"***Improvement of Shear Resistance of Unreinforced Masonry (URM) Walls made of Recycled Clay Bricks***"

A THESIS SUBMITTED TO
THE FACULTY OF ARCHITECTURE AND ENGINEERING
OF
EPOKA UNIVERSITY

BY

KEVIN ZACE

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

JULY, 2021

**Approval sheet of the Thesis**

This is to certify that we have read this thesis entitled "***Improvement of Shear Resistance of Unreinforced Masonry (URM) Walls made of Recycled Clay Bricks***" and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

________________________

Dr. Enea Mustafaraj
Date: 09/07/2021

Examining Committee Members:

| | | |
|---|---|---|
| Dr. Enea Mustafaraj | (Civil Engineering) | __________________ |
| Assoc. Prof. Dr. Hüseyin BILGIN | (Civil Engineering) | __________________ |
| Dr. Erion Luga | (Civil Engineering) | __________________ |

**I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.**

Name Surname: Kevin Zace

Signature: _______________

# ABSTRACT

***Improvement of Shear Resistance of Unreinforced Masonry (URM) Walls made of Recycled Clay Bricks***

Zace, Kevin

M.Sc., Department of Civil Engineering

Supervisor: Dr. Enea Mustafaraj


Buildings made with unreinforced masonry (URN) are one of the most common kinds of construction in the world. All of this building stock is vulnerable to destruction in the event of an earthquake due to its poor capacity to withstand lateral stresses. The experimental campaign on investigating the structural performance of masonry walls by conducting diagonal compression tests is described in this study.

In the laboratory, three diagonal compression tests were performed on two specimens with nominal dimensions of 1.2 x 1.2 x 0.25 m. The major goal was to look at the structural behavior of two different types of masonry panels: unreinforced and reinforced.

Mortar type used was type “O” and wall panels were built. A plain wall, a polypropylene strengthens on one side and the third was a polypropylene strengthen on both sides. Three diagonal compression tests were carried out completely according to the American Society for Testing and Materials' technical requirements (ASTM International). Material properties of masonry component materials were established for each panel prior to testing.

The ultimate drift and ductility are two more parameters to consider while evaluating the behavior of URN. URM buildings are subjected to lateral loads as a result of seismic shaking, which cause lateral deformation of the structure. Ductility is described as a material's capacity to deform without rupturing, or, in the case of URM structures, the structure's ability to deform without collapsing. The decrease in stiffness was often noticed at load levels around the ultimate load, when the first crack forms but is unable to grow owing to the existence of external reinforcement.

The testing findings revealed that the panel's shear strength is highly influenced by the mortar type (mortar strength), since the fractures spread through the joints without harming the bricks in all cases.

The highest shear strength was achieved by W3-PP-2s, 0.376 MPa which was 4 times higher than the shear strength of the plain panel of W1. Additionally, W3-PP-2s achieved higher ultimate diagonal load of 159.424 kN. W3-PP-2s, on the other hand, had a lower deformation capacity and were more brittle than plain panels, with an ultimate drift ratio of 0.434.

***Keywords:*** fiber reinforced polypropylene, mortar type "O", diagonal compression test, structural behavior, unreinforced masonry

# ABSTRAKT

Përmirësimi i rezistencës ndaj forcave prerese e mureve të tullesse ndertuar me tulle te ricikluar së pa perforcuar

Zace, Kevin

Master Shkencor, Departamenti i Inxhinierisë së Ndërtimit

Udhëheqësi: Dr. Enea Mustafaraj

Ndërtesat e bëra me muraturë të pa përforcuar (URN) janë një nga llojet më të zakonshme të ndërtimit në botë. I gjithë ky stok i ndërtesës është i prekshëm ndaj shkatërrimit në rast të një tërmeti për shkak të kapacitetit të tij të dobët për të përballuar streset anësore. Fushata eksperimentale mbi hetimin e performancës strukturore të mureve të muraturës duke kryer teste diagonale të ngjeshjes është përshkruar në këtë studim.

Në laborator, u kryen tre teste diagonale të ngjeshjes në tre mostra me dimensione nominale 1.2 x 1.2 x 0.25 m. Qëllimi kryesor ishte të shikojmë sjelljen strukturore të dy llojeve të ndryshme të paneleve të muraturës: të pa përforcuar dhe të përforcuar.

Llaçi i përdorur ishte tipi "O" dhe u ndërtuan panele muri. Një mur i thjeshtë, një polipropilen i perforcuar në njërën anë dhe e treta ishte një forcim polipropileni në të dy anët. Tre teste diagonale të kompresimit u kryen plotësisht sipas Shoqërisë Amerikane për Testimin dhe Kërkesat Teknike të Materialeve (ASTM International). Karakteristikat materiale të materialeve përbërëse të muraturës u përcaktuan për secilin panel para testimit.

Zhvendosja dhe duktiliteti përfundimtar janë dy parametra të tjerë që duhen marrë parasysh gjatë vlerësimit të sjelljes së URN. Ndërtesat URM i nënshtrohen ngarkesave anësore si rezultat i lëkundjeve sizmike, të cilat shkaktojnë deformim anësor të strukturës. Duktiliteti përshkruhet si aftësia e një materiali për të deformuar pa u prishur, ose, në rastin e strukturave URM, aftësia e strukturës për të deformuar pa u shembur. Ulja e ngurtësisë shpesh vërehet në nivelet e ngarkesës rreth ngarkesës përfundimtare, kur plasja e parë formohet, por nuk është në gjendje të rritet për shkak të ekzistencës së përforcimit të jashtëm.

Gjetjet e testimit zbuluan se forca e prerjes së panelit ndikohet shumë nga lloji i llaçit (forca e llaçit), pasi thyerjet përhapen nëpër nyje pa dëmtuar tullat në të gjitha rastet.

Forca më e lartë e prerjes u arrit nga W3-PP-2s, 0.376 MPa e cila ishte 4 herë më e lartë se forca e prerjes e panelit të thjeshtë të W1. Për më tepër, W3-PP-2 arriti një ngarkesë më të madhe diagonale përfundimtare prej 159,424 kN. W3-PP-2, nga ana tjetër, kishin një kapacitet më të ulët deformimi dhe ishin më të brishtë se panelet e thjeshta, me një raport përfundimtar të zhvendosjes prej 0.434.

***Fjalët kyçe:*** polipropileni i përforcuar me fibra, llaçi i tipit "O", prova e kompresimit diagonal, sjellja strukturore, muratura e pa perforcuar.

# TABLE OF CONTENTS

# LIST OF TABLES

# LIST OF FIGURES

# CHAPTER 1

# INTRODUCTION

## 1.1 Problem Statement

Buildings made of unreinforced masonry (URM) are one of the most common types of construction in the globe, as well as in Southern Europe and the Mediterranean basin. Despite the fact that these areas have a medium-to-high level of seismic danger. The URM structures are susceptible because they were intended (or were not designed at all) to solely withstand gravity stresses.

Many of those structures have been harmed by a combination of poor construction practices, seismic and wind loads, foundation settlements, and material degradation (C. Faella, 2010).As a result of these factors, there is a rising need to enhance the overall structural response of these structures in order to avoid seismic damage. This improvement could be achieved by using external shear strengthening techniques.

## 1.2 Thesis Objective

The main objective of this study is to investigate the structural behavior of masonry panels. Two main specimen types have been tested: unreinforced and strengthened panels.

The structural performance of panels constructed of one type of mortar was tested using a strengthening technique called textile reinforced mortar (TRM).

## 1.3 Scope of works

For this study, 3 panels with nominal dimensions of 1.2 x 12 x 0.25m made of solid clay brick were constructed using one mortar type: ASTM type "0" mortar. These composition was aimed at replicating the mortar used in and existing old buildings.

A total of 2 diagonal compression test were conducted in order observe the structural behavior of two types of panels; plain and reinforced. One strengthening technique is investigated and comparisons of improvement of shear strength, drift and energy dissipation are done.

To have a better understanding of the interaction between masonry assemblage components; day brick, mortar, and brick/mortar interface, the same tests are reproduced in a FEM modeling of plain and reinforced panels (polypropylene).

## 1.4 Organization of the thesis

This thesis is divided in 5 chapters. The organization is done as follows:

In Chapter 1, the problem statement, thesis objective and scope of works is presented. Chapter 2, includes the literature review. Chapter 3, consists of the methodology followed in this study. In Chapter 4, the experimental results. In Chapter 5, conclusions are stated.

# CHAPTER 2

# LITERATURE REVIEW

## 2.1 Introduction

Masonry structures in Albania and in most countries around the world represent a significant percentage of the existing building inventory. Until modern building code standards for seismic resistance were developed, many of these buildings were planned and constructed. Unreinforced masonry (URM) walls under moderate and high seismic demand show poor seismic efficiency.

This behavior is linked to the rapid deterioration of the capacity for stiffness, strength, and energy dissipation that corresponds to the masonry wall's sudden brittle failure. For this reason, the design of new strengthening and repairing techniques have been the focus of many experimental researches done in the last few years by many researchers worldwide. For the reinforcement of URM walls, fiber-reinforced polymer (FRP) composites of different matrix and fiber compositions may provide solutions.

Valluzzi, Tinazzi, Modena and Marshall and Sweeney have used cement- and polymer-based matrixes em. While much of the research on FRP composites as well as field applications has concentrated on repairing members of reinforced concrete (RC), available masonry literature shows high potential with advantages linked to lower installation costs, better corrosion tolerance, ease of usage, and minimum changes in the size of the member after repair.

Bedding glass, biomass, and aramid FRP bars for this form of use. Disturbance to occupants and loss of usable space are also minimized. In addition, the elastic properties of the current structure remain intact from the structural point of view so there is no weight addition, and stiffness improvements which be engineered case by case.

Strengthening by inserting FRP bars into mortar joints, often referred to as near-surface-mounted (NSM) strengthening, or structural repointing, will greatly improve the shear potential and provide URM walls with pseudo ductility.

## 2.2 Materials Properties

The overall behavior of a URM framework is closely dependent on the individual properties of components of masonry. Therefore, it is of vital importance to determine the physical and mechanical parameters of brick, mortar and masonry assemblage units to understand a structure's global behavior.

For example, if the compressive strength of the masonry is required, the compressive strength of the brick is used, which could then be a useful parameter in determining other additional properties such as Elasticity Modulus (E) and masonry stress-strain behavior (H.B. Kaushik, 2007)

### 2.2.1. Bricks

Bricks are a significant structural part of clay or silicate URM structures. As a construction material, the clay brick is made of solid clay, or clay with admixtures fired at a particular temperature (ranging from 700-1100 ° C) to avoid crumbling when water is in contact.

When the topsoil layer was stripped before a layer of clay or shale was achieved, the brick manufacturing process began with the preparation stage, mining. The drilling method was carried out using either a hand shovel or a mechanical excavator (Figure 1).

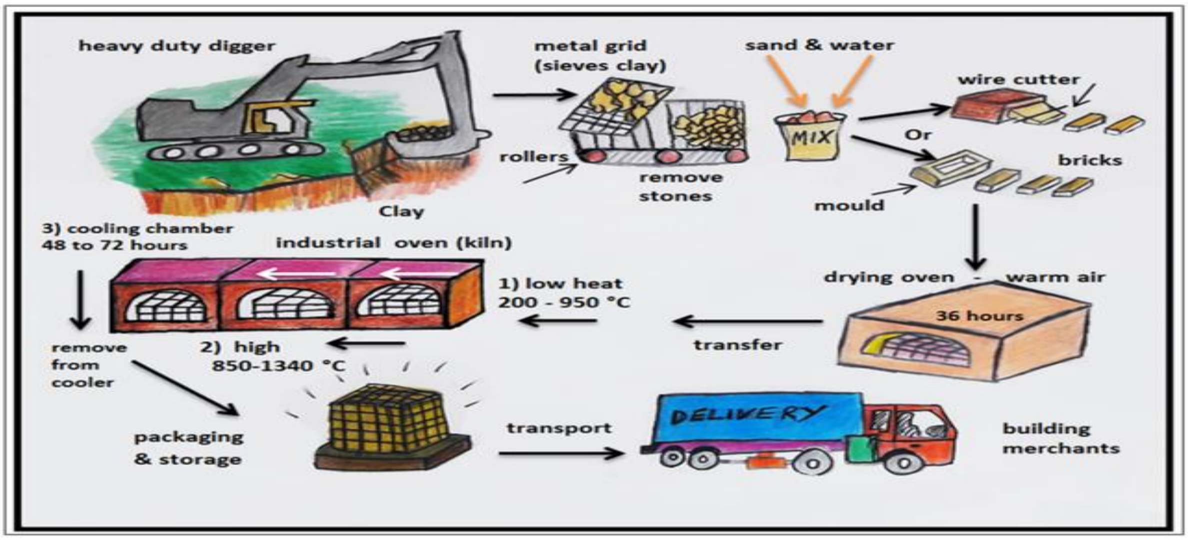


***Figure 1*** Brick making process (schematic view) *(Industry, 2006)*

The molding of bricks was performed by hand in the early stages, later wooden or metal molds were used. The next accomplishment at the point of development was the method of wire cutting involving the cutting of the bar of clay compressed by a pug mill or auger machine by wires. The drying process then took place and covered the freshly shaped bricks against rain, wind, sun and frost. Firing of bricks is performed in a kiln at a specified temperature (Lumantama, 2012).

(1) Color, (2) texture and (3) the degree of porosity are the key influences influencing the physical properties of the brick. The mechanical qualities of bricks are closely related to longevity and compressive power. The brick color and mechanical properties have no clear relationship. (4) The division of bricks by color is dependent on their color. Just true for the one that is made of the same clay material. A darker color suggests a higher firing temperature, thus a higher intensity of compression. The texture of the brick is a physical property influenced by the degree of vitrification (formation of glassy layers during high temperatures, causing the clay particles to bond together). (S. Karaman, 2006)

The tolerance to freeze-thaw cycles, which is influenced by the size of pores in the brick, pore structure contact, and weathering, is one of the key criteria for the longevity of brick units (K. Elert, 2003).

The strength of the brick depends closely on the consistency of the clay content used as well as the temperature at which it is fired; the greater the temperature, the higher the vitrification degree, the more bonded the clay elements are (K. Elert, 2003) (S. Karaman, 2006).

Bricks are weak in tension due to high porosity and brittleness, but very strong in compression. The compressive strength also depends on the porosity level; the higher the porosity level, the lower the strength (K. Elert, 2003). By increasing the firing temperature, the level of porosity decreases.

Bricks are classified as solid (when the net cross-sectional area is 75% or more than the gross cross-sectional area of each plane parallel to the bearing surface), perforated (when the net cross-sectional area is between 25-75% of the gross cross-sectional area) and hollow (when the net area is less than 25 percent of the gross cross-sectional area of the brick) (Mustafaraj, 2016)

According to ASTM C 62-04 (ASTM, 2004), bricks should be manufactured from clay, shale or similar naturally occurring earth materials, molded, pressed or extruded during the manufacturing process, and subjected to firing. It should be subject to visual inspection when the brick is delivered to the site, and it should be provided that it is free from defects, deficiencies and other surface treatments that would impair the brick's strength or performance during the construction process. The physical specifications are identified in (Table 1).

***Table 1*** Physical requirements for bricks, ASTM C 62-04 ***(ASTM, 2004)***

| *Designation* | *Minimum Compressive Strength gross area (MPa)* | | *Maximum Water Absorption by 5-h Boiling, (%)* | | *Maximum Saturation Coefficient** | |
|---|---|---|---|---|---|---|
| | **Avg. of 5 Brick** | **Individual** | **Avg. of 5 Brick** | **Individual** | **Avg. of 5 Brick** | **Individual** |
| Grade SW | 20.7 | 17.2 | 17.0 | 20.0 | 0.78 | 0.80 |
| Grade MW | 17.2 | 15.2 | 22.0 | 25.0 | 0.88 | 0.90 |
| Grade NW | 10.3 | 8.6 | no limit | no limit | no limit | no limit |

*The saturation coefficient is the ratio of absorption by 24-h submersion in cold water to that after 5-h submersion in boiling water.

**-Grade SW (Severe Weathering)**: Brick intended for use where high and uniform resistance to damage caused by cyclic freezing is desired and where the brick may be frozen when saturated with water.

**-Grade MW (Moderate Weathering)**: Brick intended for use where moderate resistance to cyclic freezing damage is permissible or where the brick may be damp but not saturated with water when freezing occurs.

**-Grade NW (Negligible Weathering)**: Brick with little resistance to cyclic freezing damage but which are acceptable for applications protected from water absorption and freezing.

According to Sneck, the most important parameter affecting the fresh mortar and the hardened mortar is the suction rate of the brick and, as a consequence, the properties of the entire assemblage (Sneck).

Direct checks, which consist of moving the brick to collapse, are the easiest way to assess the characteristic properties of bricks. However, sample extraction might not be an easy job to do in the case of antique URM houses. NDT (non-destructive testing) can be used to predict in-situ material properties if the following test can be performed: 1) ultrasonic pulse velocity test; 2) Schmidt hammer test; 3) porosity test; 4) scratch test.

The tensile strength is an important parameter to be determined; the ability of the material to withstand maximum stress. Depending on the applied load, there are various tensile strength estimates.

1. Direct tensile strength (axial tensile strength), measured on a cylindrical specimen with a ratio of height/diameter 1.
2. Flexural tensile strength (rupture module) measured when the masonry units are exposed to an axial load applied between the two end supports of the unit in the center (ASTM, 2014);

3. Splitting tensile power, measured when the line load is applied parallel to the unit length on both surfaces (International, 2013);

In the absence of tests, then according Hilsdorf, 1967, tensile strength can be measured using the empirical formulas (Hilsdorf, 1967) ;

$$\boldsymbol{ft = 0.26 * f^{0.67}} \qquad \textbf{\textit{Equation 1}}$$

$$\boldsymbol{ft = 0.72 * f_{cb,splitting}} \qquad \textbf{\textit{Equation 2}}$$

$$\boldsymbol{ft = 0.72 * f_{cb,flexural}} \qquad \textbf{\textit{Equation 3}}$$

where $f_{t,axial}$ is the uniaxial tensile strength and $f_{cb}$ is the compressive strength of brick.

In 1971, Sahlin suggested that the ratio of tensile strength to compressive strength for solids was 1:20 and 1:30 for hollow bricks. The rupture modulus ranges between 10-30 percent of the compression power, while the tensile strength varies between 30-40 percent of the rupture modulus (Sahlin S. , 1971).

### 2.2.2 Mortar

Mortar is a building medium made up of a proportional combination of water, sand and lime or cement used as binders. Mortar features are usually correlated with the properties of the binder (McKay, 1947) (Mulligan, 1942) (A. Palomo, 2004). The mortars used nowadays are based on cement and have a 1:1:6 volume ratio of cement: lime: sand (c:l:s). They are relatively rigid and have higher pressures, with c:l:s of 1:2:9, distinguished by very low strength but high ductility, than the lime-based mortars. The main types of mortar are:

1. Cement mortar: Portland cement, sand and water dry mix sets fast

and has elevated mechanical strength and low porosity.

2. Lime mortar: low mechanical strength, high deformation capacity, high workability and self-repair ability of sand, water and quicklime (hydraulic or non-hydraulic) .

3. Cement-lime mortar: lime, Portland cement, sand and water equal mixture, is workable, provides strong bonding, and has high deformation potential and compressive strength as well as crack healing ability.

4. Lime-pozzolan mortar: a lime mortar using pozzolanic ingredients, in contrast to cement mortars, has higher mechanical strength, high porosity and low compressive strength.

According to ASTM C 270-03, (ASTM, 2003) classification of mortar is done according to the Table 2 below:

***Table 2*** Types of mortar ASTM C 270-03 ***(ASTM, 2003)***

| *Mortar Type* | *Proportion by Volume* | | | *Binder: Aggregate ratio* | *28 Days Compressive strength* |
|---|---|---|---|---|---|
| | Cement | Lime | Sand | | |
| **M** | 4 | 2 | 15 | 1:3 | 17.2 |
| **S** | 2 | 2 | 9 | 1:3 | 12.4 |
| **N** | 1 | 2 | 6 | 1:3 | 5.2 |
| **O** | 1 | 2 | 9 | 1:3 | 2.4 |
| **K** | 1 | 3 | 12 | 1:3 | 0.5 |

The mortar conditions are strongly influenced by the following factors: 1) freeze-thaw cycles, 2) leakage of water, 3) crystallization of salt, 4) chemical interaction, 5) bio-deterioration, etc. Workability, plasticity, water retaining capability, compressive strength and bond strength formed between bricks and mortar are the most important features of mortar.

The compressive strength of pure lime mortars varies from 1.0 MPa to 2.0 MPa; the strength rises to 5 MPa for hydraulic mortars, while the compressive strength can go up to 17 MPa for cement-mortar. (Mustafaraj, 2016)

The compressive strength of the mortar depends on the bonding agent's consistency, as well as the ratio of sand to cement or lime. It is the ability of the mortar used to bond with the bricks that controls the general ability of URM to withstand the cracking of the in-plane shear during a seismic event.

### 2.2.3 Masonry assemblage

Nowadays, in various manufacturing processes, there are different kinds of masonry units developed by different raw materials such as clay, calcium silicate, stone and concrete. Masonry assemblage is a composition of two components with very different characteristics, such as bricks and mortar: stiffer bricks and softer mortar. It is known as a substance that is usually inelastic, extremely inhomogeneous, and anisotropic. Masonry can undergo substantial mortar failure due to combination chemical, physical and mechanical corrosion due to mortar joints.

Masonry is known to be a quasi-brittle substance with an unordered internal structure comprising a "large number of potential failure zones in the form of grain boundaries if randomly oriented" (Bakeer T. , 2009) .Quasi-brittle refers to the incremental decrease in resistive force as the micro-cracks are widened and become macro-cracks after achieving the full load.

A significant element playing an important role in the mechanical properties of masonry is the formation of brick/mortar bonds. (A.W. Hendry, 2017) Indicated that it is primarily affected by: aggregate properties; mortar water content and water retentivity; surface roughness, structure of the pore and initial rate of brick absorption; construction quality (Khalaf, 2001)

The so-called "bond" which has aesthetic as well as structural functions, forms the distinct framework of brick units. (a)  Running bond; (b) Flemish bond; (c) American or common bond; (d) Herringbone bond; and (e) stack bond are the most common types of bonds used worldwide (Figure 2).

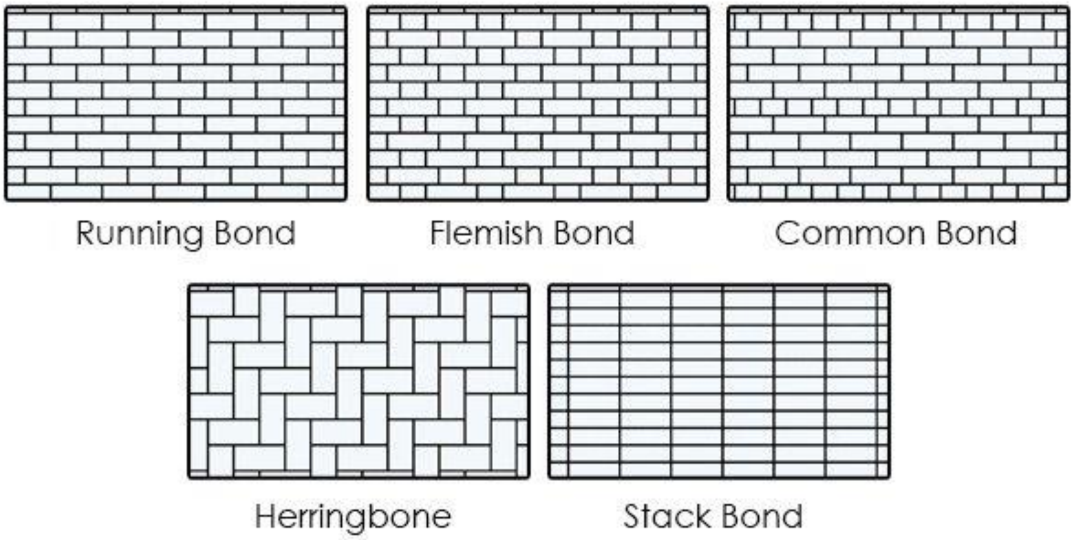


***Figure 2*** The most common types of bonds: (a) Running bond; (b) Flemish bond; (c) American or common bond; (d) Herringbone bond; and (e) stack bond *(Lourenco, 1998)*

### 2.2.4 Masonry compressive strength

The compressive strength of Masonry determines the prism's ability to withstand compressive forces and ranges to around 20-50 percent of the compressive strength of the brick. Such a low value is due to low mortar strength; the greater the strength of the mortar, the greater the strength of the prism. (Priestly T. P., John Wiley & Sons ) (R.G. Drysdale, 1994) Workmanship, the characteristics of the masonry units, the thickness of the mortar joints, the age of the mortar, and the suction rate of the bricks all influence the compressive strength of masonry (Sahlin S. , 1971) It is also determined by mortar and brick thickness; the heavier the masonry, the thicker the bricks are in contrast to the mortar. It is proposed that the optimal joint thickness is between 5-10 mm. Any value above would reduce the overall masonry strength in compression (Deodhar S. , 2000).

## 2.3 Unreinforced Masonry Walls

For unreinforced masonry shear walls, researchers (Hendry) (Meyer, 2004 ) have identified two distinct forms of behavior during the past decade. The basic form of the shear strength expression is based on the Mohr Coulomb shear friction expression for low axial compression stresses, as demonstrated in Equation 4 below.

$$\boldsymbol{\tau_m = \tau_o + \mu\sigma_n} \qquad \textbf{\textit{Equation 4}}$$

where $\tau_m$ and $\sigma_n$ are the average shear and normal stresses, $\tau_o$ is the shear bond strength and $\mu$ is the coefficient of internal friction. In parametric form, Equation 4 can be expressed as:

$$\boldsymbol{V_{n=fn(f^{\wedge}m,N)}} \qquad \textbf{\textit{Equation 5}}$$

where $V_n$ represent the nominal shear strength of the strength of the masonry wall and N is the axial compression force. As demonstrated from experimental studies, values for the constants $\tau_o$ and $\mu$ vary considerably and are influenced by test method and type of masonry. (Priestly, 1992) Recommended a typical range of values of $0.1 \leq \tau_o \leq MPa\ and\ 0.3 \leq\ \mu \leq 1.2$.

The wall achieves peak strength and its behavior changes as the axial compression force exceeds a sufficiently high stress, with the failure mode being the mixture of shear and masonry crushing. Shear strength decreases with even greater compression pressures when compression loss of masonry dominates response to loads. Therefore, Equation 4 does not apply in these cases. This compression failure corresponds to the second part of the curve shown in Figure 3

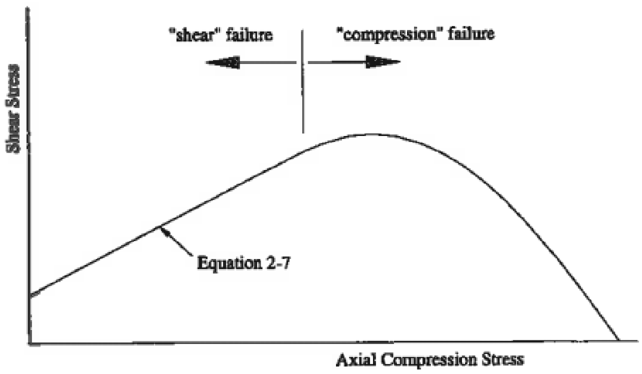




Figure 3 Correletaion between axial compression stress and shear stress

*(Jianguo, 1987)*

## 2.4 Failure of Shear Walls

Shear walls, also called 'racking loads', are needed to withstand horizontal loads. A shear wall subject to horizontal loads can collapse in one of three ways, absent premature lap-splice or reinforcement bond failure: by sliding horizontally, in flexure or in shear. (Park, 1986) Many variables, such as wall aspect ratios, will affect the mode of failure. Stress ratios for axial compression. From the materials used in wall building, wall boundary conditions and resistance characteristics. Figure 5 illustrates these types of faults diagrammatically. Therefore, because the prevailing mode of failure of a shear wall may be other than shear, the term 'shear wall' may be especially representative

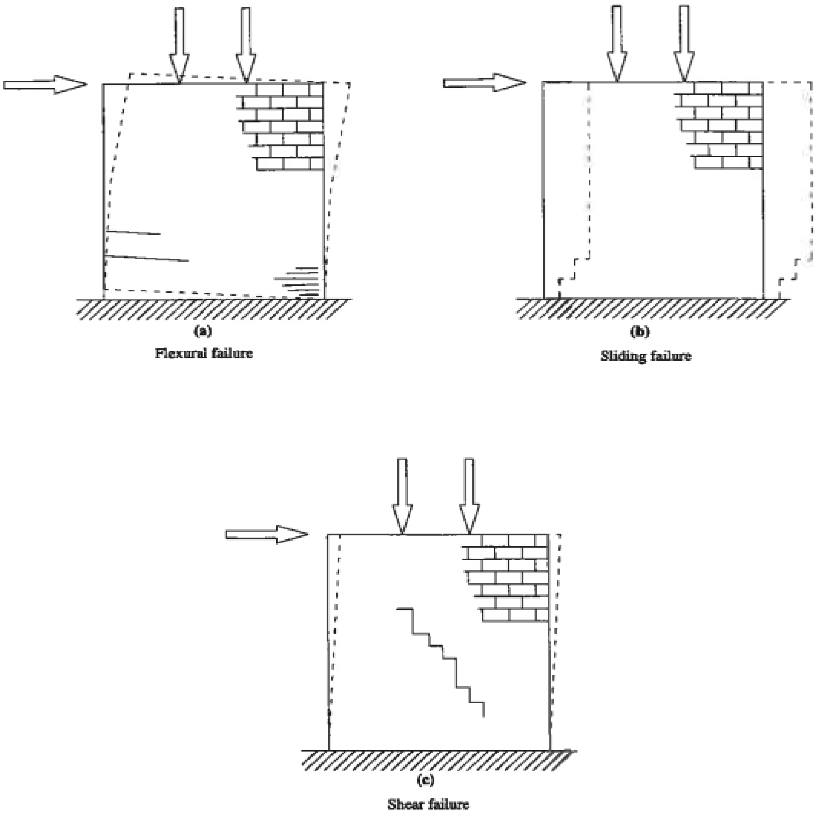


*Figure 4 Reinforced* masonry shear walls failure modes *(Bahman Ghiassi, 2012)*

### 2.4.1 Flexural failure

This type of failure occurs when the wall behaves as a vertical cantilever due to the yield of the vertical reinforcement near the wall heel or the crushing of the masonry at the wall toe. This is generally the preferred mode, as failure is ductile and, in conjunction with reinforcement, dissipates energy effectively.

### 2.4.2 Sliding failure

Sliding shear is the movement on the base or the other mortar bed of entire parts of the wall and is resisted by the vertical reinforcement dowel action anchored at the base and by friction on the mortar bed. (Prietsly, 1976)However, in any case where there is a low friction coefficient, such as by using a friction breaker or water proof membrane, or where the wall is placed on a smooth finished slab, this form of failure may become important. In unreinforced masonry walls, this failure can generate a specific problem.

### 2.4.3 Shear failure

This form of failure is defined by the introduction of apparent diagonal cracking along the shear wall where, under increasing forced lateral displacements, the principal tensile stresses surpass the tensile strength of the masonry. Two types of shear failure are likely depending on the volume and anchorage of horizontal reinforcement: a 'ductile shear failure' and a brittle shear failure (Meyer, 2004 )Whenever there is adequate horizontal reinforcement with proper anchorage, after the initiation of diagonal cracking, redistribution of the stresses across the shear wall will be achieved. Therefore, under increasing horizontal loads, the initial diagonal cracks do not open, but instead form new sets of diagonal cracks and gradually spread throughout the walls of the masonry, accompanied by high energy dissipation that results in ductile behavior. Failure occurs gradually in the event that, under cyclic lateral loading, the strength of the masonry wall deteriorates. At severely cracked portions of the wall diagonals, partial crushing of the masonry finally leads to complete loss of strength. Ductile shear failure is described as this type of failure.

If the horizontal reinforcement quantity and/or anchorage is not sufficient to transfer the tensile stresses across the diagonal cracks, these cracks open extensively and lead to a significant X-shaped diagonal crack pair, resulting in a relatively sudden and destructive failure. This form of failure is referred to as “Sudden shear failure ".

## 2.5 Shear resistance

Unreinforced masonry walls serve as brittle structural elements with limited energy dissipation ability during a shear failure, especially when exposed to high compression stresses (P.B.Shing, 1989) (Haluk Sucuoglu and Hugh D. McNiven, 1991) (Tomaževič, 1999) In order to enhance lateral resistance and ductility, masonry walls are often supplied with steel reinforcement, both horizontally and vertically.

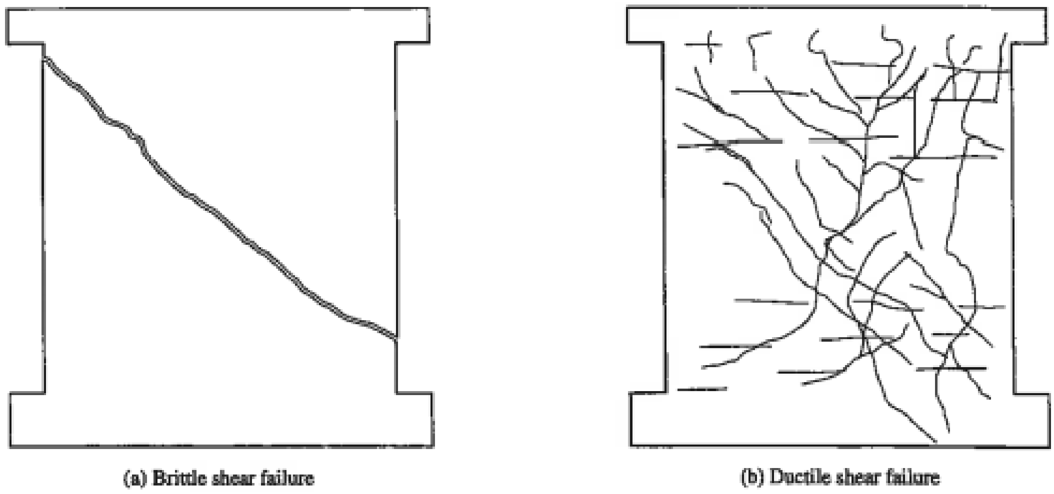


***Figure 5*** Modes of shear failure *(Mann & Müller, 1973)*

The horizontal reinforcement prevents the separation of the cracked parts of the wall from shear failure if a masonry wall is reinforced horizontally, thus improving the resistance and energy dissipation capacity of the wall when subjected to cyclic loading. A single diagonal crack causes significant weakening in strength and eventual brittle collapse in the case of reinforced masonry walls, see Figure 5.

## 2.6 Calculation of the shear strength of unreinforced masonry walls

There are three main factors influencing the shear strength of unreinforced masonry wall as follows:

(1) Influence of strength of materials

The block and mortar strength both impact the shear strength of the masonry wall. The higher the strength grade of Block or Mortar. The better the shear strength of the wall for masonry. The compressive strength of masonry will completely embody the impact of the strength of Block and mortar and masonry consistency on the masonry wall shear strength.

(2) Influence of dimension of masonry wall.

From the experimental evidence at home and abroad, it iss seen that the height ratio to the width of the masonry wall $\lambda_h$(= H/B. H is the height of the masonry wall. The shear strength of the masonry wall is strongly determined by the width of the masonry wall. The larger λh, the greater the masonry wall bending moment caused by horizontal force, the lower the masonry wall shear power. According to the experimental results of 60 pieces of masonry wall, the factor of the effect of the ratio of height to width of masonry wall $\psi$ may be calculated by the following formula.

$$\psi = 0.96 - 0.68 lg\ \lambda_h \qquad \textbf{\textit{Equation 6}}$$

(3) Influence of vertical compressive stress $\sigma_y$

If the ratio of $\sigma_y$ /$f_m$ ($f_m$ is the mean compressive strength of the masonry wall) is less than 0.5 or so, the horizontal slip of the shear section caused by vertical compressive stress can be minimized or avoided by friction of the shear section. In addition, the bending moment of the masonry wall caused by horizontal stress can be minimized and the bad effect of Ah on the shear strength of the masonry wall can be weakened. Vertical compressive stress is advantageous to the shear strength of the masonry wall at this point. With the rise of a $\sigma_y$, the shear strength of the masonry wall increases.

Longitudinal cracks created by $\sigma_y$ may weaken the rigidity of the masonry wall if the ratio of $\sigma_y / f_m$ is greater than 0.5 or so. Finally, under the action of vertical compression and typical horizontal stress, diagonal-compression collapse of the masonry wall is induced. $\sigma_y$, by comparison, is not advantageous to the shear strength of the masonry wall at this time. The shear strength of masonry wall goes down with the increase of $\sigma_y$.

The mean value of the shear strength of the masonry wall can be determined by the following formula according to the experimental findings of the aseismic behavior of twenty - one bits of brick masonry wall (Jianguo, 1987) (Jingqian, 1991):

$$\boldsymbol{f_{v,m} = f_m[0.02 + 0.88\left(\frac{\sigma_y}{f_m}\right) - 0.9\left(\frac{\sigma_y}{f_m}\right)2]\varphi} \qquad \textbf{\textit{Equation 7}}$$

where $f_{v,m}$ = average value of shear strength of unreinforced masonry wall:

$f_m$ = average value of compressive strength of masonry. to be taken according to the stipulation of literature (The Design Code of Masonry Structures GBJ3 - 88, 1998)

Average shear load value - The unreinforced wall bearing power can be estimated using the following formula:

$$\boldsymbol{V_{u,m} = f_m[0.02 + 0.88\left(\frac{\sigma_y}{f_m}\right) - 0.9\left(\frac{\sigma_y}{f_m}\right)2]\varphi A} \qquad \textbf{\textit{Equation 8}}$$

where A = area of cross section of masonry wall.

The mean value of the ratio by the equation 8 and the experimental value ($f^0_{v.m}$) of the measured value ($f_{v.m}$) is 1.056 and the coefficient of variance is 0.185. The comparison of the outcomes of the evaluation with measured values is seen in Fig. 6.

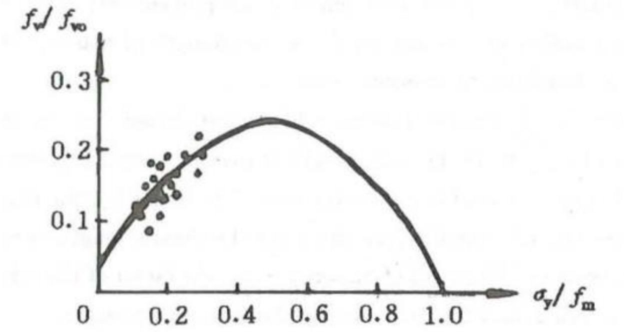


***Figure 6*** The comparison of the outcomes of the evaluation between the experimental and measured value (Hilsdorf, 1967)

The mean value of the ratio of the measured value ($f^1_{v.m}$) to the experimental value is 1.212 and the variance coefficient is 0.235.

The design values of the strength of the materials are followed in line with the stipulation of literature, (The Common Unified Standard for Building Structures Design GBJ68 - 84, 1984) the shear load - bearing capability of the cross-section of the unreinforced masonry wall can be determined by the following formula:

$$V \leq f_m[0.02 + 0.45\left(\frac{\sigma_y}{f_m}\right) - 0.22\left(\frac{\sigma_y}{f_m}\right)2]\varphi A \qquad \textit{Equation 9}$$

Where V = design value of shear of masonry wall;

f = design value of compressive strength of masonry.

## 2.7 Unreinforced masonry buildings in Albania

Buildings in Albania can be divided into two types: those constructed before WWII, which are mostly small three-story residential structures, and those built after WWII, which are multi-story and condominium structures. After previous earthquakes, all types of structures were severely damaged.

Between 1950 and 1990, four large factories dominated the production of construction materials, especially clay and silicate bricks with standard dimensions of 250 x 120 x 65 mm (Kadiu, 2007)

The first edition of the national building codes was published in 1978-79, and the second edition was revised in 1989. Since the majority of the unreinforced masonry buildings were constructed prior to 1978, it is reasonable to presume that they do not comply with Albanian Design Codes.

A significant portion of Albania's building stock, roughly 62 percent, is made up of URM structures constructed before the 1990s based on ADC-89 provisions or without any compliance with any codes at all if built before 1978 (Albania Institute of Statistics, Tirane, Albania)

## 2.8 Strengthening techniques used in existing URM structures

Various stabilization methods have been developed and applied in the history of architecture to increase defects related to low structural integrity of URM systems under seismic actions. The key goal of strengthening techniques is to improve low masonry parameters like tensile and shear strength. These methods are classified as conventional or modern depending on the process and materials used. (Mustafaraj, 2016)

### 2.8.1 Confinement of URM with Constructional Columns

The masonry walls are confining at all corners and wall intersections, as well as the vertical boundaries of door and window openings, using constructional columns. (M.A. ElGawady) The structural integrity would be significantly enhanced if the constructional columns are connected to ring beams at the floor level. The masonry structure is contained at the same storey by both constructional columns and ring beams. This method could increase resistance in both directions (out-of-plane and in-plane). (Earthquake Eng. Struct. Dynam) Discovered that this method could improve lateral resistance by 1.5 times and lateral deformations and energy dissipation by 50%. (S. Paikara) Tested this technique on half-scale specimens under cyclic loading and found that the energy dissipation of the wall as well as the deformability in the in-plane direction had improved. This confined device is recommended for newly constructed masonry structures in Eurocode 8 because the building's integrity can be guaranteed. The application of this technique to existing structures would be difficult and expensive.

### 2.8.2 Confinement of URM with Ring Beam

The reinforced concrete ring-beams were normally used in masonry structures to improve its mechanical behavior. Masonry structures confined with constructional columns and ring beams are required to perform well in earthquakes. (H. Okail) concluded in a study on confined masonry structures that the confining elements preserve the mechanical efficiency (ductility and strength) of the masonry panels. Furthermore, with a higher reinforcement ratio and more confining features, the masonry structure's strength can be maintained during an earthquake. If the current ring beam is initially damaged or weak, retrofitting/strengthening may be performed to restore the ring beam's original function. A masonry ring-beam reinforced with composites was retrofitted into the masonry building. (Retrofitting of masonry building with reinforced masonry ring-beam) The results showed that the load-bearing ability of the masonry ring-beam reinforced with composites is good. This method, like constructional columns, is simple to install on newly constructed buildings.

### 2.8.3 Tie Bars

Tie bars may also be used to improve the masonry building's structural integrity. The tie bar's aim is to apply compression stress to the masonry wall either horizontally or vertically, similar to the post-tension technique. (A. Darbhanzi) Conducted a series of tests on masonry panels retrofitted with vertical steel ties in some cases where the base settled unevenly and the building inclined, and the tie bars can be used to redress the inclined sections back to their original level. The findings showed that vertical relations would significantly improve the seismic ability of masonry structures in terms of both strength and ductility. It should be remembered that the bar's surface treatment should be done with caution to prevent corrosion.

### 2.8.4 Fibre/Textile-reinforced Mortar

A masonry structure's mortar is usually too thin to recognize its tensile strength. As a result, a masonry element's tensile and flexural strength are often overlooked in favor of its compressive strength. Mortar mixed with fiber/textile can be used to increase stress and flexural resistance, and thereby strengthen the masonry structure's integrity. The use of fibre/textile additives in the mortar helps to increase the tensile strength of the mortar. (F. Porto) used plaster and hybrid glass fibres to reinforce the masonry infill walls. The results showed that it is not only effective in preventing masonry panel expulsion in out-of-plane directions, but also in reducing global in-plane damage. Similarly, (A. Martins, 328-342) used the Textile-Reinforced Mortar (TRM) technique to avoid brittle failure. The ductility and strength in the out-of-plane direction were both increased as a result of the experiment. It should be noted, however, that the increase in honesty is not as significant as the methods described above. Steel Reinforced Grout (SRG) is a type of mortar that is rendered by embedding ultra-high tensile strength steel chords in the mortar in a convex masonry substrate (Santis). However, the efficiency of SRG is influenced by the masonry surface roughness and curing conditions. As a result, the findings of this analysis were found to be inadequate for a thorough understanding of this methodology. The re-pointing and grout injection methods are very similar in this technique.

### 2.8.5 Mortar Joint Treatment

Upgrading the element strength of masonry structures or masonry bearing walls, including vertical and horizontal masonry components, will increase the load resistance of the entire structure, increasing the masonry structure's or masonry element's ability to resist unexpected external loadings. This is the most common term used when retrofitting/strengthening masonry structures.

The masonry units in the buildings may still be of good quality, but the mortar may be weak or not entirely filled. As a result, the mortar may be replaced or refilled with a stronger bonding material. The most commonly used methods are grout injection and re-pointing.

Filling voids and cracks with grout is how grout injection is done (M. Schuller). Different types of grouts have been created for filling spaces varying in size from very small cracks to wide voids and empty joints. This method has been found to be successful in restoring the initial stiffness and strength of masonry, but no substantial change in the initial stiffness or strength has been observed. Even if the grout can be replaced with a stronger material, the difference isn't important. The addition of 2% Ordinary Portland cement to the mortar had little or no impact on the ultimate acceleration resistance, according to (R. Tetley). However, if used in conjunction with other methods, this technique's effectiveness may be increased. (D. Tinazzi) Performed research on masonry structures using FRP rods and re-pointing techniques. The findings showed that the most efficient retrofitting technique is re-pointing combined with FRP laminates. It should be noted that this method can only function effectively if the mix's mechanical properties, as well as its physical and chemical compatibility with the retrofitted masonry, are met (P. Alcaino). The preservation of original aesthetics and compatibility in terms of physicochemical and mechanical characteristics are the most critical considerations in the retrofitting of masonry heritages (M. Apostolopoulou). The former implies that after retrofitting, the authenticity of masonry heritages must be preserved, while the latter implies that masonry and the retrofitting material must be compatible in terms of physicochemical and mechanical efficiency. Incompatible retrofitting materials can cause decay mechanisms or even result in catastrophic outcomes. The use of grout injection and re-pointing will help to keep masonry heritage looking its best. As previously stated, the physical and chemical compatibility between masonry heritage and retrofitting materials is critical, while the interaction between retrofitting material and masonry is still not known clearly. As a result, recent research on the design and selection of restoration mortar is connected with compatibility evaluation to ensure the long-term longevity of masonry heritage. This research provided a methodological approach for the selection of restoration mortars based on fragility analysis. In the report, setting criteria during the characterization of retrofitting materials and the investigation of masonry heritage allows for the selection of the best mortar while still meeting the set compatibility and performance requirements.

### 2.8.6 External Steel Reinforcement

This technique involves installing steel elements next to the original masonry piece, which may or may not be tied together. Small cracks are likely to appear during an earthquake, and they will grow and spread if external loading exceeds the structure's load carrying capacity. The modern steel framework, on the other hand, has a significant stiffness and will prevent masonry wall cracking from spreading (A. Hamid) (D. Rai). In such situations, the stronger steel system will carry the external load, while the initial masonry system can serve as a structural element rather than carrying loads. (Taghdi) Performed research by directly connecting steel members to masonry walls, and the findings showed that the reinforced wall's lateral strength was increased by around 4.5 times in the in-plane direction. Other studies have found that this steel reinforcement device improves the masonry structure's resistance, ductility, and energy absorption substantially (D. Rai) (Taghdi). Since steel is a good retrofitting material, this technique is very effective in improving a structure's load resistance. As a result, this method can be used on masonry structures that are fragile or need to be significantly improved. However, since the appearance of steel can alter the original masonry structure's aesthetics, it is not a suitable retrofitting solution for masonry heritage. Furthermore, the high cost of implementation in developed countries is a problem.

## 2.9 Recycling of Bricks

Salvaging, cleaning, and reusing old bricks from a demolished brick wall has several advantages. Many individuals choose ancient bricks with an aged look to give a new project character. Of course, there are financial benefits as well as the joy of doing something good for the environment. Bricks may be salvaged for reuse by a competent individual with no prior construction expertise. The work is potentially hazardous and should be undertaken with caution. Cleaning bricks is a difficult task, but the rewards are numerous.

### Safety First

Depending on the height of the brick wall, dismantling it might be quite risky. Appoint a leader with relevant construction experience, and make sure you have the required number of personnel on hand to remove the wall. Hard helmets, breathing masks, safety goggles, and steel-toed shoes should all be used. Helpers must be trained in the jobs they undertake and must be closely monitored. Working with muriatic acid while working with falling bricks or crumbling walls is risky job that must be done with caution.

### Dismantle the Brick Wall

Brick walls that are less than waist high pose a risk of falling bricks, while those that are more than waist height provide a serious risk of harm from collapse. Begin at the top of the wall and work your way horizontally along the top course of bricks, removing one brick at a time.

Drop down to the following course, traveling horizontally, after removing the top course; never start at the center or bottom of the wall. Bricks can be dismantled by more than one person as long as they work on the same course of bricks. Use the 2-

pound sledgehammer to forcefully smash the mortar joint with the cold chisel. To remove the mortar surrounding the brick's border, repeat the operation.

## Chip the Mortar

To ensure that the new mortar sticks to the brick, the old mortar must be removed. With a brick hammer – an 8-inch-long hammer with a 1-inch-square hitting surface on one side and a curved, tapered end on the other — chip away at the old mortar. Simply strike the clumps of mortar with the striking surface and scrape off the remnants with the tapered end. By soaking the bricks in water and repeating the chipping procedure with the hammer, you can loosen resistant mortar.

## Reusing bricks keeps them out of landfills.

One of the most obvious advantages of brick recycling is that it prevents them from being thrown away. Bricks make up a large portion of the construction material removed during the demolition of ancient structures and residences. Unlike poured concrete slabs, stairs, and timber components, however, bricks are only restricted by their placement. While it may be difficult to recycle some wood or concrete steps, the possibilities for bricks are virtually endless.

## Brick recycling reduces the need for new bricks.

The brick-making process consumes a significant amount of energy, materials, and even certain hazardous chemicals. When bricks are recycled, they reduce the amount of trash generated throughout the production process.

## How Are Bricks Recycled?

It's essential to create plans for the preservation of bricks for recycling before demolishing any brick construction. While tearing down a brick wall with a heavy or harsh equipment is relatively simple, recycling bricks is a bit more delicate. If the bricks are to be saved, they must be carefully removed one by one, and the mortar must be removed in a variety of methods. The demolition procedure might be less accurate if the brick material is to be used in a crushed form.

## Strength

No guarantees can be made about the strength of certain recycled bricks. The crushing strength of modern bricks is categorized in reference to a sample taken from bulk amounts of newly made bricks, therefore using this approach to measure the crushing strength of recovered bricks would be impracticable.

Modern bricks are manufactured using more regulated manufacturing processes than in the past, resulting in product uniformity. A recycled material should be expected to have more diversity in its characteristics. However, the use of reused brick in two- and three-story home building is unlikely to be limited by strength requirements.

## Water Absorption

A clay brick absorbs a considerable amount of water. There is no assurance that samples of recovered bricks reflect a consignment of consistent units in the calculation of flexural strength in the design of structural brickwork. This property, on the other hand, is unlikely to limit the use of recovered bricks in the sorts of building for which they are often recommended.

## Movement Joints

Although the irreversible moisture movement that happens over the life of clay bricks will have occurred in reclaimed bricks, cyclic thermal movement will still occur in brickwork erected with them.

# CHAPTER 3

# METHODOLOGY

## 3.1 Introduction

Destructive tests on masonry panels were used in this work to assess the major mechanical parameters of bricks, mortar, and the masonry assemblage. The testing protocols are set by the American Society for Testing and Materials (ASTM), which lays out all of the stages that must be performed. Many scholars that have experimented with unenforced clay brick masonry all throughout the world have adopted these criteria. (A. Borri, 2015) (N. Ismail, 2011) (A. Brignola, 2008) (C. Faella, 2010) (J. Milosevic, 2013) (D. Dizhur, 2011) (Boem, 2015)

## 3.2 Determination of bricks parameteres

In this section, we'll look at how to figure out what physical and mechanical properties are needed for bricks. Physical requirements, sampling and testing processes, weight and water absorption determinations, compressive and tensile strength, and dimensioning are all computed according to ASTM guidelines.

### 3.2.1 Physical requirements

According to (ASTM, 2004) bricks should be made of clay, shale, or similar naturally occurring earthy material, molded, pressed, or extruded during the manufacturing process, and fired. When the brick is brought to the job site, it should be visually inspected to ensure that it is free of faults, flaws, and other surface treatments that would compromise the brick's strength or performance during the construction process. Table 1 lays out the physical criteria.

### 3.2.2 Sampling and testing procedur

These test methods provide the standard approach for determining the distinctive parameters of clay bricks such as modulus of rupture, compressive strength, and water absorption, as well as weight, size, and void area, (ASTM, 2014)

#### 3.2.2.1 Sampling

Full-size bricks were chosen for the tests on the condition that they were representative of the lot of units from which they were chosen, that they were free of or brushed to remove dirt, mud, mortar, or other foreign materials unrelated to the manufacturing process, and that they were free of or brushed to remove dirt, mud, mortar, or other foreign materials unrelated to the manufacturing process. At least 10 individual random bricks were selected and marked with an individual unique number for the assessment of modulus of rupture, compressive strength, and absorption (based on ASTM standards).

#### 3.2.2.2 Weight determination

This procedure begins with drying the test specimens for at least 24 hours in a vented oven at 110 to 115°C. Following the drying process. At a temperature of 24°C, the specimens were chilled. Then, five full-size dry specimens were weighed on a scale with a capacity of at least 3000 g and a 0.5 g sensitivity. The weight of each specimen was recorded separately, with the average of the five being calculated to the closest 0.1 g.

### 3.2.2.3 Modulus of Rupture ( Flexure Test)

Five full-size dry units were tested. The test specimen is supported flatwise (the load is delivered in the direction of the unit's depth) over a span of about 25.4 mm during the testing operation. The average of each specimen reported to the closest 0.01 MPa is used to compute the modulus of rupture.

$$\boldsymbol{S = \frac{3W(\frac{l}{2}-x)}{bd^2}} \qquad \text{Equation 10}$$

S = modulus of rupture of the specimen at the plane of failure, Pa;

W= maximum load indicated by the testing machine. N;

l= distance between the supports. Mm;

b= net width (face to face minus voids). of the specimen at the plane of failure. mm;

d = depth. (bed surface to bed surface). of the specimen at the plane of failure mm;

x = average distance from the mid-span of the specimen to the plane of failure measured in the direction of the span along the centerline of the bed surface subjected to tension. mm.

### 3.2.2.4 Compressive Strength

Five dry half-brick test specimens acquired by any method that produces, without fracturing or splitting a specimen with roughly level and parallel ends, the entire height and breadth of the unit, and a length equal to one half of the unit's entire length of 25.4 mm. The flatness of the brick sample is tested: (the load is applied in the direction of the depth of the brick). The compressive strength is obtained by taking the average of the specimens and rounding it up to the closest 0.01 MPa as follows:

$$\boldsymbol{C = \frac{W}{A}} \qquad \text{Equation 11}$$

C = compressive strength of the specimen. $Kg/cm^2$;

W= maximum load, N, indicated by the testing machine;

A = average of the gross areas of the upper and lower bearing surfaces of the specimen, $cm^2$.

#### 3.2.2.5 Water Absorption

For testing, four brick examples were used. The test specimens should be dried and cooled. Submerge the dry, cooled specimen in clean water for 5 and 24 hours, without first partial immersion. The specimens were then removed, washed dry with a moist towel, and weighed using a balance with a capacity of at least 2000 g and a sensitivity of 0.5 g.

The cold water absorption of each specimen was calculated to the closest 0.1 percent as follows:

$$\boldsymbol{Absorption}, \% = \frac{\mathbf{100}(\boldsymbol{Ws}-\boldsymbol{Wd})}{\boldsymbol{Wd}} \qquad \textbf{\textit{Equation 12}}$$

where :

Wd = dry weight of the specimen;

Ws = saturated weight of the specimen after submersion in cold water.

#### 3.2.2.6 Measurements of Size

Using steel standards graded in 1-mm divisions, ten entire dry full-size items typical of the lot were evaluated, including the extremes of color range and size determined by eye inspection. Each specimen's average width, length, and height are determined to the nearest 0.8 mm.

## 3.3 Specifications of mortar properties

According to ASTM C 270-03, the classification of standard mortars to be used in construction is described in Table 2 (ASTM, 2003).

### 3.3.1 Determininination of mortar strength

Two levels of tamping are used to condense 50-mm test cubes. The cubes are cured in the molds for one day before being peeled and soaked in lime water till they are evaluated. The specimens are analyzed as soon as they are removed from the storage water. The load is imparted to the specimen faces that were in touch with the mold's true plane surfaces. The testing machine's total maximum load is recorded, and the compressive strength is computed as follows:

$$\boldsymbol{ʄm = \frac{P}{A}} \qquad \textit{\textbf{Equation 13}}$$

**ʄm** = compressive strength in MPa;

**P** = total maximum load in N;

**A** = area of loaded surface in $mm^2$.

To the closest 0.1 MPa, the average compressive strength of all approved test specimens manufactured from the same material and tested at the same time was computed.

## 3.4 Determination of masonry assemblage compressive strength

The techniques for constructing and testing masonry prisms are included in this test technique and processes for assessing the compressive strength of masonry ʄmt, which are used to determine whether or not masonry ʄ^m meets the stipulated compressive strength. To begin, the top and bottom faces of the prisms' length and breadth at the margins are measured to the closest 1.3 mm (ASTM International, 2004). Figure 8 depicts the failure mode of the assembly.

Each masonry prism's compressive strength is computed by dividing the greatest compressive load it can withstand by its net cross-sectional area. In addition, the $h_p/t_p$ ratio for each prism is determined using the height and least lateral dimension of that prism. The adjustment factor from Table 3 is then calculated.

***Table 3*** Height to Thickness Correction Factors for Masonry Prisms Compressive Strength ***(ASTM International, 2004)***

| $h_p/t_p$* | 1.3 | 1.5 | 2.0 | 2.5 | 3.0 | 4.0 | 5.0 |
|---|---|---|---|---|---|---|---|
| Correction factor | 0.75 | 0.86 | 1.0 | 1.04 | 1.07 | 1.15 | 1.22 |

* $h_p/t_p$ – Ratio of prism height to least lateral dimension of prism

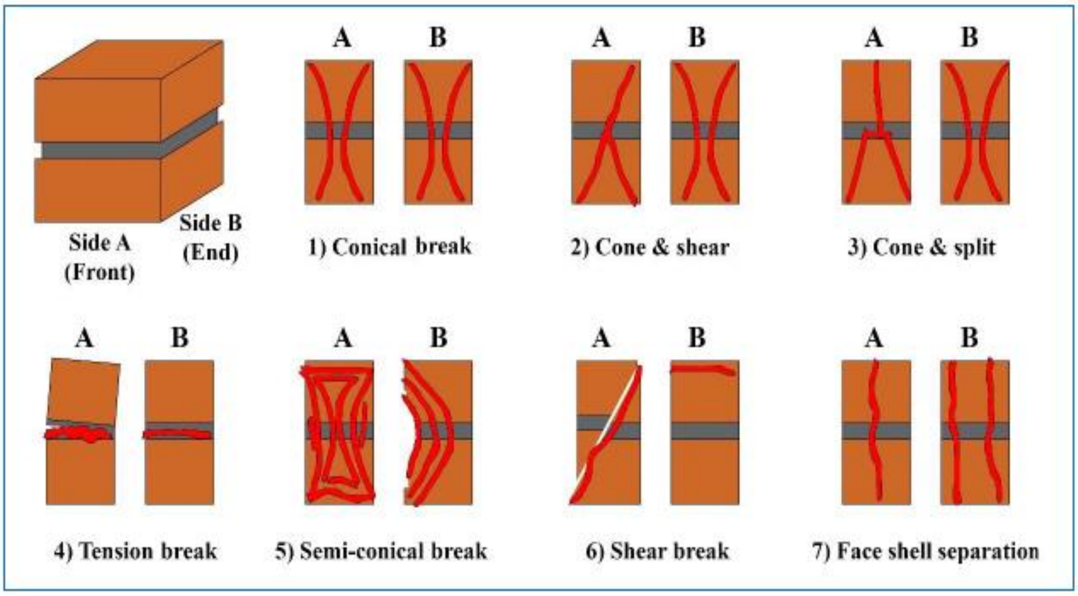


**Figure 7** Failure modes of masonry prims (reproduced after ***(ASTM International, 2004)***)

## 3.5 Determination of diagonal tensile strength (shear strength)

(ASTM International, 2002) The typical testing protocol calls for a 45-degree rotation of the specimen and vertical loading along one of the wall's diagonals. However, the wall's masonry bond strength is poor, as well as the potential of unintentionally adding more stress to the overall state of stress. As shown in Figure 10, the test set-up was adjusted so that the wall specimen remained upright in its original location and the loading mechanism was rotated. Two loading shoes are put on two diagonally opposite corners of the panel and are linked by four high strength steel rods positioned along the compressed diagonal in the mobile test set-up (Figure 8). Between the top loading shoe and a metallic plate attached to the steel rods, a hydraulic jack with a 50-tonne capacity was added. When loaded, the four steel rods connecting the loading shoes created tension pressures, diagonally squeezing the wall allowing for the desired failure mode: diagonal cracking and/or bed joint sliding failure. The applied stress was steadily raised until it reached a breaking point. Two diagonally positioned displacement gauges installed to every wall panel throughout a gauge length of 1000 mm, aligned parallel and perpendicular to the loading direction, recorded the deformations of the wall specimen (compression and elongation of diagonals).

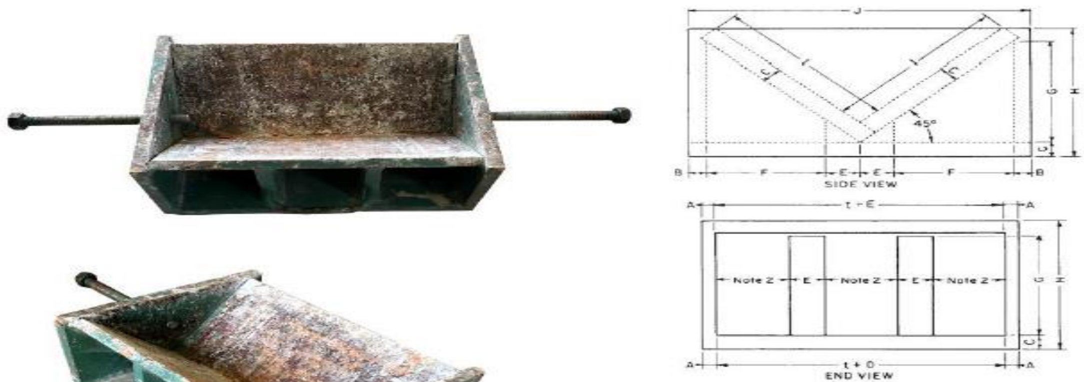


Table of Metric Equivalents

| | Metric Units, mm | | Metric Units, mm |
|---|---|---|---|
| A | 10 | F | 89 |
| B | 13 | G | 114 |
| C | 16 | H | 146 |
| D | 22 | I | 152 |
| E | 25 | J | 254 |

***Figure 8*** Loading shoes used for diagonal compression test ***(Mustafaraj, 2016)***

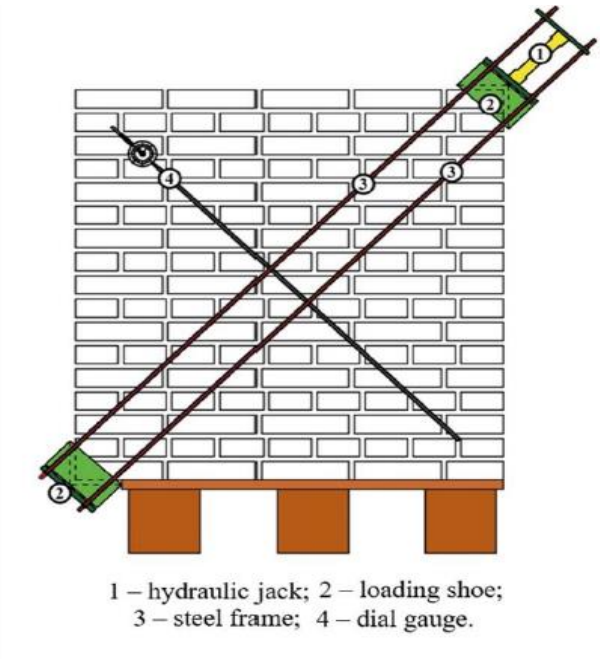


***Figure 9*** Diagonal compression test set-up (schematic view) *(Mustafaraj, External Shear Strengthening of Unreinforced Damaged Masonry Walls, 2016)*

The load distribution along the corners of the wall panels was carefully considered during the application of the diagonal compression test in order to avoid an excessive concentration of compressive stresses at the surface of metallic plates. This test method determines the diagonal tensile or shear strength of 1.2 by 1.2-m masonry assemblages by compressing them along one diagonal and inducing a diagonal tension failure, with the specimen splitting apart parallel to the load direction (Figure 9). The test should be performed on at least three specimens with the same size and kind of masonry units, mortar, and craftsmanship, according to this technique. The specimens should not be moved for at least 7 days and should be preserved in the laboratory for at least 28 days to achieve proper cure.

The calculation procedure is as follows:

$$\boldsymbol{s_s = \frac{0.707P}{A_n}} \qquad \textbf{\textit{Equation 14}}$$

where:

$S_S$— shear stress (MPa);

P- load exerted along the compression diagonal (N);

$A_n$ - net area of the specimen (mm2);

$$\boldsymbol{A_n = \frac{w+h}{2} t * n} \qquad \textbf{\textit{Equation 15}}$$

where:

w — width of specimen (mm);

h — height of specimen (mm);

t — total thickness of specimen (mm);

n - percent of the gross area of the unit that is solid, expressed as a decimal

$$\Upsilon = \frac{\Delta \boldsymbol{V} + \Delta \boldsymbol{H}}{\boldsymbol{g}} \qquad \textbf{\textit{Equation 16}}$$

where:

$\Upsilon$ -shearing strain (mm/mm);

$\Delta \boldsymbol{V}$ — vertical shortening (mm);

$\Delta \boldsymbol{H}$ - horizontal extension;

g — vertical gage length;

$$\boldsymbol{G = \frac{s_n}{\Upsilon}} \qquad \textbf{\textit{Equation 17}}$$

where:

G- modulus of rigidity, MPa

## 3.6 Stifness

The Shear Modulus, G, is computed as the ratio of shear stress to shear strain and may be used to determine material stiffness. The secant modulus of $0.05\tau_{max}$, and $0.70\ \tau_{max}$ of the stress-strain response curve may be used to calculate the shear modulus. The Modulus of Elasticity, E, which is linked to shear modulus by the following Equation 18, where v = 0.25 is accepted by (G. Pande, 1998) , may be used to quantify the stiffness of a wall specimen.

$$\boldsymbol{E = 2G * (1 + v)} \qquad \textbf{\textit{Equation 18}}$$

## 3.7 Experimental Process

Three wall panels were employed in this study's experimental campaign: There are two types of wall panels: unreinforced (plain specimen) and reinforced. The dimensions of the walls are 1200 x 1200 x 25 mm. The solid clay bricks used in this project were salvaged from previous wall panels that had been damaged and were originally purchased in Fier, Albania. The clay bricks were made on-site at the facility, utilizing clay quarries close by. All of the bricks were backed at the same time and have features that are almost comparable. The remaining components came from Fushe-Kruje, which is famed for its cement and lime manufacturing. CEM II/B-L 32.5 R cement was chosen for its decreased water consumption and increased workability, and it was delivered in 25 kilogram bags.

## 3.8 Construction of Wall Panels (Unstrengthen Specimens)

The wall panels were made of substantial clay bricks and were put to the test. One type of mortar, type "0," was utilized for the mortar. The goal of this mixture was to replicate the mortar used in existing old buildings.

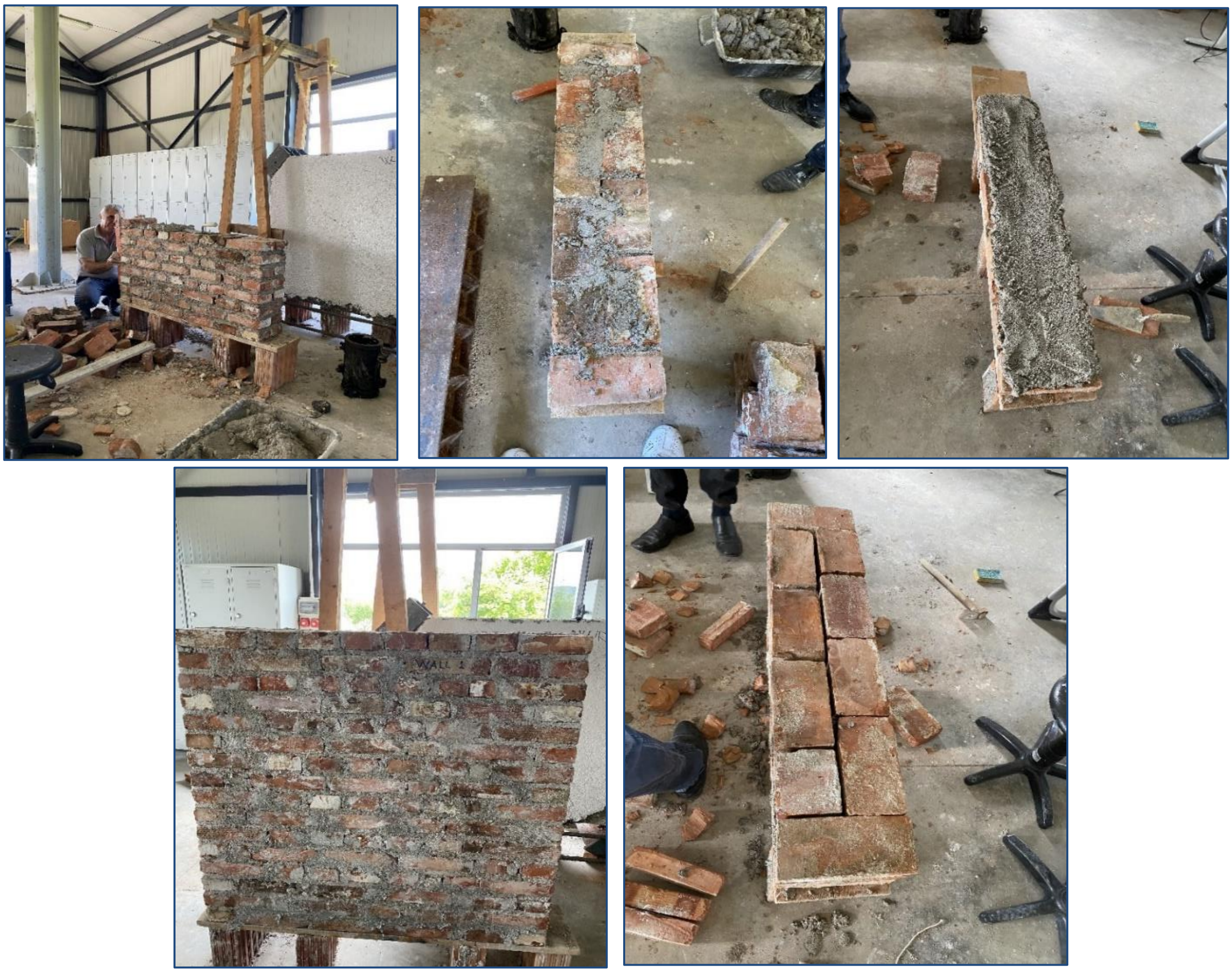

***Figure 10*** Construction process plain walls

The wall panels were constructed using two leaf, English bond, and recycled clay bricks with typical nominal dimensions of 243.7 mm x 119.2 mm x 58.5 mm and 15 mm thick hydraulic cement mortar joints with a volumetric mix ratio of cement: lime: sand of 1:2:9, which was found to be a good representative of existing old buildings.

All of the wall panels were constructed in a laboratory by skilled masons utilizing the English bond, which is the most common bond used in URM structures across the world, including Albania. Before testing they were allowed to cure for 28 days. After that period of time had passed, a coat of white lime paint was added to offer a better medium for evaluating the crack development.
A total of three (3) wall panels were tested, out of which one (1) was plain wall, one (1) was polypropylene fiber reinforced on one side and one (1) was polypropylene fiber reinforced on both sides prior to testing. In total, 3 diagonal compression tests were performed on 3 specimens with nominal dimensions of 1.2m x 1.2m x 0.25m.
All of the wall panels were created in a laboratory and tested on their own. The testing method was created in such a way that no damage to the walls was produced. In Section 3.5, the experiment setup is detailed in full. The testing proceeded until the wall panels collapsed; once the major diagonal fracture appeared, the ultimate load suddenly dropped (it reached to zero up to a few tones).

## 3.9 Reinforcing techniques

The aim of strengthening of URM is to increase resistive capacity of the masonry under combined tensile and compressive forces. As discussed in Section 2.8, there are a number of techniques that can be used to reduce the risks associated with natural disasters and structural deterioration over time, as well as to improve load resisting capacity and overall structural performance, thereby extending the service life of URM structures. In the next section, we'll go through the chosen strengthening technique.

### 3.9.1 Textile Reinforced Mortar (TRM) — plastering with polypropylene fibers (W-X-PP)

Plastering the ordinary walls with a 25 rum thick layer of fiber reinforced mortar on both sides is a method of strengthening with polypropylene fibers (Figure 11). The mortar is made with a 1:1 sand-to-cement ratio, 1.5 percent fibers by volume, and a water-to-cement ratio of 0.5, which has been proven to be the right quantity for workability. Cracking and shear capacity, as well as toughness, are improved by the fibers. Table 4 summarizes the technical characteristics of the fibers. They do not, however, have a significant impact on the matrix's compressive strength (mortar). The mix is prepared by dry mixing the fibers with the sand and cement, then adding water at the end, resulting in a plater mix with medium workability.

***Table 4*** Technical specifications of polypropylene fibers.

| | |
|---|---|
| **Chemical base** | 100% polypropylene fibre |
| **Specific gravity** | 0.91g / $cm^3$ |
| **Fibre length** | 12mm |
| **Fibre diameter** | 18 micron-nominal |
| **Melt point** | 160°C |
| **Ignition point** | 365°C |
| **Thermal conductivity** | Low |
| **Electrical conductivity** | Low |
| **Specific surface area of fibre** | 250$m^2$ / kg |
| **Acid resistance** | High |
| **Alkali resistance** | 100% |
| **Tensile strength** | 300 - 400 N / $mm^2$ |
| **Module elasticity** | ~ 4000 N / $mm^2$ |

***Figure 11*** Plastering process with polypropylene fibers (PP).

# CHAPTER 4

# Experimental Results

## 4.1 Introduction

The experimental testing program was undertaken to study the structural performance of URM, namely the in-plane performance of diagonal shear cracking and/or bed joint sliding mode of failure, which was reinforced with ferrocement jacketing. Polypropylene reinforced mortar is a type of mortar that is reinforced with polypropylene fiber Glass and carbon-reinforced plastics.

During a diagonal compression test (ASTM E 519-02), the wall panel's behavior was studied, and critical characteristics such as kinds of failure modes and shear strength were found. In-plane performance dictated by diagonal shear cracking mode of failure received significant attention. (Mustafaraj, 2016)

The diagonal compression test was carried out according to Section 3.5's instructions. Three diagonal compression tests were carried out on three type 0 mortar specimens measuring 1.2m x 1.2m x 0.25m.

Material properties of brick, mortar, and masonry assemblage were established before to testing of wall panels, as detailed in Sections 3.2 and 3.3.

## 4.2 Brick properties

Determination of brick parameters was done according to the ASTM C 62-04, explained in details in Section 3.2

### 4.2.1 Brick Dimensioning

The method of measuring is described in figure 12, and the results of brick dimensioning are presented in table 5.

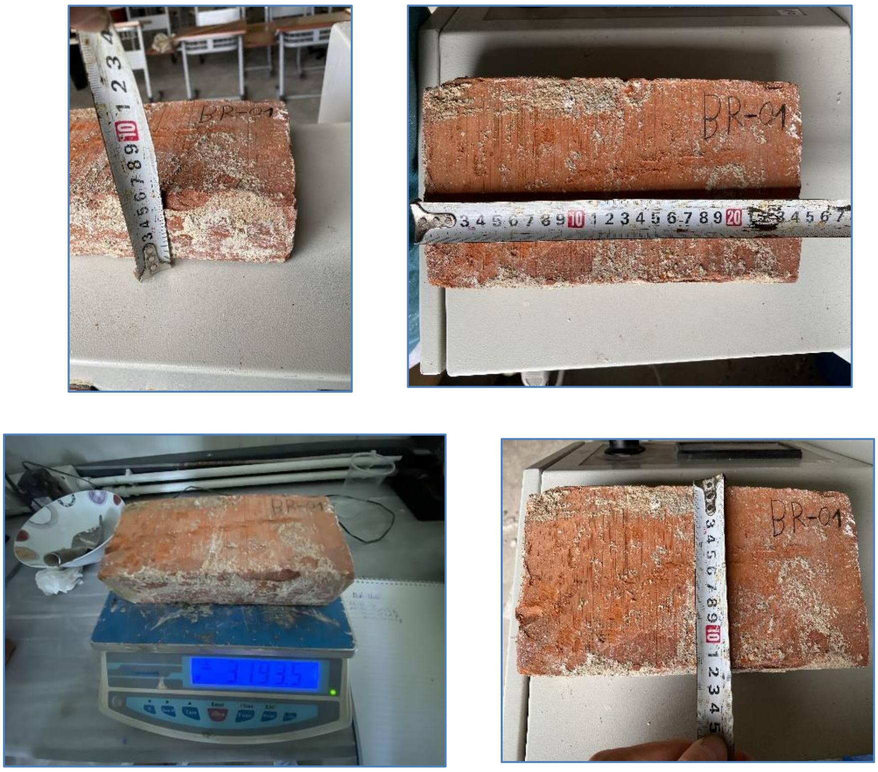


***Figure 12.*** Determining brick dimensions

Table 5 Dimensioning of the bricks

| Sample name | Length (l) (mm) | Width (w) (mm) | Thickness (t) (mm) | Area ($mm^2$) | Weight (g) |
|---|---|---|---|---|---|
| BR-01 | 241 | 119 | 59 | 28679 | 3193.5 |
| BR-02 | 245 | 117 | 57 | 28665 | 2874 |
| BR-03 | 243 | 120 | 58 | 29160 | 2736 |
| BR-04 | 242 | 119 | 59 | 28798 | 3243.5 |
| Average Brick (BR-X) | 242.75 | 118.75 | 28.25 | 28826.6 | 3011.75 |

***Table 6*** Determination of weight per unit area of the bricks

| Sample name | Dry Weight (g) | Area ($mm^2$) | Weight per unit area ($g/mm^2$) |
|---|---|---|---|
| BR-01 | 3193.5 | 14219 | 0.22 |
| BR-02 | 2874 | 13965 | 0.2 |
| BR-03 | 2736 | 14337 | 0.19 |
| BR-04 | 3243.5 | 14278 | 0.22 |
| Average Brick (BR-X) | 3011.75 | 14199.75 | 0.21 |

### 4.2.2 Brick Dimensioning

The procedure of determining the water absorption is describing in details in the Figure 13 and the results are tabulated in table 7.

***Figure 13*** Determination of brick water absorption.

*Table 7* Determination of water absorption

| Sample name | Dry Weight (g) | Fully Saturated Weight (g) | Water Absorption (%) |
|---|---|---|---|
| BR-01 | 3193.5 | 3627.5 | 13.59 |
| BR-02 | 2874 | 3438 | 19.62 |
| BR-03 | 2736 | 3296.5 | 20.48 |
| BR-04 | 3243.5 | 3651.5 | 12.57 |
| Average Brick (BR-X) | 3011.75 | 3503.3 | 16.56 |

## 4.3 Compressive strength

The test technique described in Section 3.2.2.4 was used to assess the compressive strength of bricks. Table 8 shows the results of a compression test on four clay bricks chosen at random.

***Table 8*** Determination of compressive strength

| Sample name | Length (l) (mm) | Width (w) (mm) | Area ($mm^2$) | Ultimate Load (kN) | Compressive Strength (MPa) |
|---|---|---|---|---|---|
| BR-01 | 141 | 119 | 16779 | 312.4 | 18.61 |
| BR-02 | 145 | 117 | 16965 | 325.7 | 19.19 |
| BR-03 | 143 | 120 | 17160 | 319.2 | 18.6 |
| BR-04 | 142 | 119 | 16898 | 322 | 19.5 |
| Average Brick (BR-X) | 142.75 | 118.75 | 16951.5 | 319.825 | 18.86 |

## 4.4 Mortar Properties

Compressive and flexural strength were the key properties of mortar that needed to be assessed. The testing process was carried out according to Section 3.3. The flexural and compressive tests on mortar are presented in Figure 14. Table 9 show the results, which show that mortar samples were taken for each wall specimen.

***Figure 14*** Determination of mortars flexural and compressive strength

As it may be seen from the Table 9, the average compressive and tensile strengths of mortars are 4.3MPa and 1.77 MPa, respectively. Based on the test results, the types of mortar can be categorized as Type “0" according to ASTM C 270-03 (ASTM, ASTM C 270-03 Standard Specification for Mortar for Unit Masonry, 2003).

***Table 9*** Results of the compressive and flexural strength of mortars from wall panels

| Specimen Name | Mean Compressive Strength of Mortar (MPa) | Mean Flexural Strength of Mortar (MPa) |
|---|---|---|
| W-01 | 4.7 | 1.9 |
| W-02 | 3.9 | 1.7 |
| W-03 | 4.3 | 1.7 |
| Average | 4.3 | 1.77 |

## 4.5 Masonry Compressive Strength Results

Section 3.4 was used to determine the compression strength of the masonry assembly. The fractures have propagated in both mortar joints and brickwork, as seen in Figure 14. The prism's average compressive strength was determined to 8.94 be MPa. The compressive strength of the masonry assemblage was higher than that of the mortar but lower than that of the bricks individually, as predicted. The results of 3 tested prisms are presented in Table 10.

***Figure 15*** Determination of masonry assemblage compressive strength

***Table 10*** Results of the compressive strength of masonry assemblage.

| Prism Name | Length (mm) | Width (mm) | Height (mm) | Load (kN) | Hp/tp | Correction Factor | Compressive Strength (MPa) |
|---|---|---|---|---|---|---|---|
| P-1 | 241 | 119 | 204 | 219.7 | 0.85 | 1.5 | 11.49 |
| P-2 | 245 | 117 | 198 | 139 | 0.81 | 1.5 | 7.27 |
| P-3 | 243 | 120 | 201 | 157 | 0.83 | 1.5 | 8.07 |
| Average (P-X) | 243 | 118.7 | 201 | 171.9 | | | 8.94 |

## 4.6 Failure modes (crack pattern)

A heterogeneous structure's exact crack pattern is difficult to anticipate. The failure mode is highly reliant on the characteristics of the mortar and bricks, as discussed in Section 2.5. Because the mortar joints are the weakest link in the masonry assemblage, and because the load is delivered diagonally, crack propagation is likely to occur diagonally across the mortar layers. Nonetheless, anticipating which course of mortar layer will be applied is very difficult.

All of the failure mechanisms of the tested specimens are provided in this section. The findings of the experiments revealed that all of the specimens examined had a similar failure pattern, which was primarily characterized by a step-like crack along one of the diagonals. Nonetheless, the panel's fracture propagation, maximum deformation, and ultimate load bearing capability were shown to be highly reliant on the reinforcing technique.

### 4.6.1 Plain wall (PP-Plain)

Cracking was detected in the unstrengthen wall panel along the squeezed diagonal, mostly via the mortar joints in a diagonal step pattern. Sliding along the mortar bed joints was noticed, followed by diagonally expanded fractures (Figure 16). Tension failure followed by shear-sliding along the compressed diagonal in a step-like manner can be classified as the overall failure mechanism.

Shear sliding began at the third course from the top of the panel, proceeded horizontally along the bed joint for about 250 mm, and then propagated entirely via the mortar joints in a diagonal step-like pattern. Along the compressed diagonal, the remaining plain panels have a step-like design. The fractures appeared in the mortar joints even in such situations.

***Figure 16*** Failure mode of plain wall

The unstrengthen wall built with type "0" mortar displayed a step-like diagonal fracture with a failure in the mortar joints, as shown in Figure 16, owing to the mortar's relatively low strength.

### 4.6.2 Polypropylene reinforced wall panels (W-X-P)

The wall panel PP-1Side exhibited a deep crack along the compressed diagonal, followed by some other cracks parallel to it (Figure 17).


***Figure 17*** Failure mode of polypropylene reinforced mortar strengthened wall panel on 1 side

### 4.6.3 Summary of failure modes

Cracking occurred mostly via the mortar joints, and failure was linked with the formation of a stair-like fracture along the diagonal in all of the examined specimens. The panels' overall reaction may be classified as diagonal tension failure followed by shear sliding along fractured diagonal stepped joints.

## 4.7 Shear stress-strain response

Figure 18 - Figure 21 depicts the shear stress-strain relationship. Prior to fracture initiation, the experimental curve for all wall panels was roughly linear, followed by a nonlinear section of the curve up to the maximum strength. This same pattern of behavior was seen in other research as well. (Mustafaraj, 2016) (Borri, Castori, Corradi, & Speranzini, 2011) (A. Borri, 2015).

The curves are plotted on a scale of maximum strain, $\varepsilon_{max}$, 0.005, which corresponds to a drift of 0.5 percent (which is the allowable drift limit for masonry structures, as explained in detail in Section 0 where the maximum allowable drift ranges between 0.5-0.6 percent) and is considered to be an optimum value where the comparisons of all three experiments can be presented.

### 4.7.1 W-1 (Plain wall)

The plain wall is very fragile, and the stress-strain response is quite rapid. The maximum shear stress that the plain wall reaches is 0.095 MPa and the displacement of the shear strain for that maximum is 0.008 mm. The biggest shear strain that the wall have is 0.01 mm.

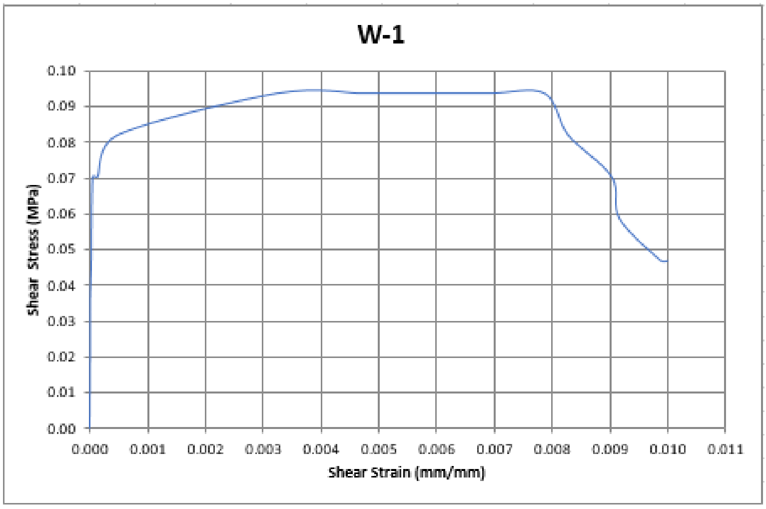


***Figure 18*** Stress-strain response of plain wall (W-1)

## 4.7.2 W-2 (PP-1S)

The reinforced in one side wall is not so fragile, and the stress-strain response is quite rapid. The maximum shear stress that the plain wall reaches is 0.19 MPa and the displacement of the shear strain for that maximum is 0.0025 mm. The biggest shear strain that the wall have is 0.0225 mm.

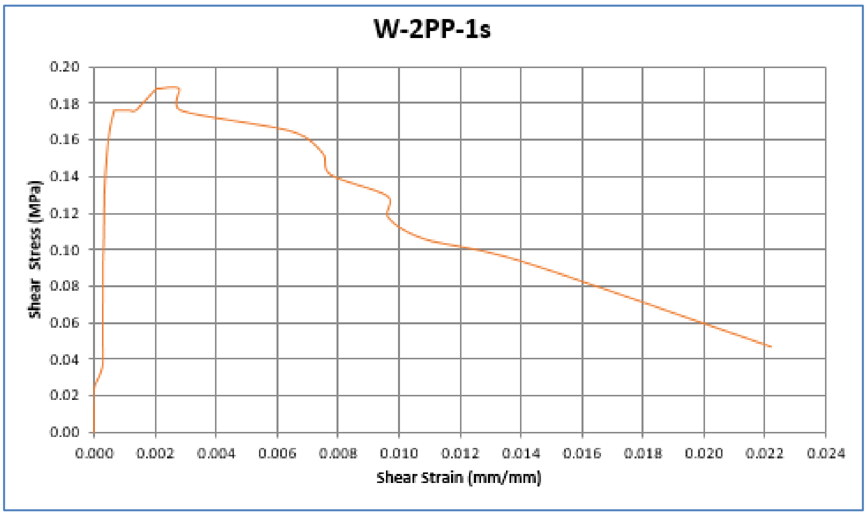


***Figure 19*** Stress-strain response of (W-2PP-1S)

## 4.7.3 W-2 (PP-2S)

The reinforced on both sides wall is compact, and the stress-strain response is quite rapid. The maximum shear stress that the plain wall reaches is 0.37 MPa and the displacement of the shear strain for that maximum is 0.001 mm (the values for the shear stress remains the same 0.001 till 0.003 mm of shear strain). The biggest shear strain that the wall have is 0.005 mm.

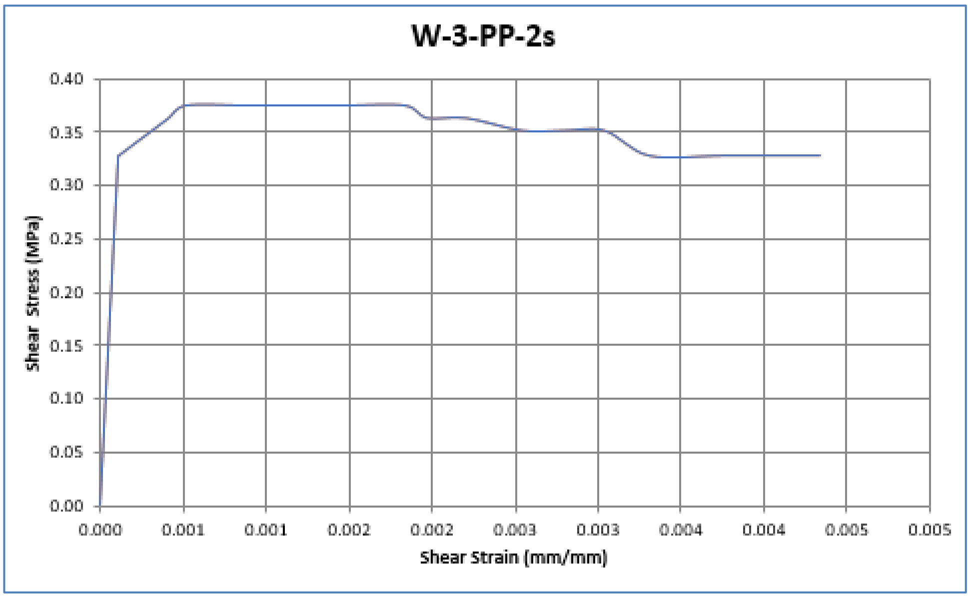


***Figure 20*** Stress-strain response of (W-2PP-2S)

## 4.7.4 Plain vs PP-1S vs PP-2S

In Figure 21, it is presented the comparison of the average stress-strain diagrams of the strengthening techniques together with the plain panel.

W3 (PP-2S) exhibits the highest ductility, followed W2 (PP-2S) and the last one is W1 (Plain wall). W3 (PP-2S) despite the high shear strength, deformation capacity was limited. W2 (PP-2S) of of all 3 wall panels reached the highest deformation capacity.

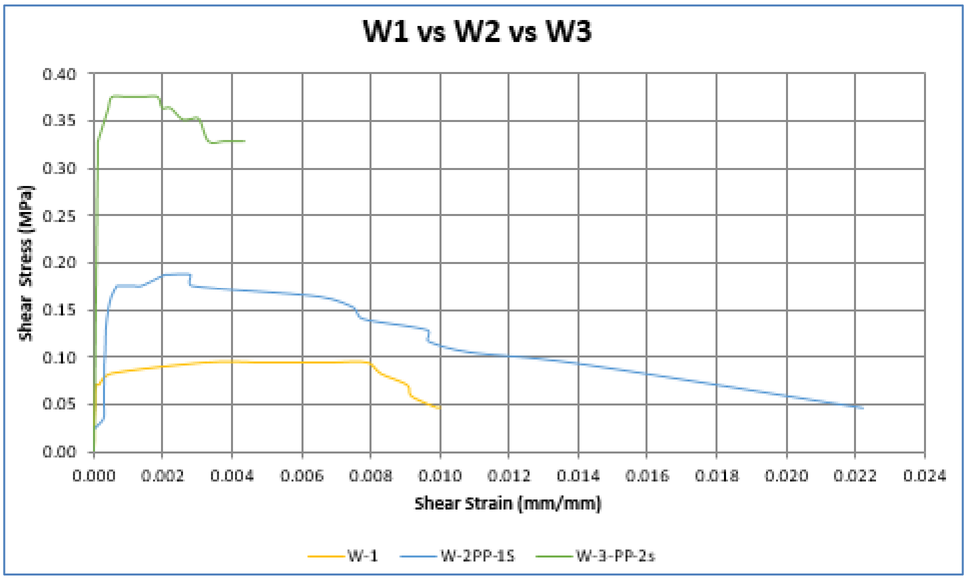


***Figure 21*** Comparison of strengthening techniques for wall panels

## 4.8 Shear Strength, Stiffness, Ultimate drift and Ductility

The ultimate drift and ductility are two more parameters to consider while evaluating the behavior of URN. As indicated in the previous chapter, URM buildings are subjected to lateral loads as a result of seismic shaking, which cause lateral deformation of the structure. Ductility is described as a material's capacity to deform without rupturing, or, in the case of URM structures, the structure's ability to deform without collapsing. The decrease in stiffness was often noticed at load levels around the ultimate load, when the first crack forms but is unable to grow owing to the existence of external reinforcement.

***Table 11*** Summary of mechanical parameters for Plain, PP-1S and PP-2S wall panels

| Panel Name | P (kN) | v (MPa) | δ (%) | G (MPa) | E (MPa) |
|---|---|---|---|---|---|
| W1 | 39.856 | 0.094 | 1.001 | 569 | 1422.5 |
| W2-PP-1s | 79.712 | 0.188 | 2.219 | 164 | 410 |
| W3-PP-2s | 159.424 | 0.376 | 0.434 | 667 | 1667.5 |

The testing findings revealed that the panel's shear strength is highly influenced by the mortar type (mortar strength), since the fractures spread through the joints without harming the bricks in all cases.

The highest shear strength was achieved by W3-PP-2s, 0.376 MPa which was 4 times higher than the shear strength of the plain panel of W1. Additionally, W3-PP-2s achieved higher ultimate diagonal load of 159.424 kN. W3-PP-2s, on the other hand, had a lower deformation capacity and were more brittle than plain panels, with an ultimate drift ratio of 0.434.

# CHAPTER 5

# CONCLUSIONS

## 5.1 Conclusions

The structural performance of unreinforced and reinforced masonry panels was studied in this research. The panels were constructed using local materials with similar characteristics: all of the clay bricks were salvaged from earlier walls that had been demolished and repurposed. In addition, one type of mortar (type "0") was utilized to represent very ancient unreinforced masonry structures. Professional masons built the wall panels, which had nominal dimensions of 1.2 x 1.2 x 0.25 m, in a controlled setting within Epoka University's civil engineering laboratory.

Wall panels were built. A plain wall, a polypropylene strengthed on one side and the third was a polypropylene strengthed on both sides. Three diagonal compression tests were carried out completely according to the American Society for Testing and Materials' technical requirements (ASTM International). Material properties of masonry component materials were established for each panel prior to testing.

The following conclusions may be formed based on the test results and the numerical analysis results:

1 The findings of the experiments revealed that all of the specimens examined had a similar failure pattern, which was primarily characterized by a step-like fracture along the compressed diagonal, mostly in a diagonal step pattern through the mortar joints However, the strengthening approach had an impact on fracture propagation, maximum deformation, and ultimate load bearing capability of the panels.

2 The wall panels, as predicted, displayed brittle behavior and low shear resistance, with an average shear strength of 0.219 MPa. W3-PP-2s had the maximum shear resistance of 0.376 MPa, which was 2 times greater than W2-PP-1s which was 0.188 MPa.

3 The recommended strengthening strategies have been shown to work.
4 Experimental analysis showed that W3-PP-2s technique provided more satisfactory results in terms of higher resistance and more ductility levels.

Strengthening of URM buildings with polypropylene fibers materials appears to be an attractive alternative for improvement of structural performance against lateral loadings.

# REFERENCES


A. Borri, G. C. (2015). Determination of shear strength of masonry panels through different test. *International Journal of Architectural Heritage vol 9*, 913-927.

A. Brignola, S. F. (2008). Identification of shear parameters of masonry panels through the in situ diagonal compression test. *International Jornal of Architectural Heritage, vol 3* , 52-73.

A. Palomo, M. B.-V.-R. (2004). *Historic Mortars: Characterisation and Durability. New Tendencies for Research.* Prague, Czech Republic.

A.W. Hendry, F. M. (2017). *Masonry Wall Construction.*

Albania Institute of Statistics, I. (Tirane, Albania). *Population and housing, Census 2011*. 2012.

ASTM. (2003). *ASTM C 270-03 Standard Specification for Mortar for Unit Masonry.* West Conshohocken, PA: ASTM International.

ASTM. (2003). *ASTM C 270-03, Standard Specification for Mortar for Unit Masonry.* West Conshohocken, PA: ASTM International.

ASTM. (2004). *ASTM C 62-04 Standard Specification for Building Brick (solid masonry units made from clay or shale)* . West Conshohocken.

ASTM. (2014). *ASTM C67-14, Standard Test methods for Sampling and Testing Brick and Structural Clay Tile* . West Conshohocken.

ASTM International. (2002). *ASTM E519-02, Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages.* West Conshohocken, PA.

ASTM International. (2004). *ASTM C1314-04-Standard Test Method for Compressive Strength of Masonry Prisms.* West Conshohocken, PA.

ASTM International. (2014). *ASTM C67-14, Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile, .* West Conshohocken, PA.

Bakeer, T. (2009). *Collapse analysis of masonry structures under earthquake actions* (8th Edition ed.). Dresden, Germany: TU Dresden.

Bakeer, T. (2009). *Collapse analysis of masonry structures under earthquake actions, 8th edition.* Dresden, Germany: TU Dresden.

Boem, N. G. (2015). Experimental and analytical study to evaluate the effectiveness of an in-plane reinforcementfor masonry walls using GFRP meshes. *Construction and Building Materials vol 88*, 94-104.

Borri, A., Castori, G., & Corradi, M. (2015). Determination of shear strength of masonry panels through different tests. *International Journal of Architectural Heritage, 9*, 913–927.

Borri, A., Castori, G., Corradi, M., & Speranzini, E. (2011). Shear behavior of unreinforced and reinforced masonry panels subjected to in situ diagonal compression tests. *Construction and Building Materials, 25,* 4403–4414.

Brignola, A., Frumento, S., Lagomarsino, S., & Podesta, S. (2008). Identification of shear parameters of masonry panels through the in situ diagonal compression test. *International Journal of Architectural Heritage, 3*, 52–73.

C. Faella, E. M. (2010). Shear capacity of masonry walls externally strengthened by a cement-based composite material: An experimental campaign. *Construction and Building Materials vol 24*, 84-93.

Calderini, C., Cattari, S., & Lagomarsino, S. (2009). In-plane strength of unreinforced masonry piers. *Earthquake Engineering Structure Dynamics, 38*(2), 243–267.

CEN. (2005). *EN 1996-1-1: Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures."* . Brussels, Belgium: European Committee for Standardization.

Corradi, M., Borri, A., & Vignoli, A. (2002). Strengthening techniques tested on masonry structures struck by the Umbria–Marche earthquake of 1997–1998. *Constr Build Mater, 16*(4), 229–239.

D. Dizhur, a. J. (2011). Diagonal tension strength of vintage unreinforced clay brick masonry walls panels . *Construction and Buildings Materials vol 43*, 418-427.

Deodhar, S. (2000). Strength of Brick Masonry Prisms in Compression. *Journal of the Institution of Engineers (India), vol 81, no 3*, 133-137.

Deodhar, S. V. (2000). Strength of Brick Masonry Prisms in Compression. *Journal of the Institution of Engineers (India), 81*(3), 133-137.

Drysdale, R. G., Hamid, A. A., & Baker, L. R. (1994). *Masonry Structures, Behavior and Design.* Englewood Cliff: Prentice Hall Inc.

G. Pande, J. M. (1998). *Computer methods in structural masonry.* London: E & FN Spon .

Gumaste, K. S., Nanjunda Rao, K. S., Venkatarama Reddy, B. V., & Jagadish, K. S. (2006). Strength and elasticity of brick masonry prisms and wallettes under compression. *Materials and Structures, 40*(2), 241-253.

H.B. Kaushik, D. R. (2007). Uniaxial compressive stresss-strain model for clay brick masonry. *Current Science, vol 92, no 4*.

Haluk Sucuoglu and Hugh D. McNiven. (1991). Seismic Shear Capacity of Reinforced Masonry Piers. *Journal of Structural Engineering*.

Hendry, A., Sinha, B., & Davies, S. (1997). *Design of Masonry Structures, Load Bearing Brickwork Design,* (Third Edition ed.). UK: E&FN Spoon, UK.

Hilsdorf, H. (1967). Investigating Into the Failure Mechanism of Brick Masonry Loaded in Axial Compression. *International Conference of Masonry Structural Systems.* Texas: University of Texas.

International, A. (2013). *ASTM C1006-07-Standard Test Methods for Splitting Tensile Strength of Masonry Units.* West Conshohocken.

J. Milosevic, A. G. (2013). Experimental assessment of shear strength parameters on rubble stone masonry specimens. *Construction and Building Materials, vol 47*, 1372-1380.

Jianguo, C. X. (1987). Shear Strength of Brick Masonry with Reinforced Network in . *Journal of Hunan University*.

Jingqian, Z. B. (1991). Experimental Study on Seismic Behaviour of Brick Masonry . *Journal of Building Structures*.

K. Elert, G. C. (2003). Durability of bricks used in the conservation of historical buildings- influence of composition and microstructure. *Journal of Cultural Heritage, vol 4, no 2* , 91-99.

K. Elert, G. C. (2003). Durability of bricks used in the conversation of historic buildings- influence of composition and microstructure. *Journal of Cultural Heritage, vol 4*, 91-99.

Kadiu, F. (2007). *Shkenca dhe Teknologjia e Materialeve (Albania).* Tirane, Albania: Erik Botime.

Kalali, A., & Kabir, M. Z. (2012). Experimental response of double-wythe masonry panels strengthened with glass fiber reinforced polymers subjected to diagonal compression tests. *Engineering Structures, 39*, 24-37.

Karaman, S., Gunal, H., & Ersahin, S. (2006). Assesment of clay bricks compressive strength using quantitative values of colour components. *Construction and Building Materials, 20*(5), 348-354.

Kaushik, H. B., Rai, D. C., & Jain, S. K. (2007). Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression. *Journal of Materials in Civil Engineering, 19*(9), 728-738.

Kaushik, H. B., Rai, D. C., & Jain, S. K. (2007). Uniaxial compressive stress-strain model for clay brick masonry. *Current Science, 92*(4), 497-501.

Khalaf, A. H. (2001). *Masonry wall construction.* London: Spon Press.

Krawlinker, H. (1979). Possibilities and limitations of scale-model testing in earthquake engineering. *Proceedings of the second US national conference on earthquake engineering*, (pp. 283-292). Stanford, California.

Lourenco, P. (1998). Experimental and numerical issues in the modelling of the machanical behavior of masonry. *Structural Analysis of Historical Constructions 2*.

Lourenço, P. B. (1996). *Computational strategies for masonry structures.* Delft, Netherlands: Delft University of Technology.

Lumantama, R. (2012). *Material characterization of New Zealand clay bricks unreinforced masonry buildings.* Auckland.

Lumantarna, R. (2012). *Material characterization of New Zealand clay brick unreinforced masonry buildings.* Auckland, New Zealand: University of Auckland.

Magenes, G., & Calvi, M. G. (1997). In-plane seismic response of brick masonry walls. *Earthquake Engineering Structure Dynamics*, *26*(11), 1091–1112.

Mann, W., & Müller, H. (1973). Failure criteria for laterally loaded masonry and their application to shear walls (in German). *Die Bautechnik*, *50*(12), 421–425.

Mayes, R., & Clough, R. W. (1975). State-of-the-art in seismic shear strength of masonry-an evaluation and review. *EERC 75-21.* College of Engineering, University of California. .

McKay, W. B. (1947). London, United Kingdom: Green and Co.

Meyer, U. (2004 ). IN PLANE SHEAR STRENGTH OF VERTICALLY. *13th International Brick and.* Amsterdam.

Montes, P. F. (2001). Behaviour of a hemispherical dome subjected to wind loading. *Journal of Wind Engineering and Industrial Aerodynamics, 89,* 911-924.

Mulligan, J. (1942). *Handbook of brick masonry construction.* New York, United States: McGraw-Hill.

Mustafaraj, E. (2016, June). External shear strengthening of unreinforced damaged masonry walls. Tirana: Epoka University.

Mustafaraj, E. (2016). *External Shear Strengthening of Unreinforced Damaged Masonry Walls.* Tirana.

N. Ismail, R. P. (2011). Diagonal shear behaviour of unreinforced masonry walletes strengthened using twisted steel bars. *Construstion and Building Materials, vol 25*, 4386-4393.

P.B.Shing, J. (1989). Inelastic Behavior of Concrete Masonry Shear Walls. *Journal of Structural Engineering*.

Page, A. W. (1982). An experimental investigation of the biaxial strength of brick masonry. *6th International Brick/Block Masonry Conference*, (pp. 3–15). Rome, Italy.

Pande, G., Middleton, J., & Krajl, B. (1998). *Computer Methods in structural masonry.* London, UK: E & FN Spon.

Park, R. (1986). Ductile Design Approach for Reinforced Concrete Frames.

Paulay, T., & Priestley, M. J. (1992). *Seismic design of reinforced concrete and masonry buildings.* New York, USA: John Wiley & Sons, .

Priestly, T. P. (1992). *Seismic Design of Reinforced Concrete and Masonry Buildings.* John Wiley & Sons, Inc.

Priestly, T. P. (John Wiley & Sons ). *Seismic design of reinforced concrete and masonry buildings* . New York, USA: 1992.

Prietsly. (1976). *Design Expression for the In-Plane Shear Strength.*

R.G. Drysdale, A. H. (1994). *Masonry Structures, Behavior and Design.* Englewood Cliff: Prentice Hall Inc.

S. Karaman, H. G. (2006). Assesment of clay bricks compressive strength using quantitative values of color components. *Construction and Building Materials , vol 20,* 348-354.

Sahlin, S. (1971). *Structural Masonry.* Englewood Cliffs, New Jersey: Prentice-Hall Inc.

Sahlin, S. (1971). *Structural Masonry.* Englewood Cliffs, New Jersey.

Sneck, T. (n.d.). Dependence of masonry properties on the interaction between masonry units and mortar. *Proceedings of the North American masonry Conference.* Colorado: 1978.

*The Common Unified Standard for Building Structures Design GBJ68 - 84.* (1984). Building Industry .

*The Design Code of Masonry Structures GBJ3 - 88.* (1998). Beijing: Building Industry Publishing House of China.

Tomazevic, M. (1999). Earthquake-Resistant Design of Masonry Buildings. In *Series on Innovation in Structures and Construction, Masonry Materials and Construction Systems* (Vol. 1). Imperial College Press.

Tomaževič, M. (1999). *Earthquake-resistant Design of Masonry Buildings.* Imperial College Press.

Triantafillou, T. C. (1998). Strengthening of masonry structures using epoxy-bonded FRP laminates. *ASCE Journal of Composites for Construction, 2,* 96-104.

Turnašek, V., & Cacovic, F. (1971). Some experimental results on the strength of brick masonry walls. *2nd international brick masonry conference*, (pp. 149–156). Stoke-on-Trent, UK.

Yokel, F. Y., & Fattal, S. G. (1976). Failure hypothesis for masonry shear walls. *Journal of Structural Division, 102*(3), 515–532.