İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

ANALYSIS, REPAIR AND STRENGTHENING OF HISTORICAL MASONRY STRUCTURES; CASE STUDY: MEHMET AGA MOSQUE

M.Sc. Thesis by
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Department : **CIVIL ENGINEERING**

Programme: **EARTHQUAKE ENGINEERING**

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

TARİHİ YIĞMA YAPILARIN MODELLENMESİ, ANALİZİ VE GÜÇLENDİRİLMESİ: MEHMET AĞA CAMİİ ÖRNEĞİ

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Civil Engineer

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ABBREVIATIONS

Eurocode 6	: European Committee for Standardization, 2005
CH	: High Plasticity Clay
DBYYHY	: Deprem Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelik, 2007
SPT	: Standard Penetration Test
EQ2	: Increased TSC spectrum (50 years, % 2)
EQ10	: DBYYHY spectrum (50 years, % 10)
CQC	: Complete Quadratic Combination
FRP	: Fibre reinforced polymer
SW	: Southwest
NW	: Northwest
SE	: Southeast
NE	: Northeast

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SYMBOL LIST

E	: Elastic modulus
t_m	: The thickness of mortar
t_u	: The height of brick
E_m	: Elastic modulus of mortar
E_u	: Elastic modulus of unit
ρ	: Coefficient associated with the deficient bond between the unit and mortar
τ_{em}	: Shear strength of the masonry wall
τ_o	: Fracture strength of the wall
μ	: Friction coefficient
σ	: Vertical stress of the wall
f_k	: Characteristic compressive strength of the masonry
f_b	: Normalized mean compressive strength of the masonry unit
f_m	: Compressive strength of the mortar
K,α,β	: Various constants
A_o	: Effective ground acceleration coefficient
Z3	: Soil class
T_A,T_B	: Soil characteristic periods
R_a	: Seismic load reduction factor
I	: Building importance factor
n	: Live load participating factor
S (T)	: Spectral coefficient
T	: Natural period
f	: Natural frequency
υ	: Poison’s ratio
γ	: Unit weight
K	: Stiffness of foundation
c	: Soil cohesion coefficient
ϕ	: Shear friction angle of the soil

ANALYSIS, REPAIR AND STRENGTHENING OF HISTORICAL MASONRY STRUCTURES; CASE STUDY: MEHMET AGA MOSQUE

SUMMARY

Historical structures have important architectural and cultural values. Preservation of the historical structures is an essential issue. The construction types of the historical structures are generally masonry. The tensile strength of masonry is lower whereas, the compressive strength is higher. Therefore, the tensile stress locations in the structure are critical. Historical masonry structures consist of several components such as walls, arches, domes, vaults and pillars. Earthquake and settlement are main damage reasons for historical masonry structures. Repair and strengthening of an historical structure require specification. Intervention should be kept at a minimum level. There are several intervention methods for historical masonry structures. After giving a summary of the masonry material, components of historical masonry structures, damages, repair and strengthening principles; historical Mehmet Aga mosque is investigated in this research work. The mosque is located in Istanbul, which is one of the oldest cities in the world having many historical and cultural heritages. The mosque is located in the North Anatolian fault line. Several estimations show that a major earthquake would probably strike Istanbul in a short time period. For the research, existing damages on the mosque were determined by in situ survey. The mosque was modeled and analyzed considering its self-weight and two different earthquake loadings having 2% and 10% exceeding probability for 50 years. Stress concentrations, especially tensile stresses, were investigated and compared to the existing damages in order to determine the damage reasons. Moreover, some repair and strengthening interventions were recommended to prevent further damages.

TARİHİ YIĞMA YAPILARIN MODELLENMESİ, ANALİZİ VE GÜÇLENDİRİLMESİ: MEHMET AĞA CAMİİ ÖRNEĞİ

ÖZET

Tarihi yapılar büyük sanatsal ve kültürel öneme sahiptir. Tarihi yapıların korunması çok önemli bir konudur. Tarihi yapılar çoğunlukla yığma yapım tekniği ile inşa edilmişlerdir. Yığma yapıların basınç dayanımı göreceli olarak yüksek, çekme dayanımı düşüktür. Bu yüzden, yığma yapılarda çekme gerilmesi oluşan bölgelerin tesbit edilmesi önemlidir. Tarihi yığma yapılar, duvar, kubbe, kemer, tonoz ve sütun gibi elemenlardan oluşur. Deprem etkisi ve yapı zemininde oluşan oturmalar, tarihi yığma yapılarda iki temel hasar nedenidir. Tarihi yapıların onarım ve güçlendirilmesi çok özel bir konudur. Tarihi yapılara yapılacak olan müdahaleler mümkün olduğu ölçüde minimum düzeyde tutulmalıdır. Tarihi yapıların onarım ve güçlendirilmesinde birçok yöntem vardır. Bu çalışmada, tarihi yığma yapıların malzeme özellikleri, elemanları, bu yapılarda oluşan hasarlar ve bunların onarım ve güçlendirilmesi hakkında özet bilgi verildikten sonra, tarihi Mehmet Ağa Camii incelenmiştir. Yapı İstanbul'da bulunmaktadır. İstanbul dünyanın en eski şehirlerinden biridir ve birçok kültürel mirasa sahiptir. Yapı, Kuzey Anadolu Fay hattının çok yakınındadır. Bir çok çalışma, kısa zaman içinde İstanbul'da muhtemelen büyük bir deprem olacağını göstermektedir. Yapıdaki mevcut hasarlar saha çalışmalarıyla belirlenmiştir. Yapı modeli hazırlanmış ve yapının kendi ağırlığı ile deprem yükleri altında analiz edilmiştir. 50 yılda aşılma olasılıkları %2 ve %10 olan iki farklı deprem yüklemesi kullanılmıştır. Mevcut hasarların nedenlerini belirlemek amacıyla özellikle çekme gerilmesi bölgeleri araştırılmış ve mevcut hasarlarla karşılaştırılmıştır. Gelecekteki olası hasarların önlenmesi için bazı onarım ve güçlendirme önerileri sunulmuştur.

1. INTRODUCTION

Historical structures are architectural and cultural heritage of a country and the world. They reflect the previous civilizations. The main responsibility of the present generation is to carry them to the next generations safely. Therefore, analysis, repair and strengthening of the historical structures are very important issues.

Construction types of the historical structures are mostly masonry. Masonry is constituted by interconnecting the masonry units with or without mortar. Most common masonry units are brick, stone and adobe. Moreover, wood was used for various purposes in the historical masonry structures. The masonry has low tensile strength whereas its compressive strength is higher. Therefore, masonry structures are very vulnerable under tensile stresses. In order to determine the material properties of the existing masonry structures, there are many assessment methods. These can be classified under three main groups; destructive, non- destructive, and in-situ test methods. The masonry material can be assumed single material for simplifications of analyses.

Walls, domes, arches, vaults and pillars are the main components of historical masonry structures. The other components can be aligned as buttresses, ties, piers, drum and weighting towers.

The main damage reasons of masonry structures are earthquake and settlement, where earthquake is more important. The soil condition of historical buildings generally gets to reach equilibrium since the structure was constructed a long time ago. There are many damage patterns for historical masonry structures such as out of plane rapture, cracks due to greater tensile stresses, corner and junction damages, radial cracks on domes, moving the abutments of the arches and vaults, frost effect and rapture and buckling of the ties.

Repair and strengthening of the historical masonry structures is a critical issue. It is slightly different from the other ordinary buildings. It requires combinations of many disciplines such as history, civil engineering, architecture and archaeology, etc.

Intervention should be kept at minimum level and it must be compatible with Venice Charter 1964 (Venice Charter, 1964). Intervention techniques can be classified into two main groups on the brink of reversible and irreversible techniques. Reversible techniques are primarily preferable. There are several repair and strengthening techniques for historical masonry structures.

In the present study, Mehmet Aga mosque was investigated. The mosque was constructed in 1585 by architect Davud Agha. Mehmet Agha of Darussade commissioned the mosque. The mosque experienced several earthquakes in 1766, 1894 and 1999. Two interventions have been performed to the building so far. It is still in service as a mosque. The mosque is located in Fatih province of Istanbul. It is located on seismic region and very close to the North Anatolian Fault Line. Istanbul is one of the oldest cities and it was capital of several Empires. Istanbul has many historical masterships, mostly from Ottoman Empire, such as mosques, bathhouses, churches and aqueducts. For Istanbul, a major earthquake is expected to strike in near future.

Three-dimensional model of the building was prepared using SAP2000 software (Wilson and Habibullah, 1998). The mosque was analyzed considering its self-weight and two different earthquake loadings. Then, the load cases were combined. Excessive stresses were investigated. Existing damages on the mosque were determined by in-situ survey. However, they could not be determined thoroughly because of past interventions. Currently, existing damages do not threaten the structural safety. However, some repair and strengthening interventions could be recommended in order to prevent further damages.

2. MATERIAL PROPERTIES AND CONDITION ASSESMENT OF MASONRY CONSTRUCTION

Masonry construction mainly consists of units and mortar. Most common units are clay, stone and concrete. Mortar is used for connecting the units to each other. Compressive strength, tensile strength, durability, shear strength, water absorption coefficient and thermal expansion of both the units and the mortar affect the load bearing capacity of masonry (Ünay, 2002).

Masonry is very strong in compression, while poor in tension. Three different failure modes can be seen in masonry, namely; tensile cracking, compressive crushing and shear failure.

The masonry is very complex, because it is non-homogeneous, anisotropic and brittle and it has weak mortar- strong unit balance. However, according to analysis purposes, some simplifications may be made for modeling the masonry material.

2.1 Constituent Materials of the Masonry

The masonry is mainly composed of units and mortar. Bricks, blocks, adobes, ashlars, irregular stones and others are typical masonry units (Lourenco, 2002). Wood and iron were also used for reinforcement in historical masonry constructions. Recently, concrete and steel are also used in masonry constructions.

2.1.1 Mortar

Mortar is used for integrating the units such as brick or stone. Mortar is mainly composed of sand, water and binding material. The traditional mortar basically consists of lime and sand. The rate of lime putty/sand is approximately 1/3 (Robert et.al., 2004). Organic and inorganic substances are added to mortar in order to increase physical properties and accelerate carbonating (Öztürk, 2006). The mortar can contain several materials such as bitumen, chalk, brick dust, natural cement and pigments and animal hair (Lourenco, 2002, Robert et.al., 2004). Khorassan mortar is a mortar type, which is composed of tile, brick powder and lime. It is light and gets hard slowly. It has relatively high tensile strength, pores and low density (Öztürk, 2006).

Strength and binding characteristics of the mortar affect the masonry performance considerably, although amount of the mortar is very less compared to the units. Compressive, tensile and bond strength of the mortar determine the strength of the masonry. Strength of the mortar depends on the properties and mixture ratio of constituent materials (Özen, 2006).

Strength of mortar should be less than strength of the units. It is supposed that any possible damage would occur in mortar region instead of units, because repairing of the mortar is comparatively easier and cheaper (Ünay, 2002).

2.1.2 Brick

Historical evidences shows that, brick was used since very early times. In historical structures, the brick was produced by burning pure kaolin and clay material in the ovens under high temperature. It is known that, the brick was produced under the sun heat in the earlier eras when there was no oven technology (Ünay, 2002). Clay is the main ingredient of brick (Şener, 2004). The strength of the clay brick is mostly affected by the properties of the constituent materials, baking temperature, drying process and porosity. Due to producing process, it shows different characteristics in horizontal and vertical direction (Özen, 2006). The brick is more ductile than stone (Sofronie, 2001); therefore it may be stronger in out of plane bending.

The compressive strength of brick is very strong where the tensile strength is poor. Elastic modulus of the brick vary 5000 and 10000 MPa and Poison’s ratio of the brick is about 0.15-0.2 (Croci, 1998). Some mechanical properties of brick are given in Table 2.1.

Table 2.1 : Mechanical Properties of Brick (Ünay, 2002).

Compressive strength(MPa)	Tensile Strength (MPa)	Shear Strength (MPa)
10-30	2.7-5	10-20

The oldest brick type is adobe. It is composed of soil, water and organic materials such as straw and manure. It is not durable against environmental conditions, such as humidity and rain. Mud layers are applied on adobe walls to prevent damage from rain, snow, etc (Özen, 2006).

2.1.3 Stone

Stone is one of the oldest masonry materials and it has high durability. Stone has widespread use in historic structures since it can be found easily (Ünay, 2002). The compressive strength of stone is high where the tensile strength is poor. Due to the mineral composition of the stone, the physical properties of stone highly changes (Özen, 2006). Mechanical properties of several stone types are given in Table 2.2.

Table 2.2 : Mechanical Properties of Stone (Ünay, 2002).

Stone Type	Compressive Strength (MPa)	Shear Strength (MPa)	Tensile strength (MPa)	Elasticity Modulus (GPa)
Granite	30-70	14-33	4-7	30-55
Marble	25-65	9-45	1-15	25-70
Limestone	18-35	6-20	2-6	10-55
Sandstone	5-30	2-10	2-4	13-50
Quartzite	10-30	3-10	3-4	15-55
Serpentine	7-30	2-10	6-11	23-45

The stone can be used either in its natural shape or be reshaped as a structural masonry material. Rubble is natural or minor shaped stones and ashlars are accurate rectangular forms of the stone. Dry masonry is made up without mortar using stones with smooth surfaces (Özen, 2006).

2.1.4 Wood

Wood can be found easily in nature. Since it is light, its workmanship is easy. However, the wood is short lived. Tensile and bending strength of the wood are strong. By reason of tensile and bending strength, it was used such as slab, tie and balk element in historical masonry structures (Yılmaz, 2006).

2.2 Mechanical Properties of Masonry

The masonry structure has high compressive and low tensile strength. The structural forms of masonry constructions are designed accordingly i.e. to take advantage of the ample compressive strength. The masonry material is very brittle, such that sudden failure occurs in tension loading. Figure 2.1 shows typical behavior of quasi-fragile materials under uniaxial loading and definition of the fracture energy. Fracture energy is the absorbed energy until the failure time. It can be determined by calculating the area under stress-strain diagram (Alvarenga, 2002, Özen, 2006).

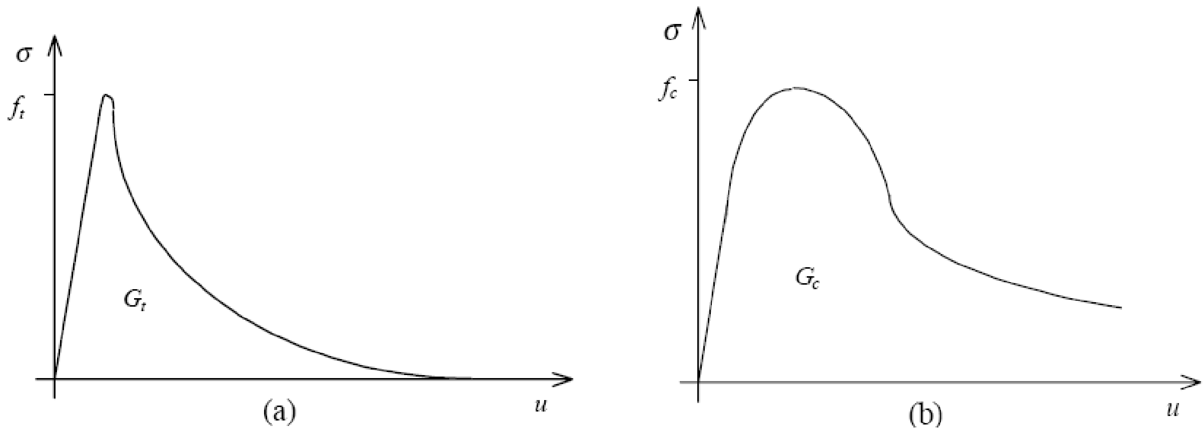


Figure 2.1 : Typical behavior of quasi-fragile materials under uniaxial loading and definition of the fracture energy a) tension loading b) compression loading (Alvarenga 2002).

Overall strength of masonry is mostly affected by the bond strength at the unit-mortar interface (Özen, 2006). Generally, the weakest contact in masonry assemblages is the bond between unit and mortar (Lourenco, 2002). The perpendicular forces to the mortar-unit joint are resisted by the tensile bond strength where the parallel forces to the mortar-unit joint are resisted by the shear bond strength of masonry (Ünay, 2002). Tensile failure or shear failure can occur at the unit–mortar interface (Lourenco, 2002).

Strength of stone masonry depends on the material properties and bond type of units. The stone is massive and stiff. The strength of stone does not much effect to stone masonry. The joint behavior of unit and mortar determines the strength of stone masonry. The shear strength of the stone masonry is approximately 25% of the compressive strength (Ünay, 2002).

Different types of stone masonry are shown in Figure 2.2 (Lourenço, 1998).

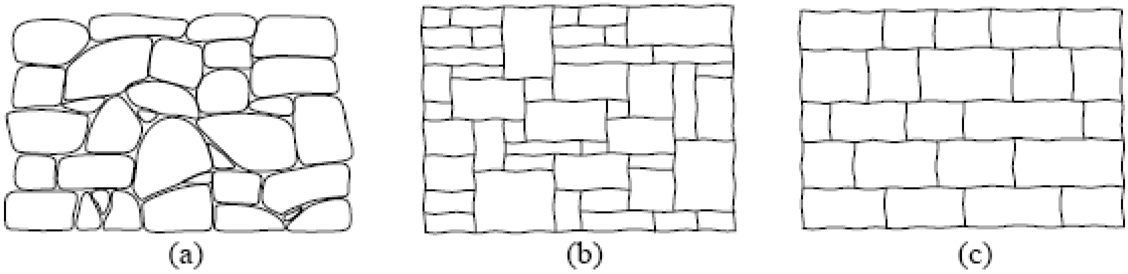


Figure 2.2: Different kinds of stone masonry: (a) rubble masonry, (b) ashlar masonry; c) coursed ashlars masonry (Lourenço, 1998).

Strength of the brick masonry is very close to the strength of brick units. It depends on material quality, mortar type and bond condition of units (Ünay, 2002).

Figure 2.3 shows the different bond types of brick masonry (Lourenço, 1998).

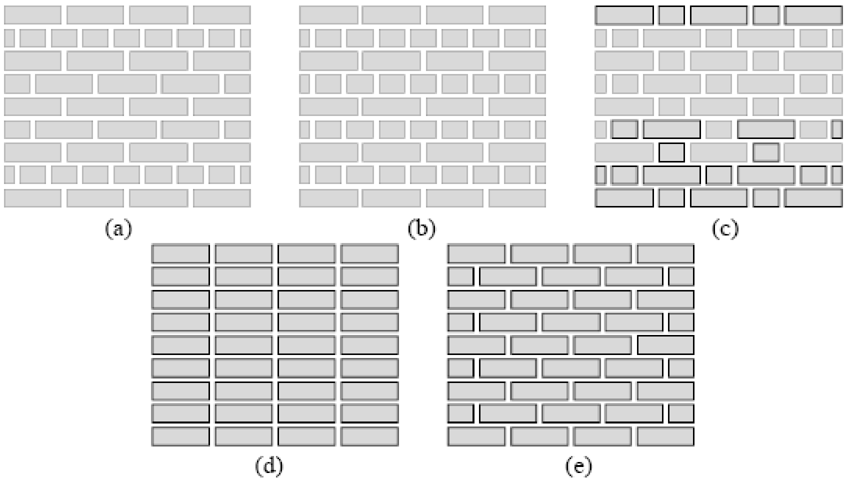


Figure 2.3: Different arrangements for brick masonry: a) American (or common) bond, b) English (or cross) bond, c) Flemish bond, d) stack bond, e) stretcher bond (Lourenço, 1998).

Elastic modulus (E) can be determined from stress-strain diagram. Generally, the slope of beginning linear part of the diagram is used as elastic modulus. In fact, the elasticity modulus of masonry changes in each direction and due to different loadings (Özen, 2006), since masonry material is nonhomogenous and anisotropic. However, the elastic modulus of masonry is considered only under compression condition in this chapter.

The elastic modulus of masonry depends on elastic modulus and dimensional ratios of constituent materials and the bond characteristic of the mortar-unit interface.

Elastic modulus (E) can be determined using several methods. Eurocode 6 (European Committee for Standardization, 2005) suggests a formula which is given in equation 2.1.

$$E = 1000\, f_k \qquad \textbf{(2.1)}$$

where; f_k is the characteristic compressive strength of masonry.

Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik (DBYBHY, 2007) offers equation 2.2 for the elastic modulus of masonry (Ministry of Reconstruction and Resettlement**,** 2007).

$$E_d = 200 f_d \qquad \textbf{(2.2)}$$

Shear modulus (G) could be taken 40 % of the elastic modulus (E) (European Committee for Standardization, 2005).

The Poison's ratio of the masonry generally varies between 0.20 and 0.25. In case cracking or joint sliding, Poison's ratio can be greater than 0.5 (Dialer, 2002).

2.2.1 Compressive Strength of Masonry

Compressive strength of masonry is very high when compared to the tensile strength of masonry. The compressive strength of masonry can be determined by testing the masonry prisms composed of units and mortar. The strength is equal to divide maximum load by the cross-sectional area of the masonry prism (Ünay, 2002). The masonry compressive strength depends on constituent materials strength, joint properties, relative mortar thickness etc (Özen, 2006).

Mortar is softer than the brick. Since mortar and brick have different elastic properties, under uniaxial compressive loading, the mortar tries to expand laterally more than units. However, there is continuity between mortar and units. Because of that, the mortar is restricted by units laterally by means of cohesion and friction. Hence, the shear stresses, which occur at the brick-mortar interface, generate bilateral tension couple and uniaxial compression in the brick and they produce triaxial compression in the mortar (Oliveira, 2000).

Several codes comprise empirical formulas in order to determine the compressive strength of masonry. Eurocode 6 (European Committee for Standardization, 2005) defines the characteristic compressive strength of masonry as follows:

$$f_k = K\, f_b^{\alpha}\, f_m^{\beta} \quad \textbf{(2.3)}$$

f_k : Characteristic compressive strength of the masonry (MPa)

f_b : Normalized mean compressive strength of the masonry unit (MPa)

f_m : The compressive strength of the mortar (MPa)

K, α, β : Constants

In masonry structures, such as arches, vaults and pillars components are generally exposed to eccentric normal force (Brancich et al., 2002). Load eccentricity affects the compressive strength. Brancichs' study shows that, there is an inverse ratio between eccentricity and the compressive strength of component (Brancich et al., 2002) (Figure 2.4).

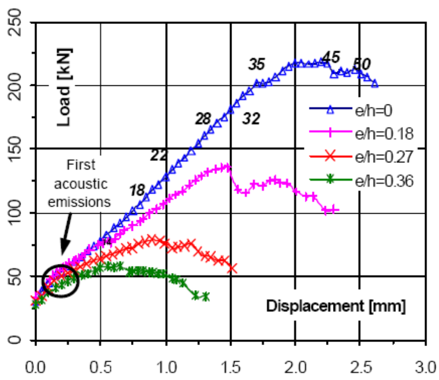


Figure 2.4: Load displacement diagram due to different eccentricity ratios (Brancich et al., 2002).

2.2.2 Tensile Strength of Masonry

Masonry shows brittle behavior and has high risk under tensile stress. Generally, there is no load component that exposes the masonry to direct tensile stresses in historic structures. The tensile stress caused by bending is more important. The tensile stress occurs in domes, vaults, arches and pendentive components, under flexural loads. Shortening and extension strains that occur by reason of moisture and temperature changing also cause the tensile stress (Ünay, 2002).

The tensile strength of masonry is generally equal to the flexural bond strength between mortar and units. If the tensile bond strength is higher than unit strength, the tensile strength of masonry is of the same value as the tensile strength of unit (Özen, 2006).

2.2.3 Shear Strength of Masonry

Shear strength of masonry depends on the binding condition between mortar and units. DBYYHY 2007 (Ministry of Reconstruction and Resettlement**,** 2007) offers

$$\tau_{em} = \tau_0 + \mu\sigma \quad \textbf{(2.4)}$$

for determining the shear strength of masonry, where; τ_{em} is the shear strength of the masonry wall, τ_0 is the fracture strength of the wall, μ is the friction coefficient which can be taken as 0.5 and σ is the vertical stress of the wall (Ministry of Reconstruction and Resettlement**,** 2007). Equation 2.4 shows that, if the vertical stress of masonry increases, also the shear strength of masonry increases. Figure 2.5 shows the shear stress-strain diagrams under various vertical stresses (Van der Pluijm, 1993).

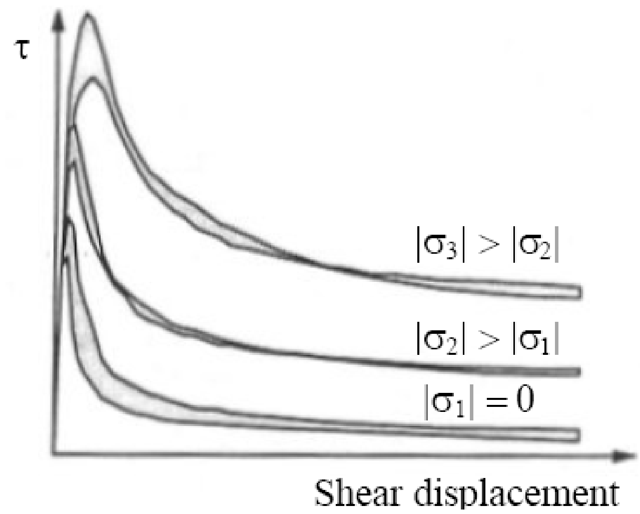


Figure 2.5: Typical experimental shear stress-displacement diagrams (Van der Pluijm, 1993).

2.3 Condition Assessment

Analysis of a structure requires determining the material properties such as strength, elastic modulus and unit volume weight along with the overall load carrying capacity of the structure. In order to determine the properties, there are several methods. However, performing material tests on historical structures has some difficulties compared to the ordinary structures. Historical structures have highly architectural and cultural value. It is important to preserve these characteristics. Therefore, use of destructive testing techniques is generally not convenient in historical structures. Extracting specimens from a historical structure would generally be not permitted. Furthermore, even though small specimens are extracted, small specimens do not always reflect the true structural behavior of the overall system (Ünay, 2002). In case of removing large specimens is necessary, damaged components of a structure which constructed in the same era can be used. If the component is non-uniform, obtained strength from experiments may not give exact result. The chemical test is obligatory for deciding the compatible repair material. Chemical test specimen is very small and its extraction does not constitute a problem in terms of structural safety (Sesigür et al., 2007).

There are several methods for condition assessment of the masonry structures. They can be assorted to three parts; destructive test methods, non-destructive test methods and in situ test methods.

2.3.1 Destructive Test Methods

Destructive test methods mainly involve extracting cores from the structure and testing in the laboratory. The specimens should be taken out and carried without damage as much as possible. Number of specimens should be as minimum as possible. Test goals are determining the chemical, physical and mechanical properties of the masonry construction (Şen, 2006).

2.3.2 Non- destructive Test Methods

Tests are performed without reasonable damage. The properties such as strength and stiffness are not found directly. General information about the material condition is collected establishing relationships with the non-destructive test results and physical characteristics. Surface penetrating radar, rebound hammer, infrared termography, impact-echo, stress wave transmission, ultrasonic velocity testing and tomographic imagining are some of the non-destructive test methods that are applicable to masonry structures (Şen, 2006).

2.3.3 In-Situ Test Methods

In-situ test methods such as borescope, mortar evaluation, flatjack methods, in-situ shear tests, in-situ bond tests are used instead of destructive test methods. Specimens are not carried to laboratory. The engineering properties are obtained which cannot provide by non-destructive methods directly. It is applied with installing loading equipments on structure (Şen, 2006). They can give reliable outcomes. However, the expertise is necessary to choose the best locations and to interpret the data (Teomete, 2004)

2.4. Modeling Strategies for Masonry Material

The modeling of a historical structure is a complex task since behavior of masonry is not isotropic, elastic or homogenous. Since, stone, brick and mortar have different physical and mechanical properties; it is difficult to determine the material properties of masonry (Ünay, 2002). However, due to the purpose of analysis, some modeling simplifications can be done.

Three different strategies may be used for modeling the masonry. Namely, detailed micro-modeling, simplified micro-modeling and macro modeling (Figure 2.6). Complexity of analysis tool does not mean that it will provide better results than a simple tool (Lourenco, 2002). The modeling strategy should be chosen due to the analysis purpose.

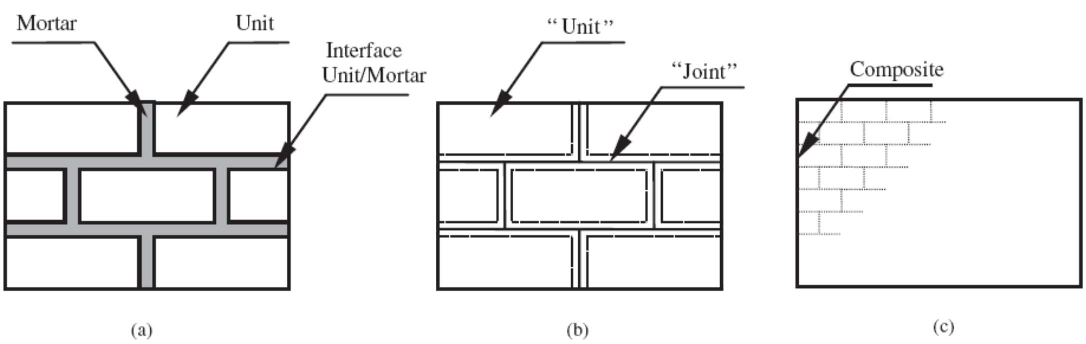


Figure 2.6: Three modeling strategies for masonry: a) detailed micro-modeling, b) simplified micro modeling, c) macro modeling (Lourenco, 2002).

Detailed micro-modeling: Unit, mortar and unit-mortar interface are modeled particularly. To obtain a better understanding about the local behavior of masonry, this modeling method is necessary (Lourenco, 2002).

Simplified micro-modeling: The mortar and unit-mortar interface are modeled as a lumped element in the mortar joint. Two different components are modeled, namely; unit and joint element (Lourenco, 2002).

Macro-modeling: Unit, mortar and unit-mortar interface are modeled as a homogeneous continuum element. This method is suitable in case the structure consists of solid walls with adequately large dimensions and stresses across or along a macro-length will be mainly uniform. It provides a compromise between efficiency and accuracy (Lourenco, 2002).

Homogenization approach (assuming the masonry as a unified material) aims to determine the behavior of masonry from geometry and behavior of the representative basic cell (Figure 2.7) (Zucchini et al., 2002).

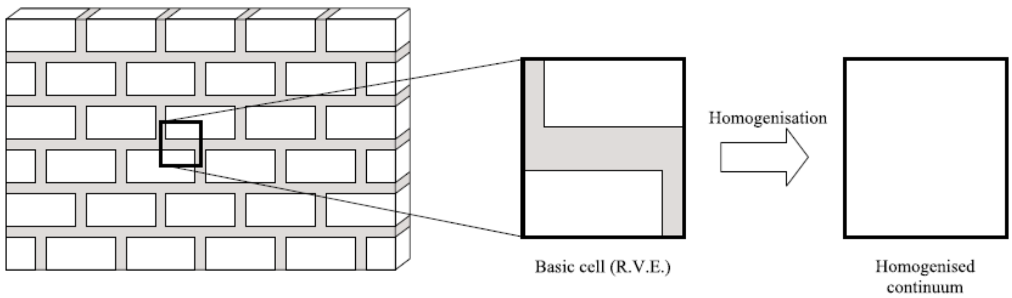


Figure 2.7: Basic cell for masonry and objective of homogenization (Zucchini et al., 2002).

The elastic modulus of the homogenized continuum may be determined by use of Equation 2.5 (Lourenco et al., 2001).

$$E = \frac{t_m + t_u}{\dfrac{t_m}{E_m} + \dfrac{t_u}{E_u}} \times \rho \qquad \textbf{(2.5)}$$

where; t_m is the thickness of mortar, t_u is the height of brick, E_m and E_u are the elastic modulus of mortar and unit. ρ is a coefficient associated with the deficient bond between the unit and mortar (Lourenco et al., 2001).

3. COMPONENTS OF HISTORICAL MASONRY STRUCTURES

Historical masonry structures compose of various elements. Arches, vaults, domes, walls and pillars are main components of historic masonry structures. Furthermore, such as buttresses, ties, drum, pendentives and clamps elements were used in historical masonry structures for different purposes.

3.1 Arches

A horizontal space is spanned by beam easily replacing across an opening. In historical structures, this achieved using stone lintels. In the lintel, tensile stresses occur at the bottom part while the compressive stresses occur at top level. Since the stone has very low tensile strength, cracks take place at bottom part of the lintel. Because of that, it is necessary relatively large cross section for spanning over only a short space (S´anchez, 2007). Therefore, in order to span over longer distances with smaller cross sections, arch shape was invited.

Arches are curved shape components of the historical masonry structures. Main function of the arches are carrying the dome and transferring the loads to the main piers or walls safely (Sesigür et al., 2007). Arch action is the main load bearing principle for arched structures. It creates compressive stress internally and a pair of force externally which tries to push the supports apart (Beckmann, 1995).

Usually, only compressive stress occurs under self-weight in the arched structure along a virtual path, which called "thrust line", inside its thickness. If the thrust line moves outside the middle third of cross section, which called central core, cracks take place due to tensile stresses. As long as the thrust line is inside the thickness, the arch safety is preserved (S´anchez, 2007).

The thrust line can be obtained by a simple analogy. If a cable is suspended between two points under its self-weight, a catenary shape is formed and the cable is subject to pure tensile stresses. Then, if the cable is rigidly inverted 180 degree, about two support points, the weight would act in the contrary direction and the cable would be subject to pure compressive stresses. Thus, the best arch shape is obtained which resists only its self-weight (S´anchez, 2007).

Required arch shape depends on the applied load pattern. In order to determine the arch shape under other load patterns, equivalent weights can be applied on the cable model. Figure 3.1.a shows the shape of thrust lines under various loading pattern. Since the arches are generally under vertical uniform loading, the line of thrust is usually parabolic (S´anchez, 2007). Superposition of the thrust lines for some typical arched structures shown in Figure 3.1.b.

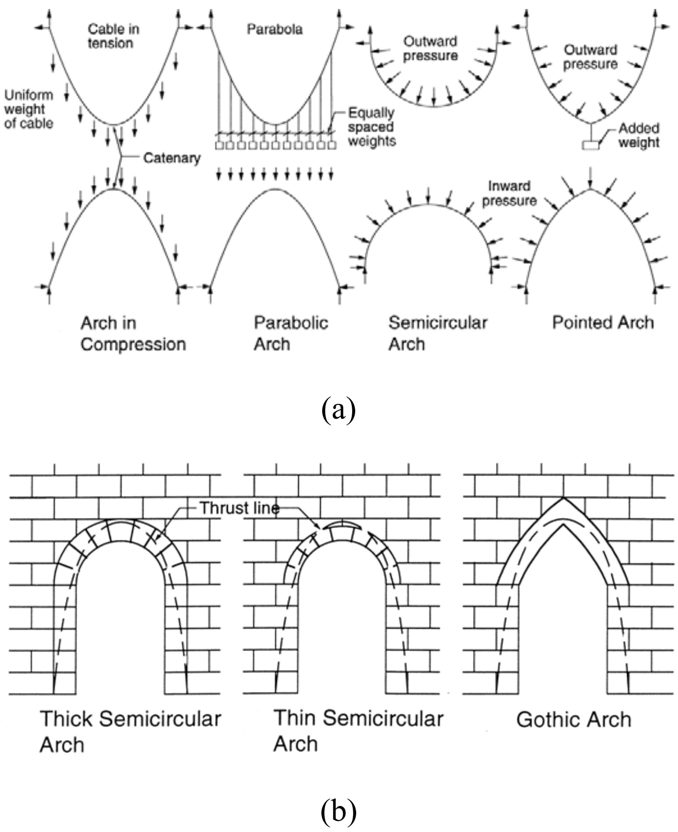


Figure 3.1: Basic of the line of thrust: (a) cable arch analogy; (b) thrust lines in typical arched structures (S´anchez, 2007).

Parabolic curvature of the arch carries only axial forces under equal loads. Normally, no bending moment occurs in the arches due to its curvature properties. However, because of the interaction with other components, this is generally not possible. Arches are obliged to carry the horizontal thrusts in complex structures. Arches are imposed significant tensile stress under lateral forces. Increasing the dead loads reduce the tensile stresses. Cross-section dimensions of arches are large in historic masonry structures for using the contribution of self-weight to the arch stability (Ünay, 2002).

Arches are constructed placing the units one on the other with a curvature shape. Keystone is placed last at top level. The lower level of the arch is called Stirrup Level. The distance between two-stirrup points, which are springing of arch, is arch span. The deflection is defined as the height between keystone and stirrup. Load bearing capacity of arches increase due to raising the rate of deflection by arch span. Horizontal thrust goes up while the span length and total load increase and it is inversely proportional to the height.

Main arches are generally composed of stone or brick. Short span decorative arches made of marble (Ünay, 2002).

Ties are used in arched structures to resist the thrust that occurs due to the dome or arch behavior and for preventing the moving the abutments apart. Because, opening the abutments outward affects the arch stability negatively. Generally, ties were made of wood or iron materials in historical masonry structures. Mostly, single or double ties were used at stirrup level. In some historic structures, an addition tie was used between keystone and stirrup level (Ünay, 2002).

3.2 Vaults

Vaults were employed to cover the rectangular areas in historical structures (Sesigür et al., 2007) Vault is obtained elongating the arch along the perpendicular direction to its axis. Load bearing principle is the same with arches. However, their construction techniques are different (S´anchez, 2007).

Various vault types can be seen in Figure 3.2. Cross and Groin vaults are supported by pillars or piers. Thus, they are used for covering multi-unit volumes (Sesigür et al., 2007). Combining the barrels vaults with great ingenuity, nice buildings were constructed (S´anchez, 2007).

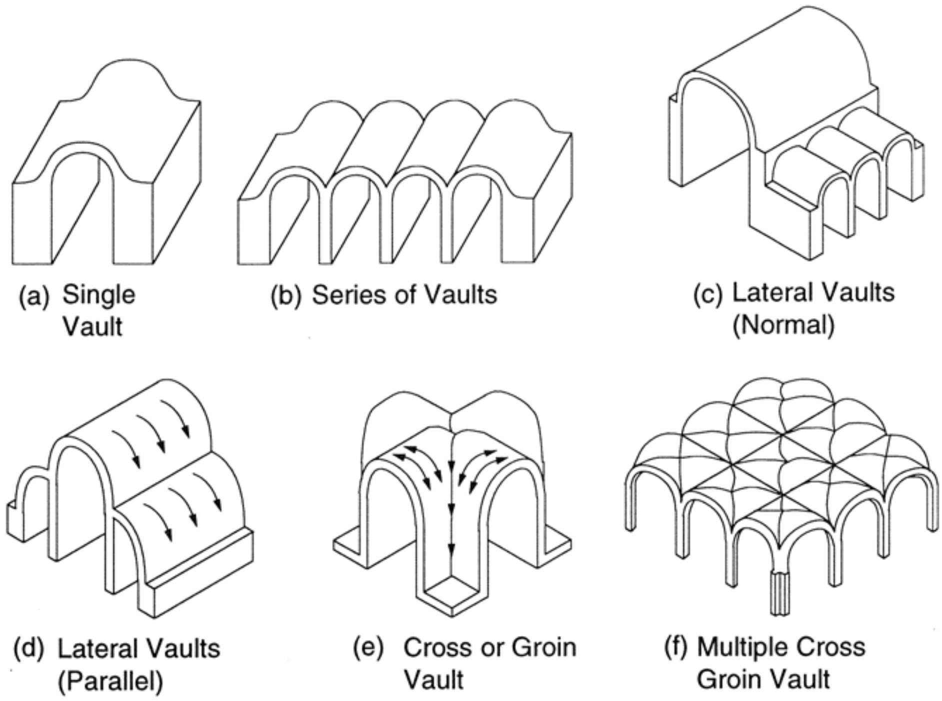


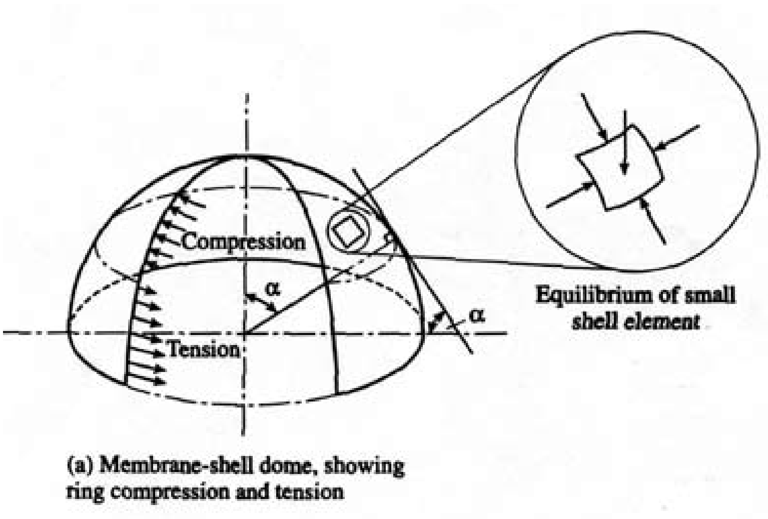
Figure 3.2: Examples of combined barrels: a) single vault, b) series of vaults, c) lateral vaults (normal), d) lateral vaults parallel, e) cross or groin vault, f) multiple cross groin vault (S´anchez, 2007).

3.3 Domes

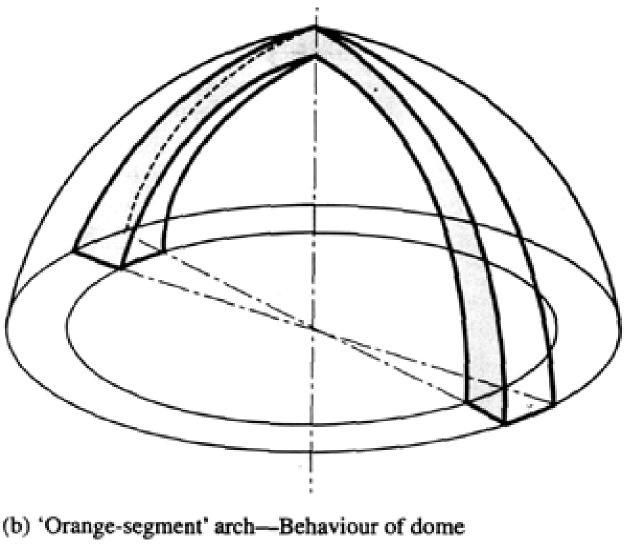
Dome is obtained rotating an arch about its vertical axis (Sesigür et al., 2007). The real behavior of a dome is very complex. It is exposed to the forces in three dimensions. It produces bending stresses as well as compression and tension. However, two assumptions may be made for simplifying. Each of them reflects a part of overall behavior (Beckmann, 1995).

The first assumption is assuming the dome as a shell element of which thickness is very small compared to the other dimensions (Figure 3.3.a). So, due to its small thickness, it has a very low bending strength. Compressive stresses occur along meridians. Circumferential stresses change from compression to tension at lower dome part due to the load distribution and the shape of dome (Beckmann, 1995).

In the second assumption, the dome is taken into considerations as a series of arch (Figure 3.3.b). If the thrust line of one such arch does not get closer to the intrados or extrados, the arch is strong and stable enough. If each arch strong and stable enough, then the dome is stable and strong enough in spite of that the cracks may occur along meridians. Real behavior is mixture of these two assumptions (Beckmann, 1995).



(a)



(b)

Figure 3.3: Behavior of dome, a) Membrane-shell dome, b) "Orange-segment arch" (Beckmann, 1995).

The dome requires a continuous support. Therefore, it needs the circular support, which called drum. Square planned structures require transition components. Most common transition components are pendentives, tromps and Turkish triangles in historical masonry structures. Sometimes semi-domes were used in order to support the main dome (Sesigür et al., 2007). Tensile forces, which occur along parallel directions in the lower part of dome, are deactivated by massive and heavy drum in the structure that has large dome (Ünay, 2002).

Square and polygon supports may also be used. Thick walls or thin walls with lintels or arches are necessary while the square support is used for supporting the dome (S´anchez, 2007).

Weighing towers and buttresses were used for resisting the dome thrusts in historical masonry structures.

The dome was generally made of brick. In addition, it was constructed using stone or wood in historical masonry structures (Sesigür et al., 2007).

3.4 Walls

Walls resist the self-weight, vertical and lateral loads. They transmit the superstructure loads to the foundation. They are continuous components. The wall thickness is wide in historical masonry structures since out of plane forces are resisted by self weight of the walls. The wall thickness is defined considering oblique loads and earthquake loads (Sesigür et al., 2007).

Cross section stresses of the walls should be equal or close each other and should be spreaded regularly. Mortar ties units each other. Lintels, which are placed at various wall levels, were used for assurance the wall stability. In some cases, the walls were supported by buttresses in order to resist the oblique loads (Sesigür et al., 2007).

The clamp element was used interconnecting stone units. It was covered by leaden material in order to prevent against corrosion effect. Thus, splitting of the stone is prevented (Bal et al., 2007).

3.5 Pillars

Pillars carry significant compressive stress. They can be composed either one-piece stone or multi-piece stone which is obtained interconnecting the stone parts with pins (Figure 3.4) (Sesigür et al., 2007).

(a)

(b)

Figure 3.4: Pillars, a) One-piece pillars (Roman Form), b) Multi-piece pillars (Propylaia, Atina) (Sesigür et al., 2007).

4. DAMAGE PATTERNS, REPAIR AND STRENGTHENING OF HISTORICAL MASONRY STRUCTURES

Earthquake and settlements are main damage reasons for historical masonry structures. Moreover, there are several other damage causes. Such as crack occurrence, out of plane rapture and differential settlement damages are formed in historical masonry structures due to these effects. In order to provide the structural safety, damaged components and structural insufficiencies are repaired and strengthened.

4.1 Damage Patterns

Damage reasons in historical structures are earthquake, settlement, poor material quality, air pollution, other disasters, poor labor, traffic, etc. (Öztürk, 2006). Self-weight is also important in historical masonry structures. The main damage or collapse reasons are seismic effect and soil movement for historical masonry structures. Earthquake effect is more important than settlement. The soil has reached equilibrium since the historical structure constructed a long time ago. However, changing the soil condition due to environmental effects may cause damage in historical structures.

Damage patterns for historic masonry structures can be aligned as follows:

- Generally, collapse mechanism of masonry structure starts with out of plane rupture (Henry, 1990). This is the most common failure case. Wall overturns towards the outside from vertical plane. Connection condition between different parts of the masonry structure is important in terms of out of plane behavior. Lack of connection between orthogonal aspects causes overturn of wall. If the connection is adequate, loads can be transmitted to the orthogonal walls in plane safely (Figure 4.1) (Şen, 2006).

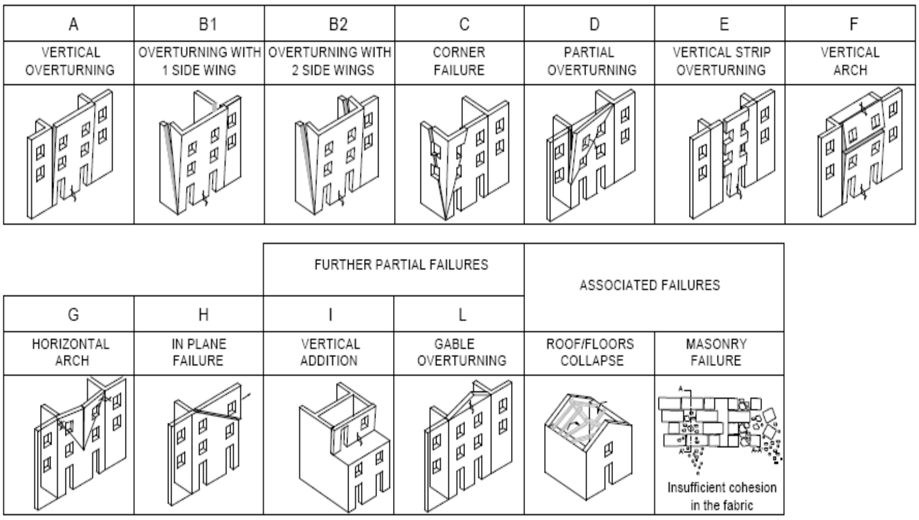


Figure 4.1: Out of plane mechanism in masonry structures (D'Ayala et al., 2002).

- Crack Occurrence is very common damage for historic masonry structures. Generally, tension and shear cracks occur due to differential settlement or excessive lateral loads (Beckmann, 1995). Cracks can be active or passive. New cracks are clearer and have sharp edges while the old cracks are more unclean, and have round edges. Location and distribution of cracks in structure give idea about stress dispersion and cause of cracking (Sesigür et al., 2007).
- Temperature fluctuation and frost effect cause the enlargement of cracks. While the movement caused by temperature fluctuations, sand grains and mortar particles tend to fall into the existing cracks. They open the crack and do not permit them for closing in the second part of the temperature cycle. Then, these particles gradually increase the crack width. It is called ratchet action. In the same way, if water gets in the crack, it can expand by frost effect and will gradually widen the crack (Beckmann, 1995).
- Differential settlement causes damage in masonry structures. Generally, it shows itself by in plane inclined cracks. Settlement condition determined due to the crack direction (Sesigür et al., 2007).
- Crack occurrence due to different settlement in masonry structures can be seen in Figure 4.2.

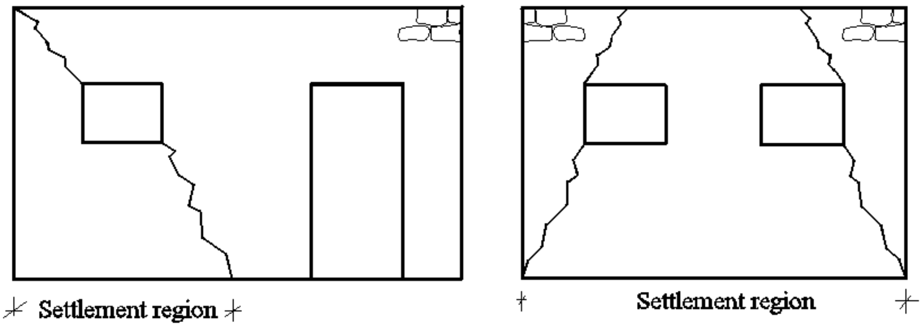


Figure 4.2: Differential settlement damages in masonry structures

- Rotation of the walls is another damage pattern. It gives clear information about the translation of the structure. It should determine using photogrammetric methods if necessary. The rotation might occur because of the construction mistake (Sesigür et al., 2007).
- Dome damages: When the tensile stresses in the circumferential direction exceed the tensile strength of masonry, radial cracks occur on the dome (Figure 4.3). Increasing the cracks causes that the dome behavior approaches the independent arches along meridians. Then, the bending moment may get important and instability problem may occur for the dome (S´anchez, 2007 and Croci, 1998b).

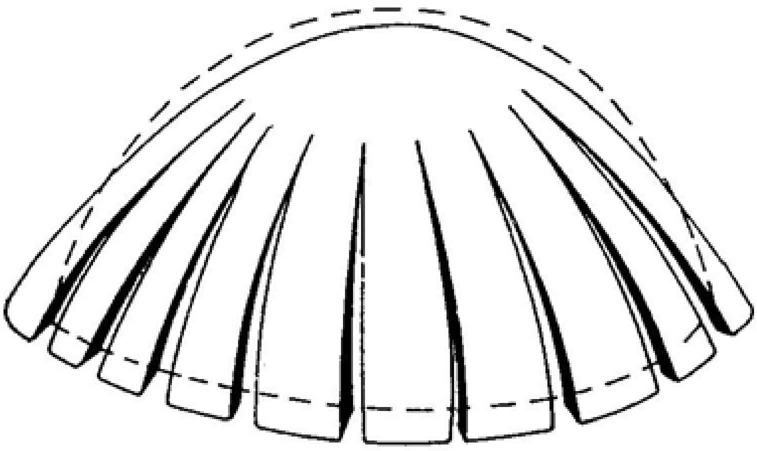


Figure 4.3: Typical cracking of a dome (Heyman, 1988).

The dome produces cracks under differential settlement condition, similarly to arches and vaults but that is more complex (Beckmann, 1995).

- Arch and Vault damages: Structural problems cause cracking and join openings in the arches and vaults (Beckmann, 1995). Decreasing the abutment thrusts and combined bending (M+N) effect are main damage reasons in these components (Sesigür et al., 2007). When the springing reaction declines, the horizontal thrust tries to push the abutments away and the arch tends to spread. It is fulfilled by opening the joints of voussoir stones and cracks be formed. In addition, differential settlement causes the opening of joints. Incompatibility between the arch shape and load distribution is another damage reason. In this case, if the thrust line moves closer than one-third of the arch thickness towards an arch side, joint openings will be formed on the opposite face of the arch (Figure 4.4) (Beckmann, 1995).

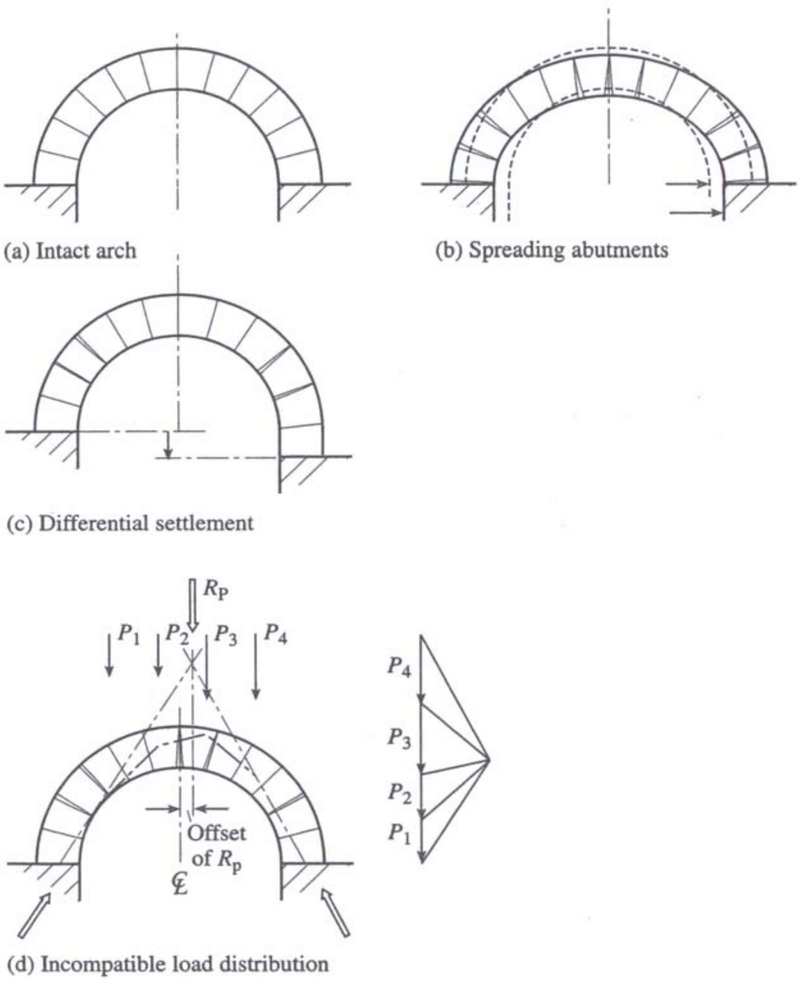


Figure 4.4: Cracking patterns of arch; a) intact arch, b) Spreading abutments, c) Differential settlement, d) Incompatible load distribution (Beckmann, 1995).

- Theodossopoulos et al. (2004) performed experiments on a cross vault model under service loads and support movement. The study showed that the model has reasonable strength under service condition and it is highly affected by geometric changes. Movement of the abutments created tensile stresses, which are greater than the flexural strength of abutments and vertices. However, membrane tensile failure at the groins dominated at last. (Theodossopoulos et al., 2004) Pointing separation damage in vaults shown in Figure 4.5.

Figure 4.5: Pointing separation of the vault (Sesigür et al., 2007).

- Tie damages: Extreme corrosion is the most common deterioration type in the iron ties. Most observed damages are the buckling and rupture of the ties due to motions in the structure (Figure 4.6). Putrefaction damage may occur on the wood ties (Sesigür et al., 2007).

(a) (b)

Figure 4.6: Damages in iron Ties, a) buckling damage, b) rapture damage (Sesigür et al., 2007).

- Water penetration into rubble core masonry create the frost expansion (Beckmann, 1995) (Figure 4.7)

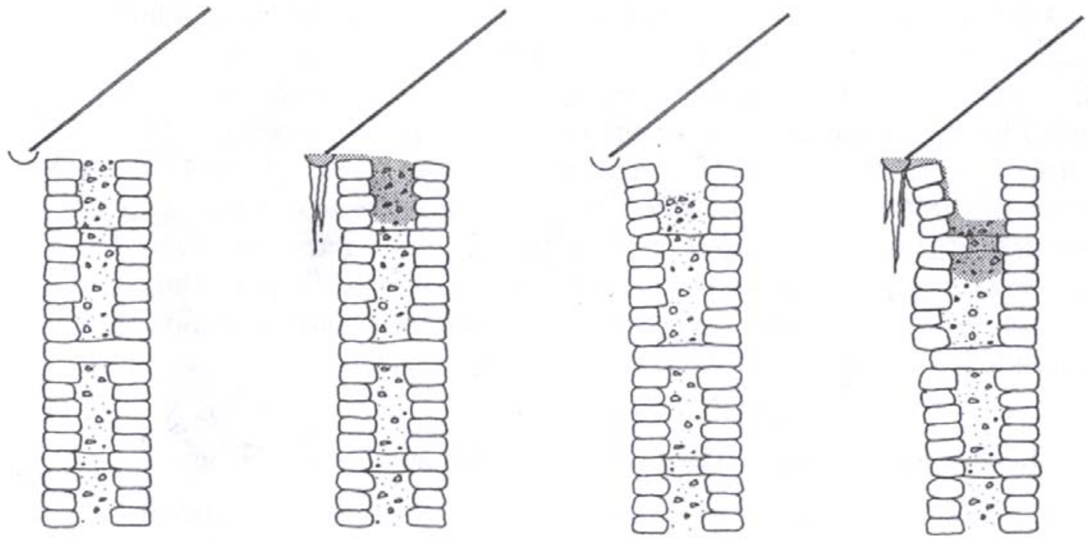


Figure 4.7: Deterioration of rubble-cored masonry wall due to frost expansion (Beckmann, 1985).

- Crash case comes about due to compressive stress concentration or weakness of the structural elements (Sesigür et al., 2007).
- Shear strength increase due to vertical load. Uppermost part of walls is very vulnerable under earthquake loads because, vertical load acting on them is insufficient.
- Wood element damages: Joint deterioration of the wooden frames is a common damage. Because of material deterioration or loss of the connecting property of conjunction elements, the joint loses the bearing capacity (Sesigür et al., 2007).

4.2 Repair and Strengthening

Historical structures are cultural heritages of past civilizations. Having high art and aesthetic values, these structures reflect historical and cultural characteristics of the time when they were built. Now, passing historic heritages down to next generations is very important issue. Therefore, repair and strengthening of historical structures is thoroughly important issue. It is not possible to use all intervention methods, which is used for ordinary structures. Preservation of its aesthetic and historical value is primarily important (Penelis, 2002).

Modern intervention aspects can be aligned as removability, respect to original concept and minimum intervention, construction safety, material durability and compatibility and optimization between cost and existing financial resources (Loureço, 2004).

Intervention should be kept minimum and should be in proportion to safety and durability objectives (Loureço, 2004). When the structural intervention is performed, original structure should remain the same.

Intervention requires a multi-disciplinary study. Structural restoration requires the collaboration of specialists in many disciplines such as structural engineering, archaeology, architecture, chemical engineering (Osman, 2005). Essential information for intervention is collected performing the structural, historical and architectural investigations, structural survey, in situ research, laboratory testing and monitoring of structure (Loureço, 2004). For example, determining the past seismic events can provide useful information about the structural behavior.

Before repair or strengthening process, the reasons of observed damages must be investigated if they are active or passive. If the damage cause is still active, it must be removed before intervention. Otherwise the repair process would be short lived. After treating the problem, subsequent repair will be substantially cosmetic intervention. If the defects cannot be eliminated, the masonry should be strengthened to resist the cracking reason. Even if the cracks are dead, also further environmental effects such as earthquake and possible changing in soil conditions should be considered before deciding the cosmetic repair (Beckmann, 1995).

Venice charter, which is issued in 1964, determines the basic intervention principles for historic structures. All interventions to historic structures should correspond to this specification. Conservation and restoration principles of Venice charter can be summarized as follows (Venice Charter, 1964):

- The main purpose of the conservation and restoration is to safeguard the heritage as work of art and historical evidence.
- All science and techniques must be put account, which can contribute to conservation and restoration of the cultural heritage.
- Restoration process requires highly specialization. Purpose is to prevent and reveal the aesthetic and historic value of the heritage.
- Restoration requires respecting the original material.

- Replacing the missing part must be distinguished as different from the original monuments and they must be reflected its own period.
- Before and after any restoration process, archeological and historical investigations should be investigated. Construction date, damage patterns, past repairs and strengthening and past function of the heritage should be determined.
- In case traditional techniques are insufficient, any modern technique, which has been proved by experiments and scientifically, may be used for restoration.
- All periods contributions to the architectural heritage must be respected. Revealing of the previous period states may be permitted in only particular cases.
- All aspects of the restoration work should be documented precisely and the documentations should be kept by the related public institutions.

4.2.1 In Situ Survey

Intervention process requires information about existing condition and behavior of structure. It is obtained by in situ survey. These investigations can be aligned as follows (Sesigür et al., 2007):

- Construction system and existing condition of the load-bearing walls should be determined
- Condition of the metal and wood elements should be investigated
- Non-destructive drilling can be performed in order to find out the properties of the unreachable components.
- Cross-sectional area losses and joint deteriorations of the steel beams of the jack arches should be determined
- All determined deteriorations should be showed on a detailed load-bearing system relief in addition to the normal relief. Crack widths and depths, vertical and horizontal declinations, visible and invisible metal elements and other helpful information should be given in these reliefs.
- It is necessary to determine the all-material properties and foundation system.

4.2.2 Classification of Interventions

Intervention techniques are classified in two categories; namely, reversible and irreversible techniques.

4.2.2.1 Reversible Techniques

Reversible interventions are primarily preferable for historical masonry structures. Because;

- They can be replaced without any damage in case their inefficiency and low durability are proven (Osman, 2005)
- They can be replaced easily in case of better techniques or materials are improved (Penelis, 2002).
- They do not damage to the aesthetic and historic value (Penelis, 2002)
- There are many reversible intervention techniques such as external buttresses, strengthening of arches by ties, confining ring around the dome, using reinforcement laminates, improvement of ductility, stiffness and strength of the existing diaphragms etc. Since they can be replaced, there are few restrictions for using these techniques in historical structures (Penelis, 2002 and Osman, 2005)

4.2.2.2 Irreversible Techniques

Reversible methods may not be always applicable and sufficient for solving the existing problem. In that case, irreversible techniques are used. These methods cannot be undone without damage to the original texture. Because of that, they should be considered if there is no other proper reversible method. Some of these techniques are grouting, underpinning, strengthening of foundations etc. (Penelis, 2002).

4.2.3 Repair and Strengthening Materials

Intervention material must be compatible with existing one and must have enough durability. Compatibility between new and existing materials is agreement of the mechanical, physical and mineralogical properties such as strength, stiffness and permeability as well as to aesthetic harmony. Durability means that the lifetime of the new material should be at least equal to the lifetime of the existing material. Highly knowledge is necessary about both new and existing materials for providing compatibility and durability (Penelis, 2002).

Using the same composition of traditional material for the repair and strengthening material is the best way to provide the compatibility and durability. However, it is not possible all the time due to various reasons such as the detailed analysis is difficult for recomposition of the existing material (Penelis, 2002 and Penelis et al., 1989). In addition, properties of existing material may not be convenient and sufficient. In that case, new material can be used. For using the new materials, insufficiency or unsuitability of traditional material has to be proved as per Venice Charter. New materials such as high strength steel, epoxy resin, stainless steel, various cements, FRP etc. are used for repair and strengthening of historical masonry structures (Sesigür et al., 2007).

Using impermeable mortar such as Portland cement together with porous units is one of the compatibility problems and it can cause problem. When the rain penetrates into the masonry, impermeable mortar bed collects the water and then it may cause the spalling due to subsequent frost effect. Another compatibility trouble is using modern strong mortar between masonry units. Since the mortar is strong, it would not permit slight sliding of units. Then, cracks will occur through the units. This is undesired because; repair of the units is more difficult than repair of the joints. In addition, it looks worse aesthetically (Beckmann, 1995).

4.2.4 Repair and Strengthening Techniques

There are several repair and strengthening techniques for historical masonry structures. Local strengthening of components particularly remedies the overall structural behavior. In addition, there are several overall structural strengthening methods, which affect the structural behavior positively.

4.2.4.1 Urgent Intervention

Sometimes, urgent temporary intervention is necessary for historical structures in case instability or fatigue risk due to excessive loading or earthquake. This should be minimum in order to prevent these interventions becoming permanent. These are hanging the structure, caring decorations and partial demolishing. Partial demolishing should be used in case there is no other method for providing structural stability (Sesigür et al., 2007).

4.2.4.2 Repair of Cracks

Primarily, it is necessary to determine if cracking is progressing or static. If it is static, the cracking reason probably will not recur, so cosmetic repair would be enough. Minor cracks, of which the cause has been eliminated, may only need filling and pointing in order to prevent the water ingress (Beckmann, 1995).

Masonry wall cracks, radial dome cracks and vault cracks are repaired. Crack repairing method changes due to crack wide and location. Steel or FRP can be used between crack edges for providing the tensile strength. In historical masonry structures, brick or stone particles is placed between two edges of the crack and then a proper mortar injection is applied. Using the similar property mortar to the original material is most common method for repairing thin cracks or repairing the cracks or gaps, which placed on the thick walls (Sesigür et al., 2007).

4.2.4.3 Repair and Strengthening of Masonry Walls

Cracks, out of plane rapture, buckling, junction separation, and frost expansion damages are formed in masonry walls under various loading patterns and conditions. Repair and strengthening techniques, in order to remove and prevent these damages, are given below.

- For crack repair, the injection is applied with tensile resisted elements, in case the crack wide is greater than 10 mm or if there are some fallen units in the wall. In this purpose, adjacent units are taken out, stitch or steel connection elements are placed and the gaps are filled with a proper mixture under low pressure. (Figure 4.8). After process, it should be controlled and if necessary, it should be repeated (Sesigür et al., 2007).

In some cases, even injection does not provide the reliable stress transmitting. In this instance, creating a vertical lintel along the crack may be helpful (Sesigür et al., 2007).

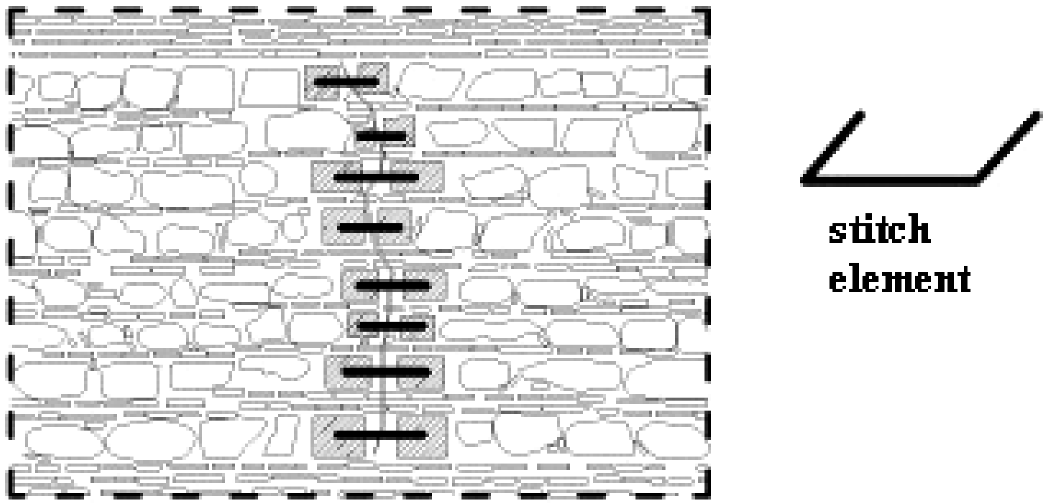


Figure 4.8: Repair of the wide crack in the wall (Sesigür et al., 2007).

- Walls with reasonable damage may be strengthened covering one or two face of the wall with the reinforcement mesh and performing shotcrete. First, the wall surface should be cleaned. The reinforcement mesh should be tied to either the wall or mesh of the other face. Then, shotcrete should be applied on the wall surface. Thickness of the concrete varies between 30 mm and 80 mm for various cases. Thus, shear capacity and ductility of wall increase due to section enhancement and reinforcement mesh. It is important to supply the enough anchorage for integration between added parts and existing walls to each other. The anchorage should be provided both in the same floor and between two floors. However, this method can be used rarely in historical structures, it may be used only for the walls that have no adornment. If necessary, this type of strengthening should be kept at minimum level for historical masonry structure (Sesigür et al., 2007 and Celep, 2005)

This process should be performed carefully. Due to increasing of the wall stiffness excessively, the component may collect great shear forces and this may cause the damage concentration at the strengthened walls (Sesigür et al., 2007).

- The lateral stability of a wall or pier depends on their thicknesses and vertical load imposing on them. Masonry walls may be added in order to increase the overall thickness of wall for providing the lateral stability of the walls (Beckmann, 1995)

- Corner separations of the walls can be combined creating reinforced concrete or steel lintels at upper wall levels or placing metal connection elements. (Figure 4.9) In addition, FRP bands can be used at corner junctions (Sesigür et al., 2007).

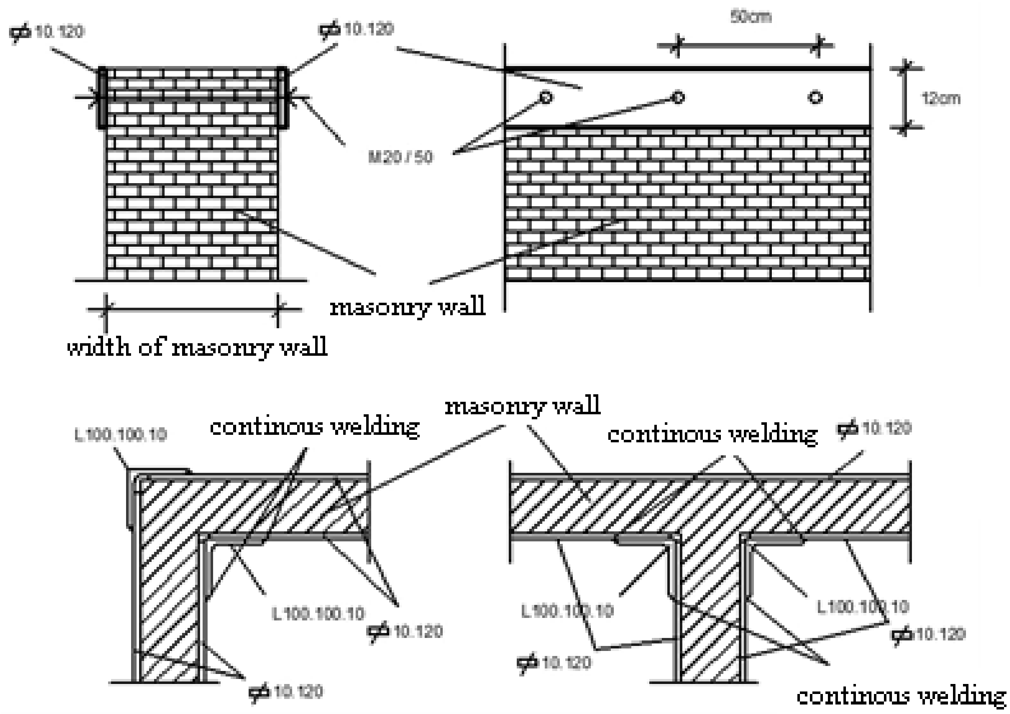


Figure 4.9: Steel lintels a) steel lintel arrangement (d) corner conjunctions of lintels (Sesigür et al., 2007).

- Walls can twist due to eccentric loads, high height/thickness ratio and insufficient lateral constraint. Figure 4.10 shows buckling damage and possible intervention for walls (Beckmann, 1995).
- Two wall surfaces of rubble core masonry should be tied each other. Cavities should be filled in order to prevent the water collecting and frost effect (Beckmann, 1995).
- In some structures, the façade wall and cross walls may not have proper bond and they may have even gaps of 20-30 mm between them. To provide the junction resistance, pre-cast concrete elbow ties may be inserted and bed them into the rebates with a similar mortar to the original one (Figure 4.11) (Beckmann, 1995).

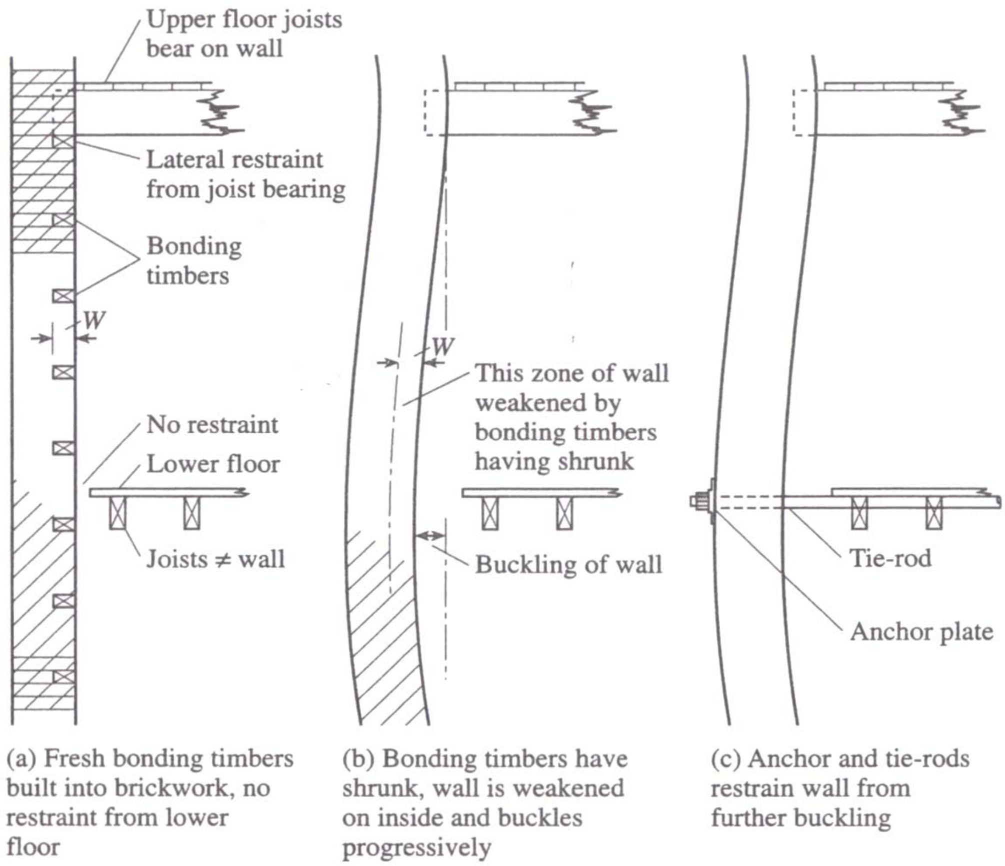


Figure 4.10: Buckling of wall and remedial anchoring (Beckmann, 1995).

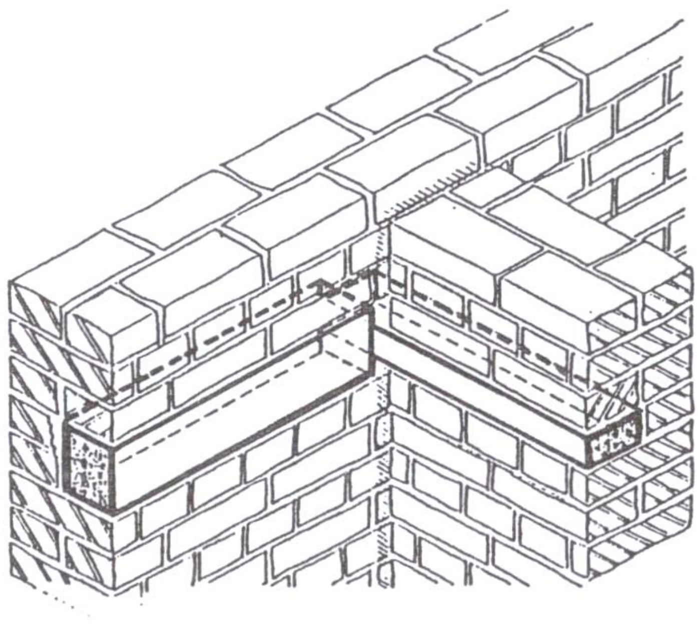


Figure 4.11: Strengthening of wall junctions with pre-cast concrete elbow ties (Beckmann, 1995).

- In order to increase the out of plane strength of the wall, vertical FRP band is placed on the middle part of wall. It increases the rupture load and ductility (Kanıt et al., 2006). In addition, strengthening of wall corners increases the out of plane strength.
- Arches, vaults and bad-supported domes produce lateral thrusts acting walls and piers. This thrust can be compensated creating buttress. However, the settlement under buttress support must be prevented to provide the load bearing of the buttress (Beckmann, 1995).
- The adobe walls can be covered with mud mortar. This protects the adobe walls against weather effects such as rain and wind (Öztürk, 2006).

4.2.4.4 Repair and Strengthening of Domes

Occurrence of radial cracks due to tensile stress on the lower part of the dome is typical dome damage. The best measure is creating a tensional ring on the underside of the dome (Figure 4.12) (Sesigür et al., 2007). Tightening the ring for closing of cracks is not true, because the particles, which are inside the crack gap, would prevent closing (Beckmann, 1995).

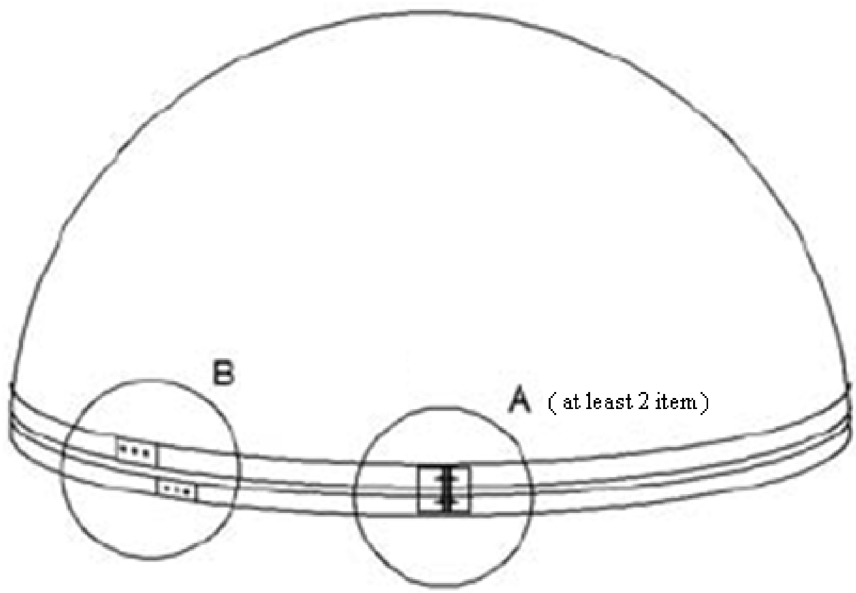


Figure 4.12: Creating a tensional ring at lower part of dome (Sesigür et al., 2007).

Stainless steel material should be used for tensional ring or the ring should be covered with concrete in order to prevent the corrosion effect. In case the dome has no damage, FRP material may be used for tensional ring (Sesigür et al., 2007).

4.2.4.5 Repair and Strengthening of arches and vaults

Earthquake and soil movement are main damage reasons for arches and vaults. Repair and strengthening ways can be aligned as follows:

- Buttress can be created in order to resist the lateral thrusts
- Ties are arranged between springing of arches and vaults in order to prevent spreading the abutments apart. Architecturally, it is not desirable. It can be arranged at invisible or visually unimportant level (Beckmann, 1995).
- Existing ties are rearranged and ties are got resistible to the horizontal thrusts (Sesigür et al., 2007).
- Fallen or crushed components are replaced (Sesigür et al., 2007).
- Cracks are filled with relatively weaker mortar (Sesigür et al., 2007).

4.2.4.6 Repair of Pillars

Pillar cracks can be remedied by confining circular reinforcement element. It increases safety of the pillar. (Figure 4.13) (Sesigür et al., 2007).

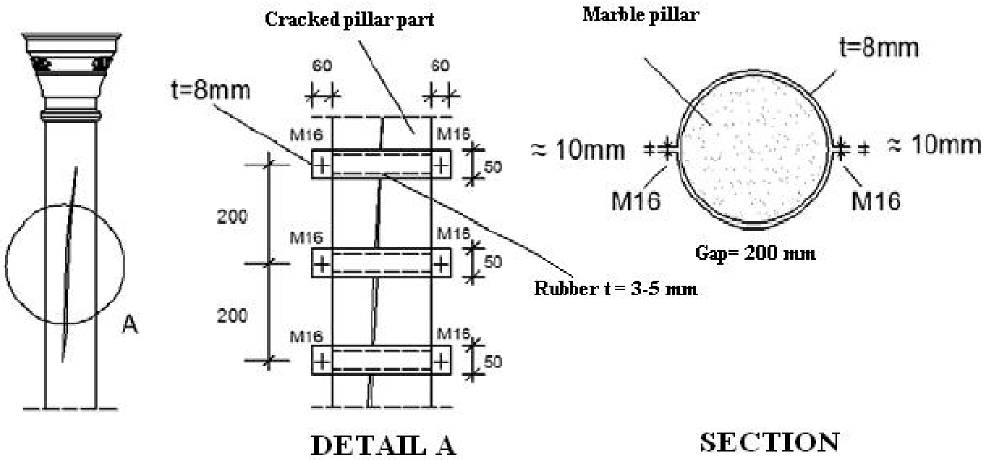
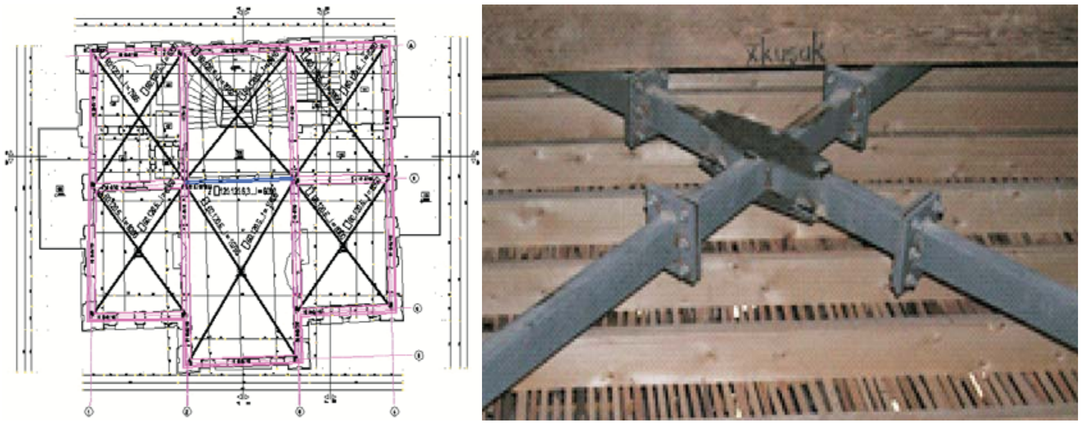


Figure 4.13: Repairing of the pillar crack (Sesigür et al., 2007).

4.2.4.7 Strengthening of Floors

Floors spread the lateral loads to the walls with diaphragm effect due to stiffness of walls. In order to increase the floor stiffness, reinforced concrete or steel lintels may be created. Horizontal steel truss provide stiffness like reinforced concrete slabs without any additional loads since it is lighter than reinforced concrete slabs. (Figure 4.14) (Sesigür et al., 2007).



(a) (b)

Figure 4.14: Developing stiffness of floors by steel trusses; a) Layout view and b) Detail of truss (Abdülaziz Hunting Mansion- Izmit/Turkey) (Sesigür et al., 2007).

4.2.4.8 Strengthening of the Foundation

If necessary, both foundation and soil should be strengthened. For a historical building, it is supposed that the settlement of the cohesive soil had to be finished a long time ago. However, if there is liquefaction risk in the soil, ground water level should be reduced several meters below the foundation. Much attention should be given for preventing the differential settlement. Other measures are enlarging the foundation width and transmitting the superstructure loads to the firm soil stratum (Sesigür et al., 2007). Injection can be applied to soil in order to decrease the soil permeability and to increase the soil shear strength (Öztürk, 2006). This process should be performed carefully. Otherwise, it may cause differential motions in the structure.

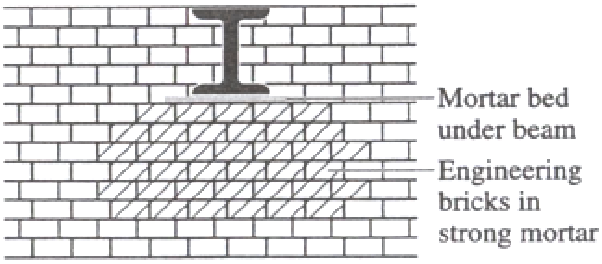
4.2.4.9 Overall Structural Strengthening

In addition to repair and strengthening of the components, there are several intervention methods in order to remedy the overall structural behavior given as follows:

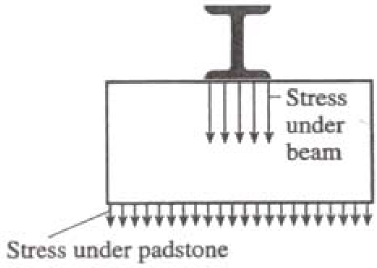
- Removing the overmuch weight from upper level of the vaults and soil roofs makes positive effect under earthquake loads due to decreasing the weight of the structure (Sesigür et al., 2007)
- Stiffness and mass distribution in historical masonry structures are generally regular. However, adding the new parts to the structure may change this regularity.

In this case, torsion disorder can be decreased creating pointing between different parts (Sesigür et al., 2007).

- Massive structures may have some weak parts such as a cloister of mosques. Two parts behave differently. Due to this, damage concentration may occur in the connection regions. Pointing arrangement between two parts is a good measure if possible. However, it is generally not likely in historical structures (Sesigür et al., 2007).
- Seismic isolation is another method. It decreases the structural stiffness and increases the period and damping. Therefore, the structure collects less lateral loads under seismic action. Seismic isolation is used in case the function of structure is important and the structure should continue its service (Sesigür et al., 2007). It is not convenient for adjacent building. If subsoil of structure is soft, seismic isolation may increase the earthquake loads (Öztürk, 2006).
- Inserting reinforced concrete or steel frames into the masonry is a mistake. It causes loss of aesthetic and recovering is not possible. In addition, the behavior of this composite system is generally unknown (Sesigür et al., 2007)
- Damaged wooden elements are replaced by new ones.
- In order to improve the structural behavior of a masonry tower structure, the masonry is confined by horizontal metallic reinforcements on several tower sections along the tower height. It improves the wall connections (Modena et al., 2002).
- In case changing of the building function, overstressing may occur at supports of beams, which located on walls. Spreading the stresses over a greater length of wall is a solution. It can be done rebuilding underside the support with strong bricks and bedding strong mortar on it. It reduces the stress concentration. It should not be used if the beam bearing is near the end of wall (Figure 4.15.a). The other solution is insertion of a padstone for spreading high bearing stress. However, these local strengthening change the nature and sometimes appearance of the masonry. (Figure 4.15.b) (Beckmann, 1995). Because of that, they should be used in case there is no other convenient solution for strengthening of historical masonry structures.
- In order to increase the structural interaction, ties can be placed between walls at roof level (Sesigür et al., 2007).



(a)



(b)

Figure 4.15: Solution of overstressing problem a) using strong bricks, b) using padstone (Beckmann, 1995).

5. CASE STUDY: MEHMET AGA MOSQUE

Mehmet Aga mosque located in Fatih, Istanbul, which is one of the most historical city in the world. Istanbul was the capital of several empires and it has many historic heritages from various civilizations such as Ottoman Empire and Byzantine Empire.

Mehmet Aga mosque was constructed in 1585 (Okçuoglu, 1994) (Figure 5.1). It has been in service since its construction date. The mosque belongs the Ottoman Empire time. It was commissioned by Mehmet Agha of Darussade. Architecture of the mosque is Davud Agha who is a student of great Ottoman Architect Sinan.

Figure 5.1: Mehmet Aga mosque.

The mosque was subjected to several sesismic effects since ever. The most important ones are the earthquake of 1766, 1894 and 1999 and so far it was subjected to two interventions; it was repaired in 1743 and restored in 1982 (D.İ.B Fatih Müftülüğü, 1991). In situ investigations are performed in order to determine the crack formations and damages. However, due to the previous maintenance and repair works, most of the cracks and damages, which occurred probably during past seismic effects, could not be seen; only the recent damages and cracks could be inspected during in situ investigations. Such as cracks at the edges of several windows, partly stone wall deterioration, tie damages were inspected. Any soil problem did not observe.

Three-dimensional Finite Element Model was prepared for the mosque using Sap 2000 software (Wilson and Habibullah, 1998). Various structural analyses were carried out under self-weight and earthquake loads. Although existing damages do not threaten the structural integrity of the building, some repair and strengthening interventions were recommended in order to prevent further damages.

5.1 Architectural Description

Mehmet Aga Mosque has a square shaped main part called "Harim". This part is covered with the main dome, which has a height of 17.3m and a diameter of 11.0m, and with five tromps. The pendentives are placed in between the main dome and tromps. The arches that bear the main dome are supported by four pilasters and by the four corners of body walls (Figure 5.2, 5.3, 5.4, 5.5, 5.6 and 5.7)

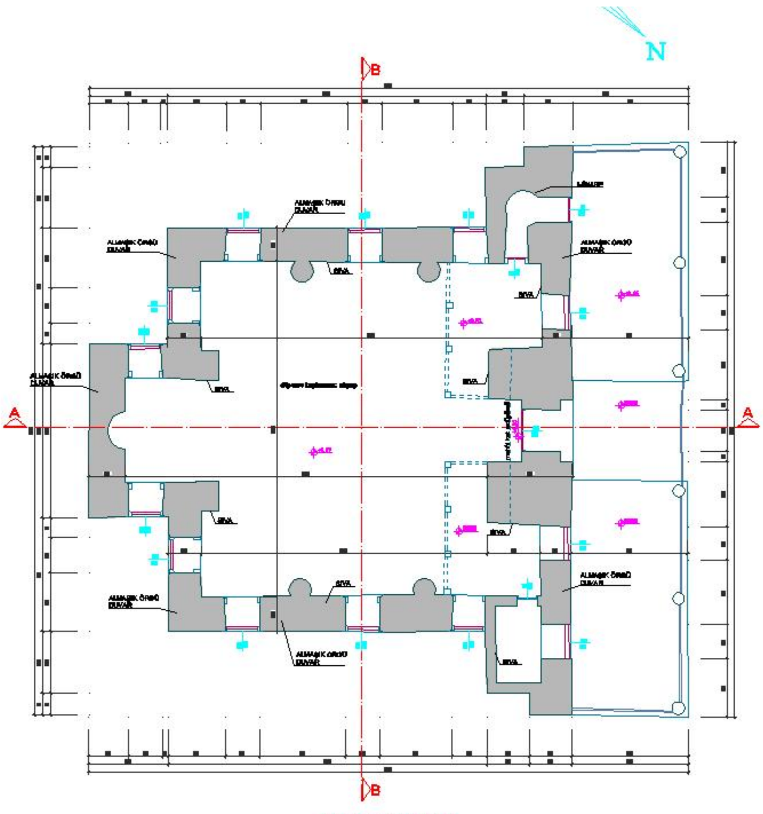
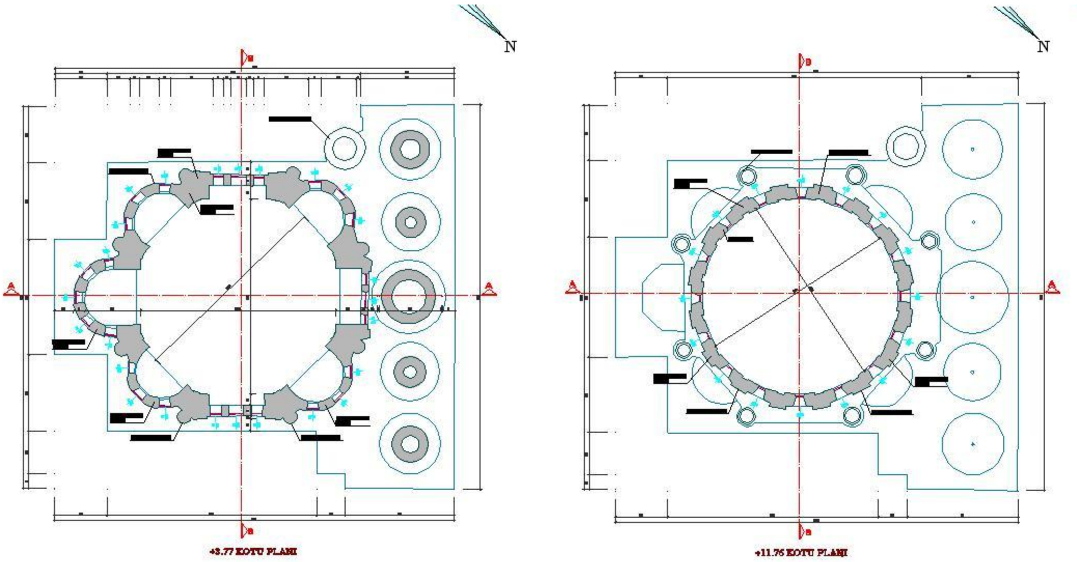


Figure 5.2: Plan of ground level



(a) (b)

Figure 5.3: Upper level plan views; a) +8.77 height, b) +11.76 height

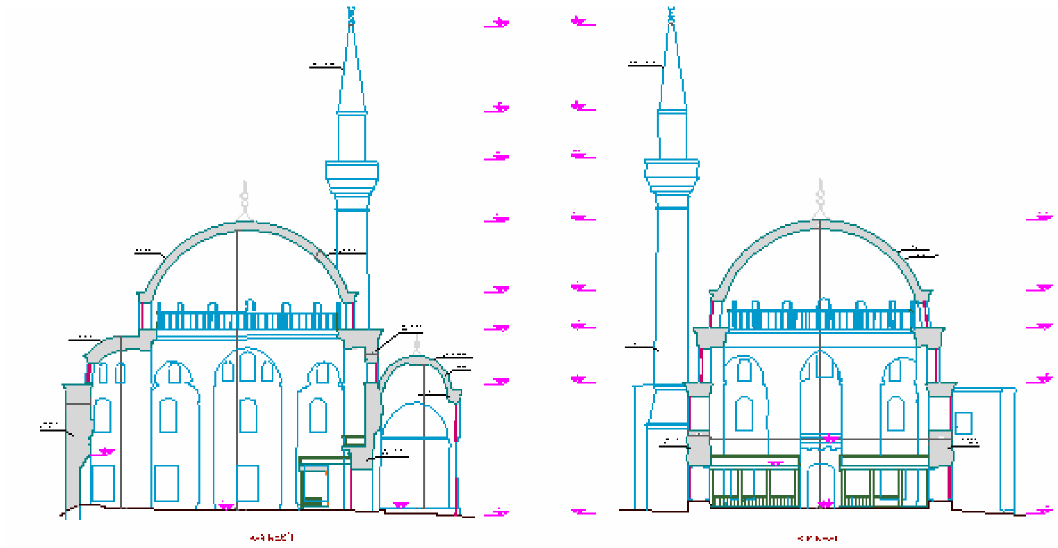


Figure 5.4: Section views; a) SE-NW section, b) NE-SW section

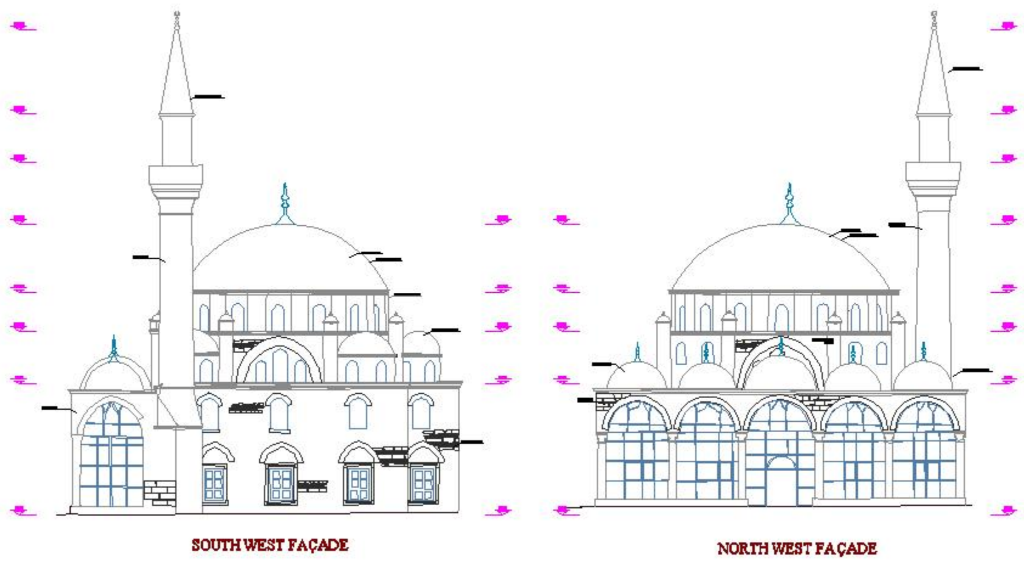


Figure 5.5: South-west and north-west façade views

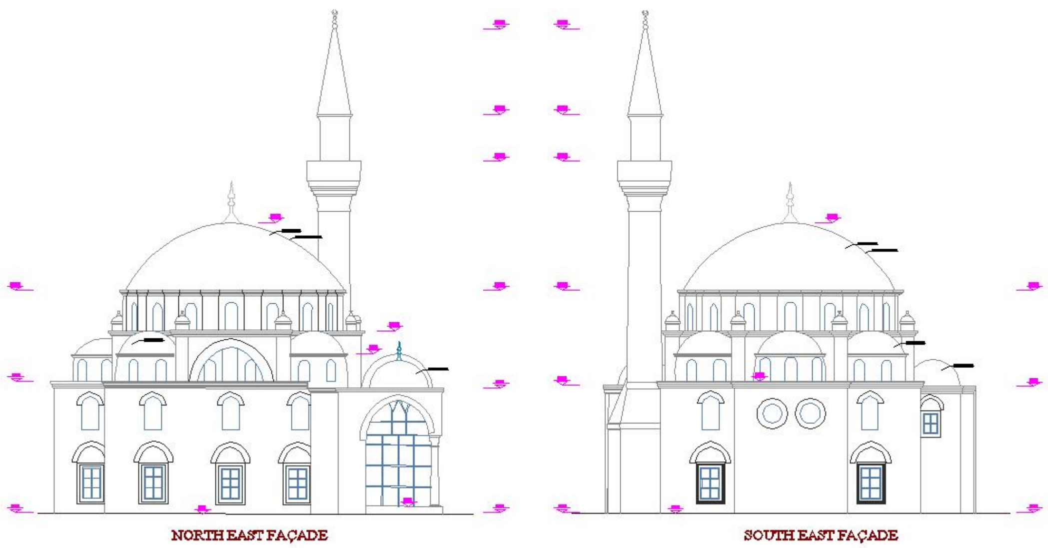


Figure 5.6: North-east and south-east façade views

The mosque has a wooden interior balcony having a height of 2.8 m. External body walls of the building consist of one stone row alternated with three brick rows, which called “almaşık örgü” and its average thickness is 1.3 m. The main dome is made of brick. The cloister of the mosque consists of five parts covered by five small domes, which are supported by the six marble columns having stone heads through the arches (Figure 5.8). All arches have iron ties. The mosque has a minaret of 29 m height. The minaret is made of ashlars. Although most of them were changed, there are still some original nice ceramics, beautiful calligraphies and handicrafts on the walls (Figure 5.9).

Figure 5.7: Mosque interior; arches, tromps, pendentives and arch ties

Figure 5.8: Front façade- the cloister of the mosque.

Figure 5.9: Calligraphies, handicrafts and ceramics

5.2 Observed Damages

The mosque has experienced Istanbul earthquakes of 1766, 1894 and 1999. It was repaired in 1743 and restored in 1982. In addition, since the mosque is still in service, maintenance works is being done continuously. Because of that, damages, which occurred due to the past earthquakes, could not be observed completely. Furthermore, no damage in the soil of the building is observed. The observed damages can be aligned as follows:

- Some ashlars are deteriorated in the cloister of the mosque (Figure 5.10).

Figure 5.10: Stone deterioration on the wall of the cloister.

- Few slight radial cracks are observed on one of the tromps.
- Generally, the edges of windows openings have cracks (Figure 5.11).

Figure 5.11: Cracks of window edges

- There are buckling damages at connection points of some iron ties (Figure 5.12).

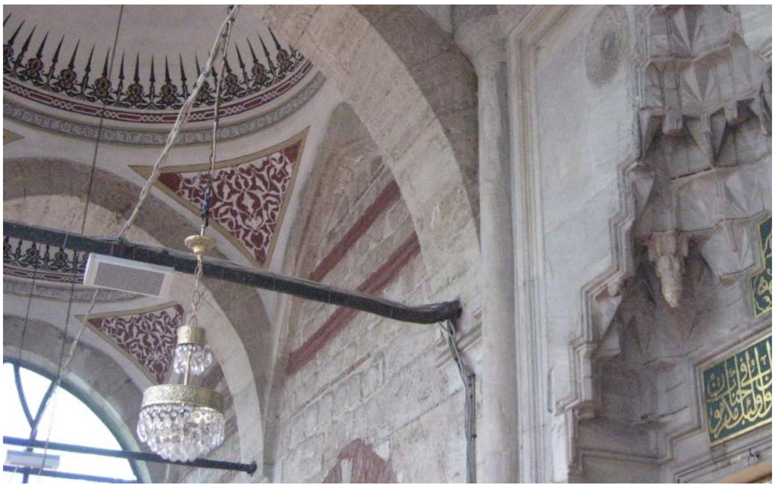


Figure 5.12: Buckling damage of iron ties.

5.2 Seismicity of Area

Istanbul is located in highly seismic region. It is so close to the North Anatolian Fault Line, which is a major earthquake source of Turkey. There are various estimations showing that a great earthquake will probably strike to Istanbul in very near time. Seismic Risk Zone Map of Turkey and Istanbul are given in Figure 5.13 and 5.14 (Url-1, Ministry of Public Works and Settlement, Government of Republic of Turkey, 1996)

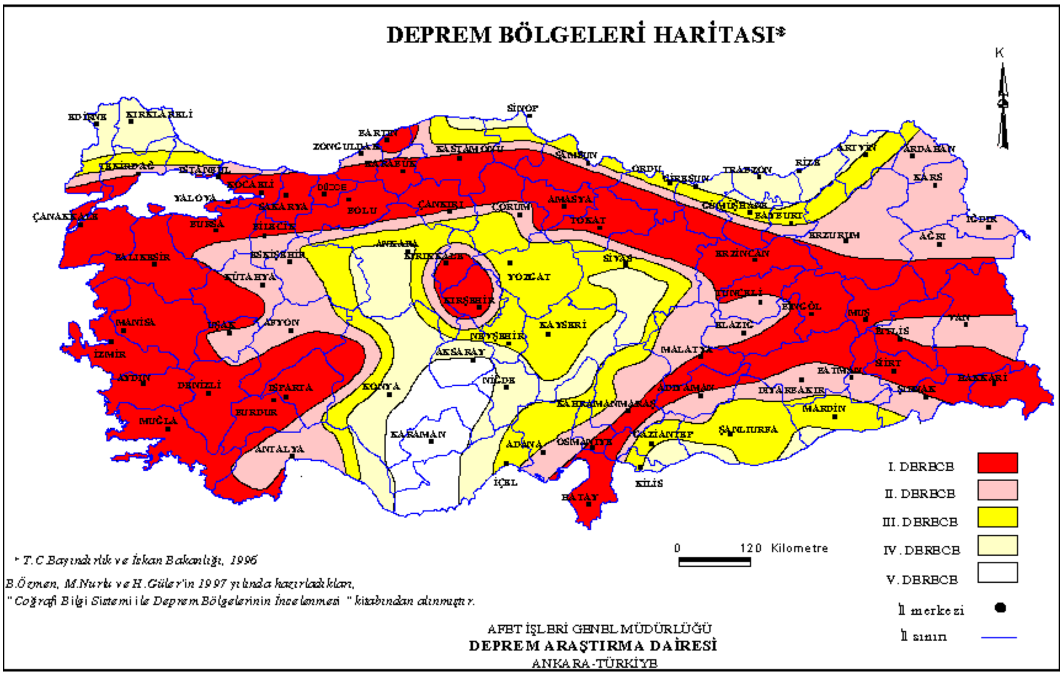


Figure 5.13: Seismic risk map of Turkey (Url-1)

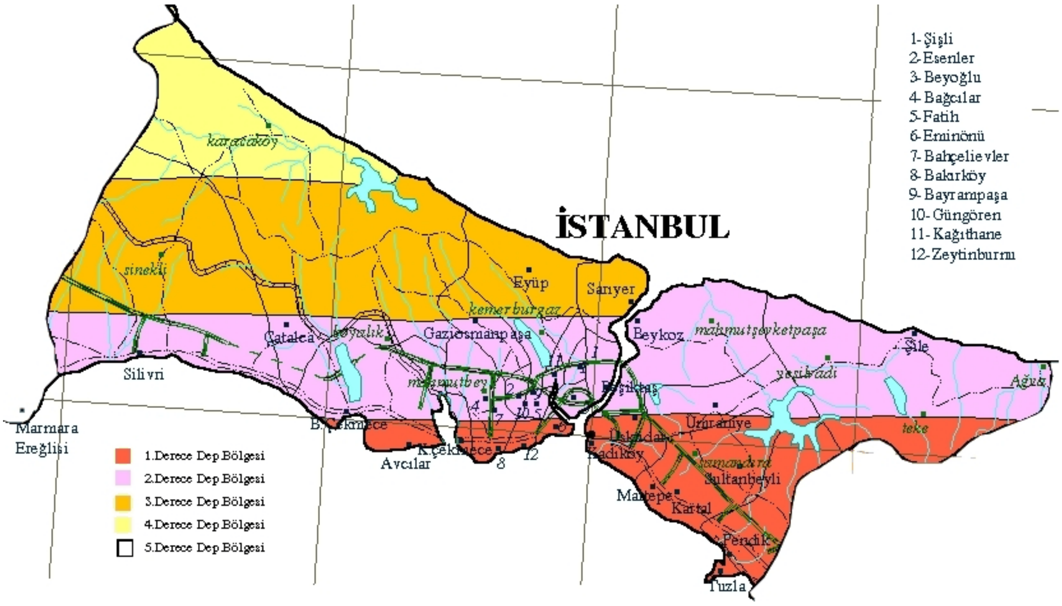


Figure 5.14: Seismic risk map of Istanbul (Url-1)

In fact, the building is located in the second highest seismic risk zone according to the Turkish Seismic Zone Map (Url-1, Ministry of Public Works and Settlement, Government of Republic of Turkey, 1996). However, the building is assumed located in the highest seismic risk zone, which has 0.4 g maximum effective ground acceleration within DBYBHY 2007 (Ministry of Reconstruction and Resettlement**,** 2007).

5.3 Soil Condition

The mosque is located in higher placement; the subsoil surface does not any slope. The foundation of the building located in Gungoren formation, which is one of the well known soil formation of Istanbul. The subsoil of the building is composed of firm-stiff consistency, high plasticity clay (CH). Partially, there are gravel and sand strata. Geotechnical investigation has been performed for this location by digging exploratory pits and this investigation has been carried out by Istanbul Metropolitan Municipality. SPT values for Gungoren formation range between 20 and 40. Soil stiffness can taken as $K = 20\,MN/m^3$ for analyses evaluating the soil properties and SPT values. Soil properties are given as follows as a whole:

Specific gravity of soil, $\gamma = 18\,kN/m^3$

Cohesion coefficient, $c = 60\,kN/m^2$

Shear friction angle, $\phi = 10^o$

Soil class Z3 within the Turkish Seismic code 2007; characteristic soil periods are $T_A = 0.15\sec$ and $T_B = 0.60\sec$.

5.4 Numerical Analysis of the Structure

5.4.1 Material Properties

The mosque consists of ashlars, brick and mortar in between units. Main components of the building are body walls, domes, tromps, pilasters and pendentives. In fact, each component has different material composition; body walls are composed of the mixture of brick and stone masonry and the walls of cloister consist of ashlars. The main dome is made of brick masonry. However, the building is assumed to be made of an equivalent single material having the same elastic modulus, unit weight and Poison's ratio. Unified material properties for analyses are determined after literature survey and compared to the properties of the same time-period and similar type structures, and they are given as follows:

Elastic Modulus, $E = 2\,GPa$

Poison's ratio, $\upsilon = 0.2$

Unit weight, $\gamma = 20\ kN/m^3$

Other material properties:

Unit weight of wood (wooden balcony), $\gamma_w = 7\ kN/m^3$

Elastic modulus of iron, $E_i = 180\ GPa$

Unit weight of iron, $\gamma_i = 78.5\ kN/m^3$

Poison’s ratio of iron, $\upsilon_i = 0.3$

Unit weight of lead (lead covers), $\gamma_l = 114\ kN/m^3$

Unit weight of marble, $\gamma_m = 27\ kN/m^3$

Since it is difficult to determine the stiffness of foundation exactly, sensitivity analyses were carried out using different foundation of stiffness values (Güler et al., 2004). In addition, modal analyses were performed for different elastic modulus values in order to determine the effects to results of analyses.

5.4.2 Structural Analysis Model

Historical masonry structures usually have very complex geometry. Thus, it is necessary to make some idealizations when preparing the analysis model. The three-dimensional analysis model is developed using the architectural plans and sections of the building, which is drawn in AutoCAD format (Autodesk, Inc). Then, the structural model is prepared in Sap2000 structural analysis software (Wilson and Habibullah, 1998) (Figure 5.15).

Finite element model of the structure is composed of masonry body walls, a main dome and small domes, arches, tromps and pilasters. External thick masonry walls were modeled by three-dimensional solid elements whereas, the domes, tromps, thick arches and pendentives were defined by two-dimensional shell elements. Frame elements were used for the pilasters, thin arches and iron ties.

Generally, it is not easy to determine reasonable number of finite elements to obtain response of the structural system with rational accuracy. In this study, totally 2418 solid elements, 1530 shell elements and 201 frame elements were used for constituting the structural analysis model of the mosque.

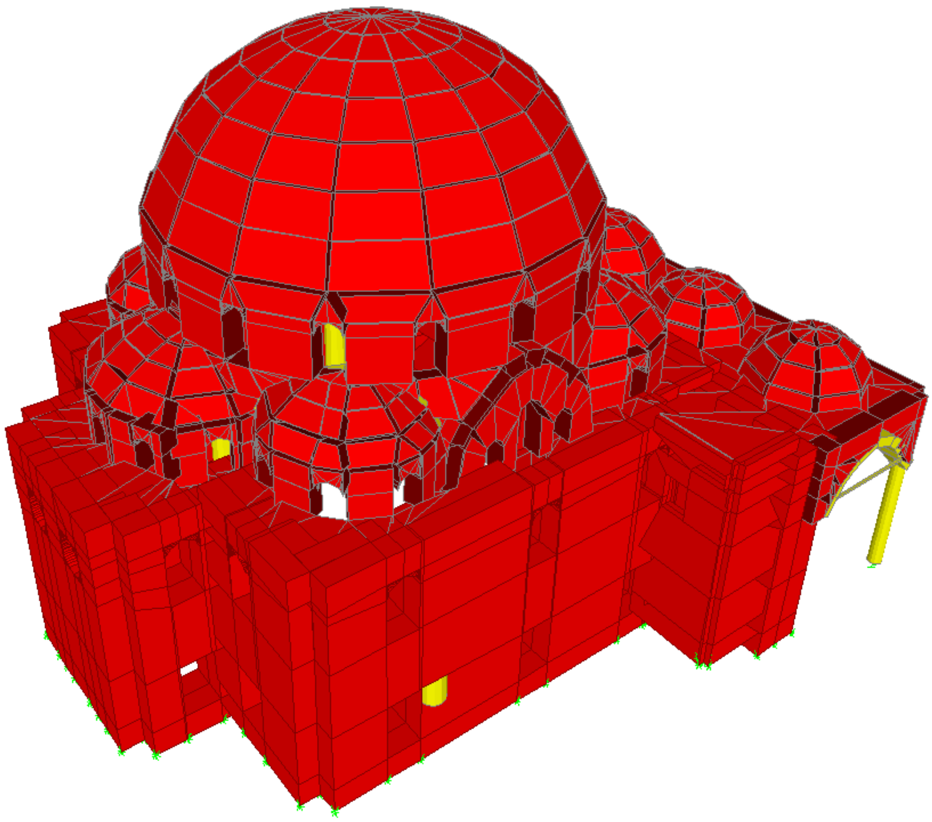


Figure 5.15: Sap2000 structural analysis model

During the modeling, various minor simplifications were made in order to keep the number of the finite elements of the model to avoid geometric complexity of the structure. However, maximal attention was given in order to obtain the real behavior of the building and for identifying the structural response with an acceptable accuracy.

5.4.3 Self-Weight and Earthquake Analyses of the Structure

Self-weight and lateral seismic loads analyses were carried out elastically due to relatively brittle behavior of masonry structures for insight of the structural behavior.

First, free vibration modal analysis was performed to obtain the modal periods, frequencies and mode shapes. Then, self-weight analysis was carried out under gravity loads. Seismic analyses were performed using the response spectrum of DBYBHY 2007 and obtained stresses were combined with those responses of gravity loads (Ministry of Reconstruction and Resettlement, 2007).

Snow load was taken as $0.75\,kN/m^2$ (Region 2) within TS498 Design Loads for Buildings (TS498, 1997). The weight of the chandelier hanging on the main dome assumed as 5 kN.

5.4.3.1 Free Vibration Analysis

Free vibration analyses were performed and first five natural periods and frequencies are given Table 5.1.

Table 5.1: Free vibration modal periods (s)

Mode	1	2	3	4	5
T (Sec)	0.382	0.352	0.201	0.193	0.185
f (Hz)	2.618	2.841	4.975	5.181	5.405

Elastic modulus and foundation stiffness, which were used for analyses, are supposed values. Because of that, sensitivity analyses were carried out using various E and K values in order to determine the effects of these values on analyses results. Comparison of modal periods for various elastic modulus and foundation stiffness values are given in Table 5.2.

Table 5.2: Comparison of modal periods

Mode	1	2	3	4	5
T (K = 20 MN/m^3, E = 2GPa)	0.382	0.352	0.201	0.193	0.185
T (K = 20 MN/m^3, E = 1GPa)	0.446	0.418	0.254	0.244	0.213
T (K = 20 MN/m^3, E = 3GPa)	0.354	0.323	0.197	0.159	0.153
T (K = 10 MN/m^3, E = 2GPa)	0.480	0.437	0.277	0.193	0.185
T (K = 30 MN/m^3, E = 2GPa)	0.339	0.315	0.187	0.180	0.167
T (Fixed support, E = 2GPa)	0.229	0.221	0.174	0.147	0.140

As expected, the periods increase with decreasing the elastic modulus and foundation stiffness. However, the effect of the foundation stiffness is much more than the elastic modulus.

Sensitivity analysis showed that changing the elastic modulus and foundation stiffness would not affect the spectral earthquake loads significantly, because period values mostly range between characteristic soil periods. In this interval, spectrum coefficient is the same and maximum value of S (T) = 2.5. Figure 5.16, 5.17, 5.18, 5.19 and 5.20 indicate the first five mode shapes.

As seen Figure 5.14 and 5.15, the first two mode shapes occur in the two orthogonal directions. The first mode shape occurs in the northeast - southwest direction and the second mode shape occurs in the southeast- northwest direction. First two modes have almost the same natural period. It shows that the rigidity and participating mass of first two modes are very close to each other. However, the higher mode shapes are too complex to make comments about them.

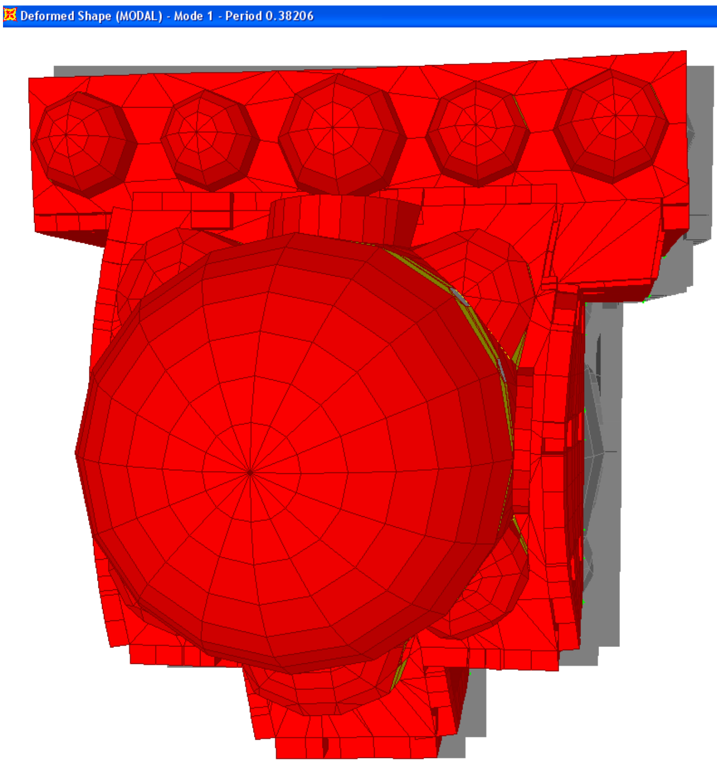


Figure 5.16: First mode shape

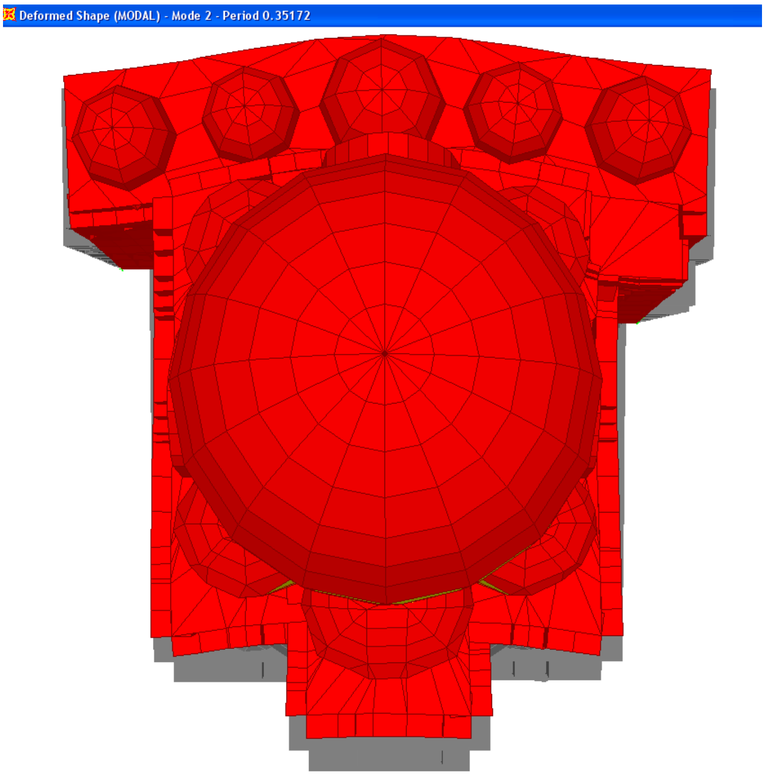


Figure 5.17 Second mode shape

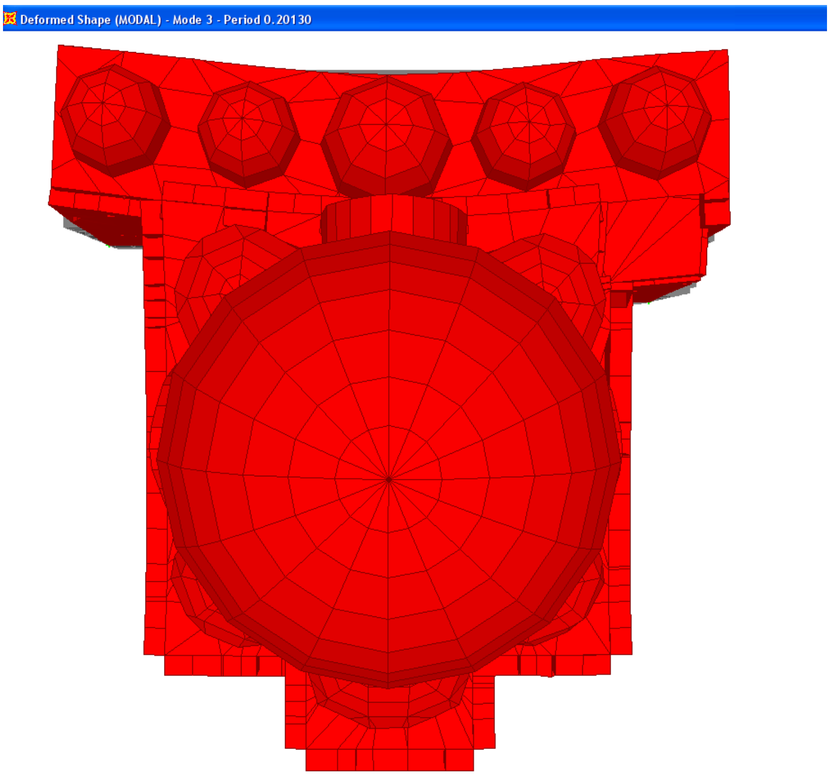


Figure 5.18: Third mode shape

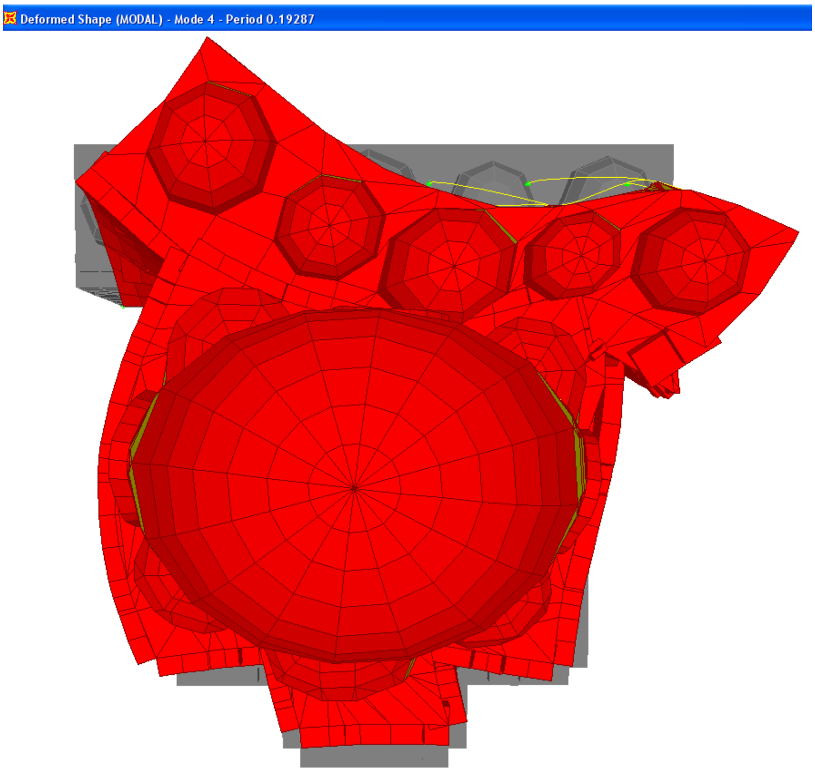


Figure 5.19: Fourth mode shape

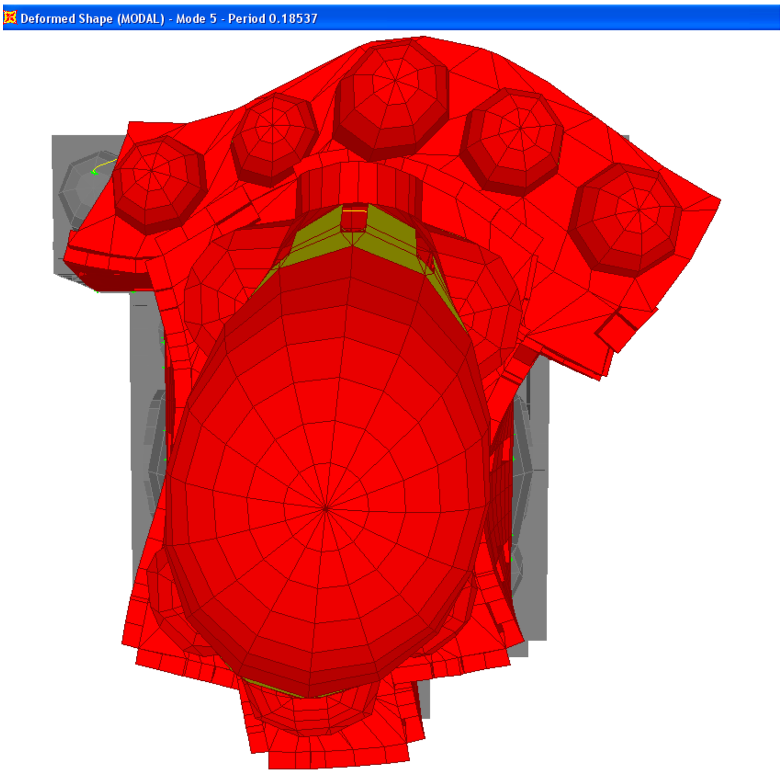


Figure 5.20 Fifth mode shape

5.4.3.2 Self-Weight Analysis

Linear elastic self-weight analysis has been performed in order to understand the structural behavior under gravity loads. Live loads and dead loads are included in the analysis (Figure 5.21).

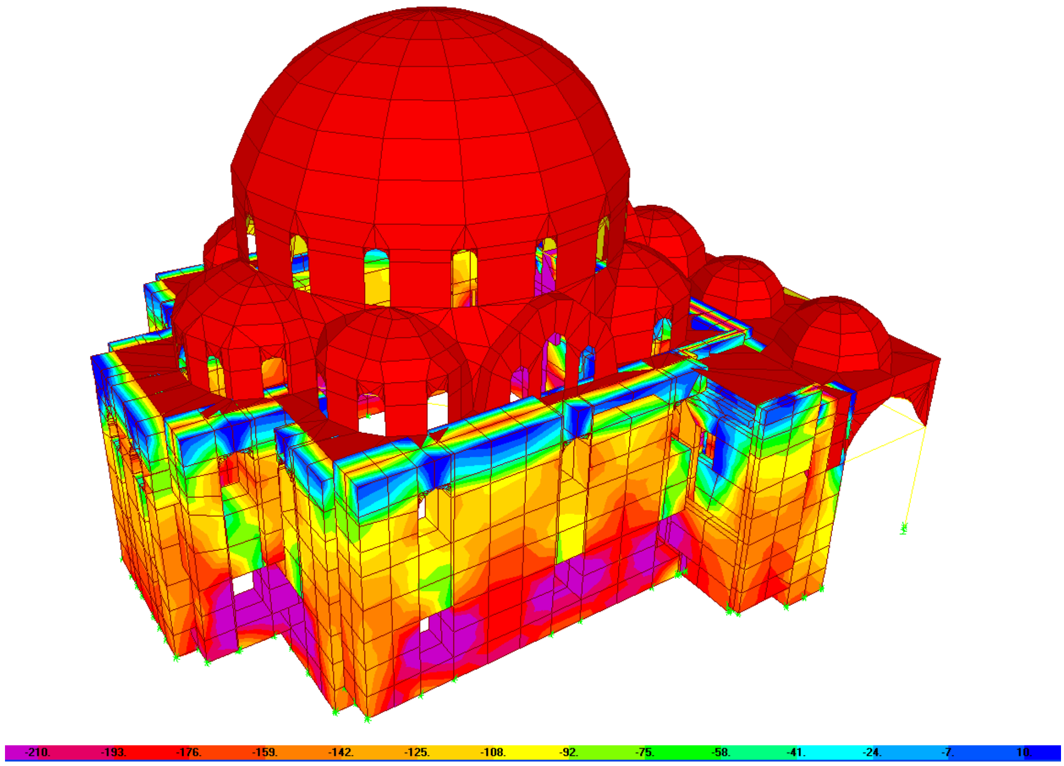


Figure 5.21: Self-weight analysis- Vertical stresses (kN/m^2)

Vertical normal stresses in the solid masonry walls of the building are given Figure 5.22, 5.23 and 5.24. As expected, the vertical normal stresses increase up to down. The compressive stress at the bottom level of the body walls is about 0.20 MPa. In addition, stress concentrations are observed on the edges of openings. The compressive stress on the edges of the below openings level increases up to 0.28 MPa. Small tensile stress occurs at the top wall levels because of the oblique loads transmitting by the dome and tromps and the insufficient vertical normal loads.

Elastic modulus and foundation stiffness do not affect the vertical stresses notably. As expected, the effect of the foundation stiffness may become significant, when the distribution of the foundation stiffness is not uniform along the subsoil surface.

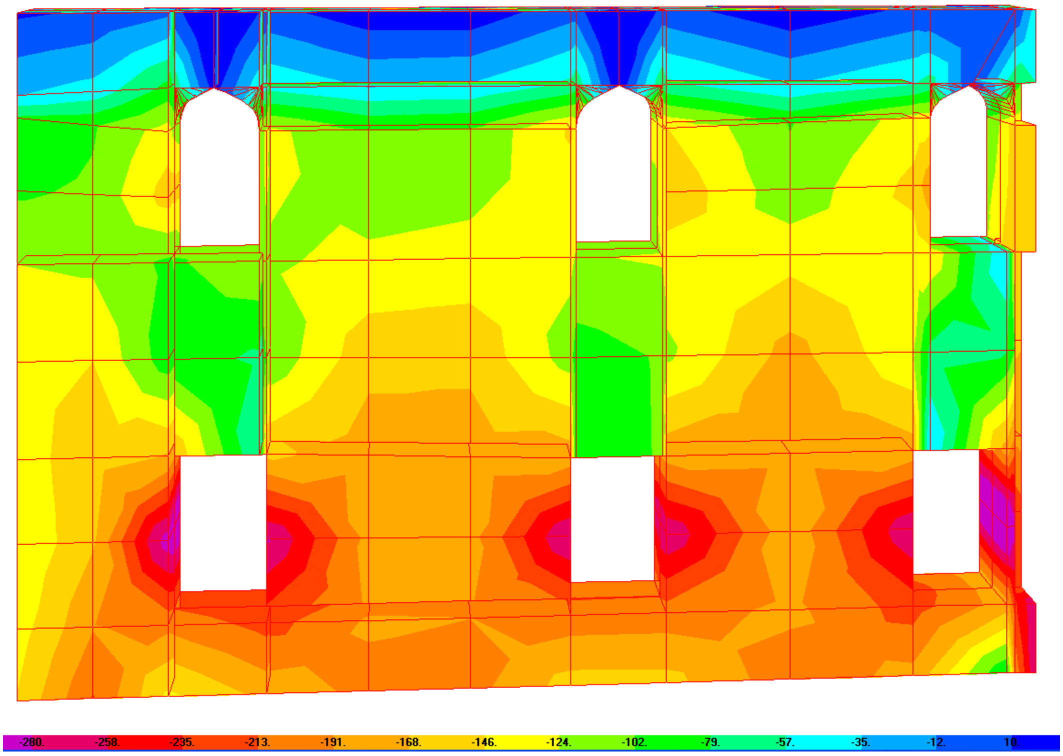


Figure 5.22: Vertical normal stresses in the northeast wall of the building (kN/m^2)

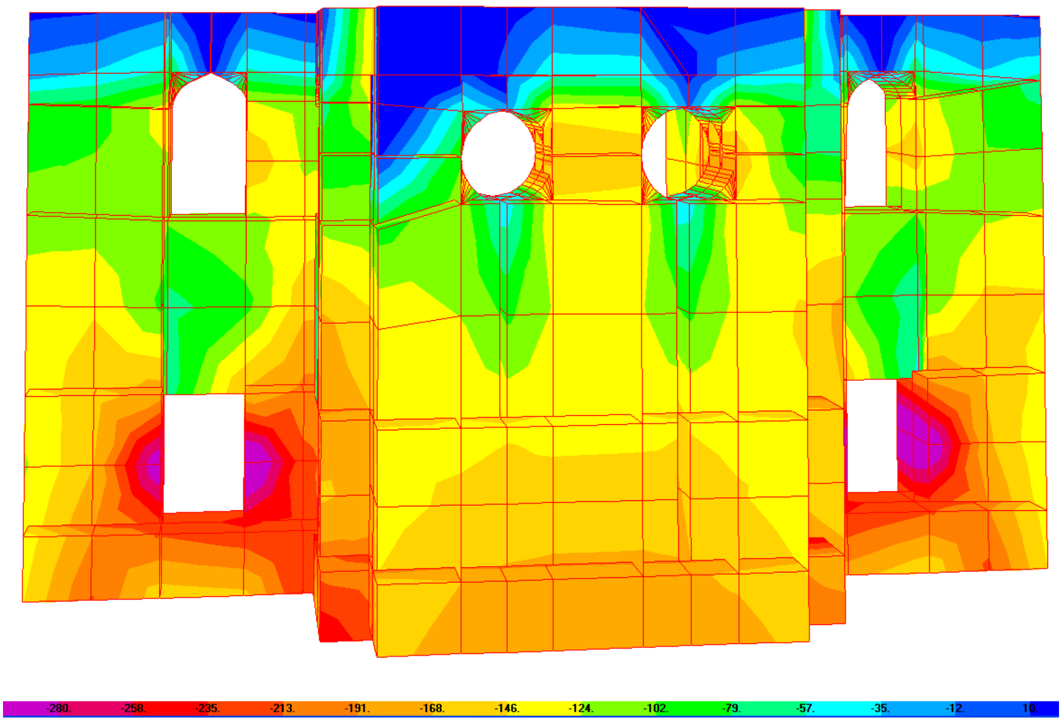


Figure 5.23: Vertical normal stresses in the southeast wall of the building (kN/m^2)

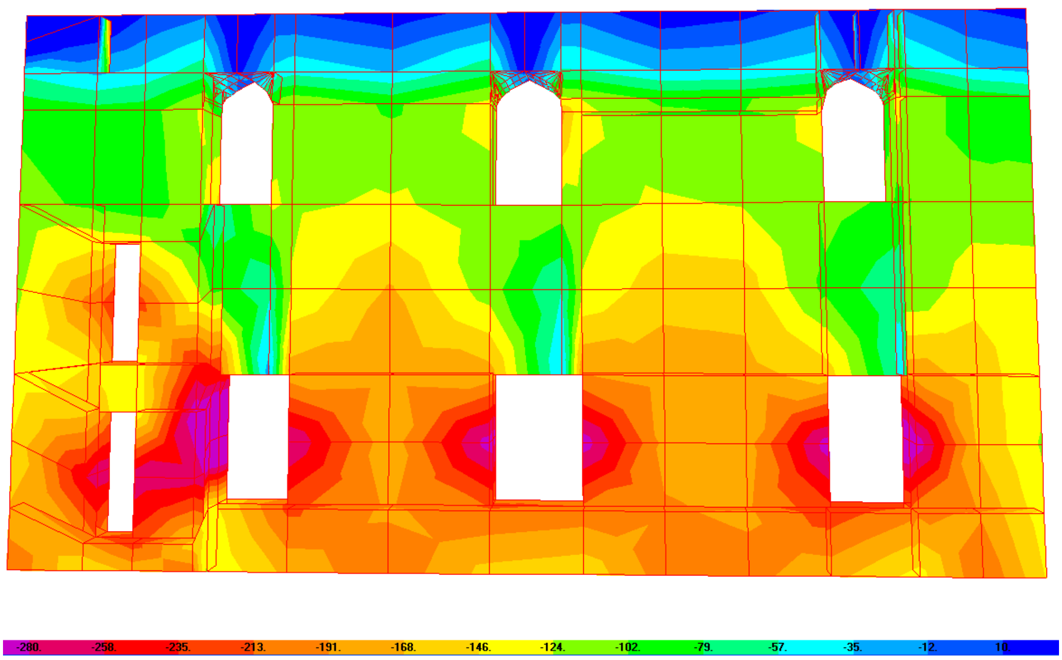


Figure 5.24: Vertical normal stresses in the southwest wall of the building (kN/m^2)

Figure 5.25 shows the stress contours of the main dome along the meridians. The dome stresses along meridians increases up to down. The compressive stress on the bottom of the main dome is about 0.20 MPa.

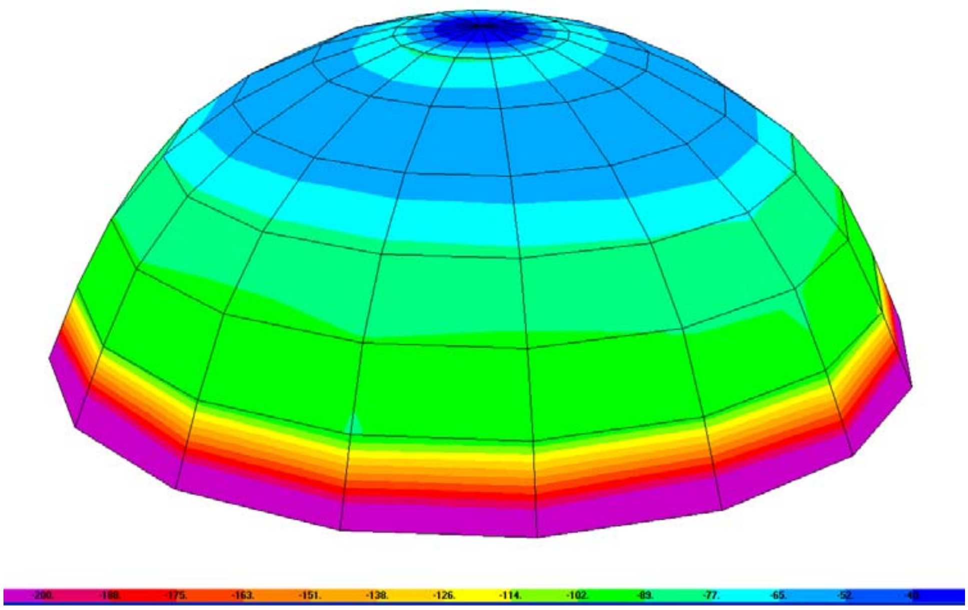


Figure 5.25: The stresses along meridians of the main dome (S22) (kN/m^2)

As mentioned in Chapter 3, the tensile stresses occur along circumferential direction at the lower part of the domes. Variation of the tensile stresses can be seen in Figure 5.26 on the bottom part of the dome and its maximum value is 0.05 MPa.

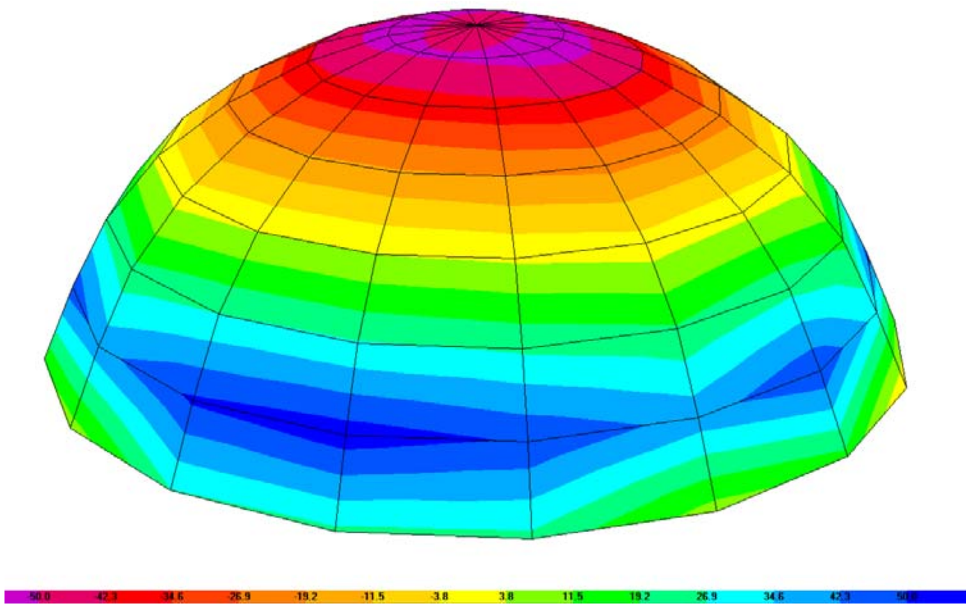


Figure 5.26: The stresses along circumferential direction of the main dome (S11) (kN/m^2)

Four pilasters have an average compressive stresses of 0.2 MPa. On the other four wall jags, which are the other support points of the main dome, stress concentrations are observed.

5.4.3.3 Seismic Analyses

Seismic analyses were performed by using the response acceleration spectrum of DBYBHY 2007 (Ministry of Reconstruction and Resettlement**,** 2007). Since the building is historical structure, taking the earthquake loads as the same with ordinary buildings is not convenient. Ordinary buildings are supposed to be designed smaller time projection (Sesigür et al., 2007). However, historical heritages should be standing along the centuries. Because of that, in addition to existing spectrum, an increased spectrum was used for analyses, which has smaller exceeding probability and greater return period (Table 5.3)

Table 5.3: Comparison of two spectrums

Spectrum	Exceeding Probability	Return Period
DBYBHY 2007 spectrum (EQ10)	50 years, % 10	475 years
Increased DBYBHY 2007 spectrum (EQ2)	50 years, % 2	2475 years

EQ2 is obtained multiplying the ordinates of the normal spectrum (EQ10) by 1.5 (Ministry of Reconstruction and Resettlement, 2007).

Mode superposition method was used for computation which given in DBYBHY 2007 and Complete Quadratic Combination (CQC) rule were applied to combine the modal responses (Ministry of Reconstruction and Resettlement, 2007).

The earthquake analysis parameters within Turkish Seismic Design Code (DBYBHY, 2007) are given as follows (Ministry of Reconstruction and Resettlement, 2007):

- Seismic load reduction factor, $R_a = 2$
- Building importance factor, $I = 1.2$
- Live load participating factor, $n = 0.3$

Spectral analyses were carried out in two principal directions and they were combined with self-weight.

Maximum compressive stresses under self-weight and EQ10 are concentrated at the openings, corners and wall jags, which are the supports of the main dome (Figure 5.27 and 5.28). The maximum compressive stresses at supports are about 2.2 MPa.

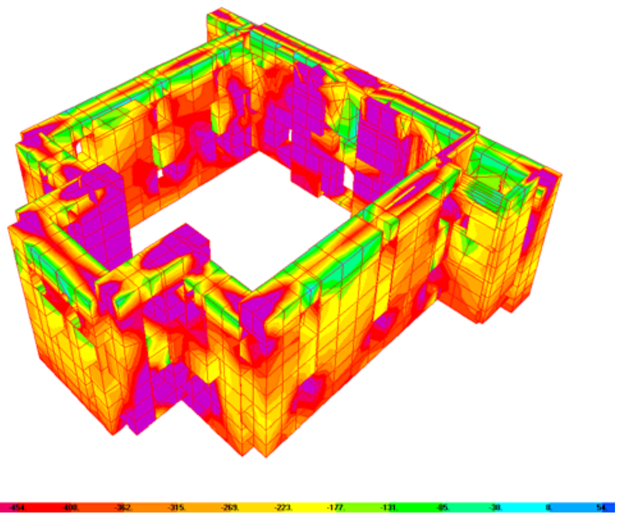


Figure 5.27: Maximum compressive stresses under self-weight and EQ10 (kN/m^2).

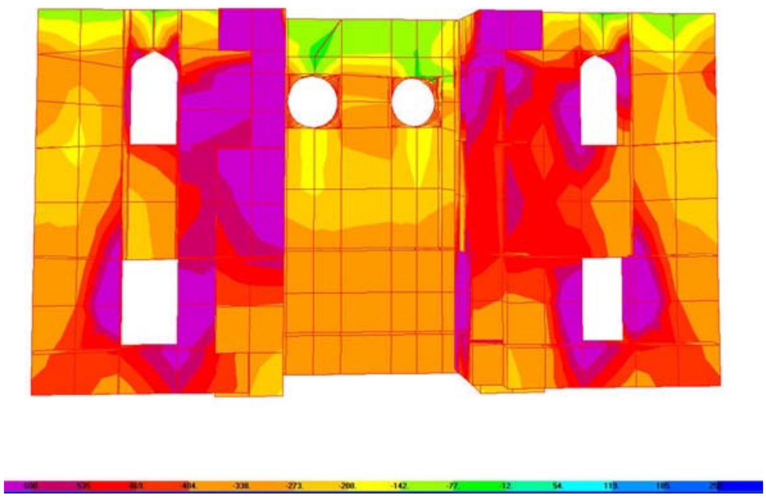


Figure 5.28: Maximum compressive stresses under self-weight and EQ10 at southeast wall (kN/m^2).

Maximum tensile stresses under self-weight and EQ10 are concentrated at the openings, corners and wall jags and upper wall levels (Figure 5.29 and 5.30). Upper wall levels are more vulnerable, since they have not enough vertical loads acting on them. Maximum tensile stresses at support points are about 2.0 MPa.

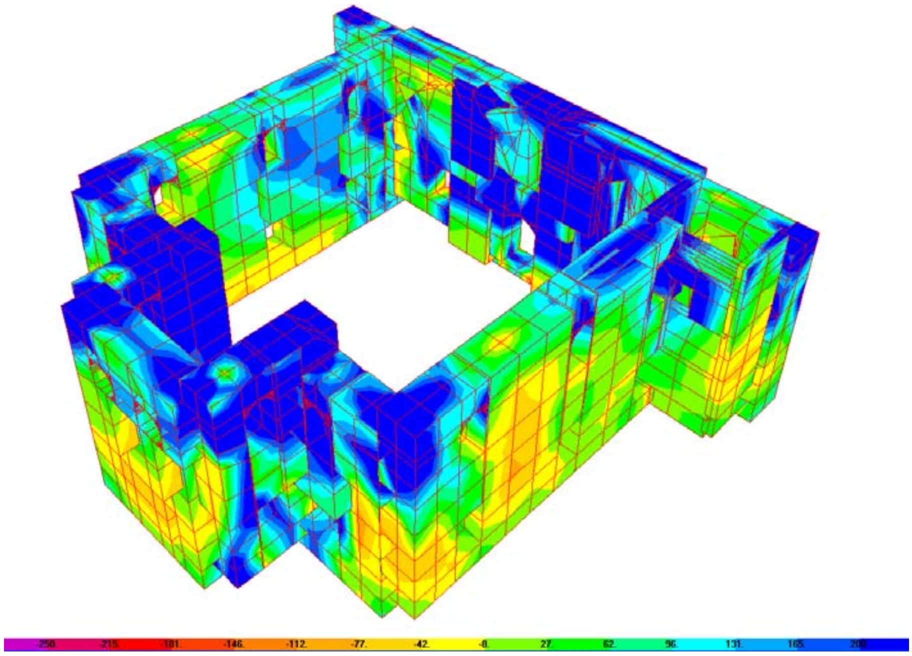


Figure 5.29: Maximum tensile stresses under self-weight and EQ10 (kN/m^2).

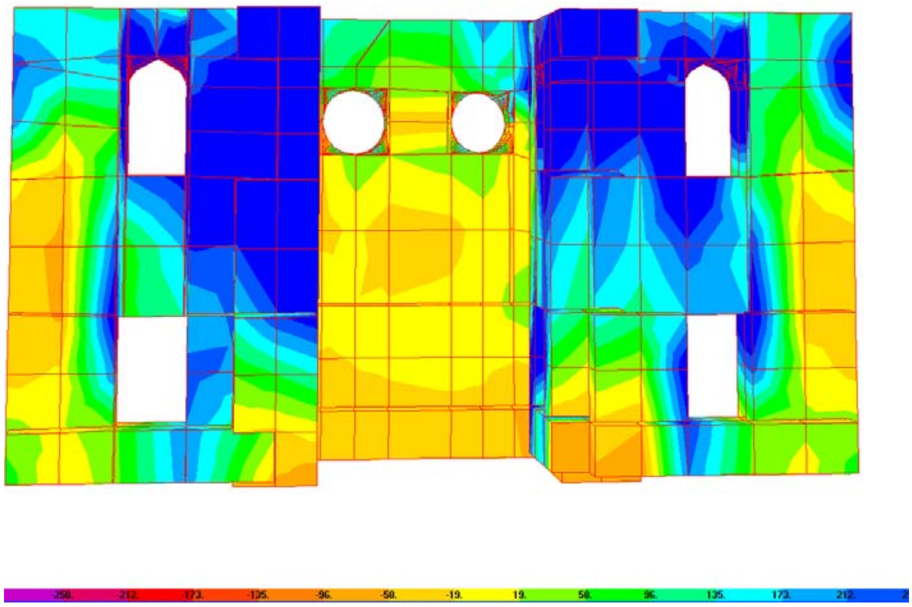
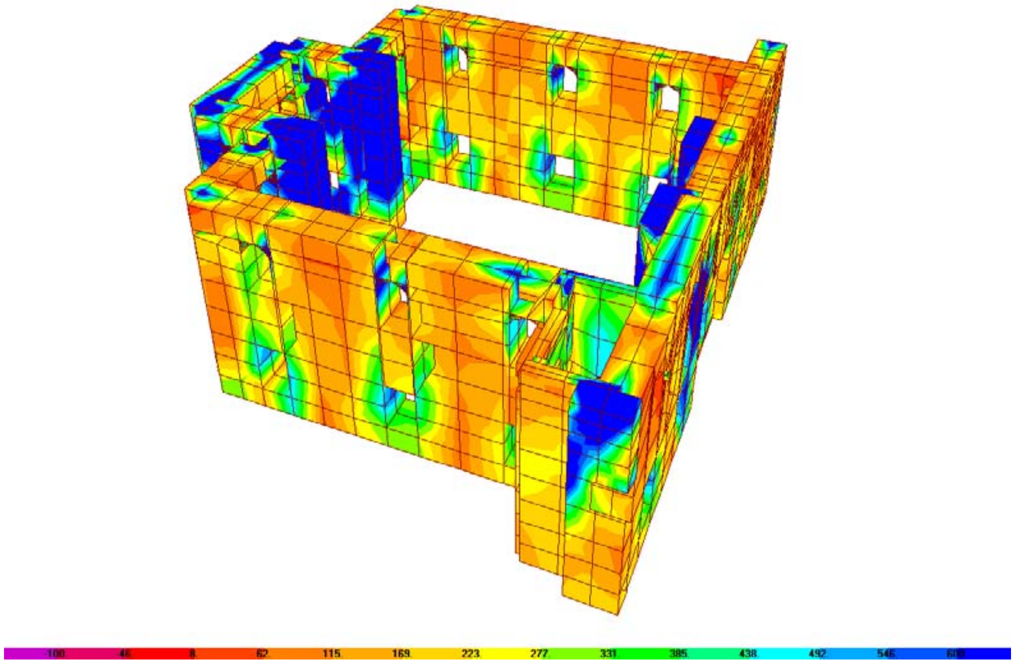


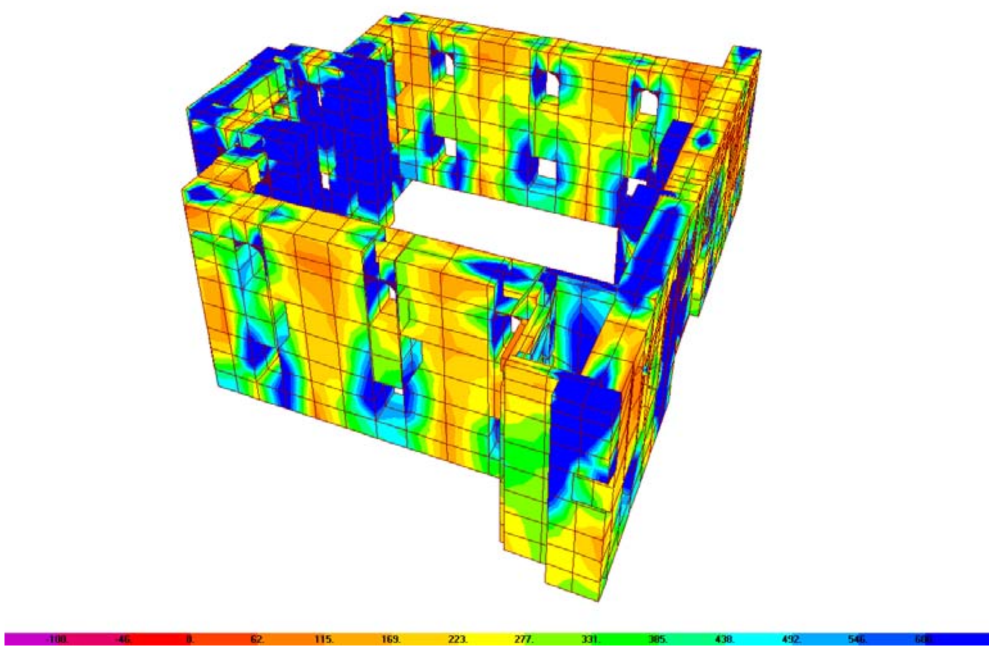
Figure 5.30: Maximum tensile stresses under self-weight and EQ10 (kN/m^2).

Locations of the tensile stresses are so important for masonry structure because it has low tensile strength. Stress concentrations around the openings may explain the cracks, which are observed in situ investigations.

Figure 5.31 and 5.32 shows the comparison of S33 vertical stresses for EQ10 and EQ2 (earthquake loads only). Blue color shows maximum tensile stress where the compressive stresses are presented by pink color. As expected, stresses for EQ2 are higher than EQ10. Stress increasing is about 40-50% between two different earthquake loadings.

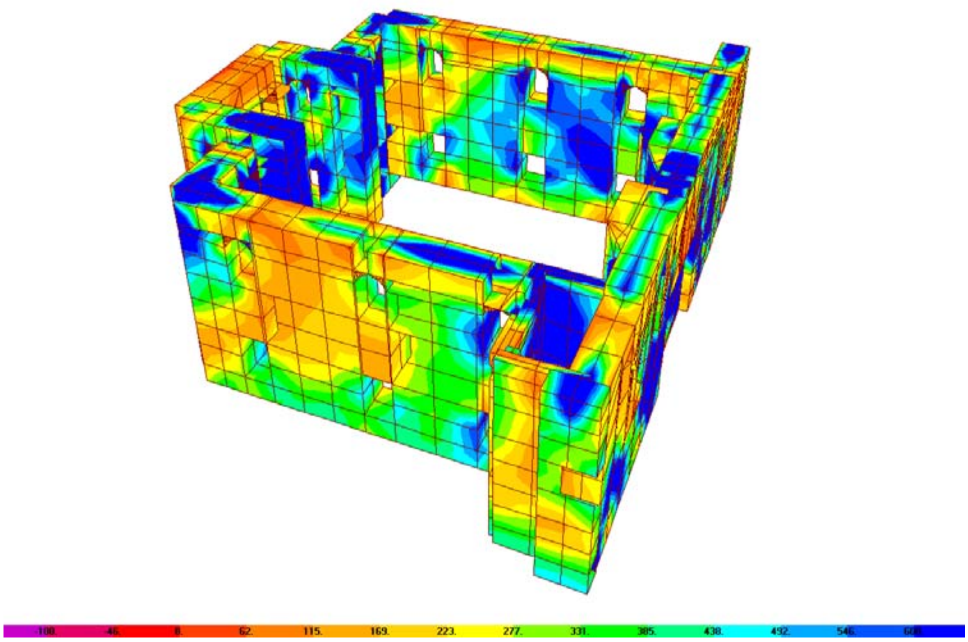


(a)

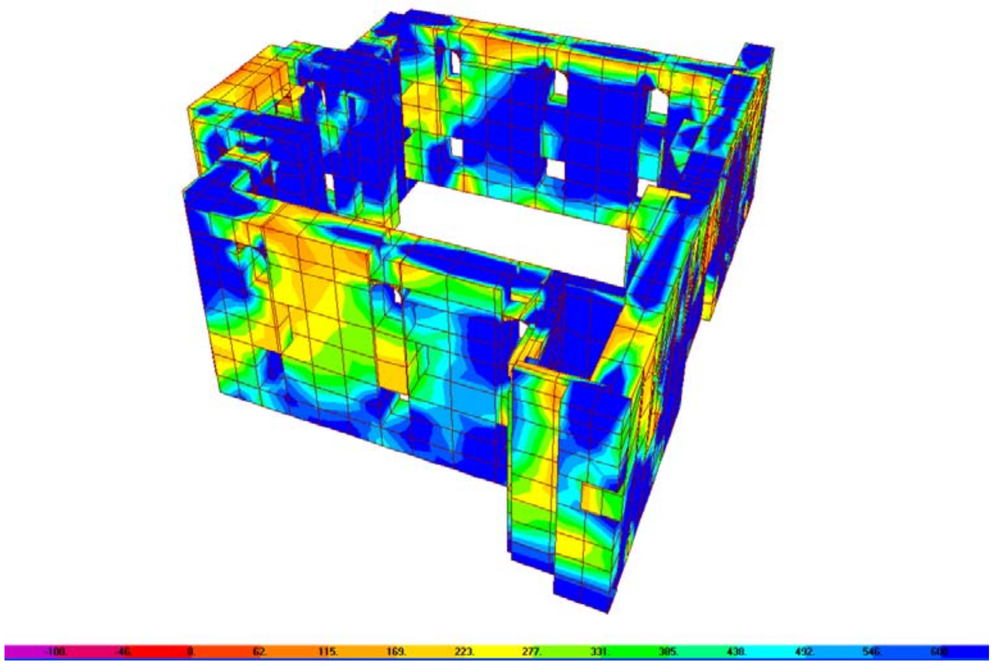


(b)

Figure 5.31: Comparison of the S33 vertical stresses for SE-NW (X) earthquake direction (kN/m^2), a) EQ10, b) EQ2



(a)



(b)

Figure 5.32: Comparison of the S33 vertical stresses for NE-SW (Y) earthquake direction (kN/m^2), a) EQ10, b) EQ2

Maximum tie stresses are about 44 MPa in compression and 80 MPa in tension.

Maximum compressive stress along meridians (S22) of the main dome is about 0.3 MPa for EQ10 and self-weight loading (Figure 5.33). Maximum tensile stress along the circumferential direction (S11) is over 0.35 MPa on lower part of the main dome (Figure 5.34).

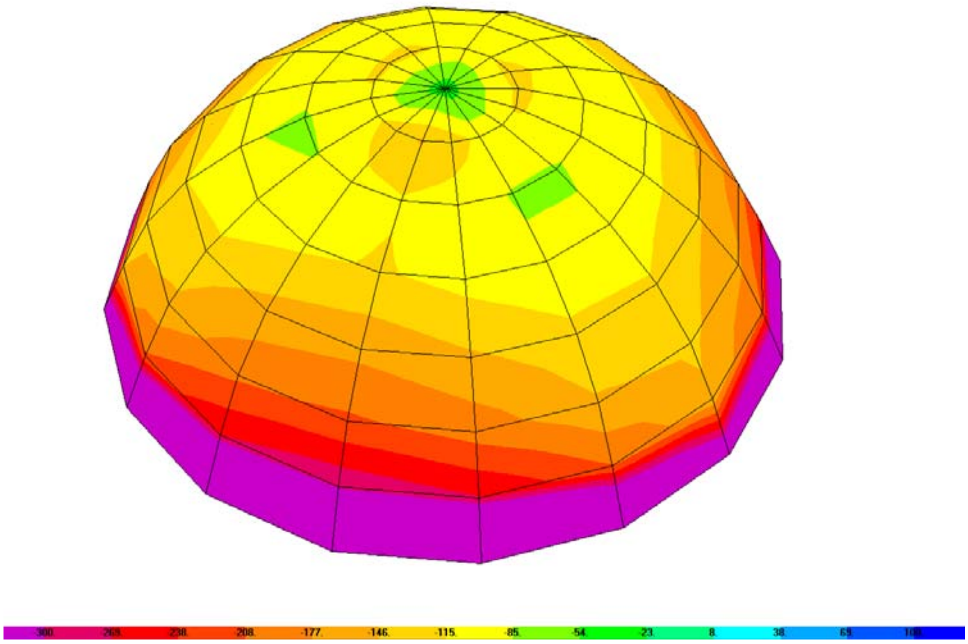


Figure 5.33: Stress contours of the main dome along meridians (kN/m^2)

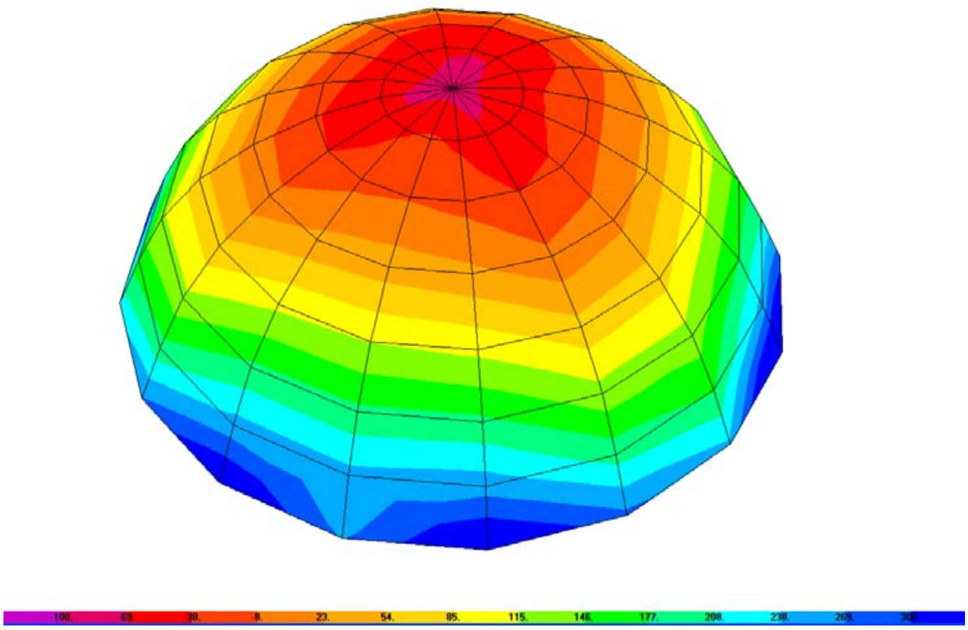


Figure 5.34: Stress contours of the main dome along circumferential direction (kN/m^2)

Tromps, pendentives and drum stresses along meridians (S22) can be seen in figure 5.30 and 5.31. In compression, stress concentrations are formed on the pendentives and the main arches, especially on the NW main arch. The maximum compressive stress is over 1.5 Mpa (Figure 5.35). The maximum tensile stress is about 1.5 Mpa. Stress concentrations occur at main arches and drum sitting points (Figure 5.36)

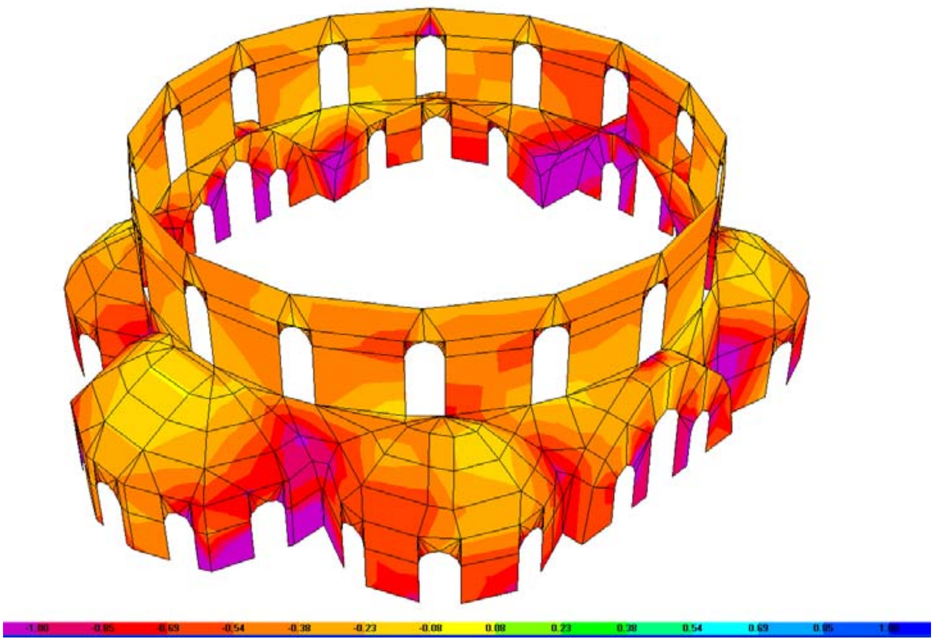


Figure 5.35: Stress contours along meridians for maximum compression (S22) (kN/m^2)

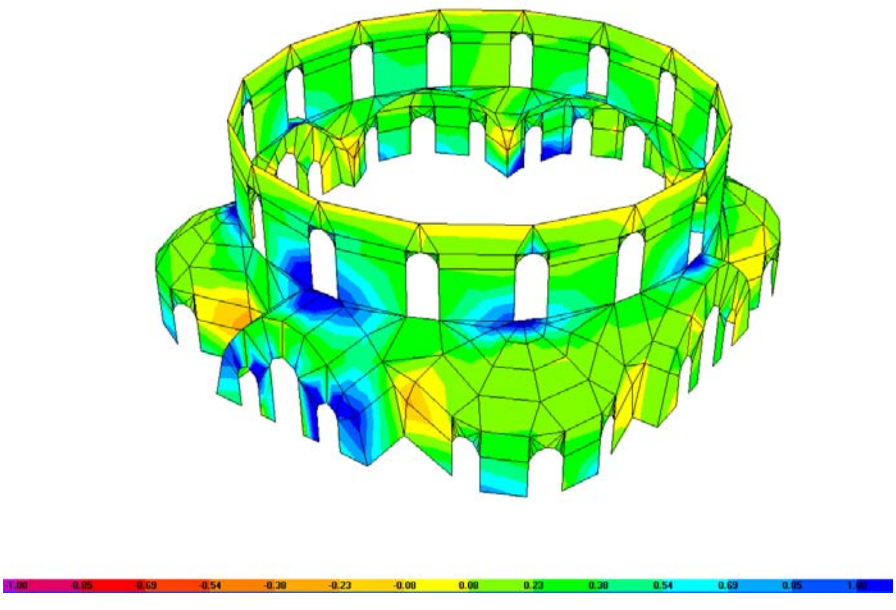


Figure 5.36 Stress contours along meridians for maximum tension (S22) (kN/m^2)

Circumferential stress contours (S11) are given in figure 5.37 and 5.38. In compression, stress concentrations occur on main arches and junction points of drum, tromps and pendentives (Figure 5.32). Maximum tensile stress on NW main arch is about 1.8 Mpa. Tensile stresses increase on the openings and NW main arch (Figure 5.33)

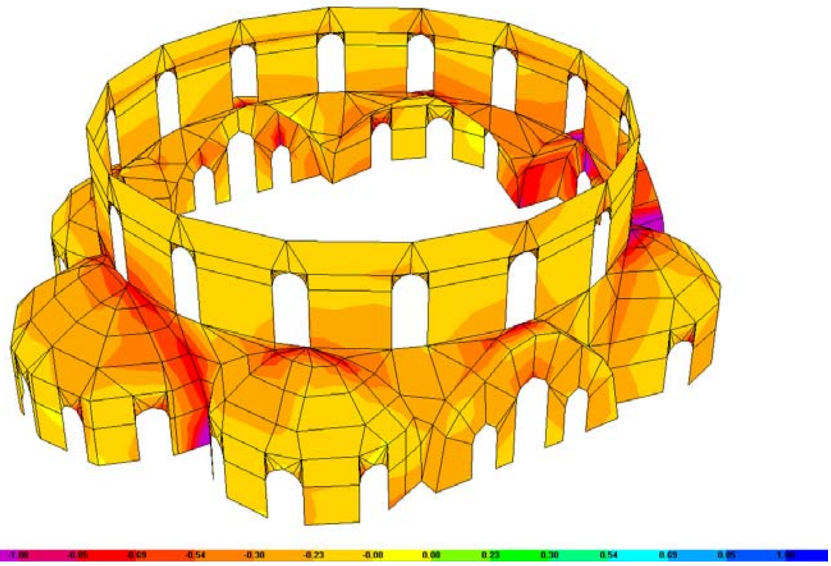


Figure 5.37: Stress contours along circumferential direction for maximum compressive stress (S11) (kN/m^2)

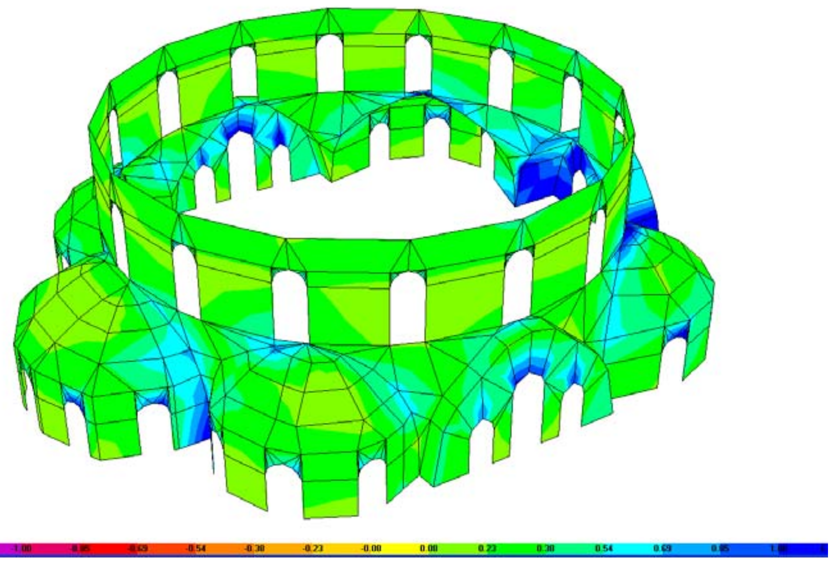


Figure 5.38: Stress contours along circumferential direction for maximum tensile stress (S11) (kN/m^2)

Maximum soil settlement is 2 cm under EQ10 and self-weight loading. It increases up to 2.8 cm under EQ2 and self-weight loading.

5.5 Repair and Strengthening of the Building

One of the main interventions for domes is arrangement of tensional ring at the bottom level of the dome. In this study, dome strengthening was made using tensional ring in order to decrease the circumferential tensile stresses. Figure 5.39 and 5.40 shows the comparison of dome circumferential stresses without and with the tensional ring. As seen figures, tensile stresses decrease significantly when tensional ring is used. Blue color shows maximum tensile stresses.

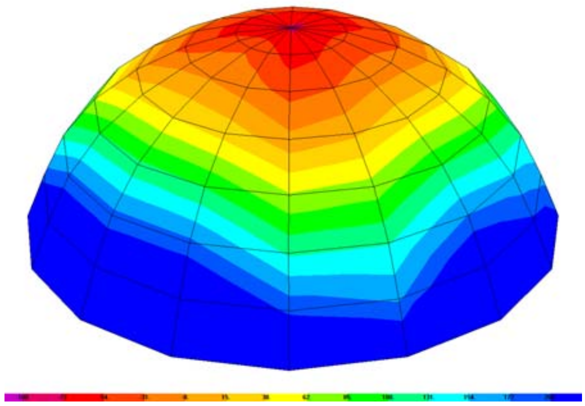


Figure 5.39: Maximum stress contours of the main dome without tensional ring (SW+EQ10) (kN/m^2)

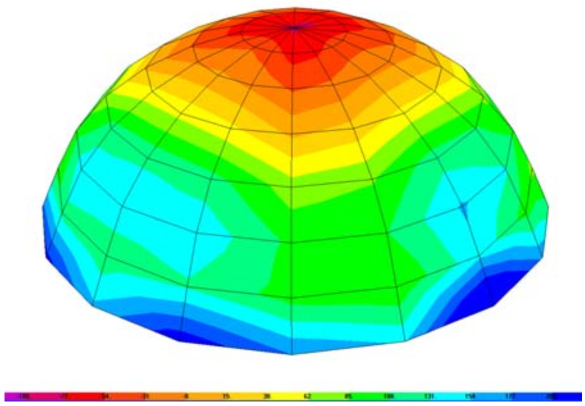


Figure 5.40: Maximum stress contours of the main dome with tensional ring (SW+EQ10) (kN/m^2)

In addition, FRP strips may be used for decreasing the tensile stresses on the domes or tromps. Since there is no damage on the main dome, using FRP strips may be more proper for strengthening.

Existing opening cracks do not threat the structural safety. However, cosmetic repair is necessary for opening cracks. These cracks may be filled.

Damaged iron ties can be replaced or can be fixed in order to increase their load bearing capacities.

6. CONCLUSIONS

Istanbul is one of the oldest cities in the world. It had been capital of various empires. Istanbul has many historic structures from several civilizations, mostly from Ottoman Empire, such as mosques, bridges, palaces, churches, bathhouses, and aqueducts. One of these building is Mehmet Aga Mosque. It belongs to Ottoman time and it was constructed in 1985 by Architect Davud Agha. The building is located in Fatih province. It is still in service. The mosque has experienced several earthquakes including great Istanbul earthquakes of 1766, 1894 and 1999. Two interventions, a repair and a restoration, have been performed so far. Because of that, possible past damages could not be investigated thoroughly. However, there are still cracks, buckling and deterioration damages in several components.

In this study, after giving various general aspect of the investigation, repair and strengthening of the historical buildings, three-dimensional FEM model was prepared for Mehmet Aga Mosque first. Historical structures generally have very complex geometry. It is very difficult to reflect all geometrical shape. Because of that, meaningful several simplifications were made when preparing the analysis model. Global behavior of the structure was obtained using a reasonable number of finite elements.

The masonry shows brittle behavior and its tensile strength is relatively low where the compressive strength of masonry is high. Non-linear behavior appears due to stress concentrations, which causes cracking of masonry when the stresses reach the strength value of masonry. Increasing of the capacity beyond the elastic response appears due to redistribution of stresses beyond the elastic limit of masonry material. However, overall structural behavior can be obtained by elastic analysis.

Body walls of the building consist of brick, stone and mortar. Main dome is made of brick. However, unified material was assumed in analyses for simplification. Material properties were chosen by considering the same era structures and literature survey. Stiffness of foundation was determined by evaluation of soil properties. In addition, sensitivity analyses were performed using various elastic modulus and stiffness of foundation values in order to determine the effect of these values on the analysis results. Stresses are insensitive to the elastic modulus as well as foundation stiffness, which is uniformly distributed along subsoil of structure.

Free vibration analysis was performed in order to determine the natural periods, frequencies and mode shapes. Self-weight and earthquake analyses were carried out. Elastic analyses were used in this study. Stress concentrations were investigated and compared to the existing damages in order to determine the damage reasons. Since the tensile strength of masonry is low, tensile stress concentrations are primarily important. Excessive tensile stresses occur at circumferential dome direction and main support points of upper shell elements. Generally, compressive stress does not threat the building.

Addition of a tensional ring is the one of the main strengthening intervention method for dome type components. It decreases the circumferential tensile stress and displacements. Another intervention method is covering the support region of the dome with FRP strips to provide integrity, to carry the support thrusts and to prevent the moving the supports of the domes apart. This method is very effective in case it is implemented to rotationally symmetric surfaces. It is not convenient for vaults due to their rectangular shapes.

Crack repair, replacing the missing and damaged units with special mortars, adding buttresses for domes, vaults or walls, adding or repair and strengthening of ties, stitching the wide wall cracks can be aligned as intervention techniques for historical masonry structures. However, it should be kept in mind that interventions must be kept at minimum level for historic structures and they should remain the same after intervention due to their highly art and cultural values.

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RESUME

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