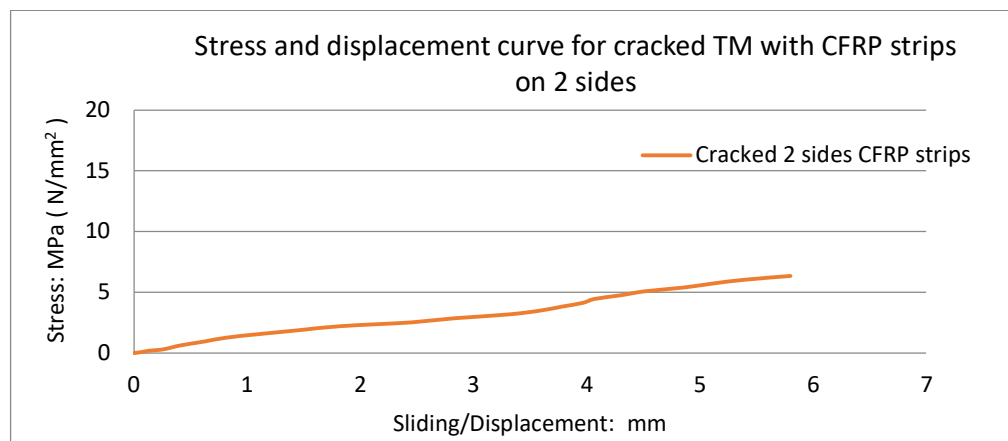


Figure 5.18: stress & displacement/sliding of cracked triplet with 2 sides CFRP strips

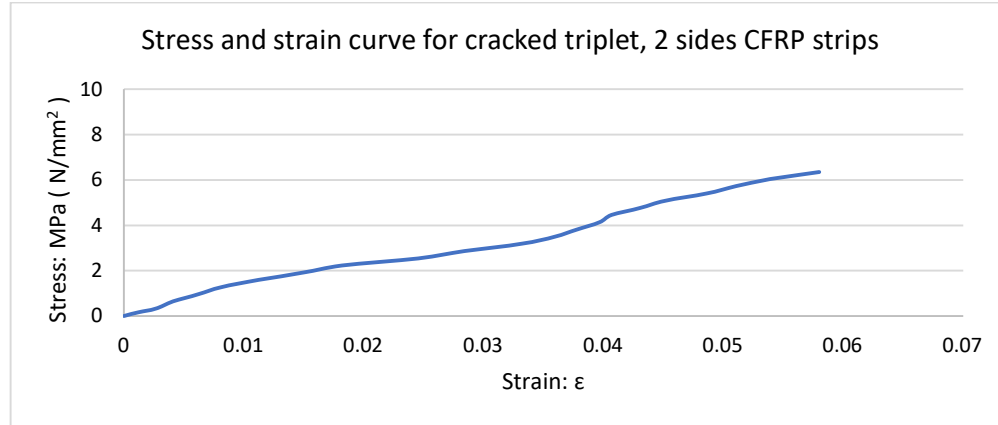


Figure 5.19: Stress and strain of cracked triplet with 2 sides CFRP strips

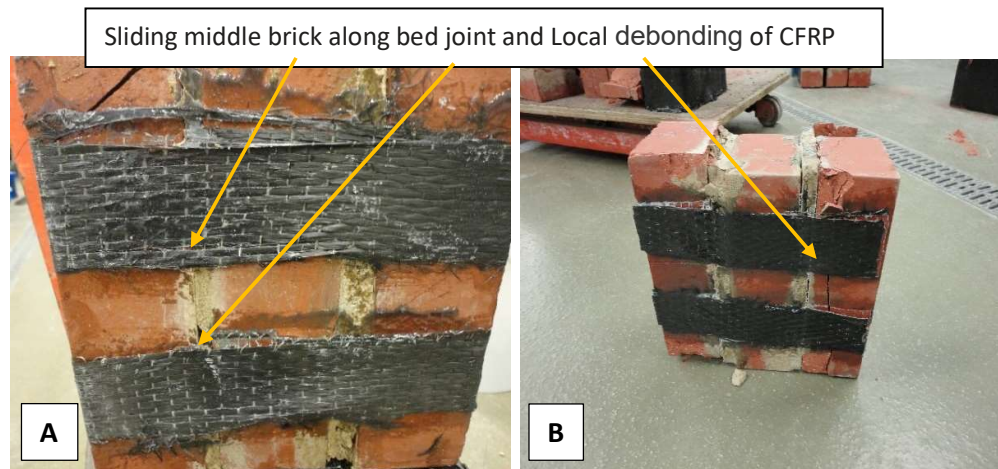


Figure 5.20: Cracked triplet masonry with two sides CFRP strips

5.10. Cracked triplet with 4 sides CFRP net strips

The results for the CFRP net strips on four sides of cracked triplet shown in figures 5.21 and 5.22. The maximum load just before failure was about 45KN that shows reinforcement with CFRP net strips on four sides makes triplet 4 times stronger than samples without initial damage and CFRP net strips on four sides and CFRP net strips on two sides are the same in case of maximum load at failure point. The main reason for failure of triplet was sliding middle brick along bed joint and delamination and debonding of CFRP from bricks as can be seen in figure 5.23.

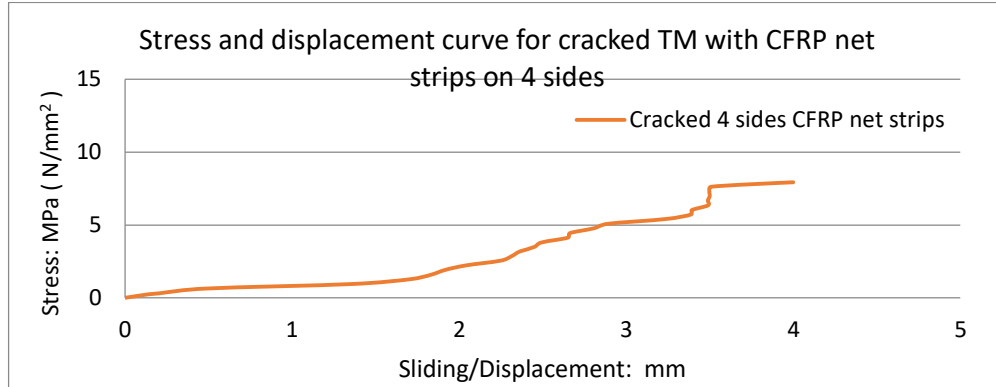


Figure 5.21: Stress & displacement/sliding of cracked triplet with 4 sides CFRP net strips

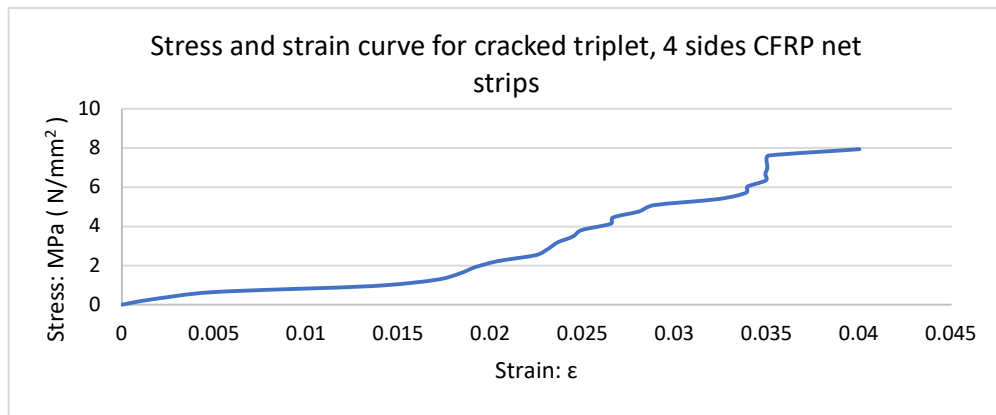


Figure 5.22: Stress and strain of cracked triplet with 4 sides FRP net strips

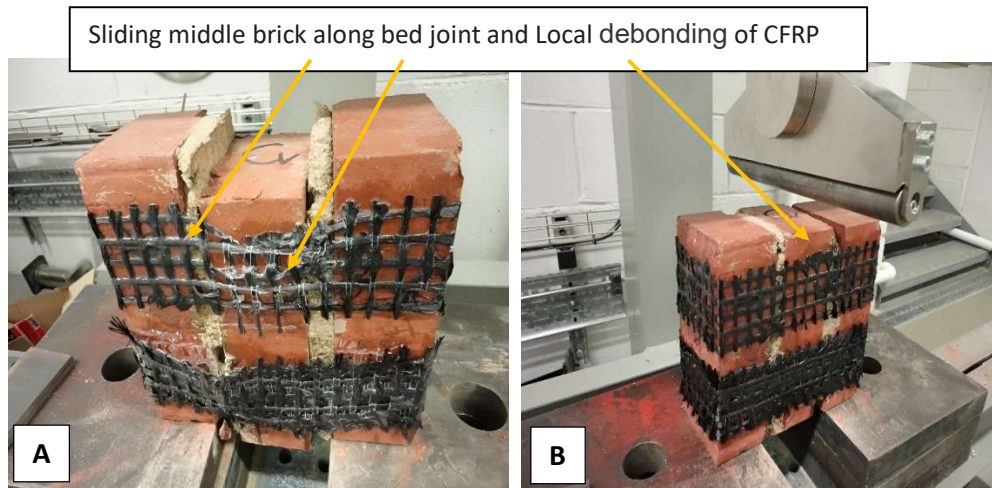


Figure 5.23: Cracked triplet masonry with four sides CFRP net strips

5.11. Cracked triplet with 2 sides CFRP net strips

The results for the CFRP net strips on two sides of cracked triplet shown in figures 5.24 and 5.25. The maximum load just before failure was about 45KN that shows reinforcement with CFRP net strips on two sides makes triplet 4 times stronger than samples without initial damage and CFRP net strips on four sides and CFRP net strips on two sides are the same in case of maximum load at failure point. The main reason for failure of triplet was sliding middle brick along bed joint and delamination and debonding of CFRP from bricks as can be seen in figure 5.26.

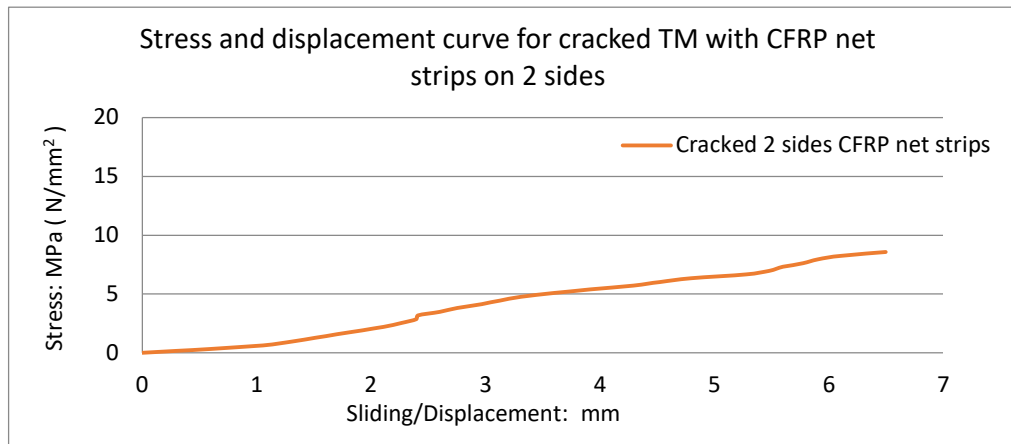


Figure 5.24: Stress & displacement/sliding of cracked triplet with 2 sides CFRP net strips

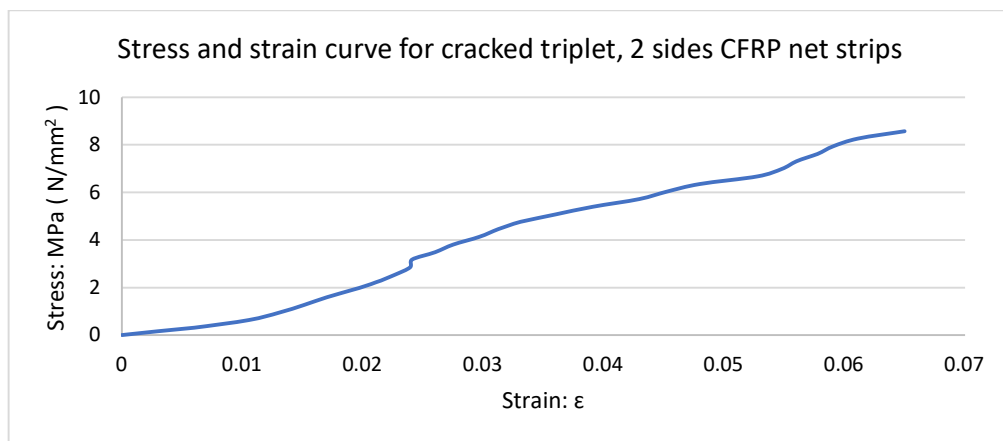


Figure 5.25: Stress and strain of cracked triplet with 2 sides CFRP net strips

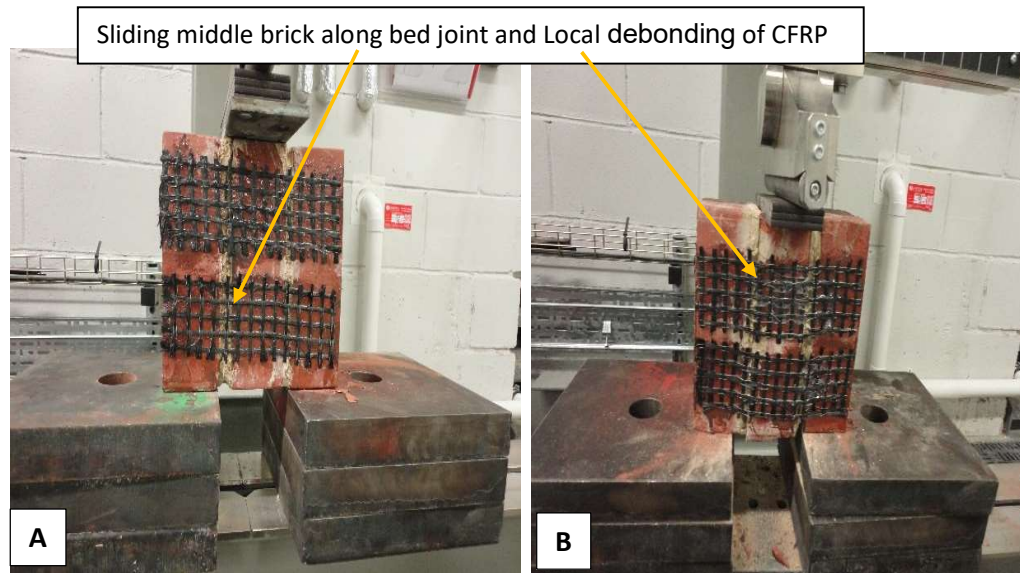


Figure 5.26: Cracked triplet masonry with two sides CFRP net strips

5.12. Cracked triplet with 4 sides CFRP

The results for the CFRP sheets on four sides of cracked triplet shown in figures 5.27 and 5.28. The maximum load just before failure was about 50KN that shows CFRP reinforcement at four sides makes triplet 4 times stronger than samples without initial damage. The main reason for failure of triplet was sliding middle brick along bed joint, delamination and debonding of CFRP sheets from bricks as can be seen in figure 5.29. There is no significant difference between CFRP application on four sides and two sides.

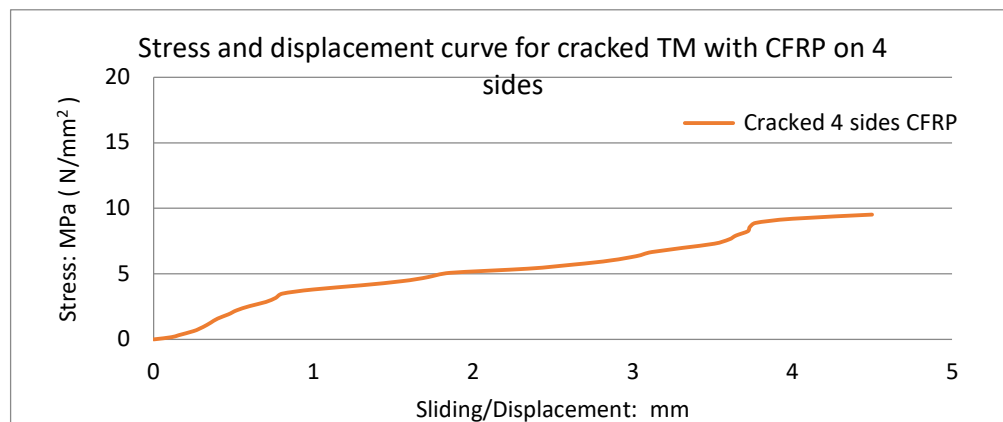


Figure 5.27: Stress & displacement/sliding of cracked triplet with 4 sides CFRP sheets

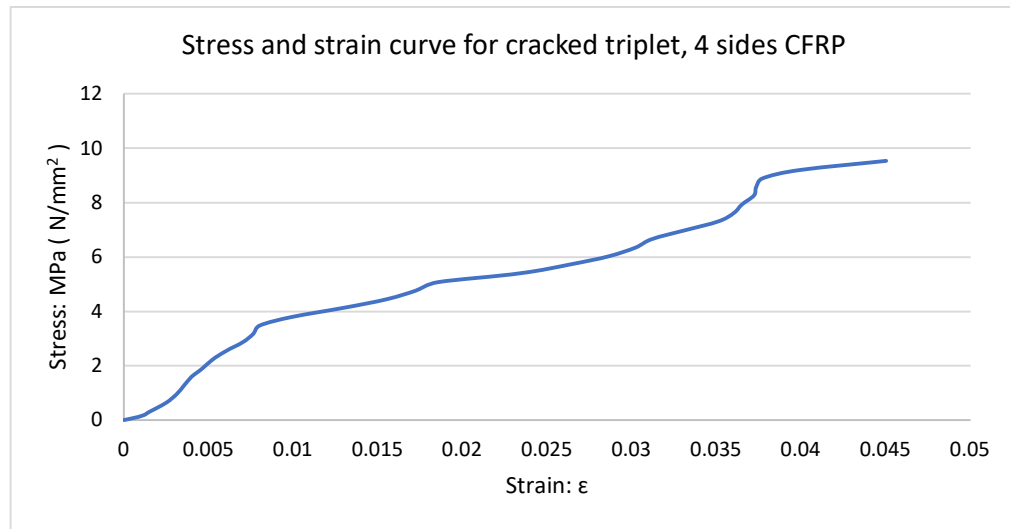


Figure 5.28: Stress and strain of cracked triplet with 4 sides CFRP sheets

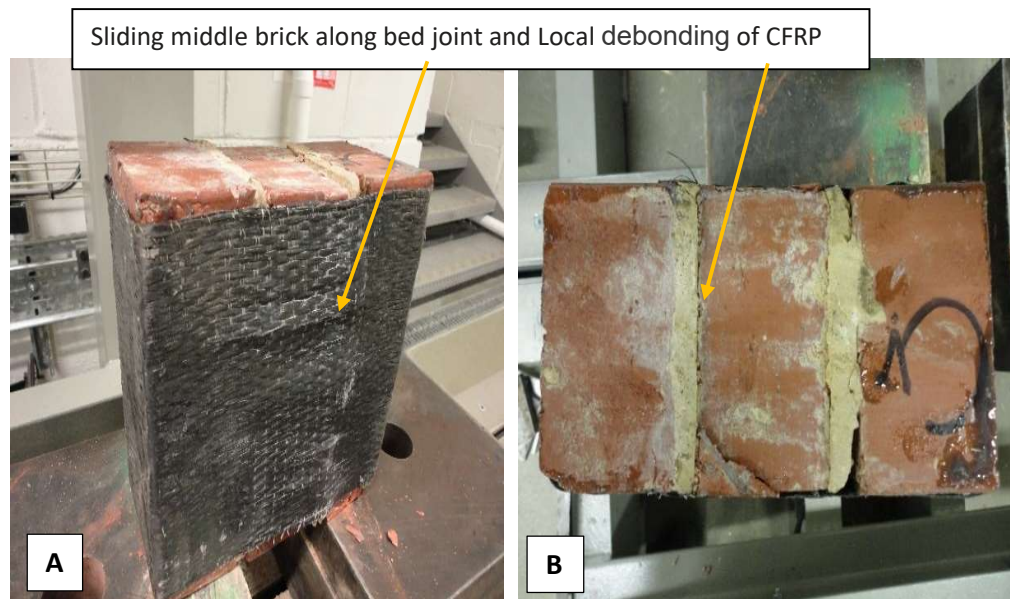


Figure 5.29: Cracked triplet masonry with four sides CFRP sheets

5.13. Comparison of maximum load, stress & sliding of cracked triplets with different CFRP configurations

A comparison of experimental tests with different CFRP configurations is presented in Figures 5.30 and 5.31. There is no significant difference between the two sides and four sides CFRP sheets application. CFRP strips on two sides and four sides are nearly the same in case of stress and sliding. These results indicate that after debonding the middle brick from the FRP composite, the extra length of carbon fiber on the side bricks did not make a stronger bond in triplets.

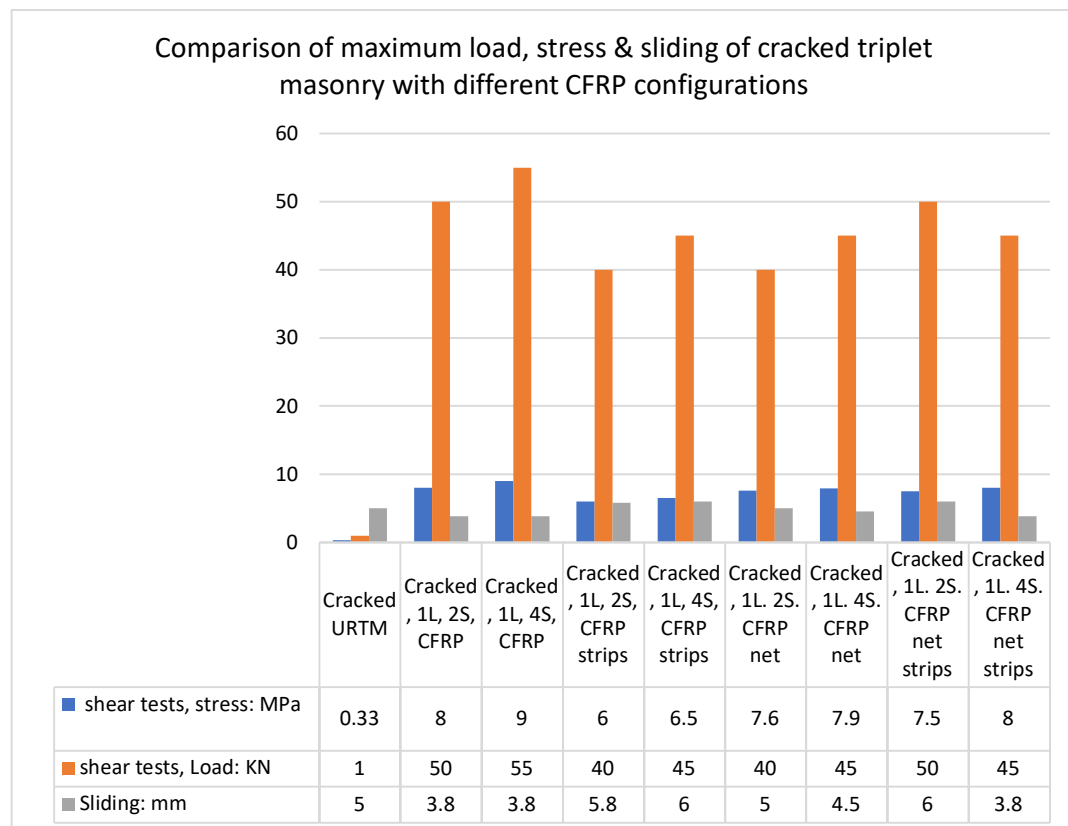


Figure 5.30: Comparison of stress, load and sliding/of cracked triplet masonry

5.14. Comparison of maximum stress & sliding of cracked masonry triplets with different CFRP configurations

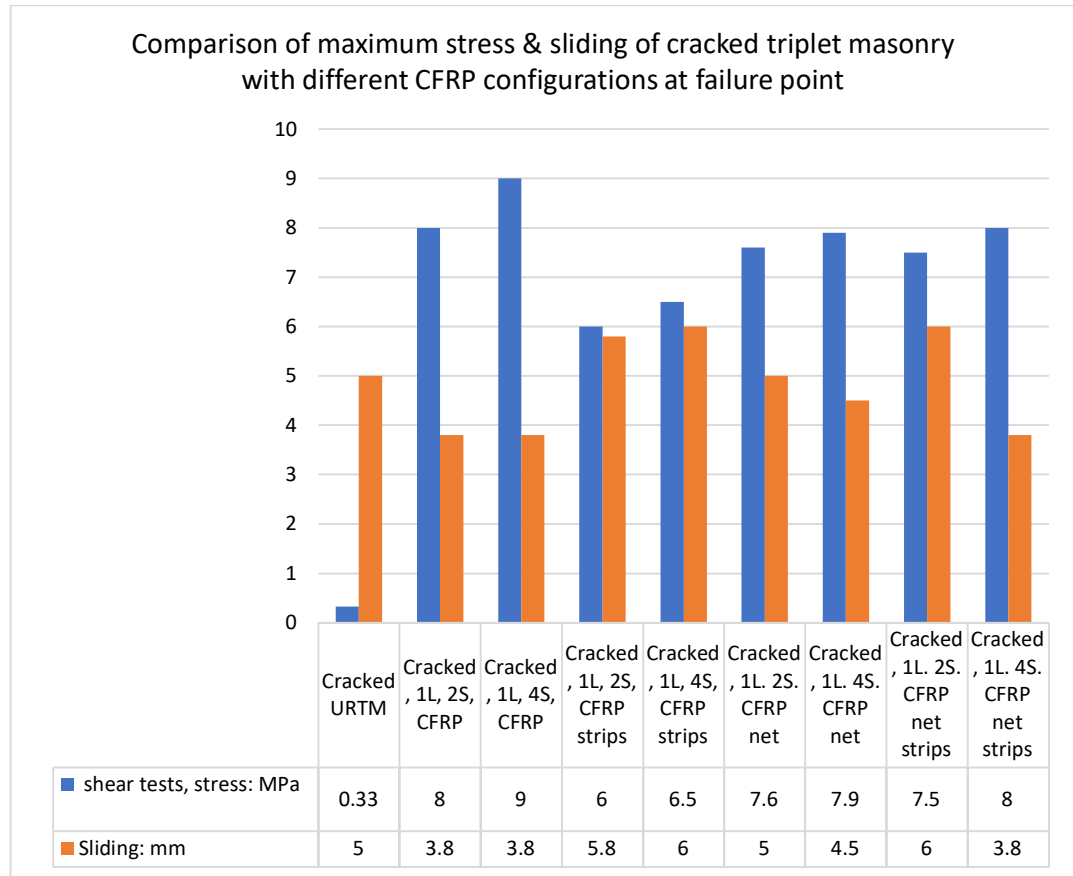


Figure 5.31: Comparison of maximum stress & sliding of cracked triplet masonry with different CFRP configurations at failure point

5.15. Comparison of stress, load and sliding of cracked triplet masonry and normal triplet masonry

Eight different arrangements of CFRP were used to repair cracked triplet masonry. The samples were tested under shear loading and the test results show triplet masonry were three to four time stronger than the samples without initial damage. The increase of shear strength and maximum displacement at the failure of repaired triplets are 20% to 40% less than reinforced triplets without damage. The cracking patterns at the maximum load of the repaired triplet with CFRP were 15% to 30% less than triplets without cracks. Triplet masonry with an initial crack and repaired with externally bonded CFRP may recover the undamaged initial lateral stiffness, depending on the retrofitting arrangement.

The failure of repaired triplet was characterized by progressive delamination of the CFRP followed by a loss of strength due to pulling off the CFRP reinforcement, generating a relatively ductile failure. Figure 5.32 to Figure 5.47 shows the comparison of triplet masonry cracked samples with different CFRP configurations.

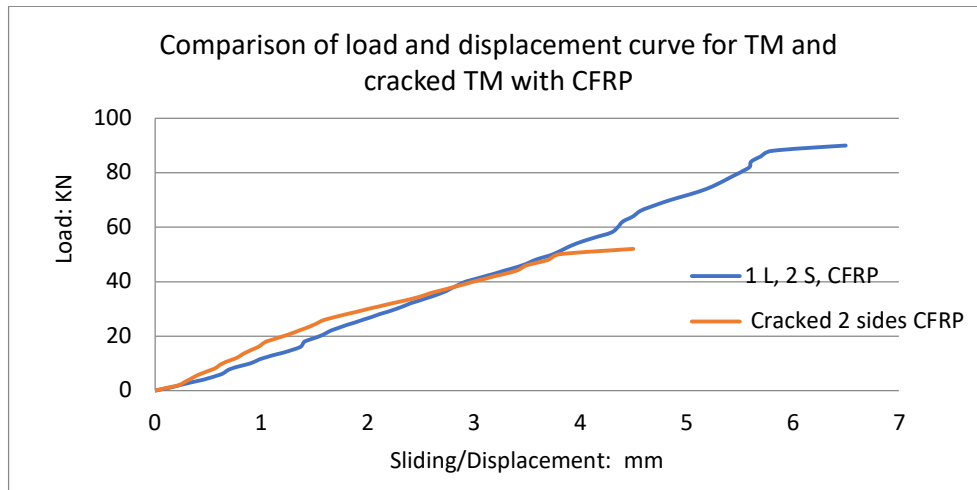


Figure 5.32: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP

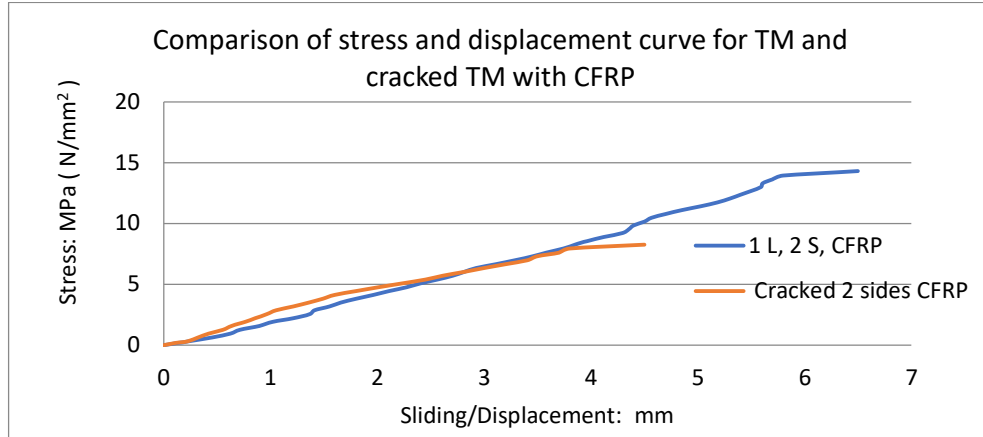


Figure 5.33: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP

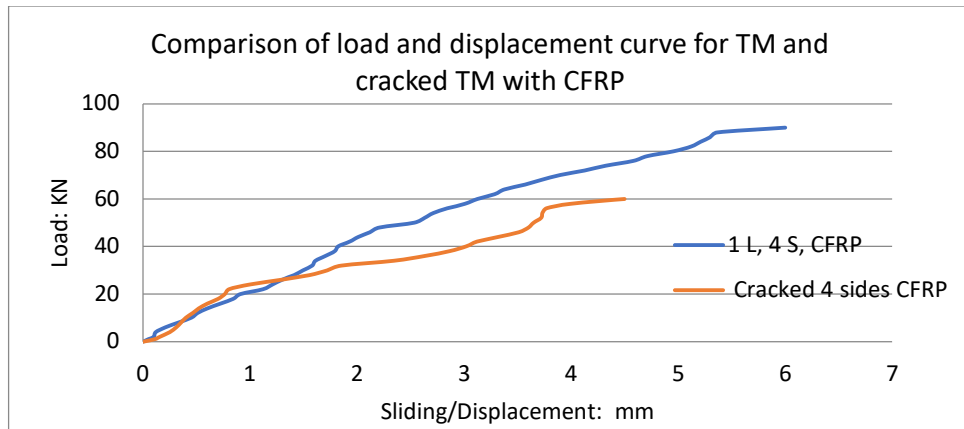


Figure 5.34: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP

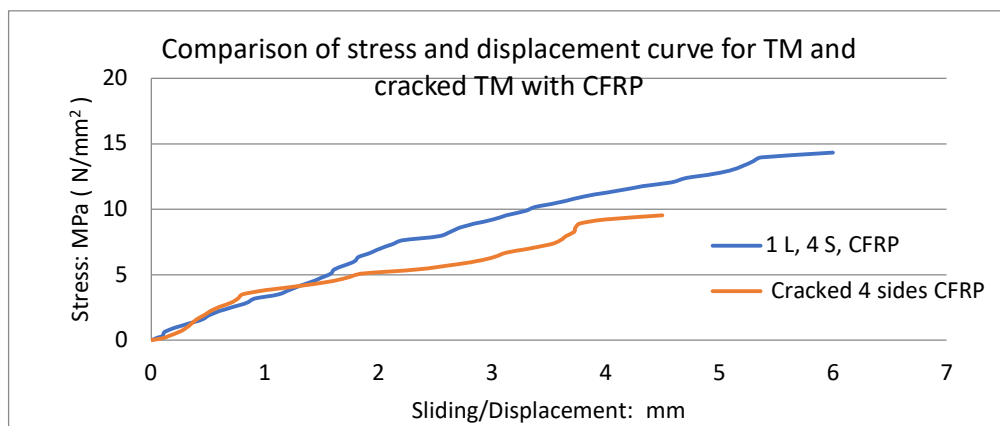


Figure 5.35: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP

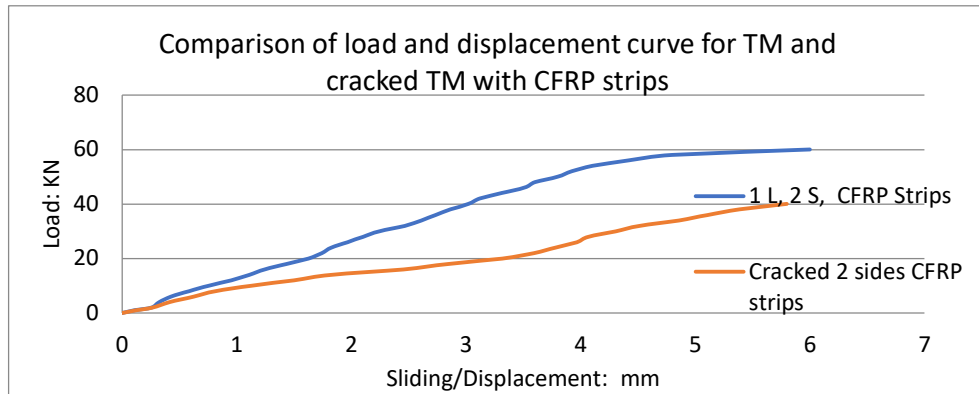


Figure 5.36: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP strips

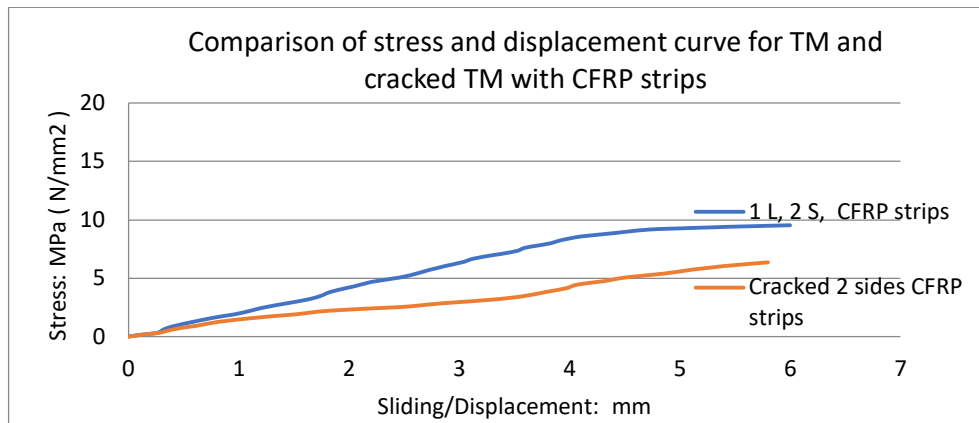


Figure 5.37: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP strips

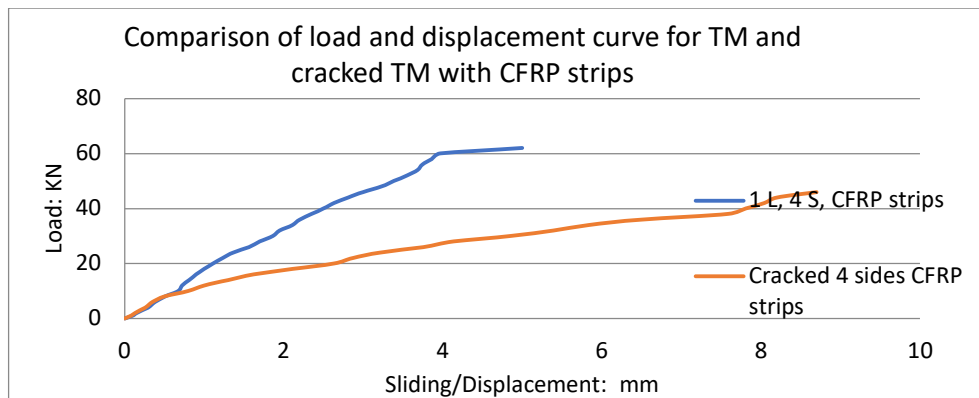


Figure 5.38: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP strips

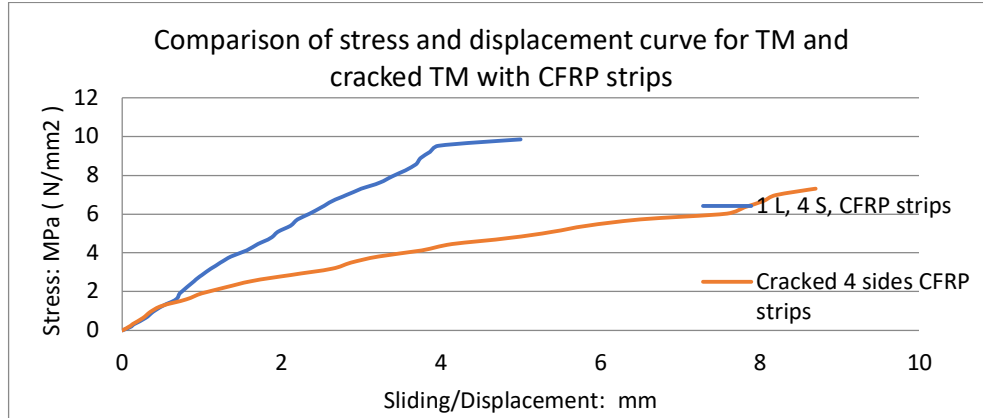


Figure 5.39: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP strips

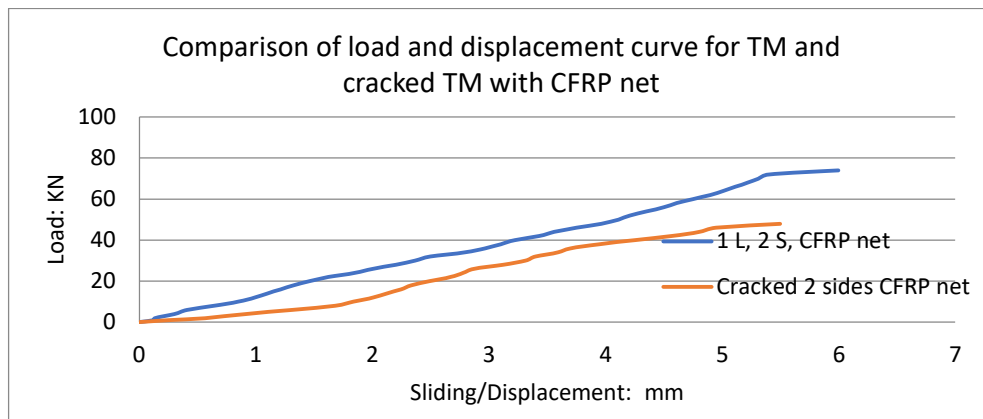


Figure 5.40: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP Net

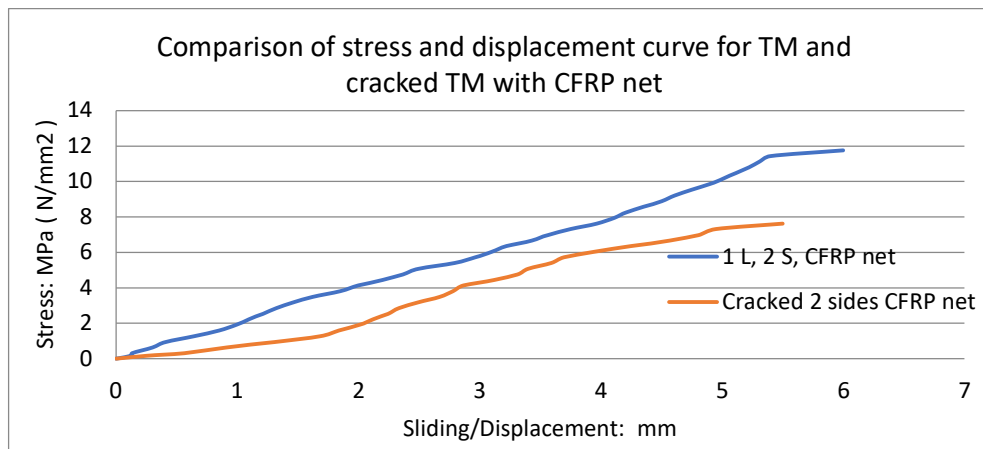


Figure 5.41: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP Net

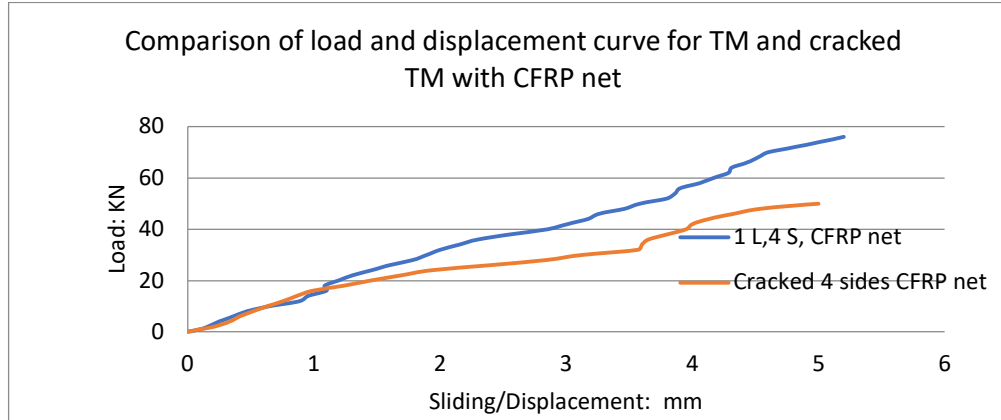


Figure 5.42: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP Net

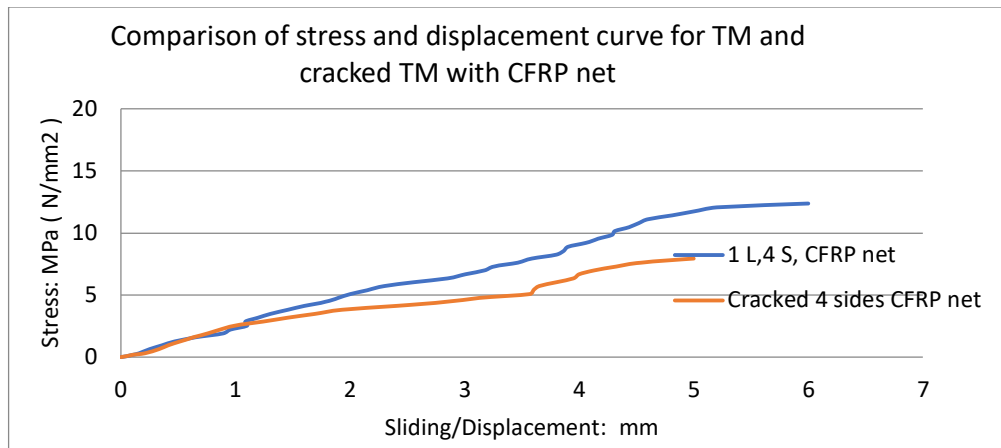


Figure 5.43: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP Net

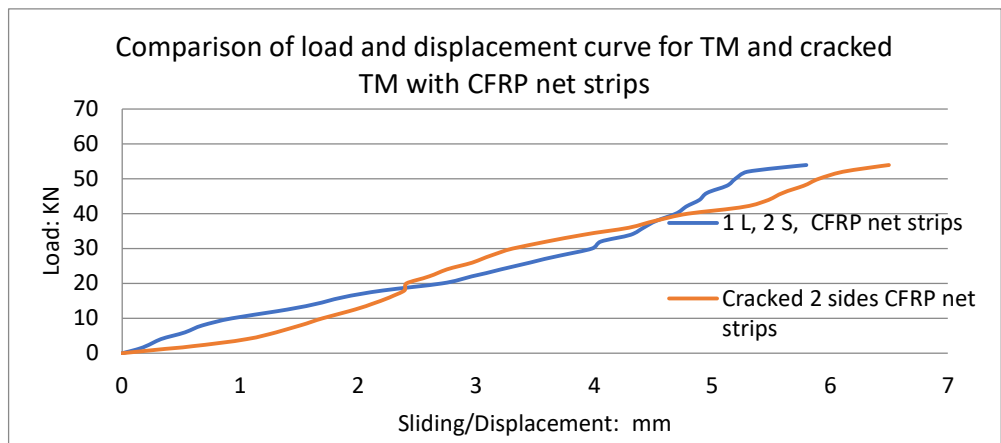


Figure 5.44: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP Net strips

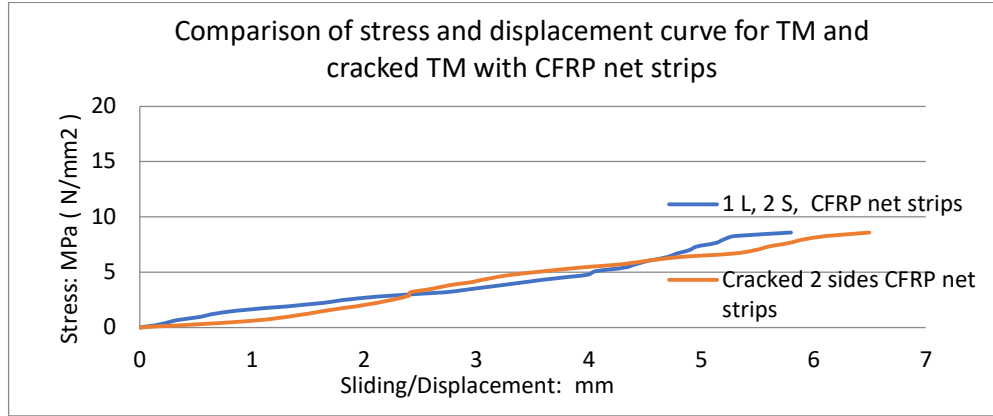


Figure 5.45: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP Net strips

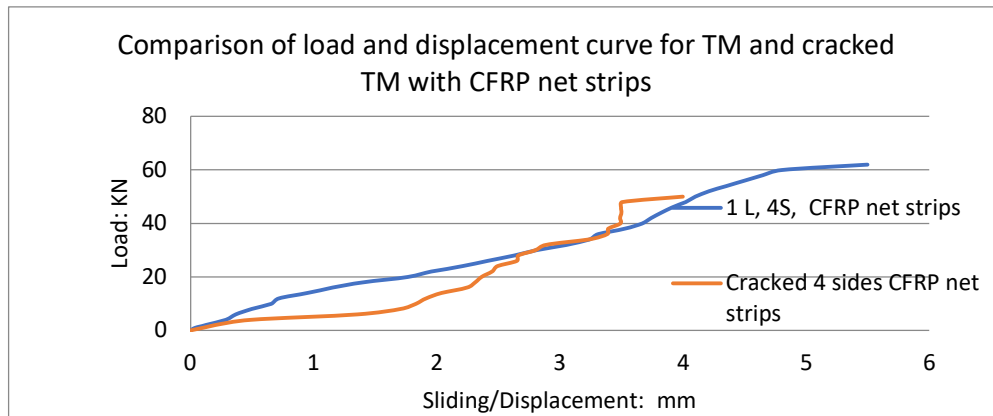


Figure 5.46: Comparison of Load and Sliding curve for TM and Cracked TM with CFRP Net strips

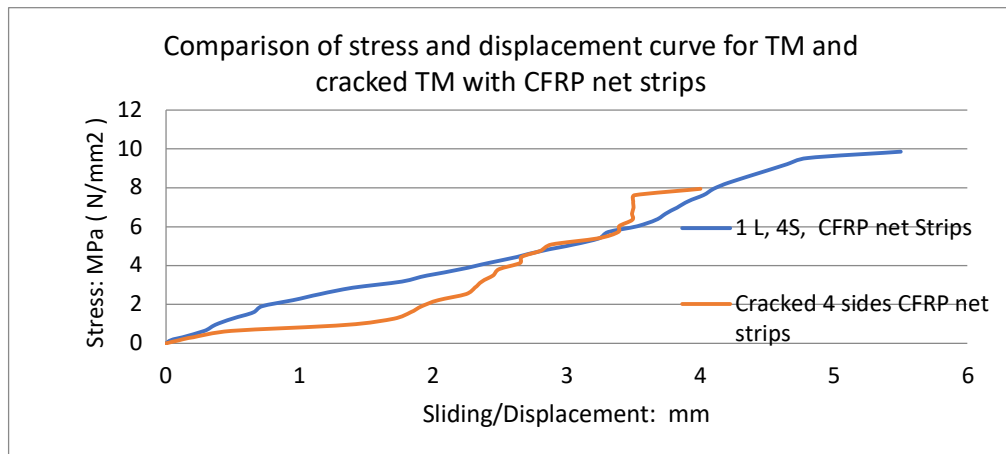


Figure 5.47: Comparison of Stress and Sliding curve for TM and Cracked TM with CFRP Net strips

5.16. New equations for maximum shear strength of reinforced cracked triplet masonry with CFRP

The maximum shear strength of reinforced cracked triplet masonry for all CFRP configurations can be calculated as the sum of the resistance provided by brick–mortar joint and CFRP as stated in Equation (4.5) in chapter four:

$$S_m = S_{m0} + S_{frp} \quad (4.5)$$

where:

S_m = Shear strength of CFRP reinforced masonry.

S_{m0} = shear resistance URTM (for cracked triplet masonry, assumed $S_{m0}=0$).

S_{frp} = resistance obtained by FRP composites.

For normal triplet masonry S_{m0} is available in design guidelines such as IS: 1905 (BIS 2002a). or can be obtained from the experimental shear tests.

$$S_{m0} = \tau_{max}$$

The expressions that calculate S_{frp} need to be developed. The sliding of the middle brick during the loading will produce tensile stress in FRP composites and the compressive stress on the bed joint keeps increasing with increases in sliding of the middle brick until the specimen reaches failure either due to de-bonding or due to rupture of the FRP composite. Thus, the maximum shear strength of CFRP could be expressed as Equation (5.1).

$$S_{frp} = \rho \times (\sigma_{frp} \times (A_f/A_t) \times K \times \mu_f) \quad (5.1)$$

where:

$\rho = 0.70$ for CFRP sheets in cracked masonry triplets

$\rho = 1.2$ for CFRP strips in cracked masonry triplets

σ_{frp} , is the tensile strength of the CFRP which, is given by the factory.

The average tensile strength of the CFRP and CFRP net is 3500 Mpa.

S_{frp} : is the maximum shear strength of masonry reinforced with CFRP

A_t is the area of triplet masonry

A_f is the area of Triplet masonry covered by CFRP

$K = 1$, for one layer of CFRP on two sides

$K = 0.7$, for one layer of CFRP on four sides

μf is the coefficient for CFRP

$\mu f = 0.007$ for CFRP

Table 5.1 shows the comparison of the maximum shear strength bond of different configurations of experimental triplet masonry strengthened with CFRP and the results from equations 5.1:

Test	CFRP configurations on triplet masonry	A_f/A_t	$S_{frp} = (\sigma_{frp} \times A_f/A_t \times K \times \mu f) \times 0.80$ $S_m = S_{m0} + S_{frp}$ (MPa)	Experimental shear tests (MPa)	Error: %
1	One-layer CFRP on two sides	0.51	8.7	8	8
2	One-layer CFRP on four sides	0.75 0.7	9	9	0
3	One-layer CFRP strips on two sides	0.23	6.76	6	10
4	One-layer CFRP strips on four sides	0.35 0.7	7.2	6.5	9
5	One-layer CFRP net on two sides	0.51	8.7	7.6	12
6	One-layer CFRP net on four sides	0.75 0.7	9	7.9	12
7	One-layer CFRP net strips on two sides	0.23	6.76	7.5	9
8	One-layer CFRP net strips on four sides	0.35 0.7	7.2	8	10
	$S_{m0} = \tau_{max} = 0$				

Table.5.1: Triplet masonry strengthened with CFRP and the results from Equations 5.1.

Figure 5.48 also, illustrate the comparison of the maximum shear strength of different configuration of experimental cracked triplet masonry strengthened with CFRP and the results from equation 5.1. The compared results for all CFRP configurations show this equation is reliable and the errors are between 0% to 12%.

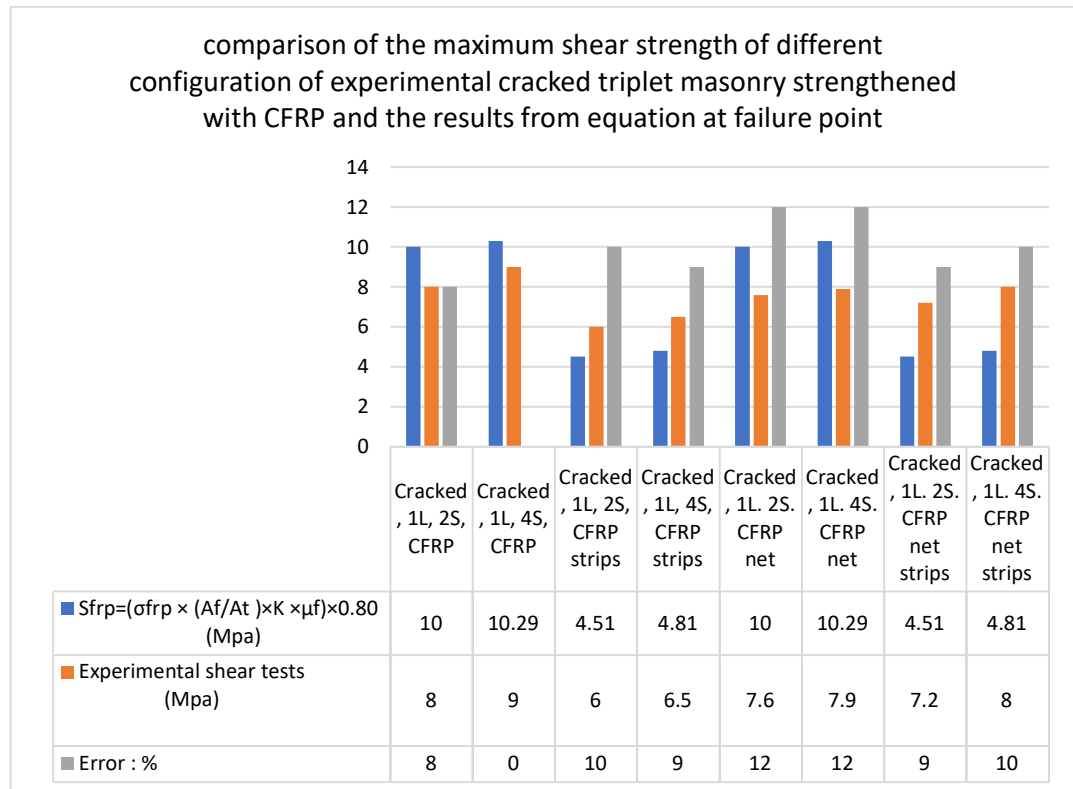


Figure 5.48: Comparison of the maximum shear strength with different CFRP configurations in experimental tests and the results from equation 5.1.

5.17. Cracked triplet's conclusions

Eight different configurations of CFRP overlays were used to repair masonry triplets with in-plane shear cracks. The samples were tested under shear loading and the test results were compared to the responses of triplets with no initial damage and strengthened with CFRP. It was found that the increase of shear strength and maximum displacement at the failure of repaired samples almost is similar to the triplets with no initial damage and with the same amount of CFRP overlays.

The results for cracked triplets are summarized as follows:

1. The triplet samples repaired with CFRP and tested under shear loading showed significant improvement in bond characteristics, strength, and ductility. The test load capacity of triplet masonry increased by 3 to 4 times in comparison with samples without initial damage
2. Repaired cracked triplets can maintain resistance up to similar sliding values as undamaged strengthened walls. Also, the highest ductility in terms of maximum deflection at ultimate load is observed for all samples.
3. The shear strength of cracked triplets strengthened with the same amount in terms of area reinforced with CFRP and CFRP net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity. However, the gain in shear strength increased with an increasing amount of CFRP.
4. The influence of the CFRP was observed on deformational characteristics of masonry triplets such as ultimate displacement and the initial slip of the load-displacement curve. Ultimate displacement and the initial slip of the load-displacement response of triplets with higher CFRP sheets were lower than triplets with lower CFRP strips.

5. De-bonding failure pattern was observed in triplet specimens glued with CFRP composites in full and strip configuration. However, tearing of FRP composites was observed in the case of triplet specimens glued with lower FRP composites
6. Remarkable increases in shear capacity and ductility and depending on the dimensions and orientation, retrofitting with CFRP increases ductility under in-plane shear prevent sliding, and increases the ultimate strength.

Chapter 6

Finite element simulation of triplet masonry

6.1. Introduction

Unreinforced masonry (URM) is a construction of clay brick, concrete block, or stone units bound together with lime or cement mortar. Due to the nonhomogeneous nature and different mechanical properties of each constituent material, masonry walls exhibit highly nonlinear behavior and various failure mechanisms. Its mechanical behavior is extremely complex and developing an accurate and comprehensive numerical model to evaluate the structural response of the masonry walls is still a challenging task for structural engineers. URM structures are widely used in the world and it is important to develop a numerical model that reflects the actual behavior of masonry in an accurate and comprehensive way (D'Altri et al. 2020).

Mohammadipour and Willam, (2016) studied the brick–mortar interface, which is the weakest part of the masonry composite and plays an important role in the failure of this assemblage. Interface cohesion and friction are important in many areas such as masonry constructions. Finite element modeling is needed to help the analysis and design of large-scale masonry structures. Traditionally, material interface formulations exhibit tensile de-bonding and shear sliding, but they normally disregard the action of the interface surface layer. Whereas in many applications this approach may be adequate, progressive loss of interface bond can only be explained properly by considering all details including the effects of material mismatch and the surface roughness and chemical bond between the materials. This requires interface studies and materials modeling for the prediction of failure, de-bonding in tension, and delamination in shear load, which still missing.

There have been significant advances in numerical strategies developed to simulate the nonlinear behavior of masonry structures in recent years, and studies are ongoing for further development. In general, numerical modeling of masonry structures can be classified into two main groups; macro and micro modeling. Determining which modeling technique will be used in the analysis depends upon the accuracy and precision desired from the simulation (Lourenço 1996).

According to (Ravi et al, 2014), the Numerical investigation can provide detailed information about the behavior of masonry, which can be compared with the actual behavior of masonry structures. The analysis of unreinforced brick masonry can be

performed using the finite element method. The masonry can be numerically studied by developing a complete model with appropriate boundary conditions.

Lourenço et al., (1997) developed a plane stress model to describe the inelastic behavior of quasi-brittle orthotropic materials. Orthotropic material behavior was defined as a Hill-type criterion for compression and a Rankine-type criterion for tension. Internal damage due to these failure mechanisms was represented with two internal parameters, one for damage in tension and one for damage in compression. Comparison between calculated and experimental results of masonry shear walls showed good agreement.

Koksal et al. (2016) presented a new approach to nonlinear finite element modeling of masonry walls subjected to a combination of vertical and lateral loads. Two different experimental studies on the masonry walls with dry and mortar joints were used to validate the proposed approach for masonry wall simulations.

According to Doran et al., (2022), the micro-modeling approach can be studied in two parts; detailed micro-modeling and simplified micro-modeling. In detailed micro modeling, masonry bricks, and mortar and the interaction between brick-mortar interface are modeled separately by adopting appropriate constitutive laws for each component. While brick and mortar are modeled with continuum elements, discontinuous interface elements are used in bed and head joints to represent the brick–mortar interactions.

Parisi et al., (2015) studied the micromechanical model of tuff stone masonry by considering a pressure-dependent failure rule for each component. The numerical simulations were performed to predict the nonlinear response of tuff stone masonry subjected to different boundary conditions and load patterns. The micromechanical FE model reproduced the experimental behavior and observed damage satisfactorily.

Drougkas et al., (2016) presented the experimental results and detailed micro-modeling simulation of a historic masonry pillar in compression. Elastic modulus, compressive strength, and failure mode obtained from numerical simulation are compared to the experimental outcomes. In conclusion, the simulation of phenomena related to masonry compressive effects using micro-models is much more accurate in terms of damage patterns and their development.

AlGohi et al. (2017) studied the lateral response of URM walls subjected to different levels of axial load, numerically and experimentally. The concrete damage plasticity

constitutive model was used in numerical simulations in ABAQUS software. According to the results, the plastic-damage constitutive model could predict the appropriate failure mode for URM walls.

D'Altri et al. (2018) proposed a novel 3D detailed micro-model, which is characterized by textured units, consisting of one brick and mortar layer, to represent the behavior of masonry walls subjected to in-plane and out-of-plane loading conditions.

Simplified micro-modeling was developed to eliminate the aforementioned disadvantages of detailed micro-modeling. In this technique, the masonry unit is modeled by expanding half of the mortar thickness in all directions to ensure the overall structural geometry of the masonry wall is unchanged. While the masonry unit is represented by continuum elements, the nonlinear behavior of mortar joints and interaction between unit mortar are lumped into a discontinuous interface element. It is widely used in the numerical analysis of masonry due to its simplicity and a significant reduction in analysis time compared to detailed micro-modeling (Doran et al., 2022).

Bolhassani et al. (2015) performed the simplified micro-modeling of assemblages. Mortar joints and units were combined into a homogeneous material and modeled using the concrete damage plasticity constitutive model. Cohesive surface-based behavior was employed to model the mortar joints. It is suggested that the results from the simulation are generally in good agreement with the behavior observed in the experiments for the hollow and grouted prisms, shear, and diagonal tension assemblages.

Abdulla et al., (2017) proposed a simplified micro-model approach utilizing surface-based cohesive behavior to capture the elastic and plastic behavior of masonry joints and a Drucker Prager (DP) plasticity model to represent masonry units to simulate nonlinear behavior and failure modes of masonry under in-plane and out-of-plane loading conditions.

Doran et al. (2017) investigated the effects of different types of lime mortars on masonry shear walls under compression and shear conditions. A new fabricated joint material approach was proposed to describe the constitutive behavior of both mortar and mortar–brick interface. Doran et al. (2020) studied the grout injection strengthening technique for the structural rehabilitation of traditional URM walls. A simplified micro-modeling strategy based on the isotropic damage model is presented. The numerical results were compared with the test results of four test walls to evaluate the accuracy of

the proposed methodology. Interface elements are used to describe the interaction between brick-mortar and brick-brick in micro modeling. Brick-mortar interface interaction has a crucial role in the nonlinear analysis of masonry structures. By using the interface elements in the micro and simplified micro models, the potential failure mechanisms inside the mortar joint such as brick/mortar decohesion and shear cracking across the mortar layer could be captured. For this reason, interface elements have a broad application in the analysis of masonry structures, and ongoing studies are still being carried out for their development. The first attempt to implement interface elements in masonry walls was performed by Page (1978).

Sacco and Toti (2010) presented a special interface model combining damage and friction to reflect the mortar-brick decohesion mechanisms. To perform simple structural analysis, a cohesive zone model implemented in finite element code was used as a constitutive model for interface elements. As a result of the study, the ability of the proposed model to simulate the behavior of the unreinforced and reinforced masonry arches in terms of ultimate load, nonlinear behavior, and collapse mechanism is presented. The results of the finite element show the accuracy and convergence of the numerical procedure.

According to Hamrat et al, (2019), the repair and strengthening of damaged or undamaged structural members with externally bonded fiber-reinforced polymer (CFRP) sheets have become one of the important challenges confronting civil engineers and researchers. CFRP composite systems when bonded to various damaged or undamaged structural members can significantly improve their mechanical behavior in serviceability conditions, at failure mechanism, and reduce crack propagation by increasing both the stiffness and the load capacity resistance.

The present study aims to explore the effectiveness and reliability of the modeling strategies to reflect the experimental behavior of URM triplets with and without CFRP under in-plane loading. This study mainly focuses on choosing an accurate finite element modeling approach to represent the behavior of URM triplets in terms of load-displacement responses and failure patterns. The commercial finite element software (ANSYS) was used to perform the three-dimensional nonlinear finite element analysis. Validation of the modeling strategies has been presented, and a comparison against

experimental results has been discussed to evaluate the capabilities and shortcomings of the modeling approaches.

6.2. Micro and Macro-modelling

Lourenco, et al., (1996) conducted those numerical simulations are fundamental to understanding the structural behavior and support the derivation of logical design rules but nonlinear finite element analyses will be always helpful for the validation of the design of complex masonry structures under complex loading conditions. Aside from failure analysis, also the serviceability limit states can be successfully validated with numerical analyses. Another important aspect is the safety of existing structures under existing or new loading conditions. Reliable numerical models are necessary to assess and strengthen existing masonry structures. According to (Lourenco, et al., 1996), masonry is a composite material that consists of units and mortar joints Fig. 6.1a. A comprehensive analysis of masonry must include a representation of units, mortar, and the unit/mortar interface, Fig. 6.1b. In this case, units and mortar are represented by continuum elements whereas the unit-mortar interface is represented by discontinuous elements. The Young's modulus, Poisson's ratio, and optionally, inelastic properties of both unit and mortar are taken into account. The interface represents a possible crack or slip. A simplified micro-modeling of masonry will be preferably used here, see Fig. 6.1c. (Lourenco, et al., 1996).

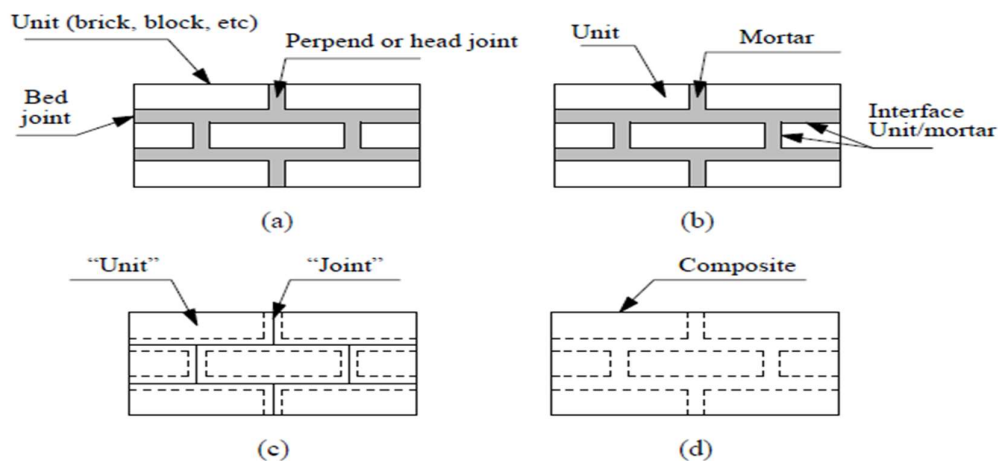


Fig. 6.1. Modelling strategies for masonry structures: (a) masonry sample; (b) detailed micro-modelling; (c) simplified micro-modelling; (d) macro-modelling. (B. Lourenco, et al. 1996).

Micro-modeling methods are good for small structural components with a particular interest in strongly heterogeneous conditions of stress and strain.

Several attempts to use interfaces for the modeling of masonry were carried out in the last decade with reasonably simple models. In the big structure analyses, the knowledge of the contact between units and mortar is, generally, negligible for the global structural behavior. In these cases, a different approach can be used, such as macro-modeling, where a distinction between individual units and joints is not made, see Fig. 6.1d. Instead, the material is considered an anisotropic composite, and a relation is established between average masonry strains and average masonry stresses (Lourenco, et al.1996).

6.3. Micro-modelling

It is noted that one modelling strategy cannot be promoted over the other because different application fields exist for micro- and macro-models. Micro-modelling studies are necessary to give a better understanding about the local behavior of masonry structures.

According to Lourenco, et al. (1996), the micro-models are, probably, the best tool available to understand the behaviour of masonry. The benefit of using this method is that all the different failure mechanisms can be considered such as, (a) cracking and sliding along the bed or head joints, (b) cracking of the units in direct tension, (c) diagonal tension cracking of the units, (d) masonry crushing, commonly identified with the splitting of units in tension as a result of mortar at high values of normal stress, as illustrated in Fig. 6.2. It is clear from the described phenomena that (a and b) are joint mechanisms, (c) is a unit mechanism and (d and e) are combined mechanisms involving units and joints. (Lourenco, et al.1996).

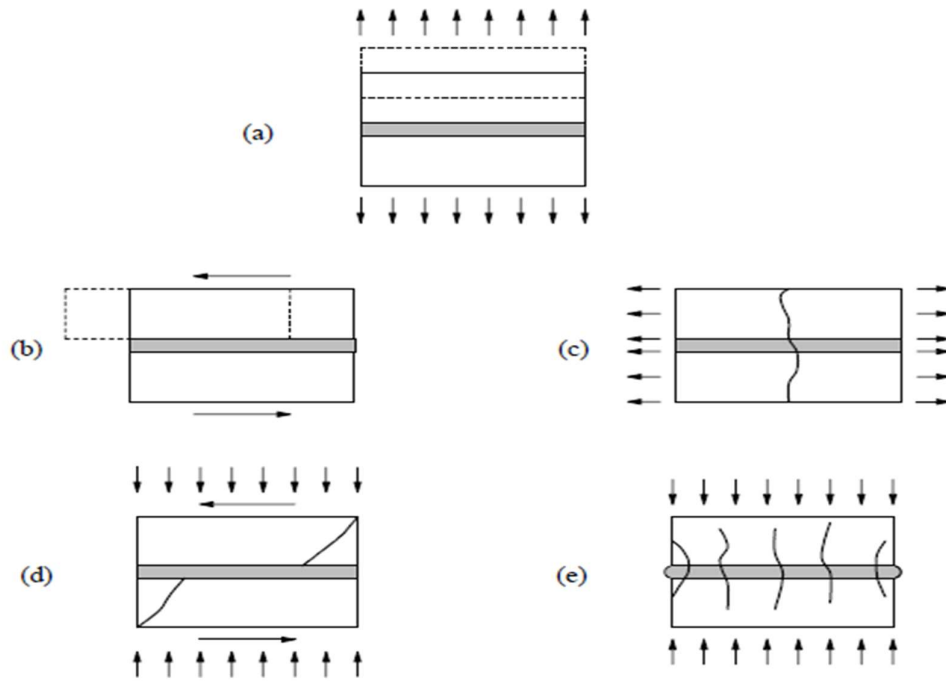


Fig. 6.2. Masonry failure mechanisms: (a) joint tension cracking; (b) joint slipping; (c) unit direct tension cracking; (d) unit diagonal tension cracking; (e) masonry crushing. (Lourenco, et al.1996).

6.4. Macro-modelling

The investigation of masonry structures with a large number of units and joints can only be carried out with macro models. The effective fundamental behaviour of masonry features anisotropy arising from the geometrical arrangement of units and mortar, even if the properties of these constituents are isotropic. The modern plasticity framework is enough to apply equally well to both isotropic and anisotropic plasticity models. An accurate analysis of masonry structures from a macro-modelling (or composite) perspective requires a material description for all stress states. (Lourenco, et al.1996).

According to (Lourenço 1996), in macro modelling, masonry is regarded as a homogeneous continuum without any distinction between units, mortar, and unit–mortar interface. The macro modelling approach offers great advantages in terms of computational cost and analysis time. Despite these benefits, it is unable to provide an accurate prediction about the evolution of damage and collapse mechanisms at the local level in the masonry structure.

Accurate micro or macro modelling of masonry structures requires a detailed experimental material test. However, the properties of masonry are influenced by many

factors, such as material properties of the units and mortar, arrangement of bed joints, dimension of units, joint width, quality of workmanship, environment and age.

6.5. Modelling by ANSYS and verification against results from experimental studies in this research

The behaviour of masonry walls depends upon the characteristics of brick and mortar. In this chapter, the compression and shear behaviour of FRP-reinforced triplet masonry was studied analytically by using and developing the ANSYS software. Ansys allows more meshing options for finite element (FE) modelling and is aware of engineering units such as inches and millimetres so it can accept input in a mixture of inconsistent units and give output expressed with similar units. Other programs such as universal distinct element code (UDEC), 3DEC, ADINA, Abaqus and Midas, are not aware of engineering units. This can be more flexible, allowing unusual units but also involves doing a lot of conversions by hand and has the risk of choosing the wrong units. In addition, Ansys has two modes for FE modelling, Ansys APDL mechanical which is a best-in-class finite element solver with structural, thermal, acoustics, transient and nonlinear capabilities to improve FE modelling and Ansys workbench offers better Graphic User Interface (GUI) and allows importing files from CAD or APDL.

Different CFRP-reinforced specimens were analysed and the behaviour of these samples was compared with results from the experimental tests.

The assumptions during analytical modelling:

1. Masonry samples and CFRP composites follow the elastic theory and the load-deflection relationship is non-linear and elastic.
2. The cross-sectional area of the samples remains constant during loading.
3. Masonry samples are considered to be made up of a non-homogeneous and isotropic material.

In this study, non-homogeneous modelling and non-linear elastic behaviour are assumed for the URM triplets. The compression and shear behaviour of strengthened masonry can be compared together especially having in mind that the displacement of the triplet masonry is one of the main objectives of this research.

6.6. Finite element analysis of triplet masonry under compression test

The micro and macro modelling help the numerical study of compressive strength tests on triplet masonry samples under different regular compressive loads. Two different models were created using a commercially available finite element analysis package ANSYS. Overall, there are two methods accepted for masonry modelling, macro-modelling and micro-modelling. The macro-modelling approach does not make a distinction between individual brick units and joints but only treats masonry as a homogeneous anisotropic composite. The micro-modelling technique treats each component of masonry material separately with its specific constitutive law and failure criteria. Micro-modelling is often used for the detailed analysis of small structures to find stress and strain states. Also, the detailed brick-and-mortar interfaces can be studied in micro-modelling approaches. Interface elements are used to describe the interaction between brick-mortar and brick-brick in micro modelling. Brick-mortar interface interaction has a crucial role in the nonlinear analysis of masonry structures. By using the interface elements in the micro and simplified micro models, the potential failure mechanisms inside the mortar joint such as brick/mortar decohesion and shear cracking across the mortar layer could be captured. On the other hand, macro-modelling is appropriate for big structures where the interaction between brick-and-mortar joints is not significant.

6.7. Formulation of the models

Brickwork strength is reliant on the physical features of the masonry unit, the mortar and the bond among them. In the current study, models with two different materials are presented which are brick unit and mortar joints. Both materials are replaced with an equivalent material property, assuming it to be a homogenised material. In this method, the behaviour of the masonry is roughly approximated by linear elasticity and perfect interface bonding. The brick unit was modelled using solid 185. Solid 185 is used for the 3-D modelling of structures. It is defined by eight nodes. Each node has three degrees of freedom: translations in the nodal x, y, and z directions. The element has large deflection, large strain, plasticity, hyperelasticity, creep and stress stiffening capabilities. Deformations of nearly incompressible elastoplastic materials and fully

incompressible hyperelastic materials can also be simulated as it has mixed formulation capability. This is available as a homogenous structural solid element and layered structural solid element (Figure 6.3).

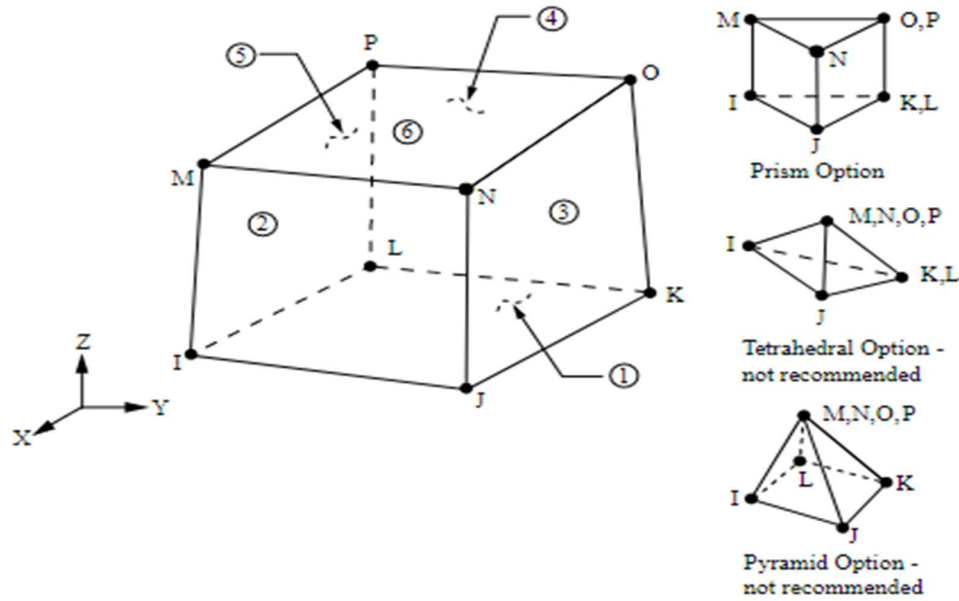


Figure 6.3: Solid 185 structural solid geometry (Ansys modelling guide, 2010)

The mortar joint was modeled using solid 65, solid 65 is used for 3-D modeling of solids with or without reinforcing bars (rebar). Solid 65 is capable of cracking in tension and crushing in compression. In concrete applications, for example. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. Up to three different rebar specifications may be defined. The concrete element is similar to a 3-D structural solid but with the addition of special cracking and crushing capabilities. The most important aspect of this element is the treatment of nonlinear material properties (Figure 6.4).

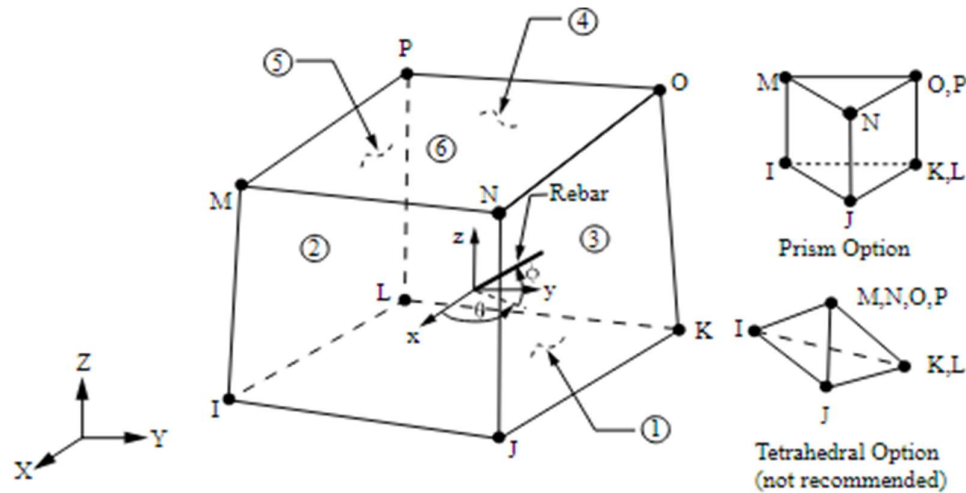


Figure 6.4: Solid 65 structural solid geometry (Ansys modelling guide, 2010)

6.8. Micro modelling of triplet masonry in this study

The bonded brick masonry prism was made with three solid clay London bricks having dimensions of 220 mm x 104mm x 65 mm in 10mm thick cement mortar having a ratio of 1:4, (M4). The bricks and the mortar joints had been represented by separate layers of elements with their properties and modulus of elasticity. The applied loading was on the top surface of the model. The brick and the mortar joint were modeled using the micro-modeling approach and assuming a perfect bond between the brick unit and the mortar joint.

6.9. Finite element analysis for compression tests on triple masonry by ANSYS

Nonlinear finite element analysis helps researchers to conduct more detailed investigations on the behaviour of masonry structural elements. Several three-dimensional finite element structural models are proposed in this chapter to predict the nonlinear behaviour of brick, mortar and masonry. Numerical simulations are carried out with finite element software ANSYS 22. Comparison of finite element compression

modelling and experimental compression tests show good agreement and finite element modelling are close to the experimental study.

6.10. Finite element modelling of triplet masonry

The modeling and prediction of the behavior of masonry are challenging work for researchers. Solid 65 element was used to model the masonry and this element has eight nodes with three degrees of freedom at each node. This element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. Solid 65 element requires linear isotropic and multi-linear isotropic material properties to properly model concrete as can be seen in Table 6.1.

	Density (kg/m ³)	E (N/mm ²)	Poisson's ratio
Brick unit (220×103×65) mm	2200	6520	0.20
Mortar, M4	1700	4005	0.30
Masonry triplet	2000	5015	0.20
CFRP	1800	240000	0.20

Table 6.1. Material properties for modelling

The mortar in triplet is modelled by SOLID 65 (CONCRET 65) finite elements nonlinear plasticity material model. Different constants for application of the Willam and Warnke's material model [5] in ANSYS are require (Table 6.2).

1	Shear transfer coefficient for an open crack (β_t)	0.4
2	Shear transfer coefficient for a close crack (β_c)	0.8
3	Uniaxial cracking stress (f_r)	2E+006Pa
4	Uniaxial crushing stress (f_c)	3.33E+007Pa

Table 6.2: Implementation of the Willam and Warnke material model

The value of β ranges from 0 to 1, with 0 representing a smooth crack (complete loss of shear transfer) and 1 representing a rough crack (no loss of shear transfer) (Kwan et al., 1999 and Terec et al., 2010).

6.11. Finite element simulations of masonry triplets under compression load

Different configurations of strengthened triplet masonry with CFRP and analysed by ANSYS in this research, are as follows:

- 1 - Reinforcement with one layer of CFRP wrapping on both sides
- 2 - Reinforcement with one layer of CFRP wrapping on four sides
- 3 - Reinforcement with one layer of CFRP strips on both sides
- 4 - Triplet constructed without any strengthening as control samples

Each test was repeated three times.

The masonry prisms were constructed with three clay solid London bricks having dimensions of 220 mm x 104 mm x 65 mm in 10mm thick cement mortar having a ratio of 1:4, (M4), Figure 6.5.

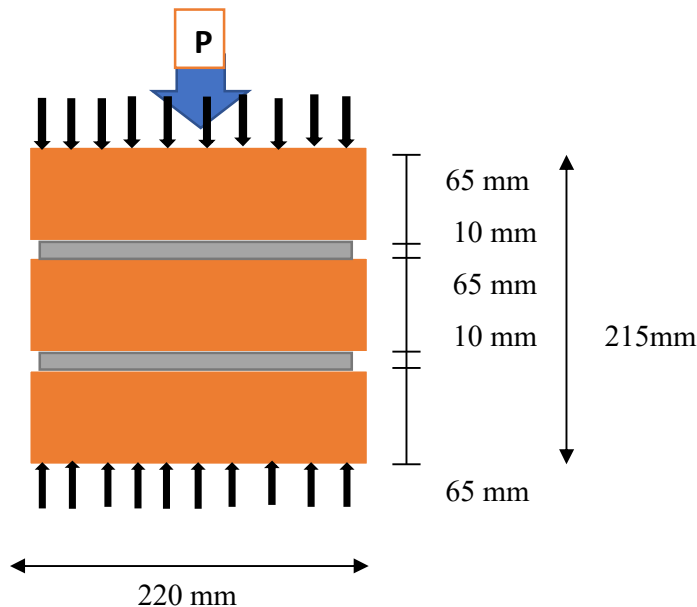


Figure 6.5. Triplet bricklayers

6.11.1. Loading and boundary conditions

In the micro model of shear triplets, the increasing load was applied at the top surface of the triplet and the bottom were assumed to be fixed with all degrees of freedom. The geometry and loading conditions for the triplets are shown in Fig 6.5. A perfect bond was assumed to exist between the triplet's surface and the CFRP composite. The interface between brick and mortar was modeled using surface to surface contact algorithm. The brick surface was considered as the target surface due to its higher modulus of elasticity and the mortar surface was considered as a contact surface.

6.12. Comparison of experimental compression test and finite modelling for unreinforced triplet masonry (URTM)

Figures 6.6. presents a comparison of the results for URTM (control samples) in experimental tests and numerical simulation for URTM. The results show good agreement and assuming linearity for this triplet within indicated limits of loading is realistic. The triplet mesh, boundary conditions, displacement and stress are shown in Figures 6.7, 6.8 and 6.9. As can be observed, the proposed numerical model adequately reproduces the average stiffness of the masonry triplet, and also, the first part of the damaged response. For a load value around 250 kN, a diminution of the masonry stiffness occurs due to the start of elements damages in brick-and-mortar interfaces, which is accurately reproduced by the numerical model. On the other hand, maximum loads cannot be reproduced with same precision, since the numerical model leads to a logical unstable calculation when damages in bricks and mortar are significant. In the numerical simulation of triplet brick prism under compression load, it is observed that the brick-mortar interface layers and the sides and corner regions are subjected to maximum stress as shown in Fig 6.9.

The numerical the numerical results are obtained from the micro modelling with the application of load in sub-steps. Similar way of loading (in sub-steps) is carried out, so that the variation in behaviour of brick masonry in each step while loading can be plotted.

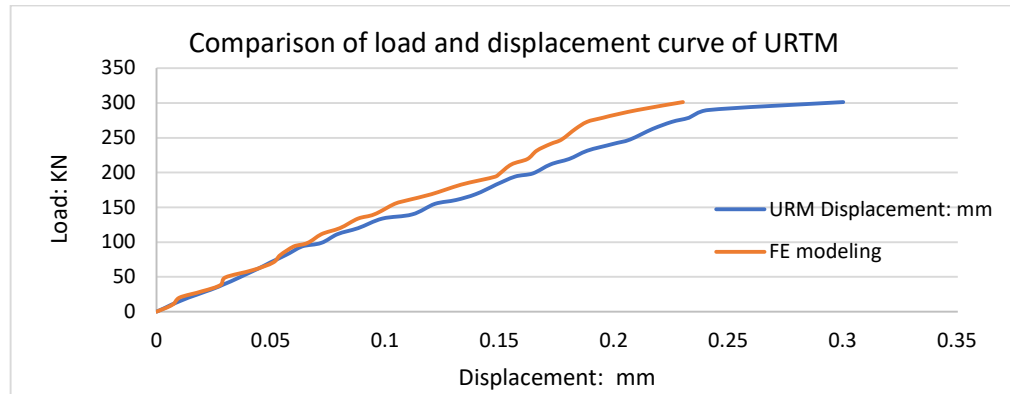


Figure 6.6.: Comparison of Load and displacement curve of URTM

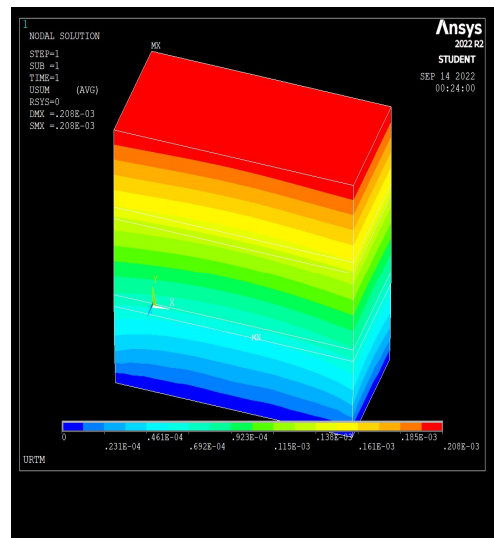
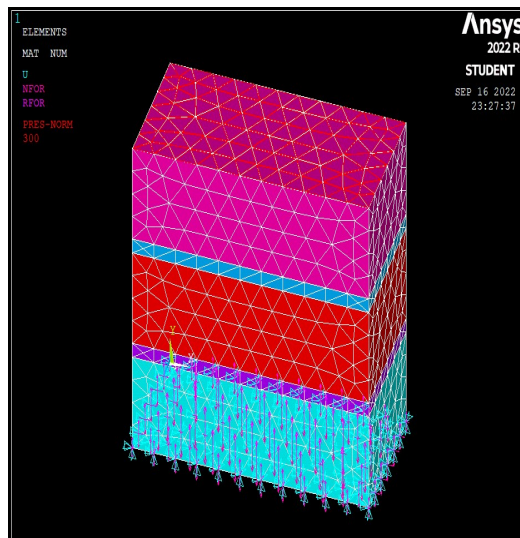


Figure 6.7: Triplet masonry mesh, boundary conditions Figure 6.8: Displacement at 250KN

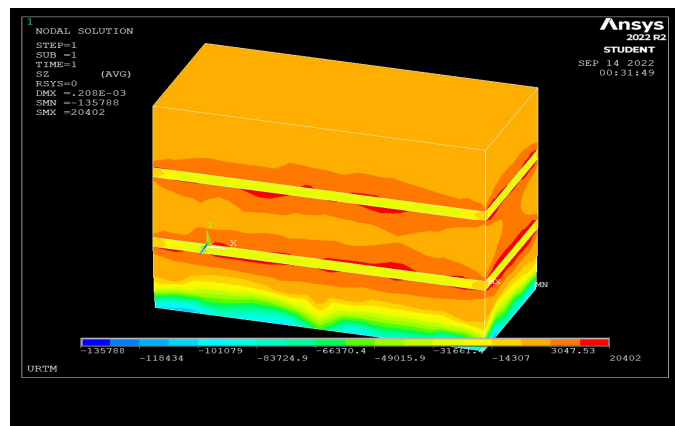


Figure 6.9: Stress distribution at 250KN

6.13: Analyses for one-layer CFRP on both sides of triplet masonry

Figures 6.10 compares the results for one layer of CFRP on both sides of triplet masonry in the experimental and theoretical models. The results show good compliance and displacement in theoretical modelling and the experimental test is nearly the same.

Figures 6.11 to 6.13 show the displacement and stress by FE modelling.

The numerically obtained results for the reinforced triplet with CFRP sheets at both sides and experimental compressions test are compared in Fig. 6.10. As can be observed, the proposed modelling provides a satisfactory result and show that the numerical approach adopted correctly reproduces the experimental behaviour in terms of maximum compression strength with CFRP sheets on both sides. The maximum load at the failure point was about 500KN. The crack was produced by debonding of the masonry units from the mortar and travelled through the triplet thickness until debonding of the epoxy from the masonry units. At this point, the triplet fails because the tensile stresses are no longer transferred to the CFRP. The FE modelling in Figure 6.13 shows maximum stress before failure in the masonry sample. The corresponding principal stress distribution shows that compressive crushing was started in brick units at the mortar region and CFRP experienced relatively larger stress, indicating the brick-and-mortar interface delamination. It was also observed that the triplets with CFRP strengthening gave more stiffness as compared to URTM.

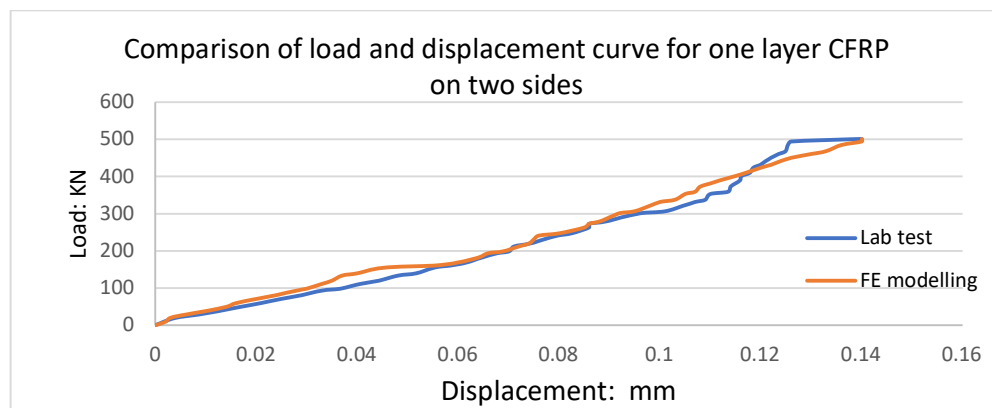


Figure 6.10: Comparison of load and displacement curve for CFRP at two sides

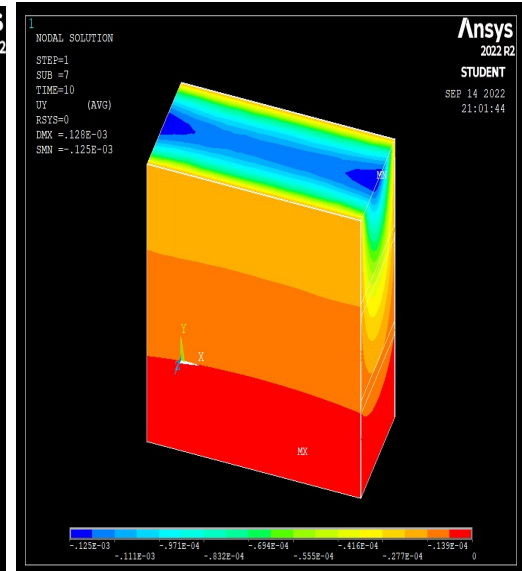
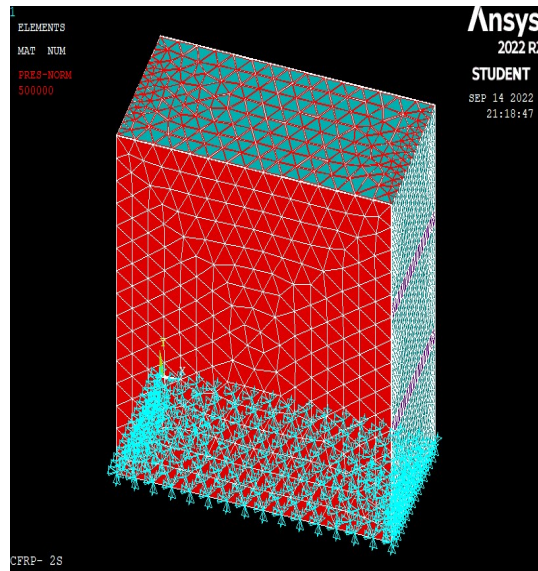


Figure 6.11: Triplet's mesh, boundary conditions

Figure 6.12: Displacement, triplet with CFRP on both sides

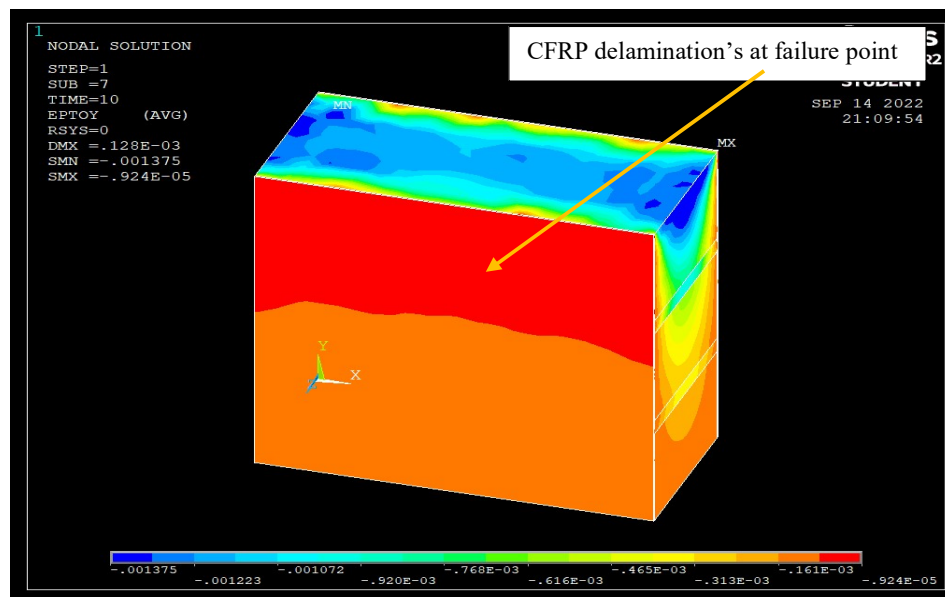


Figure 6.13: Compressive stress at 500KN load

6.14: Analyses for one-layer CFRP on four sides of triplet masonry.

Figure 6.14 presents the results for one layer of CFRP on four sides of triplet masonry in the experimental and theoretical models. The displacement of the theoretical modelling at 600KN load is slightly higher than the displacement of the experimental test, which is too small and can be neglected. The boundary condition and displacement of the FE modelling are shown in Figures 6.15 and 6.16. As can be observed, the proposed numerical model adequately reproduces the average stiffness of the masonry triplet, and also, the first part of the damaged response. For a load value of around 500 kN, a diminution of the masonry stiffness occurs due to the start of elements damages in brick-and-mortar interfaces, which is accurately reproduced by the numerical model. Figure 6.17 shows the maximum stress transferred from triplet to carbon fibre due to cracks in spacemen before failure. The numerical the numerical results are obtained from the micro modelling with the application of load in sub-steps. Similar way of loading (in sub-steps) is carried out, so that the variation in behaviour of brick masonry in each step while loading can be plotted.

The corresponding principal stress distribution shows that compressive crushing was started in brick units at the mortar region and CFRP experienced relatively larger stress, indicating the brick-and-mortar interface delamination. It was also observed that the triplets with CFRP strengthening gave more stiffness as compared to URTM.

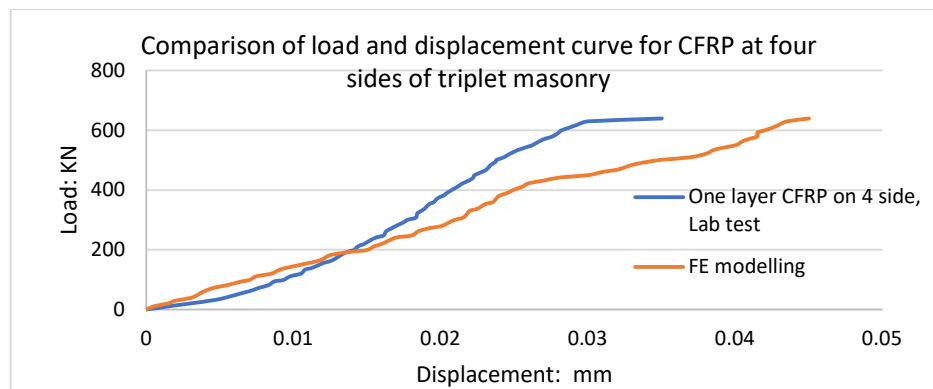


Figure 6.14: Comparison of load and displacement curve for CFRP at four sides of triplet masonry

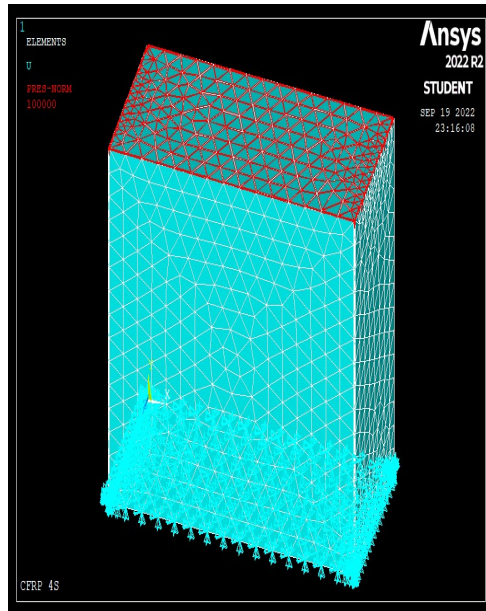


Figure 6.15: Triplet's mesh, boundary conditions

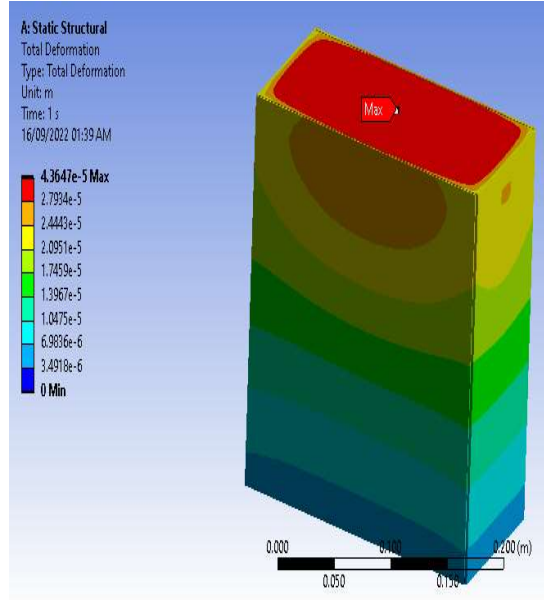


Figure 6.16: Displacement at 600KN for CFRP at four sides of triplet masonry

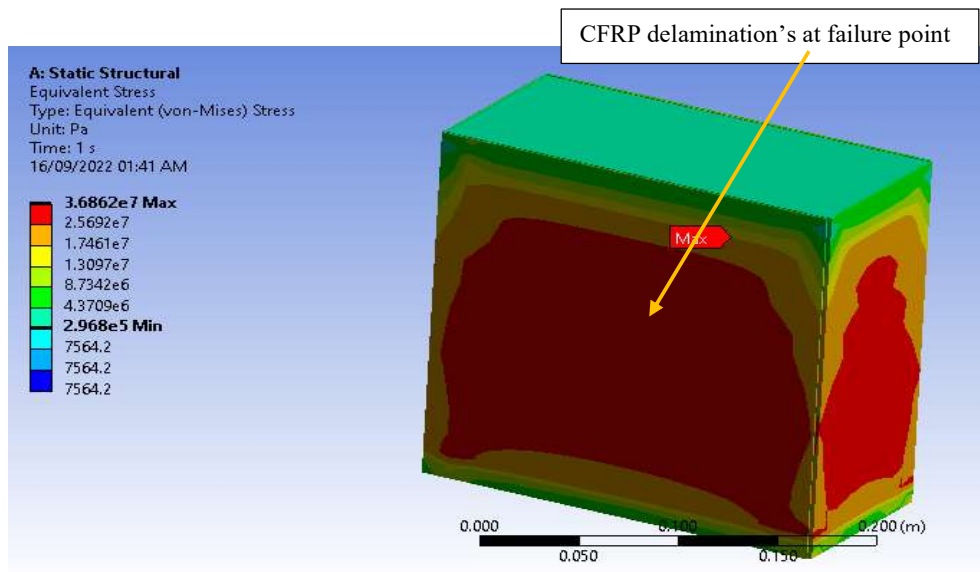


Figure 6.17: Maximum stress transferred to CFRP failure of triplet masonry

6.15: Analyses for one-layer CFRP strips on both sides of triplet masonry

Figures 6.18 illustrate a comparison of the results for CFRP strips at both sides of triplet masonry in experimental and theoretical FE modeling. The proposed modeling provides a satisfactory result and shows that the numerical approach adopted correctly reproduces the experimental behavior in terms of maximum compression strength with CFRP strips on both sides. The maximum load at the failure point was about 600KN.

The displacement of the theoretical modeling after 500KN loading is slightly higher than the displacement of the theoretical modeling, which can be neglected. Figures 6.19 and 6.20 show the applied load on top brick and displacement by Ansys. Figure 6.21 presents the transferred stresses from triplet to carbon fiber and CFRP delamination before failure. The corresponding principal stress distribution shows that compressive crushing was started in brick units at the mortar region and CFRP experienced relatively larger stress, indicating the brick-and-mortar interface delamination. It was also observed that the triplets with CFRP strengthening gave more stiffness as compared to URTM. The un-strengthened triplets showed brittle failure, while strengthening increased its deformation capacity.

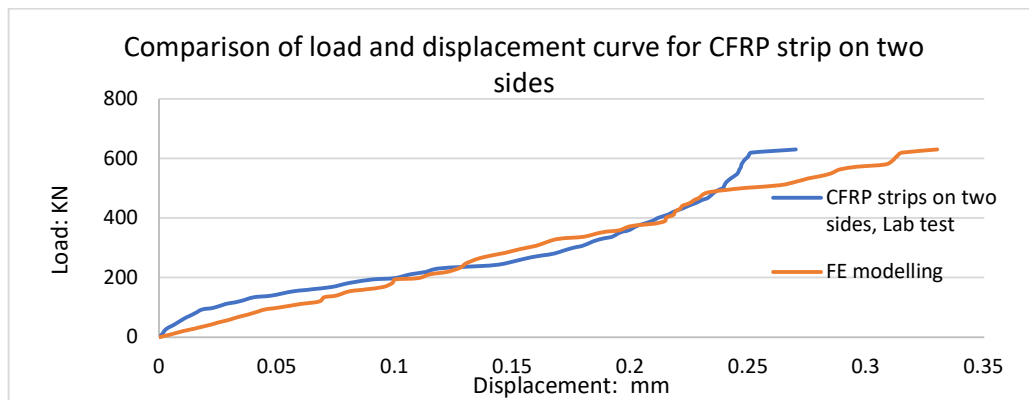


Figure 6.18: Comparison of Load and Displacement curve for CFRP strip at two sides

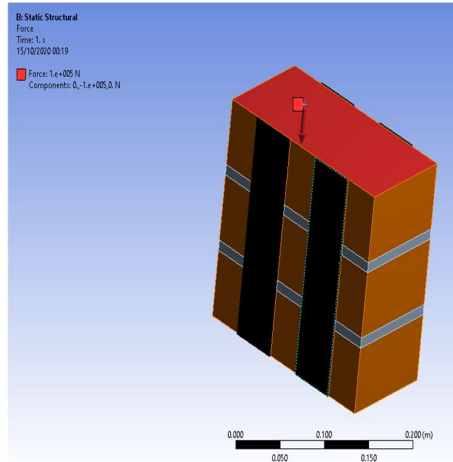


Figure 6.19: CFRP strips at both sides

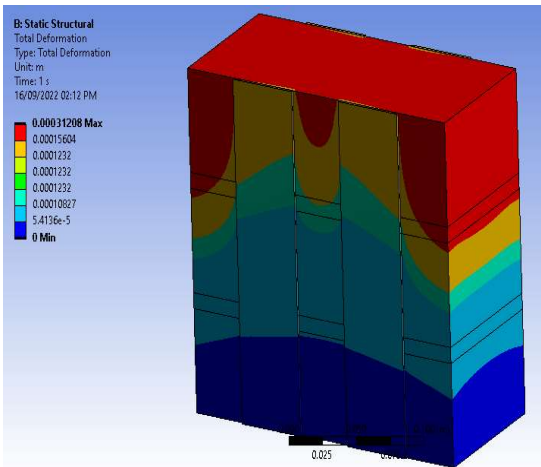


Figure 6.20: Displacement for CFRP strips at both sides of triplet masonry for 600 KN

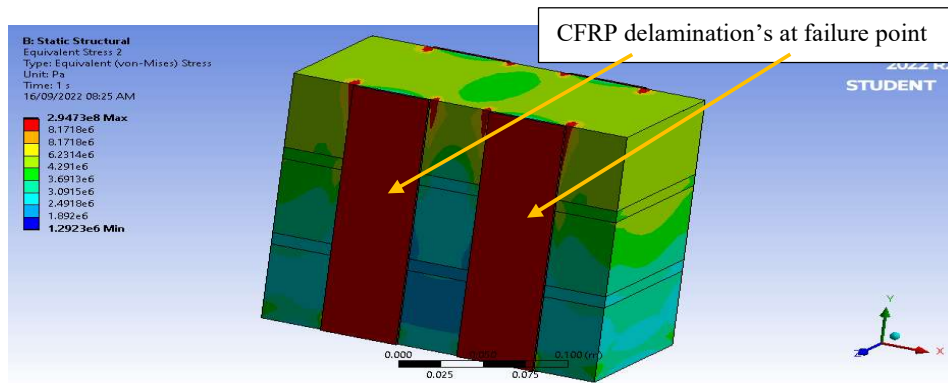


Figure 6.21: Stress for CFRP strips at both sides of triplet masonry

6.16. Finite element simulations of masonry triplets under shear loading and comparison with experimental tests

The shear strength of unreinforced masonry triplets depends on the strength of the mortar used in the triplet. In micro modelling, the bricks and mortar are modelled separately through their properties and contact between them. The purpose of micro-modelling is to show the behaviour of masonry from the information on unit properties and brick-mortar interfaces. The micro modelling plan is mainly suitable for small constructions where the contact between the units and mortar is the main interest.

According to Prakash and Alagusundaramoorthy, (2007), the micro modelling of triplet masonry helps to recognise the behaviour and shear transfer mechanism through the

mortar joint. The mechanism of shear transfer at a joint can be isolated into sliding, interlocking and overriding.

Rahman and Ueda (2016) conducted the in-plane shear strength of the strengthened walls is significantly improved by using the FRP and although the CFRP increases the shear capacity of a masonry wall.

6.16.1. Shear triplets load and boundary conditions

In the micro model of shear triplets, the increasing load was applied at the top surface of the middle brick and the bottom of the side bricks was assumed to be fixed with all the degrees of freedom arrested. The geometry and loading conditions for the triplets are shown in Fig 6.22. A good bond was assumed to exist between the triplet's surface and the CFRP composite. The interface between brick and mortar was modeled using surface to surface contact algorithm. The brick surface was considered as a target surface due to its higher modulus of elasticity and the mortar surface was considered as a contact surface.

6.16.2. Load deformation behaviour

The displacement in FE modelling was not close to the displacement in experimental tests. This error was corrected by developing a new user-defined function (UDF) in Ansys software.

The load resistance was constant initially before the cracking of the bed joint till 40 kN. After cracking through the bed joints, the CFRP started to resist the load. The ultimate load resistance from the finite element model was found to be different, depending on CFRP configurations. This is due to the perfect bond assumption made in the modelling between the triplet surface and the CFRP surface. The non-linear shear behaviour of triplets is mainly due to the surface interaction of brick and mortar.

The distribution of stress on the retrofitted specimens is shown in the figures for each CFRP configuration. The shear stress concentrations at the interface were reduced due to FRP retrofitting. This shows that the shear failure of masonry can be eliminated through retrofitting with CFRP composites. There was observed significant stress transferred to FRP around the cracked elements.

6.16.3. Effect of type and area of CFRP on triplet

This FE research demonstrates the increase in load resistance of triplets due to an increase in FRP composite. Experimental results showed that the load resistance of masonry triplets was increased due to retrofitting CFRP composites. In the case of shear triplets, CFRP retrofitting increased the strength and deformation capacity. The overall bond between the CFRP and triplet surface was intact up to failure.

In the present investigation, triplets modeling was conducted on shear triplet specimens under compression to find the shear strength of the brick masonry. The behavior of three shear triplet specimens retrofitted with CFRP composites with different configurations was also studied. The test results of the retrofitted triplets were used to validate the micro modelling.

The theoretical modelling consisted of nine different triplet masonry configurations with CFRP and CFRP net composites and they were analysed under shear loading.

- 1 - One Layer CFRP on Two sides of Triplet Masonry
- 2 - One Layer CFRP on Four sides of Triplet Masonry
- 3 - One Layer CFRP Strips on Two sides of Triplet Masonry
- 4 - One Layer CFRP Strips on Four sides of Triplet Masonry
- 5 - One Layer CFRP Net on Two sides of Triplet Masonry
- 6 - One Layer CFRP Net on Four sides of Triplet Masonry
- 7 - One Layer CFRP Net Strips on Two sides of Triplet Masonry
- 8 - One Layer CFRP Net Strips on Four sides of Triplet Masonry
- 9 - Un-reinforced triplet masonry (URTM)

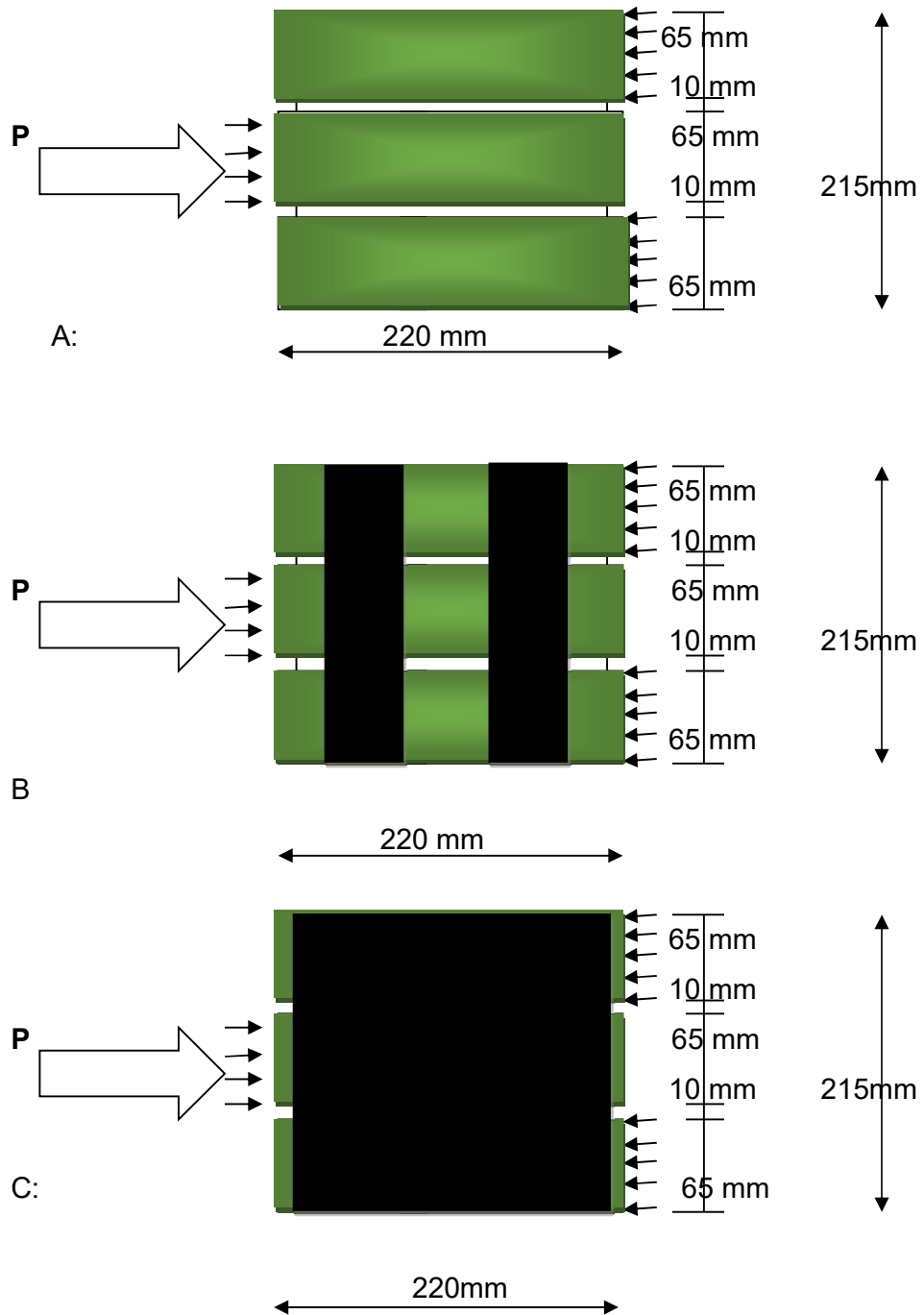


Figure 6.22. Modelling for shear analyses, A: Unreinforced masonry prism,
 B: Reinforced masonry prism with two vertical CFRP at both sides,
 C: Reinforced masonry prism (cover with CFRP at both sides).

6.17. Finite element analyses by ANSYS and UDF functions for un-reinforced and reinforced triplets

In this study, the theoretical modelling by Ansys was much lower than the experimental tests and not reliable. It is normal to find disagreement between analytical finite element modelling and experimental tests. The small difference can be neglected, but significantly high or low results from FE modelling compared to experimental tests should be addressed. In this study the comparison of the load-displacement graphs from the theoretical modelling with the results from the experimental tests showed significantly low displacement and were not accurate, thus the new user-defined functions for finite element modelling were developed.

The reason for lower sliding in FE modelling than the experimental tests is, the finite element program in Ansys assumes a perfect bond between brick and mortar. The perfect homogeneous material is the other assumption in finite element analysis. Therefore, this error can be corrected by using a newly developed formula as a user-defined function (UDF) to create correct sliding and 3D modelling for eight CFRP configurations as shown in table 6.3. By using the new functions, it is possible to predict correct sliding in 3D modelling at any load or stress point. These equations are based on quadratic equations, trial and error and verification against experimental tests. Also, these functions are useful for different types of brick and mortar as well. The UDF functions in this study are steps forward in finite element modelling. The findings of this study will directly benefit engineers to predict correct wall's displacement in case of shear loading.

In this function, σ_b (MPa) is bond stress and UX (mm) represents sliding, the software's user-defined function recognises UX as sliding.

	CFRP configurations on triplet masonry (one layer)	Equation number	Equations (UDF) (UX) = f(σ_b)
1	URTM	6.16.1	$(\sigma_b * UX)^2 - (15 - \sigma_b) * UX + (\sigma_b^2) = 0$
2	CFRP on two sides	6.16.2	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.45 * \sigma_b) = 0$
3	CFRP on four sides	6.16.3	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.25 * \sigma_b) = 0$
4	CFRP strips on two sides	6.16.4	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.55 * \sigma_b) = 0$
5	CFRP strips on four sides	6.16.5	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.40 * \sigma_b) = 0$
6	CFRP net on two sides	6.16.6	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.50 * \sigma_b) = 0$
7	CFRP net on four sides	6.16.7	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.35 * \sigma_b) = 0$
8	CFRP net strips on two sides	6.16.8	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.70 * \sigma_b) = 0$
9	CFRP net strips on four sides	6.16.9	$((\sigma_b) * (UX))^2 - (15 - \sigma_b) * UX + (0.50 * \sigma_b) = 0$
UX (mm) represent Sliding and σ_b (MPa) is Bond Stress			

Table 6.3: The newly developed formula as a user-defined function to create correct 3D modelling for theoretical analyses.

6.18. Finite element analyses for triplet masonry without CFRP (URTM)

The unreinforced masonry triplet is simulated under shear loading to investigate the shear behaviour of masonry structures. The ultimate load obtained from the experiment is applied as pressure on the prism. The material properties obtained in this research are used for brick and mortar in the numerical simulation. The displacement in FE modelling with Ansys was very low, therefore the user-defined function (UDF) was developed to obtain correct sliding. Figures 6.23 shows a comparison of the results for URM (control samples) in the experimental and theoretical models for URTM with and without UDF. Figure 6.23 shows 0.40 mm sliding by Ansys and with the new UDF formula, the sliding will be 2.5mm at 10KN loading, which is correct and reliable. Figure 6.24 illustrates the interfacial shear stress concentrations between the brick-and-mortar surfaces. The failure starts from the lateral sides of the interface layer

sweeping to the centre of the specimen. Figure 6.25 illustrate brick-and-mortar interfaces sliding. As can be observed in Figures 6.26 and 6.27, the proposed FE model adequately reproduces the stress between brick and mortar. The higher stress and cracks occur in a mortar with a lower modulus of elasticity than in a brick. The whole mortar experienced relatively larger shear deformation and indicating the interface delamination. The sliding result from Ansys software was very low as shown in figure 6.28 and the correct sliding UDF function can be seen in figure 6.29.

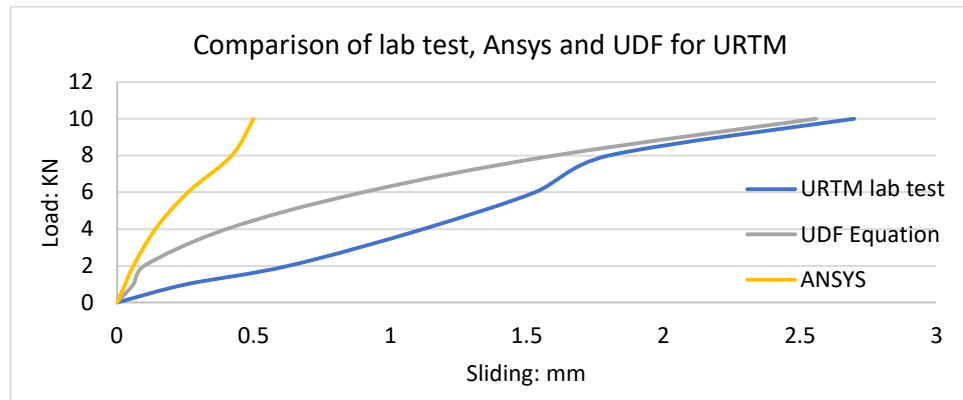


Figure 6.23: Comparison of lab test, Ansys and UDF for URTM

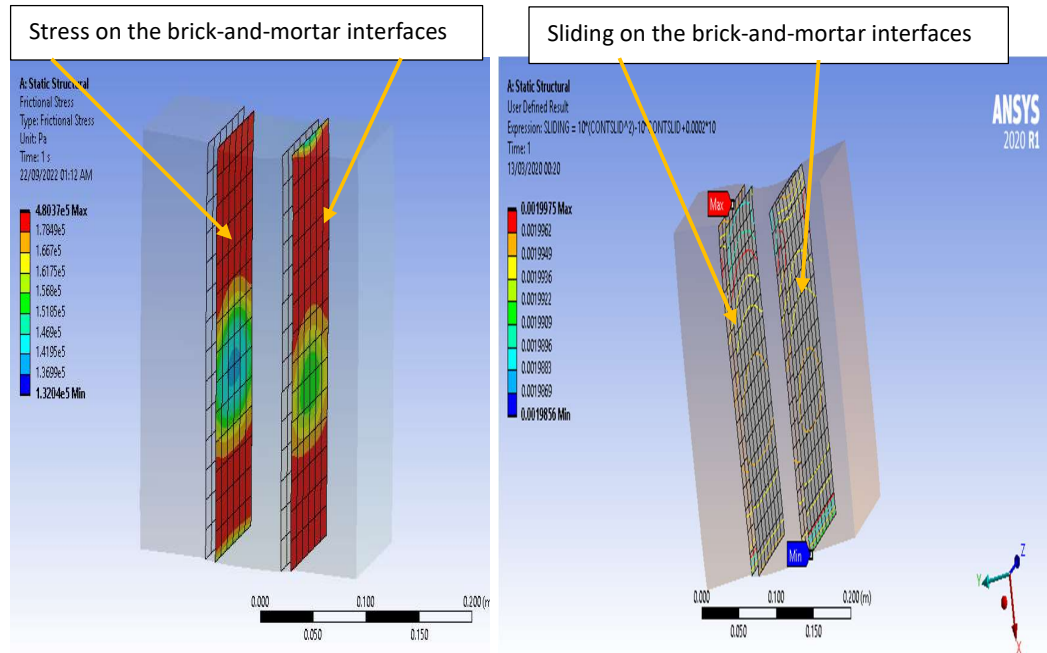


Figure 6.24: The bond stress from UDF function Figure 6.25: The interface sliding from UDF function.

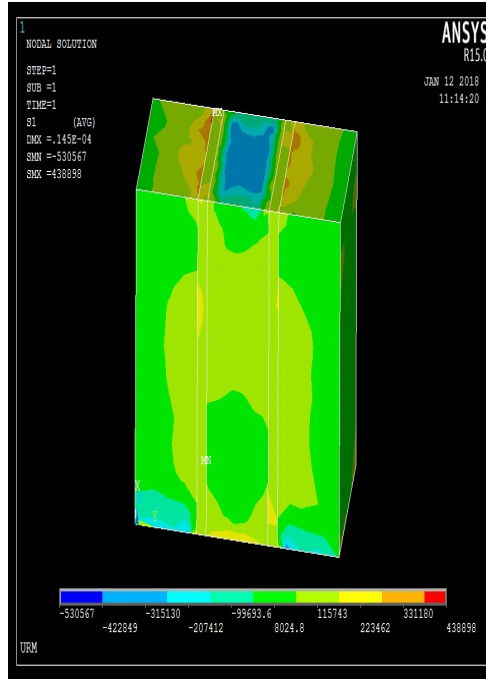


Figure 6.26: Stress without UDF function.

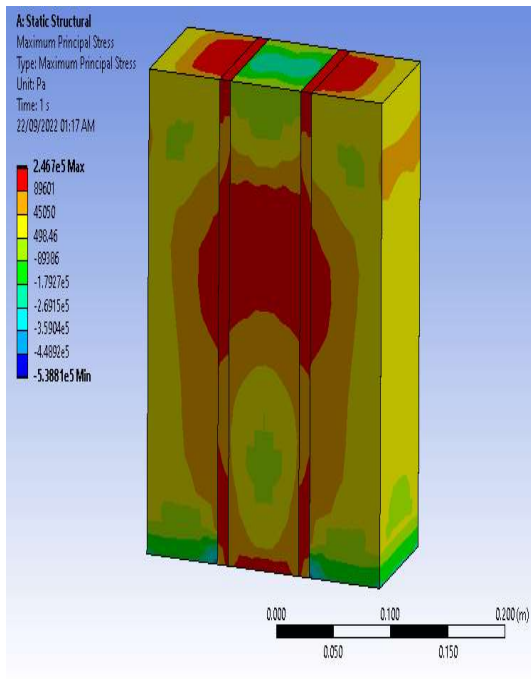


Figure 6.27: Stress from UDF function.

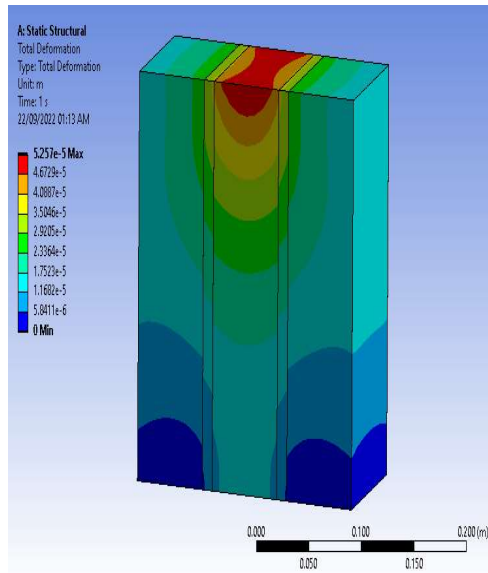


Figure 6.28: Sliding without UDF function.

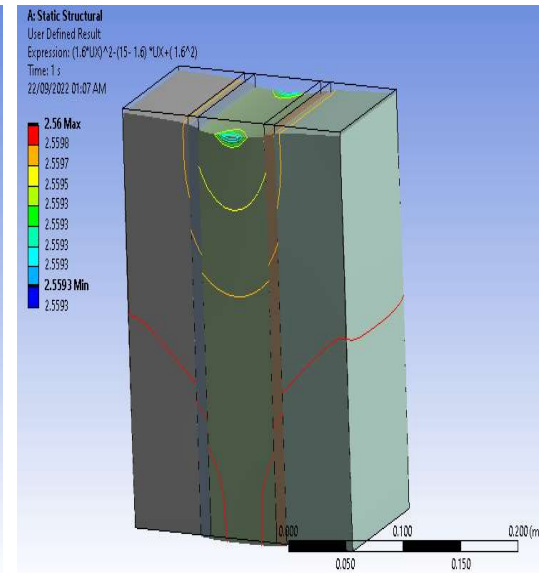


Figure 6.29: Sliding from UDF function.

6.19. Finite element analyses by ANSYS for one-layer CFRP on two sides of triplet masonry.

Figures 6.30 shows a comparison of the results for one-layer CFRP on two sides of triplet masonry in the experimental and theoretical models. The result in figure 6.32 for 90KN load shows 0.9 mm sliding/slipping by Ansys and with the new UDF formula, the sliding is 6.0 mm, as shown in Figure 6.33, which is a reliable answer as shown in Figure 6.30. Figure 6.35 illustrates the interface gap between brick-mortar and CFRP at the failure point. Figure 6.34 shows stress transfers from mortar and bricks to CFRP and the failure of all triplets starts from the brick-and-mortar interface layer sweeping to the CFRP on the specimen. The mechanics of interface cracks consider two isotropic elastic solids which are in contact through their interface. The finite element modeling for all triplets with CFRP showed that the interfacial shear stress concentrations between the brick-and-mortar surfaces were reduced due to retrofitting with CFRP composites and also showed that the stiffness and load resistance of CFRP retrofitted triplets increased linearly with the increase of CFRP composites. The FE results of masonry triplets show that the stress transfers to CFRP composites and makes the stronger bond between brick and mortar. Figure 6.36 shows the progressive crack propagation along with the interface where the crack length and delamination increase during a simulation.

In experimental tests and also in finite element analyses, debonding CFRP from triplets was the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. As shown in 6.34 stress transfers from mortar and bricks to CFRP and the failure of all triplets starts from the brick-and-mortar interface layer sweeping to the CFRP on the specimen. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets with sufficient accuracy.

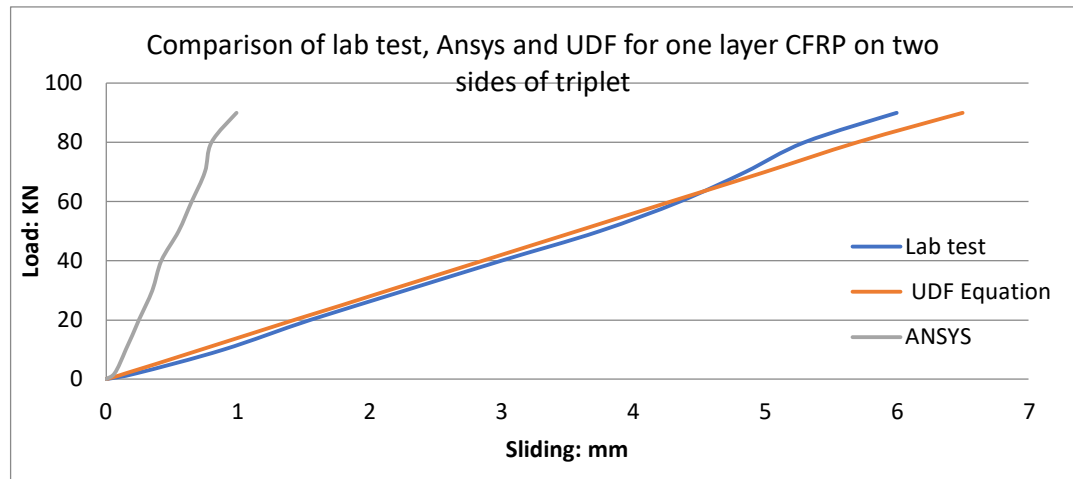


Figure 6.30: Comparison of lab test, Ansys WB and UDF for one-layer CFRP on two sides of triplet masonry

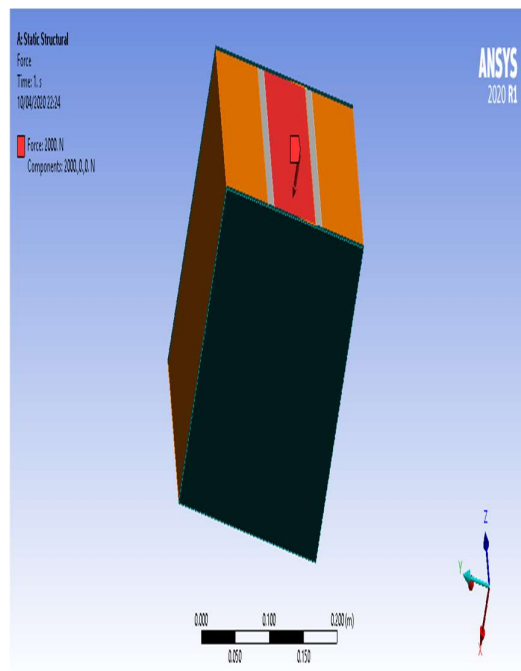


Figure 6.31: One-layer CFRP on two sides of triplet masonry

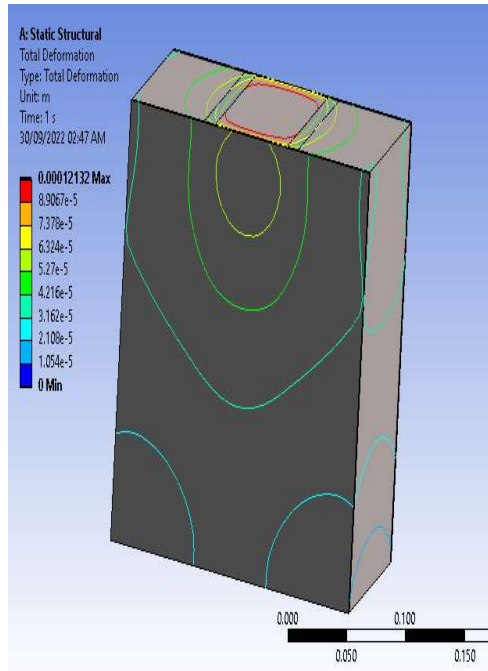


Figure 6.32: The sliding result from normal ANSYS software without UDF

CFRP delamination's at failure point

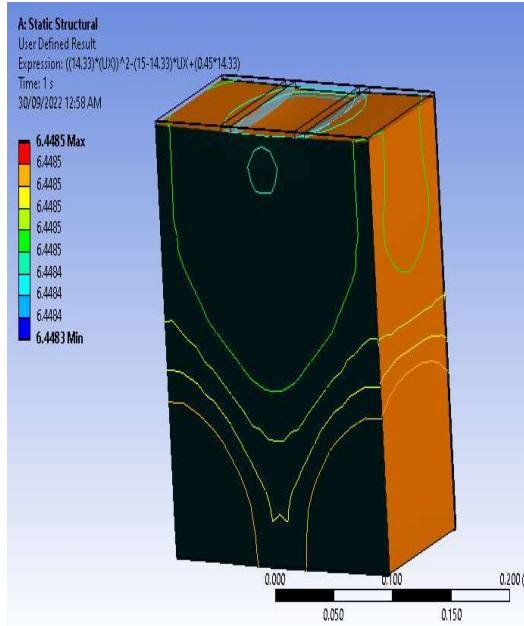


Figure 6.33: Sliding result from UDF Function.

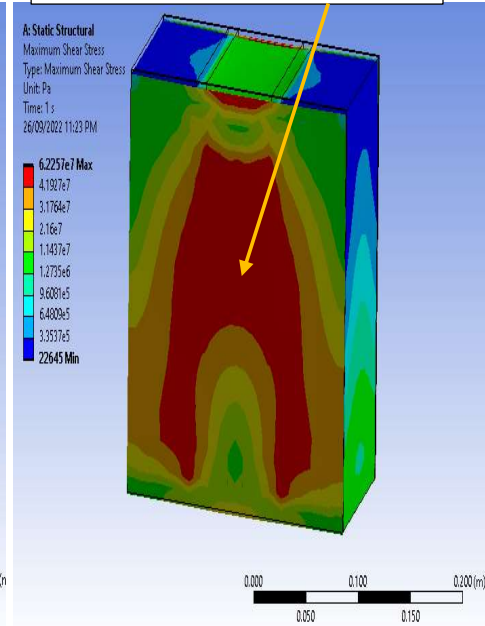


Figure 6.34: Stress result from UDF Function.

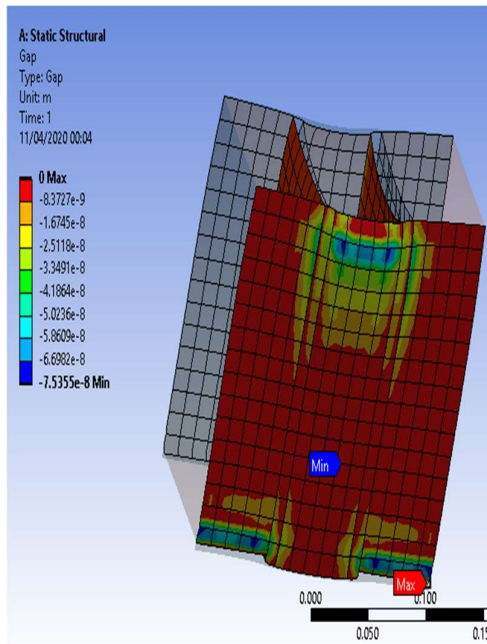


Figure 6.35: The gap between CFRP, mortar and brick at the failure point

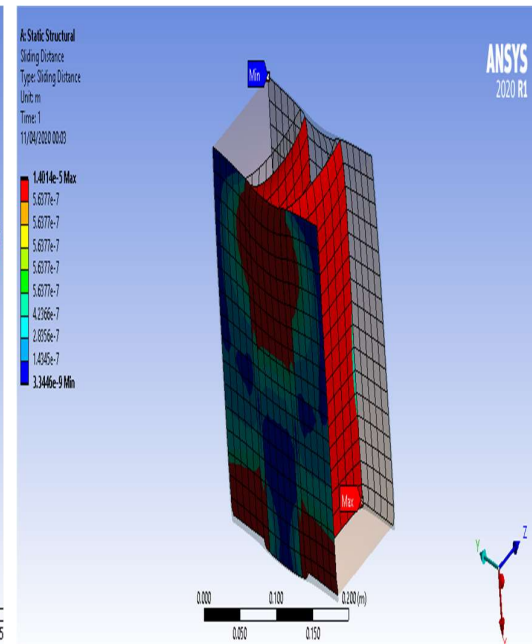


Figure 6.36: The sliding between brick, mortar and CFRP interfaces.

6.20. Finite element analyses for one-layer CFRP on four sides of triplet masonry.

A comparison of the results for one-layer CFRP on four sides of triplet masonry in the experimental and theoretical models is shown in Figure 6.7. The sliding in FE modeling without UDF is too low and with UDF function is similar to the result from the experimental test. As given in Figure 6.41, the failure mechanism produced by the model is the same as reported by experimental observations. Also, the numerical output in terms of load and a sliding curve is too close to the experimental curve as shown in 6.37. The result in figure 6.39 for 80kN load shows 0.7 mm sliding/slipping by Ansys and with the new UDF formula, the sliding will be 5.2mm as shown in Figure 6.41. Interface sliding and the gap between brick, mortar, and CFRP at the failure point are shown in Figures 6.40 and 6.43. Figures 6.42 and 6.44 present shear stress and interface stress respectively at the failure point. The largest shear stress occurred at the middle brick-and-mortar joint. The whole mortar experienced relatively larger shear deformation and indicating the interface delamination. For a load value around 80 kN, a reduction of the masonry stiffness occurs due to the start of elements damages in brick-and-mortar interfaces, which is accurately reproduced by the UDF function in FE modeling. In experimental tests and also in finite element analyses, debonding CFRP from triplets was the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets correctly.

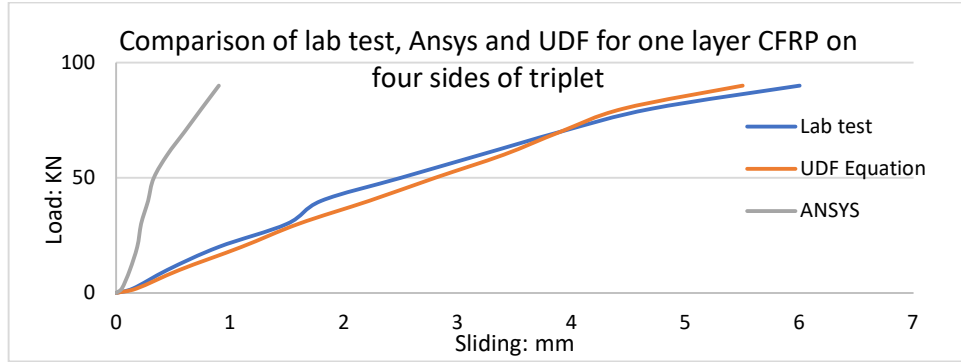


Figure 6.37: Comparison of lab Test, Ansys WB and UDF for one-layer CFRP on four sides of triplet masonry

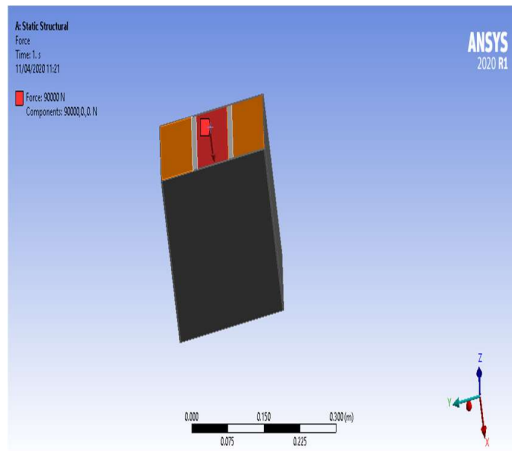


Figure 6.38: One-layer CFRP on four sides of triplet masonry

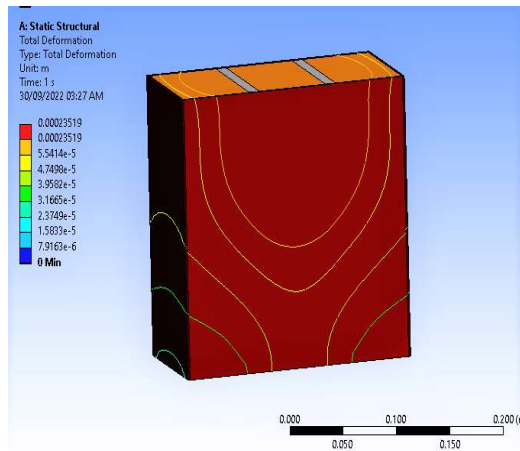


Figure 6.39: The sliding result from ANSYS software without UDF Function.

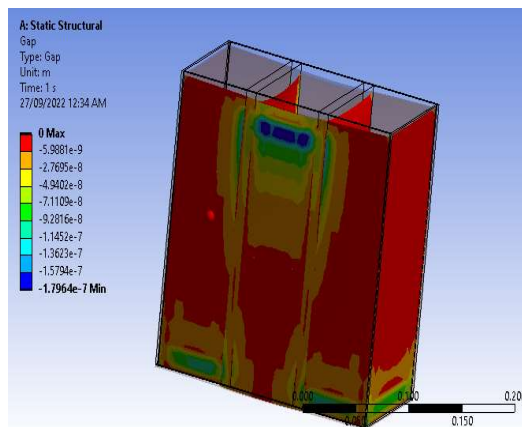


Figure 6.40: The interfaces gap at Max load between CFRP, mortar and brick.

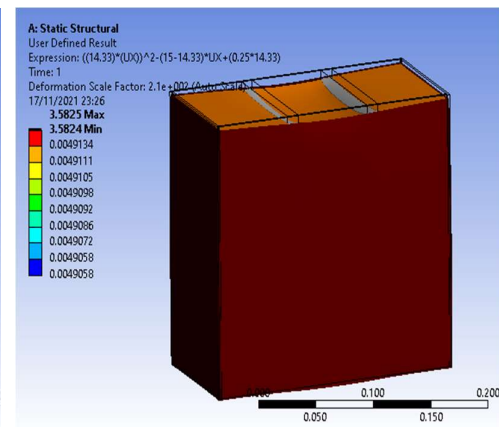


Figure 6.41: The sliding result software with UDF function.

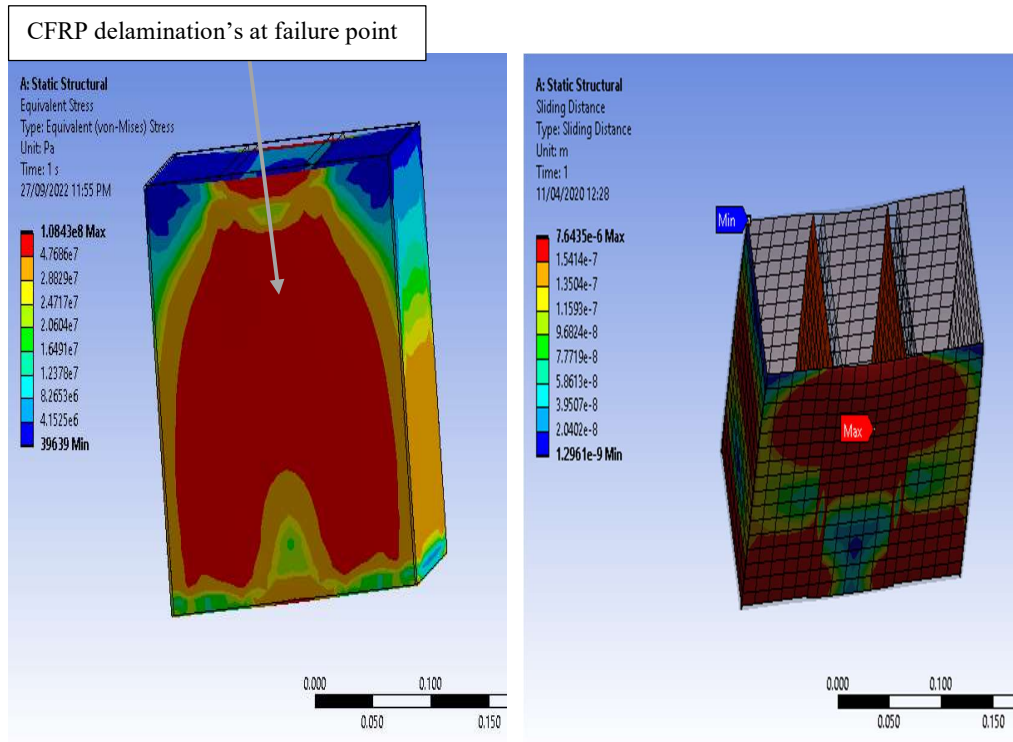


Figure 6.42: Shear stress at failure point

Figure 6.43: The interfaces sliding result with UDF function.

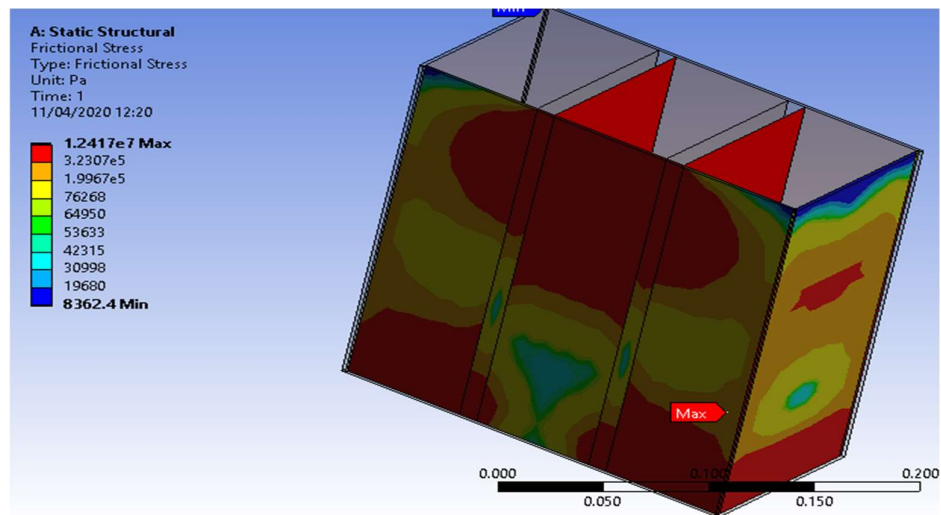


Figure 6.44: The interface stress result with UDF function.

6.21. Finite element analyses for one-layer CFRP strips on two sides of triplet masonry

Figures 6.45 illustrates a comparison of the results for one-layer CFRP strips on four sides of triplet masonry in the experimental and theoretical models. FE modelling with the UDF equation and experimental test are too close and show good agreement. Figures 6.47 and 6.48 show the theoretical modelling without and with UDF respectively. The sliding without UDF at 60KN load is 0.75mm and with the new UDF function is 5.0mm. Figure 6.49 illustrates the shear stress failure propagation. The shear failure indicates that the crack was produced by debonding of the masonry units from the mortar and travelled through the triplet thickness until debonding of the epoxy and CFRP from the masonry units. At this point, the triplet fails because the tensile stresses are no longer transferred to the CFRP. In experimental tests and also in finite element analyses, debonding CFRP from triplets was the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. Interfaces sliding and shear stress with UDF at the failure point are shown in figures 6.50 and 6.51. The reason for failure is debonding of the masonry units from the mortar followed by debonding of epoxy and CFRP from the masonry units. The FE results of masonry triplets show that the stress transfers to CFRP composites and makes the stronger bond between brick and mortar. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets correctly.

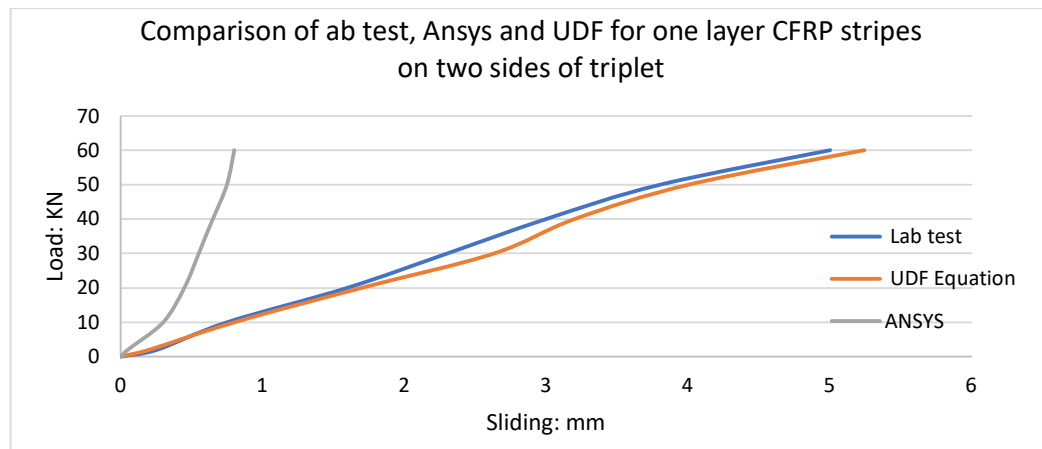


Figure 6.45: Comparison of lab test, Ansys and UDF for one Layer CFRP strips on two sides of triplet

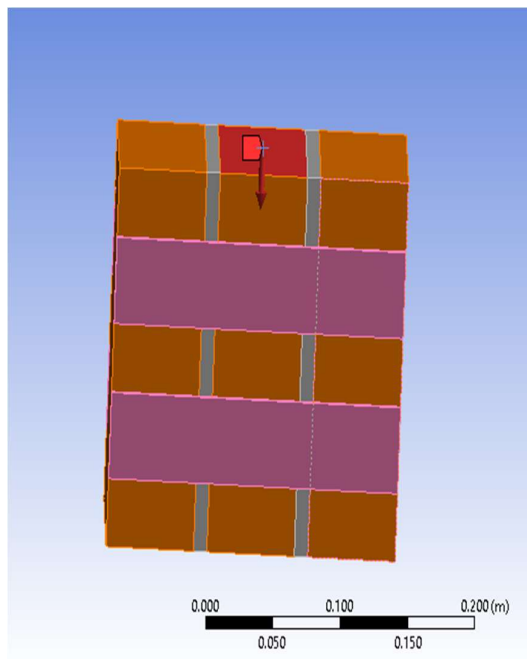


Figure 6.46: One-layer CFRP strips on Two sides of triplet masonry

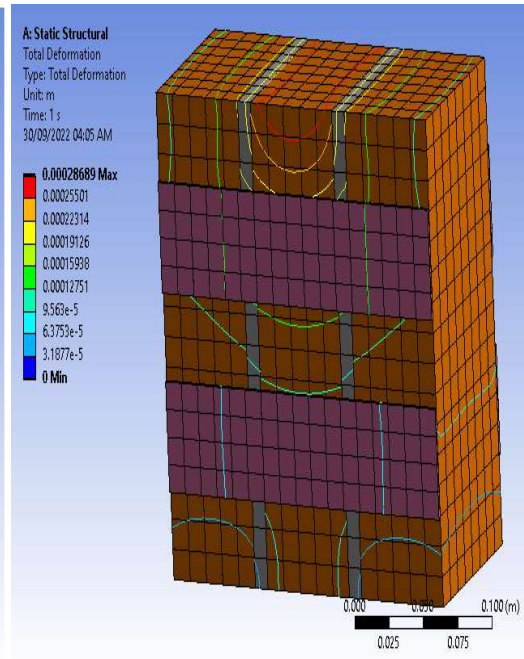


Figure 6.47: The sliding result without UDF

CFRP delamination's at failure point and cracked mortar

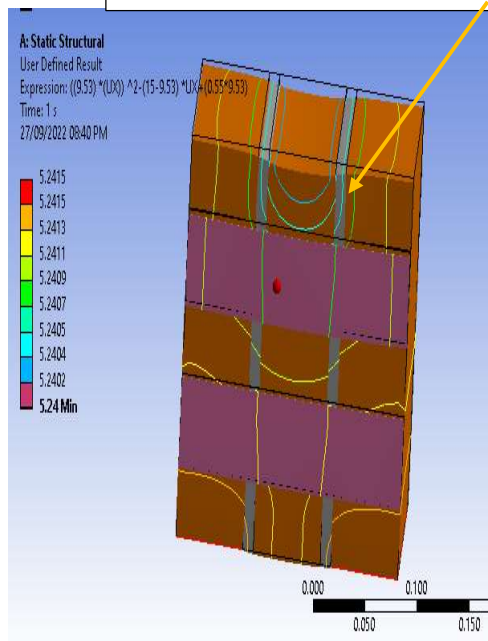


Figure 6.48: The sliding result with UDF

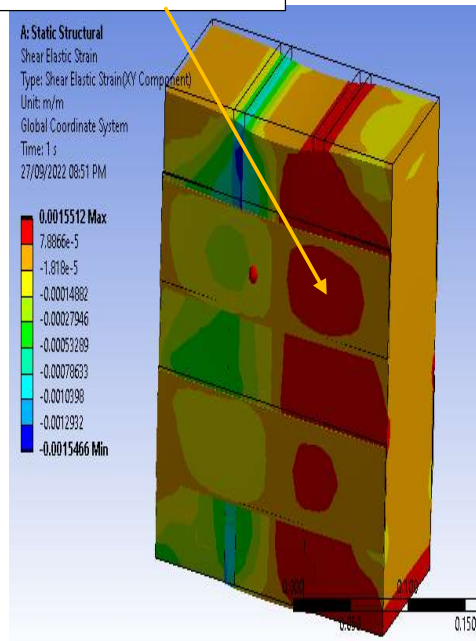


Figure 6.49: Shear stress with UDF

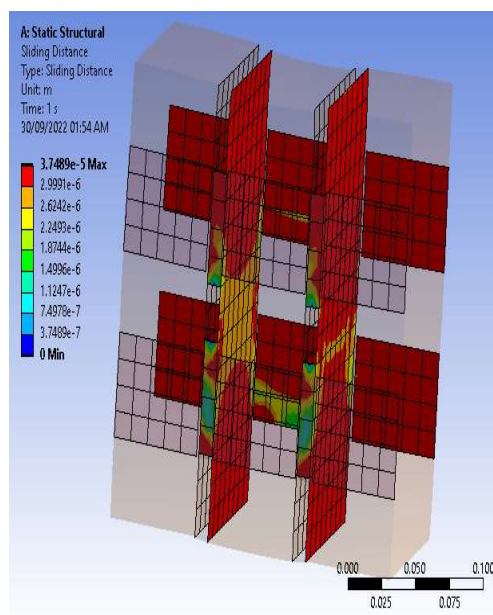


Figure 6.50: Interface sliding with UDF Function.

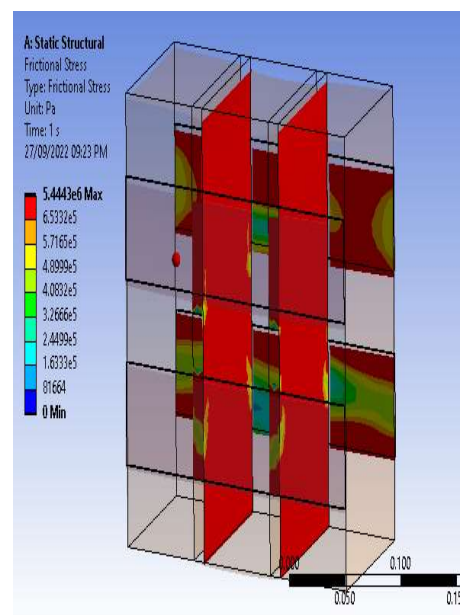


Figure 6.51: Interfaces shear stress with UDF

6.22. Finite element analyses for one-layer CFRP strips on four sides of triplet masonry

The numerically obtained results for the reinforced triplet with CFRP strips on four sides and the experimental shear test are compared in Figure 6.52. As can be observed, the proposed modeling provides a satisfactory result and show that the numerical approach adopted correctly reproduces the experimental behavior in terms of sliding and shear strength with CFRP strips on four sides of the triplet. The result in figure 6.54 for 60KN load shows 0.5mm sliding by Ansys and with the new UDF formula, the sliding will be 4.1mm as shown in Figure 6.55. Figure 6.57 illustrates the shear stress failure propagation and the largest shear stress occurred at the middle brick and mortar joint. The whole mortar experienced larger shear stress and indicating the interface delamination. Also, interface stress and interface sliding can be seen in Figures 6.56 and 6.58.

As can be observed, the proposed numerical model with UDF function adequately reproduces the real sliding and shear stress of the masonry triplet. On the other hand, a diminution of the masonry stiffness occurs due to the start of elements damages and significant joint slipping, which is accurately reproduced by the numerical modeling. In experimental tests and also in finite element analyses, debonding CFRP from triplets was the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost.

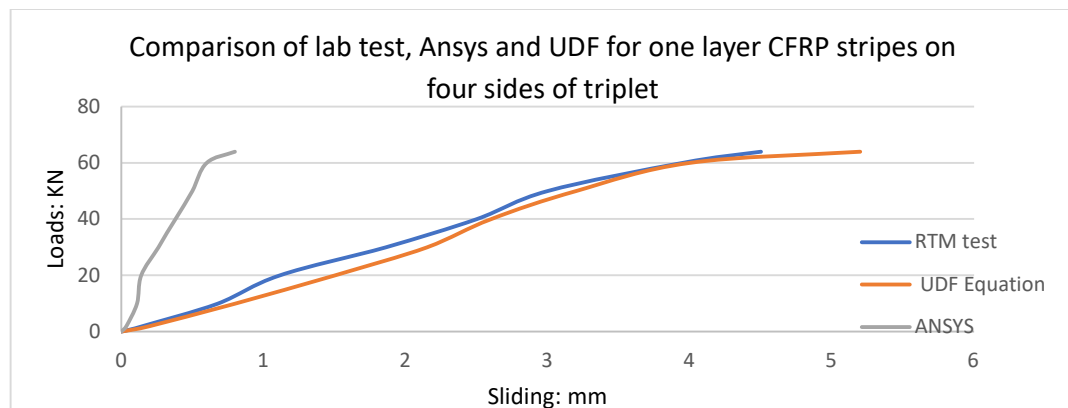


Figure 6.52: Comparison of lab test, Ansys and UDF for CFRP strips on four sides of triplet masonry

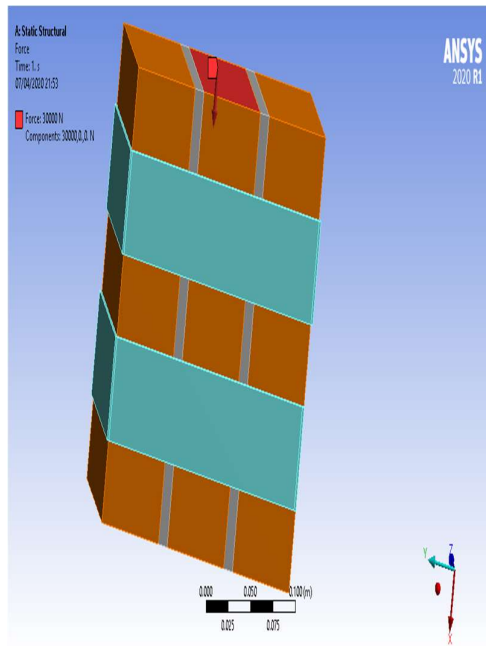


Figure 6.53: One-layer CFRP strips on four sides of triplet masonry

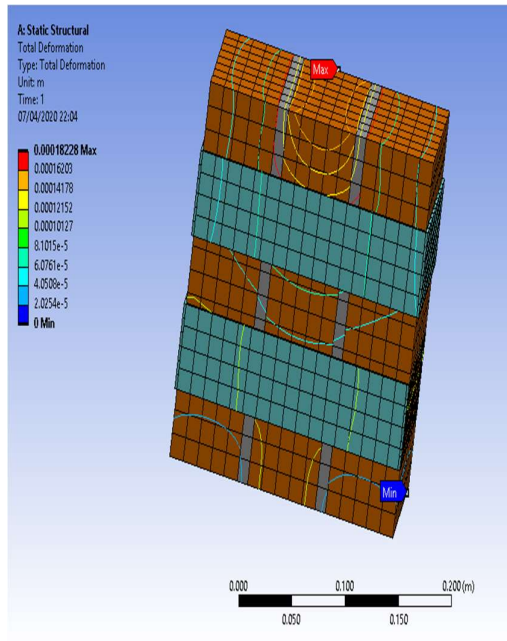


Figure 6.54: The sliding result from ANSYS without UDF function

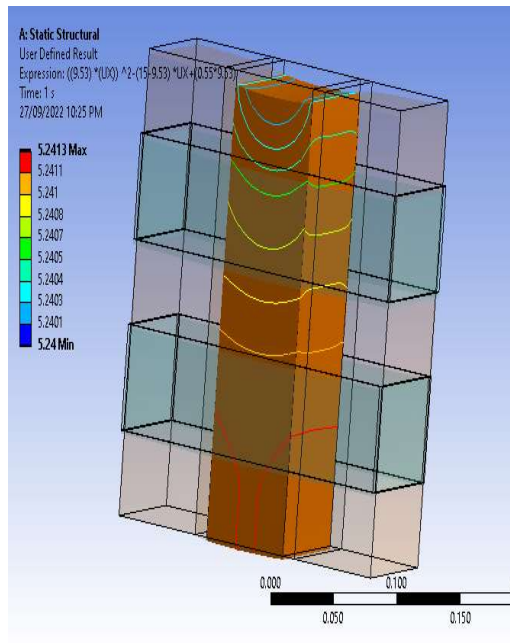


Figure 6.55: Sliding result with UDF Function

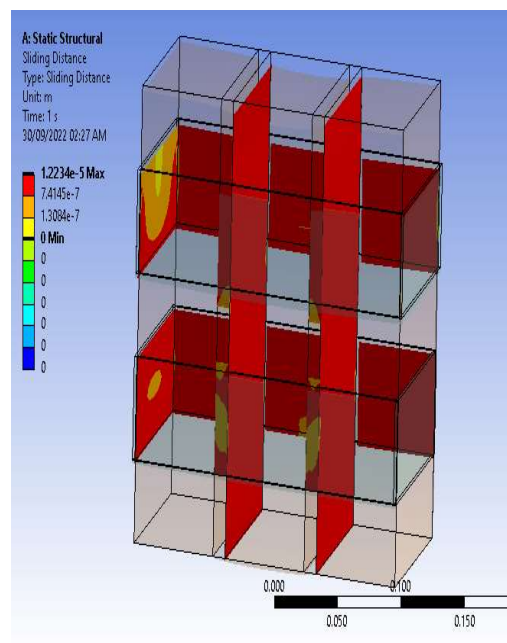


Figure 6.56: Interfaces sliding result from UDF

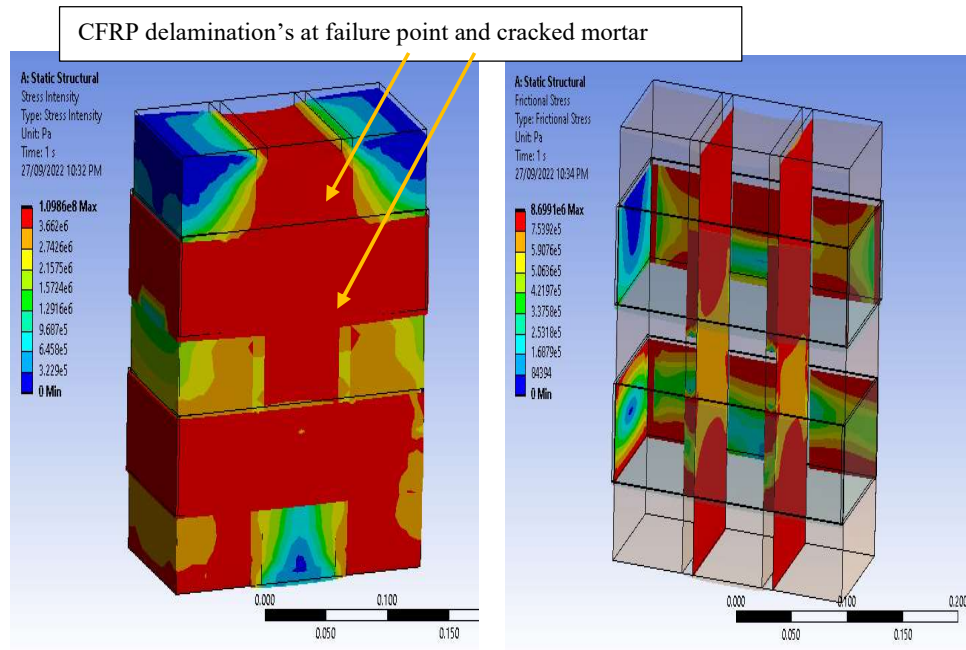


Figure 6.57: Shear stress result with UDF

Figure 6.58: The interfaces stress with UDF

6.23. Finite element analyses for one-layer CFRP net on two sides of triplet masonry

The numerically obtained results for the reinforced triplet with CFRP net on two sides and experimental shear test are compared in Figure 6.59. As can be observed, the proposed modeling provides a satisfactory result and show that the numerical approach adopted correctly reproduces the experimental behavior in terms of sliding and shear strength with CFRP net on two sides of the triplet. The result in figure 6.60 for 75 KN load shows 0.5mm sliding by Ansys and with the new UDF formula, the sliding will be 5.7 mm as shown in Figure 6.61. Figure 6.64 illustrates the shear stress at the failure point and interface sliding can be seen in Figure 6.63. According to the micro-modeling approach, interfaces reproduce the contact zone between bricks and mortar. As can be observed, the proposed numerical model with the UDF function effectively reproduces the real sliding and shear stress of the masonry triplet. On the other hand, a reduction of the masonry stiffness occurs due to the start of elements damages and significant joint slipping, which is accurately reproduced by the numerical modeling. In experimental

tests and also in finite element analyses, debonding CFRP from triplets was the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets with good accuracy.

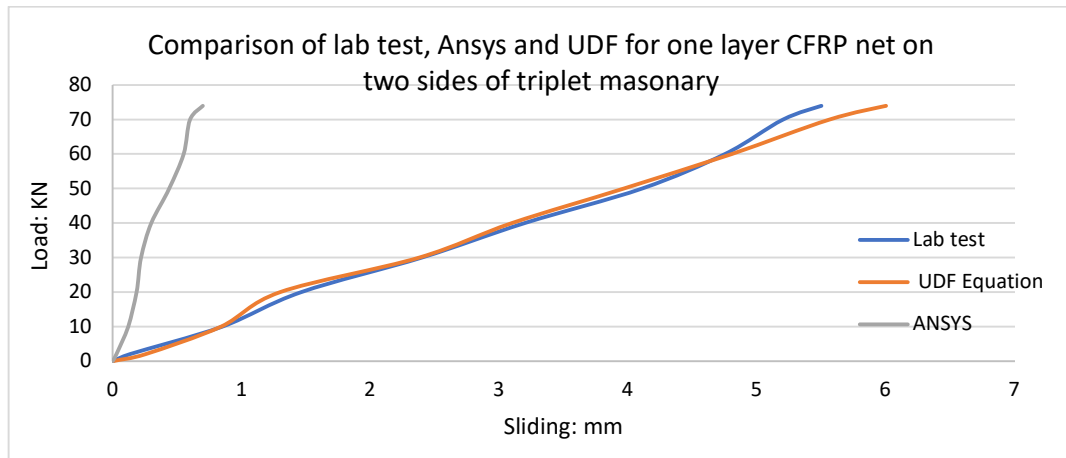


Figure 6.59: Comparison of lab test, Ansys and UDF for one-layer CFRP net on two sides of triplet

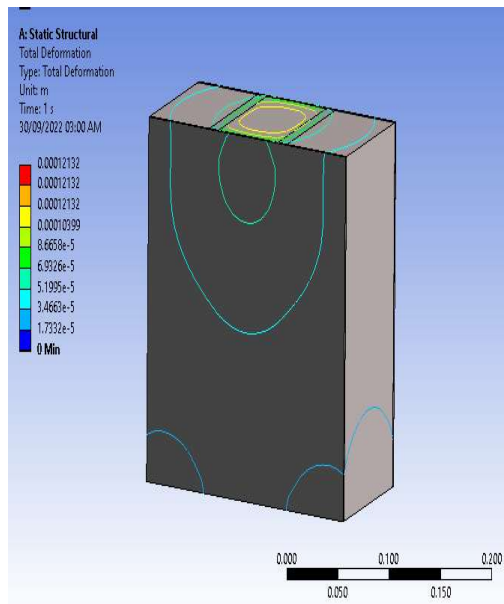


Figure 6.60: Slipping by ANSYS without UDF

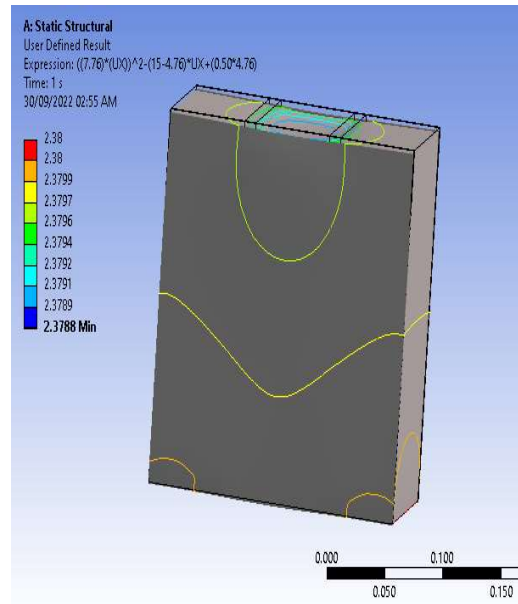


Figure 6.61: Slipping result with UDF

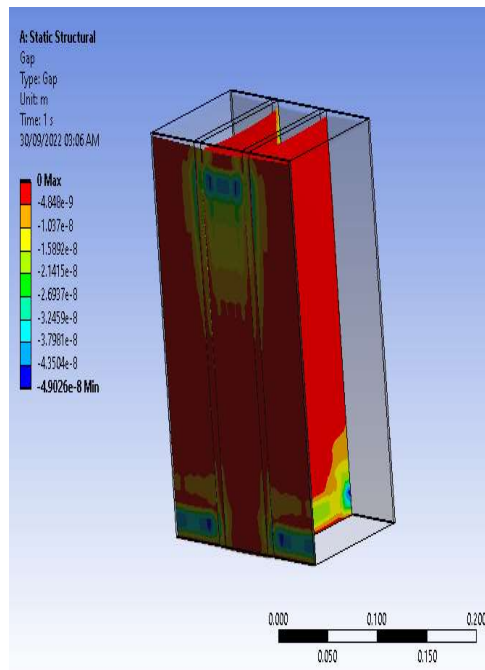


Figure 6.62: The interfaces gap with UDF

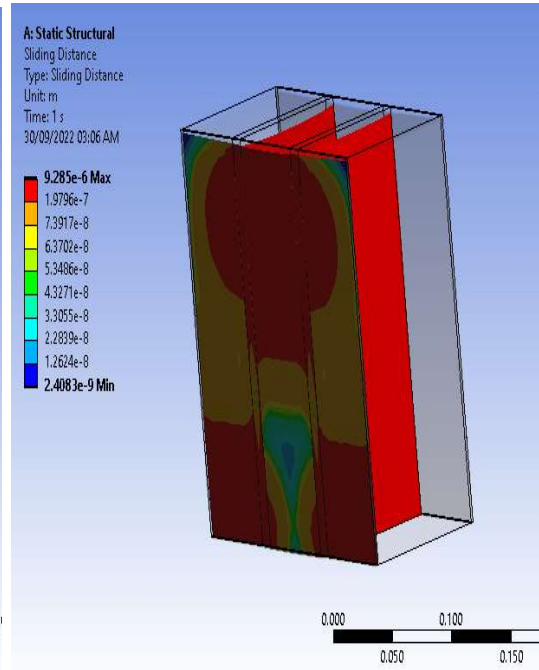


Figure 6.63: Interface sliding result with UDF

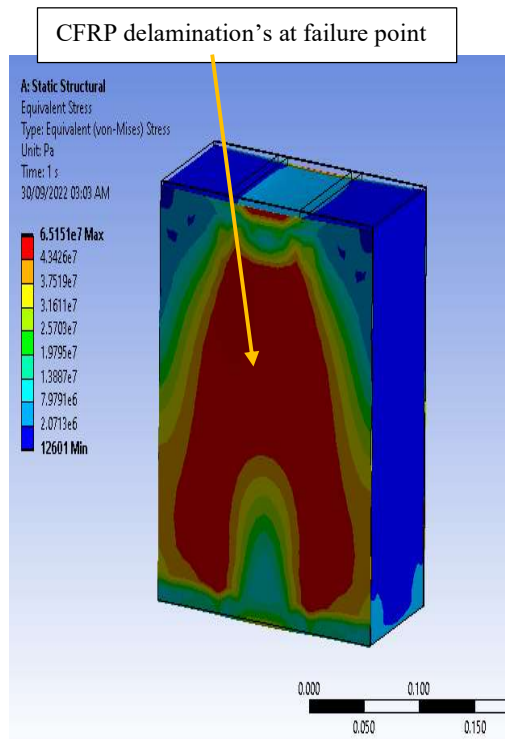


Figure 6.64: The shear stress

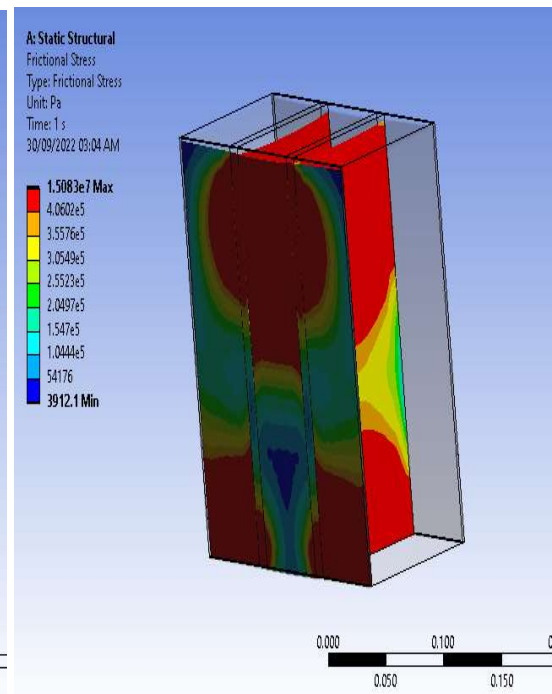


Figure 6.65: Interface frictional stress

6.24. Finite element analyses for one-layer CFRP net on four sides of triplet masonry

Figure 6.66 illustrates a comparison of the results for one-layer CFRP net on four sides of triplet masonry in the experimental and theoretical models. FE modeling with the UDF equation and the experimental test shows good agreement. Figures 6.67 and 6.68 show the theoretical modeling without and with UDF respectively. The sliding without UDF at 70KN load is 0.5 mm and with the new UDF function is 4.5 mm. Figure 6.70 shows the shear stress at the failure point. The shear failure indicates that the crack was produced by debonding of the masonry units from the mortar and travelled through the triplet thickness until debonding of the epoxy and CFRP from the masonry units. At this point, the triplet fails because the tensile stresses are no longer transferred to the CFRP. Interfaces sliding and shear stress distribution with UDF at failure point are shown in figures 6.71 and 6.72. The reason for failure is debonding of the masonry units from the mortar followed by debonding of epoxy and CFRP from the masonry units. The FE results of masonry triplets show that the stress transfers to CFRP composites and makes the stronger bond between brick and mortar. Also, the interfaces gap between brick, mortar, and CFRP net at the failure point is shown in Figure 6.69. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets with satisfactory accuracy.

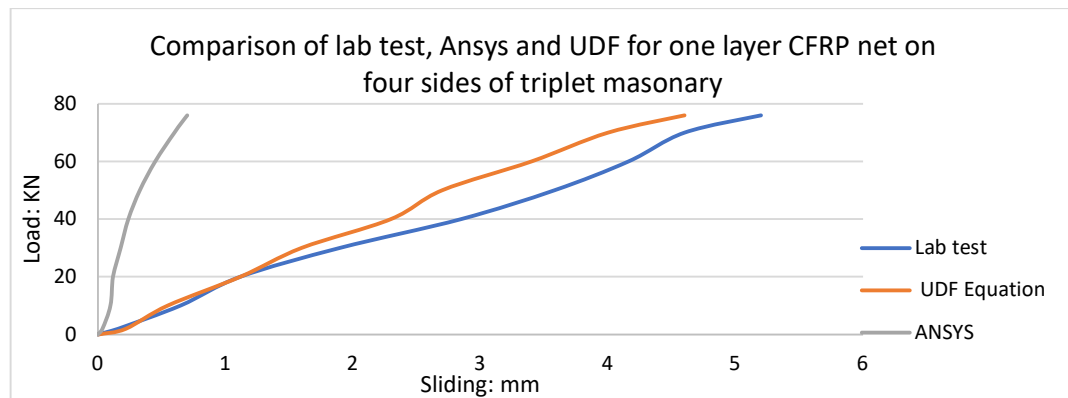


Figure 6.66: Comparison of lab test, Ansys and UDF function for one-layer CFRP net on four sides of triplet masonry

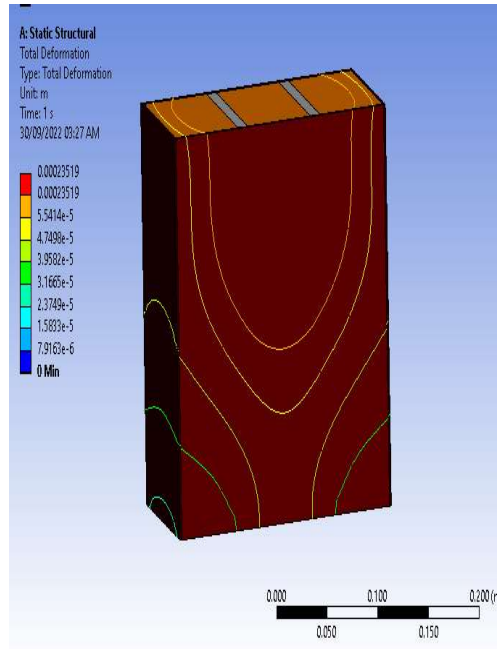


Figure 6.67: Sliding with ANSYS without UDF

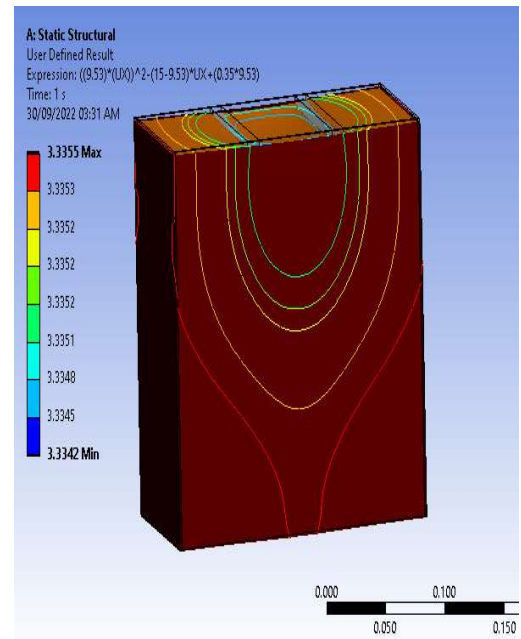
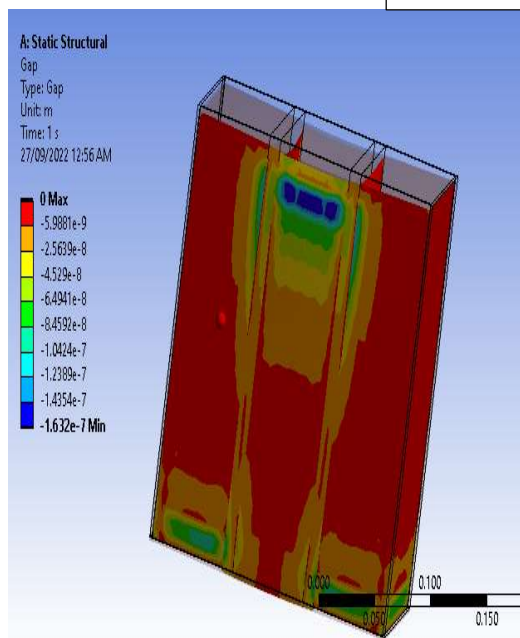
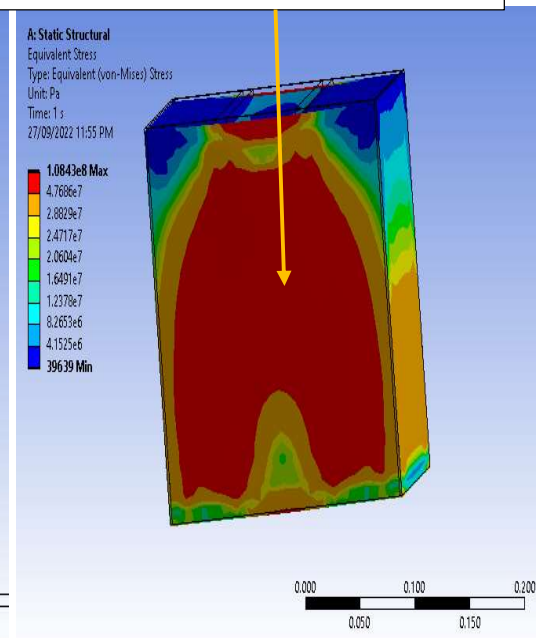


Figure 6.68: Sliding with UDF function.

CFRP delamination's at failure point and cracked mortar



Figures 6.69: The interfaces gap at Max load between CFRP net, mortar and brick



Figures 6.70: Shear stress transferred from brick and mortar to CFRP net

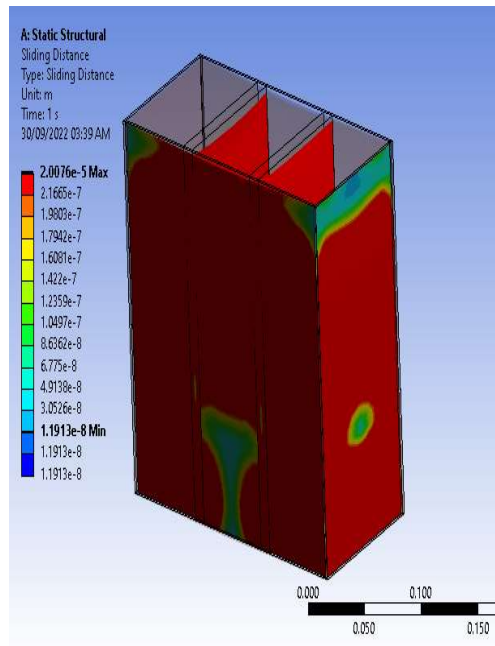


Figure 6.71: The interfaces sliding

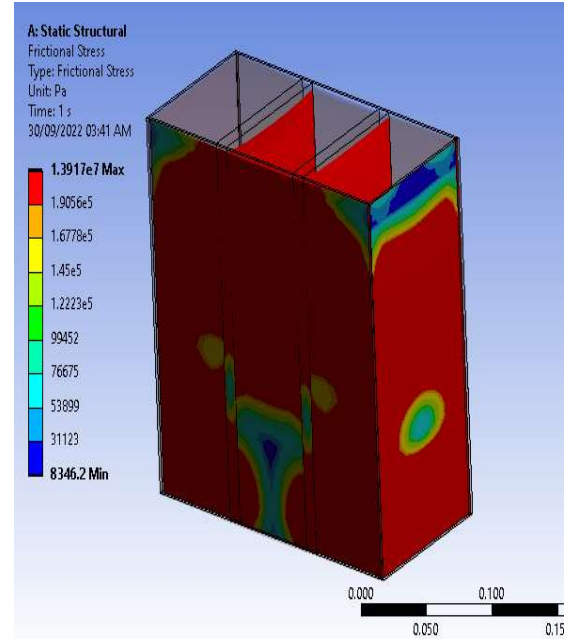


Figure 6.72: Interface frictional stress

6.25. Finite element analyses for one-layer CFRP net strips on two sides of triplet masonry

The numerically obtained results for the reinforced triplet with CFRP net strips on two sides and the experimental shear test are compared in Figure 6.73. As can be observed, the proposed modeling provides an acceptable result and show that the numerical approach correctly reproduces the experimental behavior in terms of sliding and shear strength with CFRP net strips on two sides of triplet masonry. The results show good agreement and displacement in theoretical modeling and the experimental test is nearly the same. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets with sufficient accuracy. The result in figure 6.74 for 58 KN load shows 0.6 mm sliding by Ansys and with the new UDF function, the sliding will be 5 mm as shown in Figure 6.75. Figure 6.78 shows the shear stress distribution and interface frictional stress can be seen in Figure 6.79. As can be observed, the proposed numerical model with UDF

function adequately reproduces the real sliding and shear stress of the masonry triplet. On the other hand, a reduction of the masonry stiffness occurs due to the start of elements damages and joints slipping, which is accurately reproduced by the numerical modeling. Also, the Interfaces gap and interfaces sliding between triplet components and CFRP are shown in Figures 6.76 and 6.77.

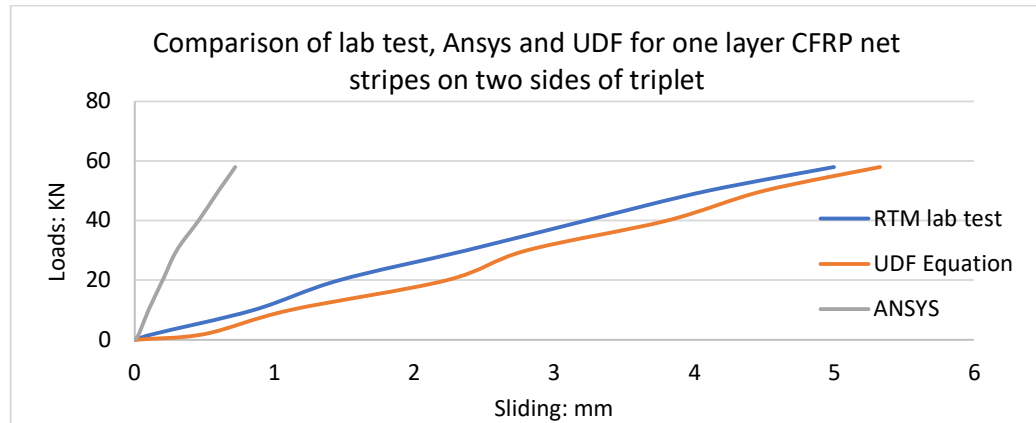


Figure 6.73: Comparison of lab test, Ansys and UDF for one-layer CFRP net strip on two sides of triplet masonry.

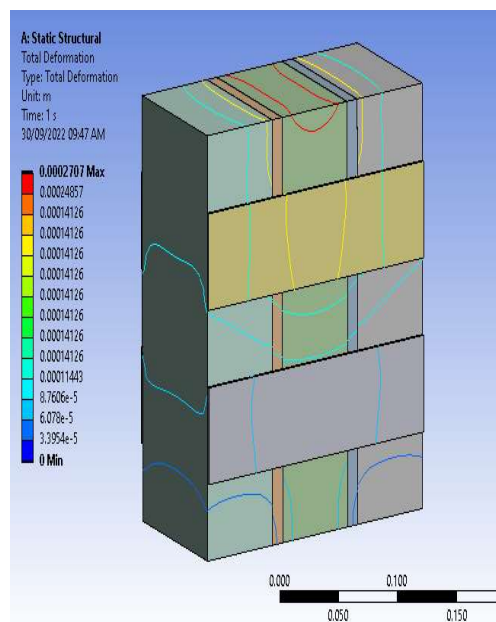


Figure 6.74: Slipping without UDF

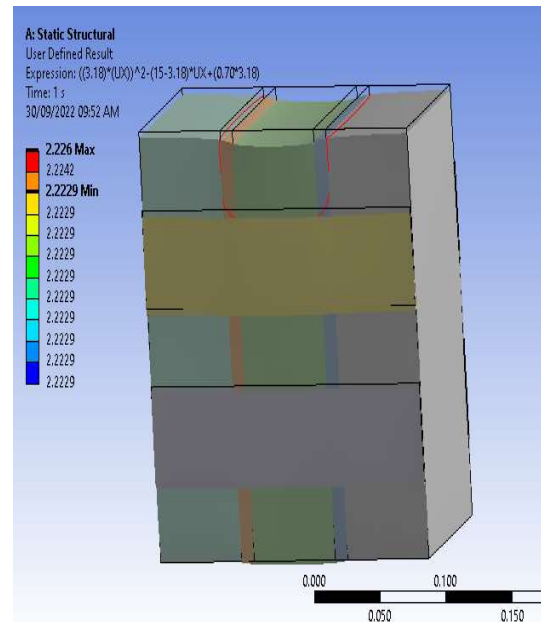


Figure 6.75: Slipping with UDF function

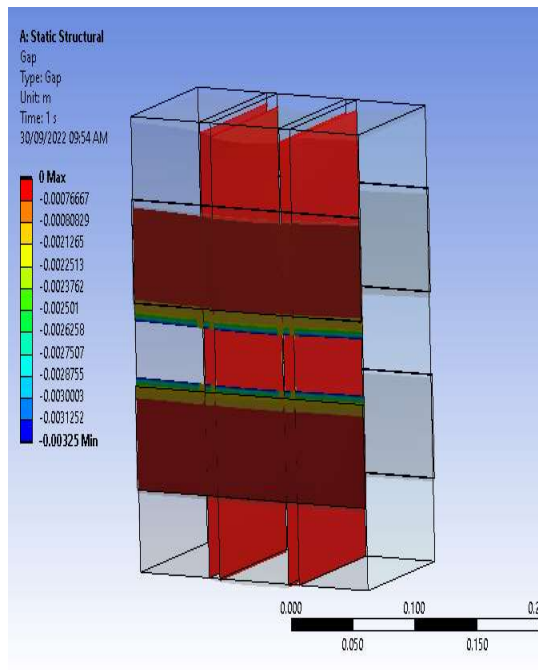


Figure 6.76: Interface gap between triplet and CFRP

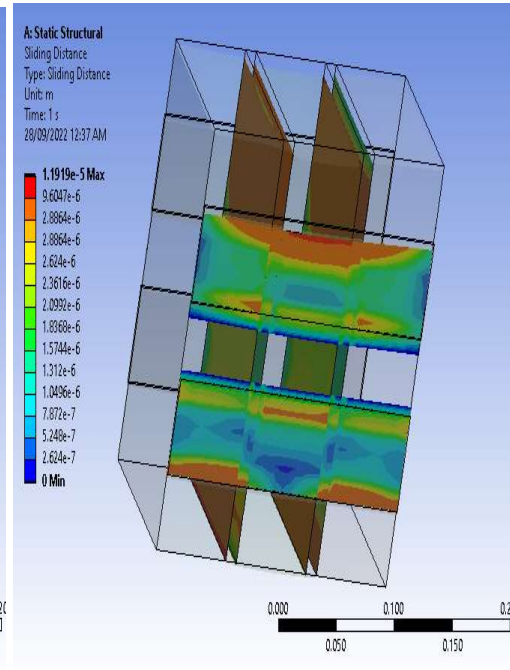


Figure 6.77: The interfaces sliding

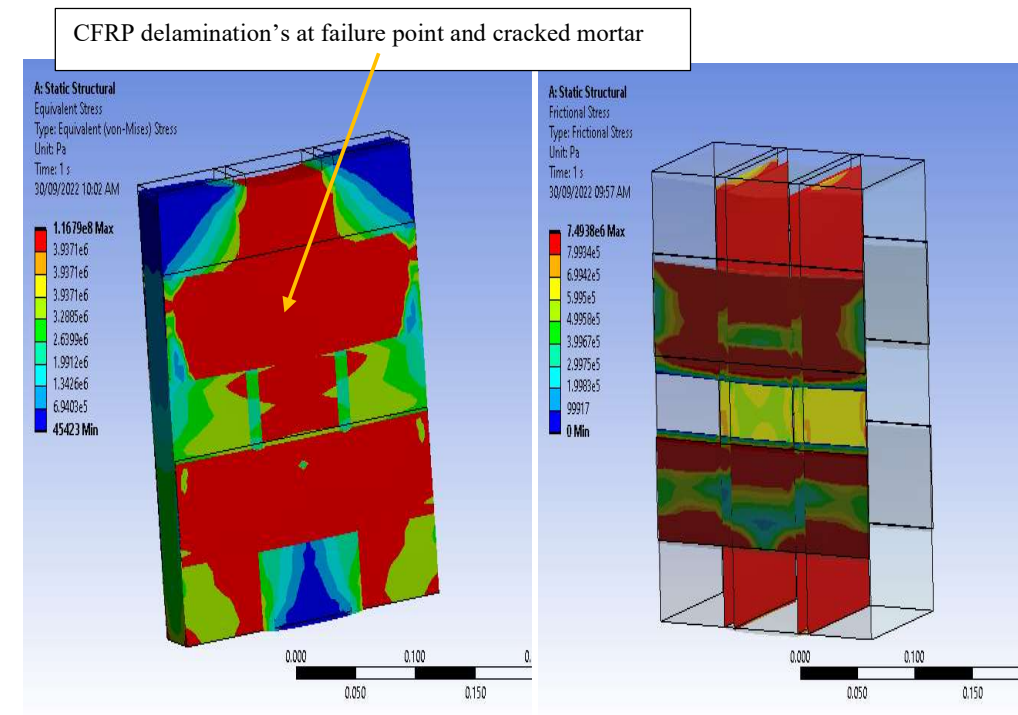


Figure 6.78: Equivalent shear stress

Figure 6.79: Interface frictional stress

6.26. Finite element analyses for one-layer CFRP net strips on four sides of triplet masonry

Figures 6.80 illustrates a comparison of the results for one-layer CFRP net strips on four sides of triplet masonry in the experimental and theoretical FE models. The result in figure 6.81 for 72 KN load shows 0.5 mm sliding by Ansys and with the new UDF function, the sliding is 5.0 mm, as shown in Figure 6.82, which is acceptable and displacement in theoretical modeling and the experimental test are too close. Figures 6.83 and 6.84 illustrate the interface gap and interface sliding between brick-mortar and CFRP at the failure point receptively. The reduction of the masonry stiffness occurs due to the start of elements damages and joints slipping, which is accurately reproduced by the numerical modeling. The stress distribution in mortar, bricks, and CFRP net strips at the failure point is shown in Figure 6.85. Also, frictional stress between brick and mortar and CFRO net strip is shown in Figure 6.86.

As can be observed, the proposed modeling provides an acceptable result and shows that the numerical approach with the UDF function correctly reproduces the experimental behavior in terms of sliding and shear strength with CFRP net strips on four sides of triplet masonry. In experimental tests and also in finite element analyses, debonding CFRP from triplets was the main reason for failure of CFRP-strengthened triplet masonry, this failure mostly occurs when the bonding between the reinforcement composite and masonry interface is lost. The crack observed in the mortar layer and the brick during the experimental test and those predicted in the proposed model were similar. This means that the numerical model with UDF function can simulate the failure modes of shear triplets with sufficient accuracy.

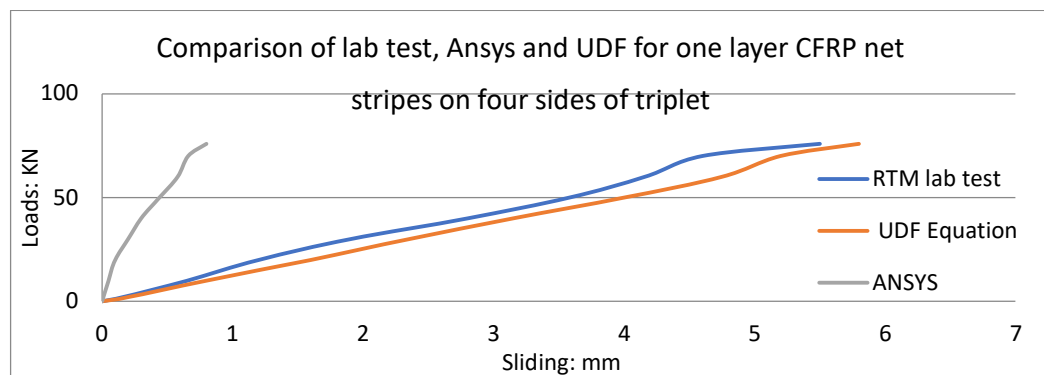


Figure 6.80: Comparison of lab test, Ansys and UDF for one Layer CFRP net strip on Four sides of triplet

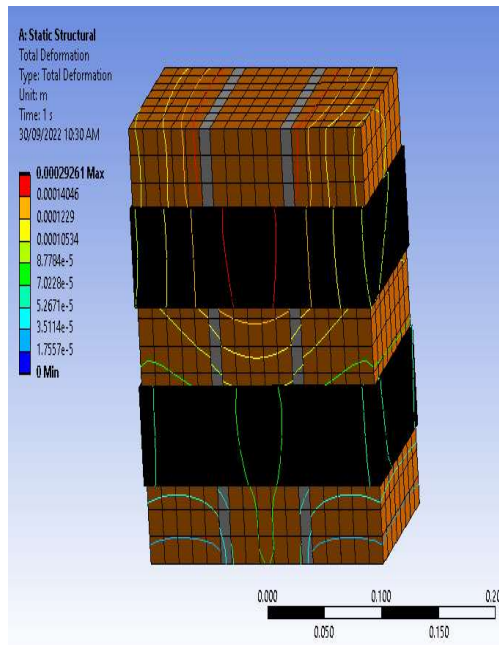


Figure 6.81: slipping without UDF function

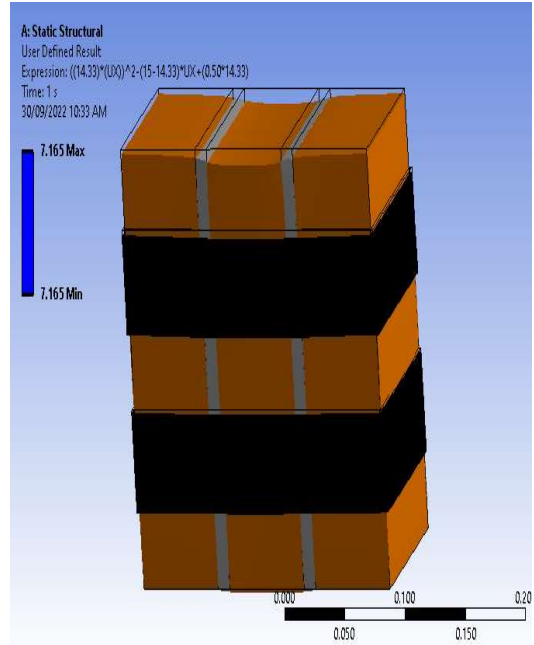


Figure 6.82: Sliding result with UDF function

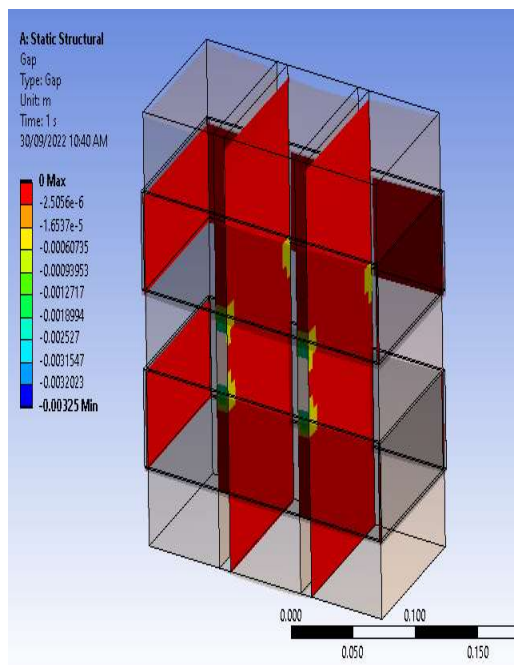


Figure 6.83: Interfaces gap

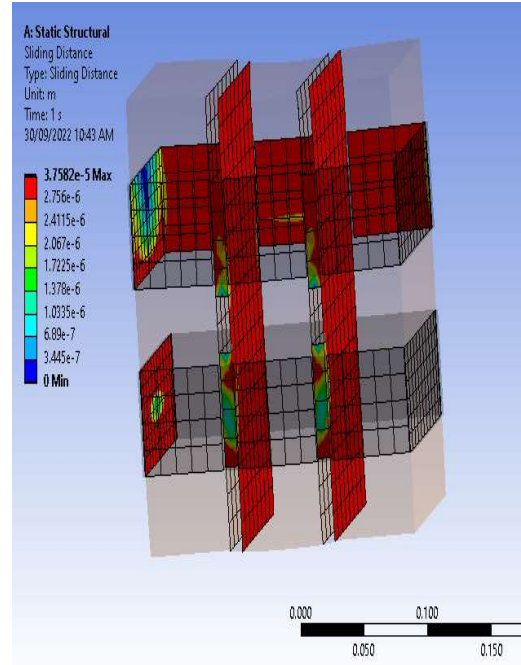


Figure 6.84: Interface sliding

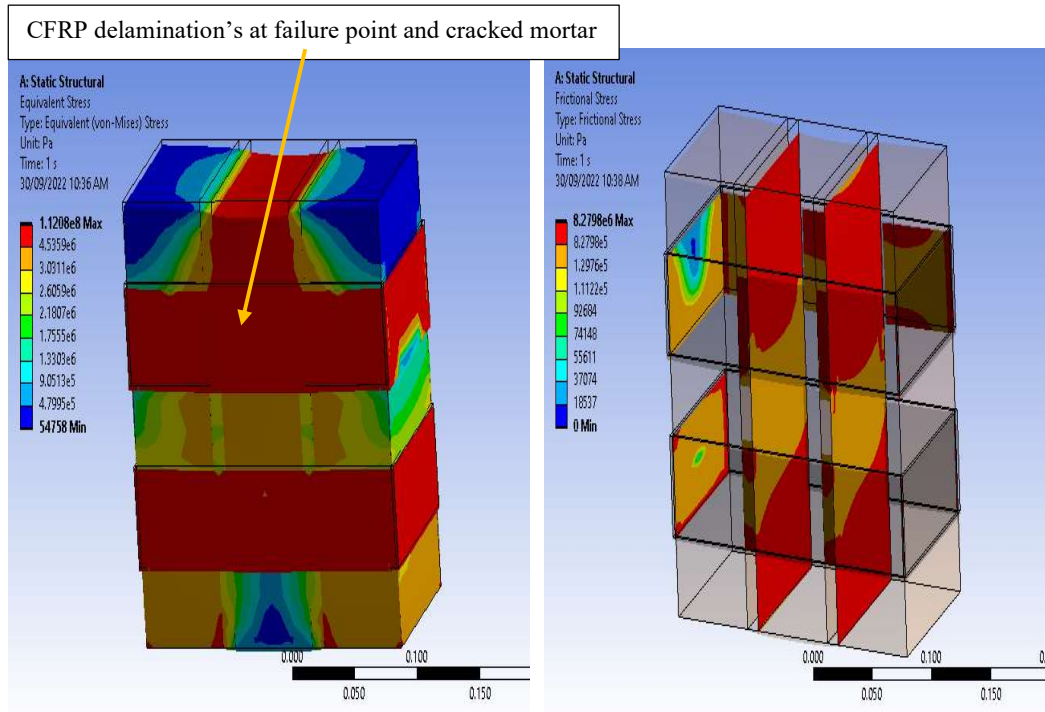


Figure 6.85: Equivalent shear stress

Figure6.86: Frictional stress

Figure 6.87 illustrates the sliding comparison by Ansys, experimental tests and UDF equations at failure point for different CFRP configurations of on triplet masonry (S represents side of triplet).

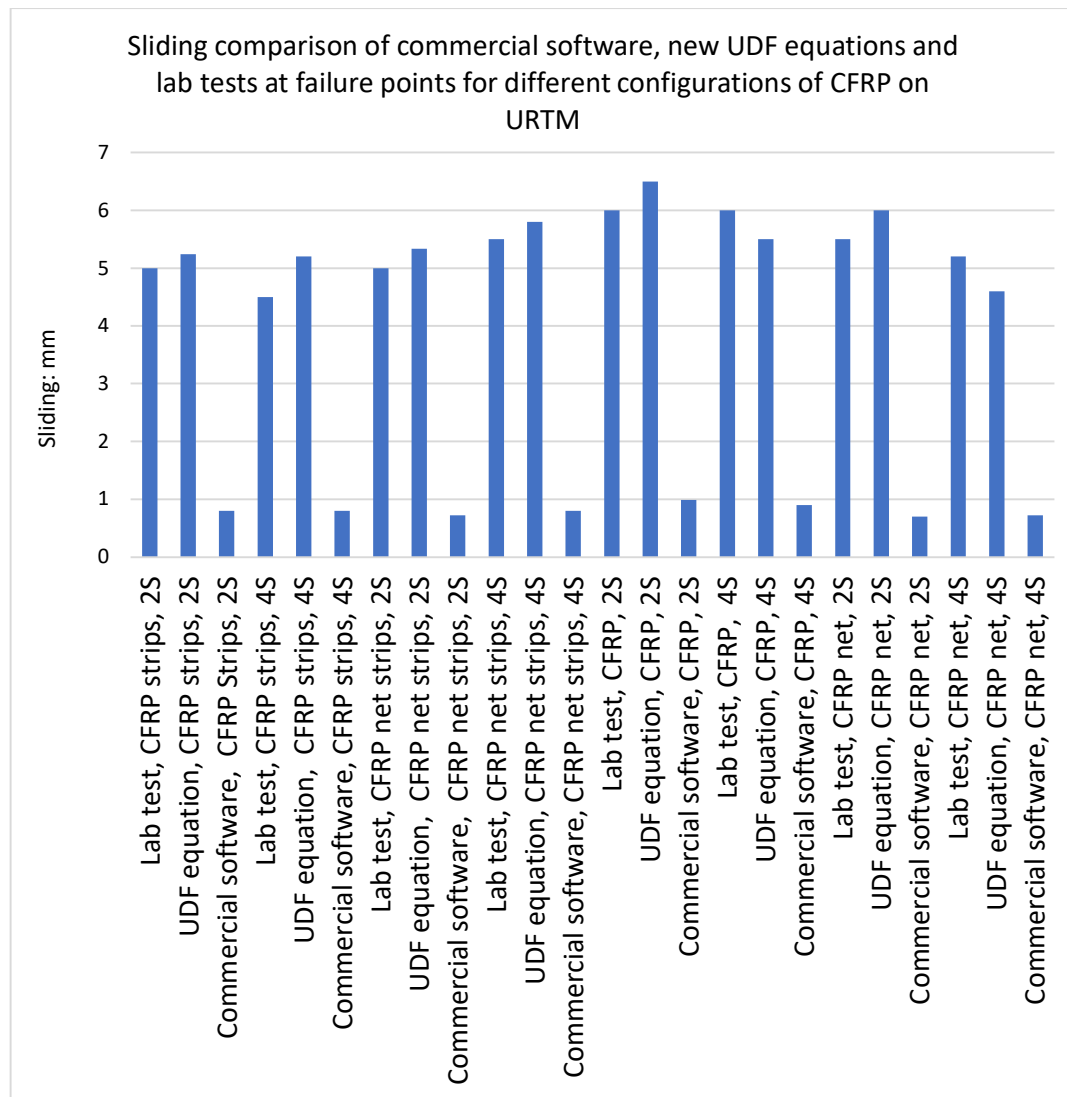


Figure 6.87: Sliding comparison of commercial software, new UDF equations and lab tests at failure points for different CFRP configurations on URTM

Table 6.4 illustrates the results of sliding from Ansys, UDF function, and lab tests at failure point and error for all CFRP configurations. This table shows acceptable errors and a good agreement between new UDF functions and lab tests for shear loading.

	CFRP configurations on triplet masonry (one layer)	Load at failure point (KN)	Lab test sliding (mm)	Ansys sliding (mm)	UDF function sliding (mm)	Error, Ansys (%)	Error, UDF function (%)
1	URTM	10	2.7	0.05	2.56	98	5.2
2	CFRP on two sides	90	6	0.99	6.5	83.5	7.7
3	CFRP on four sides	90	6	0.9	5.5	85	8.3
4	CFRP strips on two sides	60	5	0.8	5.24	84	4.6
5	CFRP strips on four sides	64	4.5	0.8	5	82	10
6	CFRP net on two sides	74	5.5	0.7	6	87	8.3
7	CFRP net on four sides	76	5	0.7	4.6	86	8
8	CFRP net strips on two sides	58	5	0.72	5.3	85.6	5.7
9	CFRP net strips on four sides	76	5.5	0.8	5.04	85.5	8.4

Table 6.4: Comparison the new Ansys, UDF functions and lab tests at failure point and errors

6.27. Comparison of experimental tests, theoretical equations and finite element modelling (UDF functions) for stress and sliding of triplet masonry

This section is a comparison of the finite element analyses by new UDF functions, theoretical equations, and experimental tests for reinforced triplet masonry with different CFRP configurations. based on stress and sliding and a comparison of experimental tests, and theoretical equations from chapter four and table 4.5. The user-defined function to create correct 3D modeling is shown in table 6.3. The new functions make it possible to predict correct sliding/slipping in 3D modeling at any stress point and the errors are between 5% to 25%. Figures 6.88 to 6.96 illustrate the sliding comparison of experimental tests, UDF functions, and new theoretical equations from chapter four for different CFRP configurations of triplet masonry.

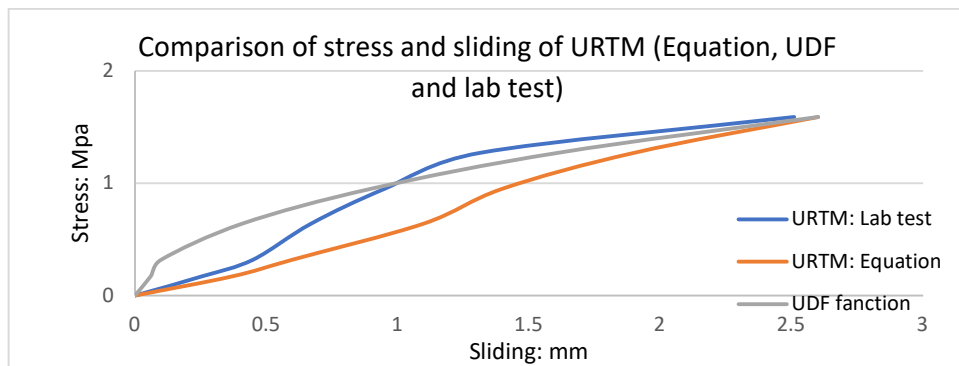


Figure 6.88: Comparison of Stress and Sliding of URTM (Equation, UDF and Lab Test)

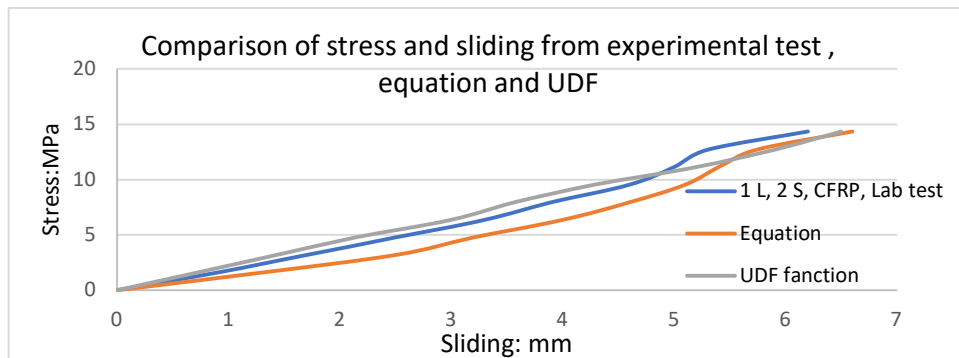


Figure 6.89: Comparison of Stress and Sliding from Experimental Test, Equation and UDF

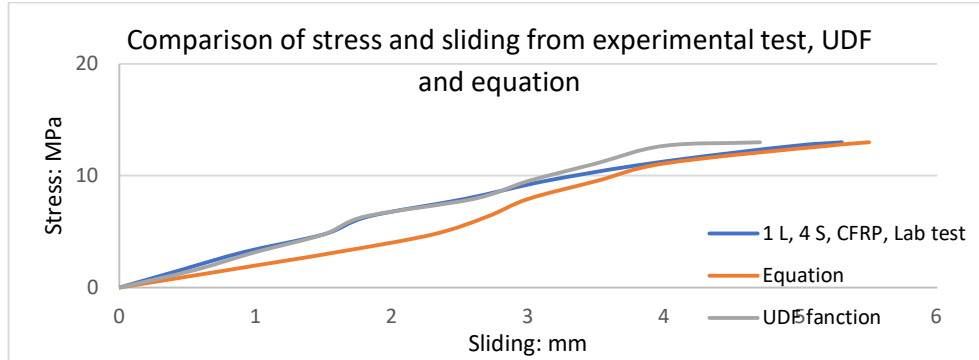


Figure 6.90: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

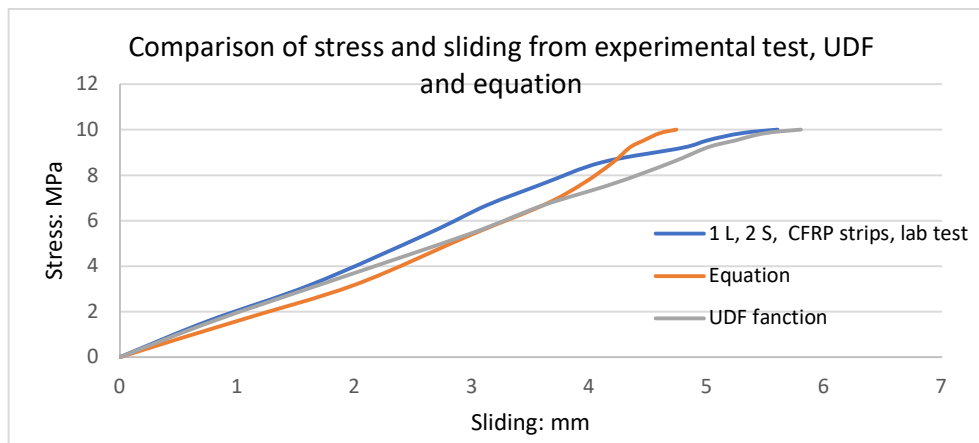


Figure 6.91: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

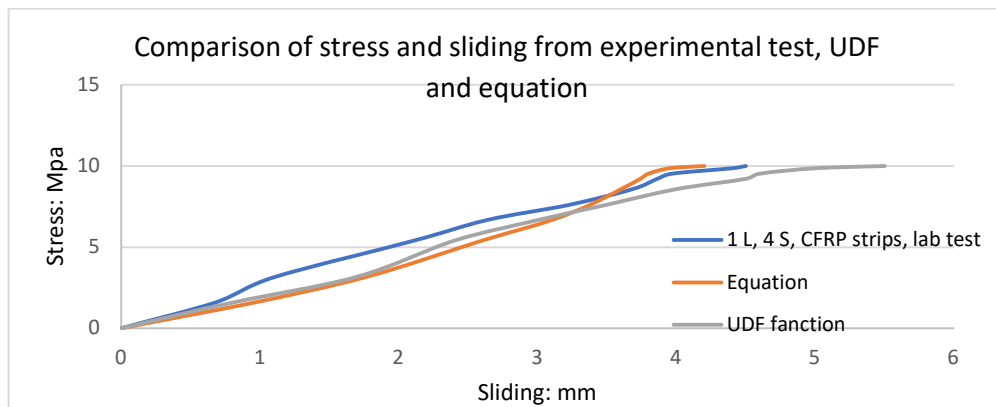


Figure 6.92: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

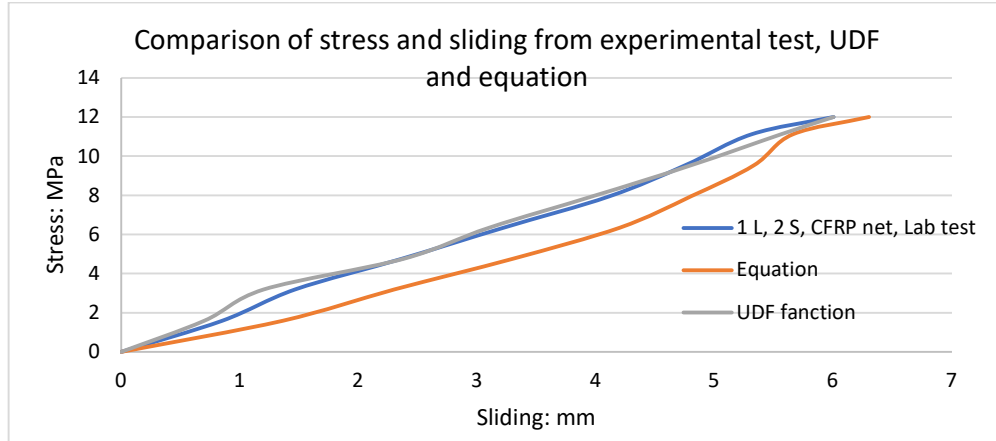


Figure 6.93: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

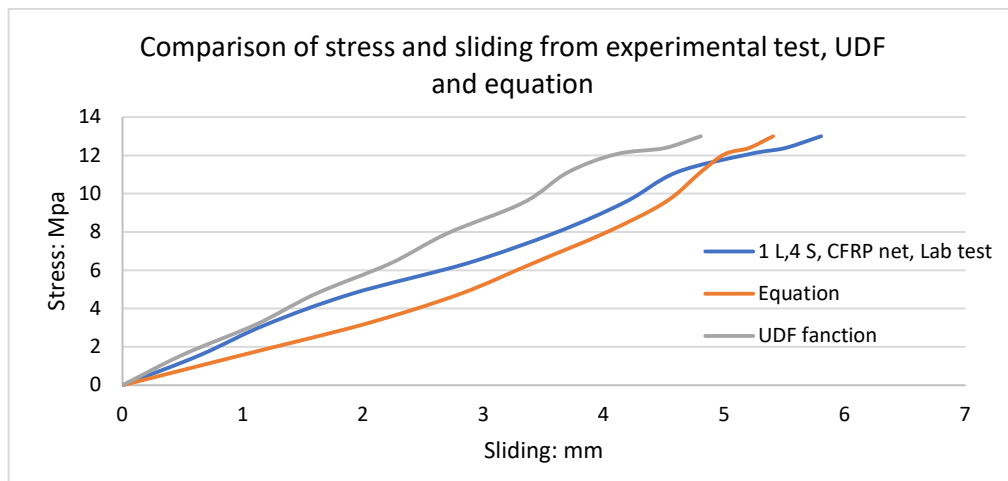


Figure 6.94: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

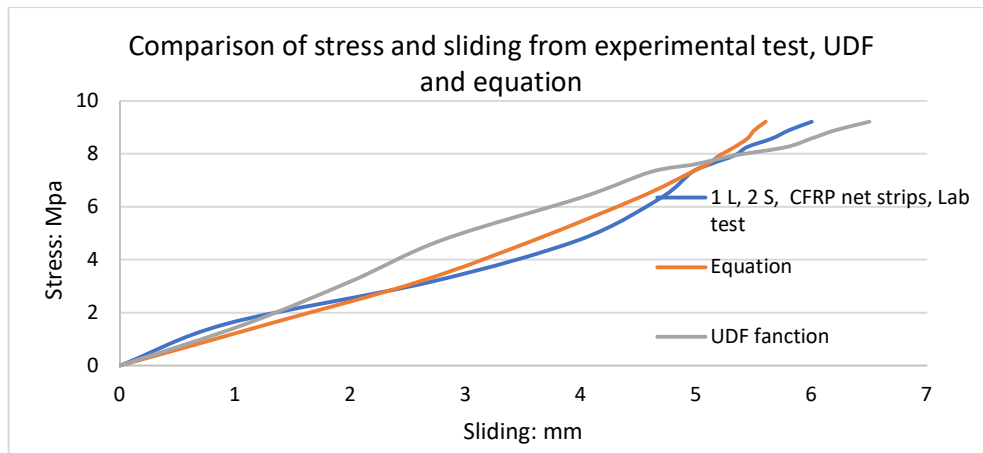


Figure 6.95: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

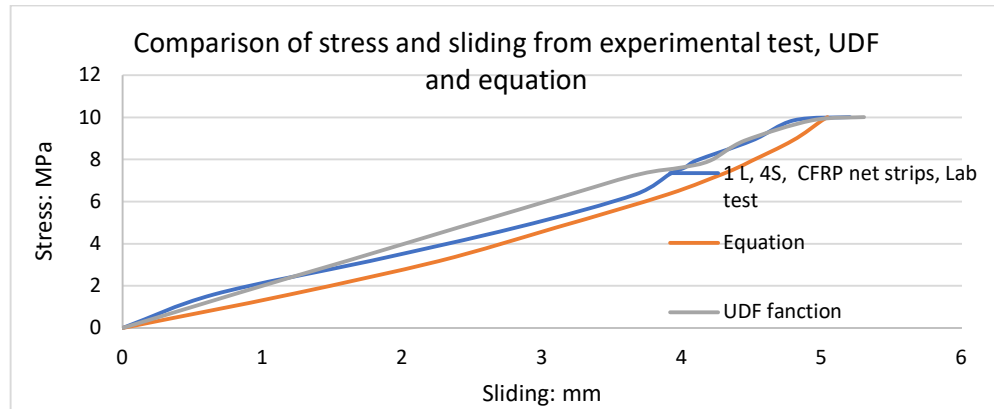


Figure 6.96: Comparison of Stress and Sliding from Experimental Test, UDF and Equation

6.28. Finite element simulations of cracked triplet masonry under shear loads

The repair and strengthening of damaged or undamaged masonry walls with fibre-reinforced polymer (CFRP) sheets has become one of the important challenges confronting civil engineers and researchers. CFRP composite systems when bonded to various damaged or undamaged structural members can significantly improve their mechanical behaviour in serviceability conditions, at failure mechanism and reduce crack propagation by increasing both the stiffness and the load capacity resistance, Hamrat et al, 2019.

Experimental results showed that the load resistance of masonry triplets was increased due to retrofitting CFRP composites. In the case of shear triplets, CFRP retrofitting increased the strength and deformational capacity. Each type of element was represented with its properties and initial modulus of elasticity obtained from exponential tests. The failure of masonry under shear load was related to the debonding CFRP composites.

In this study, the theoretical modelling by Ansys was much lower than the experimental test and was not reliable. Therefore, this error was corrected by using the developed UDF formula as a user-defined function to create correct 3D modelling. By using the new formulas, it is possible to predict correct sliding/slipping in 3D modelling at any load point. In this study, eight configurations of externally bonded triplet masonry with

carbon fibre were tested under shear loading up to failure. The tests were analysed by Ansys without UDF and with UDF functions from table 6.3 and compared with the results from experimental tests.

6.29. Numerical simulations of cracked URTM triplet masonry under shear loading

The unreinforced cracked masonry triplet is simulated under shear loading to investigate the shear behaviour of masonry structures. The sliding comparison of the finite element (FE) modelling, experimental test and UDF function for cracked URTM (control samples) are shown in Figure 6.97. Figures 6.99 and 6.70 show the theoretical modelling without and with UDF respectively. The displacement in FE modelling with Ansys was very low, therefore the user-defined function (UDF) was developed to obtain correct sliding. As shown in Figures 6.99 and 6.70, sliding without UDF at 2KN load is 4 mm and with the new UDF function is 9 mm, which is correct and reliable. The proposed FE model adequately reproduces the sliding of the middle brick in a cracked triplet. Figure 6.71 illustrates shear stress failure propagation. The shear failure is very low and indicates the lack of appropriate connection between mortar and bricks in cracked URTM.

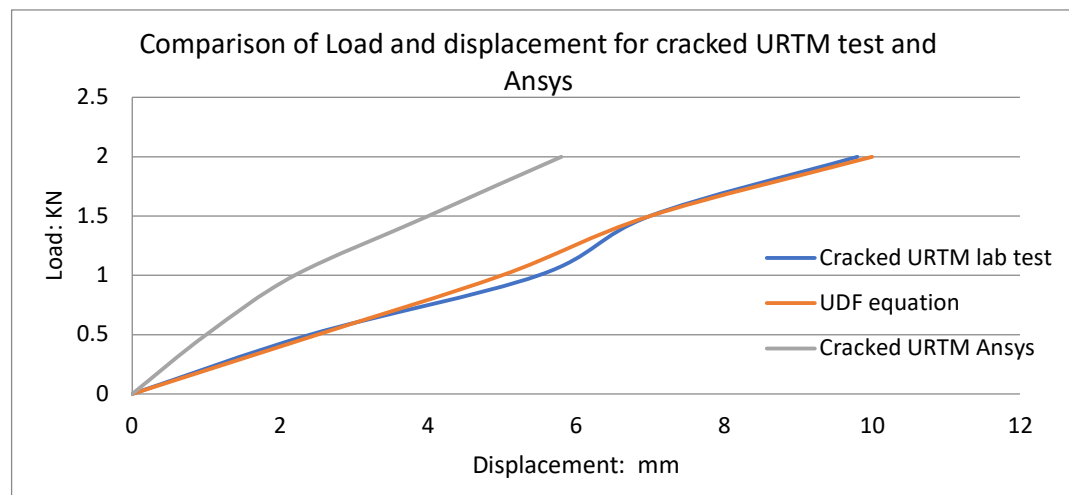


Figure 6.97: Comparison of cracked URM triplet, lab test and UDF function

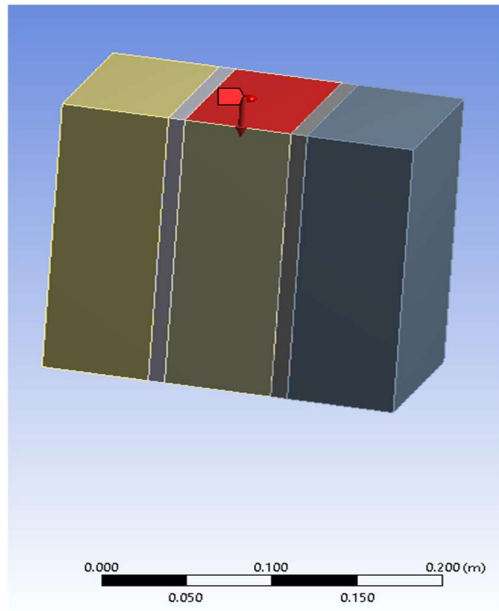


Figure 6.98: Cracked URM triplet

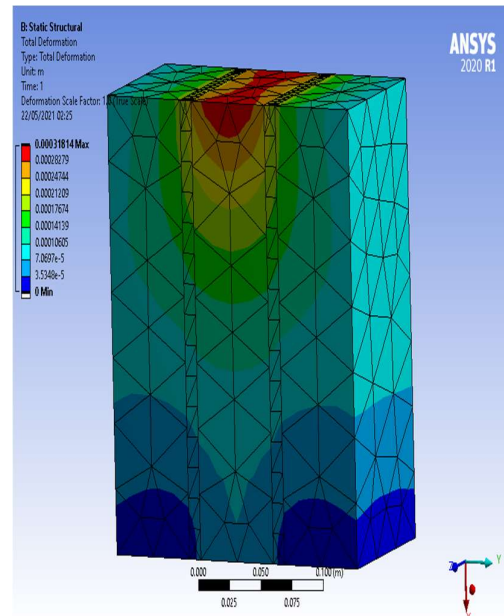


Figure 6.99: Sliding by Ansys without UDF

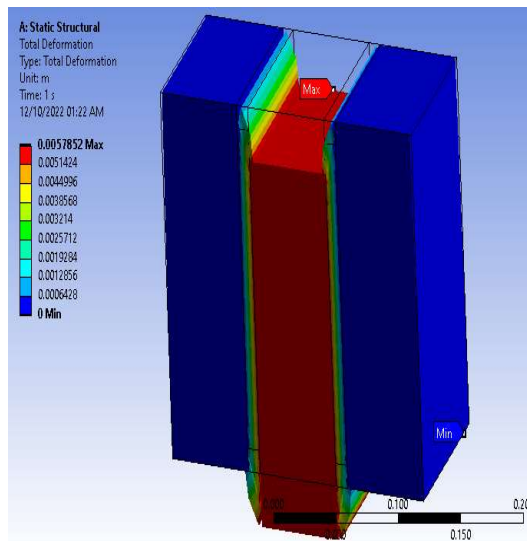


Figure 6.70: Sliding URTM triplet by UDF

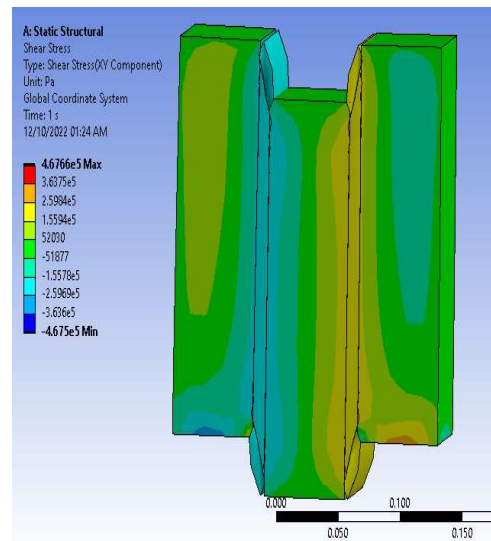


Figure 6.71: Shear stress URTM triplet by UDF

6.30. Numerical simulations of cracked masonry triplet reinforced with CFRP on two sides

Figures 6.72 illustrates a comparison of the results for one-layer CFRP on two sides of cracked triplet masonry in the experimental tests and theoretical study with and without UDF function. The result shows 3.8 mm, 3.6 mm, and 0.222 mm sliding at 50KN load respectively. Figure 6.76 shows, stress transfers from cracked and damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure of all triplets. The largest shear stress occurred at the middle brick, mortar and then CFRP composite. In absence of a bond between brick and mortar in a cracked triplet, the CFRP experienced relatively larger stress, indicating the interface delamination. The finite element modeling for all triplets with CFRP showed that the stiffness and load resistance of CFRP retrofitted cracked triplets increased linearly with the increase of CFRP composites. Triplets reinforced with CFRP on two sides are 3.5 times stronger than uncracked masonry triplets. It was also observed that the triplets with CFRP strengthening gave more stiffness as compared to URTM.

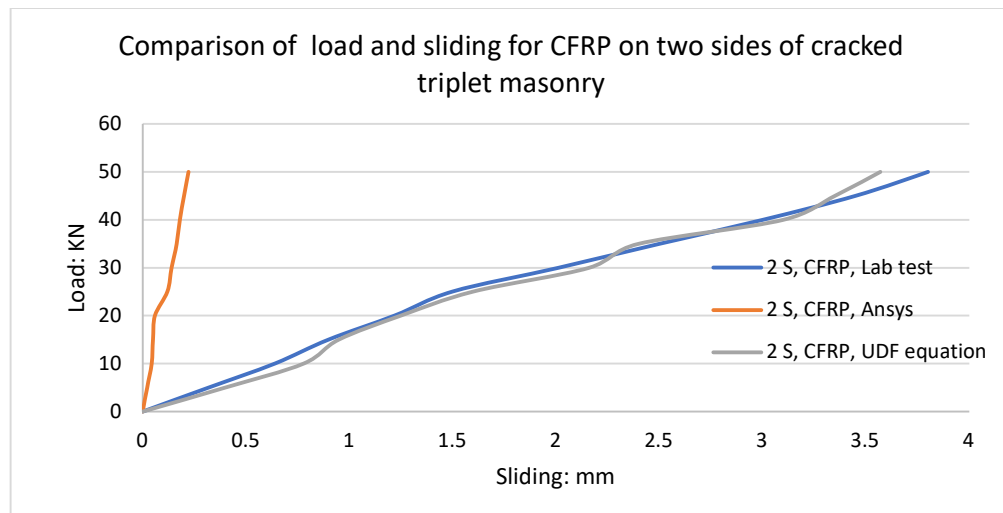


Figure 6.72: Comparison of Displacement for CFRP on Two sides of Cracked Triplet masonry.

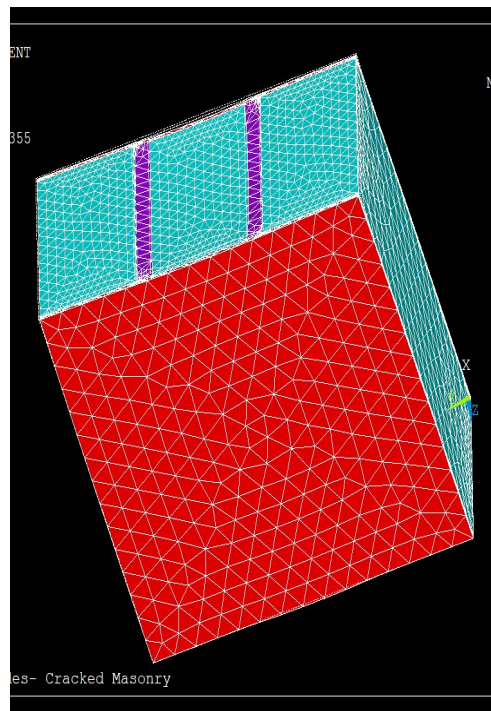


Figure 6.73: CFRP on two sides of Cracked triplet

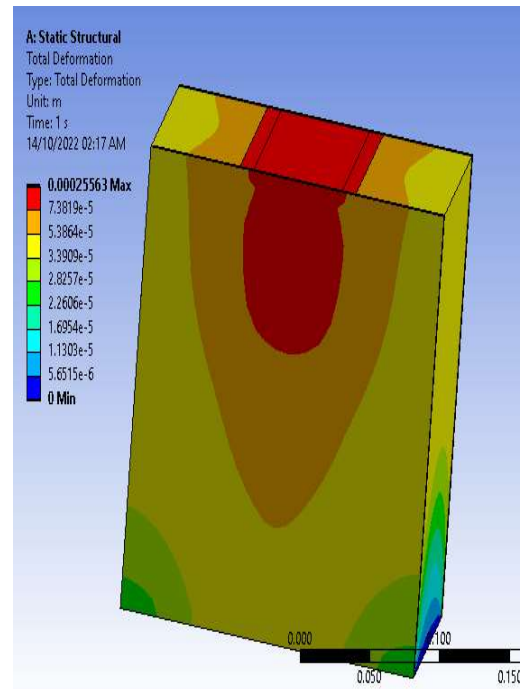


Figure 6.74: Sliding without UDF function

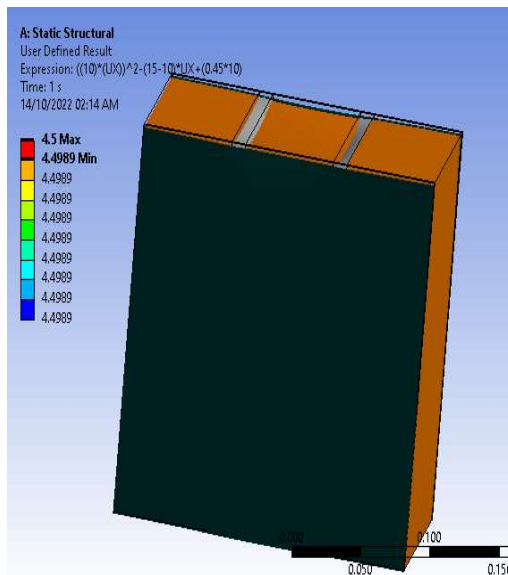


Figure 6.75: Sliding with UDF function

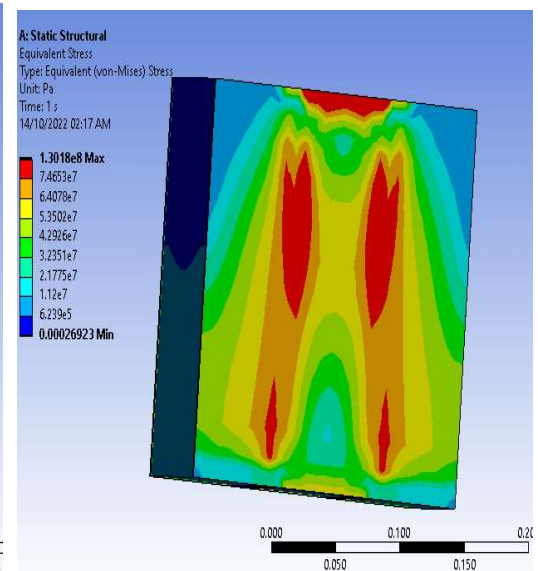


Figure 6.76: Shear stress, cracked triplet masonry

6.31. Numerical simulations of cracked masonry triplet reinforced with CFRP net on four sides

The displacement comparison of the theoretical modeling without and with UDF function and experimental test for CFRP net on four sides of cracked triplet masonry is shown in Figure 6.77. The sliding in experimental tests and numerical simulations with UDF function at 48 KN is 4.5 mm, but without UDF is 0.2 mm (Figures 6.79 and 6.80). As Figure 6.81 shows, stress transfers from cracked and damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure of all triplets. Also figure 6.81 does not show shear stress in the mortar between breaks which is normal for cracked and unbonded triplets. The largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in a cracked triplet, the CFRP experienced relatively larger stress. The finite element modeling for all triplets with CFRP showed that the stiffness and load resistance of CFRP retrofitted cracked triplets increased linearly with the increase of CFRP composites. Cracked triplets reinforced with CFRP net on four sides are 4.5 times stronger than uncracked masonry triplets.

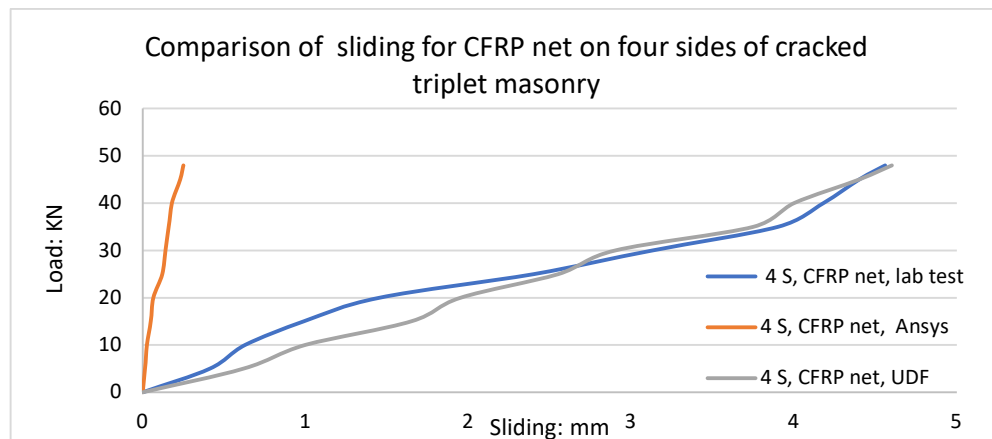


Figure 6.77: Comparison of sliding in lab the test and Ansys with and without UDF for CFRP net on four sides of cracked triplet masonry

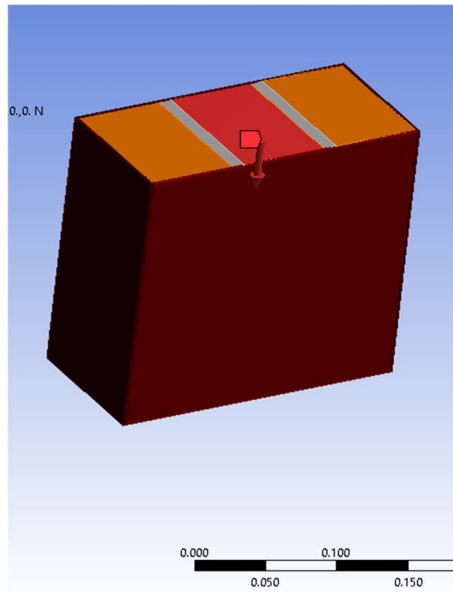


Figure 6.78: CFRP net on four sides of cracked triplet masonry

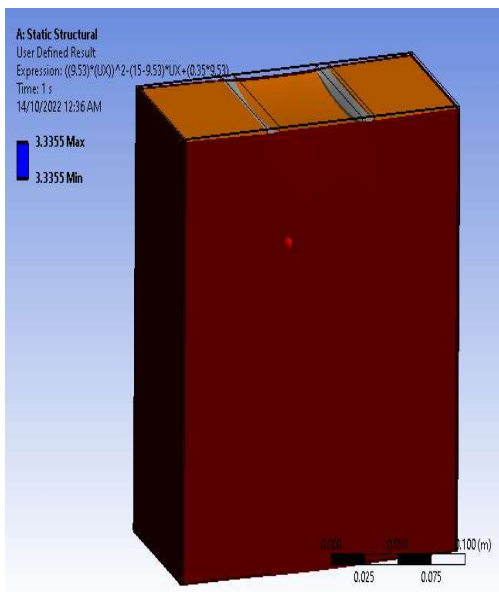


Figure 6.79: Sliding in cracked triplet with CFRP net on four sides using UDF function

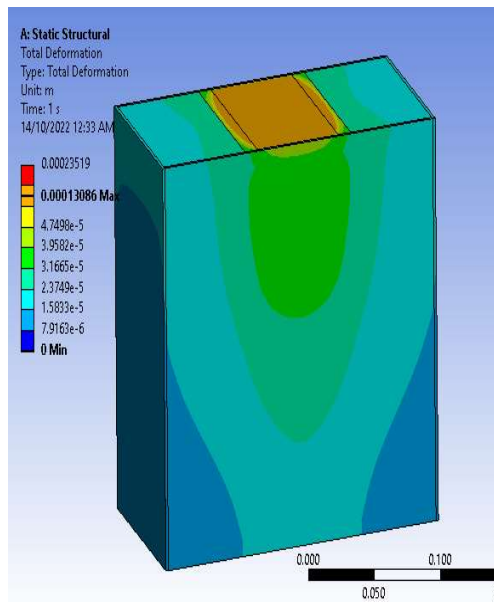


Figure 6.80: Sliding in cracked triplet with CFRP net on four sides without UDF function

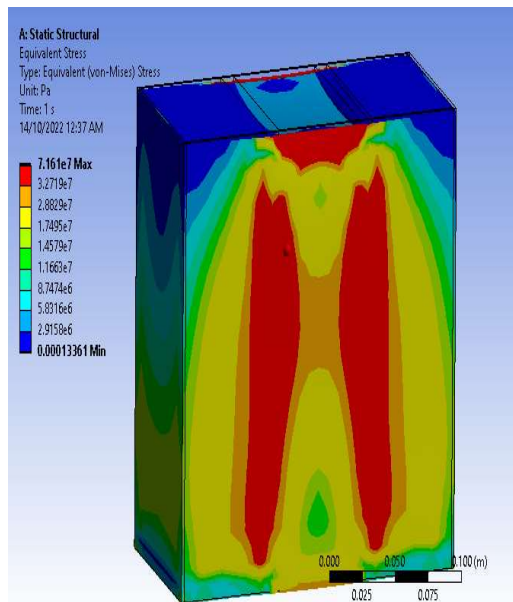


Figure 6.81: Stress in cracked triplet masonry with CFRP net on four sides

6.32. Numerical simulations of cracked masonry triplet reinforced with CFRP net on two sides

Figure 6.83 illustrates a displacement comparison of the theoretical modeling without and with UDF function and experimental test for CFRP net on two sides of cracked triplet masonry. The maximum sliding in experimental tests and numerical simulations with the UDF function at 45 KN is 5 mm, but without UDF is 0.2 mm (Figures 6.84 and 6.85).

As Figure 6.86 shows, stress transfers from cracked and damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure of all triplets. The finite element modeling for all triplets with CFRP showed that the stiffness and load resistance of CFRP retrofitted cracked triplets increased linearly with the increase of CFRP composites. Cracked triplets reinforced with CFRP net on two sides are 5 times stronger than uncracked masonry triplets. Also, figure 6.86 does not show shear stress in the mortar between breaks which is normal for cracked and unbonded triplets. The largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in a cracked triplet, the CFRP experienced relatively larger stress, indicating the interface delamination. It was also observed that the triplets with CFRP strengthening gave stiffness as compared to URTM.

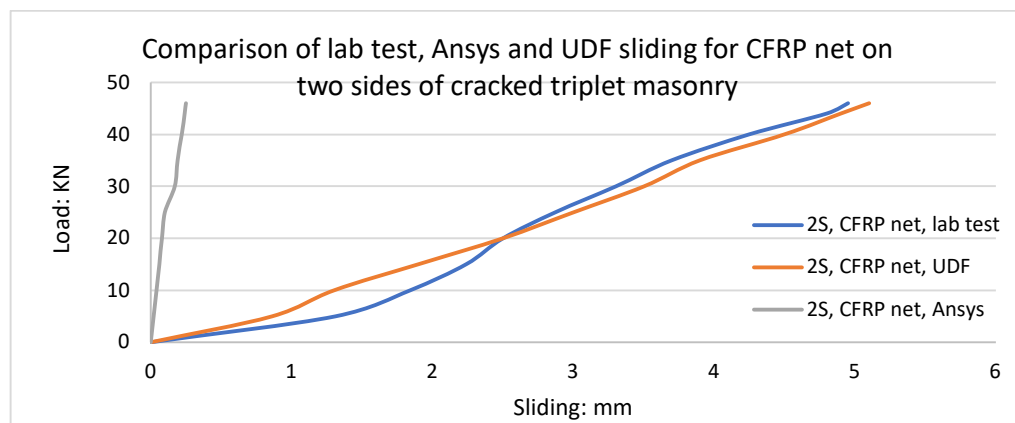


Figure 6.82: Comparison of lab test and UDF sliding for CFRP net on two sides of cracked triplet masonry.

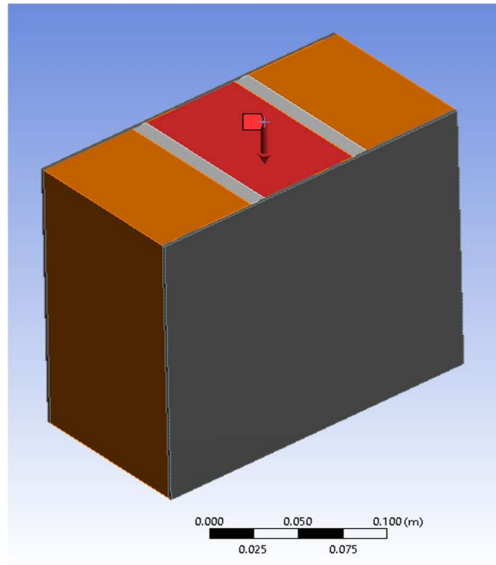


Figure 6.83: CFRP net on two sides of cracked triplet masonry.

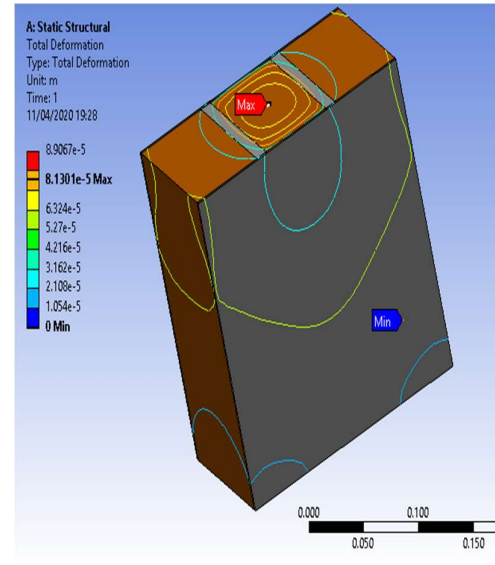


Figure 6.84: sliding of CFRP net on two sides of cracked triplet masonry without UDF

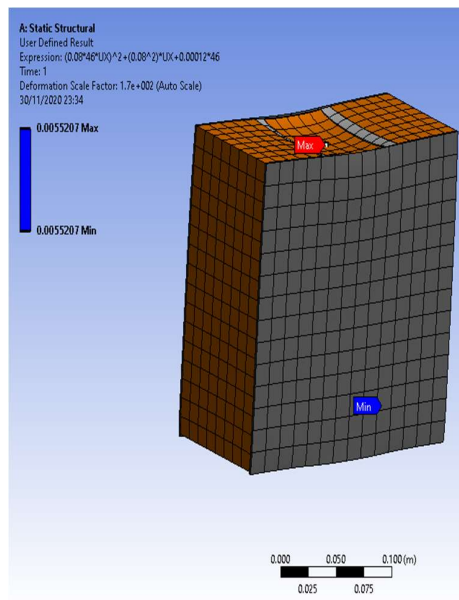


Figure 6.85: sliding of CFRP net on two sides of cracked triplet masonry with UDF

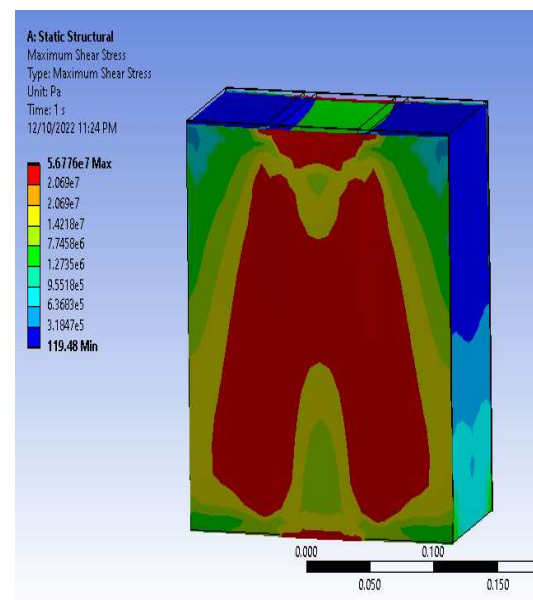


Figure 6.86: Stress of CFRP net on two sides of cracked triplet masonry with UDF

6.33. Numerical simulations of cracked masonry triplet reinforced with CFRP strips on four sides

Figure 6.87 illustrates the sliding comparison of the theoretical FE modeling without and with UDF function and experimental test for CFRP strips on four sides of cracked triplet masonry. The maximum sliding in experimental tests and numerical simulations with the UDF function at 42 KN is 7 mm, but without UDF is 0.5 mm (Figures 6.89 and 6.90).

As Figure 6.91 shows, stress transfers from cracked and damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure of all triplets. The largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in a cracked triplet, the CFRP experienced relatively larger stress, indicating the interface delamination. The finite element modeling showed that the stiffness and load resistance of CFRP retrofitted cracked triplets increased linearly with retrofitting CFRP composites. Cracked triplets reinforced with CFRP strips on four sides are 4 times stronger than uncracked masonry triplets. Also, figure 6.91 shows stress distribution in triplet, stress from middle brick transfers to CFRP composite, and because of deboned triplet before reinforcement.

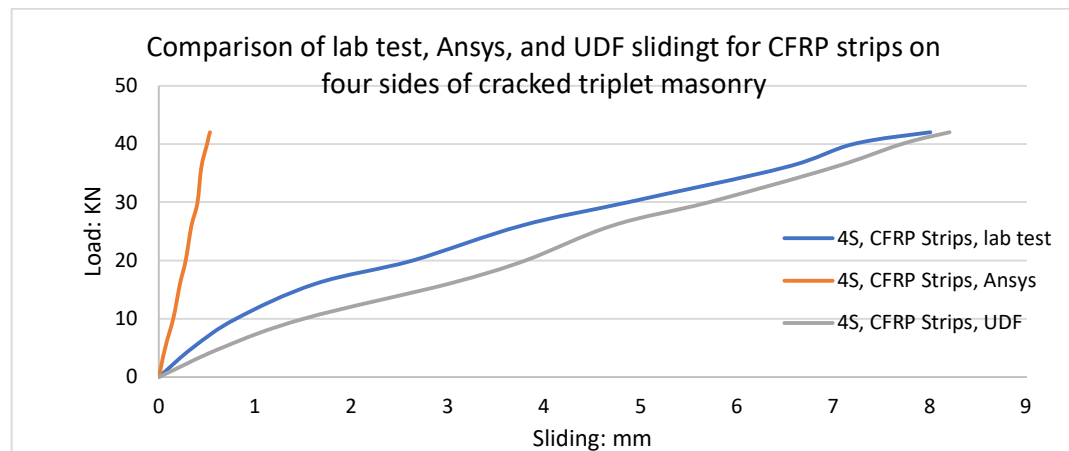


Figure 6.87: Sliding comparison of lab test and FE modelling with and without UDF function for CFRP strips on four sides of cracked triplet masonry.

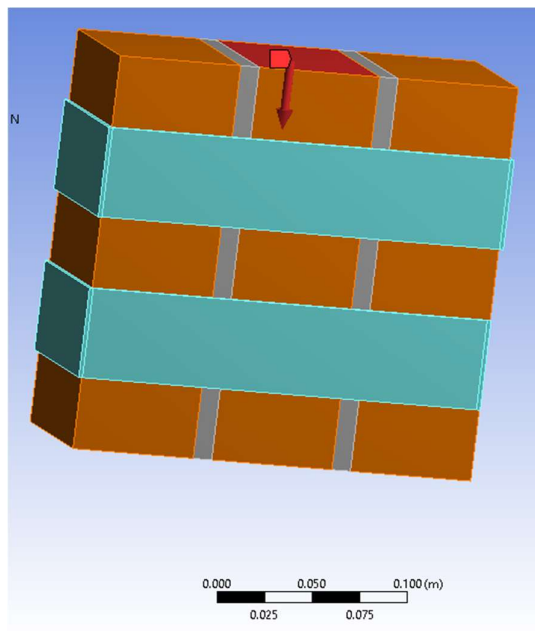


Figure 6.88: CFRP strips on four sides of cracked triplet masonry

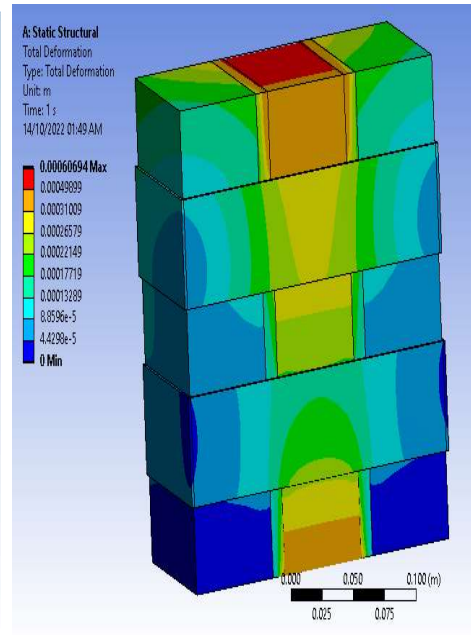
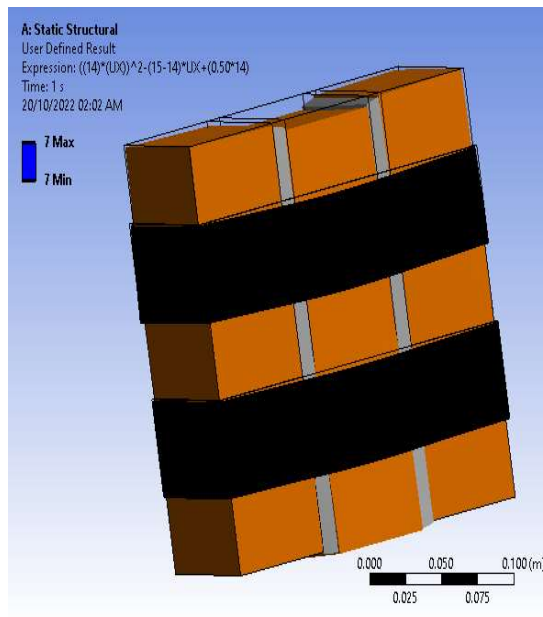


Figure 6.89: Sliding of CFRP strips on four sides of cracked triplet without UDF



Figures 6.90: Sliding of CFRP strips on four sides of cracked triplet with UDF function

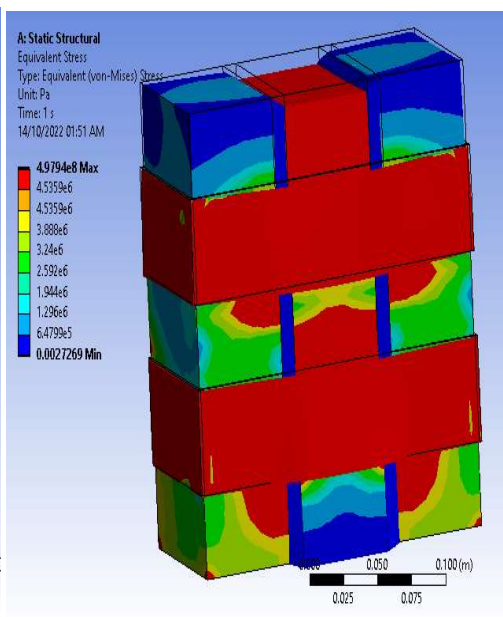


Figure 6.91: Stress of CFRP strips on four sides of cracked triplet

6.34. Numerical simulations of cracked masonry triplet reinforced with CFRP strips on two sides

Figure 6.92 illustrates a displacement comparison of the theoretical modeling without and with UDF function and experimental test for CFRP strips on two sides of cracked triplet masonry. The maximum sliding in experimental tests and numerical simulations with the UDF function at 38 KN is 5 mm, but without UDF is 0.277 mm, as shown in Figures 6.94 and 6.95.

Figure 6.96 shows, stress transfers from cracked and damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure. The finite element modeling for triplets with CFRP strips on two sides showed that the stiffness and load resistance of CFRP retrofitted cracked triplets increased significantly. Cracked triplets reinforced with CFRP strips on two sides are 4 times stronger than uncracked masonry triplets. Also, figure 6.96 does not show shear stress in the mortar between breaks which is normal for cracked and unbonded triplets. The largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in cracked triplets, the CFRP experienced relatively larger stress, indicating the interface delamination.

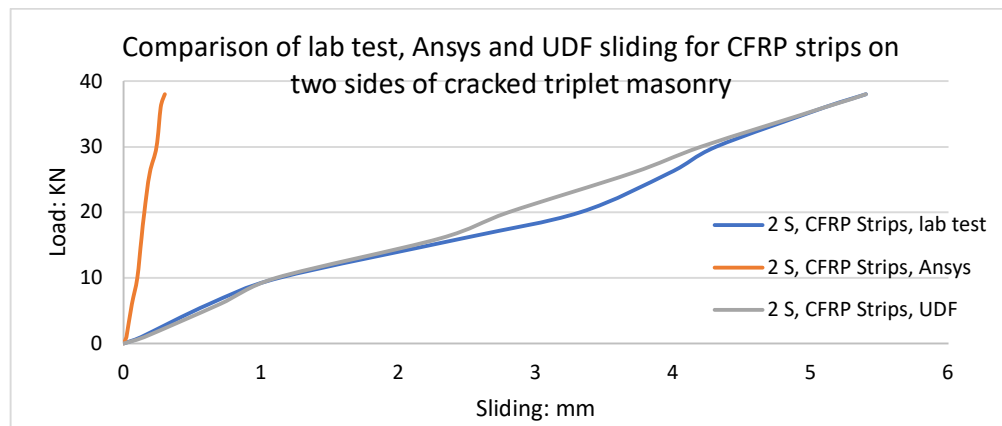


Figure 6.92: Sliding comparison of lab test and UDF function for CFRP strips on two sides of cracked triplet masonry.

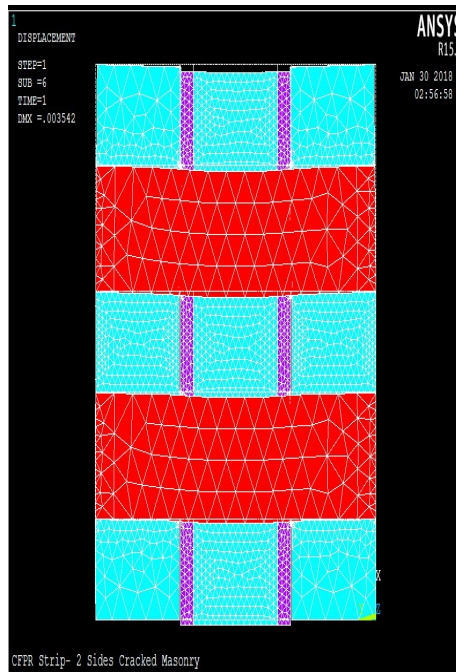


Figure 6.93: CFRP strips on two sides of cracked triplet masonry

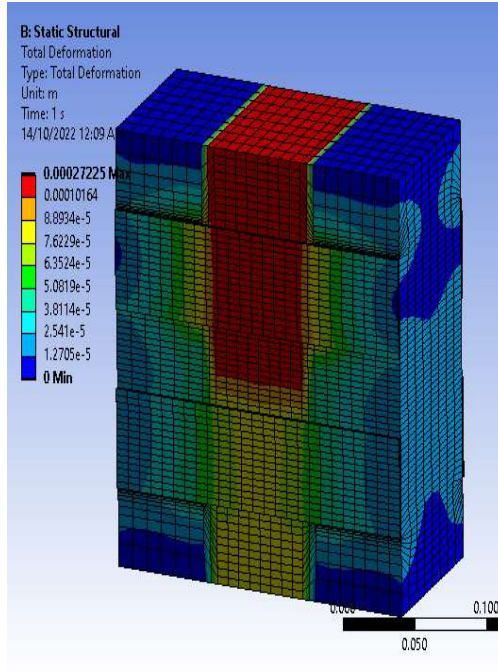


Figure 6.94: Sliding without UDF function

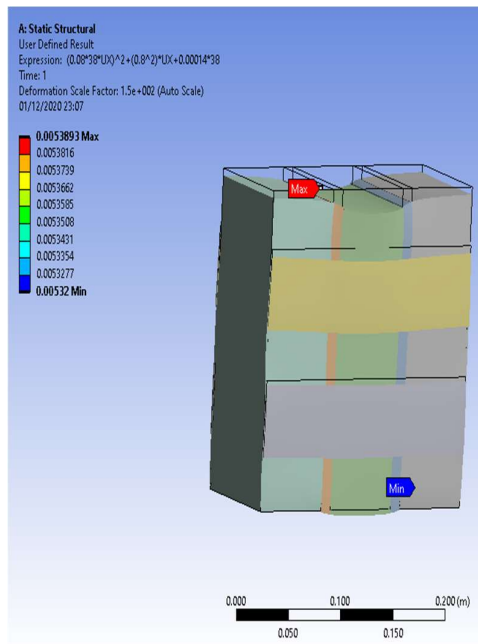


Figure 6.95: Sliding with UDF function

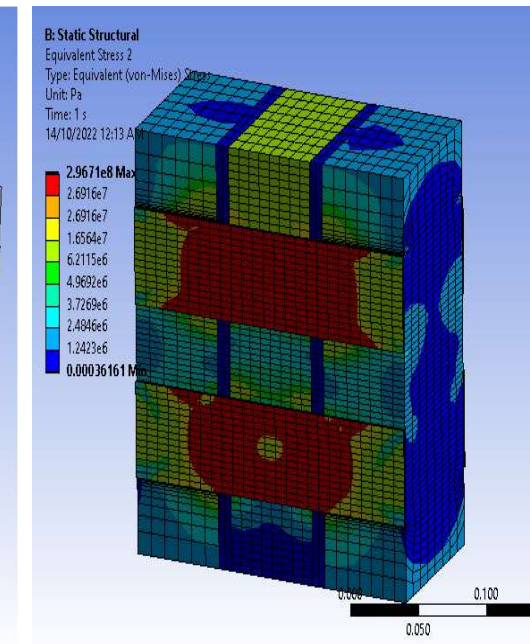


Figure 6.96: Stress of CFRP strips on two sides of cracked triplet masonry

6.35. Numerical simulations of cracked masonry triplet reinforced with CFRP net strips on four sides

Figure 6.97 illustrates the sliding comparison of the numerical simulations with and without UDF function and experimental lab test for CFRP strips on four sides of cracked deboned triplet masonry. The maximum sliding in experimental tests and numerical simulations with the UDF function at 48 KN is 3.5 mm, but without UDF is 0.6 mm, as shown in Figures 6.99 and 6.100.

Figure 6.101 shows, stress transfers from cracked/damaged triplet to CFRP composites and debonding of CFRP from specimens is the main reason for the failure. The finite element modeling for triplets with CFRP strips on four sides showed that the stiffness and load resistance of CFRP retrofitted cracked triplets increased significantly. Cracked triplets reinforced with CFRP strips on four sides are 4.5 times stronger than uncracked masonry triplets. Also, figure 6.101 shows stress distribution in triplets, stress from middle brick transfers to CFRP composite, and because of deboned triplet before reinforcement, shear stress in the mortar between breaks is very low, and the largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in a cracked triplet, the CFRP experienced relatively larger stress, indicating the interface delamination.

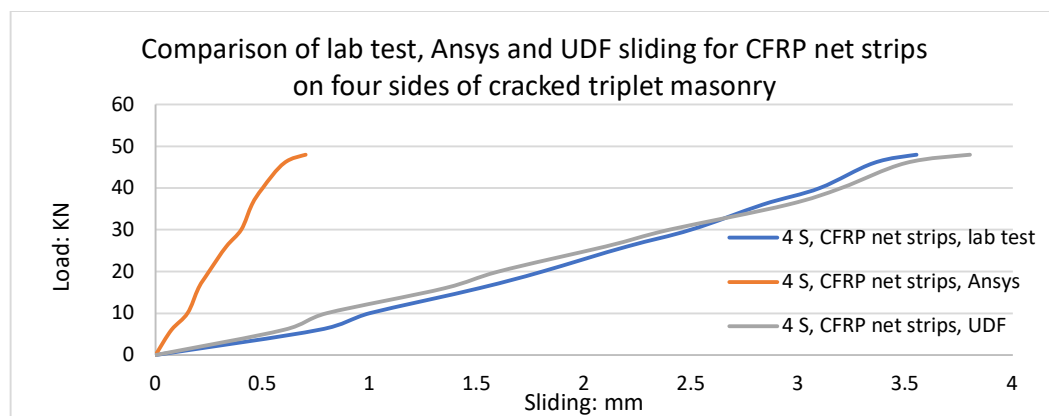


Figure 6.97: Comparison of lab test, Ansys and UDF for CFRP net on four sides of cracked triplet

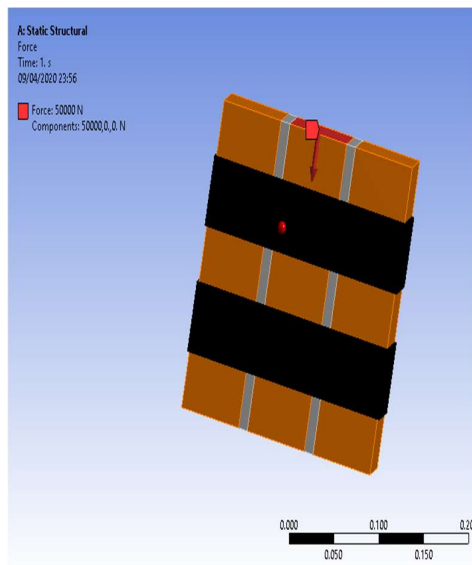


Figure 6.98: CFRP strips on four sides of cracked triplet masonry

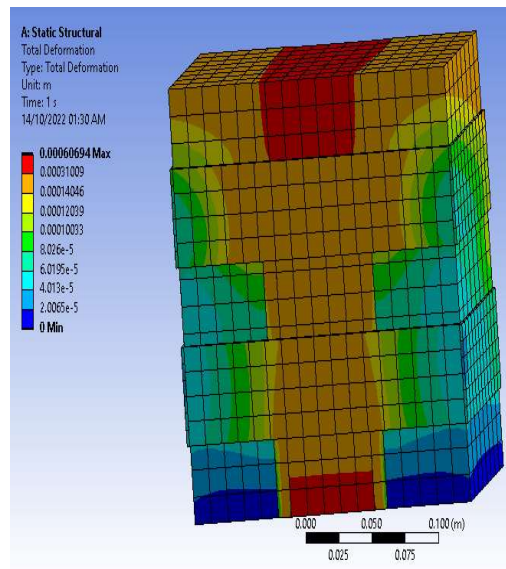


Figure 6.99: Sliding without UDF function

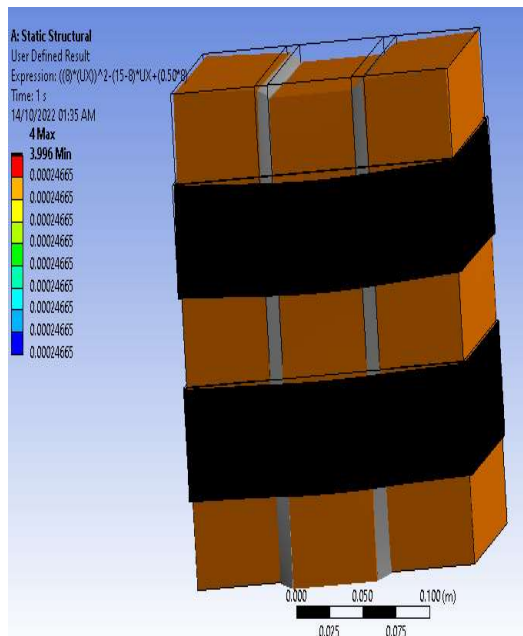


Figure 6.100: Sliding with UDF function

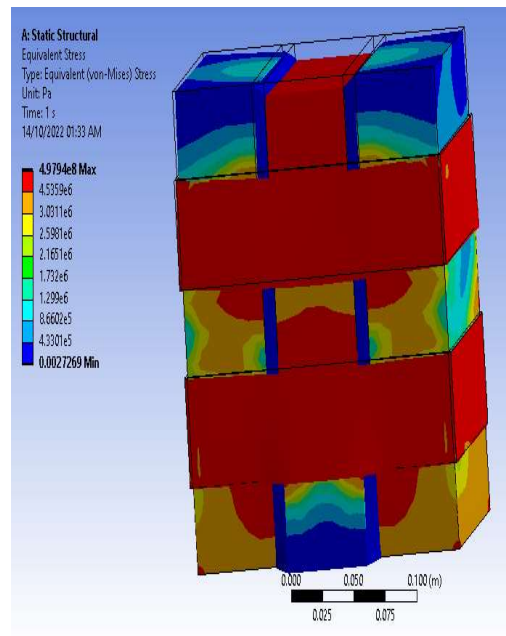


Figure 6.101: Stress of CFRP net on four sides

6.36. Numerical simulations of cracked masonry triplet reinforced with CFRP net strips on two sides

Figure 6.102 shows a sliding comparison of the numerical simulations with and without UDF function and experimental lab test for CFRP net strips on two sides of cracked triplet masonry. The maximum sliding in experimental tests and numerical simulations with the UDF function at 52 KN is 6 mm, but without UDF is 0.37 mm, as shown in Figures 6.104 and 6.105.

Figure 6.106 shows that stress transfers from cracked/damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure. The finite element modeling for triplets with CFRP net strips on two sides showed that the load resistance of CFRP retrofitted cracked triplets increased significantly. Cracked triplets reinforced with CFRP strips on four sides are 4.5 times stronger than uncracked masonry triplets. Also figure 6.106 shows stress distribution in the triplet, stress from the middle brick transfers to CFRP composite, and because of the deboned triplet before reinforcement, shear stress in the mortar between breaks is very low, and the largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in cracked triplets, the CFRP experienced relatively larger stress, indicating the interface delamination.

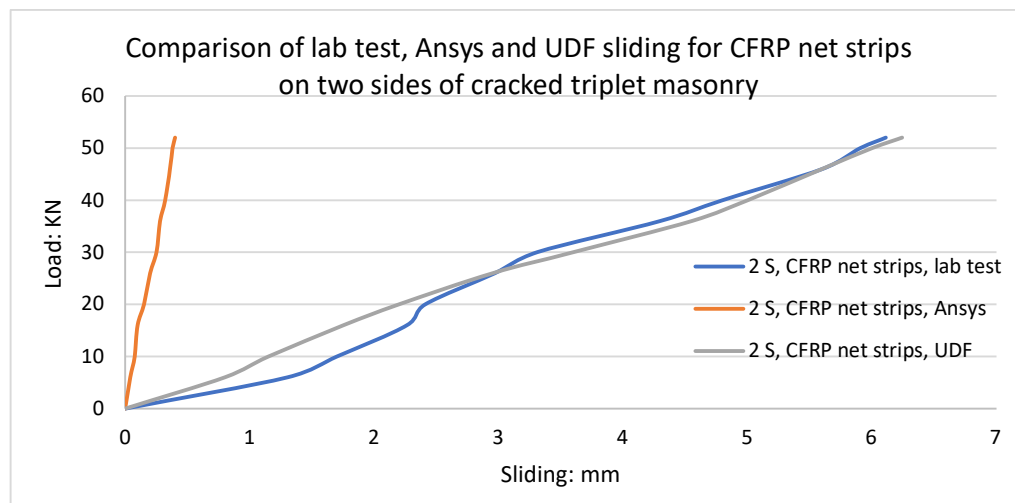


Figure 6.102: Comparison of lab test and UDF sliding for CFRP net strips on two sides of cracked triplet masonry.

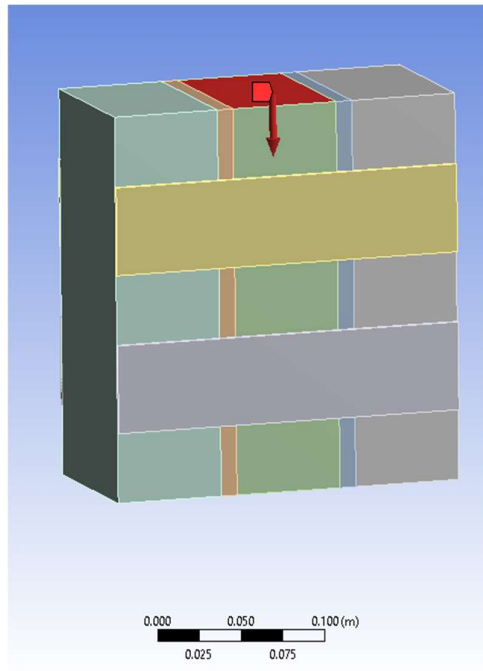


Figure 6.103: CFRP net strips on two sides of cracked triplet masonry

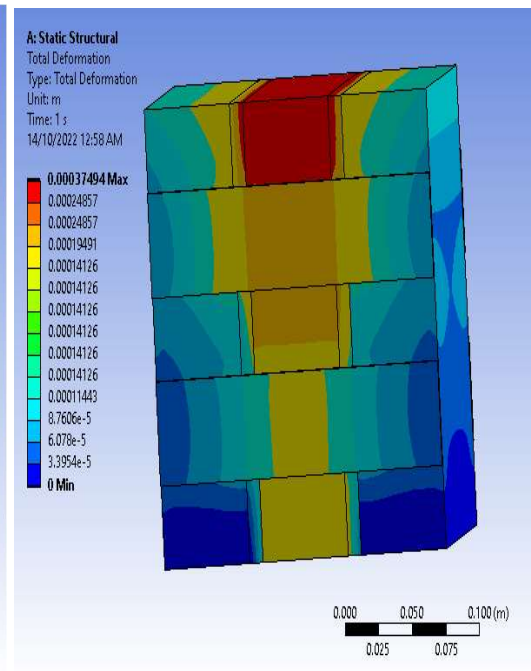


Figure 6.104: Sliding without UDF function

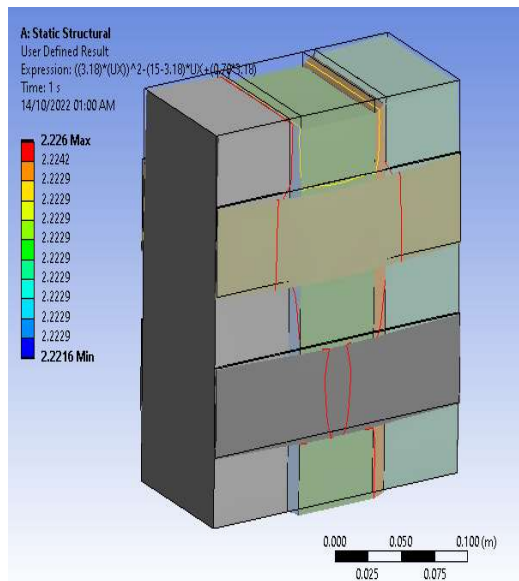


Figure 6.105: Sliding with UDF function

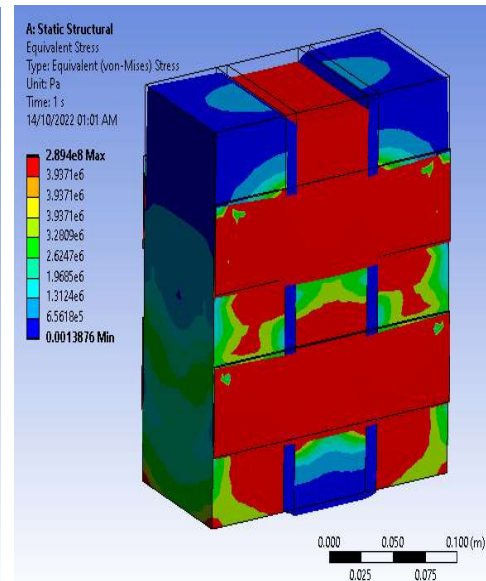


Figure 6.106: Stress of CFRP net on two sides

6.37. Numerical simulations of cracked masonry triplet reinforced with CFRP on four sides

Figure 6.102 shows the sliding comparison of the numerical simulations with and without UDF function and experimental lab test for CFRP net strips on two sides of cracked triplet masonry. The maximum sliding in experimental tests and numerical simulations with the UDF function at 52 KN is 6 mm, but without UDF is 0.37 mm, as shown in Figures 6.104 and 6.105.

Figure 6.106 shows that stress transfers from cracked/damaged triplets to CFRP composites and debonding of CFRP from specimens is the main reason for the failure. The finite element modeling for triplets with CFRP net strips on two sides showed that the load resistance of CFRP retrofitted cracked triplets increased significantly. Cracked triplets reinforced with CFRP strips on four sides are 4.5 times stronger than uncracked masonry triplets. Also figure 6.106 shows stress distribution in the triplet, stress from the middle brick transfers to CFRP composite, and because of the deboned triplet before reinforcement, shear stress in the mortar between breaks is very low, and the largest shear stress occurred at the middle brick and CFRP composite. In absence of a bond between brick and mortar in a cracked triplet, the CFRP experienced relatively larger stress.

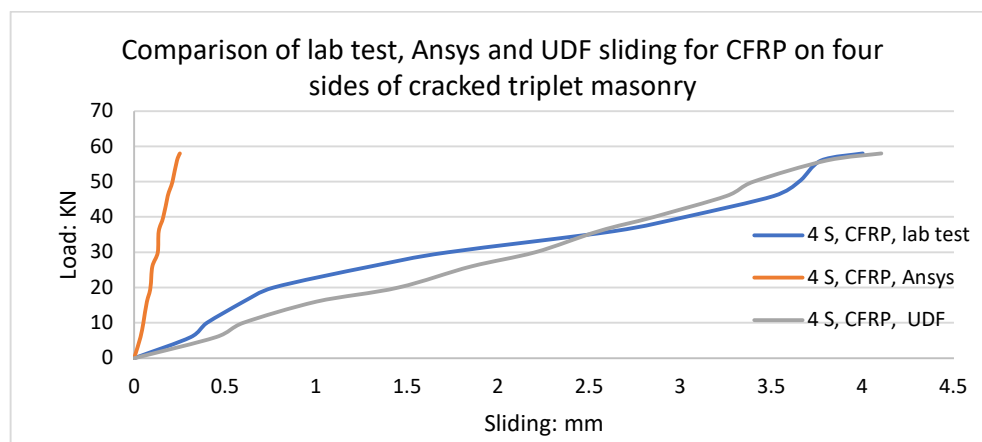


Figure 6.107: Comparison of lab test, Ansys and UDF sliding for CFRP on four sides of cracked triplet masonry

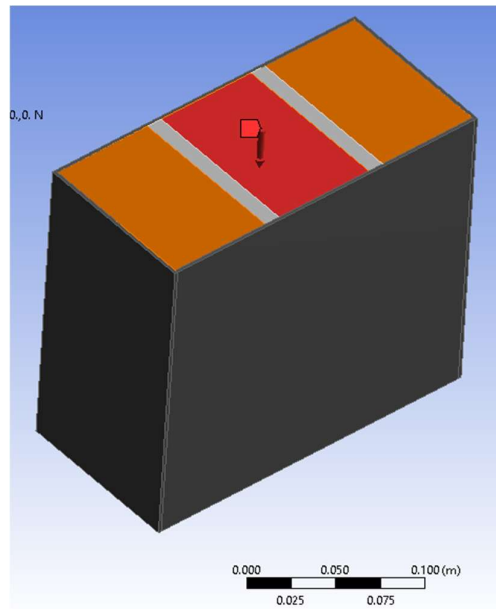


Figure 6.108: CFRP on four sides of cracked triplet masonry

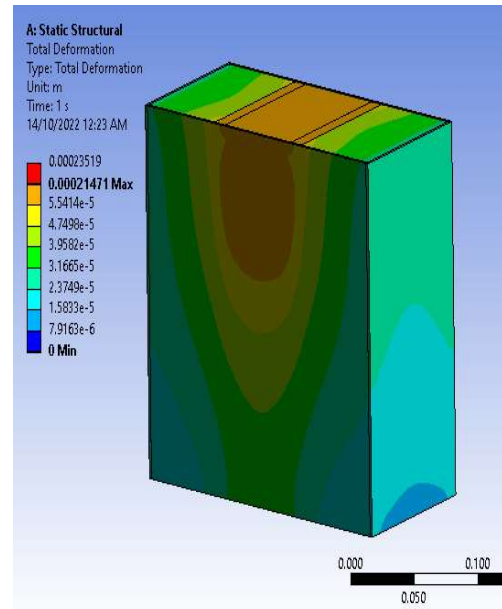


Figure 6.109: Sliding without UDF function

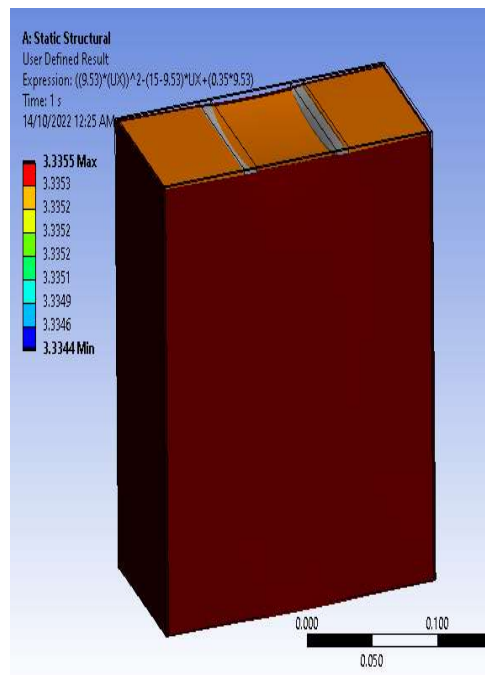


Figure 6.110: Sliding with UDF function

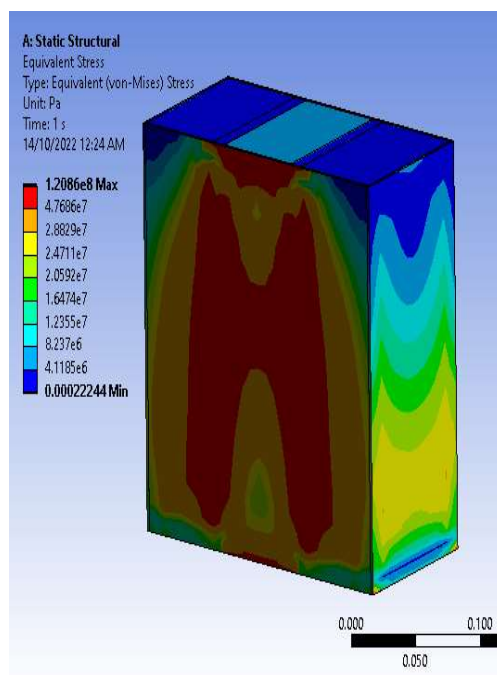


Figure 6.111: Stress of CFRP on four sides

Figure 6.112 illustrates the comparison of numerical simulations at maximum stress, load, and displacement of cracked masonry triplets with different CFRP configurations at failure points under shear loading.

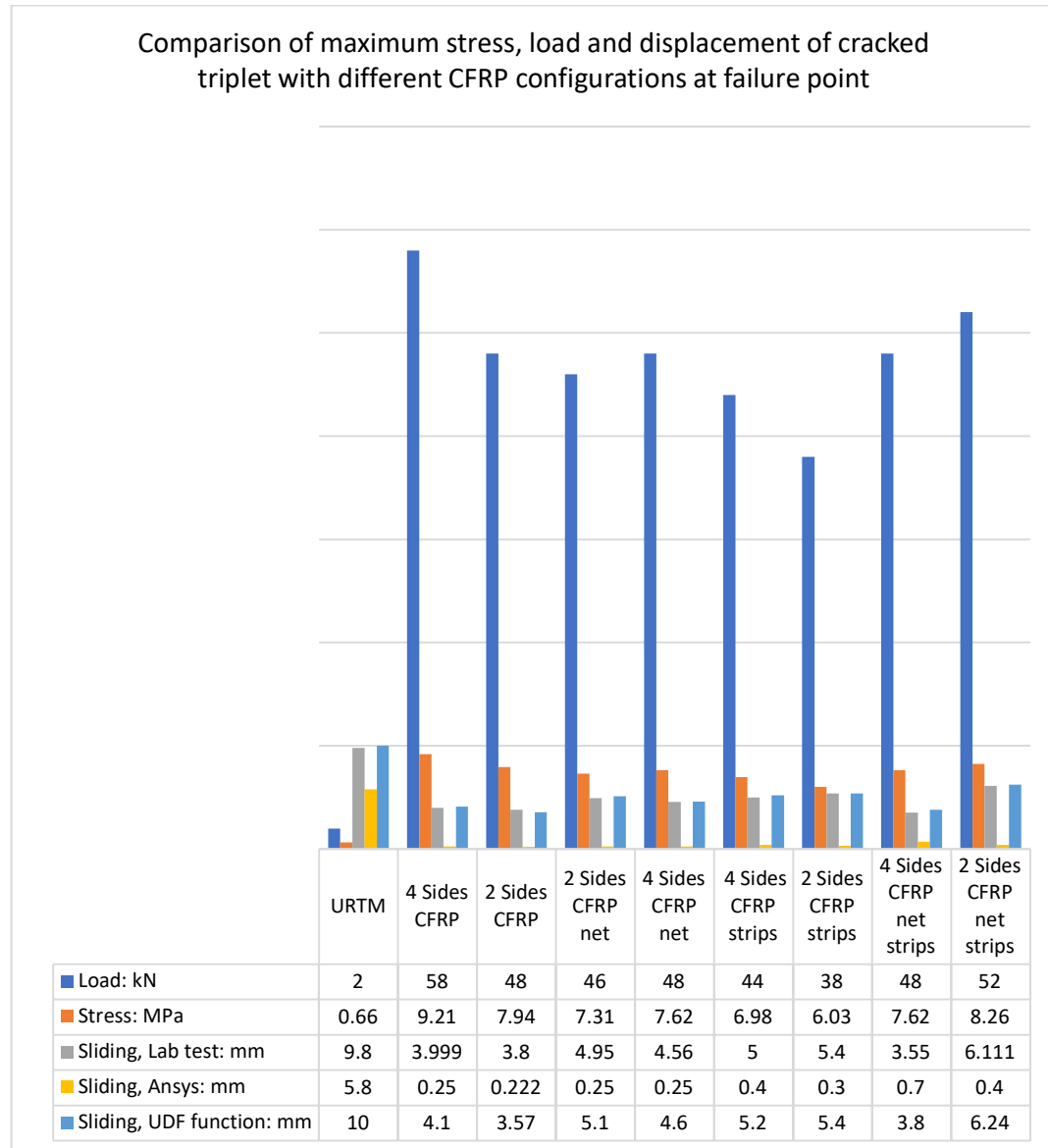


Figure 6.112: Comparison of stress, load and displacement of cracked triplet masonry

Table 6.5 illustrates the cracked triplet results of sliding from Ansys, UDF function, and lab tests at failure point and error for all CFRP configurations. This table shows acceptable errors and a good agreement between new UDF functions and lab tests for shear loading.

	CFRP configurations on triplet masonry (one layer)	Load at failure point (KN)	Lab test sliding (mm)	Ansys sliding (mm)	UDF function sliding (mm)	Error, Ansys (%)	Error, UDF function (%)
1	URTM	2	9.8	5.8	10	41	2
2	CFRP on two sides	50	3.8	0.222	3.57	94	6
3	CFRP on four sides	58	4	0.25	4.1	93.7	2.4
4	CFRP strips on two sides	38	5.4	0.3	5.4	94	0
5	CFRP strips on four sides	42	8	0.53	8.2	93	2.44
6	CFRP net on two sides	46	4.95	0.25	5.1	95	2.94
7	CFRP net on four sides	48	4.56	0.25	4.6	94.5	0.87
8	CFRP net strips on two sides	52	6.11	0.4	6.24	93	2.08
9	CFRP net strips on four sides	48	3.55	0.7	3.8	85.5	6.5

Table 6.5: Comparison the newly developed formula and lab tests at failure point

6.38. conclusions

6.38.1. Summarized results for compression and shear triplets

The finite element analysis was used to improve understanding and prediction of the mechanism of shear, sliding and CFRP debonding that can occur with CFRP strengthened triplets. The finite element analysis showed good characterization of the behavior of masonry triplet strengthened with CFRP and the UDF approach for shear loading and has demonstrated good agreement with the observed failure mechanisms.

1. The comparison of the load-displacement graphs from the theoretical modeling with the results from the experimental tests were not accurate for London solid brick and mortar M4, thus the new equations for finite element modeling were developed to predict correct sliding.
2. The interfacial shear stress concentrations between the brick-and-mortar surfaces are reduced due to retrofitting of shear triplets with CFRP composites and it was also found that the strengthened triplets had greater ductility than the URM triplet.
3. Shear strength of triplets reinforced with the same amount in terms of area reinforced with CFRP and CFRP net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity.
4. Comparison of the finite element analyses by developed UDF functions, new theoretical equations and experimental tests for reinforced triplet masonry with different CFRP configurations showed good agreement. The new functions make it possible to predict correct sliding/slipping in 3D modeling at any stress/load point and the errors are between 5% to 25%.

6.38.2. . Summarized results for cracked triplets

1. Repaired cracked triplets can maintain resistance up to similar sliding values as undamaged strengthened walls.
2. The triplet samples repaired with CFRP and tested under shear loading showed significant improvement in bond characteristics, strength, and ductility. The test load capacity of triplet masonry increased by 3.5 to 5 times in comparison with samples without initial damage.
3. Shear strength of cracked triplets strengthened with the same amount in terms of area reinforced with CFRP and CFRP Net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity. However, the gain in shear strength increased with an increasing amount of CFRP.
4. Comparison of the load-displacement graphs from the theoretical modeling with the corresponding results from the experimental investigation were not accurate, thus the new equations (UDF) were developed.

Chapter 7

**Finite element analyses of URM walls
using newly developed functions**

7.1. Finite element analyses by ANSYS and new UDF functions for URM and reinforced 4.00×3.00×0.10m masonry wall

The in-plane reaction of masonry walls depends mostly on their failure mechanisms. In case of poor-quality mortar, seismic loads or shear loads, the wall starts cracking and moving or sliding the upper part on the bottom part. This behaviour is called sliding shear failure. This theoretical study was designed to investigate the in-plane strength of URM brick walls on large scale. Based on experimental data on triplet masonry, the walls were modelled by FE software with new UDF functions using solid bricks and mortar reinforced with CFRP sheets and strips. The CFRP composites are applied every 50cm for strengthening the structure. The technique of strengthening with CFRP sheets, increased the strength of the wall four times and three times for walls strengthened with CFRP strips, compared to the URM masonry structure. In this section, the finite element modelling was based on triplet failure. The failure of triplets is the result of bond failure. Therefore, in finite element modelling, the lateral load was increased up to the failure point in triplets and the wall was fixed at the bottom. According to the results from chapter 6 that shows theatrical analyses by ANSYS were too low. Thus, this result can be corrected by using functions 6.16.1, 6.16.2 and 6.16.4 in table 6.3 from chapter 6, for the URM function, CFRP sheets on two sides and CFRP strips on two sides of a masonry wall.

	CFRP configurations on Triplet Masonry (One Layer)	Equation number	Equation $f(UX) =$
1	URTM	6.16.1	$(\sigma_b * UX)^2 - (15 - \sigma_b) * UX + (\sigma_b^2) = 0$
2	CFRP on two sides	6.16.2	$(\sigma_b * UX)^2 - (15 - \sigma_b) * UX + (0.45 * \sigma_b) = 0$
3	CFRP strips on two sides	6.16.4	$(\sigma_b * UX)^2 - (15 - \sigma_b) * UX + (0.55 * \sigma_b) = 0$
UX (mm) represent Sliding and σ_b (MPa) is Bond Stress			

Table 7.1: The newly developed a user-defined function (UDF) to create correct 3D modelling for theoretical analyses.

7.2. Masonry wall without reinforcement

The results for unreinforced masonry walls are illustrated in Figures 7.1 to 7.4. As can be seen in the figures, the displacement in finite modeling without the UDF function is very low and it was corrected with user-defined function 6.16.1.

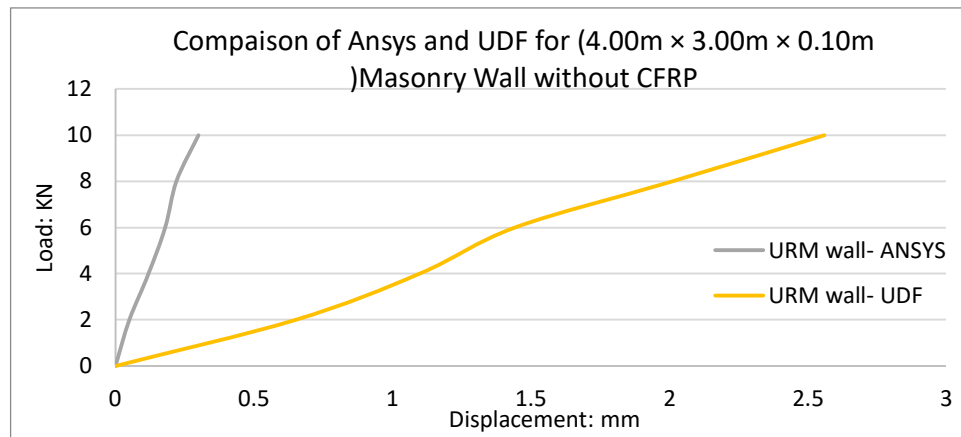


Figure 7.1: Comparison of Ansys and UDF for masonry wall without CFRP

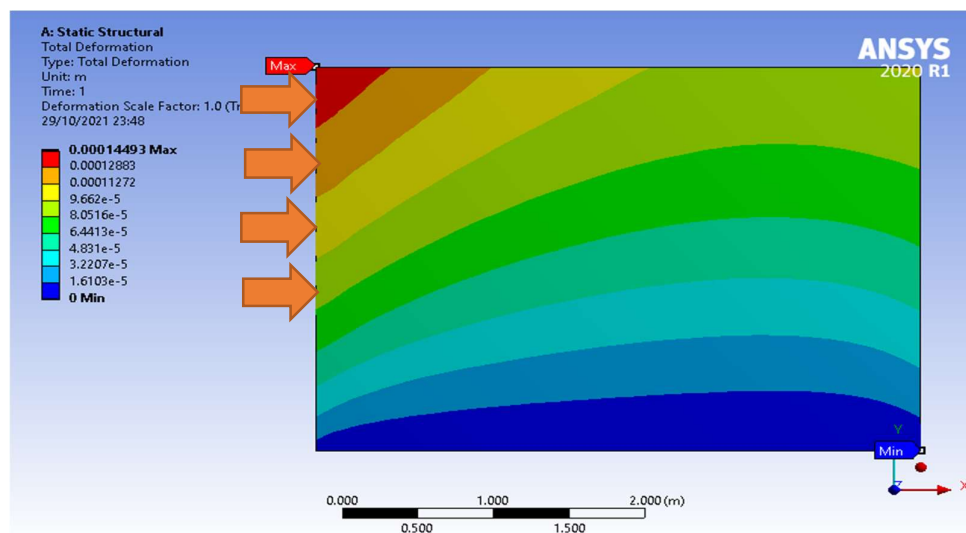


Figure 7.2: Displacement unreinforced masonry wall at 10 KN load without UDF function

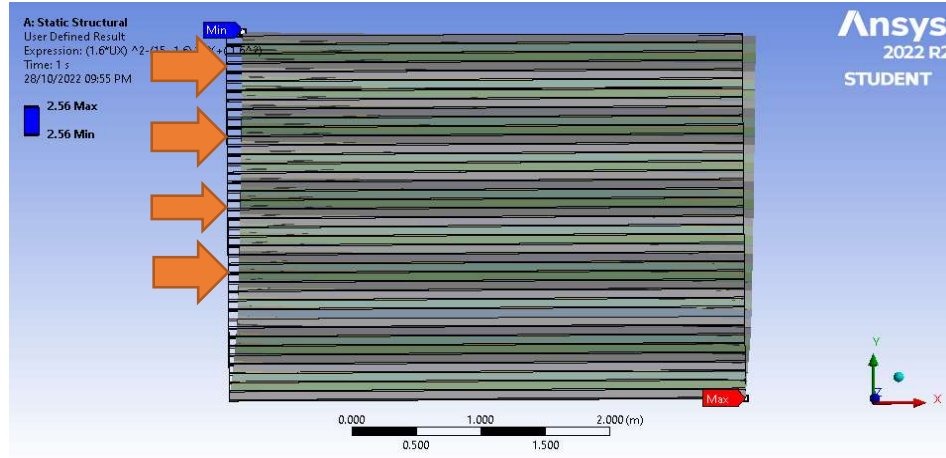


Figure 7.3: Displacement unreinforced masonry wall at 10 KN load with UDF function

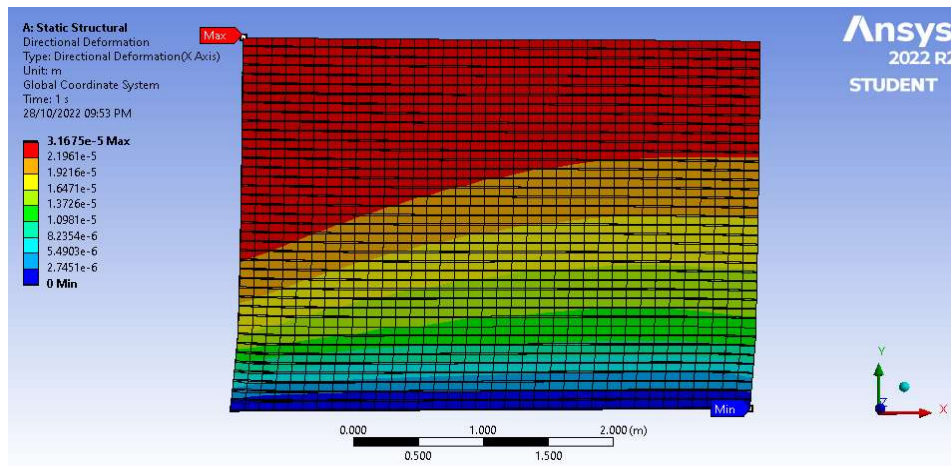


Figure 7.4: Stress for unreinforced masonry wall at 10 KN load

7.3. Masonry wall with reinforcement

The results for a masonry wall reinforced with CFRP sheets on both sides are illustrated in Figures 7.5 to 7.8. As can be seen in the figures, the displacement in finite modeling without the UDF function was very low and it was corrected with user-defined function 6.16.2. The FE results of masonry walls show that the stress transfers to CFRP composites and makes the stronger bond between brick and mortar, also the load resistance of CFRP retrofitted masonry structure was increased. The numerical simulations of walls showed that the UDF function can be used to predict the correct displacement of walls.

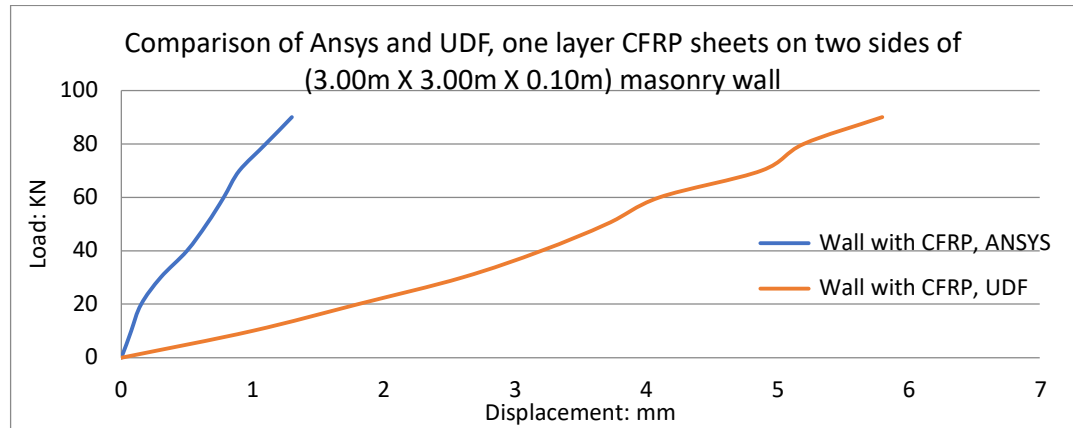


Figure 7.5: Comparison of Ansys and UDF for of masonry wall with CFRP sheets on both sides

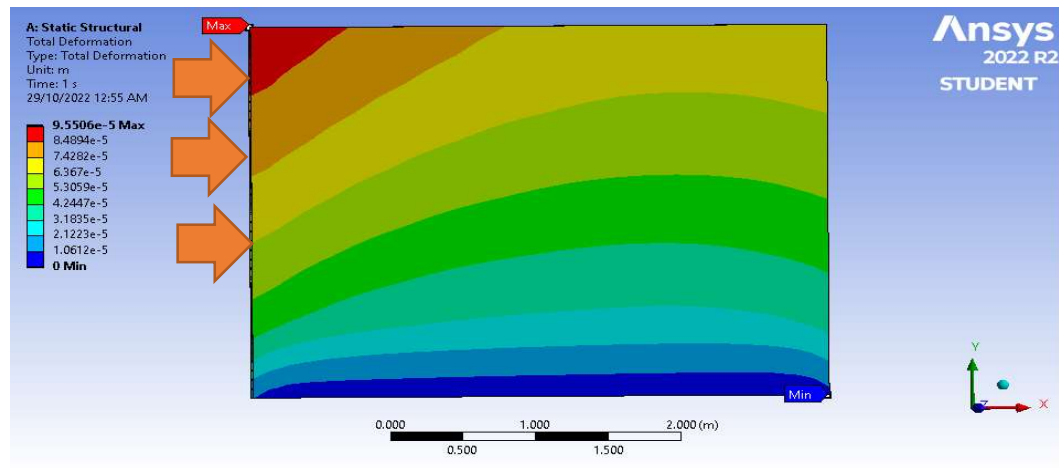


Figure 7.6: Displacement for CFRP sheets on two sides of a masonry wall at 90 KN load without UDF

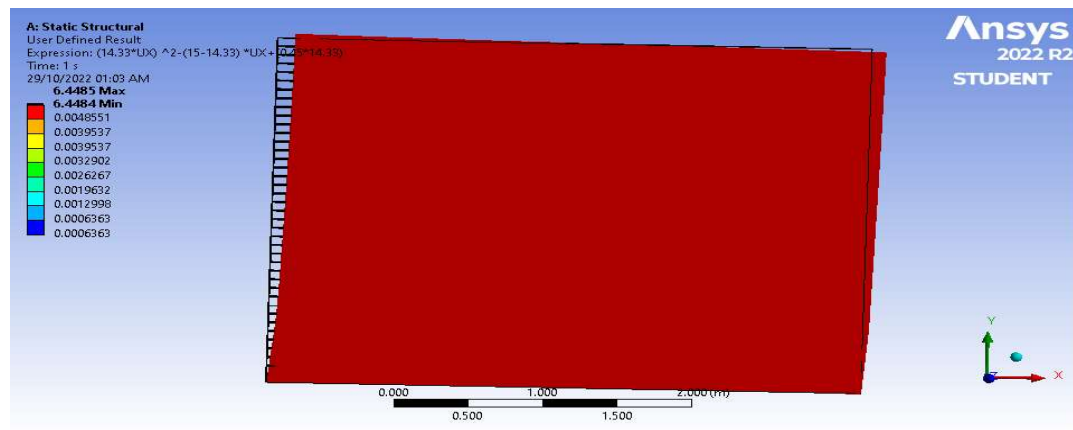


Figure 7.7: Displacement for CFRP sheets on two sides of a masonry wall at 90 KN with UDF function

Figure 7.8 illustrates the maximum shear stress in CFRP sheets at wall's failure point which shows sliding shear failure (sliding the upper part on the bottom part).

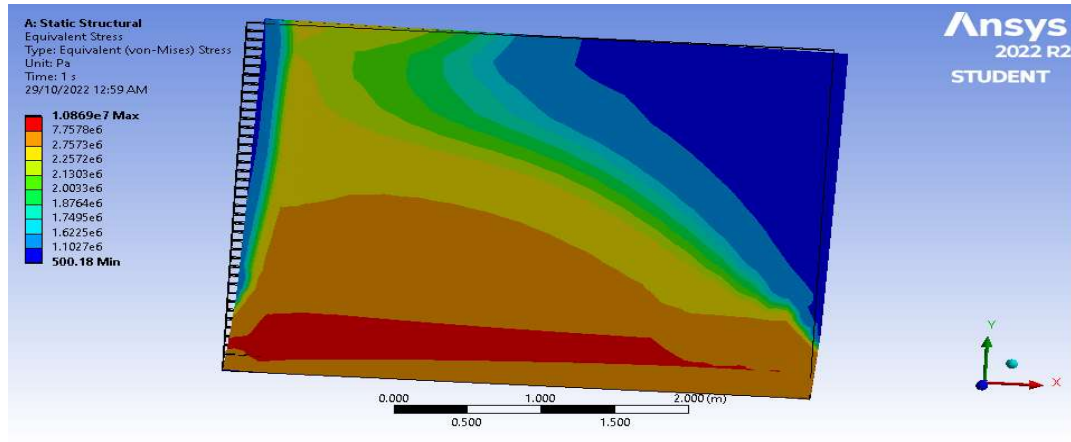


Figure 7.8: Stress for CFRP sheets on two sides of a masonry wall at 90 kN load with UDF function

7.4. Masonry wall with CFRP strips on both sides of wall

The results for a masonry wall reinforced with CFRP strips on both sides are illustrated in Figures 7.9 to 7.12. As can be seen in the figures, the displacement in finite modeling without the UDF function was very low and it was corrected with by user-defined function 6.16.4.

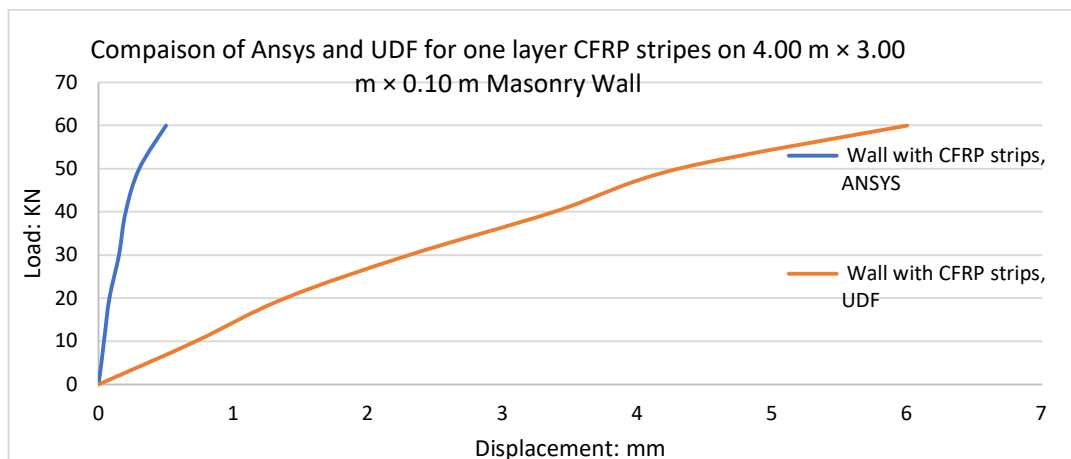


Figure 7.9: Comparison of Ansys and UDF for of masonry wall with CFRP strips on both sides

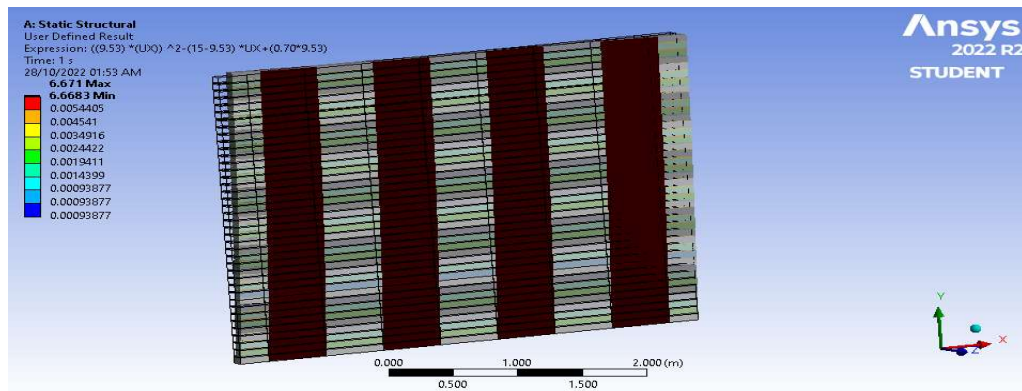
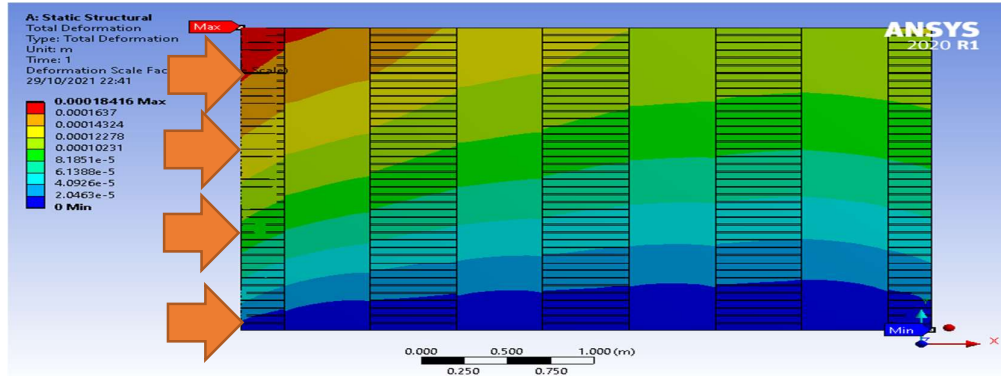
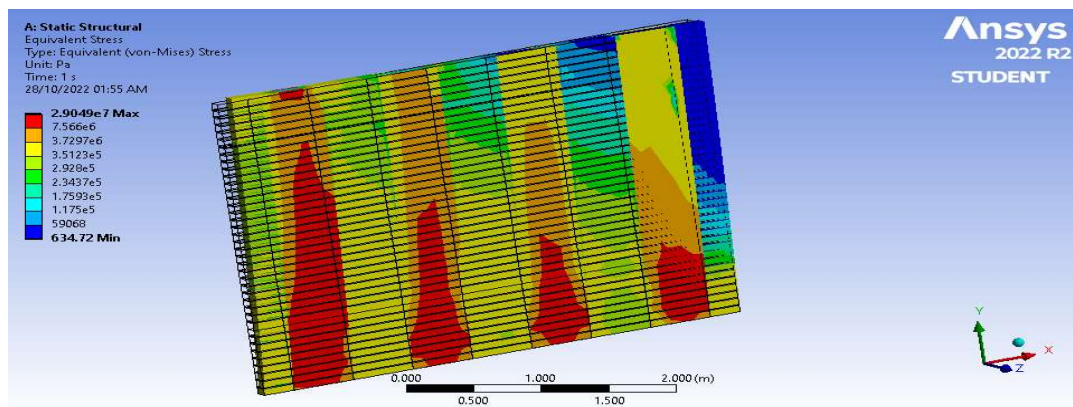


Figure 7.12 illustrates the maximum shear stress in CFRP strips at the wall's failure point which shows sliding shear failure (sliding the upper part on the bottom part).



Chapter 8

Conclusions, findings and recommendations

8.1. Research findings

This chapter provides a brief summary of the significant findings and conclusions of the experimental and analytical phases of this research program as well as suggestions for future work to expand the body of knowledge in the area of FRP-strengthening masonry walls. This thesis is also a contribution to improving the design of reinforced masonry structures with different CFRP configurations under shear loads.

The objective of this research was to investigate the compressive and shear behaviour of masonry with and without CFRP and to contribute to the understanding of the strengthened masonry walls with CFRP. The compressive and shear behaviour of CFRP-reinforced masonry made from solid clay bricks and mortar (M4) has been investigated in this research.

The present study has shown that CFRP composites offer great benefits for the reinforcement of masonry elements. The present investigation has demonstrated that FRP composites offer benefits for the strengthening of masonry elements. FRP systems have been proven to increase the flexure and shear capacities of URM elements. Analytical studies were presented for determining the flexural capacity limit of the strengthened walls.

First, an experimental study was carried out for measuring the technical characteristics of brick, mortar and triplet Masonry samples.

Second, a hundred masonry triplets were constructed and strengthened with different configurations of CFRP to study the behaviour of unreinforced and strengthened masonry triplets under shear and compressive loading. In addition, the reinforced cracked triplet masonry with CFRP was studied. Also, new equations were developed to predict stress and sliding as well.

Third, the Ansys software for finite element modelling was used. The comparison of results shows the finite element modelling did not give a realistic image of the behaviour of masonry triplets in the case of the sliding and displacement, the ultimate loads were very low. Therefore, the new user-defined functions for finite element modelling were developed and verification of these models with regard to the

experimental results are conducted. The proposed model with user-defined functions showed good agreement with the results of the laboratory tests.

The experimental study on reinforced triplets, repaired cracked triplets and FE analyses with UDF functions in this study are steps forward in brick masonry walls and finite element modeling. The findings of this study will directly benefit engineers in reinforcing masonry structures and predicting the correct displacement in walls in case of shear loading. Based on the experimental and analytical outcomes, this research will contribute to the efforts of the worldwide research community by providing a deeper understanding of the behavior of CFRP, in particular its ability to increase the shear and compression capacity of the masonry walls. This study generates the required data for engineers who are involved in the field of repairing and strengthening of existing masonry structures damaged by extreme loading.

The following conclusions for strengthened triplets under shear loads, compression loads, cracked triplets and finite element simulations can be drawn.

8.2. Masonry triplets under compression load

Based on the literature review, in the first stage, an experimental test program was established, aiming to identify the mechanical properties of masonry and its components. The second stage was a compression test on unreinforced and CFRP-reinforced triplets.

The experimental study on the triplet masonry produced results for the compression behavior of strengthened masonry under uniform pressure in terms of displacement, and maximum strength with and without FRP. Twenty-eight masonry triplets were constructed in the laboratory to study masonry triplets under compression loads with different CFRP configurations.

The following conclusions for strengthened triplets under compression loads can be drawn.

1. In this study, the de-bonding of the CFRP from the masonry surface is the reason for failure. In contrast with URM triplets, reinforced triplets were stable after failure.

2. The strength of masonry triplets under compression was increased 3 times due to CFRP composite retrofitting.
3. Most effective strengthening in terms of ultimate load capacity is for the triplet sample with CFRP reinforcement at four sides and the highest ductility in terms of maximum deflection at ultimate load is observed for all samples. This finding is useful for reinforcing old buildings' columns.
4. The increase of the stiffness of URM triplets with CFRP sheets and strips is significant for the first layer and not so effective for the additional layers on top of the first layer.
5. Comparing one layer and two layers of CFRP or CFRP strips on triplet masonry showed It is no significant bond improvement between one layer and two layers. Because the layers stick together and act as one layer. Depending on CFRP configurations, two layers make triplets 5% to 15% stronger, which is not sufficient to apply two or three layers.

8.3. Masonry triplets under In-Plane or shear load

Composite material, masonry can present different failure modes, depending on the geometry, material, and load factors. According to the literature review, additional experimental studies were carried out to investigate the behavior of triplet masonry under shear loading with a different configuration of CFRP sheet and CFRP net.

In the un-strengthened samples, the failure was brittle. In this triplet, some material comes loose after the ultimate load. At the final state, all strengthened triplets did not have material loos and wider cracks were mostly observed in the un-strengthened triplets. In this stage, the experimental tests were carried out with One-layer CFRP.

To investigate the effectiveness of extra layers of FRP composite on masonry triplets, the experimental tests were set up with retrofitting triplets with two layers of CFRP with eight different configurations. Comparing one layer and two layers of CFRP or CFRP strips on triplet masonry showed It is no significant bond improvement between one layer and two layers. It proved that layers stick together and act as one layer. Depending

on CFRP configurations, two layers make triplets 5% to 15% stronger, which is not sufficient to apply two or three layers. Triplets that were covered on two sides by CFRP were 20% to 40% stronger than strips and CFRP strips are 5% to 10% stronger than CFRP net strips.

Also, the equations to predict shear stress and sliding were developed. By using the new equations, it is possible to predict correct stress at any sliding distance for solid London brick and mortar M4 reinforced with eight different CFRP configurations and unreinforced triplet masonry. The equations in this study are a step forward in theoretical study and will directly benefit engineers to predict the correct wall's resistance in case of shear loading.

The following conclusions for strengthened triplets under shear loads can be drawn.

1. All types of reinforced triplets were at least 4 to 6 times stronger than triplets without CFRP and shear failure of brickwork started with cracking and sliding through the mortar joints. The failure occurred along with the interface between the brick-and-mortar joint.
2. Highest ductility in terms of maximum sliding at ultimate load is observed for all samples. Also, according to this experiment, it is no significant difference between one layer and two layers of CFRP on triplet masonry.
3. The increase of the stiffness of URM walls with CFRP sheets and strips is significant for the first layer and not so effective for the additional layers on top of the first layer.
4. The comparison of the load-displacement graphs from the theoretical modeling with the results from the experimental tests was not accurate, thus the new user-defined functions for finite element modeling were developed.
5. The interfacial shear stress concentrations between the brick-and-mortar surfaces are reduced due to retrofitting of shear triplets with CFRP composites.

6. Shear strength of triplets reinforced with the same amount in terms of area reinforced with CFRP and CFRP net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity
7. Debonding failure pattern was observed in triplet specimens glued with CFRP composites in full and strip configuration. However, tearing of FRP composites was observed in the case of triplet specimens glued with lower FRP composites.
8. In contrast with URM triplets, strengthened samples were stable after failure.

8.4. Cracked triplets Conclusions

The objective of this study was to investigate the strength and stiffness of masonry walls repaired with different layouts of FRP by comparing them to similar undamaged walls strengthened with the same retrofit schemes. The effect of CFRP with different configurations on damaged or cracked walls has not been studied properly. This research addresses a method to reinforce cracked masonry walls to acceptable levels. The results showed that the application of the CFRP composite on unreinforced masonry (URM) has a great influence on strength of masonry triplets as well as changing of failure modes and ductility.

Eight different configurations of CFRP overlays were used to repair masonry triplets with in-plane shear cracks. The samples were tested under shear loading and the test results were compared to the responses of triplets with no initial damage and strengthened with CFRP. It was found that the increase of shear strength and maximum displacement at the failure of repaired samples almost is similar to the triplets with no initial damage and with the same amount of CFRP overlays.

The results for cracked triplets are summarized as follows:

1. Repaired cracked triplets can maintain resistance up to similar sliding values as undamaged strengthened walls.

2. The triplet samples repaired with CFRP and tested under shear loading showed significant improvement in bond characteristics, strength, and ductility. The test load capacity of triplet masonry increased by 3 to 5 times in comparison with samples without initial damage.
3. Shear strength of cracked triplets strengthened with the same amount in terms of area reinforced with CFRP and CFRP net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity. However, the gain in shear strength increased with an increasing amount of CFRP.
4. The influence of the CFRP was observed on deformational characteristics of masonry triplets such as ultimate displacement and the initial slip of the load-displacement curve. Ultimate displacement and the initial slip of the load-displacement response of triplets with higher CFRP sheets were lower than triplets with lower CFRP strips.
5. De-bonding failure pattern was observed in triplet specimens glued with CFRP Composites in full and strip configuration. However, tearing of CFRP composites was observed in the case of triplet specimens glued with lower CFRP composites.
6. Remarkable increases in shear capacity and ductility and depending on the dimensions and orientation, retrofitting with CFRP increases ductility under in-plane shear prevents sliding, and increases the ultimate strength.
7. Highest ductility in terms of maximum deflection at ultimate load is observed for samples. Also, according to this experiment, it is no significant difference between one layer and two layers of CFRP on cracked triplet masonry.
8. Comparison of the load-displacement graphs from the theoretical modeling with the corresponding results from the experimental investigation was not accurate, thus the new equations (UDF) were developed.

8.5. Finite element analyses conclusions

The finite element analysis was used to improve understanding and prediction of the mechanism of shear, sliding and CFRP debonding that can occur with CFRP strengthened triplet. In this study, the theoretical modelling by Ansys was much lower than the experimental tests and was not reliable. It is normal to find disagreement between analytical finite element modelling and experimental tests. The small difference can be neglected, but significantly high or low results from FE modelling compared to experimental tests should be addressed. In this study the comparison of the load-displacement graphs from the theoretical modelling with the results from the experimental tests showed significantly low displacement and were not accurate, thus the new user-defined functions for finite element modelling were developed.

The reason for lower sliding in FE modelling than the experimental tests is, the finite element program in Ansys assumes a perfect bond between brick and mortar. The perfect homogeneous material is the other assumption in finite element analysis. Therefore, this error can be corrected by using a newly developed formula as a user-defined function (UDF) to create correct sliding and 3D modelling for eight CFRP configurations. By using the new functions, it is possible to predict correct sliding in 3D modelling at any load or stress point. These equations are based on quadratic equations, trial and error and verification against experimental tests. Also, these functions are useful for different types of brick and mortar as well. The UDF functions in this study are steps forward in finite element modelling. The findings of this study will directly benefit engineers to predict correct wall displacement in case of shear loading.

The finite element analysis showed good characterization of the behaviour of masonry triplet strengthened with CFRP and UDF approach for shear loading has demonstrated good agreement.

8.5.1. Conclusions for shear loading are summarized as follows:

1. The comparison of the load-displacement graphs from the theoretical modeling with the results from the experimental tests were not accurate for London solid brick and mortar M4, thus the new equations for finite element modeling were developed to predict correct sliding.

2. The interfacial shear stress concentrations between the brick-and-mortar surfaces are reduced due to retrofitting of shear triplets with CFRP composites and it was also found that the strengthened triplets had greater ductility than the URM triplet.
3. Shear strength of triplets reinforced with the same amount in terms of area reinforced with CFRP and CFRP net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity.
4. Comparison of the finite element analyses by developed UDF functions, new theoretical equations and experimental tests for reinforced triplet masonry with different CFRP configurations showed good agreement. The new functions make it possible to predict correct sliding/slipping in 3D modeling at any stress/load point and the errors are between 5% to 25%.

8.5.2. Conclusions for shear loading on cracked triplets are summarized as follows:

1. Repaired cracked triplets can maintain resistance up to similar sliding values as undamaged strengthened walls.
2. The triplet samples repaired with CFRP and tested under shear loading showed significant improvement in bond characteristics, strength and ductility. The test load capacity of triplet masonry increased by 3.5 to 5 times in comparison with samples without initial damage.
3. Shear strength of cracked triplets strengthened with the same amount in terms of area reinforced with CFRP and CFRP Net was found to be nearly the same. This can be mainly attributed to the similar values of modulus of elasticity. However, the gain in shear strength increased with an increasing amount of CFRP.

4. Comparison of the load-displacement graphs from the theoretical modeling with the corresponding results from the experimental investigation was accurate, thus the new equations (UDF) were developed.

8.6. Suggestions for further investigations

The effectiveness of the FRP composites for reinforcement of the masonry structures has been proved further investigations are needed to clarify the behavior of FRP-strengthened masonry walls. Some of the areas that need to be investigated are indicated below:

1. Possible application of different combinations of composite materials for different types of masonry walls.
2. Further information about the durability of composite materials used in the reinforcement of URM walls under different environmental conditions would be very beneficial.
3. Investigation of surface preparation methods and the amount of impregnating resins are also needed.
4. Since debonding of FRP from the masonry surface is the main reason for failure and has a direct relationship with the porosity of the masonry unit and mortar, the different types of masonry units and mortar should be investigated.
4. Investigate the strengthened masonry walls in high temperatures.
5. Investigate large-scale strengthened masonry walls with different CFRP configurations under combinations of in-plane and out-of-plane loadings.

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