INVESTIGATION OF MATERIAL PROPERTIES FOR THE TURKISH MASONRY BUILDINGS

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INVESTIGATION OF MATERIAL PROPERTIES FOR THE TURKISH MASONRY BUILDINGS

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ABSTRACT

INVESTIGATION OF MATERIAL PROPERTIES FOR THE TURKISH MASONRY BUILDINGS

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Unreinforced masonry construction is still widespread among many urban and rural areas, in developing countries such as Turkey. In order to prevent any loss of lives during an earthquake, ensuring the safety of these buildings is crucial. The buildings which are located in earthquake prone regions should be investigated and assessed so that necessary precautions can be taken. Therefore, several questions and concerns regarding the application of the assessment procedure, the assumptions and risk decision for the current methods of masonry building assessment are raised.

This thesis addresses these concerns and investigates the material characteristics of masonry buildings in Turkey and its effect on assessment methods. In this regard, ten buildings were selected for the study and detailed information were collected on the field regarding the selected buildings. After that, preliminary assessments were conducted on the selected buildings according to the method described in RBTEIE (2013) and the results were evaluated. Then, axial compression, diagonal tension and sliding shear tests were conducted in the laboratory, on the wall specimens obtained from the selected buildings. The test results were discussed along with the comparison with the strength values given in TEC (2007) and new material characteristics for masonry buildings were recommended accordingly. Finally, assessment methods described in TEC (2007) and RBTEIE (2013) were conducted to the selected buildings with different material characteristics for all the earthquake zones. The assessment results showed that, the assessments conducted with the allowable stress values provided by TEC (2007) might lead to inaccurate and unsafe results.

Keywords: Masonry, Material Properties, Assessment

ÖZ

TÜRKİYE'DEKİ YIĞMA BİNALARIN MALZEME ÖZELLİKLERİNİN İNCELENMESİ

Çobanoğlu, Adil Baran
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Donatısız yığma yapılar, Türkiye gibi gelişen ülkelerin kentsel ve kırsal alanlarında hala yaygın olarak kullanılmaktadır. Olası bir deprem durumunda can kaybını önlemek adına bu gibi binaların güvenliğini sağlamak çok önemlidir. Bu yüzden, ileri deprem bölgelerindeki binaların incelenip değerlendirilmesi ve gerekli önlemlerin alınması şarttır. Bu bağlamda binaların değerlendirilmesi ve değerlendirme sırasında yapılan varsayımlar hakkında çeşitli sorular ve endişeler ortaya çıkmaktadır.

Bu tez, belirtilen endişeleri ele alarak Türkiye'deki yığma yapıların malzeme özelliklerini inceleyip bu özelliklerin risk değerlendirme methodları üzerindeki etkisini araştırmaktadır. Bu kapsamda, çalışma için on bina seçilmiş ve binalar hakkında detaylı bilgi saha çalışmalarıyla toplanmıştır. Öncelikle bütün binalar için RBTEIE'de belirtilen yöntemle (2013) ön değerlendirme yapılmıştır ve sonuçlar irdelenmiştir. Daha sonra, laboratuar ortamında sahadan alınan numuneler üstünde eksenel basınç, diyagonal çekme ve kayma deneyleri yapılmıştır. Deney sonuçları, DBYBHY'de (2007) verilen dayanım değerleriyle karşılaştırılarak incelenmiş ve yeni malzeme özellikleri önerilmiştir. Son olarak, seçilen binaların farklı malzeme özellikleri ve farklı deprem bölgeleri için DBYBHY (2007) ve RBTEIE'de (2013) belirtilen yöntemle detaylı değerlendirmeleri yapılmıştır. Değerlendirme sonuçları, DBYBHY'de (2007) verilen malzeme özellikleri kullanıldığında hatalı ve güvenli olmayan sonuçların ortaya çıkabilieceğini göstermektedir.

Anahtar Kelimeler: Yığma Bina, Malzeme Özellikleri, Risk Değerlendirmesi

To My Family

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CHAPTER 1

INTRODUCTION

1.1 Introduction

From the beginning of the civilized human life, one of the most basic needs had been sheltering. It had been essential for people in terms of protection from both the wild life and the nature. Being the only construction alternative at the time, masonry construction had been the easiest way to fulfill the need. From simple cottages to complex structures, the use of unreinforced masonry governed the construction industry through centuries.

Contemporarily, unreinforced masonry (URM) construction is still widespread among urban and rural areas. Due to the reasons such as the availability and durability of materials, low cost and maintenance, reasonable insulation performance and most importantly, constructing with little engineering knowledge enabled masonry construction to maintain its popularity, especially for residential buildings (Hendry, 2001). Most of the residential building stock is composed of masonry structures all over the world. In developing countries such as Turkey, the amount of masonry construction was quite high in the midst of 20th century and started decreasing towards the end of the century. A report prepared by the Housing Development Administration of Turkey states that in urban areas, 48% of all buildings are brick masonry or timber framed, while 30% of them are reinforced concrete frame type, and 22% are made from adobe or rubble masonry. In rural areas the rate of any kind of masonry buildings for housing increases to 82% (Erdik and Aydınoğlu, 2002).

Although masonry construction is widespread among many areas, it comes with several drawbacks. According to Arya et.al, (1986) most of the masonry construction is conducted without any engineering knowledge and guidance. Since more than 90% of the population in the Middle East is still living and working on such buildings that are in the moderate or severe seismic zones as exhibited by the past experience most of the live losses occurred due to the collapse of such buildings during earthquakes such as Elazığ (Turkey, 2010), Bam (Iran, 2003) and Kashmir (Pakistan, 2005) earthquakes. The rising population in the developing countries along with deficiency of traditional building materials and construction techniques, lack of awareness and necessary skills, are the main causes of the risk to the human life especially in developing countries.

In this retrospect, ensuring the safety of these buildings during an earthquake is crucial in order to prevent any loss of lives. The buildings that are located in earthquake prone regions, should be investigated and assessed so that necessary precautions can be taken beforehand. Depending on the assessment results, the buildings that are under high earthquake risk should either be strengthened to resist expected earthquakes in that region or renewed completely.

After the devastating earthquakes that resulted with the loss of thousands of lives in Turkey, the importance given to the assessment and renewal of existing masonry buildings increased dramatically. The implementation of new codes Turkish Earthquake Code (TEC, 2007) and Guidelines for the Assessment of Buildings under High Risk (RBTEIE, 2013) as per Law no. 6306 gave a better understanding for the assessment of existing structures. In these documents, the assessment techniques for existing reinforced concrete buildings are provided in details whereas the assessment of masonry buildings are conducted with rather less accurate and more primitive methods. The assessment process often results with the renewal of the existing buildings and urban renewal projects became widespread along the country due to the economical benefits rather than the consciousness of reducing seismic risk, which in turn is the achieved objective. The popularity of the urban renewal projects, increased the importance of the assessment methods, while raising several questions and concerns regarding the application of the procedure, the assumptions and risk decision for the current methods of masonry building assessment.

One of the main concerns about the assessment procedure for an existing masonry building is the determination of the material properties while assessing the capacity of the walls. A better understanding of the material characteristics of masonry is essential since it may lead to more accurate assessment results. At this point, TEC (2007) provides allowable stress values to be used if the capacity values cannot be obtained through experiments conducted on the existing building. TEC (2007) recommended values which depend on the masonry type. Unit compressive strength and mortar class are specified, while the allowable compressive and shear strength values of walls are also presented depending on the masonry unit type. Currently, those tabularized allowable stress values are used as the material strength values in the assessment of all of the existing masonry buildings. This situation obviously calls for a detailed investigation of the accuracy and safety of the reported material strength values.

1.2 Literature Review

Many researchers studied masonry material characteristics, their correlation to each other and factors affecting these properties. The easiest method to determine the material characteristics of masonry is to perform appropriate laboratory tests on specimens constructed with the exact same ingredients that are used on the building site. Although this method provides reliable test results, the accuracy and possibility of finding such specimens are questionable for existing masonry buildings. Therefore, conducting experiments on specimens obtained from actual buildings provide a better understanding of the material properties of existing buildings. There are numerous publications on testing of masonry walls to obtain material strength along with many numerical simulation results with various levels of sophistication. A brief review of the literature conducted on both laboratory and in-situ material tests of masonry walls are discussed in the following paragraphs.

Several codes implement methods to determine the material characteristics of the masonry walls. For the determination of the compressive strength, FEMA 356 (2000) and Eurocode 6 (2005) recommends to use one of the following methods: i)testing of prisms that are extracted from the existing walls, ii) testing of prisms that are fabricated from actual extracted masonry units or iii) estimating the compressive strength using flatjack compression tests. If none of the tests are available, FEMA 356 (2000) provides default lower bound compressive strength values varying between 2.1 MPa and 6.2 MPa depending on the quality of the masonry condition, while Eurocode 6 (2005) gives equations based on the masonry unit and mortar to calculate the compressive strength of masonry. In order to determine the shear strength of masonry walls, both FEMA 356 (2000) and Eurocode 6 (2005) recommends to use an in-situ shear test. If the test is not applicable, FEMA 356 (2000) provides default lower bound shear strength values varying between 0.09 MPa and and 0.16 MPa depending on the observed masonry condition. On the other hand, Eurocode 6 provides shear strength values depending on masonry unit and mortar class that varies from 0.10 MPa to 0.30 MPa.

Gumaste et al., (2006) investigated the properties of brick masonry using 2 types of bricks from India with various types of mortars. The strength and elastic modulus of brick masonry under compression were evaluated. In order to observe the size effect and different bonding arrangements, various sizes of prisms and wallettes were tested and the failure mechanisms were observed. Combination of four types of masonry specimens using table moulded bricks and wire-cut bricks with three different types of mortar were tested. The compressive strength values varied between 0.67 MPa and 3.18 MPa for the moulded brick masonry while the compressive strength varied between 5.3 MPa and 14.9 MPa for the wire-cut brick masonry. For the first type of specimens the modulus of elasticity varied between 260 and 735 MPa. On the other hand, the modulus of elasticity values were observed in a range of 2393-5232 MPa. Different failure modes were observed between different types of specimens. Empirical relationships for masonry strength depending on brick and mortar strength were derived as the main result of the study.

Russell (2010), studied the characterization and seismic assessment of unreinforced masonry buildings in order to develop a better understanding of the response of URM buildings. During the study, Russell (2010), also investigated the diagonal tension (shear) strength of unreinforced masonry having different mortar properties and bond patterns. A total of nine diagonal tension tests were conducted and the results showed that the capacities were between 0.04 MPa and 0.5 MPa. Even when the mortar type and other variables were kept constant, the results indicated a significant variation in the diagonal tension strength of the wall panels. The modulus of elasticity values were also reported and they varied from 1960 MPa to 2560 MPa. These experiment results along with some other experiments were used to develop a procedure for assessing the performance of unreinforced masonry buildings.

Sarangapani et al., (2005) examined the influence of bond strength on masonry compressive strength with an experimental research using local bricks and mortars. 14 different combinations of prisms were prepared and tested in the laboratory to determine the masonry compressive strength when the brick-mortar bond strength was varied over a wide range without changing the strength and deformation characteristics of the brick and mortar. The compressive strength results varied from 2.15 MPa to 5.24 MPa. Flexure bond strength and shear bond strength tests were also conducted to determine the brick-mortar bond strength. Triplet shear tests were made to obtain the shear bond strength and the results were between 0.054 MPa and 0.265 MPa for different types of bond enhancers. A relationship between the masonry prism compressive strength and bond strength was obtained and it was concluded that an increase in bond strength, while keeping the mortar strength constant, led to an increase in the compressive strength of masonry.

Venkatarama Reddy et al., (2007) studied the methods of improving the shear-bond strength of soil-cement block masonry and the influence of shear-bond strength on masonry compressive strength. An experimental study was conducted on the laboratory made specimens. Specimens were prepared for a combination of two different types of mortar and six different types of bond enhancing technique for both compression and triplet shear tests. Compressive strength results varied from 2.75 MPa to 4.08 MPa while shear strength values were between 0.03 MPa and 0.25 MPa. Venkatarama Reddy et al., (2007) concluded that no significant changes were determined for the compressive strength and stress-strain characteristics of soil-cement block masonry due to changes in shear-bond strength and masonry had a higher strain capacity than the block and the mortar.

Tomazevic (2009) investigated the shear failure mechanism of walls, characterized by the formation of diagonal cracks, along with the sliding shear failure mechanism. A series of laboratory tests were conducted to observe both failure mechanisms separately. Five different types of masonry units were used to prepare different specimens for both diagonal tension testing and triplet shear tests. The tensile strength of the specimens varied from 0.17 MPa to 0.22 MPa. On the other hand, shear strength values were between 0.16 MPa and 0.28 MPa. The results were analyzed to point that in the case of the diagonal tension failure, the results of the Eurocode 6 based calculations did not comply with the actual resistance of masonry walls. The results of analysis were concluded as more realistic, where the diagonal tension mechanism and tensile strength of masonry were considered as the critical parameters. Due to the reason that the results, which were based on the Eurocode 6 assumed sliding shear mechanism, were not in favor of structural safety, it was recommended that the diagonal tension shear mechanism should also be considered, besides sliding shear.

Calvi et al. (1996) expressed that the availability of analytical models which can relate the properties of materials to the properties of the masonry walls, determines the usefulness of results from individual material tests. Despite much progress has been made in the field of fundamental models recently, it appears that the confidence in the general reliability of these methods is not yet enough. Andreini et. al, (2013) mentioned that the only method to determine reliable mechanical characteristics values seems to be the testing on the masonry as a composite, since these characteristics can almost never be directly correlated to those of the components, except for a few combinations of mortar and masonry units. Therefore, in-situ material tests present a better understanding for the mechanical properties of an existing building.

Xie et al., (2014) investigated an in-situ axial compression test method on brick masonry. Two walls of an existing building were tested in order to determine their compressive strength. After the test had been conducted on a number of points for each of the walls, the obtained data were analyzed and compressive strengths were calculated as 3.76 MPa and 4.15 MPa respectively. The test results showed that, the compression strength of masonry were much larger than the standard strength and Xie et al., (2014) concluded that the accuracy of this method needs further verification.

Maheri and Sherafati (2012) studied the effects of environmental conditions, particularly humidity and temperature, on the shear strength of brick walls. Factors such as material type and age of the building were also evaluated. After the devastating Bam earthquake in 2003, a very large scale project of seismic retrofitting for existing school buildings was introduced in Iran and an extensive amount of field tests had become available for the shear strength of brick walls of existing buildings from different parts of the country. In-situ tests were conducted on all the buildings by removing the neighboring bricks near the selected test brick and inserting a hydraulic jack to load the brick horizontally. The results for 4 buildings were presented in the study. Eight walls were tested in each building and the mean shear strength values for the four buildings were 0.72 MPa, 0.47 MPa, 0.26 MPa and 0.65 MPa respectively. However, the individual test results varied from 0.13 MPa to 1.23 MPa. Results for all the buildings showed that the shear strength increased with the increasing humidity level while the absorption rate of bricks and the daily temperature of the location of the building may also have some effect on the shear strength.

Brignola et al., (2008) examined the mechanical interpretation of the in-situ diagonal compression test on masonry panels. An experimental campaign was conducted on 24 masonry panels in Italy. Out of 24 panels, 3 of them were solid brick masonry, two were hollow brick masonry and one of them was a concrete block masonry while the rest of the specimens were several types of stone masonry. Diagonal tension tests were also conducted for each type of panel. The tensile strength of solid brick masonry values varied from 0.13 MPa to 0.26 MPa. On the other hand, the tensile strength of hollow brick masonry panels were 0.34 and 0.30 MPa, while concrete block masonry tensile strength was obtained as 0.411 MPa. The behavior of different masonry typologies were simulated and numerical interpretation of the tests were presented. Through a non-linear numerical modeling, a methodology was also proposed for the evaluation of the tensile strength and the shear modulus of masonry.

Lumantarna et al., (2014) investigated the material properties of unreinforced clay brick masonry buildings in New Zealand, which were built between 1880 and 1930 by means of in-situ testing. Samples were extracted from six buildings and compression, bond wrench and shear bond tests were conducted. Compressive mean strength values were varied from 3.3 MPa to 14.7 MPa depending on the building. Shear strength of the samples were obtained by triplet shear tests with axial pre-compression loads. Depending on the building and axial load on the specimen, the shear strength results varied from 0.15 MPa to 1.12 MPa. The experimental results indicated that the mortar bed-joint shear strength increased with increasing axial compressive load and the mortar bed-joint cohesion is better characterized using the mortar compressive strength than using the masonry compressive strength.

The mechanical characteristics of masonry is extremely variable since it is dependent on many factors such as temperature, humidity, spatial variability, construction techniques workmanship etc. as it can be clearly seen in the literature. There are many numerical/analytical models which try to define these characteristics in terms of masonry units and mortar type generally. Nevertheless, their reliability are questionable when existing buildings are considered. In order to conduct an accurate assessment of an existing building, having a clear understanding of the material characteristics is essential and the most appropriate way to determine these are test results conducted on actual materials.

1.3 Objective and Scope

Considering the above discussion, a research project was initiated including several faculty members of Structural Mechanics Division of Civil Engineering Department of METU with the support of Ministry of Environment and Urbanization in order to investigate the assessment method and criteria for masonry buildings. The project is composed of several parts which can be summarized as follows: 1-Selection of ten buildings to be used throughout the project, 2- Determination of the material characteristics for the selected buildings, 3-Linear elastic assessment of selected buildings and conducting nonlinear analyses on these buildings to estimate the seismic performance, 4- Pushover experiments on two of the selected buildings, 5-Comparison of the existing assessment methods with the ones in the literature and 6- Development of a more reliable assessment method.

This thesis focuses on the first two phases and partly the third phase of the research project which consists of the building selection process, determination of the material characteristics for each building and assessment of the buildings with the recent codes. This study aims to:

1. Determine the typical material characteristics for the selected masonry buildings by conducting laboratory tests and compare them with the strength values provided by TEC (2007),

2. Obtain a better feeling of the preliminary assessment method and the calculated performance scores described in RBTEIE (2013) for masonry buildings,

3. Recommend material characteristics for the assessment of masonry buildings

4. Conduct risk assessment according to the rules of existing guidelines by using different approaches for material strength and identify the impact of assumed material strength for building assessment.

This study is composed of four chapters. In the first chapter, brief background about the unreinforced masonry construction and its use in Turkey is given. The importance given to assessment and renewal projects in recent years is presented and one of the main concerns regarding the assessment process is revealed: determination of the material characteristics of an existing masonry building and its effects on the assessment results. A literature review about the laboratory and in-situ tests of masonry walls is presented. Finally, the objective and scope of the study is given.

In chapter 2, the building selection process and detailed information regarding the selected buildings is given. In addition to this, the preliminary assessment method described in RBTEIE (2013) is conducted for the selected buildings. The results are comparatively evaluated.

Chapter 3, deals with the laboratory experiments conducted on the wall specimens gathered from the selected buildings. Firstly, extraction process of wall specimens from the selected buildings is described. Then, axial compression test procedure and the results of the experiments are presented. Diagonal tension tests and their results are given subsequently. In addition to these, shear test procedure and the results are also presented. Finally, all the test results are discussed along with the comparison with the strength values given in TEC (2007) and recommendations were made accordingly.

In chapter 4, the assessment procedures, described in TEC (2007) and RBTEIE (2013), are presented. The procedures were applied to 10 selected buildings both with the recommended strength values in TEC (2007) and the actual material characteristics obtained from the tests for all the earthquake zones. The results were compared and the importance of material characteristics on the assessment methods is discussed.

CHAPTER 2

BUILDING INFORMATION

In this chapter, building selection for the research, detailed information about the buildings and the preliminary assessments conducted on these buildings are presented. Ten buildings, nine of which were located in Altındağ, Çubuk, Çankaya, Gölbaşı, and Mamak districts of Ankara and one located in Kırşehir, were investigated during the course of this study.

2.1 Building Selection

The investigated buildings were provided by the Ministry of Environment and Urbanization. As a part of the urban renewal project, these buildings had been evaluated by the licensed companies and later approved by officials to be under high earthquake risk.

Ten buildings were selected for this research out of the building inventory in Ankara. The buildings were selected according to several criteria. Firstly, the buildings were investigated on the field and confirmed that no residents were living in it. The structural system of the building is determined to check whether it was an unreinforced masonry building and the masonry unit type was also determined. Then, the building was inspected for any existing damage which can cause dangerous damage during the specimen removal process. After that, it was checked whether the building had sufficient number of walls from which the specimens could be taken from. Since transportation of the removed wall specimens to the vehicle was very challenging, the proximity of the building to the loading area was also considered. The soil characteristics were not determined on the field, but they were obtained from the investigation report submitted to the ministry. If the buildings were qualified as eligible according to these criteria, necessary permissions were taken from the ministry to conduct the gathering of the specimens regarding the building. The general information about the buildings are summarized in Table 2.1.

Table 2.1 The general information about the buildings

Building ID	Masonry Unit Type	Number of stories	Story Height (m)	Building dimensions (m x m)
1	Hollow clay brick	2	2,8	8,9 x 14,0
2	Hollow clay brick	2	2,72	7,3 x 12,5
3	Solid clay brick	3	2,72	10,5 x 18,0
4	Solid clay brick	3	2,85	14,5 x 16,2
5	Cellular concrete block and adobe	1	2,36	9,0 x 9,1
6	Solid concrete brick	2	3	9,0 x 10,7
7	Solid clay brick	2	2,72	10,1 x 12,1
8	Hollow clay brick	2	2,73	10,5 x 16,9
9	Hollow clay brick	3	3	9,2 x 12,3
10	Solid clay brick	3	2,75	11,9 x 21,7

The locations of the buildings are shown on the map of Ankara along with the earthquake zones in Figure 2.1.

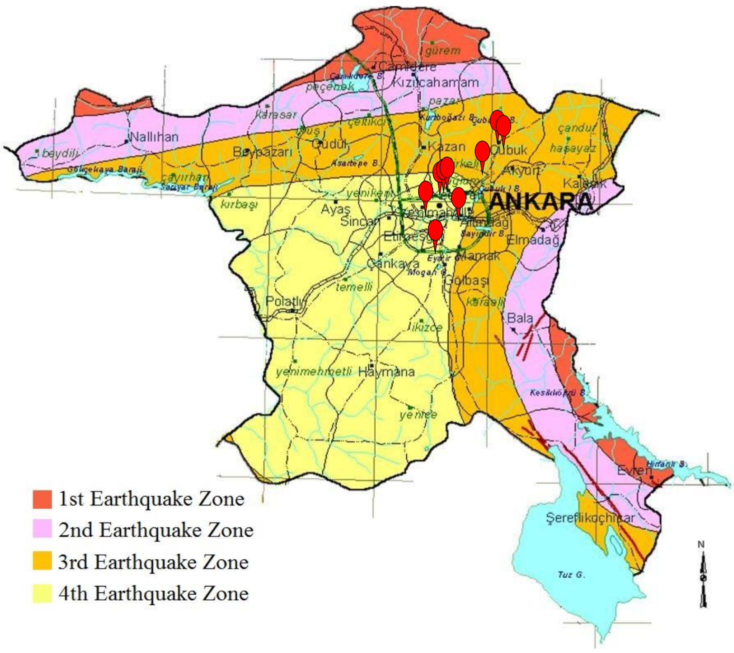
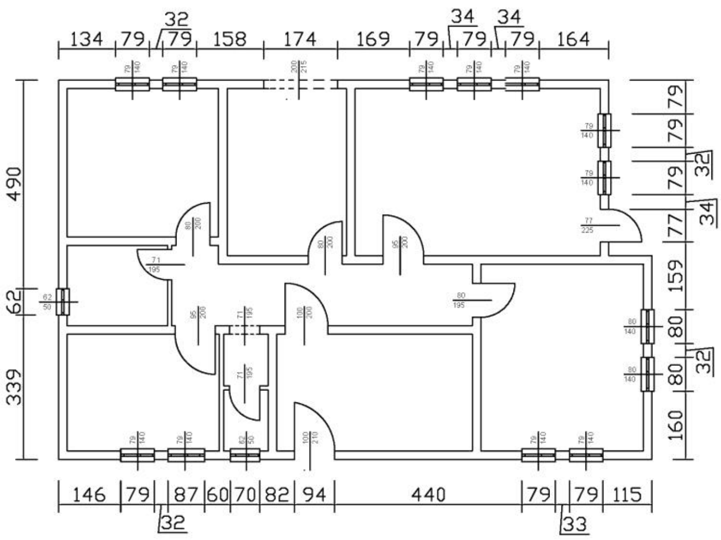


Figure 2.1 Locations of the buildings

2.2 Detailed Building Information

Building 1 was located in Çubuk/Ankara. It was a two story structure constructed with hollow clay bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 1 is shown in Figure 2.2. Table 2.2 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.2 Building 1 Information

Table 2.2 Detailed information about Building 1

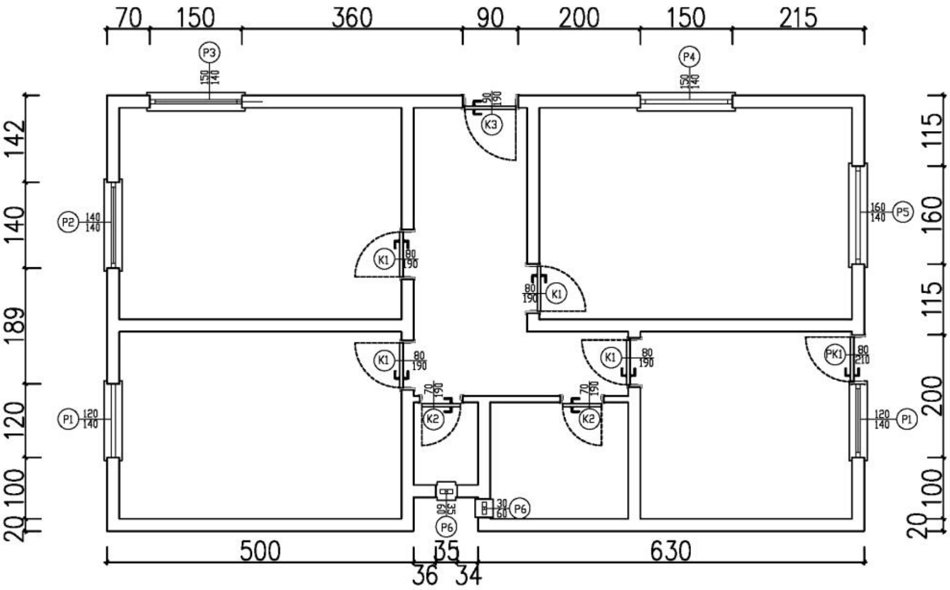
Address		Esenboğa District, Atatürk Road, Şeyh Şamil Street, No:2 ÇUBUK – ANKARA
Lot and block ID		550-8
Construction year		Unknown
Structural system		Unreinforced masonry
Type of masonry unit		Hollow clay brick
Void ratio of the masonry unit		0.49
Earthquake zone		3
Soil class		C
Number of stories		2
Story height (m)		2.80
Basement floor		None
Plan geometry		Regular
Building dimensions (m x m)		8.9 x 14.0
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.2
Wall ratio*	**x-direction (%)**	29.4
	y-direction (%)	24.5
Slab thickness (cm)		13
Average plaster thickness (cm)**		3.5
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 2 was located in Gölbaşı/Ankara. It was a two story structure constructed with hollow clay bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 2 is shown in Figure 2.3. Table 2.3 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.3 Building 2 information

Table 2.3 Detailed information about Building 2

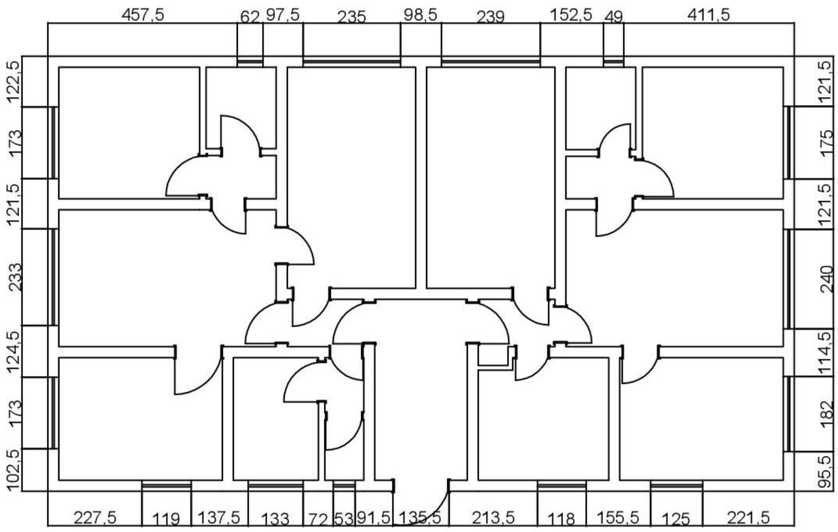
Address		Seğmenler District, Seğmenler Road, No:74 GÖLBAŞI – ANKARA
Lot and block ID		83-6
Construction year		1990
Structural system		Unreinforced masonry
Type of masonry unit		Hollow clay brick
Void ratio of the masonry unit		0.53
Earthquake zone		4
Soil class		D-Z4
Number of stories		2
Story height (m)		2.72
Basement floor		None
Plan geometry		Regular
Building dimensions (m x m)		7.3 x 12.5
Structural arrengement		Contiguous
Height difference between contiguous buildings (if exists)		Exists
Slab alignment difference between between the contiguous buildings (if exists)		Exists
Typical wall thickness (m)		0.23
Wall ratio*	**x-direction (%)**	34.5
	y-direction (%)	21.0
Slab thickness (cm)		12
Average plaster thickness (cm)**		4.6
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		Cracks in various locations

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 3 was located in Aydınlıkevler/Ankara. It was a three story structure constructed with solid clay bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 3 is shown in Figure 2.4. Table 2.4 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.4 Building 3 information

Table 2.4 Detailed information about Building 3

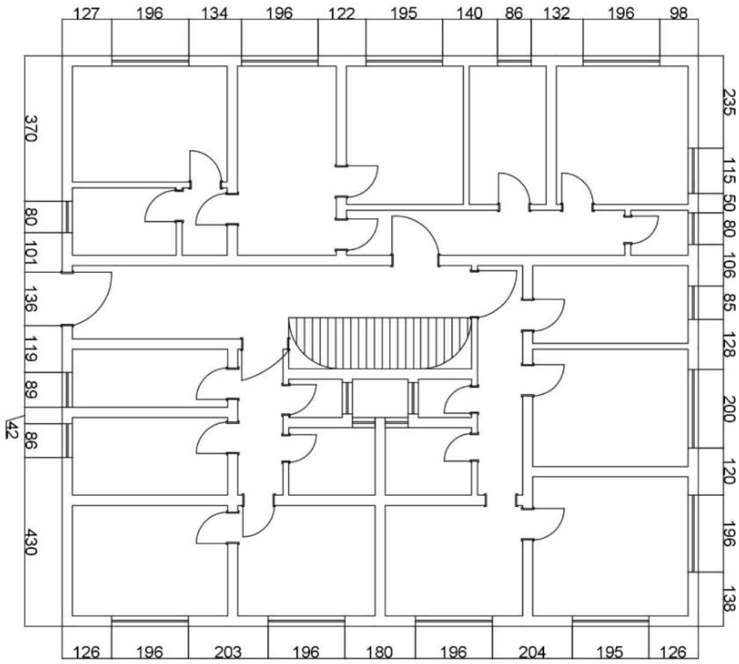
Address		Aydınlıkevler District, Çember Street, No:20 ALTINDAĞ - ANKARA
Lot and block ID		2736-10
Construction year		1950
Structural system		Unreinforced masonry
Type of masonry unit		Solid clay brick
Void ratio of the masonry unit		-
Earthquake zone		4
Soil class		
Number of stories		3
Story height (m)		2.72
Basement floor		Exists
Plan geometry		Regular
Building dimensions (m x m)		10.5 x 18.0
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.27
Wall ratio*	**x-direction (%)**	25.5
	y-direction (%)	25.5
Slab thickness (cm)		12
Average plaster thickness (cm)**		6.0
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 4 was located in Aydınlıkevler/Ankara. It was a three story structure constructed with solid clay bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 4 is shown in Figure 2.5. Table 2.5 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.5 Building 4 information

Table 2.5 Detailed information about Building 4

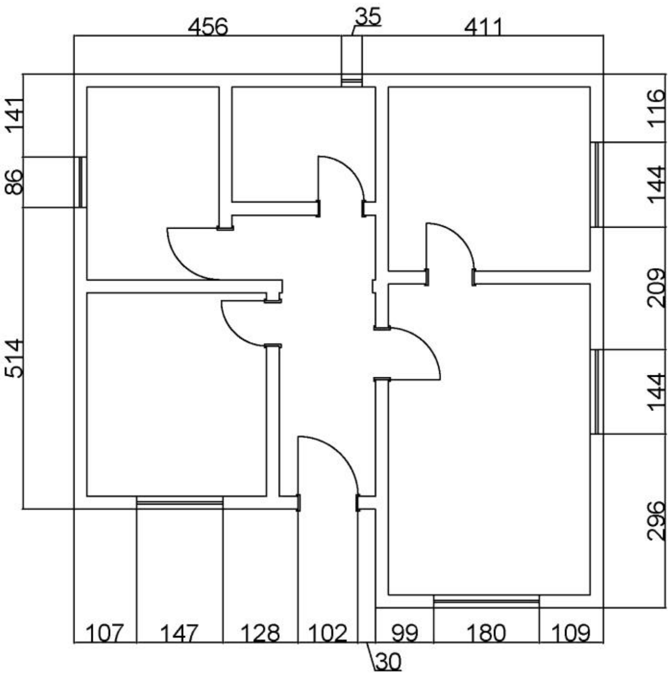
Address		Aydınlıkevler District, Şehit Bülent Ay Street, No:49 ALTINDAĞ – ANKARA
Lot and block ID		4459-12
Construction year		1960
Structural system		Unreinforced masonry
Type of masonry unit		Solid clay brick
Void ratio of the masonry unit		-
Earthquake zone		4
Soil class		D-Z4
Number of stories		3
Story height (m)		2.85
Basement floor		Exists
Plan geometry		Regular
Building dimensions (m x m)		14.5 x 16.2
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.26
Wall ratio*	**x-direction (%)**	22.1
	y-direction (%)	32.5
Slab thickness (cm)		15
Average plaster thickness (cm)**		4.6
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 5 was located in Mamak/Ankara. It was a one story structure constructed with cellular concrete blocks and adobe. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 5 is shown in Figure 2.6. Table 2.6 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.6 Building 5 information

Table 2.6 Detailed information about Building 5

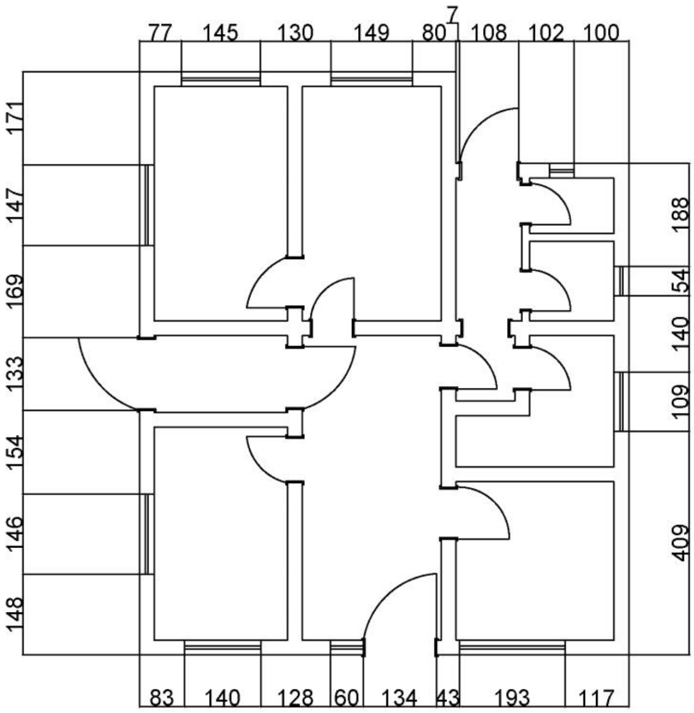
Address		Cengizhan District, 863 Street, No:16 MAMAK – ANKARA
Lot and block ID		50988-1
Construction year		1970
Structural system		Unreinforced masonry
Type of masonry unit		Cellular concrete block and adobe
Void ratio of the masonry unit		0.53
Earthquake zone		4
Soil class		D-Z4
Number of stories		1
Story height (m)		2.36
Basement floor		None
Plan geometry		Regular
Building dimensions (m x m)		9.0 x 9.1
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.22
Wall ratio*	**x-direction (%)**	24.3
	y-direction (%)	30.7
Slab thickness (cm)		15
Average plaster thickness (cm)**		3.5
Horizontal lintels		None
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 6 was located in Merkez/Kırşehir. It was a two story structure constructed with solid concrete brick. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 6 is shown in Figure 2.7. Table 2.7 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.7 Building 6 information

Table 2.7 Detailed information about Building 6

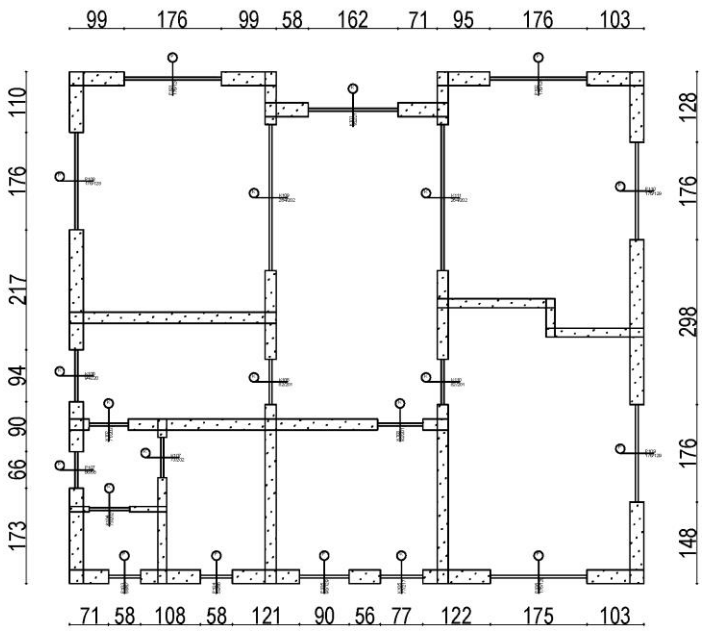
Address		Nasuhdede District, Şehit Kemal Akça Road, No:2 MERKEZ - KIRŞEHİR
Lot and block ID		1441-34
Construction year		1977
Structural system		Unreinforced masonry
Type of masonry unit		Solid concrete brick
Void ratio of the masonry unit		-
Earthquake zone		1
Soil class		
Number of stories		2
Story height (m)		3.00
Basement floor		None
Plan geometry		Irrgeular
Building dimensions (m x m)		9.0 x 10.7
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.27
Wall ratio*	**x-direction (%)**	22.1
	y-direction (%)	31.0
Slab thickness (cm)		12
Average plaster thickness (cm)**		7.4
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 7 was located in Çubuk/Ankara. It was a two story structure constructed with solid concrete bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 7 is shown in Figure 2.8. Table 2.8 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.8 Building 7 information

Table 2.8 Detailed information about Building 7

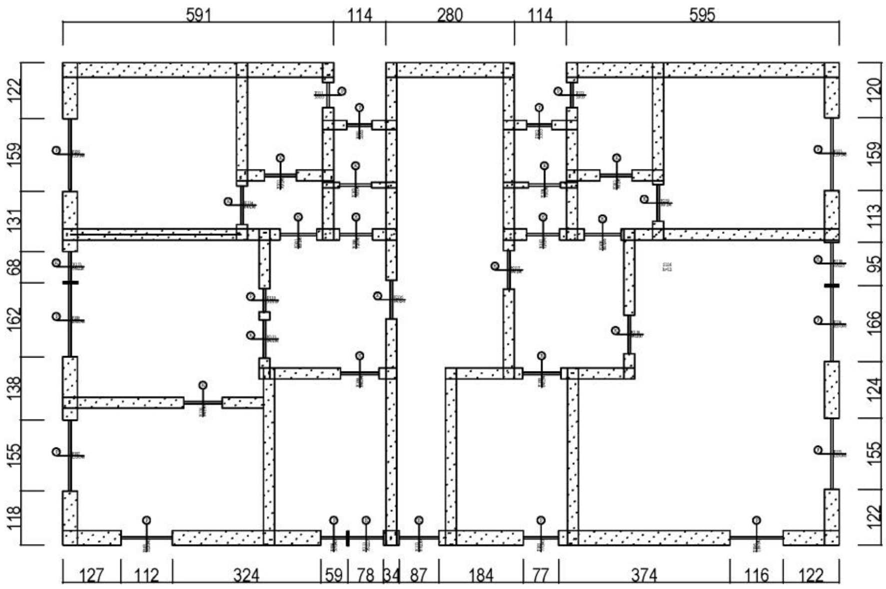
Address		Yıldırım Beyazıt District, Defne Street, No:5 ÇUBUK – ANKARA
Lot and block ID		2116-4
Construction year		Unknown
Structural system		Unreinforced masonry
Type of masonry unit		Solid clay brick
Void ratio of the masonry unit		-
Earthquake zone		3
Soil class		C
Number of stories		2
Story height (m)		2.72
Basement floor		None
Plan geometry		Regular
Building dimensions (m x m)		10.1 x 12.1
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.25
Wall ratio*	**x-direction (%)**	12.0
	y-direction (%)	12.6
Slab thickness (cm)		13
Average plaster thickness (cm)**		4.8
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 8 was located in Çubuk/Ankara. It was a two story structure constructed with hollow concrete bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 8 is shown in Figure 2.9. Table 2.9 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.9 Building 8 information

Table 2.9 Detailed information about Building 8

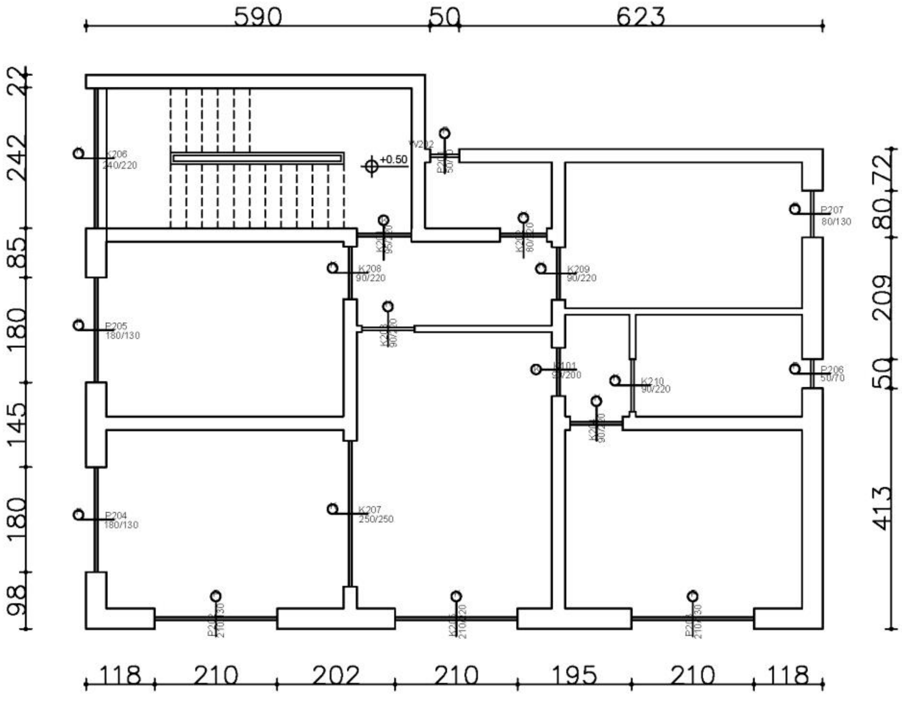
Address		Yavuz Selim District, Selimiye Street, No:5 ÇUBUK – ANKARA
Lot and block ID		5047-8
Construction year		Unknown
Structural system		Unreinforced masonry
Type of masonry unit		Hollow clay brick
Void ratio of the masonry unit		0.54
Earthquake zone		3
Soil class		C
Number of stories		2
Story height (m)		2.73
Basement floor		None
Plan geometry		Regular
Building dimensions (m x m)		10.5 x 16.9
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.32/0.24
Wall ratio*	**x-direction (%)**	25.8
	y-direction (%)	25.5
Slab thickness (cm)		12
Average plaster thickness (cm)**		3.1
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 9 was located in Aydınlıkevler/Ankara. It was a three story structure constructed with hollow concrete bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 9 is shown in Figure 2.10. Table 2.10 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.10 Building 9 information

Table 2.10 Detailed information about Building 9

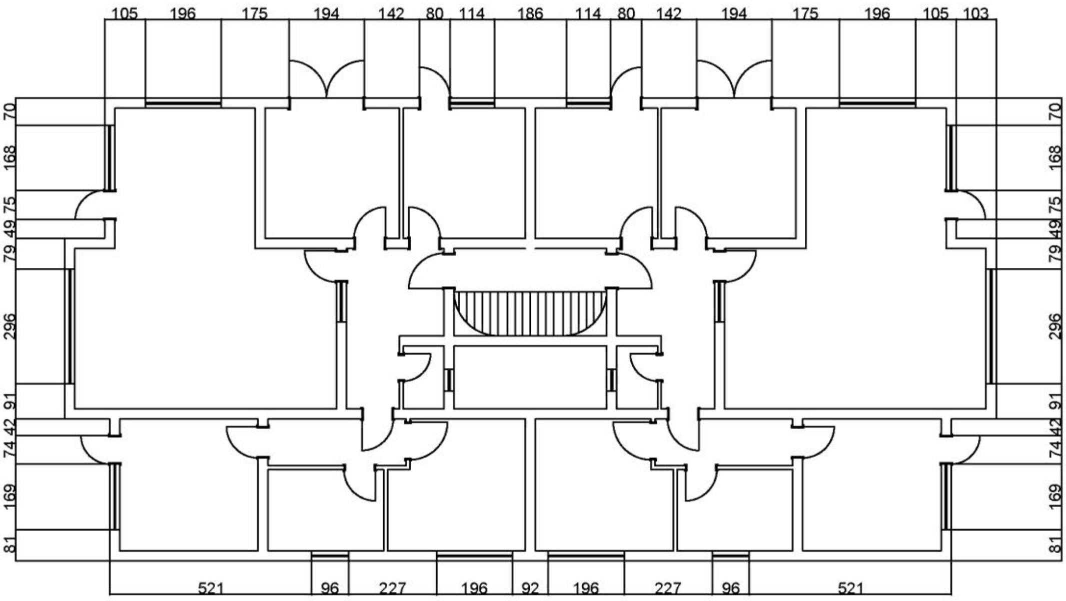
<table>
<tr><td colspan="2">Address</td><td>Aydınlıkevler District, Çağdaş Street, No:31 ALTINDAĞ – ANKARA</td></tr>
<tr><td colspan="2">Lot and block ID</td><td>4507-18</td></tr>
<tr><td colspan="2">Construction year</td><td>Unknown</td></tr>
<tr><td colspan="2">Structural system</td><td>Unreinforced masonry</td></tr>
<tr><td colspan="2">Type of masonry unit</td><td>Hollow clay brick</td></tr>
<tr><td colspan="2">Void ratio of the masonry unit</td><td>0.22</td></tr>
<tr><td colspan="2">Earthquake zone</td><td>4</td></tr>
<tr><td colspan="2">Soil class</td><td>C</td></tr>
<tr><td colspan="2">Number of stories</td><td>3</td></tr>
<tr><td colspan="2">Story height (m)</td><td>3.00</td></tr>
<tr><td colspan="2">Basement floor</td><td>Exists</td></tr>
<tr><td colspan="2">Plan geometry</td><td>Regular</td></tr>
<tr><td colspan="2">Building dimensions (m x m)</td><td>9.2 x 12.3</td></tr>
<tr><td colspan="2">Structural arrengement</td><td>Seperate</td></tr>
<tr><td colspan="2">Height difference between contiguous buildings (if exists)</td><td>-</td></tr>
<tr><td colspan="2">Slab alignment difference between between the contiguous buildings (if exists)</td><td>-</td></tr>
<tr><td colspan="2">Typical wall thickness (m)</td><td>0.23</td></tr>
<tr><td rowspan="2">Wall ratio*</td><td>x-direction (%)</td><td>33.1</td></tr>
<tr><td>y-direction (%)</td><td>19.0</td></tr>
<tr><td colspan="2">Slab thickness (cm)</td><td>12</td></tr>
<tr><td colspan="2">Average plaster thickness** (cm)</td><td>4.2</td></tr>
<tr><td colspan="2">Horizontal lintels</td><td>Exist</td></tr>
<tr><td colspan="2">Roof type</td><td>Hipped tile</td></tr>
<tr><td colspan="2">Soft story</td><td>None</td></tr>
<tr><td colspan="2">Existing damage</td><td>None</td></tr>
</table>

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

Building 10 was located in Bahçelievler/Ankara. It was a three story structure constructed with hollow concrete bricks. The slab of the building was made of reinforced concrete and it had no RC beams or columns. The picture and floor plan of Building 10 is shown in Figure 2.11. Table 2.11 summarizes brief information about the building.

a) Appearance of the building



b) Plan of the building

Figure 2.11 Building 10 information

Table 2.11 Detailed information about Building 10

Address		Bahçelievler District, 46 Street, No:104 ÇANKAYA – ANKARA
Lot and block ID		2614-6
Construction year		Unknown
Structural system		Unreinforced masonry
Type of masonry unit		Solid clay brick
Void ratio of the masonry unit		-
Earthquake zone		4
Soil class		Z2-C
Number of stories		3
Story height (m)		2.75
Basement floor		Exists
Plan geometry		Regular
Building dimensions (m x m)		11.9 x 21.7
Structural arrengement		Seperate
Height difference between contiguous buildings (if exists)		-
Slab alignment difference between between the contiguous buildings (if exists)		-
Typical wall thickness (m)		0.26
Wall ratio*	**x-direction (%)**	8.70
	y-direction (%)	8.40
Slab thickness (cm)		12
Average plaster thickness (cm)**		5.4
Horizontal lintels		Exist
Roof type		Hipped tile
Soft story		None
Existing damage		None

*:The ratio of the length of walls in one direction to the floor plan area

**: The plaster thicknesses for each specimen was measured and the average thickness is reported.

2.3 Preliminary Assesment of the Buildings

A preliminary assessment is made for all the buildings according to the method recommended in RBTEIE (2013). The following procedure is conducted for ten selected buildings:

- The base point for the building is determined according to the number of stories of the building and the earthquake zone in which the building is located.
- The effect of structural system of the masonry building (i.e. unreinforced masonry, confined masonry or reinforced masonry) is determined in terms of positive performance points.
- Depending on the visible quality of the material and workmanship and existing structural damage, negative performance points are assigned to the buildings accordingly.
- The effects of geometry of the plan, wall ratio of the building and existence of any bond beams or lintels are represented by means of negative performance points.
- The vertical irregularities in plan,soft story effect, out of plane behavior of the building and the effect of contiguous buildings are also taken into account in terms of negative performance points.
- A final performance point is determined considering the above criteria

These performance points can be sorted to provide a general understanding of risk priority through the buildings without complex assessment methods. The results of the preliminary assessments are given in Table 2.12. As the result of preliminary assessment, masonry buildings that were under high earthquake risk had performance points between 60 and 90 with a mean of 73. This shows that for masonry buildings classified as high risk according to the RBTEIE (2013), the expected performances scores would be around 70.

Table 2.12 Preliminary assessment results

Building ID	1	2	3	4	5	6	7	8	9	10
Base point	110	120	110	110	130	100	110	110	110	110
Quality of the material	-20	-20	-10	-20	-20	-20	-20	-20	-20	-20
Quality of workmanship	-10	-10	-5	-10	-10	-5	-10	-10	-10	-10
Existing damage	0	-5	0	0	-5	0	0	0	0	0
Plan geometry	0	0	0	0	0	-10	0	0	0	0
Wall ratio	-5	-5	-10	-10	-5	-5	-5	-5	-10	-10
Bond beams or lintels	0	0	0	-5	0	0	-5	0	0	0
Vertical irregularites	0	-5	0	0	0	0	0	0	0	0
Soft story	0	0	0	0	0	0	0	0	0	0
Contiguous building effect	0	-10	0	0	0	0	0	0	0	0
Total performance point	**75**	**65**	**85**	**65**	**90**	**60**	**70**	**75**	**70**	**70**

CHAPTER 3

MATERIAL TESTING

In this chapter, the material test results conducted on specimens are presented. Test specimens were obtained from the buildings whose properties are described in Chapter 2. Masonry wallets were extracted from the buildings and shipped to METU Structural Engineering Laboratory. The details of wallet selection, cutting, removal and shipment in addition to the test results are given in the following sections.

3.1 Specimens for Testing

The selection, removal and shipment process is conducted in accordance with ASTM C1532/1532M-12. Specimens with a size of about 70 cm x 70 cm were marked on the walls which were qualified as suitable for the tests. The walls were selected carefully so that the building would not have any structural damage upon the removal of the walls. Specimens were taken from load-bearing walls without any prior cracks or damage that were placed on the ground floor. In order to avoid any structural damage on the rest of the building after the extraction of test specimens, some wallets were also selected from walls underneath windows.

The marked walls at aforementioned positions in each building were photographed and then cut using a saw with the brand Husqvarna K760. The removal process was carefully performed by spraying water. It was assured that the specimens maintained their integrity and did not get damaged. In order to minimize the damage to specimen removed from existing masonry walls, the process was followed exactly as recommended in ASTM C1532/1532M-12. Firstly, the bottom part was cut and shimmed to take up the weight of the specimen. Then, the top of the specimen was cut and finally the sides were cut. Pictures showing the whole process of obtaining wall specimens are given in Figure 3.1.

Figure 3.1 Examples from specimen removal process

As described in ASTM C1532/1532M-12, the removed specimens were carefully taken out for transportation. The specimens were confined with rigid styrofoam and placed in the vehicle so that they would not move and take any damage during transportation. Finally, the specimens were shipped to the laboratory, taken out from the vehicle and stacked carefully using the crane in the laboratory.

3.2 Laboratory Tests

The tests were conducted using a displacement controlled test machine (Figure 3.2) at METU, Department of Civil Engineering, Structural Mechanics Laboratory. Use of this machine allows obtaining the softening branch on the load-deformation graph after the specimen reached its capacity. This curve was successfully obtained in all the diagonal tension experiments, while for the compression tests it could not be obtained due to the observed brittle failure mode.

Figure 3.2 Displacement based test machine

The tests were conducted according to the provisions below:

- ASTM C1314-12: Standard Test Method for Compressive Strength of Masonry Prisms
- ASTM E519/E519M-10: Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages
- EN 1052-3 Methods of Tests for Masonry – Part 3: Determination of Initial Shear Strength
- ASTM C1552-03a: Standard Practice for Capping Concrete Masonry Units, Related Units and Masonry Prisms for Compression Testing
- ASTM C1532-03a: Standard Practice for Selection, Removal, and Shipment of Masonry Assemblage Specimens from Existing Construction

Depending on the proposed testing method, the wall assemblages were cut down into the appropriate dimensions when necessary. Since the cutting process was performed by spraying water, the specimens were stored until moisture was removed. The places, which the measurements were going to be taken by LVDT's, were marked on the specimen and drilled carefully without damaging the specimens. During the course of this study, compression tests, diagonal tension tests and sliding shear tests were conducted. The details of each test and results are presented in the following subsections.

Specimens extracted from the walls were usually covered with plaster. The brick wall thickness and plaster thickness were measured and reported for each specimen along with the test results herein. The tests were conducted for plaster-covered specimens. Strength values were always computed based on gross area of specimens including the thickness of plaster if there is any.

3.2.1 Compression Tests

The compression tests were conducted in order to determine the strength of the specimens under axial compression. The tests were conducted according to ASTM C1314-12 provisions. The specimens were tested in two different directions:

- Tests parallel to the load bearing direction of the wall
- Test perpendicular to the load bearing direction of the wall

The specimens were prepared for the tests first by capping of the ends. Due to the irregularities on the surface of the walls that occurred during the removal, capping all the specimens were necessary and it was conducted with a gypsum cement capping material, as stated in ASTM C1552-03a, for the top and bottom surfaces of the walls. After waiting approximately half a day for the cap to cure, the specimens were placed in the test machine with their bearing end plates attached. Then, the centroidal axis of the specimen was perfectly aligned with the center of loading head and a spherical ball bearing was placed on the top of the bearing plate.

Steel rods of 6 mm thickness were inserted into the previously drilled holes by applying epoxy. After the curing of epoxy, LVDTs were placed such that displacements were measured between the ends of the steel rods. Afterwards the instruments including the LVDTs and the load cell were connected to the computer and their calibrations were conducted. Due to the safety reasons, the necessary equipment was tied to the test machine assuring that they would fall down upon sudden failure. Specimen dimensions were measured and after being photographed, specimens were ready for testing. Pictures showing the axial compression test setup (Figure 3.3) are given below.

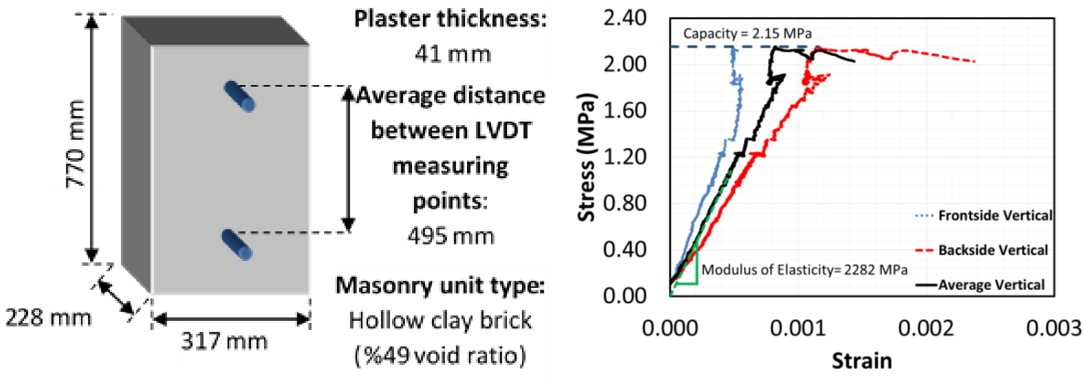
Figure 3.3 Axial compression test setup

After the experiment was conducted, the failed specimen was photographed and the load-deformation data were extracted. In order to calculate the compressive strength of the masonry wall, the procedure in ASTM C1314-12 was taken into account.

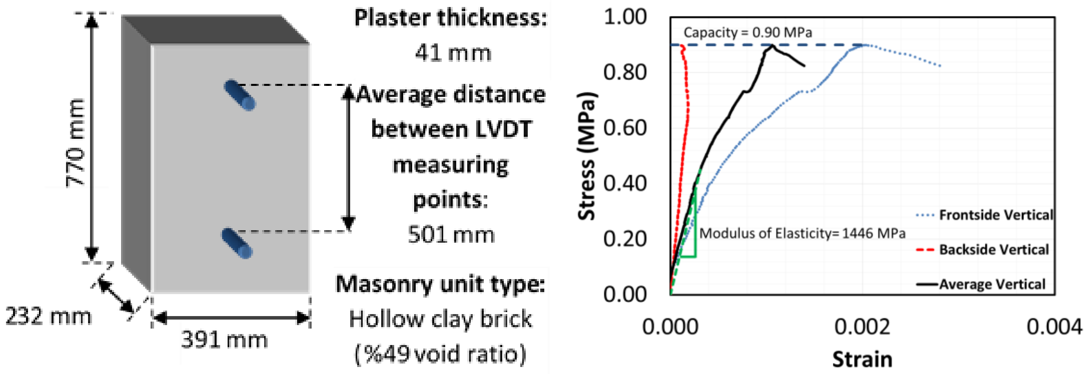
Each compressive load data measured in kilograms was multiplied with the gravitational acceleration constant and divided by the gross cross-sectional area of the specimen in order to obtain stress in Pascal. Then depending on the height-thickness ratio of the specimen, a correction factor was determined from ASTM C1314-12 Table-1 and multiplied with the compressive stress found previously. On the other hand, the axial deformation data were divided by the distance between the measurement points in order to obtain the strains of the specimen. For each test, the modulus of elasticity was calculated according to ASTM C1314-12. In this procedure, a straight line was drawn from the origin to the point corresponding to 33 percent of the specimens' strength.

The data that was obtained through the calculations were displayed as stress -strain graphs and the important values were shown on it. The experiment results obtained from axial compression tests were given below with the geometric properties of each specimen. Test results including the dimensions of the specimens, measured stress-strain response, and the pictures of the failed specimens are presented in Figures 3.4 to 3.20 for specimens extracted from buildings 1 to 10, respectively.

Three types of irregularities were observed on the stress vs. strain curves of some specimens. First type of irregularity was the decrease in the strain while the stress was increasing after linearly elastic part of the curve (for example see Figure 3.13). The main reason for this type of irregularity is the out of plane bending that occurred due to the inhomogeneous section of the masonry wall units that are composed of unequal plaster thickness on both sides. Despite the fact that the geometric centroidal axis of the specimen was perfectly aligned with the center of loading head, the inhomogeneity led to out of plane bending of the specimen. The second type of irregularity observed, was the increase in the strain under constant or decreasing stresses (for example see Figure 3.6). A mechanical glitch in the test setup caused the electromotor to halt under low rates of loading. Increasing the speed of the loading enabled to continue the experiment, however this caused the second type of irregularity on the stress vs. strain curves. Finally, third type of irregularity was the incomplete stress vs. strain curves as shown in Figure 3.10. This was caused because the LVDT's were taken out before the specimen reached its capacity to prevent them from taking any damage.



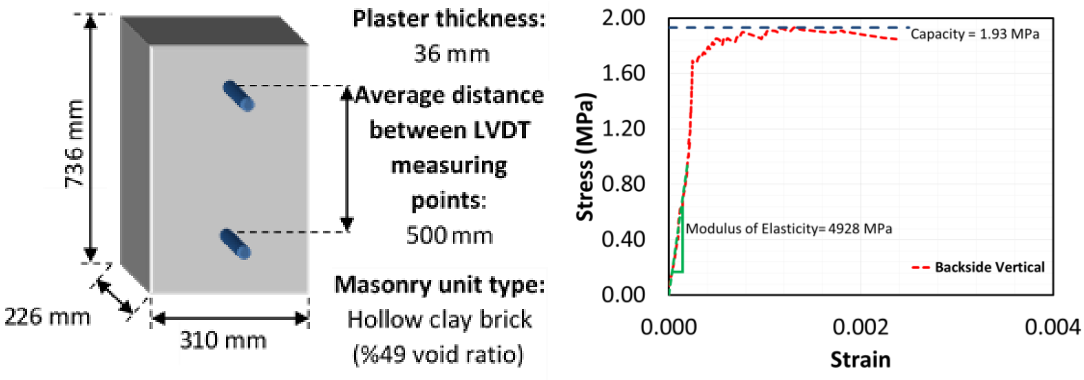
a) Geometric properties and axial compression test result of Specimen 1



b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.4 Building 1 compression test results along the axis parallel to the load bearing direction of the wall

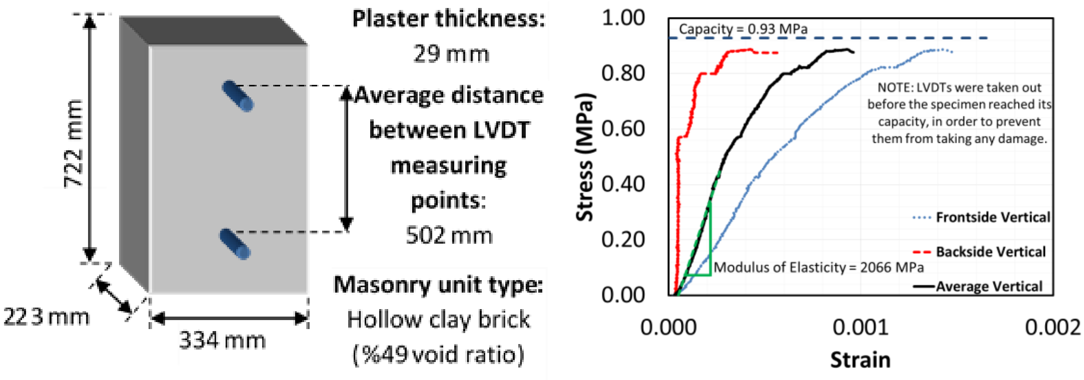


a) Geometric properties and axial compression test result of Specimen 2*

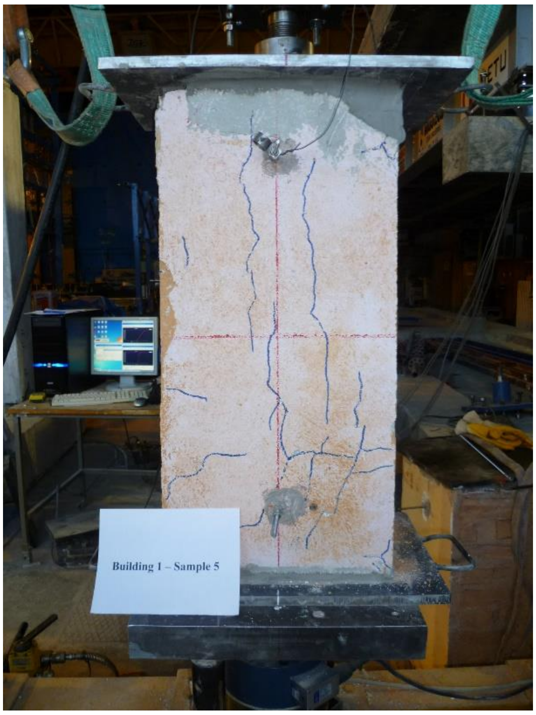
b) Specimen 2* at the end of the test

* The test for the second specimen of Building 1 compression test along the axis parallel to the load bearing direction of the wall was repeated with another specimen since the failure was caused by a pipe installed in the second specimen.

Figure 3.5 Building 1 compression test results along the axis parallel to the load bearing direction of the wall



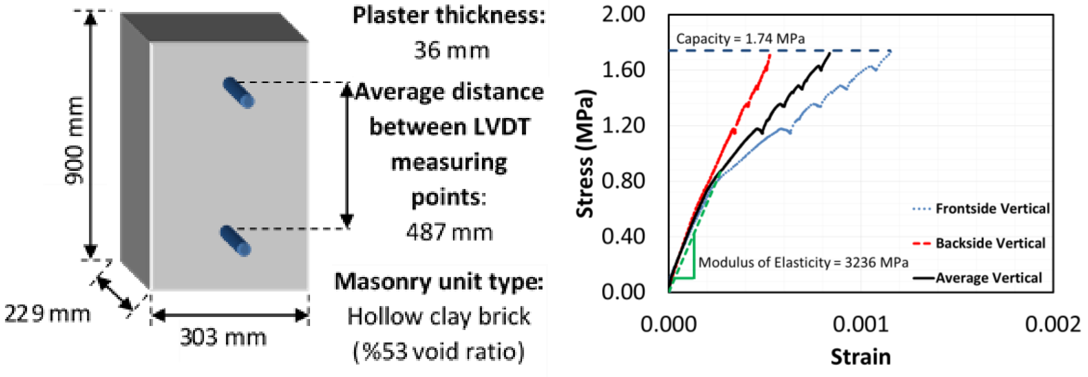
c) Geometric properties and axial compression test result of Specimen 3



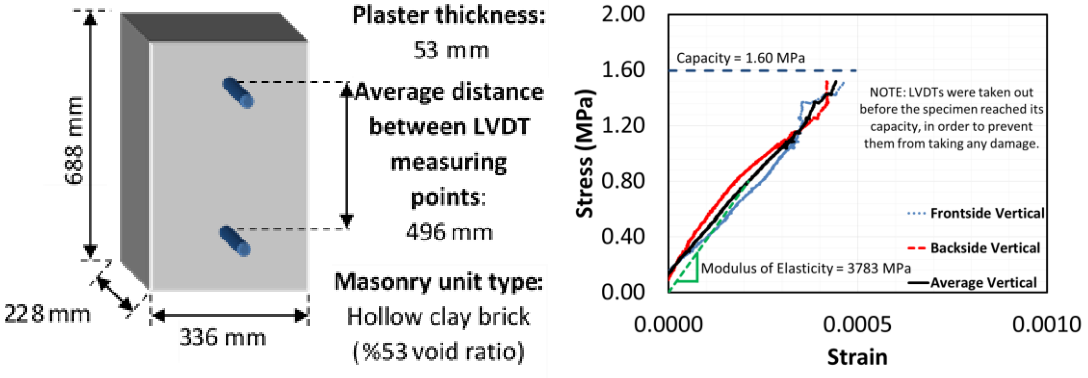
d) Specimen 3 at the end of the test

Figure 3.6 Building 1 compression test results along the axis perpendicular to the load bearing direction of the wall

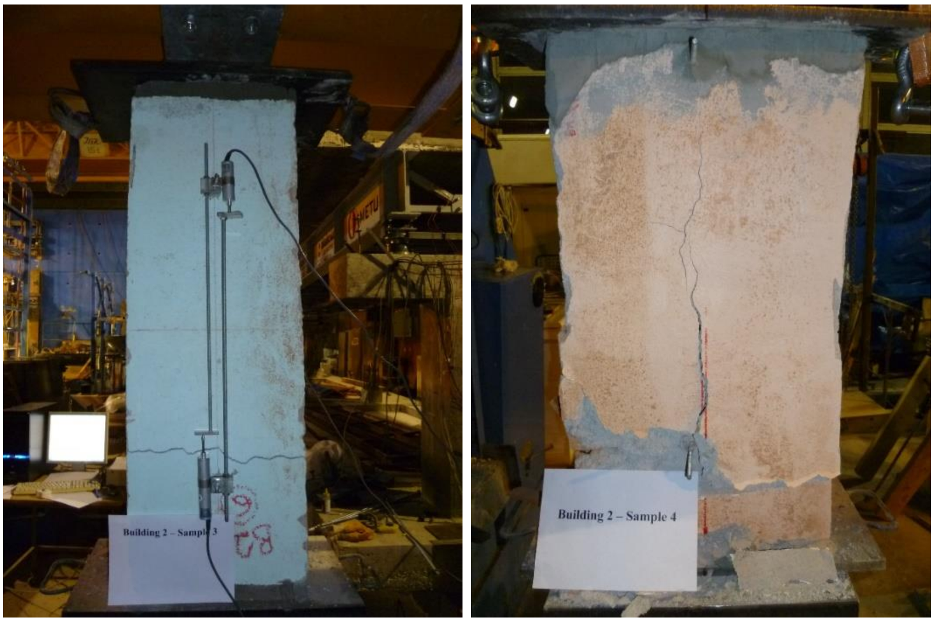
For building 1, a fourth specimen was not tested since the specimen was found to be severely damaged during cutting process.



a) Geometric properties and axial compression test result of Specimen 1

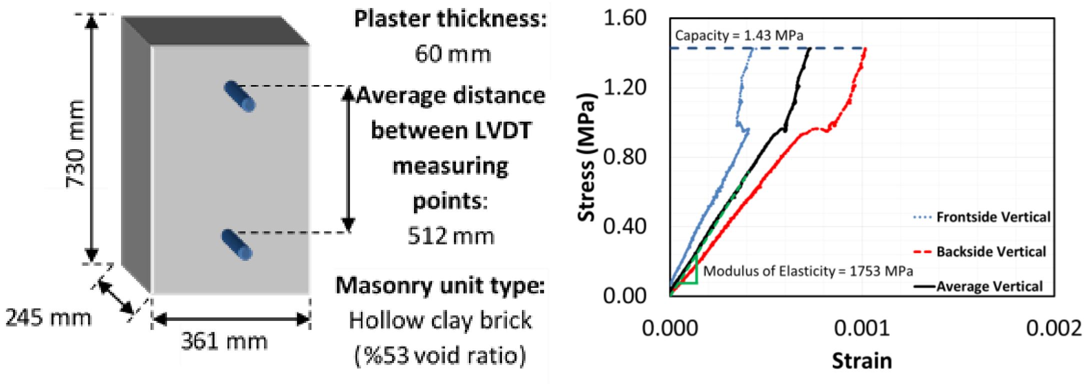


b) Geometric properties and axial compression test result of Specimen 2

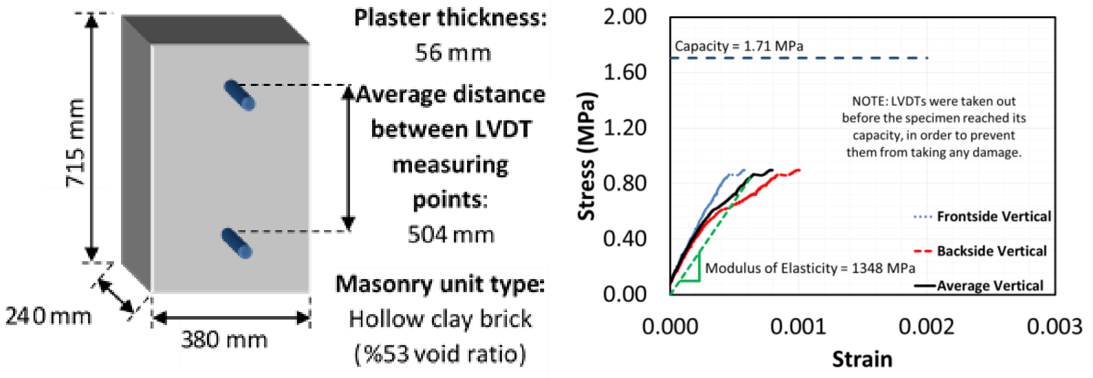


c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.7 Building 2 compression test results along the axis parallel to the load bearing direction of the wall



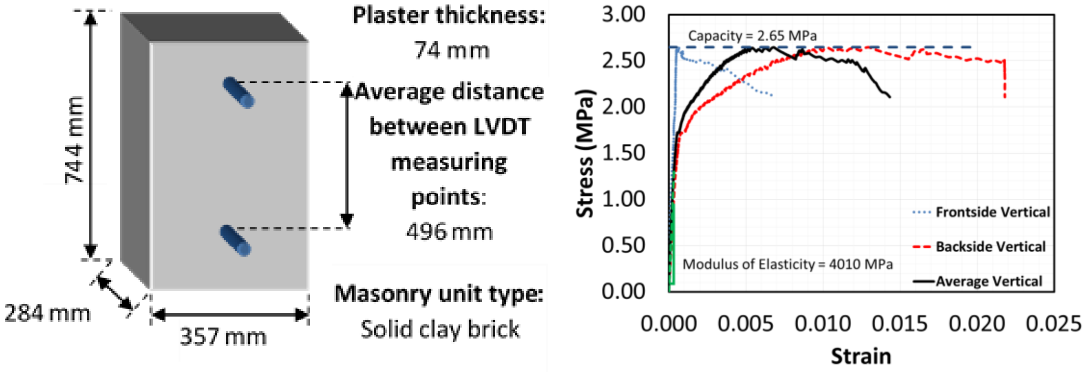
a) Geometric properties and axial compression test result of Specimen 3



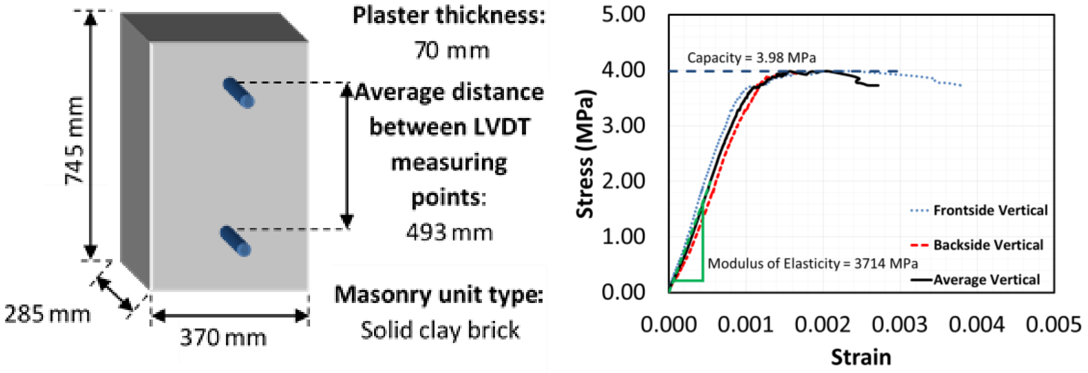
b) Geometric properties and axial compression test result of Specimen 4

c) Specimen 3 at the end of the test

Figure 3.8 Building 2 compression test results along the axis perpendicular to the load bearing direction of the wall



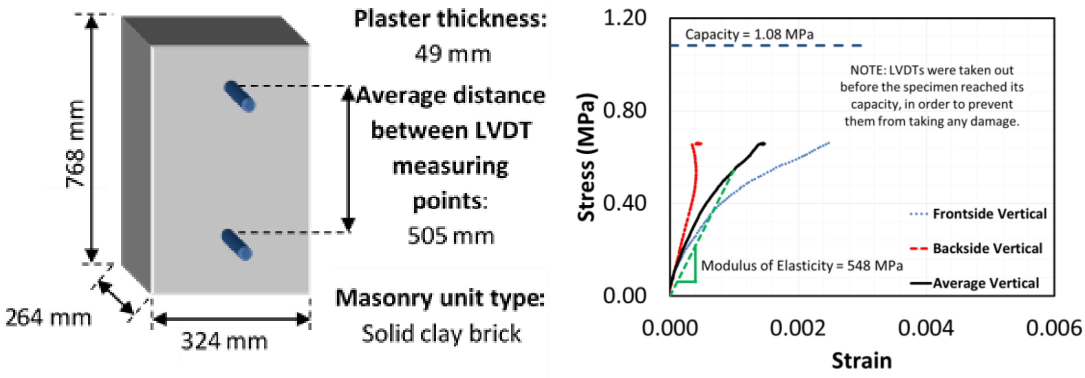
a) Geometric properties and axial compression test result of Specimen 1



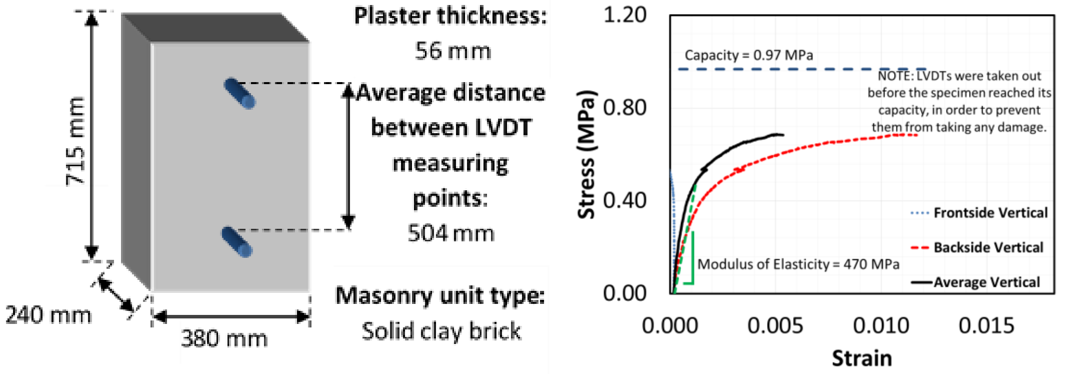
b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.9 Building 3 compression test results along the axis parallel to the load bearing direction of the wall



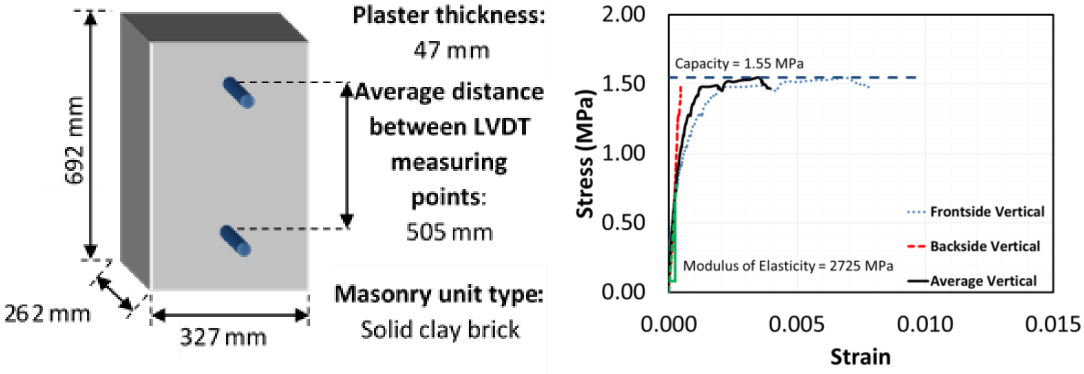
a) Geometric properties and axial compression test result of Specimen 3



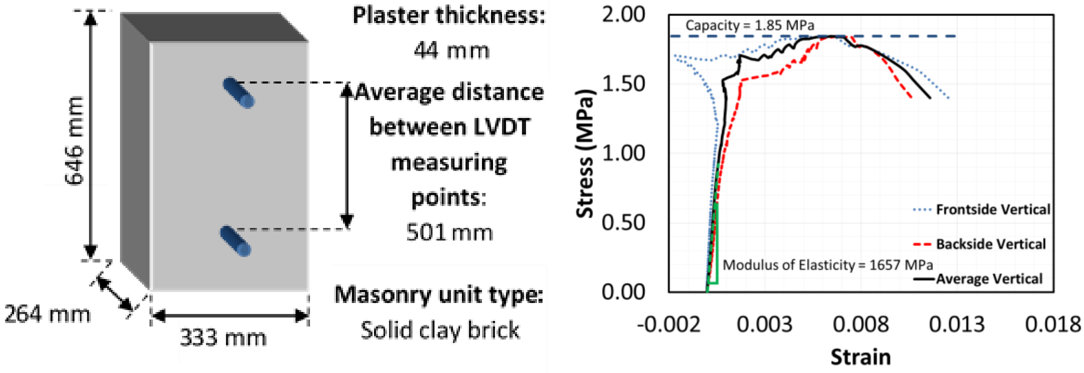
b) Geometric properties and axial compression test result of Specimen 4

c) Specimen 3 and Specimen 4 at the end of the test

Figure 3.10 Building 3 compression test results along the axis perpendicular to the load bearing direction of the wall



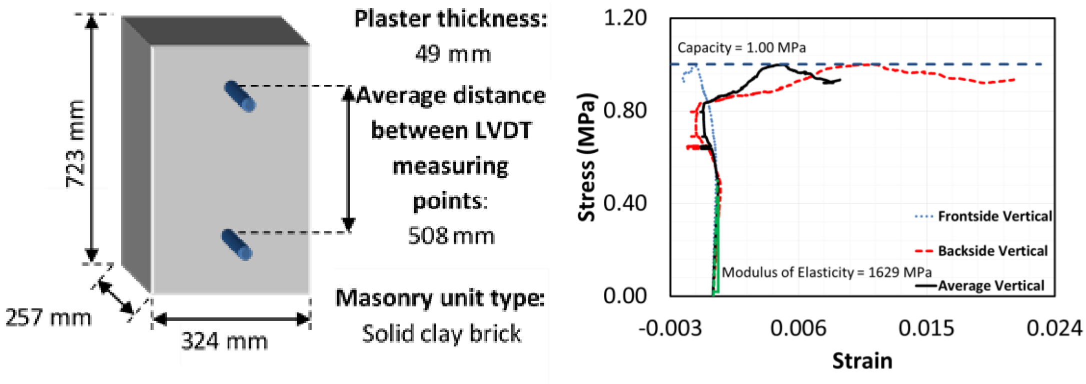
a) Geometric properties and axial compression test result of Specimen 1



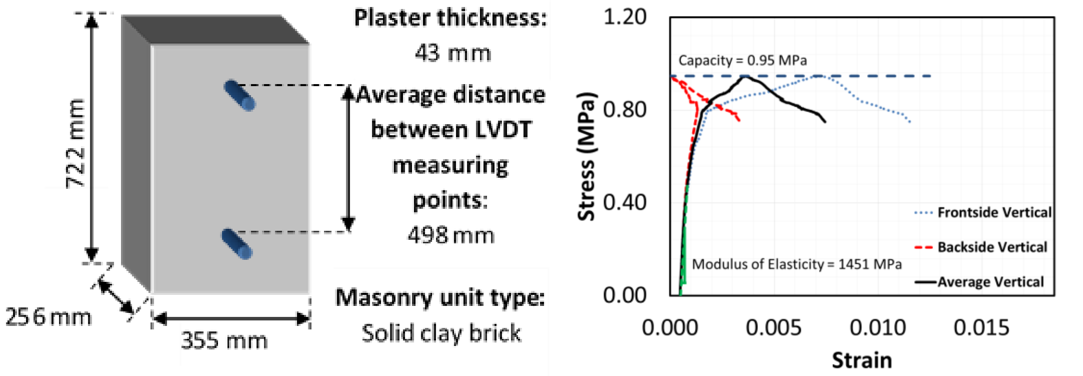
b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.11 Building 4 compression test results along the axis parallel to the load bearing direction of the wall



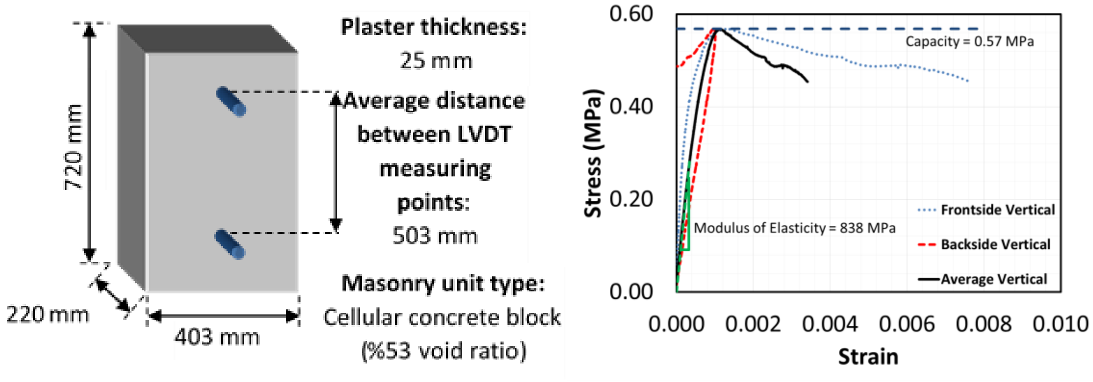
a) Geometric properties and axial compression test result of Specimen 3



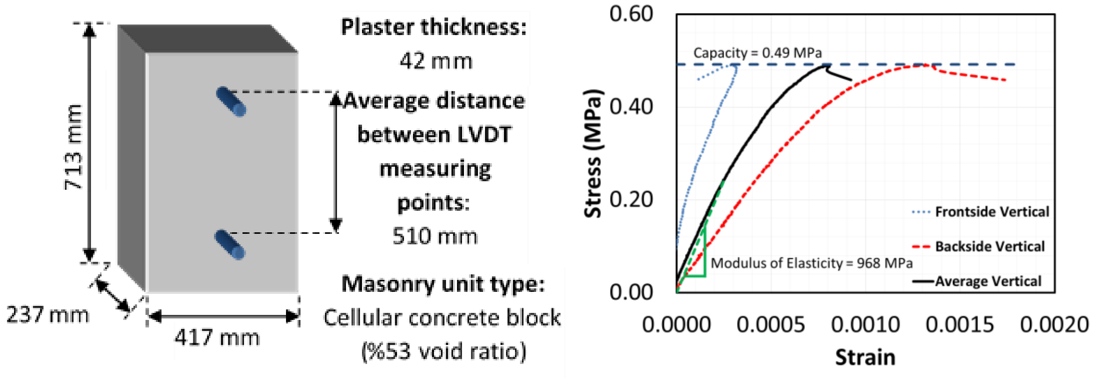
b) Geometric properties and axial compression test result of Specimen 4

c) Specimen 3 and Specimen 4 at the end of the test

Figure 3.12 Building 4 compression test results along the axis perpendicular to the load bearing direction of the wall



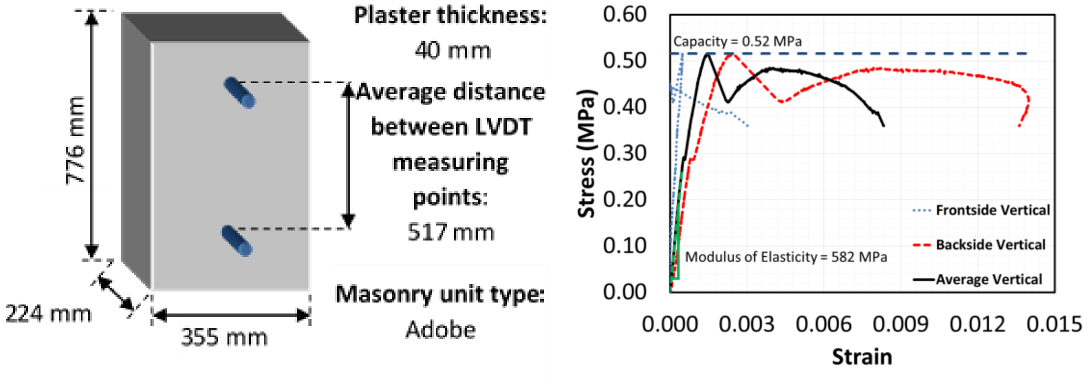
a) Geometric properties and axial compression test result of Specimen 1



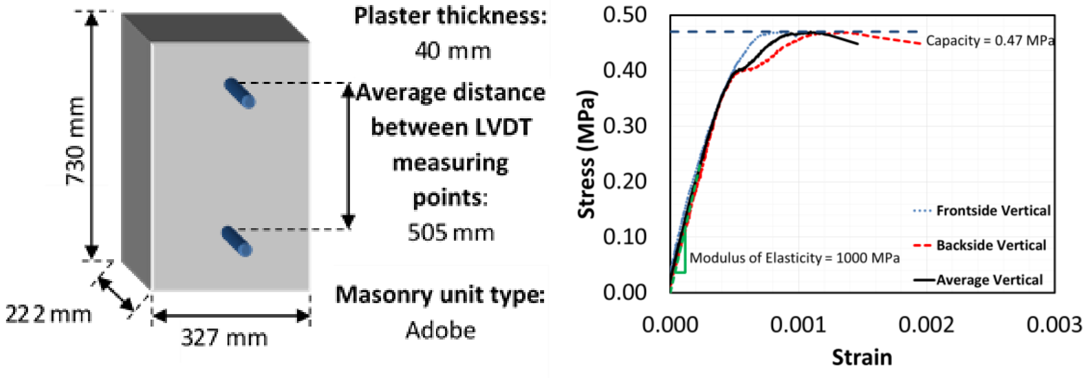
b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.13 Building 5 compression test results along the axis parallel to the load bearing direction of the wall (Cellular concrete block)



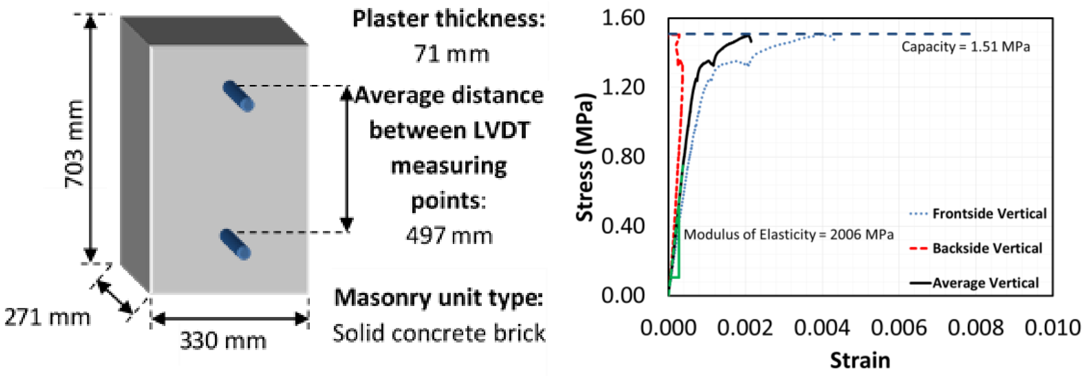
a) Geometric properties and axial compression test result of Specimen 3



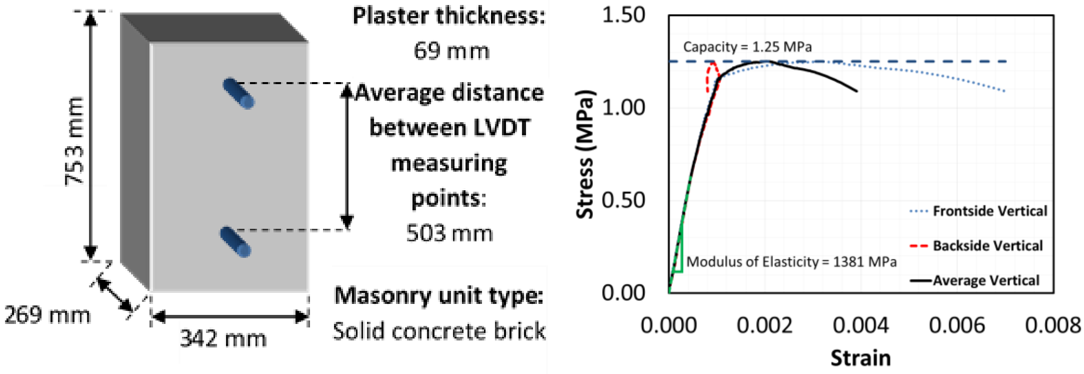
b) Geometric properties and axial compression test result of Specimen 4

c) Specimen 3 and Specimen 4 at the end of the test

Figure 3.14 Building 5 compression test results along the axis parallel to the load bearing direction of the wall (Adobe)



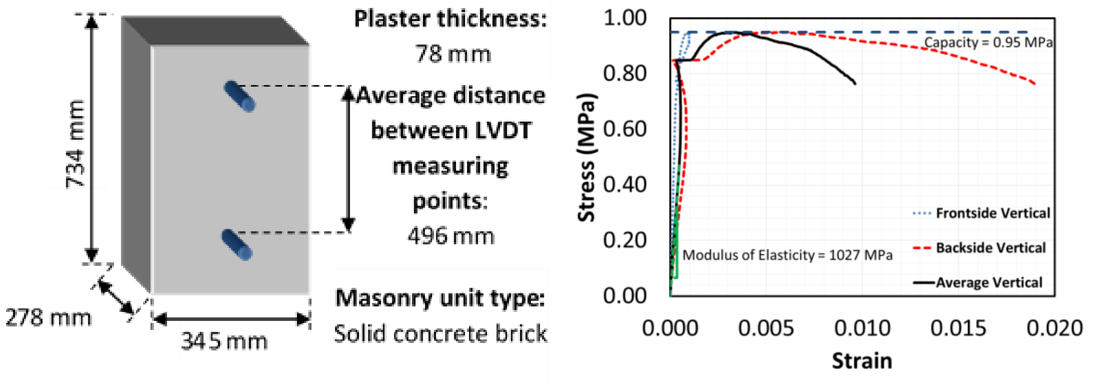
a) Geometric properties and axial compression test result of Specimen 1



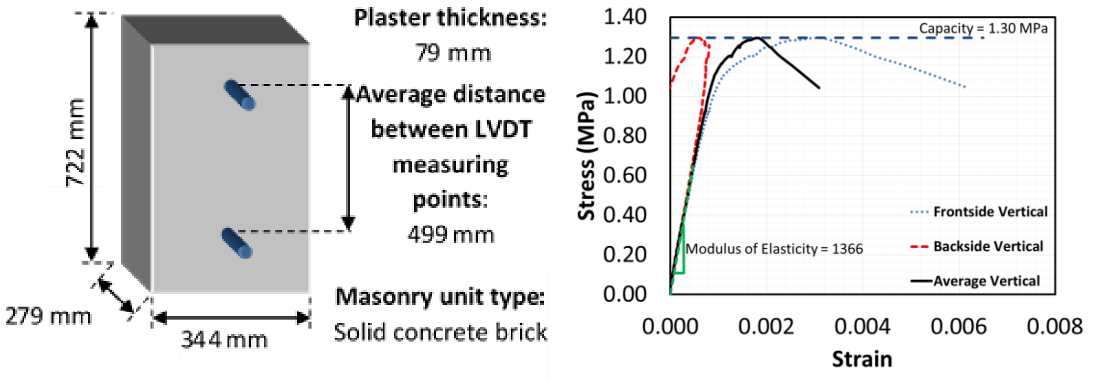
b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.15 Building 6 compression test results along the axis parallel to the load bearing direction of the wall



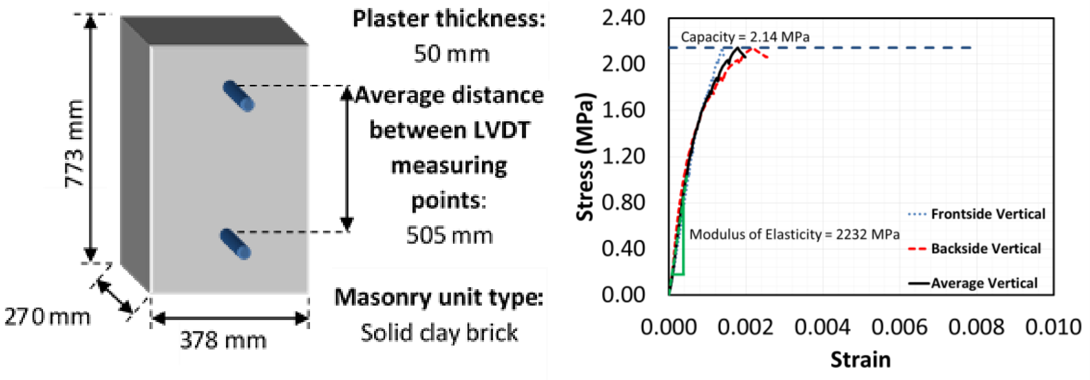
a) Geometric properties and axial compression test result of Specimen 3



b) Geometric properties and axial compression test result of Specimen 4

c) Specimen 3 and Specimen 4 at the end of the test

Figure 3.16 Building 6 compression test results along the axis perpendicular to the load bearing direction of the wall

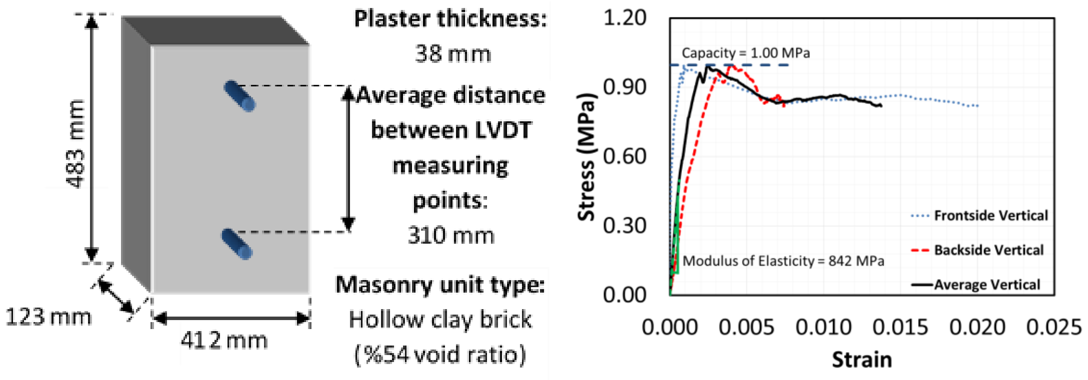


a) Geometric properties and axial compression test result of Specimen 1

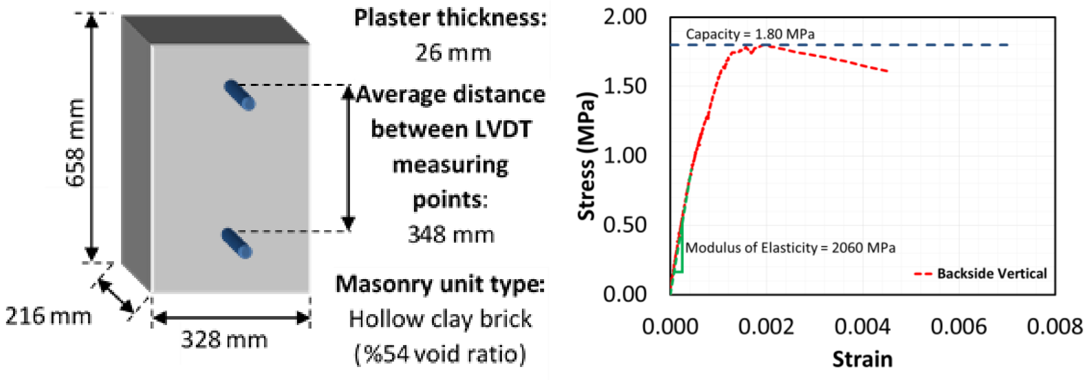
b) Specimen 1 at the end of the test

Figure 3.17 Building 7 compression test result along the axis parallel to the load bearing direction of the wall

Only one specimen was tested during the investigation of materials from building 7. The main reason of this is the fact that this building was planned as the test structure for the second phase of the project. Therefore, it was aimed not to reduce the horizontal load carrying capacity of the building and stiffness properties of the building.



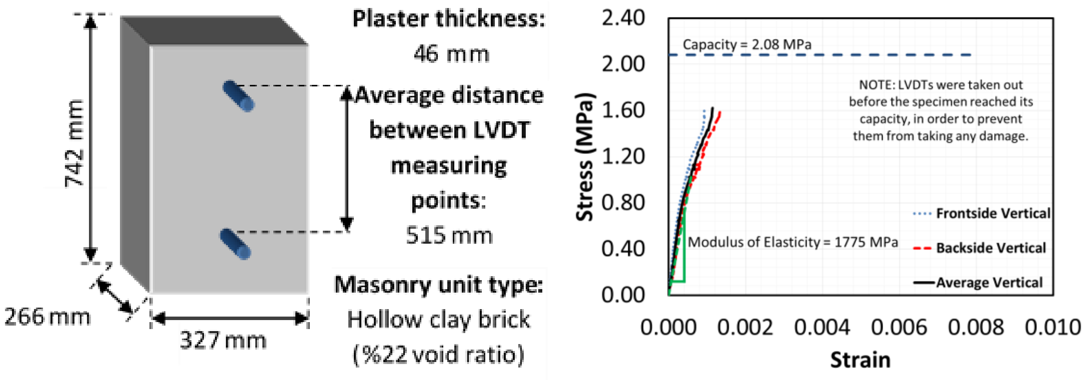
a) Geometric properties and axial compression test result of Specimen 1



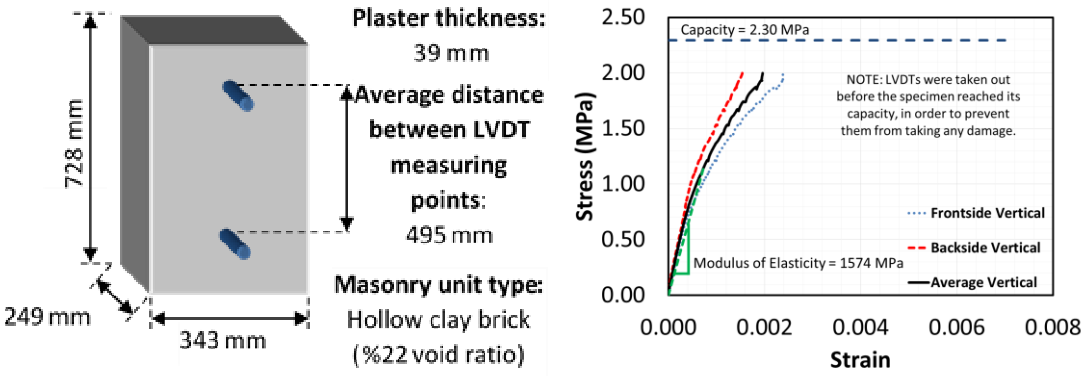
b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 at the end of the test

Figure 3.18 Building 8 compression test results along the axis parallel to the load bearing direction of the wall



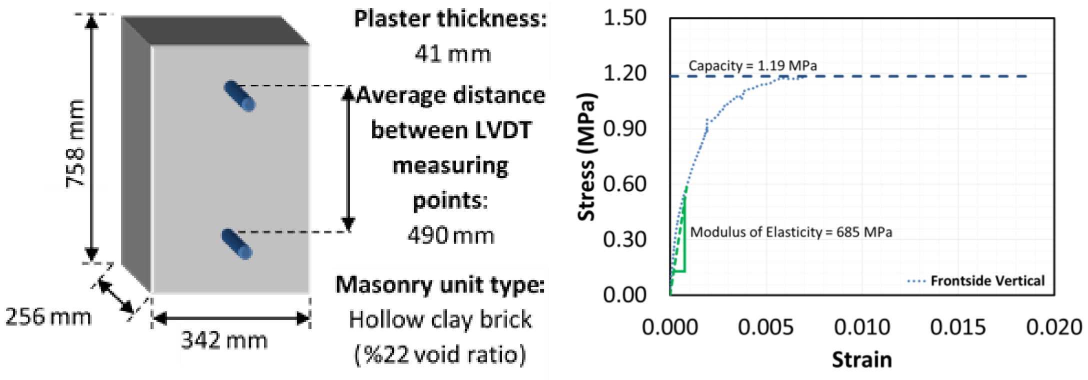
a) Geometric properties and axial compression test result of Specimen 1



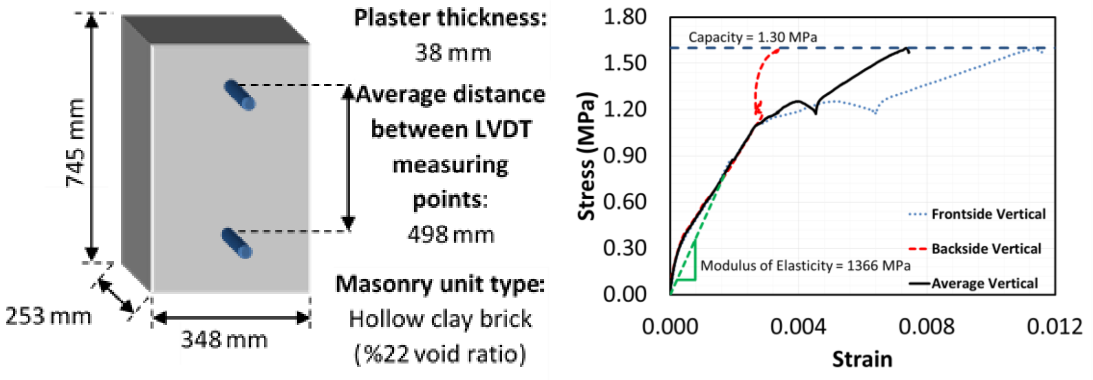
b) Geometric properties and axial compression test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.19 Building 9 compression test results along the axis parallel to the load bearing direction of the wall



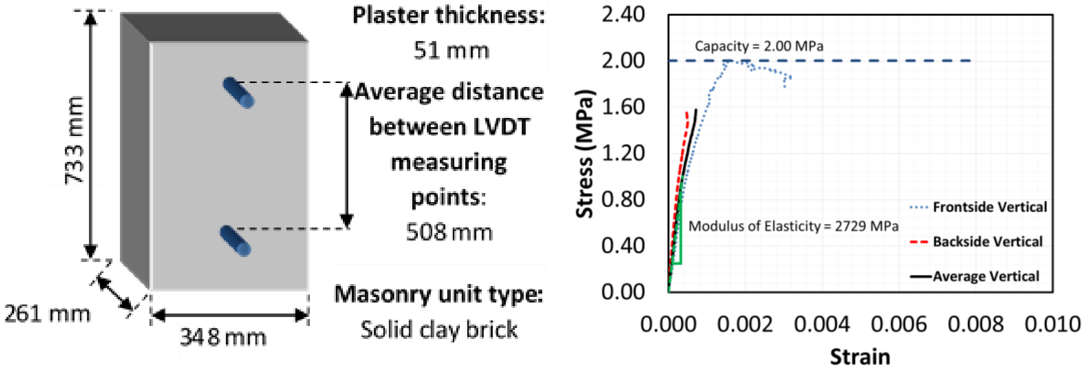
a) Geometric properties and axial compression test result of Specimen 3



b) Geometric properties and axial compression test result of Specimen 4

c) Specimen 3 and Specimen 4 at the end of the test

Figure 3.20 Building 9 compression test results along the axis perpendicular to the load bearing direction of the wall



a) Geometric properties and axial compression test result of Specimen 1

a) Specimen 1 at the end of the test

Figure 3.21 Building 10 compression test results along the axis parallel to the load bearing direction of the wall

In order to ensure the safety of the building during the removal process, less number of specimens had to be taken and only one axial compression test was conducted for Building 10.

All the data obtained from test results and calculations are summarized in Table 3.1.

Table 3.1 Test results

Building ID			1	2	3	4	5		6	7	8	9	10
Masonry Unit Type			Hollow clay brick	Hollow clay brick	Solid clay brick	Solid clay brick	Cellular concrete block	Adobe	Solid concrete brick	Solid clay brick	Hollow clay brick	Hollow clay brick	Solid clay brick
Modulus of Elasticity (MPa)		1	2282	3236	4010	2725	838	582	2006	2232	842	1775	2729
		2	1446 (4928)*	3783	3714	1657	968	1000	1381	-	2060	1574	-
		3	2066	1753	548	1629	-	-	1027	-	-	685	-
		4	-	1348	470	1451	-	-	1366	-	-	1366	-
Compressive Strength (MPa)	**Parallel to the load bearing direction**	1	2.15	1.74	2.65	1.55	0.57	0.52	1.51	2.14	1.00	2.08	2.00
		2	0.90 (1.93)*	1.60	3.98	1.85	0.49	0.47	1.25	-	1.80	2.30	-
	Perpendicular to the load bearing direction	3	0.93	1.43	1.08	1.00	-	-	0.95	-	-	1.19	-
		4	-	1.71	0.97	0.95	-	-	1.30	-	-	1.30	-
Diagonal Tension Strength (MPa)		1	0.31	0.24	0.08	0.15	0.14	0.12	0.29	0.36	0.27	0.17	0.20
		2	0.24	0.27	0.12	-	-	-	0.22	-	-	0.19	-
Shear Strength (MPa)		1	0.09	0.14	0.20	0.06	-	-	0.19	0.09	0.19	0.09	0.21
		2	0.23	0.21	0.35	0.09	-	-	0.17	0.18	0.15	0.11	0.19
Void ratio			0.49	0.53	-	-	0.53	-	-	-	0.54	0.22	-
Unit weight (kN/m^3)			11.96	11.54	17.01	15.85	10.18	18.51	14.11	20.30	10.86	14.93	19.91

*The test was repeated with another specimen since the failure was caused by a pipe installed in the second specimen.

3.2.2 Diagonal Tension Tests

The diagonal tension tests were conducted in order to determine the diagonal tensile of masonry walls by loading the walls along one diagonal, thus causing a diagonal tension failure with the specimen splitting apart parallel to the direction of loading. The tests were conducted according to ASTM E519/E519M-12 standards.

The specimens were prepared for the tests firstly by placing the two diagonal corners of the specimen into two steel loading shoes (Figure 3.21) as specified in ASTM E519/E519M-10. The loading shoes were essential to transfer the load correctly to the specimen.

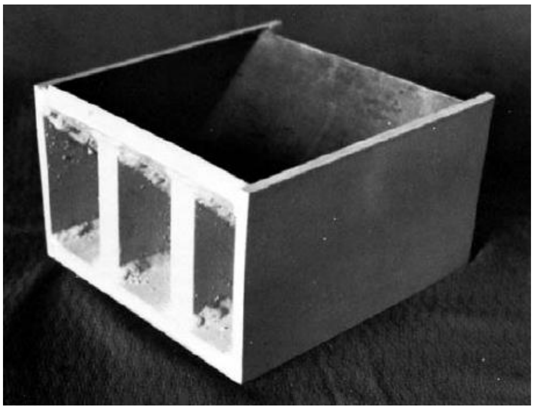


Figure 3.22 Steel loading shoe

In order to achieve sufficient force transfer between the specimen and the loading shoes, a gypsum-cement based mortar was used. After waiting approximately half a day for the cap to dry, the specimens were placed in the test machine. Then the centroidal axis of the specimen was aligned with the center of trust of the machine and a spherical headed support was placed on the top of the upper loading shoe.

Steel rods of 6 mm thickness were inserted into the previously drilled holes by applying epoxy. After the epoxy had dried, LVDTs were placed such that displacements were measured between the ends of the steel rods. Afterwards the instruments including the LVDTs and the load cell were connected to the computer and their calibrations were conducted. Due to the safety reasons, the necessary equipment was tied to the test machine assuring that they would fall down upon sudden failure. Specimen dimensions were measured and after being photographed, specimens were ready for testing. Pictures showing the diagonal tension test setup are given in Figure 3.22.

Figure 3.23 Diagonal tension test setup

After the experiments were concluded, the failed specimens were photographed and the load-deformation data were extracted. In order to calculate the diagonal tensile strength of the masonry wall, the procedure in ASTM E519/E519M-10 was employed. Accordingly, the diagonal tensile strength was computed using Equation 1.

$$S_s = \frac{0{,}707\,P}{A_n} \qquad (1)$$

where, S_s is the shear stress on gross area in MPa, P is the applied load in N and A_n is the gross area of the specimen in mm^2, which is calculated using Equation 2.

$$A_n = \left(\frac{w+h}{2}\right)t \qquad (2)$$

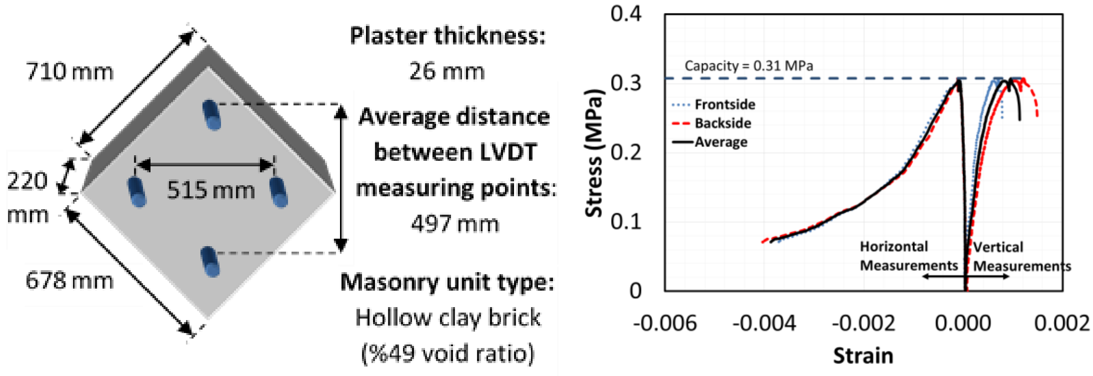
where, w is the width of the specimen in mm, h is the height of the specimen in mm and t is the total thickness of the specimen in mm.

On the other hand the axial deformation data was divided by the distance between the measuring points of LVDTs and strain values were obtained. Then, shearing strain was calculated using Equation 3.

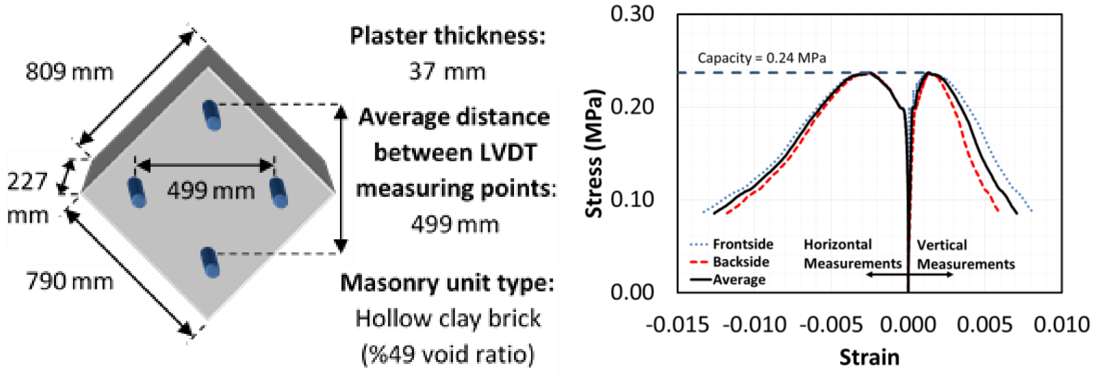
$$\gamma = \frac{\Delta V + \Delta H}{g} \tag{3}$$

where, γ is the shearing strain, ΔV is the vertical shorthening in mm, ΔH is the horizontal extension in mm and g is the vertical gage length in mm.

The data that was obtained through the calculations were displayed as a stress versus strain graph and the important values were shown on it. The experiment results obtained from diagonal tension tests are given in Figures 3.23 to 3.32 for specimens extracted from buildings 1 to 10, respectively.



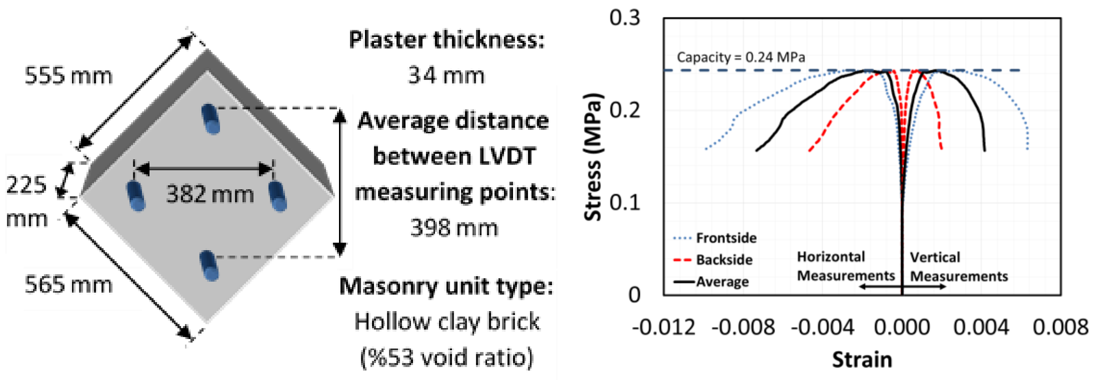
a) Geometric properties and diagonal tension test result of Specimen 1



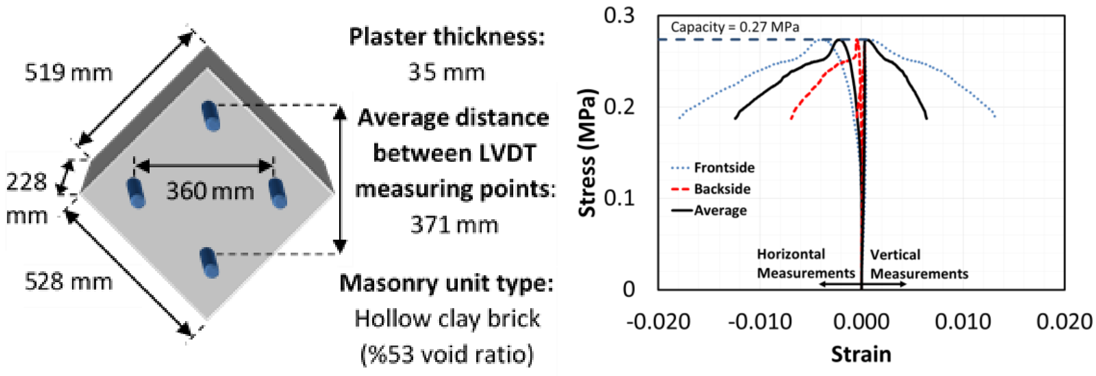
b) Geometric properties and diagonal tension test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.24 Building 1 diagonal tension test results



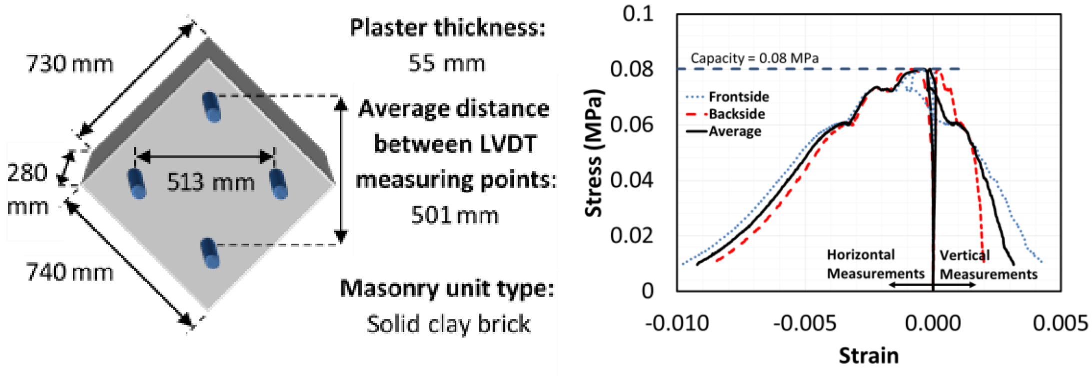
a) Geometric properties and diagonal tension test result of Specimen 1



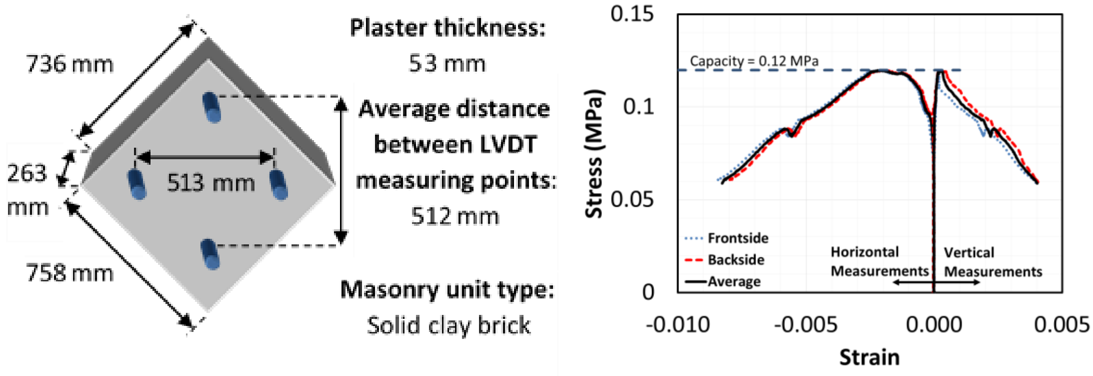
b) Geometric properties and diagonal tension test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.25 Building 2 diagonal tension test results



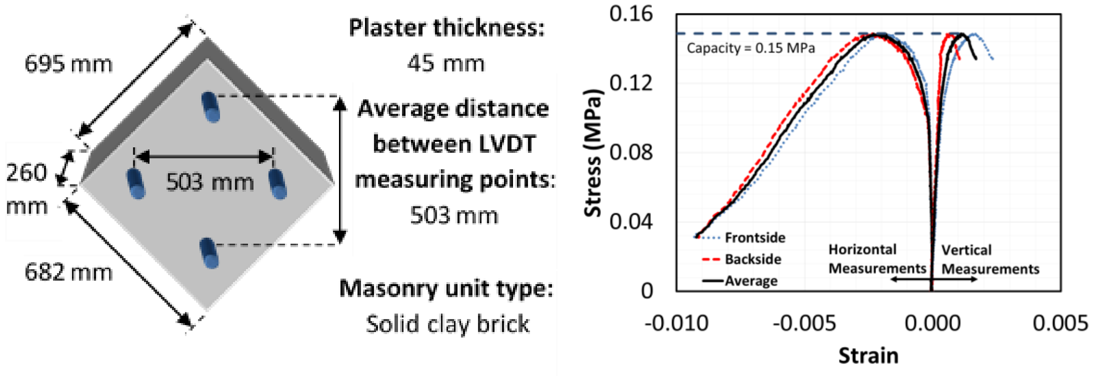
a) Geometric properties and diagonal tension test result of Specimen 1



b) Geometric properties and diagonal tension test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.26 Building 3 diagonal tension test results

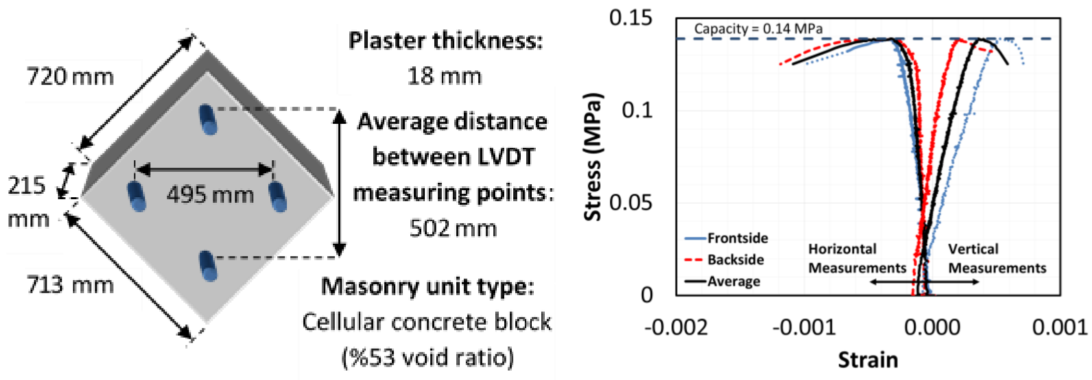


a) Geometric properties and diagonal tension test result of Specimen 1

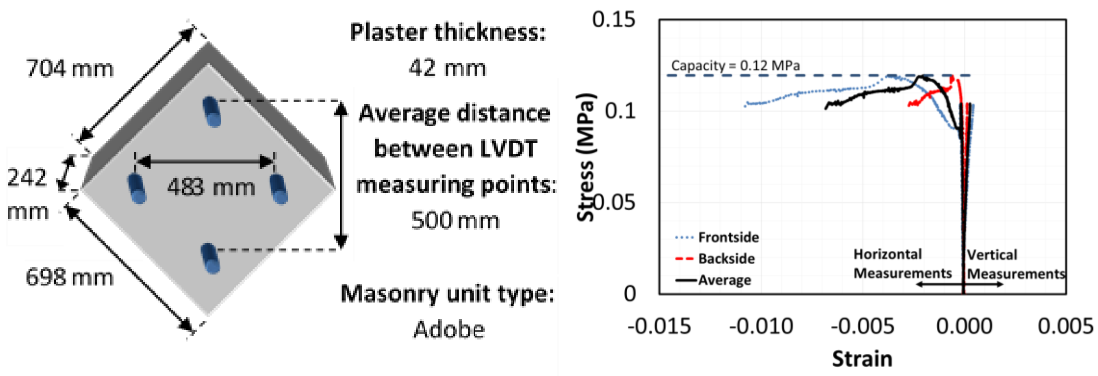
b) Specimen 1 at the end of the test

Figure 3.27 Building 4 diagonal tension test result

Due to the complications that occurred during the removal process, less number of specimens had to be taken and only 1 diagonal tension test was conducted for Building 4.



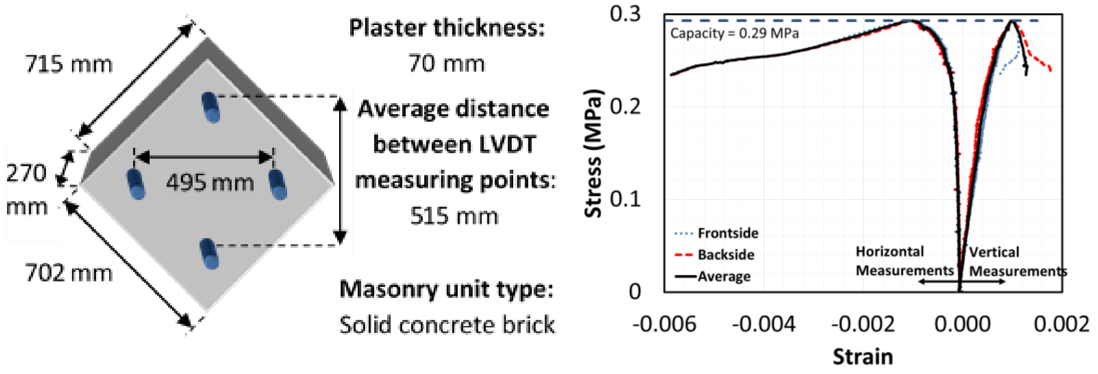
a) Geometric properties and diagonal tension test result of Specimen 1 (cellular concrete block)



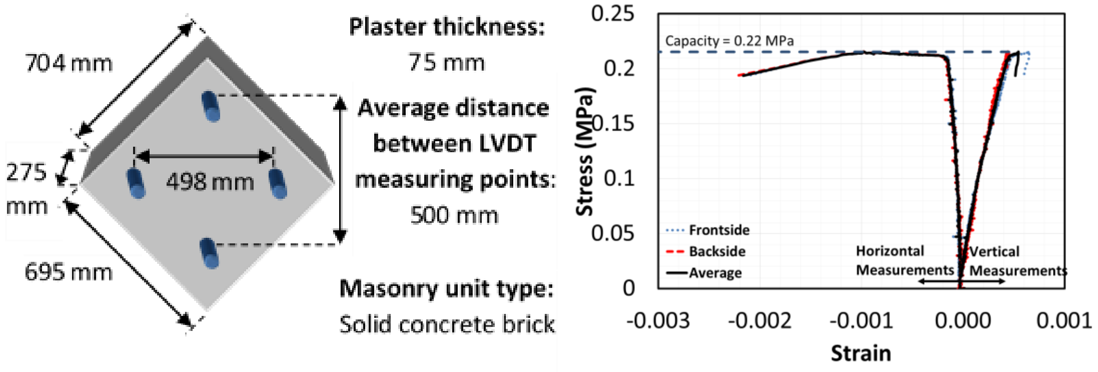
b) Geometric properties and diagonal tension test result of Specimen 2 (adobe)

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.28 Building 5 diagonal tension test results



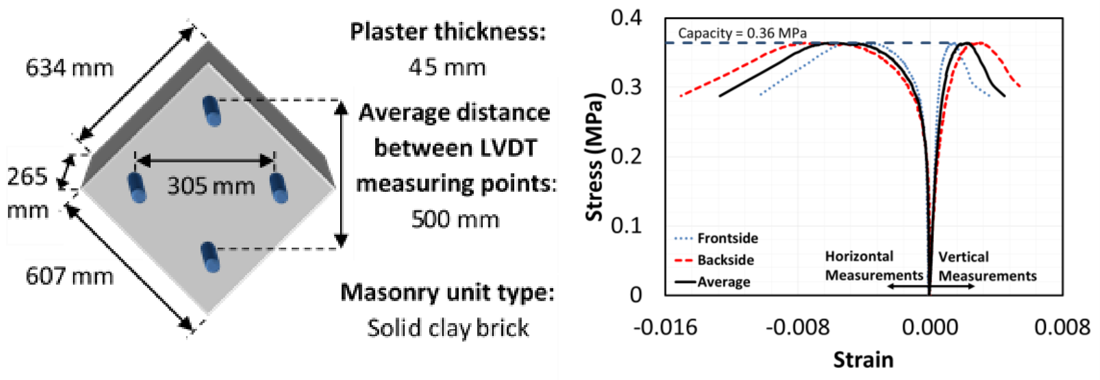
a) Geometric properties and diagonal tension test result of Specimen 1



b) Geometric properties and diagonal tension test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.29 Building 6 diagonal tension test results

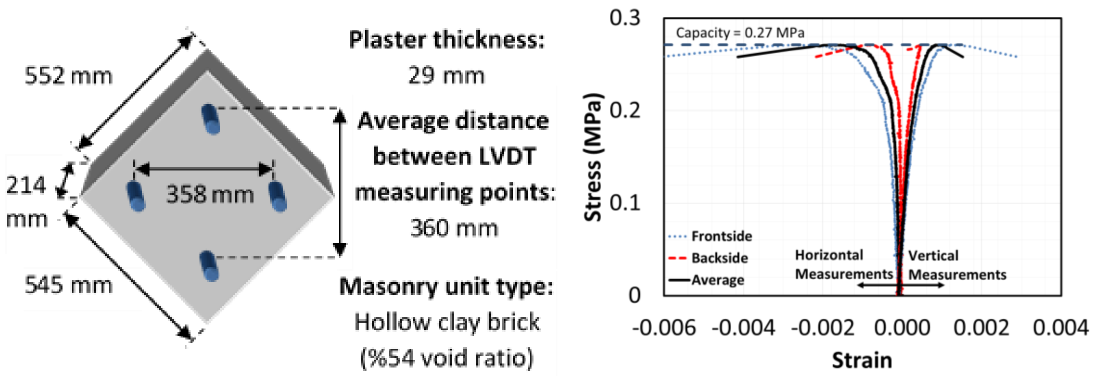


a) Geometric properties and diagonal tension test result of Specimen 1

b) Specimen 1 at the end of the test

Figure 3.30 Building 7 diagonal tension test result

Only one specimen was tested during the investigation of building 7. The main reason of this is the fact that this building was planned as the test structure for the second phase of the project. Therefore, it was aimed not to reduce the horizontal load carrying capacity of the building and stiffness properties of the building.

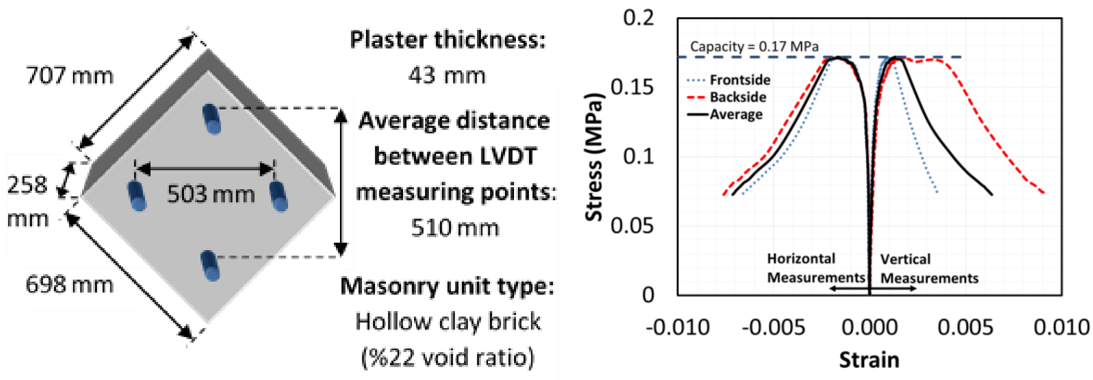


a) Geometric properties and diagonal tension test result of Specimen 1

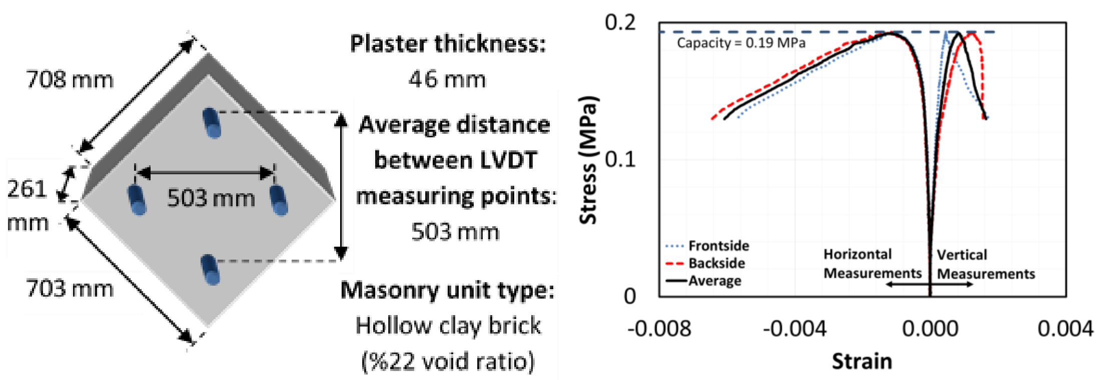
b) Specimen 1 at the end of the test

Figure 3.31 Building 8 diagonal tension test result

Only one specimen was tested during the investigation of building 8. The main reason of this is the fact that this building was planned as the test structure for the second phase of the project. Therefore, it was aimed not to reduce the horizontal load carrying capacity of the building and stiffness properties of the building.



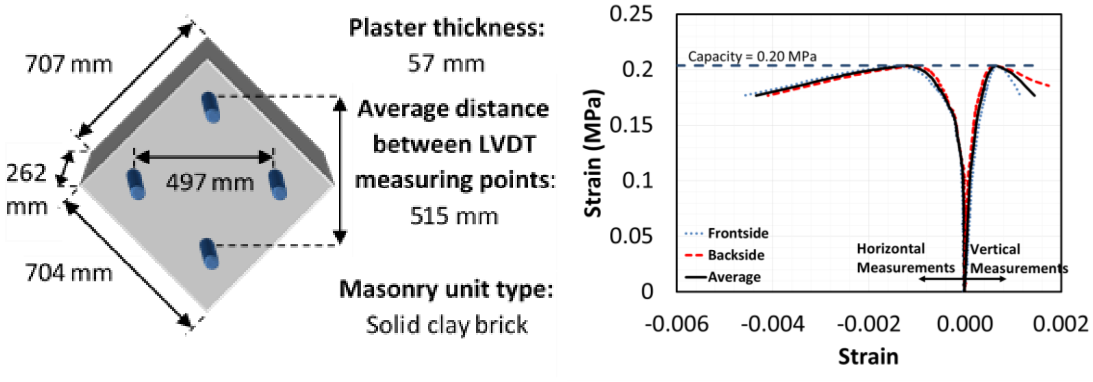
a) Geometric properties and diagonal tension test result of Specimen 1



b) Geometric properties and diagonal tension test result of Specimen 2

c) Specimen 1 and Specimen 2 at the end of the test

Figure 3.32 Building 9 diagonal tension test results



a) Geometric properties and diagonal tension test result of Specimen 1

b) Specimen 1 at the end of the test

Figure 3.33 Building 10 diagonal tension test result

In order to ensure the safety of the building during the removal process, less number of specimens had to be taken and only 1 axial compression test was conducted for Building 10.

The data obtained from diagonal tension test results and calculations are summarized in Table 3.1.

3.2.3 Sliding Shear Test

The sliding shear tests were made in order to determine the strength of the specimens under shear stress. Masonry shear bond strength was determined using a triplet test in accordance with European Testing Standard EN 1052-3 Methods of Tests for Masonry – Part 3: Determination of Initial Shear Strength. The specimens were tested along the axis perpendicular to the load bearing direction of the wall.

The specimens were prepared for the tests first by capping the ends. Due to the irregularities on the surface of the walls that occurred during the removal, capping all the specimens were necessary and this was applied with a gypsum cement capping material as stated in ASTM C1552-03a to upper and lower surfaces of the walls. After waiting approximately half a day for the cap to dry, the specimens were placed in the test machine with their bearing plates. In order to eliminate the contribution of the capping to the shear strength of the specimen, the cap is cut along the specimen where the bearing plates would be placed as shown in Figure 3.33. Then, the specimens' dimensions were measured and after being photographed, specimens became ready for testing. Photos representing the triplet test setup (Figure 3.34) are given below.

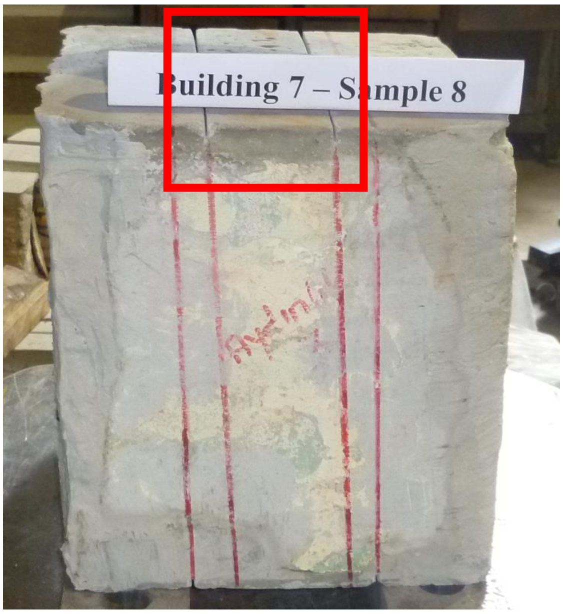
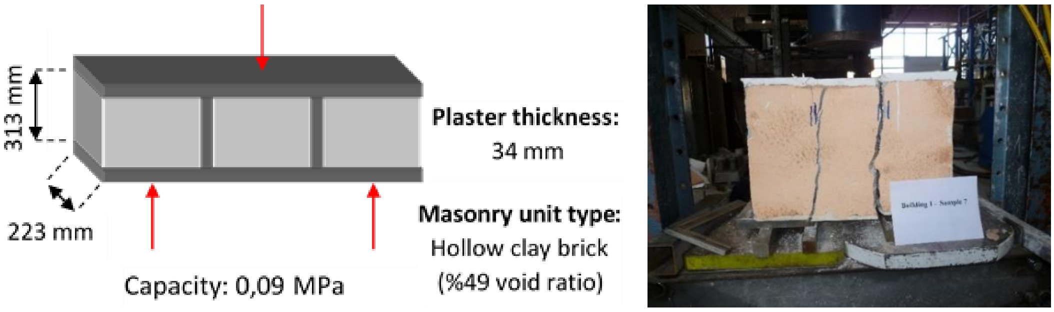


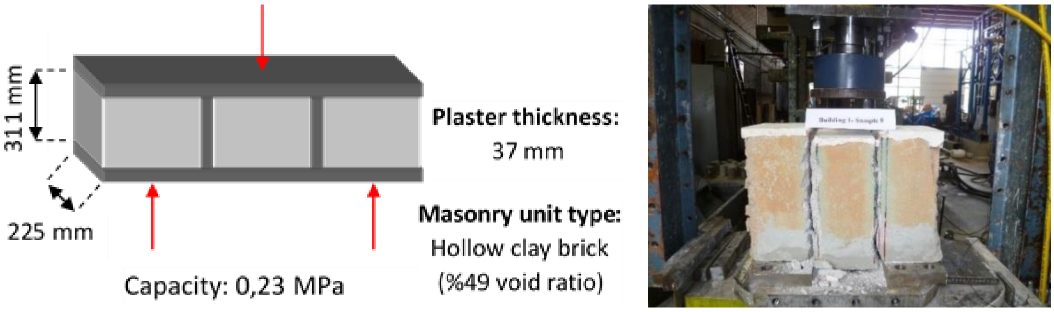
Figure 3.34 The cutting of the cap to eliminate its contribution to shear strength

Figure 3.35 Sliding shear test setup

After the experiments were finished, specimens were photographed and the load-deformation data were extracted. In order to calculate the shear strength of the masonry wall, the following procedure was used: Each compressive load data was multiplied with the gravitational acceleration and divided by the cross-sectional area of the specimen under shearing stresses. The maximum calculated stress value is taken as the shear strength. The experiment results obtained from sliding shear tests are given in Figures 3.35 to 3.43 for specimens extracted from buildings 1 to 10, respectively.

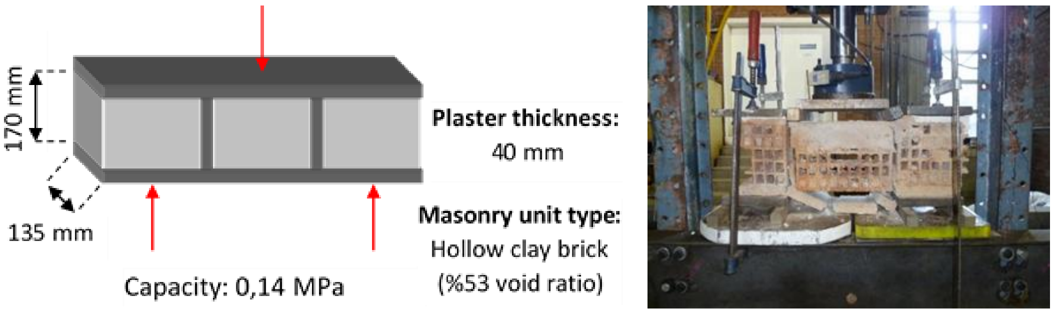


a) Geometric properties and triplet test result of Specimen 1

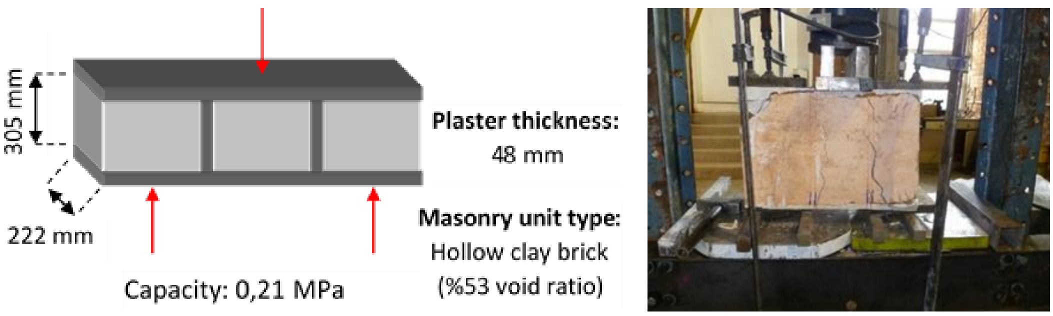


b) Geometric properties and triplet test result of Specimen 2

Figure 3.36 Building 1 triplet test results

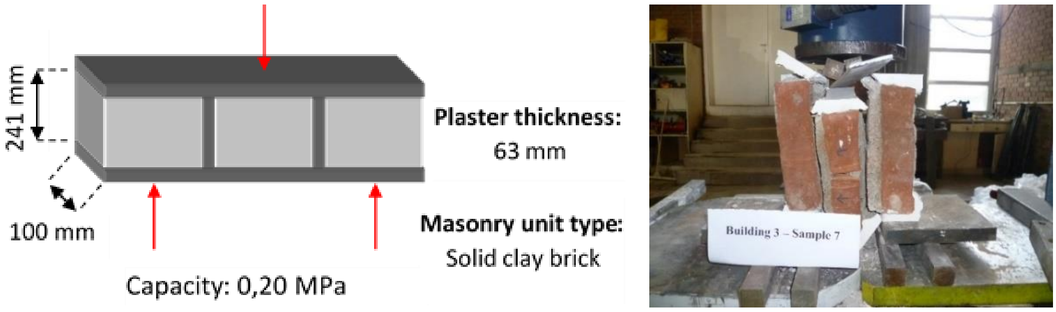


a) Geometric properties and triplet test result of Specimen 1

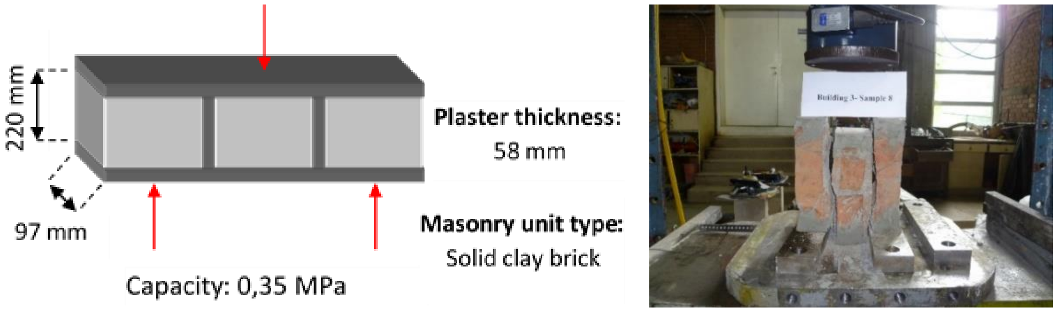


b) Geometric properties and triplet test result of Specimen 2

Figure 3.37 Building 2 triplet test results

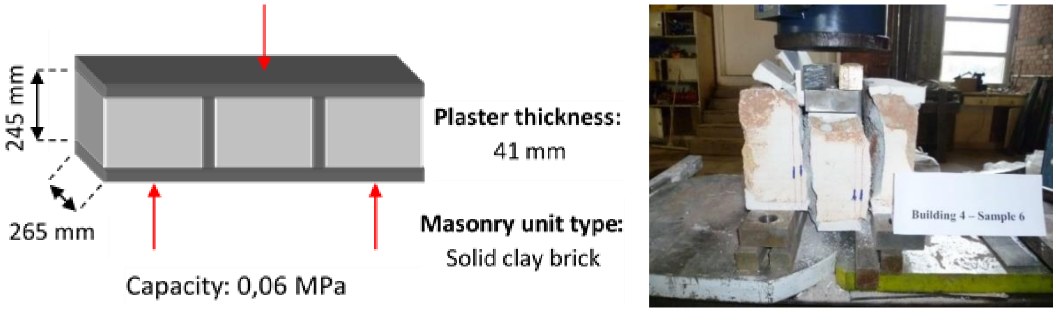


a) Geometric properties and triplet test result of Specimen 1

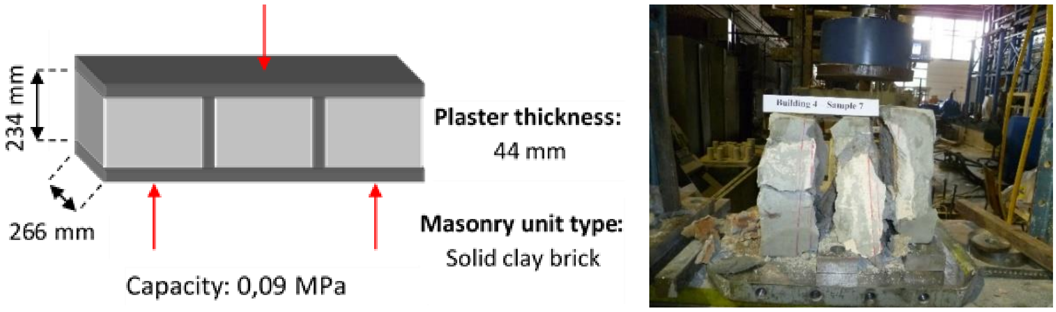


b) Geometric properties and triplet test result of Specimen 2

Figure 3.38 Building 3 triplet test results

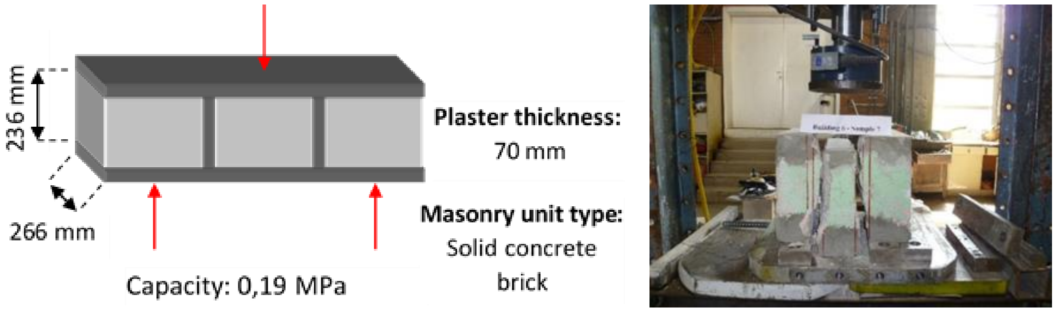


a) Geometric properties and triplet test result of Specimen 1

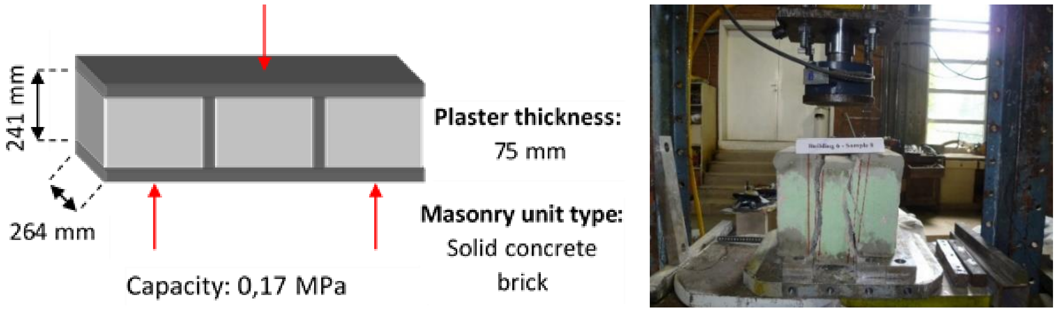


b) Geometric properties and triplet test result of Specimen 2

Figure 3.39 Building 4 triplet test results

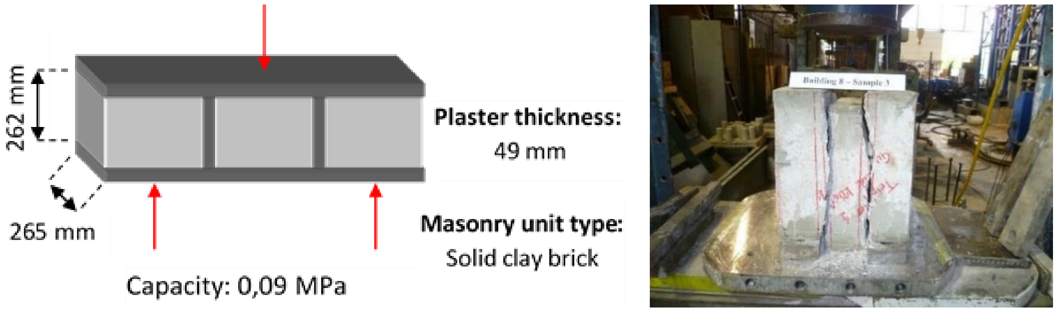


a) Geometric properties and triplet test result of Specimen 1

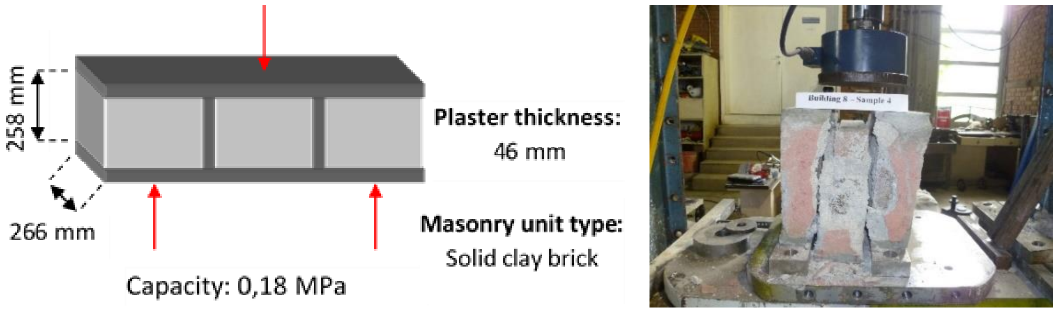


b) Geometric properties and triplet test result of Specimen 2

Figure 3.40 Building 6 triplet test results

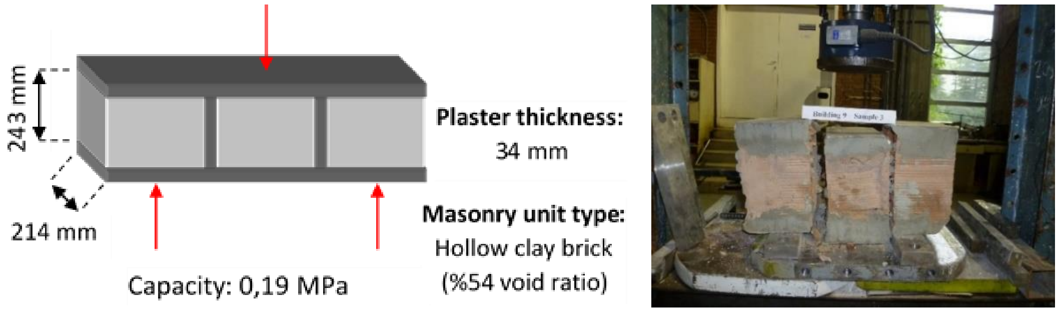


a) Geometric properties and triplet test result of Specimen 1

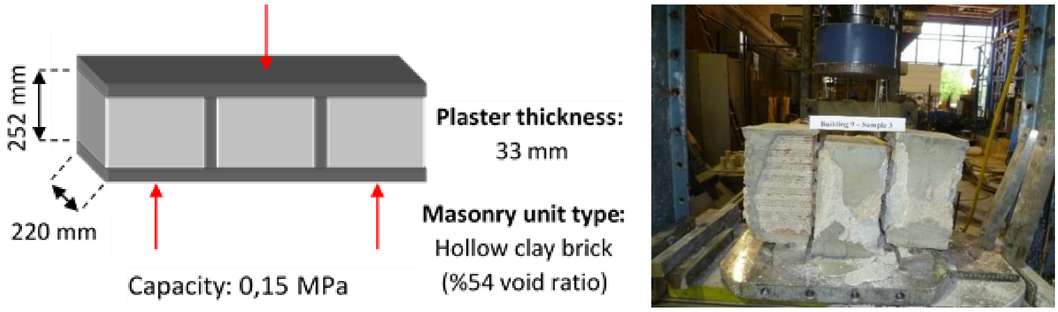


b) Geometric properties and triplet test result of Specimen 2

Figure 3.41 Building 7 triplet test results

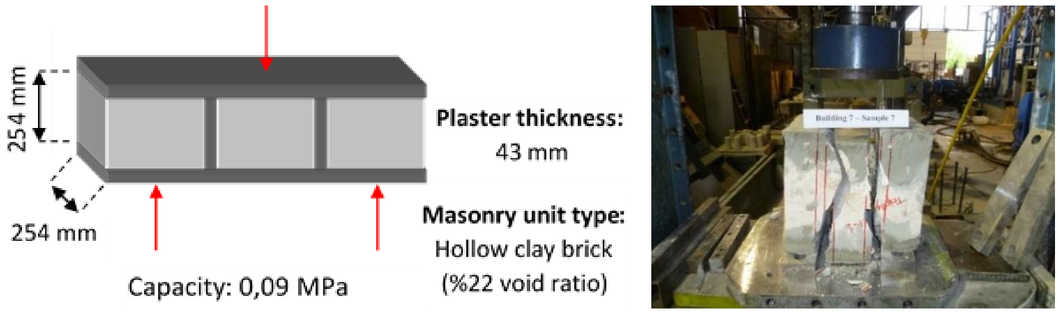


a) Geometric properties and triplet test result of Specimen 1

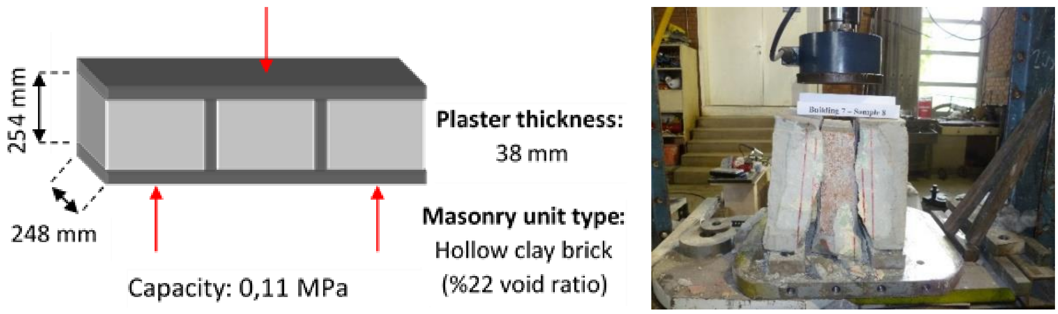


b) Geometric properties and triplet test result of Specimen 2

Figure 3.42 Building 8 triplet test results

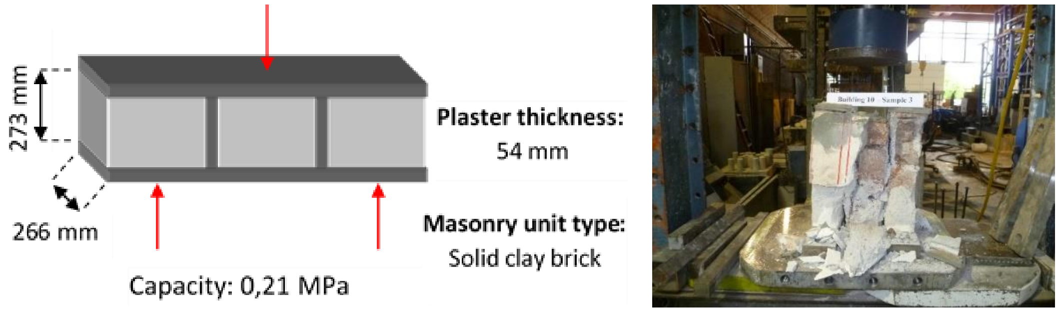


a) Geometric properties and triplet test result of Specimen 1

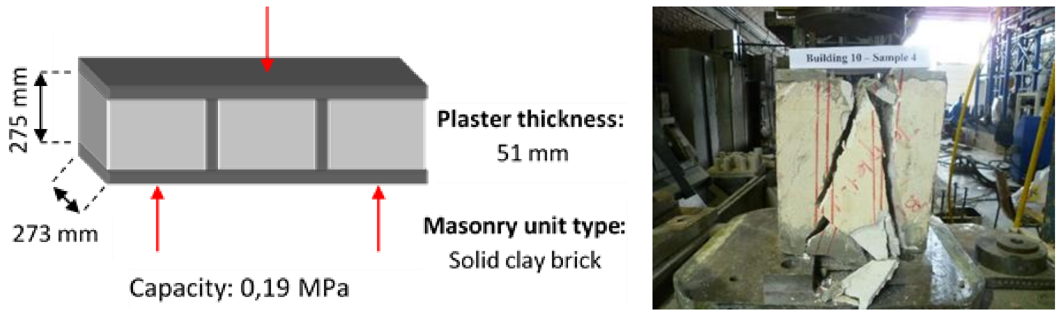


b) Geometric properties and triplet test result of Specimen 2

Figure 3.43 Building 9 triplet test results



a) Geometric properties and triplet test result of Specimen 1



b) Geometric properties and triplet test result of Specimen 2

Figure 3.44 Building 10 triplet test results

The data obtained from sliding shear test results and calculations are summarized in Table 3.1

3.3 Discussion of Results and Recommendations

The results obtained from the tests are given in Table 3.1. The material strength values given in Table 3.1 are compared with the allowable stress values recommended by TEC (2007) and the obtained modulus of elasticity values are compared with the values calculated with the equation given in TEC (2007). The allowable compressive and shear stress values depending on the masonry unit type provided by TEC (2007), are presented in Table 3.2 and Table 3.3, respectively.

Table 3.2 Allowable compressive stress values of walls (TEC, 2007)

Masonry unit and mortar type	**Allowable compressive stress (MPa)**
Hollow brick (void ratio less than %35, lime mortar with cement)	1.0
Hollow brick (void ratio between %35 and 45%, lime mortar with cement)	0.8
Hollow brick (void ratio more than %45, lime mortar with cement)	0.5
Solid brick (lime mortar with cement)	0.8
Stone (lime mortar with cement)	0.3
Autoclaved aerated concrete	0.6
Solid concrete block (cement mortar)	0.8

Table 3.3 Allowable shear stress values of walls (TEC, 2007)

Masonry unit and mortar type	**Allowable shear stress (MPa)**
Hollow brick (void ratio less than %35, lime mortar with cement)	0.25
Hollow brick (void ratio more than %35, lime mortar with cement)	0.12
Solid brick (lime mortar with cement)	0.15
Stone (lime mortar with cement)	0.10
Autoclaved aerated concrete	0.15
Solid concrete block (cement mortar)	0.20

The strength values that are presented in TEC (2007) show similarities with the capacity values obtained from the tests of specimens obtained from the field. This is a sign that can be interpreted as; the factor of safety for material capacities recommended for new design of masonry buildings were absent for some of the existing buildings.

The obtained capacity values are sorted according to the masonry unit types and presented in Table 3.4 while comparing with the range of capacity values given in TEC (2007). The values given in Table 3.2 and Table 3.3 are allowable stress values and while converting these into capacity values, a factor of safety coefficient of 2 must be used according to TEC (2007). Therefore, the capacity values presented in Table 3.4 for TEC (2007), were determined by multiplying the given allowable stress values in the code by the coefficient of 2.

Table 3.4 Comparison of test results with the values given in TEC (2007)

Properties	Masonry Unit Type					
	Hollow Clay Brick		Solid Clay Brick		Cellular and Solid Concrete Brick	
	Test results	TEC (2007)	Test results	TEC (2007)	Test results	TEC (2007)**
Compressive Strength (MPa)	0.9-2.3 (0.9-1.7)*	1.0-2.0	1.55-3.98 (0.95-1.1)*	1.60	0.49-1.51 (0.95-1.93)*	1.60
Shear Strength (MPa)	0.09-0.23	0.24-0.50	0.06-0.35	0.30	0.17-0.19	0.40
Diagonal Tension Strength (MPa)	0.17-0.31	NA	0.08-0.36	NA	0.14-0.29	NA

*: The compressive strength along the axis perpendicular to the load bearing direction of the wall

**: TEC (2007) provides capacities only for solid concrete brick

The obtained results imply that the specimen capacities show differences from the ones given in TEC (2007). It can be concluded that the compressive strength is related with the masonry unit type as presented in TEC (2007). On the other hand, it can be seen that shear and diagonal tension capacities do not show any correlation with the type of masonry units. The most significant reason is the fact that shear and diagonal tension strength are mostly related with the quality of the mortar. Therefore, categorizing these strength values depending on the quality of mortar, which is observed on the field, instead of the masonry unit type would be a better approach.

The capacity values can differ from region to region, building to building and even from wall to wall within the same building. The most accurate way of determining the capacity values would be performing tests on the specimens obtained from buildings. Nevertheless, in the absence of these tests it is essential to use lower bound values instead. Some lower bound capacity values can be recommended considering that the performed tests provide only a limited database for the subject. The recommended values are presented in Table 3.5. The recommended values must be used considering the thickness of both the masonry unit and the plastering together, since the tests were conducted without removing the plastering on the specimen.

Table 3.5 Recommended strength values based on masonry unit types

Masonry Unit Type	Compressive Strength (MPa)	Shear Strength (MPa)			Diagonal Tension Strength (MPa)		
		Observed mortar quality			Observed mortar quality		
		Poor	Medium	High	Poor	Medium	High
Hollow clay brick	0.90	0.05	0.1	0.2	0.1	0.15	0.25
Solid clay brick	1.00						
Cellular concrete	0.40						
Solid concrete brick	0.85						

In order to determine the shear and diagonal tension strength depending on the mortar quality, one can conduct tests on specimens extracted from the building as done in this study. In the absence of experiments, one can use the recommended strength values. If there are cracks or mortar discontinuities observed on the walls or mortar and plaster can be easily removed by hand, one can use the recommended capacity values for poor quality mortar. If none of these criteria are encountered the use of medium quality strength values are suggested. In the case of at least one test showing that strength found in the experiment is higher than the recommended values for high quality mortar, the use of high quality recommended values are suggested.

The compressive strength values were determined using the mean value and subtracting one and a half times the standard deviation value for each masonry unit type. Shear and diagonal tension values on the other hand were determined by categorizing the results into 3 levels according to the quality of the mortar used. The recommended lower bound values are given in Figure 3.44 and Figure 3.45.

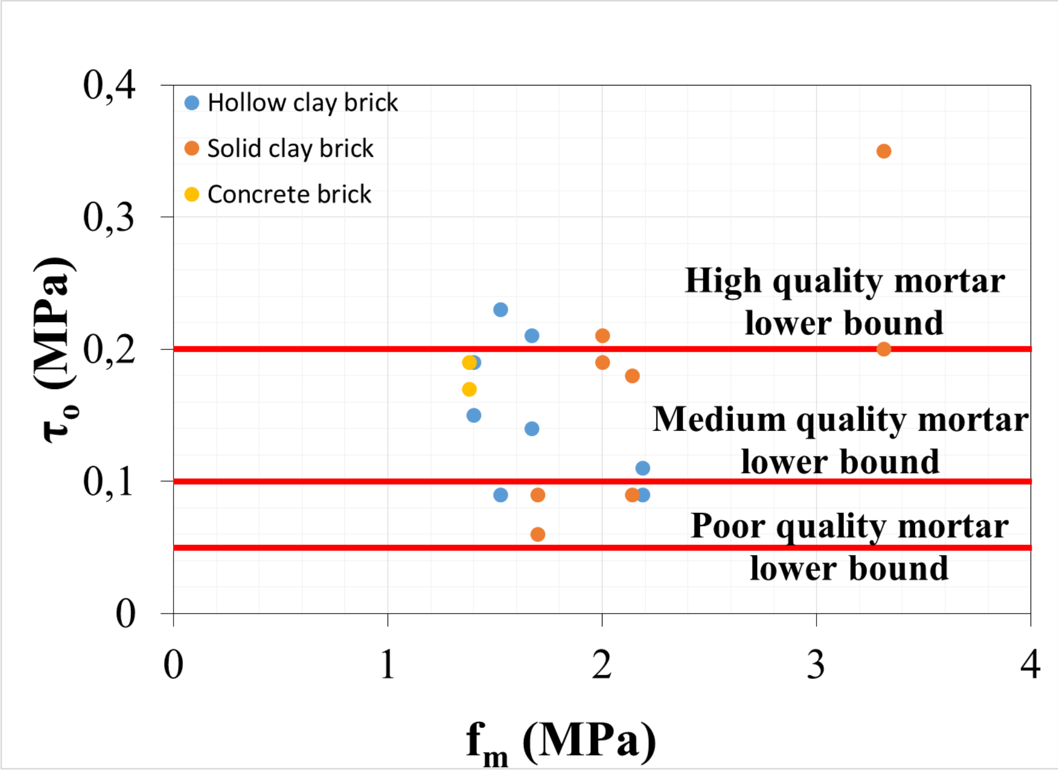


Figure 3.45 Relationship between shear strength and compressive strength

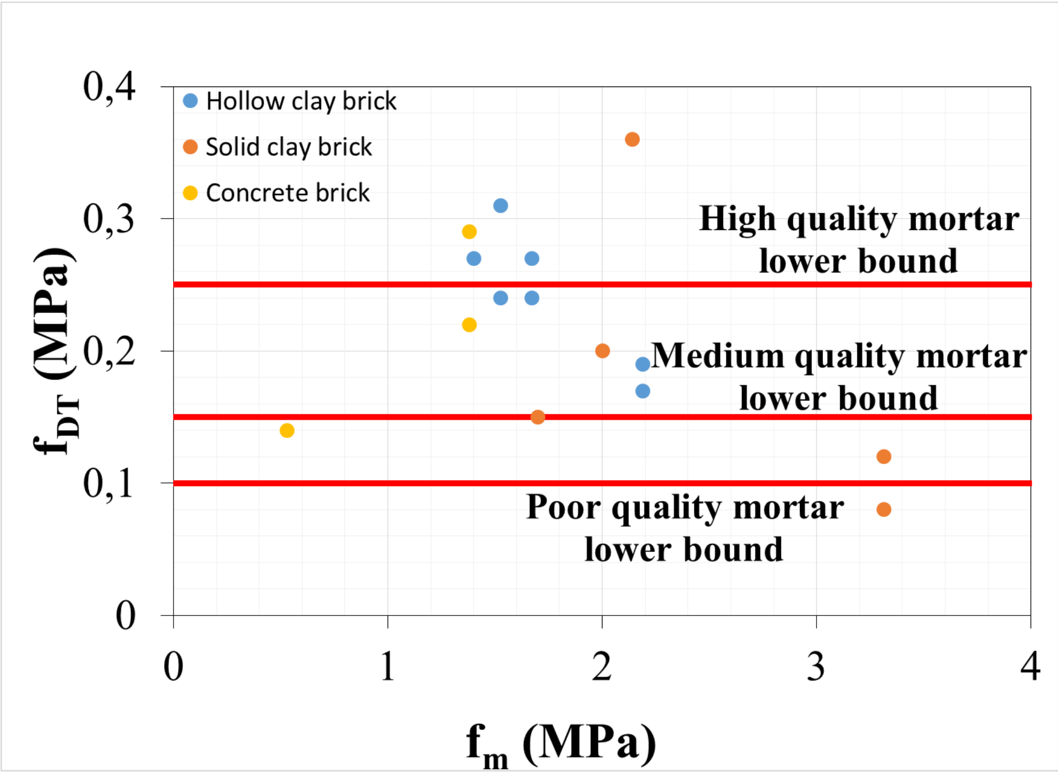


Figure 3.46 Relationship between diagonal tension strength and compressive strength

The relationship between the modulus of elasticity and the compressive strength based on the test results is presented in Figure 3.46. While the modulus of elasticity value is given in TEC (2007) as 200 times the capacity of the wall, the results show that a linear correlation with a slope of 1200 according to the test results is appropriate.

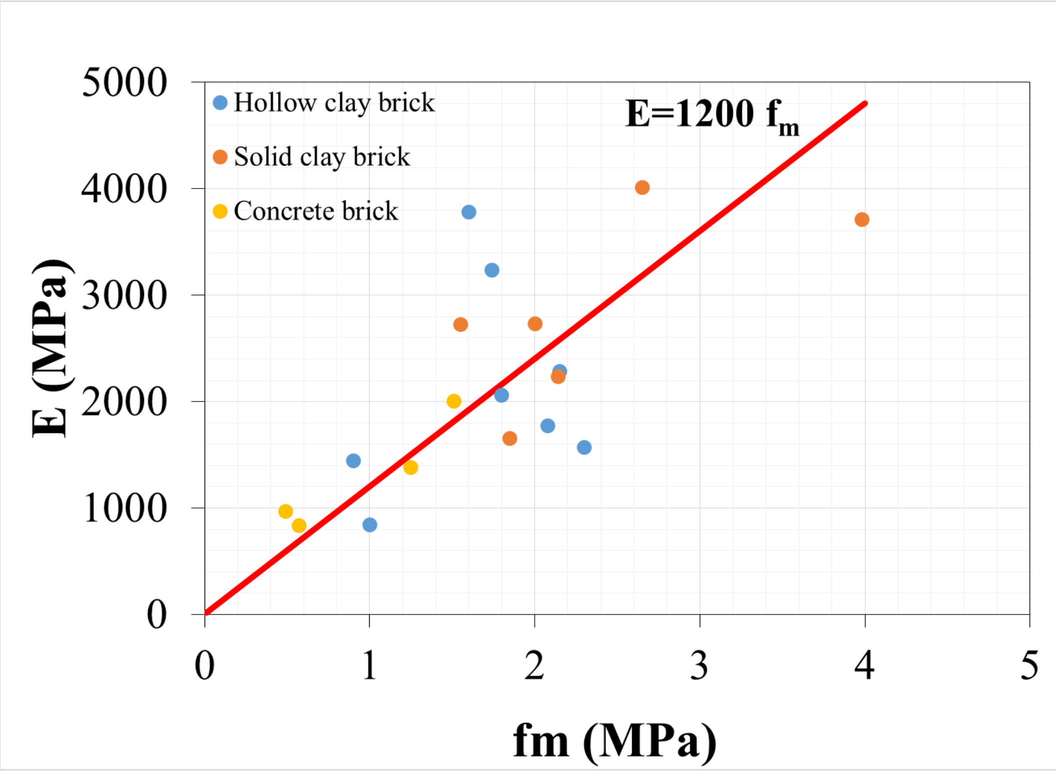


Figure 3.47 Relationship between the modulus of elasticity and compressive strength

All in all, the following equation is recommended for the calculation of the modulus of elasticity of masonry units.

$$E = 1200 f_m$$

To sum up, the recommended strength values present lower bound material characteristics for the conducted material tests. When compared with the allowable stress values given in TEC (2007), it can be concluded that the recommended values provide a safer approach to the subject.

CHAPTER 4

EFFECT OF MATERIAL PROPERTIES ON THE ASSESSMENT OF EXAMINED BUILDINGS

4.1 General

For the assessment and strengthening of existing buildings in Turkey, recently implemented codes namely the Guidelines for the Assessment of Buildings under High Risk (RBTEIE, 2013) and the Turkish Earthquake Code (TEC, 2007) are employed, respectively. Despite providing a new and detailed assessment method for reinforced concrete buildings, RBTEIE (2013) recommends to use the method defined in TEC (2007) for the assessment of masonry buildings with a change in the acceptance criteria. RBTEIE (2013) suggests using a different limit for the ratio of shear force contribution of the vulnerable walls to the total story shear demand. This limit is 50% in RBTEIE (2013), whereas TEC (2007) suggests to use a limit of 20%. In addition to this, there is also a difference in the recommended knowledge factors where TEC (2007) presents a factor of 0.75, while RBTEIE (2013) suggests a factor of 0.9. For the investigation conducted in this chapter, similar knowledge factor of 0.9 is used in order to eliminate the effect of it on the assessment results.

In this chapter; the assessment methods of masonry buildings for RBTEIE (2013) and TEC (2007) are described, the results of the assessments conducted for the ten selected buildings with the material strength values obtained from the test results, the allowable stress values provided in TEC (2007) and the material characteristics recommended in Chapter 3 are presented. Finally the effects of material characteristics on the assessment results are discussed.

4.2 Assessment Method of Masonry Buildings described in RBTEIE (2013) and TEC (2007)

The assessment procedure given in RBTEIE (2013) and TEC (2007) for masonry buildings are described briefly below. The objective of the assessment procedure is to determine whether the building is under high risk for a given seismic hazard level. This is accomplished by finding the vulnerable walls and comparing the calculated base shear force ratio (shear force carried by vulnerable walls to the total base shear force) to the limits given in TEC (2007) or RBTEIE (2013). The following procedure is conducted for the assessment of the selected buildings:

- From the obtained plan of the building, the dimensions and locations of each wall is determined.
- Necessary geometric properties such as the effective pier height and total wall thickness of the walls are determined.
- Vertical loads acting on each wall in the building and compressive stresses for each wall are calculated.
- The axial compression capacity of each wall is computed and the acting compressive stresses are checked whether they reach the calculated capacities.
- Equivalent seismic load procedure defined in TEC (2007) is computed with the seismic load reduction factor taken as 2 and the base shear force is computed.
- Depending on the relative shear stiffness of each wall segment, the shear force acting on each wall is computed by using the total story shear computed based on building weight and possible torsion effects.
- The shear stresses for each wall are computed for the shear force determined in the previous step and compared with the shear strength (or the allowable shear stress value in the case of TEC 2007 approach). If the strength of the wall is exceeded the wall is classified as vulnerable.
- Total shear force acting on the vulnerable walls divided by the total story shear force is compared with the limit given by TEC (2007) or RBTEIE (2013). If the limit is exceeded, the building is found to be under high seismic risk.

For each building, the assessment procedure is conducted by using three different material strength values: i) experimentally obtained material capacities, ii) allowable stress values provided by TEC (2007) and iii) the recommended capacity values for each earthquake zone.

4.3 Discussion of Results

The results of the assessment of 10 selected buildings using the material characteristics obtained from the test results described in the previous chapter, the allowable stress values provided in TEC (2007) and the values recommended in Chapter 3 are presented herein. Noting the fact that similar buildings can be seen all over the country, the assessment was also conducted for the same buildings with different earthquake zones. All results are given in terms of the ratio of base shear force of vulnerable walls to the total story shear of the critical story, named hereafter as the base shear ratio, given in percentages.

For the assessments conducted using the material strengths obtained from the laboratory tests, the minimum capacity values are taken as the compressive or shear strength of the wall, if two specimens were tested for the related material property. In order to assess the buildings with the material strengths given in TEC (2007), the allowable stress values depending on the masonry unit type for compressive and shear strength of the walls are used. For the assessment of buildings using the recommendations, the material strengths are determined by considering the masonry unit type for compressive strength and considering the observed mortar quality for the shear strength of the walls. The recommended material strengths used for the assessment of buildings are presented in Table 4.1.

Table 4.1 Material strengths used for the assessment

Building ID	Compressive Strength (MPa)			Shear Strength (MPa)		
	From laboratory tests	From TEC (2007)	From recommended values	From laboratory tests	From TEC (2007)	From recommended values
1	0.90	0.5	0.90	0.09	0.12	0.05*
2	1.60	0.5	0.90	0.14	0.12	0.05*
3	2.65	0.8	1.00	0.20	0.15	0.10**
4	1.55	0.8	1.00	0.06	0.15	0.05*
5	0.49	0.8	0.40	0.06	0.20	0.05*
6	1.25	0.8	0.85	0.17	0.20	0.10**
7	2.14	0.8	1.00	0.09	0.15	0.05*
8	1.80	0.5	0.90	0.15	0.12	0.05*
9	2.08	1.0	0.90	0.09	0.25	0.05*
10	2.00	0.8	1.00	0.19	0.15	0.05*

*: The observed mortar quality was selected as poor.

*: The observed mortar quality was selected as medium.

The assessment results of buildings considering their actual earthquake zones for three different cases are presented in Table 4.1. The light blue marks indicate that the base shear force ratio limit is exceeded according to TEC (2007), while the darker blue marks point out that the base shear force ratio limit is exceeded according to RBTEIE (2013). Obviously, a dark blue mark suggests that both code limits are exceeded.

For the case where actual material strength from laboratory tests are used, three and two buildings were found as under high risk (HR) according to TEC (2007) and RBTEIE (2013), respectively. On the other hand, upon using TEC (2007) default values, only one building was found as under high risk according to RBTEIE (2013) and three buildings were found as HR according to TEC (2013). The use of recommended strength values presented in Chapter 3 resulted in three buildings being under high risk according for both TEC (2007) and RBTEIE (2013). This situation shows the effect of assumed material properties on the assessment results.

Table 4.2 Assessment results consdering the buildings' actual earthquake zone

Building ID	Base shear force ratio					
	Material strengths from laboratory tests		Material strengths from TEC (2007)		Material strengths from recommended values	
	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)
1	12	50	7	50	12	60
2	0	0	0	0	0	20
3	0	0	0	0	0	0
4	7	0	7	0	17	0
5	0	0	0	0	0	0
6	35	0	26	0	53	19
7	53	38	26	14	53	38
8	0	0	0	0	0	0
9	0	0	0	0	0	0
10	0	0	0	0	0	0
Number of buildings under HR for TEC (2007)	3		3		4	
Number of buildings under HR for RBTEIE (2013)	2		1		3	

Although all the assessment results are based on the buildings' actual earthquake zone, one must consider that similar buildings can be encountered all over the country. With this perspective, it is important to investigate the behavior of these buildings under different earthquake zones. Therefore, all the buildings are assessed for all possible earthquake zones in TEC (2007). The results for the assessments conducted for each earthquake zone are presented through Table 4.2 and Table 4.4. The light blue marks indicate that the base shear force ratio limit is exceeded according to TEC (2007), while the darker blue marks point out that the base shear force ratio limit is exceeded according to RBTEIE (2013) and TEC (2007).

Table 4.3 Assessment results using the material strengths from the laboratory tests

Building ID	Base shear force ratio							
	4th Earthquake zone		3rd Earthquake zone		2nd Earthquake zone		1st Earthquake zone	
	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)
1	0	0	12	50	12	74	12	74
2	0	0	0	20	0	20	0	20
3	0	0	9	0	33	0	50	31
4	7	0	58	20	63	43	89	68
5	0	0	0	0	15	0	81	47
6	0	0	0	0	15	0	35	0
7	0	14	53	38	72	64	85	83
8	0	0	0	0	0	0	0	0
9	0	0	0	16	29	58	39	63
10	0	0	0	0	0	0	0	0
Number of buildings under HR for TEC (2007)	0		4		6		8	
Number of buildings under HR for RBTEIE (2013)	0		3		4		6	

Table 4.4 Assessment results using the material strengths provided by TEC (2007)

Building ID	Base shear force ratio							
	4th Earthquake zone		3rd Earthquake zone		2nd Earthquake zone		1st Earthquake zone	
	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)
1	0	0	7	50	12	60	12	74
2	0	0	0	20	0	20	0	33
3	0	0	9	0	46	31	50	31
4	7	0	50	4	58	20	61	38
5	0	0	0	0	0	0	0	0
6	0	0	0	0	15	0	26	0
7	0	0	26	14	53	55	72	58
8	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	16
10	0	0	0	0	0	0	0	0
Number of buildings under HR for TEC (2007)	0		4		5		6	
Number of buildings under HR for RBTEIE (2013)	0		2		4		4	

Table 4.5 Assessment results using the recommended material strengths

Building ID	Base shear force ratio							
	4th Earthquake zone		3rd Earthquake zone		2nd Earthquake zone		1st Earthquake zone	
	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)	X (%)	Y (%)
1	0	44	12	60	12	82	59	93
2	0	20	0	20	40	60	40	100
3	0	0	31	0	50	31	68	83
4	17	0	58	20	67	43	92	73
5	0	0	0	0	81	0	81	67
6	0	0	15	0	35	19	53	19
7	0	0	53	38	72	64	85	78
8	0	0	0	0	34	47	72	79
9	0	0	29	58	39	63	59	92
10	0	0	0	0	0	5	0	95
Number of buildings under HR for TEC (2007)	2		6		9		10	
Number of buildings under HR for RBTEIE (2013)	0		4		7		10	

The assessments conducted using the material strengths obtained from the laboratory tests and considering TEC (2007) base shear force ratio limit for the 4th, 3rd, 2nd and 1st earthquake zones, result with 0, 4, 6 and 8 buildings being under high risk for each zone, respectively. Taking the RBTEIE (2013) base shear force ratio limit into account with the 4th, 3rd, 2nd and 1st earthquake zones for the assessment results, the number of buildings that are under high risk are 0, 3, 4 and 6, respectively.

From the assessments results, which considered the allowable stress values provided by TEC (2007) as the material strengths and TEC (2007) base shear force limit ratio, for the 4th, 3rd, 2nd and 1st earthquake zones, it is clearly seen that 0, 4, 5 and 6 buildings are under high risk for each earthquake zone respectively. On the other hand, when the RBTEIE (2013) base shear force ratio is taken into account for the same earthquake zones, the number of buildings that are under high risk are 0, 2, 4 and 4, respectively. It is important to observe that number of buildings found HR using the actual material strength data is higher than the number found by using TEC (2007) default values. This shows that use of TEC (2007) values may not always safe results.

The assessments conducted using the recommended material strengths and considering TEC (2007) base shear force ratio limit for the 4th, 3rd, 2nd and 1st earthquake zones, result with 2, 6, 9 and 10 buildings being under high risk for each zone, respectively. Taking the RBTEIE (2013) base shear force ratio limit into account with the 4th, 3rd, 2nd and 1st earthquake zones for the assessment results, the number of buildings that are under high risk are found as 0, 4, 7 and 10, respectively. As expected, the use of recommended values provides the safest estimate among the three cases.

According to these results it is clearly seen that the assessment of masonry buildings are much more crucial for the buildings located in higher earthquake zones and comparing the assessment results for a specified earthquake zone might give a better understanding of the effect of material characteristics on the assessment results.

For the 1st earthquake zone, 8 buildings are under high risk with material strengths obtained from the laboratory tests and 6 buildings are under high risk when the allowable stress values provided by TEC (2007) are taken into account, while using the recommended material strengths results with all of the buildings being under high risk assuming that TEC (2007) base shear ratio limit is valid. On the other hand, the number of buildings that are under high risk are 6, 4 and 10 respectively if RBTEIE (2013) base shear force limit ratio is taken in account. Depending on these results, it can be concluded that using the allowable stress values given in TEC (2007) for the assessment of masonry buildings, might provide inaccurate and unsafe results. This can be clearly seen by examining the assessment results of each building separately.

Assuming that RBTEIE (2013) base shear force ratio limit is sufficiently accurate, the assessment results of Buildings 1, 4 and 6 showed similarities. The results of assessments using 2nd earthquake zone are taken into account for Buildings 1 and 4 while 1st earthquake zone is considered for Building 6. All the assessment results despite the material characteristic values used, indicate that these buildings are under high earthquake risk. However, using TEC (2007) allowable stress values for the assessment underestimated the base shear force ratio when compared with the assessment results that used the experimentally found by using strength values. On the other hand, using the recommended values for material characteristics overestimates the base shear force ratio, thus providing a safer approach.

The assessment results of Buildings 5, 7 and 9, also showed similarities, assuming that RBTEIE (2013) base shear force ratio limit is valid. Results from 1st earthquake zone is considered for Buildings 5 and 9, whereas 3rd earthquake zone results are taken into account for Building 7. For each building, the assessment results using TEC (2007) allowable stress values indicated that buildings 5, 7 and 9 are not under high earthquake risk, while the results obtained by using the actual capacities clearly show that the building is under high risk instead. This difference is very important since it implies that the allowable stress values provided by TEC (2007) might lead to inaccurate assessment results. Meanwhile the assessment results that used recommended values, show that it overestimates the base shear force ratio and remains on the safe side.

Results from 1st earthquake zone are used to discuss the findings from Buildings 2 and 3, assuming that RBTEIE (2013) base shear force ratio limit is valid. The results of both of the buildings showed that they are both under high earthquake risk regardless of the material strength used in the assessment. However, allowable stress values given by TEC (2007) and recommended capacity values, provides a safer approach this time.

For Buildings 8 and 10, assessment results using 1st earthquake zone are considered, assuming that RBTEIE (2013) base shear force ratio limit is valid. The results obtained by using the actual capacity values and the allowable stress values show that these buildings are not under high earthquake risk. On the other hand, using the recommended capacity values for the assessment still provides a safe approach.

Considering the above discussion, it is clearly seen that, the reliability of the material characteristics given by TEC (2007) are questionable and it can be concluded that using the allowable stress values provided by TEC (2007) for the assessment of masonry buildings might lead to inaccurate results, when compared with the assessment results that used material strengths obtained from laboratory tests. Therefore, considering the use of the recommended values for the assessment of masonry buildings would be a safer approach.

CHAPTER 5

CONCLUSION

5.1 Summary

In this study, the material characteristics of masonry buildings in Turkey and its effect on assessment methods were investigated. The main objectives were to determine the material characteristics for the selected masonry buildings and compare them with the strength values provided by TEC (2007), investigate the importance of material characteristics on the assessment methods defined in TEC (2007) and RBTEIE (2013), obtain a better feeling of the preliminary assessment method and the calculated performance scores described in RBTEIE (2013) for masonry buildings and recommend material characteristics for the assessment of masonry buildings in Turkey.

In this regard, firstly ten buildings were selected for the study and detailed information regarding the selected buildings were collected on the field. After that, the preliminary assessment method described in RBTEIE (2013) was conducted for the selected buildings and the results were comparatively evaluated. Then, axial compression, diagonal tension and sliding shear tests were conducted in the laboratory on the wall specimens obtained from the selected buildings. All the test results were discussed along with the comparison with the strength values given in TEC (2007) and recommendations for new material characteristics for masonry buildings were made accordingly. Finally, assessment methods described in TEC (2007) and RBTEIE (2013) were conducted to the selected buildings with different material characteristics for all the earthquake zones. The results were compared and the importance of material characteristics on the assessment methods were discussed.

5.2 Conclusions

Conclusions drawn from this study can be stated as follows;

- The preliminary assessment results showed that the performance scores of vulnerable masonry buildings varied between 60 and 90 for the selected buildings.
- The material strengths obtained from the laboratory tests were similar in some cases with the allowable stress values recommended by TEC (2007). In some other cases, even lower strength values than those recommended by TEC (2007) are found. This can be interpreted as; the factor of safety for material capacities can be absent for some of the existing buildings.
- Depending on the laboratory tests, it is clearly seen that the compressive strength of masonry walls depend on the masonry unit type as stated in TEC (2007). On the other hand, diagonal tension and shear strengths show no correlation with the masonry unit type. The quality of mortar seems to be related with the diagonal tension and shear strengths instead.
- Compressive, diagonal tension and shear strength values were recommended considering the fact that the conducted tests only provide a limited database for the subject. Thus, the recommended strength values should be considered as lower bound values. In the current form of the TEC (2007) expressions, wall strength is computed based on only bed joint sliding failure mode. It is clear that this mode has increasing capacity with increasing axial stress. This situation may cause unreasonable estimates of wall strength for high axial stresses. It is recommended to include diagonal tension failure mode in wall strength calculations using the recommended diagonal tensile strength values for an accurate assessment approach.
- The assessment results showed that, the assessments conducted with the allowable stress values provided by TEC (2007) might lead to inaccurate and unsafe results when compared with the assessments conducted with the material capacities obtained from the laboratory tests. Thus, it can be concluded that the reliability of the allowable stress values given in TEC (2007) for masonry walls are questionable.

- The recommended strength values provide a safer approach to the subject, when compared with the material capacities obtained from laboratory tests and the stress values provided by TEC (2007), according to the assessment results.
- The database for the material characteristics of existing masonry buildings in Turkey should be expanded with further experiments, so that the reliability of allowable stress values presented in TEC (2007) along with the recommended strength values presented in this study, can further be investigated.

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