AN INVESTIGATION OF STRENGTHENING OF HISTORICAL MASONRY CONSTRUCTIONS BY STEEL SKELETON

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

BY

BİLGE KÜÇÜKDOĞAN

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE
OF
MASTER OF SCIENCE IN BUILDING SCIENCE
IN
ARCHITECTURE

AUGUST 2007

Approval of the Thesis;

AN INVESTIGATION OF STRENGTHENING OF HISTORICAL MASONRY CONSTRUCTIONS BY STEEL SKELETON

Submitted by **BİLGE KÜÇÜKDOĞAN** in partial fulfillment of the requirements for the degree of **Master of Science in Building Science, Middle East Technical University** by,

Prof. Dr. Canan Özgen
Dean, Graduate School of **Natural and Applied Sciences** _______________

Assoc. Prof. Dr. Güven Arif Sargın
Head of Department, **Dept. of Architecture** _______________

Assoc. Prof. Dr. Ali İhsan Ünay
Supervisor, **Dept. of Architecture, METU** _______________

Examining Committee Members:

Dr. Erhan Karaesmen (*)
Dept. of Architecture, METU _______________

Assoc. Prof. Dr. Ali İhsan Ünay (**)
Dept. of Architecture, METU _______________

Prof. Dr. Mehmet Emin Tuna
Dept. of Architecture, Gazi University _______________

Assist. Prof. Dr. Saadet Toker
Dept. of Architecture, Süleyman Demirel Üniversity _______________

Assist. Prof. Dr. Ali Murat Tanyer
Dept. of Architecture, METU _______________

Date: _______________

(*) Head of Examining Committee
(**) Supervisor

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: Bilge Küçükdoğan

Signature :

ABSTRACT

AN INVESTIGATION OF STRENGTHENING OF HISTORICAL MASONRY CONSTRUCTIONS BY STEEL SKELETON

KÜÇÜKDOĞAN, Bilge
M.S., Department of Architecture
Supervisor: Assoc. Prof. Ali İhsan Ünay

August 2007, 114 pages

Historical masonry structures are important cultural assets which reveal the social, archaeological, aesthetic, economical, political, architectural and technical features of their times. Within the course of the time, the structures have been exposed to the destructive effects of the nature and the man. Some has been able to survive somehow and others were totally ruined. Most of the remained structures are in vulnerable condition to upcoming effects and for the continuity of their presence, structural strengthening applications are needed. A variety of applications are used with different levels of respect to original fabric and different extents of intervention within the principles of international charters that regulate the intervention on historical monuments.

In this study, a method of strengthening for the historical masonry constructions is developed in a general sense by the use of steel skeleton systems. In the proposed methodology, it is aimed to approach the intact structural conditions as much as possible in the strengthened structure. For the study a 3D model is created to compare the behaviors of the intact and the modified structure. In the modified model some structural elements are replaced by the steel skeleton system as a strengthening application. The behavioral investigation of the two models is performed in the finite element platform. Finally, it is certified that this methodology successfully efficient in approaching the original intact condition of the structure under concern as well as complying with the restoration principles.

Keywords: Historical constructions, Masonry, Strengthening of historical masonry, Steel skeleton, Structural analysis of monuments

ÖZ

TARİHİ YIĞMA YAPILARIN ÇELİK İSKELET SİSTEMLERİ İLE GÜÇLENDİRİLMESİ ÜZERİNE BİR İNCELEME

KÜÇÜKDOĞAN, Bilge
Yüksek Lisans, Mimarlık Bölümü
Tez Yöneticisi: Doç. Dr. Ali İhsan Ünay

Ağustos 2007, 114 sayfa

Tarihi yığma yapılar, yapıldıkları dönemin sosyal, arkeolojik, estetik, ekonomik, politik, mimari ve teknik özelliklerini yansıtan önemli kültür varlıklarıdır. Zaman içerisinde bu yapılar doğal ve insan kaynaklı çeşitli afetlere maruz kalmış, bir kısmı bir şekilde ayakta kalabilmişken büyük çoğunluğu ise tamamen yıkılmıştır. Ayakta kalan yapıların dışarıdan gelebilecek bir etkiye karşı oldukça savunmasız ve güçsüz durumda oldmaları nedeniyle ayakata kalabilmeleri için bir takım güçlendirme işlemlerine gerek duyulmaktadır. Orjinal dokunun korunması ve müdahalenin kapsamı gibi iki ana unsur üzerinde tarihi yapılara uygulanacak müdahaleleri düzenleyen ve uluslararası hükümlere uygunluk gösteren çok geniş bir yelpaze de güçlendirme işlemleri yapılmaktadır.

Bu çalışmada, çelik iskelet sistemlerini kullanma esasına dayalı bir güçlendirme yöntemi geliştirilmiştir. Bu yöntemde, güçlendirilen yapının, orjinal durumundaki davranışına yakınlaştırılması amaçlanmıştır. Çalışma dahilinde davranış özelliklerini gözlemlemek üzere üç boyutlu bir yığma yapı model hazırlanmış ve ikinci bir model olarak bazı taşıyıcı elemanlar çelik iskelet sistemleri ile değiştirilmiştir. Davranışı belirlemek üzere yapılan analizler sonlu elemanlar platformunda gerçekleştirilmiştir. Sonuç olarak, bu yöntemin restorasyon prensiplerine uyarak yapının orjinal davranışına yakınlaşma da başarılı sonuçlar verdiği gözlemlenmiştir.

Anahtar Kelimeler: Tarihi Yapılar, Yığma Yapı Sistemleri, Tarihi Yığma Yapıların Güçlendirilmesi, Çelik İskelet Sistemleri, Tarihi Yapıların Analizi

To,

Necip HABLEMİTOĞLU and Alaettin KÜÇÜKDOĞAN

PREFACE

"Türk Genci**,** devrimlerin ve cumhuriyetin sahibi ve bekçisidir. Bunların gereğine, doğruluğuna herkesten çok inanmıştır. Yönetim biçimini ve devrimleri benimsemiştir. Bunları güçsüz düşürecek en küçük ya da en büyük bir kıpırtı ve bir davranış duydu mu, "Bu ülkenin polisi vardır, jandarması vardır, ordusu vardır, adalet örgütü vardır" demeyecektir. Elle, taşla, sopa ve silahla; nesi varsa onunla kendi yapıtını koruyacaktır.Polis gelecek, asıl suçluları bırakıp, suçlu diye onu yakalayacaktır. Genç, "Polis henüz devrim ve cumhuriyetin polisi değildir" diye düşünecek, ama hiç bir zaman yalvarmayacaktır. Mahkeme onu yargılayacaktır. Yine düşünecek, "demek adalet örgütünü de düzeltmek, yönetim biçimine göre düzenlemek gerek" Onu hapse atacaklar. Yasal yollarla karşı çıkışlarda bulunmakla birlikte bana, başbakana ve meclise telgraflar yağdırıp, haksız ve suçsuz olduğu için salıverilmesine çalışılmasını, kayrılmasını istemeyecek. Diyecek ki, "ben inanç ve kanaatimin gereğini yaptım. Araya girişimde ve eylemimde haklıyım. Eğer buraya haksız olarak gelmişsem, bu haksızlığı ortaya koyan neden ve etkenleri düzeltmek de benim görevimdir. İşte benim anladığım Türk Genci ve Türk Gençliği!"

Mustafa Keman ATATÜRK, 5 Şubat 1933

This is the preface not only for an MS study but also it is a preface for my officially not long ago started journey to the thousand years of history which is in the form of architectural beings. I believe that with this study my somehow suppressed interest to architectural history and archaeology and my desire to engineering have melted in the same pot for the very first time. The outcome was a resonance from which I have realized that I am finally on the right track.

The study definitely would not have been possible without the support, encouragement supports, patience, love, appreciation and friendship of the precious people around me from whom I learned the beauty and the importance of the sharing.

First a very special thank you to my supervisor Assoc.Prof .Dr. Ali İhsan Ünay for his invaluable guidance, never-ending patience and support, his great contributions, his relieving sessions at the times I felt overwhelmed and lost, kindness and his smiles that I will always remember with a gratitude . I strongly believe that you, my professor, are the main reason which made this study enjoyable, pleasing and satisfying not the subject. In every session that we met to

work together, there were absolutely no stress, no anxiety, no counting of the minutes and no anger but a calming music, smiling faces, creative solutions and memories of the good old travels. Thank you..

I would like to say few words to try to thank Engin and Erhan Karaesmen whom I am indebted and speechless of telling my gratitude. I was not expecting anything unusual when I took the course named "CE 480-Design and Construction of Special Structures" in undergraduate and I was not aware that this selection was the milestone that changed the course of my future. It was in that course that the very first spark took place in my mind on this subject. But more than that my hope for life flourished for the first time after tough depressing months following my family tragedy when I had the chance to know these two precious people more closely. It is hard to tell that how I relieved by talking and even seeing these people. They gave me the courage and support I needed to go on. It was not only courage and support that they gave generously but also their knowledge and experience on life, on technical subjects and on arts, politics and on anything that you might imagine broadening my mind in even small talk on the phone. They became a sincere father and a humorous and considerate sister with whom I could share and talk anything. I really do not know how to tell my gratitude and indebtness to this family. A talkative person like me is speechless right now. Yes, speechless for the first time. Thank you for letting me in your world of joy, intellectuality and love "Engin and Erhan Hocam".

I offer my sincere thanks to my beloved family; my mother Emel Küçükdoğan, my grandfather Adem Hablemitoğlu, my sister İdil Belgin Küçükdoğan and my dear aunt Efser Koçakoğlu for their unique love and support throughout my life and their endless faith in me. I am really sorry that I could not spend much time with you for the last few years because of the theses I have but your existence have always given me the support to overcome the difficulties in life. My dear father Alaettin Küçükdoğan and my dear uncle Necip Hablemitoğlu to whom I dedicated this study, you were a bit in hurry to leave this world. But in any case what I

learned from you and the things we shared still enlighten my life. Your determination in pursuing your belief and your ideas will be an endless courage for me to follow the path I want without any hesitation and any slightest fear.

Sincere thanks to all my friends whom I found beside me whenever I needed and felt blue for their refreshing energy, their encouraging and calming words and their love. Many thanks to:

Chorists of J.S.Bach Choir, Gökçen, Derya, Öncü, Esra, Güçlü, Moldiyar, Janar, Hamdi, Can, Bülent and many others for their absolutely limitless joy and energy, for the times we sang in the streets, in the cafes or anywhere on earth during days and nights; Duygu Küçük, for her true and pure friendship, her intellectual capability to discuss any single object with a brand new point of view, her musical taste and her breathtaking singing; Özgür Şanlı for being the only one that shares and understands the my events of mischance, carelessness and weirdness without any questioning and her amazement towards life

The members of the Arinna Sailing Team, Tayfun, Serhan, Koray, Oğuz and Semih for their willingness to share the mystery of the deep blue with me, the moments we spent on the boat with the biggest swordfish of the Mediterranean. A special thanks to Pelin and Utku, it was a fun and a fantastic experience to be with you in the same boat talking, discussing and laughing for hours; Utku Kartal, for his sincere friendship and sense of humor, many other things of the last 5 years ; Semih Demirer, to whom I can never tell enough how I am grateful to him, for being always with me starting from the onset of this study with his valuable advices, courage, patience and emotional support, his endless faith in me and all the beauties he brings to my life and all the hope that lighten my world.

My colleagues: Özge Göbelez, Sami Demiroluk and Zerrin Ardıç Eminağa for their courage, support in every single step of this study, the more than hundred lunches that we had together with pleasure of talking, their patience in listening to

me and their sincere friendship: Güzide Atasoy, with for her positive energy and smiles, her invaluable advices and her encouraging words; Hazım Yılmaz, with whom we have a sinusoidal relationship, for his advices, his fun, dancing sessions in the office, tough discussions and laughs we had ; Tuba Eroğlu for the last minute emergency help in technical matters ; Dilşad Koyuncu and Özgecan Canarslan for their encouraging friendship and support in my studies at the Department of Architecture.

I am thankful to Glenn Gould, Dinu Lipatti, William Kempff, Pablo Casals, Emma Kirby, Kathleen Ferrier, Cecilia Bartoli and Rene Jacobs for their accompany during the tough days of this study with their calming interpretations and to Zorba for reshaping the concept of freedom for me in the planes of Crete.

Finally, and most importantly, I am profoundly grateful to Mustafa Kemal Atatürk. I owe him my being able to write this MS dissertation as a woman while other women in other parts of the world still fight for their gender rights.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Architectural Heritage and Conservation in General

Sheltering has been one of the basic instincts of the man starting from its existence. The search for protection has blossomed in a way that the concepts of creativity, functionality, uniqueness, variety, eternality and identity have melted in tangible architectural beings. From the very first settlement areas to today's modern high technology structures, each has its own story and conveys invaluable information from its time. The tracks of the development of civilization have been reflected to architectural preferences in structures which are, in a way, witnesses of the history.

The phrase "architectural heritage" defines the structures which reveal cultural, social, archaeological, aesthetic, economical, political, architectural and technical features of their time. The concepts of identity, continuity, spiritual and symbolic attributes have been assigned to these monuments as the self definition of societies or even nations. Building the present on the concrete foundations of the past has become of primary importance for the countries of the developing world.

The past in the form of architectural beings has been partially or entirely worn out on the course of time. The natural and man-made factors fasten the process of deterioration threatening the existence of the structure. Several conventions have been issued especially after 1950s to protect and to slow down the destruction of monuments with heritage value. The conventions commonly underline the importance of the continuity of the presence of the monuments not only for the country they belong but also for the collective history of the humanity.

Conservation and restoration of the historical monuments necessitate a multidisciplinary approach enveloping a wide range of professions as engineering, architecture, history, archaeology and chemistry as fundamental sciences. The coordination and communication among the variety of the professionals and organizations are of primary importance. A systematic investigation of the monument of concern, called condition assessment study, is applied before any intervention on historical monuments. The study includes the historical, architectural and structural survey of the monument with field research and laboratory testing, diagnosis and safety evaluation steps. Based on the outcome of the assessment study, necessary strengthening and restoration proposals are prepared complying with the international charters. The degree of intervention and the preservation of the authenticity are two aspects that should be considered before any intervention on historical monuments.

1.2 Argument

Historic buildings especially masonry ones that survived from the destructive effects of natural/man-made disasters still remain in partially standing with perfect load transfer mechanisms. However, their stability at present does not guarantee their survival from any upcoming expected / unexpected event. These buildings should be strengthened with specific methods in which the material used is compatible with the original fabric in terms of behavior and the method is compliant with the Conservation Charters underlying the importance of respect to authentic fabric while being differentiated by the original material. Within this framework, the strengthening of the partially collapsed buildings by using steel skeleton can be proposed as a method complying with the above mentioned criteria of the strengthening applications.

1.3 Objectives

The study aims to propose a structural strengthening method for partially collapsed historical masonry structures with steel skeleton system in which steel skeleton functions as a load carrying member as well as completing the physical appearance. One of the main objectives of this study is to evaluate the effectiveness and validity of the method in reflecting the original behavior of the intact model.

The study points out the characteristics of masonry, structural analysis methods and the important evaluation steps before any intervention to historical masonry structures as well as proposing the method. The intention of the study is not to define the method comprehensively and in a detailed way considering the evaluation steps but rather to give a general perspective and outline for the use of the method. The study aims to contribute to the general understanding and perception of the inner structure of the historical masonry buildings and to provide helpful and useful basis for the restoration and retrofitting of those structures.

1.4 Methodology

In this study for the evaluation of the effectiveness and the validity of strengthening application with steel skeleton, the structural behaviors are aimed to be observed. Two different models are considered for the evaluation.

The first model, a conceptual model, is developed inspired by the existing historical masonry domed structures. From this model with the removal of some structural elements, a partially collapsed model is obtained. This partially collapsed model is strengthened by the steel skeleton. The strengthened model is taken as the second model. Then, the two models, intact model and strengthened model, respectively, are analyzed under vertical (gravity) and lateral (response spectrum) loading cases. The behaviors are compared based on the deformed shape, modal time period, and stress and displacement variations.

1.5 Disposition

This study consists of seven chapters. The introduction gives the general information about the architectural heritage concept and the importance of the conservation applications. The objectives of the study as well as the brief information about the methodology are provided.

Chapter 2 focuses on the general characteristics and the damage agents of the masonry. Types, material characteristics and mechanical properties of stone and brick masonry are summarized as well as failure patterns. Damage types and the factors that alter the physical and chemical properties of the masonry are discussed in this chapter.

Chapter 3 is a review of the structural analysis principles and methods of analysis in general. Structural loads and the behavior of basic structural elements are explained to provide a basis for the analysis in succeeding chapters. The features of the analytical modeling are covered within the scope of the chapter.

Strengthening of the masonry is discussed in Chapter 4 from a wide range of aspects. Structural intervention types, international restoration principles for historical constructions and methodologies used during the strengthening applications are briefly considered. The importance of the condition assessment and its steps and techniques of structural interventions are elucidated.

A proposed method for the strengthening of historical masonry structures is introduced in Chapter 5. The conceptual background of the proposed strengthening method by steel skeleton is provided.

Chapter 6 has the finite element analyses of the 3D models on which the method explained in Chapter 5 is applied. Finite element analyses are conducted for both the intact model and the strengthened model in which some structural elements are replaced by steel skeleton. Analysis results are provided in graphical and tabular forms to question the validity of the proposed method.

Final chapter consists of the summary of the study, the conclusions drawn from the analysis and recommendations for future studies.

CHAPTER 2

MASONRY IN GENERAL: CHARACTERISTICS, AGENTS OF DAMAGE AND DECAY

2.1 Masonry in Historical Perspective

The history of civilization has witnessed a continuous progress in construction activities of the man starting from prehistoric cave dwelling times. The very first activities were the protection of entrance with some rubbles, bush, pieces of stones, the earth etc and the enlargement of the interior by the use of hard stones and bones. Sedentariness accelerated the construction activities of sheltering and created the fertile base for the development and the utilization of new materials. The primitive and the easiest technique was to lay up the pieces of stones or the earth in an order and create an enclosed space by covering the top with branches and similar organic materials, which can be considered as very early masonry. At that point, stone and the earth were the primary materials of construction [1].

The first artificial construction material, the brick, has a long past more than 8000 years on the lands of Mesopotamia where the sources of stones were scarce. It passed through an evaluation process in which emerged as moulded mud and then strengthened by the use of clay as a material. In time, during handling of clay the significance of temperature was realized and by firing the clay got considerable strength. Around 3000 B.C, big cities, towers and complicated infrastructure systems were achieved from the different types of brick by Sumerians [2]. The remained brick architectural heritage is limited in number and mostly date backs medieval times. However, brick masonry is still a commonly encountered construction material in traditional construction practice of rural areas.

Masonry, especially stone masonry, was the predominant construction material until the emergence of steel and concrete. In other words, the use of stone in masonry is old as the very first construction activities. The inherit properties of stone and the advancement on the working of stone after the introduction of the metals has lead wide use of the stone in antiquity. As stated by Sevgili et al [1], a symbolic meaning was attributed to stone masonry and many prestigious monuments and structures were built by stone to convey the idea of prosperity and the eternality as in the case of Memphis and Ghiza pyramids in Egypt. In later periods, large prehistoric monuments in Mexico, the great walls of China, Roman, Byzantium structures, and fortresses are among the examples of masonry in monumental scale [1]. It must be underlined that old and historic in structural sense is described by the symbolic concept corresponding to the word "stone". Therefore, in this study main emphasize will be given to the stone masonry and its structural features in the succeeding chapters.

2.2 Types and Material Properties of Masonry

Masonry is a heterogeneous material which is composed of masonry units and binding mortar. Its characteristics are dependent on that of units and the unit/ mortar interface. Since masonry covers a wide range of materials from mud to natural building stone, material properties and behavior greatly changes among different masonries which, in turn, makes it impossible to reach a generalization on the features of masonry.

Durability, workability, absorption capability, hardness, sensitivity to temperature, color, texture, porosity are among the significant material properties that are inherit in each material and altered later by physical and chemical conditions. Durability is the ability of material to resist external effects such as environmental and chemical effects. Water absorption capacity and compressive strength are two parameters for the evaluation of durability [3]. Hardness and workability of the material are the two conversely related properties. In other words, the higher the hardness, the less workable is the material [4].

The types of masonry and their brief definitions are proposed by Croci [5] as follows:

- Brickwork – Regular alternating layers of brick and mortar
- Stonework – Natural stone elements (various kinds and shapes) held together by mortar
- Sack Masonry– At external faces, two thin walls of stone or brick and internal fill of loose or cemented aggregates
- Mixed Masonry – Alternating layers of brick and stone
- Dry stone Masonry – Perfectly cut stone without mortar
- Adobe – Sun dried mud

Two main masonry types namely brick and stone masonry and the binding material, mortar, will be covered in a detailed way since they are the basic constituents of masonry construction.

2.2.1 Brick Masonry

Brick has been one of the widely used materials in the construction activities of the man starting from the very first settlements in Neolithic period. The use of mud to cover the branches and between the stone pieces then turned into a construction material which was molded and roughly shaped and dried under the sun. The hardened material is called sun dried brick or adobe which is still widely used in rural areas where the clay deposits are available. Later, the molded earth was burnt to accelerate the drying process which, in fact, was a brilliant attempt since the strength of the clay mud was positively correlated with the temperature at which it was treated. Moreover, the burning process picked up the pace of the production rate of the brick which, in turn, triggered the construction activities [2], [4], [6]. Brick is generally used in the construction of walls, in curvilinear elements such as vaults, domes and arches. Its lightness solves the weight problem in the covering of an enclosure.

The fundamental constituent of the brick, clay, has some certain characteristics as being easily formed into any shape when mixed with water, retaining its shape when dried and hardening when burned. Clays are mainly composed of hydrated silicates of alumina and small amounts of other minerals called impurities as iron oxide, calcium oxide, magnesia, potassa, soda and sand. Clays used for brick production are classified under two groups [7]:

- Surface clays – found in the deposits at the site of the rock from which they are formed (Residual clays), deposited by sedimentation after the transportation by water (Sedimentary clays) or formed by the pressurization of sedimentary clays (Shales)

- Fire clays – Found at deeper levels. High resistance to temperature and uniform in character

The strength of brick masonry has a wide range of values due to fact that it depends on the composition, drying process of brick, the baking temperature, quality of the brick, the kind of mortar used and the pattern of laying of brick. [3], [5], [8]. The porosity and absorption are also significant factors affecting the strength features of the brick as well as its durability.

2.2.2 Stone Masonry

Stone has always been a part of architectural journey of the human being although its use is considerably declined with the introduction of concrete and steel after the industrial revolution. The meaning attributed to stone as well as its material characteristics make it an indispensable material for centuries [1].

Stone, a concretion of mineral matter, is the primary building material taken from the crust. Geological factors have been important criteria for the use of stone since the extraction, transportation, working of the stone and the construction technique is dependent on those. Stones are divided into three groups according to their geological origin as igneous, sedimentary and metamorphic rocks [7], [9].

Igneous rocks result from the crystallization of magma or the accumulation and consolidation of volcanic ejecta. As magma cools, minerals crystallize and the resulting rock is characterized by interlocking mineral gains. Magma that cools slowly beneath the surface produces intrusive igneous rocks which are fine grained and relatively hard. On the other hand, magma that cools at the surface produces extrusive igneous rocks the grains of which are visible without magnification. Igneous rocks are relatively hard when compared with other types of stones. Granite, basalt, gabbro, obsidian and andasite can be listed as examples of this type [10], [11].

Sedimentary rocks originate by consolidation of rock fragments, precipitation of mineral matter from solution, or compaction of plant or animal remains. In other words, they are composed of materials derived by mechanical and chemical weathering that disintegrate and decomposed preexisting rocks. The properties of the rocks depend on the weathering agents and vary within a wide range of hardness from very soft to medium. Sandstone, conglomerate, limestone, limestone and siltstone are commonly observed examples in historical constructions [7], [12].

Metamorphic rocks result from the alteration of other rocks, usually beneath the surface, by heat, pressure and the chemical activity of the fluids. Marble is a metamorphic rock produced when the agents of metamorphism are applied to sedimentary rocks limestone and dolostone. Other than marble, quartize, schist and slate are among this group of rocks found in historical constructions [7], [13].

2.2.3 Mortar

Mortar has been used as a binding material between the masonry units for more than 7000 years. Although its composition greatly varies between different periods and different regions, basically it is composed of a binder, a mixture of inorganic compounds (aggregates) and water. The proportions of ingredients are the key factor in the strength of the mortar [14].

The types of mortar used in construction activities are listed by Croci [5] along below lines:

- Plaster mortars – contain no aggregates and harden quickly in water without shrinking
- Aerial lime mortars – composed of slaked lime, sand and water and harden with the presence of CO_2 (carbon dioxide) in the air.
- Hydraulic lime mortars – composed of specially heat treated aerial lime, sand and water and harden under water in the absence of air
- Pozzolonic mortars – composed of slaked lime, pozzolona and water. Widely used by Romans
- Cement mortars – composed of cement which is hydraulic binder produced by burning of calcareous and clayey materials with pulverized gypsum in high temperature and water. It is the most resistant and recent type of mortar.
- Bastard mortars – obtained my mixing lime and cement

The binding role of mortar makes it considerably significant especially for the shear and flexural strengths of masonry. The bond strength is affected from the content of cementitious material, water content and surface texture of the masonry [3].

2.3 Mechanical Properties of Masonry and Failure Patterns

The heterogeneous character of masonry makes it difficult to precisely determine the strength and stiffness characteristics of a masonry structure which, in turn, hinders the assessment of the behavior. The inherit properties of individual elements namely masonry unit and mortar have a significant share in the overall strength beside the state of the interface between the materials. However, the bonded behavior differs from the individual behaviors of elements.

Masonry in historical constructions generally work under compression or at least designed to work under compression therefore the compressive strength features of elements are important inputs for the assessment studies. Figure 2.1 [3] demonstrates the stress strain relationships of masonry unit, mortar and masonry prism. As seen, the compressive strength of masonry prism is smaller than that of masonry unit. On the other hand, mortar has the lowest strength value from which it can be concluded that the increase in mortar strength is reflected directly to the strength of the prism [3]

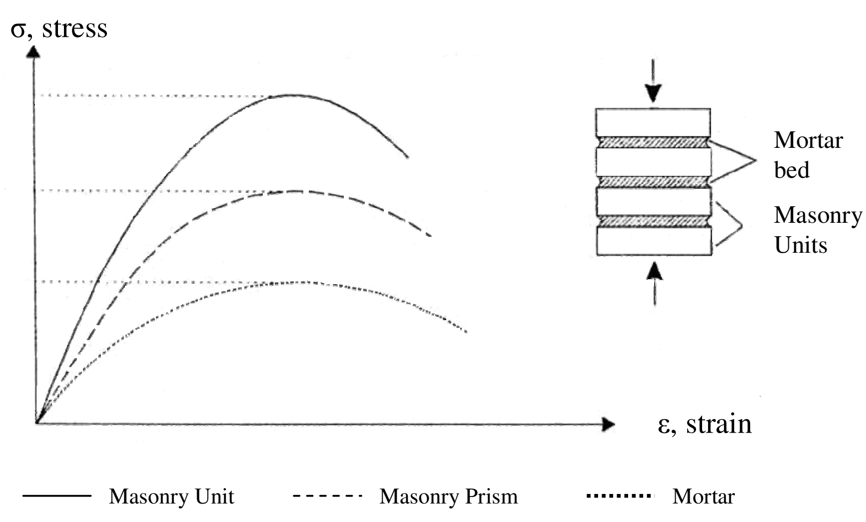


Figure 2.1 Stress- strain relationship of single elements and prism of masonry.

As for the brick masonry, the compressive strength value for a good quality clay brick varies between 10 MPa to 30 MPa. On the other hand, that of stone varies significantly among the different types within the range of 5 MPa to 70 MPa in which igneous rocks have the highest values. Furthermore, the thickness of mortar is considerably influential in the compressive strength of the masonry [3], [5], [7].

The most distinctive characteristic of the masonry is its low tensile strength which, in fact, is one of the main damage sources. The tensile strength of masonry depends on the bond between masonry unit and mortar since the tensile strength of the masonry elements is higher than that of the bond. A good bond is achieved when the balance between retentivity of the mortar and the suction of the masonry units. The resistance of mortar is in a way dictates the failure pattern among the masonry. In the case of brick, tensile strength is approximately 5 to 8 % of the compressive strength, i.e., 0.5 MPa - 2.5 MPa. As for natural building stones, depending on the type tensile strength varies between 2 MPa to 45 MPa. However, in analysis of masonry structures the tensile strength contribution is totally neglected and the structure is assumed to be unable to withstand tension [3], [7], [9], [15]. Flexural tensile resistance of masonry is the indicator of the load bearing capacity of the structure in which it is used as beam and is referred much more than the direct tensile strength [4].

The shear resistance is provided by the bond between mortar and masonry units in usual masonry and for the dry masonry by the friction between the units. The movements along the parallel plane to the joints result in shear stresses which triggers the initiation and propagation of cracks in joints. Shear strength values changes within the range of 2- 45 MPa for natural building stones and 10- 20 MPa for bricks [3], [16].

Modulus of elasticity is the value that indicates the nonlinear relationship between the stress and strain. It is determined by the slope of approximately linear part of stress strain curve. A typical stress strain curve can be seen in Figure 2.1. The value has a range for stone between 10000 MPa and 70000 MPa and for bricks 5000 MPa to 10000 MPa [3], [15].

The complexity and uncertainty of the behavior of the unit and mortar interface, the geometry of the structure, strength characteristics of the material and load propagation through the structure are the key factors in the failure mechanism investigations. Failure type is important in the sense that the accurate assessment of the vulnerability depends on the probable patterns [3].

Mortar joints are the planes of weakness since the homogeneity is not present in the joints and as a result failure may occur only in joints or as a combined mechanism including both the unit and the mortar. By the consideration of the abovementioned key factors critical failure mechanism is determined. Critical failure mechanism is defined as the lowest bound after which the failure initiates. Failure can be classified according to the type of the forces that cause the problem as follows: Failure under tension forces, failure under shear forces and failure under compression forces [17].

Failure caused by tensile forces has mainly initiated along with the bond between units and mortar. Fracture patterns change with the bedding plane which defines the angle between the vertical axis and the direction of the laying of the masonry units. A masonry unit has two joints, bed and head joints respectively that, indeed, behave differently under tension. Bed joint is the horizontal layer on which a masonry unit is laid while head joint is the vertical mortar between two adjacent masonry units. The strength of the head joint and bed joint as well as the masonry unit are superimposed when the direction of stress is parallel to bed joints. On the other hand, when the direction of stress is parallel to head joints, the resistance is dependent on the strength of the bond unless the masonry unit has higher tensile strength than the applied stress. Figure 2.2 [2, 17] shows the fracture patterns in uniaxial tension at different angles (0º, 45 º and 90 º) [2], [3], [17], [18].

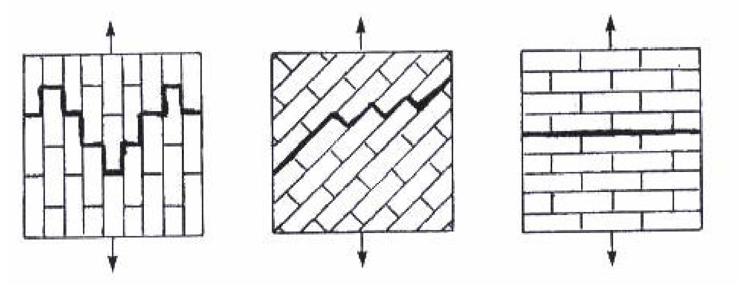


Figure 2.2 Sketches showing the fracture patterns in uniaxial tension stressed at 0º, 45 º and 90 º angles to the bedding plane**.**

The distinct variance in strain characteristics of mortar and masonry units is the main reason of the compression failure. Moreover, other mechanical properties such as shear and tensile strength, coefficient of friction of the bond, modulus of elasticity and Poisson’s ratio are related to this kind of failure. The development of cracks parallel to load direction is generally observed in horizontally laid bedding plane. In Figure 2.3, fracture patterns occurred under uniaxial compression is presented [2], [17].

Compression as explained above can be uniaxial as well as biaxial. In the case of biaxial compression, the plane of compression determines the direction of masonry cracks. In the biaxial case, which is the combination of tensile and compression forces, the cracks are parallel to the compression plane while perpendicular to that of tension [17].

Figure 2.4 [17] shows the two different combinations of biaxial stress situation. Shear in masonry result in two fundamental form of failure in masonry depending on the resistance of mortar, diagonal shear and sliding shear failure respectively. Diagonal failure occurs when the combination of principle tensile stresses and compressive stresses is present at the same time on the masonry. On the other hand, sliding (joint) shear is the relative movement of masonry units in a parallel direction to the mortar joint. Both failures are in a way prevented by the use of mortar with sufficient cohesive characteristics. The sketches of the failures are exhibited in Figure 2.5 [3], [18], [19].

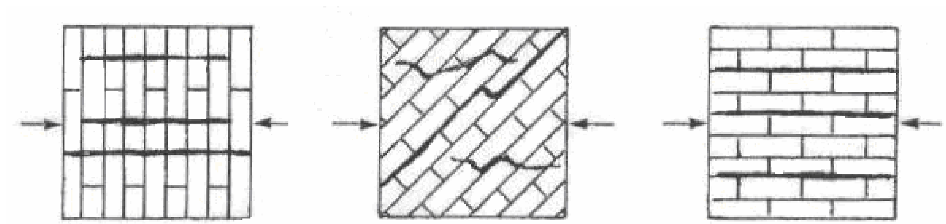
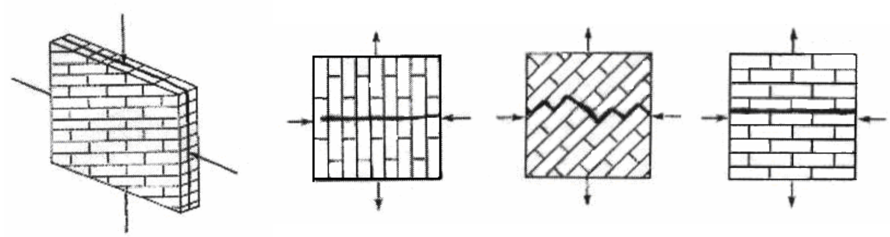
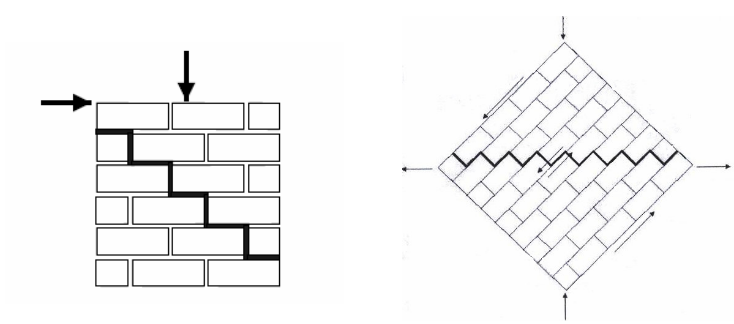


Figure 2.3 Sketches showing the fracture patterns in uniaxial compression stressed at 0º, 45 º and 90 º angles to the bedding plane.



(a) (b)

Figure 2.4 Sketches showing the fracture patterns in biaxial compression (a) and biaxial tension and compression stressed at 0º, 45 º and 90 º angles to the bedding plane.



(a) (b)

Figure 2.5 Sketches showing the fracture patterns in shear failure ; (a) sliding (joint failure) (b) diagonal failure

2.4 Damage and Decay in Masonry Structures

Historical masonry structures have been exposed to the destructive effects of nature and man within the course of time. These effects have caused a variety of damages from tiny cracks to total disintegration. Damage and decay, in general comprehension, may originate from the inherit characteristics of buildings such as imperfections in design, natural phenomenon as disasters, environmental factors or a combination of both. Regardless of the source, it is a fact that damage and decay alter the structural behavior of structures [5].

2.4.1 Damage and Damage Agents

Damage is defined as the change in the structural behavior as a result of mechanical actions or the decrease in the structural efficiency. Mechanical actions causing damage can be classified under two categories as static and dynamic actions. Static actions are those that are always present on structures such as dead and live loads and those that are resulted from the deformations or strains imposed on the structures. On the other hand, dynamic actions are the ones involving movement produced by earthquakes, wind, vibrating machines, explosions etc. The decrease of the efficiency, however, is related with the strength- stress ratio of structures. External effects as well as aging, creep etc. cause a decrease in the bearing capacity of a structure, which, in turn, reduce the strength - stress ratio. As a result, structures become more susceptible to any potential effect [5], [20].

The level of damage depends on the type of the structure and the type and the intensity of the actions. However, the visible signs can be grouped under two classes: cracks and crushing [5].

Cracks are commonly encountered signs of damage in historical masonry with a variety of forms. Cracks in masonry generally occur during the adaptation of the structural system to newly altered situation. That is, with any change in load transfer mechanism, as a reaction cracks are induced. As previously stated, masonry is weak in tension and there observed cracks along the tension zones. The evaluation of crack patterns provides clues for the determination of the cause. Especially, long term investigation of the cracks is significant in the sense that the progressive alterations in cracks can be followed up and necessary measures be taken accordingly. Cracks exhibit different patterns in different structural elements depending on the dimensions and material characteristics of the element [5], [21], [22].

Crushing is a phenomenon that occurs when the compression stress on the masonry goes beyond the strength of the material. It starts with tiny cracks parallel to loading direction and after a certain time detachment of the flakes is observed. The final state, crumbling, suddenly occurs and may result in even collapse if other structural elements do not compensate the alteration in load transfer mechanism [5].

The widening of head joints, masonry slackening, split of blocks, detachment of walls in a corner and similar occurrences are damages observed in historical masonry.

There are some alterations which result in damage in masonry structures. First of all, indirect actions as soil settlement and earthquakes are major causes of damage in historical constructions. Soil settles most of the time nonlinearly and creates a significant change in pressure distribution beneath the foundations. As a result of this differential settlement, inward, outward and horizontal movement can occur above the foundation level.

On the other hand, the risk imposed by the seismic action is of primary importance for the integrity of historical constructions. It has been known from the history that many monuments and structures from the ancient past destroyed by the devastating effects of earthquakes. As Croci states, most damage and collapse are generated by the horizontal components of movements during the release of energy. Since historic buildings are not designed to resist lateral forces as the contemporary ones. Earthquake hits with hundreds of shocks and in which building becomes more and more disconnected resulting in decrease in overall stiffness. The length of earthquake and frequency content are important factors affecting the level of damage. Elimination of structural elements, modifications in supports, structural elements, additional loading imposed on the structure, excavations or any other activities affecting the foundations may be counted among the alterations damaging the masonry structures [1], [5], [23], [24].

Creep is another source of problem in any material which is defined by the time dependent deformation of the material under sustained loads. The age of masonry, the level of sustained loads, the humidity and temperature and the time are some of the factors that creep depends on. Creep by enforcing stress redistribution, may result in excessive deformation and as a result a total collapse may be observed as seen in sudden collapse of Civic Tower of Pavia, Italy in 1989. For the detailed information about the creep behavior, the references Anzani et al and Ferretti et al can be referred [25], [26], [27], [28].

2.4.2 Decay

By Croci, decay is defined as the detrimental change of a material's characteristics as a result of environmental conditions. Decay may result from physical and chemical factors as well as biological nature. The main causes of deterioration which involve the loss of substance can be listed as follows [5], [29]:

- Crystallization of salts
- Air
- Frost action
- Biodeterioration

Crystallization of salts is one of the most damaging decay agents in masonry. By the penetration of moisture inside the masonry through water vapour, rain leaking or ground water absorption, soluble salts present in construction materials are dispersed throughout the material. The fluctuations in moisture content by wind and temperature activate these salts. Under drying conditions, with the evaporation of the water, salts are deposited on the surface and within the pores of the masonry. The growth of salty crystals on the surface is called efflorescence. Sufficient cycles of wetting and drying triggers a process of disintegration near the surface [5], [29], [30].

The changes in the composition of the atmosphere with the advancement in the industrial activities as well as natural weathering agents have accelerated the deterioration process of stone in last millennium. Human activity in dense urban areas results in emissions of different pollutants of which sulphure compounds, ozone, nitrogen oxides and carbon dioxide are known to be responsible from the decay of stone. The rainwater is acidified by the pollutants in the air and this acidified rainwater constitutes the main source of damage on surface of masonry. The acid in water attack on metals and various minerals present in the composition of stones. However, chemical attack is not the only process occurred on the surface due to environmental conditions. Besides chemical attacks, physical and mechanical agents are always active without depending on the presence of acidic components. They lead mechanical break up which, as a result, increases the area of the mineral surface to be exposed to moisture and weathering agents. The two photographs of same sculpture taken 60 years apart below illustrates the dramatic effect of air pollution and weathering on stone surfaces (Figure 2.6), [15], [29], [31], [32], [33].

Water as a substance has the salient property of volumetric expansion upon freezing. This feature of water makes it highly destructive within pores and fissures of masonry in cold weather. The expansion of water in pores and fissures create internal tensions and hydraulic pressures within the stone. The pressures and tensions result in the formation of cracks to resist the stress imposed. Depending on the mechanical properties of masonry, the resistance and the rate of weakening process varies. It has been observed that for a stone to show visible signs of damage it must be exposed to several cycles of freezing and thawing. The severity of the frost damage is directly related with the pore size distribution, moisture content of masonry and intensity, rate and duration of freezing as well as mechanical properties of masonry [15], [29], [34], [35].

Biologic agents as certain plants, microorganisms, fungi, lichen and algae are known to be affecting stones in historical constructions in a way that they have contributed the decay processes of the material. Biodeterioration is considered to be the succeeding stage of the initial deterioration caused by inorganic agents. The increase in the content of pollutants in the atmosphere and deteriorated surface has lead to a condition suitable for the proliferation of microorganisms with the presence of water. These organisms may change the color of stone, alter the chemical and mechanical properties, and weaken the strength characteristics resulting in desquamation, chipping and exfoliation. Other than microorganisms, trees, climbers and creepers are also agents of damage and deterioration in materials of historical constructions. Figure 2.7 exhibits examples for deterioration caused by microorganisms and plants [36], [37], [38], [39].

Figure 2.6 A dramatic example of stone decay in 60 years of time in the industrial atmosphere of the Rhein-Ruhr Area.

Figure 2.7 Examples of biodeterioration on masonry. [36], [40]

CHAPTER 3

STRUCTURAL ANALYSIS OF MASONRY STRUCTURES

3.1 General

Historic buildings are considered to be the living witnesses of the civilization progress of the man and accepted as the primary elements reflecting the facts about art, architecture and engineering of their times. Their conservation and perpetuity are of primary importance which necessitates a special evaluation of their current physical, aesthetic and cultural states. In these kinds of evaluations, knowledge from a variety of fields as archeology, civil engineering, history of architecture, archaeometry, chemistry, urban planning, computer sciences, ethics, is needed to be integrated [41].

The underlying motivation for structural analysis is to observe a structure's capability to resist any action within the limits of stability, in other words, the comparison of the resistance of a structure with the effects of actions. Contemporary structures are designed to react within certain safety margins however; in the case of historical structures safety margins are not as clear as contemporary ones. Indeed, masonry heritage in general is massive in character and most of the time overdesigned. However, this massiveness and overdesign do not ensure their stability. Analysis methods do come into picture at this point to evaluate reaction to the action. Observation and experience of failures were the main methods for structural design and analysis until early 19th century. After then, new methods were developed for structural analysis as a result of advancement in mathematics and graphic statistics. The following century witnessed a considerable development in numerical methods and the introduction of computers and their use in calculations has increased the reliability and accuracy of results and brought a new dimension to the studies on historical masonry. Analysis processes has become easier and more accurate than ever before [43].

Structural analysis of historic built heritage differs from that of modern buildings and has some salient aspects that make it a complex task. These aspects are categorized as follows: missing geometric data, mechanical properties of materials, current damage state and inapplicability of regulations and codes [42].

- Geometric features of a building are essential for a complete and precise analysis. Current non intactness of historic buildings results in the use of assumptions in place of missing parts which in a way impede to understand the original behavior. Since masonry constructions are massive in nature the geometry of inner elements are almost impossible to be determined without harming the structure.

- Masonry in nature is a complex heterogeneous material and characterization of mechanical properties of materials is a tedious and complicated task. Although experimental studies on material properties provide valuable information of the properties, due to workmanship and use of natural materials there exist large variability in mechanical properties. Besides, long construction periods result in changes in the core and constitution of structural elements.

- Historic buildings are suffered from different kinds of damages during their life time that affect their load transfer mechanisms. Damage in a structure can be known to a certain extent however the level of knowledge might stay well behind in defining the actual state. Their inclusion in structural analysis introducing new loads, deformations complicates the model.

- Regulations and codes for the analysis of modern buildings are not applicable for historic buildings. Each historical structure has its own features of materials, damage state, geometry and structural elements necessitating a unique study that has to be planned with the consideration of historical, structural and architectural aspects.

Today, sophisticated simulations that take into account above complications are adopted in structural analysis with the expense of computational time. Analytical models in those simulations that exhibit the original state contribute to detect the sequences and agents of damage that structures suffered by comparing with the present condition besides their function as determining the vulnerable parts and estimating the limits [5].

3.2 Structural Loads and Load Combinations

Structural behavior studies evaluate the performance of structures and the level of performance is dependent on the ability of structural elements to safely carry and transfer loads. Therefore, expected loads and load combinations are key parameters at the onset of behavioral studies.

Structural loads can be classified in many different ways. However, classification according to source as natural and service loads and direction of application as vertical and horizontal loads is commonly used [43]. As the name implies source based classification takes into account the starting basis of loads. Gravity, earthquake, wind, snow, hydrostatic and soil pressure loads are among nature based loads. Of these, gravity and earthquake loads are the most essential ones. Gravity loads in other words, the self weight of structure involve structural, architectural, nonstructural partitions and covering components of a building. They are important in the sense that the structure should be able to carry its own weight to be called as stable and standing and any analysis on structures is obliged to start with the estimation of gravity loads. Gravity loads are permanent and static loads which are imposed on structures during their life time. Loads on structures due to earthquake constitute the significant threat to the historic buildings since the amount energy emitted from ground shaking has to be dissipated by building. Earthquake loads are dynamic loads and seriously affect overall stability of buildings. The amount of knowledge on structure itself and seismicity of the region where it is located is two important ingredients in the calculation of earthquakes loads therefore the exact values of loads are impossible to be determined. Service loads are the loads resulting from the usage of structure. Furniture, temporary structural components and humans produce service loads. As for the classification by the direction of application, it considers the way that loads are applied on structures. Gravity, snow, moving loads are defined as vertical loads while earthquake, wind, soil and water pressure are accepted as horizontal loads. Historic buildings generally are not designed to resist horizontal components of forces therefore destructive effect of structural loads imposing lateral actions is considerably high.

Besides aforementioned loads, loads result from indirect actions as soil settlements, creep, thermal effects, shrinkage of materials and etc. have to be considered in structural analysis of historic constructions [5]. Differential support settlements due to soil deformation are one of the major causes of damage to the buildings and their inclusion in modeling and analysis stages increases the accuracy and reliability of results.

Load combination is the superposition of loads acting on a structure. In modern design codes, different factors are assigned for different loads to simulate probable conditions by load combinations. However, in the case of historic buildings the recommended factors in the codes are not directly applicable due to their material, constructional and existing damage features. Therefore, historic buildings have to be evaluated within their own framework and load combinations have to be assigned by considering the past of building with identified current decay and damage state and by past experiences combined with engineering judgment. It should be underlined that load combinations involving earthquake and support settlement loads constitute the most critical situation since both loads alter the original load transfer mechanisms resulting in excessive stress concentrations in some parts leading to failure [5], [43].

3.3 Basic Structural Elements and Their Features

A clear understanding of some basic structural elements and their way of reaction to actions is essential in investigation of general behavior. Proper functioning of each element in a way ensures the stability of the structure. Herein some fundamental elements used in historical masonry constructions are listed and their specific features are provided.

- Beams and columns

  One of the elementary forms of building is achieved by post and lintel, columns and beams systems until Roman times by stone masonry. Extraction of long units was a great challenge for that time therefore beams spanning between columns were short when compared with contemporary buildings. As previously mentioned, main insufficiency of masonry is its low tensile strength capacity. In Figure 3.1, a schematic representation of a beam is provided with gray lines showing the exaggerated deformed state. In a simple supported beam bending action is observed creating tension zones at bottom and compression zones at the top. Cracks due to weakness in tension are commonly encountered in flat lintels. Shear may also constitute a source of failure in beams constructed by soft stones [44].

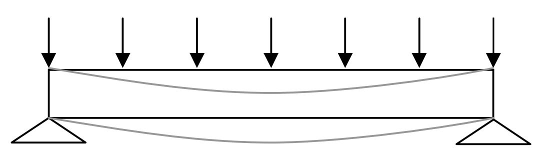
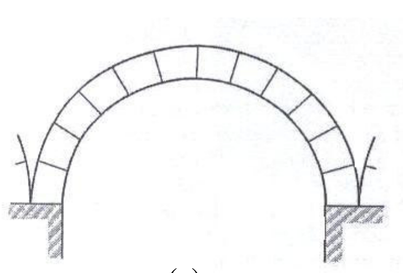


Figure 3.1 A modal beam with deformed shape under loading.

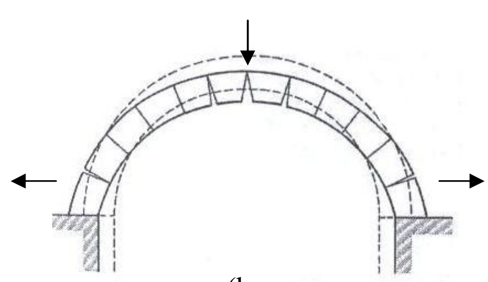
As for columns, they are vertical supporting elements which transfer loads to the foundations. Columns work in compression therefore they are able to carry vertical loads up to the limit strength of masonry used. However, horizontal loads that produce shear stresses jeopardize these elements. This threat differs among masonry types –presented in chapter 2- from slight displacement between blocks to complete disintegration.

- Arches

The form of arches has lots of advantages as a load carrying and transferring element and has been widely used as a perfect solution of architectural and engineering necessities staring from Roman times. The way the form acts amplifies the benefit gained from the use of masonry since loads on arches compress the blocks forming the arch and solidify the form. The curvature of arch affects its reaction to loading and pointed and parabolic arches have known to be stronger as a result of less trust on abutments [5]. Lateral thrust exerted on abutments in arches is seen in Figure 3.2 [44] and resistance of abutments is of paramount importance for the stability of the arch. Depth of voussoirs is another point to be underlined in the concerns of stability. As stated by Feilden [44], the deeper the voussoirs, the stronger the arch. Cracks in arch form are precursor of abutment problems,-spreading, settlement etc. - while those in voussoirs indicate thermal movements or excessive loadings.



(a)



(b

Figure 3.2 General arch form : (a) Intact arch ; (b) arch with cracks due to lateral thrust.

- Vaults

  Vault form is created by the use of series of arches to cover a space. There are different types of vaults used in gothic and medieval Islamic architecture. Abutments are also important in the case of vaults as in arches. There have been introduced various solutions to increase the resistance of abutments in vaulted structures. Thick walls constructed for continuous vaults and buttresses for intersecting vaults are commonly encountered structural features supporting the form. Ribs in certain vault types act as separating members for the webs of vaults and outline the shape as well as carrying loads [5], [44].

- Domes

  Domes are spatial form of arch formed by rotating it around a vertical axis. It is known that the achievement of stable masonry dome had not been easy mainly due to self weight. As seen in Figure 3.3 [45], weight and other vertical loads on domes are spread uniformly on its curved edge supports and push the supports vertically and horizontally outward. This thrust action results in tension in the perimeter, which in turn generates circumferential cracks and at the limit state leads to collapse of dome. There used some supporting elements as buttresses, pendantives and cupolas to stabilize the dome against thrust action therefore domes should be investigated as a whole with their supporting elements [45].

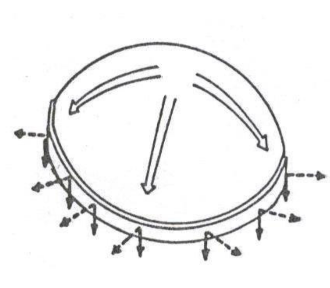


Figure 3.3 Single dome and thrust actions on its planarly curved supports.

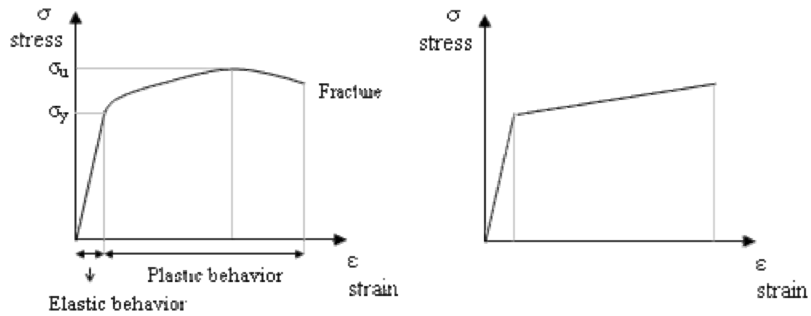
- Walls

  Wall characteristics change from geographic region and period in history and it is not possible to provide a generalization of features for masonry walls [44]. Existence of binding material, type of masonry and material used, pattern of bonding are among the sources of differences. In many of historic constructions, walls have load carrying function and depending on the loads imposed various types of behavior are observed. Investigation of cracks and failures in walls reveals actual sources of disturbance which should be considered with materialistic characteristics.

There exist some other structural elements like piers, trusses, frames and foundations however in here only the ones that are common in almost all of stone masonry are taken into account. In the case of foundations, generalization in behavior is almost impossible since each building has specific system of foundations.

3.4 Methods of Analysis According to Behavior of Masonry Structures

In structural analysis, regardless of construction material used, behavior of structures is idealized to simulate the actual performance with different assumptions on stress-strain relationships of the material. Stress – strain relationship is the characterization of the behavior of material and as seen in Figure 3.4 (a) has some certain points important in making the assumptions. Yielding point in a stress strain curve represents the limit stress level before which the material is capable to return its original shape upon the removal of loading. After that point material do not return its original undeformed shape, i.e. there observed some permanent deformations. Behavior after yielding point is called plastic behavior in general. Ultimate point shows the maximum stress that the material can withstand before fracture. For simplicity purposes, stress – strain curve is used in idealized linear form (Figure 3.4 (b)).



(a) (b)

Figure 3.4 Stress-strain curve : (a) a general curve; (b) a linearized curve.

Based on above mentioned stress- strain diagram, three common idealizations are used for analysis as elastic behavior, plastic behavior and nonlinear behavior. Figure 3.5 [47] illustrates the applications of these idealizations.

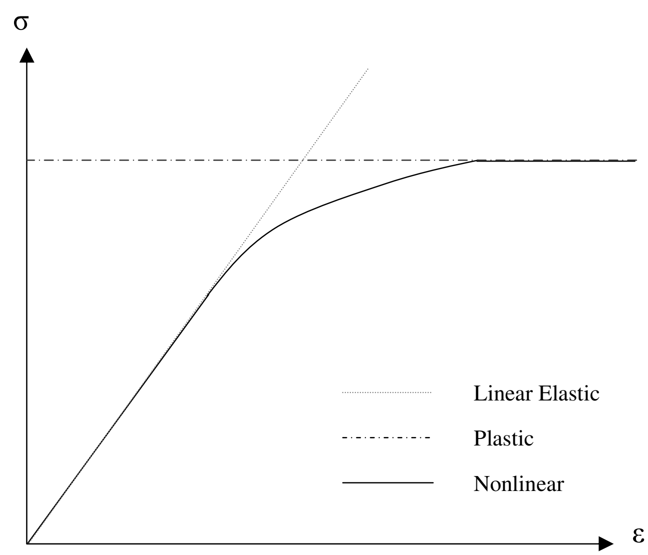


Figure 3. 5 Three different types of behaviors

3.4.1 Linear Elastic Behavior

Linear elastic model considers the behavior of the material within the recoverable limits of deformation. The model is not as complex as the other two models and acceptable for the masonry provided that loading is short term and tensile cracking does not exist. Its easiness in computation and validity in wide range of materials make the linear elasticity most common model in structural engineering. The fundamental assumptions of the model are explained by Macleod [46] as follows:

1. As stress increases the resulting strain increases in a linear proportion.
2. As stress decreases the resulting strain decreases in the same linear proportion.
3. Strain perpendicular to an applied strain is linearly proportional to the applied strain (Poisson's ratio effect)
4. The material is homogeneous and continuous.

Although the linear elastic analysis method is practical to apply, beyond the beginning of cracking it is not able to represent the behavior of historical masonry construction. Furthermore, the assumption of homogeneity and continuity of material seems not a correct characterization of masonry which is a composite material. However, salient approaches of homogenization aim to combine the different actions of units and mortar and reach a representative values for the overall behavior of masonry [42], [47], [48].

3.4.2 Plastic Behavior (Limit Behavior)

Plastic models are based on the evaluation of the structural loads, stress distributions and collapse mechanisms at failure. The model is used for the verification of ultimate limit states and its effectiveness and accuracy is directly related with the tensile stresses present. There are two bounds for plastic analysis as lower bound (static) and upper bound (kinematic) [42]. Identification of tension zones and their contribution to bearing capacity are significant steps in the limit analysis and the behavior of compression zones whether it remains in elastic range or plastic range changes the methods to be used. The static approach distributes the stresses which are statically admissible and by considering the equilibrium conditions a lower bound for the limit load is determined [5]. An example of static approach is thrust line analysis in which plastic hinges is developed when the line of thrust passes outside the entire cross-section leading failure [3]. As for kinematic approach, as stated by Croci [5], the structure is transformed into a "mechanism" by creating a certain number of plastic zones at which stresses are thought to reach the limit values.

These limit values represents the upper bound for that mechanism. The mechanism approach assumes masonry with no tensile strength and infinite compressive strength, perfect fit of the blocks, negligible strains and no sliding between adjacent masonry units [49]. The assumptions bring about valid results for dry masonry blocks, masonry with deteriorated mortar and materials with finite tensile capacity since the mortar contributes to tensile strength and ignorance of strength would lead erroneous results [4].

3.4.3 Non-linear Behavior

Non-linear behavior aims at evaluating the complete response of a structure from the elastic range, cracking and crushing, up to the failure and therefore non-linear analysis is accepted as the most powerful analysis method. Non-linear analysis necessitates the comprehensive mechanical characterization of the materials and a keen detailing in modeling for the best representation of the reality. Nonlinearity in the model can be in physical properties of the material, point of application of the loads or contact of bodies [42].

3.5 Analytical Modeling in General

Analytical modeling is the definition of the structure by adequate number, size and type of materials considering the geometrical features, joint restraints, connection states and load conditions applied on structure [43]. It is an important stage in the evaluation of structures since the behavior of a structural component or the whole structure is obtained through analysis of this model. Prior to computer-aided methods, analysis carried out by hand calculations and models had to be simple enough to get through many equilibrium equations with many unknowns. Computer based environment accelerates the processing of equations and enables the model be more complicated and detailed with the expense of computational time which is still shorter than the time necessary for traditional methods [46].

Structural system of historical constructions is complex in nature and a special care should be given in the modeling stage. In modeling, structural components are divided into discrete elements with idealizations in geometry. The structure under concern can be made of linear elements, two dimensional elements or three dimensional elements. The nature of the problem dictates the type of elements to be used. Reflectivity of the model to the reality depends on some fundamental assumptions on material characteristics, geometric features and the number and type of elements. Definition of joint restraints and connection of elements should be taken into consideration.

Discontinuous nature of masonry necessitates different modeling strategies for the appropriate constitutive description of its anisotropic behavior. Computational modeling frameworks are categorized by Lourenço [50] referring to the study of Rots as follows (Figure 3.6, [50]):

- **Detailed micro-modeling** – Continuum elements define units and mortar in the joints and discontinuous elements represent the interface of units and mortar.

- **Simplified micro-modeling**- Continuum elements represent expanded units while discontinuous elements cover the behavior of mortar joints and unit-mortar interface.
- **Macro-modeling** – units, mortar and the interface defined by a fictitious homogeneous continuum.

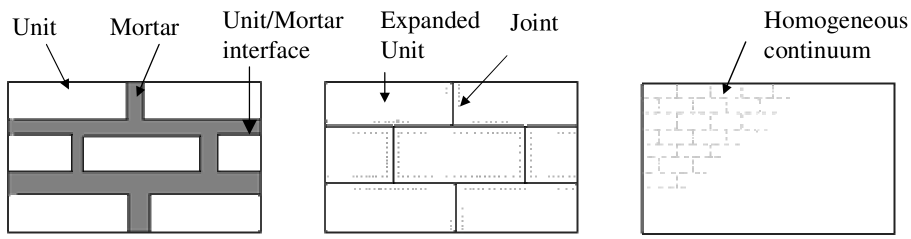


Figure 3. 6 Modeling of masonry: (a) detailed micro-modeling; (b) simplified micro-modeling; (c) macro-modeling

The choice of modeling strategy is based on the problem requirements. In other words, application fields of micro and macro modeling are different. Micro-modeling is preferred when the local behavior of masonry structures are sought. On the other hand, macro modeling gives better results for the global behavior of an entire structure. For low stress levels homogeneous model is acceptable for the prediction of deformations. However, for high stress levels, nonlinear behavior of mortar and its local failures should be included within the model [3], [50].

The principles of analytical modeling can be summarized as follows [43], [46]:

- Model should be simple as well as satisfactory in defining the structure. Complex models, unless necessary, do not ensure the reliability of approximations of the actual state of structure.
- The sizing of elements should be determined with the consideration of its structural affects.

- Detailed behavior of a specific part should not be extracted from the model of the complete structure. A refined model with corresponding boundary and connection conditions should be created.

3.6 Structural Analysis of Masonry Constructions with Finite Element Method

The Finite Element Method is a promising numerical method for the analysis of continua and structures with capability of generating and solving several algebraic equations simultaneously on computer based environment. The method aims to calculate the stresses and deflections in a structure by dividing it in a finite number of elements and calculating each element and their interactions entirely to approximate the general behavior [51].

Finite element method was first proposed in 1940s by Hrennikoff and Courant separately concentrating on different aspects of it with a common thought of discretizing the continuum to a set of sub-domains. In the method, a model is created by small elements which have simple geometry and assemblage of the elements with proper joints.. Elements-lines, 2D or 3D elements are separated from each other by fictitious cuts represented by lines. Triangle as finite element type was the first type used in this method by Cough in 1960's since construction of any shape with triangles was convenient. Triangular element approach was based on the stiffness method which required the direct solution of equations. On the other hand, a method called "dynamic relaxation" by Otter in 1965 based on mesh system formulated the equations of motion that lead to a static solution. A combination of two methods is used as finite element method today [4].

Finite Element Method has some certain steps in which a clear sequence of the system is observed. The very first step is to idealize the design structure so that it is detailed enough to represent the actual structure and at the same time not complicated. The idealized structure and successively created meshed model are highly crucial in the accuracy and efficiency of the analysis. In the design material properties-failure domains etc, definition of elastic limits and boundary conditions should also be stored. Then, with the inclusion of the definition of number of increments and intensity of loading a solution model is prepared. The next step is the numerical solution of the model to determine the displacements and stresses. The results of analysis are processed by the computer and state of stress and deformation is provided for each element by evaluating it within failure domain. The results are presented in tabular and graphical form with the characteristics of model. Finally the results are interpreted with the consideration of mathematical model and the relation between nodes, elements, loads and restraints. However, taking into account the hundreds of outputs and data, using graphical form of outputs make it easy to interpret the behavior of the structure under concern. The interpretation of deformations, especially the deformed shape provides significant information about the behavior [52], [53].

Structural analysis of masonry constructions with finite element methods has gained attention within the engineering, architecture and restoration circles during the last decades with the increase in the concern of architectural heritage. The method enables the analysis of complex structures as historical constructions with arbitrary shapes, load and support conditions. Variance in material characteristics and geometric features of a structure can easily be reflected in the mesh generation stage which in turn increases the reflectivity level of the reality of the model [51].

CHAPTER 4

STRENGTHENING OF HISTORICAL MASONRY IN GENERAL

4.1 Vulnerability of Masonry Structures

Vulnerability is defined as the susceptibility to physical attack or damage [54]. It has become a key concern in the assessment studies of existing building stock and architectural heritage and aims to define the actual state of weaknesses at present. The case of historical heritage has a special place due to their significance as surviving representatives of the past [55].

Historical masonry structures have been subjected to external actions of natural and man-made disasters throughout their life time which has lead to structural weakness, deformations, failures and collapses [4]. Damage agents as lateral loads due to earthquakes, strong wind effects, support failures due to soil structure interaction, excessive loading and concentrated loads as a result of any change in load transfer mechanisms have affected the initial strength and safety level of the structures. Furthermore, climatic and environmental effects has resulted in the decay of the material used and altered the physical and chemical properties. These damages and decays in the structure make the structure vulnerable to any further external and internal action which accelerates the deterioration process and threatens the stability. Besides natural occurrences in structures and its surroundings, man made factors are among the reasons of susceptibility of architectural heritage. Overconcentration in population with complex infrastructure and systems, economic activities and destruction as a result of public unconsciousness have contributed in an unpleasant way [56].

Although masonry is a durable and strong material, its strength mostly comes from its ability to resist compression. Any action imposing tension in the structure generally results in damage. To increase the resistance to lateral actions is one of the primary concerns to eliminate the vulnerability in this respect. Seismic vulnerability, within this framework, is a significant step in the evaluation of historical buildings since earthquakes are the most destructive occurrences not only causing cracks and partial failures but also threatening the integrity [57].

Before any intervention to improve the current state of historical buildings, a complete investigation to determine the sources of vulnerability should be carried out by considering the stability, capacity, safety in short/long term, climatic and environmental conditions as well as inherit properties of construction material.

4.2 Structural Intervention in General

The concerns towards the protection of architectural heritage has accepted to be started during 19th century with Viollet-le-Duc and hastened in 20th century. Historical structures that had been used like public assembly buildings had undergone certain repair works and more or less stayed standing. However, the ones that remained from ancient past and abandoned in the course of time were destructed by natural and man-made actions. Many of structures in latter definition are now in vulnerable condition and their integrity and stability are under peril even due to self weight.

Structural interventions on architectural heritage aim to decrease the vulnerability against certain actions and to sustain their existence for the future generations. The establishment of independent international highly specialized legal organizations initiated by UNESCO as ICOMOS (International Council on Monuments and Sites), ICCROM (International Centre for the Study of the Preservation and Restoration of Cultural Property), ICOM (International Council on Museums), IUCN (World Conservation Union) and etc has outlined a good framework for the study, conservation and presentation of cultural property. There have been proposed some criteria by the organizations for selection of degree of intervention which has been issued as international charters [58].

4.2.1 Principals of Intervention

The Athens Charter adopted at the First International Congress of Architects and Technicians of Historic Monuments in 1931 is accepted as the first attempt to regulate the restoration applications which later constituted the base for Venice Charter. International cooperation for the welfare of the monuments, the use of modern techniques and materials in restoration works, preservation of sites and structures by legislative measures were underlying points of the congress [59].

International Charter for the Conservation and Restoration of Monuments and Sites issued after the congress in Venice in 1964 first of which was held in 1931 in Athens, i.e. The Venice Charter is important in the sense that it defines the fundamental principles for the conservation of the cultural heritage. There are 14 articles in the charter to define monuments, their values and conservation ethics to set the rules for restoration and excavation activities. Key points of articles related to structural intervention are listed below [55], [60].

- The aesthetic and historic value of the monument must be preserved and revealed by respecting for the original material and authentic documents

- Modern techniques can be used provided that the inadequacy of traditional techniques is proved and the efficacy of modern techniques is verified by scientific data;

- Unity of style is not the aim of the restoration therefore the valid contributions of all periods must be respected;

- Replacement of missing parts must integrate harmoniously with the whole, but at the same time must be distinguishable from the original;

- Additions cannot be allowed except in so far as they do not detract from the interesting parts of the building, its traditional setting, the balance of its composition and its relation with its surroundings.

There exist several other charters on specific cases or particular parts of restoration applications as underwater heritage, timber structures, and wall paintings etc. or recommendations on analysis, guidelines on education and so on. However, the Venice Charter conveys the general attitude towards the conservation and restoration of cultural heritage.

4.2.2 Levels of Intervention

The purpose of intervention is to increase the life time of structure by considering the aesthetic and authentic nature. Therefore, a detailed investigation and decision process is needed to intervene historical structures as minimum as possible. The interventions vary according to the nature and value of the structure or site.

As seen in Figure 4.1 [61], heritage conservation activities have some certain levels. The increase in the extent of intervention decreases respect for historic fabric. Preservation, stabilization, consolidation, restoration and rehabilitation are considered as the interventions with maximum respect for historic character. On the other hand, renovation and modernization activities are the least respectful ones in terms of intactness of historic fabric. Reassembly, replication, reconstruction, moving and fragmentation have categorized as the interventions with moderate respect to historic texture and moderate level of intervention [61]. Although it is not possible to describe all interventions in detail, a brief summary of each will provide a general understanding about the intervention activities [44],[61].

- Preservation – Keeping a structure or a site in its existing state. It composes of repairs and stabilizations to prevent further decay, deterioration and damage.

- Stabilization – It is to safeguard a structure with a minimum amount of work involving temporary emergency reinforcing, protective coverings.

- Consolidation – To ensure the durability and integrity of cultural property it is the physical addition or application of adhesive or supportive materials into actual fabric. It is undertaken when the structural elements lose their strength and the integrity of the structure is under danger.

- Restoration – It aims to revive the original concept and appearance of a structure by removing later material and by replacing missing elements and details.

- Rehabilitation – It is defined as the keeping structure in use. It is possible to use it for contemporary needs while preserving its features. The original use is the best way of preserving the structure's historic, architectural and cultural values. On the other hand, adaptive re-use rehabilitation may need some modifications to serve for new usage purpose of the structure.

- Reassembly – It is called anastylosis and preferred when justified by archeological evidence and when it makes a ruin more comprehensible. Reassembly may also be undertaken by the structural necessity like repairing a deteriorated part or observing historic construction techniques.

- Replication / reproduction –It is the making of the exact copy of a structure or an artifact in order to replace some missing or decayed parts to maintain the unity. If a cultural property is threatened by its environment, it is moved to a safer place and a reproduction is substituted to maintain the unity of site or building.

- Reconstruction – It is the reproduction of a building, site feature or artifact that no longer exists. The authenticity of the reconstruction depends on sufficient documentary information.

- Moving – It is the relocation of a structure or part of site to another site. It is preferred when no other measure to sustain the continuity of structure or site is left. There is high potential of damage to historic fabric during moving.

- Fragmentation – It is the retaining of the portions of a structure and reassembling on the original site or elsewhere.

- Renovation – It is the process of extensive changes and/or additions made to an existing building internally and externally to renew. The conservation of historic fabric is not the first priority in this intervention.

- Modernization – It is the conscious attempt to hide or alter the old appearance of heritage features to achieve a modernized appearance.

The determination of the level of intervention is the most critical stage of the heritage conservation activities. The process should be based on historical research, site and analytical analysis and documentation to identify and safeguard the architectural heritage.

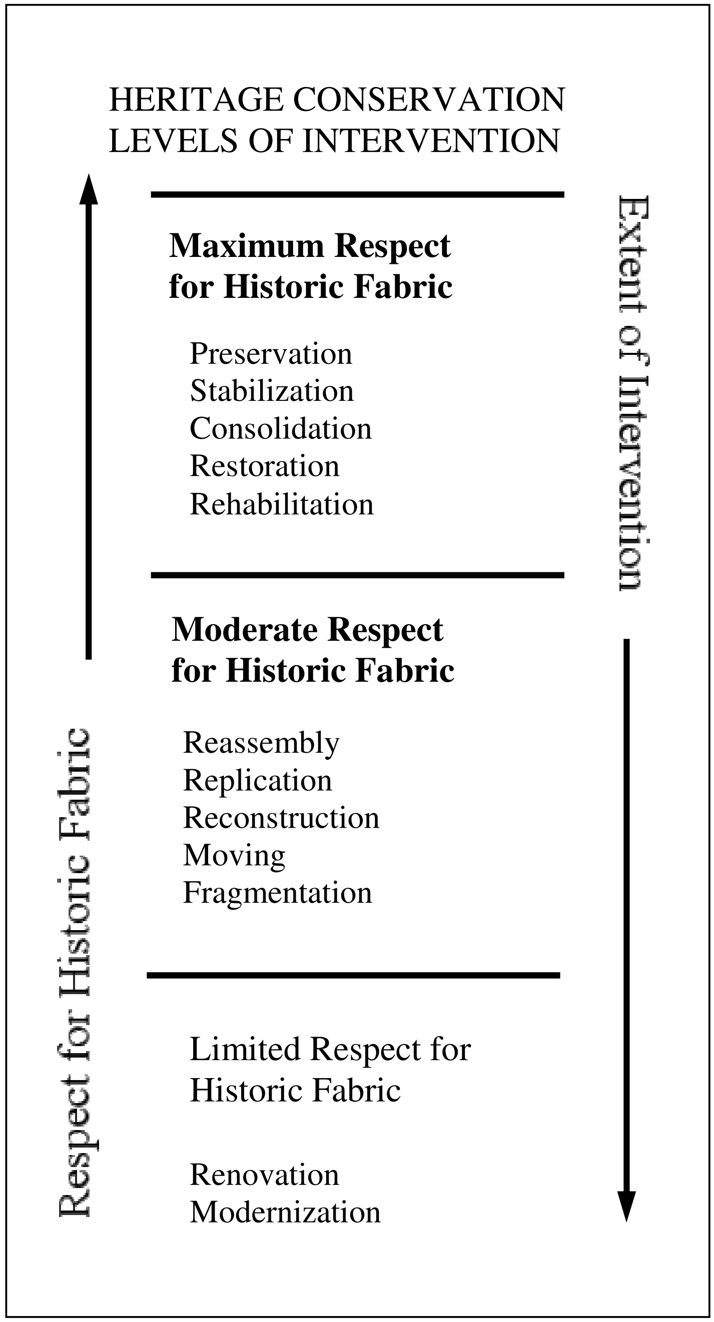


Figure 4. 1 Levels of Intervention and their sequence according to the extent of intervention and respect for historic fabric.

4.3 Condition Assessment Methodology

Condition assessment is the task of evaluating the current condition of the component based on observed or reported characteristics [62]. Condition assessment has started to be widely used in recent decades in the field of earthquake engineering to strengthen building stocks against destructive effects of earthquakes. Unlike contemporary ordinary structures, historical constructions need a special treatment in the assessment process since any intervention on this kind of constructions bases on not only the results of the present state evaluation – scientific insight- but also based on the historical and cultural context of the structure. The inherit complexity of material characteristics, structural features and uncertain past histories of changes and damages makes the assessment of the heritage a demanding task.

The investigation of historic structures goes beyond simple technical considerations and requires multidisciplinary approach. The assessment of historical constructions has some certain steps which are recommended by ICOMOS and many experts in the field of conservation of cultural heritage. The steps are listed below and then explained in an elaborate way in next sections [1], [41], [63], [65]:

- Acquisition of data: Historical, structural and architectural investigations and information
- Survey of the structure
- Field research and laboratory testing
- Diagnosis
- Safety evaluation

4.3.1 Acquisition of Data: Historical, Structural and Architectural Investigations

A historic structure is a representative of an earlier tradition that no longer exists and definition and understanding of its cultural and historic significance provide a fundamental starting point and orient further activities.

Historical survey aims to clarify the techniques, skills used in its construction and to determine changes in the structure and its environment. It covers the reading and the interpretation of historical documents, writings, drawings and photographs related to the structure of concern. The history of events of the structure enables to correlate the causes and effects in a way that can contribute to diagnosis stage [63]. Any recorded earthquakes, failures, reconstructions, additions and structural modifications can be invaluable sources of information for the evaluation of the structure provided that their reliability is assured.

4.3.2 Survey of the Structure

The survey of structure involves the direct observation of the structure of concern and creating its detailed drawings if not available. This process necessitates a qualified team to reveal the whole geometry and its dimensioning and to identify the damage and ongoing environmental effects on the structure.

Mapping of visible damage gives an idea of the possible structure behavior and helps to identify the critic parts that need detailed examination. The determination of zones of crashing and cracking/ the separation of elements can be useful in the detection of the changes in load transfer mechanism. This survey also observes the material characterization and pattern of change in materials throughout the building. The improvement of the knowledge of the survey is then achieved by in-situ and laboratory tests [41], [63].

4.3.3 Field Research and Laboratory Testing

This step involves the test, measurement and identification of materials and deformations both in the field and laboratory. As mentioned previously, the characteristics of the used material are not the same throughout a structure therefore a generalization of the characteristics of masonry is not effective in explaining the reality. Therefore, to identify the mechanical, physical and chemical properties of masonry some tests are conducted grouped under two major headings as laboratory and in-situ tests and under two subheadings as destructive and nondestructive.

Mechanical tests determine the behavior of materials as resistance, deformability etc and are generally conducted in laboratories. For the tests to be conducted samples are needed with varying diameters and if they are block in various sizes according to the type of the masonry to be tested. The fundamental tests applied in this category are the axial compression test to measure the resistance and obtain the stress-strain curve, triaxial compression test to measure the effect of lateral expansion and indirect tensile strength test to measure shear and tensile strength Mechanical tests can be on blocks, bricks, mortar as well as actual portions of the wall. However, transportation of large elements may not be feasible and carrying out the tests directly in situ is preferred in the testing of masonry walls [5], [63].

Physio-chemical tests aim to get physical parameters as specific weight, porosity and pore size distribution, freeze-thaw resistance, water absorption and salt crystallization, chemical characteristics as presence of water soluble salts, sulphates, chlorides, petrographic composition type, the type of binder, the type of aggregates and their relative proportions. Mercury porosimetry for porosity and pore size distribution, electron microscopy for the evaluation of decay, X-ray diffraction for the detection of the presence of soluble salts, optical and petrologic analysis for the classification of bricks in terms of origin, burning temperature etc, and for the classification of stones and mortars, resistance test for the particle abrasion are among the tests applied on samples taken from the structure of concern and conducted in laboratories. There are several in situ tests some of which are destructive and some are nondestructive. Endoscopic examination, for instance, necessitates the drilling of small diameter holes and uses optic fibres to reveal the morphological variations in masonry, internal cavities and cracks etc. On the other hand, sonic and ultrasound tests are effective in the determination of decayed layers and nondestructive to historic fabric applicable at the site. There exist several other tests enhanced by the developments in technology as Infra Red Thermography, Fibre optics microscopy, Digital Image Processing or advanced ultrasound measurements etc. The studies between [65] and [73] can be referred for detailed information about tests and their procedures.

Monitoring, which consists of a network of sensors positioned to measure, periodically or continually cracks, deformations, stresses etc connected to a feed unit and recording unit, is the another field application that is widely used in the evaluation of historical constructions. The advantages of the use of monitoring is underline by Croci [5] as follows: the recording of active of active phenomena like settlement etc by easing the determination of trend, speed and risk situations, control of the structural improvements during repairs to strengthen or restore, in order to check its conformity to the design and control of structural behavior, where there is movement in nearby zones (for instance, excavations, drilling etc.) It provides an accurate knowledge of the behavior of the structure. There are two types of monitors used in the investigation of historical constructions: Static and dynamic. Static monitor records the evaluation of deformation in structural elements, forces and rigid movements as rotations, settlements etc with high frequency of acquisition (period 1s to 1-2 days). On the other hand, one of the dynamic monitors, passive monitor, records the displacements by stand-by sensors which are activated when a seismic action occurs or when wind or traffic vibration exceed the trigger level (period milliseconds). Active monitor measures the modes of vibration with changing levels of frequency [63], [74].

4.3.4 Diagnosis

Diagnosis stage consists of both qualitative and quantitative investigations and aims to define the causes and their probable results at critical circumstances. Therefore, its accuracy affects the decision at the following safety and risk evaluation stage. Qualitative investigations cover all the related documents obtained in data acquisition and information attained through the survey of the structure. Field research and laboratory testing as well as experimental and mathematical modeling are involved in quantitative investigations [41].

For the diagnosis stage to be accurate, the uncertainty in material, geometry, geotechnical data should be minimized, the failures, crack patterns and their causes should be defined, the analytical model of structure (explained in Chapter 3) should reflect the reality as much as possible based on above mentioned knowledge with reliable assumptions and different combinations of external effects should be taken into consideration. Experimental results are used for the validation and calibration of the mathematical model. The results of structural analysis should be investigated thoroughly to identify the behavior and features of structural deficiencies.

4.3.5 Safety Evaluation

Safety evaluation is defined by Croci [5] as: *"It is the subsequent judgment on the capacity of the structure to resist specific actions such as loads, earthquakes, etc. and the potential risked involved."*

It is important in the sense that based on the judgment are decided the need and the extent of remedial measures. Poor judgment may result in conservative which means over intervention or inadequate which results in unsafe solution for the conservation of the structure.

Safety evaluation is a difficult task that requires the combination of solid scientific background, intuition, experience and intense observation. Methods of analysis for new constructions are not reliable to be safely used for historical structures due to their complexity, uncertainties in material characteristics, lack of knowledge of past occurrences and alterations. Therefore, assessing the safety level should envelope the preceding steps and their results with bearing in mind the principles of Venice Charter [5], [41]. On the other hand, the precise conduction of the above steps does not totally clear out the uncertainty and subjectivity involved in the overall assessment of historical constructions. Experience and expertise in the evaluation have significant contribution.

The lifetime of the decided intervention that assumes certain safety level is another aspect that should be considered in the evaluation process. Since a safe situation in short term may not necessarily be safe in the long term. An optimum decision should involve the probable future situations which consequently affect the extent of intervention.

4.4 Techniques of Intervention

Intervention activities on the present state of monuments which are based on the above mentioned methodology can basically be classified under two groups as traditional techniques and modern techniques. The use of traditional techniques and old materials is respectful to the historical value of monuments in the way that the integrity and overall sense of completeness are achieved. The replacement of deteriorated material with the material of same composition is preferably used when the material is available and traditional techniques satisfy the envisaged safety requirements for the structure of concern. However, most of the time safety measures required by strongly influential factors such as earthquakes, soil settlements can not be obtained by the use of old materials and old techniques and since the safety and integrity of historical constructions are of primary concern, modern techniques are preferred to get the required safety level. At this point, an important term comes into picture, “reversibility”, which in a way tries to balance the safety level and the authenticity of the structure. Reversibility in restoration defines the capability of an intervention to turn back to original untouched state. Decisions during safety evaluation process may turn out to be erroneous or inefficient to solve the problem and it is highly important that the applied action can be replaced without damaging the original fabric. The development of better techniques and materials is another factor that encourages the replacement of previous actions. Therefore, reversibility should be considered in the restoration applications with bearing in mind that it is a preference rather than strict requirement [5], [55], [74].

Techniques of intervention are classified by Penelis as reversible and irreversible. The reversible techniques are listed as follows [74]:

- Restoration of stone or marble monuments with dry joints
- External ties
- Prestressed unbonded stitches
- External buttresses
- Ties at springing of arches
- Internal steel curbs for confinement
- Rings at the perimeter of the domes
- Improvement of the strength, stiffness and ductility of existing diaphragms etc.

The techniques below can be classified as irreversible [74]:

- Grouting
- Deep rejoints
- Stitching of walls with Prestressed rebars
- Interconnections of marble or stone parts with bonded dowels
- Rebuilding of part of the facings of walls where these have fallen bodily

- Reinforcement of masonry with incorporated steel bars
- Strengthening applications on foundations
- Underpinning
- Reinforcement with fibre-reinforced plastics (FRP) etc.

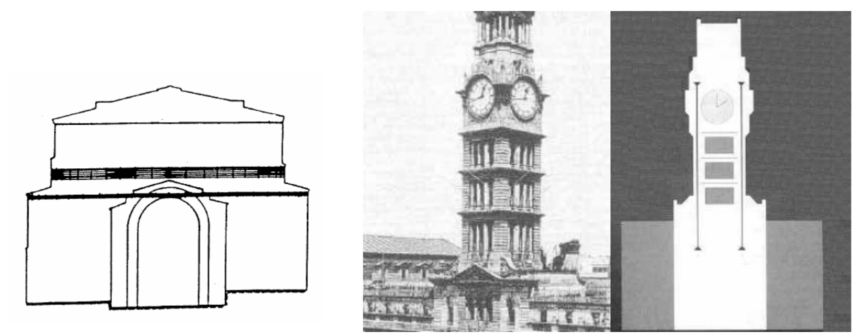
Unlike the reversible techniques, irreversible techniques impose restrictions on the structures and can affect the original fabric. In the decision process of irreversible applications two essential requirements should be taken into account: durability and compatibility. By the term *durability* the lifetime criterion is referred underlining that lifetime of new application to be at least equal to that of original material. *Compatibility* is defined as the congruousness of the physical, chemical, mechanical and mineralogical properties of new materials with the original fabric. Strength and stiffness characteristics, bonding, coefficient of thermal expansion, permeability and reaction to the efflorescence are among the basic indices of compatibility [55], [74].

Each of above listed techniques has certain situation of application and necessitates a clear understanding of the state of the monument. Among the techniques prestressing and post strengthening by fibre- reinforced plastics have gained a particular interest recently and widely used as modern efficient techniques for the conservation of the historical constructions.

4.4.1 Prestressing Masonry

Prestressing is a simple and effective way for overcoming the low tensile strength of masonry. The weakness in tension resistance constitutes a significant source of damage in historical masonry monuments and has a damage range from tiny cracks to complete disintegration of units. Therefore, any attempt to increase the tensile resistance helps to consolidate the stability and integrity of monuments in masonry.

Prestressing has two different applications: pre-tensioning and post-tensioning. Pre-tensioning starts with the installation of tendons. Then, masonry is built around tensioned tendons and after the curing masonry tendons are released. By this way, the tension in tendons is transferred to the masonry as compression. The application is quite different in post-tensioning. Tendons are tightened after the completion of the masonry. Although pre-tensioning is relatively easy to apply in new constructions, post-tensioning is commonly preferred in the strengthening of historical constructions. It gives the flexibility of introducing desired level of load externally on a wall, column, curvilinear elements etc. (Figure 4.2 [55], [75]). The studies conducted by the VSL Strengthening solutions research group [75] reveal that post tensioning exhibits an effective solution in walls subjected to axial, out of plane lateral and in plane shear loading. There observed increase in the threshold value of cracking, in the flexural and shear resistance and the improvement in the cracking behavior [55], [75], [76], [77].



(a) (b)

Figure 4. 2 Examples of prestressing: (a) Prestressed rings at the dome of Rotunda, Thessaloniki; and (b) Vertical prestressing of GPO Tower, Sydney.

Prestressing by post tensioning can be applied vertically and horizontally. Horizontal post tensioning can externally be performed on the structure by anchoring the tendons. It should be underlined that horizontal post tensioning relatively more respectful to the historical fabric than vertical prestressing in which bottom anchorage is needed [4].

One aspect which should be taken into account in prestressing masonry constructions is the loss of effectiveness of forces in tendons in long term. As a result of variety of factors, the stress in the tendons decreases in time therefore a periodic investigation and maintenance can be needed for the effective functioning of the application.

4.4.2 Strengthening of Masonry with Fiber Reinforced Polymers

With the advancement in material technology a variety of new applications has been introduced for the restoration and strengthening of the historical masonry structures. The use of fibre reinforced polymers has stood out among other applications in the family of polymers which are defined as the substances composed of molecules with large molecular mass composed of repeating structural units and seen as a promising use with its basic features. The advantages of FRP can be listed as follows [78], [79]:

- High strength and stiffness
- Lightness
- Insensitivity to corrosion
- Reduced installation and maintenance costs
- Reversibility
- Color options for aesthetic appearance
- Different forms of laminates, tendons and fabrics with practically unlimited length

FRP has higher strength and stiffness when compared with steel and has become an alternative for steel ties in post-tensioning. It gives the equivalent resistance provided by steel tendons in addition to its minimization of the impact on appearance of the strengthened member. Its light weight does not alter the dynamic properties of the structure on which FRP is applied therefore it does not complicate the structural features. Corrosion constitutes a significant problem in metals and may result in the loss of functioning in metal element and decay in the historical fabric. FRP's insensitivity to corrosion makes it promising and advantageous choice for the restoration activities. Furthermore, reversibility is ensured in many FRP applications since adhesive materials can be removed from the surface [78], [79].

There exist various FRP composites which are used extensively in various industries. Among these, carbon (CFRP), glass (GFRP) and aramid (AFRP) fibers are mainly used for strengthening of masonry. Except AFRP, CFRP and GFRP have certain ranges of strength: 2000-2500 MPa and 1100-1300 MPa. The tensile strength of AFRP has a wide range depending on the manufacturer. Depending on the constitution of the fiber matrix, three composites exhibits different characteristics in terms of mechanical, physical and chemical behavior. GFRP's are sensitive to ultra-violet light and highly affected from the alkalis and moisture. On the other hand, CFRP tendons have resistance to chemicals, moisture and ultraviolet light and a tendency to rupture under shear or lateral loading. As for AFRP, the composition of epoxy resin and fibres results in low modulus of elasticity with high strength and high resistance to the chemicals. Besides, AFRP is easy to shape and can be used at various levels of structure by placing sheets that wraps the entire exterior. Its flexibility of strength values make it preferable in the place of steel [78], [80].

For the strengthening the masonry, two alternative applications of the FRP composites are used: externally bonded laminates and Near Surface Mounted (NMS) [81]. Installation of laminates necessitates sandblasting and puttying of the surface. Then Laminates are obtained by impregnation of the fiber ply between two coats of saturant which enables the laminate become a part of the strengthened member. It has been efficiently used for flexural and shear strengthening of masonry members. It is possible to use prefabricated FRP by providing adhesion of the laminate to the substrate as well as hand lying up of laminates. In NMS method grooves are cut onto the surface of masonry that is being strengthened. FRP bars encapsulated into the grooves by a paste and once the curing time is passed, paste hardens and transfers the stresses from the substrate of the member to the bar. NSM does not require surface preparation and can easily be anchored adjacent strengthened members [81], [82]. Another feasible application of NSM technique is structural repointing, i.e., the placement of FRP bars in mortar bed joints. They act as a shear reinforcement to resist in-plane loads as well as increasing the flexural capacity of the masonry walls [82].

The following studies of Balsamo et al [83], Triantafillou [84], Shrive [85], Albert et al [86], Galati et al [87], Cruz et al[88], Valluzzi et al [89], Tanazzi et al [90], and Tumialan et al [91] may provide further information about FRP composites and the experiments conducted on different masonry members.

CHAPTER 5

STRENGTHENING OF HISTORICAL MASONRY STRUCTURES BY STEEL SKELETON

5.1 Use of Steel in Strengthening Applications

Steel is an alloy material which has been started to be used widely since the early 20^{th} century in the construction activities. Steel has some salient features as malleability, high strength, durability and ductility which increase its popularity among other materials. Malleability is defined as the ability of the steel to be rolled into any shape and form, for instance, thin sheets, wires, rod, bars or beams and etc. This feature gives the flexibility for any application involving the use of steel. The strength varies with the chemical composition of the steel. The amount of certain metals and elements affect the strength. By this way, it is possible to produce specific steel with predefined strength capacity. One of the highlighted features of the steel, durability is the resistance of the material to external effects. The durability of the steel is modified by the amount of carbon during the production and wide range of durability options are provided for the steel. Other than these four basic features, steel is lighter that any other material. These features make the steel as a promising construction material.

Beside its use as a construction material, steel is preferred in many of the strengthening applications for the historical masonry structures. As well as its above mentioned features, the easiness of reversibility of the applications is an underlying reason for the selection of steel. A variety of applications from simple connections to big scale interventions has been practiced with the use of steel in this field. Some of the applications are listed below in which generally prestressing technique discussed in Chapter 4 is applied:

- Ties at abutments of arches
- Rings at the perimeter of the domes
- Confinement of structural elements by steel ties
- Structural repointing by steel bars
- Steel stitching
- Miscellaneous other applications depending on the structure of concern

Arches as a result of their form have the tendency to exert lateral thrust at the abutments, which result in cracks or in the later stages instability in the form. Ties connecting the two abutments of the arches are commonly preferred solution in this kind of problems. Figure 5.1 has the schematic sketch and the real application of this method.

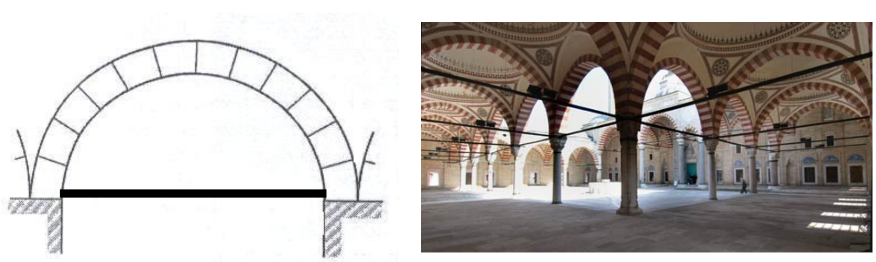


Figure 5.1 Steel ties used to overcome the lateral thrust action in arches (Selimiye Mosque, Edirne) [92]

For the domes, lateral thrust action is circumferential and as a result steel ties are not effective to prevent the action. Instead, steel rings are used at the perimeter of the dome. The rings enfold the dome and resist against the spread of the drum. The tie used for this purpose is prestressed before being erected in its place creating a strong confinement around the perimeter (Figure 5.2).

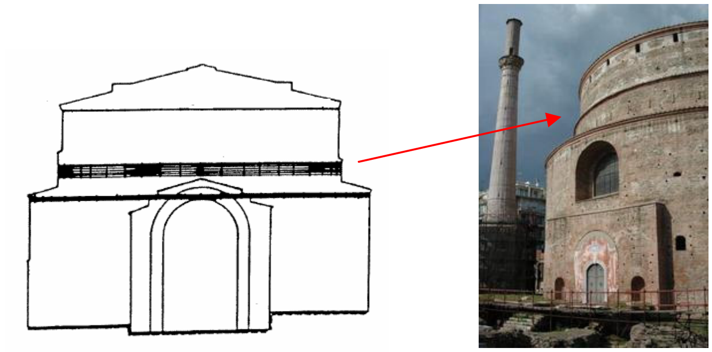


Figure 5.2 Prestressed steel ties used to support the dome (The Rotunda, Thessaloniki) [93]

As a common application similar to the supporting of the dome, steel ties are also used for the confinement of specific structural elements or the whole slender structure. Examples of the use of the method on pillar and slender tower are presented in Figure 5.3.

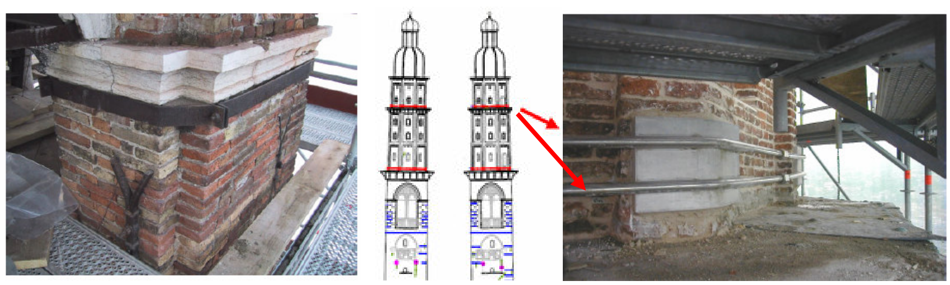


Figure 5.3 Steel confinements around pillar and around a tower [94]

Structural repointing is a bed joint reinforcement technique in which steel bars are placed in few centimeter excavated joints between the blocks and consolidated by the FRPs or mortar. The efficiency of the method increases with the regularity of the masonry facings since the weakness of the joints is aimed to be improved [95]. Figure 5.4 shows the examples of the application of this method.

Figure 5.4 Structural repointing application on masonry walls [95]

Separation at the corners of the walls is commonly encountered problem in masonry structures which threatens the structural integrity. A method called steel stitching is used to prevent further separation. Steel strips are installed by removing the upper masonry blocks extending to a distance to the correspondent masonry course in the corner. After the installation, removed units are placed using rich cement mortar. For the existing cracks grouting is applied. The schematic representation of the method is given in Figure 5.5. [96].

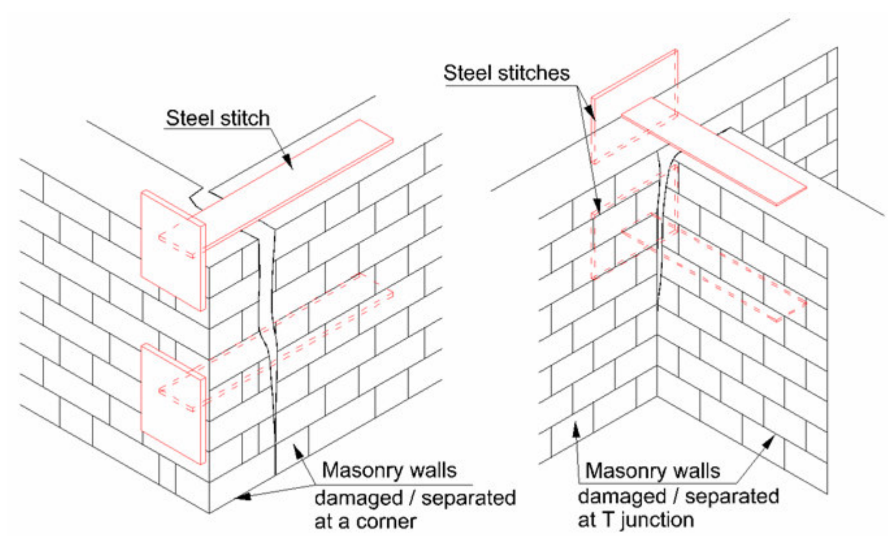
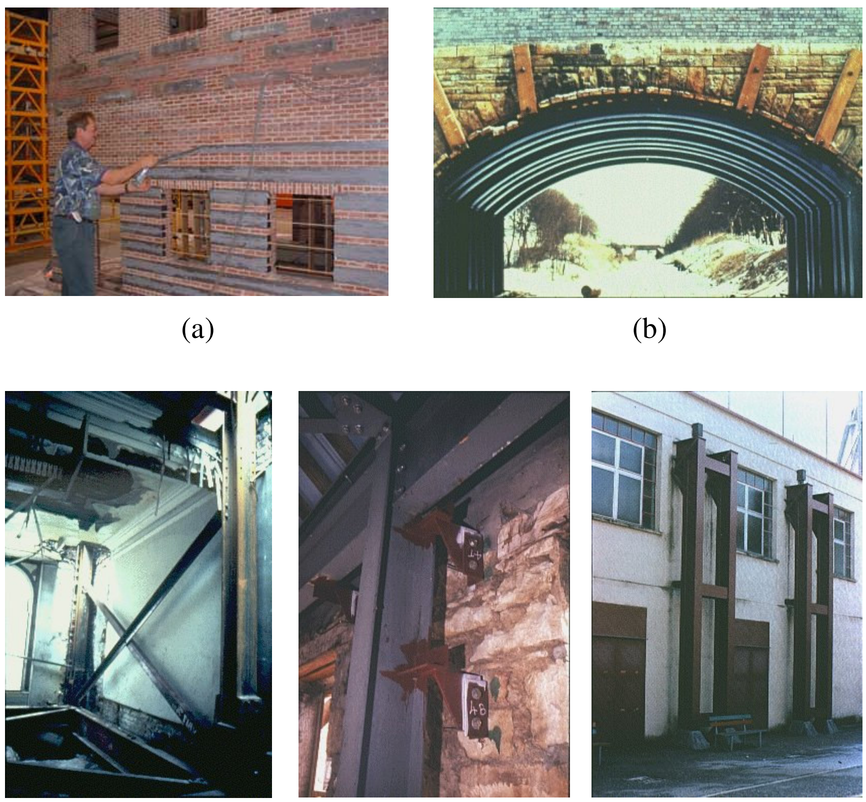


Figure 5.5 Strengthening of corners using steel stitching

Other than above mentioned commonly used restrengthening applications for masonry, a considerable amount of methods are developed and utilized for the specific cases. Few selected examples are provided in Figure 5.6 (a)-(d). Figure 5.6 (a) shows the FRP application with the use of steel wire meshes where historic fabric is considerably intervened. In Figure 5.6 (b) a bridge in Scotland is strengthened by steel members curved to a required radius under the masonry. Vertical and horizontal load carrying resistance is provided for the masonry walls by steel columns and bracings in Figure 5.6 (c),(d) and (e). Regardless of the method applied, the preservation of the historic fabric and the reversibility of the method (if possible) should be taken account before the intervention.



(a) (b)

(c) (d) (e)

Figure 5.6 Miscellaneous use of steel in strengthening applications [97], [98]

5.2 Method of Strengthening Historical Masonry by Steel Skeleton

Historical structures have exposed to destructive effects of the surrounding environment during their life time and damaged within a spectrum starting from cracks to partial collapses. The case of partial collapses has specifically preferred in this study since the form may not be readable from the remaining parts and load carrying mechanisms are altered making the structure more vulnerable to external effects. Strengthening applications are unavoidable to ensure the perpetuation of the structures.

In this study, it is aimed to propose a method using steel skeleton to strengthen a partially collapsed masonry structure while giving the sense of the original appearance. The method is applicable for a wide range of structures since the general principle is to provide the form while supporting the structure. The method is formed in general sense not for a specific type of structure in this study. The methodology was first developed conceptually and then the structural validity was evaluated analytically on a model by a finite element analysis program.

Before explaining further the methodology followed in this study, a brief literature is provided to set the grounds. The use of steel in the completion of missing parts is not a new application. There are several examples in the world and Turkey in which steel frames are used to give the sense of the form or to strengthen and provide the functioning of the structure. Figures below present an example from each case. Figure 5.7 shows the cases where steel frames draw the main outline of the missing form without contributing the structural load carrying system in a dome of a palace in Baeza, Spain. There is no clear evidence of the presence of the dome when the remaining walls are observed and this steel arches ease the perception of the complete structure.

Figure 5.7 Steel arches are used to draw the outline of the dome

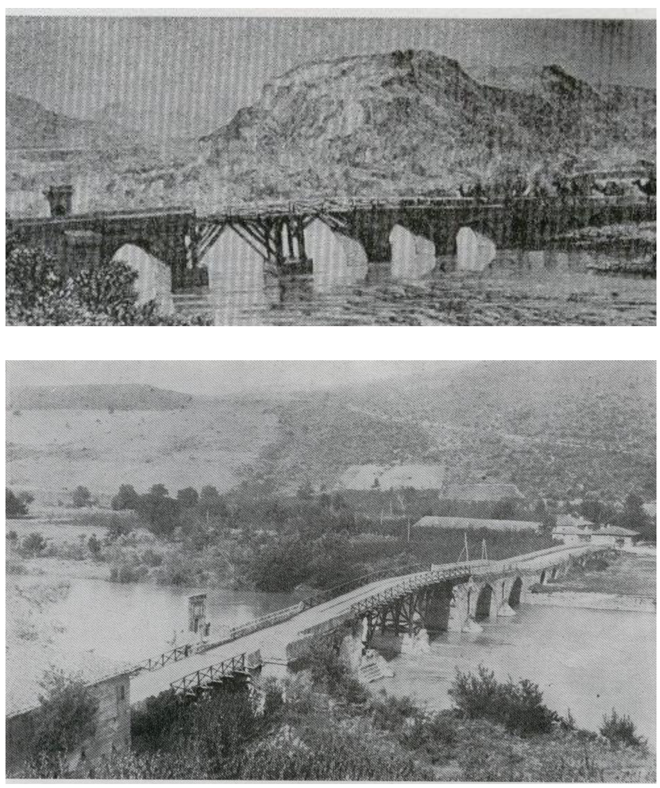


Figure 5.8 First attempt to complete the bridge in 1950s [99]

On the other hand, in Figure 5.8 and Figure 5.9 steel is used to complete the missing parts of a bridge (Geyve Sultan II. Bayezıd Bridge) in which steel enables the proper functioning. It is for sure that the method applied in this bridge provided the necessary strength for the passage of the vehicles and connected the two sides of river however the form and the continuity of the arches are lost as seen from the photographs. Two successive strengthening applied on the same bridge can be seen in Figure 5.8 and Figure 5.9.

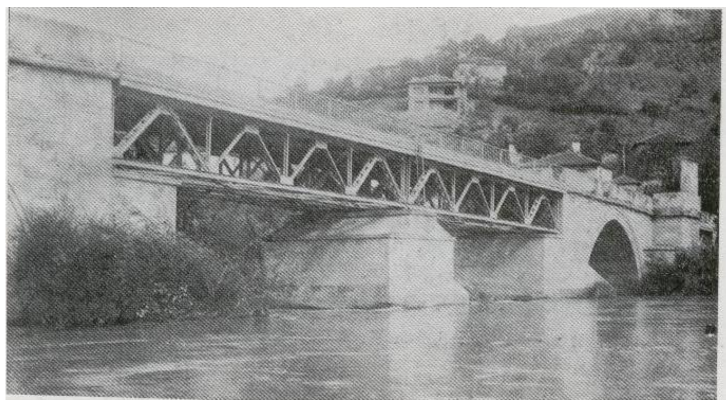


Figure 5.9 Final situation of the bridge connected with steel trusses [99], [100]

A study conducted by Örmecioğlu and Ünay [57] constituted a starting point for the proposed method. The study proposed the use of steel skeleton for the first time to complete an existing partially collapsed structure. A structural model of the Ani Church of the Redeemer had been prepared and steel frames following the form of the structure had enfolded the model. Analysis had been conducted by the researchers to see the improvement in the behavior of the structure after it had been restrengthened by steel skeleton. The photograph of the church and its corresponding 3D model can be seen in Figure 5.10.

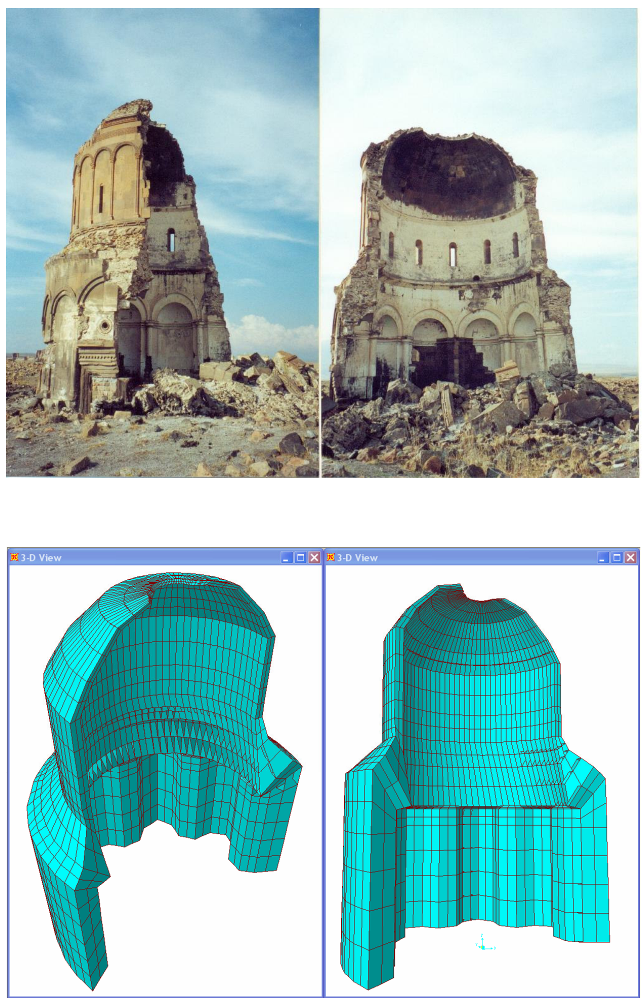


Figure 5.10 The images of the church and its corresponding mesh model [57]

The gravity and response spectrum analysis results revealed that the time periods of the structure, hoop and meridional stresses was significantly reduced in the strengthened model in Figure 5.11. However, since a specific partially collapsed structure is used in the work, the question of how effective the steel skeleton is to reflect the features of the masonry half has not been answered thoroughly.

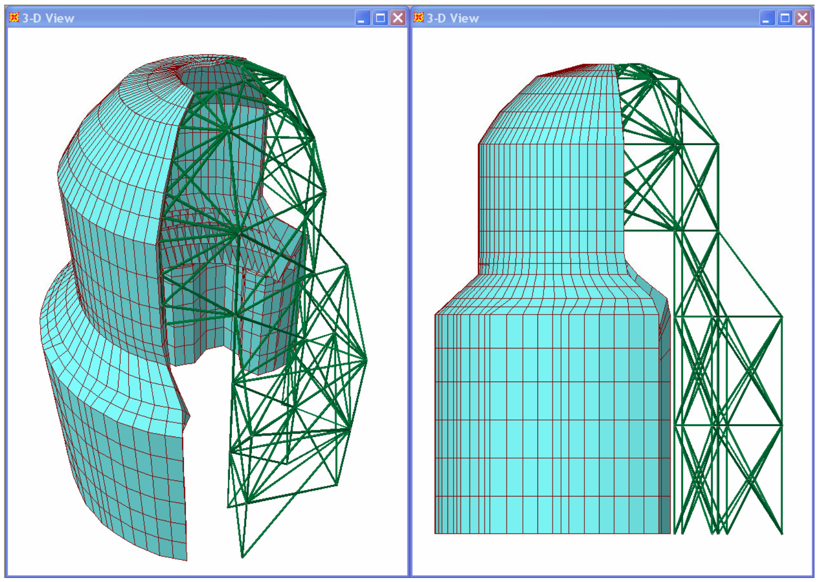


Figure 5.11 Strengthened model of the Church the Reedemer, Ani [57]

In this study, a different approach is carried out on the same conceptual based. An intact model is developed and then some structural elements are removed. By this way, a partially collapsed model is generated. Then, the partially collapsed model is completed by steel frame elements as a strengthening application. The whole study is based on the comparison of the behavior of the strengthened model with the intact one.

The model created is composed of one dome supported by four main arches that rest on four pillars. It is similar in the structural characteristics and form to the Mihrimah Sultan Mosque of the masterbuilder Sinan. The model corresponds to the inner part of the historical skeleton which contains no walls that contribute to load carrying system and give extra stiffness and massiveness to the structure. A 3D mesh of the model is given in Figure 5.12.

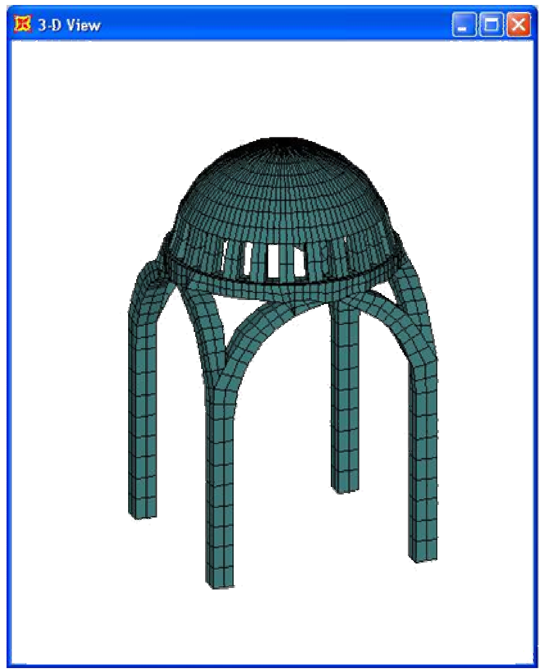


Figure 5.12 The model used in this study

Some small scale analyses are conducted to determine the worst case of removal in which the stability of the model is threatened. The cases where half of the structure remains are not taken into account like in the case of Ani since the structure is more stable when the half of it collapsed instead of the collapse of a structural element (arch or pillar). Fundamental time periods of the models are used to compare the level of the instability in the models since the higher period value is observed in highly damaged structure. Firstly, an arch is removed and the analyses are carried out. The modal periods are provided in the Figure 5.13 (a). Secondly, as a more drastic case one of the pillars is removed. The results indicates that the removal of one pillar result in higher period values i.e., more severe damage (Figure 5.13 (b)).

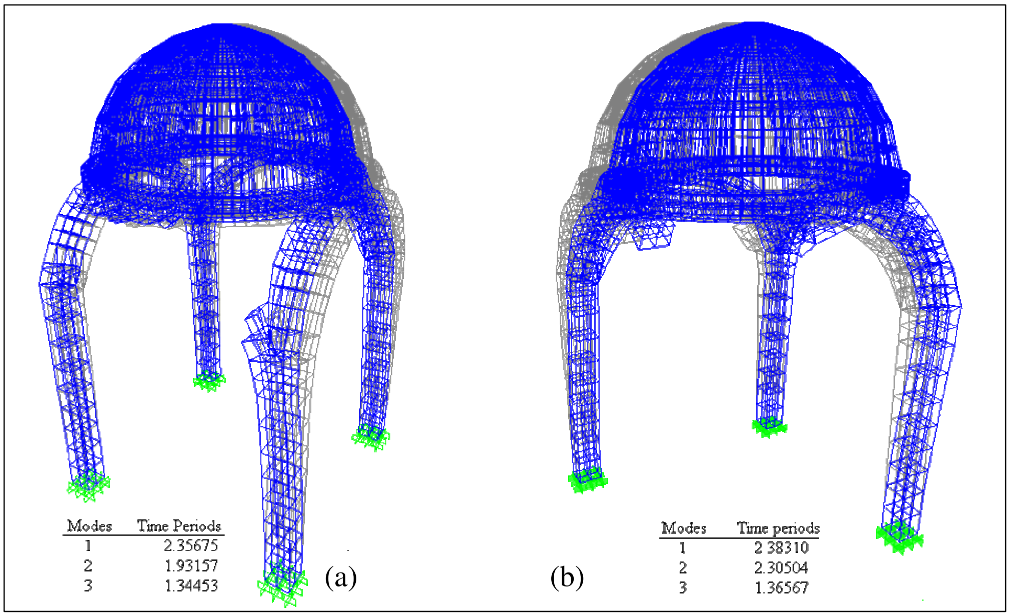


Figure 5.13 Deformed shapes of the models in mode 1 with time period values (a) an arch missing and (b) a pier missing

The model with one pillar collapsed is selected for further investigation as a partially collapsed model. Steel skeleton is constructed to complete the geometry and form as well as to support the upper structure following the grids of the solid elements with different steel sections for different places. The partially collapsed model and the strengthened model are given in Figure 5.14.

The methodology applied in this study aims to evaluate the performance of the method-strengthening with the steel skeleton- in approaching the behavior of the original intact model not to evaluate the success of the strengthening method in reducing the stresses or deformations. Therefore, the effect of the method on rehabilitating the existing deficiencies of the structure is not considered. The stress values on specific points and locations and values of maximum displacements and modal periods are used as parameters to evaluate the effectiveness of the strengthened method to reflect the behavior of the intact model.

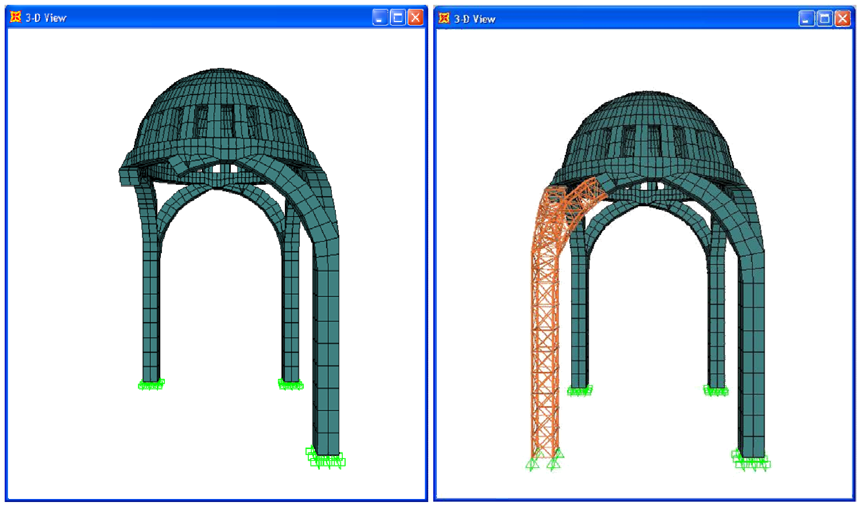


Figure 5.14 Partially collapsed model and the completed model

CHAPTER 6

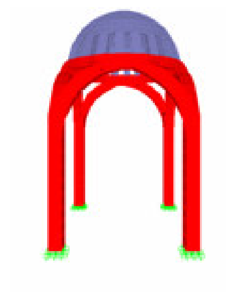
FINITE ELEMENT ANALYSIS OF THE PROPOSED STRENGTHENING METHOD

6.1 Geometric and Material Characteristics of the Conceptual Model

For the verification of the validity of the proposed strengthening method explained in Chapter 5, a conceptual model is used. The second model is developed through the modification of this conceptual model with a steel pillar.

In the analytical model, round- off values are used in the dimensioning of the members instead of the actual dimensions of the inspired mosque since the main purpose was to analyze and observe the functioning of the steel skeleton in structures with complex elements, not to investigate a specific structure. The height of the structure from foundation level to the top of the dome is 38.5 m with column height of 16.5 m. The pillars are 2 x 2 m in cross-section. The diameter of the dome is taken as 19 m with thickness varying from 1 m to 0.50 m at the top. A series of windows are placed on the drum element beneath the dome with dimensions of 1.37 m by 3.5 m.

In the construction practice of similar historical masonry structures generally different materials are used for the dome and the rest of the structural elements. In order to reflect this common tradition, two different masonry types are defined for the model: Masonry 1 and Masonry 2, successively. The self weight and modulus of elasticity values of the Masonry 1 is higher than those of the Masonry 2, which, in a way depicts the general application in domed structures with heavy structural elements and light dome. Masonry 1 is used in pillars, main arches and the drum while the dome is made of Masonry 2. Figure 6.1 illustrates the masonry types on the model with specific properties taken for each type.



Properties	Masonry 1	Masonry 2
Unit Mass (KN-s2/m4)	2.55	2.24
Unit Weight (KN/m3)	25	22
Modulus of Elasticity (MPa)	4500	1500
Poisson's Ratio	0.2	0.2

Figure 6.1 Masonry types and their characteristics values

As explained in Chapter 5, steel is used for the strengthening of the modified model. It has unit weight of 76.82 KN/m^3 with elasticity modulus of 199947 MPa. The Poisson’s ratio is taken as 0.3 for steel frames. Three different types of steel pipe sections are utilized in the erection of the steel skeleton. The thickest pipe is placed along the corners of the pillar working as load carrying elements. Thin sections are used for the diagonal and horizontal elements in the model. Pipe dimensions can be seen in Figure 6.2 along with representative sketch of the skeleton.

Types	**t3 (m)**	**tw (m)**
Pipe 1	0.25	0.012
Pipe 2	0.2	0.012
Pipe 3	0.1	0.0035

Figure 6.2 Steel sections and their dimensions

6.2 Description of the Finite Element Models

The proposed strengthening method is tested through the finite element models created in SAP2000 structural analysis software. Geometric representation of the models is done by 3D solid elements which are known to capture the behavior of complex 3D models better than frame and shell elements. During the modeling, the interface between masonry and mortar is not considered. That is, macro modeling approach explained in Section 3.5 is used which assumes a fictitious homogeneous isotropic medium for masonry, mortar and the interface. Besides, nonlinear material properties are not included since the models are not the representatives of the specific structures. Two separate models are generated for the analysis stage. Model 1 is the intact masonry structure modeled by using 3008 solid elements and 5272 joints. Figure 6.3 presents the side and plan mesh views of the intact model. On the other hand, Model 2 is created by removing one pillar and some parts of the neighboring arches which is, then, enfolded by the steel frames as explained in a detailed way in Chapter 5. At the end, this modified model is composed of 2912 solid elements, 5154 joints and 411 steel frames. The mesh of the strengthened model can be seen in Figure 6.4. The two models are identical in material, geometric and restraint characteristics excluding the steel skeleton. Table 6.1 below gives the total weight and the contribution of different materials to total sum. As seen from the values, the intact model is heavier than the model with steel reinforcement. This difference in the weight, in a way, is the precursor of the probable differences in the base shear values which will come into picture later in the response spectrum analysis case.

Table 6.1 Weight of the finite element models.

Weight	**Intact Model**	**Model with steel**
Masonry 1 (kN)	20399.46	16992.69
Masonry 2 (kN)	7803.56	7803.56
Steel (kN)	0	146.41
Total (kN)	28203.02	24942.66

In the connection of masonry and steel, to assure the continuity in rigidity along the masonry and steel frames extra diagonal elements are provided.

6.3 Finite Element Analysis of the Models

In order to illustrate the efficiency and the compatibility of the models, two major analyses are performed for each model: gravity analysis and dynamic analysis based on the current Turkish Seismic Code. As stated by many researchers ([42], [47], [48], [101]), linear elastic analysis gives satisfactory results in the overall behavior of the whole structure but generally is unsatisfactory for a detailed investigation of specific parts.

As for this study, the performance of whole structure is under concern and some local overstressed and over deformed regions are determined not to investigate further but to compare the two models whether they show the same behavior in the same locations. Therefore, all major analyses are conducted to observe the linear elastic behavior of the models. The performed analyses for the verification of the methodology are listed below in organized manner:

Intact Model *(Model 1)*
- Gravity Analysis
- Response Spectum Analysis
- Modal Analysis

Strenghtened Model *(Model 2)*
- Gravity Analysis
- Response Spectum Analysis
- Modal Analysis

Gravity analysis is done under the effect of the weight of the structures in which the static performances under permanent loads are observed. As for response spectrum analysis, the behavior of the structure is determined in seismic zone 1 using the spectrum of the region that corresponds to high seismicity in Turkish

Earthquake Code [93]. Modal analyses of the models are also performed to see the structural response and to determine the fundamental periods of the structures. The first three modes are taken for the comparison of the deformed shape and the periods. All the finite element analyses are conducted in the SAP2000 environment in which the models are created.

6.4 Discussion of the Results

For the comparison of the two models, the fundamental periods of the two structures are taken into account as the first parameter. Since the period values are one of the identifying characteristics of the buildings as well as the modal shapes which represents the way the structures responds to the excitation. The results of the analysis indicate the expected behavior for a massive and slender structure which is relatively high period compared with that of concrete structures with same height. The fundamental period values for the models are 1.91s and 1.92 s respectively for Model 1 and Model 2. For the periods at Mode 2 and Mode 3 the difference between the values is in the order of 0.01s. The models show almost the same response in terms of period indicating that with the steel pillar and arch elements similar response characteristics are achieved. Besides, the modal periods of two models are provided below in Table 6.2

Table 6.2 Modal periods of the models

Mode	Model 1	Model 2
1	1.912051	1.922844
2	1.911622	1.903237
3	1.267227	1.252625

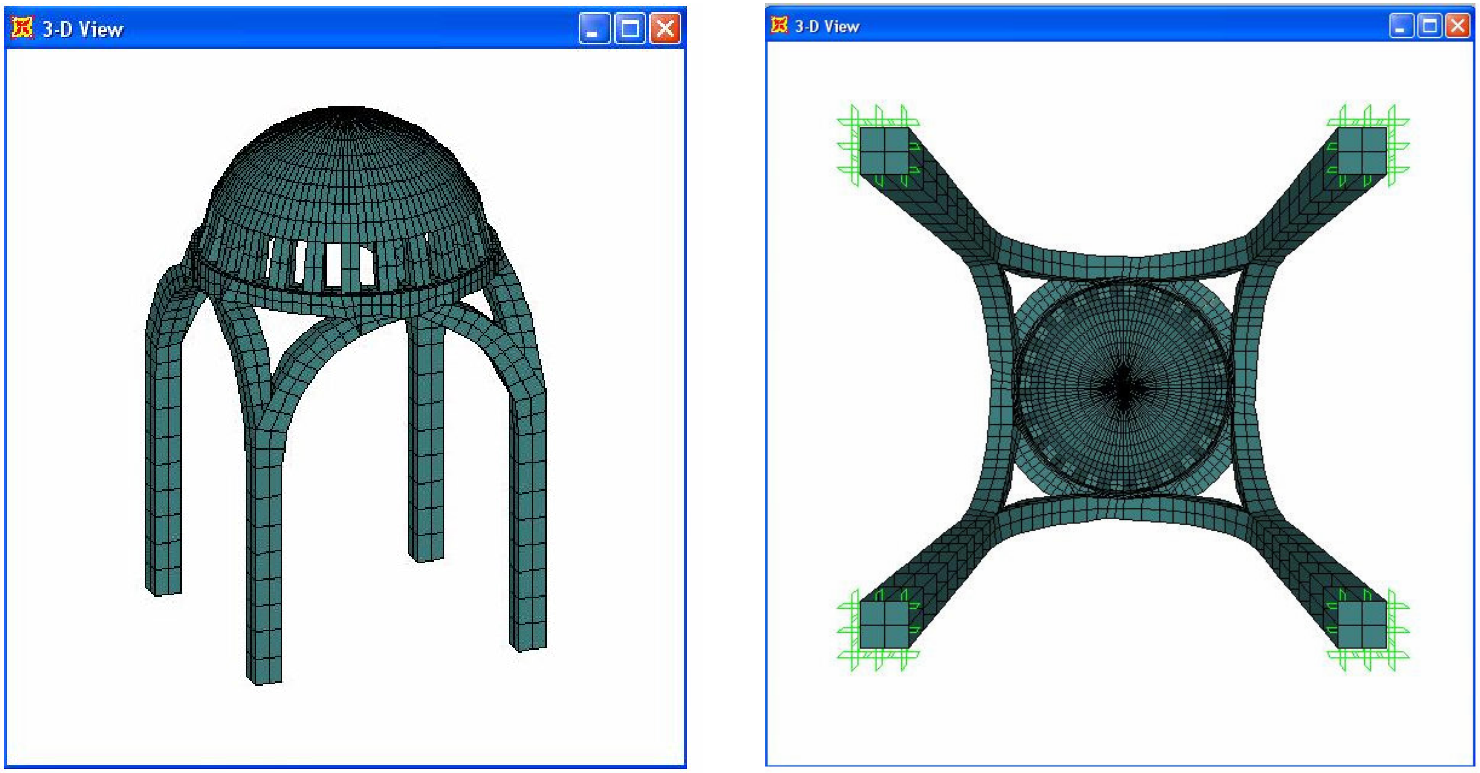


Figure 6.3 The finite element model of the intact structure

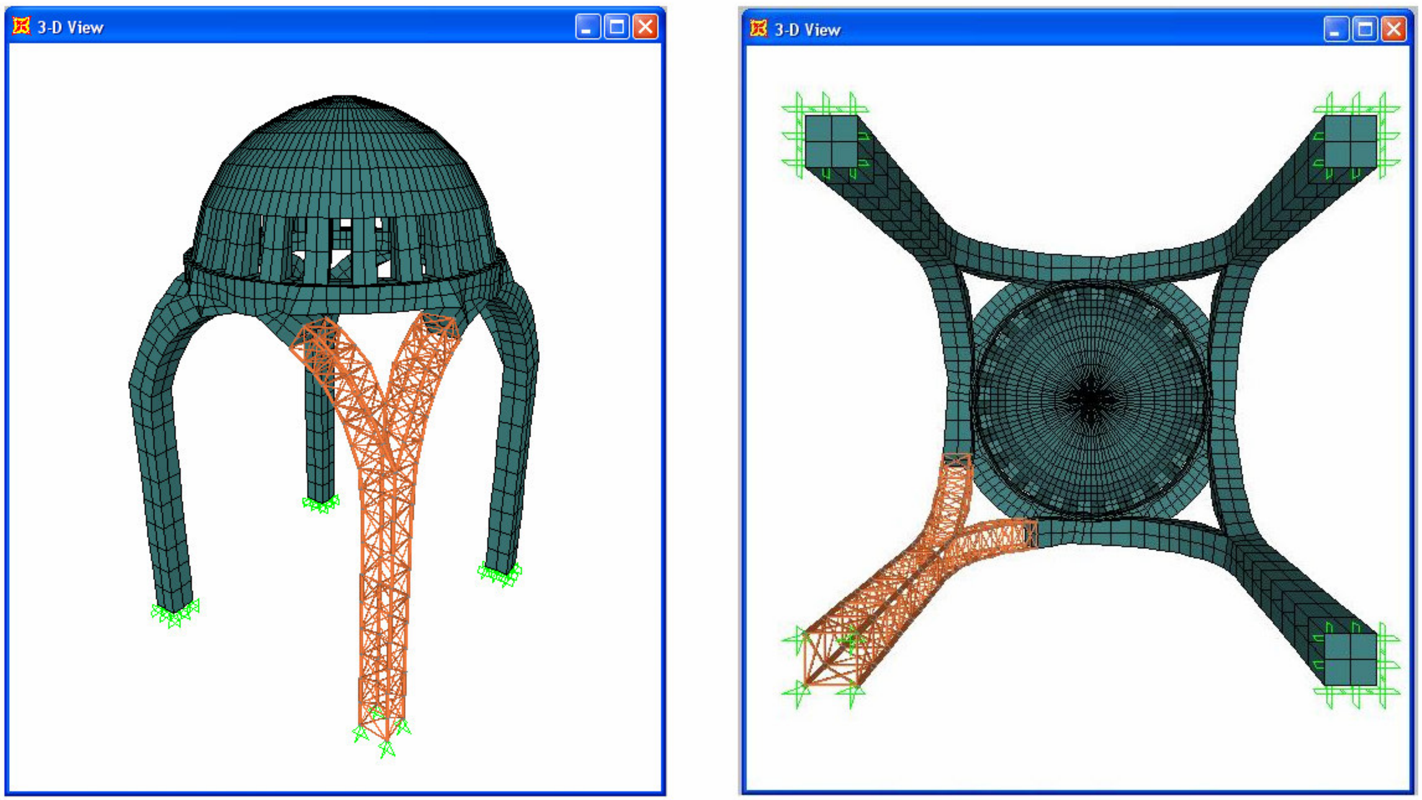
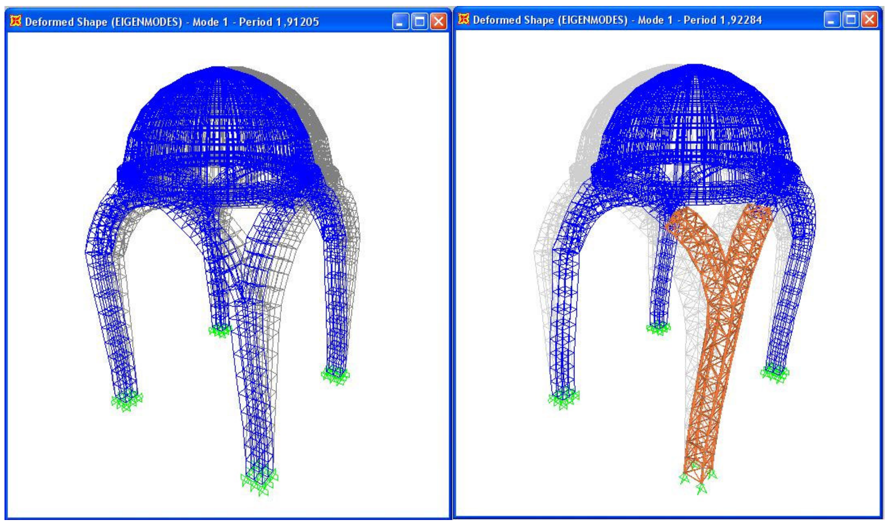


Figure 6.4 The finite element model of the strengthened structure

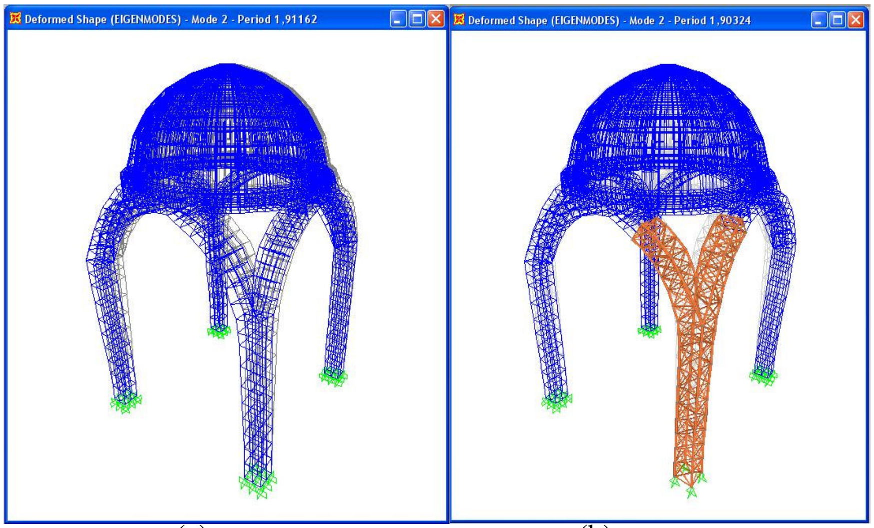
Modal deformed shapes of the models are also similar as seen in the following three figures for the first three modes. The gray lines in the figures show the undeformed shape of the model. Different behaviors in the deformed shape is observed in the first mode due to the newly added material (steel) with different stiffness in a way creating a heterogeneous structure However, the general trend and the location of the deformation lines are similar as seen in Figure 6.5.



(a) (b)

Figure 6.5 Modal deformed shapes of Model 1 (a) and 2 (b) for Mode 1

As for the second mode, the deformed shapes are seen in Figure 6.6. The effect of the difference in overall stiffness can be observed along the steel framing with a direction change as well as some amount of torsion in Model 2.



(a) (b)

Figure 6.6 Modal deformed shapes of Model 1 (a) and Model 2 (b) for Mode 2

The third mode behaviors of the two models are almost identical. Figure 6.7 depicts the deformed shapes in which the direction of the torsion is also same.

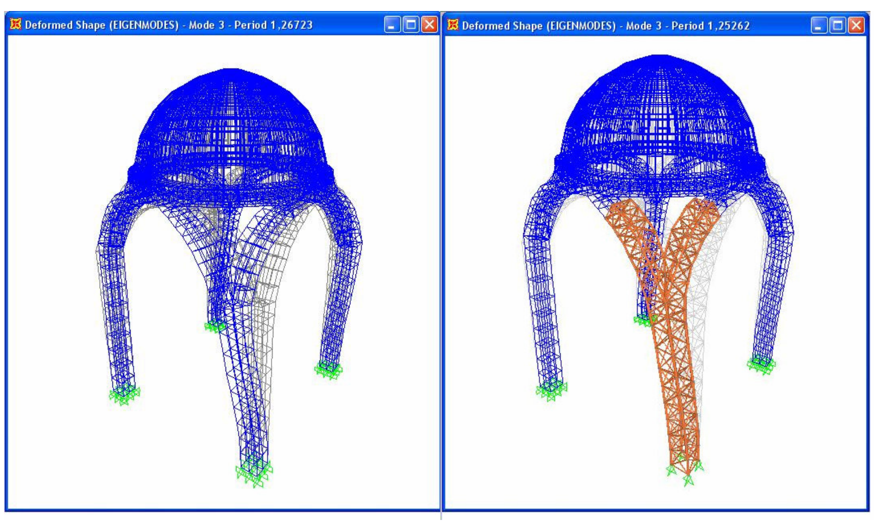


Figure 6.7 Modal deformed shapes of Model 1 (a) and Model 2(b) for Mode 3

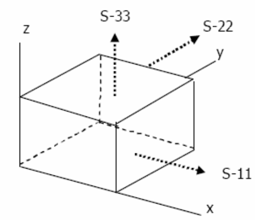


Figure 6.8 The sketch showing the local axes and stress directions on solid element

Figure 6.8 shows the sketch of the local axes and stress directions on solid element. The gravity analyses results on the vertical direction (S33) are provided in the Figure 6.9 and Figure 6.10 for the models. As seen in the figures, the overall behavior and the stress concentrations of the members are almost the same along the structures regardless of the presence of the steel skeleton. Figure 6.9 present the stress distribution of the models on the deformed plan view. The deformed shapes of the intact model and the model with steel indicate that the responses of the structures to the gravity loading are very close to each other. Although the color-stress scale beneath the models uses different ranges for the stresses and their corresponding colors, the same tensile stress concentration is observed in the outer side of the pillar where it connects with two arches. Similarly, high compression regions in the outside of the lowest pillar elements and in the inner arch-pillar connection elements are seen on the same locations in two models.

A different view of the stress distribution is provided in Figure 6.10 to observe the situation on the dome and drum elements. It is seen from the figure that the response of the drum and the dome are the same and no stress concentration is observed due to the change in load transfer mechanism as a result of the replacement of the pillar and arch elements with steel. The abovementioned high tension and compression locations are clearly seen in the figure.

The maximum principle stress values along the finite elements are presented in Figure 6.11. The distribution in the pillars is as expected and the two models respond similarly with close values which are discussed in the later paragraphs. The stress concentrations at the points where the window elements and the dome elements connected are more visible in the strengthened model. In other words, in the upper corners of the window element there observed compressive stress increase due to the presence of the steel skeleton although the increment of the increase is small.

The structural behaviors of the models under gravity loading are investigated further with numerical values compared through three different parameters as follows:

- The stresses along a pillar (a neighboring pillar to the steel pillar)
- The stress valus at the locations of maximum determined in the intact model
- Displacement values at locations of maximum deformation

The stress variation in one of the pillars (pillar 2) is determined to observe the difference in stresses between the intact model and the strengthened model. The stress values are taken along the height on each solid element both in inner and the outer corner. The vertical red lines in Figure 6.12 show the corners on the pillar 2 that the values are taken for the determination of the variation.

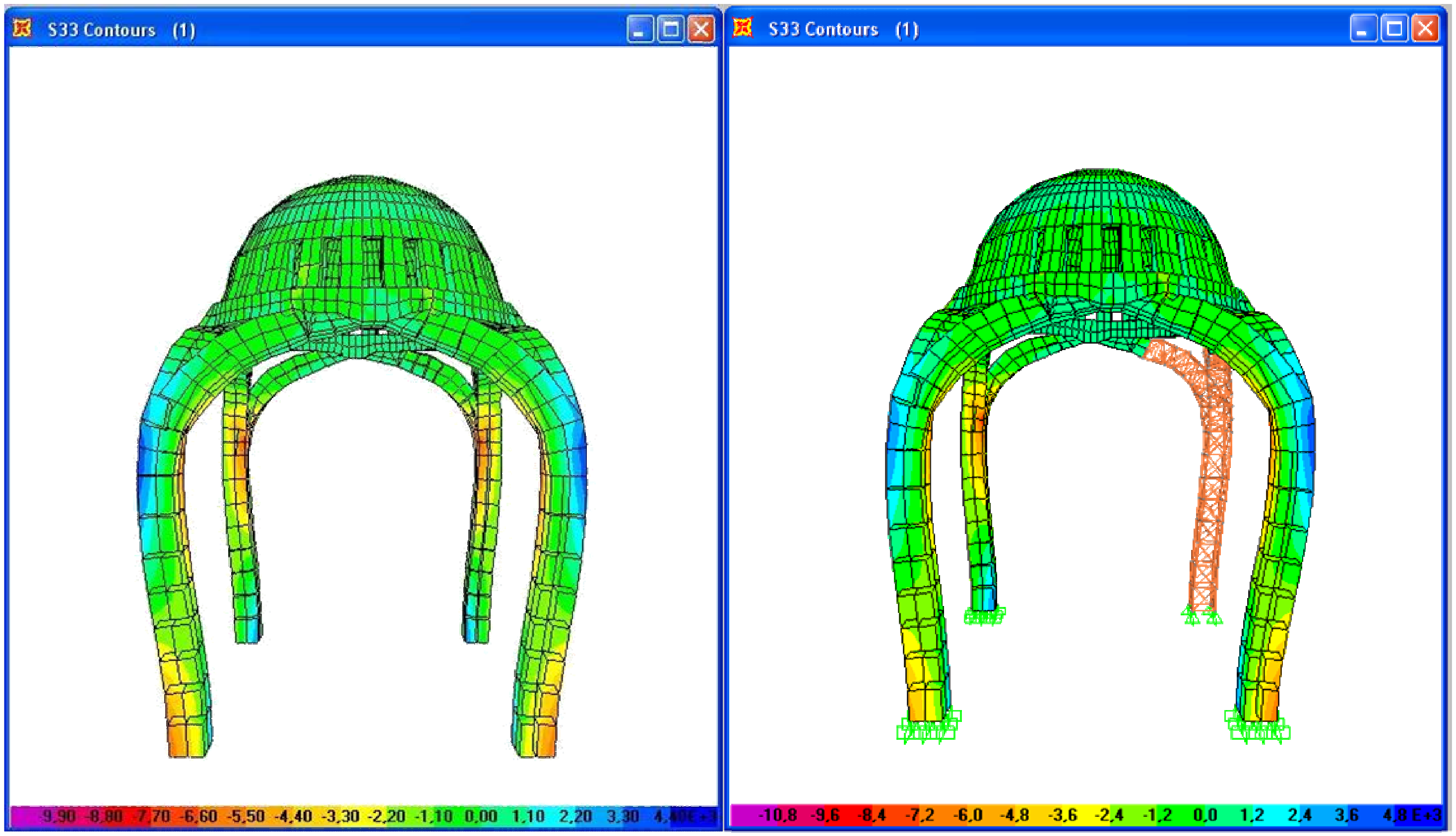


Figure 6.9. The stress,S33, distribution of the Model 1 under gravity loading (side view)

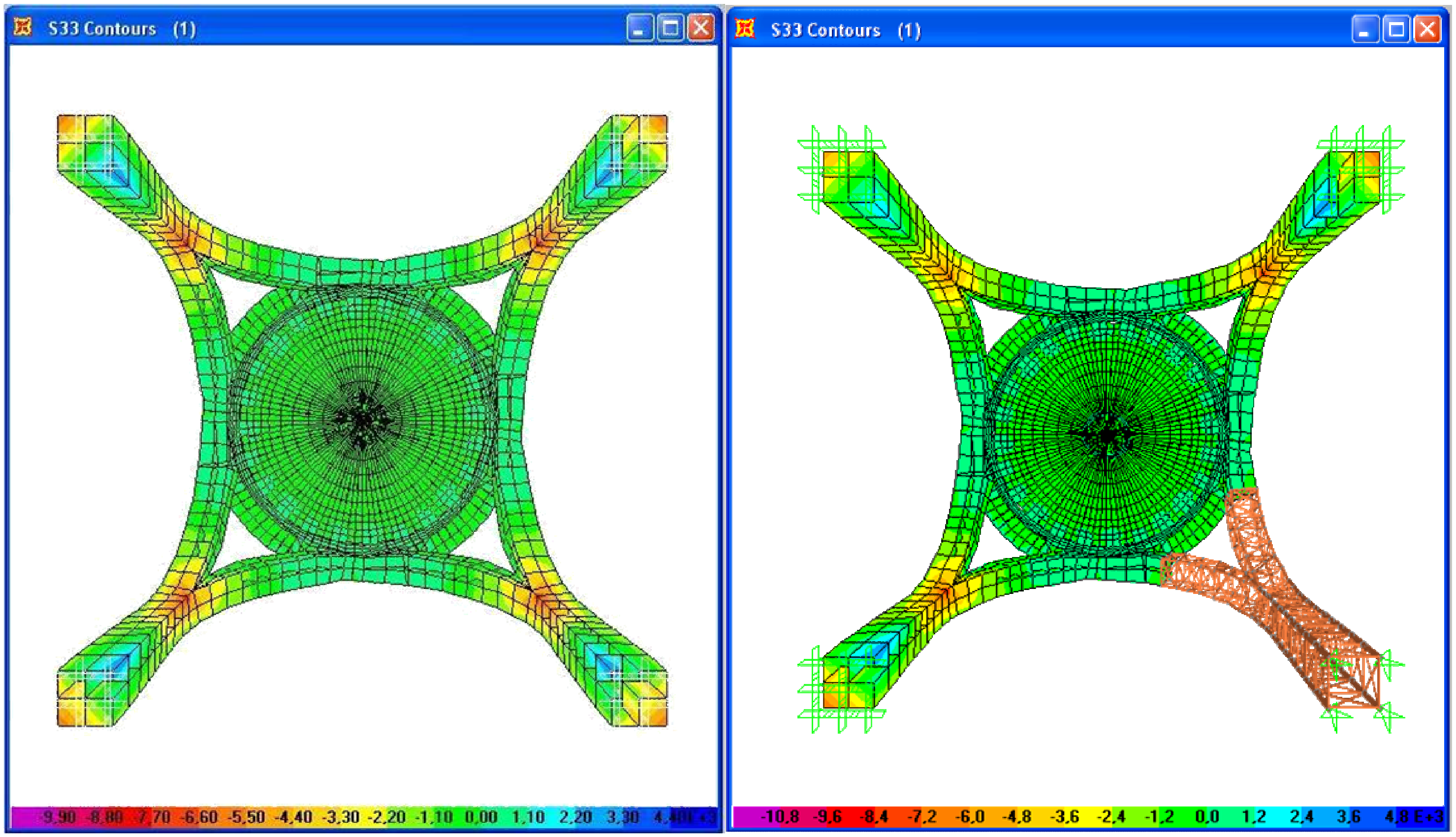


Figure 6.10. The stress,S33, distribution of the Model 1 under gravity loading (plan view)

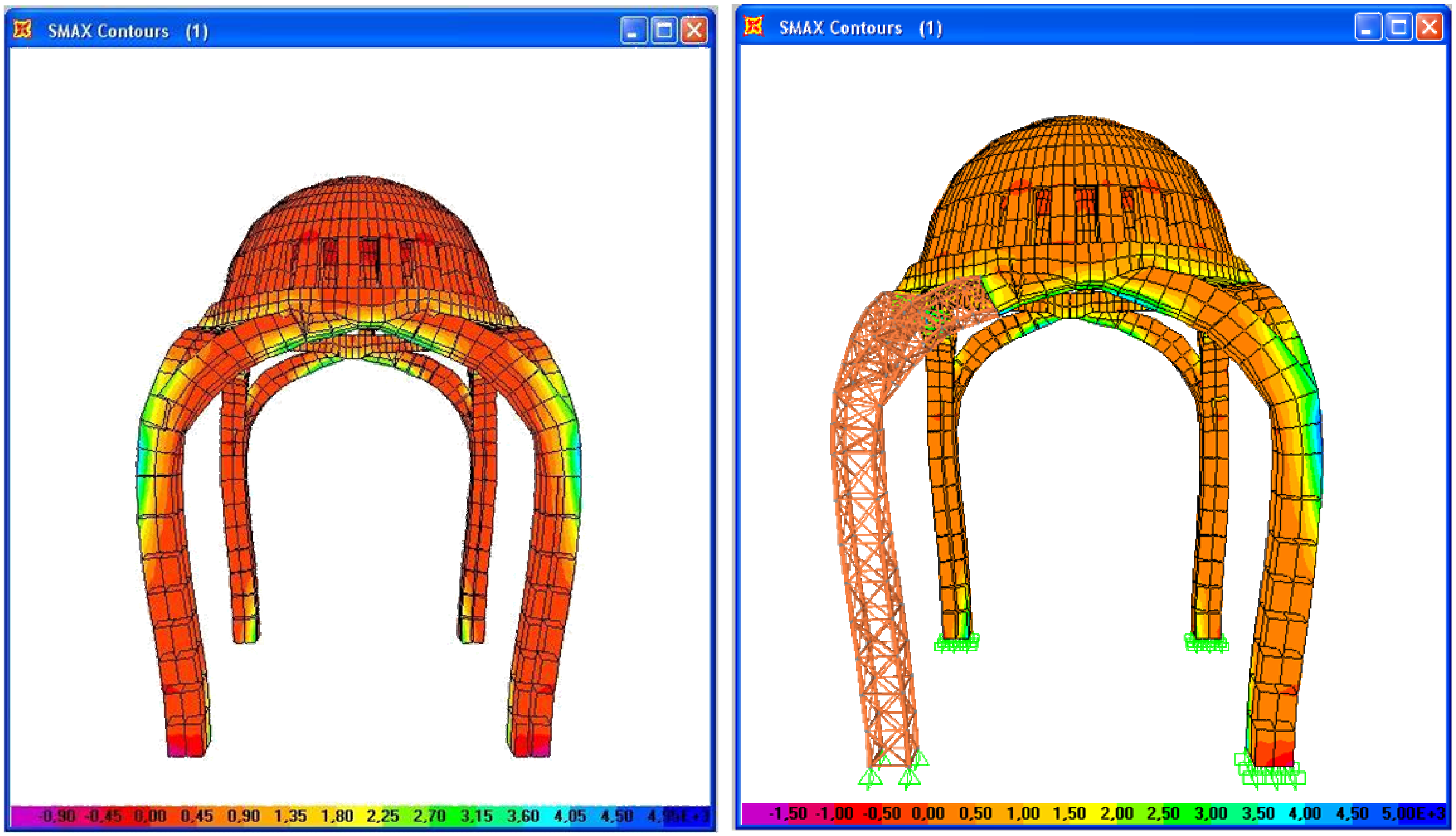


Figure 6.11 The stress,S_{max}, distribution of the Model 1 under gravity loading

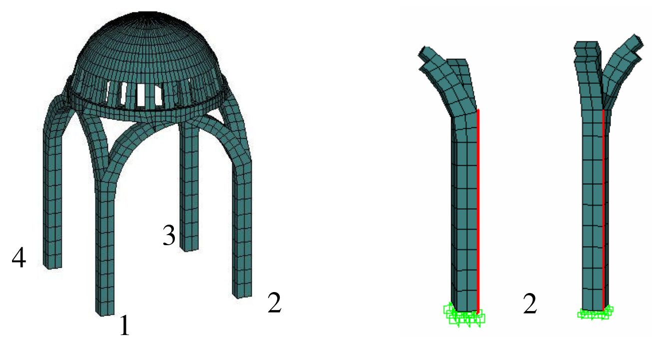


Figure 6.12 The corners on pillar 2 where the stress values are taken

The stress values (S33, Maximum principle stress, S_{max} and minimum principle stress, S_{min}) of the pillar 2 in both of the models are then compared by graphics and the results are tabulated for inner and outer corners separately. The charts presenting the variation of the stress along the corners are provided in Figure 6.13 and Figure 6.14.

Figure 6.13 indicates that the pillar 2 in Model 1 and Model 2 has the same stress variation of S33 up to 10 m. Stresses in Model 2 slightly diverges from the Model 1 after that height however the trend of the curve is similar with that of the Model 1. As for S_{max} the variation is vice versa. That is, up to a certain height the curves are parallel to each other and after 6 m they converge into same curve. The variation of Smin in Model 2 is almost the same with that in Model 2. As for Figure 6.14, the outer corner of the models, the curves in S33 variations has the same trend and values along the height. A slight divergence is observed in the last 10 m of the S_{max} variation and in the first 10 m of the S_{min} variation.

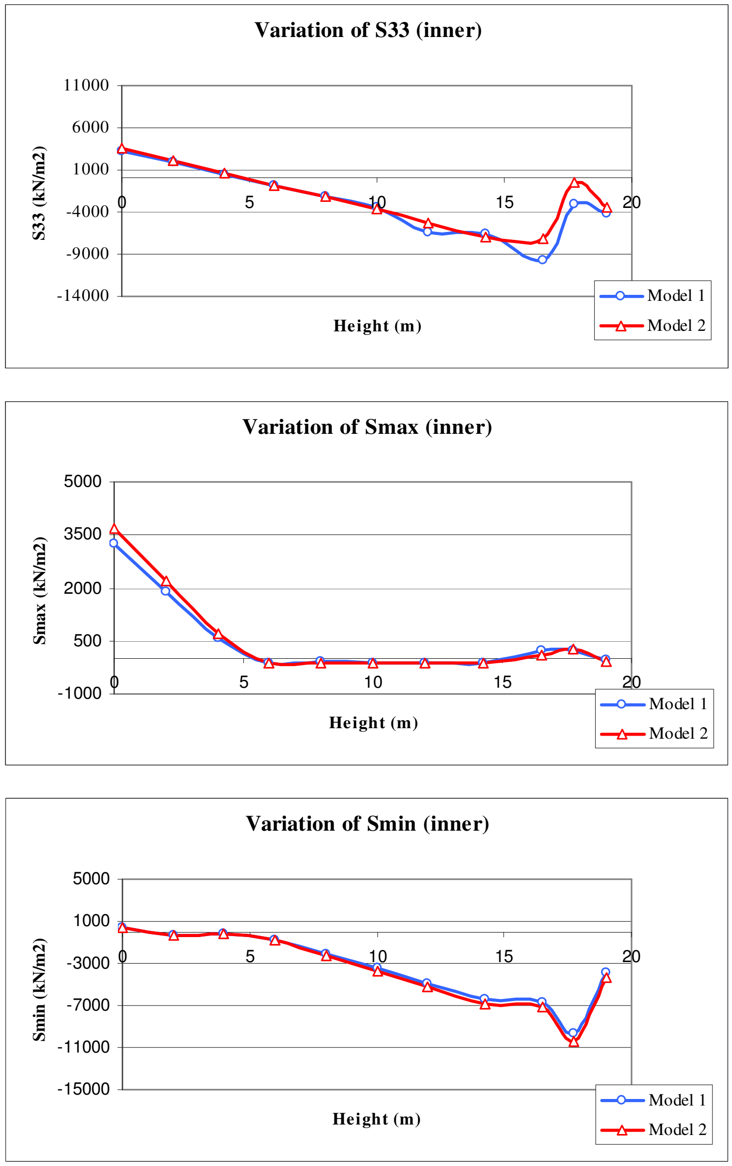


Figure 6.13 Stress variations along the height of the pillar 2 under gravity analysis (inner)

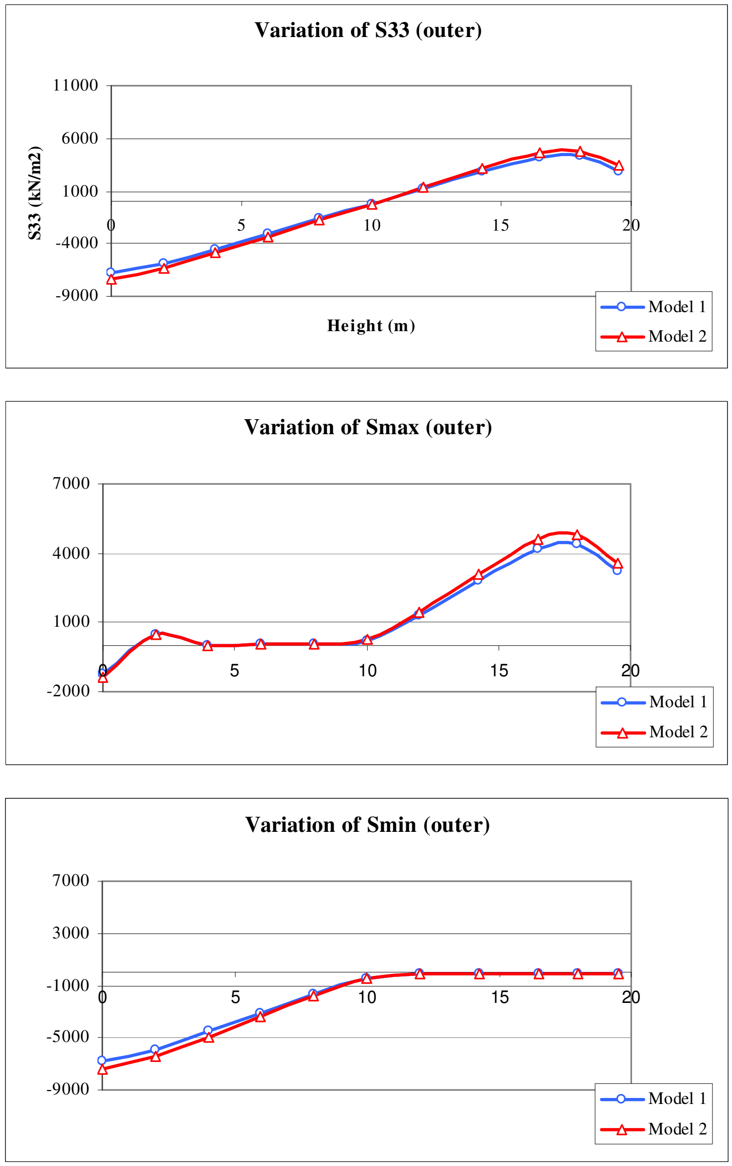


Figure 6.14 Stress variations along the height of the pillar under gravity analysis (outer)

Table 6.3 has the numeric values of the gravity analysis results, S33, S_{max} and S_{min} for the two models in comparative manner. Stress values in the table have the unit of 10^{-3} MPa.

Table 6.3 The stress values along the height of the pillar 2

INNER CORNER

	MODEL 1			MODEL 2		
Height	**S33**	**Smax**	**Smin**	**S33**	**Smax**	**Smin**
19	-4130.5	-14.75	-3954.76	-3360.38	-73.17	-4345.07
17.75	-3085.37	217.09	-9714.21	-407.92	253.34	-10466.48
16.5	-9659.61	231.06	-6677.23	-7081.84	100.87	-7083.01
14.25	-6567.32	-109.65	-6396.41	-6881.35	-120.07	-6886.41
12	-6391.85	-104.47	-4933.28	-5275.28	-114.67	-5281.37
10	-3481.33	-102.62	-3489.13	-3692.78	-112.46	-3701.57
8	-2155.56	-95.44	-2168.86	-2240.6	-104.64	-2255.71
6	-862.57	-109.55	-839.57	-760.62	-119.47	-807.99
4	510.49	556.18	-168.13	680.04	723.61	-176.15
2	1876.8	1883.63	-323.52	2169.65	2176.6	-360.38
0	3242.14	3256.94	374.5	3671.25	3687.21	436.04

OUTER CORNER

	MODEL 1			MODEL 2		
Height	**S33**	**Smax**	**Smin**	**S33**	**Smax**	**Smin**
19.5	2853.19	3218.24	-93.74	3455.92	3589.53	-99.685
18	4344.18	4391.1	-120.05	4782.07	4830.71	-126.163
16.5	4184.11	4188.14	-73.84	4572.21	4576.58	-79.905
14.25	2812.48	2821.85	-64.08	3078.26	3088.45	-69.453
12	1254.24	1280.52	-41.64	1382.94	1411.161	-45.223
10	-230.87	231.62	-450.31	-233.22	260.183	-480.942
8	-1660.52	41.55	-1678.19	-1789.24	45.472	-1808.824
6	-3122.6	40.18	-3130.92	-3375.67	44.029	-3388.816
4	-4524.81	-19.93	-4531.54	-4906.73	-21.269	-4914.136
2	-5888.39	444.88	-5893.2	-6389.73	483.396	-6394.972
0	-6809.24	-1253.33	-6823.14	-7391.05	-1352.495	-7406.2

Locations of maximum stresses under gravity loading in vertical direction are selected as other parameters to compare the two models in terms of behavior. As previously discussed in Figure 6.9 and Figure 6.10, the two models are said to be presenting the similar behavior in general sense. Stress concentration locations of

Model 1 are determined and the values are compared with the stresses in the same locations of the Model 2. The maximum stress points are shown on the Figure . A and B points designates the maximum compression stress points while C and D indicates the maximum tensile strength points of the whole model. The points are selected randomly since the model is symmetric, i.e., the same values are valid for other maximum stress locations on other pillars.

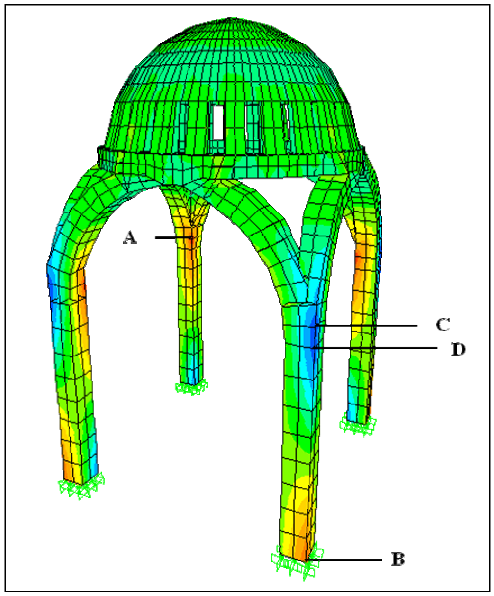


Figure 6.15 Maximum stress points under gravity loading in vertical direction

The S33 stresses of the points shown in Figure 6.15 are compared with that of Model 2. As seen from the Table 6.4 and Figure Figure 6.16 below, an increase is observed in the stress values of the Model 2. However, this increase is not a drastic one when the units of the values are taken into account which is kN/m^2 i.e. 10^{-3}MPa.

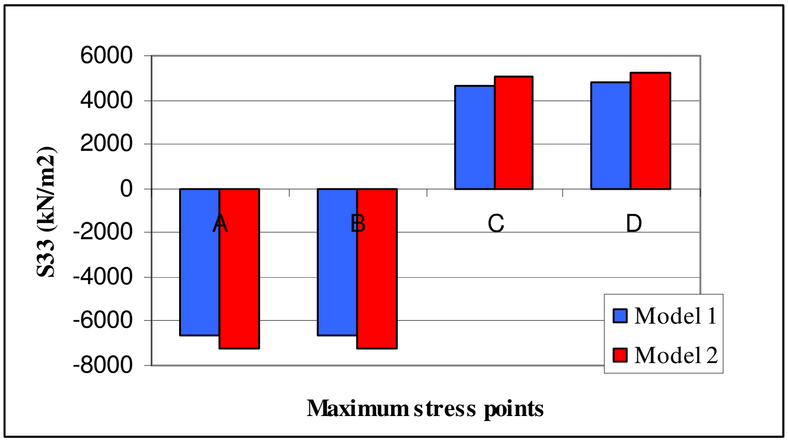


Figure 6.16 S33 of the models on points A,B,C and D

Table 6.4 Stress values of the models at A,B,C and D

	A	B	C	D
Model 1	-6672.06	-6664.82	4627.91	4780.65
Model 2	-7196.45	-7231.94	5056.39	5223.24

The last parameter considered in the comparison of the models is the displacements at some specific points on pillar, arch, drum, window and dome elements. The points are selected from the locations of maximum displacements and marked on the deformed shape as in the Figure 6.17. Eight points are indicated in the figure as follows: 2 points from the pillar, 2 from the arch, 2 from the dome, 1 from the drum and 1 from the window.

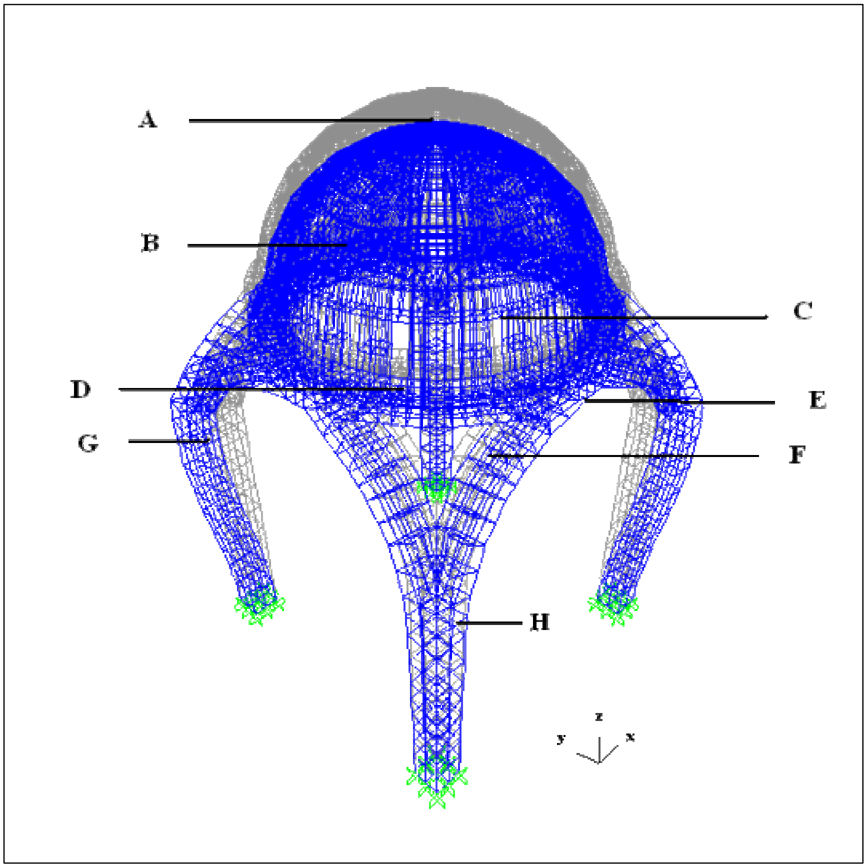


Figure 6.17 The points where the maximum displacements occur in the structural elements

The displacements of specified points reveal that slight differences are observed although the order is around few millimeters in most of the points. For the points A, B, C and D change in direction is observed with an increase in the values in x-direction (Figure 6.18). The increase is expected since a slight increase in the stresses is pointed out in preceding paragraphs. As for y direction, a similar increase is observed on the dome and the drum points as well as the points on the main arch (Figure 6.19). The graph representing the values in z-direction reveals that for B, D and H, the deformation values do not change while for the remaining points the order is between 0.1- 8 mm (Figure 6.20).The exact values are given in Table 6.5.

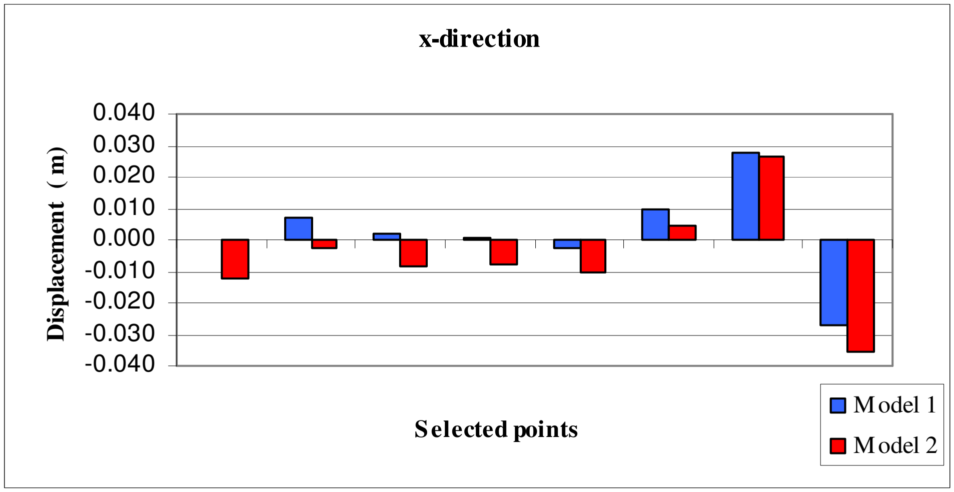


Figure 6.18 Displacements of critical points at x- direction

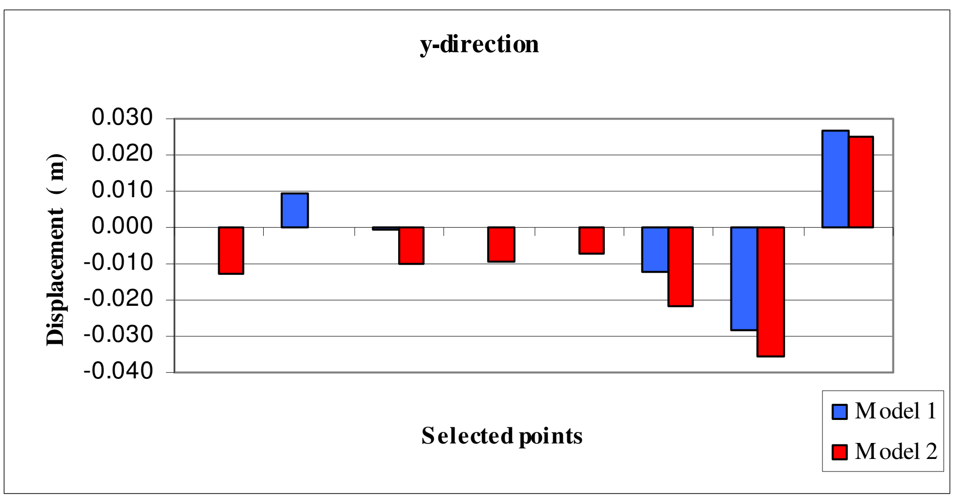


Figure 6.19 Displacements of critical points at y- direction

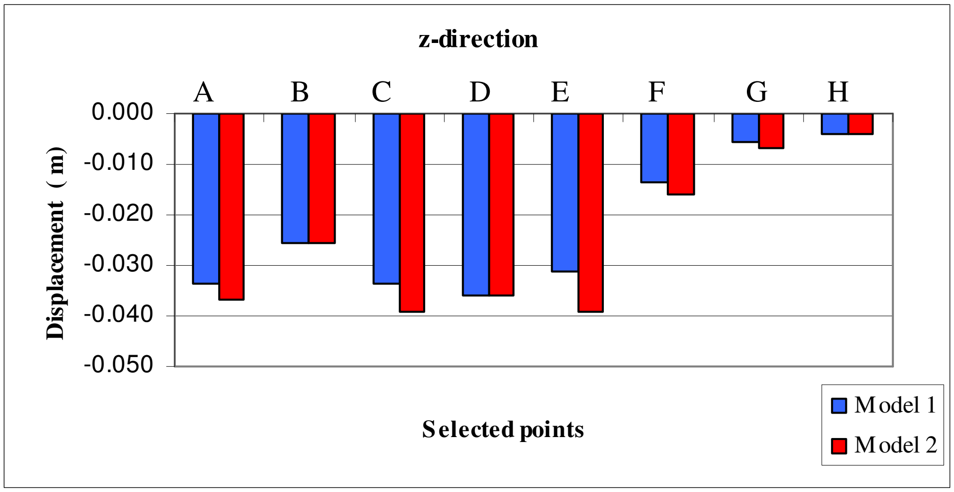


Figure 6.20 Displacements of critical points at z- direction

Table 6.5 Exact displacement values of the selected points for the first analysis

X-direction

	A	B	C	D	E	F	G	H
Model 1	0.00015	0.007248	0.002203	0.000936	-0.00238	0.009722	0.027804	-0.027185
Model 2	-0.012554	-0.002278	-0.008471	-0.007599	-0.010475	0.00449	0.026668	-0.035376

Y-direction

	A	B	C	D	E	F	G	H
Model 1	0.000153	0.009427	-0.000611	0.000227	-0.000074	-0.011993	-0.028162	0.02662
Model 2	-0.012502	0.000226	-0.010274	-0.00958	-0.00703	-0.021484	-0.035742	0.02506

Z-direction

	A	B	C	D	E	F	G	H
Model 1	-0.03351	-0.025713	-0.033788	-0.035897	-0.031071	-0.013656	-0.005725	-0.00386
Model 2	-0.036871	-0.025594	-0.039099	-0.036122	-0.039114	-0.015924	-0.006801	-0.004131

The same investigations are conducted for the second analysis under response spectrum analysis applied by using the spectrum from Turkish Seismic Code, zone 1. It is deducted from the results that the stress concentrations and displacements are observed in approximately same locations of gravity analysis with different values.

In Figure 6.21, the contour maps of Model 1 and Model 2 are seen in S33 direction on their deformed shape. The inclination in the same direction is observed for two models with similar high stress concentrations which are more comprehensible in Figure 6.22. The observations from the two figures are investigated further with the stress variations on the neighboring pillar of the steel pillar. Figure 6.23 has stress variations of the inner and the outer corner of the same pillar (pillar 2) shown in Figure 6.12. A divergence in the curve is recognizable after 5 m in the inner corner which may result from the stiffness and the mass change in the Model 2. However, the curves show similar trends along the height. In the case of the outer corner, the effect of the steel pillar is clearly seen. For the first 7 m the stress variation is completely different while after that height similar trend with some offset is noticed. The weight difference between the two models may be responsible from the difference since the weight is included in the calculation of the equivalent seismic load applied laterally on the structure. The exact S33 values along the height of the inner and the outer corner are given in Table 6.6.

Table 6.6 S33 values of inner and the outer corner of the pillar

INNER CORNER			OUTER CORNER		
Height	**MODEL 1**	**MODEL 2**	**Height**	**MODEL 1**	**MODEL 2**
19	4609.32	5558.23	18	6860.07	6613.66
17.75	11348.02	4437.66	16.5	6850.09	4436.98
16.5	7447.89	10882.94	14.25	4623.58	1621.72
14.25	6218.86	6978.66	12	1732.79	1174.11
12	3273.19	5895.35	10	1121.28	3848.28
10	410.9	3023.9	8	3868.88	6512.43
8	2325.74	292.14	6	6601.2	9343.2
6	5078.23	2448.22	4	9505.36	9343.2
4	7933.34	7913.3	2	12025.08	1180.39
2	10550.88	10460.13	0	15297.27	14992.36
0	13719.85	13556.98			

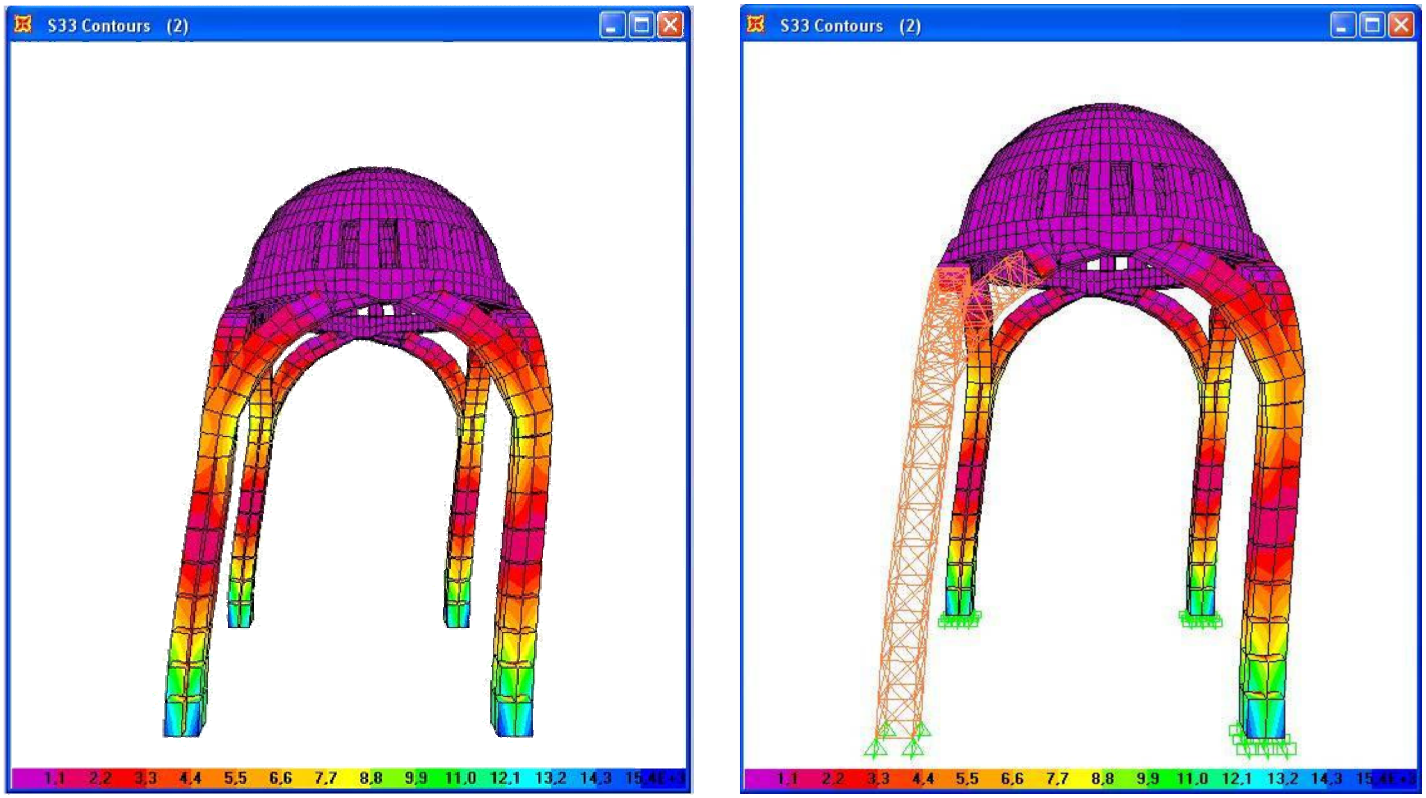


Figure 6.21 The stress,S33, distribution of the Model 1 and Model 2 under lateral loading (side view)

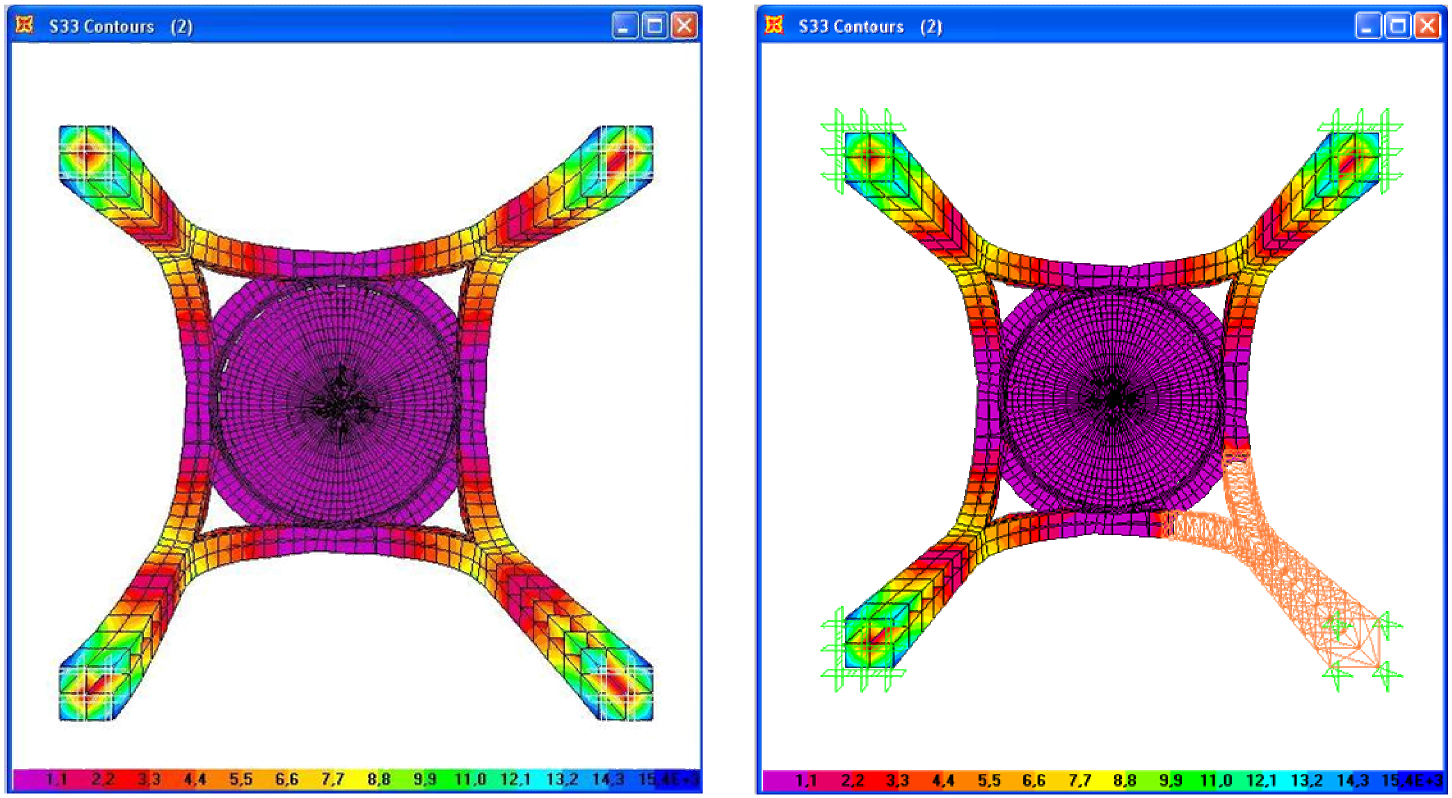
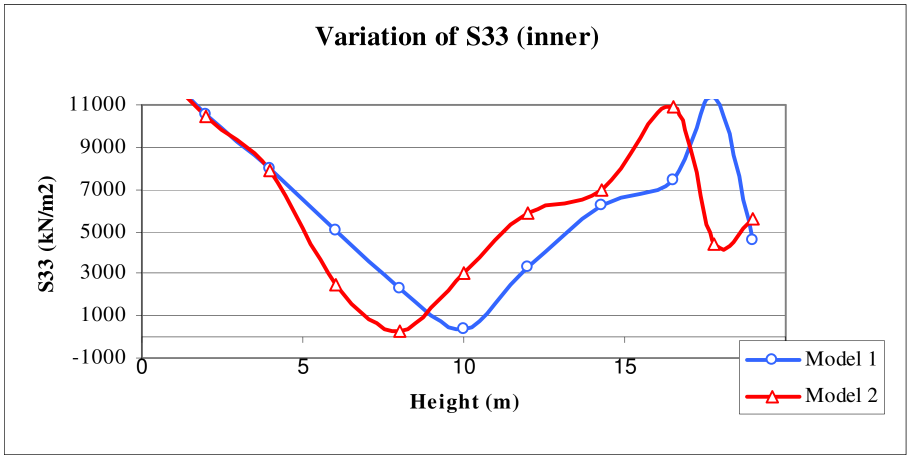


Figure 6.22. The stress,S33, distribution of the Model 1 and Model 2 under lateral loading (plan view)



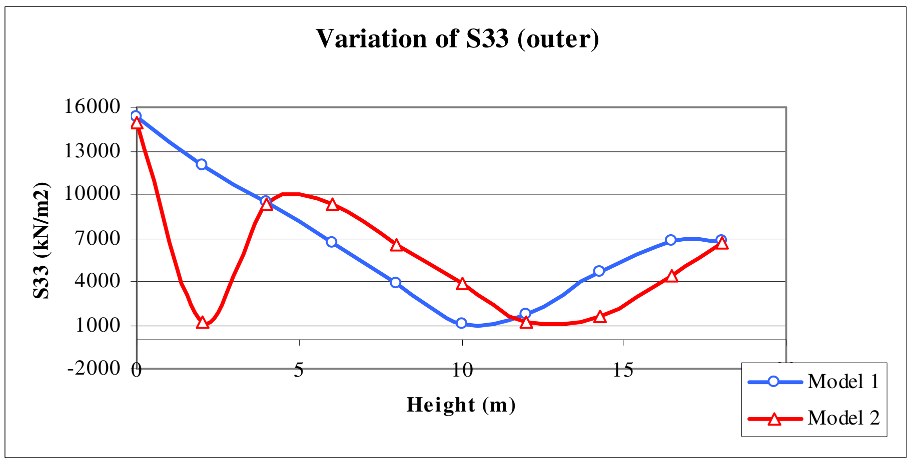


Figure 6.23 Stress variations along the pillar in vertical direction

A new three points are used for the comparison of the stress values on locations of maximum stress for the second analysis. Figure 6.24 shows the stress check points for the models. The results are presented both in tabular and graphical form in Figure 6.25 and Table 6.7. The results reveal that a consistent decrease takes place in the maximum stress points in the models.

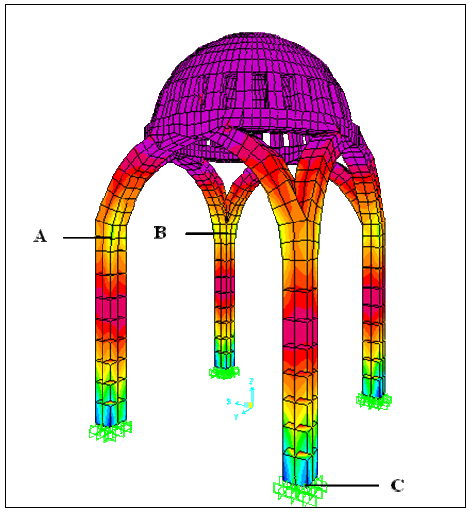


Figure 6.24 Maximum stress points under lateral loading in vertical direction

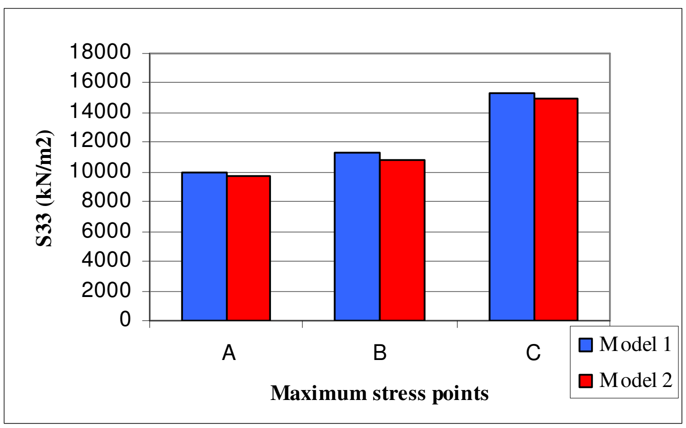


Figure 6.25 Stress comparison of the models on points A, B and C.

Table 6.7 Stress values of the models at A,B and C

	A	**B**	**C**
Model 1	9971.93	11348.22	15297.27
Model 2	9703.8	10882.94	15010.83

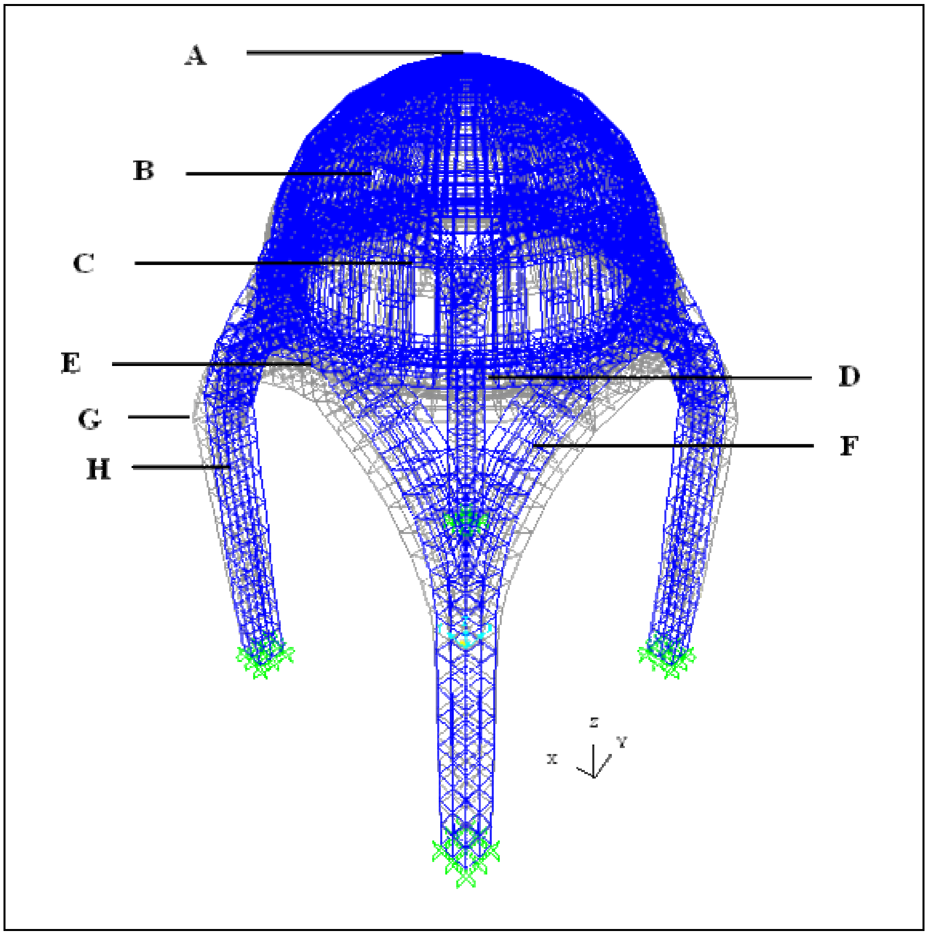
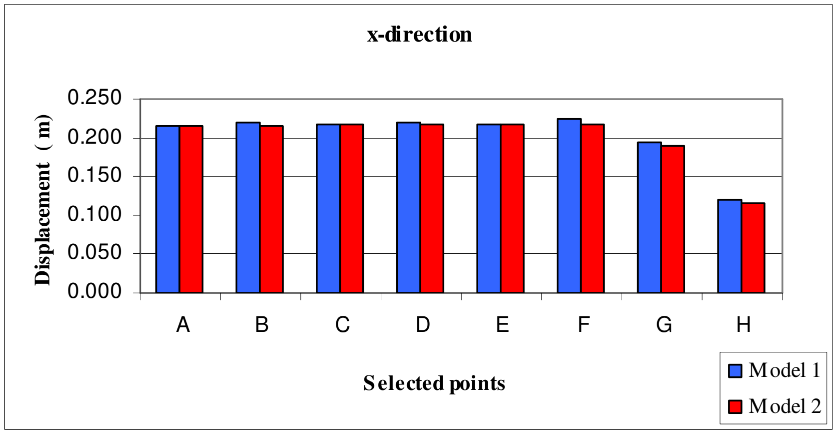
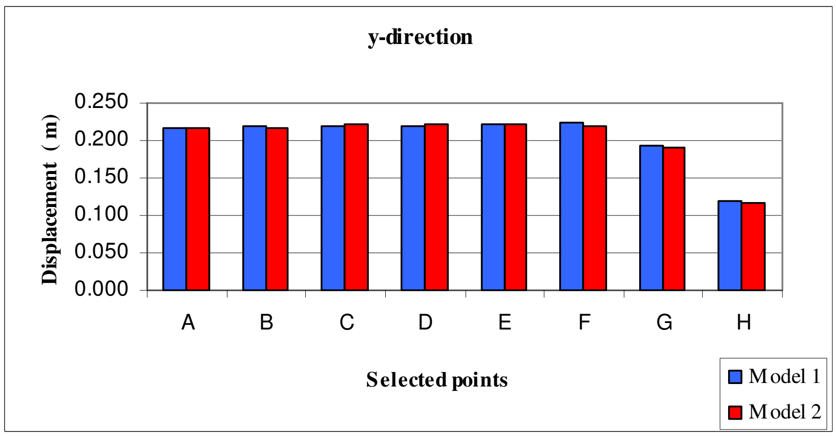


Figure 6.26 The points where the maximum deformations occur in the structural elements for the second analysis case

The displacements on specified points are shown in the Figure 6.26. The displacement of the points of Model 2 in x-direction has very slight difference with that of the Model 2. The same trend is observed for the displacements in y-direction. As in the case of z-direction, higher displacements are seen for the points on the dome, the drum, the window and the middle of the arch (A, B, C, D and E) while very close displacements are observed for the points on pillar and the arch. (Figure 6.27 and Table 6.8)





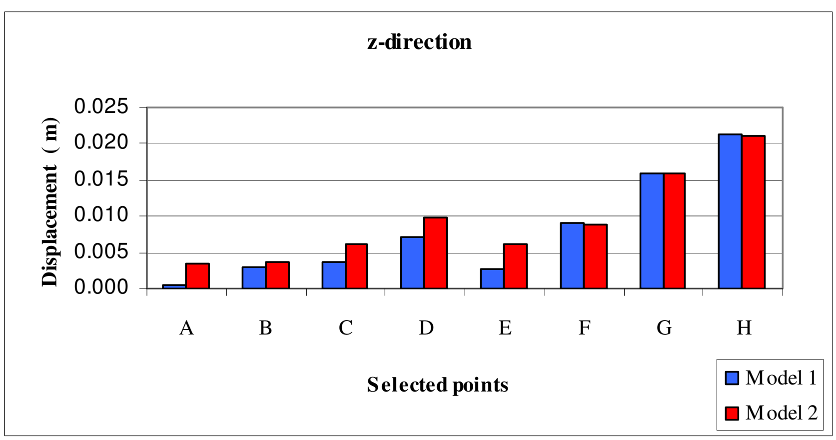


Figure 6.27 Displacements of Model 1 and Model 2 at the critical points at x, y and z direction for the second analysis

Table 6.8 Exact displacement values of the selected points for the second analysis

X-direction

	A	B	C	D	E	F	G	H
Model 1	0.216094	0.218967	0.21872	0.220091	0.217108	0.223515	0.193773	0.120184
Model 2	0.215696	0.215093	0.218344	0.217182	0.217532	0.218303	0.188705	0.11688

Y-direction

	A	B	C	D	E	F	G	H
Model 1	0.216416	0.218412	0.219083	0.219333	0.22046	0.22265	0.193618	0.12008
Model 2	0.21577	0.217251	0.221433	0.220537	0.222252	0.218498	0.190206	0.116963

Z-direction

	A	B	C	D	E	F	G	H
Model 1	0.000436	0.002822	0.003667	0.007026	0.002644	0.009181	0.016035	0.021387
Model 2	0.003338	0.003748	0.006089	0.009736	0.006197	0.008833	0.015933	0.021052

CHAPTER 7

CONCLUDING REMARKS

7.1 Conclusions

Historical masonry constructions constitute significant part of the architectural heritage which acts as a link between the past and the present. The concept of identity is generally attributed to the historical monuments. The conservation of the present condition or rehabilitation of the deficiencies in the monuments gains importance as the reflection of this common sense of identity. The time, the destructive effects of the nature and man-made factors are the main sources of damage to prevent which some measures of conservation are employed.

Condition assessment study is conducted before any intervention on historical monuments which includes acquisition of the data, survey of the structures, field research and laboratory testing, and safety evaluation. Analyses conducted on the representative models constitute the backbone of the assessment study in which the structural behavior is tried to be estimated. Structural analysis of historical monuments is a demanding task due to material characteristics of masonry, missing geometric data, existence of the damages and inapplicability of the codes and regulations. Analytic modeling techniques are used to simulate the historical masonry structures and to conduct the structural analysis. Of these, finite element method is the most powerful and suitable tool for the analysis of historical constructions with its salient features in reflecting the properties of the actual structure. The structure is represented by finite number of mesh elements in which the accuracy increases with the increasing number of the elements

Depending on the decision based on the assessment study, different levels of interventions are suggested in which the respect to historical fabric decrease with increasing degree of intervention. Within this framework, the concept of reversibility is taken into account which defines the capability of an intervention to turn back to original untouched state in a way balancing the safety level and the authenticity of the structure. Prestressing and strengthening by fiber reinforced fibers are two promising applications which is being used in a variety of situations.

In this study a strengthening method with steel skeleton which strengthens the structure as well as completing the form and appereance is proposed for partially collapsed historical masonry structures. Since in most of the partially collapsed structures the form is not readable from the remaining parts and the structures are vulnerable to any excitation. The use of steel in strengthening applications is not new and ties at the abutments of arches, rings at the perimeter of the domes, confinement of structural elements by prestressed ties, structural repointing, steel stitching and miscellenaous applications can be listed as the examples of this use. However, the method proposed suggests a different approach in using the steel. During the study, the effect of the strengthening method in reducing the stresses or deformations is not taken into account since the aim is to compare the behavior of intact model and the strengthened model.

For the evaluation of the validity of the method and the efficiency of the method in reflecting the initial intact condition, a conceptual 3D masonry model, structurally and formly similar to Mihrimah Sultan Mosque of the Masterbuilder Sinan, is generated in finite element platform. Then a second model is developed by removing one of the pillar and replacing it with steel skeleton composed of different steel sections. The connection is supported by extra steel elements to assure the continuity of the rigidity along the intersection. The two models are compared under same analysis cases (gravity and response spectrum analysis) in terms of structural behaviors. In the comparison the following parameters are taken into account :

1. Modal periods and deformed shapes
2. Overall stress distribution
3. Stress variation along a pillar
4. Stress values at locations of maximum stresses of the intact model
5. Displacement values at locations of maximum deformationof the intact model

The major observations and corresponding conclusions drawn from the analyses results are summarized as follows:

- The modal period values of the two model- the intact model and the strengthened model- are almost the same. The difference is in the order of 0.01s. The modal period, or natural frequency are one of the dynamic properties of the structures which in a way controls the response to a dynamic load. Therefore, same periods can be interpreted as more or less same response under an excitation.

- First mode deformed shapes under gravity loading of the models exhibit differences because of the replacement of one pillar by steel skeleton. However, second and third modes of two models show similar trends.

- The overall stress distribution of models along the vertical direction have the same distribution in general sense under gravity loading. The locations of maximum stresses, tension and compression concentration zones are the same in two models. Similar to the gravity analysis, in the response spectrum analysis the distribution in general is the same with varying values in some locations.In both analysis, the introduction of steel skeleton to the model does not affect the upper structure drastically, i.e., the drum and the dome.

- The stress variation in vertical direction, S33, maximum principle stresses,S_{max} , and minumum principle stresses, S_{min} along the inner and outer corners of a pillar neighboring the replaced one between the intact model and the strengthened exhibits same trends with minor divergences between certain height values in the gravity analysis. On the other hand, there observed differences in variation in which the trends are similar after a certain height in the results of the response spectrum analysis. The stiffness and the mass dissimilarity between the models can be proposed as the reason for the distinct behaviors.

- The numeric values of vertical stresses at the locations of maximum stresses reveals that there occured an increase in the order of 1 MPa between two models where the strengthened model has higher values for the gravity analysis case. Nevertheless, for the second analysis case the decrease in the stress values of the points in the strengthened model is noted.

- When the displacements at the locations of maximum deformation of the intact model, in the directions perpendicular to vertical (x and y directions) difference in direction (+ or -) is observed, albeit very small in the gravity analysis. As for vertical direction (z-direction) the displacements are either equal in two models or bigger in the order of mm in the strengthened model. In the case of response spectrum analysis, the displacements are almost equal in the majority of the points in horizontal directions. Since the loading is in horizontal direction, the closeness of the displacements are of importance. However, in the vertical, an increase in the displacement of the strengthened model is observed for the points on the dome, drum, window and arch elements while no recognizable change is noted for the points on the lower arch and pillar elements. The change may result from the different weight of the two models and the stiffness of the steel.

- The extra diagonal elements in the connections between the masonry and the steel frame are observed to be effective in providing the necessary rigidity in the connections since there occured no hinging and abrupt deformation in the system at that points.

The analyses results show that the method applied for the strengthening of the partially collapsed historical masonry structures is proved to be effective in reflecting the structural behavior of the untouched original structure.

7.2 Recommendations for Further Studies

In this study the applicability and the efficiency of the method is studied in a general sense. Its contribution to load carrying system and its effectiveness in the reduction of the stresses and deformations are not covered within the scope of the study.

For further studies, the model used for analysis can be enhanced by the inclusion of the nonlinear material characteristics as well as by the increase of the number of finite elements. Moreover, the behavior of the models beyond the elastic range, that is, nonlinear range can be investigated to obtain more precise results by nonlinear static and dynamic response history analyses. The observation of the bahavior under earthquakes with different magnitudes, frequency content and duration may give more detailed results about the vulnerability of the system and the effectiveness of the method. The parts where the masonry and the steel are connected can be investigated in a detailed way.

Finally, experimental set-up for the proposed model can be developed for the validation of the model in a more realistic way.

REFERENCES

1. Sevgili,G., Küçükdoğan, B., and Karaesmen,E., *Old Masonry in Seismic Zones*, Earthquake Engineering: Essentials and Applications, Earthquake Engineering Research Center, METU, July 18-20, 2005

2. Bowyer, J., *History of Building*, Crosby Lockwood Staples, London. 1973

3. Ünay A.İ., *A Method for the Evaluation of the Ultimate Safety of Historical Masonry Structures*, Ph.D. Dissertation, METU, Ankara, 1997

4. Şener, İ.N., *An Innovative Methodology and Structural Analysis for Relocation of Historical Masonry Monuments: A Case Study in Hasankeyf*, M.S. Dissertation, METU, Ankara, 2004

5. Croci, G., *The Conservation and Structural Restoration of Architectural Heritage*, Computational Mechanics Publications, Southampton, 2002

6. Vitruvius, *Mimarlık Üzerine On Kitap; The Ten Books on Architecture*, English Translation by Morris Hicky Morgan, Turkish Translation by Suna Güven, Şevki Vanlı Mimarlık Vakfı Yayınları, İstanbul, 1998.

7. Erdoğan T., *Materials of Construction*, ODTÜ Geliştirme Vakfı Yayıncılık ve İletişim A.Ş., Ankara, 2002

8. British Geological Survey Natural Environment Research Council, *Brick Clay*, Mineral Planning Fact Sheet, November, 2005

9. Dimes, F.G., *The Nature of Building and Decorative Stones*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume 1, p.19-36, Butterworth-Heinemann, 1990

10. Dimes, F.G., *Igneous Rocks*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume 1, p.37-60, Butterworth-Heinemann, 1990

11. Wicander,R., and Monroe, J.S., *Essentials of Geology*, Wadsworth Publishing Company, California, 1998

12. Dimes, F.G., *Sedimentary Rocks*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume 1, p.61-134, Butterworth-Heinemann, 1990

13. Dimes, F.G., *Metamorphic Rocks*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume 1, p.135-149, Butterworth-Heinemann, 1990

14. Moropoulou, A., Biscontin,G.,Bakolas, A., Micjailidis,P. and Basiotis,J., *Historic Mortars in Mediterranean Monuments*, Proceedings of the 4th International Symposium on the Conservation of Monuments in the Mediterranean, Technical Chamber of Greece, 1997

15. Winkler, E.M., *Stone in Architecture: Properties, Durability*, Springer, New York, 1997

16. Corradi, M., Tedeschi, C., Binda, L. and Borri, A., *Experimental Evaluation of Shear and Compression Strength of Masonry Wall Before and After Reinforcement: Deep Repointing*, Construction and Building Materials (in Press), 2007

17. Page, A.W., *An Experimental Investigation of the Biaxial Strength of Brick Masonry*, Proceedings of the Sixth International Brick Masonry Conference, Rome, 1982

18. Oliveira, D.V., *Mechanical Characterization of Stone and Brick Masonry*, University of Minho, Department of Civil Engineering, Report 0- DEC /E-4, 2000

19. Marzahn, G., *The Shear Strength of Dry Stacked Masonry Walls*, Retrieved on March 13, 2007, from http://aspdin.wifa.uni-leipzig.de /institut/lacer/lacer03/ l03_24.pdf

20. Croci G., *Strengthening of monuments under the effect of static loads, soil settlement and seismic actions-Examples*, Proceedings, 3th International Seminar on Historical Constructions 2001. Paulo Lourenço and Pere Roca (eds.), pp. 1167-1190. Guimarães, 2001

21. Mills, R., *Structural Failure and Repair*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume 2, p.55-70, Butterworth-Heinemann, 1990

22. Beckmann, P., *Structural Aspects of Building Conservation*, McGraw-Hill Book Company Europe, England, 1994

23. Karaesmen, En., and Karaesmen,E., *A Discussion on Some Concepts of the Archeoseismology*, Seminar Notes of METU Faculty of Natural Sciences Department of Archaeometry Seminar Series, November, 1998

24. Balderrama, A.A., *Earthquake Damage to Historic Masonry Structures*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume 2, p.107-113, Butterworth-Heinemann, 1990

25. Anzani, A., Binda, L. and Mirabella Roberti, G., *The Effect of Heavy Persistent Actions into the Behavior of Ancient Masonry*, Materials and Structures, Volume 33,no.4, p.251-261, 2000

26. Ferretti, D., and Bazânt,Z.P., *Stability of Ancient Masonry Towers: Stress Redistribution due to Drying, Carbonation and Creep*, Cement and Concrete Research, Volume 36, p.1389-1398, 2006

27. Ersoy, U., Özcebe,G. and Tankut,T., *Reinforced Concrete*, Middle East Technical University Ankara, 2003

28. Pina-Henriques, J. and Lourenço, P.B., *Testing and Modelling of Masonry Creep and Damage in Uniaxial Compression*, Retrieved on March 12, 2007, from http://www.civil.uminho.pt/masonry/Publications/Update_Webpage/2003_Pina_Lourenco.pdf

29. Honeyborne, D.B., *Weathering and Decay of Masonry*, Conservation of Building and Decorative Stone, J.Ashurst and F.G.Dimes (Eds.),Volume1, p. 155-178, Butterworth-Heinemann, 1990

30. Snethlage, R. and Wendler, E., *Moisture Cycles and Sandstone Degradation*, Dahlem Workshop on Saving Our Architectural Heritage: Conservation of Historic Stone Structures, N.S.Baer and R.Snethlage (Eds.), Hohn Wiley&Sons Ltd, England, 1997

31. Torraca, G., *Air Pollution and the Conservation of Building Materials*, Air Pollution and Conservation Safeguarding Our Architectural Heritage, J.Rosvall and S. Aleby (Eds.), p. 199-208, Elsevier Science Publishers B.V, The Netherlands, 1988

32. Fassina, V., *Environmental Pollution in Relation to Stone Decay*, Air Pollution and Conservation Safeguarding Our Architectural Heritage, J.Rosvall and S. Aleby (Eds.), p. 133-174, Elsevier Science Publishers B.V, The Netherlands, 1988

33.Lorusso, S., Troili, M., Buonocore, F. and Marabelli, M., *Air Pollution and Historic Monuments*, Proceedings of 4th International Symposium on the Conservation of Monuments in the Mediterranean, Ed. Technical Chamber of Greece, Athens, Vol 2, pp. 153-160, 1997

34. Grossi, C.M., Brimblecombe, P., and Harris, I., *Predicting Long Term Freeze-thaw Risks on Europe Built Heritage and Archaeological Sites in a Changing Climate*, Science of the Total Environment, Volume 377, Issues 2-3, p.273-281, 2007

35. Scherer, G.W., *Internal Stress and Cracking in Stone Masonry*, Fracture of Nano and Engineering Materials and Structures - Proceedings of the 16th

European Conference of Fracture, Alexandropoulis - Greece, July 3-7, CD-Rom, 2006

36. *Photo of Decayed Masonry*, Retrieved on April 5, 2007 from http://books.nap.edu/books/0309096251/html/images/p2000d421g73001.jpg

37. Warscheid, T., Braams,J., *Biodeterioration of Stone: A Review*, International Biodeterioration & Biodegradation, Volume 46, p.343-368, 2000

38. Zanardini,E.,Abbruscato, P., Ghedini,N., Realini,M. and Sorlini, C., *Influence of Atmospheric Pollutants on the Biodeterioration of Stone*, International Biodeterioration & Biodegradation, Volume 45, p.35-45,2000

39. Nuhoglu,Y., Oguz, E., Uslu, H., Ozbeh,A., Ipekoglu, B., Ocak, I., and Hasenekoglu, I., *The Accelerating Effects of the Microorganisms on Biodeteriration of Stone Monuments Under Air Pollution and Continental-cold Climatic Conditions in Erzurum Turkey*, Science of the Total Environment, Volume 364, p.272-283, 2006

40. *Photo of Decayed Masonry*, Retrieved on April 15, 2007, from http://www.healthy-homes.co.uk/Dryrot.jpg

41. ICOMOS, *Recommendation for Analysis, Conservation And Structural Restoration of Architectural Heritage*, 2003

42. Lourenço, P.B., *Analysis of Historical Constructions : From Thrust Line to Advanced Simulations*, Proceedings of Historical Constructions, P.B.Lourenço, and P.Roca (Eds.) 91-116, Guimaraes, 2001

43. Ünay, A.İ., *Tarihi Yapıların Depreme Dayanımı*, ODTÜ Mimarlık Fakültesi Basım İşliği, Ankara, Türkiye, 2002

44. Feilden.B.M., *Conservation of Historic Buildings*, Butterworth & Co. Ltd, England, 1989

45. Karaesmen,E., Ünay,A.İ., *A Study of Structural Aspects of Domed Buildings With Emphasis on Sinan's Mosques*, Proceedings of IASS, Domes from Antiquity to the Present, MSU Symposium, Istanbul, 1988

46. MacLeod, I.A., *Analytical Modeling of Structural Systems, An Entirely New Approach With Emphasis on the Behavior of Building Structures*, Ellis Horwood Limited, Southampton, 1990

47. Orduna, A., Lourenço, P.B., *Limit Analysis as a Tool for the Simplified Assessment of Ancient Masonry Structures*, Proceedings of Historical Constructions, P.B.Lourenço, and P.Roca (Eds.), p.511-520, Guimares, 2001

48. Lourenço, P.B., Zucchini A., Milani G. and Tralli A., *Homogenization Approaches for Structural Analysis of Masonry Buildings*, Structural analysis of Historical Constructions, New Delhi, 2006

49. Gilbert,M., and Melbourne, C., *Rigid-Block Analysis of Masonry Structures*, The Structural Engineer, Volume 72(21), p. 356-361,1994

50. Lourenço, P.B., *Historical structures: Models and modelling*, EPMESC VII: Computational Methods in Engineering Science, Eds. J. Bento et al., Elsevier, Amsterdam, Países Baixos, p. 433-442, 1999

51. Cook, R.D., *Concepts and Applications of Finite Element Analysis*, Wiley and Sons, 1974

52. Carlton, D., *Application of the Finite Element Method to Structural Engineering Problems*, The Structural Engineer, Volume 71(4), p.55-59, 1993

53. Toker,S., and Ünay, A.İ., *Mathematical Modeling and Finite Element Analysis of Masonry Arch Bridges*, G.U. Journal of Science, Volume 17(2), p.129-139, 2004

54. *Definition of Vulnerability,* Retrieved, April 18, 2007, from http://en.wikipedia.org/wiki/Vulnerability

55. Penelis, G.G., *Structural Restoration of Historical Buildings in Seismic Areas*, Progress in Structural Engineering Materials, Vol.4, 64-73, 2002

56. Avramidou, N., *Vulnerability of Cultural Heritage to Hazards and Prevention Measures*, Proceeding of Advanced Research Workshops 4, ARIADNE-4, Czechoslovakia, August 18-24, 2001

57. Örmecioğlu, H.T. and Ünay, A.İ., *Seismic Strengthening of Historical Structures by Enfolding Steel Skeleton*, 5th International Conference on Behavior of Steel Structures in Seismic areas (Stessa2006), 14th-17th August 2006, Vol.1, pg. 835-841, Yokohama, Japan, 2006

58. Kokkinos, Ch., Moropoulou, A., Nicolacopoulos, P., *A Conceptual Approach to the Conservation of Cultural Heritage*, Proceedings of 4th International Symposium on the Conservation of Monuments in the Mediterranean, Ed. Technical Chamber of Greece, Athens, Vol 2, pp. 427-436, 1997

59. ICOMOS, *Athens Charter for the Restoration of Historic Monuments*, Retrieved, February 12, 2007, from http://www.icomos.org/athens_charter.html

60. ICOMOS, *International Charter for the Conservation and Restoration of Monuments and Sites (The Venice Charter, 1964)*, Retrieved, February 12, 2007, from http://www.international.icomos.org/charters/ venice_e.htm

61. Oberlander, J., Kalman, H., and Lemon, R., *Principles of Heritage Conservation*, British Columbia Heritage Trust,Mary McKinnon(Ed.), Technical Paper Series; 9 ,Victoria , 1989

62. *Definition of Condition Assessment*, Retrieved, April 09, 2007, from http://www.armstrongassoc.net/terms.html

63. Croci, G., *General Approach to the Protection and Restoration of Historic Buildings and Assessment of the Effects of Earthquakes on Buildings*, Proceedings of Protection of Architectural Heritage Against Earthquake, 28-09.-2.10.1992, 47-86, Istanbul-Ankara, 1992

64. Syrmakezis, C.A., *Tentative Guidelines for Protection and Rehabilitation of Historical Buildings Subjected to Earthquakes*, , V.Petrini and M.Save (Eds), CISM Courses and Lectures no.359, p.295-322, Udine, 1996

65. Moropoulou, A., Koui, M., Kourteli, Ch., Papasotiriou, D., Theoulakis, P., Tsiourva, Th., Achilleopoulos, N., Karakandas, Ch., Romanos, A., Tokatlidou, K., Kolliadis, Th., Zarifis, N., Van Grieken, R., Delalieux, F., Silva, B., Molina, E., Vicente, M. A., Vicente,S., Zerra, F., and Macri, F., *Techniques and Methodology for the Preservation and Environmental Management of Historic Complexes – The Case of the Medieval City of Rhodes*, Proceedings of the 4th International Symposium on the Conservation of Monuments in the Mediterranean, Technical Chamber of Greece, Vol.4, 603-630, 1997

66. Pinto, A., Molina, J., Pegon, P., Renda, V., *Protection of the Cultural Heritage at ELSA Laboratory*, Proceedings of Historical Constructions, p . 973-982, 2001

67. Berger, F., *Assessment of Old Masonry by Means of Partially Destructive Methods*, Structural Repair and Maintenance of Historical Buildings, C.A. Brebbia (Ed), Computational Mechanics Publications, Basel, p. 103-117, 1989

68. Kingsley,G.R., Livingston,R.A., Taylor, T.H., and Noland, J., *Nondestructive Evaluation of a 19th Century Smokehouse in Colonial Williamsburg, Virginia*, Structural Repair and Maintenance of Historical Buildings, C.A. Brebbia (Ed), Computational Mechanics Publications, Basel, p. 149-158 1989

69. Exadaktylos, G.E., Papadopoulos, Ch. Th., Stavropoulou, M.Ch. and Athanassiadou,A., *In Lab and In situ Assessment of Masonry Stones' Mechanical Properties Through Micro-drilling Technique*, Proceedings of 8th International Conference on Structural Studies, Repairs and Maintenance of Heritage Architecture, C.A.Brebbia (Ed), p. 173-182, WIT Press, Southampton, 2003

70. Nappi, A., and Cote,P., *Nondestructive Test Methods Applicable to Historic Stone Structures*, Report of the Dahlem Workshop on Saving Our Architectural Heritage : The Conservation of Historic Stone Structures, Berlin, 3 – 8, 1996 N.S.Baer and R.Snethlage (Eds), p. 151-166, John Wiley & Sons, Chichester, 1996

71. Zanzi, L.,Saisi, A., Binda, L., and Cardarelli, E., *Sonic Tomography of the Stone Pillars of a 17th century Church*, Proceedings of 7th International Conference on Structural Studies, Repairs and Maintenance of Heritage Architecture, C.A.Brebbia (Ed), p. 339-348, WIT Press, Southampton, 2001

72. Beconcini M.L., BEnnati, S. and Salvatore, W., *Tensile and Low-cyle Compressive Tests on a Natural Structural Stone*, Proceedings of 7th International Conference on Structural Studies, Repairs and Maintenance of Heritage Architecture, C.A.Brebbia (Ed), p. 483-496, WIT Press, Southampton, 2001

73. Baker, I.O., *A Treatise on Masonry Construction*, John Wiley & Sons, Inc, New York, 1909

74. Penelis, G.G., *Techniques, Materials and Design in Structural Restoration*, Proceedings of the 4th International Symposium on the Conservation of Monuments in the Mediterranean, Technical Chamber of Greece,Vol.3, 281-294, 1997

75. VSL, *VSL International, Post-tensioned Masonry Structures*, Retrieved on April 12, 2007, from http://www.vsl.net/downloads/ VSL_technical_reports/ PT_Masonry_ Structures.pdf

76. Post tension masonry structures, Retrieved April 12, 2007, from http://www.masonrymagazine.com/12-04/ptm.html

77. Foti D. and Monaco, P., *Post-tensioned masonry: State of Art*, Progress in Structural Engineering Materials, Volume 2, p.311-318, 2000

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

79. . Valluzi M.R., and Modena,C., *Experimental Analysis and Modeling of Masonry Vaults Strengthened by FRP*, Proceedings of Historical Constructions, P.B.Lourenço, P.Roca (Eds.), p. 627-635, Guimaraes, 2001

80. Shrive, N.G., Masia, M.J. and Lissel, S.L., *Strengthening and Rehabilitation of Masonry Using Fibre-reinforced Polymers*, Proceedings of Historical Constructions, P.B.Lourenço, P.Roca (Eds.), p.1047-1056, Guimaraes, 2001

81. Turco, V., Galati, N., Lorenzis, L., Modena, C,. and Nanni, A., *Bond Between Near Surface Mounted FRP Rods and Masonry in Structural Strengthening*, Advancing with Composites, Plast 2003, Ed. I. Crivelli-Visconti, Milan, May 7-9, 2003, Milan, Italy, pp. 209-217, 2003

82. Tumialan, G., Galati, n., Namboorimadathil, S.M., and Nanni, A., *Strengthening of Masonry with FRP Bars*, Proceedings of ICCI 2002, San Francisco, CA, June 10-12, 2002

83. Balsamo,A., Battista, U., Herzalla, A. and Viskovic,A., *The use of Aramidic Fibres to Improve the Structural Behavior of Masonry Structures Under Seismic Actions*, Proceedings of Archii 2000, Bethlehem, Israel ,September 10-2, 2001

84. Triantafillou, T.C., *Strengthening of structures with advanced FRPs*, Progress in Structural Engineering and Materials, Volume 1, Issue 2 , Pages 126 - 134, 2005

85. Shrive, N.G., *The Use of Fibre Reinforced Polymer to Improve Seismic Resistance of Masonry*, Construction and Building Materials, 20 , p. 269-277, 2006

86. Albert, M.L., Elwi, A.E. and Cheng, J.J.R., *Strengthening of Unreinforced Masonry Walls using FRPs*, Journal of Composites for Construction, p.76-84, 2001

87. Galati, N, Garbin,E., and Nanni, A.*, Design Guidelines for the Strengthening of Unreinforced Masonry Structures Using Fiber Reinforced Polymers (FRP) Systems*, Final Draft Report for BONDO and TECHFAB, University of Missouri-Rolla , 3\3\2005

88. Cruz, H., Moura, J.P., and Machado, J.S., *The Use of FRP in the Strengthening of Timber-reinforced Masonry Load-bearing Walls*, Proceedings of Historical Constructions, P.B.Lourenço, P.Roca (Eds.), p.847-856, Guimaraes, 2001

89. . Valluzzi M.R.1; Tinazzi D.; Modena C., *Shear Behavior of Masonry Panels Strengthened by FRP Laminates*, Construction and Building Materials, Volume 16, Number 7, October 2002, pp. 409-416(8)

90. Tinazzi, D., C. Modena, and A. Nanni, *Strengthening of Masonry Assemblages with FRP Rods and Laminates*, International Meeting on Composite Materials, PLAST 2000, Proceedings, Advancing with Composites 2000, Ed. I.Crivelli-Visconti, Milan, Italy, May 9-11, 2000, pp. 411-418.

91. Tumialan, J.G., Morbin, A., Micelli, F., and Nanni, A., *Flexural Strengthening of URM Walls with FRP Laminates*, Third International Conference on Composites in Infrastructure (ICCI 2002), San Francisco, CA, June 10-12, 2002

92. *Photograph of Selimiye Mosque in Edirne*, Retrived, June 17, 2007, from http://www.edirneden. com/getresim.php?id=337

93. *Photograph of the Rotunda in Thessaloniki*, Retrived, June 17, 2007, from http://img2.travelblog.org/Photos/1/6686/t/25147-The-Rotunda-0.jpg

94. Valluzi, M.R., Porto, F. and Modena, C., *Structural Investigations and Strengthenıng of the Civic Tower in Vicenza,* Retrived, June 17, 2007, from http://www.onsiteformasonry.bam.de/Results/papers/SFR2003.pdf

95. Valluzi, Binda L. And Modena, C., *Mechanical Behaviour of Historic Masonry Structures Strengthened by Bed Joints Structural Repointing*, Construction and Building Materials, Volume 19, Issue 1, p. 63-73, 2005

96. City University London, *Repair and Strengthening of Brick/ Block Masonry,* Retrived, June 18, 2007, from http://www.staff.city.ac.uk/earthquakes/ Repairstrengthening

97. *Photograph of steel strengthening*, Retrived, June 18, 2007, from http://www.kuleuven.ac.be/ bwk/materials/Teaching/master/wg16/l0100.htm

98. *Photograph of steel strengthening*, Retrived, June 18, 2007, from http://www. hughesbros.com /Aslan100/Aslan%20GFRP%20Masonry.html

99. Çulpan, C., *Türk Taş Köprüleri Ortaçağdan Osmanlı Devri Sonuna Kadar*, Türk Tarih Kurumu Basımevi, Ankara, 1975

100. *Photograph of Geyve II. Beyazıd Bridge, Retrived*, June 18, 2007, from http://www.kenthaber.com/Resimler/2006/04/14/00152352.jpg retrieved June 19, 2007

101. Özen, G.Ö., *Comparison of Elastic and Inelastic Behavior of Historical Masonry Structures at the Low Load Levels,* M.S. Dissertation, METU, Ankara, 2006