COMPARISON OF ELASTIC AND INELASTIC BEHAVIOR OF HISTORIC MASONRY STRUCTURES AT THE LOW LOAD LEVELS

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ABSTRACT

COMPARISON OF ELASTIC AND INELASTIC BEHAVIOR
OF HISTORIC MASONRY STRUCTURES AT THE LOW LOAD LEVELS

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Conventional methods used in the structural analysis are usually insufficient for the analysis of historical structures because of the complex geometry and heterogeneous material properties of the structure. Today's computing facilities and methods make FEM the most suitable analysis method for complex structural geometry and heterogeneous material properties. Even the shrinkage, creep of the material can be considered in the analysis. Because of this reason Finite Element Method (FEM) is used to analyze such structures. FEM converts the structure into finite number of elements with specific degree of freedoms and analyses the structure by using matrix algebra. However, advanced FEM methods considering the inelastic and time dependent behavior of material is a very complex and difficult task and consumes considerable time. Because of this reason, to analyze every historical structure is not feasible by applying advanced inelastic FEM, whereas elastic FEM analysis at low load levels is very helpful in understanding the behavior of the structure.

The analysis of a masonry gate in the historical city, Hasankeyf is the case study of this thesis. Different common software are used in FEM to compare the stresses, deformations, modal shapes etc. of the same structure. Besides the inelastic behavior of the structure is investigated and compared with the elastic behavior of the structure. The study is intended to show that at the low load levels elastic FEM analysis is sufficient to understand the response of the structure and is preferable to the inelastic FEM analysis unless a very complex analysis is required.

Keywords: Masonry Structures, Finite Element Analysis, Linear Analysis, Non-linear analysis

ÖZ

TARİHİ YIĞMA YAPILARIN DÜŞÜK YÜK SEVİYELERİNDE DOĞRUSAL VE DOĞRUSAL OLMAYAN DAVRANIŞLARININ KARŞILAŞTIRILMASI

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Yapısal analizde kullanılan geleneksel yöntemler tarihi yapıların karmaşık geometrik yapısı ve homojen olmayan malzeme özellikleri nedeniyle bu tür yapların analizinde yetersiz kalmaktadır. Günümüz hesaplama olanakları ve yöntemleri karmaşık geometri ve homojen olmayan malzeme özellikleri için sonlu elemanlar yöntemini en uygun analiz yöntemi haline getirmektedir. Malzemenin akması, büzülmesi bile analizde hesaplanabilir. Bu gibi nedenlerle sonlu elemanlar yöntemi tarihi yapıların analizinde kullanılmaktadır. Sonlu elemanlar yöntemi yapıyı sonlu sayıda ve belirli serbestlik derecelerine sahip elemanlara bölerek matrix cebir yöntemleri kullanılarak yapıyı analiz eder. Fakat malzemenin elastik olmayan ve zamana bağlı davranışını hesaba katan ileri derecede sonlu elemanlar yöntemleri çok karmaşık ve çok fazla zaman alan yöntemlerdir. Bu nedenle bütün tarihi yapıların ileri dereceli sonlu elemanlar yöntemleri kullanılarak analiz edilmesi pek makul değildir, fakat düşük yük seviyelerinde

elastik sonlu elemanlar yöntemleri yapının davranışının anlaşılmasına yardım eder.

Hasankeyf'de yer alan yığma bir kapı bu tezin örnek çalışmasıdır. Çalışmada sonlu elemanlar yönteminde kullanılan değişik yazılımlar gerilmeler, şekil değiştirmeler, mod şekil değiştirmeler kullanılarak karşılaştırılmış, ayrıca yapının elastik olmayan analizi yapılmış ve elastik davranışla karşılaştırılmıştır. Çalışma düşük yük seviyelerinde elastik sonlu elemanlar yöntemlerinin yapının davranışını anlamak açısından çok ileri düzeyde bir irdeleme gerekmedikçe ileri derecedeki sonlu elemanlar yöntemlerine tercih edilebileceğini göstermeyi amaçlamaktadır.

Anahtar Kelimeler: Yığma Yapılar, Sonlu Elemanlar Analizi, Elastik Analiz, Elastik Olmayan Analiz

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“Hence to fight and conquer in all your battles is not supreme excellence; supreme excellence consists in breaking the enemy's resistance without fighting”

Sun-Tzu

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INTRODUCTION

1.1. Importance of Historic Monuments and Ethics of Conservation

Historic monuments are the cultural reflections of a society. They create a strong link between the past and the present by presenting the economical, social and technical situation of the ancestors of a society. Historic monuments are also works of art. For these reasons historic monuments are unique and invaluable. The uniqueness of historic monuments makes them worthy to be well preserved and conserved.

A lot of important civilizations have survived in Anatolia. These civilizations are the main resources of the rich historical heritage of Anatolia. Departing from the fact that historic monuments constitute the majority of the living history, it might not be so pretentious to claim that they are the proofs showing the spirit and charm of the heritage. Therefore, conservation and restoration of ancient monuments have a great importance in order not to lose our past, to transfer it to the present, and consequently, to reveal the cultural evolution [1, 2].

Most of the historic monuments are works of art. The signs of true genius can be found in the innovative designs of historic monuments. For this reason, the conservation and restoration of historic monuments are different from the processes applied on ordinary buildings. Since they are unique and priceless, conservation and restoration of historic monuments require a good cooperation of engineering, architecture and the science of history. The history of the structure and its surroundings should be well examined. Besides, a thorough inspection should be carried out on the construction methods, construction materials and the functions of the structure before any restoration process is performed. These all constitute the very important step, which could be named as “To Understand the Building”. The main principle of restoration and conservation of historic monuments is to keep the building as original as possible. Repair is the keyword in restoration of historic structures, which principally means that replacement of deteriorated architectural features should be avoided as much as possible. If replacement is urgency, the materials similar, preferably identical, to the original one should be used. Each property should be recognized as a physical record of its time, place and use; thus, changes that create a false sense of historical development must be avoided. All of these methods should retain and preserve the historical character of the building. It is very important to conserve the original concept in order to enlighten the past correctly and carry it to the future with its original characteristics [3].

1.2. Information on the Methodology of Conservation and Restoration

Restoration and conservation of historic monuments is based on a sequence of activities. The most important of these steps are anamnesis and analysis, diagnosis, therapy, control and prognosis. Anamnesis and analysis cover assessing the safety of the structure by different methods. Diagnosis, which is the first step of any study, is a judgment on the cause and the nature of the factors that have affected the structure. Prognosis is a tool to compare possible restoration alternatives, to define the critical parameters and to obtain an optimal solution for any kind of uncertainties, such as future loading [2,4].

Most of the existing historic monuments are masonry due to the fact that it is one of oldest and the most widely used construction material throughout history.

Another important characteristic of masonry for its widespread utilization is its durability. Masonry historic monuments are constructed of either stone or brick blocks with mortar in between. Masonry structures should not be considered as a continuum. In fact, these kind of structures are an assemblage of stone or brick elements linked by mortar joints. These two units, masonry and mortar, make the behavior of the structure different than those of reinforced concrete or steel structures. The deformation and failure mechanism of the structure is governed by the block nature of the masonry [2,5].

The primary step for the structural conservation and restoration of a historic monument is to understand the behavior of the structure under different load conditions. However, due to complex geometrical and material properties, classical analysis methods are not adequate to have a comprehensive information on the behavior of masonry structures.

1.3. Argument

There are basically two factors which seriously damage or even lead to collapse of historic monuments. These are foundation problems and earthquakes. Foundation problems are not as significant as the problems caused by earthquakes since these kind of structures have attained soil-foundation equilibrium due to their old age [1]. The evaluation of currently constructed buildings for different load effects depend on reliable methods, but the investigation of seismic behavior of old masonry structures lacks scientific background [5]. The evaluation of historic monuments and their real behavior represent a very complex problem because of non-linear characteristics of the construction material, complex geometric form of the structure, lack of structural data and unknown damages, which have already existed, formed within the structure through its lifetime. For these reasons, conventional analysis methods of structural mechanics are not adequate for the analysis of historic structures.

Today's computing facilities and methods make FEM the most suitable analysis method for complex structural geometry and heterogeneous material properties. Even the shrinkage, creep of the material can be considered in the analysis. Because of this reason, Finite Element Method (FEM) is used to analyze such structures. The complete behavior of the structure, from elastic range to the failure, the locations of the damaged parts and the ultimate load capacity of the structure can be observed by inelastic FEM. However, advanced FEM considering the inelastic and time dependent behavior of material is a very complex and difficult task and takes a considerable amount of time. Thus, it might not be practical and even feasible to analyze every historic structure by means of advanced inelastic FEM.

1.4. Objectives

It is a well known fact that masonry is weak under tensional forces. Because of this reason, the lateral load capacity of a masonry structure is very low since lateral loads create tensional forces in the structural members.

Although inelastic analysis gives complete information about the behavior of the structure, the application of this kind of an analysis is quite difficult and time consuming. Thus, if it can be shown that the elastic behavior of a masonry structure at the ultimate lateral load is not quite different than the inelastic behavior, this might help engineers that are involved in the structural restoration works, in the sense that they can decrease the analysis time, meaning money and time economy, and they can have a general information about the behavior and the susceptible parts of the structure easily. After this step, it might only be suitable to investigate the susceptible parts by nonlinear methods.

In the light of the information given in the previous paragraph, the aim of the study is to compare elastic and inelastic behavior of historic masonry structures at low load levels.

1.5. Procedure

The study is conducted in three phases. In the first phase, a literature survey was carried out on the material and strength properties of masonry materials. Besides, analysis methods are investigated to decide on the best analysis method for the study.

In the second phase a case study structure is selected. The case study structure was chosen as Hasankeyf Gate, which is a masonry gate located in the historic city of Hasankeyf. The reason for choosing such a gate is the proportion among the dimensions of the structure. Detailed information about the gate is given in the 4th Chapter. Finite element models of the gate are created by using different software for linear and non-linear analysis. The model is verified by comparing the modal deformed shapes. Afterwards, a dynamic analysis is performed to verify the rigidity of the structure.

In the third phase, linear and nonlinear analysis are performed. The ultimate lateral load capacity of the structure is determined by non-linear analysis. The same load is applied to the structure for the linear analysis. Results of these analyses are compared in terms of deformations at ultimate loads and the locations of the cracks. The discussion part gives information about the relevant cases for application of both of the analysis methods.

1.6. Disposition

The study consists of five chapters. The introduction part gives general information about conservation and preservation of historical buildings. Continuing with general features about elastic and inelastic analyses and their application points, this section focuses on the procedure of the study. Concise information about the case study is given in here and afterwards, the sequence of the analyses is explained briefly. In the second chapter, a literature survey on the strength and material properties of masonry is presented. The types of masonry materials such as clay bricks, stone and mortar are summarized. The strength characteristics of these masonry materials, such as the compressive strength, tensile strength and the shear strength, are presented, with information on elasticity modulus of masonry materials.

The third chapter gives information about the loads acting on masonry structures. This section also explains types and modes of failure of masonry elements and masonry structures. After giving information about the loads acting on masonry structures, the emphasis is given on the analysis methods of masonry structures.

The verification of the finite element model and the analysis results for linear and non-linear analysis of the case study are presented in the 4th Chapter. In the first parts, information on the historic city of Hasankeyf and the case study gate is given. The verification of the finite element model follows this part. Lastly in this chapter, linear analysis and non-linear analysis are performed and the results are compared.

The last chapter consists of summary, general conclusions and recommendations for future studies.

CHAPTER-2

STRENGTH AND MATERIAL PROPERTIES OF MASONRY MATERIALS

2.1. Masonry as a Construction Technique through History

Masonry is the construction technique that the blocks are compiled one on each other piece by piece by either with or without mortar. The blocks are stone or brick, stone usually being natural form or artificially shaped forms are mostly bricks which are adobe or fired clay brick. Mortar is usually made from clay, chalk, bitumen or cement. The use of blocks and the mortar in a combining way as a construction technique is called masonry construction technique. Different combination of units in the masonry construction can be used. Figure 2.1 shows different stone masonry arrangements. The use of undressed rough stone without a pattern is called rubble masonry. Rubble masonry construction technique is generally used for the construction of walls. Smooth square or rectangular stones are used in the ashlar masonry construction technique. Coursed ashlar masonry technique requires stones of the same height within each course, but each course can vary in height.

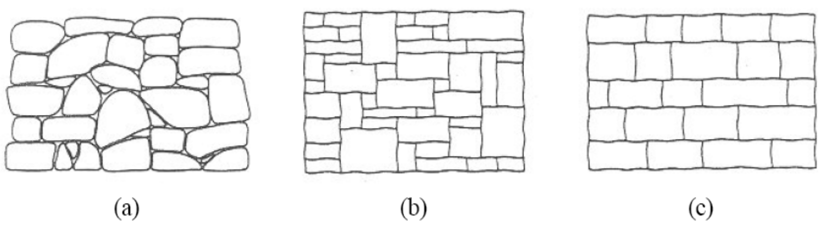


Figure 2. 1 Different kinds of stone masonry; a) rubble masonry b) ashlar masonry c) coursed ashlar masonry [6]

Figure 2.2 shows different arrangements for brick masonry. In common bond technique, a header is a brick laid such that the small end only appears on the face of the wall. A brick is laid such that the long, narrow side only appears on the face of the wall in the stretcher bond technique. Cross bond is made from alternating courses of headers with courses of stretchers. As the name implies, stack bond has each course, made of stretchers, stacked right on top of the below it.

Timber and masonry are the oldest and the most widely used construction materials through history. Owing to its durability, the majority of the historical buildings that still stand today are masonry. Locating the units one on top of each other, by either using mortar or not, brings masonry about a very simple and a very durable construction method [6,7].

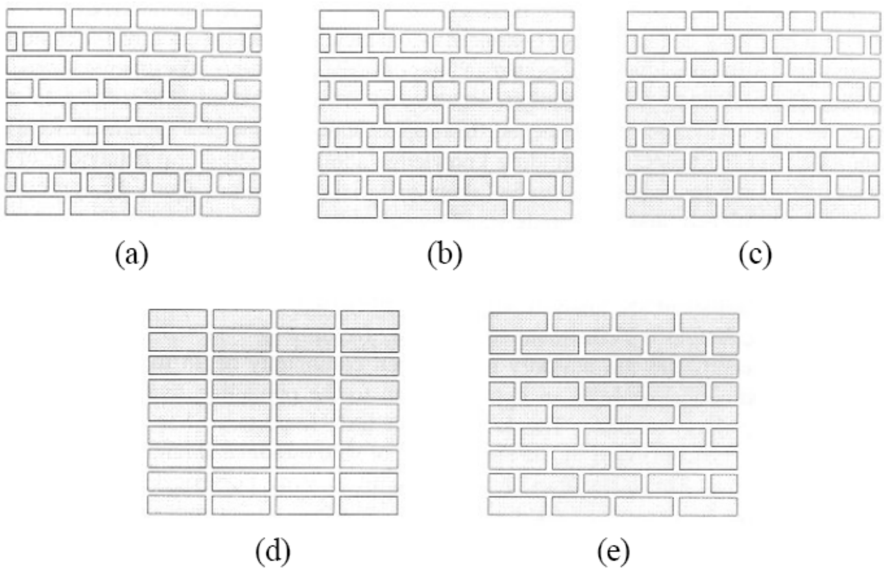
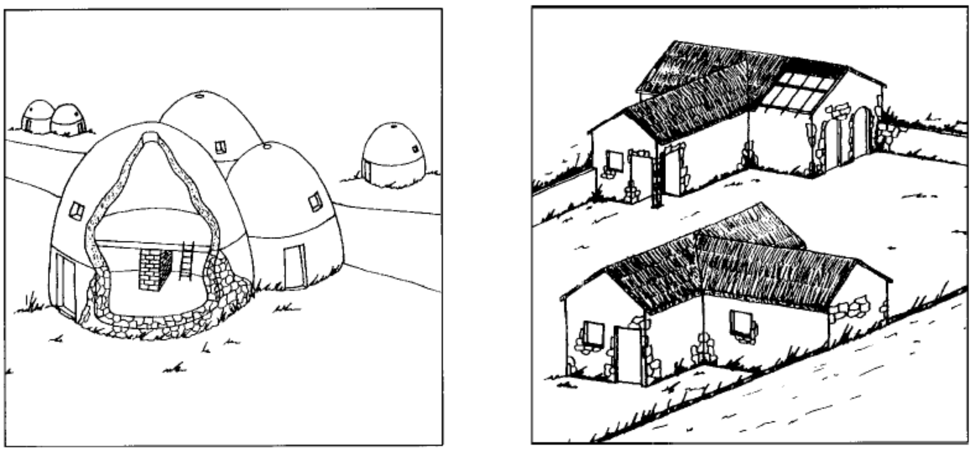


Figure 2. 2 Different arrangements for brick masonry; a) American (common) bond b) English (cross) bond c) Flemish bond d) Stack bond e) stretcher bond [6]

Most probably, the first masonry material was natural stone. Regular shaped stone substituted natural stone as tools and construction techniques developed. The first bricks were made from mud or clay dried by the sun. These bricks were then laid with mud or mortar into walls. In the valleys of the Nile and the Mesopotamia, this process has been used to construct dwellings [4]. Figure 2.3 shows the evolution of housing from huts, to apsidal houses and finally to rectangular houses.

Masonry houses near Lake Hullen, Israel (9000-8000 BC) are among the oldest examples of masonry building construction [6,8]. Another earliest example of masonry construction is the Walls of Jericha (8000 BC). These structures were made from limestone and the joints were filled with earth [3]. Figure 2.4 and Figure 2.5 shows Walls of Jericho.



(a) (b)

Figure 2. 3 Examples of prehistoric architecture of masonry in the Ancient Near East; a) beehive houses from a village in Cyprus (c.5650 BC) b) rectangular dwellings from a village in Iraq [6].

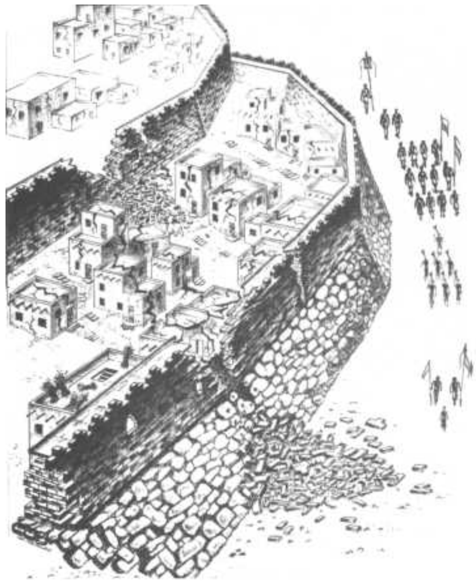


Figure 2. 4 A representation of the north side of Ancient Jericho [36].

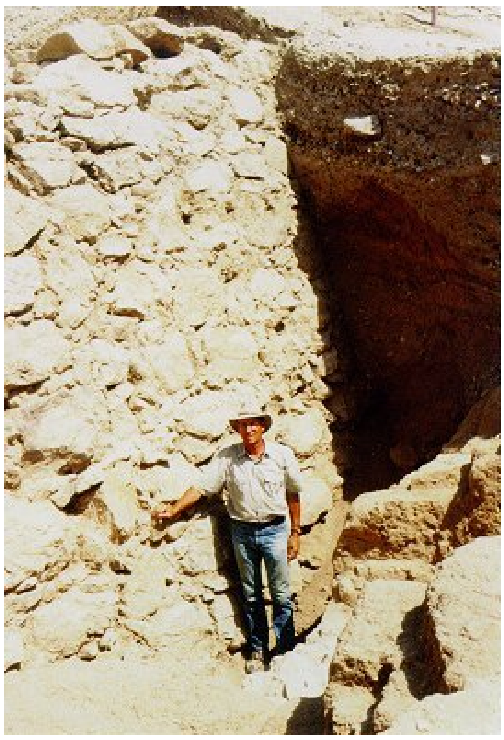


Figure 2. 5 A section of the collapsed wall of Jericho [36].

Egyptian pyramids present the most stable structural shape with their huge dimensions. As the construction of masonry evolved, the behavior of the structure started to be more important. Lintels started to be used as the supports for the openings in the wall.

As for the evolution of the form, Greek Architecture was based on rules of proportion and symmetry. Limestone was used as the construction material. The Parthenon (5^{th} Century BC) is one of the most famous examples of this era (Figure 2.6)

The Romans constructed not only temples but also roads, bridges, aqueducts and introduced many innovations related to materials and structural concepts. The quality of the bricks was improved and the size of the bricks became more standardized. Figure 2.7 shows some general types of Roman masonry walls.

Figure 2. 6 A view of Parthenon [37].

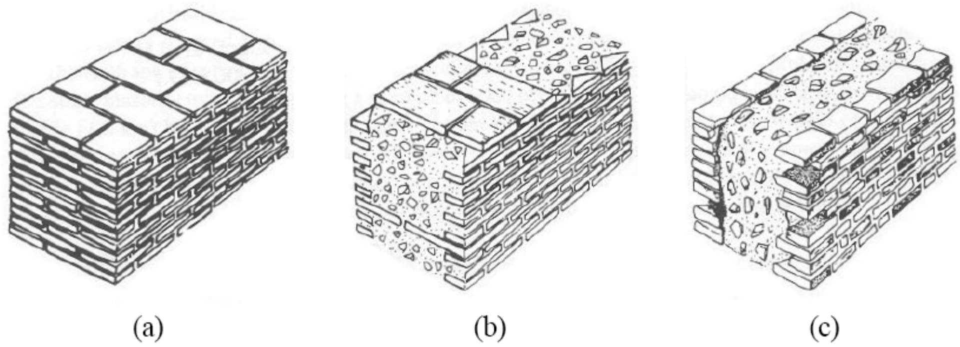


Figure 2. 7 Roman masonry walls; a) bonded brick wall b) brick faced wall with header courses c) brick faced wall [7]

In the course of time, the structural shape evolved from linear to curved or arched forms that enabled to span larger distances. Figure 2.8, Figure 2.9 and Figure 2.10 depict important examples of curvilinear structural forms.

Figure 2. 8 The Pont du Gard aqueduct, Nimes, France[1] [38].

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[1] The Pont de Gard was constructed before the Christian era on the Gard river by the Roman architects and engineers.

Figure 2. 9 The Church of Hagia Sophia, İstanbul[2] [39].

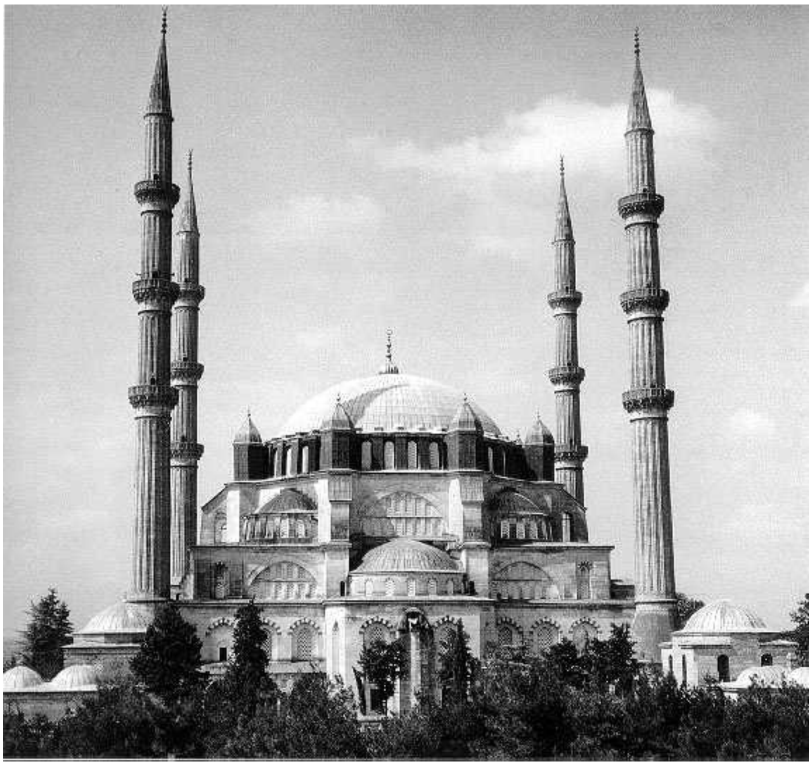

Figure 2. 10 Selimiye Mosque, Edirne[3] [40].

[2] The first church on the site was built by the Eastern Roman Emperor Constantius. Constantius church was consecrated in 360 AD. Since it was the greatest of its time it was first known as the Great Church. Later it became known as Holy Wisdom, a name attributed to Christ by theologians of 4th century.

[3] The mosque was commissioned by Sultan Selim II and was built by architect Sinan between the dates 1568-1574.

Masonry also played an important role in the Gothic Architecture. Following the Gothic Architecture, Renaissance Architecture, brought new concepts to masonry construction. The buildings were characterized by regular forms and geometrical symmetry [7]. Figure 2.11 shows a view of the Amiens Cathedral and Figure 2.12 shows St. Maria del Fiore church at Florence. With the invention of cast-iron and the reinforced concrete, masonry has lost its importance in the construction world.

Figure 2. 11 Amiens cathedral[4] [41].

[4] Amiens cathedral is the tallest of the large 'classic' Gothic churches of the 13th century. After a fire destroyed the former cathedral, the new nave was built in 1220 and finished in 1247.

Figure 2. 12 St. Maria del Fiore churh[5].

2.2. Components of Masonry

The components of masonry are the mortar and the block units .The mechanical properties and arrangement of the mortar and the units usually determine the global mechanical properties of masonry structures. The following sections give general information about mortar, stone and brick, respectively.

2.2.1 Mortar

Mortar is a mixture of cementitious material with constituents and water. When used as a structural material in masonry structures, it creates a bind between masonry units by becoming an artificial sandstone with the setting up of the cementing material [1].

Traditional mortar was made from lime, putty or slaked lime combined with local sand. Other ingredients such as brick dust, natural cements and pigments were

[5] The church was started to be built in 1296. The bell tower was built between 1334 and 1387 by Giotto. The octagonal dome was designed by Brunelleschi in 1418 and was built between 1420 and 1436.

also added to mortar for different purposes. Until the advent of Portland cement, the basic combination of mortar remained practically unchanged [9].

The performance of a masonry building is highly dependent on the strength and binding characteristics of mortar, although the amount of mortar is very little when compared to masonry units. The compressive, tensile and shear strength as well as the bind strength between the masonry units, are indications of the strength characteristics of the masonry constructions.

The compressive strength of mortar mainly depends on the proportion of the mixture materials. However, it is very difficult to represent the strength of historic mortar because of the difficulty of making strength tests on historic mortar. By using similar mixture ratios, it is possible to obtain artificial historic mortar. For example, compression tests performed by Oliveira [11] on mortar samples gave results of approximately 5 MPa compression strength. However, the overall strength of a masonry construction is mostly affected by the bind strength between the mortar and the masonry block. Forces perpendicular to the mortar-block joint is resisted by tensile bind strength, whereas forces parallel to the mortar-block joint is resisted by the shear bind strength of the masonry [1,2]. Different test set-ups can be used for the determination of the tensile bind strength of masonry. Figure 2.13 shows possible test set-ups for determination of tensile bind strength of masonry.

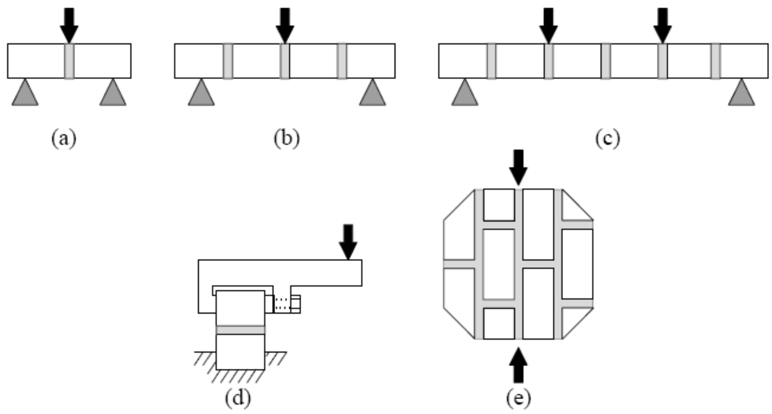

Figure 2. 13 Possible test set-ups for determination of bind strength; a) three point bending with a single joint b) three point bending with a longer specimen c) four-point bending d) bind-wrench test e) splitting test [6]

The shear bind strength of masonry can be determined by using couple or triplet tests. Figure 2.14 shows the test set-ups for couple and triplet tests. Not only the properties of the mortar and the masonry unit, but also the pressure on the unit mortar interface affects the shear bind strength of masonry. As the pressure increases, the shear bind strength also increases.

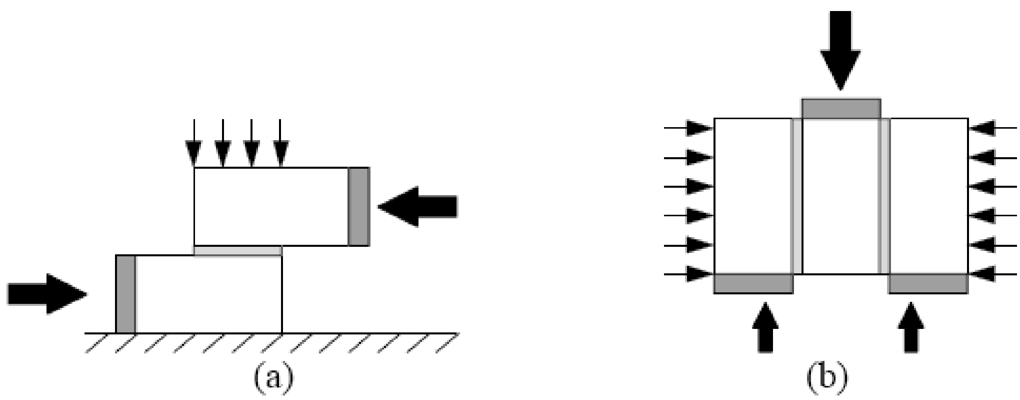

Figure 2. 14 Possible test set-ups for determination of shear bind strength; a) couple test b) triplet test [6].

It is usually thought that strength of mortar itself is an indication of a strong masonry system. In fact, this is not the case. A good quality mortar should be softer than masonry units to account for the settlements, lateral deformations etc. Also the vapor permeability of mortar should be more than the vapor permeability of the masonry units. It is preferred deteriorations to occur on the mortar, since the replacement of mortar is easier when compared to replacement of masonry blocks [9,10,12].

2.2.2. Stone Masonry Units

Being very durable and easily available, stone is one of the oldest construction materials. If the structure is constructed with good design and good materials, it can stand for centuries [1,2,13].

Structural stones can be classified considering mineralogical constituents and shape of the stone units. According to mineralogical constituents, structural stones can be classified in three main groups. Argillaceous stones contain clay, shale and slate. Lime of carbonate of calcium are the main constituents of calcerous stones, whereas siliceous stones are mainly composed of silica or quartz [1].

The structural stone can be used as a structural material either in its original shape or as reshaped. Natural or minor shaped stones are called rubble, whereas accurate rectangular forms are called ashlars. By using perfectly shaped stones, a good resistance masonry structure can be constructed without mortar, which is called dry masonry.

The physical properties of stone is highly variable depending on the mineral composition. Stone is very strong in compression but very weak in tension. Generally, the tensile strength of stone is approximately 10% of its compressive strength and the shear strength of stone is 25% of its compressive strength. Table 2.1 shows the physical properties of masonry. The compressive strength of masonry is such sensitive to mineral composition that, Corrodi [14] showed that the compressive strength of pink colored calcareous stone is 90 MPa, whereas the compressive strength of white colored calcareous stone is almost half of the pink colored one. Also test results obtained by Oliveira [11] showed that the compressive strength of stone prisms has a very wide scatter (from 42 to 75 MPa).

Table 2. 1 Physical Properties of Stones [1]

Stone	Compressive Strength	Modulus of Rapture	Shear Strength	Tensile Strength	Modulus of Elasticity
Granite	30~70 MPa	10~35 MPa	14~33 MPa	4~7 MPa	30~55 GPa
Marble	25~65 MPa	8~25 MPa	9~45 MPa	1~15 Mpa	25~70 GPa
Limestone	18~35 MPa	3.5~14 MPa	6~20 MPa	2~6 MPa	10~55 GPa
Sandstone	5~30 MPa	2.5~16 MPa	2~10 MPa	2~4 MPa	13~50 GPa
Quartzite	10~30 MPa	5~15 MPa	3~10 MPa	3~4 MPa	15~55 GPa
Serpentine	7~30 MPa	9~15 MPa	2~10 MPa	6~11 MPa	23~45 GPa

2.2.3. Brick Masonry Units

Archeological evidence has shown that brick is the oldest processed construction material. Bricks can be manufactured either by drying the material in the sun (adobe) or by burning the material in the kiln [1,2,13].

The word "adobe" comes from the Arabic "at-tub" and from the Coptic "tObe". One of the oldest reference for the use of adobe is from the City of Çatalhöyük 7500 BC. Adobe is made from soil mixed with water and organic materials such as straw and animal dung. Material properties of the soil, drying process, effect of additives and construction process affects the strength of adobe. Addition of sand and organic materials such as straw increases the strength of adobe. Adobe walls can be decayed by the rain effect. Because of this reason, layers of mud are applied on the sides of the adobe walls. Figure 2.15 shows the production of adobe bricks by traditional methods.

Figure 2. 15 Production of the adobe bricks by traditional methods [16].

The basic ingredient of brick is clay, which is composed of mainly silica and alumina. The type of the clay may be soft and plastic surface deposits or hard sandstone, shale and slates [1,2,13].

The strength of clay bricks depend on several factors. The properties of the ingredients of the brick, drying process, baking temperature and pore structure are the most pronounceable factors that affect the strength of clay bricks. Bricks can have different material properties for the horizontal and the vertical directions depending on the production process. Tests performed by Oliveira [7,11] on 4 different brick samples made from the same material and by the same method have revealed 56.8 MPa compression strength on vertical direction and 51 MPa on horizontal direction. The modulus of elasticity was determined as 12.75 GPa on vertical direction and 10. 45 Gpa in horizontal direction. Figure 2.16 shows the strain-stress graphics for four prismatic brick specimens. Note that the different behavior in vertical and horizontal directions.

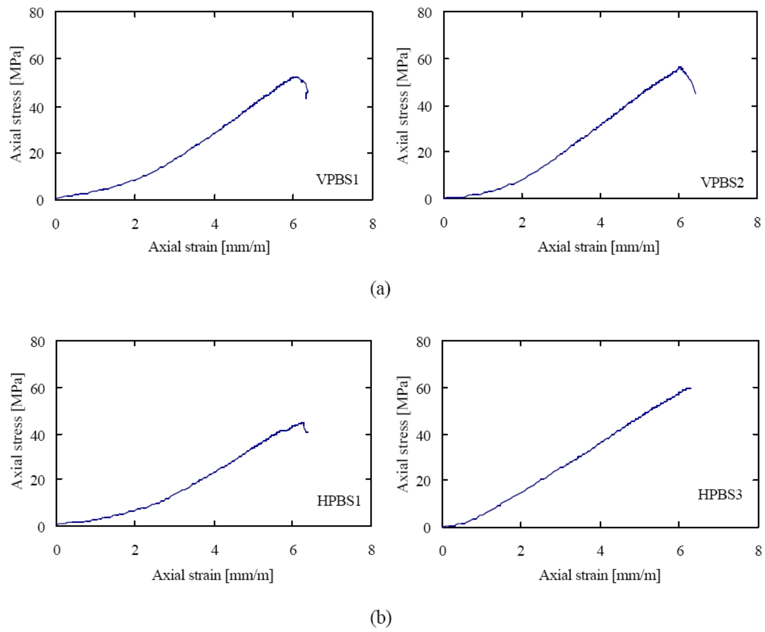

Figure 2. 16 Stress-strain graphics for four prismatic brick specimens tested under axial displacement control; a) vertical direction b) horizontal direction [7].

Bricks are strong in compression but weak in tension. Usually the tensile strength of brick is 8% and the shear strength is 30% of the compressive strength of the brick. Clay bricks can have compressive strengths up to 100 MPa, however 10 to 40 MPa compressive strength is adequate for the construction of domestic buildings. The modulus of elasticity varies between 5000-10000 Mpa. Tensile strength of bricks is usually 3 to 5 Mpa. The bricks are stronger than the stones because of their ductility. Ductility causes the bricks to absorb stress concentrations much more easier than stones [2,15].

2.3. Mechanical Properties of Masonry Materials

Masonry is a composite material being composed of mortar and stone or brick units. However, the behavior of these composing materials is different from each other. Mortar is much softer than the stone or brick units. This composite nature and complex geometry of masonry leads to a very complex structural behavior.

Unfortunately, the strength and stiffness properties of the constituent materials do not reflect the structural and stiffness properties of the masonry structure itself. This is a result of large variability of mechanical properties of the materials and the result of size effect. This means two different masonry structures constructed of the same material but with different sizes and material orientations do not behave the same.

2.3.1. Compressive Strength of Masonry Materials

The compressive strength of masonry is much higher when compared to the tensile strength of masonry. This property is so important that the structural form of masonry constructions is based on compression forces [6,18].

The investigation of stress-strain curves of masonry can be helpful in understanding the behavior of masonry under compressive forces. Figure 2.17 shows the typical behavior of masonry under uniaxial compression. As it can be seen from the stress-strain curve, the deformation does not change linearly. The area under the stress-strain curve is called as the fracture energy, the energy absorbed until the time of failure.

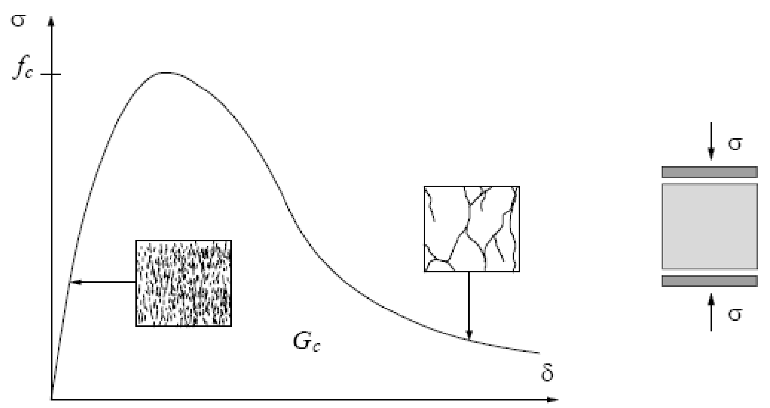

Figure 2. 17 Typical behavior of quasi-brittle materials under uniaxial compression [6].

As stated before, the mortar usually shows a softer behavior when compared to brick or stone units. This different strain characteristic of composing materials of masonry causes a different character in terms of stresses. Under uniaxial compressive loading, the mortar tries to expand laterally more than the stone or brick units. However, because of the continuity between the units and the mortar, combined by cohesion and friction, the mortar is confined laterally by the units. Because of this reason, shear stresses develop at the mortar-brick interface that causes triaxial compression in the mortar and bilateral tension couple with uniaxial compression in the unit [11]. Figure 2.18 shows the prism under compressive loading normal to bed joints and stress states for brick and mortar elements.

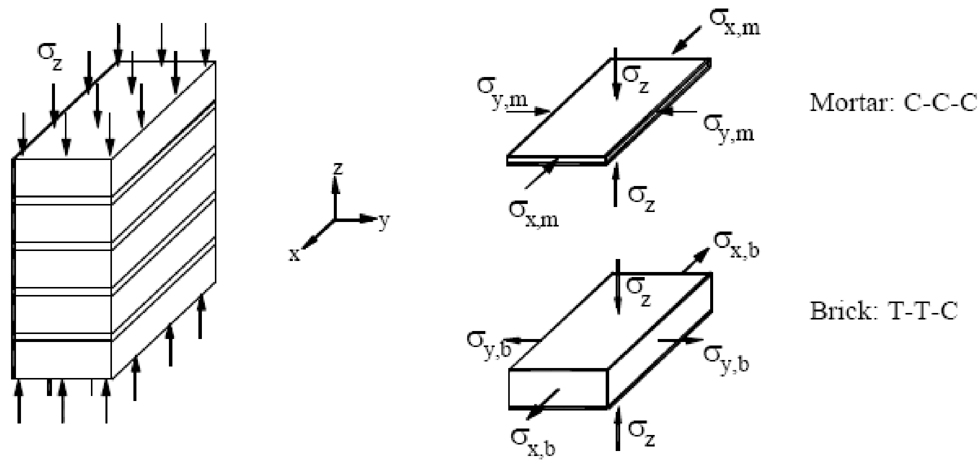

Figure 2. 18 Masonry prism under compressive loading normal to bed joints and stress states for brick and mortar elements (C: compression, T: tension) [11].

The compression strength of masonry depends on the properties of the joints, strength characteristics of the constituent materials, relative thickness of the mortar and the geometry of the structure. Some codes such as Eurocode EC6, FEMA356, ACI530, ASCE6 and RILEM offer some empirical equations for the determination of compressive strength of masonry. These equations generally specify compression strength of masonry by the relative dimensions and the strength characteristics of the mortar and the units. For example in EC6, the compression strength of masonry is defined as follows;

$$f_k = K(f_b^{'})^{\alpha}(f_m)^{\beta} \tag{2.1}$$

where;

f_k is the characteristic masonry compressive strength (MPa)

f_m is the average mortar compressive strength (MPa)

$f_b^{'}$ is the normalized compressive strength

K, α, β are some constants

In this equation normalized compressive strength is defined as;

$$f_b^{'} = f_b \delta_m \delta_s \tag{2.2}$$

where;

f_b is the compression strength of brick masonry unit (MPa)

δ_m is the moisture factor

δ_s is the shape factor

Another important factor affecting the compressive strength of masonry is the eccentricity of the loading. In an experimental and analytical study, Brencich [19] showed that as the eccentricity increases, the compressive strength of masonry structural elements decreases.

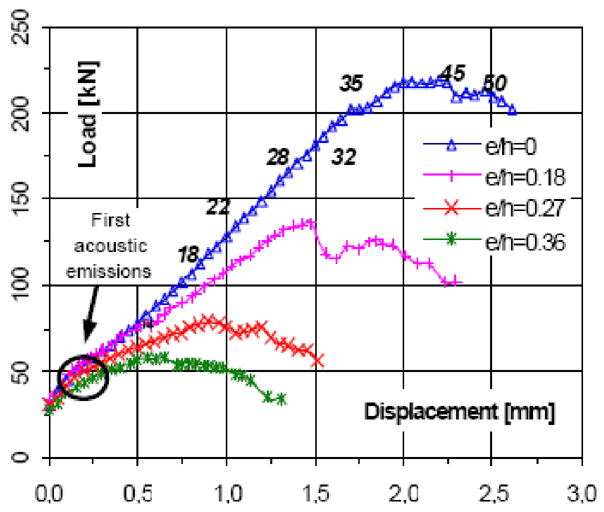

Figure 2. 19 Load-displacement diagrams for different eccentricity rates [19].

2.3.2. Tensile Strength of Masonry Materials

Flexural and diagonal tension members in piers and walls, which carry shear forces, generally face with tensile stresses. In addition, changes in temperature and moisture may cause tensile stresses [1].

The tensile strength of masonry is such low that, some analytical models for the analysis of masonry assumes that the material model has no tensile strength. The tensile strength of masonry can be determined by direct tensile bind strength or flexural bind strength test. Although the direct tensile bind strength test is difficult to perform, it gives the complete tensile stress-displacement diagram as well as the correct tensile strength [7]. Figure 2.20 shows the possible tests for the determination of tensile strength.

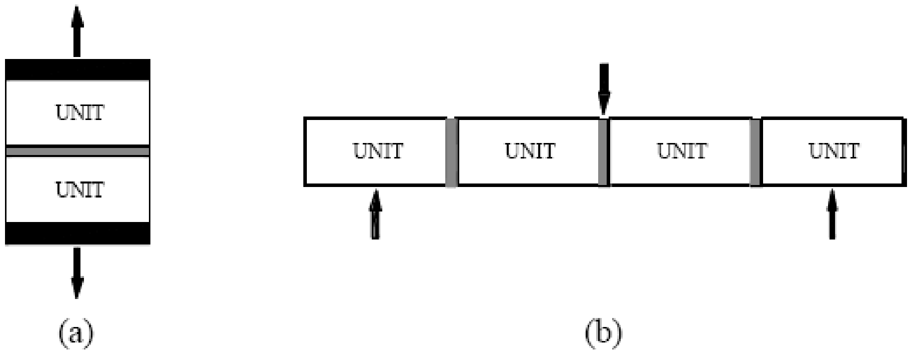

Figure 2. 20 Direct tensile bind strength and flexural bind strength tests [7].

Typical stress-strain diagram for masonry under uniaxial tension is shown in Figure 2.21. When the stress-strain diagram is investigated, it can be observed that the behavior is very brittle ending with a sudden failure.

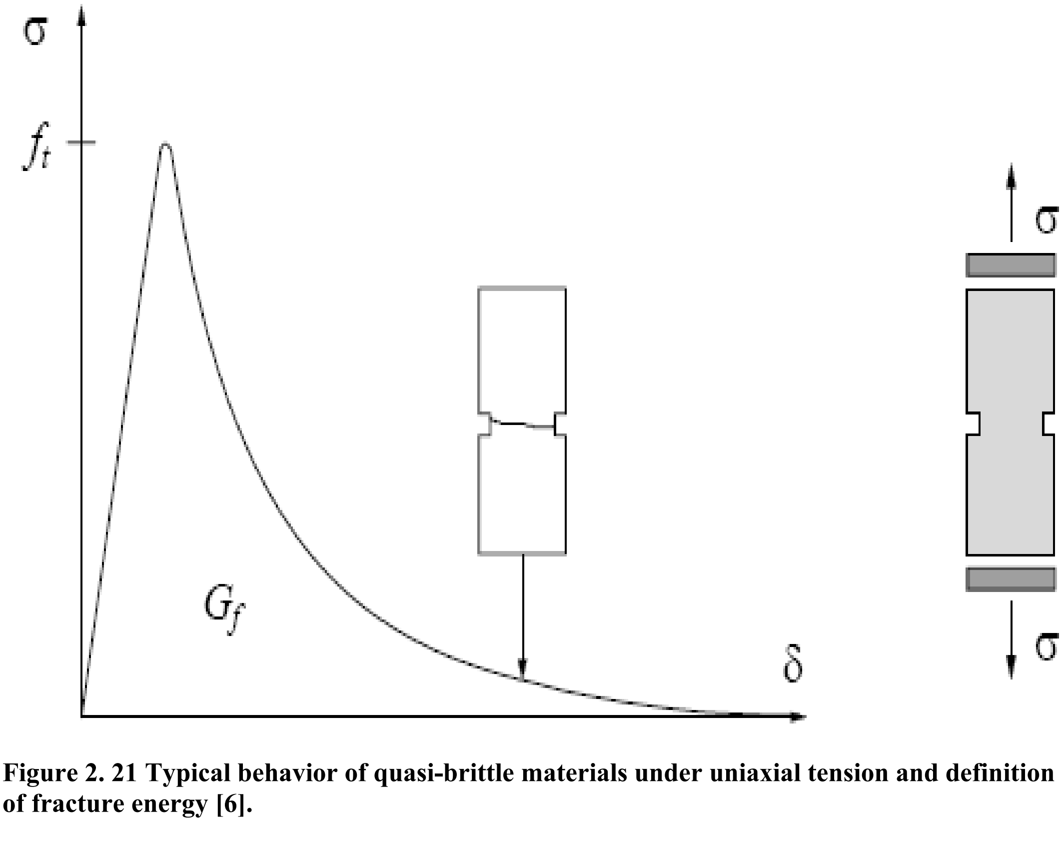

Figure 2. 21 Typical behavior of quasi-brittle materials under uniaxial tension and definition of fracture energy [6].

The tensile strength of masonry is usually equal to the tensile bind strength between the joint and the unit. In case the tensile bind strength is higher than that of the unit, the tensile strength of masonry is equal to the tensile strength of units.

2.3.3. Shear Strength of Masonry Materials

The shear strength of masonry is dependent on the shear bind between the unit and the mortar, as stated earlier in this chapter. Figure 2.22 shows the stress-strain diagram of masonry under shear loading. As it can be seen from the figure, the shear strength increases as the pressure increases.

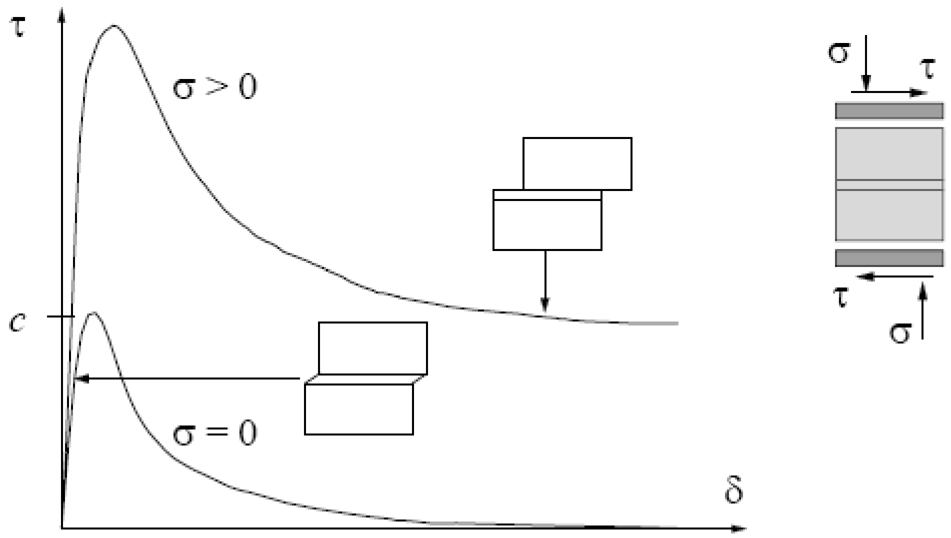

Figure 2. 22 Behavior of masonry under shear direct shear [6].

Various standards offer different methods for the determination of shear strength of masonry. For example Italian Standard D.M. 20.11.1987 is based on the determination of shear strength of masonry by using 6 test results. The characteristic shear strength of masonry is given as;

$$\tau_k = f_{vk0} = k \times f_{vm} \tag{2.3}$$

where;

$$f_{vm} = \frac{1}{6}\sum_{i=1}^{6} f_{vk0,nom}^{i} \tag{2.4}$$

k is a constant (0.7)

FEMA 356 uses the below equation for the expected shear strength of masonry. The equation is given as;

$$v_{me} = \frac{0.75 \times (0.75 v_{te} + \frac{P_{CE}}{A_n})}{1.5} \tag{2.5}$$

where;

P_{CE} is the expected gravity compressive load

v_{te} is the average bed-joint shear strength, which is given by the equation;

$$v_{te} = \frac{v_{test}}{A_b} - P_{D+L} \tag{2.6}$$

where;

v_{test} is the test load at first movement of masonry

A_b is the sum of the mortared area of bed joints above and below the test unit

P_{D+L} is the stress due to gravity loads at test location

2.3.4. Modulus of Elasticity and Poisson's Ratio of Masonry Materials

The modulus of elasticity of masonry depends on the modulus of elasticity of the mortar and the unit as well as the volumetric ratios of the constituent materials. In reality, the modulus of elasticity of masonry varies for different directions and loading conditions. However, for simplicity, this chapter covers modulus of elasticity only under compression.

The modulus of elasticity can be determined by investigation of the stress-strain curve, which is obtained by experiments performed on masonry samples. The fundamental shape of the stress-strain curve for masonry is shown in Figure 2.23. Generally, the linear portion of the stress-strain diagram is used for analysis purposes (E_0). The E_0 value can be determined by means of several methods. One of these methods offers to determine E_0 by using the following formula;

$$E_0 = 1000 \times f_k \tag{2.7}$$

where;

f_c is the characteristic masonry strength.

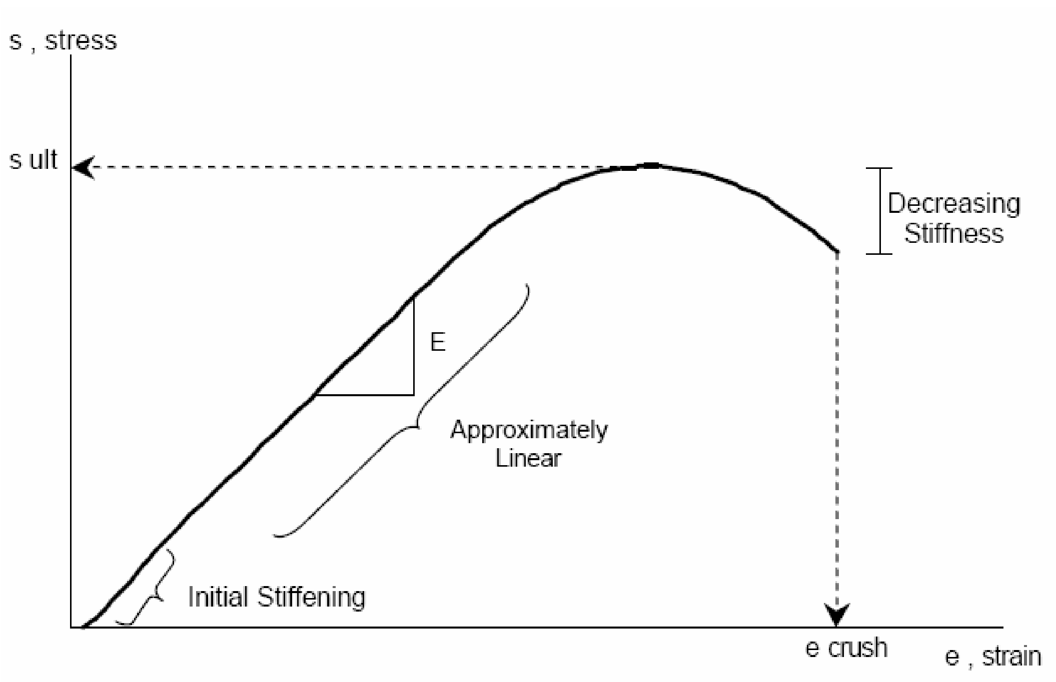

Figure 2. 23 Fundamental stress-strain diagram for masonry under compression

The stress-strain relationship can be modeled by a parabolic function. One example is given in Figure 2.24. In that example the relation between stress and strain is given by the equation;

$$\sigma = f_{cm} \frac{\varepsilon}{\varepsilon_{cmy}} \left(2 - \frac{\varepsilon}{\varepsilon_{cmy}} \right) \tag{2.8}$$

The initial stiffness obtained from equation 2.8 is;

$$E_0 = 2 \frac{f_{cm}}{\varepsilon_{cmy}} \tag{2.9}$$

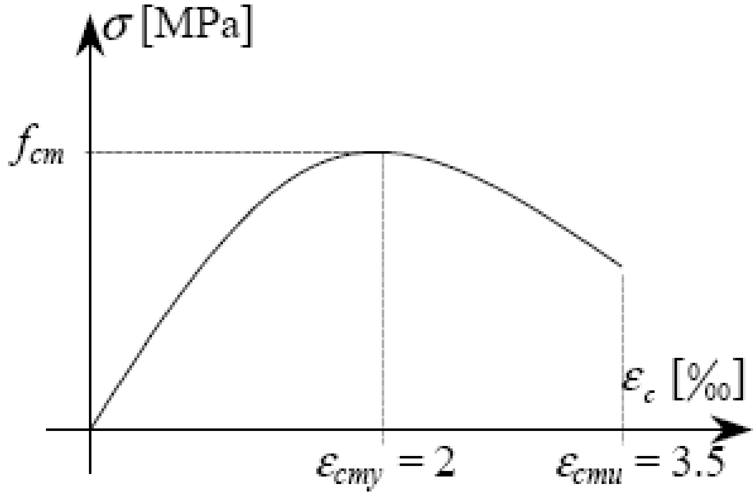

Figure 2. 24 Parabolic stress-strain relationship for masonry [20].

Some methods relate the modulus of elasticity of masonry to the masonry geometry and the material properties. One of these methods is given in Reference-19 as follows;

$$\frac{1}{E_M} = \frac{\eta_b}{E_b} + \frac{\eta_m}{E_m} + 2\eta_m \eta_b \frac{\nu_b E_m - \nu_m E_b}{\eta_m (1-\nu_b) E_m + \eta_b (1-\nu_m) E_b} \left(\frac{\nu_m}{E_m^2} - \frac{\nu_b}{E_b^2} \right) \tag{2.10}$$

where;

η_m, η_b are the volume fractions of mortar and brick

E_M is the modulus of elasticity of the masonry

E_b, E_m are the modulus of elasticity of the brick and the mortar

v_b, v_m are the Poisson's Ratio of the brick and the mortar

The volume fractions of the brick and the mortar can be calculated by using the following equations;

$$\eta_m = \frac{t_m}{t_b + t_m} \tag{2.11}$$

$$\eta_b = \frac{t_b}{t_m + t_b} \tag{2.12}$$

where;

t_m is the thickness of the mortar

t_b is the thickness of the brick

The Poisson's Ratio of masonry is generally between 0.2-0.25. However, as a result of discontinuous material behavior, a masonry structure can have a 'quasi'-Poisson Ratio larger than 0.5 because of cracking or sliding in the joints [21].

CHAPTER-3

STRUCTURAL ANALYSIS OF MASONRY STRUCTURES

3.1.Importance of Structural Analysis of Historic Masonry

Understanding structural behavior of masonry structures is the initial and the most important step for structural conservation of historical built heritage. Safety assessment and determination of the most effective structural restoration method are highly dependent on understanding structural behavior. However, masonry possesses a composite behavior with distinct directional properties, which are given in Chapter-2. Besides this complex structural and material behavior, the difficulties for the analysis of historic structures are listed below [18, 24];

- The geometric data is usually missing
- The information about the inner core of the structural elements is not adequate
- The determination of the mechanical properties of materials of the structure requires very advanced and expensive testing methods
- The mechanical properties of the materials are variable due to the nature of the materials and workmanship
- The construction sequence is usually unknown and very long
- The existing damage is not exactly known
- There were no regulations or codes at the time of construction.

As can be inferred from above discussion, structural conservation and restoration of a historic monument is a very complex task and requires specific training. Understanding the load effects, failure modes, creating structural models and choosing an analysis method are the key issues for the assessment of a historic structure.

3.2. Load Effects on Masonry Structures

A good understanding of a structure begins with estimating the loads acting on the structure and the effects of these loads on the structure. This section of the chapter defines the basic loads on masonry structures. Afterwards, the effects of these loads on masonry structures are explained briefly as failure modes and types of failure in masonry structures.

3.2.1. Loads Acting on Masonry Structures

In the design and analysis of modern structures, load intensities are obtained by structural design codes. However, when the concern is historic structures, this method is not applicable since there were no design guidelines at the time of the construction. Hence, the load estimations and their combination is based on historical documents, observations, past experiences and engineering judgment [1]. The most common expected loads on a historic structure are:

- Self weight of the structure
- Earthquake loads
- Differential settlements of supports
- Soil pressure and ground movement
- Creep

- Thermal loads
- Snow loads and ice pressure
- Impact loads
- Surcharge on walls

Self-weight of the structure includes the weight of the structural elements, weight of the architectural elements etc. Since masonry is very strong under compression forces, the masonry structures are usually very resistant to gravity loads.

Earthquake loads and differential settlements of supports are usually the main reasons for the damage or collapse of masonry structures. Damage due to support settlement is less common with respect to earthquake damage, since the soil and structure has reached equilibrium with time. However, construction in urban areas can cause the ground profile to change and this may cause support settlement problems.

Earthquakes are always the number one enemy of masonry structures erected in highly active seismic zones. In other words, masonry structures are highly vulnerable to earthquakes [25]. The high seismic vulnerability of masonry structures is due to [26];

- Highly nonlinear behavior
- Very small tensile strength
- Uncertain arrangement of blocks and mortar joints
- Significant scatter of mechanical properties throughout the building
- Composite geometry and morphology
- Extreme mass

Determination and application of earthquake load to the analysis model is also a difficult process. Since the lifetime of a historical structure is much longer than that of a contemporary one, the usage of design spectrums derived by using recently registered earthquake data may not be valid for historical structures. Application of earthquake load by the lumped mass assumption is not valid for masonry structures either, since the mass of such kind of buildings is uniformly distributed through the structure.

There are various methods for the determination of earthquake load and its application to the analytical model. Among the various procedures, modal response analysis and non-linear time history analysis are the most popular ones. However, a non-linear modal response analysis is more favorable since non-linear time history analysis requires considerable time and effort. Lourenço (2001) offers using non-linear static analysis in which the seismic load is determined by using a shear coefficient (the percentage of the total weight of the structure that is accounted for the lateral earthquake load). Figure 3.1 shows elastic and inelastic design spectrum that is offered by various codes. However, as stated in the previous paragraphs, these spectrums are derived for recent buildings and the application of these spectrums requires more attention since the life-time of historic masonry structures are much more longer than those of the existing ones.

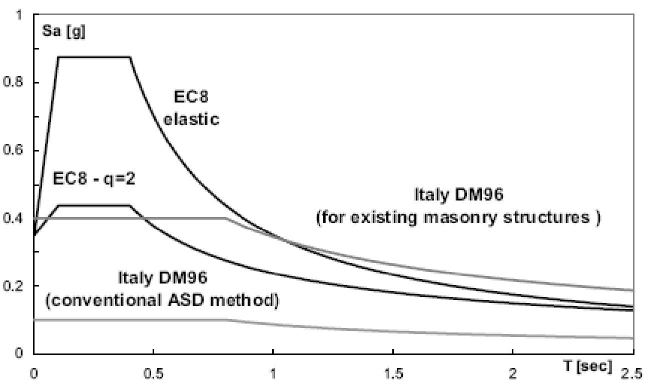

Figure 3. 1 EC8 elastic and reduced spectra and Italian Seismic Code spectra [26].

Another important enemy of historic structures is the differential settlements of supports. However, this effect is observed rarely when compared to earthquake-induced damages since soil-structure equilibrium has reached in time, as mentioned in the previous paragraphs. Figure 3.2 shows another type of soil-structure interaction effect, which is called as doming and dishing. This phenomenon is a result of heaving or shrinking of expansive soils.

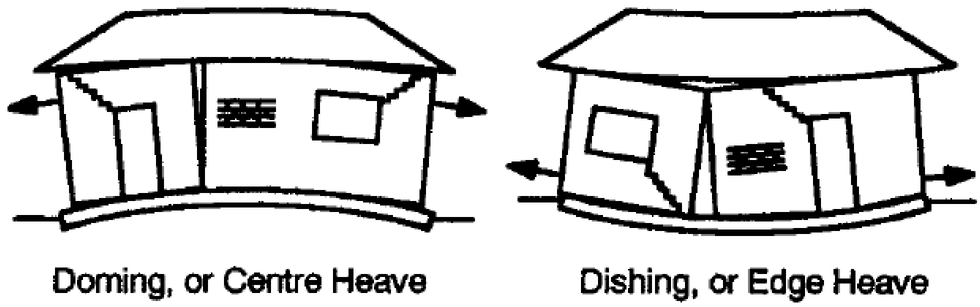

Figure 3.2 Doming and dishing differential foundation profiles [27].

Thermal loads are usually a result of fire. It is a well known fact that fires in the buildings can reach temperatures about 1000 °C and as result of this high temperature, the compressive strength of masonry elements decrease considerably that may cause cracking and crushing in the load carrying elements.

The load effects due to chemical and physical reasons are thermal expansion of calcite, free thaw cycles and salt crystallization. These effects usually cause the masonry material to loose their strength capacity in time.

3.2.2. Types of Failure in Masonry Structures

This section of the chapter describes types of failure in masonry structures. The section is divided into two parts. First part is about the failure modes of masonry. The theory of failure of units and mortar is discussed in Chapter-2. This section only describes the corresponding failure shapes of masonry. Second part of the chapter describes the global failure modes of masonry structures under different load conditions, the emphasis being on the earthquake load. The basic failure modes of masonry are splitting failure due to compression, stepped and vertical cracks due to tension and lastly sliding and stepwise failure due to shear forces.

3.2.2.1. Failure Modes of Masonry

The non-homogenous complex material properties of masonry lead to very different failure modes. The type of failure depends on the orientation of the load, and the direction of head and bed joints.

As stated in Chapter-2, the main reason for the compressive failure of masonry is the different strain characteristics of mortar and masonry units. The failure usually occurs as a splitting failure of which crack pattern is shown in Figure 3.3.

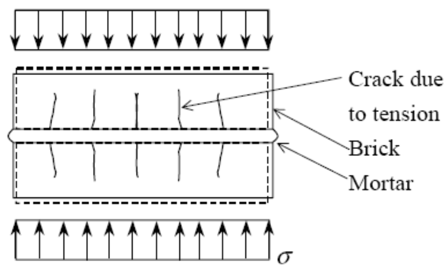

Figure 3.3 Sketch of the splitting failure mechanism [20].

Figure 3.4 shows the failure mechanisms of brick specimens under compressive loading. As can be observed from the figure, the brick specimens act as a whole prism and the cracks go through the cubes continuously. The confinement effect due to testing machine platens can be observed, there are no cracks at the top and bottom faces of the specimens [11].

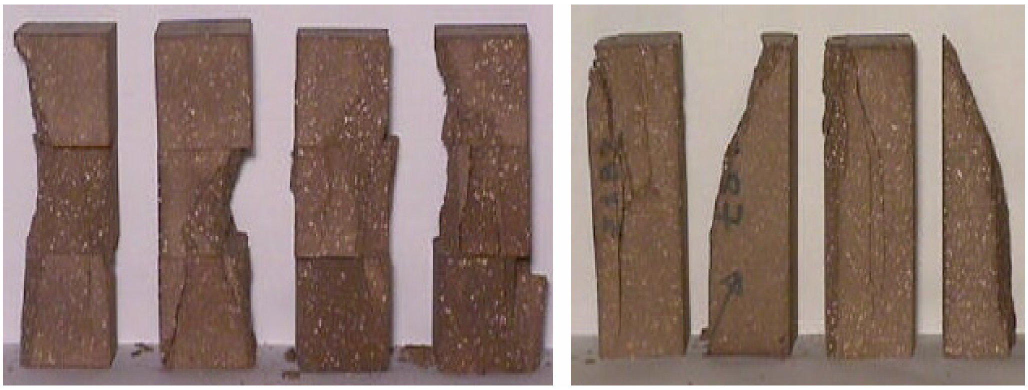

Figure 3.4 Failure mechanisms of the prismatic brick specimens [11].

Figure 3.5 shows the experimental stress-strain diagrams for tension in the direction parallel to the bed joints. The first type of the crack occurs as a stepped crack through head and bed joints. In the second type of failure, the cracks are almost vertical through head and bed joints [6].

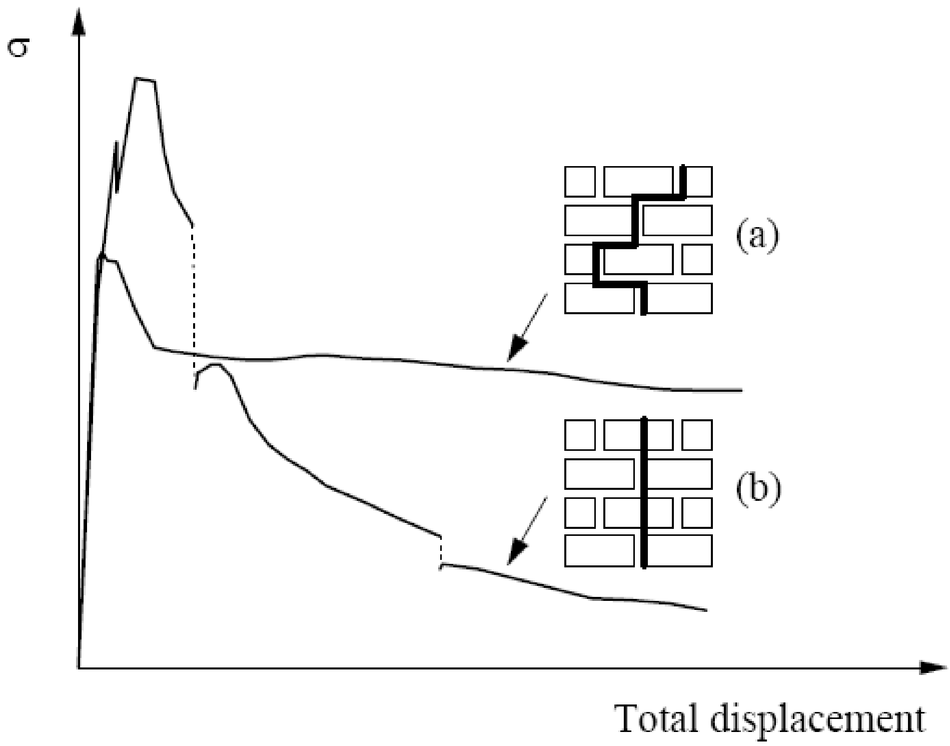

Figure 3.5 Stress-displacement diagrams for tension in the direction parallel to bed joints, a) a stepped crack through head and bed joints b) vertical crack through head joints and units [6].

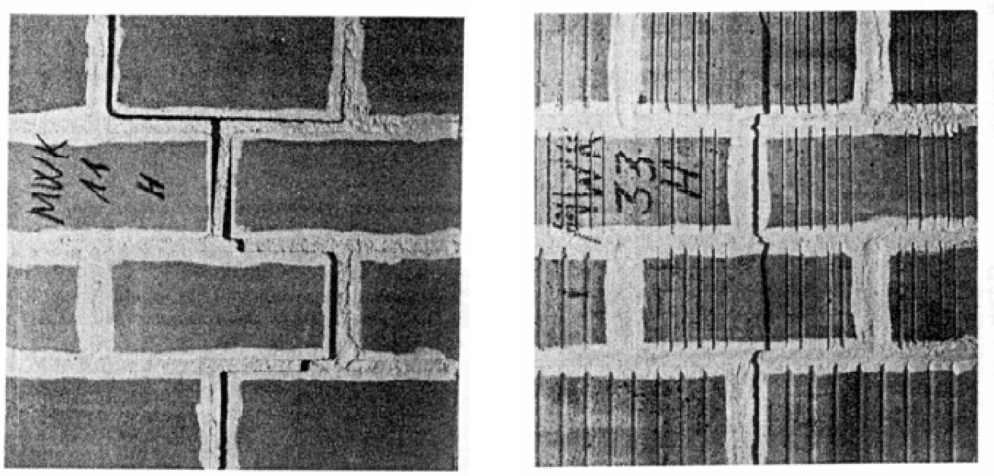

Figure 3.6 Stepped crack and vertical crack photos of the specimens under tensional loading [6].

Shear failure is usually as a sliding failure along joints or a stepwise failure along diagonal. The corresponding failure patterns are shown in Figure 3.7 and Figure 3.8.

Figure 3.7 Sliding failure along joints [2].

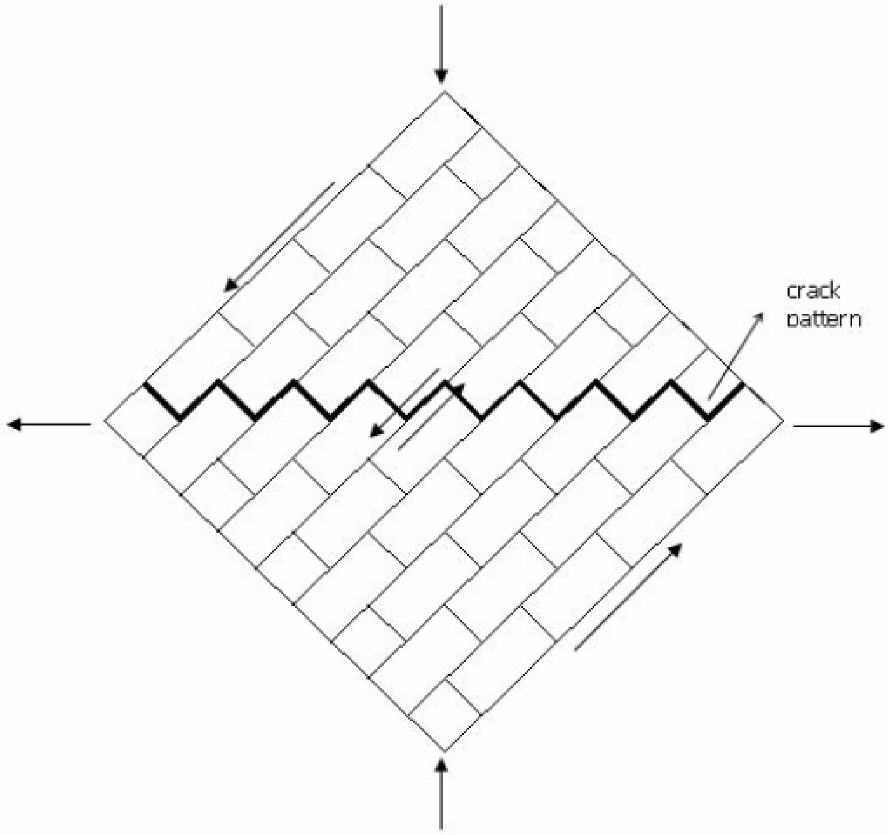

Figure 3.8 Stepwise failure along diagonal [2].

As mentioned before, the crack patterns are related with the direction of loading and also the direction of head and bed joints. Figure 3.9 shows the crack pattern of masonry under uniaxial compression for different bedding plane angles. As can be seen from the figure, under uniaxial compression, the cracks are parallel to the direction of the compressive load. Figure 3.10 shows the crack pattern of masonry under uniaxial tension for the same bedding plane angles given in Figure 3.9. The perpendicular orientation of the cracks can be observed.

The cracks are parallel to the compression and perpendicular to tension in case of biaxial tension-compression. In biaxial compression, masonry cracks in the plane of compression [30]. Figure 3.11 shows the fracture patterns in biaxial stress states for masonry.

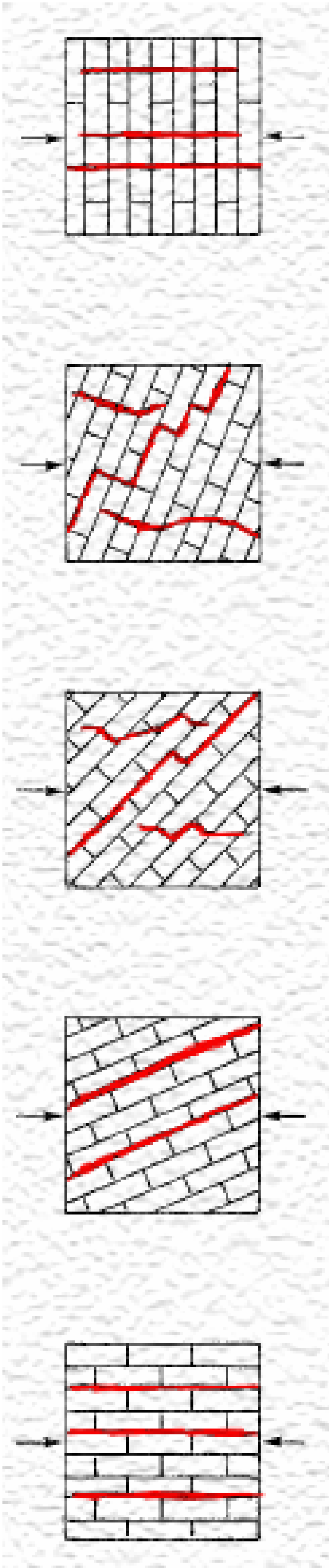

Figure 3.9 Crack patterns of masonry under biaxial compression.

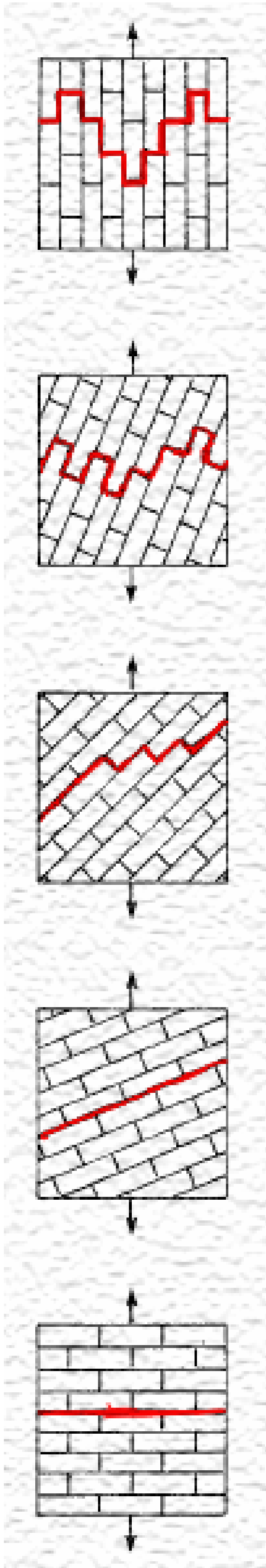

Figure 3.10 Crack patterns of masonry under biaxial tension.

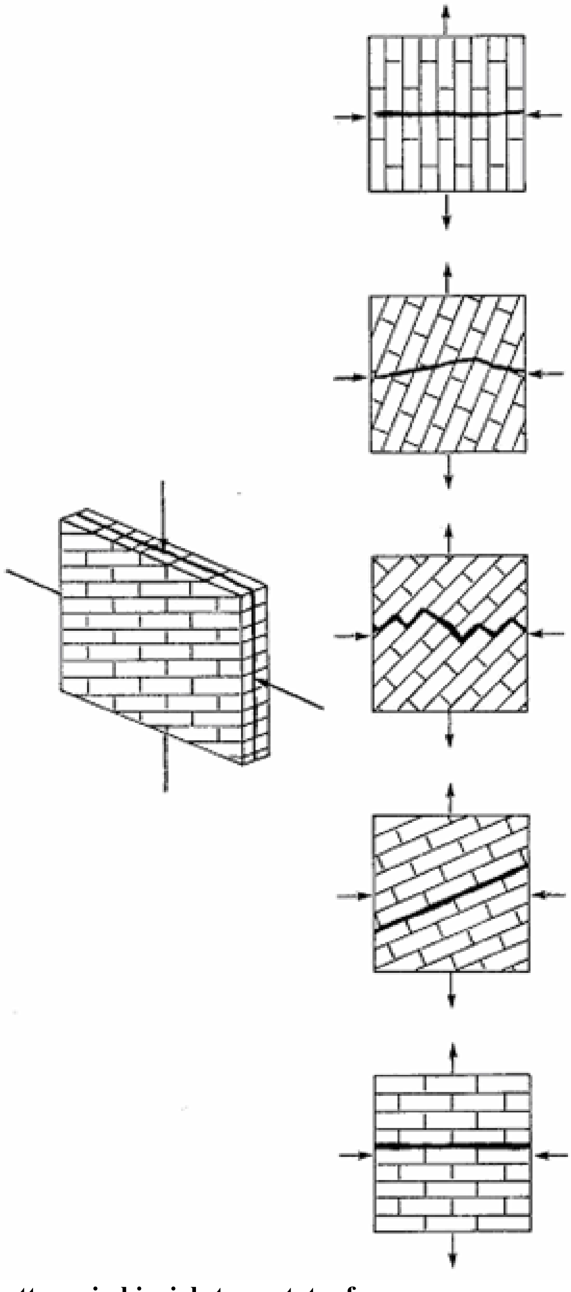

Figure 3.11 Crack patterns in biaxial stress states for masonry.

3.2.2.2. Major Causes and Types of Failure in Historic Structures

As stated in the previous paragraphs, foundation settlement and earthquake are the most important enemies of masonry structures. The low tensile strength and brittle behavior of masonry makes it very sensitive to the support settlements. Figure 3.12 shows the cracks due to support settlement of a masonry church. Figure 3.13 shows the extended cracking along the Orta-Capu tower in Greece due to a landslide.

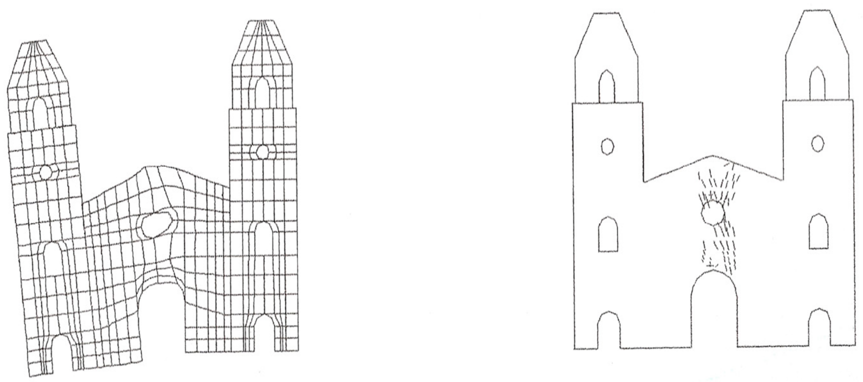

Figure 3.12 Deformed shape and cracks of a masonry church due to support settlement [28].

Figure 3.14 shows the expected failure mechanisms of a load-bearing wall of a Portuguese Castle due to soil pressure.

Figure 3.13 Extended cracking along the Orta-Capu tower due to a land slide [25].

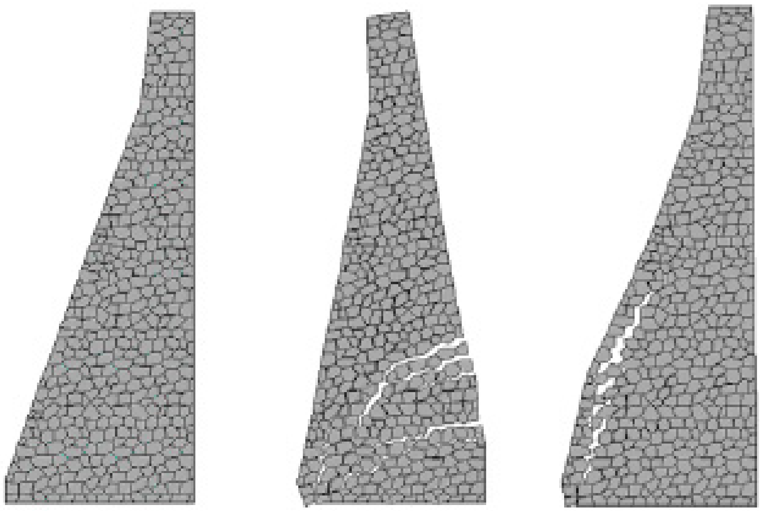

Figure 3.14 The undeformed shape and expected failure mechanism of a load bearing wallof a Portuguese Castle [29].

The collapse mechanism of masonry structural elements under earthquake loads varies greatly. Generally, the type of structural element and the direction of lateral load determine the collapse mechanism. Figure 3.15 shows seismic behavior and collapse mechanisms for different structural elements. The structural elements given in this figure are a single column, post-and-lintel and a roman arch, respectively. Figure 3.16 is an illustration of a panel masonry wall that is loaded horizontally in plane. As can be observed from the figure, the collapse mechanism is in the form of sliding or overturning, or a combination of both [31].

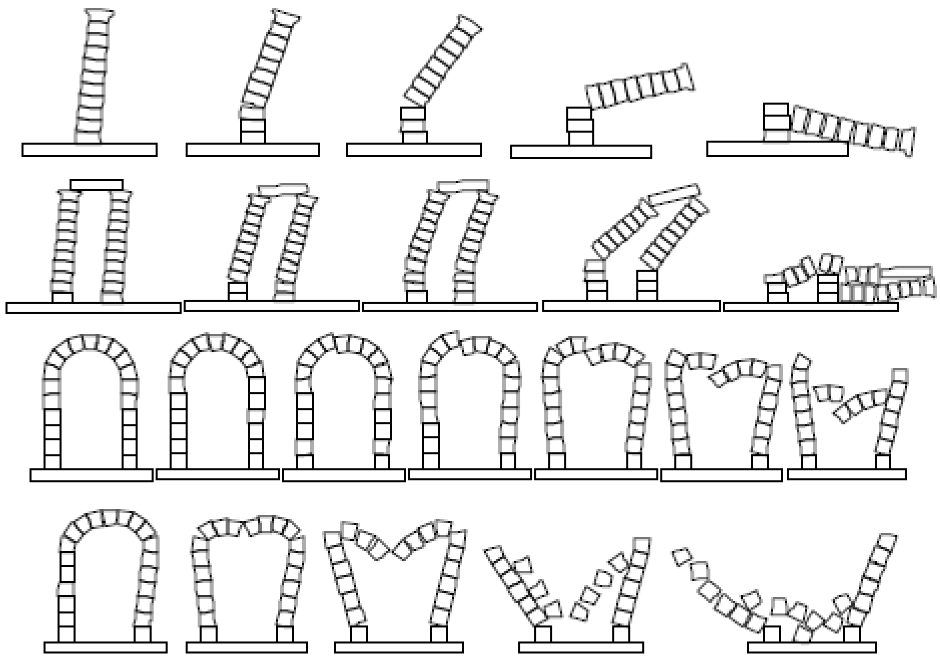

Figure 3.15 Seismic behavior and collapse patterns for different structural elements [5].

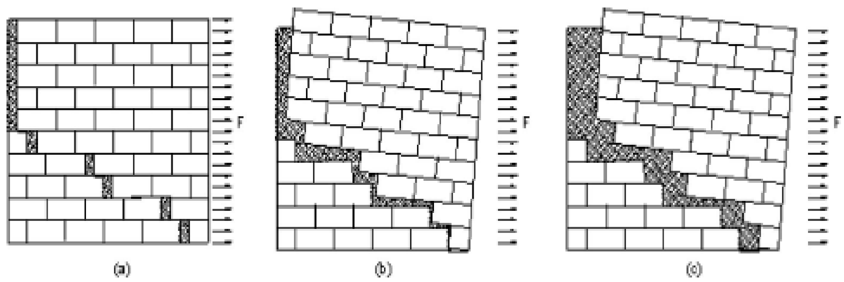

Figure 3.16 Collapse mechanisms of a panel wall under horizontal loading, a) sliding b) overturning c) mixed mode [31].

Damage to masonry buildings can be divided into two groups. The first one is a result of seismic actions perpendicular to the wall (out-of-plane) that leads to overturning. The latter is a result of the forces that act parallel to the wall (in-plane) that cause "X" shaped cracks [32]. Figure 3.17 shows basic damage modes of a masonry building. Figure 3.18 shows the expected collapse sequence for the S. Giorgio in Trignano bell tower. As can be seen from the figure, the masonry material disintegrates because of the lateral load due to earthquake, which results in a total collapse.

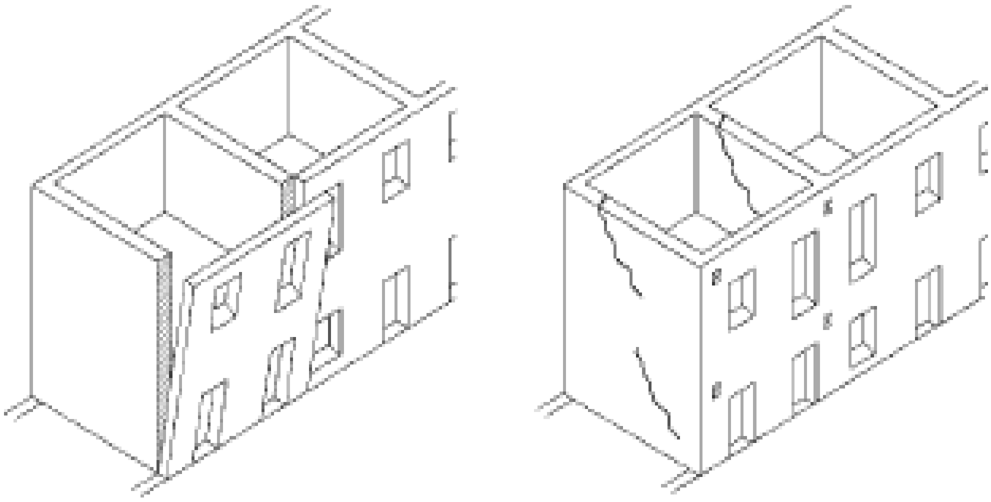

Figure 3.17 Basic damage modes of a masonry building [32].

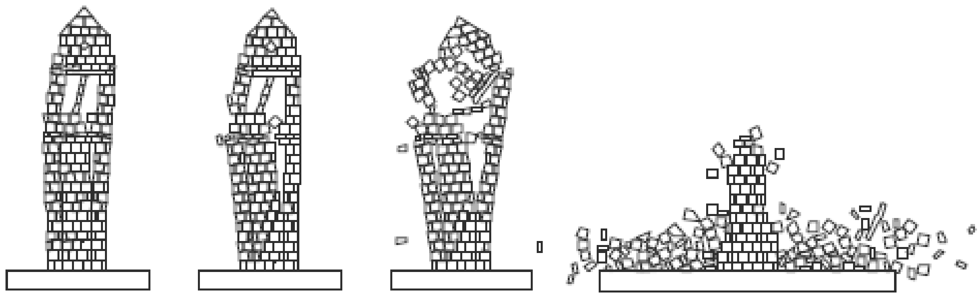

Figure 3.18 Collapse sequence of a masonry tower [5].

3.3. Numerical Modeling and Analysis of Masonry Structures

This part of the thesis is about numerical modeling and analysis methods of masonry structures. The definition of material properties of the materials, the definition of the geometry of the structure, as well as the definition of the loads acting on a masonry structure is different from those in contemporary buildings.

3.3.1. Structural Analysis in History

The basic method for transferring the knowledge of construction to the apprentices was based on the experience of the mason masters in ancient times. However, this transformation was quite different from the methods that modern structural engineers are used to. Today's structural design is based on construction practices and the art of structural engineering. The loads that are expected to act on the structure during its life time, the mechanical properties of the construction materials and the structural analysis methods are rely on a group of codes and norms with a strict supervision of the employer. However, in ancient times the design process was based on experience. No structural analysis was performed. Rather, the geometrical proportions that were obtained through experience were used. For example, the masons of medieval had no methods for calculating the amount of buttressing and they used their observation and experience for the margin of safety. The geometrical proportions was so important that, the general idea " if a building is proportionally correct, it is structurally correct" was a general and excepted rule. The theoretical explanations of structural behaviour were tried to be explained in Renaissance with Leonardo da Vinci. Long after Leonardo, Simon Stevinus published a book on statics and this book provided the basis for 19th century in graphics statics, that enabled the solution of structural problems by using drawings [6,18]. Since then the structural analysis evolved from classical statics to the advanced finite element analysis methods with the improvement of new numerical and calculation methods.

3.3.2. Modern Structural Analysis of Masonry

With the development of computers and new numerical methods, the structural analysis of masonry has become easier, however it is still very difficult when compared to those of modern structures.

Among the various analysis methods, the complex geometrical and mechanical features of masonry make the finite element method best suitable for the analysis of masonry structures. By using finite element method, it is possible to model and analyze very different geometrical shapes, complex loads or support conditions.

The basic logic of finite element method is subdivision of the structure into small sized elements that are called finite elements. The connection points of the elements are called nodes. The stresses and displacements are calculated for each finite element and these results are transferred to the whole structure. In other words, the finite element converts the structural problem into a finite degrees of freedom [1, 2, 33].

The first step in the finite element analysis is the idealization of the structure to obtain the geometrical model. The geometrical model consists of finite elements. After the application of the loads to the meshed model, the solution is performed by using numerical methods. The type of the finite elements and numerical methods has been developing for the intended use with the aid of computers.

3.3.3. General Principles of Analytical Modeling

The aim of analytical modeling is to represent the behavior of a real structure in mathematical terms. Because of this reason, before any analysis, a model should be developed in order to understand the effects of loads, understand the load transfer mechanism within the structure and to determine the load capacity etc. The need for a simple model arises from the complexity of the nature of historical masonry construction materials. Under this circumstance, some idealizations are needed. The most important idealizations through analytical modeling are;

- Idealization of geometry

- Idealization of material behavior

The idealizations will be explained briefly in the following sections; however a good analytical should be as simple as possible as long as it is adequate to represent the effects of loads on the structure that is to be investigated [1,18,24].

3.3.3.1. Idealization of the Geometry

One of the basic principle of creating an analytical model is creating a geometrical model. However, it is difficult to distinguish between the structural and decorative elements in case of historic masonry structures. As a general rule, the geometric idealization should be as simple as possible providing that the model is adequate for the problem being analyzed [18,24].

The geometrical representation can be made by using frame, shell or solid elements. No element type is superior to the other. The decision of the element type is completely dependent on the complexity of the problem. For example it would be unnecessary to use solid elements for the out of plane investigation of a masonry wall. Instead of solid elements, it would be enough to use shell elements for such a kind of investigation. When the concern is the investigation of a thick wall for in-plane loading for example, using shell elements would be a bad choice since it would be very difficult to investigate the stresses through the thickness of the wall.

3.3.3.2. Idealization of the Behavior

The basic idealizations for the analysis of masonry are elastic behavior (linear), inelastic behavior (non-linear) and plastic behavior.

The basic assumption of elastic analysis is that the material obeys Hooke's law. The increase or decrease of strain is directly proportional to the decrease or increase of stress. The deformations are fully recovered when the applied actions are removed. Linear elastic behavior is mostly valid for the masonry under tensional loading.

Plastic analysis methods are usually performed for determining the load at failure. However, the application of plastic analysis is not practical for large structures [18, 24]. When plastic analysis methods are concerned, the main assumption is that masonry has no tensile strength and infinite compressive strength.

The most powerful and realistic idealization method for the analysis of masonry is the non-linear material assumption. The behavior of the structure can be observed through elastic range up to the time of failure. The cracks through loading can be simulated by redistribution of stresses where cracks occur. However, a reliable non-linear analysis requires the material properties to be determined through experimental testing.

The method for numerical representation depends on the scale of the problem and the intended calculations. A detailed micro-element represents the behavior of mortar and masonry separately. Simplified micro-element represents the blocks as a continuum, however the mortar interface is assumed to be a lumped interface. The macro models are used for plastic analysis and they represent the mechanical properties of masonry as a homogeneous material [6]. Figure 3.19 shows the three alternative representations for the modeling of masonry.

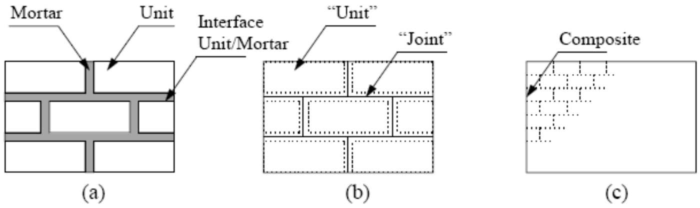

Figure 3.19 Modeling strategies for masonry structures, a) detailed micro-modeling b) simplified micro-modeling c) macro-modeling [6].

The number of parameters for an elastic analysis is much less then the parameters required for an inelastic analysis. For a complete inelastic analysis the number of parameters about the mechanical properties of the masonry materials is quite high, and it is difficult to determine all these mechanical properties (i.e. the compressive, tensile and shear strength as well as the stress-strain diagrams, the modulus of elasticity, the Poisson's ratio of the materials in three principal directions are required). Also non-linear analysis requires quite a long time. However, the complete behavior of the structure can be observed through elastic range up to the failure phase. Plastic analysis is used for the capacity determination and it is usually valid for simple structural forms, most of the time application of plastic analysis methods are not adequate for the investigation of large and complex geometrical forms. As a general comparison non-linear analysis gives all the information about the behavior of the structure, whereas plastic analysis can determine only the ultimate load capacity. Elastic analysis is the simplest method for the analysis and gives general information about the behavior of the structure. As can be inferred from discussions about the analysis of historic masonry, it can not be claimed that one analysis method is superior to other. The scale of the problem, the availability of the mechanical properties and the intended use determine the type of structural analysis.

CHAPTER-4

LINEAR AND NON-LINEAR ANALYSIS OF A MASONRY STRUCTURE: A CASE STUDY, HASANKEYF GATE

4.1. Information on the Historical City of Hasankeyf and the Case Study Structure

There are a lot of historical buildings in Anatolia since it hosted many civilizations throughout history. Although these historical monuments are unique and are the symbols of our heritage, they are not given the importance they deserve. Conservation of historical monuments is one of the most important points for continuity of history and thus, for protection of cultural heritage [2].

The name of Hasankeyf comes from the Arabic genitive construction, "Hısn Keyfa". It means "steep rock" and has been used for centuries. Chronologically, Hasankeyf was the indispensable city of the Mesopotamian civilizations, the Byzantine stronghold of the east, the treasured city of the Islamic period. The earliest knowledge about Hasankeyf belongs to the Byzantine period, but the cave dwellings prove that it dates back further. Hasankeyf district is entirely interesting with its caves that belong to various times. Besides the caves, the natural residences carved into rocks, cave dwellings with water ways, examples of antechambers and multi-storied architecture, a seven-niche mosque carved into rocks, some churches and cemeteries could be listed as the most striking landmarks.

Unfortunately, the Hasankeyf settlement is under the risk of being flooded by the reservoir of Ilisu Dam which is planned to be constructed on the Tigris River. Because of the fact that Ilisu Dam reservoir will cover the integrated natural and architectural heritage in Hasankeyf, a comprehensive documentation of the site and monuments must be done before the construction of the dam.

The case study structure is a masonry gate, constructed in the 12th century, which is located in Hasankeyf. The ratio of its height to its thickness makes this structure suitable to observe the in-plane and out-of-plane behavior more clear. Figure 4.1 shows the general appearance of the structure. The width and the height of the structure are 6.85 and 14.5 meters respectively. The thickness of the structure is about 1.6 meters. As can be seen from Figure 4.1 the left side of the Gate is supported by the ground which is considered as translational restraints in the modeling section.

Figure 4.1 A photo of the Hasankeyf Gate.

4.2. Information on the Software Used in the Study

Through the study SAP2000 and ANSYS software are used. SAP2000 is a finite element package used mainly by civil engineers. It can analyze general structures, such as bridges, buildings, dams, solids etc. ANSYS is a very advanced finite element software usually used for advanced simulations and academic purposes. Any kind of physical problem such as fluid flow, contact analysis, static and dynamic analysis can be simulated by ANSYS.

The solid element used by SAP2000 is an eight-node finite element used for modeling solids. Each solid element has its own local coordinate system for defining material properties and interpreting output. Stresses in the element local coordinate system are evaluated at the integration points and approximated to the joints of the element. Each solid element has six quadrilateral faces, with a joint located at each of the eight corners as shown in Figure 4.2 [34].

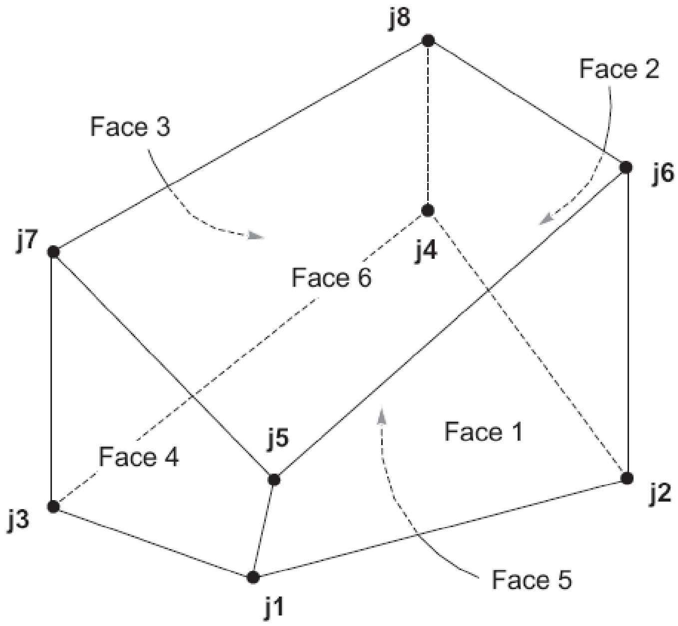

Figure 4.2 Solid element joint connectivity and face definitions of SAP2000.

The solid elements of SAP2000 has three translational degrees of freedom at each joint. Rotational degrees of freedom are not active for the solid element. The stresses are evaluated by using the standard Gauss integration points of the elements and extrapolated to the joints [34].

ANSYS has numerous kind of solid elements. SOLID65 element is used in this study. SOLID65 is used for three-dimensional modeling of solids with or without reinforcing bars. When the amount of reinforcing bars are assigned to 0, the element is adequate for the representation of the behavior of masonry. SOLID65 element is capable of cracking in case of tension and crushing in case of compression. The elements are defined by eight nodes with three degrees of translational degrees of freedom. The geometry, node locations and coordinate system of this element is shown in Figure 4.3 [35].

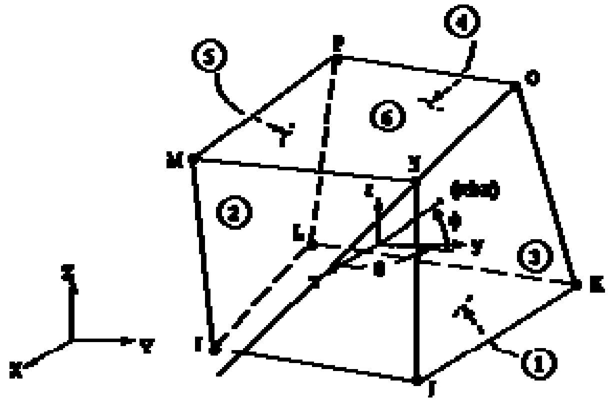

Figure 4.3 The node locations, geometry and coordinate system of SOLID65 element.

4.3. Model Definition

A finite element model of the Gate is created with the same material characteristics by using both SAP2000 and ANSYS Software. For the analysis cases some assumptions about the material and strength characteristics of the Gate are made by choosing the general values that are usually offered in the literature. The material is also assumed to have a homogeneous isotropic behavior. The reason for these assumptions are that there were no experimental data for the material characteristics of the Gate. However, since the aim of the study is to compare elastic and inelastic analysis, these assumptions are thought to be not very dramatic for the study. The constituve model is a macro model with the given elastic material properties:

- Modulus of Elasticity (E) : 10000 MPa
- Poisson's Ratio (υ) : 0.2
- Unit weight (γ) : 2 t/m^3

For the nonlinear analysis, the material characteristics are also chosen from the literature considering the general stress-strain curves that are obtained by experiments performed by various researchers. As stated in the previous chapters, one of the most predominant characteristic of masonry is that it has a very low tensile strength. This property is so important that the form of masonry structures has been determined such that the structural elements are always in compression. The uniform compression strength of the macro model elements is assumed to be 0.2 MPa, while the compression strength is 30 MPa. The high compression strength may seem extreme; however some macro model techniques assume 0 tension strength and infinite compression strength. The shear transfer coefficients are taken 0.1 for open cracks and 0.9 for closed cracks. This means that 90% of the force is redistributed to the adjacent nodes when a crack opens and 10% of the force is redistributed when a crack closes. The stress-strain relation for the macro element model is given in Figure 4.4. As can be seen from the graph, the material behaves elastic until the strain is 0.003. When this value is exceeded, softening behavior starts and the material is fully crushed when the strain is 0.008.

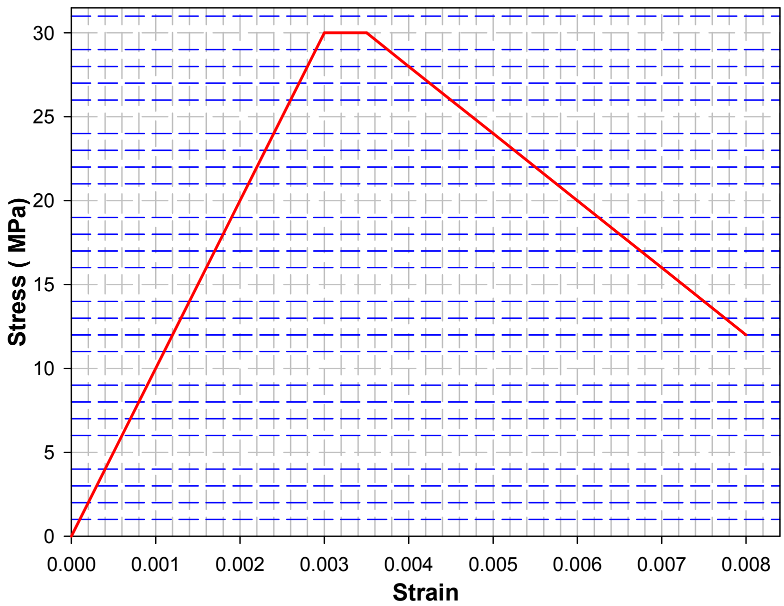

Figure 4.4 Stress-strain relation for the macro element model

Figure 4.5 and Figure 4.6 shows the finite element model of the Gate created with the same material properties and with the same geometry by different software. The model is restrained for the translational movement from the bottom and left side. Figure 4.7 shows the restraints of the model. The green triangles are the points where the model is restrained against translational movement.

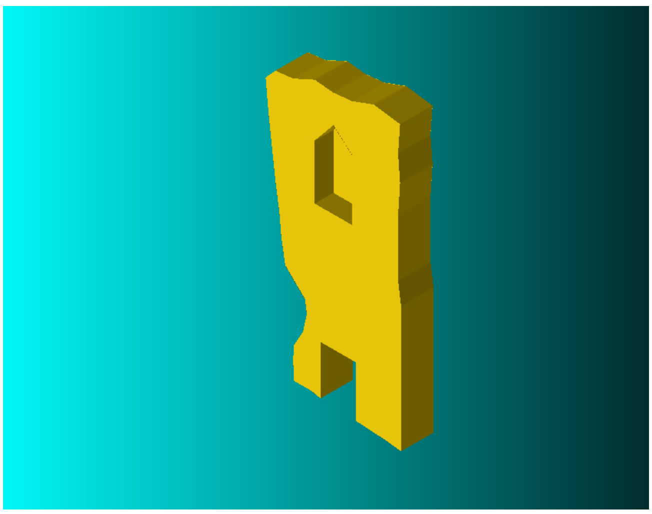

Figure 4.5 The finite element model of the Gate created by ANSYS

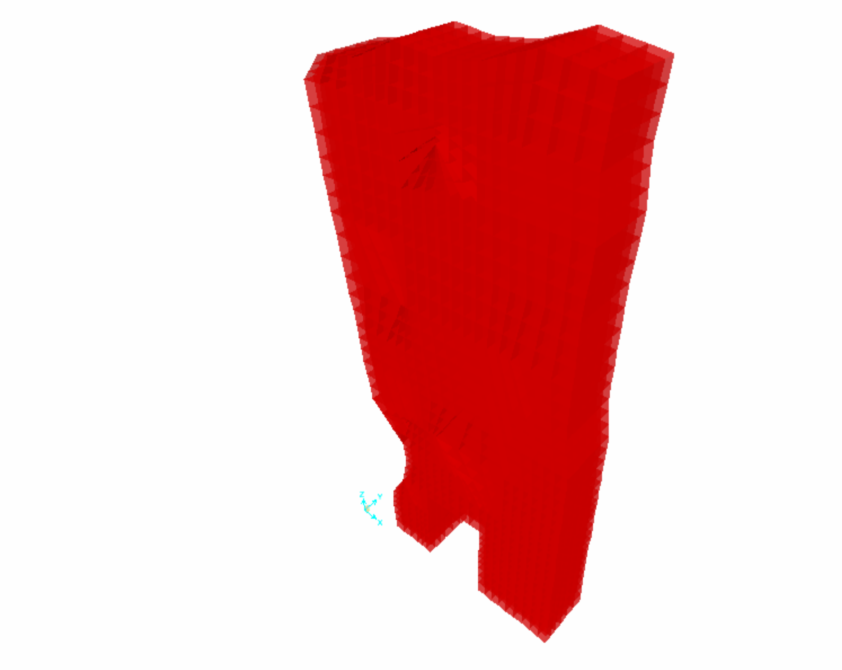

Figure 4.6 The finite element model of the Gate created by SAP2000

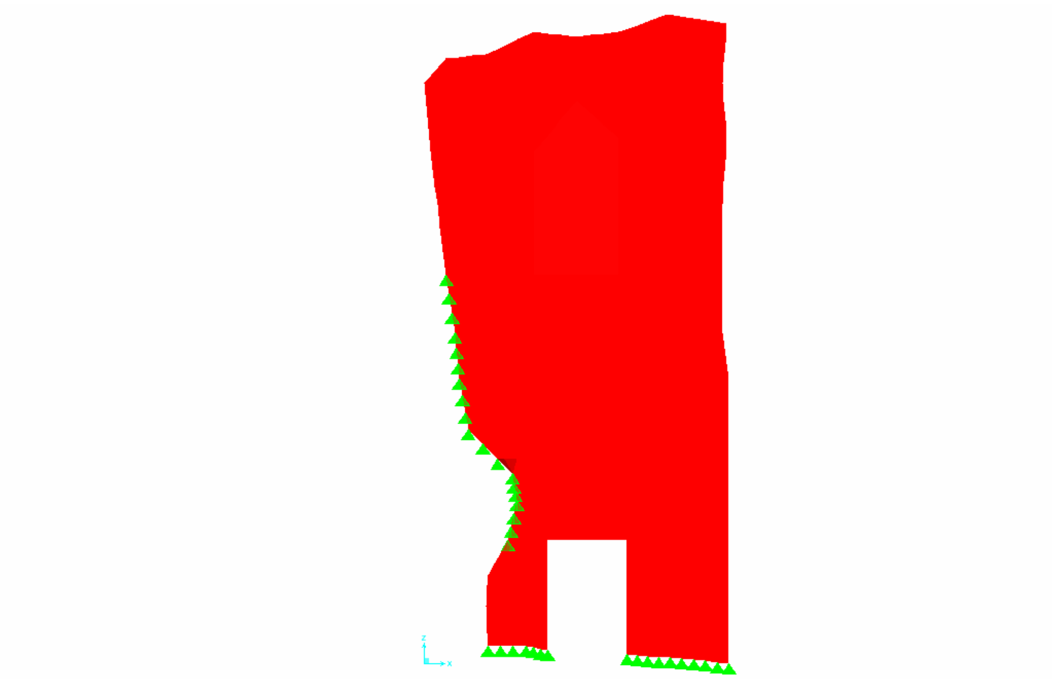

Figure 4.7 Restraints of the finite element model

4.4. Model Verification

Before any analysis it is a must that the model must be verified. The best way for the verification is that to compare the analysis results with the experimental results. However, for a full scale analysis it is almost impossible to test the whole structure. In the scope of the study, the model verification is based on comparisons between different software and also some expected behavior of a masonry structure.

4.4.1. Modal Verification

For the modal verification, the modal deformed shapes for ANSYS and SAP2000 are compared and it is observed that the modal deformed shapes are almost identical. The following figures show the deformed shapes for the first 6 modes. In fact, a real structure has nearly infinite number of modes. However, for applications in practice, all of the modes are not concerned. For an ordinary building the number of modes considered is usually 3 or 4. When Figures 4.8 and 4.9 are investigated, it can be observed that the first three modes are deformations in the strong and weak direction and also torsion effect. As the number of modes increases, the modal deformed shapes become more complicated which is due to different deformation combinations of the nodes. This phenomena can be observed for the Gate also. When Figure 4.9 is investigated, it can be observed that the deformed shapes become more complicated when compared with the first 3 modal deformed shapes.

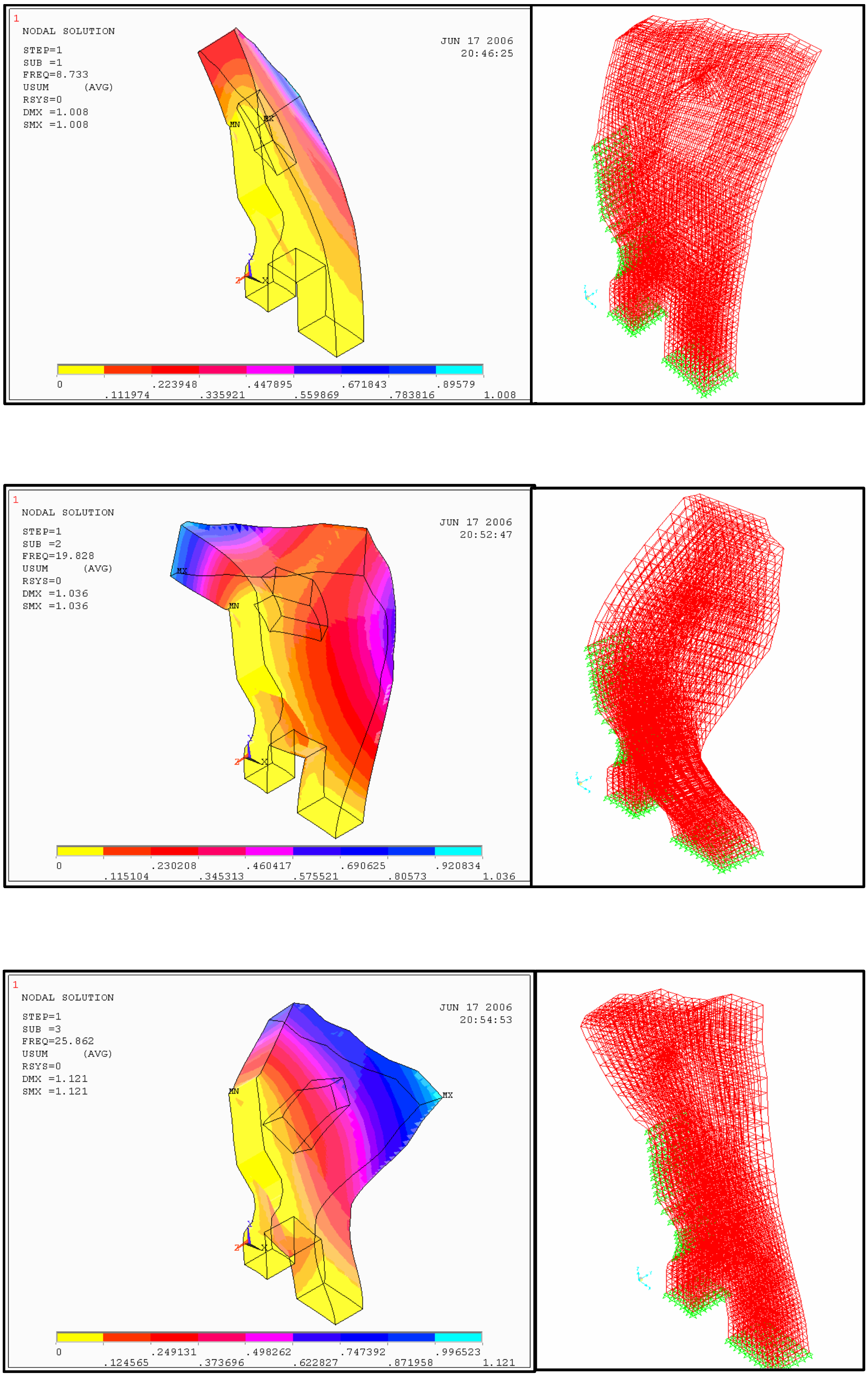

Figure 4.8 Modal deformed shapes for Modes 1,2 and 3, respectively.

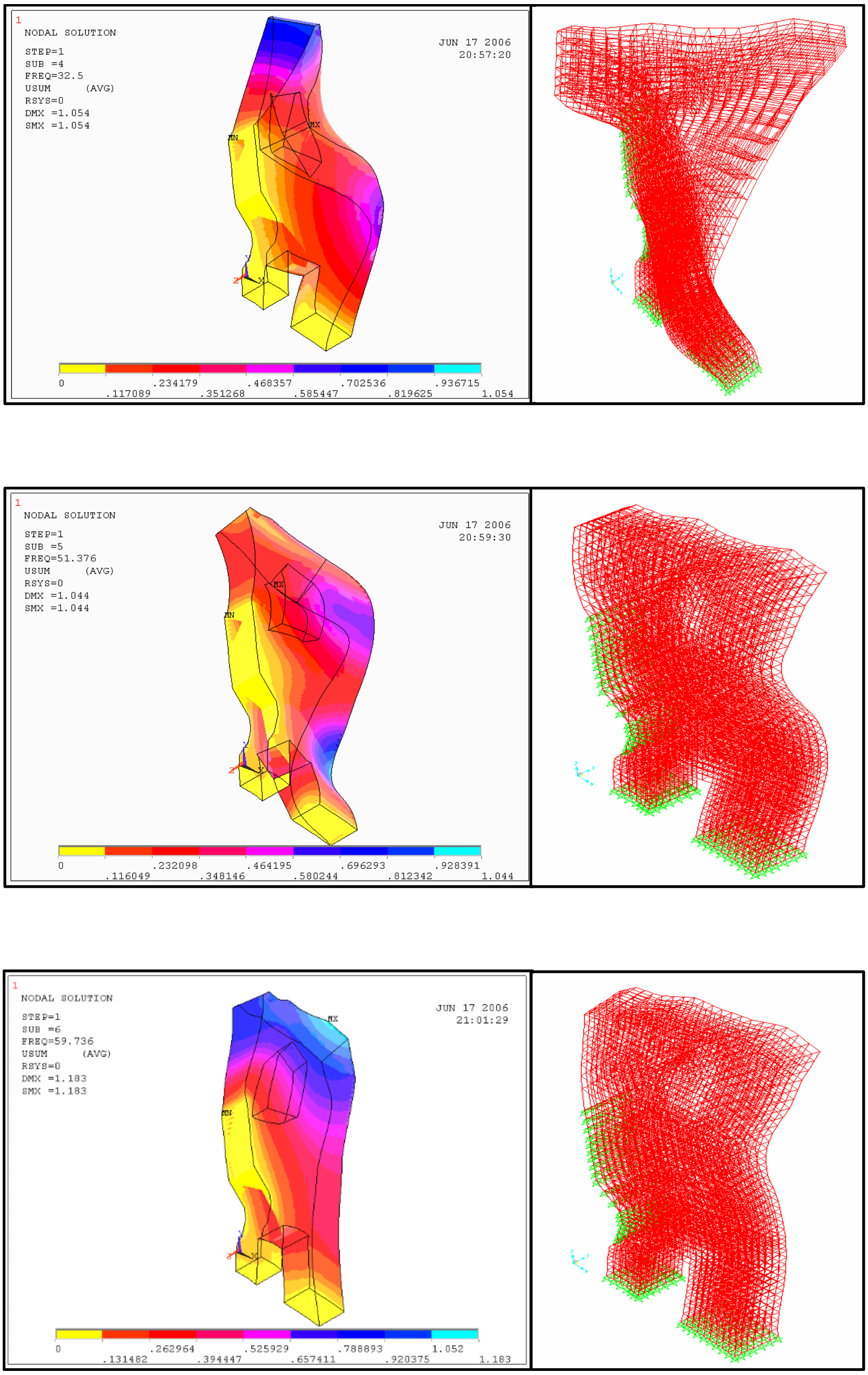

Figure 4.9 Modal deformed shapes for Modes 4,5 and 6, respectively.

The corresponding modal periods for the first 6 modes are given in Table 4.1. One of the expected behavior of this kind of a masonry structure is low modal periods. The results verify this expectation. Also the calculated modal periods are very close to each other for SAP2000 and ANSYS.

Table 4.1 Comparison of the Modal Periods for SAP2000 ad ANSYS

MODE	ANSYS	SAP2000
1	0.1145 (s)	0.1122 (s)
2	0.0504 (s)	0.0496 (s)
3	0.0386 (s)	0.0381 (s)
4	0.0308 (s)	0.0304 (s)
5	0.0195 (s)	0.0193 (s)
6	0.0167 (s)	0.0166 (s)

Figure 4.10 and Figure 4.11 show the modal periods and the percentage error corresponding to the modal periods assuming ANSYS as the reference. As can be seen from the figures the modal periods are very close to each other and the maximum percentage error is 2%.

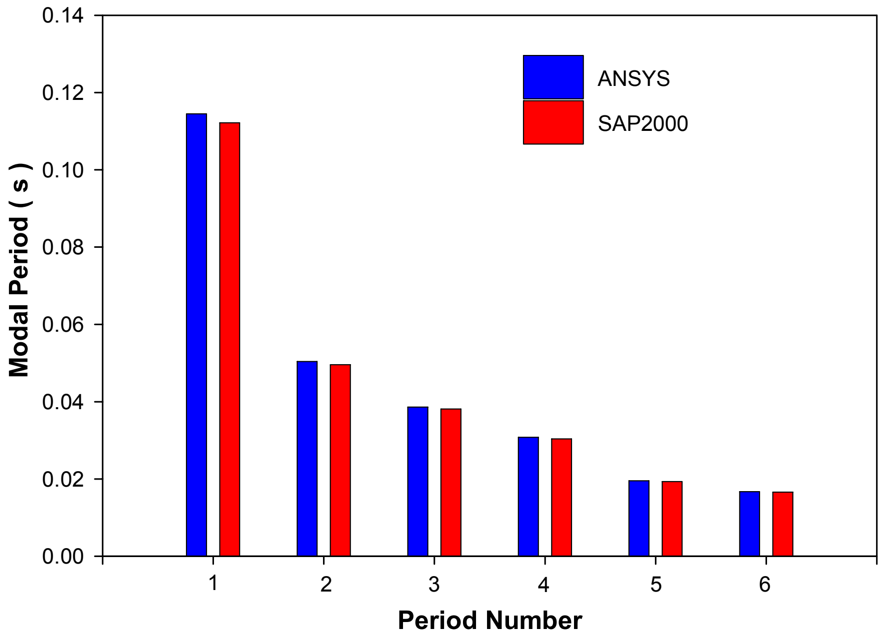

Figure 4.10 Vertical bar chart of the modal period values for ANSYS and SAP2000.

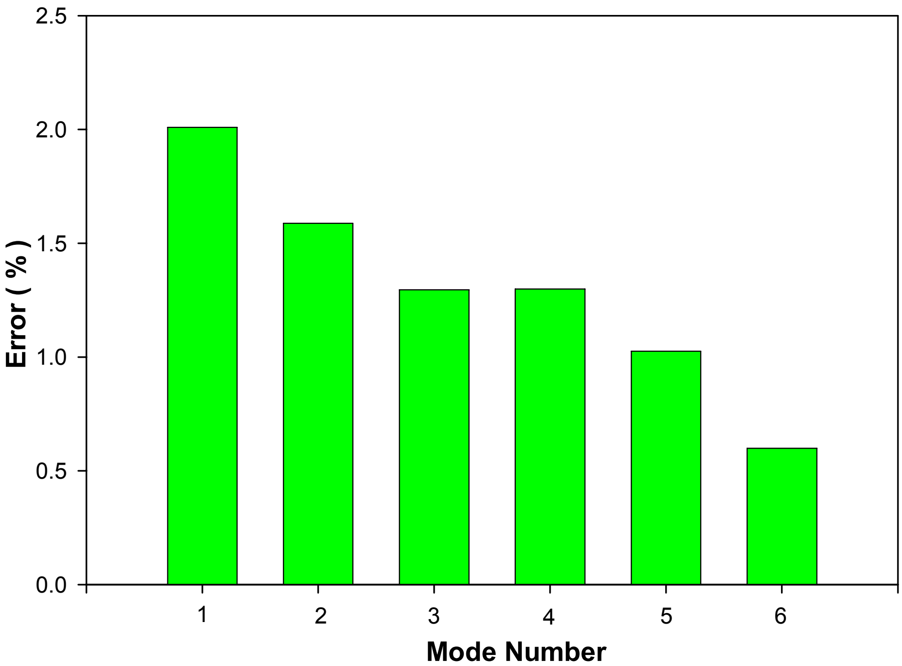

Figure 4.11 Vertical bar chart of the percentage error for modal periods assuming ANSYS as the reference

4.4.2. Dynamic Verification of the Model

The low modal periods mean that the structure is rigid. However, the rigidity of the structure is also verified by performing a dynamic analysis. For this purpose, one horizontal component of the 1989 Loma Prieta Earthquake recorded at Gilroy Gavillan College is applied to the base of the structure and the acceleration at the top of the structure is observed. The earthquake data is chosen randomly, since the verification of the rigidity of the structure is not expected to be influenced by the earthquake. The maximum acceleration of the applied earthquake was 3.19 m/s^2 and the maximum acceleration observed at the top of the structure was 3.22 m/s^2. Besides this observation, the acceleration traces at the base and at the top of the structure are very similar in pattern. These observations are another verification of the rigid behavior of the structure. Figure 4.12 and Figure 4.13 show the acceleration traces of the base and the top respectively.

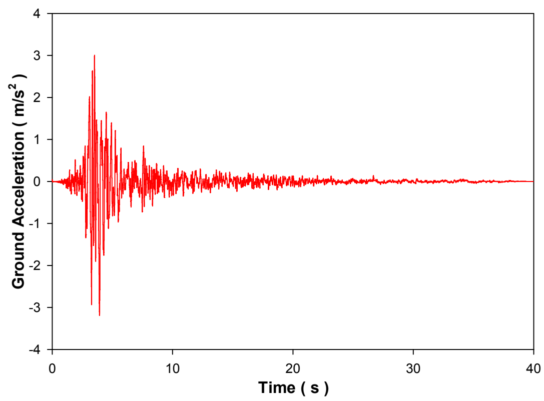

Figure 4.12 Acceleration trace of one horizontal component of the 1989 Loma Prieta Earthquake recorded at Gilroy Gavillan College applied to the base of the structure (PGA=3.19m/s^2).

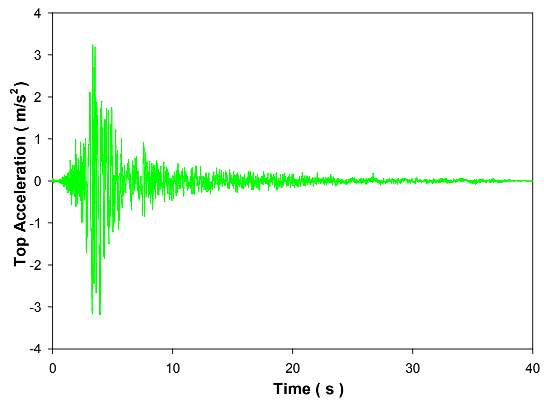

Figure 4.13 Acceleration trace observed at the top of the structure (Max Acc.=3.22m/s^2).

4.5. Analysis Results

After the verification of the model, linear and non-linear analysis are performed. Three load cases were considered for the analysis. These were;

- An inertial force to the right in the strong direction (Load Case-1)
- An inertial force to the left in the strong direction (Load Case-2)
- An inertial force to the left in the weak direction (Load Case-3)

For the load cases described above, a constantly increasing inertial force is applied until a mechanism was observed by the nonlinear analysis. For the linear case, the maximum load at the time of failure obtained by the non-linear analysis was applied to the structure and some observations are made. Figures 4.14, 4.15 and 4.16 show the direction of the inertial force for 3 different load cases for a better understanding.

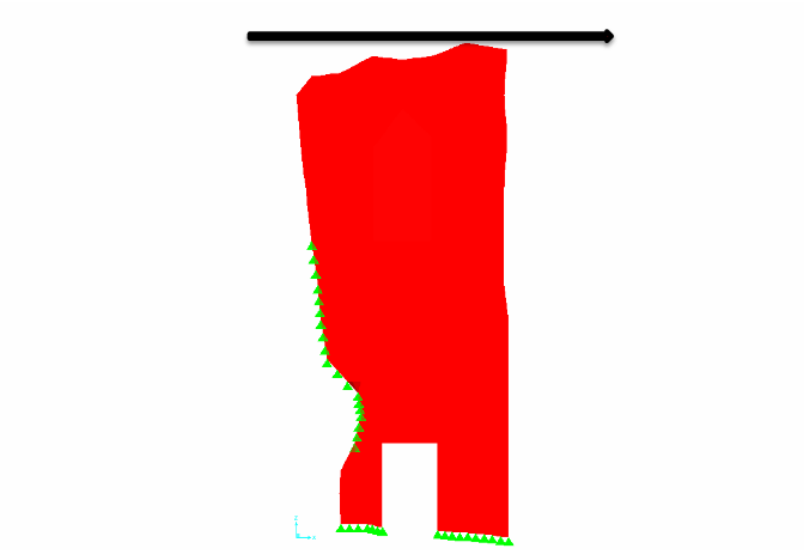

Figure 4.14 Load Case-1, inertial force is to the right (in-plane).

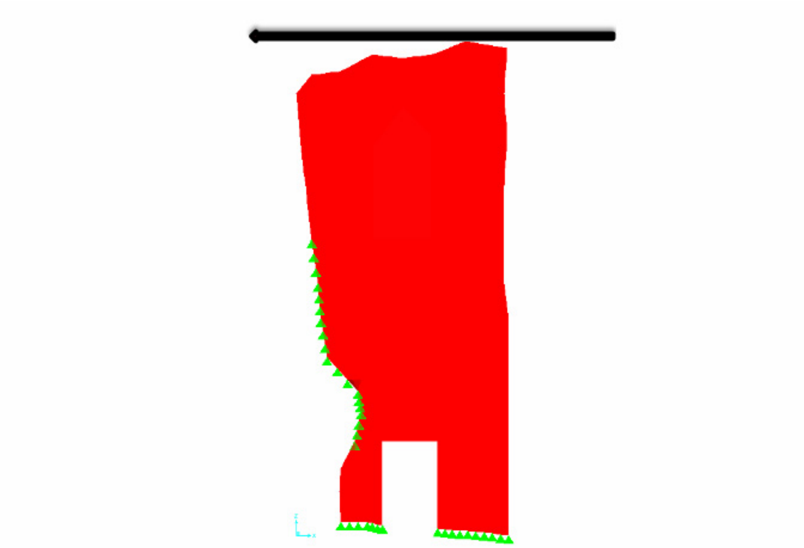

Figure 4.15 Load Case-2, inertial force is to the left (in-plane).

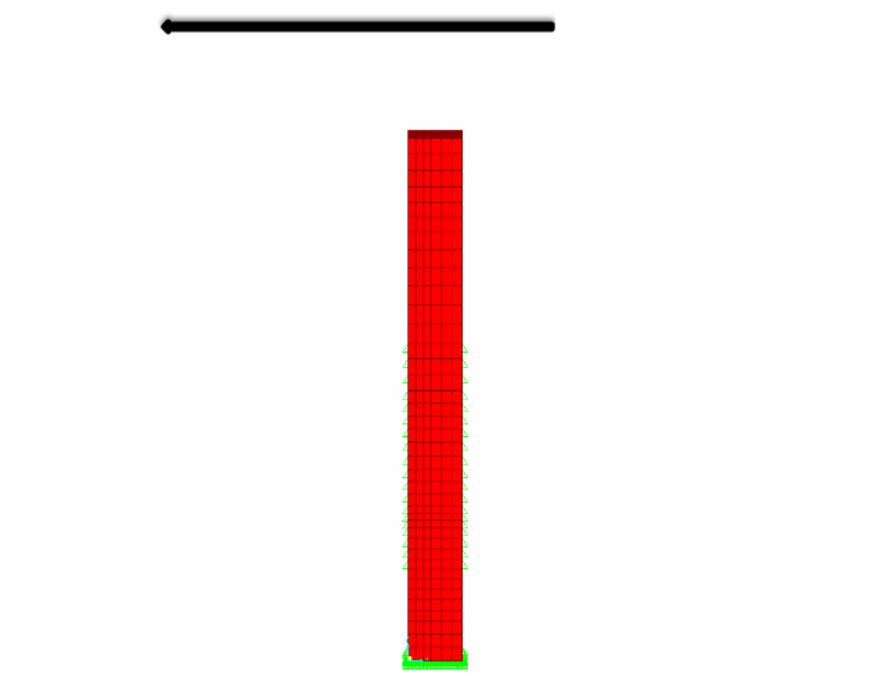

Figure 4.16 Load Case-3, inertial force to the left (out-of-plane).

4.5.1. Load Case-1 Analysis Results

For Load Case-1, a non-linear analysis is performed and this analysis result is assumed to be the reference analysis for Load Case-1. The load displacement diagram is given in Figure 4.17. When the load displacement diagram is investigated it is observed that a very brittle failure occurs when the inertial force is 5.78 m/s^2. The maximum observed deformation is 0.29 mm. The small displacement at a large force can be explained by the model restraints and very little tensile strength. This phenomenon implies that a very sudden crack occurs and the structure fails because of this sudden crack. The initial cracks are shown in Figure 4.18 and the cracks for the unconverged solution are shown in Figure 4.19 (red points show the location of the cracks).

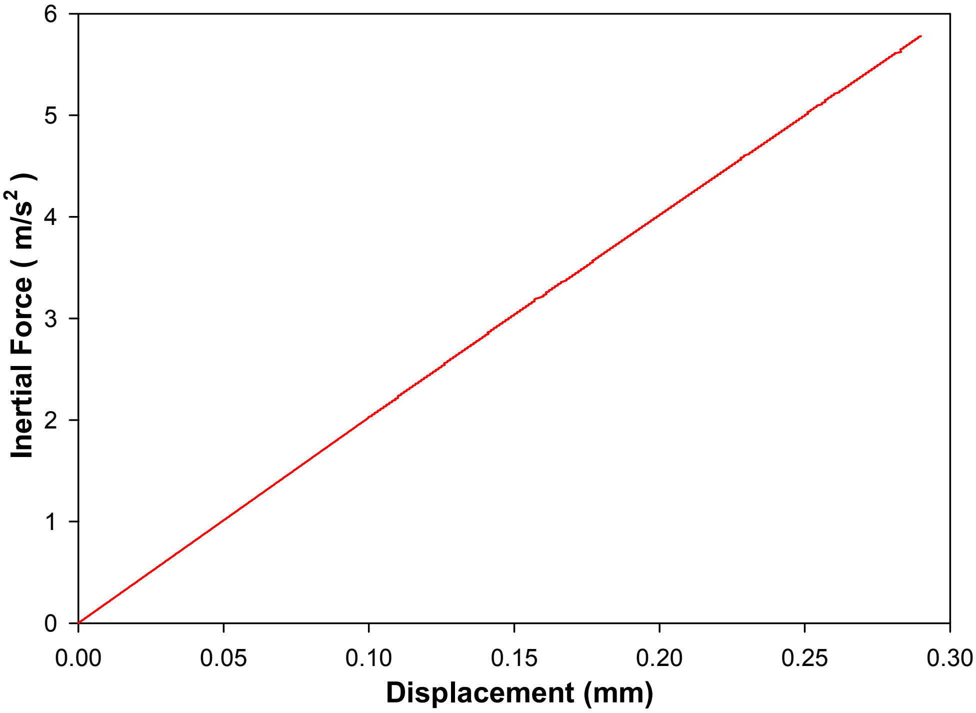

Figure 4.17 Load-displacement diagram for Load Case-1.

Figure 4.20 shows in-plane displacement for the non-linear analysis. In Figure 4.21 the same displacements are shown for linear analysis. For non-linear analysis, the maximum displacement is observed as 0.29 mm and for the same load level, linear analysis gives a maximum displacement of 0.25 mm.

Figure 4.22 shows the linear analysis results for the case where the positive stress is greater than 0.2 Mpa in any direction which implies the locations of the cracks. When the Figure 4.23 is investigated, it can be observed that the stresses are in harmony with the crack pattern obtained by non-linear analysis.

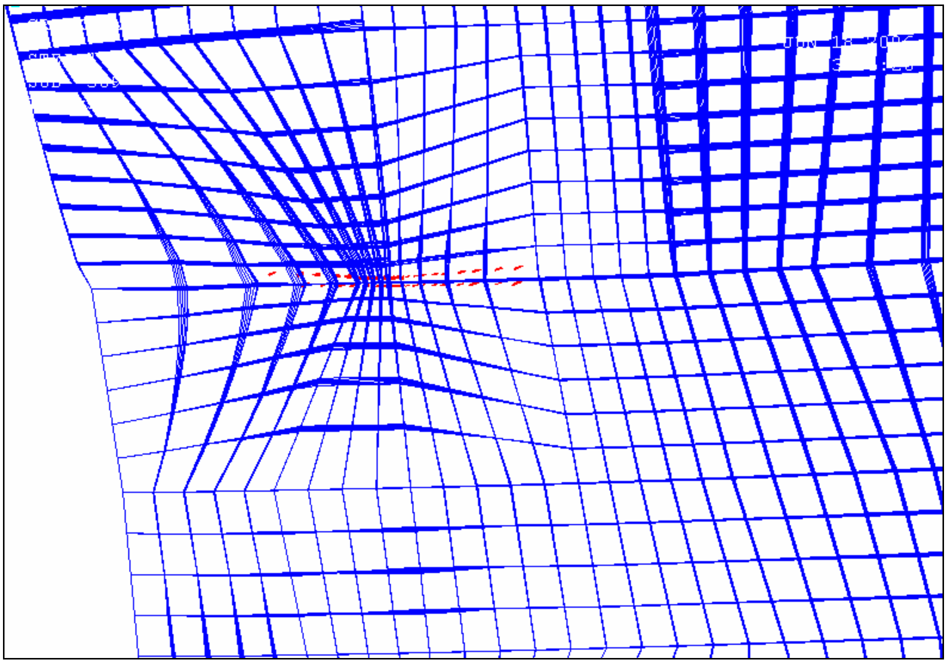

Figure 4.18 Crack pattern at ultimate load for Load Case-1.

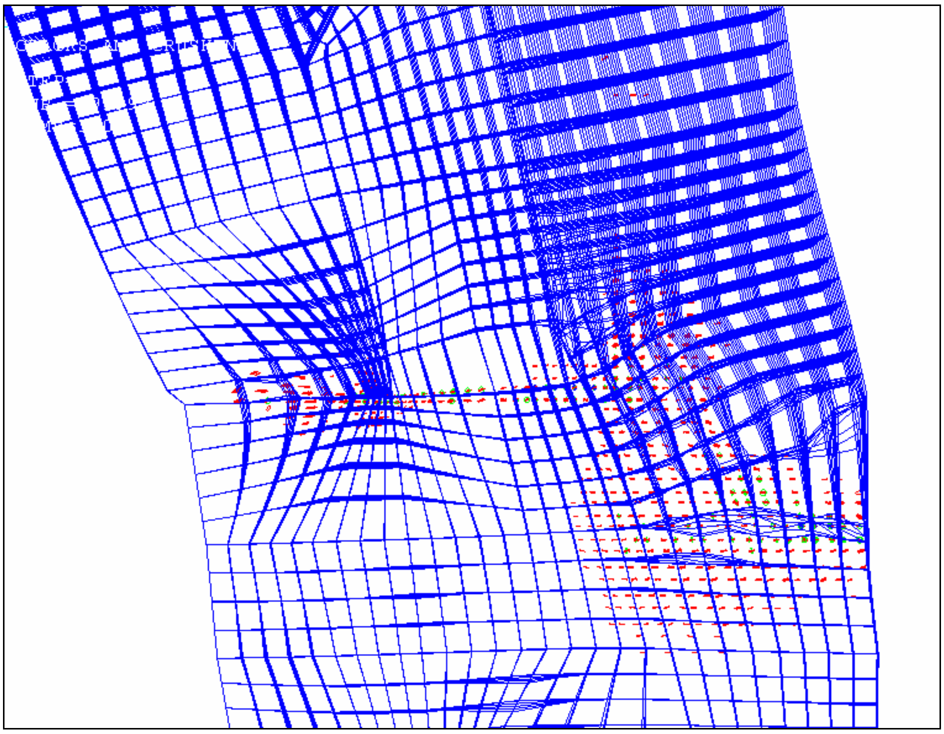

Figure 4.19 Crack pattern for unconverged solution for Load Case-1.

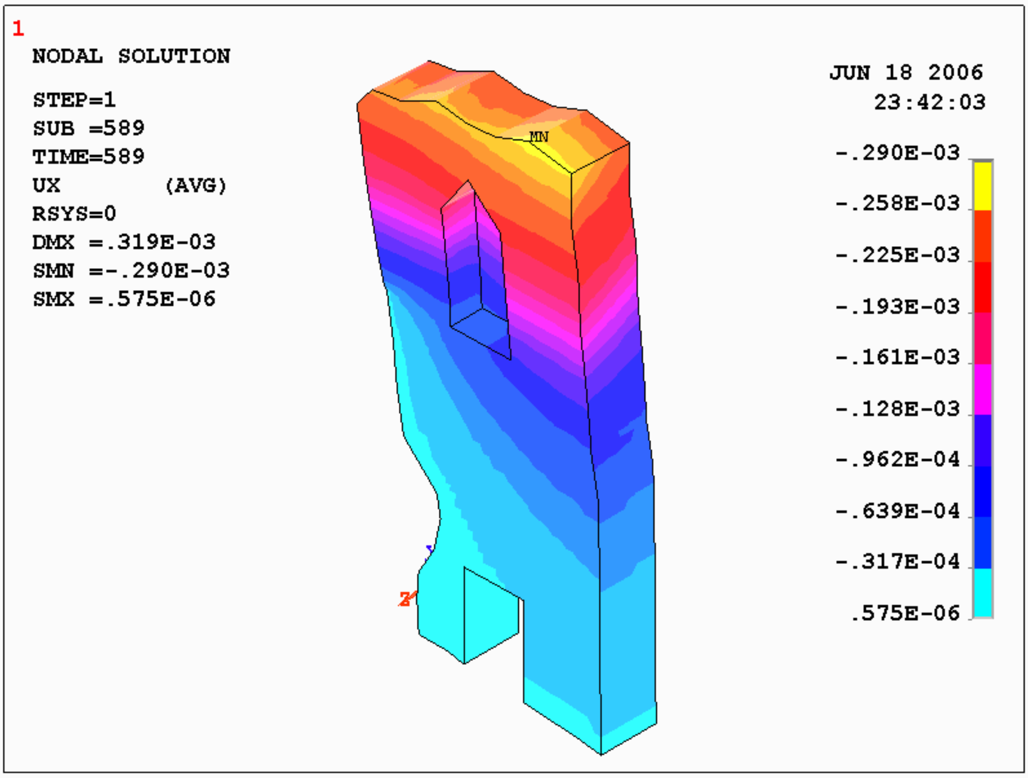

Figure 4.20 X component of displacement at ultimate load for Load Case-1 (non-linear analysis).

Figure 4.21 X component of displacement at ultimate load for Load Case-1 (linear analysis).

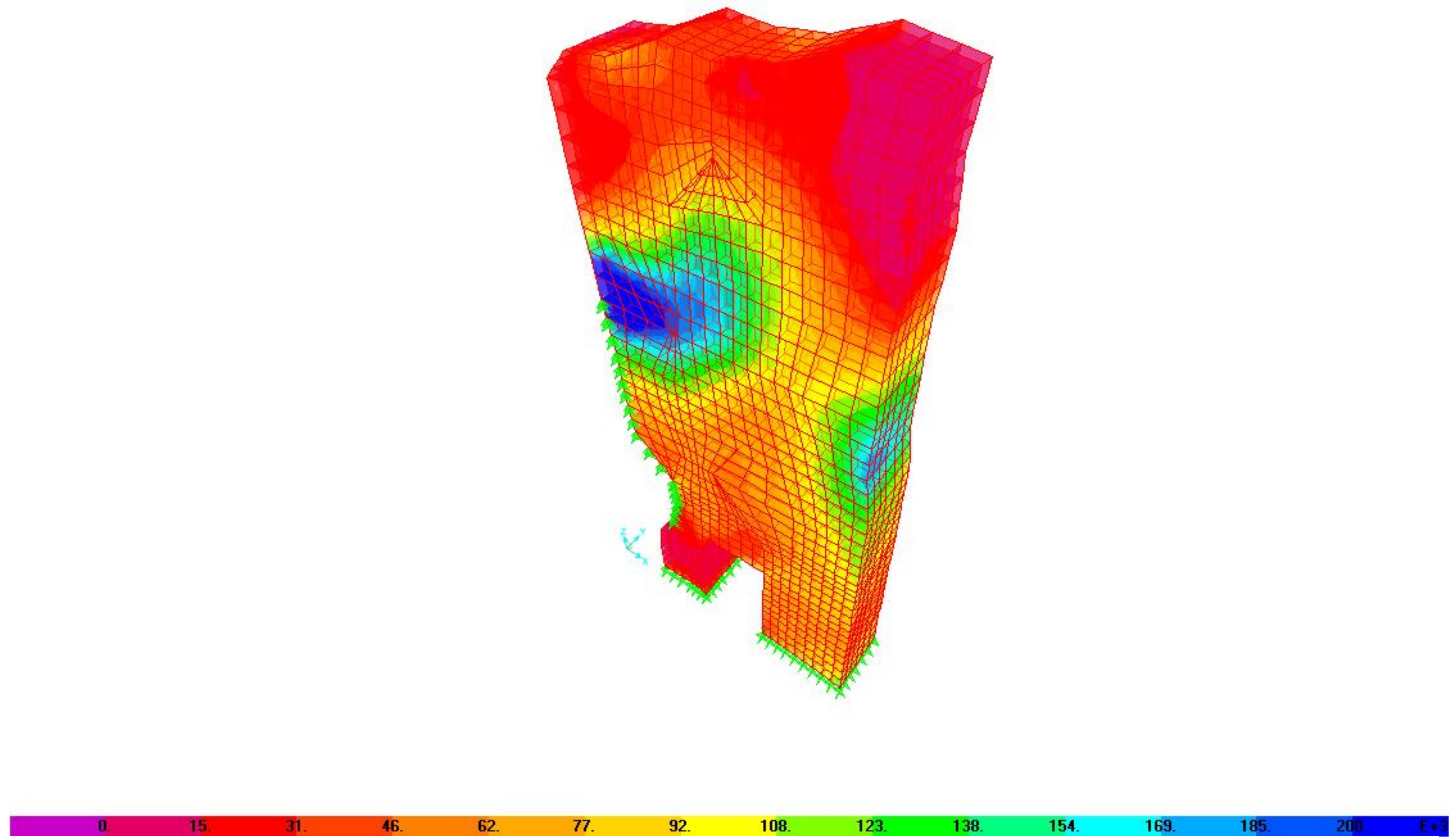

Figure 4.22 The diagram for the positive stress distribution for Load Case-1. The contours are scaled such that the blue contours show the locations where the stress is greater than 0.2 Mpa.

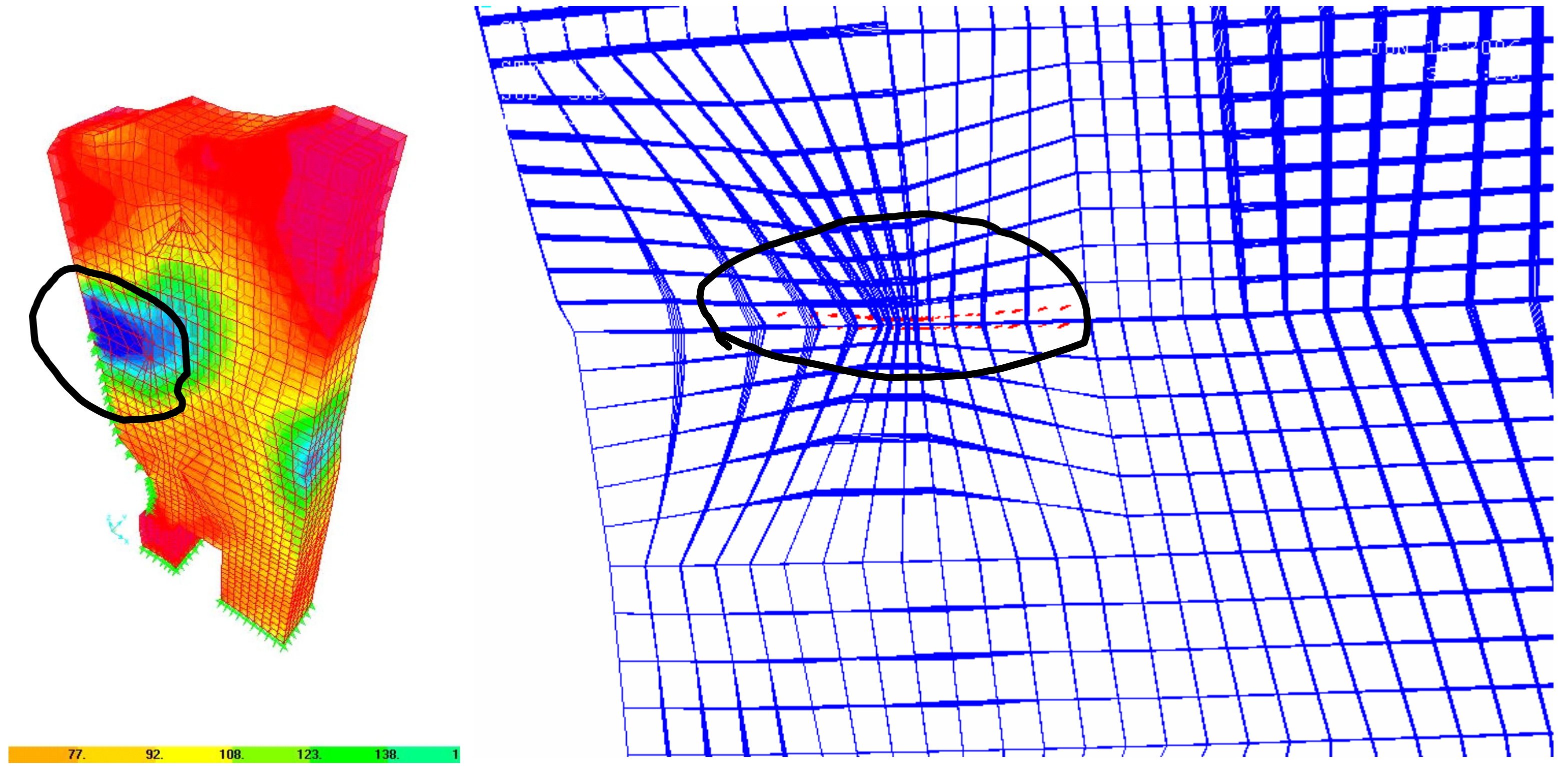

Figure 4.23 The comparison of the crack locations for linear and non-linear analysis for Load Case-1.

4.5.2. Load Case-2 Analysis Results

The analysis results are similar to the Load Case-1. The maximum inertial force is 1.45 m/s^2 and the maximum deformation is 0.118 mm. Figure 4.24 shows the load displacement diagram which implies a brittle failure. Figure 4.25 shows the initial crack pattern and Figure-26 shows the crack pattern for the unconverged solution.

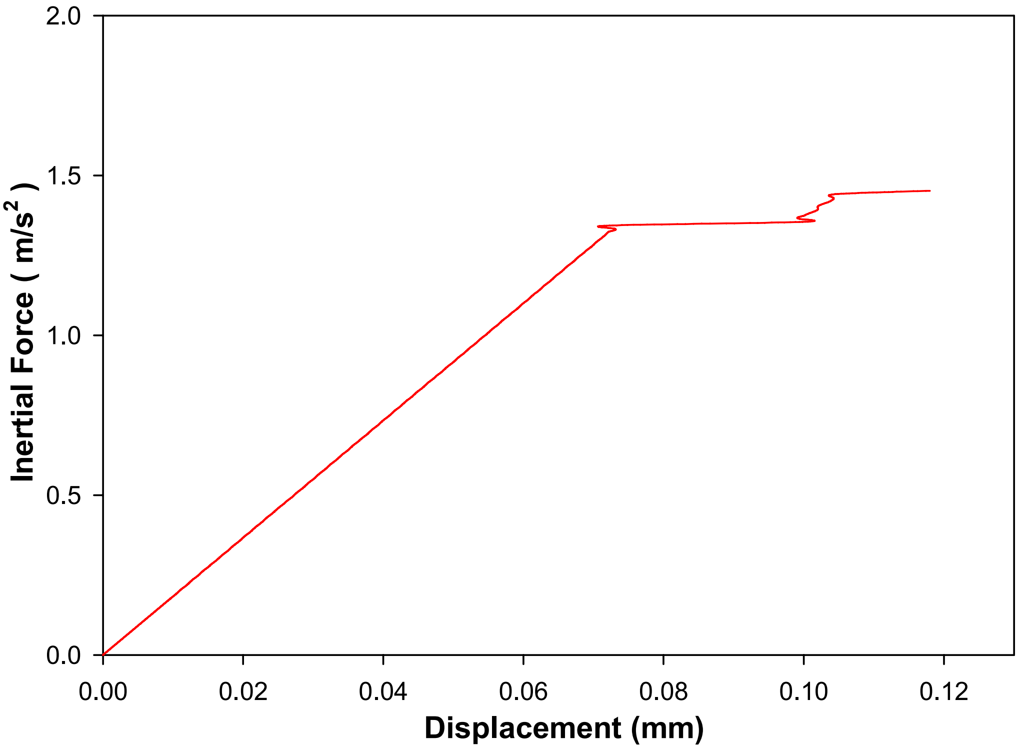

Figure 4.24 Load-displacement diagram for Load-Case-2.

Figure 4.27 shows the in-plane displacement for the non-linear analysis. In Figure 4.28 the same displacements are shown for linear analysis. For non-linear analysis, the maximum displacement is observed as 0.118 mm and for the same load level linear analysis gives a maximum displacement of 0.07 mm. The difference for this analysis case seems to be a result of that linear analysis could not capture the nonlinear deformation caused by the cracks (which yields more deformation). This can be seen from the load deformation graph, the linear behavior is lost when the deformation is about 0.07 mm.

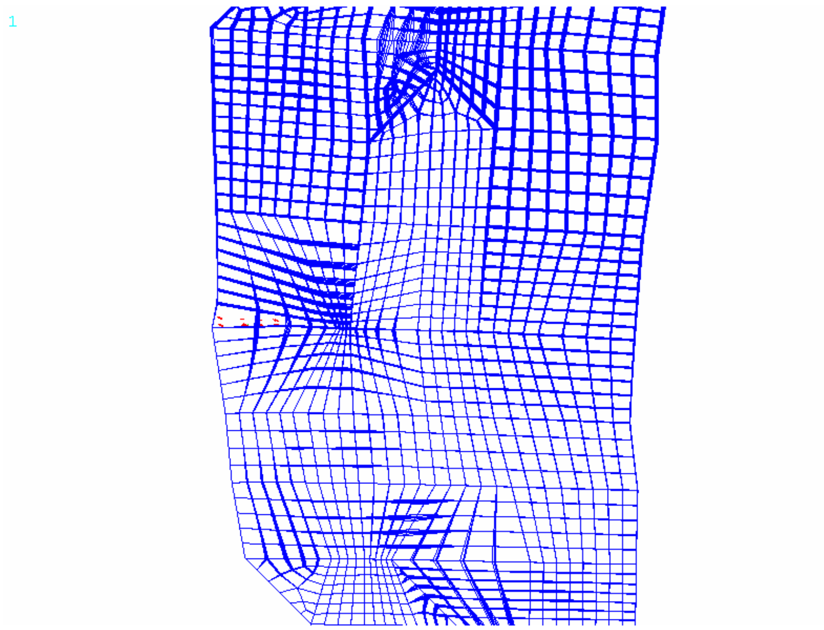

Figure 4.25 Crack pattern at ultimate load for Load Case-2.

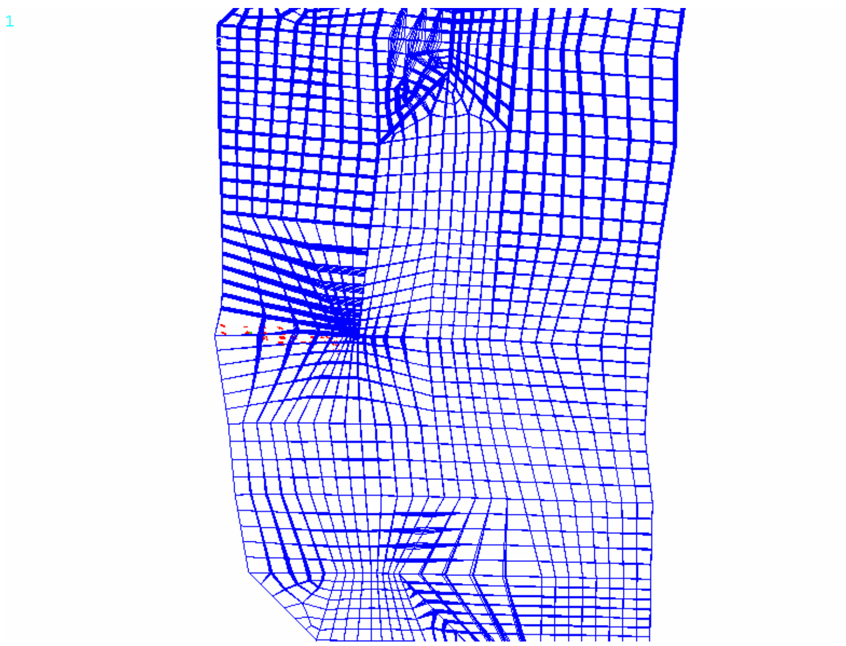

Figure 4.26 Crack pattern for unconverged solution for Load Case-2.

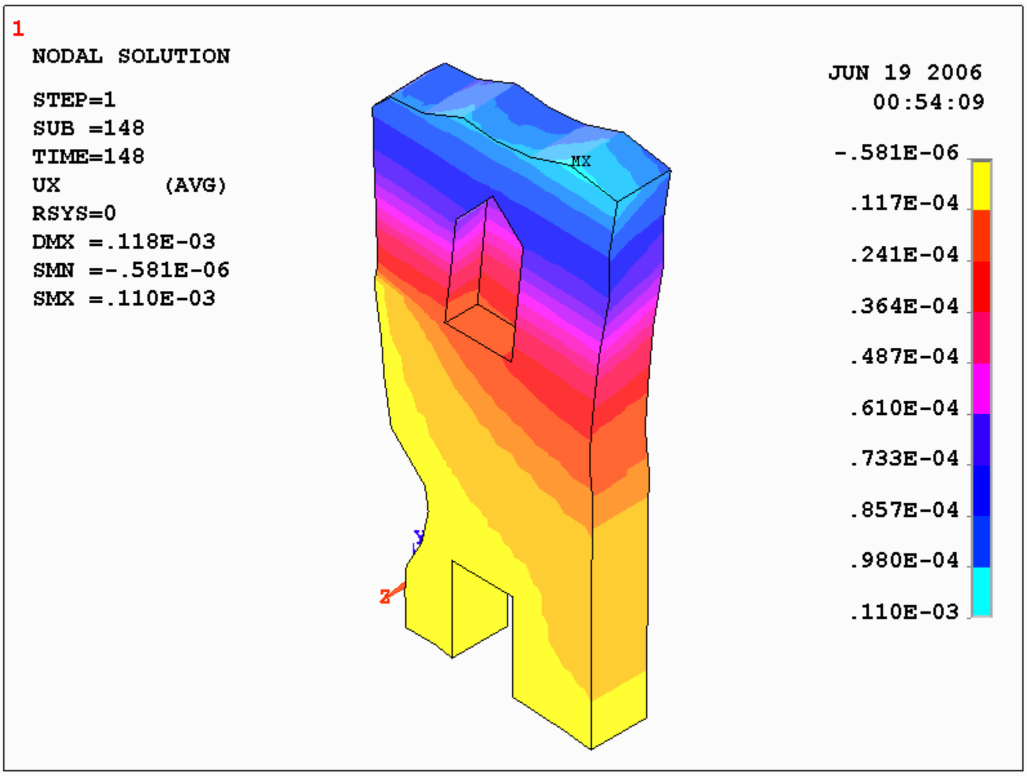

Figure 4.27 X component of displacement at ultimate load for Load Case-2 (non-linear analysis).

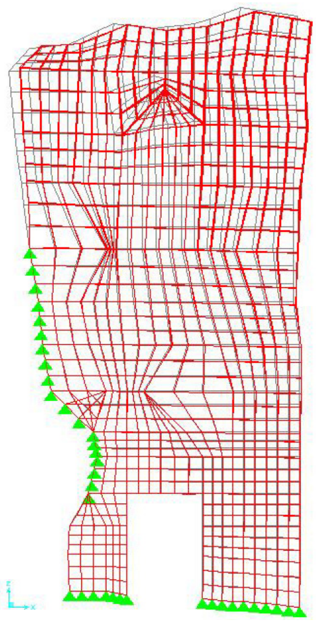

Figure 4.28 X component of displacement at ultimate load for Load Case-2 (linear analysis).

Figure 4.29 shows the linear analysis results for the case where the positive stress is greater than 0.2 Mpa in any direction. When the Figure 4.30 is investigated, it can be observed that the stresses are in harmony with the crack pattern obtained by non-linear analysis.

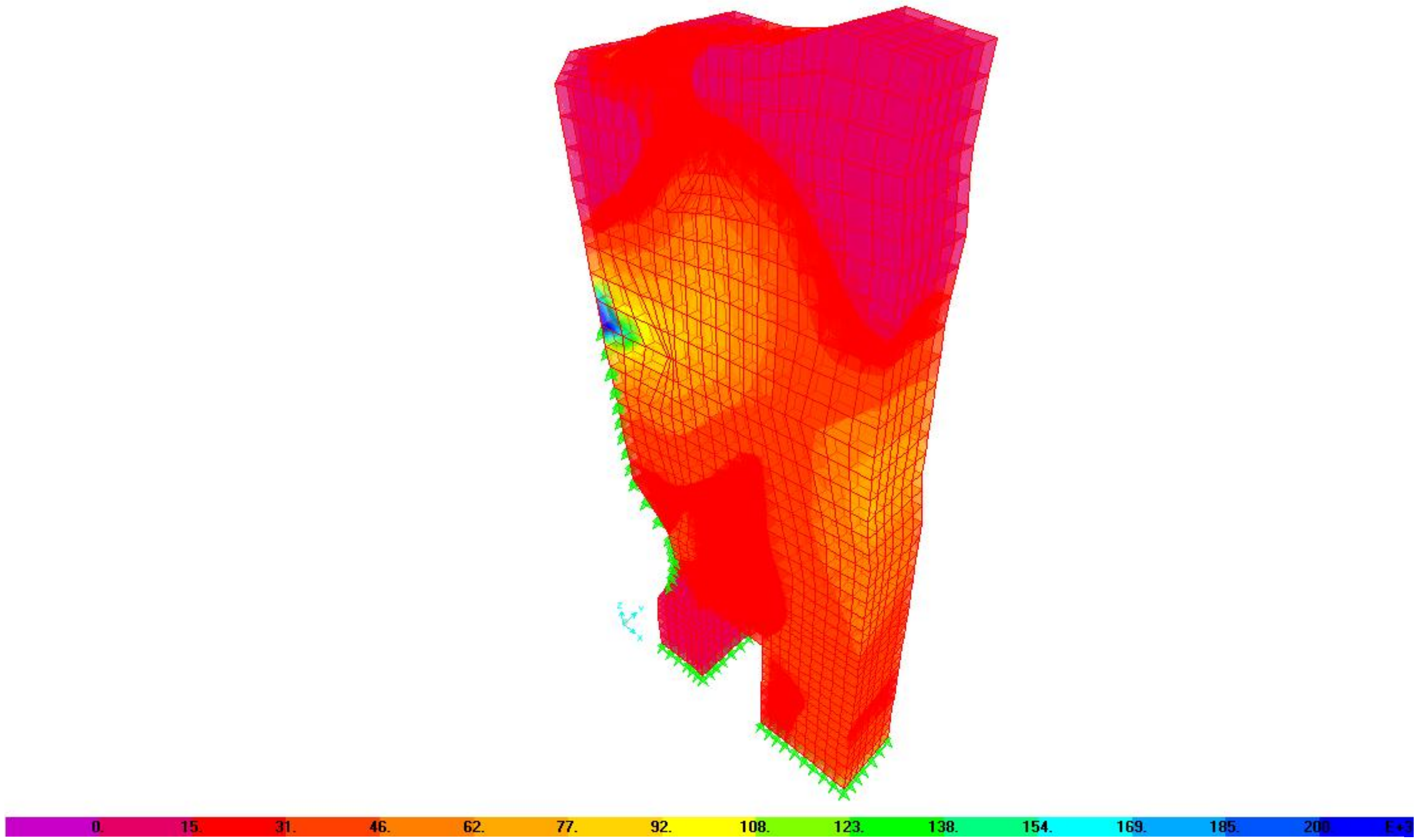

Figure 4.29 The diagram for the positive stress distribution for Load Case-2. The contours are scaled such that the blue contours show the locations where the stress is greater than 0.2 Mpa.

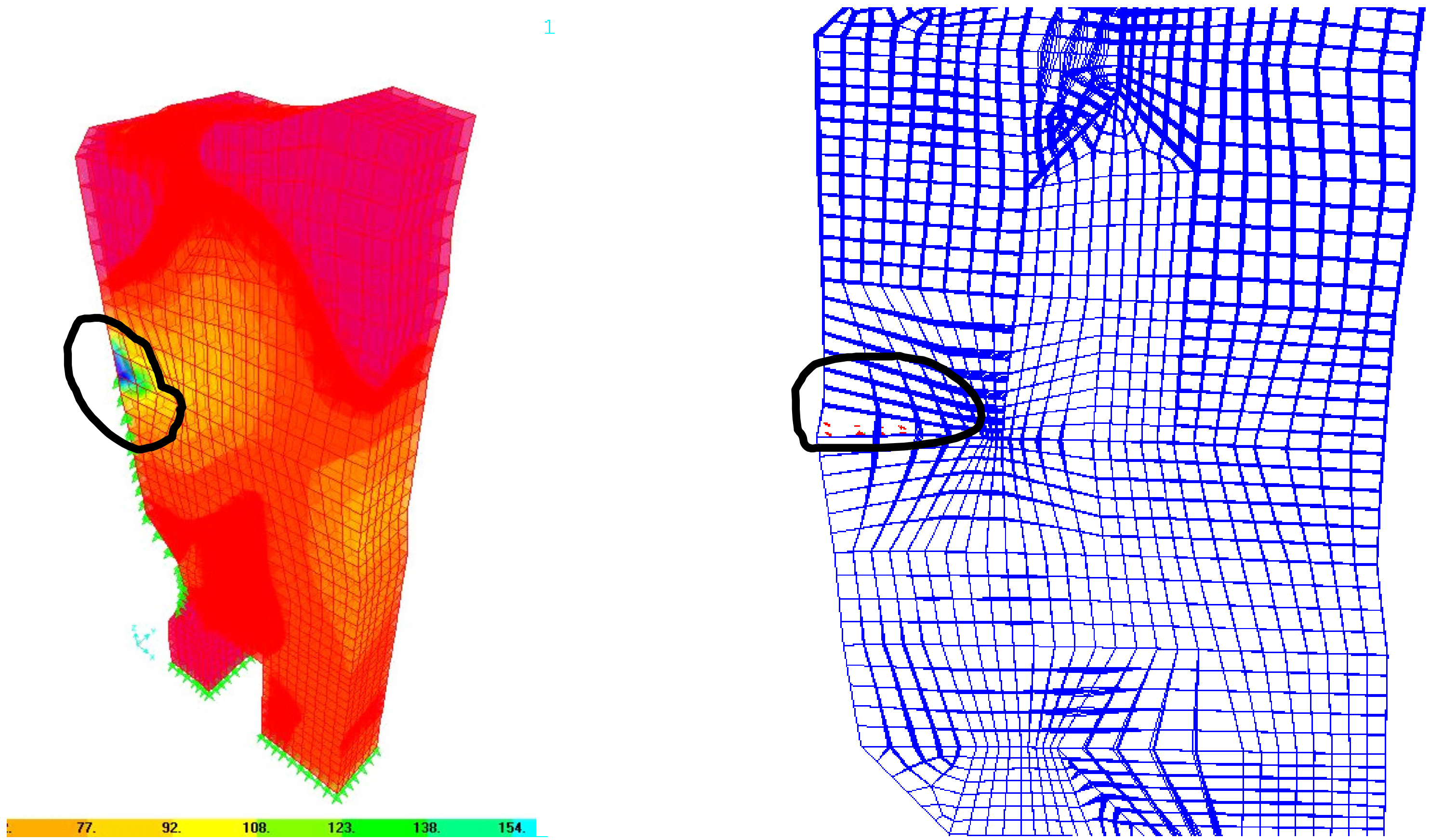

Figure 4.30 The comparison of the crack locations for linear and non-linear analysis for Load Case-2.

4.5.3. Load Case-3 Analysis Results

Load Case-3 is a simulation of the structure for the out of plane behavior. The maximum inertial force is 0.405 m/s^2 and the corresponding displacement is 0.298 mm. Figure 4.31 shows the load deformation plot of the structure. Figure 4.32 shows the initial crack pattern and Figure 4.33 shows the crack pattern for the unconverged solution.

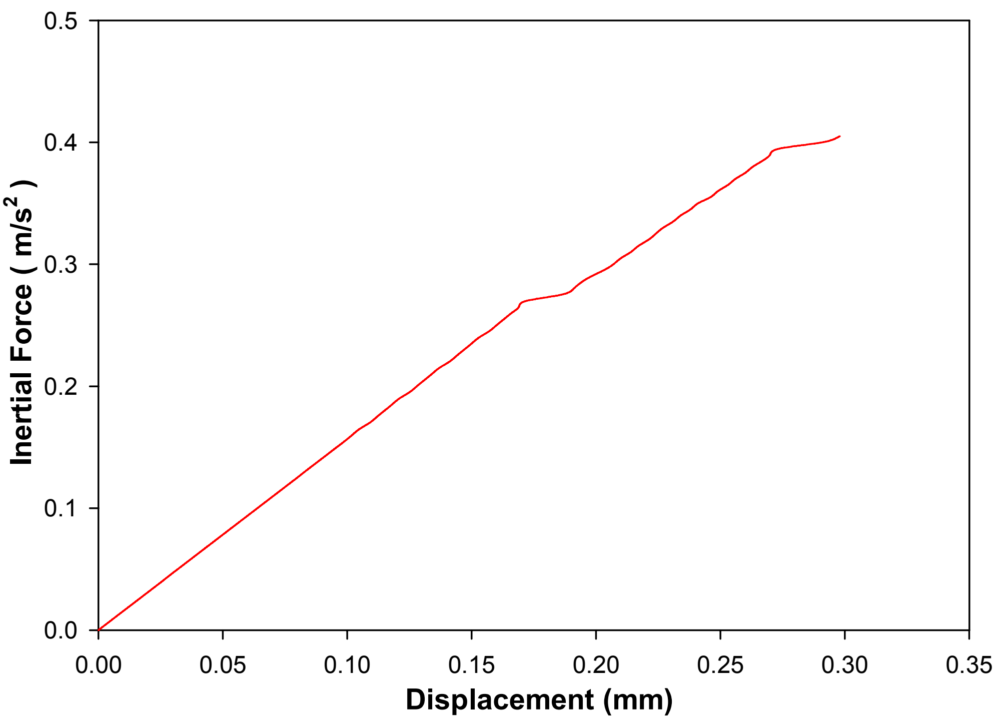

Figure 4.31 Load-displacement diagram for Load Case-3

Figure 4.34 shows the out-of-plane displacement for the non-linear analysis. In Figure 4.35 the same displacements are shown for linear analysis. For non-linear analysis, the maximum displacement is observed as 0.298 mm and for the same load level linear analysis gives a maximum displacement of 0.213 mm.

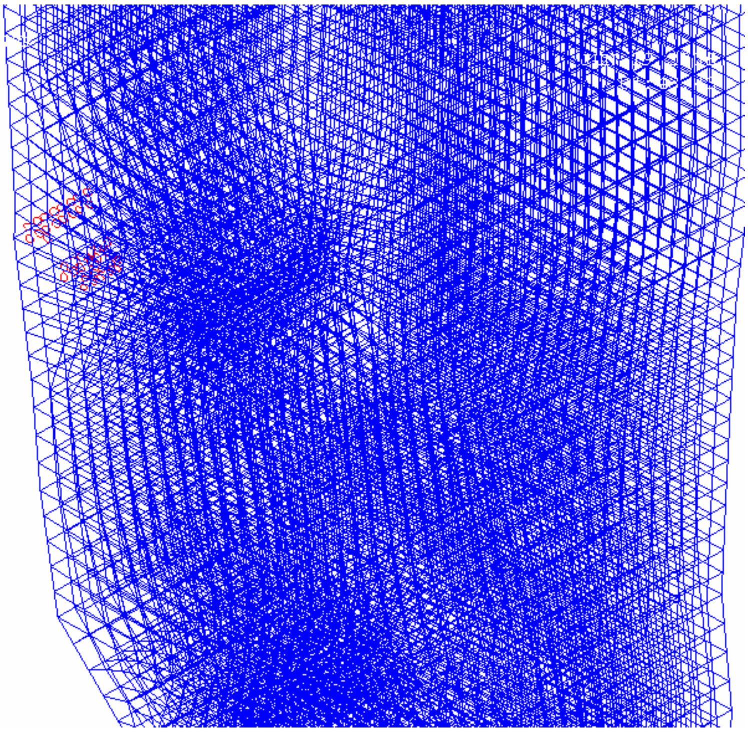

Figure 4.32 Crack pattern at ultimate load for Load Case-3.

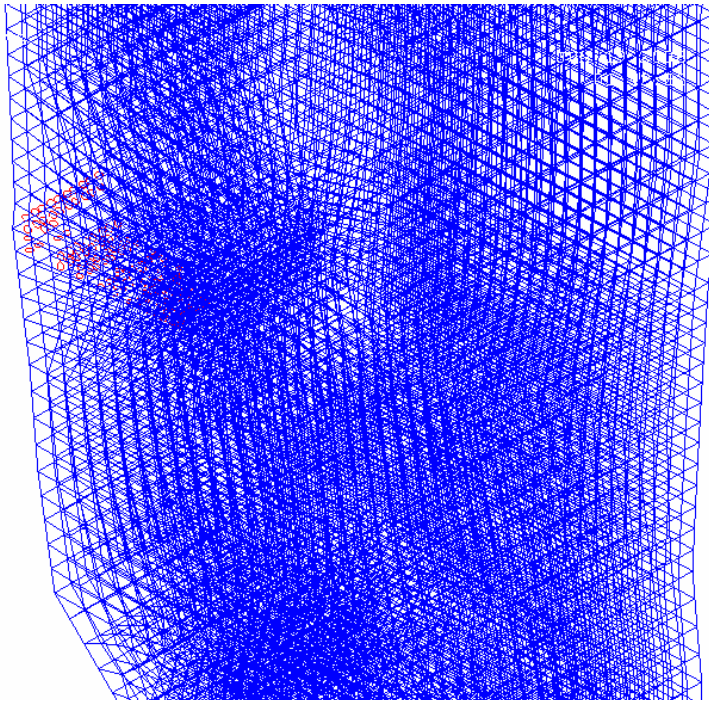

Figure 4.33 Crack pattern for unconverged solution for Load Case-3.

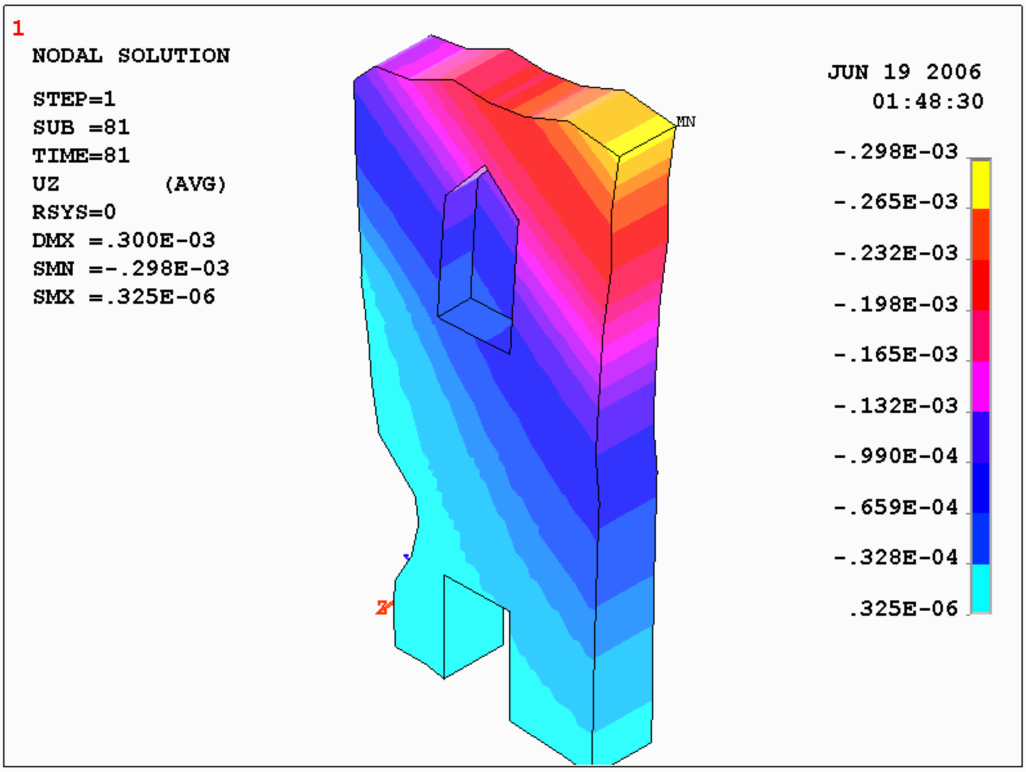

Figure 4.34 Z component of displacement at ultimate load for Load Case-3 (non-linear analysis).

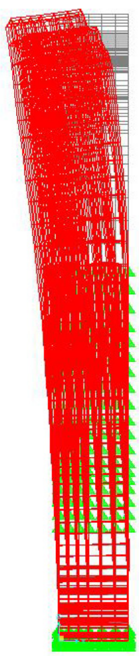

Figure 4.35 Z component of displacement at ultimate load for Load Case-3 (linear analysis).

Figure 4.36 shows the linear analysis results for the case where the positive stress is greater than 0.2 Mpa in any direction. When the Figure 4.37 is investigated, it can be observed that the stresses are in harmony with the crack pattern obtained by non-linear analysis.

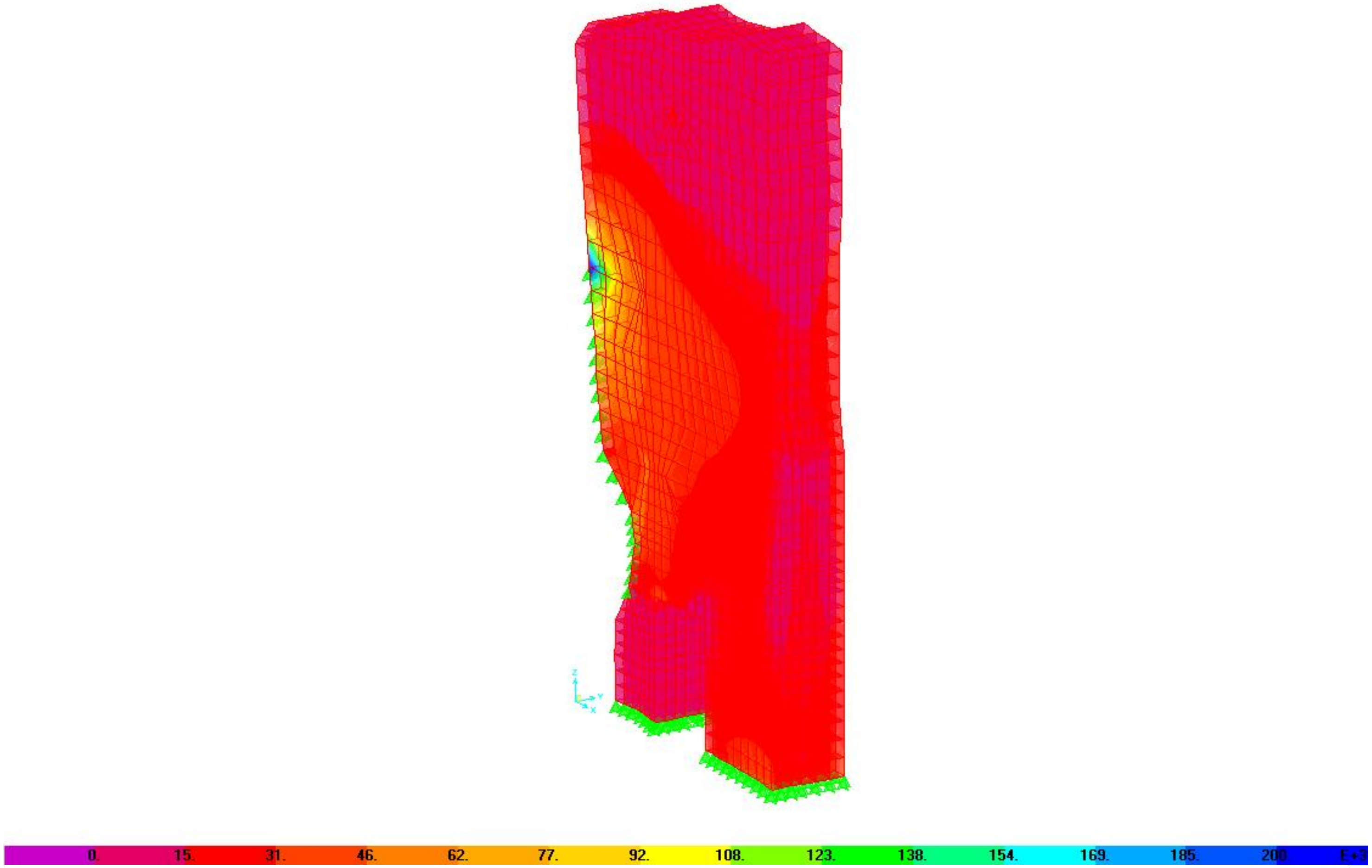

Figure 4.36 The diagram for the positive stress distribution for Load Case-3. The contours are scaled such that the blue contours show the locations where the stress is greater than 0.2 Mpa.

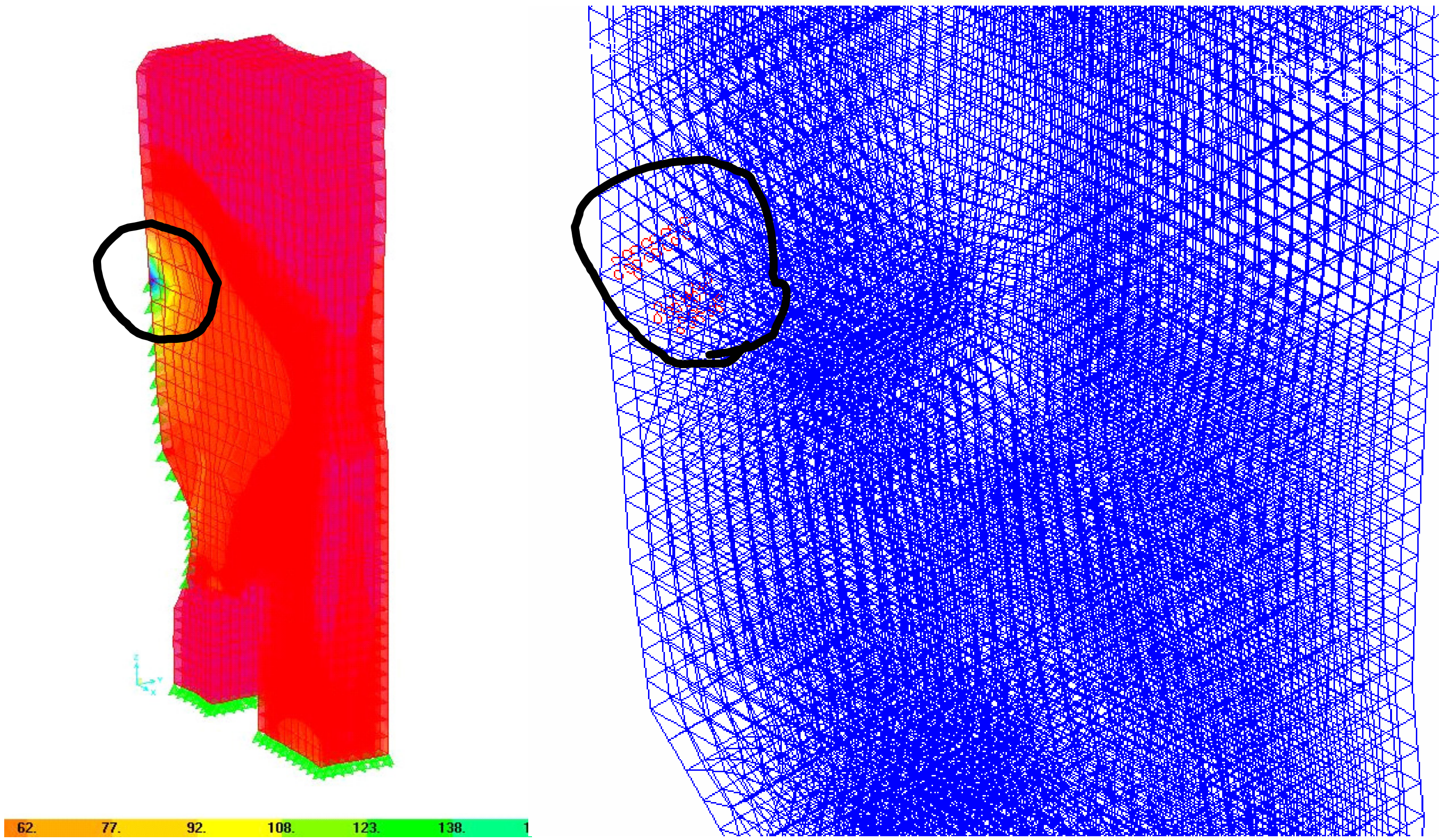

Figure 4.37 The comparison of the crack locations for linear and non-linear analysis for Load Case-3

4.6. Discussion of Analysis Results

Before any analysis attempt, the finite element model is tested by considering the modal deformed shapes and dynamic behavior. The modal deformed shapes are obtained for the first 6 modes by using both ANSYS and SAP2000 software. The related figures show a clear similarity for the modal deformed shapes. The corresponding modal periods are observed to be very close to each other with a maximum percentage error of 2. The dynamic verification of the model is performed by applying an earthquake load to the base of the structure. The maximum acceleration at the base and at the top of the structure are very close to each other, which is another verification of the models created by ANSYS and SAP2000.

After the verification of the model, a lateral load is applied to the structure for 3 different directions. The capacity of the structure (the maximum lateral load that the structure can stand) is determined by performing a non-linear analysis. After determination of the capacity of the structure, the maximum load that the structure can withstand is applied to the same structure for elastic analysis and observations are made.

For Load Case-1, the capacity of the structure is determined as 0.58g, which means that the structure can carry 58% of its self-weight for the given direction. For a masonry structure this lateral force may seem to be excessive, however this is a result of the translational restraints at the left side of the structure that increases the lateral load capacity of the structure. When the maximum lateral displacement is investigated, the value is very small when compared with the lateral force. This behavior is a clear indication of a very brittle failure as can be understood from the crack pattern figures. When the same maximum load is applied to the structure for the linear analysis, the maximum deformation at the maximum load is very close to the non-linear analysis result. The macro model assumes that cracks occur when positive stress in any direction exceeds 0.2MPa in any direction. When the elastic analysis results are compared with the crack patterns obtained by non-linear analysis, it can be observed that the locations and the expected cracks are very similar to non-linear analysis.

The capacity of the structure is determined as 0.15g for Load Case-2. The maximum deformation is determined as 0.118mm for the non-linear analysis. However, at the maximum load the lateral deformation is determined as 0.07mm for the linear analysis. This difference is quite large. The reason for this difference can be that the non-linear behavior could not be captured by elastic analysis, which yielded less deformation. This phenomena can be understood when the load-deformation curve for Load Case-2 is investigated. As can be understood from the figure, the linear behavior is lost when the lateral deformation is about 0.07 mm. The locations of the cracks determined by elastic analysis by considering the stress values are similar to the locations of the cracks obtained by non-linear analysis. The overall behavior of the structure is again very rigid as can be understood from the force-deformation diagram.

Load Case-3 is a representation of a lateral load applied in the out-of-plane direction. The capacity of the structure is expected to be less then the capacity of the structure for the strong direction. The maximum lateral load that the structure can withstand is determined as 0.05g which is a quite small value when compared to the capacity of the structure in the strong direction. However, the structure is more flexible for the out-of-plane direction and although the capacity is less than the capacity for the strong direction, the deformations are expected to be larger than the deformations for the in-plane direction. When the load-deformation graph is investigated, this expected behavior can be observed, the maximum deformation is 0.298mm. When the figure showing the stress distribution where the positive stress is greater than 0.2MPa is investigated, the locations of the cracks are very similar to the locations of the cracks obtained by non-linear analysis. The summary of the results are given in Table-4.2.

Table 4.2 Summary of the analysis results

LOAD CASE	Capacity	Max. Disp. (non-linear)	Max. Disp. (linear)
1	0.58g	0.29mm	0.250mm
2	0.15g	0.118mm	0.070mm
3	0.05g	0.298mm	0.213mm

CHAPTER-5

CONCLUDING REMARKS

5.1. Summary

The aim of the study is to investigate the linear and non-linear behavior of masonry structures. For this reason, a masonry gate located in the Historical City of Hasankeyf was chosen as the case study structure. The dimensions of the structure were favorable for the investigation of the in-plane and out-of-plane behavior of the structure separately. Two different finite element models were created by using two different software (ANSYS and SAP2000) and analysis were performed for different load cases.

The models were first verified by comparing the modal deformed shapes and modal periods by using ANSYS and SAP2000. The modal deformed shapes were almost the same and the largest percentage error for the model periods was obtained as 2%. The model was also verified by applying a time history function from the base and the top and ground accelerations were nearly the same which implies a very rigid behavior for the given restraints. By performing a non-linear analysis for the 3 different loading conditions to the structure, the load capacity, maximum deformation and crack patterns are obtained. When the same loads are applied to the model that is created for a linear analysis it was observed that the maximum deformations and the susceptible parts of the structure could be easily determined without a non-linear analysis.

5.2. Conclusions

It is very difficult to determine the strength and stiffness characteristics of masonry, since it is composed of two different materials (blocks and mortar). Because of this reason, conventional analysis methods are not adequate to fully understand the behavior of masonry. Finite Element Method overcome the difficulties for analyzing masonry structures. However, analysis of masonry by the Finite Element Method is also a quite difficult task, requiring very detailed application and supervision.

When the force deformation plots obtained for different lateral load cases are concerned, it can be concluded that masonry is very weak under tension and very strong under compression. Because of its rigid and highly nonlinear behavior, combining with extreme mass, earthquakes are the most dangerous threat masonry structures. However, determination of earthquake load that a masonry structure demands is very difficult because of the uncertain material and stiffness properties of masonry. Also the design spectrums that are used for determination of earthquake loads are derived for modern buildings. The application of these kinds of spectra to historical masonry structures requires particular attention since the life-time of historical masonry structures is quite long when compared to that of modern buildings.

The best method for the analysis of historical masonry buildings is nonlinear finite element analysis method. However, any nonlinear analysis method requires determination of material properties that was discussed to be very difficult, time consuming and expensive. Also the nonlinear analysis itself requires quite a long time.

Unless a very detailed analysis for a local part of the structure is required, it is seen that a linear analysis is adequate to analyze the structure (For the same model the time required for the non-linear analysis was 3000 times greater than the time required for the linear analysis). As far as the analysis results that are performed for different load cases concerned, it can be observed that the maximum deformations at the ultimate loads and the locations of the cracks are nearly the same. Because of this reason, as an alternative to the nonlinear analysis methods, linear analysis methods can be adequate when the whole structure is under investigation.

5.3. Recommendations for Further Studies

The most uncertain point of application of linear analysis for the determination of earthquake effect is the maximum load that a historical masonry can withstand. As can be observed from the study, the comparison of linear and nonlinear behavior was performed after the determination of the ultimate earthquake load by a nonlinear analysis. For the use of linear analysis, the maximum earthquake load that a historical masonry structure demands must be determined. As a future study, the adequacy of earthquake design spectrums can be inspected for different masonry structural systems. Also as a second alternative specific earthquake spectrums for historical centers can be developed. This can help the researchers to determine the earthquake load capacity of different kind of historical masonry structures.

REFERENCES

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

2. Şener, İ.,N., 2004, *An innovative Methodology and Structural Analysis for Relocation of Historical Masonry Monuments: A Case Study in Hasankeyf*, Ms. Dissertation, METU, Ankara

3. *Standards for Historic Preservation and Guidelines for Rehabilitating Historic Buildings*, Moncton Heritage Preservation Review Board

4. Crogi, G., 1998, *The Conservation and Structural Restoration of Architectural Heritage*, Computational Mechanics Publications, Southampton

5. Azevedo, J., Sincraian, G., 2001, *Modeling the Seismic Behavior of Monumental Masonry Structures*, Proceedings of Archii 2000, Bethelem, Israel, September 10-12

6. Lourenço, P.B., 1998, *Experimental and Numerical Issues in the Modelling of the Mechanical Behavior of Masonry*, Structural Analysis of Historical Constructions II, CIMNE, Barcelona

7. Oliveira, D.V.C., 2003, *Experimental and Numerical Analysis of Blocky Masonry Structures Under Cyclic Loading*, Ph.D. Dissertation, University of Minho, Portugal

8. Lourenço, P.B., 1996, *Computational Strategies for Masonry Structures*, Ph.D. Dissertation, Delft University of Technology, Delft, Netherlands

9. Robert, C.M., John, P.S., 2004, *Repointing Mortar Joints in Historic Masonry Buildings*, Preservation Briefs, National Park Service

10. Hendry, A.W., 2001, *Masonry Walls: Materials and Construction*, Construction and Building Materials, 15; 323-330, Elsevier

11. Oliveira, D.V.C., 2000, *Mechanical Characterization of Stone and Brick Masonry*, Report 00-DEC/E-4, University of Minho, Department of Civil Engineering, Portugal

12. . Jordan, J.W., Brookes, C.L., 2004i *The Conservation and Strengthening of Masonry Structures,* The 4th International Conference on Masonry Arches, Barcelona

13. Ünay, A.İ., 2002, *Tarihi Yapıların Depreme Dayanımı*, O.D.T.Ü. Mimarlık Fakültesi Bölüm İşliği, Ankara

14. Corrodi, M. et al., 2003*, Experimental Study on the Determination of Strength of Masonry Walls*, Construction and Building Materials, 17; 325-337, Elsevier

15. Sofronie, R.A., Popa, G., Nappi, A., 2004, *Strengthening and Restoration of Eastern Churches* (http://www.unesco.org/archi2000/)

16. Sanchez, C.S., Cheng, C., 2005, *Adobe Bricks in Combination with Passive Design Strategies- Experience of Green Materials from Costa Rica*, Japan-Taiwan International Workshop on Urban Regeneration 2005, Maintenance and Green Material, Taipei, Taiwan

17. Vargas, J., Bariola, M., Blondet, M., Mehta, P.K., 1984, *Seismic Strength of Masonry*, U.S. Agency for International Development (AID) Research Project DI-84-01

18. Lourenço, P.B., 2002, *Computations on Historic Masonry Structures*, Prog. Struct. Engineering Materials, 2002, 4: 301-319, John Wiley&Sons Ltd.

19. Brancich, A., Corradi, C., Cambarotta, L., Montegazza, G., Sterpi, E*., 2005, Compressive Strength of Masonry Under Eccentric Loading,* Materials and Structures Journal, 38: 257-266, Rilem Publications SARL

20. Hansen, Z., L., 2003, *Stability of Masonry Columns*, Rapport BYG.DTU R-055, Danmarks Tekniske Universitet

21. Dialer, P., C., 2002, *Typical Masonry Failures and Repairs: A German Engineer's View*, Prog. Struct. Engng Mater., 2002; 4:332-339, John Wiley&Sons Ltd.

22. FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*

23. Italian Standard D.M. 20.11.1987

24. Lourenço, P.B., 2001, *Analysis of Historical Constructions: From Thrust Lines to Advanced Simulations*, Historical Constructions, Eds. P.B. Lourenço and P. Roca, Universidade do Minho, Guimarães, p. 91-116 (2001)

25. Syrmakesiz, C., A., 2005, *Seismic Protection of Historical Structures and Monuments*, Structural Control and Health Monitoring 10.1002/stc.89, John Wiley&Sons

26. Mele, E., De Luca, A., Giordano, A., 2003, *Modelling and Analysis of a Basilica under Earthquake Loading*, Journal of Cultural Heritage 4 (2003) 355-367, Elsevier

27. Masia, M.,J., Kleeman, P.,W., Melchers, R.E., 2004, *Modeling of Soil/Structure Interaction for Masonry Structures*, Journal of Structural Engineering 130:4(641), ASCE

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

29. Lourenço, P., B., 2002, *Guidelines for the analysis of Historical Masonry Structures*, Finite Elements in Engineering and Science, Eds. M.A.N. Hendriks et al, A.A. Balkema, Rotterdam, The Netherlands, p. 241-247

30. Page, A., W., 1982, An Experimental Investigation of the Biaxial Strength of Brick Masonry, Proceedings of the Sixth International Brick Masonry Conference, 3-15, Andil, Rome

31. Ayala, D., D., Speranza, E., 2003, *Definition of Collapse Mechanisms and Seismic Vulnerability of Historic Masonry Buildings*, Earthquake Spectra, 3: 479-509, Earthquake Engineering Research Institute

32. Decanini, L., Sortis, A., D., Goretti, A., Langenbach, R., Mollaioli, F., Rasulo, A., 2004, *Performance of Masonry Buildings During the 2002 Molise, Italy, Earthquake*, Earthqauke Spectra, 20:191-220, Earthquake Engineering Research Institude

33. Cook, D., R., Malkus, S., D., Plesha, M., E., 1989, Concepts and Applications of Finite Element Analysis, Wiley, New York

34. CSI Analysis Reference for SAP2000, 2005, Computers and Structures, Inc. Berkeley, California

35. ANSYS10 Reference Manual

36. Pictures of the Wall of Jericho, Retrieved, May 25, 2006, http ://www. biblia.com/jesusbible/joshua4.html

37. A view of Parthenon, Retrieved, Jun 02, 2006, from http://www. Students .sbc.edu/mdavis04/Parthenon-02.jpg

38. The picture of the Pont du Gard aqueduct, Retrieved, May 25, 2006, from http:// khleweke.de/ LanguedocRousillion% 202003/imagepages/image93.html

39. Hagia Sophia, Retrieved, Jun 02, 2006, from http:// www .patriarchate.org/ ecumenical_patriarchate/chapter_1/ ecclesiastical_ ministry.html

40. Selimiye Mosque, Retrieved, May 17, 2006 from http://www. Ucalgary .ca/ applied_history/tutor/islam/learning/architecture.html

41. Front view of Amiens Cathedral, Retrieved, July 04, 2006 from http ://www. bc.edu/bc_org/ avp/cas/fnart/arch /gothic/amiens1.jpg