INVESTIGATION OF A DAMAGED HISTORICAL MOSQUE WITH FINITE ELEMENT ANALYSIS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

G. ÇAĞIL KÖSEOĞLU

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

JUNE 2011

Approval of the thesis:

INVESTIGATION OF A DAMAGED HISTORICAL MOSQUE WITH FINITE ELEMENT ANALYSIS

Submitted by **G. ÇAĞIL KÖSEOĞLU** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Dr. Canan Özgen
Dean of Graduate School of **Natural and Applied Sciences** ____________

Prof. Dr. Güney Özcebe
Head of Department, **Civil Engineering** ____________

Assoc. Prof. Dr. Erdem Canbay
Supervisor, **Civil Engineering Department, METU** ____________

Examining Committee Members:

Dr. Erhan Karaesmen
Civil Engineering Department, METU ____________

Assoc. Prof. Dr. Erdem Canbay
Civil Engineering Department, METU ____________

Prof. Dr. Uğurhan Akyüz
Civil Engineering Department, METU ____________

Prof. Dr. Ali İhsan Ünay
Architecture Department, METU ____________

Assoc. Prof. Dr. Barış Binici
Civil Engineering Department, METU ____________

Date: ____________

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

Name, Last Name: G. Çağıl Köseoğlu

Signature :

ABSTRACT

INVESTIGATION OF A DAMAGED HISTORICAL MOSQUE WITH FINITE ELEMENT ANALYSIS

Köseoğlu, G. Çağıl
M.Sc., Department of Civil Engineering
Supervisor : Assoc. Prof. Dr. Erdem Canbay

June 2011, 111 pages

Historic structures form a very important part of our cultural heritage and should be well protected. Therefore, full comprehension of the structural behavior of historic structures is of prior importance.

A seriously damaged single domed mosque of 16th century Classical Ottoman Architecture was investigated in this study. Serious damages have been observed at various structural elements including the dome and the structural masonry walls, recently leading the structure's closure to service. The main objective of this study is to find out the possible reasons of the damage. The Mosque was constructed on silty-clay soil and the water table has been changed considerably due to the drought in recent years causing soil displacements. The structure is modeled with linear finite element approach. The masonry walls are modeled with homogenized macro shell elements.

The change in water table is imposed on the Mosque as displacement at foundation joints. The results of the analyses have been compared with the observed damage and the finite element model has been calibrated according to the observed damage. Some rehabilitation methods have also been proposed. Mini pile application up to firm soil (rock) was recommended to prevent the soil displacement. A steel ring around the damaged dome base was proposed to avoid any further propagation of cracks. Furthermore, the cracks on the masonry walls should also be repaired with a suitable material that is also compatible with the historic texture.

Keywords: Historic Structures, Modelling, Damage Analysis, Masonry, Structural Analysis of Historic Structures

ÖZ

HASARLI TARİHİ BİR CAMİNİN SONLU ELEMANLAR ANALİZİ İLE İNCELENMESİ

Köseoğlu, G. Çağıl
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi : Doç. Dr. Erdem Canbay

Haziran 2011, 111 sayfa

Tarihi yapılar kültürel mirasımızın önemli bir bölümünü oluşturduğundan dolayı iyi korunmaları gerekmektedir. Bundan dolayı tarihi yapıların davranışının anlaşılması çok önemlidir.

Bu çalışmada 16 yüzyıl klasik Osmanlı mimarisinde tek kubbeli hasarlı bir cami incelenmiştir. Kubbede ve taş duvarlarda gözlenen aşırı çatlaklar caminin kapatılmasına sebep olmuştur. Çalışmanın ana amacı hasarın olası sebeplerinin araştırılmasıdır. Cami siltli-kil üzerine inşa edilmiştir. Son yıllardaki kuraklıklar zemin su tablası aşırı şekilde değiştirmiştir. Su tablasındaki değişime bağlı olarak siltli kil zeminde farklı oturmalara sebep olmuştur. Yapı doğrusal sonlu elemanlar metoduyla modellenmiştir. Taş duvarlar ise homojen makro kabuk elemanlarla modellenmiştir. Su tablası değişimi zemin oturması sebebiyle camiye temel mesnetlerinde deplasman olarak verilmiştir. Analizlerin sonuçları gözlemlenen gerçek hasarla karşılaştırılmış ve sonlu elemanlar modeli hasarla uyumlu olarak kalibre edilmiştir. Bazı güçlendirme/tamir etme metodları da önerilmiştir.

Zemin oturmasına bağlı deplasmanları engellemek için sert kaya zemine kadar mini fore kazık uygulaması önerilmiştir. Kubbedeki çatlakların ilerlemesini engellemek amacıyla da kubbe kaidesi etrafına çelik plaka kasnağı konulması önerilmiştir. Taş duvarlardaki çatlaklar da uygun bir malzeme ile tarihi dokuya da uygun olacak şekilde kapıtılmalıdır.

Anahtar Kelimeler: Tarihi Yapılar, Modelleme, Hasar Analizi, Yığma Yapı Sistemleri, Tarihi Yapıların Yapısal Analizi

To Harika Köseoğlu, Peker Köseoğlu and Irmak Köseoğlu

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my supervisor Assoc. Prof. Dr. Erdem Canbay for his invaluable guidance, support and advices and his generous help throughout the research.

I would also like to thank Prof. Dr. Güney Özcebe for his support during my career in engineering profession.

Additionally, thanks to Dr. Erhan Karaesmen, Prof. Dr. Ali İhsan Ünay, Assoc. Prof. Dr. Barış Binici, Assoc. Prof. Dr. Ahmet Türer and Prof. Dr. Kemal Önder Çetin for their precious suggestions and comments for this study, Prof. Dr. Ayşıl Yavuz and Asst. Prof. Dr. Ufuk Serin for their guidance throughout the "REST533 Historic Structural Systems" classes.

Also, thanks to Volkan Kalpakcı for his advices and help.

Special thanks to my friends beside me with their endless support and encouragement through my study. Many thanks to Duygu Güleyen, for her true friendship and great understanding and Fuat Kızılpınar, for his sincere friendship and sense of humor.

Sincere thanks to my family, my mother Harika Köseoğlu, my father Peker Köseoğlu and my sister Irmak Köseoğlu for their love and support and their endless faith in me.

Finally, I am vastly greatful to Mustafa Kemal Atatürk for myself being able to write this thesis as a woman.

TABLE OF CONTENTS

LIST OF TABLES

TABLES

LIST OF FIGURES

FIGURES

CHAPTER 1

INTRODUCTION

1.1 General

Historic structures are works of art and guides for evaluation of a nation's past and its economic and cultural progress through time. Although possessing the value of being master pieces, these structures mostly adopt the rules-of- thumb rather than engineering methods, making them highly vulnerable. Therefore, they are usually damaged either partially or completely throughout the courses of time.

Several intervention methods have been proposed for damaged historic structures, each study being unique, due to the specific characteristics of each study. However, in all the methods it is seen that most of these structures, being master pieces and standing still for centuries, even though being damaged, are worth being protected and preserved. The type and quality of materials and the extent of structural damage should be considered while carrying out the analysis. Therefore, rules of preservation and restoration should be taken into consideration on application of all methods. This, especially in the case of interventions on culturally protected historic structures, limits the application of many methods.

1.2 Research Needs

Historic structures form the cultural texture of a civilization; therefore, full comprehension of their structural behavior is crucial. A complete investigation of the structure should be done before any intervention on historic monuments. The process becomes especially demanding for masonry structures due to complex material and geometrical properties and lack of data about the original state of the structure. Therefore, these structures should be treated with specific methods without worsening the state of the structure, grasping the cause of damage.

In this study, the finite element analysis method is selected to observe the complete behavior of the structure.

1.3 Objective and Scope

In the study, a seriously damaged mosque has been chosen as the case study. The main objective is to find out the reason of the damage and finally propose a suitable rehabilitation method.

The aim is to provide general perception of the behavior and analysis of historic masonry structures under different load combinations rather than a detailed step wise guide. The properties and structural analysis of masonry structures are mentioned together with pointing out the properties of the case study in detail.

1.4 Procedure

The study is carried out in several steps. Firstly, a seriously damaged case study is examined. The structure is a damaged, 16^{th} century Ottoman masonry mosque constructed by "Hassa Mimarlar Ocağı" who is guided by architect Sinan (Başkan, 1993) and is located at Ulucanlar Avenue, Ankara. The main reason of choosing the structure is the observations of formation of severe cracks propagated from the ground level up to the main dome of the mosque on such a structure that stood still for over 500 years and finally lead to its closure to service. The need to estimate the reasons of the damage and find adequate solutions to prevent it from going further, urged this study.

In the second step, the soil on which the structure sits is investigated and the ground investigation report, prepared by Middle East Technical University, Department of Civil Engineering is taken into consideration. (Canbay, Çetin, 2008) According to this report, the soil consists of three different layers. In the first layer on top, sand up to 2 meters depth and underneath the first layer, silty-clay from 7-10 meters depth and beyond that the andesite stone layer exists. In the soil investigation, the main soil problem is reported to be the high swelling-shrinkage potential of the clay layer which will be discussed in the following chapters.

In the last step of this study, the analytical model of the structure is constructed and the analysis of it, under certain load combinations, is studied. In the analysis stage, the ground data that are obtained by calculations are used as input values via SAP 2000 software. The analysis results are used for comparison with the current state and the crack patterns observed on the structure. Finally, a suitable rehabilitation method is proposed for the studied structure.

CHAPTER 2

MASONRY IN GENERAL

2.1 Evolution of Masonry Construction

Masonry is referred to the building systems formed by piling masonry units on top of each other made of stone, adobe or brick together with a binding material. It has been known to be one of the oldest construction systems where the unit material is quarried from the parent rock and then carved.

The masonry construction process is conducted through certain stages in ancient times which may be summarized as; (Camp and Dinsmoor, 1984, Crouch, 1985)

- Supplying the material.

- Quarrying of the material in guidance of the architect. (Figure 2.1)

- Transporting the material, lifting and laying the blocks in position.

Figure 2.1 Quarrying stages of limestone on Walls of Dara (Thais, 2010)

Every nation, depending on the geography of its setting, used variety of transportation and lifting methods for construction materials. Egyptians used ramps of earth for transportation of blocks and Greek used pulleys, Athens developed special systems like "Lifting Bosses", in which remains of extra stacks of stone on the face of the wall are left for handling purpose usually removed at the end of construction and a special method called "Lewis", for grasping rather smaller blocks. (Camp and Dinsmoor, 1984) (Figure 2.2)

Figure 2.2 Lewis Lifting Device (Camp and Dinsmoor, 1984)

The masonry construction has been accepted as one of the oldest structural systems originated from its ancient forms. (Crouch, 1985) Early Sumerians (3000 BC) built their dwellings by producing masonry units of roughly shaped mud bricks. (Braun, 1959) The Sumerian Ziggurat and the well known stone masonry Egyptian Pyramids (2800-2000 BC) as seen below, are one of the oldest examples of monumental architecture.

Figure 2.3 Ziggurat at Ur (Watkin, 2005)

Figure 2.4 Egyptian Pyramids (Oliveira, 2003)

In terms of load carrying systems, one of the first examples of the masonry system is “Post and Lintel” where the horizontal lintels transfer the structural load to the vertical elements called posts. An early example in use of this system is the Stonehenge in United Kingdom and was also observed at the Temple of Amun, Karnak (Figure 2.5).

Figure 2.5 The Great Temple of Amun, Karnak (Fletcher, 1996)

For longer spans, use of “Corbel Systems” came in use. At the city of Mycenae, the Aegean culture possessed an important example of this system, the Lion Gate, at the entrance passage which can be seen in Figure 2.6.

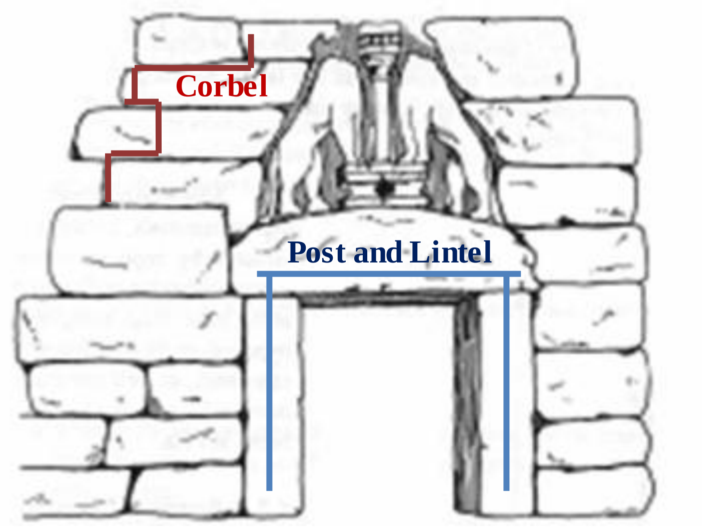


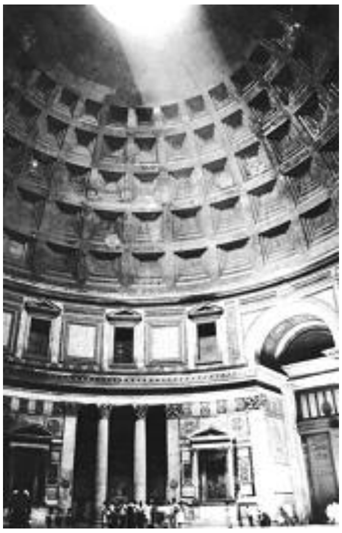
Figure 2.6 Lion Gate, Mycenae (Drysdale et al., 1999)

From the beginning of 7th century, the Greek culture created fine examples of monumental architecture with their temples, stoas and basilicas. In the middle of 7th century B.C., Doric Architecture showed the introduction of certain rules on proportions together with erection of Doric columns with 16 flutings. Their monumental temple structure generally consisted of a narrow hall that is open at one end with row of posts supporting the roof and coursed masonry walls with rough surface finishes.

This structural system was also used at ancient temples as in Temple of Parthenon at Athens being one of the most famous examples of Greek architecture where the architect also considered the optical variations such as slightly leaning inner columns and closer located corner columns. (Figure 2.7)

Figure 2.7 Temple of Parthenon (Oliveira, 2003)

Roman architecture involved one of the most important periods when improvements in many concepts of construction of buildings in terms of materials and methods were introduced. (Figure 2.8) Brick masonry construction was improved together with improving the quality of bricks especially during production stages and with variety of types as well as use of mortar and improvements on structural vaults. Roman concrete was also introduced at this period and multi layer masonry wall construction was commenced which will be discussed in following chapters.

(a) (b)

(c)

(d)

Figure 2.8 Examples of Roman architecture; (a) Dome of Pantheon, Rome (Mark and Hutchinson, 1986), (b) Baths of Diocletian, Rome (Brown, 1958), (c) Puente Romano Bridge, Mérida, Spain (Lapunzina, 2005), (d) Pont du Gard Aqueduct, Nimes, France (Lourenço, 1996)

The Byzantine monumental architecture gave rich examples starting from its early periods, with the well known example of Hagia Sophia. (Figure 2.9) Furthermore, another important period, the Islamic architecture, which initially produced rather simple and extensive early mosques, by the end of 11^{th} century, showed important architectural improvements like use of domes over large spans in Seljuk. Structures like caravanserais and mosques as well as important features like minaret were introduced in Islamic architecture. (Braun, 1959)

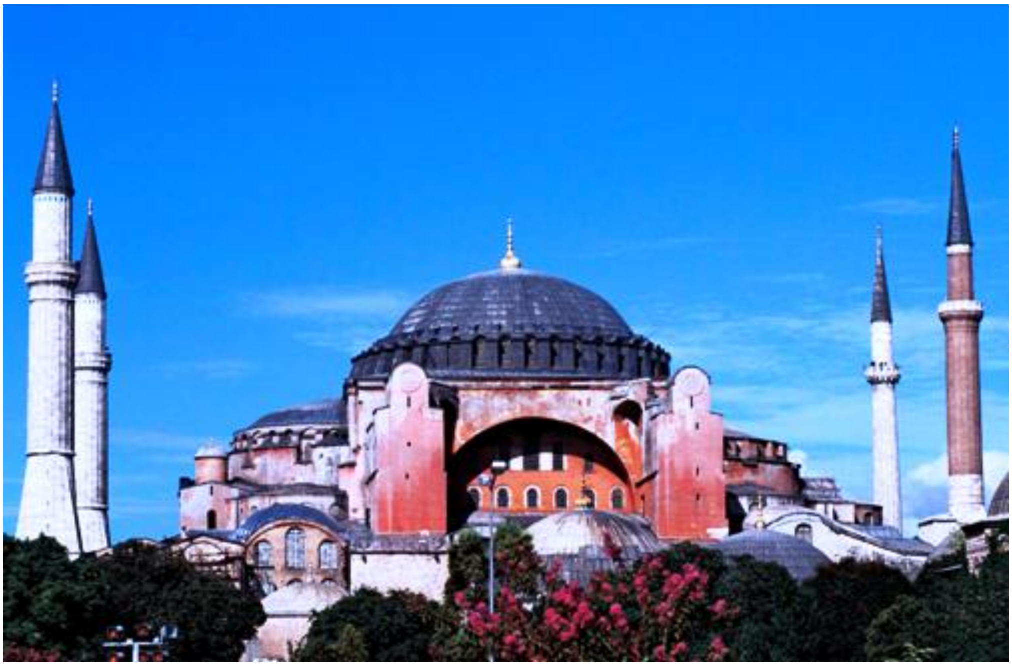


Figure 2.9 Hagia Sophia, Istanbul, Turkey (Encyclopedia- Britannica, 2010)

After the Crusades, advancements in building science and use of structural elements in structures that are especially important in terms of structural load transfer mechanisms in masonry structures were observed. Gothic architecture that is initiated at France was the period when pointed arches, flying buttresses, abutments and heavy structural walls were combined together. (Braun, 1959)

The delicate stone works of art usually combined with ornamentations, often observed in Gothic architecture was also observed in the Chartres Cathedral that is also special being one of the first Gothic Cathedrals. (Figure 2.10)

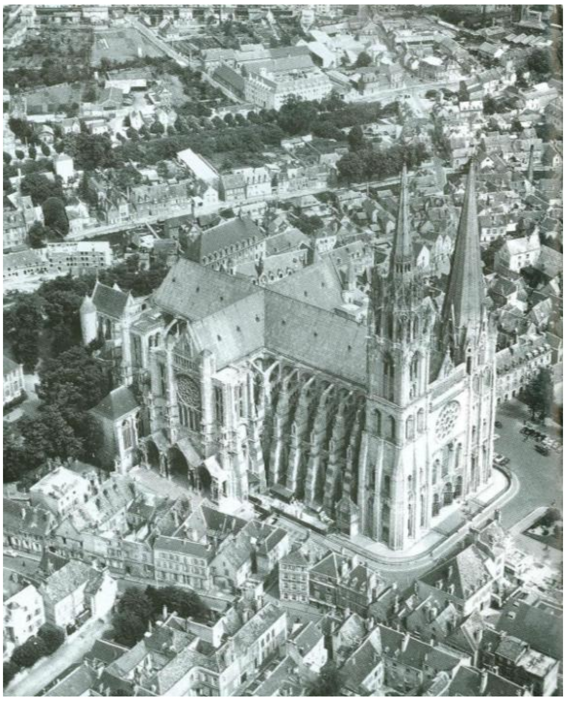


Figure 2.10 An example on Gothic Architecture; Chartres Cathedral, France (Prache, 1993)

2.2 Material Properties of Masonry

Masonry constructions are composed of two constituents; masonry units and mortar. Although the combination of these two components possesses its own characteristic properties, some properties of the final work can be derived from the constituents. General information about the units and mortar will be given in the following sections.

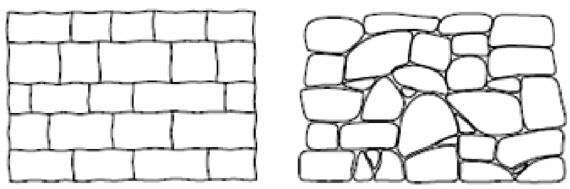
2.2.1 Masonry Units

Masonry has been a major construction system used since the earliest times. Many materials have been used for units so far. The common ones include natural stone, clay bricks, and concrete blocks.

Stone Masonry:

Stone blocks have several types each possessing different mechanical properties, strength, depending on their geological origin, mineral composition and production process. (Erdoğan, 2002)

Furthermore, according to its shape, structural stone is generally classified as shaped or natural stone. Natural stones can either be rounded or angular where angular type is more preferred for more stable structures. Masonry walls of shaped stones are further classified as "Ashlar Masonry" with perfect precision and "Rubble Masonry" if the courses are laid rather irregularly. (Figure 2.11)



(a) (b)

Figure 2.11 Examples of Types Stone Masonry Walls (Lourenço, 1998); (a) Ashlar Masonry, (b) Rubble Masonry

The stone masonry has been used in construction of many ancient structures such as Stonehenge and Cathedral of Saint-Lazare, Autun, France (Figure 2.12).

Figure 2.12 Cathedral of Saint Lazare, Autun, France (Seidel, 1999)

Adobe Masonry:

Adobe blocks are one of the oldest forms of construction materials manufactured by mixing mud with water and straw and later forming the mixture into desired shape. It is a low cost material with good insulation properties and is easily produced. However, due to post earthquake observations, it is nowadays regarded as a non-desired building system at constructions made especially in earthquake zones.

Clay Brick Masonry:

It is produced by forming the clay or shale material usually in a rectangular form and kiln-drying and burning it to obtain the desired block strength. Due to its material properties and production process it is a strong and highly durable structural material.

2.2.2 Mortar

Mortar is basically composed of a binder like cement, lime, water, aggregates and admixtures. The type and proportion of the ingredients define the mechanical properties of the mortar and it is generally used for bonding the masonry units to obtain a more stable structure with higher strength.

2.3 Mechanical Properties of Masonry

Masonry structures possess non-homogeneous and anisotropic properties as they are composed of mortar and masonry units. Therefore its mechanical properties and complex behavior is usually difficult to estimate.

As it has been mentioned before the two constituents finally form a new composite structure with its own characteristic properties and some of these will be given in this section.

2.3.1 Compressive Strength of Masonry Structures

Compressive strength of masonry is especially important at historic structures which are usually constructed to work under compression forces. Type, strength and water absorption capacity of masonry units and mortar, joint width, bonding between units and mortar as well as craftsmanship are some of the factors affecting the compressive strength of masonry work. (McNary and Abrams, 1985, Mistler et al., 2006, Hendry, 2001)

The average compressive strength of masonry units shows great variations from 5 MPa (low quality limestone units) to 100 MPa (high fired clay bricks). (Paulay, Priestley, 1992, Erdoğan, 2002)

2.3.2 Shear Strength of Masonry Structures

Shear strength of masonry wall can be defined as the resistance of masonry that is subjected to lateral loading. Eurocode 6 (European Committee for Standardization, 1999) states that the characteristic shear strength of masonry (f_{vk}) could be determined by;

$$f_{vk} = f_{vko} + 0.4\sigma_d \tag{2.1}$$

for masonry with mortar filled vertical joint,

$$f_{vk} = 0.5 f_{vko} + 0.4\sigma_d \tag{2.2}$$

for masonry with dry vertical joints.

Where $0.4\sigma_d$ is the increase in shear strength of masonry due to compressive stresses acting normal to the shear stress and f_{vko} is the initial shear strength of masonry under zero compression stresses.

The initial shear strength, f_{vko} can be determined by testing triplet specimens (Figure 2.13) (Tomazevic, 2009)

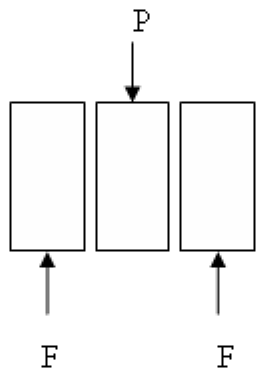


Figure 2.13 The Scheme showing Triplet Test for determining the initial shear strength of masonry

2.3.3 Flexural Strength of Masonry Structures

According to Eurocode 6 (European Committee for Standardization, 1999), flexural strength of masonry is defined as the strength in pure bending which indicates the transverse bending capacity of the masonry. The flexural resistance of the unit can be determined by testing simply supported masonry beams at two ends and applying simple beam load as seen in Figure 2.14 having sections enough to resist applied stresses without yielding. (Abrams, 1997)

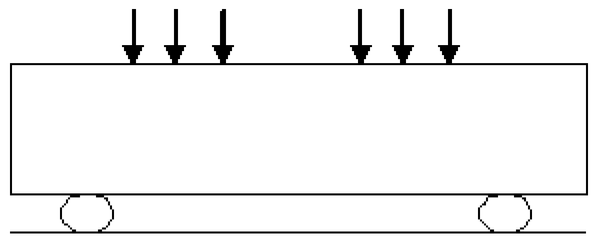


Figure 2.14 Schematic representation of simply supported masonry beam loading

2.3.4 Modulus of Elasticity

Elasticity modulus of masonry defines the stress and strain relation of the masonry. Due to the variances in material properties and testing methods, several different methods have been proposed to determine the relation between the masonry Modulus of Elasticity and its compressive strength.

In cases where no tests are available, for structural analysis purposes Eurocode 6 (European Committee for Standardization, 1999) suggests that E may be taken as;

$$E = 1000 \times f_k \tag{2.3}$$

in which (f_k) stands for the characteristic compressive strength of the unit. The variance of the stress-strain relation in units and masonry prism can be seen in Figure 2.15.

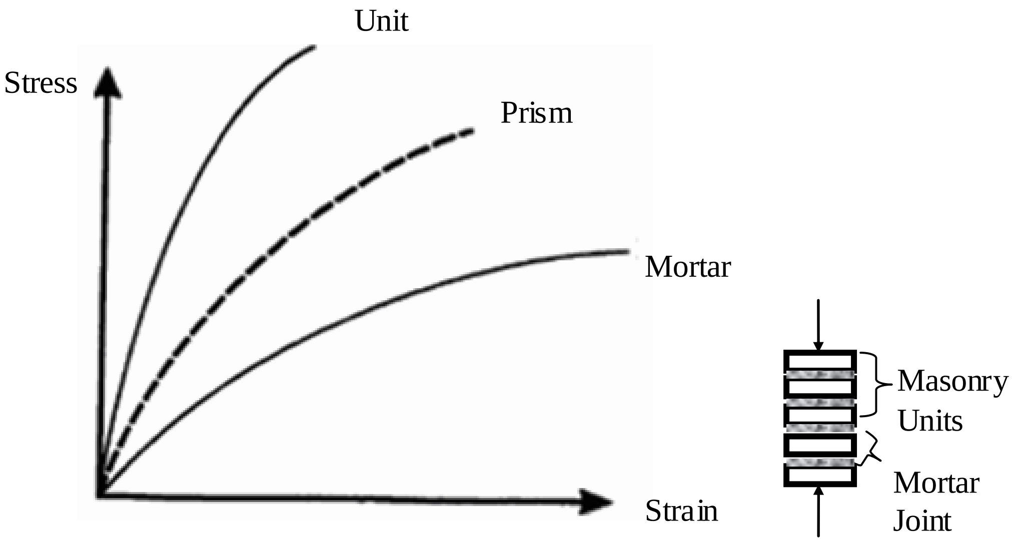


Figure 2.15 Stress-Strain Curves of Masonry Unit, Mortar and Prism (Ip, 1999)

CHAPTER 3

STRUCTURAL ANALYSIS OF MASONRY BUILDINGS

3.1 General

In the case of historic structures, handling and rehabilitation of masonry can be successful if only the diagnosis of damage is adequately perceived. This process becomes harder as these structures possess complex behaviors. In the analysis stage of investigating historic structures, the aim is to assess the state and load carrying capacity of the building and provide assurance that the final state of the building possesses good performance.

With each one being unique, studying historic buildings require specific training in the study and grasping the structural system of the structure. Understanding its behavior under different loading conditions and simulating them with adequate structural models through certain analysis methods is the basic follow through of the process which becomes especially difficult for historic structures. Some of the main reasons may be listed as;

- The details of the framework through the wall thickness are not known in details.

- The mechanical properties of structural materials cannot be deduced because of the restrictions about testing the historic texture or because of severe damages occurred in time.

- The geometric information of the structural elements is not complete including the presence of destructed or even removed elements that misinterpret the original behavior of the structure.

- The intensity and extent of damage cannot be perceived thoroughly.

- The construction process of each structure varies due to the lack of rules for the construction stages and building regulations at past.

3.2 Structural Masonry Elements

In order to perceive the structural behavior of the whole structure it is necessary to understand the structural behavior of the structural elements. Some elements often observed in historic masonry construction are Columns and Beams, Arches, Domes, Vaults, Transition Elements and Structural Walls.

3.2.1 Masonry Columns and Beams

These structural elements were evolved from its Egyptian origin where the system was named as "Post and lintel". (Braun, 1959) The posts carry vertical loads up to the compressive strength of the units and transfer these loads to the ground through foundation, if present. Lintels, on the other hand transfer the structural loads to the posts. As masonry is weak in tension, tensile cracks are often observed on flat lintels and as Feilden mentioned shear failure is often observed on soft stone units. (Feilden, 2003)

3.2.2 Arches

Arches are structural elements that transfer vertical loads to joints and was introduced initially to support openings, and further used in more developed arcuated constructions as seen in early Roman times.

The arch profile possesses certain unit elements like "Voussoir" and "Key Stone" and the representation of a sample arch profile may be seen in Figure 3.1, together with examples on different arch profiles below. (Figure 3.2) It should hereby be noted that, as also stated by Huerta, in a masonry arch under vertical load, the thrust action between the stones changes with the geometry and the curvature of the arch and it affects the stability of the profile. (Huerta, 2006)

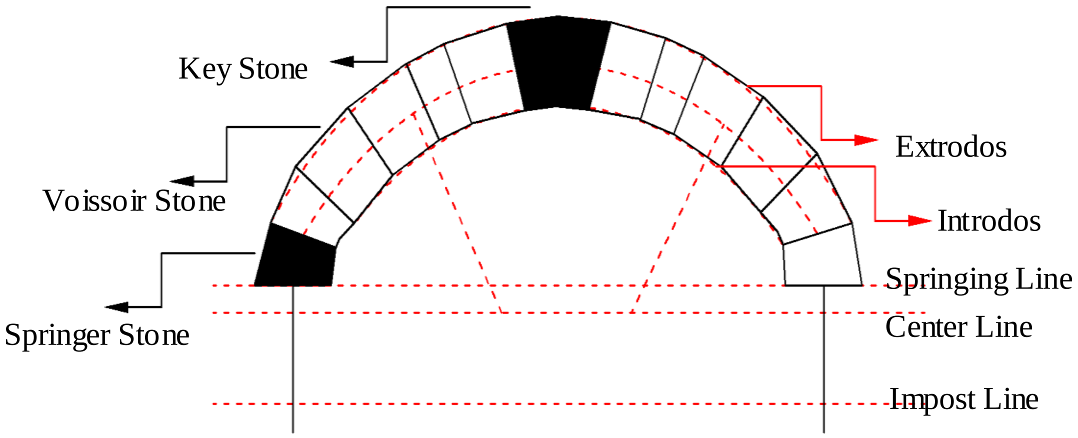
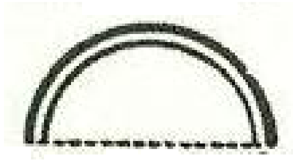
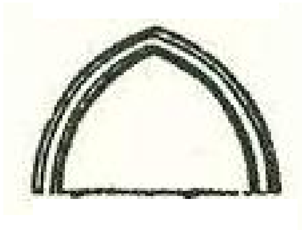


Figure 3.1 A Sample Arch Profile



(a)

(b)



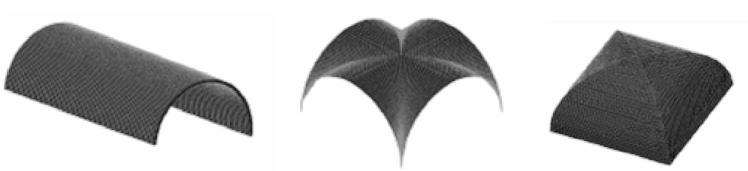
(c)

(d)

Figure 3.2 Different Types of Arch Profiles (Browne, 2005); (a) Semicircular, (b) Stilted, (c) Pointed, (d) Foliated

3.2.3 Vaults

Vaults are one of the mostly used structural elements that were also used widely in Seljuk architecture. They are basically the structural elements formed by series of arches proceeding from surrounding walls to cover a space with several types used to form continuity. Some common types include Barrel Vault, (Figure 3.3 a) with continuous extension in one direction unlike Cross Vault, (Figure 3.3 b) in which the movement is in two directions. Moreover, Cloister Vault is obtained where the two directional movements continues with breaking (Figure 3.3 c) whereas in Star Vaults, the change in sections at various parts is observed in which coursing usually emphasizes an element that is usually a star or an octagon. (Figure 3.4)



(a) (b) (c)

Figure 3.3 Types of Vaults (Szolomicki, 2009); (a) Barrel Vault, (b) Cross Vault, (c) Cloister Vault

Figure 3.4 Star Vault (Guerci, 2009)

3.2.4 Domes

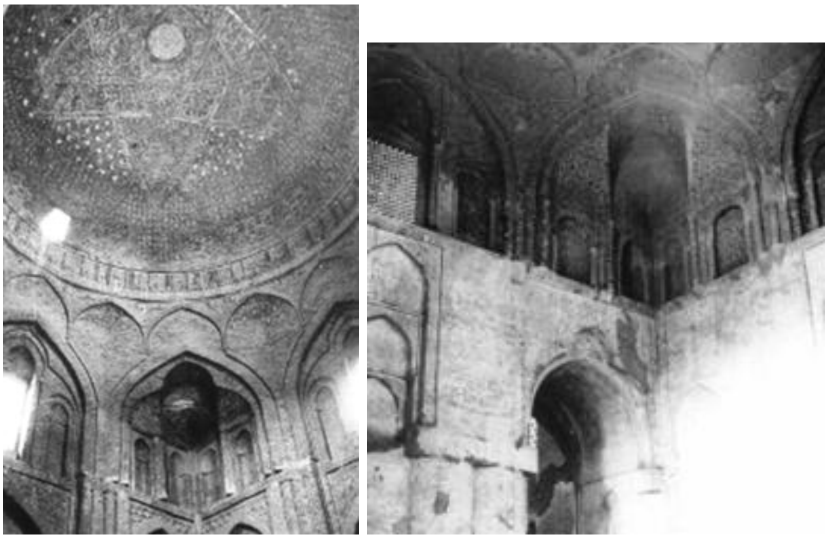
Domes are built to cover a large area and formed by rotating arches and usually sit on a ring at the base. Under loading, the dome faces compression forces whereas the ring becomes under tension in reaction to the dome. These elements have been used since Roman times and became one of the most important architectural features in Islamic architecture. (Braun, 1959)

3.2.5 Transition Elements

The transition from the circular plan of the dome to the rectangular floor plan is provided by certain transition elements. Three major types of transition elements; trompe, pendentives and band of Turkish triangles will be covered in this section.

Trompe:

Trompe (Squinch) is often used in Mosques since early Islamic times and is composed of a vault system constructed beside the central dome, on the rectangular structural walls. It transfers the loads from the dome and is usually preferred in smaller spaces of more height rather than a long span distance. (Figure 3.5)



(a) (b)

Figure 3.5 Masjid-i Jami Mosque, Isfahan, the transition zone and squinch (Edwards and Edwards, 1999); (a) North dome, (b) South dome

Pendentives:

They are commonly used transition elements that's profile is classified according to the geometry produced between the dome's circular base and the walls that it sits on. (Figure 3.6)

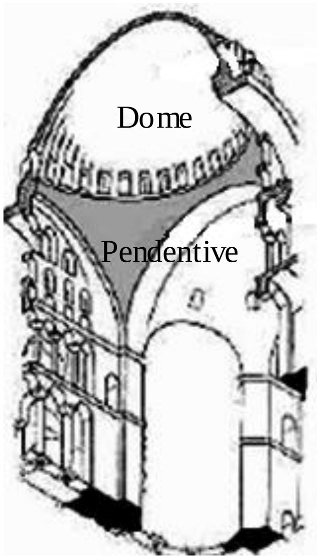


Figure 3.6 Scheme showing pendentives in Byzantine churches (Mosoarca and Gioncu, 2010)

Band of Turkish Triangle:

It is the transition element used to pass from the non-circular dome drum to the rectangular base. The linear form at the ring and the base forms a triangle and the surface of the formed geometry is treated with its protrusions. (Figure 3.7)

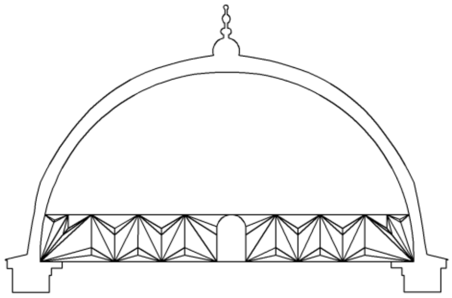


Figure 3.7 Drawing showing the Band of Turkish Triangle

Beside these major elements, “Buttresses” are separate transition elements in the form of a partial arch supporting arches or a dome facing lateral loads as seen in Üsküdar Mihrimah Sultan Mosque and Cenabı Ahmet Paşa Mosque. (Figure 3.8)

(a) (b)

Figure 3.8 Examples of Buttresses; (a) Üsküdar Mihrimah Sultan Mosque (Erzen, 1988), (b) Cenabı Ahmet Paşa Mosque

3.2.6 Structural Masonry Walls

Structural walls are the mostly used structural elements in masonry construction for carrying loads. There are different types of masonry walls due to the cross section of the wall, material type and arrangement of the courses. In terms of cross sections, masonry walls are classified into the following classes (European Committee for Standardization, 1999, Curtin et al., 2006) and can be seen in Figure 3.9.

a) *Single-Leaf Wall:* The masonry walls' width is in one unit lengths.

b) *Double- Leaf Wall:* It consists of two outer layers of masonry walls and a vertical joint, the collar joint, in between filled with bonding material like mortar. Three leaf walls have also been seen in Europe, like the Bell Tower of Sint Willi-Brordus Church in Belgium. (Verstrynge et al., 2008)

c) *Cavity Walls:* The section of the masonry wall is like the double-leaf wall however the two masonry layers are connected with wall ties and the type of cavity wall depends on the treatment of the vertical joint. If the joint section is empty, the wall is named as Cavity Wall. However, when mortar exists between the leaves the name given to the wall is Grouted Cavity Wall.

d) *Diaphragm Walls:* This type is basically like a cavity wall that consists of two leaves of masonry wall and the interior is left empty. However this specific type of wall has masonry ribs between the outer leaves that are made from the same masonry material with the wall.

e) *Piered Walls:* The masonry wall section that is similar to cavity wall is improved with an additional pier at certain locations of the wall to resist additional load concentrations.

f) *Veneer Walls:* The masonry wall has an attached veneer on the face of the wall connected with ties.

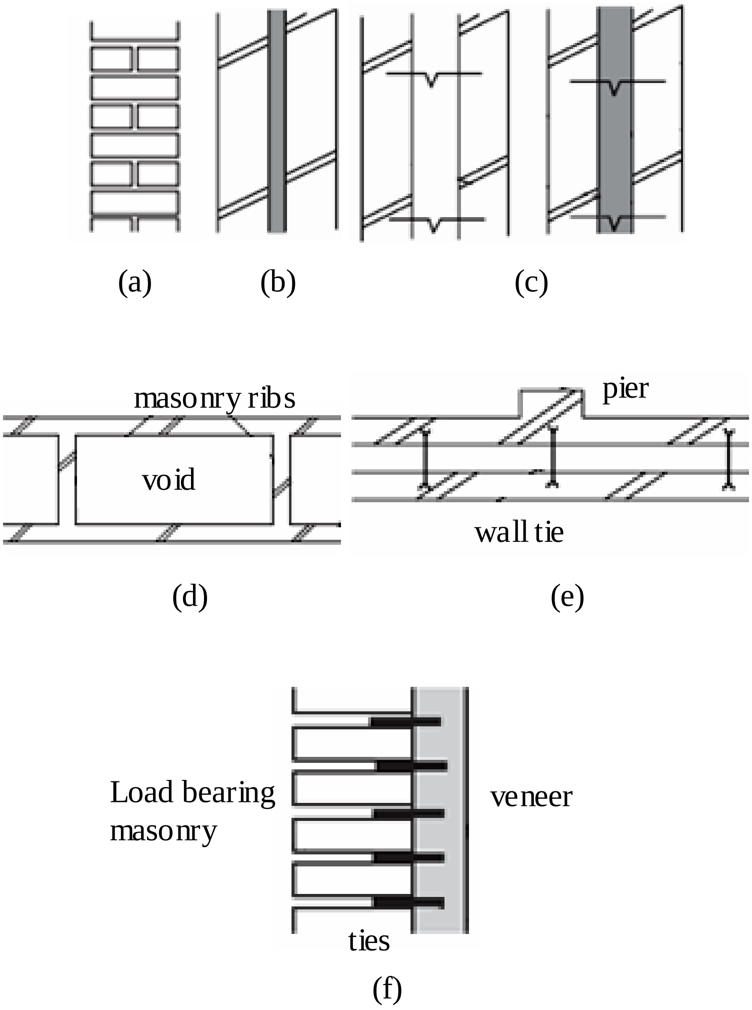


Figure 3.9 Types of Masonry Walls in terms of their cross section (European Committee for Standardization, 1999, Curtin et al., 2006); (a) Single-Leaf Wall, (b) Double- Leaf Wall, (c) Cavity Wall, (d) Diaphragm Wall, (e) Piered Wall (f) Veneered Wall

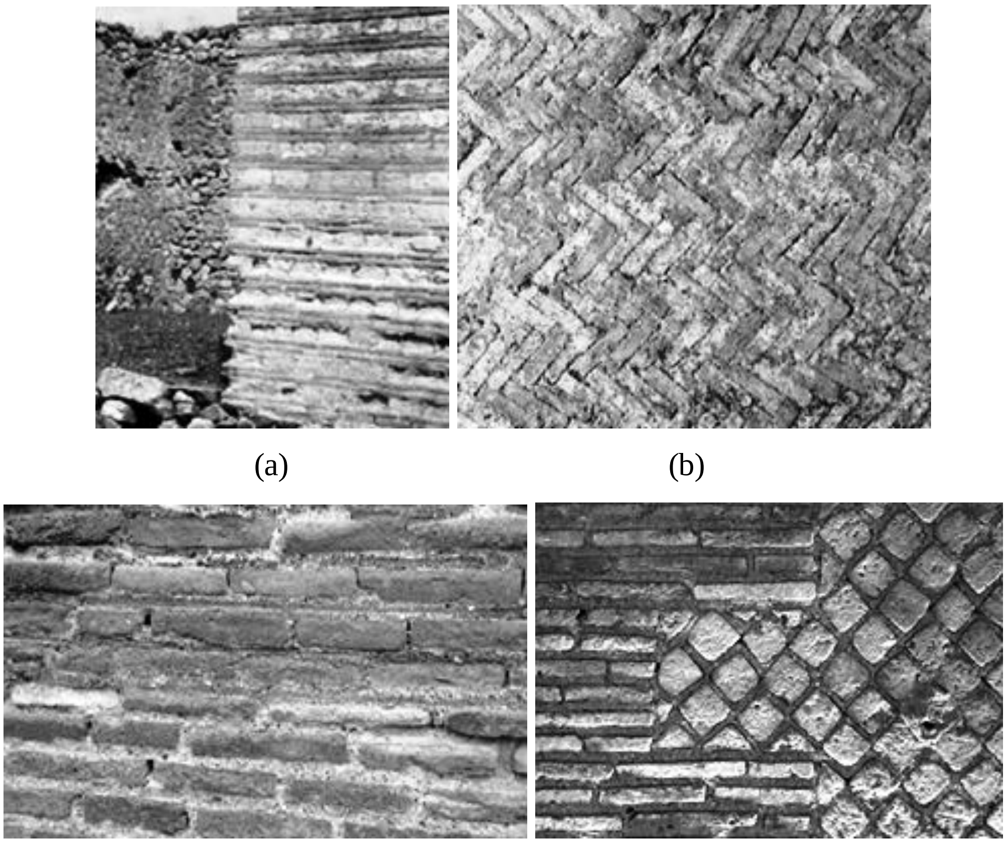
Beside these classes, masonry walls may be classified according to the type of unit material as either Stone Masonry or Brick Masonry Walls. Stone Masonry walls are further classified according to the layout of the courses since ancient times. Two of the well known classes are Opus Siliceum and Opus Quadratum. If the stone blocks are huge and laid in a manner rather irregular, then the name of the wall is Opus Siliceum. (Figure 3.10) Opus Quadratum, on the other hand consists of regular rectangular courses of stone blocks that has also been preferred in Greek city walls. (Figure 3.11)

Figure 3.10 Temple of Apollo, Delphi (Fulbright Association, 2010)

(a) (b)

Figure 3.11 Examples on types of Stone Masonry Walls (Rossi et al., 2009); (a) Opus Siliceum, (b) Opus Quadratum

Unlike Greek and Egyptian dry stone masonry walls, Roman brick masonry walls were assembled as structural walls with two outer masonry layers and mortar core in between. (Braun, 1959) When the wall consists of two layers of brick units and a mixture of stone particles and mortar as its core, the wall is named as Opus Ceamenticium. Furthermore, in Opus Reticulatum small square blocks of brick are laid diagonally forming a diamond shaped pattern whereas more regular forms of brick masonry were observed in Opus Vittatum, Opus Spicatum and Opus Latericium. However in Opus Mixtum, rather irregular courses with many forms were laid. Some of these types can be seen in Figure 3.12.



(a) (b)

(c) (d)

Figure 3.12 Examples on types of Brick Masonry Walls (Rossi et al., 2009); (a) Opus Vittatum, (b) Opus Spicatum, (c) Opus Latericium, (d) Opus Mixtum

3.3 Structural Loads

According to the specifications (TS498) the loads acting on structures are listed as,

- Dead loads and steady static loads consisting of self weight of structural elements

- Live loads, that is dependent on time and distance.

- Horizontal loads, acting on the structure horizontally like earthquake, wind, etc.

- Other loads, such as loads due to temperature changes, swelling and shrinkage action, creep, differential settlements, earth pressure, snow load and impact loads.

Masonry structures generally withstand gravity loads however due to its brittle characteristics, earthquake loads and settlements are usually threatening. Due to their complex behavior and structural composition, these structures should therefore be evaluated specifically identifying its current state by combining the engineering judgment and past experiences on type of damage.

3.4 Damages on Historic Masonry Structures

Ancient buildings were usually constructed by deducing from previous experiences. (Lagomarsino, Resemini, 2009) Therefore, it is of great possibility to observe some level of damage on these structures which would depend on the properties of structure and intensity of the mechanical action causing the damage.

The main causes of structural damage will be discussed in this chapter by also mentioning the previously conducted researches.

3.4.1 The Causes of Structural Damage

Many masonry buildings facing earthquake forces became damaged or collapsed in the past apart from a few exceptions of historical monuments remaining until today. In formation of cracks when structural elements cannot resist the altered load transfer mechanisms, crushing or collapse of the structure may be observed. (Croci, 1998) These damages occur mainly due to masonry's low tensile strength as well as its brittleness, structure's weak connections, stress concentrations around openings and improper constructions. The major reasons for damages on historic structures may be claimed to be caused by; (Bayraktar, 2006)

- Deterioration of the structural materials as a result of aging through time.

- Earthquake, ground settlements and changes in soil profile causing changes in load transfer mechanisms and stress distributions leading to serious damages together with the masonry's brittle behavior and low tensile strength.

- Inadequate alterations or restoration applications which can even lead to fatal structural errors as in removing structural elements or adding new levels.

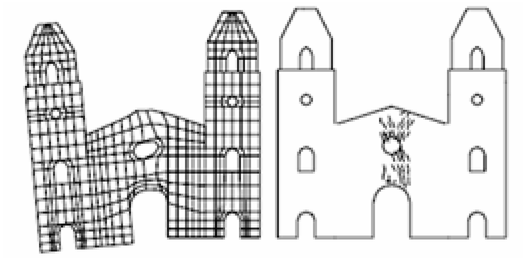
In addition to these, long-term damages have also been seen to be effective on the life of historic monuments. The collapse of the Civic Tower of Pavia, Italy is considered as one of the events that lead to arose of researches on investigating the long term effects on historic structures. (Binda et al., 2008)

The structure is an 11th century brick masonry structure that suddenly collapsed at 1989, (Figure 3.13) which was composed of thick masonry walls with regular coursed brick layers and irregular courses of stone-brick layers in-between bonded together with mortar. (Binda et al., 2008) The causes of the failure have been examined through several investigations that have been carried out by researchers and it has been deduced that in case of multiple leaf masonry, differential creep displacements formed by the leaves' different deformation characteristics and persistent loads leading to retarded strains on the structure have been effective on the structure.

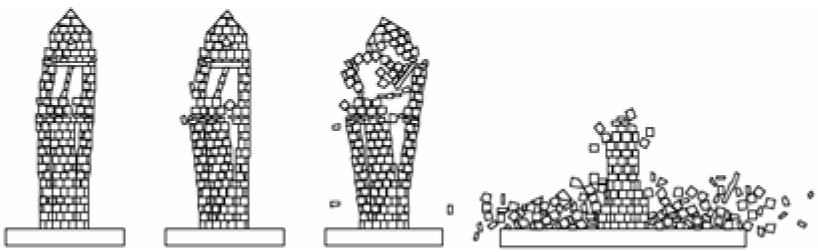
Figure 3.13 The ruins of The Civic Tower of Pavia after the collapse (Binda et al., 2008)

In addition to this type of loading, Figure 3.14 shows some examples of damages on masonry buildings including deterioration due to external weathering effects, differential settlements and earthquake actions.

(a)



(b)



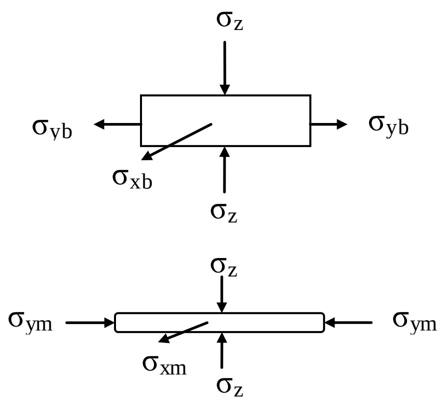
(c)

Figure 3.14 Damages on Masonry Structures; (a) Deterioration on the defense walls of a medieval castle (Juhasova et al., 2008), (b) Damage on St. Torcato church due to differential settlements (Lourenço, 1999), (c) Collapse mechanism of St. Georgio in Trignano Bell Tower,Italy after the 1996 earthquake (Azevedo and Sincraian, 2001)

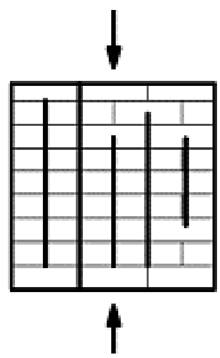
3.4.2 Failure Mechanisms of Masonry Structures

Masonry possesses non-homogeneous, anisotropic material properties therefore different types of failure mechanisms shall be observed depending on the type and direction of the load, properties of the mortar joint.

Under vertical loads, the behavior of masonry mainly depends on the elastic properties of the masonry units and the binding material. The failure is generally observed by vertical cracks through the units. (Figure 3.15) Under loading, due to different strain characteristics of these two materials, the mortar will tend to expand more than the relatively rigid masonry units. However, due to the bonding in-between, the expansion will be prevented. As a result, the masonry unit becomes under biaxial tension whereas the mortar will be under biaxial compression. When the ultimate tensile strength of the unit is reached, failure is observed.



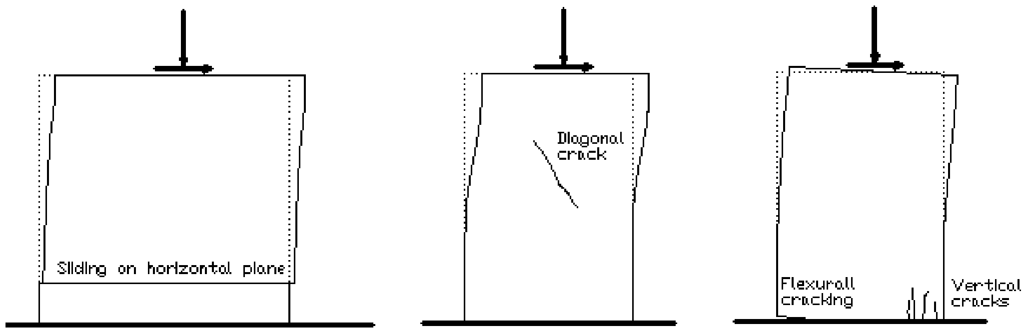
(a)



(b)

Figure 3.15 Masonry under axial compression; (a) Stresses acting on mortar and brick units, (b) Typical Fracture Pattern (Dhanasekar et al., 1985)

When masonry is under both axial and horizontal loads, type of failure mechanism depends on the level of loading and mechanical properties of masonry. If the level of axial load is low, lateral load is relatively high and the mortar is of poor quality, (Mistler et al., 2006) sliding shear mechanism is observed. (Figure 3.16 a) However, under relatively high compression forces, if the tensile stresses on masonry exceed the tensile strength of masonry units, diagonal tension occurs (Figure 3.16 b) in which the failure pattern follows the mortar bed for low strength mortar. (Lourenço, 1998) In case of flexural type of failure mechanism, on the other hand, the failure occurs by the crushing of the compression zone at the masonry. (Figure 3.16 c)



(a) (b) (c)

Figure 3.16 Sketches of Failure Patterns on masonry walls; (Calderini et al., 2009) (a) Sliding Shear (b) Diagonal Tension (c) Flexure

3.5 Numerical Modeling of Masonry Structures

Masonry structures are anisotropic, non-homogeneous complex structures requiring more considerate care. In case of historic masonry structures, special attention must be paid during the investigation and analysis stages. In past, these structures have been basically built based on the builder's experience and earlier examples. In order to inspect the state of the structure, to evaluate their performance under different loading conditions and to strengthen them where necessary, the need for modeling masonry arises.

The modeling strategies depend on the structural problem as well as its properties. (Lourenço, 1998) A simplified analysis where further assumptions should be made shall be useful for larger and more complicated structures where the overall structural behavior is to be observed. However, for more discrete observations, in which the stress-strain state, the deformations of the units and mortar is to be obtained, analysis can be achieved by developing more detailed finite element models concerning the unit-mortar interface of the structural elements. This method is preferred in analysis of certain structural elements such as masonry walls, domes or vaults under complex loading conditions. Therefore, the selection of the method greatly influences the computational cost and details of the analysis stage and should be well decided.

In the structural analysis of historic masonry structures, modeling does not respect the assumptions made for other materials governing elasticity, isotropy and homogeneity. Therefore, the representation of the material behavior that will be adopted in the analysis should also be selected appropriately.

According to material properties, the modeling methods are further classified as "Elastic", where the deformation of structural materials is assumed to be recoverable complying with the Hooke's Law (Equation 3.1), or "Plastic", where limit load of the masonry is obtained assuming the material having no tensile strength with relatively high compressive strength, or “Non- linear”, where material can be observed until failure. (Macleod, 1990)

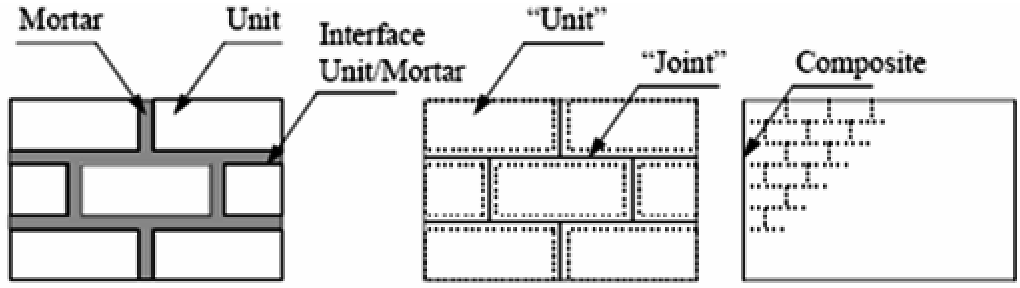
$$\sigma = \varepsilon \times E \tag{3.1}$$

According to Lourenço, there are mainly three computing strategies for the analysis of masonry structures. (Lourenço, 1999)

1. *Detailed Micro Modeling:* the units, mortar and interface are modeled including the material behavior of each constituent with the knowledge of masonry material properties. (Figure 3.17.a) Detailed modeling is advantageous especially for relatively small structural elements or sections of structural elements.

2. *Simplified Micro Modeling:* it considers the unified mortar-interface together with the masonry units, therefore with less accuracy compared to the detailed models. (Figure 3.17.b)

3. *Macro Modeling:* preferred for larger and more complex structures, where the overall behavior of masonry is more important or computational cost is rather critical. (Figure 3.17.c) In this case, the structural material should be well defined by experiments to avoid major mishandling.



(a) (b) (c)

Figure 3.17 Modeling strategies; (Lourenço, 1999) (a) Detailed Micro Modeling (b) Simplified Micro Modeling (c) Macro Modeling

In this study, the structure is modeled homogeneously using macro modeling strategy that is suitable for relatively large dimensioned structures and structural walls where the stress distributions are rather uniform.

3.6 Retrofitting Methods on Masonry Structures in General

Historic buildings that still stand today are usually damaged by the consequences of time, external effects like disasters and accidents or mishandlings. Therefore they require careful supervision during analysis and treatment of the damage.

Repair and strengthening of a historic structure usually aim to increase the strength and ductility of a damaged structure or rather to increase performance of an undamaged structure beyond its initial state. In order to accomplish either, firstly the reason of the damage should be identified carefully and then the analysis of the structure should be studied together with the proposed strengthening method. The renovations should also be fit with the rules of restoration and conservation of historic monuments, therefore certain actions should be avoided in handling these special structures like applications causing vibrations on foundation, at the structure or the ground it sits considering the brittle behavior of masonry. (Bayraktar, 2006) Hence, it is obvious that the methods should cautiously measure the state of the structure, ensure good performance of the whole structure rather than an individual member and provide the integrity of the structural members after strengthening.

Although the structural rehabilitation methods will be discussed in section 5, hereby, recommendations for handling historic masonry structures advised by the International Council on Monuments and Sites (ICOMOS) will be reviewed.

Masonry buildings are defined as stone, brick and earth based construction by the International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH). It is mentioned in the committee's charters that the initial study of a historic structure should address to identify the structural composition and material properties by carrying out material tests. It has also been seen useful to inspect the stress distribution and visualize the possible crack patterns to diagnose the causes of damage. (International Council on Monuments and Sites, 2003, Lourenço, 2006) Some of the measures taken for interventions, advised by the charter may be listed as follows;

- The proposed method should aim for the causes of the damage rather than the apparent damage only.

- The intervention should insure structure's safety and durability.

- The method should preferably be reversible considering the technical improvements.

- The materials used in the intervention should be fully compatible with the existing materials.

- The original state of the structure should not be destroyed and the application should not worsen the situation of the structure such as removal of a structural material or feature.

- The interventions should be controlled and monitoring the structure after the application procedure and documented for further investigation whereas necessary should be provided.

In order to set an example for studies on historic structures, the proposed intervention methods for strengthening masonry walls by ICOMOS include; ((International Council on Monuments and Sites, 2003)

- Re-pointing masonry wall joints with mortar

- Grouting the damaged wall

- Vertical reinforcement of the wall in longitudinal/transverse directions

- Re-construction of the wall either partially or completely

- Removal and replacement of the decayed material

CHAPTER 4

INVESTIGATION OF A DAMAGED HISTORIC MOSQUE WITH FINITE ELEMENT ANALYSIS: A CASE STUDY, CENABI AHMET PAŞA MOSQUE

4.1 Ottoman Architecture in Anatolia and the Case Study Structure

The ottoman architects (15^{th} – 19^{th} century), perceived several cultures that has influenced the Anatolian art for centuries and interpreted them by forming their own statements in which 16^{th} century has been an especially important age. This formation has also been graciously expressed in Ottoman mosques where domes have been used widely.

As it has been mentioned before, domes are roofing structures that can be used with other roof systems like vaults. It either sits on a cylindrical base which is called the “drum” or directly on the structural walls. It should be noted hereby that, the use of drums influences the spatial properties and the form of the structure itself. The related structural variations include Single-Shell Dome on Squinch, Multiple Rows of Small Domes, Double-Shell Domes and Domes without drums. (Kuban, 1987)

One of the greatest architects of the time, Sinan, has percept certain aspects in his structures which can be observed in many structures of the age. He provided a balanced structural layout using straight lines on the plan for transformation of curved sections as well as a balanced structural system considering the design of supporting elements. (Kuban, 1987) Figure 4.1 shows the development of spatial form in Ottoman architecture together with the use of internal support systems.

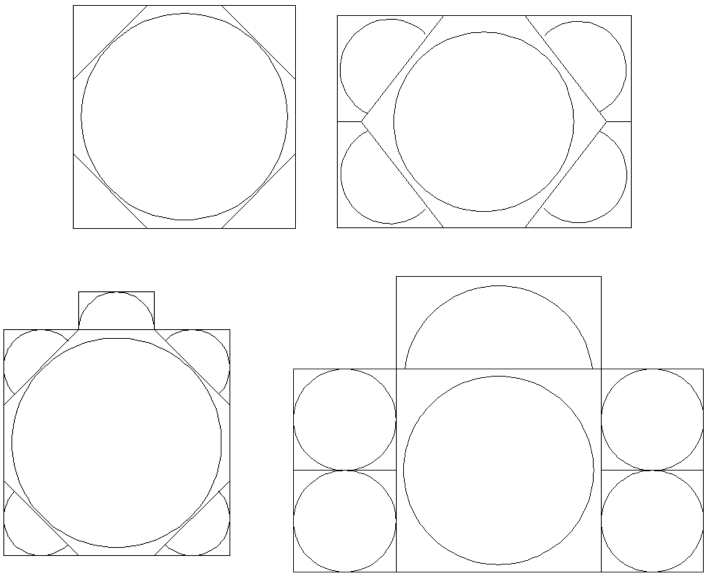


Figure 4.1 Schemes of mosques (Karaesmen, 2008)

The structure that is selected in this study, Cenabı Ahmet Paşa Mosque or also known as Yeni Mosque, is located at Ulucanlar Avenue, Ankara. (Başkan, 1993) Figure 4.2 shows the general appearance of the structure. The attributed Cenabı Ahmet Paşa was appointed as the Anatolian Governor by the period's emperor Kanuni Sultan Süleyman. The structure's construction started by the governor and could only be finished at 1565-1566 after his death. In the detailed search, the history records about the structure stated the guide architect to be the Architect Sinan. It has been acknowledged that like many of the period's buildings' constructions he has been guiding, the construction of the structure was carried out by the group "Hassa Mimarlar Ocağı" and supervised by architect Sinan since 1539. (Başkan, 1993)

Figure 4.2 Photo of Cenabı Ahmet Paşa Mosque

The case study structure is a clear example of Ottoman period Architecture formed of a single central dome and three relatively smaller domes at the last congregational area attached to the main structure. There exists a cornered minaret which is made of cut stone like the main structure. The structure has, 17.2×17.8 meters dimensioned rectangular plan and the central dome’s drum sits directly on the two- leaf structural masonry wall of 1.8 meters thickness.
The structural layout may be seen from the structural plan given in Figure 4.3 and a section of the front façade is given in Figure 4.4.

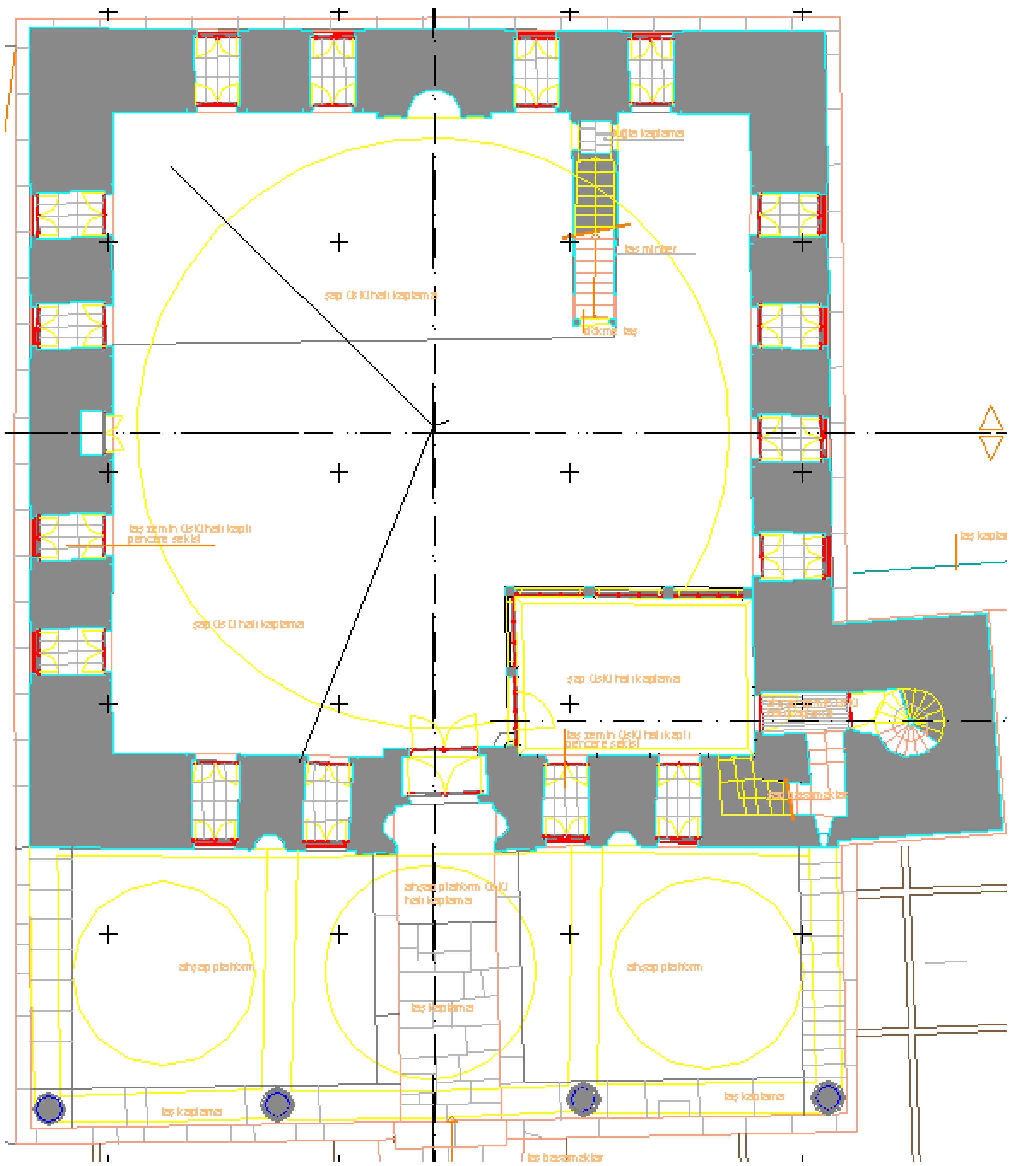


Figure 4.3 Structural Plan of the Mosque (drawings by SAYKA Construction Architecture Company, 2008)

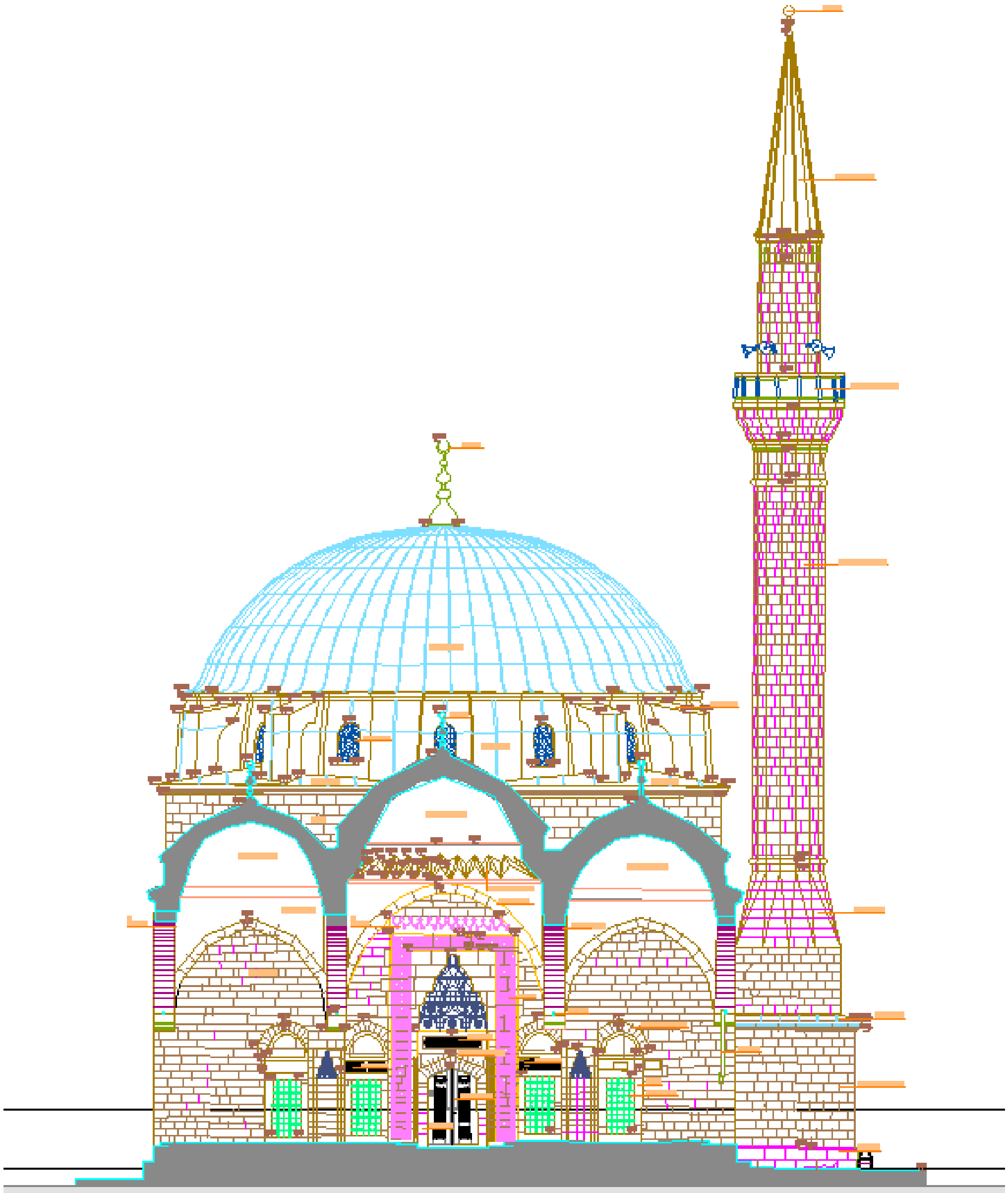


Figure 4.4 Section of the front façade of the Mosque (drawings by SAYKA Construction Architecture Company, 2008)

4.2 Information on Analysis Model

The finite element package program SAP2000 has been used in the modeling and analysis stages. It is a widely preferred engineering program that enables to analyze many civil engineering structures from simple buildings to mosques, dams, bridges and tunnels. The method of finite element analysis which involves meshing the structure into relatively smaller sub domains and obtaining the stress values of these elements rather than the whole structure, provide good representation of complex structures. Therefore, the method has seen to be adequate for this study.

Area elements are used throughout the structure making up of the structural masonry walls, pendentives, semi domes and the main dome whereas columns are preferred to be of frame elements for better representation of connections.

The area element used at the structural walls and transitional elements is the four- node quadrilateral finite element. Each area element is defined with 4 joint connectivities making up 4 faces as shown in Figure 4.5. The element has 6 total degrees of freedom consisting of 3 transitional; Ux, Uy, Uz, and three rotational; Rx, Ry, Rz degrees of freedom. The prepared model consists of 9852 nodes, 11 frame elements and 9564 area elements.

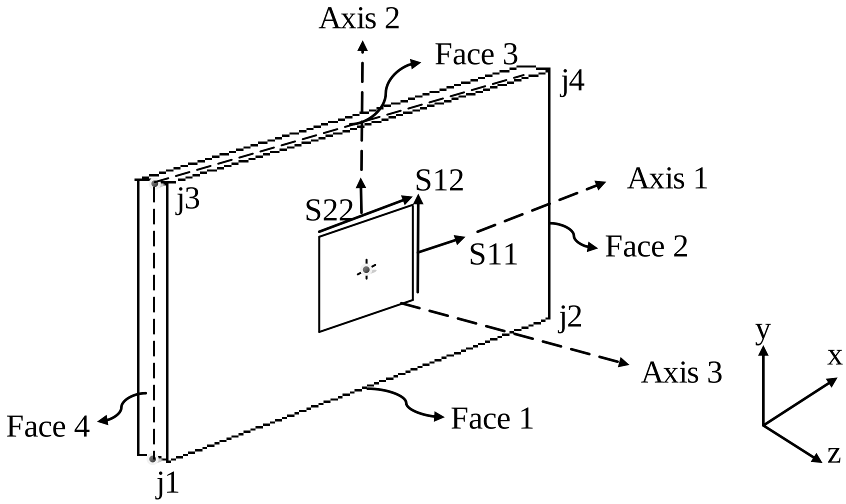


Figure 4.5 Local Axes and stress directions of the four-node quadrilateral shell element

Due to the historical value of the structure, in order to consider the rules of conservation and preservation, the material tests were prohibited. Therefore, as no experimental data is available certain assumptions have been adopted in the study. The material characteristic of the structure is chosen to be stone and its properties have been taken from the literature.

The structural masonry walls consisted of two leaves of stone masonry and the infill material in-between is assumed to be composed of mortar and straw with the walls' total thickness of 1.8 meters that is obtained from the drawings. The walls are modeled assuming a single homogeneous material; however, since the aim of the study is to identify the reasons of the damage by investigating the macro structure, the assumptions are thought not to be too influential on the result of this study. The complete finite element model of the structure may be seen in Figure 4.12.

The elastic properties of the macro-model may be listed as;

• Modulus of Elasticity (E): 30000 MPa

• Poisson's Ratio (υ): 0.2

• Unit weight of stone blocks (γ): 2.7 t/m^3

The values are obtained from the literature based on the experiments carried out by researchers on structural masonry. (Lourenço, 2006, Tóth et al., 2009) It should also be noted that, additional material loads on semi domes are also taken into account by increasing the unit weight of material. In the modeling process, certain stages have been followed. First, grids have been defined compatible with the architectural drawings to enable handling model adjustments and remodeling accurately if needed. (Figure 4.6)

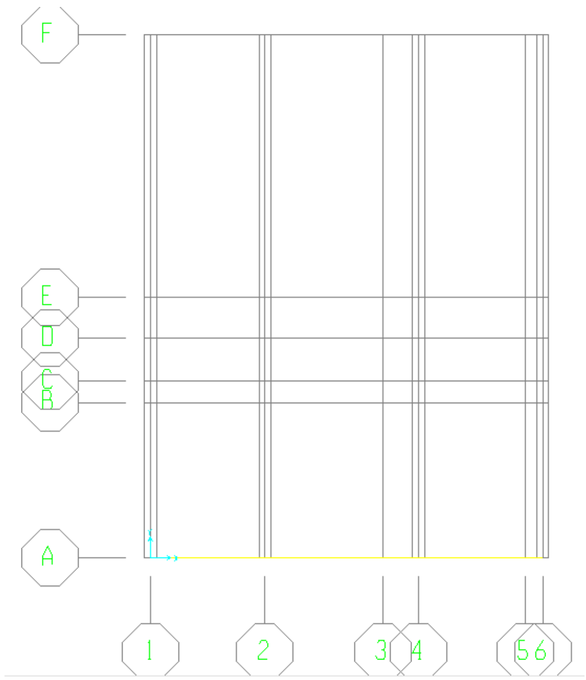


Figure 4.6 The defined grid lines

Initially a simpler model was conducted (Figure 4.7); however, as the connection details are not seen adequate, a more detailed model has been prepared.

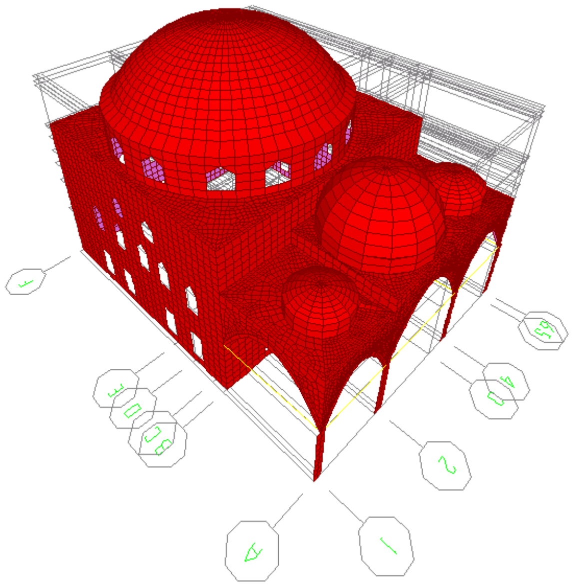


Figure 4.7 The Simplified basic Model

In the second model, the structure is formed in several steps for each structural element with specifically defined area shell sections and material properties. The material and section properties have been assigned according to the architectural drawings. The 1.80 meters thick two leaf masonry wall sections have been assumed to be of a single homogeneous section with 1.00 m thickness. After setting the defined structural wall sections, the frame elements at the last congregational area and the domes on to the grid lines, 50 cm square automatic meshing has been applied to the structural walls and the main dome. Then the openings have been pierced through the structural walls and the main dome's drum.

In the main section, arch profiles have been used to define the layout of the area elements at the transition zones and the connections. In order to do this, a circular arch frame, which is later removed, has been drawn in segments at three nodes on the surrounding walls and then the adjacent area elements have been redrawn from the existing mesh elements to the circular arch frame. Later on, the joint connectivities at the area have been checked in detail. (Figure 4.8)

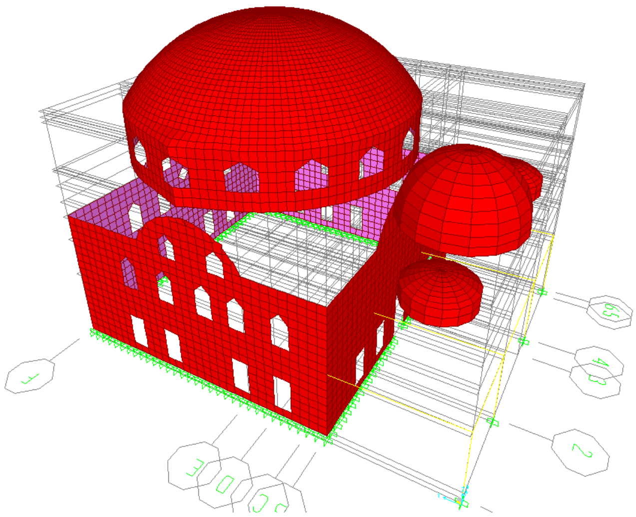


Figure 4.8 The structural model, main structure

In the semi dome sections and the inner transition zones, additional arch frames have been added each having segments along their height and area sections have been drawn from node to node. (Figure 4.9) It should be noted that, the additional masses caused by the infill material above the semi domes has been added to the system via material definitions. Therefore, the unit weight of the area elements at this region is greater than the above mentioned unit weight of stone blocks.

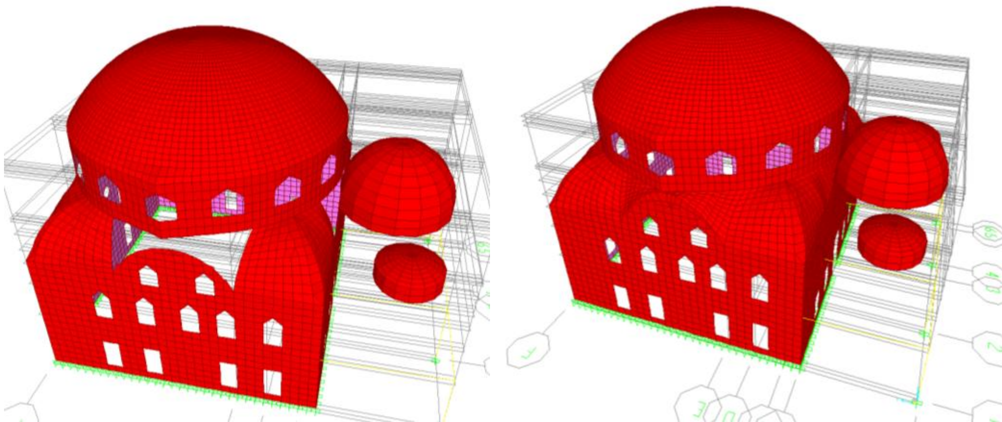


Figure 4.9 The structural model, transition zones

In the front section, arch sections have been similarly modeled between the horizontal frame elements, resembling the metal ties, and between the front domes and the front columns as seen in the structure (Figure 4.10) to aid the modeling of the pendentives.

It is hereby necessary to note that, the area meshes have been modeled with uniform sizes and the corner meshes have been specifically chosen to be of triangular 3-node mesh element for adequate representation of the structural connections between area and frame elements. (Figure 4.11) The complete model is given in Figure 4.12.

Figure 4.10 Photo of the structure's front section

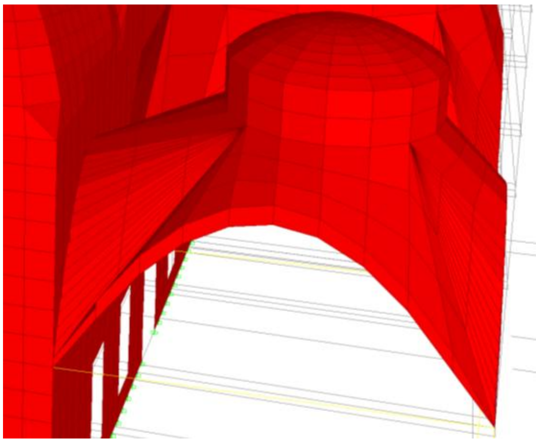


Figure 4.11 The structural model, front pendentives

Figure 4.12 Finite Element Model of the structure

4.3 Analysis of the Structure

In this study the present masonry structure shows serious crack patterns that can be followed from the ground up to the main dome. The stresses developed on the structure lead to cracks and disintegrations of stone blocks which lead to the structure's closure to service. The state of the damage (Figure 4.13) is evaluated as being caused by settlement problems. Therefore, the analysis stage of the structure shall involve the evaluations of the ground profile.

Figure 4.13 General views of cracks

Figure 4.13 (continued) General views of cracks

In this particular study, after modeling the structure, which was explained in the previous sections, the ground properties, obtained from the ground investigation report are assigned to the structure and its effects have been evaluated.

The referred ground investigation report has been prepared by Middle East Technical University, Department of Civil Engineering under the guidance of Dr. Erdem Canbay and Dr. Kemal Önder Çetin. (Canbay, Çetin, 2008) Five boring logs have been drilled around the structure's foundation and ground investigations and geotechnical experiments have been carried out. The layout of the boring logs is given in Figure 4.14 and the sections of the ground profile are given in Figure 4.15.

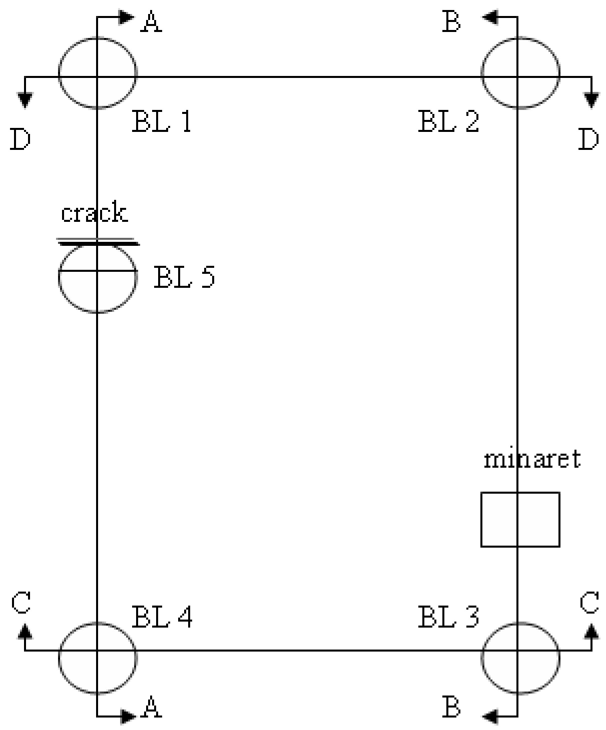


Figure 4.14 Boring Logs Layout

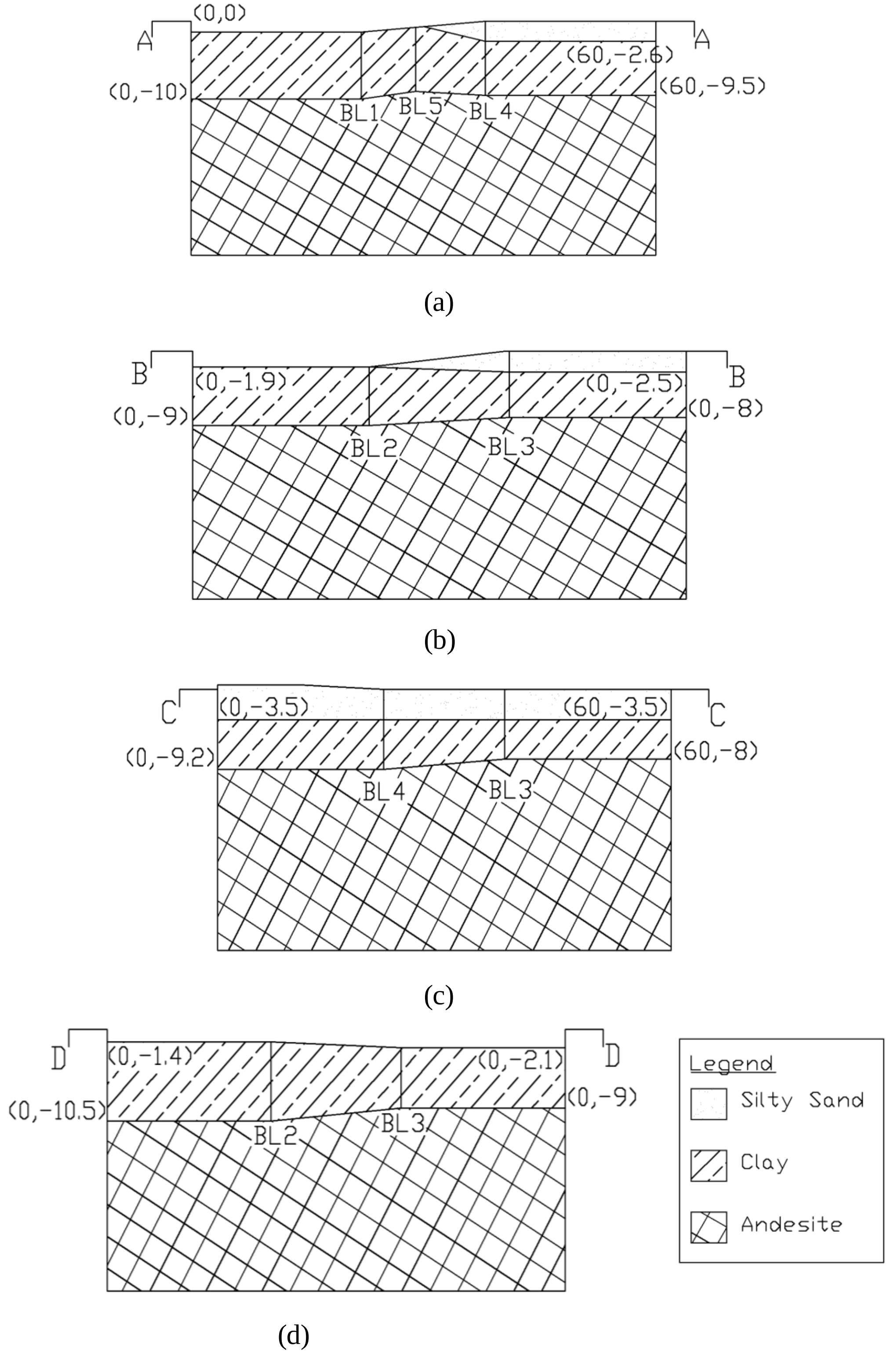


Figure 4.15 Ground profile Sections; (a) A-A, (b) B-B, (c) C-C, (d) D-D

The key points in the report may be summarized as follows; (Canbay, Çetin, 2008)

- The ground profile consists of Ankara Clay of 5-9 meters depth including a locational thin layer of silty-sand up to 2 meters depth and andesite stone below, beneath the structure's footing. (Figure 4.9)

- The saturated zone which is 3-3.5 meters above the andesite could rise up to 3 meters depth from surface.

- One of the most dangerous ground problems for the studied structure is specified as the swelling-shrinkage potential of the mentioned soil profile.

- It has also been foreseen that the uni-axial swelling-shrinkage action combined together with the changes in the water table level (will be denoted as W.T.L. from this point on) would lead to soil movements up to 3 cm. deep.

It should be noted that the ground settlement of the given profile will be treated by taking the structure's historical past into account. Being built on 1565, the soil will be assumed as it has concluded its consolidation settlements in the past 500 years. However, soil settlements due to the change in W.T.L. need to be taken into consideration in the analysis.

In light of this information, the layered ground profile and different characteristics of these soil layers would result with the differential soil settlement which in case of masonry buildings, due to brittle material properties, may lead to severe structural damages.

All in all, the soil displacements on the described soil profile are concluded to arise from two possible reasons. These are because of the change of W.T.L. due to the changed ground profile and because of soil's swelling-shrinkage. The effects of these will be explained separately from this section and will be finally combined while transferring to the model.

4.3.1 Change of Water Table Level

As mentioned before, the structure was built on 1565 therefore; it is assumed that the soil has completed its consolidation settlement. However due to the change in W.T.L., certain amount of soil settlements has been taken into consideration in the analysis.

As seen in the proposed profile (Figure 4.16), the effect of capillary rise may lead to severe changes in the W.T.L. reaching up to 3 meters depth. When this case is taken into account, the comparison between the state of ground profile where W.T.L. is at the surface and at 3 meters depth from surface will be used in calculations and will be referred as cases 1 and 2 respectively. The density of clay is taken to be 20 kN/m^3 for both cases.

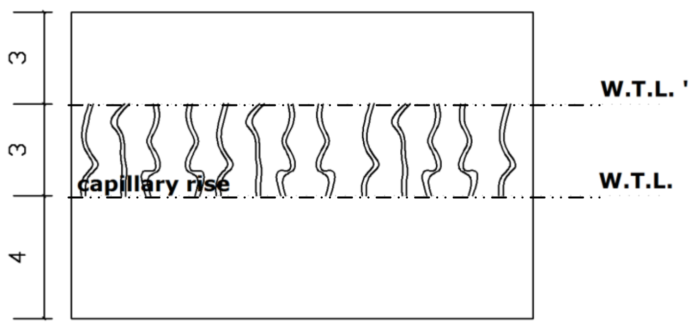


Figure 4.16 The representation of ground profile

The ground stress profile is obtained by firstly calculating the total stress acting on the soil by; (Craig, 1992)

$$\sigma'_v = \sigma_v - u \tag{4.1}$$

where;

σ'_v : effective vertical stress

σ_v : total vertical stress

u : pore water pressure

The total stress acting on a certain depth (z) of soil with the saturated density (γ_s) will be;

$$\sigma_v = \gamma_s \times z \tag{4.2}$$

and the pore water pressure of soil is;

$$u = \gamma_w \times z \tag{4.3}$$

Therefore, the effective vertical stress on soil with depth (z) shall be given as;

$$\sigma'_v = \sigma_v - u = (\gamma_s - \gamma_w) \times z \tag{4.4}$$

As mentioned before, the results of two cases in which the level of water table is variant, will be compared. In the first case the effective ground stress is evaluated as 100 kN/m^2 as seen in Figure 4.17.

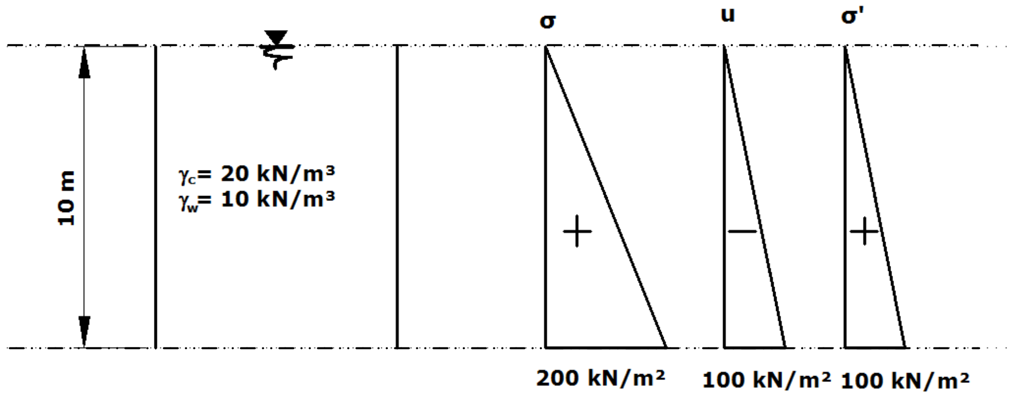


Figure 4.17 Stress Profile of Case 1, W.T.L. at the surface

In case where W.T.L. is below 3 meters from the surface, the pore water pressure would change to u= 70 kN/m^2 where the effective ground stress will be 130 kN/m^2 at 10 meters as seen in the Figure 4.18.

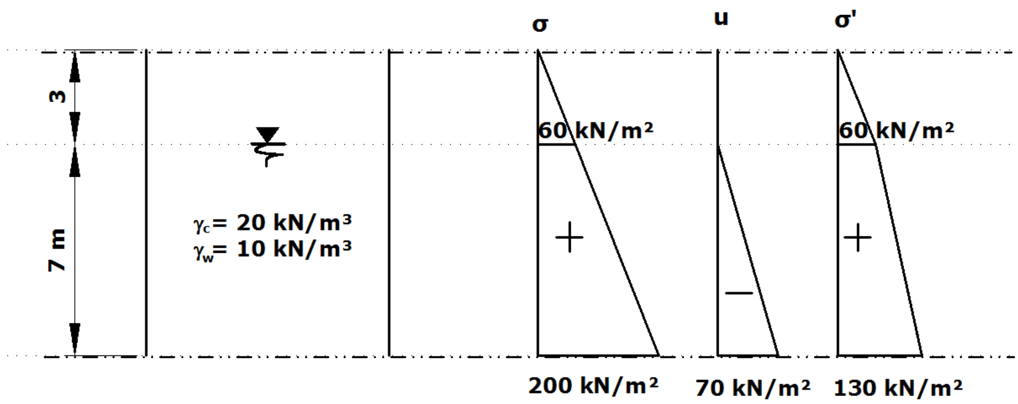


Figure 4.18 Stress Profile of Case 2, W.T.L. 3 meters below the surface

When these calculations are compared, it is seen that the 3 meters drop in W.T.L. results with 30 kN/m^2 increase in effective vertical stress which will be induced to the given ground profile.

4.3.2 The Swelling-Shrinkage

In order to explain the swelling-shrinkage behavior of soil, the three phase diagram will be referred (Figure 4.19).

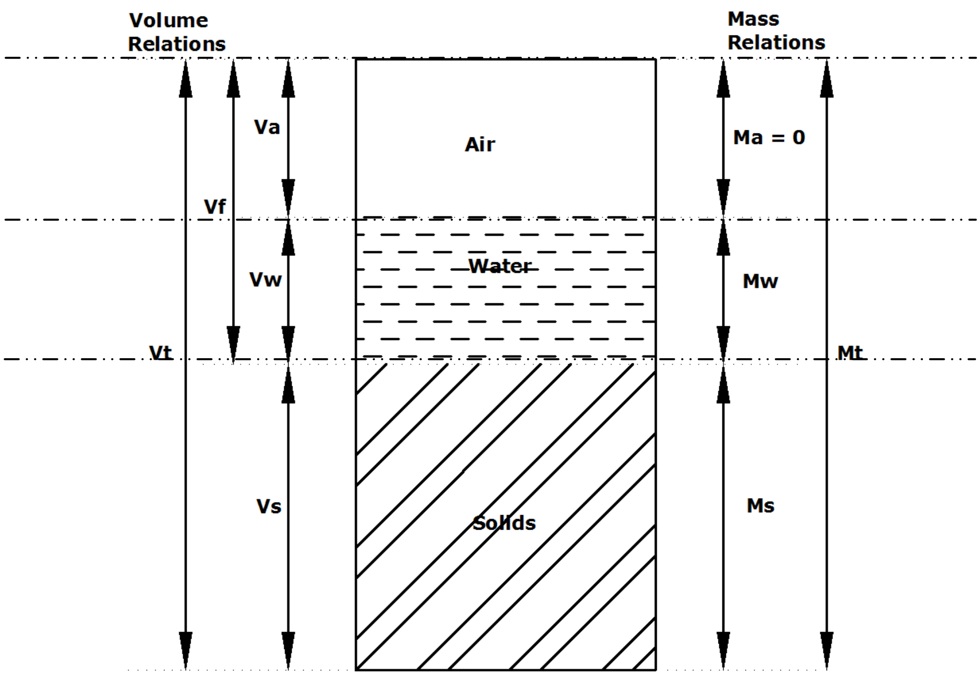


Figure 4.19 Three Phase Diagram of soil (Hillel, 1998)

It consists of representation of a soil in three physical phases that are separated to investigate their interrelations. According to this diagram, the soil mass structure determines the pore space properties where water and air masses are interchanged. In swelling soils, the pore space changes with the soil's water content.

In this study the given ratio of the soil based on its swelling-shrinkage potential is 1%. (Canbay, Çetin, 2008) Therefore 3 cm displacements due to swelling-shrinkage have been included in ground settlement profiles. The soil settlement values are derived for each log by the following equation.

$$S = m_v \times \Delta\sigma'_v \times H \tag{4.5}$$

where;

S	: total soil settlement
m_v	: modulus of volume compressibility
$\Delta\sigma'_v$	: pre-consolidation pressure
H	: depth of the soil layer

The modulus of volume compressibility (m_v) is obtained from previously studied graphical charts (Figure 4.20) whereas the depth of soil layer, i.e. the clay layer, is obtained from the boring log reports considering the effect of capillary rise. The values used in the calculations are given below and the variance of "H" value and the soil settlements due to water table change are given in Table 4.1.

Plasticity Index (PI)	: 35 (from the ground exploration report)
$N_{average}$	: 14.3 (from the ground exploration report)

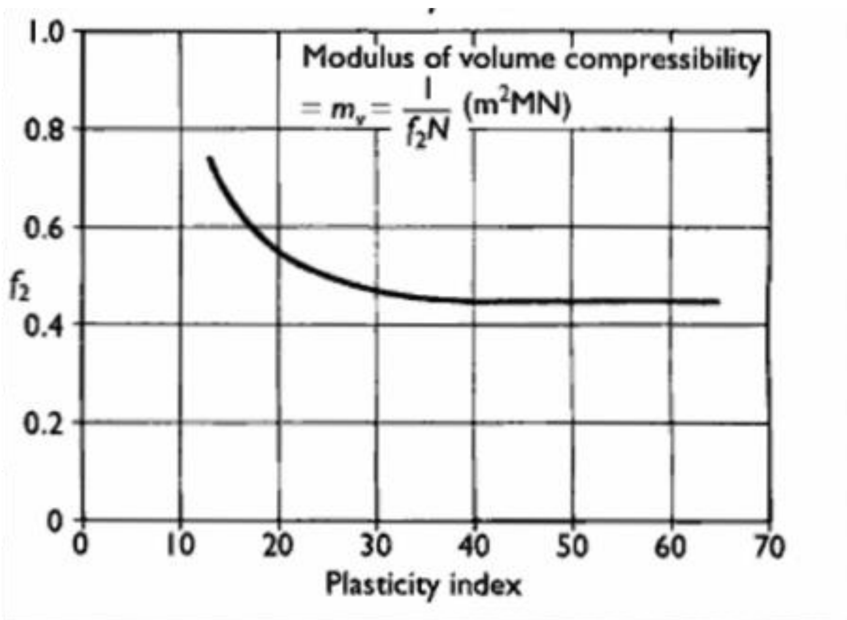


Figure 4.20 Graph of Modulus of Volume Compressibility (Stroud, Butler, 1975)

The total soil settlements are given in Table 4.1 and the resulting ground displacement profile together with the swelling-shrinkage constituent is given in Figure 4.21. The restraints where the given settlements are induced are provided in Figure 4.12. The profile is interpreted in a manner that it fits better with the structural problems that are observed, in this case the crack pattern. This adjustment is assumed not to be influential on the study as the number of boring logs is relatively sparse. It should be noted that better displacement values could be obtained with additional logs especially in estimating the soil profile.

Table 4.1 Total Soil Settlements

Bore Log ID	H (m.)		Settlement due to W.T.L. change (mm)	Total Settlement (mm)
	H_{water}	H_{clay}		
BL 1	3	4.00	26.895	56.895
BL 2	3	1.20	13.203	43.203
BL 3	3	2.10	17.603	47.603
BL 4	3	2.70	20.538	50.538
BL 5	3	1.80	16.137	46.137

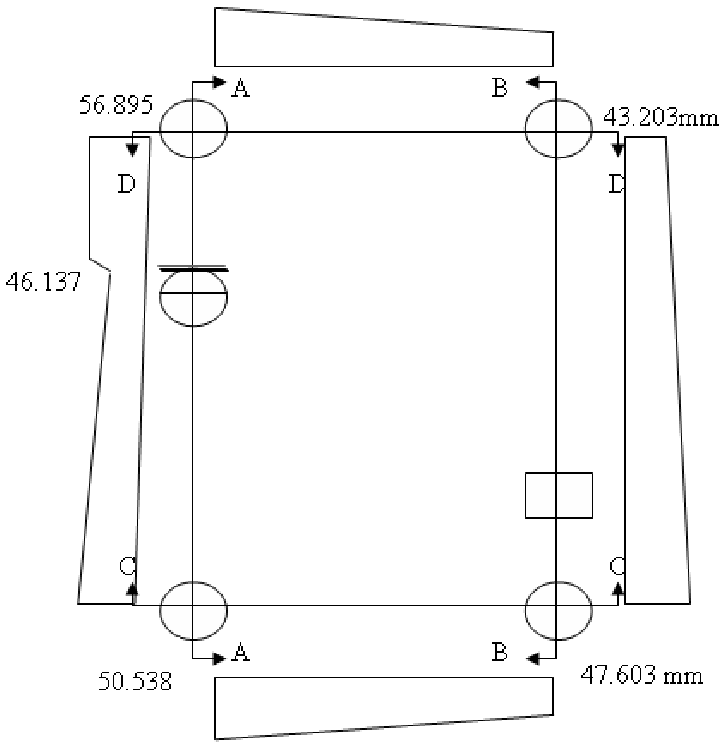


Figure 4.21 Ground Displacement Profile

At this point it is important to mention that, in the analysis stage, it is essential to validate the structural model preferably by comparing the analysis and experimental results. In this particular study based on the structure's historical situation, the verification is done by comparing the analysis results and the observations and the expected behavior. Additionally, a sensitivity analysis has been conducted in terms of elastic material properties and induced soil displacements. These comparisons are given in section 4.6 of this study.

4.4 Modal Analysis

In modal analysis, although the structures have infinite number of modes in practice, generally, first three modes are taken into account in which the deformations can clearly be observed. In this study the structural behavior has been observed in terms of the structure's two directional modal deformations and also the torsion effect. The modal deformation figures and further comments on modal analysis will be provided in the following sections.

4.5 Dynamic Analysis

In Dynamic analysis, a response spectrum analysis has been performed based on the ground observations and Turkish Earthquake Code. The linear seismic analysis method is used for the structure as specified in the earthquake code. (Equation 4.6 and Equation 4.7)

$$A(T) = A_0 \times I \times S(T) \tag{4.6}$$

$$S_{ae}(T) = A(T) \times g \tag{4.7}$$

where;

$A(T)$	: spectral acceleration coefficient
A_0	: effective ground acceleration coefficient
I	: building importance factor
$S(T)$	: spectrum coefficient
$S_{ae}(T)$	: elastic spectral acceleration
g	: acceleration of gravity

The coefficients that are selected for the analysis determined from the results of ground investigation report and properties of the structure according to the Turkish Seismic Code (Bayındırlık ve İskan Bakanlığı, 2007) are given as follows;

$R_a(T_1)$ = 2.0 (as recommended for masonry structures)
$S(T_1)$ = 2.50(as recommended for masonry structures)
I = 1.2 (for intensively but short-term occupied buildings)
A_0 = 0.20 (for seismic zone 3)

where;

$R_a(T_1)$: seismic load reduction factor

The location of the case study structure lies in the 3rd earthquake zone according to the Turkey earthquake zone map prepared by the ministry of public works, therefore, the effective ground acceleration coefficient has been taken 0.20 in the analysis accordingly.

Furthermore, the design spectra used as the input data is obtained from calculations based on the design earthquake (Bayındırlık ve İskan Bakanlığı, 2007) that has 10% probability of exceedance in 50 years. (Figure 4.22) The graph is obtained by idealization of the real system.

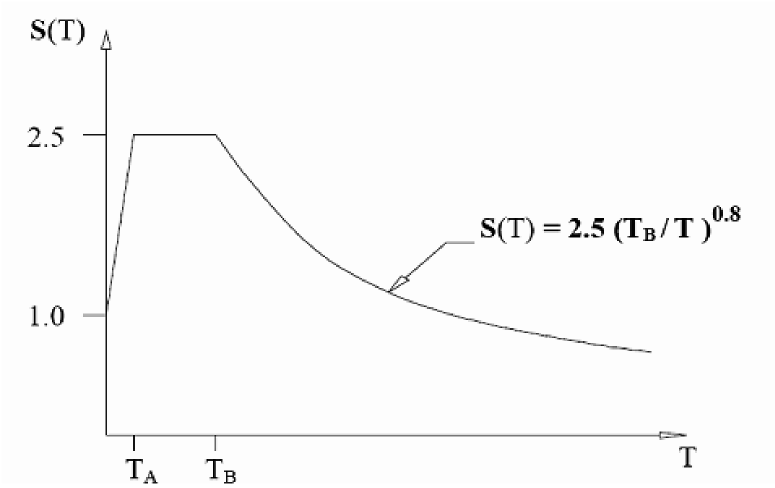


Figure 4.22 The design Spectra (Bayındırlık ve İskan Bakanlığı, 2007)

The base shear obtained from mode superposition method in x and y directions are compared. For further reference, the shear forces obtained from mode superposition method in x and y directions are as follows;

Vtbx = 3533.207 kN,

Vtby = 406.667 kN.

4.6 Analysis Results

In this study, observation of local stresses and the overall behavior of the structure are aimed; therefore by considering the previous researches, linear elastic analysis is thought to be adequate. For better identification of the reason and properties of damage, the linear analysis of the structure is carried out in gravity and dynamic analysis both combined with the soil settlement.

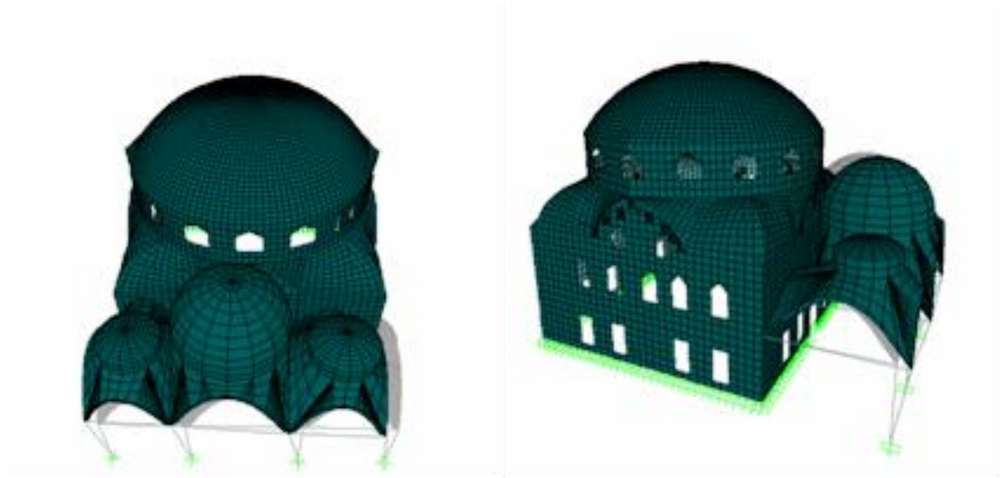
The gravity analysis is carried out under the own weight of the structure whereas in dynamic analysis Turkish Seismic Code (Bayındırlık ve İskan Bakanlığı, 2007) is considered, according to the properties of the setting of the structure. Further numerical results from the analysis are provided in terms of stress distributions, maximum deformations on the structure as well as support displacements under different load combinations.

The structure is composed of a complex geometry; with regard to this, the modal deformations of the first three modal behaviors are selected throughout the modal analysis results. It has been observed that due to the high stiffness of the main structure, the initial first three modal deformations are dominated by the last congregational area whereas the modal behavior of the complete structure is obtained from the sixth and seventh modes. Therefore the natural period of the structure is selected to be 0.07991 and 0.07746 seconds in x and y directions respectively. The total weight of the structure is 28442 kN.

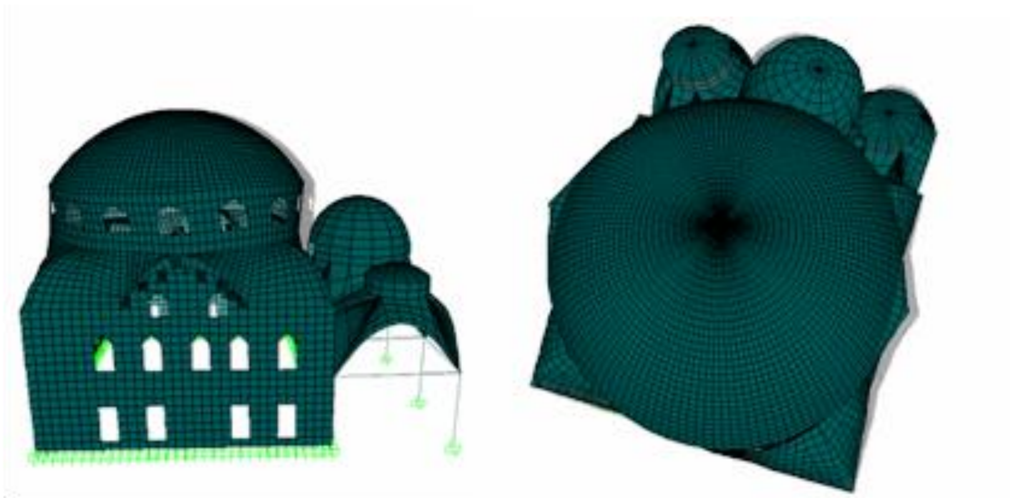
The modal periods may be seen in Table 4.2 and the corresponding deformed shapes could be seen in Figure 4.23 where the gray overlaying lines define the undeformed geometry. Totally 90 modes have been defined to obtain the desired mass participation ratios in x and y directions. The difference between deformation characteristics of the structural geometry is clearly seen in these figures. The last congregational area shows higher deformations along its height as compared to the relatively stiffer main structure.

Table 4.2 Modal Periods of the Structure

Mode Number	Period (sec.)
Mode 1	0.237
Mode 2	0.210
Mode 3	0.137
Mode 4	0.125
Mode 5	0.080
Mode 6	0.079
Mode 7	0.077
Mode 8	0.069
Mode 9	0.064
Mode 10	0.063



(a) (b)



(c) (d)

Figure 4.23 Modal deformed shapes; (a) Mode 1, (b) Mode 2, (c) Mode 6, (d) Mode 7

The analyses results are compared with the recent damaged state of the structure. To investigate the defined elastic properties and the ground settlement profile, previously mentioned sensitivity analysis is conducted, stress concentrations at selected points are considered. For better comparison, the figure showing the selected points (Figure 4.24) from the initiation point of the crack (point A) up to the dome is presented below that will also be referred to observe the stress variation and the displacement values at the selected critical wall section in sensitivity analysis and under different load combinations.

In the finite element analyses, nominal values for modulus of elasticity (E) and Poisson’s ratio (υ) were used. Elasticity modulus and Poisson’s ratio were decided as 30000 MPa and 0.2 respectively. In sensitivity study elasticity modulus (E) and Poisson’s ratio (υ) are changed as 25000 MPa, 0.15 and 35000 MPa, 0.25 respectively. These values are obtained from literature as lower and higher values considering standard deviation of stone masonry units. (Küçükdoğan, 2007, Zhang et al., 2004,) The table showing the stress variations and the mean stress changes at selected points are given in the table below.

Table 4.3 Stress Values from sensitivity analysis

Point ID	Nominal S22 values (kN/ m^2)	S22 stress (υ=0.15)	S22 stress (υ=0.25)	S22 stress (E-25)	S22 stress (E-35)
A	160365	159659	164395	133616	187114
B	155341	157348	153177	129466	181217
C	535	565	451	457	615
D	8620	8229	9547	7129	10112
E	15755	15657	16040	13096	18417
F	-1184	-1130	-1318	-999	-1370
H	7078	7068	7116	5898	8262
I	1696	1701	1688	1407	1977

Table 4.3 (continued) Stress Values from sensitivity analysis

Point ID	%Stress change (υ=0.15)	%Stress change (υ=0.25)	%Stress change (E-25)	%Stress change (E-35)
A	0,440246	-2,51302	16,68007	-16,68
B	-1,292	1,393064	16,6569	-16,658
C	-5,60748	15,70093	14,57944	-14,953
D	4,535963	-10,7541	17,29698	-17,309
E	0,622025	-1,80895	16,87718	-16,896
F	4,560811	-11,3176	15,625	-15,71
H	0,141283	-0,53687	16,67138	-16,728
I	-0,29481	0,471698	17,04009	-16,568

According to these values, it is observed that, for high υ values the stresses at selected points decreased by 0.4%; however, for lower υ values the stresses increased by an average of 1.1%. For lower E values, the stresses decreased by an average of 16%; however, for higher E values, the stresses at selected points increased by 16% on average. Additionally, soil displacements are changed and stresses are observed at selected points on the structure. Hereby it should be recalled that previously the swelling shrinkage potential of the soil profile is foreseen to be 3 cm and this value has been used in the calculations. When 1.5 cm and 6 cm swelling- shrinkage displacements are imposed, the stress concentrations and tensile stress values are almost the same with the results of originally used ground displacement profile, except the 5 kN/m^2 increase in tensile stress at point E, for 6 cm settlement induced to the structure.

Therefore according to these analyses, it can be deduced that the model is insignificantly sensitive and for such a comparative macro model in this study, the soil displacement values as well as the selected average elastic properties are adequate.

Regarding the load combinations, in "Comb1" only the own weight is considered, whereas "Comb2" combines dead load with soil displacement. For displacement values, the axes are defined as; U1, the horizontal in plane displacement, U2, the horizontal out of plane displacement and U3, vertical displacement. The local axes definition could be seen from Figure 4.5. It should also be noted that the units on the stress distributions are in kN/m^2. ($1\ kN/m^2 = 10^{-3}$ MPa)

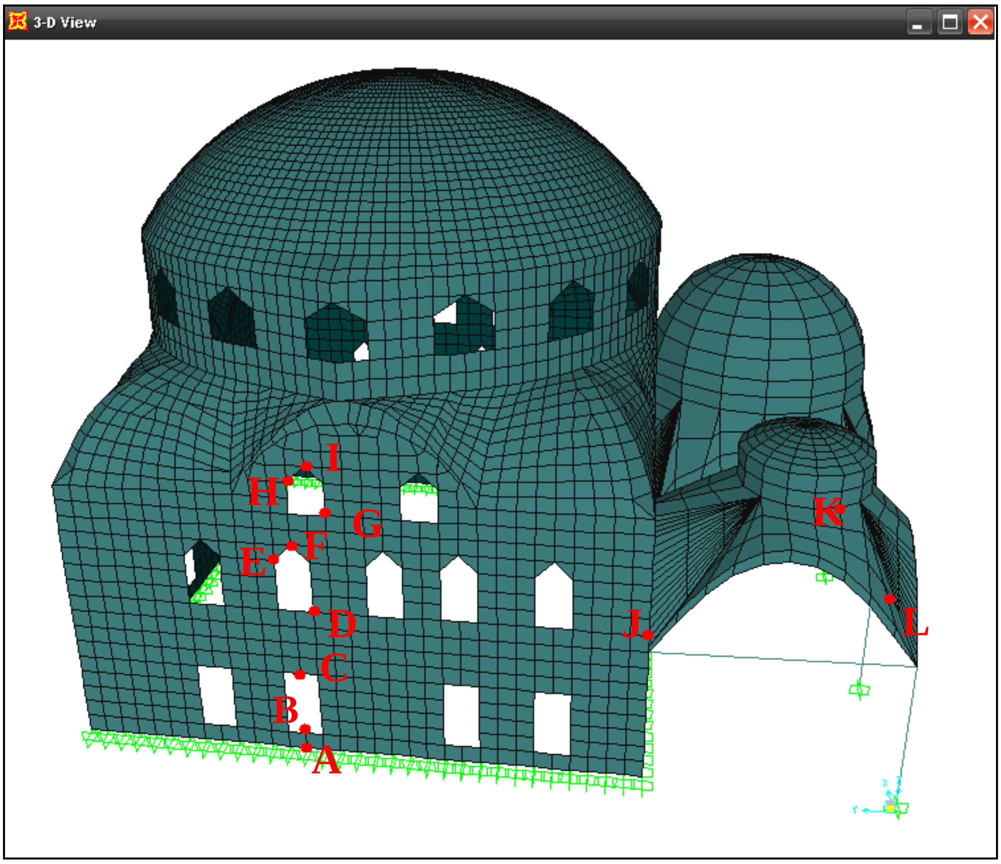


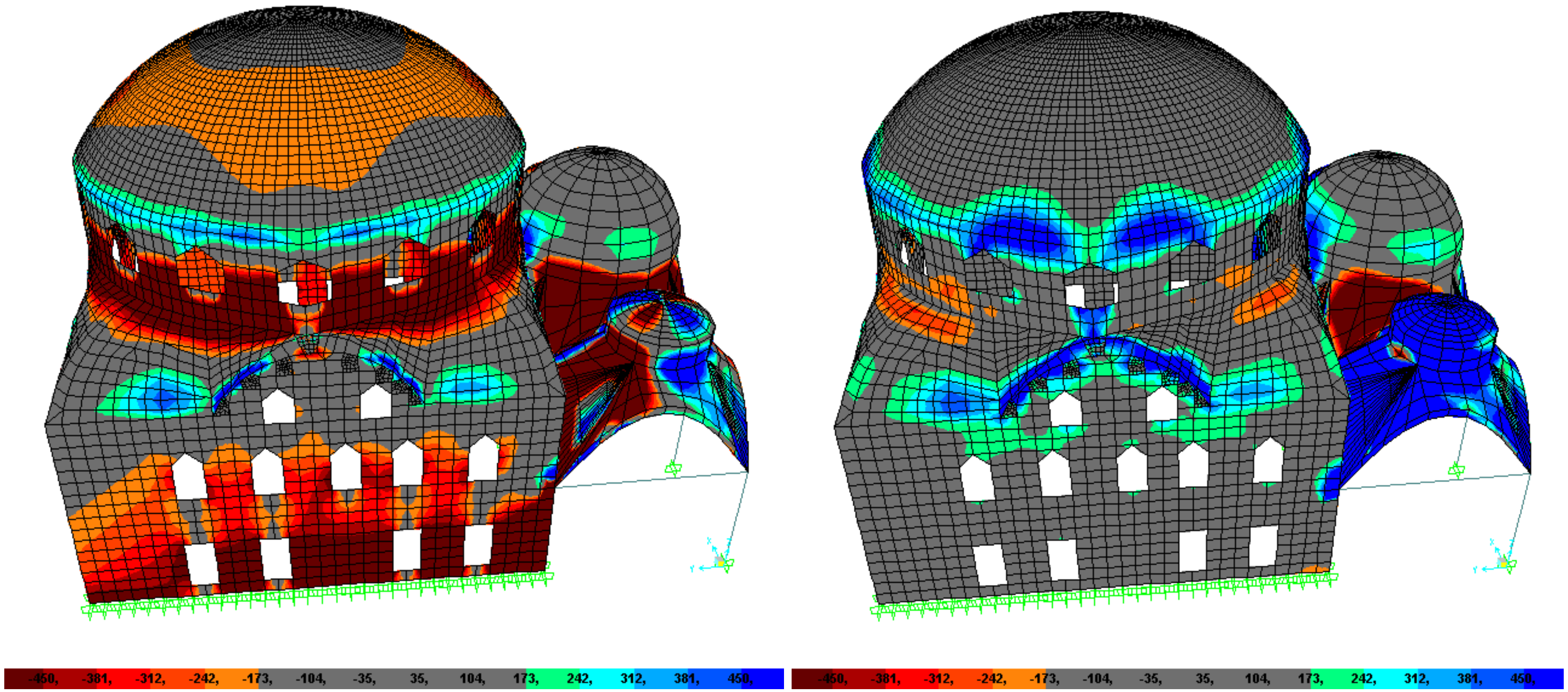
Figure 4.24 Selected Stress Points on wall section A-A

For Comb1, in gravity analysis, the stress concentration throughout the structure fits well with the expected behavior of massive masonry structure where compressive stresses gradually increase to the bottom of the structural walls. The tension areas are also apparent at dome-drum and semi dome-wall connections, and at the corners of pendentives meeting with slender front columns and structural walls. (Figure 4.25)

It is observed that the axial load along the wall height, at the structure due to the gravity loading, increased around openings and reached the maximum value of; -129.58 kN/m^2 at the foundation joint. The displacements as given in Table 4.4 shows the maximum displacement value of 0.00011 m. in vertical direction along the wall height. Also at connection points for Comb1 the S22 stress values are; for point J, 1093 kN, point K, 940 kN, point L, 489 kN.

Table 4.4 Displacement and Stress values of the Structure for Comb1

ID	U1 (m.)	U2 (m.)	U3 (m.)	S22 (kN/ m^2)	S_{MAX} (kN/ m^2)
A	0	0	0	-129	-25
B	-5.7E-07	8.86E-08	3.97E-07	89	156
C	-3.3E-05	9.38E-07	-4.5E-05	59	203
D	-0.00009	-9.3E-07	-5.5E-05	-328	71
E	-0.00012	-3.3E-06	-0.00008	-207	78
F	-0.00014	-7.5E-06	-8.7E-05	-71	237
G	-0.00016	-1.3E-05	-9.1E-05	-120	203
H	-0.00016	-1.2E-05	-0.00011	-11	119
I	-0.00016	-1.7E-05	-0.00011	-16	256

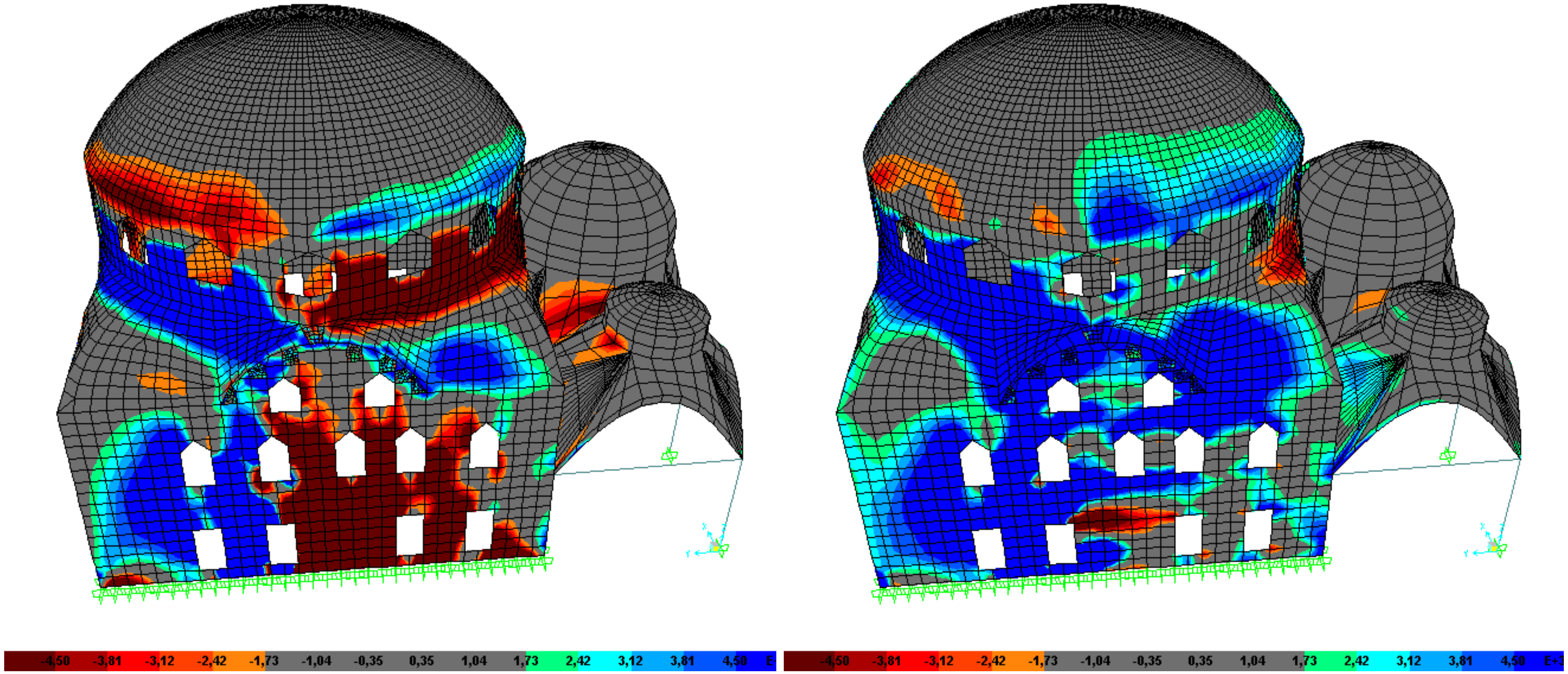


(a) (b)

Figure 4.25 Stress distributions of the model; (a) Comb1 (S22) along axis A-A, (b) Comb1, SMAX

For Comb2, the stresses under the combined action of gravity load and soil displacements are given in Figure 4.26. These stress distributions shows the compression-tension interchanges at selected unit elements along the wall which is used to estimate the possible crack pattern. Due to masonry's material properties, it is known that differential settlements could cause formation of structural damages. Together with the addition of displacement values induced to support joints, the change in tension areas is clearly visible in Figure 4.26, in which the stress concentrations are mainly at areas around openings and connections of structural elements as well as the critical section along the wall.

Compared to the Comb1 gravity analysis, it is observed from the stress distribution figures that; the ground displacement becomes highly detrimental on the masonry structure up to the main dome, and due to clamping action between the dome and the drum the tensile stresses increase at the ends of the drum.



(a) (b)

Figure 4.26 Stress distributions of the model; (a) Comb2 (S22) along axis A-A, (b) Comb2, SMAX

When the stress and displacement values at selected points are examined (Figure 4.27) a sudden stress decrease is observed after point B. From Table 4.5, note that the vertical displacement value (U3) at point A, -0.0569 m, is the displacement that has been induced to the model from ground settlement calculations by the change in water table level and soil's swelling shrinkage.

Additionally, it is seen that tensile stresses increase around openings at points E and H. These results, when compared to the results obtained from Comb1, especially show the influence of the ground settlement along the critical structural wall section. At this point, these areas are compared to the structural crack pattern as seen in detail from Figure 4.28.

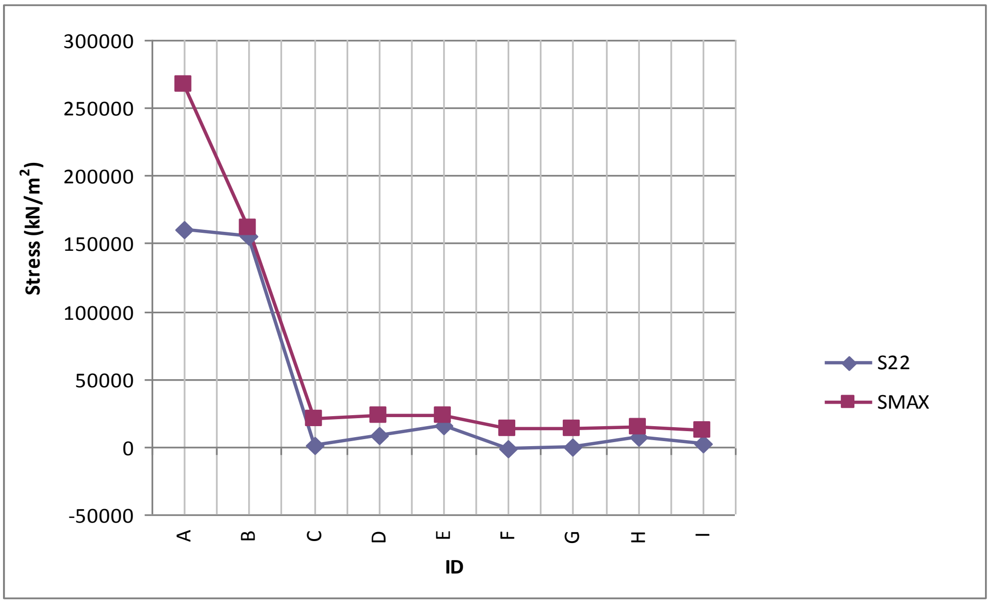


Figure 4.27 Stress Variation for Comb2

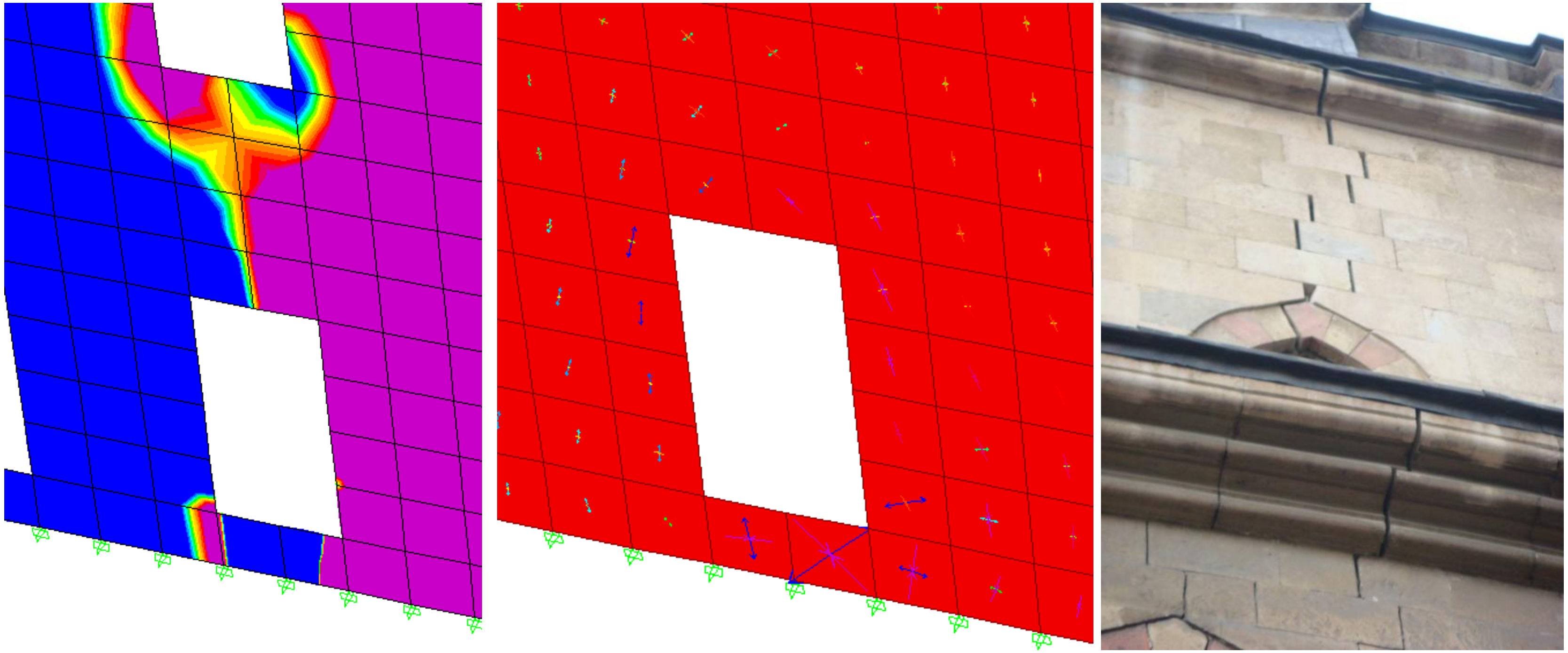
Table 4.5 Displacement and Stress values for Comb2

ID	U1 (m.)	U2 (m.)	U3 (m.)	S22 (kN/ m^2)	S_{MAX} (kN/ m^2)
A	0	0	-0.0569	160365	267343
B	-9.8E-06	0.003068	-0.05432	155341	161060
C	-0.00073	-0.00107	-0.05196	535	20441
D	-0.00209	0.001551	-0.05085	8620	22865
E	-0.00322	0.001132	-0.05314	15755	22441
F	-0.00364	0.001265	-0.05217	-1184	13081
G	-0.00441	0.001709	-0.05111	367	13045
H	-0.00514	0.002091	-0.05239	7078	14953
I	-0.0054	0.002069	-0.05191	1696	11749

At connection points for Comb2 the S22 stress values are; for point J, 3193 kN, point K, 954 kN, point L, 546 kN. Hereby, it is important that Table 4.4 and Table 4.5 should be compared relatively. The stresses in tables should not be solely evaluated.

In Comb1 loading, the levels of the S22 stresses are almost at 100 kN/m^2 (0.1 MPa) and compressive. Nonetheless, in Comb2 loading the S22 stresses are a few thousand times greater and mainly under tension. Point A is the displacement induced region and therefore stress values are very high. Very high stress points addresses probable damaged and cracked regions.

In the next step of the analysis, these results will be compared with the effect of the earthquake load.



(a) (b) (c)

Figure 4.28 Stresses on the cracked section; (a) The vertical stress distribution of Comb2, (S22) (b) Principle stresses acting on the masonry wall under Comb2, (SMAX) (c) The cracked section on the structure

In dynamic analysis, a response spectrum graph has been produced regarding the previously mentioned calculations. The investigation of the model under earthquake load in x and y direction has been carried out by the spectrum analysis that has been obtained according to the ground investigation report and Turkish Earthquake Code. (Bayındırlık ve İskan Bakanlığı, 2007, Canbay, Çetin, 2008)

The stress variation of the selected wall section under the earthquake action in x direction and the stress distribution outputs are seen in following figures. For comparison of different load combinations either with or without the effect of earthquake force, the color scales have been selected specifically in same ranges.

First of all, to investigate the behavior of the structure under earthquake action, a load combination (Comb3) of dead load and the defined earthquake load has been induced to the model.

As seen from Figure 4.29, the maximum tensile stress concentration is basically around connections of front columns with the front arch, and arches connecting with structural walls. It is estimated that these sections would be more vulnerable to an earthquake action. The tensile stress values are apparent around openings which are known to be one of the areas susceptible to failure in masonry structures facing lateral loads.

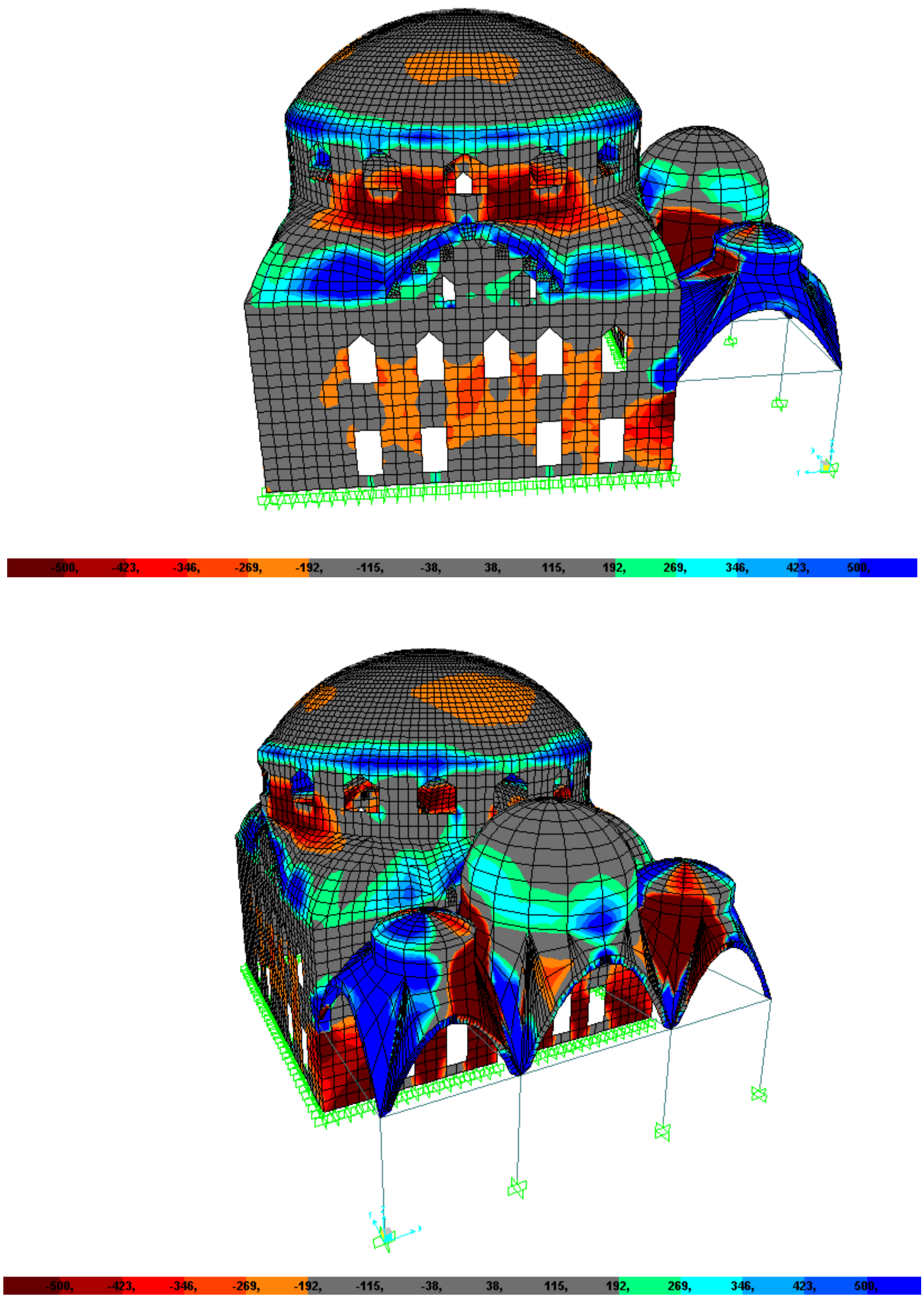


Figure 4.29 Stress distributions of the model; Comb3 (S22)

From Table 4.6 and Figure 4.30, the low displacement and stress values verify the strength of the structural wall under seismic loads. Comparison of earthquake loading (Comb3) with Comb1 (dead load) clearly shows success of wall at selected points on structural wall. Although, the S22 stresses altered from compression to tension mainly, the level of stresses are very low as compared to Comb2 case. Additionally, the changes in lateral displacements are doubled as compared to vertical loading which indicates high rigidity of the main body of the structure.

Table 4.6 Displacement and Stress values for Comb3

ID	U1 (m.)	U2 (m.)	U3 (m.)	S_{22} (kN/ m^2)
A	0	0	0	315
B	1.08E-06	2.1E-07	4.26E-07	289
C	0.000062	5.47E-06	-4.2E-05	127
D	0.000162	4.32E-06	-5.2E-05	-230
E	0.000229	3.99E-06	-7.4E-05	-37
F	0.00026	-2E-07	-8.1E-05	81
G	0.000315	-6.2E-06	-8.4E-05	174
H	0.000352	-2.6E-06	-9.7E-05	259
I	0.000381	-8.8E-06	-0.0001	250

The S22 stresses at the connection points of front columns with the front arch and arches connecting with structural walls are; Point J, 1523 kN/ m^2, Point K, 1569 kN/ m^2 and Point L, 996 kN/ m^2.

To examine the combined behavior of the soil displacements and the earthquake action, another load combination has been defined. (Comb4)

It is seen that the tensile stress areas, showing the stress concentrations are similar to the data obtained from Comb2 and Comb3. The stress contour maps (Figure 4.30), as well as the displacement and stress variation values (Table 4.6) along the critical wall section at selected points, indicate the similarity of these stress values. However, when the stress values of Comb3 and Comb4 are compared the substantial difference in S22 values, therefore the effect of soil settlement on the wall section in the presence of earthquake loads should be noted.

Table 4.7 Displacement and Stress values for Comb4

ID	U1 (m.)	U2 (m.)	U3 (m.)	S_{22} (kN/ m^2)
A	0	0	-0.0569	160810
B	-8.1E-06	0.003068	-0.05432	155541
C	-0.00063	-0.00106	-0.05195	604
D	-0.00184	0.001557	-0.05085	8718
E	-0.00287	0.00114	-0.05313	15925
F	-0.00324	0.001272	-0.05217	-1032
G	-0.00394	0.001717	-0.0511	662
H	-0.00463	0.002101	-0.05238	7352
I	-0.00486	0.002078	-0.0519	1850

Additionally, at connection points for Comb4 the S22 stress values are; for point J, 3744 kN, point K, 1584 kN, point L, 1053 kN.

According to these analysis results, it is deduced that the stress distributions on the cracked section as well as the dome and the structural walls have similar patterns in Comb2 and Comb4 with the results obtained from analysis combined with the ground displacements.

In further identification of the stress values of Comb4 compared to Comb2, a minor reduce is seen in terms of compressive stresses. However, all calculations are based on linear elastic analysis; the deduction of the last comparison should be carefully treated.

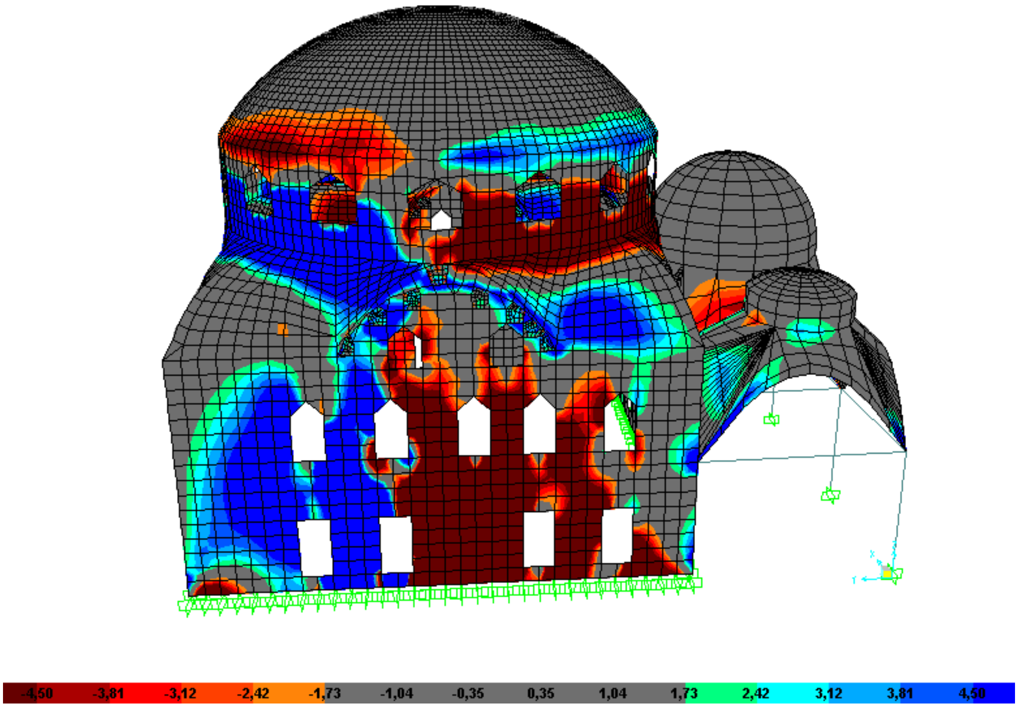


Figure 4.30 Stress distributions of the model; Comb4 (S22) along axis A-A

When the stresses at the connection points, points J. K and L are studied it is seen that, the tensile stresses at these points are rather critical at earthquake actions. The tensile stresses in Comb1 and Comb 2 especially at points K and L are similar; however, in Comb3 under the action of earthquake, the stresses at unit elements are approximately 500 kN/ m^2 greater than in Comb1.

Furthermore, regarding Turkey's high seismicity, the structural behavior of the mosque is observed for 1^{st} seismic zone. The failure pattern is aimed to be investigated referring to structure if it were located at 1st seismic zone. To further investigate the vulnerable regions behavior under more severe earthquake actions, with seismic properties of earthquake zone 1, Comb5 (structural weight and earthquake load) and Comb6 (structural weight, soil displacement and earthquake load) have been defined. According to these load combinations, an approximately linear increase in stresses has been observed at these locations. However, these tensile stress values are relatively low compared to the critical sections at the main structural wall. Therefore it is reasonable to pronounce that although the main structural damage would be due to the action of ground settlements, these sections denoted as points J, K and L would be vulnerable to an extremely severe earthquake action.

Hereby, the damaged section on the wall and probable damage pattern on the wall section will be given based on the above mentioned analysis results. (Figure 4.32)

Figure 4.31 The cracked section of the wall

Based on onsite observations, severe cracks of about 30 mm wide at the initiation point formed at the masonry wall sections propagated from the ground level and the crack width increased while propagating up to the top of the wall.

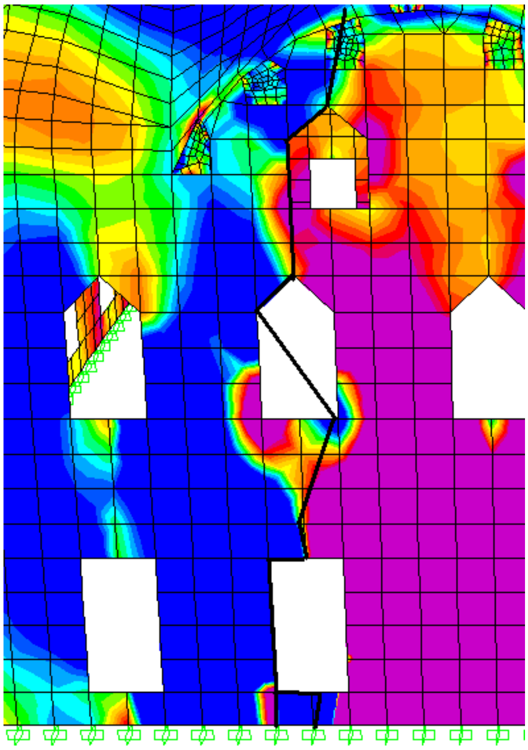


Figure 4.32 The estimated crack pattern

The estimated crack pattern derived from the analysis results fits well with the observations on the structure especially at the initiation of the crack. Comb3 and Comb4 loading combinations indicate that in addition to the existing cracks, the front columns and arch connections are possible vulnerable spots in an extremely severe earthquake. In light of these observations, it can be claimed that the reason of the observed damage which lead to severe disintegrations at the masonry structure is rather due to the proposed soil displacements caused by changes in ground water profile as verified through the conducted analysis.

CHAPTER 5

THE PROPOSED RETROFITTING METHOD

5.1 General

Structural restoration is a difficult task which involves gathering data about the state of the structure and its use, analyzing and evaluating the method to be proposed with careful investigations usually following special guidelines. It should be noted that the structural restoration of such historic buildings should include site investigations, laboratory tests, analysis of the structure as well as the analysis of the proposed rehabilitation method.

The historic structures face several damaging effects for centuries and the ones that stand still today usually show certain failure indications like cracks or deformations and require special analysis methods for damage analysis.

These structures are usually made of cut stone of many kinds, bricks or timber together with mortar and metal elements all varying due to the structure's setting and period of the construction. Therefore, the materials to be used for strengthening methods are especially important in compatibility and durability reasons in order to maintain the renovated members to work together with the old ones under different load effects. Penelis stated that unless these two terms, compatibility and durability are satisfied, the use of modern methods could only be allowed where "reversibility" is provided based on the process and result of the application. (Penelis, 1996) The referred reversible techniques include steel ties at springing line, rings around the dome drum, pre-stressed steel ties and stiffening wooden floors by addition of layer of timber planks. (Figure 5.1)

(a) (b)

Figure 5.1 Sample improvement methods by; (a) Pre-stressed cables (St. Ignatio basilica, Spain), (Croci, 2005) (b) timber planks on wooden floors (Modena et. al., 2009)

Whereas, the irreversible applications include deep pointing the masonry, rebuilding the damaged masonry walls to increase the strength, re-bonding masonry blocks, grouting for increasing masonry strength and reinforcement of the masonry structure. (Figure 5.2)

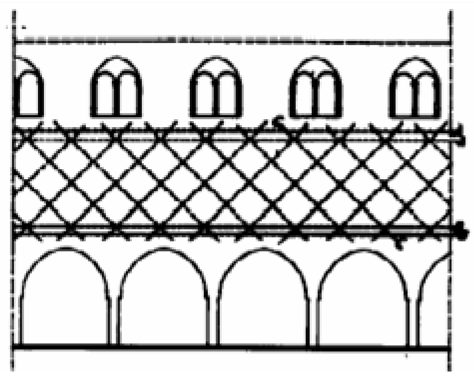


Figure 5.2 The sketch showing reinforcement of a masonry structure (Penelis, 1996)

The materials used for all intervention method on masonry structures should include use of suitable masonry block material such as stone, brick or marble and mortar. If steel is being used, it is important to take into account the corrosion problem which could cause low strength and bonding problems at the intervention. Also, Maurenbrecher mentioned the importance of the type of mortar for the rehabilitation technique that needs to be durable for a period of time. (Maurenbrecher, 2004) Therefore, the mortar used for intervention should meet the properties of the original material as much as possible for compatibility in terms of strength, thermal properties and water absorption capacity and should also have good durability.

The basic principles recommended by ICOMOS that should be considered in these structural interventions have been briefly given in section 3.6, the advised method for the analyzed structure and the types of such applications will be covered in this chapter.

5.2 The Proposed Retrofitting Method

The structure showed serious damages and therefore investigations and projects have been conducted by SAYKA Construction Architecture and Engineering Company with Middle East Technical University, Department of Civil Engineering under the guidance of Dr. Erdem Canbay and Dr. Kemal Önder Çetin. (Canbay, Çetin, 2008) The cause of the damage is firstly identified and a suitable rehabilitation method has been proposed after the analysis of the structure. With respect to the studies on the current state of the structure, the masonry mosque has been concluded to be suffering from the differential soil settlements and the method therefore include two different tasks governing the soil actions and the structural load effects.

In order to prevent further deformations to be effective on the structure and to prevent the ground deformations reach the mosque, mini piles with 0.3 meter diameter, having 3 meters rock socket depth is to be constructed into the andesite stone. (Figure 5.3)

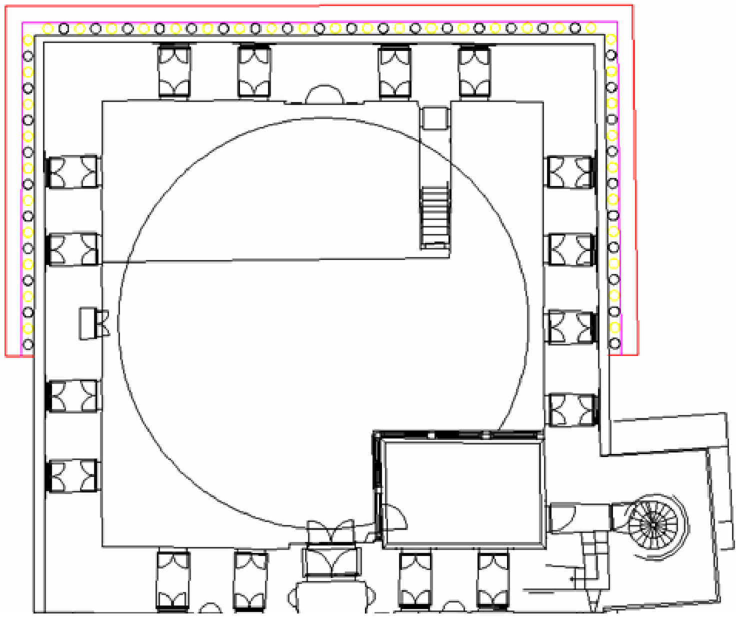


Figure 5.3 The plan showing the proposed mini piles

The pile cap beam is suggested to be anchored to the main foundation and this procedure need careful studying itself, due to lack of knowledge about the structural properties of the structure's foundation system.

Secondly, it is necessary to strengthen the damaged dome. As mentioned in previous chapters, the masonry structure that works under compression loads and face ground displacements leading to tensile forces acting on the structure, showed crack formations. The brittle characteristics of masonry cause formation of serious cracks, which in the case study start from the ground level up to the dome and lateral deformations around the drum. The proposed procedure therefore includes stabilizing the dome by introduction of a circular ring around.

The recommended method is composed of placing a prestressed steel ring around the perimeter of the dome made of a stainless steel section of 10 mm thickness and 200 mm height in eight pieces enfolding the dome. (Figure 5.4)

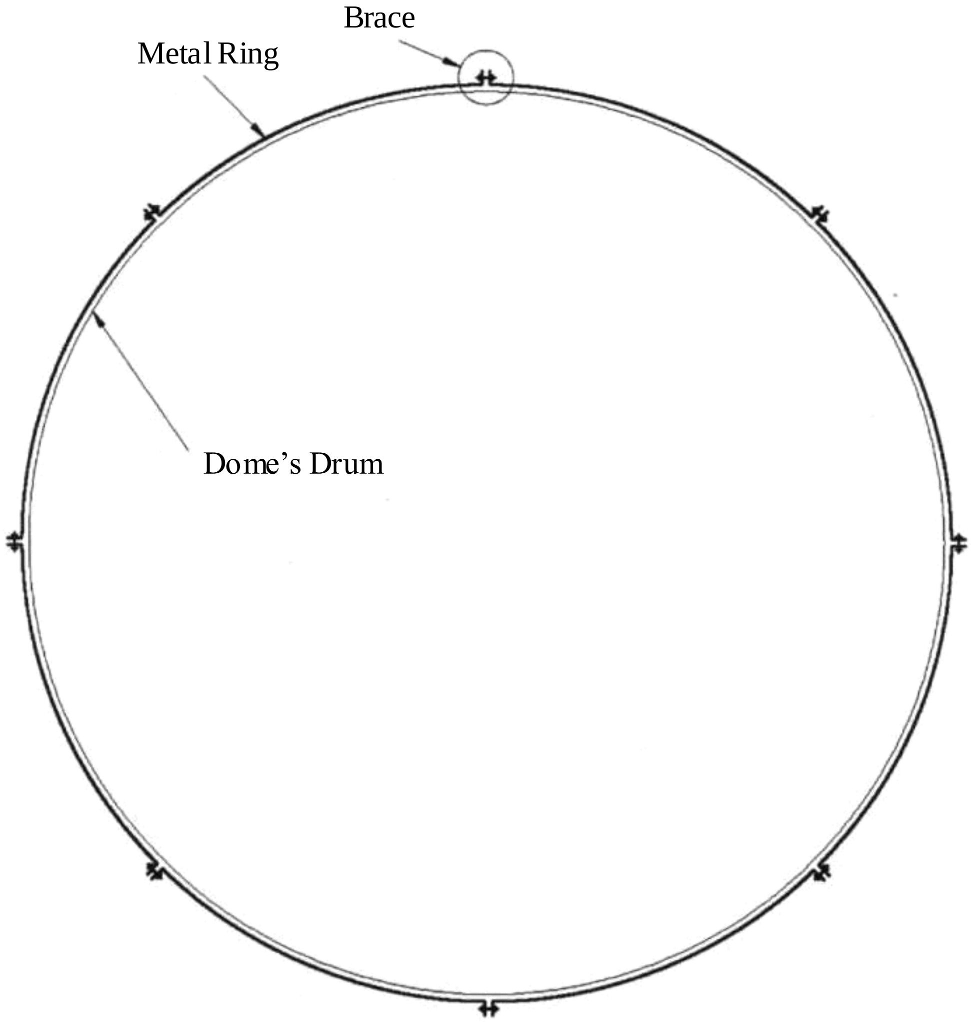
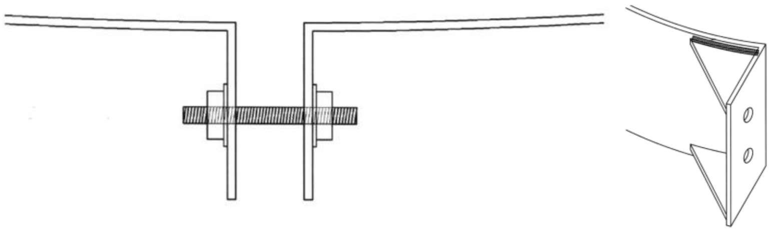


Figure 5.4 The sketch showing the proposed bracing method (Canbay, 2008)

The pieces would be connected together with two high strength Ø22 bolts, the section details can be observed in Figure 5.5.



(a) (b)

Figure 5.5 The Bracing Detail (Canbay, 2008); (a) The connection detail of the bolt (b) The corner weld detail of the steel plate

For compatibility and durability reasons and to provide sustainability of the technique, the corrosion problem should be considered. Therefore, corrosion resistant steel rim and corrosion resistant bolts should be used at connections. Additionally, the lead plates that will be used to cover the dome would provide some protection.

To investigate the method, the steel rim is modeled around the structure with frame elements of 10 mm thickness. When the stress values are compared at points B, H and below the dome drum windows, it is seen that the tensile stresses decreased by 0,008%, 2,8% and 11,1% respectively. The decrease in tensile stresses around the rim shows that the proposed method would decrease the stresses and prevent further deformations at the dome drum as expected.

The strengthening method would be concluded by proper treatment of the cracks using suitable materials compatible with the structure's historic texture, advised to be of khorasan mortar. It should also be pointed out that, due to the state of the damage and the properties of the particular application, the method should be applied by experienced and qualified professionals. The restoration applications that have been carried out so far on the structure are given in Figure 5.6.

Figure 5.6 Photos of the applied restoration applications

CHAPTER 6

CONCLUDING REMARKS

6.1 Conclusion

The structural analyses of historic buildings possess a more difficult task due to lack of information on material properties and regulations and due to the restrictions about structural investigation methods. Among other methods, Finite Element Method is proven to be one of the most capable analysis methods where detailed deformation and stress distributions can be obtained.

In this study, as the overall behavior of the structure is investigated, linear analysis has seen to be adequate. The aim of this study is to identify possible types of structural damages on historic masonry structures and to find out their reasons. A seriously damaged Anatolian Ottoman Mosque is chosen for the case study for investigation and a finite element model is conducted to obtain the deformations using numerous shell and frame elements. The structure's modal analysis as well as its stress distribution has been obtained from the case study.

It is seen from the modal periods that, local behavior of the front section of the structure dominates the first vibration modes and to identify the modal behavior of the main structure, the local modal behavior of the front section has been eliminated. Therefore, for structures having composite structural sections with comparatively different characteristics, the modal behavior should be carefully examined.

As the structure possesses serious swelling- shrinkage problems, ground profile has been produced and induced to the model considering the ground investigations and onsite explorations. The calculated displacement values has been calibrated according to the structural damage however, as the boring logs drilled during site explorations are relatively sparse the provided profile is regarded adequate.

Load combinations are defined to analyze the structural behavior under, its weight, soil displacements, earthquake loads. The overall stress distributions especially on the damaged wall section have been studied.

It has been seen that under gravity loads combined with the ground displacements, the tensile stress concentrations are intense at the cracked region following the path from the ground level up to the main dome. The crack pattern derived from the analysis results and observations on the structure provided a match in these results, therefore; it can be claimed that the reason of the observed damage which lead to severe disintegrations at the masonry structure is rather due to the proposed soil displacements caused by changes in ground profile.

Furthermore, earthquake analysis is carried out to see the critical areas in seismic action. It is observed that, the stresses developed on the damaged section are much smaller than the stresses obtained from the analysis of the load combination that consists of structural weight and ground settlements. In order to see the possible vulnerable regions at the structure under a severe earthquake, a design spectrum for 1^{st} seismic zone has been produced and combined with results obtained from the analysis of the structure that is located at the 3^{rd} earthquake zone. When the top most points on the damaged structural wall is considered, i.e. points G and I, the stresses developed in earthquake analysis for 1^{st} seismic zone gave comparatively higher values than 3^{rd} seismic zone. At front columns and arches connections, the stresses greatly increased under the 1^{st} seismic zone design earthquake. Therefore it is deduced that beside the ground displacement, vulnerable locations to damage under severe earthquake action would most likely be, the cracked section at the structural wall and the front columns and arches connections.

Finally, considering the analysis results, intervention methods have been recommended taking into account the historic value of the structure. The terms of reversibility and compatibility are considered to provide structural safety together with the properties of the original structure. To prevent further soil displacements to be effective on the structure mini pile application up to firm soil and to avoid any further propagation of cracks and disintegrations at the dome a steel ring around the damaged dome base is recommended.

6.2 Recommendations for Further Studies

Hereby, the analysis of a previously damaged structure's model is conducted with shell elements and its structural behavior is observed in the elastic range.

For further studies, the model could be improved by introducing the nonlinear material properties as well as providing material test results of the structure if would be possible, and also including an earthquake histogram data to obtain a more detailed response of the structure. The proposed method could be analyzed through further computational analysis that is also including the case scenario of the crack propagating on the dome and experimental investigations on site to observe the studied analysis together with the proposed method in this study.

REFERENCES

Abrams, D.P., “Response of Unreinforced Masonry Buildings.”, *Journal of Earthquake Engineering, Vol. 1, Issue 1*, 1997: 257-273.

Azevedo, J., Sincraian, G.E., “Modelling the Seismic Behavior of Monumental Masonry Structures.”, *Proceedings of Archi*, Bethelem, 2001.

Başkan, S., *Ankara Cenabi Ahmet Paşa Camii.*, 1st Ed., Kültür ve Turizm Bakanlığı Yayınları, Ankara, 1993.

Bayındırlık ve İskan Bakanlığı, *Deprem Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelik*, Ankara, 2007.

Bayraktar, A., *Tarihi Yapıların Analitik İncelenmesi ve Sismik Güçlendirme Metodları.*, 1st Ed., Beta Basım A.Ş., İstanbul, 2006.

Binda, L., Pina-Henriques, J., Anzania, A., Fontana, A., Lourenço, P.B., “A contribution for the understanding of load-transfer mechanisms in multi-leaf masonry walls: Testing and modelling.” *Engineering Structures, Vol. 28*, 2006: 1132–1148.

Binda, L., Anzani, A., Saisi, A., *Failures due to long-term behavior of Heavy Structures, Learning From Failure.*, WitPress, Southampton, Great Britain, 2008.

Blum, P.Z., “The Sequence of the Building Campaigns at Salisbury.”, *The Art Bulletin, College Art Association, Vol. 73, No. 1*, 1991: 6-38.

Braun, H., *Historical Architecture, The Development f Structure and Design*, Faber and Faber Limited, Great Britain, 1959.

Brown, F.E., “Roman Architecture.”, *College Art Journal, Vol. 17, No. 2*, 1958: pp. 105-114.

Browne, E.A., *Romanesque Architecture.*, Kessinger Publishing, U.S.A., 2005.

Calderini, C., Cattari, S., Lagomarsino, S., "In-plane Strength of Unreinforced Masonry Piers.", *Earthquake and Structural Dynamic, Vol. 38, Issue 2,* 2009: 243-267.

Camp II, J.M., Dinsmoor Jr., W.B., *Ancient Athenian Building Methods.*, The Stinehour Press, Athens, 1984.

Canbay, E., M.E.T.U. Department of Civil Engineering, *Tetkik Raporu, Cenabi Ahmet Paşa Camii Değerlendirme Raporu*, 2008.

Canbay, E., Çetin, K.Ö., M.E.T.U. Department of Civil Engineering, *Cenabi Ahmet Paşa Camii Cephe Değerlendirme Raporu*, 2008.

Computers and Structures Inc., CSI: Structural Analysis Program (SAP 2000)., Berkeley, California, 2000.

Craig, R.F., *Soil Mechanics.*, 5th Ed., Chapman& Hall, Great Britain, 1992.

Croci, G., "Structural Restoration of Historic Buildings: General Guidelines.", *Structural Engineering International, International Association for Bridge and Structural Engineering, Vol. 5, No. 2*, 1995:74-75.

Croci, G., *The Conservation and Structural Restoration of the Architectural Heritage.*, Computational Mechanics Publications, Southampton, Great Britain, 1998.

Crouch, D.P., *History of Architecture, Stonehenge to Skyscrapers.*, McGraw-Hill Publishing, U.S.A., 1985.

Curtin, W.G., Shaw, G., Beck, J.K., Bray, W.A., *Structural Masonry Designers' Manual.*, 3rd Ed., Blackwell Publishing, Oxford, Great Britain, 2006.

Dexter, A.R., “Advances in Characterization of Soil Structure.”, *Soil and Tillage Research, Vol. 11, Issues 3-4,* 1988: 199-238.

Dhanasekar, M., Page, A.W., Kleeman, P.W., “The Failure of Brick Masonry under Biaxial Stresses.”, *Proceedings of Institution of Civil Engineers, Part 2, 79*, 1985: 295-313.

Drysdale, R.G., Hamid, A.A., Baker, L.R., *Masonry Structures: Behavior and Design*. 2nd Ed., The Masonry Society, Colorado, U.S.A., 1999.

Edward, C. and Edward, E., “The Evolution of the Shouldered Arch in Medieval Islamic Architecture.”, *Architectural History, Vol. 42*, 1999: 68-95.

Encyclopedia- Britannica, http://www.britannica.com/bps/image/251562/96028/Hagia-Sophia-Istanbul, last visited on July 2010.

Encyclopedia- Britannica, http://www.britannica.com/bps/image/522973/15386/Cathedral-of-Santa-Maria-del-Fiore-in-Florence-constructed-between, last visited on July 2010.

Erdoğan, T.Y., *Materials of Construction*, METU PRESS, Turkey, 2002

Erzen, J., “Sinan as Anti-Classicist.”, *Muqarnas, BRILL, Vol. 5,* 1988: 70-86.

European Committee for Standardization, *Eurocode 6, BS EN 1052-1, Design of Masonry Structures, Methods of Test for Masonry Determination of Compressive Strength.* 1999.

Feilden, B.M., *Conservation of Historic Buildings.*, 3rd Ed., Butterworth-Heinemann Publications, Great Britain, 2003.

Fletcher, B., A History of Architecture, 20th Ed., Butterworth-Heinemann Publications, Great Britain, 1996.

Fulbright Association, http://www.fulbright.org/conference/2004/photos/Petronotis_Delphi1.JPG, last visited on July 2010.

Guerci, M., "A Late Seventeenth-Century Case Study in Rome: The Construction of the Palazzo Mancini, 1686-1690.", *Proceedings of the Third International Congress on Construction History*, 2009:759-766.

Hendry, A.W., "Masonry Walls: Material and Construction.", *Construction and Building Materials, Vol. 15*, 2001: 323-330.

Hillel, D., *Environmental Soil Physics.*, Academic Press, U.S.A., 1998.

Huerta, S., "Galileo was Wrong: The Geometrical Design of Masonry Arches.", *Nexus Network Journal, Vol. 8, No: 2,* 2006: 25-52.

International Council on Monuments and Sites, "ICOMOS Charter- Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage.", *ICOMOS 14th General Assembly,* Zimbabwe, 2003.

Ip, F., *Compressive Strength and Modulus of Elasticity of Masonry Prisms.*, MSc. Thesis, Carleton University, Ottawa, 1999.

Juhasova, E., Sofronie, R., Bairrao, R., "Stone Masonry in Historical Buildings- Ways to Increase Their Resistance and Durability.", *Engineering Structures, Vol. 30, Issue 8*, 2008: 2194-2205.

Karaesmen, E., *CE480 Introduction to Architectural Engineering, Lecture Notes.*, Boğaziçi University Civil Engineering Department, Istanbul, 2007.

Karaesmen, E., *Sinan teması üzerine Çeşitlemeler.*, TMMOB İnşaat Mühendisleri Odası, Ankara, 2008.

Kuban, D., "The Style of Sinan's Domed Structures.", *Muqarnas, BRILL, Vol. 4,* 1987: 72-97.

Küçükdoğan, B., *An Investigation of Strengthening of Historical Masonry Constructions By Steel Skeleton.*, MSc. Thesis, Middle East Technical University, Ankara, 2007.

Lagomarsino, S., Resemini, S., "The Assessment of Damage Limitation State in the Seismic Analysis of Monumental Buildings.", *Earthquake Spectra, Vol. 25, Issue 2*, 2009: 323-346.

Lapunzina, A., *Architecture of Spain.*, 1st Ed., Greenwood Press, Connecticut, U.S.A., 2005.

Lourenço, P.B., *Computational Strategies for Masonry Structures.*, PhD. Thesis, Delft University, Netherlands, 1996.

Lourenço, P.B., "Experimental and Numerical Issues in the Modeling of the Mechanical Behavior of Masonry.", *Proceedings of Structural Analysis of Historical Constructions*, 1998: 57-91.

Lourenço, P.B., "Historical Structures: Models and Modelling.", *Proceedings of EPMESC VII: Computational Methods in Engineering Science*, 1999: 433-442.

Lourenço, P.B., "Analysis and Restoration of Ancient Masonry Structures: Guidelines and Examples.", *Proceedings of Innovative Materials and Technologies for Construction and Restoration*, Italy, 2004: 23-41.

Lourenço, P.B., "Recommendations for Restoration of Ancient Buildings and the Survival of a Masonry Chimney.", *Construction and Building Materials, Vol. 20, Issue 4*, 2006: 239-251.

Macleod, I.A., *Analytical Modeling of Structural Systems, An Entirely New Approach with Emphasis on the Behavior of Building Structures.*, Ellis Horwood Publishing, Southampton, Great Britain, 1990.

Mark, R., Hutchinson, P., "On the Structure of the Roman Pantheon.", *The Art Bulletin, Vol. 68, No. 1*, 1986: pp. 24-34.

Maurenbrecher, A.H.P., "Mortars for Repair of Traditional Masonry.", *Practice Periodical on Structural Design and Construction, ASCE, Vol. 9, Issue 2*, 2004: 62-65.

McNary, W.S., Abrams D.P., "Mechanics of Masonry in Compression.", *Journal of Structural Engineering, ASCE, Vol. 111*, 1985: pp. 857-870.

Mistler, M., Butenweg, C., Meskouris, K., "Modelling Methods of Historic Masonry Buildings Under Seismic Excitation.", *Journal of Seismology, Vol. 10,* 2006: 497-510.

Modena, C., Casarin, F., Porto, F., Garbin, E., Mazzan, N., Munari, M., Panizza, M., Valuzzi, M.R., "Structural Interventions on Historical Masonry Buildings: Review of Eurocode 8 Provisions in the Light of the Italian Experience.", *Proceeding of the Workshop on Eurocode 8 Perspectives from the Italian Standpoint*, 2009: 223-236.

Mosoarca, M., Giancu, V., "Seismic Management and Damage Prevention of Religious Buildings Situated in Seismic Areas.", Proceedings of the International Conference on RISK MANAGEMENT, ASSESSMENT and MITIGATION, 2010.

Oliveira, D.V., *Experimental and Numerical Analysis of Blocky Masonry Structures Under Cyclic Loading.*, PhD. Thesis, University of Minho, Portugal, 2003.

Oliveira, D.V., "Cyclic behaviour of stone and brick masonry under uniaxial compressive loading.", *Materials and Structures, Vol. 39,* 2006: 247-257.

Paulay, T., Priestley, M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings.*, John Wiley & Sons Inc., U.S.A., 1992.

Penelis, G.G., "Techniques and Materials for Structural Restoration.", *Proceedings of the Eleventh World Conference on Earthquake Engineering*, Acapulco, 1996: 1388.

Penelis, G.G., “Structural Restoration of Historical Buildings in Seismic Areas.”, *Progress in Structural Engineering and Materials, Vol. 4, Issue 1*, 2002: 64-73.

Prache, A., *Chartres Cathedral*, Caisse nationale des monuments historiques et des sites: CNRS Editions, France, 1993.

Rossi, C., Russo, F., Russo, F., *Ancient Engineers' Inventions, Precursors of the Present.*, History of Mechanism and Machine Science, Vol. 8, Italy, 2009.

SAYKA Construction Architecture Company, Cenabı Ahmet Paşa Camisi Restorasyon Projesi, 2008.

Seidel, L., *Legends in Limestone: Lazarus, Gislebertus and the Cathedral of Autun.*, 1st Ed., University of Chicago Press, Chicago, 1999.

Stroud, M.A., Butler, F.G., “The Standard Penetration Test and the Engineering Properties of Glacial Materials.” *Proceedings of the symposium on Engineering Properties of Glacial Materials*, Midlands, Great Britain, 1975.

Szolomicki, J.P., “Structural Behavior of Masonry Vaults.”, *Proceedings of the 18th International Conference on the Application of Computer Science and Mathematics in Architecture and Civil Engineering*, Germany, 2009.

Sykora, D.W., Hynes M.E., Karaesmen E., *Report of an International Workshop on Preserving Historic Buildings of Major Importance.*, US Army Corps of Engineers, Waterways Experiment Station, U.S.A., 1993.

Thais, Arte e Natura, http://www.thais.it/architettura/Bizantina/highres/Foto_023.htm, last visited on July 2010.

Tomazevic, M., “Shear resistance of masonry walls and Eurocode 6: Shear versus Tensile Strength of Masonry.”, *Materials and Structures, Vol. 42*, 2009: 889-907.

Toth, A.R., Orban, Z., Bagi, K., "Discrete Element Analysis of a Stone Masonry Arch.*", Mechanics Research Communications, Vol. 36, Issue 4,* 2009: 469-480.

Triantafillou, T.C., Fardis, M.N., "Strengthening of Historic Masonry Structures with Composite Materials.", *Materials and Structures, Vol. 30,* 1997: 486-496.

T.S.E., *TS498, Yapı Elemanlarının Boyutlandırılmasında Alınacak Yüklerin Hesap Değerleri.*, 2nd Ed., T.S.E., Ankara, 2003.

Verstrynge, E., Schueremans, L., Gemert, D.V., Ignoul, S., Wevers, M., "Long-term Behavior of Historical Masonry- A Quantitative Acquisition of the Damage Evolution.", *Proceedings of the International Conference on Structural Analysis of Historical Constructions, Vol. 2,* 2008: 747-754.

Watkin, D., *A History of Western Architecture,* Laurence King Publishing, Great Britain, 2005.

Zhang, X., Koutsabeloulis, N. C., Hope, S., Pearce, A., "A finite element analysis for the stability of dry stone masonry retaining walls.", *Geotechnique 54, No. 1,* 2004: 57–60.