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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Mechanistic Behavior of Historical Masonry Assemblages of Fasil Ghenbe and Structural Response Assessment of Mentwab Castle Under Vertical Loading; A Case Study

A Thesis in structural Engineering

By Kidist Dereje

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The undersigned have examined the thesis entitled '**Mechanistic Behavior of Historical Masonry Assemblages of Fasil Ghenbe and Structural Response Assessment of Mentwab Castle Under Vertical Loading; A Case Study**' presented by **Kidist Dereje**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

For a country like Ethiopia, one of the pivotal issue, from economic as well as cultural point of view, is a conservation of heritage buildings. In that regard, understanding the structural behavior of historical structures is immensely beneficial to design and implement effective conservation and rehabilitation schemes. A comprehensive structural behavior assessment requires for a knowledge of the material properties and the use of advanced numerical tools. This study presents the experimental investigations carried out to characterize the mechanistic properties, compression and shear, of the historical masonry assemblages of Fasil Ghenbe with a FE analysis of Mentwab castle under vertical loading.

Replicas of historical masonry wall units were built in Gondar, by a well experienced masons and by the use of representative construction materials. The construction materials used for the masonry wall units were basaltic stone, slacked lime aged for six years and aggregates extracted from traditionally made quick lime. After eight months of their construction the mockups were tested for shear and compression behaviors. Additionally, index tests and chemical tests were conducted to characterize the lime mortar.

For the shear test five masonry mockups were tested on a set-up arranged to conduct a triplet shear test. The tests were executed under three different levels of confinement. For each test the confinement level or the state of stress (0.1MPa, 0.3MPa, and 0.5MPa) was kept uniform. As for the compression test, two specimens were tested. In both cases, load versus displacement behaviors were acquired from the instrumentations used.

The mechanistic nature of the masonry assemblages is characterized for compression and shear behaviors. Simple model using the Coulomb friction model is proposed for the shear behavior, using coefficient of friction and cohesion parameters. The compression behavior is mainly characterized by the peak strength and elastic modulus parameters. The results, only available experimental data, can serve for the structural analysis of the historical masonry structures of Gondar, under static loading.

The preliminary structural analysis conducted on Mentwab castle resulted in the static structural response of the castle under vertical loads only, own weight and loads from the roof/floor slabs. For this particular loading condition, the resulted stress levels were below the capacity of the masonry. Above all the numerical analysis is a breakthrough in the investigation of the historical masonry structures, on a finite element model based study.

1. INTRODUCTION

1.1 General Background

"Cultural heritages are among the priceless and irreplaceable possessions, not only of each nation but of mankind as a whole. The loss, through deterioration or disappearance of any of these most prized possessions, constitutes an impoverishment of the heritage of all the peoples in the world" (UNESCO 2002). So, appropriate conservation and restoration activities are unquestionably needed to protect such kind of precious belongings. To this goal any plan of restoration, rehabilitation and strengthening should be preceded by a comprehensive condition assessment.

Assessments of heritage structures are highly demanding and multidisciplinary tasks which involve laboratory/ in-situ tests, inspection, historical survey and structural analysis as some of the tools to comprehend the condition of the heritages. This requirement is because of the numerous changes in the life of the structure due to alteration, deterioration, misuse, damage and other changes to its as-built state.

In line with the above statement (ISCARSAH 2003) stated that studies on historical structure should integrate activities involving detailed historical investigation, deep inspection by means of non-destructive techniques and monitoring. In addition, assessment should be based on characterization of structural materials, actions, geometric data and structural behavior.

In this regard, the UNESCO World Heritage site in Ethiopia, particularly the historical masonry buildings of Gondar need a thorough investigation. Some of the historical masonry buildings are being threatened by structural problems; such as cracks and out-of-plane movement of walls. Consequently, around some of the buildings huge steel structures have been erected to protect the heritage from impending failure. However, for some, the effectiveness of these steel structures are highly doubtful.

Furthermore, there were situations where churches which have the same masonry fabric and old ages as that of the historical buildings, needed assessment on the load capacity of the existing masonry walls. In other words, the load-bearing capacity of such kind of structures, wall systems have been questioned whenever the users required to maintain or change the roof system and when there was a need to reconstruct the damaged parts of the churches.

As stated earlier, scientific knowledge of the condition of historical buildings is very crucial to properly intervene to the structures and to keep their great historical, social and economic values. Thus, with the intent to understand the structural behavior of the historical masonry buildings of Gondar. In the study, the mechanical properties of the historical masonry specimens, compression and shear properties, were characterized by experimental investigation. In addition, numerical models were used to examine the static structural responses of a masonry made heritage building.

1.2 Statement of the Problem

Some of the historical masonry buildings in Fasil compound have faced some forms of structural damages such as cracks and tilting or out-of-plane movement on the walls. Consequently, a number of resealing of cracks have occurred at different times. In addition, steel structures which are uncharacteristic of the heritage, have been constructed to protect the parts of the buildings which are thought to be liable to damage.

At this point, it must be emphasized that any intervention must be preceded by a detailed investigation of the buildings where the construction materials are properly characterized and structural behaviors of the buildings are assessed with the help of structural analysis. In this regard, the structural behavior of the historical masonry buildings of Gondar has not been researched thoroughly. Particularly, to the knowledge of the researcher, no attempt has been made in terms of characterizing the mechanistic behavior of the masonry, for the purpose of structural analysis.

1.3 Objectives

1.3.1 General Objective

The main aim of the research is to study structural aspects of historical masonry buildings of one of the World Heritage site, Gondar Fasildesis.

1.3.2 Specific Objectives

- To characterize the mechanical properties, compressive and shear behavior, of historical masonry assemblages of Fasil Ghenbe
- To model one historical masonry building from Fasil Ghebbi (compound) and to investigate its global response under vertical loading

1.4 Scope of the study

The material property of the structural wall system of the heritage buildings of Fasil Ghenbe was examined by reproducing the replica of the traditional lime-based masonry. The specimens were characterized under a monotonic loading condition, only for compression and shear properties.

In addition, the structural analysis conducted on one historical masonry building was carried out under a vertical load only, self-weight plus transferred loads from roof and floor slabs. Where the foundation conditions or other aspects related to soil-structure interaction were not taken into account. That means the building was considered to be rigidly fixed at the base.

1.5 Significance of the Study

The results of the study will give some of the engineering properties of the traditional masonry used in Fasil Ghebbi and these results will be a key input to make an in-depth investigation on the buildings of the imperial castle.

As for the numerical model of the sample building, results will benefit conservators to understand the structural behavior of the building, Mentwab Castle. And it will also be helpful to comprehend causes of structural damages or early detect conditions that might threaten the safety of the structure. In doing so, the findings of this research will contribute to design effective and efficient conservation and rehabilitation techniques, for ensuring the longevity of the heritage.

1.6 Methodology

In this subsection, the methods used to realize the objectives of the study are summarized. The study was carried out based on the recommendations given by International Scientific Committee for Analysis and Restoration of Architectural Heritage. According to (ISCARSAH 2003), studies on historical structures require both qualitative and quantitative data that are obtained from direct observation, historical research, experimental tests and mathematical models.

Accordingly, the research was conducted on historical masonry building that exists in Fasil compound. Where the data collection was followed by a scientific analysis and numerical modeling, which is used to understand the structural behavior and condition of the structure. The following flowchart summarizes the methods or activities that were used to fulfill the objectives of the study.

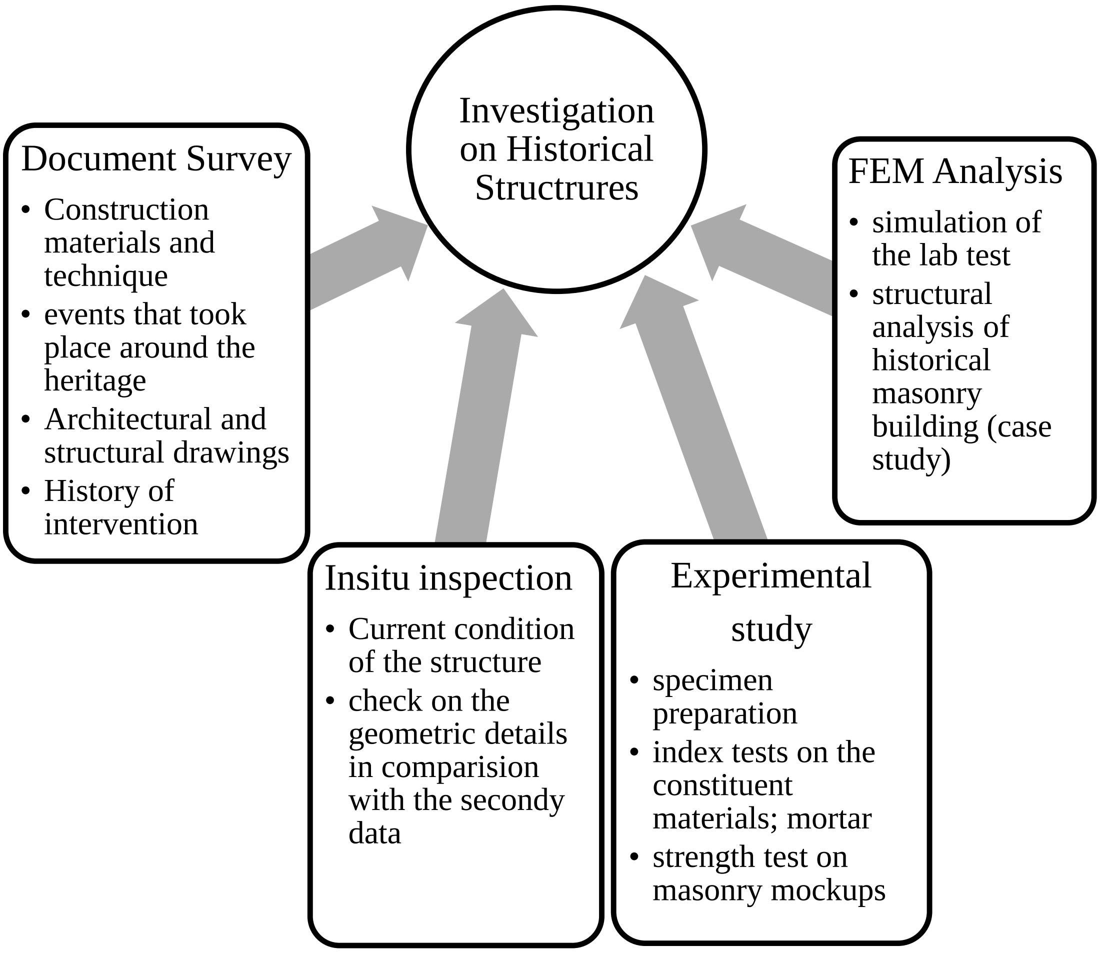

Figure1.1 Summary of the methodology

2. LITERATURE REVIEW

2.1 Looking into Buildings with History

The complex nature of historical constructions requires a scientific approach to understand their behavior and response to external actions. In this regard, there are different methods that are used to assess the condition of historical structures. These techniques are scientific methods that are used to examine their safety, distinguish between stable and progressive changes on the structure, patterns of damage (if there is any) and damage evolution leading to a catastrophic structural failure (Carpinteri, Invernizzi and Lacidogna 2005).

Among the tools which are used to examine a condition of existing heritage structures; history, in-situ inspection, monitoring (dynamic and static), laboratory as well as in-situ testing and structural analysis are the basic ones (L.Binda and A. n.d.) & (Rivas 2009). In addition to being a means to assess the condition of structures, these procedures are a prerequisite to any effective conservation and restoration activities that are held on heritage structures (Grecchi 2012).

In principle conservation and restoration of historic buildings should be preceded by a full characterization of the construction materials used on the given structure. (Milosevic, et al. 2013), reinforced this idea by stating that a detailed understanding of the structural behavior and material characteristics are essential for appropriate restoration of heritage buildings. Thus, to this goal, investigation of material properties of such constructions is handled through visual inspection, a sampling of the construction materials and laboratory testing of the samples.

(Silva, et al. 2012), (Milosevic, et al. 2013) and (Betti, Galano and Vignoli 2014) have built different types of masonry mockups and studied various mechanical property parameters of historical masonry constructions. The main concern in the characterization of the material properties is the difficulty to conduct all the necessary tests in situ and to remove samples from heritage buildings. Therefore, in order to be able to understand the material properties of such structures, laboratory mockups have been built using the similar (traditional) material, and construction techniques.

The other important tool to in researching historical structures is a structural analysis. In combination with the other tools, structural analysis is essential to simulate the performance of structures when subjected to past actions and to realize their present condition, existing damage and the likes (Roca, et al. 2010). Several studies on historic structures have demonstrated that advanced numerical analyses can offer significant information to understand structural behavior, damage and failure mechanisms of such constructions.

2.2 Numerical Modelling and Analysis of Masonry Structures

The assessment of existing buildings, even under the hypothesis of full knowledge of current conditions and materials, is not an easy and straightforward task (A.Costa, Quelhas and P. Almeida 2014). Moreover, the analysis will be more difficult when the structure is historical masonry construction. This difficulty is accredited to the presence of joints as the major source of weakness, discontinuity and nonlinearity as well as the existence of uncertainties in the material and geometric properties (G.Asteris, et al. 2015).

When it comes to the historical masonry constructions of Gondar, besides the above-stated factors the difficulties related to structural behavior assessment are rooted in, not knowing the mechanistic properties of the construction material as well as the lack of effort that was made to investigate the masonry walls by the use of numerical models.

Globally, these days a considerable research effort has been made to adequately characterize the behavior of masonry using numerical analysis, finite element methods. And as a result, numerical simulations capable of predicting the behavior of the structure from the linear stage, through cracking and degradation until complete loss of strength can be performed using different methods of modeling and analysis (B.Lourenco, G.Rots and Blaauwendraad 1995). Specifically, for masonry structures, distinct methods with different level of complexity and different application are available for modelling and analyses.

As a result of the complex typological characteristics of masonry, the numerical modelling of masonry structures through the FEM is computationally demanding task (Giordano, Mele and De Luca 2002). Despite the complexity, numerical idealization of masonry can be made with varying levels of details. Basically, the modeling strategies can be divided into two distinct categories; macro-modeling and micro-modeling.

2.2.1 Macro and Micro Modelling

In macro-modeling, a masonry element considers the effects of mortar joints implicitly using a continuum homogenized model and the properties of homogenized masonry assemblage can be obtained either from experimental tests or by using homogenization techniques. On the contrary, micro-modeling consider masonry as heterogeneous material and masonry elements are represented explicitly with a separate description of each constituent material (G.Asteris, et al. 2015).

According to (P.B.Lourenco 1996) micro-modelling is further classified into detailed and simplified micro-modeling. Detailed micro-modeling represent units and mortar in the joints by continuum elements where the unit-mortar interface is represented by discontinuous elements. The interface represents a potential crack/slip plane with initial dummy stiffness to avoid inter- penetration of the continuum.

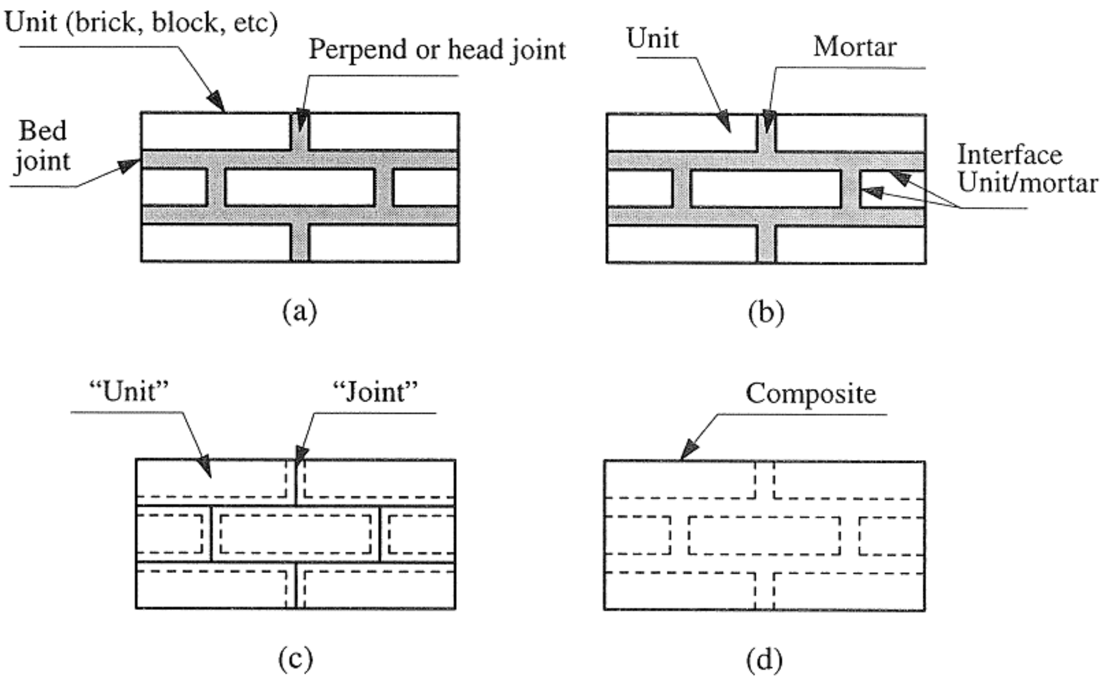

Figure 2.1 Modeling strategies for masonry structures: (a) masonry sample; (b) detailed micro-modeling; (c) simplified micro-modeling; (d) macro-modeling (P.B.Lourenco 1996)

In the case of simplified micro-modelling, a geometric expansion of the units represented by continuum elements separated by discontinuous elements that simulate the behavior of the mortar joints and unit-mortar interface.

Regarding their application, macro-modeling removes the difficulty in describing units and mortar joints separately. It is more practice-oriented due to the reduced time and computer memory requirements as well as user-friendly mesh generation. This type of modeling is valuable when a compromise between accuracy and efficiency is needed (B.Lourenco, G.Rots and Blaauwendraad 1995). In contrast, micro-modelling approach, detailed or simplified, is computationally demanding when it comes to the application for large structures. It is usually employed to analyze specific problems of little-size under a magnifying glass.

In relation to the analysis of large-size masonry constructions macro-modeling approach is more likely appropriate (Teomete and Aktas 2010). This approch have been used by a number of researchers to investigate several issues related to historical masonry constructions.

As an example; (Silva, et al. 2012) studied the behavior of an old stone masonry construction, the Gondar church (Portugal), and analyzed the seismic response of the building. (Bartoli, Betti and Borri 2015), assessed static and dynamic behavior of the Brunelleschi’s Dome of Santa Maria del Fiore in Florence, the UNESCO World Heritage site. And the results were found to be helpful to evaluate the dome’s internal stress and cracking pattern. (Betti and Virgnoli 2011) has also examined the static and seismic behavior of the Basilica of Santa Maria all’Impruneta (Italy) by using macro-modelling technique.

In general, each of the above modeling strategies has their own advantage and difficulties. At this point, it should be noted that a complex analysis is not always synonym of a better result and the choice of a method over another must depend mostly on the purpose of the analysis (Silva, et al. 2012). Therefore, the choice among these modeling approaches is made depending on the complexity of the problem and the results expected from the analysis (Soveja, Budescu and Gosav 2013).

2.2.2 Structural Analysis of Historical Masonry Structures

These are different methods and computational tools that are based on different theories and strategies, resulting in different levels of complexity, different calculation times and costs (A.Costa, Quelhas and P. Almeida 2014). In this regard, structural analysis of historical masonry structures could be conducted by linear elastic, plastic or nonlinear analysis.

Linear elastic analysis has been taken as an effective means of identifying the global tendency of building behavior, modal characteristics and areas in which the structure is subjected to stress concentrations that are able to interrupt the continuity of the masonry. However, this method of analysis is considered to be inaccurate even at low levels of loads, this is due to the complexity of historical masonry structures. Despite this limitation of the method, (Mustafaraj and Yardim n.d.) (Gosav and Soveja 2014) , have used linear analyses to simulate the structural behavior of masonry buildings of high cultural value.

Plastic analysis or limit analysis is used to assess the maximum load that a structure can sustain (load limit). As for nonlinear analysis, it is the most appropriate and recommended method for structural analysis of historical masonry constructions. This is due to the fact that, nonlinear analysis can capture the full response of the structure through the elastic stage, cracking and dislocation, up to a complete collapse. The definition of elastic and inelastic mechanical properties of masonry is required to this analysis method (Soveja, Budescu and Gosav 2013).

2.2.3 Material Models or Constitutive Models for masonry

In masonry constructions, a nonlinear material behaviour is highly evident which is characterized by a small tensile capacity and relatively large compressive strength (Atamturktur, Prabhu and Roche 2014). Since numerical models that are used to assess the existing condition of ancient masonry structures need to be reliable to decide on the necessity for strengthening or other interventions, much care should be given to the material characterization. And this can only be achieved if advanced solution procedures for the system of equations that result from the FEM discretization are supplemented by "accurate" and robust constitutive models (B.Lourenco, G.Rots and Blaauwendraad 1995).

Constitutive models are the mathematical simplification of the complex behaviors of a material. At this point, "It must be emphasized that there is no such thing as an 'exact' model for a particular material, except for the linear phenomena. And there is no a unique way to build a constitutive model for a certain behavior" (Basan and Marohnic 2016). Thus, several constitutive models can be formulated and used for historical masonry. For instance, this can be done by assuming linear elastic, elastic perfectly plastic or other plasticity models.

In conclusion, the selection of mateial models for a given sturctural analysis could be made based on the capability of the model to simulate a state of a material under which the analysis is required to be conducted. That means the state of stress which is needed to be considered, either elastic or inelastic state of the material. Or else the model could be made to reasonably represent a given experimental test result.

2.3 Studies on Historical Masonry Buildings of Gondar

Scientific studies or researches are useful in order to be capable of counteracting the dangers that threaten cultural heritages (ISCARSAH 2003). In this regard, several researchers in different times, from different countries, had come to Ethiopia and conducted research on the historical masonry buildings of Gondar.

The efforts that have been made by those researchers was mainly to investigate the physical and chemical properties of the construction materials, (Desta 2015). Extensive investigations have been conducted by a team of researchers where they have studied the historical buildings in relation to undertaken interventions, material and structural damages (ECHP-A1 2007) and (ECHP-A5 2007). The heritage buildings were also studied from an architecture point of view, (Angelini 1971). In addition, in the course of data collection, some design documents related to the reconstruction of some section of roofs and staircases of the heritage buildings were found (Getahun 2000).

In fact, several types of research have been conducted on the historical buildings. However, to the best of the knowledge that can be acquired, none of the above studies and documentation have covered the topic related with structural analysis of masonry walls of the historical buildings and mechanical properties of the masonry assemblage which is the main building block of the structures. As a matter of fact, certain structural measures or reinforcing activities have been taken on the structures to counteract against the apparent cracks and out-of-plane movements of walls. Here it is important to note that if advanced numerical models of the historical buildings were there, they would be highly advantageous for any planning and implementation of strengthening, rehabilitation activities or interventions.

Prior to the numerical models of the walls or structural analysis of the historical masonry structures, characterization of the mechanical properties of the masonry is necessary. Therefore, this research was intended to systematically address these gaps, mechanical property characterization and numerical analysis of historical masonry constructions.

3. EXPERIMENTAL STUDY

3.1 Introduction

Laboratory and in-situ tests provide essential data to characterize the material properties of the masonry used in the construction of the historical buildings. Even though sometimes they are costly and time-consuming, they represent the only means to obtain reliable data for structural evaluation and subsequent studies. Moreover, these tests allow improving and adjusting numerical tools and models to the structural analysis of this type of constructions.

The experimental study that was intended to understand the material property of the historical masonry of the Royal Enclosure mainly include compressive strength and triplet shear tests. The compressive strength test was undertaken to know the compressive strength, elastic modulus and to infer the effect of tensile stress on the masonry. And the triplet test was carried out to evaluate the cohesion and angle of internal friction. Additionally, some index tests were conducted on the constituent materials, aggregates of the mortar and the mortar itself.

In order to carry out those tests, mockups were built following the traditional construction mechanism. As much as possible the constituent materials were made to be representative of the historic masonry. In the next subsections the details of the sample preparation and index tests conducted on the constituent materials of the mortar are discussed.

3.2 Specimen Fabrication

The constituent materials that were used to build the mockups are supposed to be representative of the historic masonry. To this goal, the basaltic stone was collected from ruins of buildings that exist in Fasil compound. Plus, the lime mortar was prepared using lime putty and two types of heat treated limestone aggregates that were sieved out from the quicklime in time of slaking. Similarly, both the lime putty and the aggregates were obtained from Fasil Ghebbi.

At this point, it is important to note that traditionally slaked lime, aged for six years was used in the specimen preparation. Aging is a long-term storage of slaked lime under water. The aging process results in a substantial improvement in mortar quality. This is due to the significant morphologic changes, from prisms to plate-like crystals and particle-size reduction of $Ca(OH)_2$. These changes result in increased mortar paste plasticity, workability, water retention with a fast and high degree of carbonation (Cazalla, et al. 2000).

The masonry sample preparation was made according to the traditional practice. In other words, the materials and method of construction used to build the mockups were in a way that compiles to the original construction materials and techniques used to build the historical buildings. To ensure the compliance, the local people in the castle were fully involved in the production of the mockups.

3.2.1 Mortar preparation

The first step in the preparation of the mortar was to unearth the slaked lime from the open pit and collect the required amount in a barrel then it was gently agitated for five days consecutively, without the addition of extra water. This was done in order to break down the mud like hydrated lime and to bring it into being a uniformly mixed lime putty, Figure 3.1.

Figure 3.1 a) Hydrated lime unearthed from the slaking pit b) Hydrated lime agitated to form a lime putty

Then in order to form workable and appropriate mortar for masonry the lime putty was mixed with the aggregates. The aggregates are of two type: the first one, labeled as AG-1, was the finer aggregate which was directly collected from the sieve, retained while the washing process of the quick lime was conducted during slaking. The second one, AG-2, was obtained by crushing the larger sized aggregate that was again retained on the sieve during the process of slacking.

As these aggregates were obtained by separating the very fine and properly burned limestone, quicklime, from that of the unburned limestone through wet sieving. These aggregates are different from ordinary crushed limestone because they have passed through burning at a high temperature.

It should be noted that the production of the quicklime is not in a factory scale but simply by burning in a traditionally made kiln. As a result, in the production of quicklime, unburned limestone is also obtained with the ash, quicklime. The limestone, which is not fully changed into quicklime is the one that is being used as an aggregate, Figure 3.2.

Figure 3.2 Aggregate AG-1 and AG-2

As the mortar preparation is in accordance with the traditional method there is no clearly defined mix ratio of the ingredients. Nevertheless, the experienced masons can tell, the appropriateness of the mix proportion for the intended use. Accordingly, the lime putty was mixed with the aggregates and the lime mortar was prepared, see Figure 3.3.

Figure 3.3 Mixing of aggregate with the lime putty and prepared mortar

3.2.2 Mockups

After the preparation of the mortar, twelve samples having a size of 30x30x30 cm^3 were built using the same mortar and basaltic stone units, collected from the ruins that exist inside Fasil Ghebbi. The samples were constructed into two separate groups: one for compressive test and the other for triplet shear testing, the two groups are labeled as shown in Table 3.1.

Specimens for the compressive test were made using stone units with variable shape and dimensions, which were randomly assembled. The biggest stones, with the longest edge of about 15-18 cm, were used in corners and edges and the spaces among them were filled with a mortar and small pieces of stone.

Table 3.1 Masonry specimens

Test	Samples
Compressive strength	C1, C2, C3, C4, C5, C6
Triplet shear	TS1, TS2,TS3, TS4, TS5,TS6

Similarly, specimens for triplet tests were also built with stone units with irregular shape and dimensions but special attention was paid in order to create three horizontal layers of mortar and stone assemblages (each 10 cm height) with two nearly horizontal bed joints Figure 3.4.

Figure 3.4 Masonry specimen construction

In average the thickness of the stone units of all the samples is in the range of 4-8 cm. As compared to the cross-sectional dimensions the thicknesses of the stone units are smaller, thus, it can be said that the stone units are flat. On the other hand, the thickness of the mortar in each layer is in the range of 3-4 cm. Given the size of the masonry assemblages, it can be safely assumed that the size of the mockups is representative, for the purpose of structural analysis.

For the first 27 days, starting from the day after their construction, the samples were being cured by sprinkling water twice a day and after the 27^{th} day, they were left to harden inside a room where there was a good ventilation, Figure 3.5. This curing technique was adopted from the traditional practice.

Figure 3.5 Masonry specimens during the curing period

The specimens were tested 8 months after their construction to ensure the mortar's hardness. Hence, the mortar was prepared by an aged slaked lime, of six years, the carbonation is expected to be relatively faster. Thus, eight months is thought to be a reasonable and sufficient period to test the samples. similarly, (Milosevic, et al. 2013) tested their masonry wall samples built by lime mortar after eight months.

3.2.3 Laboratory samples

In addition to the lime-based masonry samples, two brick masonry specimens bonded by cement mortar were built. These were fabricated to be used as a trial sample during the arrangement of the triplet shear test setup. In the preparation of the trial samples, the mortar mix was 1-part cement to 3 parts sand, by volume. And the samples were cured under damp hessian, for seven days.

Although these samples were prepared for the trial purpose, the results obtained at the end of the trial tests were found to be informative to see the difference between the responses of the two masonry types, lime-based masonry, and brick masonry samples.

3.3 Index In-situ and Laboratory Tests

As stated above, the mortar preparation was in accordance with the traditional practice, where there are no clearly defined mix ratio or proportioning of the constituent materials. Consequently, only the experienced masons can tell the appropriateness of the mix for the intended use, either for masonry construction or plastering purpose.

In the study, the attempt was made to quantify the mix proportion of the constituent materials, the bulk density, and water content. There was also an effort to measure the compressive and flexural strength of the mortar. Moreover, its workability was assessed by using a method that is normally used to measure the workability of concrete. Additionally, the moisture content and particle size distribution of the aggregates was evaluated.

3.3.1 Tests on Aggregates

3.3.1.1 Gradation, and Moisture Content

The particle size distribution and moisture content of the two aggregates that were used for the preparation of the mortar were evaluated in the construction materials testing laboratory of the University of Gondar.

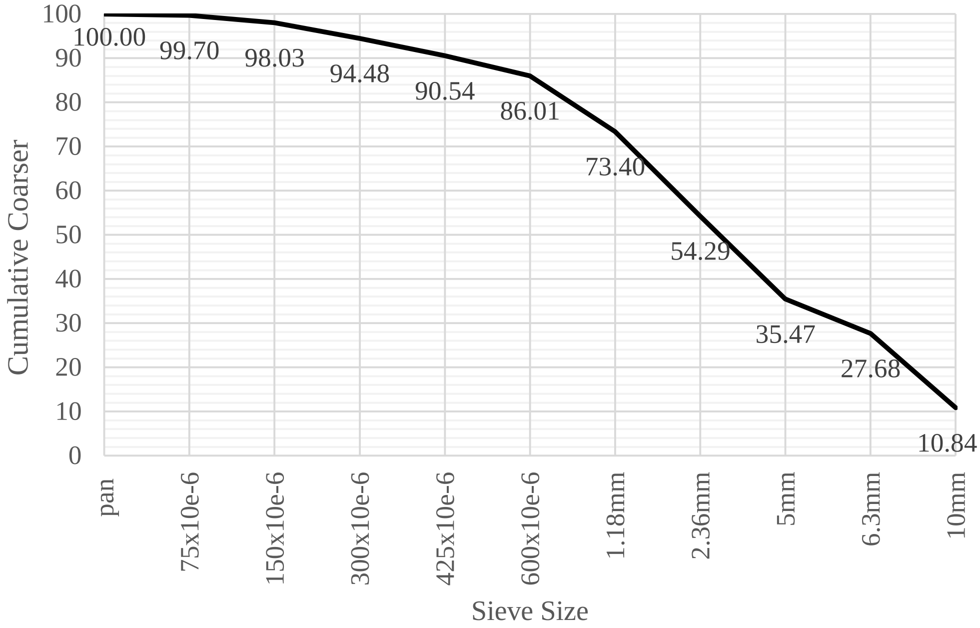

Figure 3.6. Gradation of the Fine Aggregate AG1

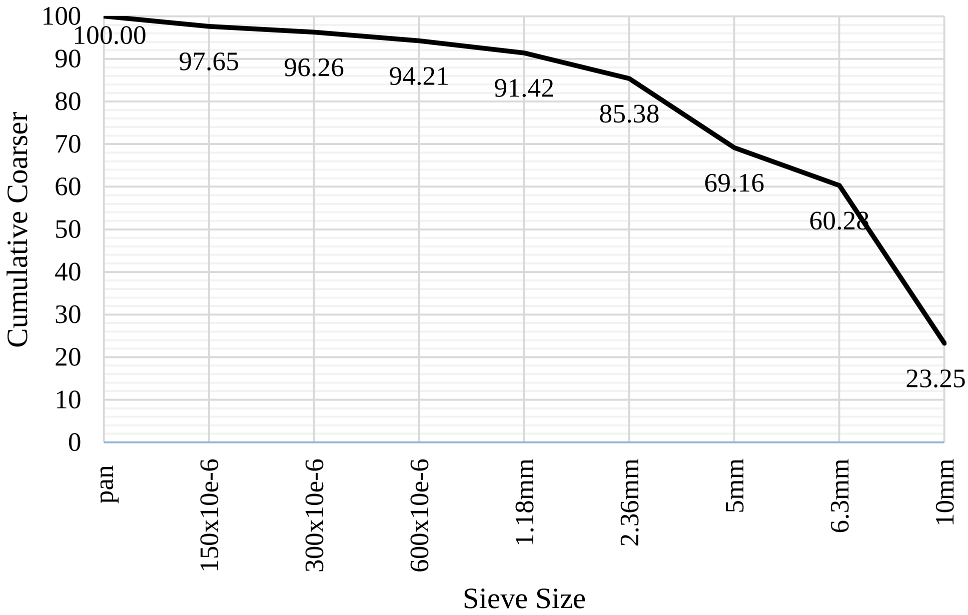

Figure 3.7 Gradation of the crushed Aggregate AG2

The moisture content of the aggregates was found to be 35.8 and 12.09 percent for fine (AG-1) and crushed (AG-2) aggregates respectively. The particle size distribution of the aggregates is presented in the charts below see Figure 3.6 and Figure 3.7.

3.3.2 Tests on the Mortar

3.3.2.1 Mix Ratio, Water Content, and Bulk Density

In the study, with an attempt to quantify the proportion of each constituent materials, a volumetric measure of aggregates and percentage of moisture content of the mix was measured. The measurement of the two aggregates and lime putty was made by the use of 20lt and 10lt buckets respectively, see Figure 3.3. Accordingly, the mix proportion of aggregate AG1 to AG2 to lime putty was found to be $1\frac{1}{2} : 1 : 1\frac{1}{4}$.

From the fresh mortar, samples were taken to determine the bulk density and water content of the mixture. As a result, the bulk density of the mortar was measured to be 1.78gm/ml, Figure 3.8.

Concerning the water content of the mortar, it was assessed by taking three samples immediately after the mortar was mixed. The three samples were collected in a plastic bag and then their initial weight and again weight after oven drying was measured. Finally, the mean value of the water content of the three samples was obtained, 25.9%.

Figure 3.8. Bulk density sample

3.3.2.2 Workable Life of Fresh Mortar

Even though the workable life of the mortar was not measured using the standard test procedure, it was observed that the mortar which was mixed once, was found to be in its plastic state for more than five hours. That means there was no any noticed problem of stiffening, which is the gradual change from fresh or plastic mortar to setting or set mortar.

In fact, during the sample preparation, the mortar was not exposed to sun and the air condition was not very hot and windy, so excessive evaporation would not be a concern. However, as it was learned from the mason, if the lime mortar is excessively exposed to sun and become stiff, one can easily regain the required plasticity simply by sprinkling water and remixing the mortar.

3.3.2.3 Consistency or Workability of the Mortar

Since the workability of the mix is the criteria that the traditional masons use as they proportion the fine and coarse aggregates with the lime putty, it was found to be valuable to understand the workability of the mix in a measurable way.

Thus, in the course of the fabrication of the masonry mockups, slump test was conducted on the lime mortar, Figure 3.9.

Figure 3.9. Slump Test

This test was chosen because it was impossible to conduct standard tests for consistency and workability of mortar. However, for some reasons the slump test was found to be reasonable to comprehend the workability of the mortar. One is due to the fact that the lime mortar is sometimes called lime concrete by the practitioners (Getahun 2000). And the other is due to the large sized aggregates (AG2) used in the mix.

Therefore, the mortar that was regarded as an appropriate mix by the traditional masons and which used to build the masonry specimens was tested. As a result, an average slump value of 4.4cm was found.

3.3.3 Flexural and Compressive Strength Test on Mortar

Mortar specimens of size 160 x 40 x 40 mm^3 for flexure test and 80 x 40 x 40 mm^3 for the compressive strength test were prepared to study the strength of the mortar. These sizes are according to the EN 1015-11 standard to evaluate mechanical properties of a mortar, flexural and compressive strength.

To this goal, wooden molds having the above sizes were made and used to cast the mortars during the fieldwork, Figure 3.10. Then the samples were brought to AAiT material testing lab. Unfortunately, most of the mortar samples were broken in an attempt to remove them from the wooden molds. Thus, the proposed strength tests were not conducted on the mortar. However, the broken samples were used for another purpose, to investigate the carbonation of the mortar.

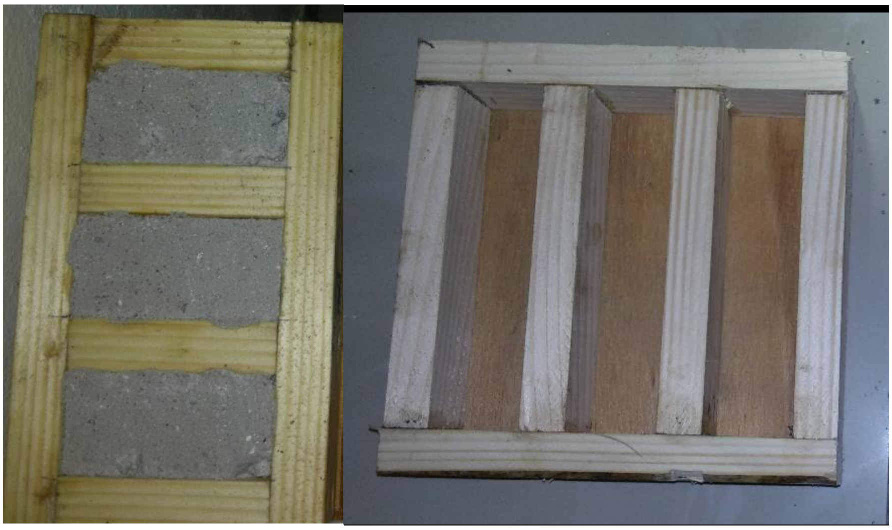

Figure 3.10 Compressive and flexural test specimens

3.4 Transporting the Masonry Samples

Once the masonry mockups were built by using the construction materials from the historical site and by the skilled masons who have worked for years in the rehabilitation of historic masonry structures, the samples were left in the Gondar for six months. This was to assure that the strength of the specimens was adequate enough to transport. Then they were transported from Gondar, Royal Enclosure to Addis Ababa, AAiT lab with considerable caution.

Although some difficulty was met during transportation, the samples were highly protected during the loading, unloading and transportation process. To this goal, individual samples were properly enclosed by protective covers; paper, Styrofoam and plastic sheet. In addition, each of them were tied up by a rope with their own wooden pallets, to safeguard them from falling down during movement.

The other protection is the mechanism that was used to minimize the effect of shock, caused by the vibration of the car, as it goes on all the way long over the uneven roadways. For this purpose, the protection against the effect of vibration was achieved by using half full sacks of sand that are laid upon the bed of the car. Then the samples were placed over those sacks and tied to the base of the car, see Figure 3.11. This way the samples were transported safely and arrived at the material testing laboratory of AAiT.

Figure 3.11. Masonry samples during Transportation

3.5 Laboratory Test Setups and Procedures

The goal of the experimental study was to evaluate the most important mechanical parameters needed for numerical modeling of historical masonry buildings, namely, the cohesion and friction coefficient by triplet tests; the compressive strength and Young's modulus through compression tests; and other tests were conducted on the mortar.

3.5.1 Triplet Shear Test

Initially, the test setup for the shear test was arranged and checked using trial sample made of brick and cement mortar, having the same size as that of the mockups built in Gondar. This setup was arranged in a way that the specimen would be subjected to a horizontal compression and vertical shear load. In this case, the specimen was supposed to be rotated 90^0 from its built condition so that the vertical shear load could be applied on the horizontal joint of the masonry, see Figure 3.12.

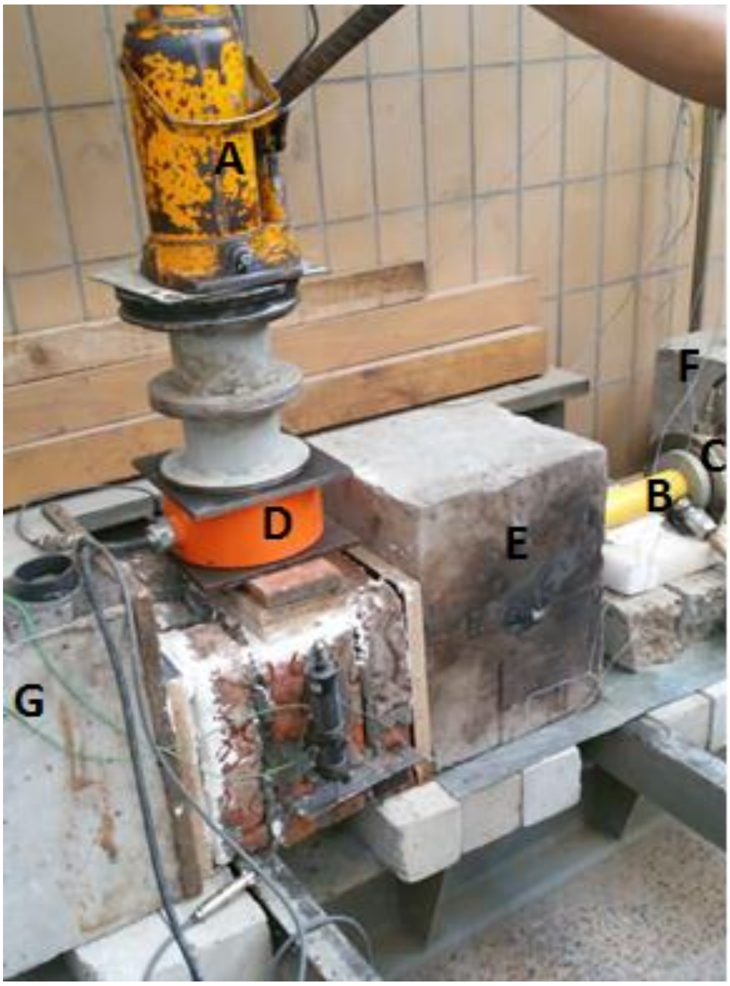

Figure 3.12. Triplet shear test setup 1

At this point, the horizontal confinement was provided by Jack B which is placed in between the concrete blocks E & F. Block E was made to freely slide over the horizontal "frictionless" bed. The frictionless surface was achieved by attaching steel sheet on the two contacting surfaces. And the contact surface was greased to minimize friction and to facilitate for the full transfer of confinement load to the specimen.

Following the horizontal confinement, vertical load was applied using Jack A. Where the incremental shear load that was applied on the specimen was recorded by load cell D. Whereas the confinement load was recorded by load cell C. The load was recorded with the interval of 5sec, from the beginning of the loading until the failure of the sample. In this manner, the shear test was conducted successfully on the brick masonry sample.

However, when it comes to the actual samples, lime-based masonry, the above setup became ineffective. Because it required rotating the sample by 90^0 from its built condition. In doing so, the lime-based masonry specimen simply crushed by its own self-weight around the base layer, see Figure 3.16. Thus, the setup was restructured in such a way that the need for 90^0 rotation of the specimen would be avoided then the setup was rearranged as depicted in Figure 3.13.

As it is shown in the figure the modification was to use vertical confinement and horizontal shear load. For the vertical confinement, the base of the sample which was a wooden pallet was strengthened to attain uniformly supported base. To this goal, small sized of woods was inserted in between the gaps under the pallet.

Regarding the horizontal shear load two concrete blocks, having a size of 10x10x50cm were used to rigidly support the upper and lower layers of the assemblage. The upper support was suspended by wire from the top and supported by the larger concrete block on the left side. As that of the lower support, it was simply placed on the bottom level.

On the other hand, the shear load at the middle layer was applied by the thick metal plate and a bundle of wooden planks. For the tests that were conducted under higher confinement levels, the bundles of the planks were not efficient so that they were replaced by a concrete block of a size 10x10x50cm. Additionally, in order to reach the level of the middle layer,15x15x15cm concrete blocks were used as a spacer.

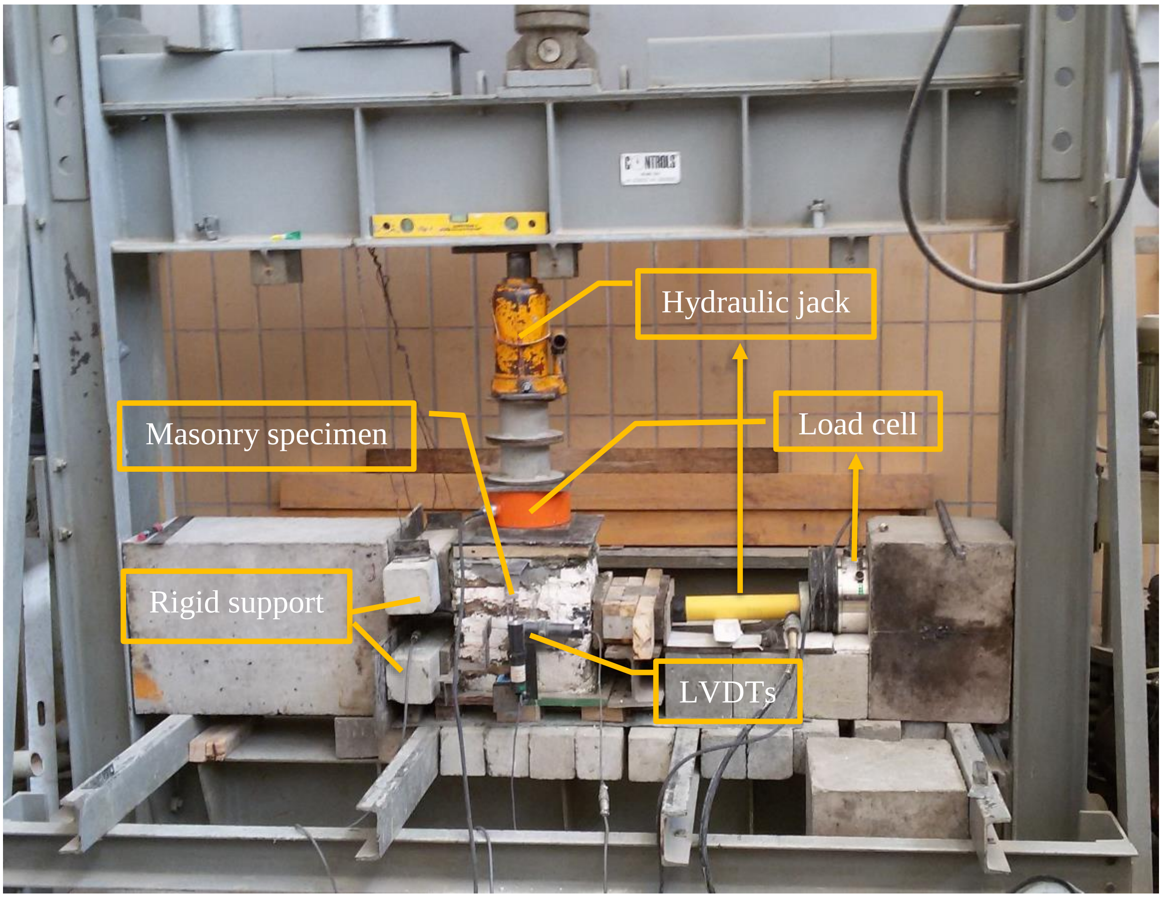

Figure 3.13 Triplet shear test setup 2

At this point it should be noted that, at locations of load applications or boundary conditions, the surface of the samples was plastered by cement mortar. This was to have an even surface for distribution/ application of loads. Then the load cells with hydraulic jacks were given their appropriate positions. And three LVDTs were used to measure the change in displacement. One was to measure the vertical displacement and the other two were attached in a way that they can capture the relative horizontal displacement, one is attached in the front and the other at the back of the sample.

In this manner, five shear test with a different level of confinement were conducted on the lime-based masonry specimens. Specimen TS1 & TS4 were tested under a confinement level of 0.1MPa, Specimen TS6 was tested under 0.3MPa and Specimen TS2 & TS5 were tested by confining them with 0.5MPa confinement load. These confinement levels were chosen as per the recommendation given on (EN1052-3 2002) and based on the estimated state of stress on the masonry structures. As a result, the behavior of the historical masonry under a combined compression and shear loading has been investigated and the shear strength parameters were determined.

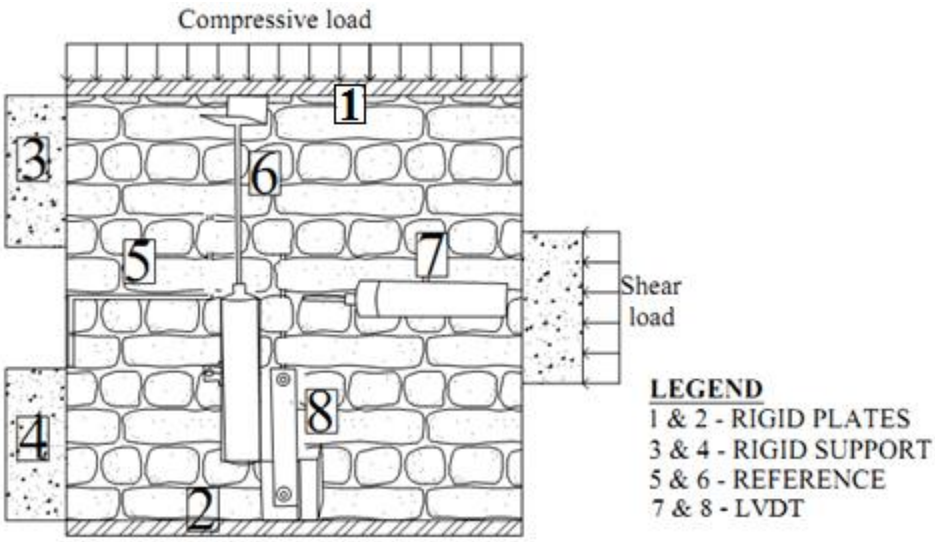

Figure 3.14 LVDT and Reference placement on the specimen

3.5.2 Compression Test

The compression test setup was not as such complicated as that of the shear test. Mainly, the test was conducted by using the frame that was used for the shear test, which served as a reaction frame to apply the load. Similarly, the base of the specimens was strengthened by filling the gaps under the wooden pallet, Figure 3.15.

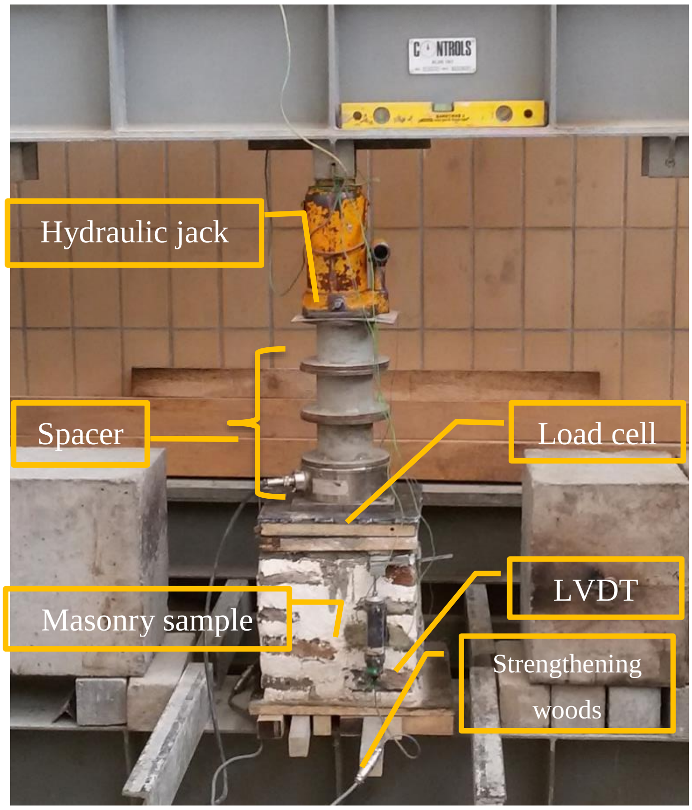

Figure 3.15 Compression Test setup

Regarding the load, a 300kN capacity hydraulic jack was used to apply the load in a displacement based manner, manually. In order to distribute the load uniformly over the surface of the specimen, the top surface was plastered and leveled by cement paste. In addition, a wooden and thick metal plate were placed over the leveled surface. During the loading process, the displacement history was measured using three LVDTs.

The transducers, as well as the references, were attached on the specimen by using metal epoxy on three different faces of the specimen, in such a way that they could measure the displacement change in the vertical direction. Accordingly, the compression test was conducted on two masonry specimens, to determine the modulus of elasticity and compressive strength of the masonry assemblage.

3.5.3 Supplementary Tests

The following tests were conducted to resolve questions related to the lime mortar that was used to build the specimens and the mortar that exists on the Gondar heritage buildings.

3.5.3.1 Rate of Carbonation

Carbonation results in the hardening of the mortar or increased mechanical strength. As the matter of fact the breakage of one the sample was the cause that brought the chance to look into the carbonation of the mortar, used to build the masonry samples. In other words, the carbonation of the lime mortar was questioned when that sample was broken when it was about to be tested for shear, meaning when it was being rotated and placed on the setup arranged to apply vertical shear and horizontal confinement.

By that time, the mortar fragments were found to be easily friable by squeezing between fingers. This was taken as an indication for the inadequacy of the carbonation around the base and in the interior part of the masonry. So, this suspicion was supposed to be verified by using appropriate test method and carbonation of the mortar was examined.

To this goal, phenolphthalein was found to be the feasible method. It is a chemical method that is frequently used to detect carbonation of mortar. The test procedure is simply to spray phenolphthalein on the freshly broken surface of the mortar and if the surface stained to a deep pink then it indicates that part of the mortar is not carbonated. On the contrary, if the surface stayed colorless or does not change color, that part is carbonated.

Accordingly, the test conducted on the mortar of the broken specimen has shown that the mortar was not fully carbonated, see Figure 3.16. Then the next attempt was to enhance the strength of the mortar by accelerating the carbonation. In an attempt to accelerate the carbonation rate and strengthen the mortar, mini carbonation chamber has been devised in Chemical Laboratory, AAiT. In fact, carbonation is affected by factors such as carbon dioxide concentration, temperature, and relative humidity. Therefore, when it was planned to set up the carbonation chamber, the above environmental factors were considered. Basically, the effort was to create an environment that has a high CO_2 concentration and moderate humidity.

Figure 3.16 Broken specimen and phenolphthalein test on the mortar

As shown in Figure 3.17, the chamber was made by using a desiccator which provides the means to enclose the mortar specimen, carbon dioxide and water vapor in one place. First, carbon dioxide was injected (from the carbon dioxide cylinder) in the desiccator to replace the environmental air by CO_2 and then hot water in a small container was placed inside the desiccator so that the steam covered the interior surface of the desiccator.

After these steps, the samples were put in place and finally, additional CO_2 was added to balance CO_2 loss when the desiccator was open. Furthermore, in order to compensate for the leakage of CO_2 and water vapor, the above procedures, meaning injection of CO_2 and steam, was repeated twice a day.

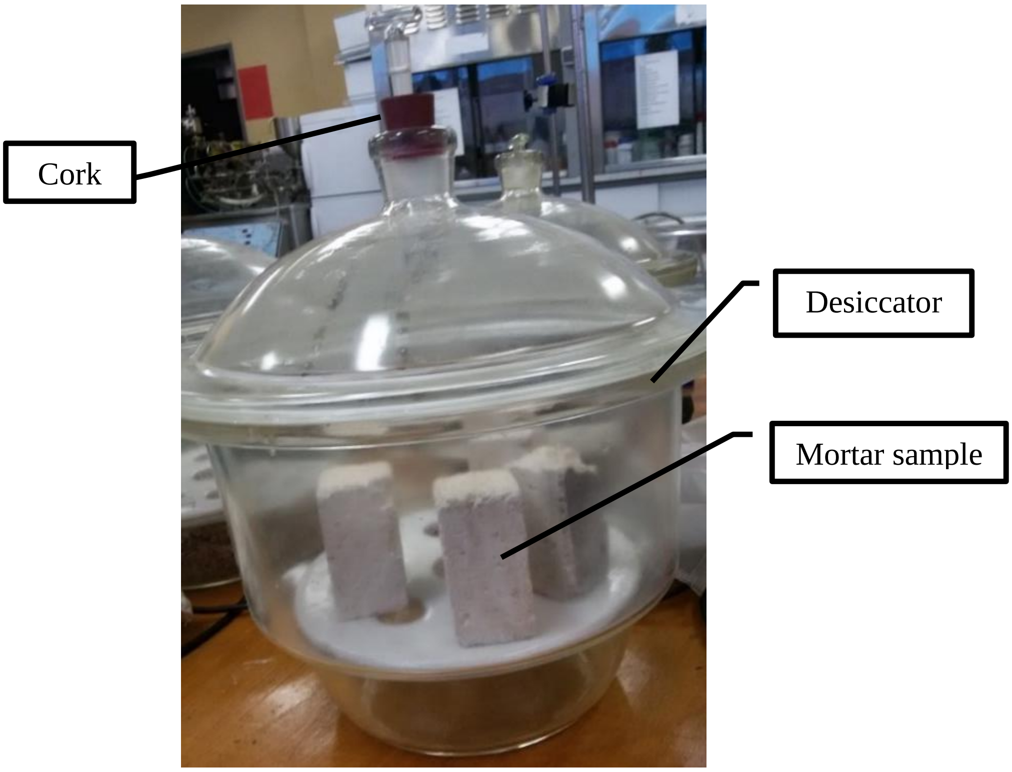

Figure 3.17 Laboratory scale mini “carbonation chamber”

In this manner, the mortar samples were treated for a week and the carbonation level was again checked using phenolphthalein. As a result, the mortar samples that were placed in the “carbonation chamber” have shown no color change when phenolphthalein was sprayed on their fresh broken surfaces, see Figure 3.18. The result was a good indication that if carbonation chamber could be made possible with a more controlled system, it would be advantageous to accelerate the rate of carbonation.

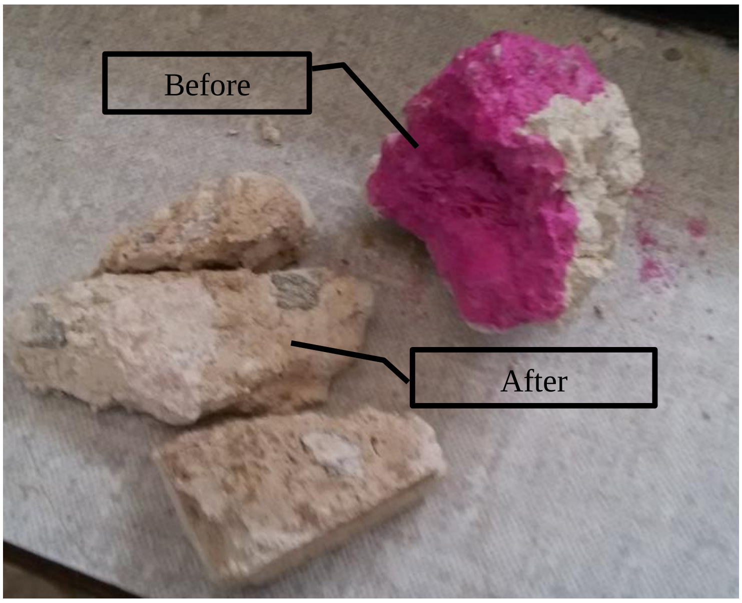

Figure 3.18 Phenolphthalein test before and after accelerated carbonation

In general, although the accelerated carbonation experimented on the mortar has given encouraging result, the application of accelerated carbonation to the masonry assemblages was found to be impossible. This is because of several reasons such as, lack of CO_2 concentration measuring equipment, room/space to set up a large size and well-controlled chamber and scarcity of budget. Therefore, the attempt made to set up the mini carbonation chamber was reported in a hope that it could give some impression for further studies.

3.5.3.2 Presence of egg in the mortar of historical buildings

As legend it is said that the castles of Gondar were built from stone masonry with a mortar consisting of egg parts. In an attempt to prove the truthfulness of this legend, (Yesuf 2014) has interviewed historian and personnel responsible to conserve and restore the historical buildings. As a result, the author concluded that the likelihood of this legend to be untrue is very high. However, for some, this question still remains.

Therefore, in this study an effort was made to examine the presence of egg in the mortar of historic buildings. Although this attempt seems to be out of the scope of the study, for two reasons it was found to be reasonable to conduct this investigate. The first and simplest reason was, the result of this test would be a checking mechanism for the representativeness of the masonry samples, reproduced to replicate the walls of the historical buildings. And the other reason was, the fact that the knowledge of the parent materials in the construction of those historical buildings is advantageous to use appropriate mortar for restoration purpose.

Having these two reasons as a motivation, mortar samples was collected from three different buildings in Fasil compound. These samples were collected from the part of the buildings where it was believed to be untouched by any restoration activity. This was made possible with the help of a conservator who have been working for several years in the royal enclosure. With his guide, it was tried to have samples from the “original” mortar and then the samples were investigated for the presence of egg.

The main focus of this investigation was on the inner part of egg. Since the chemical composition of the shell largely contains calcium carbonate (Austic 2018), it would be difficult to differentiate it from the calcium carbonate of the lime itself. Thus, by focusing only on the edible part of the egg, which is mainly protein. The mortar samples were examined under Fourier-transform infrared spectroscopy, FTIR.

FTIR spectroscopy provides information about the functional groups, contents of protein. It works by releasing infrared radiation on a sample and seeing which wavelengths of radiation in the infrared region of the spectrum are absorbed by the sample. It has to be noted that each compound has its own characteristic set of absorption bands in infrared spectrum. Characteristic bands found in the infrared spectra of proteins include the Amide I and Amide II (Gallagher n.d.). The absorption associated with the Amide I band leads to stretching vibrations of the C=O bond of the amide, absorption associated with the Amide II band leads primarily to bending vibrations of the N-H bond. Consequently, the vibration that occur in different wavelengths resulted in FTIR graph.

To have a clear understanding about Amide I and Amide II, it worth knowing that protein is a linear biological polymer for which the monomeric units are amino acids, Figure 3.19. The amino acids are linked to form a polymer by linking the amino group on one amino acid with the carboxylic acid group on another amino acid to form an amide bond.

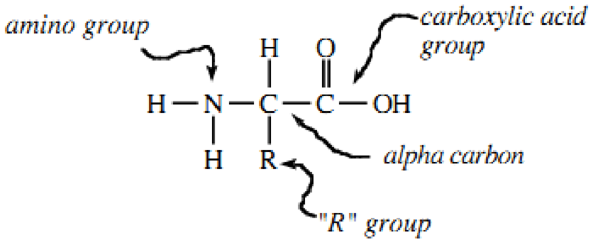

Figure 3.19 The basic building block for protein, (Gallagher n.d.)

When it comes to the investigation on the mortar samples under the FTIR spectroscopy, in addition to the three “original” mortar samples, a mortar sample from the masonry specimens was examined as a control group. As a result of the test the FTIR graph shown on Figure 3.20 was obtained.

The labels that are used on the graph indicate the buildings from which the samples were collected. ME, FA, and AD stands for Mentwab, Fasil, Adiyam Seged Eyasu buildings respectively and RE is the mortar used for restoration, taken from the masonry specimens.

The subsequent task was to interpret the graph and look for the functional groups that are indicative for the presence or absence of amino acids which are the building blocks of protein. the graphs of Figure 3.20 were interpreted with the aid of IR correlation charts.

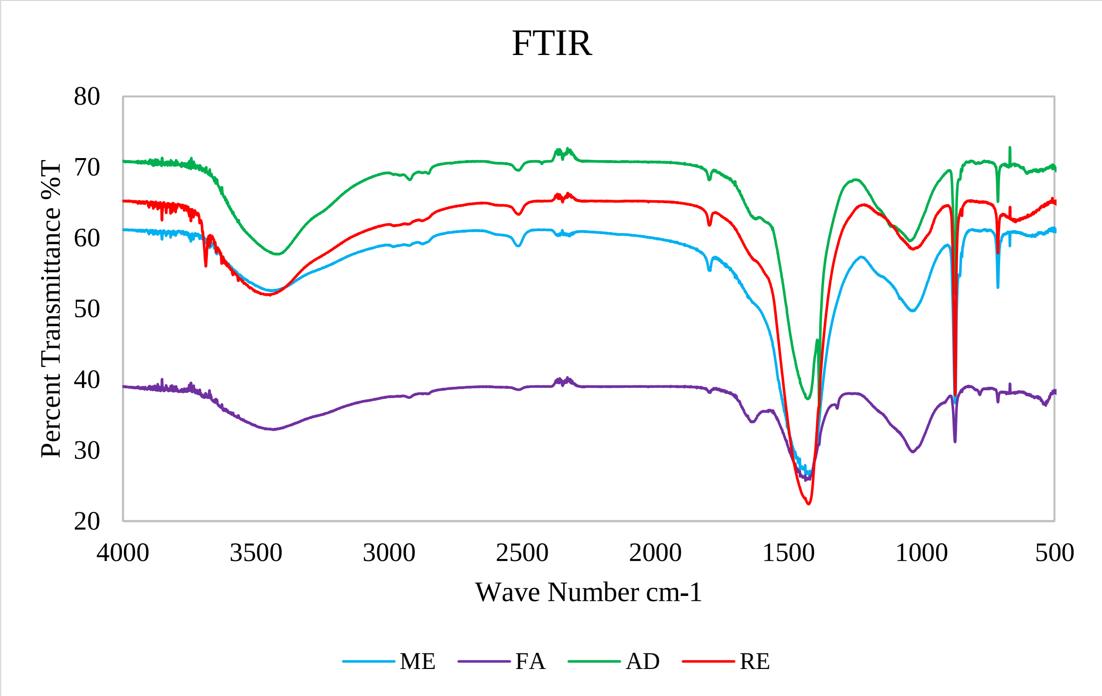

Figure 3.20 Infrared spectrum of the mortar samples

Based on IR correlation charts, if the amino groups (NH or NH_2) existed in the mortar sample, a peak value would occur in the range of 3400-3250 cm^{-1}. However, the peak values occurred at 3425cm^{-1}, and this value is a boundary range for the O-H and N-H groups. At the mentioned range, it is found to be difficult to differentiate the functional groups. Thus, there is no a clear indication about the presence of amino groups. In addition, NH_2 shows two bands, but such kind of band was not observed in the case of the mortar samples.

In conclusion, the results of the FTIR test conducted on the mortar samples did not give a clear indication about the presence of egg/ amino acid (functional groups). Thus, in order to clarify the confusion of functional groups it is advisable to conduct chemical tests/ element analysis. Having this in mind, based on the results of the FTIR test, by looking at the similitude among the trend followed by the graphs, it could be said that the "original" mortar samples and the one which is being used for restoration have similar chemical composition.

4. TEST RESULT DISCUSSION AND ANALYSIS

4.1 Triplet Shear Test

In order to investigate the local interaction between mortar and units in relation to the masonry joint shear strength, a total of eight triplet shear tests were conducted. Five of the tests were on the lime-based masonry assemblages and the remaining were on the brick masonry specimens which were built as a trial sample during the arrangement of the test setup. As a result of the test shear load versus shear displacement relation of the specimens were investigated. In addition, dilatancy and failure mechanism of the specimens were examined.

4.1.1 Shear Load–Displacement relationship of Lime based Masonry

The test is commonly used as a standard test to obtain experimental data related to masonry joint shear strength. Thus, it has resulted in shear load-shear displacement relation for different confinement levels. Table 4.1 summarizes the vertical confinement stresses, maximum shear strength and average shear strength of a pair of specimens tested under the same pre-compression load. These values, presented in the table were used later to define initial shear strength parameters of the masonry.

The shear stress is calculated by the following expression:

$$\tau = \frac{H}{2A}$$

Where H is the load in the horizontal direction (shear load) and A is the cross-sectional area of the joint section. The normal stress is also calculated from the normal pre-compression load V, and it is also based on the total area of the cross-section as:

$$\sigma = \frac{V}{A}$$

The test results summarized in the table shows that there is an increase in shear stress in relation to the increase in confinement loads.

Table 4.1 Summary of triplet shear test on lime-based masonry samples

Specimen	Vertical force (kN)	Vertical stress (MPa)	Maximum horizontal force (kN)	Shear strength (Mpa)	Average Shear strength (MPa)
TS1	9	0.1	43	0.24	0.23
TS4	9	0.1	38	0.21	
TS6	27	0.3	65	0.36	0.36
TS2	45	0.5	98	0.54	0.53
TS5	45	0.5	93	0.52	

The general shear load-shear displacement relations for the lime-based masonry mockups were depicted in Figure 4.1. The shear displacement is the result of averaging the measurements recorded by the two horizontally placed LVDTs, on the opposite faces of the specimen. In the figure 4.1 the response of TS2 is not plotted because, only the results reported in Table 4.1 were obtained during the test on specimen TS2.

In a broad view of the results, there is apparent difference regarding the increment on the shear load of each specimen, which is accredited to the difference in the level of confinement stress. As the confinement loads increase the shear capacity of the specimens also upsurge. Note that TS1&TS4 were tested under a pre-compression load of 0.1 MPa, TS6 (0.3 MPa), and TS5 (0.5MPa).

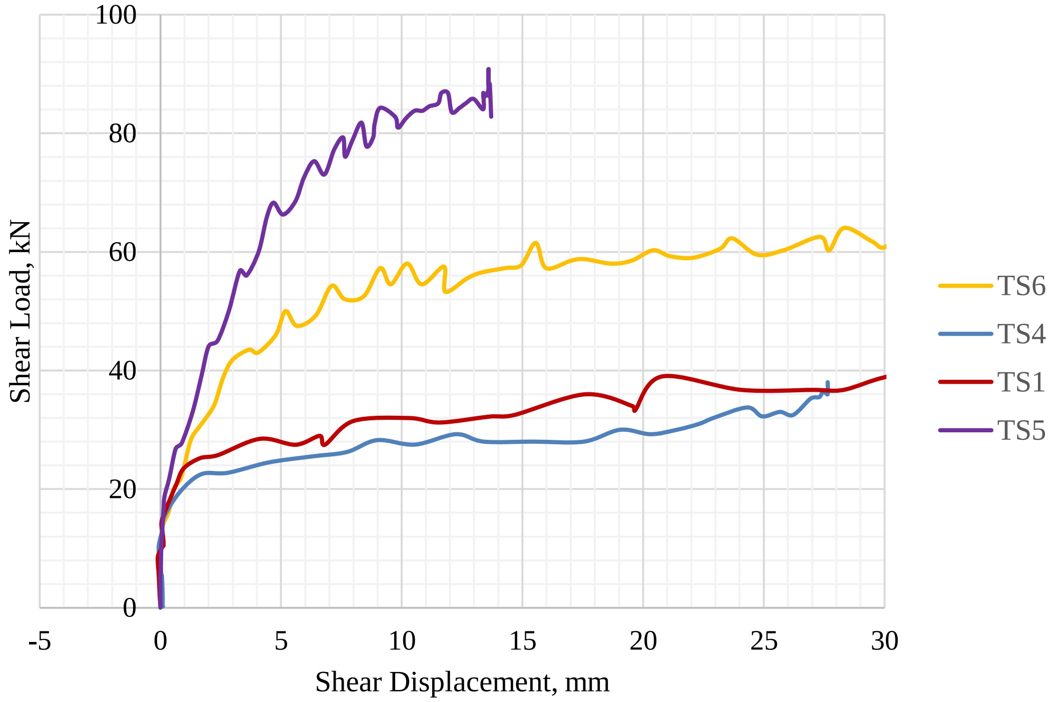

Figure 4.1 Shear load-shear displacement for the lime-based masonry samples

The results show that how the masonry assemblage responded to the combined compression and shear forces. Meaning, the behavior of the masonry assemblages in the early stages of loading and afterward were identified clearly.

When it comes to the behavior of the specimens in the early stages of loading, the initial part of the pre-peak region is characterized by the presence of negative and positive shear displacement. These characteristics indicate that the specimen bulges as the pre-compression load were applied. These features were highly evident on specimen TS1 and TS4 which can be confirmed from Figure 4.2 where the pre-peak part is highlighted.

Then by a non-linear stretch, the shear load-shear displacement history extends to peak shear load, see Figure 4.1. The non-linearity is associated with the roughness of the shear plane and friction between the constituent materials which is due to sliding, rolling friction and interlocking action among the stones and aggregates.

Similarly, the jagged nature of a graph of TS5 and TS6 was due to the same reasons, rolling friction, and interlocking actions. It is believed that these factors have more effect on these two specimens because their confinement level was higher than the others.

In contrast, the relative smoothness of the graphs of TS1 and TS4 is believed to be caused by the shear plane of the specimens that mainly incorporates the mortar. Furthermore, the specimens were tested for the lowest confinement load so that the stones that exist in different layers had a very less chance of getting in contact and creating highly rough friction surface.

Concerning the last stages of the test, although minor fluctuations are apparent on the shear load, the shear load tried to stabilize after the peak load and no shear softening was recorded. Those minor fluctuations were associated with the mobilization of various contacts/friction among the constituents, as the horizontal displacement was increased.

In general, the behavior of the masonry joints under a combined compression and shear load exhibit considerable plastic deformations associated with the inelastic sliding. In addition, the apparent difference among the specimens, see Figure 4.1 is accredited to the variation in testing condition (confinement level), the inevitable difference in the rubble stone masonry samples and interaction within the assemblage.

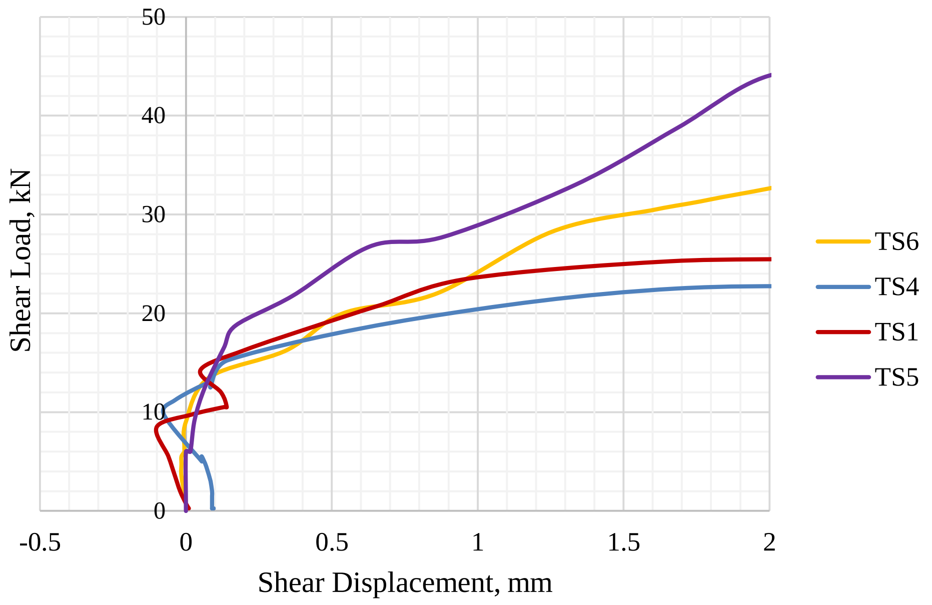

Figure 4.2 Partly potted shear load – shear displacement

4.1.2 Shear Load–Displacement relationship of Brick Masonry

The shear load-shear displacement graphs of the brick masonry samples, Figure 4.3, are characterized by a sharp initial vertical stretch. Then the peak loads are attained for a very small shear displacement. Although this is true as a general characteristic of the specimens (for BM1 and BM2), there are also some other clear differences among the results.

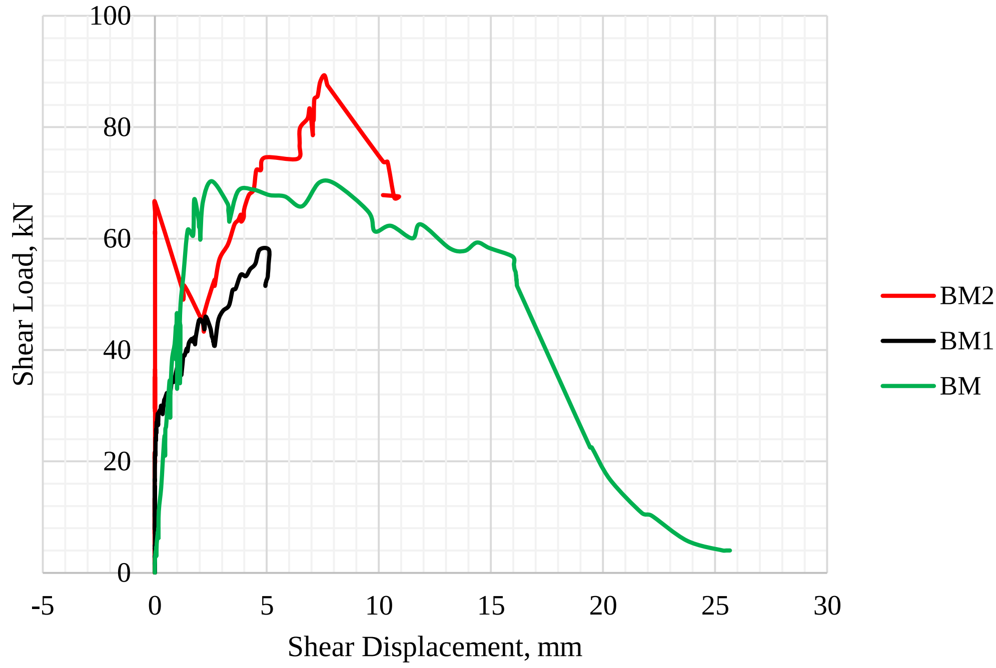

Figure 4.3 Shear load-shear displacement relation for brick masonry

It should be noted that the condition of testing and age the specimens were quite different for those three tests. It is to mean that, BM1 & BM were tested at the age of 12 and 15 days respectively, after the construction of the specimen. As that of BM2, it was tested after three months of the fabrication. The age of the specimen at the time of testing has a significant influence on the strength of the cement mortar, in return, the response of the masonry assemblage would be affected by the strength of the mortar.

For the purpose of clarification, compare the graph of BM1 and BM2. Although the confinement stress used for BM1 (0.2MPa) was greater than that of BM2 (0.1MPa), specimen BM2 is highly stiffer than BM1, in initial stages. And this is attributed to the difference in strength of the mortar of the two specimens. Other than the initial stiffness, the pre-peak behavior of the two samples also has a clear difference.

After the initial vertical stretch, with small shear displacement, specimen BM1 followed a nonlinear path to develop to a peak load. According to the author understanding, this nonlinear response is related to the minor roughness, and a relatively weak strength of the mortar.

On the contrary, specimen BM2 once it reached the limit on which it can extend on the vertical path (initial stiffness), the shear load suddenly dropped in a brittle manner. The sudden drop is due to de-bonding at the mortar joint interface. Then after, the other nonlinear path which leads to the peak load followed. This characteristic is thought to be related to the mobilization of friction among the constituents. finally, after the peak load, the specimen failed again in a brittle manner.

The other contributing factor to the noticed difference on the graphs of Figure 4.3 is the condition on which the samples were tested, this is particularly pronounced for BM and BM1. These two tests were conducted on one specimen. It could be said that a single specimen was subjected to a repeated[1] loading condition.

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[1] While conducting the first trial which is BM1, under 0.1MPa of a constant confinement stress, the test set up or the frame got suddenly disturbed when the shear load reached about 58kN. So the test was supposed to be halted at that stage. Then after rearranging and strengthen the setup, the test was repeated on the same specimen but the confinement level was doubled, 0.2MPa. the second trial resulted in the response of BM. Even though the test was not completed on the first trial, until the failure of the sample, the acquired data is found to be important to comprehend the response of the specimen in the early stages of loading.

As it is depicted on Figure 4.3, BM has no that vertical stretch which was observed on BM1 and BM2 because the bond that could give rise to this initial stiffness was broken in the first trial. So what was obtained in the second trial is that a relatively linear increment towards the peak load. After peak load is attained there is a short softening branch, corresponding to a progressive reduction of the cohesion, see BM. Then the final stage revealed that the sample failed in a brittle mode of failure.

4.1.3 Comparison of the Two Masonry Types

The difference on the test results of the two groups of masonry samples, Lime based rubble stone masonry (TS) and brick masonry made by cement mortar (BM), is mainly dependent on the distinct properties of the constituent materials. Note that, for simplicity the labels that were tagged on the samples are used here in the discussion.

Figure 4.4 shows the shear load versus shear displacement of the lime based masonry and the brick masonry assemblages. All of the specimens were tested under a confinement stress of 0.1MPa.

The basic difference on the shear load-shear displacement relationship of the samples is as a result of the difference in the mechanical properties of the binding materials and the fabric of the specimens. Regarding the apparent difference among the responses of the specimens the following characteristics were observed.

In the initial stages of loading, unlike the TS group, the samples in BM group are highly stiff until the load reached the capacity to break the bond on the mortar interface. On the other hand, the negative and positive shear displacement that was observed in the case of lime-based masonry specimens, see Figure 4.2, was not seen when it comes to the case of the brick masonry.

On the later stages, the brittle nature of the brick masonry is clearly apparent on the branch that goes down in a negative slope, after the peak load. Regarding the lime-based masonry, although its constituent materials are brittle, the stabilization of shear load without shear softening is a unique feature of these samples. As stated earlier, this property is thought to be associated with the rolling, sliding and interlocking actions among the aggregates and stones.

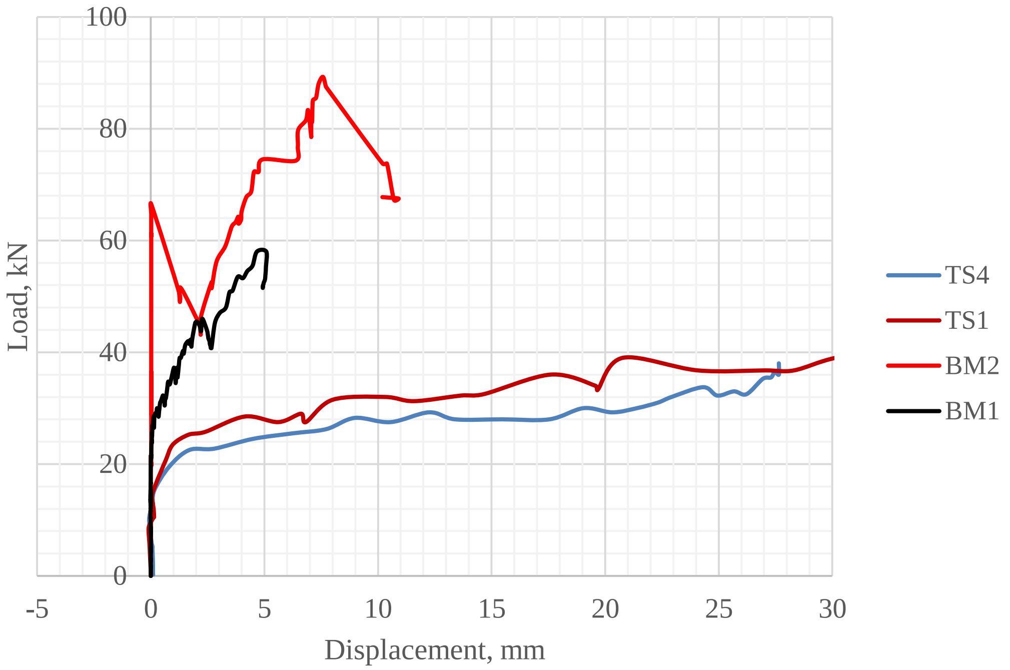

Figure 4.4 Comparison of the shear behavior of the two types of masonry samples

The other important issue regarding shear tests is the dilatancy of the specimens. Dilatancy is significantly observed in the lime-based masonry than in brick masonry samples. This is because after cracking and sliding the two sides of the cracks do not match in the case of TS specimens, which is an indication of the change in volume. This effect increases with the irregularity of the crack surfaces, therefore it tends to be stronger in TS group rather than BM. A more detailed discussion on dilatancy is presented subsequently.

4.1.4 Dilatancy

For the specimens tested under a combined compression and shear loads, the variation in the vertical displacement was measured by the LVDT positioned to capture the local vertical movement along the height of the specimens.

In other words, as the horizontal(shear) displacement was applied progressively, for the lime-based masonry samples there was a considerable change in the vertical displacement. And this change is considered as a measure of dilatancy. Figure 4.5 shows the vertical displacement a function of the horizontal displacement, where three distinct phases were distinguished.

In the initial stage, as the confinement load was applied, so that the negative vertical displacement resulted as a result of the compaction. Then once the required amount of load (pre-compression) was reached and when the horizontal (shear) displacement was being applied, the vertical displacement remains almost constant in the case of TS1 and for the other specimens, it remained constant for a short shear displacement. Then progressively the vertical displacement decreases nonlinearly. This is significantly visible as the level of confinement increases (see, TS5 and TS6).

The nonlinear progression of the displacement provides variable dilatancy, decreasing vertical displacement as the shear displacement increases. It is presumed that this behavior is associated with the changes on the interfaces due to surface wearing. The wearing of the joints interface leads to compaction of the pours of lime mortar. For the larger confinement level, compaction of the specimen was clearly noticeable, which is revealed by the negative values of the vertical displacements.

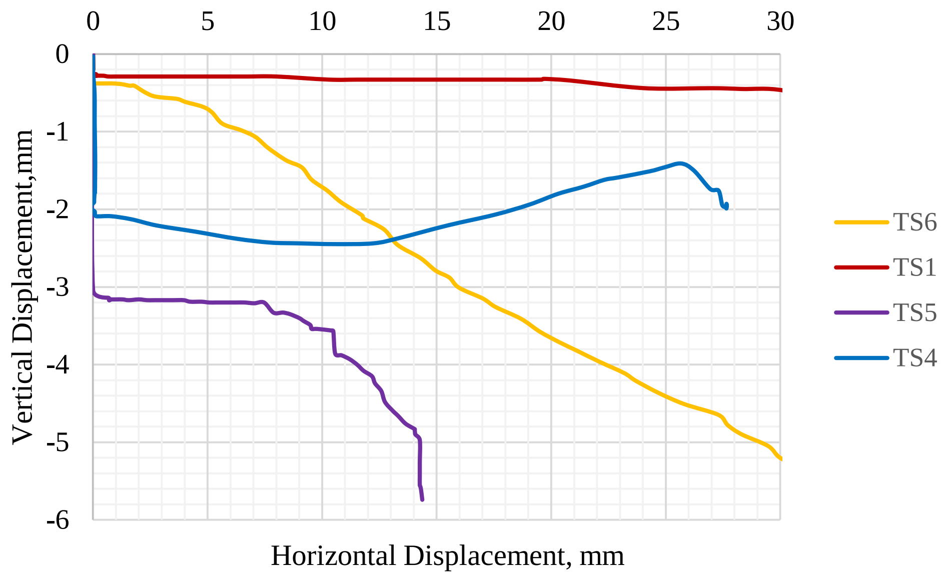

Figure 4.5 A measure of dilatancy

In Figure 4.5, for the same level of confinement TS1and TS4 have resulted in different responses. The reason for this difference is thought to be caused by the variation in the joints of the specimens. Generally, the change in the vertical displacement is dependent on the state and type of the shear plane, the shape/ roughness of aggregates in the mortar and that of the stones and the confinement level etc.

In the shear test even though the vertical displacement was fluctuating as shown in Figure 4.5. It should be noted that the confinement level or the pre-compression stress was required to be constant. In this regard, even if the hydraulic jack used for the tests was operated manually and it was unlikely to achieve a perfect constant confinement load. However, in this study the confining stress was almost constant, throughout each of the tests. Figure 4.6

shows the relation between vertical load and horizontal displacement for the tests on the lime-based masonry samples.

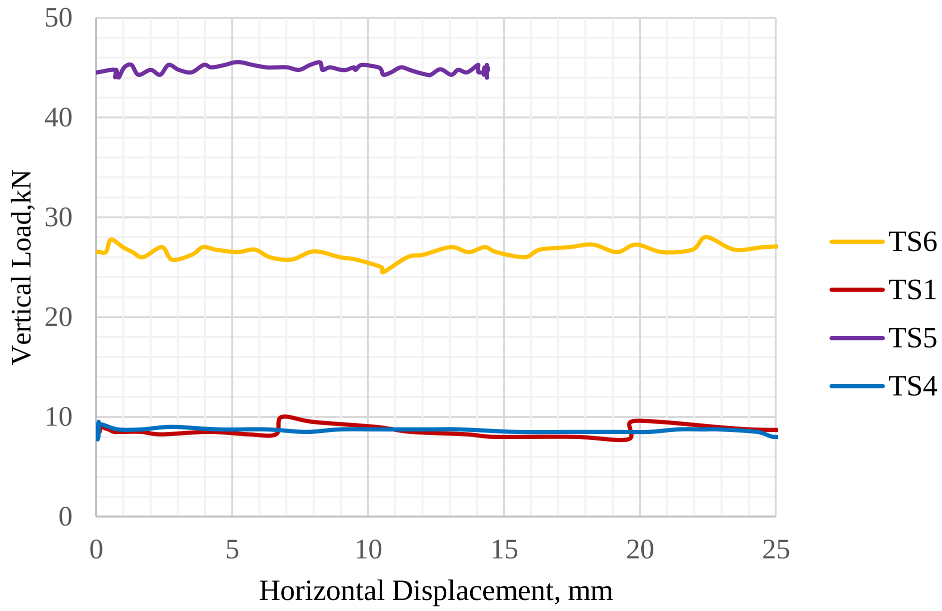

Figure 4.6 Horizontal displacement versus vertical Load

4.1.5 Normal stress versus shear stress

The shear test results show that an increase in the pre-compression stress results in a linear increase in the average shear strength of the historical masonry samples. The common explanation for this is that the strength of the joint is derived from a combination of bond shear strength and friction between the constituents (units and mortar).

Consequently, the correlation between maximum shear stress and confinement/normal stress, see Table 4.1 is expressed by the Coulomb type expression

$$\tau = c + \mu\sigma$$

Where c and μ are cohesion and coefficient of friction, respectively.

Figure 4.7 shows the relation between the normal stress and shear stress of the lime-based masonry specimens. As stated in (EN1052-3 2002), the linear regression of results from several triplet tests carried out with different compressive stress levels provides the shear strength parameters of the Coulomb's friction model, namely initial shear strength (cohesion) and the coefficient of friction.

It important to note that the good correlation between the experimental results and the linear regression line, confirms the initial assumption of Coulomb's friction law for the shear strength of horizontal bed joints in the lime-based masonry samples. As a result of the linear regression the cohesion and coefficient of friction for the lime-based masonry samples, TSs, were found to be 0.15 and 0.76 (43.5^0), respectively.

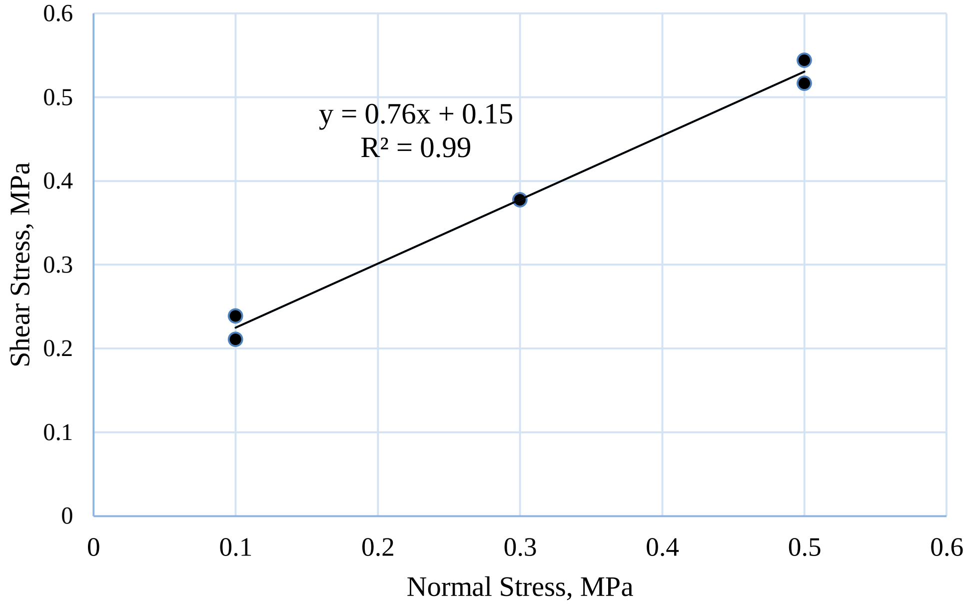

Figure 4.7 Normal Stress versus Shear Stress

4.1.6 Failure Mechanism of the Shear Test

As stated in the previous chapter, the historical masonry specimens were built with three roughly defined layers and each layer comprising a combination of mortar and basaltic stones.

Figure 4.8 Failure mode of Specimen TS1

Having this fabric of the samples, the horizontal shear load was applied to the middle layer which leads to a shear collapse mode by sliding of the medium layer. As a result, almost all the specimens followed the expected failure pattern, which is depicted in Figure 4.8.

4.2 Compression Test

The compression test was conducted to assess the compressive strength and Young's modulus of the lime-based masonry samples. Two masonry specimens were compressed effectively and used to comprehend the response of the masonry assemblage for pure compression.

To conduct the test, three displacement transducers were placed on the three faces in a way that they could capture the displacement of the specimen in the direction of load application. And the average reading from these LVDTs was reported.

4.2.1 Load-Displacement Relationship

As a result of the experimental data collected during the execution of the tests, the compressive strength fc and Young's modulus E were evaluated from the data using the following equations.

$$f_c = \frac{F_{max}}{A} \quad and\ E = \frac{F_{max}}{3 \times \varepsilon \times A}$$

Where F_{max} is maximum load reached on a specimen,

A is the specimen loaded cross-sectional area

$\varepsilon_{\frac{1}{3}}$ is the strain of the specimen when a load of 1/3 of the maximum load was achieved

Table 4.2 Summary of compression test results

Specimen	Maximum Load, kN	Compressive strength, MPa	Modulus of elasticity, MPa
C1	207	2.3	40.2
C2	217	2.4	34

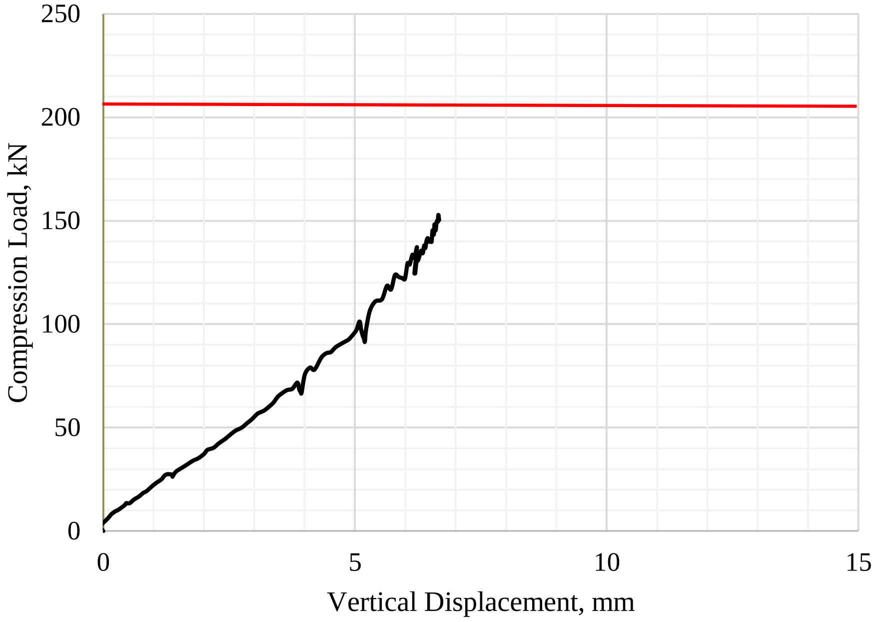

Figure 4.9 Compression test on specimen C1

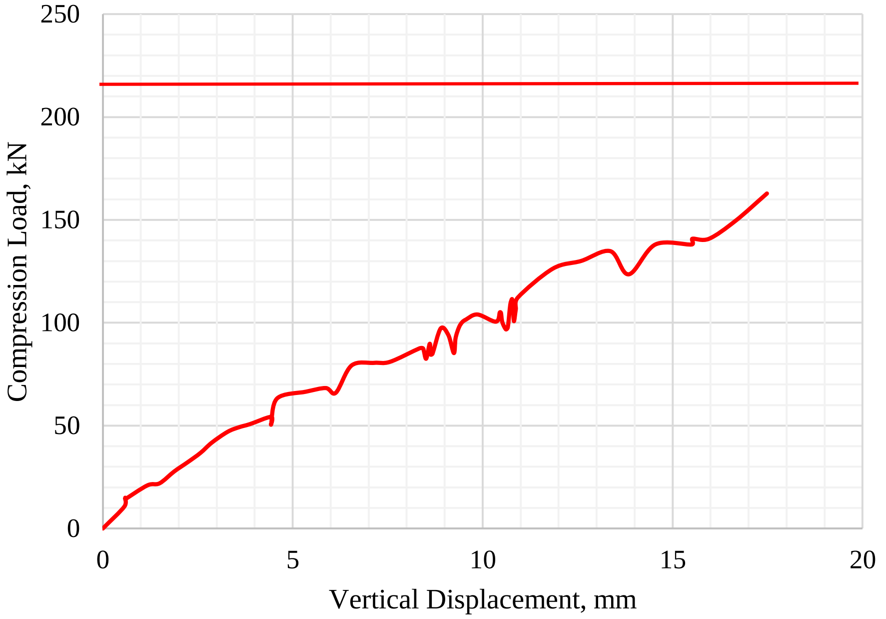

Figure 4.10 Compression test on specimen C2

The compression test results (compressive strength and modulus of elasticity) are summarized in Table 4.2. In addition, the compressive load versus vertical displacement relationships for the two specimens were depicted on Figure 4.9 and Figure 4.10.

The horizontal lines shown on the load-displacement graphs show the maximum compression load that was carried by the specimens. The displacement measurement was not recorded until the failure of the specimens, because of the detachment of the LVDTs.

In general, the experimental test resulted in fairly similar maximum strength. However, the load-displacement path followed by specimen C1 and C2 have some difference. This variation is attributed to the unique characteristics of each of the specimens, or the arrangement of the stones.

The Average compressive strength of the specimens was found to be 2.35MPa. This result is comparable with that of the values reported on some literatures. The authors, (Betti and Virgnoli 2011) and (Betti, Galano and Vignoli 2016), used a compressive strength of 2.5MPa, in their studies conducted on historical masonry buildings. Regarding the modulus of elasticity, an average value of 37MPa was obtained. As compared to values obtained from the above literatures, the modulus of elasticity calculated as a result of the experiment was found to be very small. On the research papers the modulus of elasticity of 700MPa and 1400MPa was used.

In the case of this study, the smaller value for modulus of elasticity is accredited to the sensitivity of the displacement reading that was highly affected by local damage around the stone on which the displacement transducer was attached. In other words, owing to the soft mortar at the top and bottom part of the mockup, the results for elastic modulus would be underestimated. At the same time the development of vertical displacement relative to the vertical load is found to be exaggerated. Therefore, it is important to note these facts while using the experimental results of compression for structural analysis.

4.2.2 Failure Mechanism of the compression specimen

The masonry type is rubble stone masonry where the stones were bonded by a lime mortar and arranged in random manner. This makes it is impossible to predict the failure mechanism. However, the failure mode of the masonry assemblages was observed during the execution of the tests. During the testing process, the mortar on the surface started to crack and spall off after the load exceeded 45kN.

And for specimen C1, once the load reached 150kN, the specimen disintegrated explosively. This happened when some highly stressed stones burst out from the back side of the specimen, see Figure 4.11. As for specimen C2, the stones failed in a random manner.

Figure 4.11 Specimen C1 after compression test (failure mode)

4.2.3 Effect of tensile stress on the compression behavior

Specimen C3 and C6 were tested in the presence of tensile stress. The results of these tests show that the occurrence of tensile stress the on masonry specimens significantly reduced the stiffness as well as the compressive strength of masonry assemblages.

As it is depicted in Figure 4.12 the stiffness as well as the strength of specimen C3 & C6, are much lower than that of the pure compressive tests conducted on other specimens.

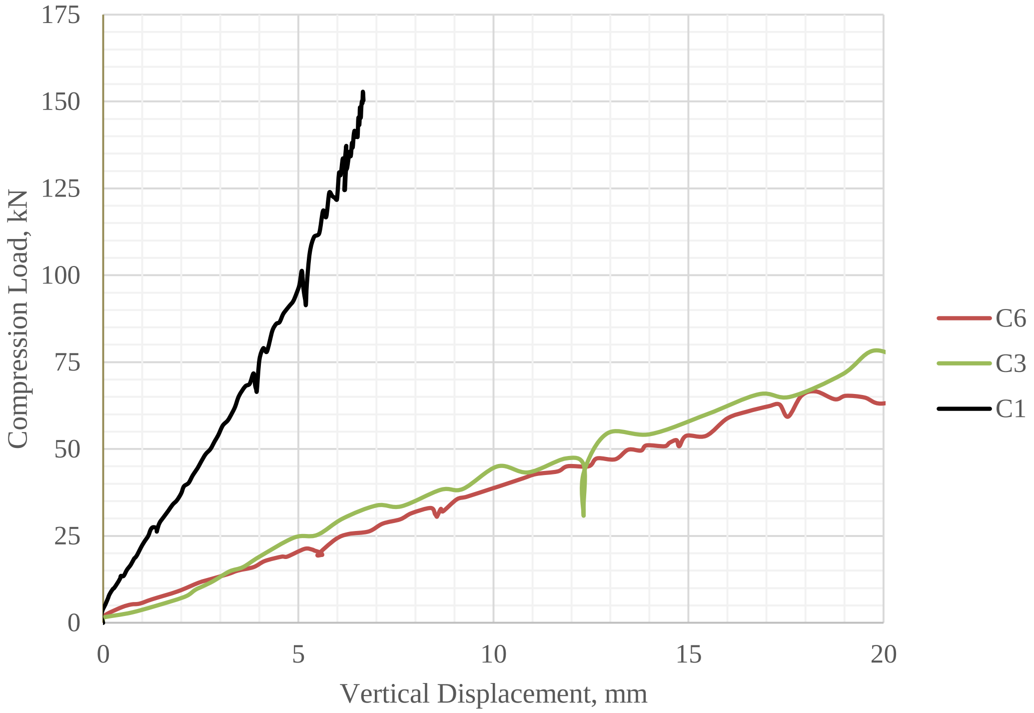

Figure 4.12 Effect of tensile stress in the compressive strength of masonry

Note that two different scenarios[2] have brought to splitting and bending stresses to the specimens, C3 and C6. The results of these tests could not tell the pure compressive capacity of the specimens. However, they are helpful to comprehend how the presence tensile stress affects the compressive strength of the masonry walls.

[2] For specimen C6, two steel plates that were placed side by side on the top of the sample were used to distribute the load coming from the hydraulic jack. Gradually, when the compressive load was being applied those steel plates started to slide against one another. And this sliding of the two plates forced the specimen to split in an early stage.

As for specimen C3, the woods that are shown Figure 3.15 were supposed to be placed at the bottom of the pallet to strengthen the base. However, the test was conducted without them. So after some steps of loading, the timber pallet bent and the loading condition became bending rather than pure compression.

5. NUMERICAL MODEL OF MENTWAB CASTLE; A CASE STUDY

5.1 Introduction

Mentwab castle is one of the buildings with most significant structural problem, needing a scientific investigations (ECHP-A1 2007). On the external face of the wall a number of plastered vertical cracks are evident. Therefore, the Castle of Mentwab is selected for the case study.

When it comes to the study of heritage buildings, historical events or alterations that affect and significantly change the original state or condition of the buildings should be considered and logically integrated within the numerical model (Roca, et al. 2010). For this purpose, despite the fact that the documentation on structural aspects of the palace are limited, important information about the Castle of Mentwab have been deduced from visual inspection and reviewed documents.

5.2 Overview of Mentwab Castle

Queen Mentwab was the last to build an imperial castle in Gondar and that was around 1750 EC. The castle in Fasil compound is an elegant, two- story building with a third smaller square shaped turret on the west and on the southeastern corner, a square tower with a dome over it. The walls are made of irregular stones and the top of the walls are enriched with remarkable battlements, while beautifully carved red stones decorate arches and openings. Some authors articulate the structure as “The most European of all the palaces in the compound”. (ECHP-A1 2007) .

The western façade of the castle has three, triple-layered, arched entrance doors on the ground and first floor, in addition one on the second floor. Those on the two upper floors appear to open out to wooden balconies which no longer exit, Figure 5.1. The eastern façade has a tall triple-layered arched door on the northern side of the first floor that opens out to an impressive staircase that leads down to the ground level, Figure 5.6.

As for functionality, the ground floor of Mentwab palace has been used as a public library (Aalund 1985). Currently the ground floor is used as office space or documentation center and the upper floors are inaccessible for other users except for janitors and maintenance workers.

Figure 5.1 western side of Mentwab Castle

5.3 Material and Geometric Details

5.3.1 Floor and Roof Slab

To begin with the first floor, the slabs of all the rooms are found to be made of concrete. Including screed and plaster of the soffit, the thickness of the slab is found to be 95mm in some rooms and 60mm in the other. As for the reinforcement bars used, a diameter of 4mm bar was observed uncovered in the slab (ECHP-A5 2007).

In relation to the roof system, as it is shown in the Figure 5.2 it is steeped roof made of two types of formations. Currently, one is concrete and the other is a traditional timber construction. The roof above the banquet hall, northern room, is made of the traditional system of primary and secondary timber beams covered with red stone slabs of a thickness 8cm, see Figure 5.3. Additionally 15cm thick lime mortar, damp proofing and 2cm thick in-situ prepared cement mortar tiles (Getahun 2000). As per (ECHP-A5 2007) the southern room is made of vertically laminated timber beams with scarf and dovetailed joints. Concerning the middle room, it is estimated that its roof is made of concrete beams.

Figure 5.2 Roof Top

Figure 5.3 Interior of roof above banquet hall

5.3.2 Wall Systems

Massive and solid masonry walls, without core infill, characterize all the historic building in Fasil Ghebbi. Alike the other buildings in the imperial compound the thickness of the wall of Mentwab castle is also impressive, having a thickness varying between 1.50 to 0.6 m. The wall systems are constituted from basaltic rubble stone set in lime mortar and red volcanic stone is used for decorations and architectural details.

Regarding the quality and layout of the masonry, some difference among different parts of the buildings is apparent. But relatively there is still homogeneous characteristics. The apparent variations in quality are ascribed to the restorations or reconstructions interventions rather than changes in the original construction.

5.3.3 Geometric Details

Construction drawings, floor plan, elevation and section drawings of Mentwab Castle were collected from the UNESCO online archive, library of Authority for Research and Conservation of Cultural Heritage (ARCCH) in Addis Ababa and Gondar. In earlier times, the ground and first floor plans of the castle were drawn by architect Sandro Angelini in 1971. Afterward these drawings were revised, detailed and replicated by the staffs of ARCCH.

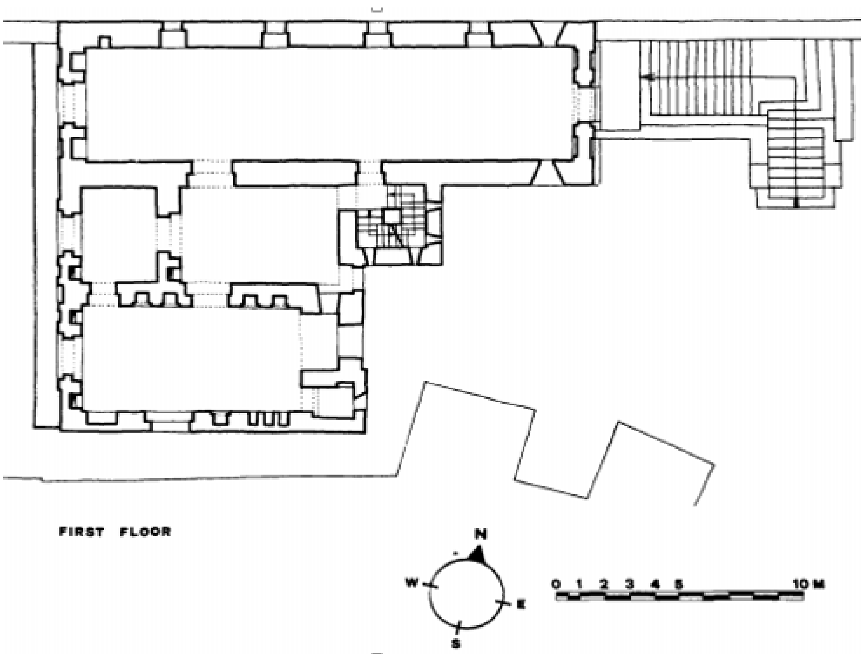

Figure 5.4 First floor plan, (Angelini 1971)

As an over-all view, the highest tip of the castle reaches about 17.75m. And excluding the staircase on the north eastern side, Figure 5.4, the castle roughly covers plan area of $264m^2$.

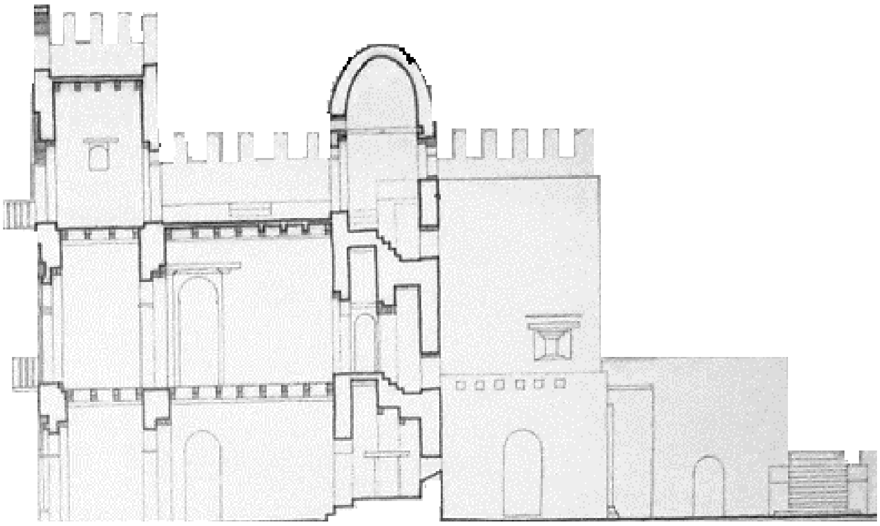

Figure 5.5 Section drawing, (Angelini 1971)

5.3.4 History of Interventions

The castle is kept in the existing condition with the series of restoration and strengthening works that were carried out in different times. And as stated earlier the knowledge of significant interventions is a plus in the study of historical structures. The major interventions that were undertaken and reported on (ECHP-A1 2007) are summarized as follows.

The earliest intervention work was done by the Italian troops in the late 1930s. At the time, they rebuilt many of the missing floor structures and roof with concrete ribbed slabs with the aim to replicate the original wooden beams supported floor system. Again in 1980s reconstruction of the floors in Mentwab Palace has took place. At this time the roof of the southern room was reconstructed by using vertically laminated timber beams. In the 1990s also reroofing of the northern upper room were carried out by replicating the traditional roof.

Concerning the wall, a number of repointing and resealing of cracks occurred in different times with different mixes. Where the strength and quality of the mixes seem relatively modest in which it does not affected the surrounding materials.

The other major intervention is the shoring that have been built by MH Engineering in 1990s. The shoring on the east and south facades of the castle are shown on Figure 5.6.

Although, the shoring system is placed to safeguard the building, its efficiency is reported as being highly doubtful.

Figure 5.6 Shoring on the east and south of the castle

5.4 Numerical Modeling

The numerical analysis was carried out on ANSYS 16.0. ANSYS is a comprehensive general-purpose finite element software which is a product of ANSYS Inc., an American Computer-aided engineering software developer formerly known as Swanson Analysis Systems Inc., SASI. The ANSYS program's capabilities have evolved according to emerging analysis needs, maturity of analysis methods and increased computing power. It is a pioneer in the discipline of nonlinear analysis and now it incorporates the necessary tools for a reliable nonlinear analysis (ANSYS Mechanical APDL Theory Reference 2013).

Using ANSYS the finite element model of the experimental test conducted on the historical masonry specimens was first analyzed. Then the analysis of Mentwab castle was run. For the two models the FEM parameters, boundary conditions, loading and other important considerations in the modeling and analysis are presented under the following subsections.

5.4.1 Simulation of the Experimental test

5.4.1.1 Shear specimen

The FE model of the shear specimens (lime-based masonry) were analyzed under different confinement levels similar to that of the experiments. To model the specimen and the plates that are used for the application of the loads, an element called SOLID185[3] was used. In addition, the contacting surfaces of the three layers of the masonry, see Figure 5.7 are connected by a contact element CONTA174[4] .

Then, the contact between the layers of the masonry model were defined as a frictional contact. Where the coefficient of friction was the input parameter given to the program. The material property parameters used in the modeling are presented in Table 5.1. Those parameters are taken from the experimental study and some are referred from literatures.

Table 5.1 Parameters of the FE model of the specimens

Material property		Reference
Unit weight, kN/m3	20	
Elastic modulus, MPa	40	
Poission’s ratio	0.2	(Betti, Galano and Virgnoli 2016)
Compressive strength, MPa	2.35	
Tensile strength, MPa	0.065	(Betti, Galano and Virgnoli 2016)
Cohesion, MPa	0.15	
Angle of internal friction	43.50	

Using the above material parameters 30x30x30 cm^3 FE model was formed and subjected to the boundary conditions and loadings that simulate the shear test. Then, similar to the experimental study a constant confinement was applied on the top and the shear load was given in a displacement based manner.

[3] SOLID185 is an element used for 3-D modeling of solid structures and the element has plasticity, stress stiffening, large deflection, and large strain capabilities.

[4] CONTA174 is used to represent contact and sliding between 3-D target surfaces and a deformable surface. In addition, the element allows the use of Coulomb friction model (ANSYS Mechanical APDL Theory Reference 2013)

The top and bottom layers of the specimen including the base were fixed. The fixity could be assigned either by a fixed support or compression only support. In the case of this model these two support mechanisms gave the same result.

As a result of the analysis shear deformation shown on Figure 5.7 was obtained. When the result of the FEM analyses was compared with that of the experimental test results, it was observed that except the lowest confinement level, the model resulted in acceptable values for the peak loads.

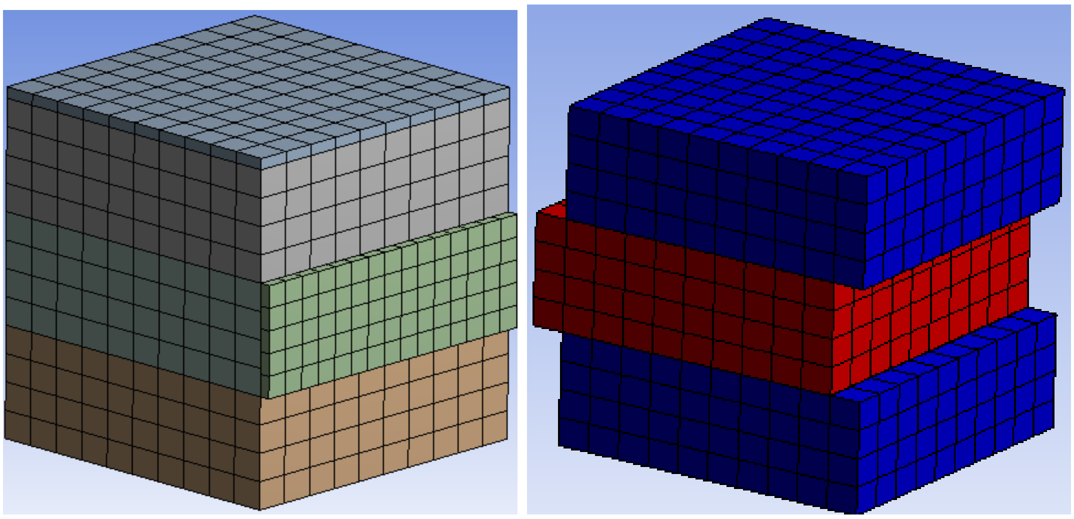

Figure 5.7 FE model of the shear test masonry specimen

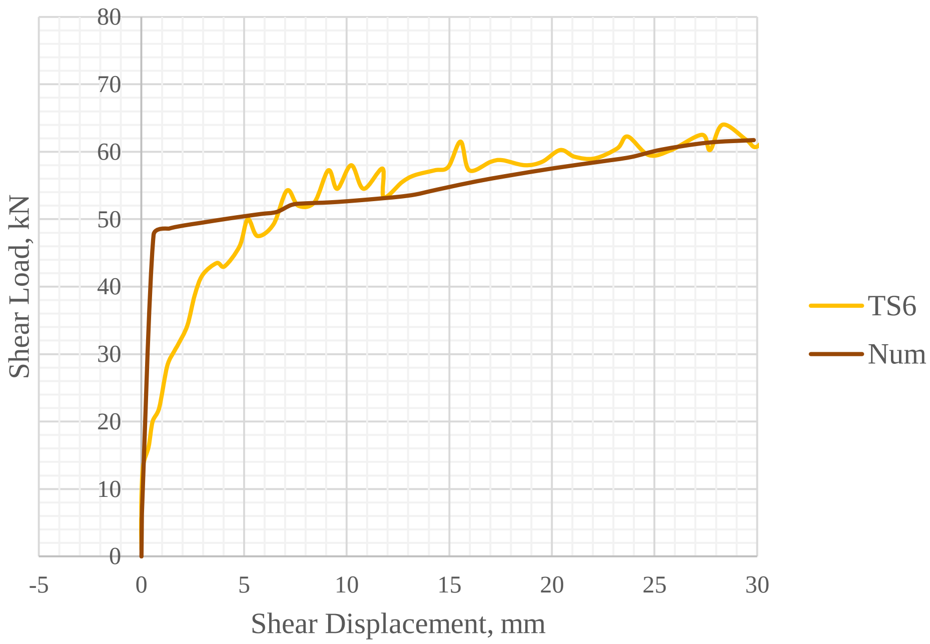

Figure 5.8 Comparison of FEM analysis and experimental result for shear

For the intermediated and higher level of confinement loads the resulted maximum shear load was almost equal to the results of the experimental study. Figure 5.8 shows the comparison between the experimental test result and the FEM analysis for the intermediate level of confinement, 0.3MPa.

However, it should also be noted that for the lowest confinement level, when the results of the experiment and FE model was compared, the FE model resulted in a highly reduced shear load. This is because the effort made to incorporate the cohesion parameter of the masonry sample, on the FE model was not successful.

Therefore, the application of this model for a large scale application, modeling of a building or other large structure, should be taken with caution, not to underestimate the shear capacity of a given structure at low confinement levels.

5.4.1.2 Compression specimen

The FE model that was used to simulate the shear test was again subjected to compression load only. Similarly, the load was applied in a displacement based approach.

As a result of the analysis conducted on the finite element model, the vertical load versus vertical displacement graph depicted in Figure 5.9 was obtained.

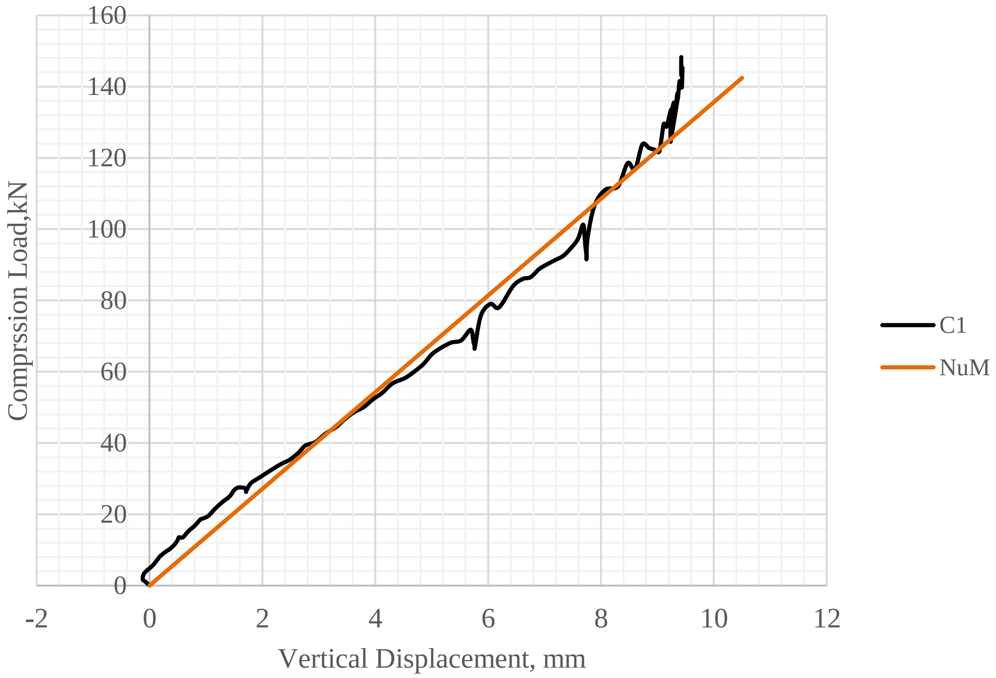

Figure 5.9 Comparison of FEM analysis and experimental result for compression

5.4.2 FEM Analysis of Mentwab Castle

The static behavior of the castle was analyzed under the vertical loads. After conducting several analyses to validate the finite element analysis, the complete 3D model of the castle was analyzed with the aim to understand the global response of the building.

In order to simplify the problem and to make the focus only on the masonry structural wall systems, the following assumptions and simplifications were considered in the modelling.

- Due to the lack of effectiveness in the connection between masonry and timber elements only the masonry elements were considered.
- No connections or continuity exists among the roof slabs covering different rooms of the building
- Roof and floor systems was not modelled explicitly, rather their self-weight and live loads were transferred to the supporting walls
- External staircases and landings are isolated from the main building, wall.
- Fence connected to the side of the wall is isolated assuming construction joint exists
- Timber roof beams are simply leant against the masonry wall without any effective restraint (Angelini 1971).

5.4.2.1 Modeling parameters

5.4.2.1.1 Geometry

For the FE analysis the 3D model of the castle was built on a CAD tool called SolidWorks[5]. Even though the whole detailed drawings of the building were not obtained, the photos collected and simple direct measurement conducted during the site visit had contributed to the develop the 3D model of the castle. After creating the volumes on SolidWorks, the masonry walls were assembled to form the 3D model of the castle which was then imported and meshed in ANSYS R16.0.

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[5] SolidWorks is a solid modeling Computer-aided design (CAD) and computer-aided engineering (CAE) computer program that runs on Microsoft Windows developed by Dassault Systèmes SolidWorks Corp.

5.4.2.1.2 Material Model

When it comes to the material model used for the castle, the model that was used to simulate the experimental tests could not be directly employed to the castle model. This is due to the fact that the model of the experimental tests incorporate masonry joint in a discrete manner, which means by defining the masonry joint as a frictional contact surface.

In the case of the castle incorporating the frictional surface as a masonry joint along the height of such a huge structure was impractical. In addition, it was not thought to be highly important for static analysis of the building under vertical loading condition. Therefore, the material model used for the castle was adopted from literature, (Betti, Galano and Vignoli 2016).

The authors have used Drucker-Prager (placticity criterion) in combination with William and Warnke (WW) failure criterion. Both DP and WW criterion have been used extensively in several scientific literatures to model inelastic behavior of masonry. For more detail on this material model interested readers are referred to publications of the above authors.

The parameters required for the above material model (DP and WW) are given in Table 5.2. Before appling the material model to the castle, it was tested on FEM masonry model of a size $30x30x30cm^3$. The result of the analysis have revealed that the material model simulates masonry as a brittle material with small amount of residual capacity after reaching the peak value.

Table 5.2 Parameters for material model of the castle

Drucker-Prager yield criterion		William and Warnke failure surface	
*Cohesion, N/mm^2	0.15	*Compressive strength, N/mm^2	2.35
*Friction angle	43.5^0	Tensile strength, N/mm^2	0.065
Flow angle	20	Closed shear transfer coeff.	0.75
		Open shear transfer coeff.	0.15
Other parameters			
*Unit weight, kN/m3	20		
Elastic modulus, MPa	700		
Poission’s ratio	0.2		

Note that the parameters with ‘*’ are those taken from the experimental study.

5.4.2.1.3 Meshing

ANSYS Workbench provides robust and easy to use meshing tools that simplify the mesh generation process. This tools have the benefit of being highly automated along with having a moderate to high degree of user control. Concerning the meshing of the castle, maximum mesh size was set to be equal to 0.3m and the MultiZone[6] meshing technique was employed. And an Element called SOLID65[7] was used.

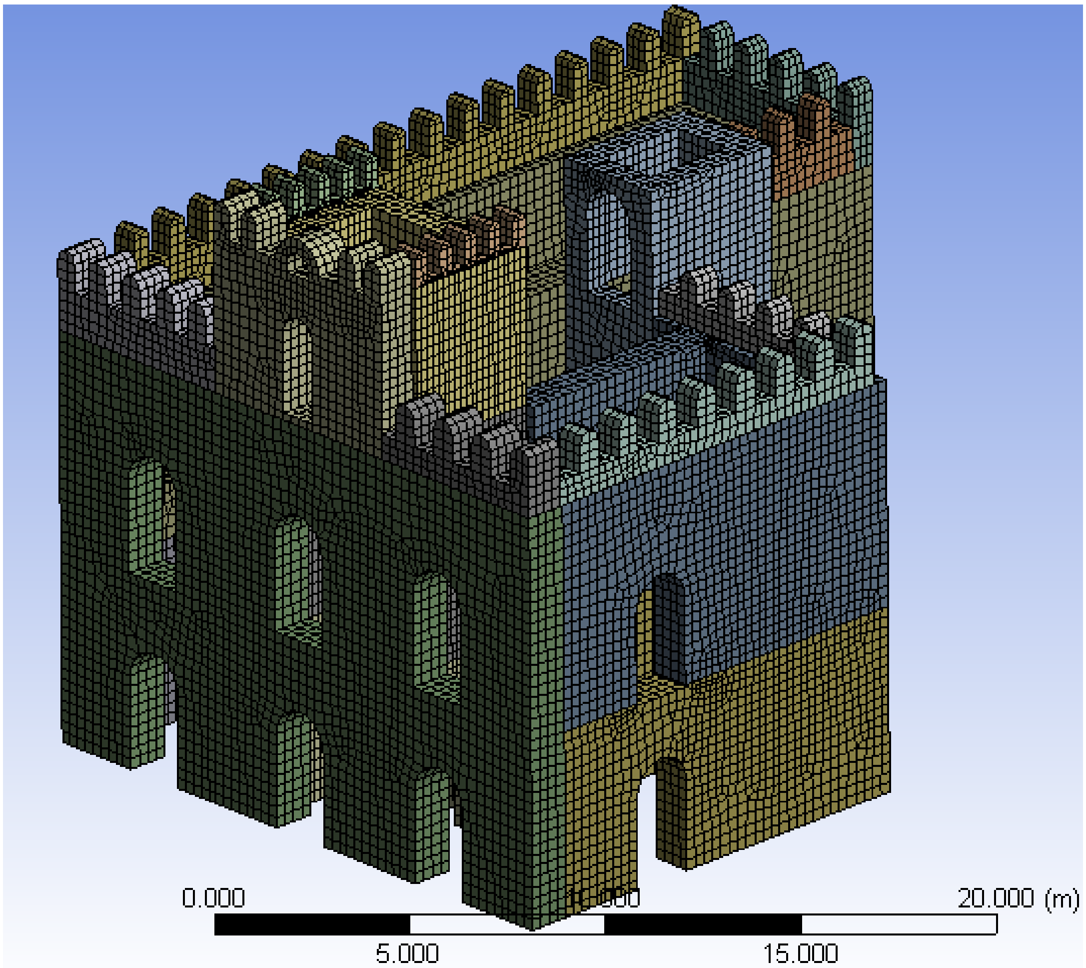

Figure 5.10 Finite element model of Mentwab castle

The quality of the mesh is one of the most critical issue in FEM modeling and a good mesh is essential for a good solution. In this regard, ANSYS provides mesh quality evaluating

[6]The Multizone mesh method is a patch independent meshing technique, which provides automatic decomposition of geometry into mapped (sweepable) regions and free regions. When the MultiZone mesh method is used, all regions are meshed with a pure hexahedral mesh if possible.

[7] SOLID65 is used for 3-D modeling of solids with or without reinforcing bars (rebar). The solid is capable of cracking in tension and crushing in compression.

methods. One of the methods is called Element quality[8]. With this quality checking mechanism the meshing of the castle were revised a number of times and finally the mesh was made to be as depicted on Figure 5.10

5.4.2.1.4 Loading and boundary conditions

In fact, historical structures experience various kinds of loads thorough out their life time. These loads could come from several environmental factors (manmade or natural). Regarding the consideration of such factors in the finite element model, all in one, would be impossible due to some limitations on the capability of the FEM tools, and other reasons.

As an opinion, it is a good practice to start from the simplest loading condition and understanding the response of the structure for that load. And then proceeding to the more complex condition. Thus, in this case study the castle was analyzed under vertical loads only. The vertical loads were derived from own weight of the walls and transferred loads from the roof and floor systems.

The calculation of loads, transferred from the roof and floor slabs, are presented in Appendix A and Appendix B. Concerning, the self-weight of the masonry walls, average unit weight was determined for the masonry specimens and the total weight of the building was calculated by the program, ANSYS. On the other hand, the foundation of the building was assumed to be highly rigid then model of the castle was mad to be fully fixed at the base. The final 3D model consists of 98869 nodes and 888389 elements. The following analysis parameters were used during the analysis; (i) initial analysis number of substeps: 20; (ii) maximum analysis number of substeps: 50; (v) minimum analysis number of substeps: 10.

Results of the static analyses on the 3D model of the castle in terms of vertical stresses are depicted in Figure 5.11- Figure 5.13. A relatively higher value of tensile stress appears on the surface of the battlements, Figure 5.11. This could be due to the connection between battements and underneath wall system.

[8] The Element Quality option provides a composite quality metric that ranges between 0 and 1. This metric is based on the ratio of the volume to the square root of the cube of the sum of the square of the edge lengths for 3D elements (ANSYS Mechanical APDL Theory Reference 2013).

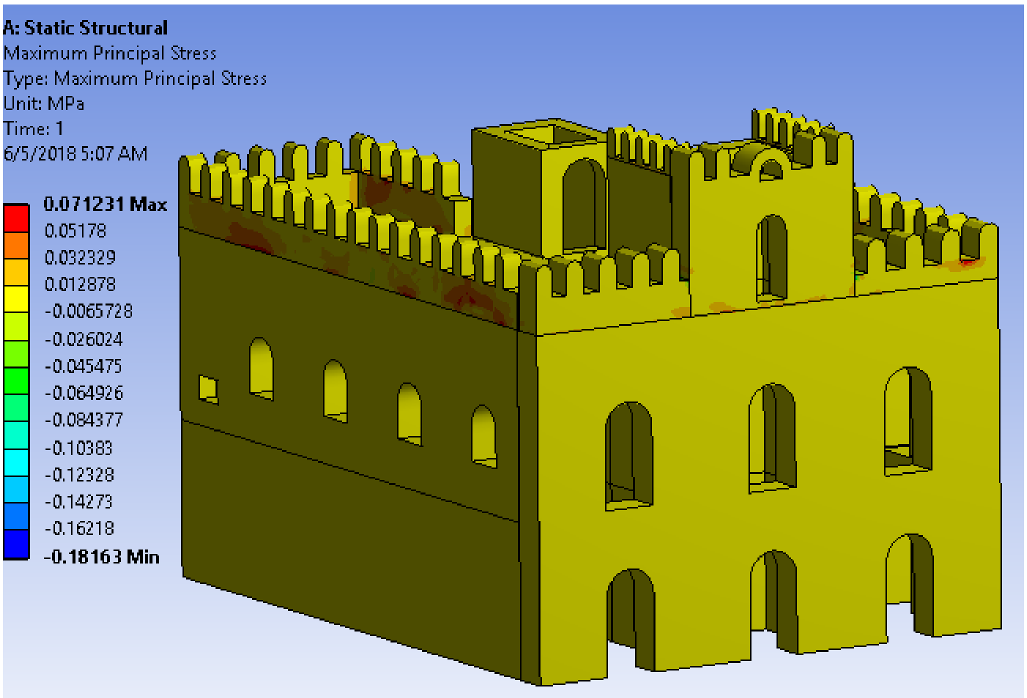

Figure 5.11 Maximum Principal stress contour on north west of the castle

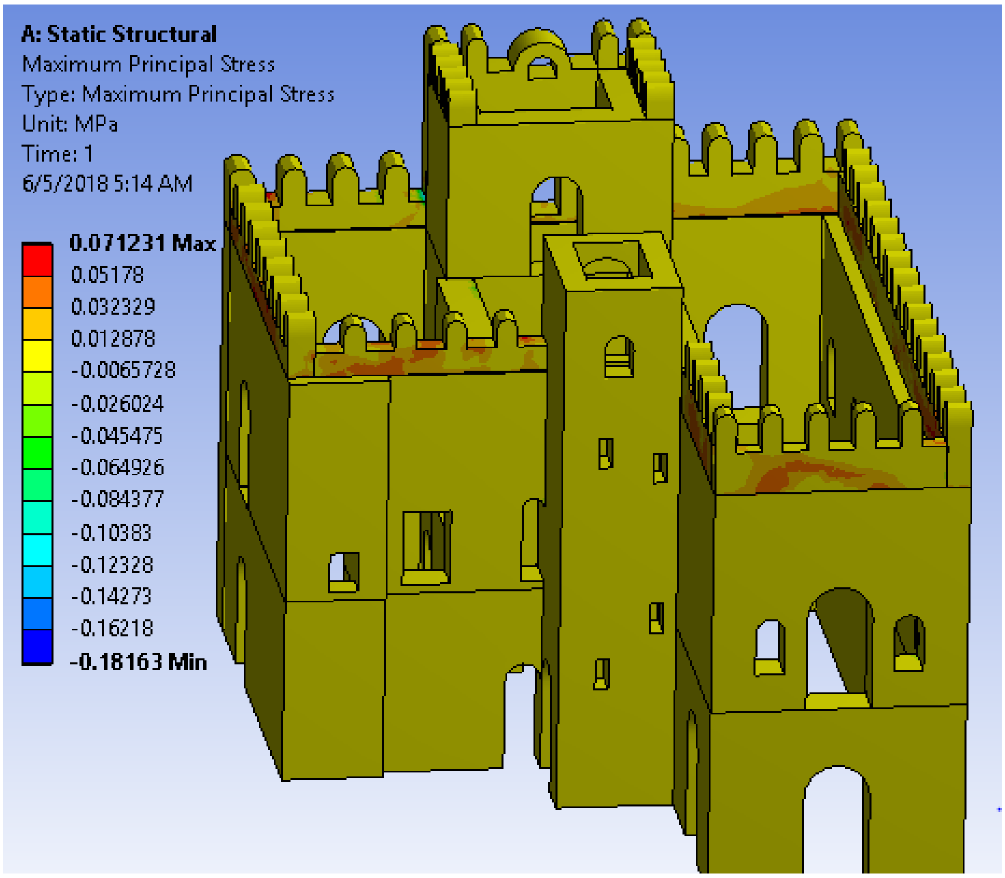

Figure 5.12 Maximum Principal stress contour on eastern side

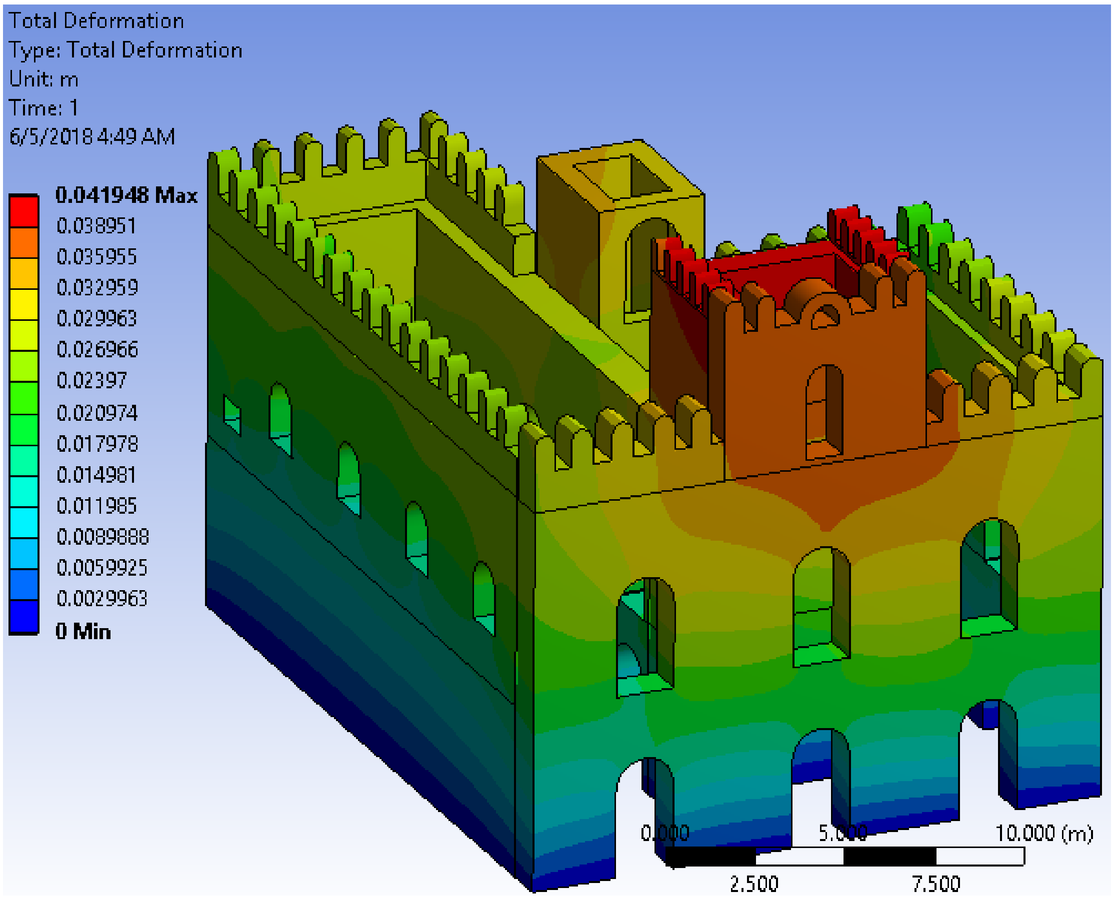

Figure 5.13 Total Deformation

The preliminary structural analysis conducted on the numerical model of the castle has resulted in the global response of the building under vertical loading condition.

As a result of the analysis, the building was found to be unaffected by the stress coming from the roof, floor and self-weight of the wall system. In general, the stress state induced on the castle by the static vertical loads is quite moderate. The average value of maximum principle stress is about 0.09MPa. In other words, the state of stress was found to be less than the capacity of the masonry tested for compression properties.

More importantly this effort, the preliminary FE model of the castle was found to be a step-forward in the study of the heritage buildings, using finite element models.

6. CONCLUSION AND RECOMMENDATION

6.1 Conclusion

In the study, with the aim to characterize the mechanical properties of the historical masonry walls of Gondar, an experimental program was undertaken by the use of a custom made laboratory test setups. The replicas of the historical masonry walls were tested for compression and shear properties.

In addition to the experimental study, finite element models were developed to simulate the laboratory tests and one historical masonry building from Fasil compound was studied by using numerical models. As a result of the experimental and numerical investigations the following conclusions were drawn.

- Most important and only available, mechanical properties defining parameters that are needed for numerical modeling (either for macro or micro modelling) of the historical masonry walls of Fasil Ghenbe were found as a result of the experimental campaign. From the shear tests shear strength parameters, cohesion and coefficient of friction, were obtained. And from the compression test average compressive strength and elastic modulus were found.
- The shear property of the masonry assemblage was found to be represented adequately by a simple friction model called Coulomb friction model. Where the shear strength was derived from a combination of bond shear strength and friction between the constituents (units and mortar). The resulting initial shear strength parameters, cohesion and coefficient of friction, are 0.15MPa and 0.76 respectively.
- The compression test allowed to evaluate the compressive strength and modulus of elasticity, E of the masonry mockups. The average compressive strength is found to be 2.35MPa. Whereas an average value of E is equal to 38MPa. Since the displacement measurement was highly sensitive to local damage on the top and bottom layers of the mortar, it is hard to firmly conclude that the masonry specimens are easily deformable or have a very small stiffness.
- From experimental campaign, particularly the shear test, it was observed that simultaneous application of a constant vertical confinement and an incremental lateral loads, on the two perpendicular faces of the specimens, in a fairly good and

controlled manner was made possible by using manually operated hydraulic jacks. This was found to be stimulating to think of more complex lab setups and conduct other tests on a large scale.

- The masonry assemblages, fabricated to investigate the behavior of the historical masonry walls of Fasil Ghenbe was found to be adequately representative, to examine the mechanistic behavior of the assemblages as well as to develop a numerical model. This was concluded based on the compliance among the construction materials and techniques used on the construction of the historical buildings. Additionally, the investigation made on the chemical composition of the mortar samples, from the heritages and masonry assemblages, strengthened this deduction. Therefore, this finding is found to be an indication for the reproducibility of the historical masonry assemblages for further experimental studies.
- As a pilot structural analysis, the resulted structural response of the castle showed the static structural response of the castle under vertical loads only, own weight and loads transferred from the roof and floor slabs. For this particular loading condition, the resulted stress levels were found to be below the capacity of the masonry.
- Most of all the numerical model of the castle can be used as a numerical laboratory to make further investigation on the structural behavior of the heritage.

6.2 Recommendation

- The experimental program held for this research have only studied some engineering properties of the historical masonry assemblages, with a limited number of specimens. And similar size of masonry specimens was used for compression as well as shear tests. For further studies, it is recommended to investigate the behaviors historical masonry walls on large-sized specimens. Because this can be beneficial to have a broad understanding on the failure mechanism and other characteristics of such structural systems.
- The contribution of advanced numerical analysis, in the design of effective restoration and rehabilitation methods should not be underestimated. Thus, the FE model of the castle should be refined and used as a numerical laboratory to comprehend the effect of various loading conditions (environmental factors including the foundation condition). And the FE model should be used to understand

the causes of the crack on the walls. Additionally, the effect of the steel structures (scaffoldings) on the stability of the building should be investigated.

- Historical constructions are a full of several uncertainties regarding the construction materials, geometric details, load history and other aspects. Therefore, from a point of view of engineering a more comprehensive experimental and desk studies are required in this field.
- Although the finite element models of the specimen as well as the building have given reasonable results for this study, modeling of masonry structures under different loading scenarios and with different levels of detail, either by macro modeling or micro modeling, need to be researched in detail.

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APPENDIX A

A. Load Transferred from the floor system

The geometry of the panels, number of ribs and rib spacing of each panel are given in Table A- 1. In addition the alignment of ribs is depicted on Figure A- 1. Regarding the loads, the dead load calculation accounts for the self-weight of the slab, lime plastering on the soffit and cement screed which have an assumed thickness of 2 mm.

In relation to live load, the castle is categorized under Category C3, EN 1991-1-1:2001. Where C3 is for areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. So an imposed load of 5kN/m^2 was used for the analysis.

Table A- 1 Geometric parameters of the floor slab

Slab Panels	Dimension, m^2	Number of ribs (N)	*Rib spacing, m
Panel-1	6.5 x 14	14	0.93
Panel-2	6.5 x 8.4	6	1.2
Panel-3	4.5 x 7.5	10	0.68
Panel-4	6 x 12.5	12	0.96
Panel-5	4.5 x 5	5	0.83

*Rib spacing was determined by counting the number of ribs of each panel, N and by dividing the length of the panel across the ribs by (N+1).

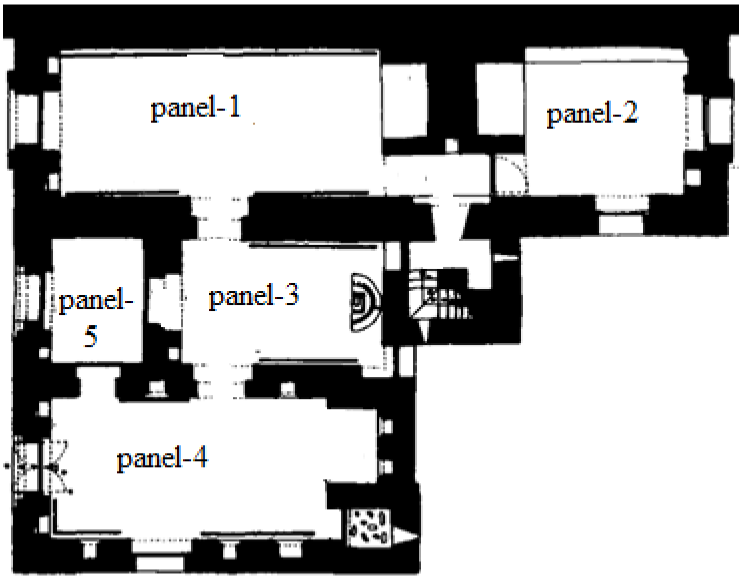

Figure A- 1 Ribs alignment (Angelini 1971)

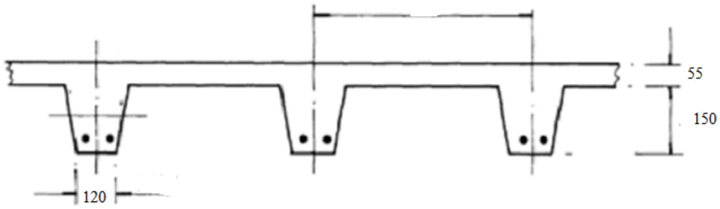

Figure A- 2 Cross section of the slab (Angelini 1971)

Loading

The dead load and live load calculation for each panel are summarized in Table A-2.

Dead load:

- Joist→ 0.12 x 0.15 x 24 = 0.432 kN/m
- Topping→ rib spacing x 0.055 x 24; (The thickness of the topping is calculated by deducting the assumed thickness of screed and plastering on the soffit)
- Cement Screed → rib spacing x 0.02 (assumed) x 23
- Lime Mortar Plastering on the ceiling → rib spacing x 0.02 (assumed) x18
- G_k = Joist + Topping + Screed + Plastering

Live load:

- $Q_k = 5kN/m^2$ x rib spacing

Table A-2 Loading on the ribs

Slab Panels	Rib spacing, m	Joist, kN/m	Topping, kN/m	Screed, kN/m	Plastering, kN/m	G_k, kN/m	Q_k, kN/m
Panel-1	0.93	0.432	1.23	0.43	0.33	2.42	4.65
Panel-2	1.2	0.432	1.58	0.55	0.43	3.00	6
Panel-3	0.68	0.432	0.90	0.31	0.24	1.89	3.4
Panel-4	0.96	0.432	1.27	0.44	0.35	2.49	4.8
Panel-5	0.83	0.432	1.10	0.38	0.30	2.21	4.15

Design load:

- Dead load = 1.35 x G_k
- Live load = 1.5 x Q_k

Floor Slab Analysis using SAP2000

The above slab panels were analyzed with the aid of computer models on SAP2000 software. The previous material and geometric parameters with the same loading condition were used to model and analyze each panel, see Figure A- 3. The number of ribs reported in Table A- 1 do not include the edge beams however, the models include those edge beams. As a result of the analysis, reaction forces tabulated in Table A- 3 were obtained.

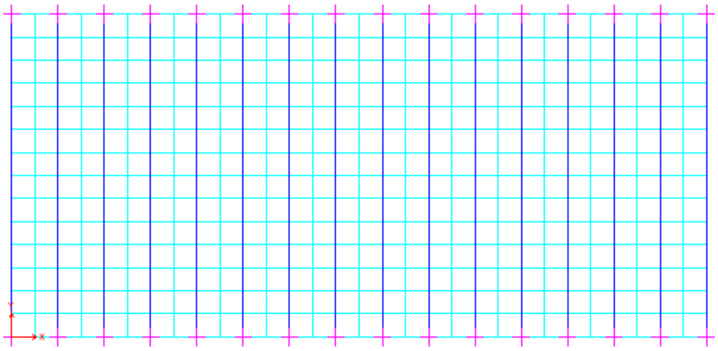

Figure A- 3 SAP2000 model of the Panel-1, plan view

The values given in Table A- 3 are the reaction forces of the joints along x-axis. Taking the advantage of symmetry, only half of the joint reactions are presented in the table.

All panels except Panel-5 have an even number of the ribs. So their line of symmetry falls on the slab. But for Panel-5, the line of symmetry is on the mid joint, Joint-4.

Table A- 3 Reaction Forces from SAP2000 Analysis

	Reaction forces, kN				
Joint no	Panel 1	Panel 2	Panel 3	Panel 4	Panel 5
1	21.98	28.26	11.02	22.08	13.66
2	30.95	38.54	15.32	28.40	19.05
3	32.04	40.81	16.16	30.32	20.10
4	32.92	41.78	16.66	31.18	20.45
5	33.26		16.86	31.56	
6	33.41		16.93	31.72	
7	33.47			31.77	
8	33.48				

Then these reaction forces were applied on the walls, as area load. When it comes to the interior walls, they carry the sum of loads that comes from the slabs on the two faces

Table A-4 Loads Transferred from Floor Slab

Slab Panels	Reaction (*R*), kN	Tributary length (*l*), m	Thickness of the wall (*s*), m	Load on Wall, kN/m2
Panel-1	33.30	1	1	33.30
Panel-2	42.41	1.4	1	30.29
Panel-3	17.21	0.75	1	22.95
Panel-4	31.67	1.04	1	30.45
Panel-5	20.71	1	1	20.71

APPENDIX B

B. Load Transferred from the roof systems

Similar to the floor slab panels, each roof slabs were analyzed independently. In the case of the roof there are four slab panels that were analyzed separately. Additionally, as it has been stated in chapter 5, the roof slab comprises two types of construction materials. Namely, concrete and traditional timber roof.

In the modelling of the panels on SAP2000, the material for the timber beams and stone slabs were defined as "*other material*". And the unit weights of both materials were found from secondary data, laboratory test reports. As per the reported results, the unit weight of timber (Tikure enchet) is 8.78kN/m^3 (AAU 2005) and that of the red volcanic stone is 14.9kN/m^3 (EGS 2005) .

In addition, the section for the beams which is 18x20 cm^2 was modelled on the "*section modeler*" of the software. On the other hand, the slabs which were made of concrete are assumed to have similar material property, thickness of topping and cross section of ribs as that of the floor slabs. However, there are additional materials that are laid on the roof slab of the middle rooms. These additional materials have not been investigated in detail, but from visual inspection the top of the roof is covered by similar tiles and it is assumed that those concrete roof slabs are also covered with damp proof course and lime mortar underneath the tiles.

The cross section of the timber made roof system is shown below on Figure B-1 and the geometric parameters of each roof slab is given in Table B- 1.

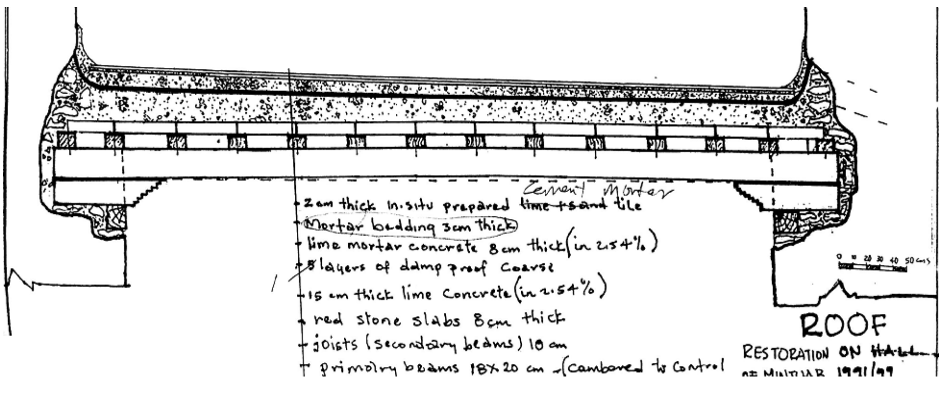

Figure B-1 Cross section of the traditional roof slab (Getahun 2000)

Table B- 1 Geometric parameters of roof slab

Roof Slab	Dimension, m^2	Number of ribs (N)	Spacing of ribs, m
Northern roof	6.5 x 19.7	29	0.68
Middle roof 1	4.5 x 7.5	8	0.94
Southern roof	6 x 12.5	20	0.63
Middle roof 2	4.5 x 5	5	1

Loading

Taking these materials into consideration, the dead load on the roof was calculated and presented in Figure B-2. As for the live load, the roof is categorized as H, which is for roofs not accessible except for normal maintenance and repair. Therefore, for the analysis an imposed load of 1 kN/m^2 was used, EN 1991-1-1:2001.

Table B- 2 Dead load on the roof

Roof Type	Dead Load on the Roof			
	Material	Thickness, m	Unit weight kN/m3	Dead load, kN/m2
Traditional Timber Roof	Cement Tile	0.02	23	0.46
	Cement screed	0.03	23	0.69
	Lime concrete	0.08	19	1.52
	Damp Proof	0.005	12	0.06
	Lime concrete	0.15	19	2.85
	Red Stone	0.08	14.9	1.192
	Secondary beams	0.1	9	0.9
	Primary beams	0.18	8.78	1.58
			G_{k1}	**8.06**
Concrete Roof	Topping	0.055	24	1.32
	Joist	0.15	24	3.6
	Cement Tile	0.02	23	0.46
	Cement screed	0.03	23	0.69
	Lime concrete	0.08	19	1.52
	Damp Proof	0.005	12	0.06
			G_{k2}	**7.65**

Table B- 3 Ultimate loading on the roof

Roof Slab	$1.35xG_k$	$1.5xQ_k$	Design load, kN/m^2
Northern roof	10.88	1.5	12.38
Southern roof	10.88	1.5	12.38
Middle roof 1	10.33	1.5	11.83
Middle roof 2	10.33	1.5	11.83

SAP2000 Model

The above material parameters and loading condition as well as geometric parameters given in Table B- 1 were used in the SAP2000 model. As an example the SAP2000 model and plan of the roof above the main hall is shown Figure B-2 and Figure B- 3. Then the analysis was conducted for ultimate loading condition, see Table B- 3.

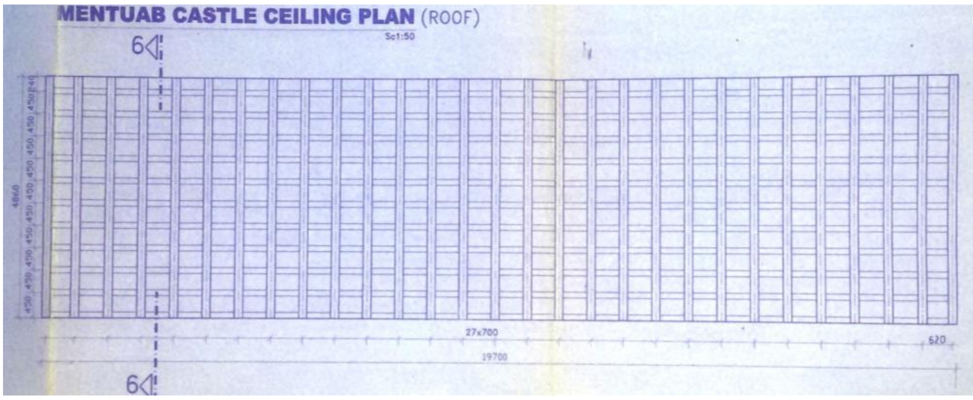

Figure B-2 Main hall ceiling plan

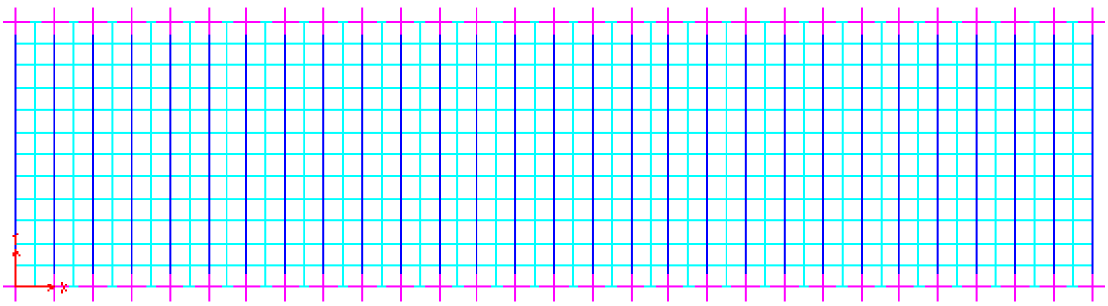

Figure B- 3 Main hall roof SAP2000 model

Load Transferred on the Wall

As a result of the analysis reaction forces tabulated in Table B-4 were obtained, then distributed over the supporting area of the wall and used as a load applied on the wall in the subsequent analysis of the wall system.

At this point it should be noted that the supporting area of the wall could be calculated and of the penetration of the slab into the wall from the drawings of (Getahun 2000). However, in order to avoid stress concentration on those supporting areas of the wall, the load transferred from the roof was distributed over the whole thickness of the wall which is equal to 1.1m. Then the calculated area and transferred loads are as given in the table below.

Table B- 4 Load transferred from roof

Roof Slab	Reaction Force, kN	Area, m^2	Loads on wall, kN/m^2
Northern roof	21.31	0.68x1.1	28.49
Middle roof 1	18.13	0.94x1.1	17.53
Southern roof	28.46	0.63x1.1	41.07
Middle roof 2	20.46	1x1.1	18.6

APPENDEX C

C. Check list for in-situ inspection

This check list is used to assess the condition of the castle, Mentwab Caste, during the in-situ inspection conducted in the course of the research. It is used to check and validate the collected secondary data concerning the geometric and physical condition of the structure. Additionally, it is used to acquire additional data and to fill the gaps due to missing drawings or documents.

1. Roof

- The constructional materials used on the exterior and interior of the roof

  ____

- Roof type

  ____

- Measure the planar dimension the top of the roof
  - Length __________________________
  - Width ___________________________
- Number of primary beams, with their type (concrete or traditional timber beams)
  - Northern roof ____
  - Middle roof 1 ____
  - Southern roof ____
  - Middle roof 2 ____

2. Floor

- Number of joists of the first floor slab in each room
  - Room 1 _________________________
  - Room 2 _________________________
  - Room 3 _________________________
  - Room 4 _________________________
  - Room 5 _________________________
- Type of construction material used for the first floor slab of each rooms

  ____

  ____

  ____

3. Wall

- Geometric detailing of southern face exterior wall
    - Number of openings ____
    - The dimension of the openings on a free hand sketch of the wall section
- Thickness of walls along the height of building
    - Interior
        - Ground floor ____
        - First floor ____
        - Second floor ____
    - Exterior
        - Ground floor ____
        - First floor ____
        - Second floor ____
- On the top of the wall, measure the dimensions of the battlements with the aid of freehand sketch of the battlements on different facades of the building
    - Height
    - Thickness
    - Spacing between battlements
    - Number of battlements on each facades
        - Northern façade ______________________________
        - Southern façade ______________________________
        - Eastern façade ______________________________
        - Western façade ______________________________
- Inspect existing structural damage, crack on the wall

  ____

  ____

  ____