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TESI DI LAUREA

in

MECHANICS OF HISTORICAL MASONRY STRUCTURES

EXPERIMENTAL AND COMPUTATIONAL APPROACHES TO HISTORICAL MASONRY STRUCTURES: A STUDY IN THE DIRECTION OF FILLING THE GAP

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Sessione I

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Chapter 1

Introduction

1.1 Motivation and study needs

The assessment of historical structures is a significant need for the next generations, as historical monuments represent the community's identity and have an important cultural value to society. Most of historical structures built by using masonry which is one of the oldest and most common construction materials used in the building sector since the ancient time. Also it is considered a complex material, as it is a composition of brick units and mortar, which affects the structural performance of the building by having different mechanical behaviour with respect to different geometry and qualities given by the components.

The most advanced national and international guidelines for assessment and risk reduction to cultural heritage constructions (Min. BB CC, 2011), (CIB, 2010) and (ICOMOS, 2002) stress the fundamental role of a knowledge path about the building consideration and proposing a methodology of structural assessment, including the aspects of data acquisition, structural and architectural investigations, field research and laboratory testing, and advanced numerical analysis, in order to have a proper justification of restorations measures, besides a full detailing of the adopted strengthening techniques.

The structural assessment has become important as a result of the need to improve existing buildings for new conditions of use. The main aspects of historical structures assessment go through the structural investigations of the building which are subjected to processes of degradation with time, leading a situation in that they become not able to fulfil the purpose for which they were built. Preliminary studies should be conducted to obtain information about the current state of the structure including the geometry, morphology, structural details, material properties, prior interventions, and existing damage. This process creates a conceptual background and indication of the construction methods that have been used for constructing the building. The understanding of the structural behaviour and material characteristics is important for any structural assessment related to historical masonry building. The diagnosis of the structure is based on qualitative and quantitative approaches, where the qualitative approach is related to the observation of structural damage and material decay, while the quantitative approach requires a series of experimental in situ and laboratory tests performed to obtain the in-situ strength of masonry materials, mechanical properties of masonry elements, and in some study cases the dynamic response of the building. Additional results could be obtained by using monitoring techniques for measuring displacements, crack's opening, settlements, internal forces, and humidity, etc..,.

The need for structural assessment of historical structures is motivated by many factors, such as: the existence of visible defects in the structure due to a particular event that affects its stability, the change of use of the building. The basic point in considering structural assessment is to establish the performance level to be fulfilled, basically the requirements of structural safety. Through the assessment of historical structures, there are challenges in diagnosis, analysis, monitoring and strengthening stages. That limits the application of structural assessment with respect to the used codes and norms having different approaches to the assessment. Clear difficulties are associated with the practical application of assessing existing masonry buildings, due to the enormous variability of structural shapes and materials that have been used, due to missing the construction drawings and structural design, also because of that the structures were built in absence of design regulations.

The definition of material properties, structural analysis, and modelling is an important branch of the structural assessment, where the mechanical characteristics of structural elements are the basic input parameters for performing either analytical or numerical structural analysis. FEM seems to be the most suitable approach for conducting the analysis, due to simulating the actual behaviour of masonry elements, which have a high degree of uncertainty in the structural behaviour especially for nonlinear analysis, so elastic approach for analysis is realistic for historical masonry structures. The obtained information is significantly important for redesign procedure for a historical structure since it gives the opportunity to investigate several scenarios with different strengthening decisions.

Hence, in order to develop an appropriate numerical modelling of historical masonry structures, different parameters are needed, which can be obtained by the quantitative approach of structural diagnosis, either by in-situ measurements on the existing structure or by laboratory tests. The obtained data can be used as input parameters for the FEM tools to have the full mechanical characterization of the masonry building. Nonetheless, defining the mechanical properties of masonry is a wide task, and these can change by time and position with respect of the same structure. The assessment of historical structures should control the whole process of investigations and calculations.

FEM is usually adopted to achieve the simulations of historical masonry structures as it is capable to predict the structural behaviour from the linear elastic phase, through cracking and softening until the complete failure. The cooperation between experimentalists and analysts has suffered in the last years due to the slow evolution of the experimental techniques compared with the development of numerical methods. This reflects negatively on the level of accuracy with respect to the obtained results. Indeed, the results obtained from the experimental tests are used as input parameters in numerical models, in which structural analyses and studies of mechanical behaviour are conducted at a given state of the structure. Nevertheless, the gap between the basic two stages of the assessment of any historical masonry structure; experimental testing and computational analysis, becomes larger due to the lack of common grounds between the two parties, which cause a misunderstanding in the conceptual principles of processing the results.

1.2 Scope of the study

Historical masonry structures are one of the most valuable cultural assets of mankind, but over the time they suffer from damage due to material degradation, differential settlements, seismic actions and other environmental effects. These damages result in a variety of forms of structural performance decrease that can cause instabilities, failure and collapse (Teomete and Aktaş, 2010). Thus, historical masonry structures need to be periodically assessed so that risk of loss can be reduced and proper maintenance or strengthening action can be taken, if needed. Nonetheless, assessment of historic masonry still remains a complex and difficult task, especially given that in today’s engineers education main attention is focused on more modern construction materials and advanced ones. Historical structures have many challenges in case of diagnosis, analysis and rehabilitation, which limits the application of structural assessment with respect to modern codes and building standards. Taking in consideration the gap and lack of communication between analysts and experimentalists, by combining experimental and numerical basis, in order to extend and improve existing structural assessment approaches.

The structural assessment requires a survey of the structure and an understanding of its historical process of construction and present state. The basic methodology of the structural assessment moves through data acquisition and diagnosis on both qualitative and quantitative approaches of the structural behaviour. If one of these stages is performed incorrectly, the final decisions will cause a poor results. Thus, the general scope of this study is to stress the role of engineering in the assessment of historical structures in both experimental and numerical evaluations. Where challenges come out because of the complexity of structures geometry, the variability of material properties, and the difference in construction techniques.

A complete mechanical characterization of masonry element is needed for performing an accurate numerical modelling, which is considered as fundamental phase of the historical structures assessment (Lourenço et al., 1998). This means all used experimental techniques play an important role in the linear and nonlinear analysis. In the case of masonry structures, the problem is somehow more complicated due to the uncertainty in the structural behaviour of masonry.

This study summarises the most recent and used experiential and computational approaches of historical masonry structures assessment. The main scope of this study is to activate and strengthen the interaction between experimental techniques and numerical modelling. Which can be done by:

- To undertake a literature review for the historical structures assessment, with respect to the experimental techniques and numerical approaches.
- To provide a catalogue of the available experimental techniques for masonry structures mechanical characterisation, by dividing it into different categories, depending on the tested material, specimens sampling, and the place of the test.
- To apply in different historical masonry structures and analyse, in situ and laboratory, test possibilities and constraints verifying the obtained results also in view of use for modelling. This part of the work aims to show what can be achieved by using different mechanical characteristics of the masonry elements implemented in the analysis.

- To discuss the theoretical basis of the FEM softwares, with respect to the element libraries, material models, and input parameters.
- To investigate the needed input parameters for different FEM softwares, in case of linear and nonlinear analysis.
- To verify the modelling outcome of structural behaviour by comparing different FEM softwares on a masonry element case, using linear and nonlinear analysis.

These objectives are intended to be useful for solving the problems of historical structures assessment and simplify the assessments approaches. A key message is organizing the contribution between the field-work and modelling engineers, who are involved in historical structures assessment.

1.3 Phases of the study

This study presents the basic stages of masonry structural assessment, going through the most used experimental and computational approaches to analyse the historical structures, and it is organized into 9 chapters as follow:

- **Chapter I:** it contains a general overview of the research and its needs beside the scope of the thesis.
- **Chapter II:** it is an introduction to historical masonry structures, introducing the development of the construction techniques and presenting the mechanical behaviour of masonry structures. Also it explains some fundamental aspects concerning masonry as structural material and its mechanical behaviour. A short review of the most frequent actions that could affect masonry structures is presented together with different types of failures.
- **Chapter III:** it is a literature review of masonry structural assessment. It explains the importance and the main procedures followed in the assessment.
- **Chapter IV:** it introduces the most used experimental techniques for masonry structures. Starting from laboratory and in situ techniques aiming to create a general reference of the experimental techniques, dividing it into different categories, depending on the tested material, specimens sampling, the place of the test, and the degree of destruction of

masonry element. A general description is included for each test, according to different international codes, such as ASTM, RILEM, and BS, complemented with the limitations and discussion of the expected results for each technique.

- **Chapter V:** is a literature review of numerical modelling methods, and the implementation for masonry structures, an overview of possible approaches for the numerical modelling of masonry structures is presented. Which includes micro modelling of the units and mortar joints, as well as modelling of masonry as macro element, then going through linear, nonlinear, static, and dynamic approaches.
- **Chapter VI:** it discusses the difference between commercial FEM softwares, with respect to the fundamentals and details proposed by the software for modelling masonry structures, beside the theoretical basis of the softwares, element libraries, material models, and input parameters needed to perform a linear and nonlinear analysis. Then an implementation of the proposed model is used to simulate the behaviour response of a masonry wall. As presented in the second part of the chapter, in order to compare different FEM softwares to employ linear and non-linear analysis.
- **Chapter VII:** it focuses on in-field experimental studies which took place in three historical structures in Bologna, of different construction ages, by following the in situ and laboratory tests on samples besides analysing the results obtained by each test. The experimental tests were carried out in Botanica Building, Palazzina della Viola, and Facoltà di Lingue e Letterature straniere. For evaluating the conservation state of masonry. Experimental results on compressive strength, shear strength, Young's modulus, and shear modulus were obtained for both brick and mortar, in order to be implemented in structural model.
- **Chapter VIII & IX:** they contain the summary and outlook of the study, and discuss the results and possible future recommendations on masonry structures assessment.

Chapter 2

Introduction to Masonry Structures

2.1 Historical introduction to masonry structures

The Egyptian Pyramids, the Colosseum in Rome, India's Taj Mahal, the Great Wall of China, some of the world's most significant architectural achievements have been built with masonry. Through civilization, architects and builders have chosen masonry for its beauty, versatility and durability (Decanini, et al., 2004).

Masonry units are man's oldest manufactured product. Sun-baked clay bricks were used in the construction of buildings more than 6,000 years ago. In order to prevent distortion and cracking of the clay shapes, chopped straw and grass were added to the clay mixture. The next big step in enhancing brick production occurred about 4,000 B.C. At that time manufactures began producing brick in uniform shapes. Along with the shaping of brick, the move from sunbaking to firing was another important change. This improved the durability of the brick (Islam, 2008).

Through the centuries, the methods for producing masonry brick have continued to evolve. As masonry units are composed of shale and clay and is fired in kilns of approximately 2,000 degrees Fahrenheit. The firing process causes the clay particles to bond chemically (Islam, 2008).

Masonry brick construction became more elaborate, the use of brick became more sophisticated. This evolution was prompted by the development of cavity walls. When originally developed, cavity walls consisted of two separate brick or stone walls with about a 5 cm air space between them. Cavity walls were developed to reduce the problems associated with water penetration. Water that would seep inside the outer wall would then run down that wall, while the inside wall would remain dry. Cavity walls soon became recognized as the best way to build, not only because they helped reduce problems with water penetration, but because they could support a heavy load such as a roof or floor. In 1850 a special block with air cells was developed. Over the years modifications to this product were introduced until the industry arrived at the standardized product we see today (Sanchez, 2005).

Masonry is still used today. As you look around you will notice that there are many aspects of society where you will see some form of masonry. Which represents the importance of masonry structures assessment.

2.2 Mechanical behaviour and properties of masonry structures

In masonry construction technique 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. Bricks are adobe or fired clay brick. Mortar is usually made from clay, bitumen or cement. The use of blocks and the mortar in a combination of way as a construction technique is called masonry construction technique (Da Porto, 2003).

There are many possibilities for the arrangement and combinations generated by the geometry of the masonry elements with respect to the stone masonry structures (Fig. 2.1) and brick masonry structures (Fig. 2.2).

The cross section of the masonry wall can be constructed by different kinds of arrangements, related to the width of the wall itself. The ancient constructions were of small width, stone units could be of the full width. If the walls were very thick, ashlars would only be used for the outer leafs and the inside would be filled with irregular stones or rubble, or more than one leaf of masonry would be used (Lourenço et al., 1998), (Fig.2.3).

The possibility of combining masonry elements with different quantities and geometry give masonry a wide range of alternatives with respect to the mechanical behaviour and structural performance. Masonry has a good ability to resist the compressive loads and a poor performance for resisting tensile loads (Andrés and Barraza, 2012).

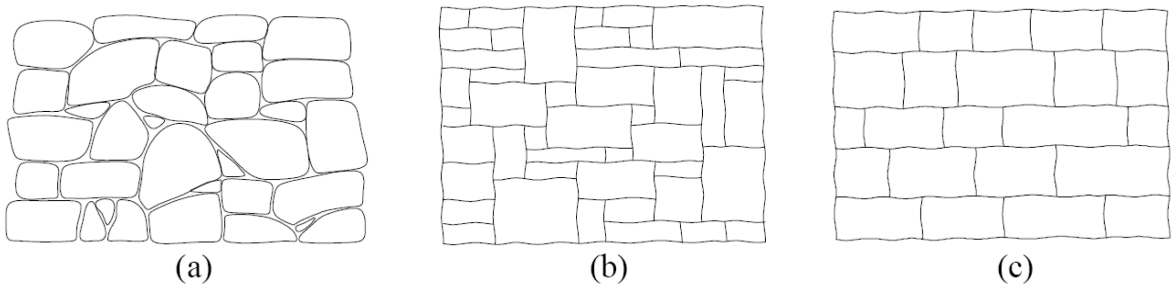


Figure 2.1: The possible arrangement for stone masonry: (a) rubble masonry; (b) ashlar masonry; (c) coursed ashlar masonry (Lourenço et al., 1998).

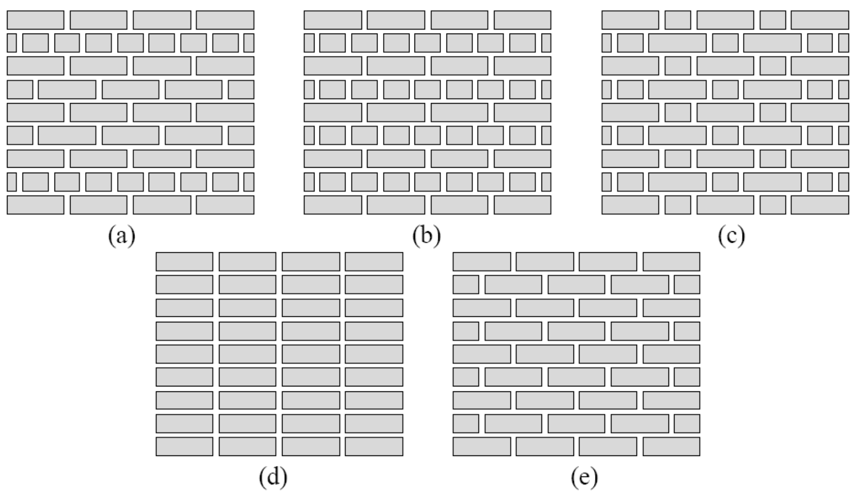


Figure 2.2: Different arrangements for brick masonry: (a) American bond; (b) English bond; (c) Flemish bond; (d) stack bond; (e) stretcher bond (Lourenço et al., 1998).

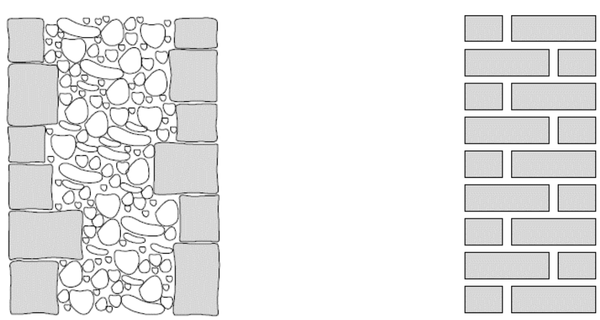


Figure 2.3: Cross section arrangement of masonry walls (Lourenço et al., 1998).

The behaviour of brick and mortar is different from each other. Mortar usually is much softer than brick units. This composite nature and complex geometry of masonry leads to a very complex structural behaviour. Unfortunately, the strength and stiffness properties of the constituent materials do not reflect the structural and stiffness properties of the masonry structure itself (Felix, 1999). 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, in a prism under compressive loading normal to bed joints, 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 (Oliveira, 2009), (Fig.2.4).

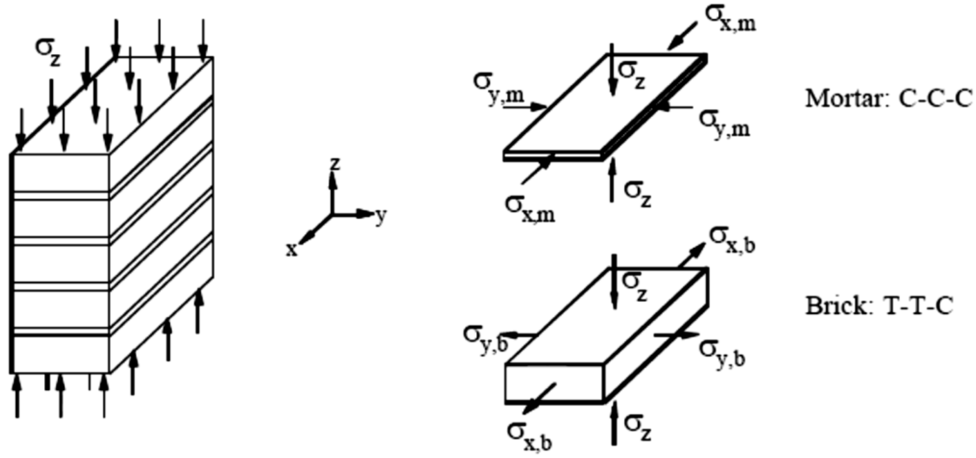


Figure 2.4: Masonry prism under compressive loading normal to bed joints and stress states for brick and mortar elements (C: compression, T: tension) (Oliveira, 2009).

Masonry elements (bricks and mortar) have a strong non-linear response as high loads are applied. The mechanical behaviour of bricks is not necessarily homogeneous and isotropic. This means that the properties are not the same in different directions. Also the behaviour changes with respect to tension or compression loads. Generally, the behaviour of bricks is described as elastic-brittle (Hossain et al., 1997). The stress-strain curve of the brick can be obtained, with respect to the direction of the applied load and measured deformation, and characteristic compression strength can be measured (Kaushik et al., 2007). A typical stress-strain curve for bricks in compression is shown in (Fig.2.5).

In order to evaluate the bricks module of elasticity (E_b), (Kaushik et al., 2007) states that the recommended range depends on the compression strength of the brick (f_b):

$$150f_b < E_b < 500f_b$$

Which it can be expressed graphically (Fig.2.6).

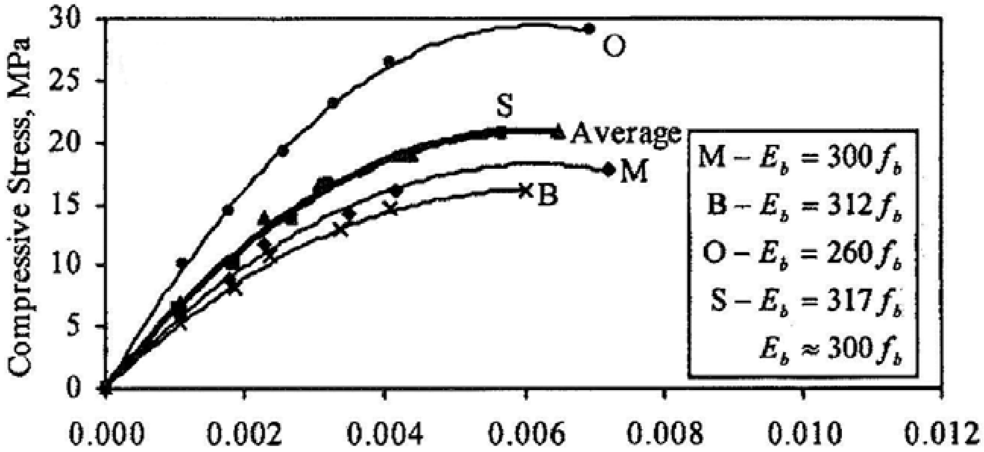


Figure 2.5: Stress-strain curve for bricks in compression (Kaushik et al., 2007).

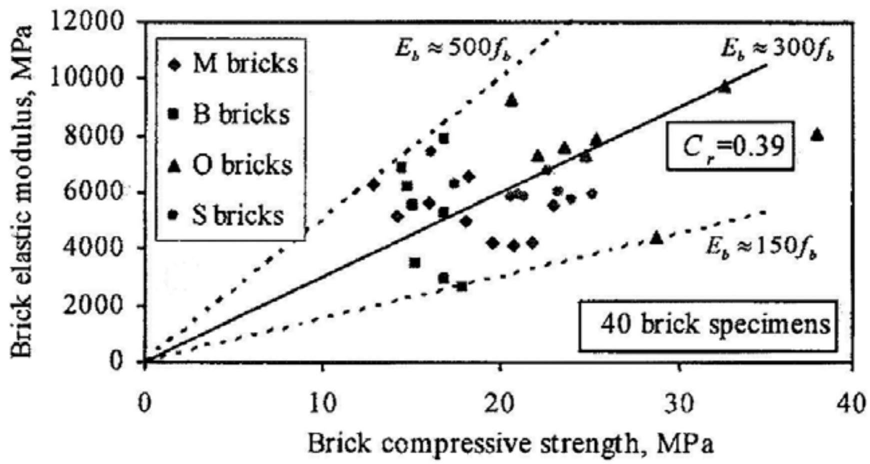


Figure 2.6: Variation of modulus of elasticity of brick (Kaushik et al., 2007).

On the other hand, the stress strain curve of new mortar and a characteristic compression strength are obtained by using a simple compression test (Fig.2.7).

In order to evaluate the mortar module of elasticity (E_m), (Kaushik et al., 2007) states that the recommended range depends on the compression strength of the mortar (f_m) (Fig.2.8):

$$100f_m < E_m < 400f_m$$

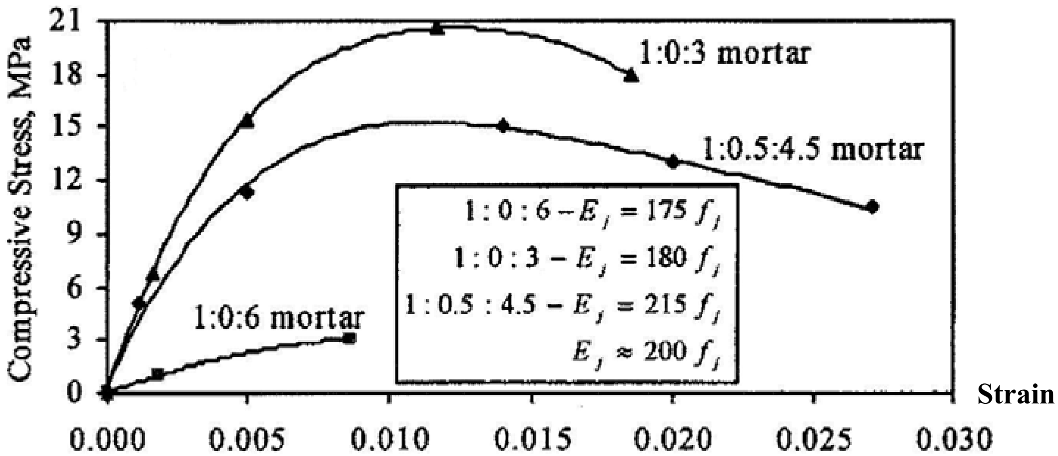


Figure 2.7: stress-strain curve for mortar in compression. (Kaushik et al., 2007)

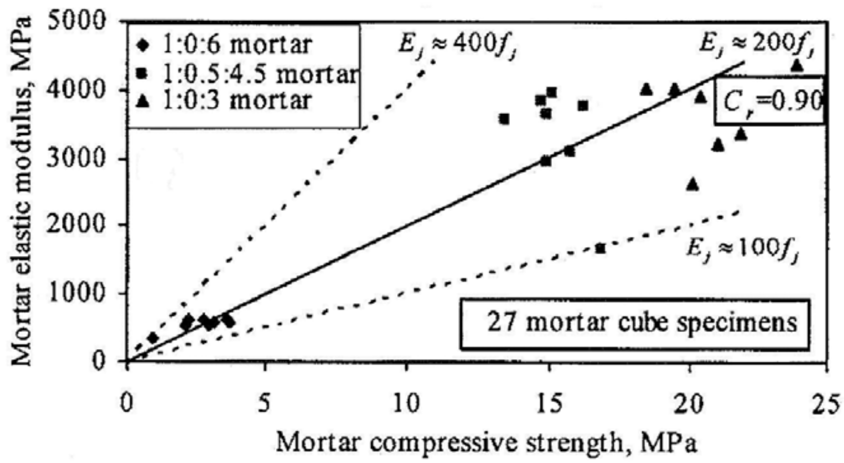


Figure 2.8: Variation of modulus of elasticity of new mortar (Kaushik et al., 2007).

From theory of elasticity (Gere & Timoshenko, 1986), the shear module of elasticity (G) is given as:

$$G_m = \frac{E}{2(1+v)}$$

where,

v = the Poisson's ratio.

The numerical analysis of the historical masonry structures requires the material mechanical properties like axial compression strength. It is not always possible to perform compression test on masonry to obtain the actual strength, although this is the basic structural property for analyzing masonry structures (Christy, 2013). On the other hand, the compressive strength of brick (f_b) and mortar (f_m) can be evaluated by standard tests. As stated by (Christy, 2013), the axial strength of the unreinforced brick masonry can be predicted from the obtained results of the brick and mortar strength and the strength. The predicted values were compared with the data reported by (Hemant et al., 2007) which included Bennet‘s equation and Dayaratnam‘s equation.

While Bennet has given a relationship between the strength of the brick masonry and the strength of the brick and the mortar as:

$$\text{Masonry strength} = 0.63{f_b}^{0.49}{f_m}^{0.32}$$

and Dayaratnam has given a relationship between the strength of brick masonry and the strength of the brick and the mortar as:

$$\text{Masonry strength} = 0.275{f_b}^{0.5}{f_m}^{0.5}$$

the generalized equation proposed by (Christy, 2013) for estimating the axial strength of the brick masonry is:

$$\text{Masonry strength} = 0.35{f_b}^{\alpha}{f_m}^{\beta}$$

where,

α and β = constants

f_b = strength of brick in MPa

f_m = strength of mortar in MPa

generally, the brick strength is greater than the mortar strength, so ‘α’ must be greater than ‘β’ as stated by (Hemant et al., 2007), which are suggested to be: $\alpha = 0.65,\ \beta = 0.25$.

Through the general mechanical behaviour of masonry elements, it appears an important aspect related with the stress-strain curve, which is represented by the softening. It can be defined as the gradual decrease in the mechanical resistance under a continuous increase of the applied load upon the masonry structure. It is a salient feature due to a process of progressive internal crack growth. Such mechanical behaviour is commonly attributed to the heterogeneity of the material, due to the presence of different phases and material defects (Lourenço et al., 1998).

This phenomenon is identified by both compressive and tensile failures where it has been observed as degradation of the cohesion in Coulomb friction models. Softening behaviour is highly dependent upon the boundary conditions in the experiments and the size of the specimen (Lourenço et al., 1998). (Fig.2.9) and (Fig.2.10) show characteristic stress-displacement diagrams for quasi-brittle materials in uniaxial compression and tension. The inelastic behaviour for both compression and tension can be described by the integral of the stress-strain diagram, which is represented by the fracture energy G_f.

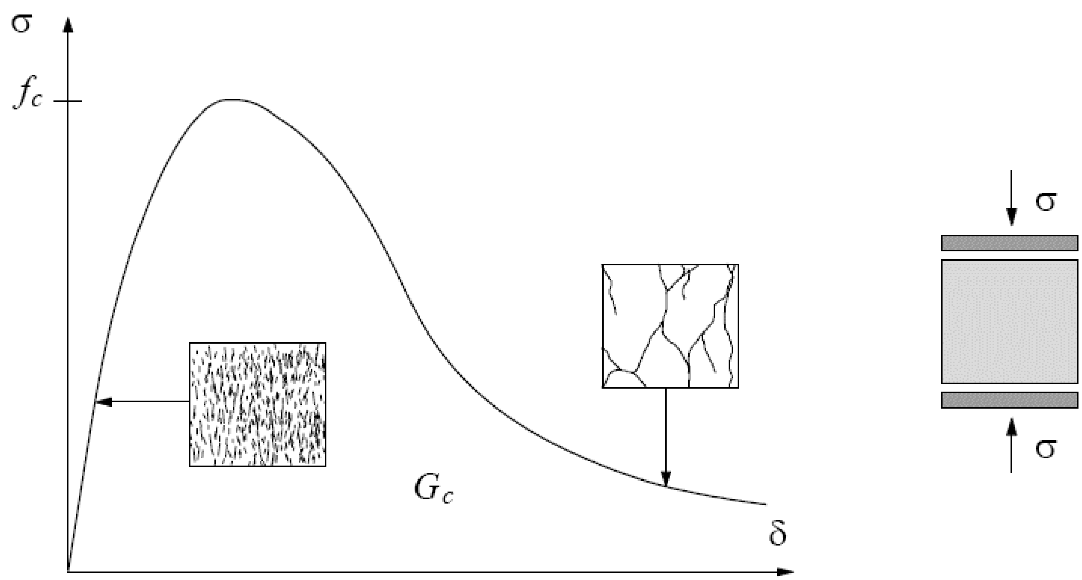


Figure 2.9: Typical behavior of quasi-brittle materials under uniaxial compression (Lourenço et al., 1998).

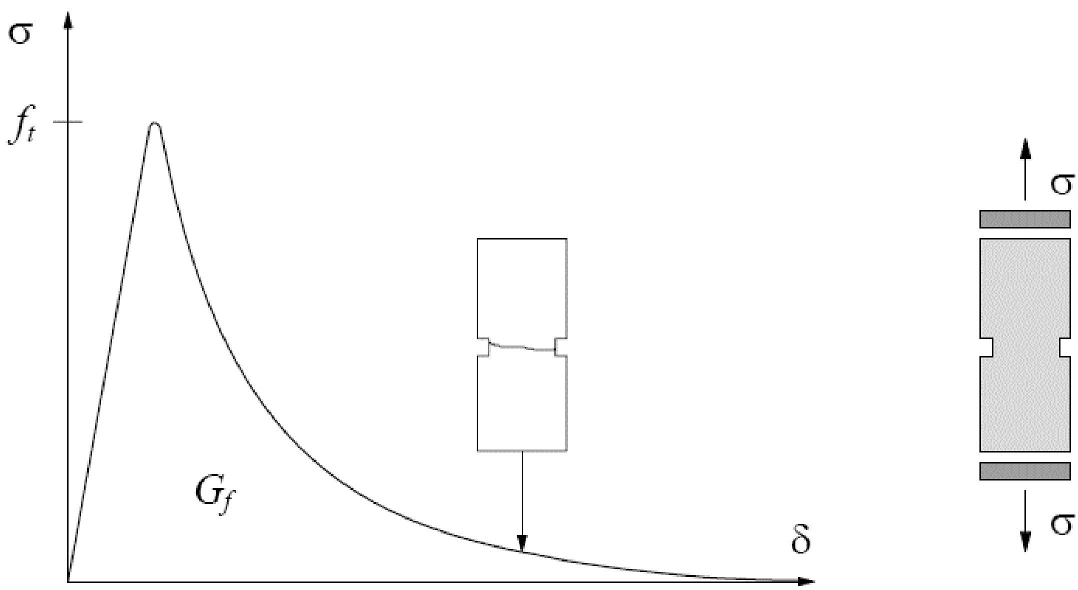


Figure 2.10: Typical behavior of quasi-brittle materials under uniaxial tension (Lourenço et al., 1998).

2.3 The acting actions and failure mechanisms of masonry structures

In the design and analysis of masonry structures, load intensities are obtained by structural design codes. However, this method is not applicable for old structures 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 (Özen, 2006). The most common expected loads on a historic structure are:

• Self weight of the structure.

• Seismic actions.

• 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. On the other hand, seismic actions and differential settlements of supports are usually the main reasons for the damage or collapse of masonry structures (Mele, 2003). 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. The high seismic vulnerability of masonry structures is due to (Mele, 2003):

• Highly nonlinear behaviour.

• 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 seismic actions 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 seismic actions 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 (Grillo, 2003).

Types of Failure

In the general case of masonry structures, there are three main failure mechanisms (Decanini et al., 2004):

a) In-plane failure

In-plane resistance of masonry walls is based on mortar strength and brick proportions. If the forces are strong enough to exceed the in-plane strength capacity of the wall, a shear failure will occur. This failure mode is characterized by brittle tensile cracking through the mortar and the masonry unit and a sudden loss of lateral load capacity.

b) Out-of-plane failure

Seismic or wind loadings induce out-of-plane bending of walls between the restraining floors. Analysis of the failure modes must take into account many different factors, such as boundary conditions, wall compressive strengths, joint tensile strengths, wall stiffness, and applied loadings.

Walls will typically remain stable under dead load and after cracking if they are within the specified height-to-thickness ratio. If the slenderness ratio is exceeded, the wall needs bracing by either a horizontal brace or vertical columns. Parapets, chimneys, and similar elements extending above the topmost line of restraint are most vulnerable to out of plane forces.

c) Connections

Out of plane loads cause walls to push against and pull away from the floors that they are connected to. Failure to have a secure connection between the two elements can cause failure by falling brick as well as floor collapse. This type of problem can be corrected and work can be performed while the building is occupied.

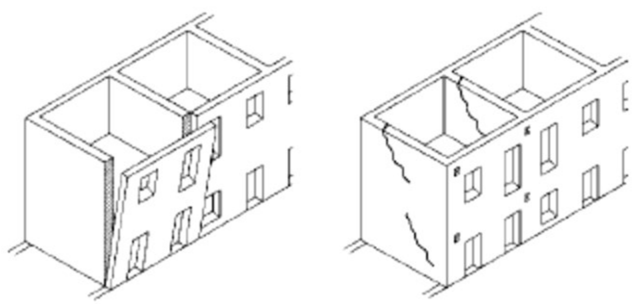


Figure 2.1: Basic damage modes of a masonry building (Decanini et al., 2004).

Chapter 3

State of Art in Masonry Assessment

3.1 Assessment for masonry structures

The preservation of historical structures is important to keep it save through the upcoming future. The historical masonry structures need to be assessed and evaluated in a way to determine and figure out the most risky primary failure mechanisms and provide a strategy for rehabilitation. So in order to have a fully structural assessment, it is needed to have: measurement survey, material tests, soil survey, long-term observations, and structural tests (Teomete and Aktaş, 2010).

Finite element models and structural analyses of the historical structures were developed by using the data obtained from destructive and non-destructive tests to characterize the materials of the structure (Binda et al., 2000). Studying the behaviour of the masonry structures is the base of the structural assessment, where several practical structural analyses techniques such as force polygon and the chain rule have been used to determine the reasons for structural damage in various masonry structures (Teomete and Aktaş, 2010), also long-term monitoring helps to increase the knowledge of the real behaviour of the structure and in the planning of maintenance intervention. In the long term, static monitoring requires accurate and very stable systems, able to relate measurements often spaced over long periods of time.

3.1.1 The importance of historical structures assessment

Historical masonry structures were built on ancient times when no appropriate theory and good knowledge were available, where it have been built according to the available knowledge and experience. So many buildings which still exist do not satisfy the present guidelines. Also the recent worldwide earthquakes make people more conscious about the safety of life and property.

Some of the famous building which becomes valuable in terms of culture and history demand longer service life (Carini and Genna, 2012). So lot of reasons may be claimed for historical structure assessment. It is summarized as follows:

- To eliminate structural problems or distress which results from unusual loading or exposure conditions, inadequate design, or poor construction practices. Distress may be caused by overloads, fire, flood, foundation settlement, deterioration resulting from abrasion, fatigue effects, chemical attack, weathering, inadequate maintenance, etc.
- To be conform to current codes and standards.
- To allow the feasibility of changing the use of a structure to accommodate a different use from the present one.
- Durability problems due to poor or inappropriate construction materials.
- Design or construction errors.
- Aggressive environments not properly understood during the design stages.
- Increased life-span demands made on ageing infrastructure.
- Exceptional or accidental loading.
- Varying life span of different structural or non-structural components.

3.1.2 The procedures of masonry structures assessment

The historical structures assessment comprises, in general, the following phases/actions:

- Acquisition of documented data about the building.
- Detailed survey of the existing condition of the building.
- Elaboration of the diagnosis (eventually, with the carrying of tests).
- Assessment of the structural safety.
- Design of the solutions for the intervention.
- Execution of the intervention.

These phases/actions depend on the actual conditions of the structure and on the objectives to be fulfilled by the assessment, where it can be by different forms, going from the non-invasive (with the imposition or not of restrictions of use), passing through different kinds of works of repair and/or strengthening, until, eventually, partial demolition followed by reconstruction. The decisions about the solutions to be adopted on the intervention will still be submitted to a cost-benefit analysis, in which all the relevant aspects will be considered, namely, the compatibility of the structural safety with respect to the cultural value of the building, and the cost to be as low as possible (CIB Commission, 2010).

3.2 Experimental techniques for masonry assessment

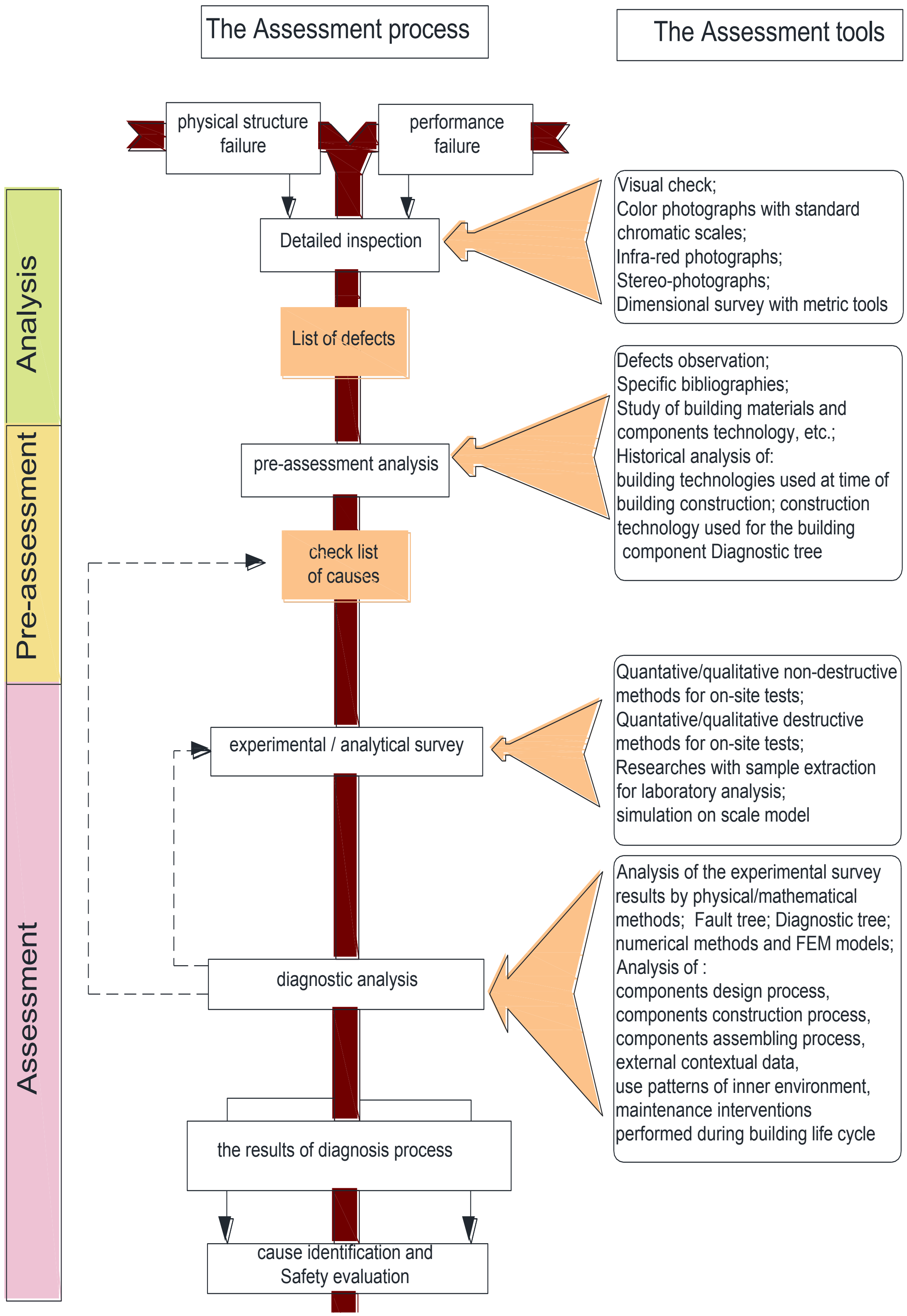
Large number of monuments are located in seismic areas, where it is needed to be controlled and investigated in order to keep it save and stable. The methodological approach to analyse the existing historical masonry structures requires a series of preliminary investigations by using experimental techniques which provide details of the structure's mechanical characteristics and define the structural behaviour of the monument. There are many challenges can be found during the analysis of historical structures, due to the complexity of its geometry, the variability of the properties of different materials, and the construction techniques have been used.

Masonry structures evaluation techniques can involve varying levels of damage or deconstruction of the masonry. Especially in historic structures, even relatively minor damage can result in expensive and difficult repairs. The use of non-destructive methods helps limit the damage caused by the testing. Additionally, gathering information about the existing masonry provides designers with confidence that they understand the current conditions and the potential causes of any distress. This knowledge and confidence typically lead to better planning and more effective designs, ultimately minimizing modifications and additions to the historic structure. (Donald, 2010)

The techniques are used in the diagnosing the historical structures are divided into destructive, semi-destructive and non-destructive methods. Destructive tests can be applied to samples and natural-scale structural elements. Both are completely destroyed during the tests. For this reason only a few representative natural-scale elements are subjected to such tests. Semi-destructive tests are also applied to samples and natural-scale elements and structures but they involve a small intrusion into the structure of the material, resulting local loss of service properties and requiring repair. There is no such intrusion in the case of non-destructive tests which are applied to mainly natural-scale elements and structures. Moreover, non-destructive tests can be applied to the same elements and structures many times and at different times whereby such methods are suitable for the diagnostic testing of building structures during both their construction and many years of their service life (Rossi, 1990).

NDT has increasingly become a major part of masonry field work. As many of the historical masonry buildings are aging and need to maintenance and repair, also it is a viable alternative to evaluate their condition and strength instead of using techniques which would limit their use or cause destruction of the buildings. On the other hand can be used to evaluate structures stability against all the external actions. The information obtained by NDT is necessary to determine the mechanical parameters for the analysis of the historical monuments and comprehension of its structural behaviour, (Akbulut and Akoz, 2004).

The assessment process presented in the chart below summarises the whole phases of this study, it can be considered as a synthesis of all investigation process which play an important role in the diagnosis and analysis of historical structures. This flow chart does not present something new but it is an attempt to indicate logical steps in the assessment with respect to the experimental and computational approaches. The assessment of structures starts from the observation of any failure that indicate the presence of structural defect which cause a condition of instability. Then it develops to deal with analysing all the causes' possibilities which may somehow be related to the defect itself. The analysis of the defect that may be found in the structure is a significant stage in the assessment process, based on accurate experimental tools and analytical surveys, where the obtained data lead to a number of possible diagnosis hypothesis. These are characterized by some degree of uncertainty. The presence of different possible reasons must be considered at the assessment process supported by operative tools.


The Assessment process
The Assessment tools
physical structure failure
performance failure
Detailed inspection
List of defects
pre-assessment analysis
check list of causes
experimental / analytical survey
diagnostic analysis
the results of diagnosis process
cause identification and Safety evaluation
Analysis
Pre-assessment
Assessment
Visual check; Color photographs with standard chromatic scales; Infra-red photographs; Stereo-photographs; Dimensional survey with metric tools
Defects observation; Specific bibliographies; Study of building materials and components technology, etc.; Historical analysis of: building technologies used at time of building construction; construction technology used for the building component Diagnostic tree
Quantative/qualitative non-destructive methods for on-site tests; Quantative/qualitative destructive methods for on-site tests; Researches with sample extraction for laboratory analysis; simulation on scale model
Analysis of the experimental survey results by physical/mathematical methods; Fault tree; Diagnostic tree; numerical methods and FEM models; Analysis of : components design process, components construction process, components assembling process, external contextual data, use patterns of inner environment, maintenance interventions performed during building life cycle

Chapter 4

Review of Experimental Techniques for Masonry

Experimental determination of material properties

The properties of masonry structures are influenced by many factors, such as; the construction technique, type of mortar, physical properties of the materials used for the mortar, and state of masonry units before casting. Where it can be deduced that in the experimental determination of mechanical properties of masonry structures, a large number of variables can be considered. As known that the structural analysis requires the material properties, for example, the modulus of elasticity of masonry is required for the linear static analysis. Stress-strain curves of masonry are required for detailed non-linear analysis of masonry structures. So There are many ways for obtaining the mechanical properties of masonry structures which are divided into destructive, semi destructive, and none destructive techniques. Which can be done in situ or in the laboratory, where it follows by many standards, such as ASTM (American Society for Testing and Materials), RILEM (The International Union of Laboratories and Experts in Construction Materials), and BS (British Standards), which are mainly used to describe the experimental techniques in this chapter.

4.1 Testing techniques for masonry unit

These test techniques provide various testing procedures commonly used for evaluating mechanical characteristics of masonry units (Fig. 4.01).

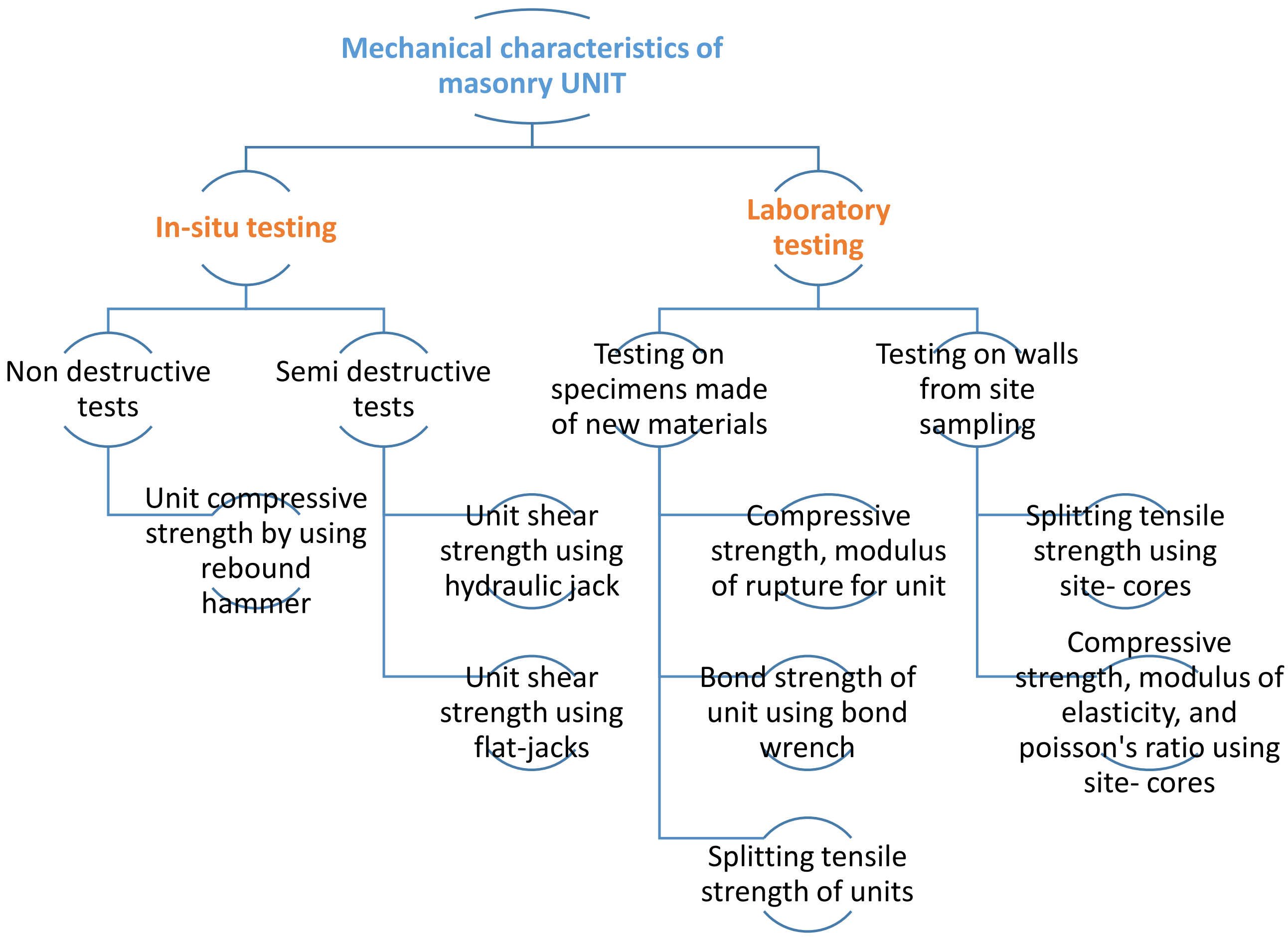


Figure 4.01: techniques for determining the mechanical properties of masonry units.

4.1.1 Compressive strength and modulus of rupture for masonry unit

Scope

This test covers the procedures for sampling and testing masonry units; it includes Young's modulus, compressive strength, and Poisson's ratio, for both brick units and masonry core specimens.

Apparatus

- Compression Loading Device: The loading device shall be of sufficient capacity to apply load at a rate conforming to the test requirements. The loading device may be equipped with a displacement transducer that can be used to advance the loading at a specified rate.
- Bearing Surfaces: Two steel plates are used to transmit the axial load to the ends of the specimen.
- Strain/Deformation Measuring Devices: Deformations or strains may be determined from data obtained by electrical resistance strain gages, compress meters, linear variable differential transformers (LVDTs), or other suitable means.
    1. Determination of Axial Strain: The design of the measuring device shall be such that the average of at least two axial strain measurements can be determined. Measuring positions shall be equally spaced around the circumference of the specimen, close to mid-height. The gauge length over which the axial strains are determined shall be at least ten grain diameters in magnitude.
    2. Determination of Lateral Strain: The lateral deformations or strains may be measured by any of the methods. Either circumferential or diametric deformations (or strains) may be measured. A single transducer that wraps around the specimen can be used to measure the change in circumference. At least two diametric deformation sensors shall be used if diametric deformations are measured. These sensors shall be equally spaced around the circumference of the specimen close to mid-height. The average deformation (or strain) from the diametric sensors shall be recorded. (ASTM D7012, 2013)

Procedure

Sampling:

- Selection of Specimens: solid masonry units shall be selected representative of the lot of units from which they are selected and shall include specimen's representative of the complete range of colours, textures, and sizes. Specimens shall be free of or brushed to remove dirt, mud, mortar, or other foreign materials

- The specimens for each sample shall be selected from cores representing a valid average of the type of masonry under consideration. This can be achieved by visual observations of the structure.
- Desirable specimen length to diameter ratios are between 2:1 and 2.5:1. Specimen length to diameter ratios of less than 2:1 are unacceptable.
- The number of specimens required to obtain a specific level of statistically results.

Modulus of Rupture (Flexure Test):

- The test specimens shall consist of whole full-size units. At least five specimens shall be tested.
- Support the test specimen flatwise, apply the load in the direction of the depth of the unit to the upper surface through a steel bearing plate.
- The rate of loading shall not exceed (8896 N)/min.

Compressive Strength test:

- Test brick specimens flatwise, the load shall be applied perpendicular to the bed surface of the brick with the brick in the stretcher position.
- The upper bearing shall be a spherically seated, hardened metal block firmly attached at the centre of the upper head of the machine. The centre of the sphere shall lie at the centre of the surface of the block in contact with the specimen. The block shall be closely held in its spherical seat, but shall be free to turn in any direction.
- Apply the load, up to one half of the expected maximum load, at any convenient rate, after which, adjust the controls of the machine so that the remaining load is applied at a uniform rate in not less than 1 nor more than 2 min.

Strain/ Deformations test:

- The spherical seat shall rotate freely in its socket before each test. Where the lower platen shall be placed on the base or actuator rod of the loading device. The bearing faces of the upper and lower platens and of the test specimen shall be wiped clean, and the test specimen shall be placed on the lower platen. The upper platen shall be placed on the specimen and aligned properly.

- The axial load shall be applied continuously and without shock until the load becomes constant, is reduced, or a predetermined amount of strain is achieved.
- Readings of deformation shall be observed and recorded at a minimum of ten load levels that are evenly spaced over the load range. (ASTM D7012, 2013)

Data Analysis

- the modulus of rupture of each unit can be calculated as given in (NCMARDL, 1993):

$$S = \frac{3W(\frac{l}{2} - x)}{bd^2}$$

where:

S = modulus of rupture of the specimen at the plane of failure, Pa

W = maximum load indicated by the testing machine, N

l = distance between the supports, mm

b = net width of the specimen at the plane of failure, mm

d = depth of the specimen at the plane of failure, mm

x = average distance from the mid-span of the specimen to the plane of failure measured in the direction of the span along the centreline of the bed surface subjected to tension, mm

- compressive strength of each unit can be calculated as given in (ASTM C67, 2013):

$$C = \frac{W}{A}$$

where:

C = compressive strength of the specimen, Pa

W = maximum load, N

A = average of the gross areas of the upper and lower bearing surfaces of the specimen, m^2,

- the relation between the shear and bulk moduli and Young's modulus and Poisson's ratio are:

$$G = \frac{E}{2(1 + v)}$$

$$K = \frac{E}{3(1-2v)}$$

where:

G = shear modulus,

K = bulk modulus,

E = Young's modulus,

v = Poisson's ratio.

- Axial strain, ε_a can be calculated as follows:

$$\varepsilon_a = \frac{\Delta L}{L}$$

where:

L = original un-deformed axial gauge length, mm

ΔL = change in measured axial gauge length, mm

- Lateral strain, ε_l can be calculated as follows:

$$\varepsilon_l = \frac{\Delta D}{D}$$

where:

D = original un-deformed diameter, mm

ΔD = change in diameter (positive for increase in diameter), mm

- The stress-versus-strain curves can be plotted for the axial and lateral directions. The complete curve gives the best description of the deformation behaviour of masonry unit having nonlinear stress-strain relationships at low- and high stress levels.
- Tangent modulus at a stress level that is some fixed percentage (usually 50 %) of the maximum strength.
- Average slope of the more-or-less straight-line portion of the stress-strain curve. The average slope shall be calculated either by dividing the change in stress by the change in strain or by making a linear least squares fit to the stress-strain data in the straight-line portion of the curve.

- Secant modulus, usually from zero stress to some fixed percentage of maximum strength.
- The value of Poisson's ratio, υ, is greatly affected by nonlinearities at low-stress levels in the axial and lateral stress-strain curves. It is desirable that Poisson's ratio shall be calculated from the following equation:

$$v = -\frac{\text{slope of axial curve}}{\text{slope of lateral curve}} = -\frac{\text{E}}{\text{slope of lateral curve}}$$

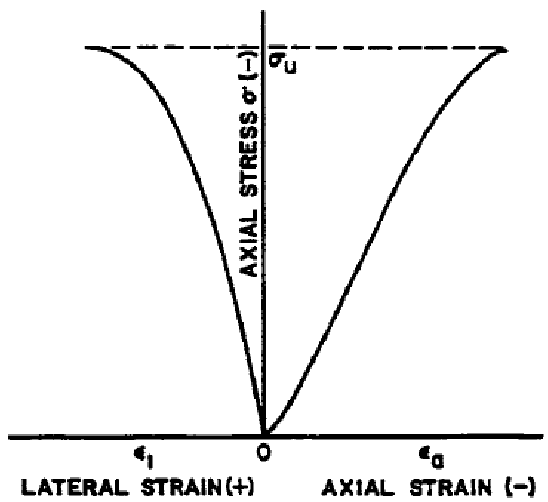


Figure 4.02: Stress-strain plot obtained by the technique (ASTM D7012, 2013).

Conclusion

Masonry stress–strain properties are required in the nonlinear analyses of structures. However, compressive stress–strain relationships for masonry are determined by testing masonry prism. Masonry is typically a nonelastic, nonhomogeneous, and anisotropic material composed of two materials of quite different properties: stiffer bricks and relatively softer mortar. Under lateral loads, masonry does not behave elastically even in the range of small deformations. Masonry is very weak in tension because it is composed of two different materials distributed at regular intervals and the bond between them is weak. Therefore, masonry is normally provided and expected to resist only the compressive forces, during compression of masonry prisms constructed with stronger and stiffer bricks, mortar of the bed joint has a tendency to expand laterally more than the bricks because of lesser stiffness. However, mortar is confined laterally at the brick–mortar interface by the bricks because of the bond between them; therefore, shear stresses at the brick-mortar interface result in an internal state of stress which consists of triaxial compression in mortar and bilateral tension coupled with axial compression in bricks. This state of stress initiates vertical splitting cracks in bricks that lead to the failure of the prisms.

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4.1.2 Splitting tensile strength for masonry unit

Scope

In this test a line load produced along surface of the masonry specimen. The compressive load applied to the masonry unit samples, imposed by means of bearing rods, results in a tensile stress distributed over the height of the specimen for it's the split length. This loading induces tensile stresses on the plane containing the applied load and relatively high compressive stresses in the area immediately around the applied load. This test method can be conducted with the rod oriented either in the longitudinal direction or in the transverse direction of the bed face.

Apparatus

- Bearing Rods: paired steel bearing rods with diameters within 1 ⁄8 to 1⁄12 of the specimen height, of a length greater than the length of the intended test area, and of straightness within 0.5 % of the specimen length shall be provided for each unit. Bearing rods that meet the straightness requirement can be reused.
- Supplemental Bearing Bar or Plate: If the diameter or largest dimension of the upper bearing face or lower bearing block is less than the length of the specimen to be tested, a supplementary bearing bar or plate shall be used. The contact surfaces of the bar or plate shall be machined to within 0.05 % of planeness as measured on any line of contact of the bearing area. The bearing bar or plate shall have a width of at least 51 mm, and a thickness not less than the distance from the edge of the spherical or rectangular bearing block to the end of the specimen. The bar or plate shall be used in such a manner that the load will be uniformly applied over the entire intended split length of the specimen.
- Testing Machine: The upper, hardened metal bearing face shall be spherically seated and attached at the centre of the upper head of the machine. The centre of the sphere shall lie at the centre of the surface of the plate in contact with the specimen.

Procedure

Laboratory Samples as states in (ASTM C1006, 2007):

- Positioning Bearing Rods: mark the intended location of the split surface on either faces, stretcher or normally exposed faces for transverse splitting, and end faces for longitudinal splitting. Spread a gypsum capping compound
- The bearing rods shall be positioned no closer to a free edge than one half the specimen height.
- Test Alignment: Align the rods with the centreline of the plates, and centre the rods in the transverse direction. Support the specimen on compressible rods or tubes that are 1.6 mm smaller in diameter than the bearing rods. Remove the compressible rods when the specimen is held in vertical orientation by the testing-machine platens.

- Rate of Loading: Apply the load without impact and load continuously at a rate less than 8900 N/min.
- Measurement: Determine the height of the specimen to the nearest 2.5 mm by averaging three heights measured near the ends and the middle and on a plane perpendicular to the bed surface. Determine the split length of the specimen to the nearest 2.5 mm by averaging at least two measurements taken on the plane of the bearing rods. The split length is the actual net length of the failure plane of the bearing rods and is equal to the gross length of the unit minus the length of any voids along this plane.

Site-Cores Samples as states in (ASTM C1006, 2007):

- Draw diametric lines on each end of the specimen using a suitable device that will ensure that they are in the same axial plane.
- Determine the diameter of the test specimen to the nearest 0.25 mm by averaging three diameters measured near the ends and the middle of the specimen and lying in the plane containing the lines marked on the two ends. Determine the length of the specimen to the nearest 2 mm by averaging at least two length measurements taken in the plane containing the lines marked on the two ends.
- Positioning Using Marked Diametric Lines: Centre one of the plywood strips along the centre of the lower bearing block. Place the specimen on the plywood strip and align so that the lines marked on the ends of the specimen are vertical and cantered over the plywood strip. Place a second plywood strip lengthwise on the cylinder, cantered on the lines marked on the ends of the cylinder. The supplementary bearing bar or plate, when used, and the centre of the specimen are directly beneath the centre of thrust of the spherical bearing block.
- Positioning by Use of Aligning Jig: Position the bearing strips, test cylinder, and supplementary bearing bar by means of the aligning jig and centre the jig so that the supplementary bearing bar and the centre of the specimen are directly beneath the centre of thrust of the spherical bearing block.
- Rate of Loading: Apply the load continuously and without shock, at a constant rate within the range 0.7 to 1.4 MPa/min splitting tensile stress until failure of the

specimen. Record the maximum applied load indicated by the testing machine at failure. Note the type of failure and the appearance of the masonry core.

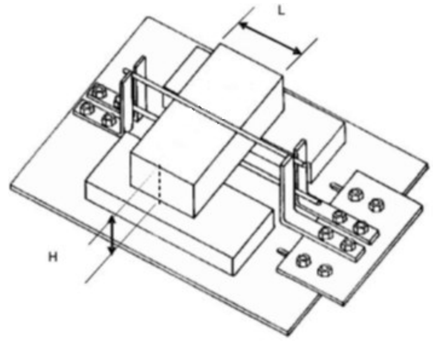

Figure 4.03: Masonry unit testing setup (ASTM C1006, 2007).

Figure 4.04: masonry site-core samples (Mazzotti and Sassoni, 2013).

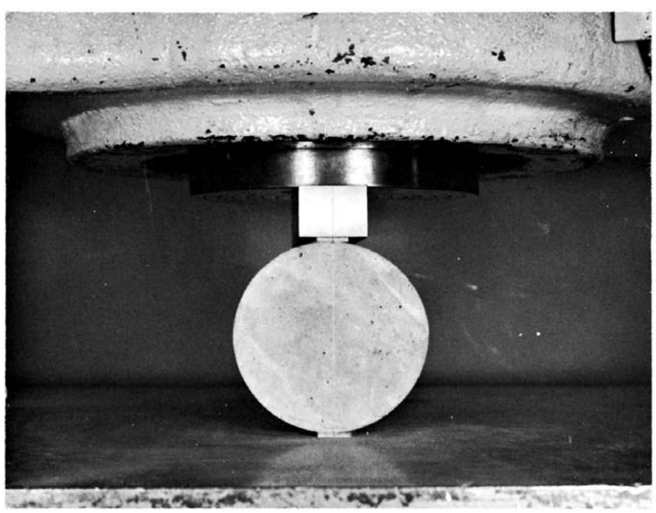

Figure 4.05: Specimen Positioned in the testing machine (ASTM C1006 ,2007).

Data Analysis

- the splitting tensile strength of the specimens can be calculated as follows:

$$f_t = \frac{2P}{\pi LH}$$

where:

f_t = splitting tensile strength, kPa

P = maximum applied load indicated by the testing machine, kN

L = split length, mm, gross length minus the length of any voids along the failure plane of the bearing rods.

H = distance between rods, mm

Conclusion

Splitting tensile strength is generally greater than direct tensile strength and lower than flexural strength (modulus of rupture). Where it is used in the design of masonry structural and evaluate the shear resistance. Tensile failure occurs rather than compressive failure because the areas of load application are in a state of triaxial compression, thereby allowing them to withstand much higher compressive stresses than would be indicated by a uniaxial compressive strength test result.

References

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MAZZOTTI, C., SASSONI, E., & PAGLIAI, G. (2013). "Determination of shear strength of historic masonries by moderately destructive testing of masonry cores." Construction and Building Materials 54: 421–431.

4.1.3 Masonry flexural bond strength by bond wrench test

Scope

Bond strength between mortar and masonry unit is a significant factor in the performance of a masonry system. This test evaluates the flexural bond strength for masonry mortar bed joint, both in situ and laboratory prepared samples. In which It is possible to check the quality of mortar under a given conditions.

Apparatus

- Tools for sample preparation:
  1. Prism alignment jig.
  2. Mortar joint template
  3. Mechanical paddle-type mortar mixer
  4. Flow table, flow mold, and caliper
  5. Cone penetrometer, unit measure, straightedge, spatula, tapping stick, and spoon.
- Bond strength test apparatus:

A bond wrench is a lever that can be clamped to the top unit of a prism and is of such mass and proportion that the stresses imposed by the bond wrench at the start of a test do not exceed 0.1 N/mm^2 in either flexural compressive stress or flexural tensile stress. Load may be applied in a number of different ways. Four typical methods are as follows:

1. By filling a container hanging from the moment arm of the wrench with lead shot or an alternative material, delivered in a steady stream. The amount delivered should be assessed by weighing on a balance accurate to within ±25g.
2. By driving a mass out along the moment arm at a steady rate and in such a way as not to cause shock or other disturbance.
3. By operation of a hydraulic ram of a suitable capacity attached to a point along the moment arm via a steel wire or articulated tie.
4. By manual application of force via a measuring device such as a load cell.

Other methods of load application may also be satisfactory provided the accuracy is comparable. In fact when the re-pointing and the jointing are weak the failure of the bond can be very sudden; in that case the load rate can be kept as low as possible by substituting the lead shots with a constant water. (Fig. 4.06) illustrates the principle of the bond wrench applied to re-pointing. (Binda et al., 2005)

- clamping device:

Clamping device is required which can firmly clamp the unit one down from the top of the prism whilst not applying any significant bending moment to the joints below. In order to calibrate the equipment determine the mass of the wrench to ±10g, the distance from the inside edge of the outer clamp face to the loading notch to within ±2 mm and the distance from the inside edge of the outer clamp face to the centre of gravity of the wrench to within ±2 mm. The type with the driven mass will require the measurements to be made with the mass in the start position. Where a container is used it should be in place but empty; if a jack is used the wire should be attached to the arm but slack.

- Displacement measurement:

To enable the monitoring of the strength of pointing when its bond strength is very low, 4 displacement transducers can be applied on the clamping devices near the area where the highest tensile stresses are applied corresponding to the most external point of the pointing and at the end of jointing so that the displacements of these two important points can be measured.

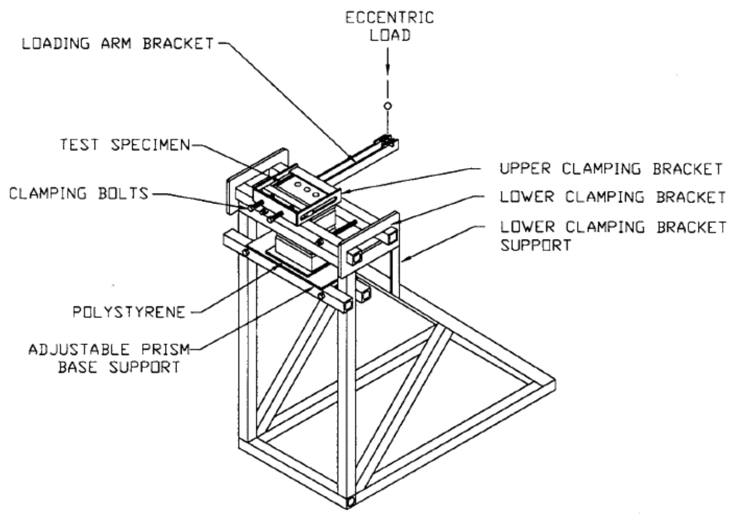

Figure 4.06: Bond Wrench Testing Apparatus (ASTM C1072, 2013).

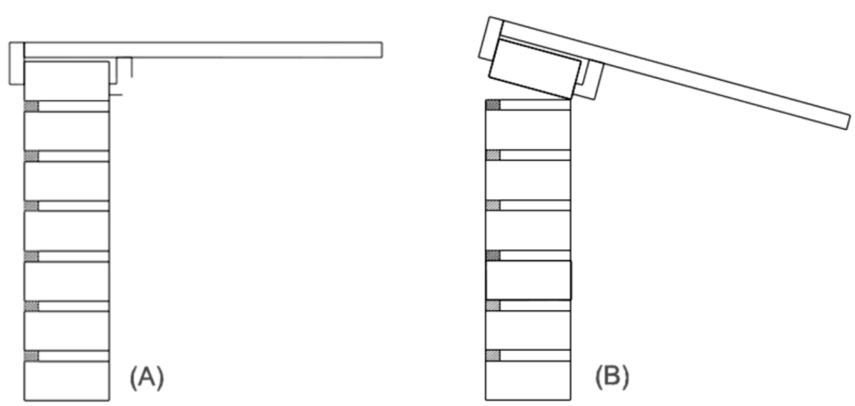

Figure 4.07: Basic principle of bond wrench test a) before and b) after the loading (Park, 2013).

Procedure

- Specimens Sampling:

In situ specimens

1. The sample shall have at least five bed joints, and contains one or more prisms.
2. Where mortar fins and extrusions project from the specimen to the extent that they may interfere with the attachment of the bond wrench, they shall be removed without causing damage to the specimen. Remove only enough material to enable proper attachment of the bond wrench.

Laboratory specimens

1. Fabricate a set of stack-bonded test prisms containing a total of not less than 15 mortar joints. Each prism shall have no more than 5 joints.
2. Proportion mortar materials by weights equivalent to volume proportions to be used in prism construction. Then mix mortar in a mechanical paddle-type mortar mixer.
3. Fabricate prism specimens.
4. Cure prism specimens.

Test Procedures:

The procedures according to (ASTM C1072, 2013) as follows:

1. Place the prism vertically in the support, and clamp firmly into a locked position using the lower clamping bracket. Orient the prism so that the face of the joint intended to be subjected to flexural tension is on the same side of the specimen as the clamping bolts. The prism shall be positioned at the required elevation that results in a single brick projecting above the lower clamping bracket as shown in (Fig.4.08).
2. Attach the upper clamping bracket to the top unit. Tighten each clamping bolt using a torque not greater than 5.7 N·m.
3. Lower base support away from the bottom of the prism so that no contact occurs during testing.
4. Apply the load at a uniform rate so that the total load is applied in not less than 1 min or more than 3 min. Measure load to an accuracy of 62 % with maximum error of 22 N.

Figure 4.08: Bond Wrench test, loading stage with displacements measurements transducers (Binda et al., 2005).

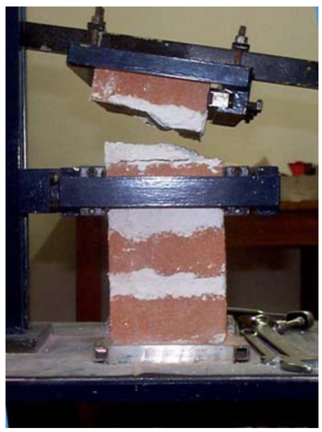

Figure 4.09: Bond Wrench test, failure after reaching the ultimate bond strength (Binda et al., 2005).

Data Analysis

- the flexural strength as given by (Park, 2013):

$$F_g = \frac{6(PL + P_I L_I)}{bd^2} - \frac{(P + P_I)}{bd}$$

where:

F_g = Flexural tensile strength, MPa

P = Maximum applied load, N

P_I = Weight of loading arm, N

L = Distance from center of prism to loading point, mm

L_I = Distance from center of prism to centroid of loading arm, mm

b = Cross-sectional width of the mortar-bedded area, measured perpendicular to the loading arm of the upper clamping bracket.

d = Cross-sectional depth of the mortar-bedded area, measured parallel to the loading arm of the upper clamping bracket.

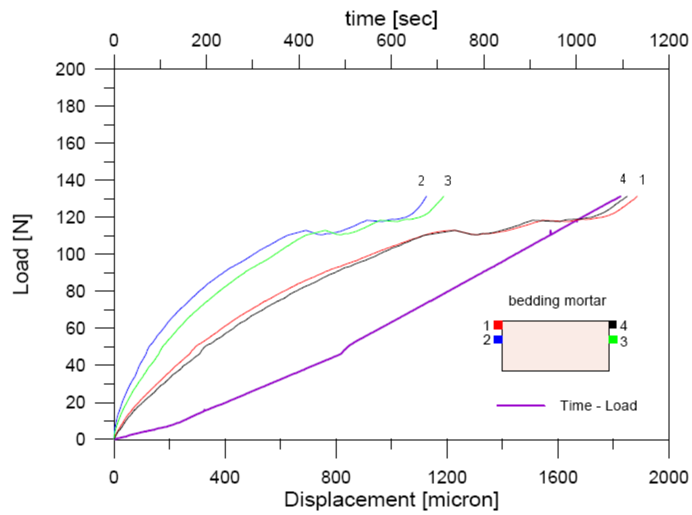

Figure 4.10: Load/displacement plot for the measured points with respect to time (Binda et al., 2005).

Conclusion

The principal of performing the bond wrench test is to determine the failure of full-scale bricks prisms, and how could be explained by a difference in bond strength developed in between the mortar and brick. Bending failure may occur either about a plane parallel to the bed joints, about a plane perpendicular to the bed joints, or both. There are two methods commonly employed to establish the flexural tensile strength of masonry. The apparatus includes a wrench to apply a bending moment, a clamping mechanism to transfer the moment to the specimen and a supporting frame. The masonry prism is fastened vertically in the supporting frame. The lever arm length is not specified, and does not seem to have any dissemble effect on the various strain gradients produced.

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4.1.4 In situ masonry unit strength by using rebound hardness

Scope

This technique is known as the Schmidt Rebound Hammer to provide a measure of relative material surface hardness without having any major destructions. Hardness is considered as the resistance of the masonry surface to localized deformation, where the measurements are widely used for the quality control of materials because they are quick and considered to be non-destructive tests when the marks or indentations produced by the test are in low stress areas.

Apparatus

A Schmidt Rebound Hammer is shown in (Fig. 4.11). This kind of hammers are available in four basic varieties, distinguished primarily by their impact energy:

- Type L (impact energy =0.075 kgm).
- Type N (impact energy = 0.225 kgm).
- Type M (impact energy = 3 kgm).
- Type P (pendulum type, impact energy = 0.09 kgm).

A type L hammer is recommended for use with most types of masonry, especially older or soft masonry.

The Schmidt Hammer is calibrated against a hardened steel test anvil, supplied by the manufacturer for that purpose. Regular calibration of the device shall follow a schedule based upon the manufacturer's recommendations. When the calibration rebound value deviates from the required value, a correction formula provided by the manufacturer may be used to modify the measured values to the correct value (RELIM Standards, 1998).

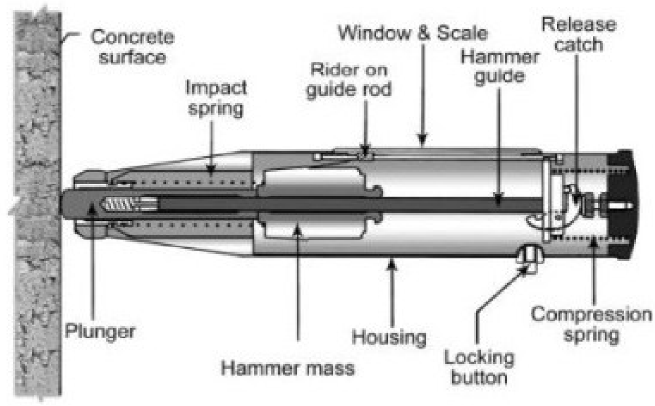

Figure 4.11: Rebound Hummer (Spectro Labs, 2011).

Procedure

The procedures of using Schmidt Hammer test as states in (RELIM MS.D.2, 1998):

- The Schmidt Hammer test should be conducted on masonry units at three or more locations for each structural element.
- The tested unit should have no free edges, no visible cracks, and be surrounded on all sides by un-cracked mortar.
- The test is to be carried out with the hammer oriented normally to the masonry surface. If the hammer orientation deviates from a horizontal position, the angle of the hammer axis with respect to horizontal shall be recorded, and the results corrected to a horizontal position using correction curves supplied by the manufacturer. The point of impact shall be centred on the unit to be tested.
- The point of impact shall be smooth and free of dirt. Where the desired testing surface is not smooth, it may be ground smooth.
- Place the tip of the plunger on the surface of the masonry unit and impact 3-4 times to seat the plunger on the masonry surface.
- Record the rebound number from ten successive impacts without removing the tip of the hammer from the masonry surface.
- Destructive or in-situ tests may be conducted to correlate the rebound number to compressive strength. The correlation to strength is useful primarily for determining the expected relative change in compressive strength between locations with different rebound numbers.

- Locations for destructive tests are chosen in areas that represent the full range of recorded rebound numbers. A minimum of four destructive tests are suggested to establish a reliable correlation.

Data Analysis

The rebound hardness for each test location shall be recorded as the mean of the five highest values from the ten successive impacts at each point. The standard deviation of the five test values shall also be reported. The variation of test results is as important as the computed means of results, and statistical tools for analysis of variance may be useful for interpretation of data. The mean and variance of rebound hardness values for the entire structure or structural element should be computed to aid in comparisons of relative material quality.

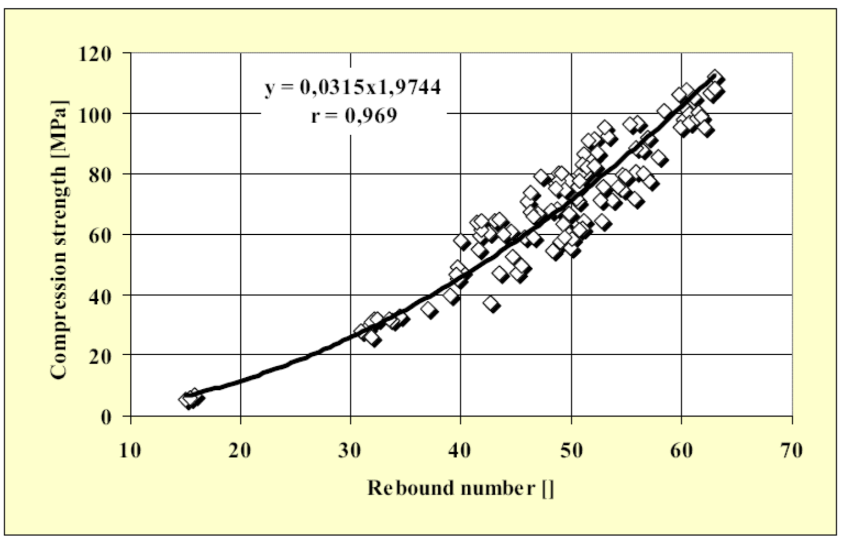

Figure 4.12: Schmidt Hammer Correction Curve (Brožovský et al., 2009).

Conclusion

The rebound hardness method is suggested only for determination of the uniformity of properties over a large area of a structure. But it may be used for prediction of masonry compressive strength only if correlated with results of controlled destructive tests conducted on masonry removed from the structure being evaluated or by in-situ compressive tests using flat-jacks. However, it is not recommended that the Schmidt Hammer be used for direct prediction of compressive strength. It evaluates only the local point and layer of masonry to which it is applied, and is unreliable for detection of subsurface flaws or for investigation of inaccessible masonry leafs. It may be suitable for detecting near surface delamination due to frost or salt action of units and stonework.

References

LABS, S. (2011). Rebound Hammer Testing Services. Accessed April 17, 2014. http://www.spectro.in/Rebound-Hammer-Test.html.

RILEM MS.D.2. 1998. " Determination of masonry rebound hardness." Materials and Structures, July : 375-377.

BROŽOVSKÝ, J., PROCHÁZKA, D., & BENEŠ, D. (2009). "Determination Of High Performance Concrete Strength By Means Of Impact Hammer." The 10th International Conference of the Slovenian Society for Non-Destructive Testing September 1-3, 2009, Ljubljana, Slovenia. Brno University of Technology, Faculty of Civil Engineering, Technology Institute of Building Materials and Elements. 233-241.

4.2 Testing techniques for mortar

These test techniques provide various testing procedures commonly used for evaluating mechanical characteristics of masonry mortar (Fig. 4.13).

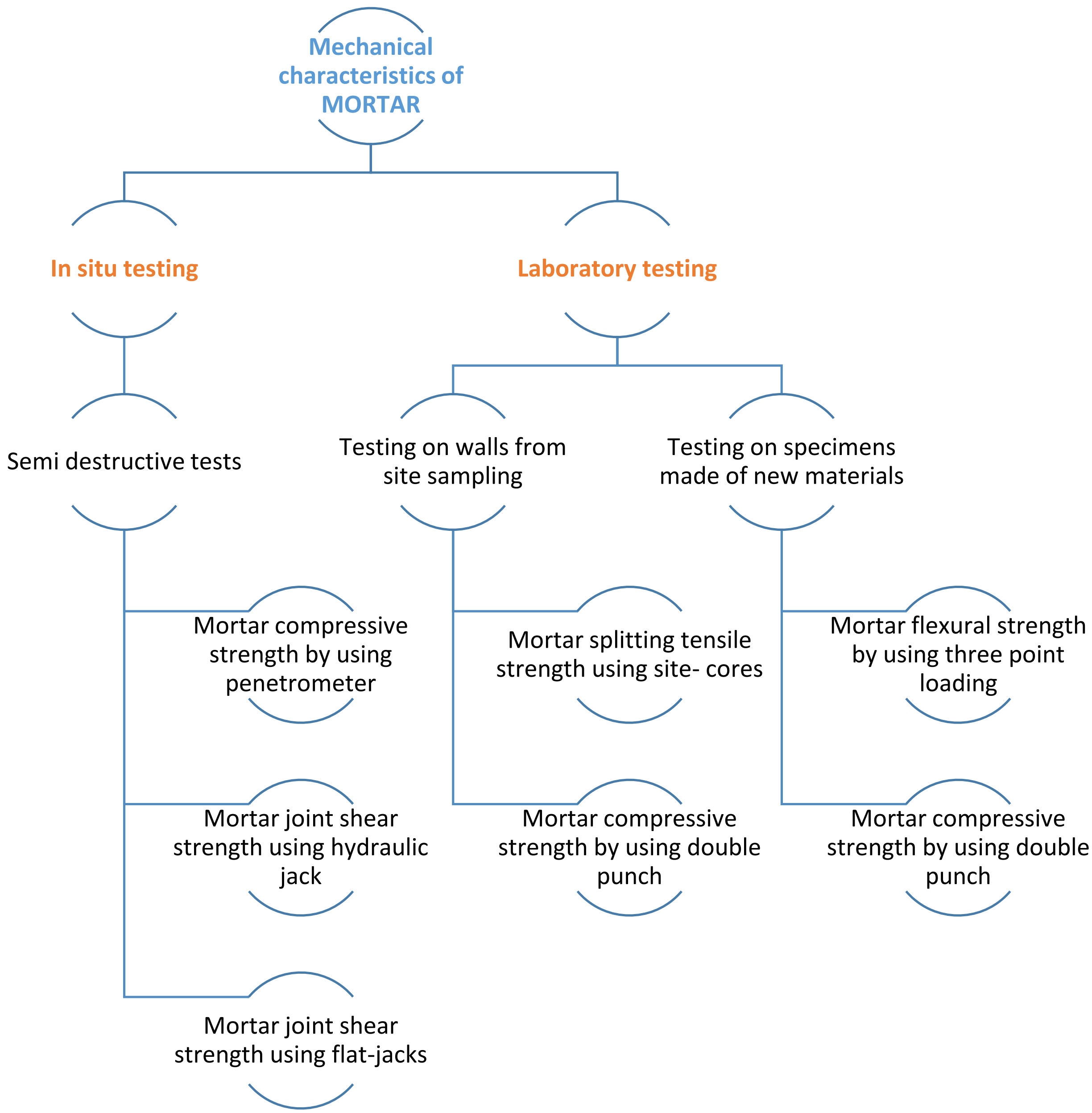

Figure 4.13: Techniques for determining the mechanical properties of mortar.

4.2.1 Load capacity for in situ mortar joint using penetrometer

Scope

The load capacity of mortar bed joint can be determine by using penetrometer, which is considered as in situ semi destructive technique. As (Gucci And Barsotti, 1995) have descired the main princpele of the test which is based on the measurement of the amount of energy required to drill a small cavity in a mortar joint. Where the test designed to find the correlation between the drilling work and the mortar compressive strength.

Apparatus

- The penetrometer device consists of hand drill connected to an ordinary 9.6 volt battery-operated with a suitable electronic circuit which is integrated to a digital screen for showing the results, (Fig. 4.14). Experimental investigations by (Gucci And Barsotti, 1995) showed that penetration energy is correlated with compressive strength, for sand mortars with compressive strength up to 4 MPa. So a different penetration device is purposed for historical masonry mortars, which are often decayed and without cohesion. It estimates the friction coefficient through the penetration of a metal pin driven by multiple strokes.

Figure 4.14: PNT-G penetrometer (Benedetti and Pelà, 2012).

Procedure

Test Preparation

- Prepare the tested masonry element by removing the plaster layer in order to have a direct contact between the penetrometer and the mortar.
- The standard apparatus uses 4mm diameter bits. Which is the minimum joint thickness in excess of the drill diameter. To avoid mechanical interaction between the drill and the

masonry unit materials the hole should be centred in the joint with width should be 8mm or greater.

- The Mortar joint must have reached a sufficient degree of resistance to penetration so that the drill does not penetrate to a depth greater than the exposed length of the pin when inserted into the hammer of the driver.
- Penetration shall not be located less than 50 mm or more than 150 mm from any other pin penetration, nor less than 50 mm from the edge of a masonry surface.
- The tests must be carried out in dry weather conditions.

PNT-G Test Procedures

- Collect the device and turn it on then reset the digital screen to zero.
- Load the spring driver unit by trigger the running button for three times.
- Place the driver unit firmly against the perpendicular to the surface of the mortar to be tested. Then trigger to drive the pin into the mortar joint surface.
- Hold triggering meanwhile the pressure during drilling has virtually no influence on the measurement then the record of the drilling work stops automatically when the bit reaches the standard depth which is around 5 mm with 4 mm diameter, and the value obtained is shown by a digital display.
- Repeat the same procedures for the other selected points.
- Clean the drill holes using the air blower device and repair it.

Data Analysis

It is sufficient to calculate the mean of the ten or more measurements, where the data is used to monitor quality variation of mortar joint to follow changes due to hardening or cyclic actions in durability testing. It is to get an indications of equivalent strength properties such as cube strength, flexural strength or tensile strength, but it is required to transform the data by using a suitable calibration curve.

Calibration indicates that the relationship between drilling energy and strength properties is not linear and therefore each individual measurement must be transformed before the mean is calculated.

Conclusion

The mechanical strength of the mortars has not an easy and direct correlation with respect to the in situ testing techniques. Because the mechanical properties of the mortars arise from physical parameters such as density, porosity, specific surface area and pore size distribution but also from the textural characteristics, with specific reference to the binder texture, the qualitative and quantitative ratios between the mortar components, the grain size and shape of the aggregate, as well as from its mineralogical composition. So generally this test allows to identify compressive strength with respect to the mechanical response of the investigated mortar, which could be used as an index of resistance for the mortar qualification.

References

ASTM C803–03. (2003). "Standard Test Method for Penetration Resistance of Hardened Concrete." ASTM standards, 5.

BENEDETTI, A., & PELÀ, L. (2012). "Experimental Characterization Of Mortar By Testing On Small Specimens." Proceedings of 15th International Brick and Block Masonry Conference. Florianópolis – Brazil. 10.

CALIA, A., LIBERATORE, D., & MASINI, N. 2013. "Approach to the study of conservation of historical masonry mortars by means of the correlation between porosimetry and penetrometric test." Built Heritage 2013 Monitoring Conservation Management, 1133-1141.

GUCCI, A. , & BARSOTTI, R. (1995). "A non-destructive technique for the determination of mortar load capacity in situ." Materials and Structures,28: 276-283.

4.2.2 In situ mortar strength by pull-out test

Scope

Pull out test is an in-situ method of testing jointing and pointing mortars. The technique can give useful characteristics data such as strength variability of mortar, variation of quality in relation to a reference sample, changes of properties with time, i.e. strength increases due to hardening and the effects of weather conditions and additives, and absolute values of mortar cube strength, provided a suitable calibration data-base.

Apparatus

The Helifix 6 mm diameter Retro-tie is the basic apparatus to be used together with a driving tool. Other equipment required:

- A 4.5 mm diameter drill bit fitted to a domestic capacity hammer-action drill to form the pilot hole.
- A tie gripper system and a pull-out force measuring unit.
- The reaction points or ring on the pull-out system should be sufficiently far from the axis of the tie to ensure that the pull-out mechanism is not influenced.

Procedure

- Select positions within an area not exceeding 2 m^2, and at each position drill a 4.5 mm diameter hole in the middle of the thickness of the mortar bed. Reject holes where the hole is significantly off-centre and too close to the unit, or where the drill drops into an obvious air void before reaching at least 30 mm depth. Add new locations if necessary until at least ten positions have been prepared. Mark each position with a number and identify the area using chalk or a marker pen, then take a record photograph or make a sketch of the layout.
- Mount a helical tie into the driving tool and then, holding the sleeve of the tool horizontal, push the exposed end of the tie gently into the pilot hole.
- Hammer the helical tie firmly, but not violently, into the hole so that the specified length of its thread (L) is embedded in the mortar using the sleeved driving tool shown. This allows the tie to rotate and cut a thread in the mortar during insertion and is dimensioned to install the correct length. (The ties have a lead-in pointed tip about 5 mm long to facilitate theadcutting, thus the total depth of insertion is circa L + 5mm). Note: experience has shown that L = 30mm gives the best results for homogeneous mortar with no pointing layer.
- After installation, a gripper is then screwed onto the end of the tie. This holds the tie fixed during the test, restraining it from rotating, and ensures a shear type failure in the test material.
- The proof loading device is then attached to the gripper and the assembly is rotated to screw down the tie and take up any slack. The tie is loaded until failure. The load applied to the tie should be increased steadily. The peak load reached during a test is held by the dead needle on the dial, and is recorded as the pull-out load. Two distinct types of reaction frame can be used: one with a number of individual legs that react against the test material, and the other with a steel ring providing the contact with the material's surface. The type with legs tends to be lighter and easier to use and can be left attached to the test unit. The reaction

points or ring should be at least 30 mm, and ideally 50 ram, away from the axis of the tie to avoid restraining the mortar under testing.

Data Analysis

Where the data is used to indicate/monitor quality variation, to follow changes due to hardening or cyclic actions in durability testing, it is sufficient to calculate the mean of the ten (or more) measurements. If absolute indications of equivalent strength properties such as cube strength, flexural strength or tensile strength are required, the data must be transformed using a suitable calibration curve. Previous calibration trials indicate that the relationship between pull-out force and strength properties is not linear, thus each individual measurement must be transformed before the mean is calculated. Examples of calibration data for cube strength are given in the Bibliography.

Conclusion

Thus, on building sites the method has application both for quality control purposes and as a means for trouble shooting. In the laboratory, it is useful for control of hardness development during curing and hardening, or as a monitoring/control parameter for other tests such as acid rain and freeze-thaw tests.

The method is limited by the yield strength of the helical steel screws used to a maximum strength of approximately 8 N/mm 2. Above this strength, the failure is by yield of the steel and the test value is no longer proportional to the strength of the mortar. Thus, for stronger mortars the technique is only suitable for proof testing. The method is not suitable for mortar beds less than 8 mm thick.

References

RILEM: MS-D.9. (1997). "Determination of mortar strength by the screw (helix) pull-out method." Materials and Structures, July: 323-328.

4.2.3 Shear strength of masonry mortar joint by single flat-jack

Scope

This test technique can be applied on both regular and irregular masonry units to measure shear strength capacity for masonry mortar joint, by removing the head joints of both ends of the tested unit, and applying single flat-jack in one end to produce a local horizontal force while the other end remains free for the horizontal deformation.

Apparatus

- Flat-jack

Using the same flat-jack mentioned before in section (single flat-jack), but taking in consideration that the flat-jack bladder should be a small rectangular one, that fits slot of the head joint of the tested masonry unit. Also the dimensions of flat-jack should not exceed the dimensions of the tested unit to prevent applying load to the mortar joints itself.

- The rest apparatus used in this test are the same mentioned in section (single flat-jack).

Procedure

It is important to choice the right place for the test, in which areas with nonparallel mortar joints must be avoided. And the tested unit should be in the stretcher orientation. Also the location of the test must be far enough from wall openings or ends, because the applied horizontal force by flat-jack must act against enough masonry to resist forces generated during the test. (Simões et al., 2012)

- State of Normal Compressive Stress

In case of using single flat-jack for measuring shear strength of masonry mortar joint, the estimated normal compressive stress on the tested unit can be based upon the location of the test unit with respect to dead and live loads acting in the structure or can be controlled by double flat-jack at the top and bottom of the test location.

- The Slots Preparation

According to (ASTM C 1531, 2009), the procedures are as follow:

1. Measure the dimensions of tested unit (width and length) with an accuracy of 0.8mm.
2. Remove the head joints on both ends of the tested masonry unit. Taking in consideration that the mortar bed joints above and below the tested masonry unit must not extend beyond the vertical ends.

3. In case of irregular masonry units, the specimen to be tested under shear load applied by flat-jack, is delimited by to vertical cuts with a distance of 350 mm.
    - Applying Flat-Jack

According to (ASTM C 1531, 2009), the procedures are as follow:

1. Insert the flat-jack into the head joint at one end where it must be centred on the tested masonry unit. Then apply a small amount of pressure enough to let the flat-jack seat in the joint, as shown in (Fig.4.15).
2. Increase gradually the pressure on the tested until, until a slip occurs or crack appears. At that moment record the indicated pressure by flat-jack gauge, as shown in (Fig.4.16).
3. Release the pressure, and remove flat-jack.
4. Fill the slots and cracks, using the original or similar materials.

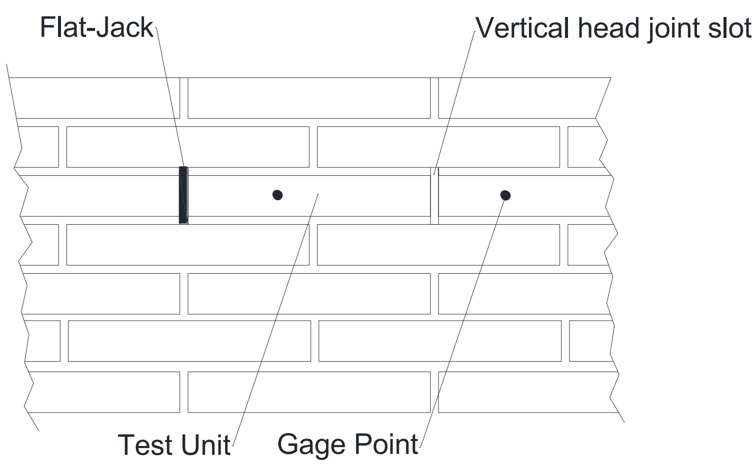

Figure 4.15: Flat-jack setup for mortar shear strength test.

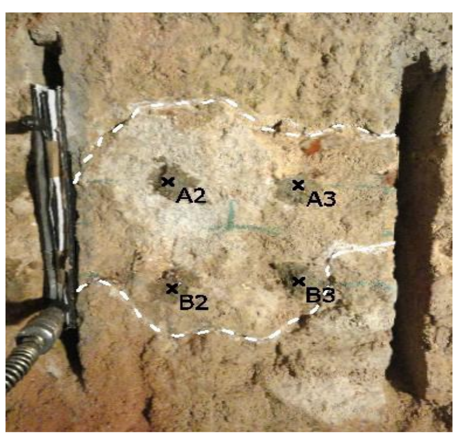

Figure 4.16: Flat-jack shear test on irregular masonry wall (Simões et al., 2012).

Data Analysis

The masonry wall shear strength is limited by shear of the mortar joints rather than shear through the masonry units.

- The horizontal force is calculated as follows:

$$P_h = K_m . A_f . p$$

$K_m =$ Dimensionless constant which reflects the geometrical and stiffness properties of the flat-jack, as determined in the calibration section.

$A_f =$ Area of the flat-jack,

$p =$ Applied pressure by flat-jack at initial crack or unit slip.

- Calculate the shear strength index of mortar bed joint τ, as:

$$\tau = \frac{P_h}{A_j}$$

where:

$P_h =$ Maximum horizontal force resisted by the tested masonry unit.

$A_j =$ Gross area of bed joint.

- The shear strength index τ, is reduced to the value that would have been obtained under zero axial load to, using the relation

$$\tau_0 = \tau - \mu(\sigma_v)$$

where:

$\mu =$ Coefficient of friction for the masonry unit.

$\sigma_v =$ Estimated normal compressive stress at the tested unit.

The value of μ varies from 0.3 to 1.6, with a mean value of 1.0 and coefficient of variation of approximately 30 %. (ASTM C 1531, 2009).

Conclusion

The existent standards regarding shear tests with flat-jacks (ASTM C 1531, 2003) and (RILEM MS-D.6, 1996) were developed for regular block masonry where the slots cross the entire thickness of the wall completely, isolating the specimens from the remaining masonry. So due to the great thickness of the traditional masonry walls the slots do not cross the entire thickness of the wall resulting in different boundary conditions, which have a major influence on the test results. On the other hand this method can be applied in case of irregular masonry walls as shown in (Fig.4.16), which consider as advantage comparing with using hydraulic jack instead of flat-jack.

References

ASTM C 1531 – 09. (2009). Standard Test Methods for In Situ Measurement Of Masonry Mortar Joint Shear Strength Index. ASTM standards, 7.

ROSSI, P.P. (1990). Non Destructive Evaluation Of The Mechanical Characteristics Of Masonry Structures. Boulder, Online Publication. University of Colorado.

SIMÕES, A, GAGO, A., LOPES, M., & BENTO, R. (2012). "Characterization of Old Masonry Walls: Flat-Jack Method." Proceedings of 15 WCEE, IST, Technical University of Lisbon, Portugal, 10.

RILEM: MS-D.6. (1996). "In situ measurement of masonry bed joint." Materials and Structures 29: 470-475.

4.2.4 Shear strength of masonry mortar joint by hydraulic jack

This test technique used for measuring the shear strength for masonry mortar joint, which can be applied just on regular masonry units, by inserting the hydraulic jack instead of one masonry unit, where the idea is determining the horizontal force required to produce the relative slip of a brick with respect to the rest of the panel and causes a movement or initial cracking of the tested unit.

Apparatus

- Sawing tool

Sawing tools are needed to prepare the testing area by removing the wall plaster and masonry units where the hydraulic jack takes a place, so it is important to use a limited and accurate tools which don't have any vibrations or distributions effect upon the structural element, such as multiuse 1500 watt electrical hummer, as shown in (Fig.4.17).

Figure 4.17: Removing plaster by electrical hummer as preparing phase of shear test.

- Hydraulic jack

Hydraulic jack equipped with a pressure gauge graduated in increments of 345 kPa or less shall be used. The jack load shall be applied at a rate not exceeding 22 kN per minute. as shown in (Fig.4.18).

Figure 4.18: Hydraulic Jack placed instead of the removed masonry unit.

- load cell

Load cell gives a measure to the applied force by the hydraulic jack, and it should be place between the spherical seat and the bearing plate neat to the test masonry unit.

- hydraulic pump

The pressure in the jack is provided by using an electrically or manually hydraulic pumps. And it is measured by gauges which calibrated to have an accuracy of 1% of full hydraulic scale. The hydraulic system must be able to maintain a constant pressure for a minimum of 5 minutes.

- Steel bearing plates

Bearing plates are used for distributing the applied force comes from the jack uniformly to the tested unit. The plates have a minimum thickness equal to 1⁄8 its vertical dimension. Also it must be placed centered to the end and less than the height of the unit by 3.2 mm, with horizontal dimension equals to the units thickness (ASTM C 1531, 2009).

- Displacement Measurement

Mechanical gauge extensometer or LVDT's can be used to obtain the masonry displacements, by measuring the distance between fixed gauge points as shown in (Fig.4.19), according to (RILEM LUM.D.5, 2004), the accuracy should be 0.1% of the gauge length, as (ASTM C 1196, 2009) requires ±0.005% of gauge length. In any case, the deformation measurements should be up to 5 mm with an accuracy of at least 0.005 mm.

Figure 4.19: Test setup for shear test by hydraulic jack.

Procedure

It is important to choice the right place for the test, in which areas with nonparallel mortar joints must be avoided. And the tested unit should be in the stretcher orientation. Also the location of the test must be far enough from wall openings or ends, because the applied horizontal force by hydraulic jack must act against enough masonry to resist forces generated during the test. (Simões et al., 2012)

- State of Normal Compressive Stress

In case of using hydraulic jack for measuring shear strength of masonry mortar joint, the estimated normal compressive stress on the tested unit can be based upon the location of the test unit with respect to dead and live loads acting in the structure.

- The Preparation

One masonry unit must be removed from one side of the tested unit with the head mortar joint to the other side, or in some cases it is possible to remove the whole unit to the other side of the test.

Taking in consideration the above and below mortar bed joints shall not extend beyond the vertical ends of the unit being tested.

- Applying Hydraulic jack

According to (ASTM C 1531, 2009), the procedures are as follow:

1. Sit the hydraulic jack and bearing plates into the space where the unit has removed next to the tested unit. Increase pressure applied by the jack gradually until the tested unit starts to displace continually under a constant level of horizontal load.
2. Record the maximum load indicated by the pressure gage or load cell.
3. Release the pressure and remove the jack.
4. Rebuild the removed unit and fill the cracks, using the original or similar materials.

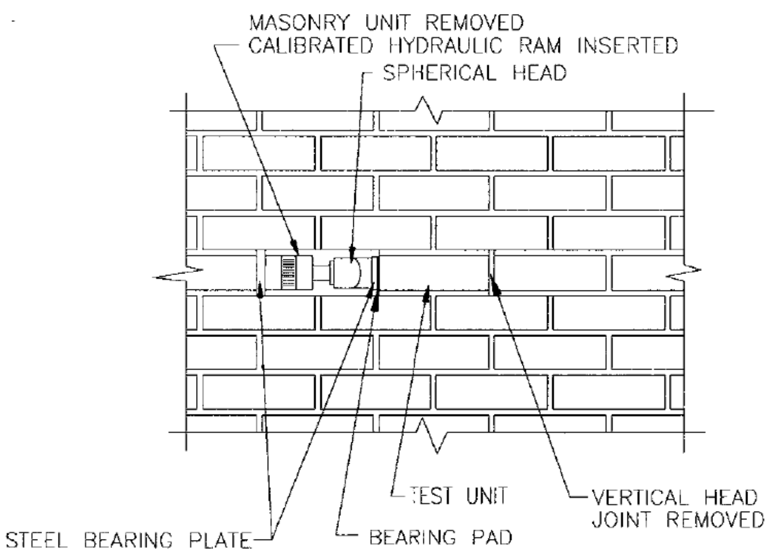

Figure 4.20: Hydraulic jack setup for mortar shear strength test. (ASTM C 1531, 2009)

Data Analysis

The masonry wall shear strength is limited by shear of the mortar joints rather than shear through the masonry units.

- Calculate the shear strength index of mortar bed joint τ, as:

$$\tau = \frac{P_h}{A_j}$$

where:

P_h = Maximum horizontal force resisted by the tested masonry unit.

A_j = Gross area of bed joint.

- The shear strength index τ, is reduced to the value that would have been obtained under zero axial load to, using the relation

$$\tau_0 = \tau - \mu(\sigma_v)$$

where:

μ = Coefficient of friction for the masonry unit.

σ_v = Estimated normal compressive stress at the tested unit.

The value of μ varies from 0.3 to 1.6, with a mean value of 1.0 and coefficient of variation of approximately 30 % (Rossi, 1990).

Conclusion

Measuring shear strength of masonry structures by using hydraulic jack, is relatively straight forward in situ technique, without having any major destruction. But it is important during the producers of the test to locate the jack in the centre of the unit, and applying the load to steel plates which cover the whole unit surface and uniformly distribute the load in order to prevent any failure in unit due to dispositioning the jack and cause punching. This in situ testing technique has the advantage of being less destructive, less time-consuming and more economical than direct shear-compression or diagonal compression tests on masonry portions. However, its suitability is limited to those cases where masonry wall shear strength is governed by shear strength of the mortar joints rather than shear capacity of the bricks.

References

ASTM C 1531 – 09. (2009). Standard Test Methods for In Situ Measurement Of Masonry Mortar Joint Shear Strength Index. ASTM standards, 7.

ROSSI, P.P. (1990). Non Destructive Evaluation Of The Mechanical Characteristics Of Masonry Structures. Boulder, Online Publication. University of Colorado.

SIMÕES, A, GAGO, A., LOPES, M., & BENTO, R. (2012). "Characterization of Old Masonry Walls: Flat-Jack Method." Proceedings of 15 WCEE, IST, Technical University of Lisbon, Portugal, 10.

RILEM TC 177-MDT. (2004). "RILEM Recommendation MDT. D.5 - In-situ stress -strain behaviour tests based on the flat jack." Materials and Structures 37: 497-501.

RILEM: MS-D.6. (1996). "In situ measurement of masonry bed joint." Materials and Structures 29: 470-475.

4.2.5 Compressive strength of mortar by double punch test

Scope

Mortar has a fundamental influence on masonry compressive strength, which responsible for creating a uniform stress distribution and smoothing the face of the masonry units. The scope of this test is determining the compressive strength of mortar in the joints. The only requirement is that an entire piece of joint can be sampled, where the determination of mortar compressive strength is important to evaluate the load bearing capacity for existing masonry structures, the idea of double punch test is loading small mortar samples by two circular steel plates until having the failure mode. (Sassoni and Mazzotti, 2013)

Apparatus

- Laboratory sample preparation:
  1. Weighting devices: for evaluating the mass of mortar components.
  2. Glass graduates: for measuring water quantity.
  3. Specimen molds: tight fitting 50 mm cube specimens.
  4. Mixer: electrically or mechanical mixer.
- Site sample preparation:
  1. Coring machine
  2. Electrical saw.

- Testing machine:

Any mechanical or hydraulic press able to carry out compression tests at controlled rate of loading. Two special punching devices with a contact surface diameter of 40 mm for specimens of 80x80 cm or of 20 mm for specimens of 50x50 cm. mainly consists of two cylindrical steel plates of smaller diameter, prepared concentrically superficially and below the specimen, as shown in

(Fig.4.21). The machine should be provided with load gage in order to get the reading at the moment of having failure of the sample.

Figure 4.21: Double punch testing machine.

The applied load through the steel plates produces a conical zone of compressions as shown in (Fig.4.22). This situation originates an increase of the diameter of the cylinder producing perpendicular tensile stress to the radial lines of the specimen. When the tensional state exceeds the mortar strength the fracture of the same takes place (Woods, 2012).

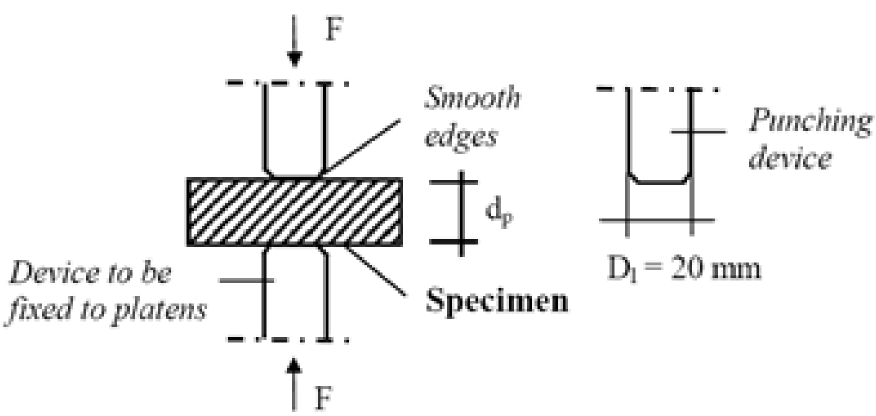

Figure 4.22: Loading schematic for double punch test (Woods, 2012).

Procedure

Mortar specimens can be sampled in-situ in case of determining the compressive strength of the mortar in an existing masonry, or prepared in laboratory, when the results can be used to compare and qualify new mortars for joint repair. The size of the specimen should be full joint thickness.

The length and width should be 100x200mm for new specimens, as much as possible near to these dimensions for the sampled ones (Binda et al., 2005).

On-site sampling of specimens:

- Samples should be taken by extracting two bricks or stones connected with a bed joint. The joint should have a certain thickness and consistency so that the two elements can be separated without spoiling the joint.
- Cored samples should be taken perpendicular to the bedding.
- Smaller specimens can be obtained, at least 50 x 50 mm.
- Sampling should preferably be carried out in dry conditions. If specimens are wet, they must be dried at 40 °C until constant mass is reached, with a variation of ±0.1% of the mass, within 24 hours.

Laboratory specimens:

- The specimens should be prepared between two bricks of the same type as those which will be used on site. In order to allow that after curing, the mortar bed to be separated from the bricks, during the preparation a gauze should be put between the brick surfaces in contact with the joint and the joint itself.
- Care should be taken to ensure that the units are in a moisture state appropriate to the objective of the tests and that the consistence of the mortar is properly adjusted to the state of the units.
- Immediately after building, each specimen shall be pre-compressed with dead weights to a level of 400 Kg/m^2.
- After the required curing time for mortar joints to harden, the joint should be separated from the bricks and the specimen cut in the right dimension for the test.
- The specimen can be tested dry or water saturated up to constant mass. Before testing the upper and lower surfaces of the specimens should be made smooth and plane by polishing them carefully or eventually by adjusting two thin smooth layers of gypsum (Binda et al., 2005).

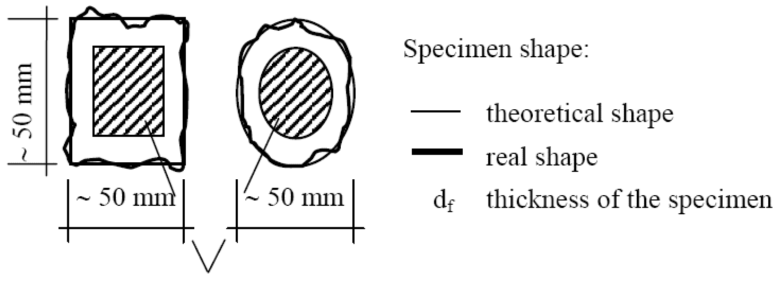

Figure 4.23: Shape and dimensions of the specimens (Binda et al., 2005).

Preparation of laboratory mortar specimens:

- Apply a thin oil coating to the interior faces of the molds.
- The proportions of mortar materials should be according to the mix design.
- The quantities of materials to be mixed at one time.
- Complete the consolidation of the mortar in the molds.
- Sorting the samples in water at temperature of [23 ± 2 °C]

Preparation of site mortar specimens:

- Removing the plaster of the wall.
- Using coring machine or electrical saw in order to have mortar samples.
- Take out the mortar joint from the core, then apply a gypsum smooth face on both faces of the sample, in order to have a normal distribution of the applying load.

Figure 4.24: Preparation of mortar samples for double punch test.

Determination of Compressive Strength:

- Place the specimen in the double punch test below the centre of the upper bearing block on the steel plate, as shown in (Fig.4.25).
- Apply the load to specimen faces that were in contact with the true plane surfaces of the mold.
- Obtain the load at the moment of failure.

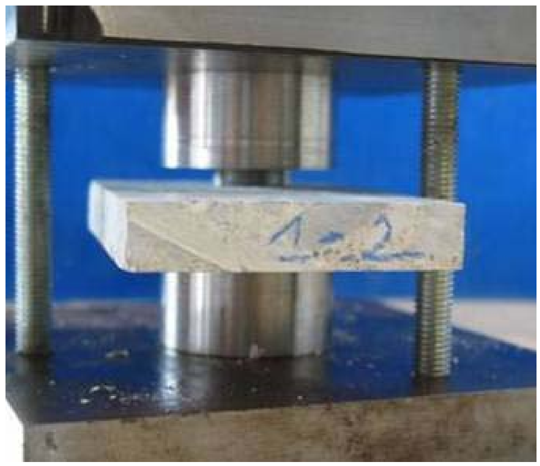

Figure 4.25: Punch test setup on laboratory mad specimen (Di Tommaso, 2013).

Data Analysis

- The compressive strength determined as given by (Henzel and Karl, 1987):

$$f_{mdp} = \frac{P}{\pi r^2}$$

$$f_m = 0.555 f_{mdp} + 3.068$$

Where:

p = Punching load, N

f_m = Compressive strength, MPa

Conclusion

The mechanical strength of the mortar is of importance for the interaction between the mortar and masonry brick interface. The strength of masonry is not directly proportional to the strength of the bedding mortar. Due to the dimension of the specimens, the value obtained is not a real compression strength, also the difficulty of carrying out standardized compressive strength tests on mortar joints that are usually very thin. The calculated strength can be used for structural analysis purposes or used for comparison with mortars for repair. However, it is important that these localized values should be representative of the whole structure.

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4.2.6 Splitting tensile test on cores with mortar joint

Scope

The splitting tensile strength test is one of the simplest tests which is performed to determine the compression and shear strength of mortar joint, by applying shear-compression stress to cylindrical cores with a mortar joint. The orthogonal planes to the mortar are subjected to pure shear, and the parallel one to shear-compression, which can be controlled in general by performing different inclinations of the mortar bed joint.

Apparatus

- Core drilling equipment:

Masonry core drilling equipment with relevant speeds for masonry structures, around 800 RPM for 100 mm core diameter. Should be fast, safe, quiet and not cause impact or vibration damage to the immediate surrounding structure.

Figure 4.26: The application of core drilling equipment.

- Testing Machine (loading device):

In order to apply and measure axial load on the specimen with sufficient loading capacity and rate conforming to the kind of the tested samples (60 ton). It shall be verified at suitable time intervals at a constant rate within the range 0.7 to 1.4 MPa/min splitting tensile stress until failure of the specimen. The upper, hardened metal bearing face shall be spherically seated and attached at the center of the upper head of the machine. The center of the sphere shall lie at the center of the surface of the plate in contact with the specimen, as shown below in (Fig.3.27) (ASTM D 3967, 2008).

Figure 4.27: Loading test machine.

Procedure

Specimen's preparation:

- In-situ Specimens

For determining the compression and shear strength of mortar joint, samples can be taken as 950 mm cores diameter on-site. Extracting them from the masonry wall. Sampling should be carried out in dry conditions, so minimum amount of water should be used during the coring. The (h/d) ratio of the specimen should be 2/1 or less (up to 1.5).

- Laboratory Specimens

Samples must be prepared in the laboratory. The masonry specimens must be cured and stored for the appropriate time as mentioned before (compression test \ wall element), then the coring can be done. (Binda et al., 2005)

In both in-situ and lanoratory specimens, general rules must be taken in consideration:

1. Specimens must be cylindrical cores with minimum radius of 25mm.
2. The circumferential surface should be smooth and straight to 0.50 mm.
3. The ends of the specimen must be parallel to each other and at right angles to the longitudinal axis.
4. Measure the diameter of the specimen to the nearest 0.25 mm, by taking the average of at least three measurements.
5. Measure the thickness of the specimen to the nearest 0.25 mm, by taking the average of at least three measurements.
6. In-situ sampled specimens must be in a minimum number of three, but for the prepared specimens in laboratory should be at least 6.

Figure 4.28: Samples preparation for Brazilian splitting test.

Test Procedures:

1. Ensure the right orientation of the specimen in the testing machine, by marking the vertical orientation of the specimen with respect to the mortar inclination angel, a strip of cardboard of the length of the specimen and 3mm thick is put on top and bottom of the specimen between machine platens and the specimen, in order to have a correct load application.
2. Position the specimen to ensure that the diametric plane of the two lines marked on the ends of the specimen lines up with the centre of thrust of the spherically seated bearing surface to within 1.25 mm.
3. Apply the compressive load to produce an approximately constant rate of loading range of 0.02 and 0.05 N/mm^2·sec. such that failure will occur within 1 to 10 min of loading (ASTM D 3967, 2008).

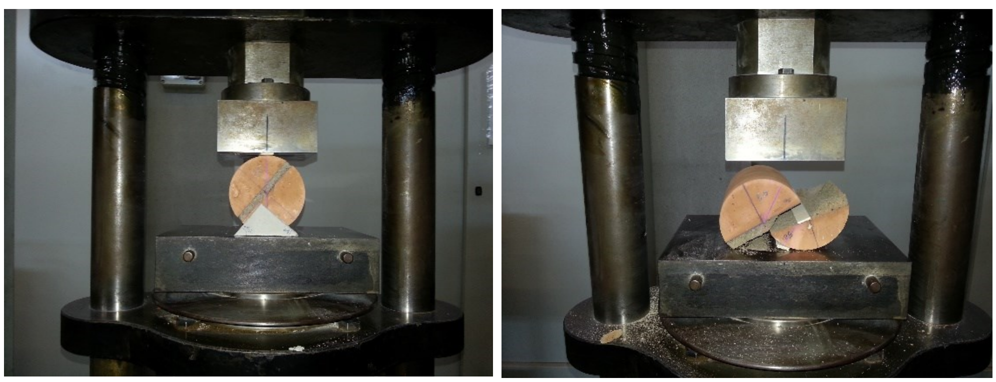

Figure 4.29: a) Placing the specimen in the testing machine. b) Failure mode of the specimen.

Data Analysis

- The stress values can be expressed as a function of the mortar joint inclination as follows:

$$f_c = \left(\frac{F}{A}\right) cos\alpha$$

$$f_v = \left(\frac{F}{A}\right) sin\alpha$$

where:

f_c = compression strength, MPa,

f_v = shear strength, MPa,

F = maximum applied load indicated by the testing machine, N

A = area of mortar layer, mm^2

α = the mortar layer inclination.

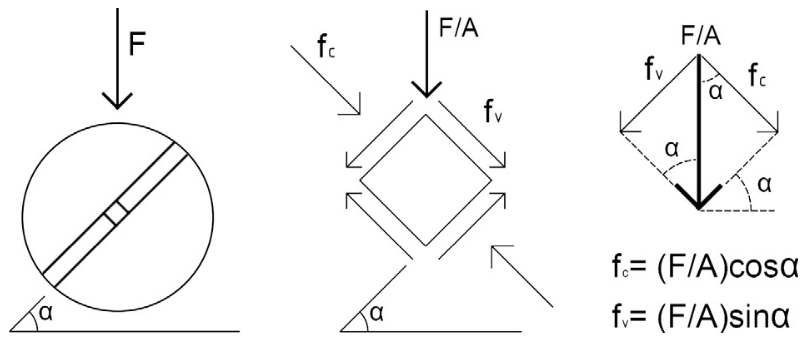

Figure 4.30: Compressive and shear stress with respect to different angles of mortar joint inclination, (Benedetti and Pelà, 2012).

Conclusion

This test is based on the idea that in a core made of two bricks and a central diametrical mortar joint, and subjected to a splitting test, with the mortar layer rotated by different angels with respect to horizontal, the stress state in the centre of the mortar joint will be a mixed compression–shear stress state, which resembles that of a masonry panel subjected to a diagonal compression test.

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4.2.7 Masonry flexural bond strength by three points loading test

Scope

Bond strength is one of the most important physical properties of masonry in design and evaluate the existing historical structures. It dictates the maximum tensile stress a masonry system can sustain. This test determine the modulus of rapture for a masonry prisms.

Apparatus

- The third-point loading method: The minimum span between supports shall not be less than 2.5 multiplied by the average depth of the specimen. The distance between each support and the adjacent distributed point load shall be one-third of the span length 63 mm. Steel rods with a maximum diameter of 25 mm shall be used to support the specimen and apply the load. The steel rods shall extend over the full width of the specimen and shall have the same nominal diameter, the loading method shown below in (Fig.4.31).

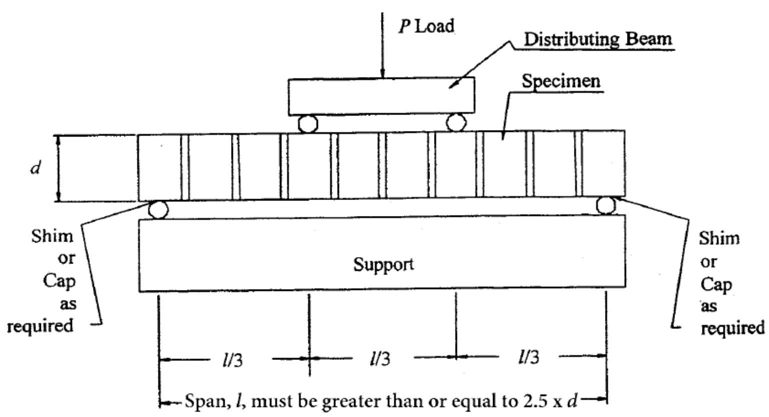

Figure 4.31: Third-Point loading test (ASTM E518, 2010).

Procedure

- Sampling:
    1. Masonry Units: Representative masonry units shall be sampled and tested by compressive strength test.
    2. Mortar: One of the types of mortar in Specification shall be used, or the mortar shall conform to that specified for the construction. Mortar shall be mixed to a workable consistency. The compressive strength, initial flow,

and water retention shall be determined in the same atmosphere as the prisms.

3. At least five test specimens shall be constructed as prisms, with 460 mm high with mortar joints (10 ± 1.5) mm in thickness. When the test is for the purpose of determining the quality of materials and workmanship during construction, the specimens shall be constructed at the site by the masons involved, utilizing the materials on the site and the same masonry construction techniques.
4. All prisms shall be cured for 28 days. The prisms together with corresponding mortar cubes shall be cured in laboratory air maintained at a temperature of 24 ± 8°C, with a relative humidity between (30-70) percent.
5. Where prisms are made during construction at the job site, they shall be constructed in a place where they will not be disturbed, but will be subjected to air conditions similar to those in the masonry structure.

Figure 4.32: Masonry prisms samples manufactured in laboratory (Park, 2013).

- Testing Procedure:

The procedure states in (ASTM E518, 2010) as follows:

1. Place the test specimen horizontally on its supports as a simply supported beam. If full contact is not obtained between the specimen and the load-applying blocks and supports, compressible shims or a bed of gypsum

capping material shall be used to level and seat the specimen thereby ensuring the uniform application of load, as shown in (Fig.3.33).

2. Apply the load at a uniform rate of travel of the moving head such that the total load is applied in not less than 1 nor more than 3 min.
3. Record the maximum applied load in Newtons as *P* and the location of the break.

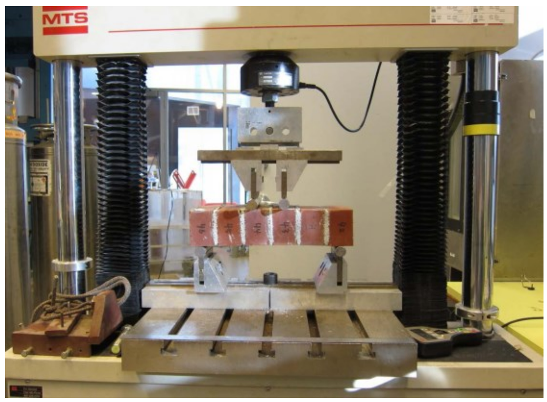

Figure 4.33: Third point test setup (Park, 2013).

Data Analysis

- Modulus of rupture can be calculated as states in (ASTM E518, 2010) :

$$R = \frac{(P + 0.75P_s)l}{bd^2}$$

where:

R = gross area modulus of rupture, MPa

P = maximum applied load indicated by the testing machine, N

Ps = weight of specimen, N

l = span, mm

b = average width of specimen, mm

d = average depth of specimen, mm

Conclusion

The three point bending flexural test provides values for the modulus of elasticity in bending, flexural stress, flexural strain and the flexural stress-strain response of the material. The main advantage of a three point flexural test is the ease of the specimen preparation and testing. However, this method has also some disadvantages: the results of the testing method are sensitive to specimen and loading geometry and strain rate.

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4.3 Testing techniques for masonry elements

These test techniques provide various testing procedures commonly used for evaluating mechanical characteristics of masonry element (Fig. 4.34).

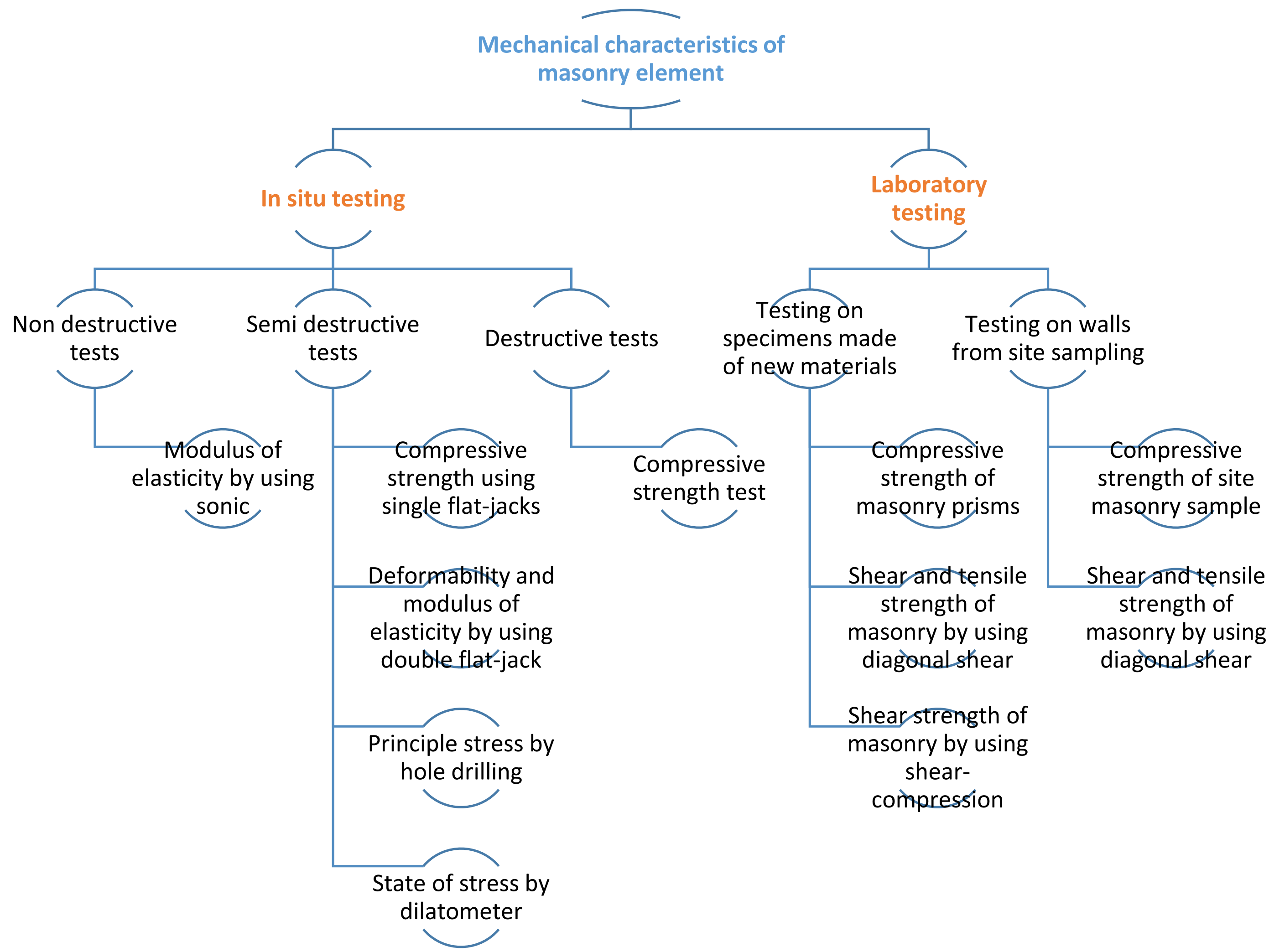

Figure 4.34: Techniques for determining the mechanical properties of masonry element.

4.3.1 Young modulus of masonry elements by pulse velocity tests

Scope

This technique is typical for non-destructive measuring of a quality parameter and its variation related to the density, stiffness, porosity, the isotropic/anisotropic characteristics and the presence of cracks and flaws (voids and fissures) (RELIM:MS-D.5, 1996). The basic concept of pulse velocity tests depends on the pulse waves moving through masonry elements. Where the velocity of pulse effected by the variation of material characteristics through measured area, which gives indication about the material quality index.

The velocity of pulse affected by many factors such as the quality of the materials, dimension of test sample, and the moisture condition. In case of humid materials, it is fast, where the velocity of wave decreases in air cavities. Basically, pulse velocity can be measured in three different ways, which are direct, semi-direct and indirect methods as shown in (Fig.4.35), (Akevren, 2010).

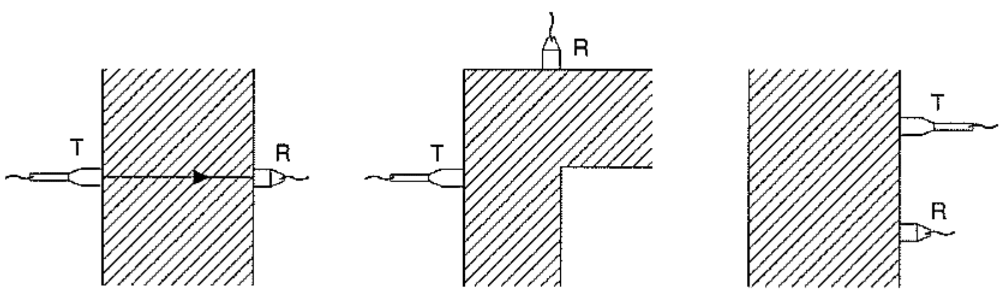

Figure 4.35: Different ways to preform Pulse velocity test, a) Direct, b) Semi-direct, and c) Indirect (Rossi, 1990).

For sonic test the pulses induced by striking the masonry with an instrumented hammer, the waveform characteristics of a low frequency ranges of 0.5 to 20 kHz. On the other hand for normal ultrasonic pulse is created by mechanical waves over 20 kHz. Such as in case of building material, waves range between 40 to 150 kHz are used. So with high frequency (> 20 kHz) (Brozovsky, 2000).

Apparatus

- Pulse Generator:

In case of sonic test, the stress wave is generated by a small, modally tuned hammer with an attached accelerator to record the input pulse. The frequency and energy content of the input pulse are governed by characteristics of the hammer. A hard hammer head will provide a high amplitude, short duration signal, suitable for transmission through large expanses of masonry, whereas a softer rubber head may be used to avoid damage to fragile masonry. The mass of the hammer determines the initial energy content of the input stress wave. Accelerometers are used to record the waveform after it passes through the masonry. A sensitivity of between 100 mV/g and 1000 mV/g is sufficient for most work (RELIM:MS-D,1 1996).

For the ultrasonic test, the pulse generator consists of circuitry for generating pulses. These electronic pulses transformed by the transducer into wave bursts with a mechanical energy of frequency in the range (20 -100) kHz, and producing repetitive pulses with a rate of 3 pulses per second at least. The decay time for the transmitting transducer should be exceeded by the time interval between pulses. The transducer must be constructed by piezoelectric, magnetostrictive, or any voltage-sensitive material, and be housed for protection. Taking in consideration that a triggering pulse shall be produced to start the time measuring circuit (Lombillo et al., 2005).

- Amplifier:

The receiving transducer and the transmitting transducer shall be similar. The voltage generated by the receiver is amplified as necessary to produce triggering pulses to the time-measuring circuit. The amplifier must have a flat response between one half and three times the resonant frequency of the receiving transducer.

- Time-Measuring Circuit:

The time-measuring circuit and the associated triggering pulses shall be capable of providing an overall time-measurement resolution of at least 1 µs. Time measurement is initiated by a triggering voltage from the pulse generator, and the time measuring circuit shall operate at the repetition frequency of the pulse generator. The time-measuring circuit shall provide an output when the received pulse is detected, and this output shall be used to determine the transit time displayed on the time-display unit. The time-measuring circuit shall be insensitive to operating temperature in the range from 0 to 40°C and voltage changes in the power source of 615 % (ASTM E494, 2010).

- Display Unit:

A display unit shall indicate the pulse transit time to the nearest 0.1 µs.

- Reference Bar:

For units that use manual zero-time adjustment, provide a bar of metal or other durable material for which the transit time of longitudinal waves is known. The transit time shall be marked permanently on the reference bar. The reference bar is optional for units that perform automatic zero-time adjustment.

- Connecting Cables:

Where pulse-velocity measurements on large structures require the use of long interconnecting cables, use low-capacitance, shielded, coaxial cables.

- Coupling Agent:

A viscous material (such as oil, petroleum jelly, water soluble jelly, moldable rubber, or grease) to ensure efficient transfer of energy between the masonry surface and the transducers. The function of the coupling agent is to eliminate air between the contact surfaces of the transducers and the concrete. Water is an acceptable coupling agent when ponded on the surface, or for underwater testing.

(Fig.4.36) shows the general principle of the test with respect to the apparatus.

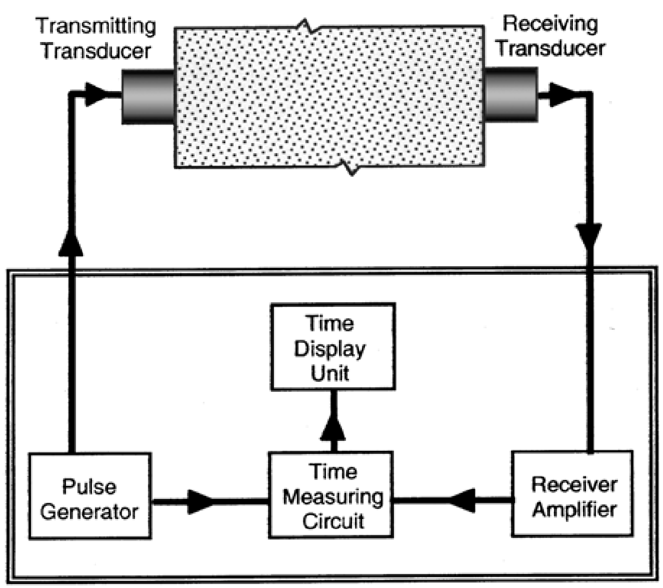

Figure 4.36: The principle work of pulse velocity test (ASTM C597, 2009).

Procedure

- Check that the equipment is properly operating by performing a zero-time adjustment as (ACI 228, 1998) states.

1. Units with Automatic Zero-Time Adjustment, Apply coupling agent to the transducer faces and press the faces together.
2. Units with Manual Zero-Time Adjustment, Apply coupling agent to the ends of the reference bar, and press the transducers firmly against the ends of the bar until a stable transit time is displayed. Then adjust the zero reference until the displayed transit time agrees with the value marked on the bar.
3. Check the zero adjustment on an hourly basis during continuous operation of the instrument, and every time a transducer or connecting cable is changed. If zero-time adjustment cannot be accomplished, do not use the instrument until it has been repaired.

- Determination of Transit Time according to (ASTM C597, 2009):
  1. Locate the transducers directly opposite each other. Because the beam width of the vibrational pulses emitted by the transducers is large, it is permissible to measure transit times across corners of a structure but with some loss of sensitivity and accuracy. Measurements along the same surface shall not be used unless only one face of the structure is accessible since such measurements may be indicative only of surface layers, and calculated pulse velocities will not agree with those obtained by through transmission.
  2. Determine the straight-line distance between centres of transducer faces.
  3. Apply an appropriate coupling agent (such as water, oil, petroleum jelly, grease, moldable rubber, or other viscous materials) to the transducer faces or the test surface, or both. Press the faces of the transducers firmly against the surfaces of the masonry until a stable transit time is displayed, and measure the transit time.

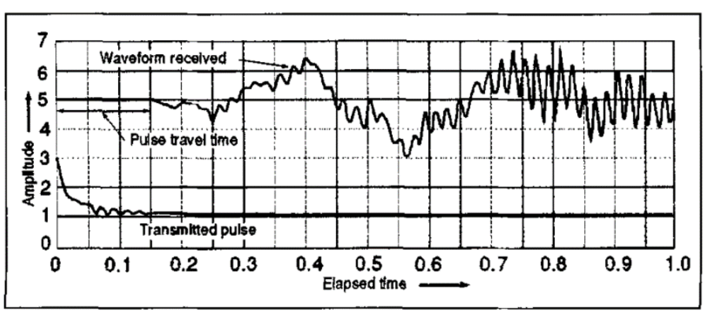

Figure 4.37: Wave record, showing both transmitted and recorded ultrasonic pulses (RELIM:MS-D.5, 1996).

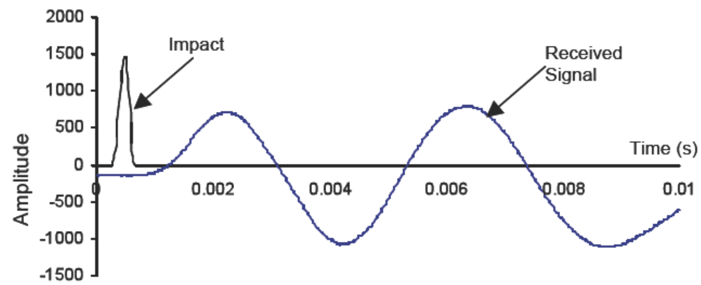

Figure 4.38: Typical signals obtained from sonic test: impact and reception (Miranda et al., 2010).

Data Analysis

- Calculate the pulse velocity as follows:

$$v = \frac{L}{T}$$

where:

V = pulse velocity, m/s.

L = distance between centres of transducer faces, m.

T = transit time, s.

- The pulse velocity, V, of longitudinal stress waves in a masonry mass is related to its elastic properties and density according to (ASTM C597, 2009) as follows:

$$v = \sqrt{\frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)}}$$

where:

E = dynamic modulus of elasticity.

μ = dynamic Poisson's ratio.

r = density.

Conclusions

The travel time allows the calculation of the velocity of the wave propagation through the masonry medium. Where the velocity of an elastic wave passing through masonry is theoretically proportional to the density, dynamic modulus and Poisson's ratio of the material. In reality, masonry can be a very inhomogeneous material. Hence, only global variations of the previous parameters can be indicative.

The use of this test for measuring masonry modulus of elasticity, can have also many other goals, where it can detect the presence of voids and flaws, and to find crack and damage patterns and detect when the physical characteristics of materials have changed.

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4.3.2 Compressive stress of masonry structures by single flat-jack

Scope

It is important to determine the state of stress in order to analyse and evaluate the current condition of existing masonry structures, in which state of stress parameter gives an indication of the structures stability against static and external actions and helps to control the repair operations. This test can be applied on different types of masonries, such as: Thin joints, thick joints, multiple leaf and irregular masonry structures and it is carried out by inserting flat-jack in slot which formed by removing the mortar bed joint, so it considers as direct, semi-destructive technique. Which cause a temporary damage can be easily repaired after testing (Chourasia et al., 2012).

Apparatus

1. Preparation equipment
    - Sawing tools

In order to have an accurate test, the surrounding masonry should not be distributed while preparing the slot. In case of regular masonry, it is possible to remove all mortar in the bed joint by stitch drilling (subsequent holes using drill), or remove it by hand using the wedge. But for irregular masonry, electrical saw is recommended.

2. Testing tools
   - Flat-jack

Flat-jack has a thin steel bladder with a hollow cavity inside capable of applying operating pressures. It consists two ports, inlet and outlet for applying the pressurized hydraulic oil. There are many shapes and sizes of flat jack bladder as shown in (Fig.4.39) depending by its functions, slot shape, and properties of masonry.

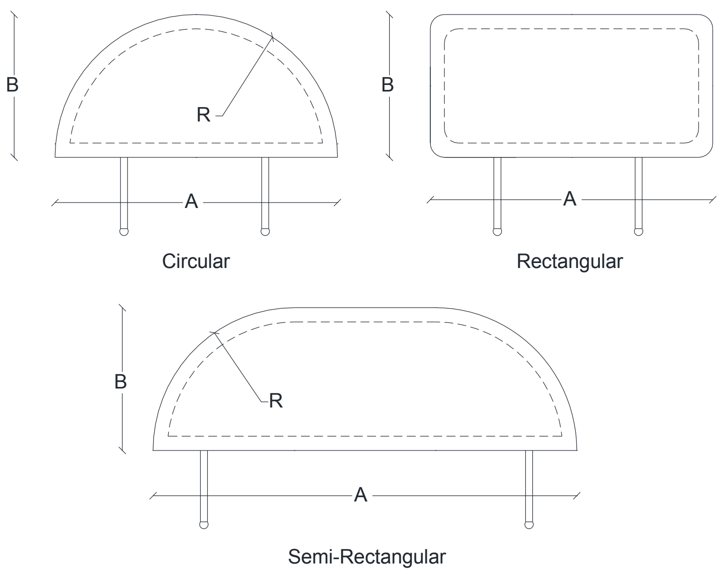

Figure 4.39: Flat-jack bladders configurations.

Circular and Semi-rectangular bladders are used to fit into slots formed by circular saws while the rectangular bladders designed to fit into slots cut by drilling holes or hand removal. On the other hand flat-jack bladder must fit the slot perfectly in the mortar joint. If not, metal shims can be used to fill the empty spaces.

Also it is important to take in consideration that the dimension [A] should be greater than or equal to the length of a single masonry unit, but it should not be less than 200 mm. The dimension [B] of the Bladder should not be less than a single masonry leaf width, and not less than 75 mm. For the radius [R], for circular and semi-rectangular bladders, it must be the same radius of the saw bladder in order to fit the slot. But for rectangular bladder, the length should be twice the width. Nevertheless. With overall flat-jack thickness which vary from a minimum of 0.625 mm to a maximum of 12.70mm, it is important not to have any significant voids [cavities] in the masonry surface, so it is possible to use a metal shims and grouting to the close area of the flat-jack (ASTM C 1196, 2009).

- Hydraulic System

The pressure in the flat-jack is provided by using an electrically or manually operated hydraulic pumps. And it is measured by gauge which calibrated to have an accuracy of 1% of full hydraulic scale. The hydraulic system must be able to maintain a constant pressure for a minimum of 5 minutes, with a maximum operating pressure of 6.9 MPa, as shown in (Fig.4.40).

Figure 4.40: Hydraulic pump (Controls group, 2013).

- Cables and Data Acquisition

The hydraulic cables connected with flat-jack, should be able to sustain the pressure of the hydraulic fluid, from pump to the jacks inlet ports, without any signs of leakage and in tight condition. The data acquisition system must be cable to store temporarily the obtained results.

- Displacement Measurement

Mechanical gauge extensometer or LVDT’s can be used to obtain the masonry displacements, by measuring the distance between fixed gauge points as shown in (Fig.4.41), according to (RILEM MDT.D.4, 2004), the accuracy should be 0.1% of the gauge length, as (ASTM C 1196, 2009) requires ±0.005% of gauge length. In any case, the deformation measurements should be up to 5 mm with an accuracy of at least 0.005 mm.

- Gauge Points

It is possible to use adhered metal discs or embedded metal inserts as gage points during the process of measuring the displacement. Gage points must be attached to the masonry by using a rigid adhesive for discs or cementations grout for plugs, to obtain a securely attach and prevent the movement to ensure the accuracy of measurement. The conical depression at their center of gage points shall be similar to the extensometer with the same angle.

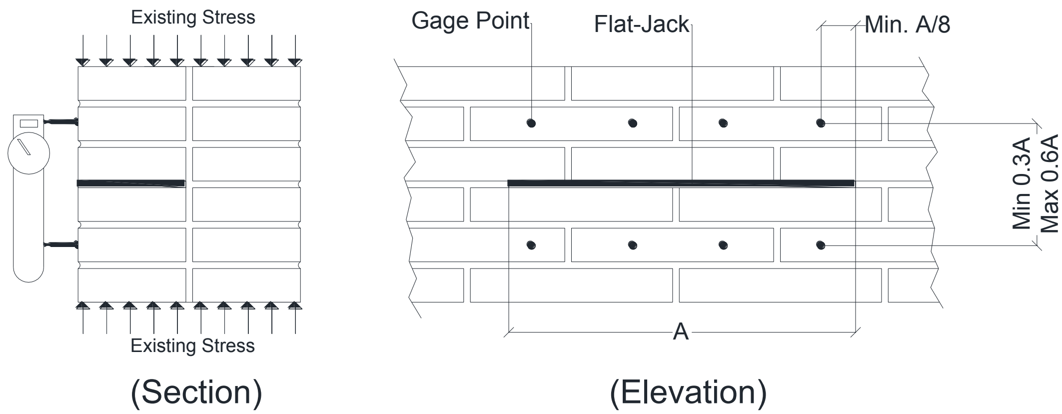

Figure 4.41: Flat-jack test setup for In situ stress measurement.

- Steel bearing plates

It is important to have steel bearing plates at each end of the test jack to distribute the load uniformly across the end of the test unit and the reaction unit. The bearing plates shall have a minimum thickness equal to 1/8 the maximum dimension in a vertical cross section (RILEM MDT.D.4, 2004).

Procedure

- The Slots Preparation

The slots prepared by cutting off and removing the mortar bed joint, by using stitch drilling holes to shape a rectangular slot, or using circular saw to provide a slot which fit the circular and semi-rectangular flat-jacks. The applied pressure by the flat-jack should be in contact with smooth masonry surface.

The slot dimensions should allow completely the flat-jack to fit into it without having any empty spaces or voids, (ASTM C 1196, 2009) states the maximum allowable gap in plane dimension not exceed of the flat-jack by 12 mm. Shims can be used to fill the remaining gap, in order to apply a uniform pressure on the masonry-unit surface. In case of large voids, it is possible to use fluid cushion shims.

- Calibration

The basic idea of flat-jack is applying a uniform pressure on the internal masonry surface, the applied pressure produced by the hydraulic pump is acting linearly with respect to the internal hydraulic pressure. So the slope of the curve between the internal and external pressure represents the coefficient of a flat-jack, which called conversion factor [K_m].

The difference caused by the inherent stiffness of the flat-jack, which resists expansion when it is pressurized. The fluid pressure produced by the pump is always greater than the applied pressure to masonry. And the procedures to get the conversion factor are, (Chourasia et al., 2012):

1. Place the calibrated flat-jack in the compression machine with at least 450 KN capacity, by using 50 mm thick steel bearing plates on the lower platen of compression machine, in which it completely covers the entire area of the flat-jack, as shown in (Fig.4).
2. Ensure that the edge with the inlet/outlet ports is coincident with the edge of the bearing plate.
3. Place steel spacers around the other edges of the flat-jack. The thickness of the spacers shall be equal to approximately $1_{1/3}$ times the combined thickness of the two sheets used in fabrication.
4. Place the upper 50 mm thick steel bearing plate on top of the shims and flat-jack, and align it to be directly above the lower bearing plate.
5. Raise the lower platen or lower the upper platen and apply a pre-load equivalent to 0.07 MPa to provide a fully contact between the bearing plates and the flat-jack.
6. Ensure that the distance between the platens must be held constant during the calibration procedure. By using a displacement-control machine or displacement gages (mechanical or electrical).
7. Pressurize and depressurize the flat-jack for three times with full pressure not exceeding the maximum range, while the distance between the platens is constant, increase the pressure with equal increments, within 5% of maximum flat-jack operating pressure. By using at least 10. At each increment, record flat-jack hydraulic pressure and force applied by the compression machine.
8. Plot load applied by flat-jack versus load measured by the machine. The slope of the line is the conversion factor [K_m], as shown in (Fig.4.44):

$$K_m = \frac{P_{Machine}}{P_{Flatjack}}$$

It is recommended to recalibrate the flat-jacks after five times using or when distortion appears excessive.

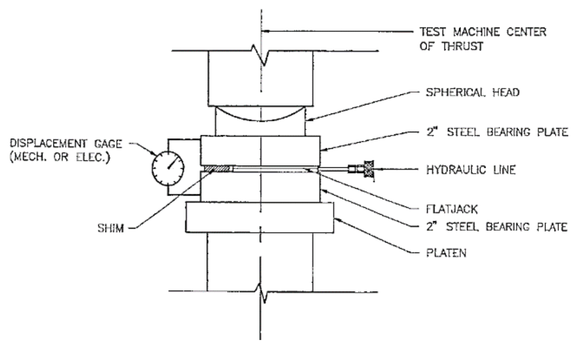

Figure 4.42: Flat-jack calibration setup (ASTM C 1196, 2009).

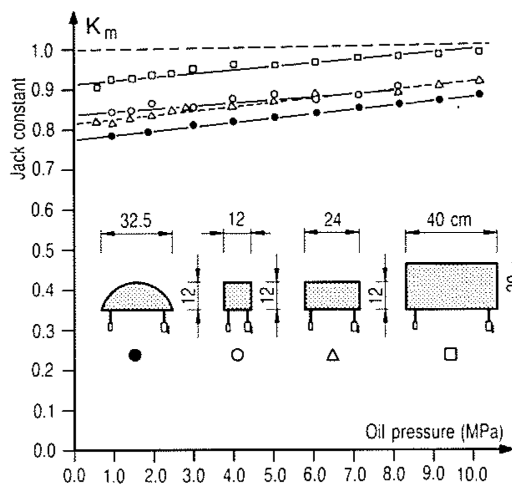

Figure 4.43: Flat-jack calibration curve (Rossi, 1987).

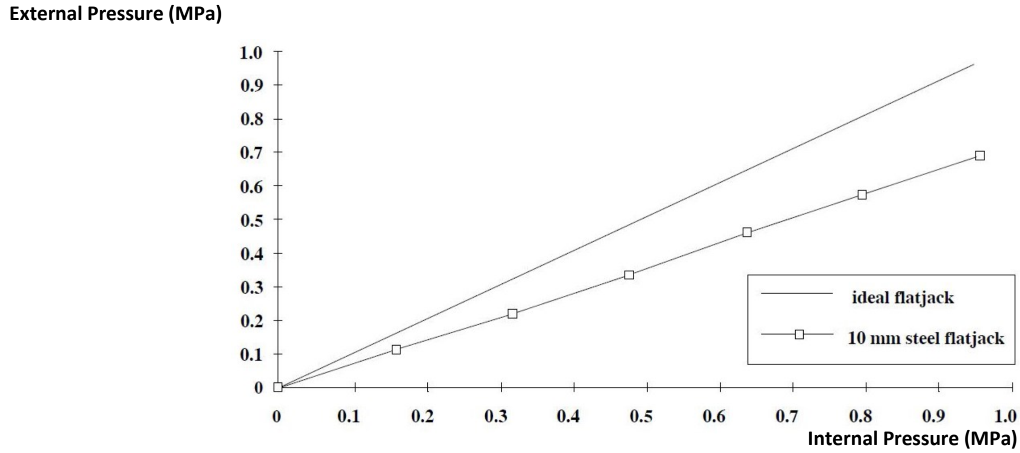

Figure 4.44: Flat-jack calibration curve (Chourasia et al., 2012).

- Positioning Of Reference Points

Setting the reference points should be done symmetrically, and it is recommended by (ASTM C 1196, 2009) to have four pairs of equally spaced points while (RILEM MDT.D.4, 2004) suggest just three pairs. The first and last pair of the reference points should be located at least a distance of [*A/8*] from the ends of flat-jack, and the vertical distance should be minimum as [*0.3A*], and maximum [*0.6A*], as shown in (Fig.4.41). And making sure about having fully attached devices to the masonry.

- Applying Flat-Jack

The main concept of this test based on the principle of releasing stress by removing the mortar bed joint in the masonry, which cause an local elimination of stresses, then placing flat-jack device which controls the stress compensation as shown in (Fig.4.45), in order to get the existing stress.

The fixed gauge points indicate the change in displacements during the test, first determining the original distance [*Li*] by measuring distances between each gauge points (Fig.4.45/ before the slot). Then, shaping the slot by removing the mortar joint to the normal direction of the measured stress. This allows the stress to release which makes the distances between gauge points decrease (Fig.4.45/ after the slot). Applying a compressive stress by to the masonry by setting flat-jack in the slot, which causes a restoration of the original displacement (Fig.4.45/ after setting flat-jack).

The applied pressure [*p*], can be related to the compressive stress in the normal direction to the slot. The internal hydraulic pressure comes from the pump to the flat-jack is higher than the actual applied stress. This is caused by the inherent stiffness of the flat-jack, which resists expansions when the jack is pressurized. Another important factor is the difference between the area of jack and the area of slot, both of these factors are taken in account while interpreting test results. (Rossi, 1987)

This test, as described above, is based on the following assumptions, (Chourasia et al., 2012):

1. The stress in place is compressive.
2. The masonry surrounding the slot is homogenous;
3. The masonry deforms symmetrically around the slot;
4. The state of stresses in the place of the measurements uniform;
5. The stress applied to masonry by the flat-jack is uniform;
6. The value of stresses (compared to compressive strength) allows the masonry to work in an elastic regime.

According to (ASTM C 1196, 2009), the procedures for in-situ stress test are as follow:

1. Measure and record the initial distance between the gage points.
2. Preparing the slots with appropriate size by removing the mortar joint with respect to the dimensions of flat-jack, and clean the slot from mortar and brick particles.
3. Measure again the distance between the gage points and record it.
4. Insert flat-jack into the slot, and place shims as required, in order to get tight fit over any internal voids.
5. Connect the hydraulic horses and develop initial stress in order to seat the flat-jack and shims, by pressurizing 50% of the estimated maximum strength of the masonry, then reduce the flat-jack pressure to zero.
6. Increase the flat-jack pressure slowly in fixed increments 25%, 50%, and 75% of estimated flat-jack pressure, and hold the pressure steady at each level.
7. Monitor the displacement, and record the pressure at which the initial distance is restored between the gages points in order to calculation the compressive stress.
8. Disconnect hoses and remove the flat-jack. The slot may be filled with mortar or other suitable material of a colour and strength similar to the original mortar.

The time taken for loading should be approximately same as that of time taken for preparation of slot to reduce the effect of creep deformations. And the allowable average deviation from the original gage length shall be the greater of ±0.013 mm or 1⁄20 th of the maximum initial deviation, with no single deviation exceeding the greater of ±0.025 mm or 1⁄10 th of the maximum deviation. Tests in which these limits are exceeded shall be considered invalid.

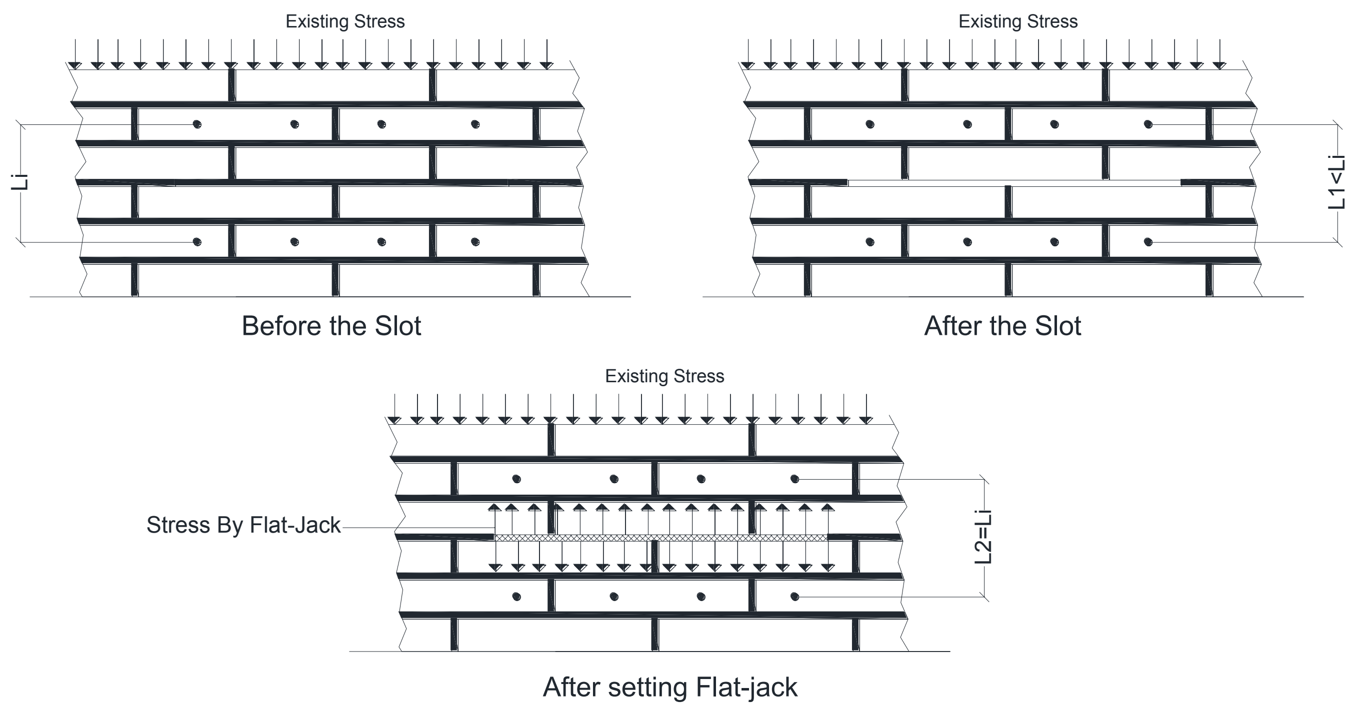

Figure 4.45: Single flat-jack test.

Data Analysis

- The stress between masonry and flat-jacks [f_m], is given by the relation, (Rossi, 1987):

$$f_m = K_m K_a p$$

where:

K_m = a dimensionless constant which reflects the geometrical and stiffness properties of the flat-jack, as determined in the calibration section.

K_a = the ratio of measured area of the flat-jack to the average measured area of the slot.

p = flat-jack pressure required to restore the gage points to the distance initially measured between them within the tolerance allowed, psi or MPa.

Conclusion

Determining state of stress of masonry structures by using flat-jack, is relatively straight forward technique without having any major destruction. Where it is can be applied also for measuring the horizontal stress affecting on the masonry wall such as the movement of footings. But it is important during the producers of the test to prevent and eliminate the voids and frogs in the mortar joints in order to get an accurate results. On the other hand flat-jack can be applied for testing single external masonry leafs, which may have a different mechanical properties from the internal one. So it is needed to use another technique to get the state of stress for the interior leafs of masonry walls such as dilatometer or hole drilling tests.

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4.3.3 Deformability properties of masonry structures by double flat-jack

Scope

Double flat-jack can be used for measuring masonry stiffness and estimating compressive strength, which are the most important parameters needed in nonlinear analysis of masonry structures. In this test, two flat-jack are pressurized together inside horizontal slots, while monitoring the strain readings of the masonry, in order to get stress-strain curve, which used to determine the modulus of elasticity.

Apparatus

The apparatus used in this test are the same mentioned in section (single flat-jack).

Procedure

- The slots preparation
    1. Remove all mortar in the bed joint, that is, pressure exerted by a flat-jack shall be directly against the surfaces of the masonry units.
    2. The plan geometry of the slot shall be similar to that of the flat-jack being used. Slots shall be parallel and aligned vertically, and shall be separated by not more than 1.5 times the length of the flat-jack.
    3. Prepare rectangular slots into which rectangular flat-jacks are to be inserted by drilling adjacent or overlapping holes (stitch drilling) and subsequently using a drill, bar, or tool to remove mortar and produce a slot of desired dimensions with smooth upper and lower surfaces.
    4. Prepare slots for circular and semi-rectangular flat-jacks using circular saws of sufficient radius to provide the depth required (ASTM C 1197, 2009).
- Deformability Test (Double Flat-Jacks Test)

The basic concept of deformability test is similar to a standard compressive test with single flat-jack, but the difference is performing the test with two flat-jacks to apply the load. The setup of the test is typical similar also, by cutting two parallel slots in the masonry wall, in order to have an isolated part from the surrounding masonry.

The initial distances between gauge points must be measured, as the isolated part of the masonry between the flat-jacks is assumed to be unstressed, so flat-jacks are introduced into both slots, and by applying the pressure, a uniaxial state pf compressive stress is created. Then by gradually increasing the pressure, the distance between gauge points decreases, and the stress-strain relationship can be determined. Also it is possible to perform loading-unloading cycles. By stress-strain curve, the value of Young's modulus can be calculated. It is possible to get the compressive strength if extended damage is acceptable (Simões et al., 2012).

The deformability test method is based on the following assumptions, (Chourasia et al., 2012):

- Masonry surrounding the slot is homogenous;
- The stress applied to masonry by flat-jacks is uniform and the state of stress in test "prism" is uniaxial, i.e. a lateral constraining effects of adjacent masonry can be neglected.

(ASTM C 1197, 2009) states the testing procedures as follows:

1. Prepare the slots along mortar joint with appropriate dimensions, clean it from any mortar and brick particles. Slots should be separated by at least five courses of masonry units, but not more than 1.5 times the length of the flat-jack.
2. Attach at least three equally spaced pairs of gauge points or electrical measuring devices. The first and last measurement points should be located at least a distance [*A/8*] in from the ends of the slot, where A is the flat-jack length as shown in (Fig.4.46).
3. Insert flat-jack into the slot, and place shims as required, in order to get tight fit over any internal voids.
4. Connect the hydraulic horses and develop initial stress in order to seat the flat-jacks and shims, by pressurizing 50% of the estimated maximum strength of the masonry, then reduce the flat-jack pressure to zero.
5. Increase the flat-jack pressure slowly in fixed increments 25%, 50%, and 75% of estimated flat-jack pressure, and hold the pressure steady at each level.

6. Monitor and record the displacement and flat-jack pressure, until it becomes highly nonlinear, as shown in (Fig.4.47).
7. Release the pressure after final displacement reading has been taken.
8. Disconnect hoses and remove the flat-jack. The slot may be filled with mortar or other suitable material of a color and strength similar to the original mortar.

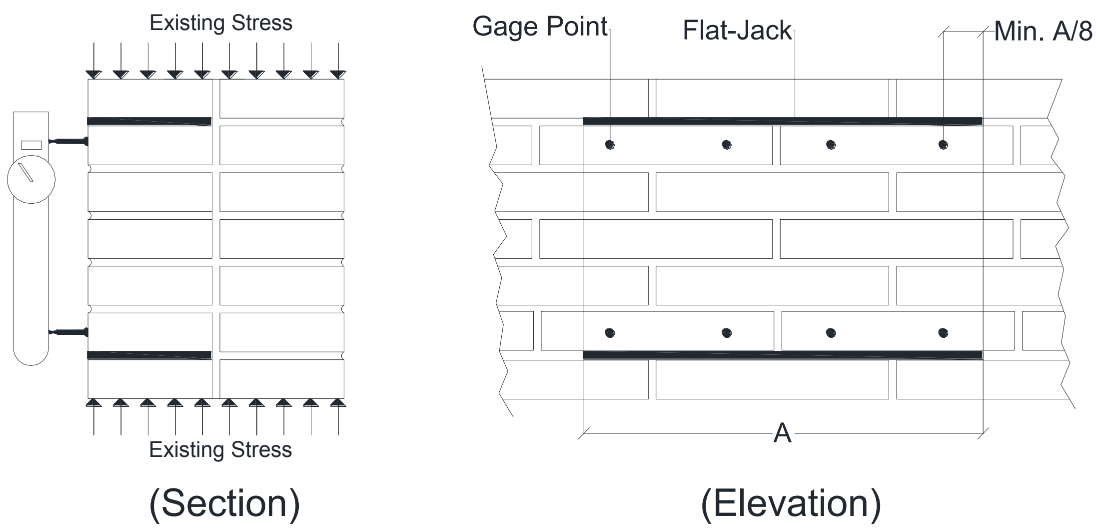

Figure 4.46: Deformation Properties Using Two Flat-jacks.

Data Analysis

- The stress between masonry and flat-jacks [f_m], is given by the relation:

$$f_m = K_m K_a p$$

where:

K_m = a dimensionless constant which reflects the geometrical and stiffness properties of the flat-jack, as determined in the calibration section.

K_a = the ratio of measured area of the flat-jack to the average measured area of the slot.

p = flat-jack pressure required to restore the gage points to the distance initially measured between them within the tolerance allowed, psi or MPa.

- The tangent modulus at any stress interval by:

$$E_t = \frac{\delta f_m}{\delta \varepsilon_m}$$

where:

δf_m = An increment of stress, psi or MPa.

$\delta \varepsilon_m$ = The corresponding increment of strain.

- The chord modulus at any point, i, is given by:

$$E_{si} = \frac{f_{mi}}{\varepsilon_{mi}}$$

where:

δf_m = Stress at point i.

$\delta \varepsilon_m$ = Strain at point i.

- The Poisson ratio (ν) is given by :

$$v = \frac{\varepsilon_h}{\varepsilon_v}$$

where:

ε_h = Horizontal strain.

ε_v = Vertical strain.

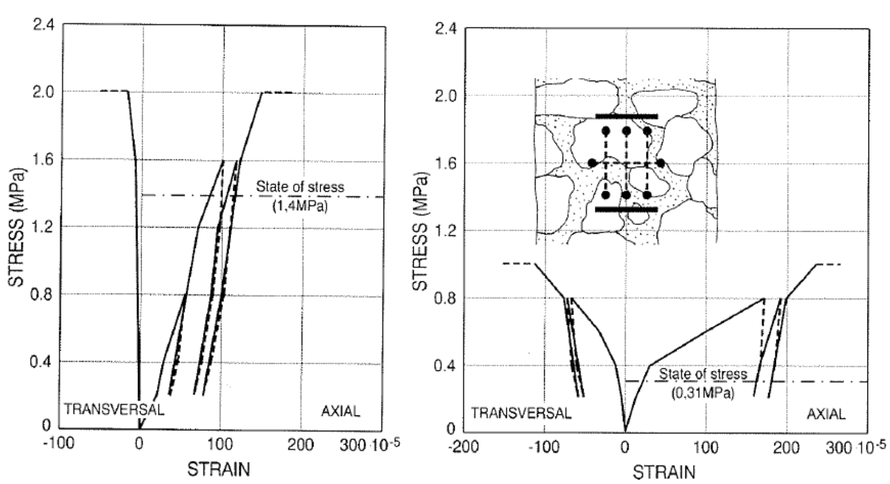

Figure 4.47: Stress-Strain diagrams obtained by double flat-jack tests on two different types of masonry (Gelmi et al., 1993).

Conclusion

Getting the deformability behaviour of the masonry structures in-situ without any major destruction, is consider one effective advantage of double flat-jack test, where it reacts directly against masonry leafs, which must be rigid enough that only the masonry between the flat-jacks deforms during the test. It is not a problem as long as the test is not performed directly above an opening, and since most walls are supported by foundations that won't deflect during the test. So it is important to ensure that enough dead load is present on the masonry above the test location, especially for single story buildings or test locations near the roof. Without enough overburden pressure, which in that situation the upper Flat-jack can actually lift the portion of the wall above it, limiting the maximum applied pressure to the test area.

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4.3.4 Modulus of deformability of masonry element by dilatometer test

Scope

The dilatometer test is an in situ semi destructive load test, applied on a masonry wall in which a uniform hydrostatic pressure on the boreholes surface is introduced by means of a cylindrical tube that can expand radially. Using double flat-jack test can only determine the deformability behavior of the external leaf of masonry wall. So it is necessary to apply dilatometer test using bore holes made by coring techniques in order to get the deformability characteristics of the internal masonry. The results obtained by the test are less reliable than those obtained by double flat-jack test, but it is useful to determine the ratio of deformability of the internal masonry leaf to the external one. (USBR 6575, 2009)

Apparatus

- The Dilatometer:
    1. Dilatometer probe: consists of a membrane mounted over a steel core which fitted with a plug at its downstream extremity. The plug is protected by a conical shaped end that helps to insert the probe into the borehole opening.
    2. Hydraulic Module: contains two cylinders in which the ends of the piston, operated by a hydraulic pump.
    3. Measuring Module: consists of an LVDT connected to a gauge by an electric cable.

4. Gauge: used to indicate the change in volume of the dilatometer probe.
5. Hydraulic Pump: A manually or electrical hydraulic pump with a pressure rating of (0-70) MPa.
6. Hydraulic Hoses: connecting the hydraulic pump to the dilatometer probe.

- Borehole Drilling:
    1. Drilling machine: This equipment includes an excavation tools, such as drills, drilling rods, drill casings, drill bits, and auxiliary tools for core drilling.
    2. Borehole Viewing Device: Some type of viewing device, such as a borehole video camera, is desirable for examination of the conditions and dimensions of the borehole's internal surface and to compare and verify geologic features observed in the core when the core recovery is poor or when retrieving oriented cores is not feasible (USBR 6575, 2009).

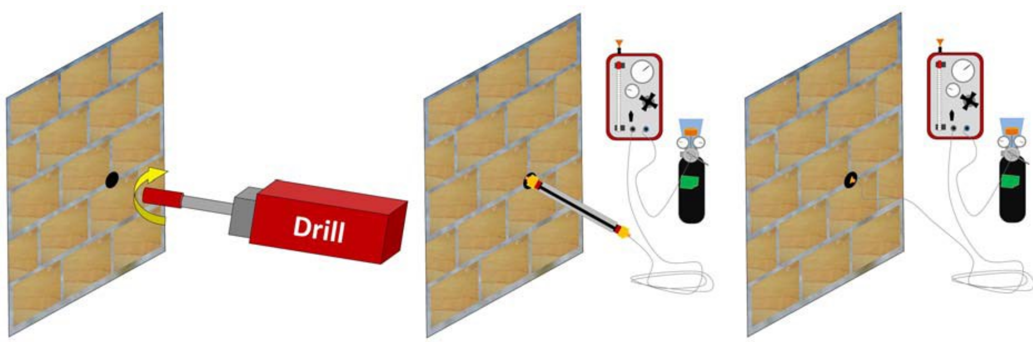

Figure 4.48: Dilatometer test setup (Lombillo et al., 2005).

Procedure

The general procedure of the test consists of drilling a hole without having any disturbance in the wall. From the results of the test, the applied pressure curve with respect to the increase of volume can be obtained, then module of deformation of the masonry can be estimated, (Fig.4.48) shows the test setup.

(USBR 6575, 2009) states the test procedures as follows:

- Drilling and Preparation: Drill the test holes, taking attention to preserve their stability. And clean the hole from any units fragments.
- Test Setup:

1. Collect the dilatometer by setting the hydraulic and electrical connections to the pump and gage unit.
2. Check the size of the borehole using the caliper, and adjust if required.
3. Record distance from centre of test membrane to the top of dilatometer assembly.
4. Insert the dilatometer into borehole, as shown in (Fig.4.49).
5. Attach additional drill rods or casing as needed to the top of the drill string to insert the probe to the required test location.

- Testing:
    1. Pressurize and expand the dilatometer membrane under a pressure just enough to ensure proper and adequate contact with the test hole walls. Record the seating pressure.
    2. Increase the pressure in 8-10 approximately equal increments to the maximum value.
    3. At each increment, keep the pressure constant while taking readings of pressure and corresponding volume. Record volume versus time to give an indication of whether the masonry behaviour is time-dependent.
    4. At the maximum test pressure, keep the applied pressure constant for at least 10 minutes or longer if specified. Readings of volume versus time at constant pressure can be tabulated to determine creep rates.
    5. Volume and pressure readings may then be taken during unloading if specified. Three cycles of loading and unloading are required.
    6. Release pressure, volume and pressure readings may then be taken during unloading if specified.
    7. Plot a pressure-volume curve and determine its slope.
    8. Release pressure and make sure membrane is fully retracted and withdraw or relocate the probe for the next test interval.

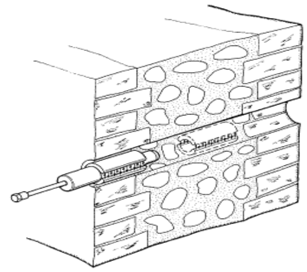

Figure 4.49: Dilatometer testing internal masonry layer (Rossi, 1990).

Data Analysis

- The modulus of deformation is based on the application of a uniform radial pressure on a cylindrical cavity in an elastic and homogeneous medium, given in (USBR 6575, 2009) as follow:

$$E_R = \frac{2(1+v)rp}{\Delta d}$$

where:

E_R = In-situ modulus of deformation, MPa

v = Poisson's ratio of masonry

r = radius of borehole, mm

p = pressure, kPa

Δd=change in diameter, mm

Conclusion

Using double flat-jack test can only determine the deformability characteristics of the superficial layer of masonry, so in order to acquire information on the deformability characteristics of the internal masonry it becomes necessary to conduct dilatometer tests using boreholes made by coring. As the portion of masonry used for this test is very limited, the values obtained by the test are less representative than those obtained by double flat-jack test. This testing technique is undoubtedly useful as it determines the ratio of the deformability of the internal masonry to the outer layer. Also it is important for testing the deformability characteristics of the foundation structures.

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4.3.5 Masonry residual stresses by hole-drilling test

Scope

The hole-drilling test applies to determine the residual stress profiles for the surface of a masonry wall. This technique is applicable for determining residual stress where in-plane stress gradients are small. The hole-drilling test is described as semi-destructive test, because the damage that it causes is localized, and easy to be repaired.

Apparatus

- At least 3 electrical strain gauges with all utilities needed for the test (glue, cables, and protections).
- Automatic equipment for recording the readings of the strain gauges used.
- Drilling machine for masonry.
- Millstone machine for preparing the masonry surface.

(Fig.4.50) shows the main apparatus used in the test.

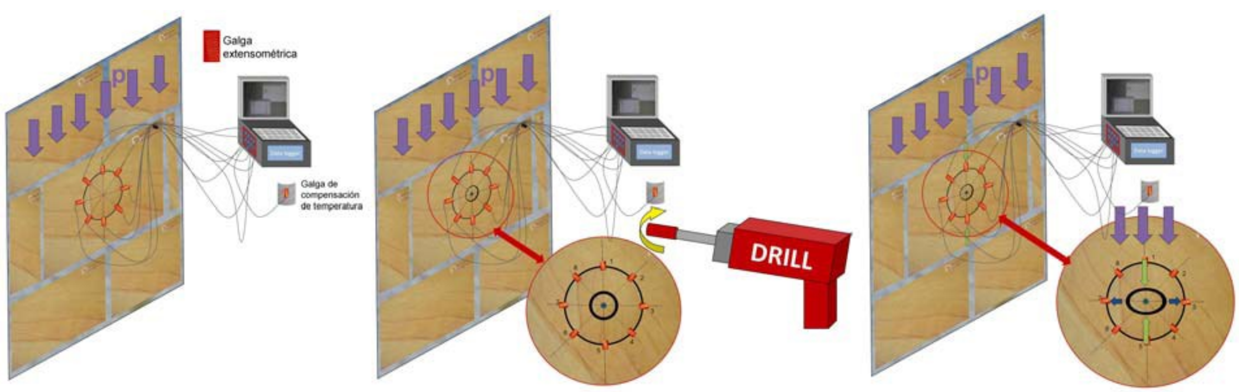

Figure 4.50: Phases of hole-drilling test (Lombillo et al., 2005).

Procedure

- Calibration:

Both constant A and B, must be determined experimentally by calibration test, in which a prismatic block sample is subjected to a uniform uni-axial compression stress. Strains are recorded in the direction of that principal compression and in the perpendicular one. Tests are repeated twice, a first one before drilling a hole and the second one after drilling it, and A and B are obtained applying the following relationships given by (ASTM E837, 2013):

$$(\varepsilon_i)_{cal} = (\varepsilon_i)_{after} - (\varepsilon_i)_{before}$$

$$A = \frac{(\varepsilon_3)_{cal} + (\varepsilon_1)_{cal}}{2\sigma_{cal}}$$

$$B = \frac{(\varepsilon_3)_{cal} - (\varepsilon_1)_{cal}}{2\sigma_{cal}}$$

Also it is possible to find out A and B by the dimensionless and material independent coefficients, values of Young modulus and Poisson ratio of the material tested given by (ASTM E837, 2013).

- Preparation:
  1. The diameter of the drilled hole (d) should be related to the diameter D by: $0{,}3 < d/D < 0{,}5$.
  2. The depth of the hole should be at least 0,4 D.
  3. Masonry surface must be prepared before insulting the strain gauges. Normally surface preparation is made by a millstone machine until a smooth surface is achieved.
  4. For avoiding experimental problems or anomalous results it is advised to use 6 strain gages instead of the minimum 3. In this case strain in each direction should be obtained by the average of the strains measured by the two gages placed in the same direction.

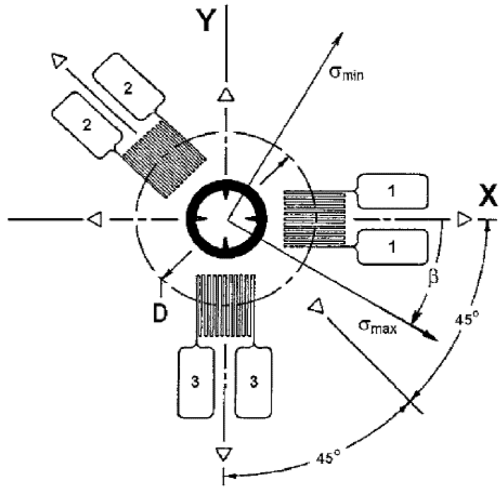

Figure 4.51: Conventional disposition of the strain gages and of the drill central circle. Where, the strain gages 1 and 3 are perpendicularly disposed and 2 is on the bisector. (D) is the diameter of the circum ference of the strain gages, while (d) is the diameter of the perforation. σ_{max} and σ_{min} are the principle stress, (β) its position with a known direction (Lombillo st al., 2005).

- Set-up:

Strains should be recorded during a certain period of time before drilling, in order to register the fluctuations around the "zero" reading. This period of time should be a minimum of 60 minutes, but longer periods are advisable if significant temperature changes are foreseeable during the duration of the test. The strain recording procedure as states in (GEOCISA, 2004) should be as follows:

a. Measurement of the "zero" reading of strains.
b. After the hole has been drilled, strains are measured again corresponding to the relieved stress situation. They should be recorded during at least 120 minutes, in order to check that any surface heating produced by the drilling process has dissipated and strain readings are stabilized, as shown in (Fig.4.53).
c. The effective strains corresponding to the stresses released by drilling are the difference between the strains measured in (b) and the strains measured in (a).

Figure 4.52: The application of Hole-drilling test (Lombillo et al., 2013).

Data Analysis

- The principal stresses and their directions are obtained from the following expressions as given by (Lombillo et al., 2013) :

$$\sigma_{max} = \frac{\varepsilon_3 + \varepsilon_1}{A} - \frac{1}{B}\sqrt{(\varepsilon_3 - \varepsilon_1)^2 + (\varepsilon_3 + \varepsilon_1 - 2\varepsilon_2)^2}$$

$$\sigma_{min} = \frac{\varepsilon_3 + \varepsilon_1}{A} + \frac{1}{B}\sqrt{(\varepsilon_3 - \varepsilon_1)^2 + (\varepsilon_3 + \varepsilon_1 - 2\varepsilon_2)^2}$$

$$\beta = \frac{1}{2} arctang \left[\frac{\varepsilon_3 + \varepsilon_1 - 2\varepsilon_2}{\varepsilon_3 - \varepsilon_1}\right]$$

- Once the principal stresses and the angle between them and the strain gauges axes have been obtained, stresses in any other direction could be obtained by application of common expressions of the theory of elasticity:

$$\sigma_{vertical} = \frac{1}{2}(\sigma_{max} + \sigma_{min}) + \frac{1}{2}(\sigma_{max} - \sigma_{min})\cos(2\alpha)$$

where:

- $\varepsilon 1$, $\varepsilon 2$ and $\varepsilon 3$ the deformations registered to 0 º, 225 º and 90 º with a direction of reference.
- σ_{max} and σ_{min} are the principal stresses.
- β is the angle between σ_{max} and the direction of $\varepsilon 1$, also it is the angle between σ_{min} and the direction of $\varepsilon 3$.

- α is the angle between the maximum principal stress σ_{max} and the vertical direction.
- A and B are the calibration constants.

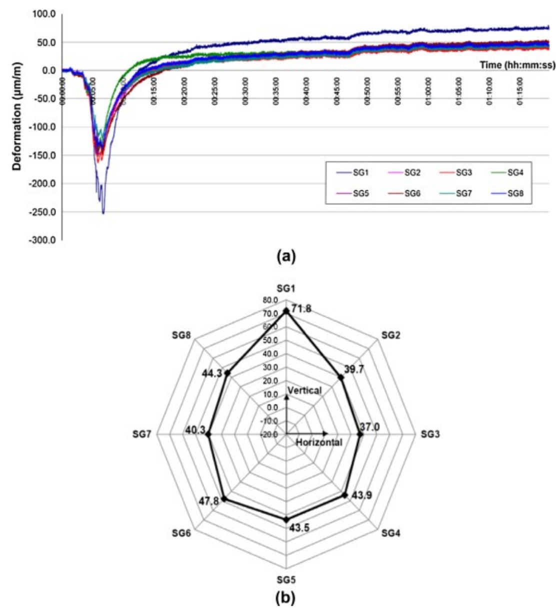

Figure 4.53: Hole-drilling recorded data, a) fluctuations of eight gauges readings, b) Difference in strain registered by each strain gauge after drilling (Lombillo et al., 2013).

Conclusion

The hole-drilling strain gage is a useful technique for measuring residual stresses in elastic materials such as masonry, where it can be termed semi-destructive if holes of very small diameters are used. The method permits the magnitudes and principal directions of residual stresses at the hole location to be determined. This is accomplished by means of an empirically determined relation between the magnitudes and directions of the principal stresses and the strain relaxation about the hole as the hole is drilled. This relation was obtained for a non-dimensional model of the hole-gage assembly in order to make the results independent of hole size. A generalization was postulated to extend the use of this calibrated solution to the measurement of residual stresses in all elastic, isotropic materials.

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4.3.6 Compressive strength of masonry prisms

Scope

Compressive strength is considered as the key value for historical structure evaluation, as it represents by the maximum stress a masonry prism can sustain under a crushing load. This technique covers the procedures for testing both laboratory made masonry prism or site sampled.

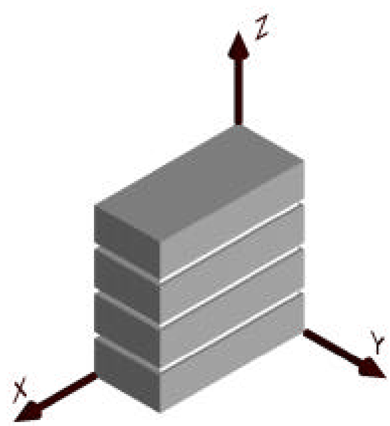

Figure 4.54: Masonry prism (Brencich et al., 2002).

Apparatus

- Testing Machine (Loading Device):

The testing machine shall be of a type having sufficient capacity and capable of providing the rates of loading with accuracy of plus or minus 1.0 % over the anticipated load range and it has a maximum apply load goes until 200 ton. The machine must be power operated and capable to apply the load continuously rather than intermittently, and without shock, (Christy, 2013) as shown in (Fig.4.53). Also it must be equipped with two steel bearing blocks with hardened faces, one of which is a spherically seated block that will bear on the upper surface of the specimen, and the other a solid block on which the specimen shall rest, as in (Fig.4.56). Bearing faces of the blocks shall have a minimum dimension at least 3% greater than the diameter of the specimen to be tested.

Figure 4.55: Axial load test setup with the data acquisition system (Christy, 2013).

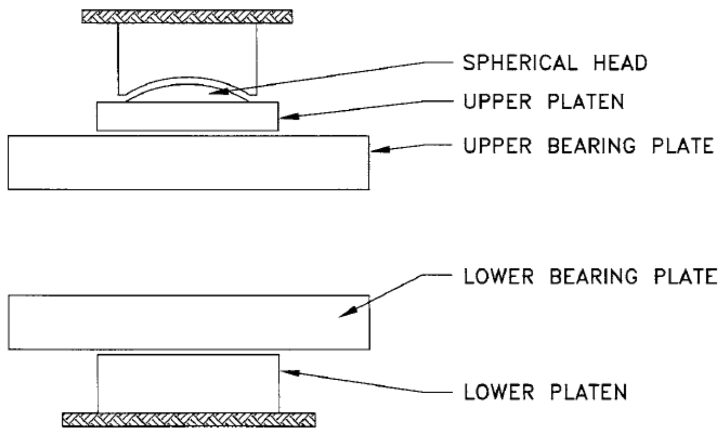

Figure 4.56: Equipment of compression testing (ASTM C1314, 2012).

Procedure

- Masonry Prism construction as states in (Felix, 1999):
    1. Prisms must be constructed on flat level base, where remain undistributed, with moisture-tight bag.
    2. Prisms minimum length shall be 100 mm.
    3. Mortar must be representative of that used in the corresponding construction, with the same thickness, method of positioning and aligning units.
    4. Prisms must be built with a minimum high of two units, and a height-to-thickness ratio, h_p/t_p, between 1.3 and 5.0.

- Curing:

Curing the constructed prisms for 48 hours, with a temperature of 24 ± 8°C. Before the testing by two days, the moisture-tight bags must be removed and keep storing with the same temperature and a relative humidity less than 80 %. For prisms obtained from field, must be stored in the laboratory with the same previous conditions of constructed prisms (CRD-C, 2001).

- Determination of Net Area:

Measuring the length, width, and height for all prisms faces to the nearest 1 mm. and determine the average. Then the net cross-sectional area of prisms considers as the net cross-sectional area of masonry units. In case of prisms obtained from field, if prisms are not of uniform length or width throughout the height of the specimen, it is considered to be minimum bearing area.

- (ASTM C1314, 2012) states the prism installation and loading in the Test Machine as follows:
    1. Clean the bearing faces, and the test specimen.
    2. Place the test specimen on the lower bearing plate.
    3. Align both central axes of the specimen with the centre of thrust of the machine. And ensure that the prism is seating uniformly.
    4. For constructed prisms, apply an initial load to the prism up to one-half of the expected total load. Then apply the remaining load at a uniform rate in not less than 1 nor more than 2 min.
    5. For prisms obtained from field, apply an initial load to the prisms up to one-fourth of the expected load. Apply the remaining load at a uniform rate in not less than 2 nor more than 4 min.
    6. Continue loading the specimen until the mode of failure is identifiable as shown in (Fig.4.57-4.58).
    7. Record the maximum load and note the mode of failure.

Figure 4.57: Failure of masonry brick prism under an axial loading (Christy, 2013).

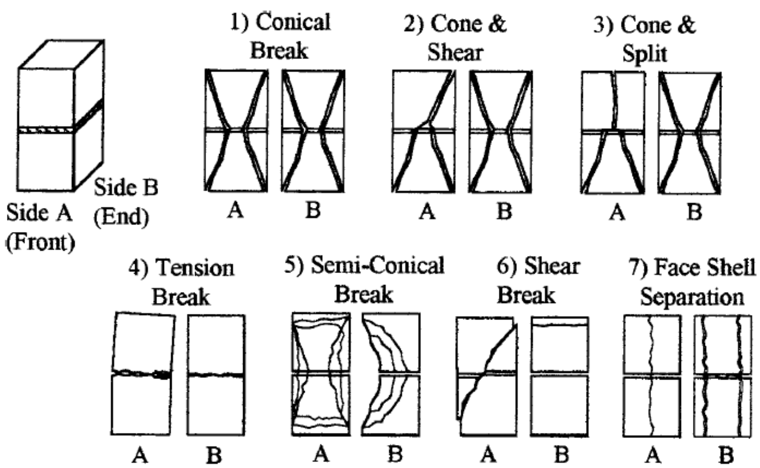

Figure 4.58: Masonry prism modes of failure (ASTM C1314, 2012).

Data Analysis

- Calculating the h_p/t_p ratio for masonry prism, then determine the correction factor from the table below.

Table 4.01: Height to thickness correction factors for masonry prism compressive strength. (ASTM C1314, 2012)

hp/tp	1.3	1.5	2.0	2.5	3.0	4	5
correction factor	0.75	0.86	1.0	1.04	1.07	1.15	1.22

- Calculating masonry prism compressive strength:

$$f_c = K_c \frac{P_{max}}{A_{net}}$$

where:

f_c = prism's compressive strength kPa.

P_{max} = prism's maximum compressive load kN.

A_{net} = net cross-sectional area m^2.

K_c = Correction factor.

Conclusion

The experimental response of the masonry prism under uniform compression revealed to be rather linear, so the compressive strength of prisms is influenced very little by the strength of the mortar. The interpretation of the moduli of elasticity for the prisms is very subjective. And it would be useful if there is a reliable method of measuring the properties of the mortar in the prism such as penetrometer.

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4.3.7 Shear-compression masonry wall test

Scope

Masonry is a typical composite construction material, which is capable to carry the compressive loads, but its capacity is low to carry the tension and shear. Where in case of lateral in-plane loads, the shear strength of masonry has the important rule of resisting the actions. So this test defines the parameters which the behaviour of masonry walls at shear are of relevant importance for the seismic resistance verification of buildings in seismic-prone areas. The test is a combination of vertical compression and in-plane shear load which applied to each masonry wall specimen.

Apparatus

1. Hydraulic system: provided by two hydraulic jacks for applying vertical pre-stress load and double-acting jack for applying in-plan horizontal load, by using electrically operated pumps.
2. Steel beam: stiff steel reaction beam for distributing the vertical load along the masonry wall, and applying shear load.
3. Steel framework: for supporting the test equipment's and the masonry wall.
4. Displacement Measurement and control mode: such as LVDT's for obtaining the masonry displacements.

Figure 4.59: Shear-Compression test setup (Di Tommaso, 2013).

Procedure

- Preparation of the specimens

1. The masonry panels shall not be less than 1.2 by 1.2 m by the thickness.
2. Specimens should not be moved for at least 7 days after the construction. And must be stored for not less than 28 days with maintained temperature of 24 ± 8°C and relative humidity between (25-75) percent.
3. Compressive strength cubes of mortar, shall be molded from a sample of each batch of mortar used to build the specimens and stored under the same conditions. The cubes shall be tested on the same day as the specimen.
4. Masonry Units, Masonry units shall be sampled and tested.

- Loading stages:

1. A compression test is usually carried out before the shear-compression test on the panels without reaching failure, in order to previously determine some masonry parameters as the Young modulus E and the Poisson's ratio.
2. Apply first the vertical pre-stress to the wall.
3. Apply the lateral load by means of a hydraulic jack, by increasing in increments of 20 kN at the same time.
4. Measure the displacement with respect to the applied lateral load.
5. The test stops when attainment of a maximum lateral load, the horizontal load dropped to 85 percent of the maximal peak value, widening of cracks beyond 3.00 mm, and local buckling of weaker areas which could result in fragile failure of the test specimen, as shown in (Fig.4.60) (Wang et al., 2006).

Figure 4.60: The failure with cracks between the mortar joints and bricks (Capozucca, 2011).

Data Analysis

- The initial value σ_0 of the vertical stress is known:

$$\sigma_0 = \frac{P_v}{A}$$

where:

P_v = Is the vertical compressive load, kN

A = the area of the transversal cross-section of the panel, m^2

- The maximum shear stress is calculated by:

$$\tau_u = \frac{T_{iu}}{A}$$

where:

T_{iu} = Maximum shear load, kN

A = the area of the transversal cross-section of the panel, m^2

- the value of the principal tensile stress σ_I in the bottom panel is expressed as given by (Corradi et al., 2007):

$$\sigma_I = \sigma_0 \left[-\frac{1}{2} + \sqrt{\langle b\frac{\tau_u}{\sigma_0}\rangle^2 + \frac{1}{4}} \right]$$

where:

b = is a shape factor that takes into account the variability of the shear stresses on the horizontal section of the wall. This parameter is assumed to be 1.5 by the Italian Standards.

- The characteristic shear stress τ_{uk} at the bottom panel is calculated:

$$\tau_{uk} = \frac{\sigma_I}{b}$$

- The shear modulus G during the elastic phase is calculated with reference to the static scheme. According to this hypothesis, the G modulus can be derived from in which it is the only unknown as given by (Corradi et al., 2007):

$$\frac{\delta_E}{0.9T_{iu}} = \frac{1.2h}{GA}\left[1 + \frac{G}{1.2E}\left(\frac{h}{d}\right)^2\right]$$

where:

d = the thickness of masonry panel, m

h = the height of masonry panel, m

E = the Young modulus obtained from the compression test. Mpa

δ_E = the relative horizontal displacement between base and the top of panel

Conclusion

This method has the advantage of allowing to evaluate shear resistance for different compression levels. Moreover, the evaluation of test results is quite straightforward, as masonry shear resistance with or without vertical loads is directly measured. However, this method has the strong limitation of being highly destructive, so that its actual applicability is limited to a very few cases. On the other hand, the behaviour of masonry walls subjected to a combination of vertical and horizontal loads depends on the geometry of the walls (height/length ratio), and mechanical characteristics of masonry and mortar bed joint, as well as on the boundary conditions. The general conclusion that the shear strength of masonry wall increases linearly with pre-compression up to a limit. So at high pre-compression, the initial bond strength reduces to zero, which means the shear strength is solely due to friction between brick and mortar separated by horizontal crack at the interface. On the other hand different strength capacity of mortar has a great influence on the shear strength.

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4.3.8 Diagonal compression test applied to masonry wall

Scope

Diagonal compression test is performed in order to obtain the diagonal tensile (shear) strength and shear modulus of masonry walls, by applying vertical loads to diagonal positioned masonry panels and monitoring the strain response with respect to the applied stress.

Apparatus

- Testing Machine:

The testing machine must have sufficient compressive load capacity (200 ton), provide the rate of loading during the test, and capable of applying the load continuously, rather than intermittently without sudden shock.

- Loading Shoes:

Two steel loading shoes must be used to apply the machine load to the masonry panel. The length of bearing of the shoe should be not less than 150 mm.

- load cell:

Load cell gives a measure to the applied force by the hydraulic jack, and it should be place between the spherical seat and the bearing plate neat to the test masonry unit.

- Pointmeter:

Displacement Measurement and control mode: such as LVDT's for obtaining the masonry displacements.

The test setup with the apparatus as shown in (Fig.4.61).

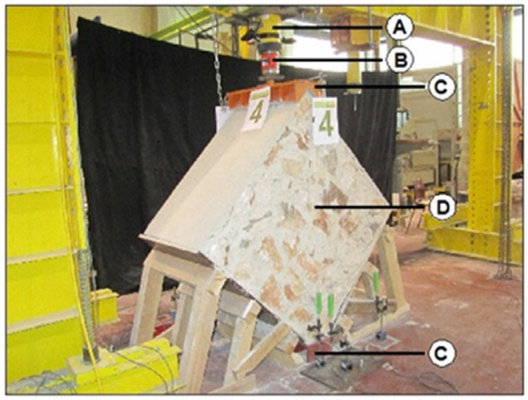

Figure 4.61: Diagonal Compression test setup, A: Hydraulic jack, B: Load cell, C: Loading shoes, D: Masonry panel (Milosevic et al., 2013).

Procedure

Preparation of Specimens

1. The masonry panels shall not be less than 1.2 by 1.2 m by the thickness.
2. Specimens should not be moved for at least 7 days after the construction. And must be stored for not less than 28 days with maintained temperature of 24 ± 8°C and relative humidity between (25-75) percent.
3. Compressive strength cubes of mortar, shall be molded from a sample of each batch of mortar used to build the specimens and stored under the same conditions. The cubes shall be tested on the same day as the specimen.
4. Masonry Units, Masonry units shall be sampled and tested.

Test Procedure

The test procedures according to (ASTM E519, 2010):

1. Place the loading shoes: the upper and lower loading shoes must be placed to be centered on bearing surfaces of the testing machine.
2. Masonry panel placement: place the specimen in a centred of the lower loading shoe, where the bed of gypsum capping material placed in, then fill the spaces between the masonry panel and the plates with the capping material. And wait the caps for at least 2 hours before testing. The testing procedure involves rotation of the masonry panel by 45° and vertical loading. But for in situ testing the wall panel remained vertical in its original orientation and the loading mechanism is rotated to obtain 45° inclination, as shown in (Fig.4.62/a).

3. Monitoring the strain: measure the changing of the vertical and horizontal diagonal under the applied load. By using compressometers and extensometers or bonded wire electrical resistance strain gages.
4. Load application: apply the loads in increments at suitable rates. Then determine the stress-strain curve. Remove the deformation measuring instruments as soon the behaviour of the panel under load indicates that it might collapse. Then apply the load until the maximum that can be applied to the specimen is determined, as shown in (Fig.4.62/b).

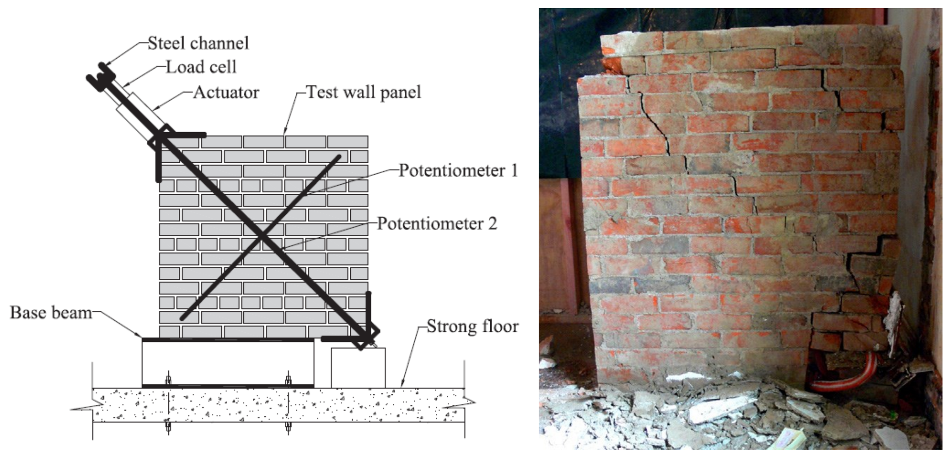

Figure 4.62: a: In-situ Diagonal Compression Test setup, b: failure mode (Milosevic et al., 2013).

Data Analysis

- The shear stress for masonry panel on the basis of net area as states in (ASTM E519, 2010) can be calculated as follows:

$$\tau = \frac{0.707F}{A_n}$$

where:

τ = shear stress on net area, MPa

F = applied load, N

An = net area of the specimen, mm^2, calculated by:

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

where:

w = width of specimen, mm

h = height of specimen, mm

t = total thickness of specimen, mm

n = percent of the gross area of the unit that is solid, expressed as a decimal.

- the shear strain as follows:

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

where:

γ = shearing strain.

ΔV = vertical shortening, mm

ΔH = horizontal extension, mm

g = vertical gage length, mm

- shear modulus of elasticity :

$$G = \frac{\tau}{\gamma}$$

where:

G = modulus of rigidity, MPa

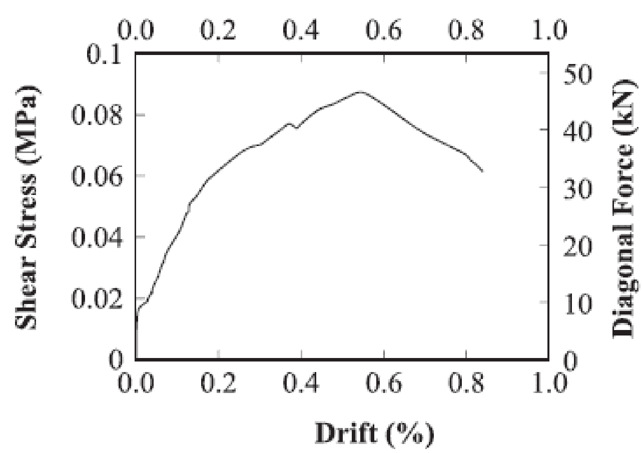

Figure 4.63: Shear stress – drift response (Dizhur and Ingham, 2013).

Conclusion

This test is considered as highly destructive test, which has a limitation of not allowing compression stress to be applied independently from shear load. Due to different mechanical properties of mortar and masonry units, different behaviour of collapse can be determined. Cracking occurred predominately through the mortar joints in a diagonal step pattern.

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Chapter 5

State of Art in Computational Masonry Approaches

5.1 Overview of computational masonry modelling

Masonry is considered the basic construction material used in the historical structures. It consists of units and mortar joints to form a composite material that behaves as one. From computational modelling and analysis viewpoint through Finite Element method, many different models can be used for simulating the behaviour of masonry structures. Micro-modelling and macro-modelling are considered the main two approaches (Mele, 2003). The first approach represents a fully detailed analysis and modelling of masonry structures, which includes a simulation of units, mortar and the interface between unit and mortar, that is the masonry units and mortar joints are considered separately as homogeneous elements, each characterized by its own mechanical properties. This approach is appropriate for small structural masonry elements in case of specific research interest in heterogeneous state of stress-strain relation. On the other hand, for the global structural behaviour where the interaction between units and mortar can be negligible, macro-modelling approach is applied, considering the masonry structure as composite material characterized by average strains-stress behaviour (Anthoine, 1992). This means as (Lourenço, 1996) states that the material parameters must be defined in large scale homogeneous states of stress. With respect to the masonry experimental tests, the necessary data must be obtained from laboratory or field tests as shown in (chapter 4).

Besides taking in consideration both micro and macro modelling, many researchers tried to produce more specific models, such as modelling bricks and mortar joints with respect to elastic analysis as shown by (Smith, 1970). Also (Page, 1978) tried to develop and apply the non-linear behaviour of masonry, considering masonry as a two-phase material. On the other hand (Ali, 1988) continued using the method to study the non-linear behaviour of masonry subjected to concentrated loads.

Modelling masonry structures requires experimental tests of the material, especially in case of micro and macro approaches. The mechanical properties of masonry are affected by many factors, such as the arrangement of bed and head joints, anisotropy of units, dimension of units, joint width, quality of construction, degree of curing, environment and age (Lourenço et al., 2012).

Researchers have been modelling the structural behaviour of masonry in recent years using the finite element method. Simple models have been used to introduce the interfaces of masonry structures, especially in case of softening behaviour and failure mechanisms (Tzamtzis and Asteris, 2003). For example, isotropic homogenized elastic behaviour has been assumed to simplify the modelling problem, ignoring the role of the mortar joints acting as planes of weakness (Lourenço, 1996). These kinds of assumptions were useful in predicting deformations at low stress levels, but not at higher stress levels where extensive stress redistribution caused by non-linear material behaviour and local failure would occur (Tzamtzis and Asteris, 2003).

Modelling of masonry structures materials based on average properties ignoring the influence of mortar joints but including the possibility of local failure, has also been developed by (Dhanasekar, 1984), who proposed a non-linear finite element model for solid masonry based on average properties derived from biaxial tests on brick masonry panels. The model is capable of reproducing the effects of material non-linearity and progressive local failure, but the masonry is modelled as a continuum with average properties, with each element in the finite element subdivision including several bricks and joints. The model, therefore, has limitations when local effects are important and cannot be used, for example, to predict the behaviour of masonry subjected to concentrated loads where local stresses and stress gradients are high.

The majority of the proposed models of masonry structures can be classified in two categories, (Tzamtzis and Asteris, 2003):

- The ‘one-phase’ material models, treating masonry as an ideal homogeneous material with constitutive equations that differ from those of the components.

- The ‘two-phase’ material models where the components are considered separately to account for the interaction between them.

The constitutive models of the first category are relatively simple to use and require less input data, and the failure criterion has normally a simple form. On the other hand, their constitutive equations are relatively complicated and are suitable at best for the study of the global behaviour of masonry. The 'two-phase' material models are relatively costly to use due to the great number of the degrees of freedom, require more input data, and their failure criterion has a complicated form due to the brick-mortar interaction. The constitutive equations of the components have normally a simple form, and they are suitable for the study of local behaviour of masonry.

5.2 Approaches of structural modelling

In this section a brief explanation of the different types of modelling approaches for masonry structures is presented.

5.2.1 Micro and macro modelling

The approach can be classified by its numerical representation, where it is possible to focus on the micro-modelling for the individual masonry structure components; unit and mortar, or the macro-modelling of masonry as a composite homogenized material. In general, it depends on the level of accuracy and the simplicity desired (Anthoine, 1992). As shown in (Fig.5.1), it is possible to use the following modelling strategies (Lourenço et al., 2012):

- Detailed micro-modelling: masonry units and mortar joints are represented by individual elements, where the interface between the unit and mortar is represented by discontinuous elements, as shown in (Fig. 5.1/b). This kind of model can represent most failure mechanisms in masonry, because the geometry of the wall is completely reproduced.
- Simplified micro-modelling: masonry units are represented by individual elements, where the behaviour of the mortar joints and unit-mortar interface is modelled by discontinuous elements. As shown in (Fig. 5.1/c), the masonry units are kept as in the “detailed micro model”, but the mortar joints and interface elements are redefined as a contact area. This

means that the general geometry is maintained, but the individual elements of joints and interface are not able to describe the Poisson's effect of mortar over masonry units. Some types of failure mechanisms cannot be reproduced in this type of model.

- Macro-modelling: masonry units, mortar and unit/mortar interface are represented by individual elements. In this case, the masonry element is considered as a homogeneous element. This type of models is able to simulate the general structural behaviour of a masonry elements but it is not able to reproduce all types of failure mechanisms (Fig. 5.1/d).

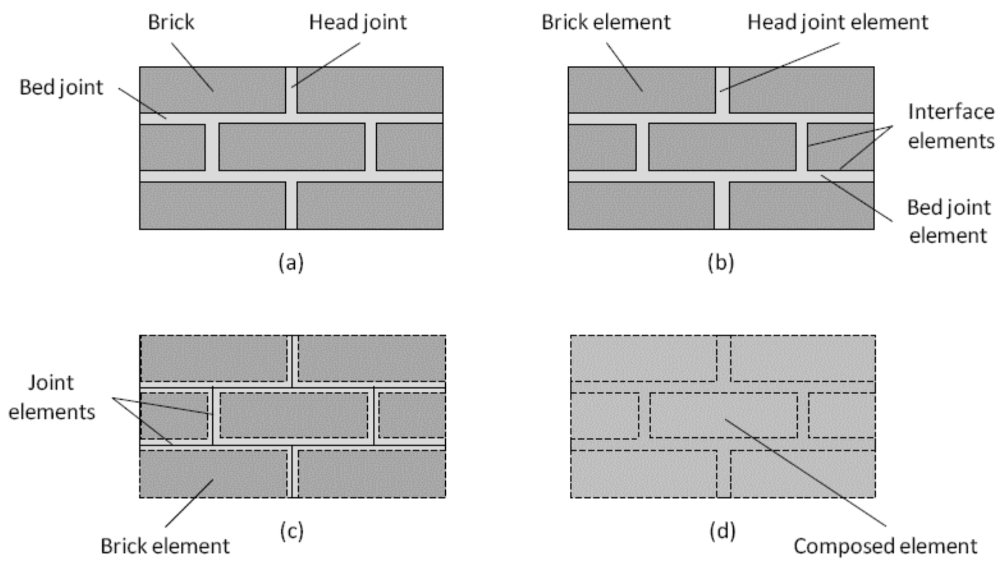


Figure 5. 1: Types of masonry models (Lourenço et al., 2012).

In the first approach, the mechanical characteristics such as Young's modulus, Poisson's ratio and inelastic properties for both masonry unit and mortar are taken into account. The interface represents a potential fracture /slip plane with initial dummy stiffness to avoid interpenetration of the continuum. In the second approach, each joint, consisting of mortar and the two unit-mortar interfaces, is lumped into an average interface while the units are expanded in order to keep the geometry unchanged. In this case, the masonry element is represented as a set of elastic blocks bonded by potential fracture/slip planes at the joints. Accuracy is lost since Poisson's effect of the mortar is not included. The third approach does not make a distinction between individual masonry units and joints but treats masonry as a homogeneous anisotropic global element, (Anthoine, 1992).

Micro and macro models can be applied in different application fields. Micro-models are necessary to give a better understanding about the local behaviour of masonry structures. On the other hand, macro-models are applicable when the structure is composed of solid walls with sufficiently large dimensions so that the stresses across or along a macro-length will be essentially uniform. Thus macro modelling is more practice oriented due to the reduced time and computational memory requirements as well as it is easy with respect to mesh generation (Anthoine, 1992).

5.2.2 Linear elastic and nonlinear analysis

The analysis of masonry structures can be classified on the basis of the relationship between stress-strain at material level and force-displacement at global structural level. These relationships can be linear, where it describes a straight line or a planar surface, or non-linear, which describes a curved line or surface. The structural analysis type to be carried out depends on the level of performance that a set of mechanical parameters can be used to perform and control the desired analysis (Andrés and Barraza, 2012). In case it is needed to assess serviceability of the construction, then mechanical, geometrical and constraint non-linearity can be neglected, where the structure is regarded in a condition with non-linearity sources are not typically important. On the other hand in case that the structure is analysed under ultimate loading conditions to assess life safety or failure prevention limit states, different non-linearity should be taking in consideration with respect to simulating the actual behaviour. Therefore, linear and non-linear analyses are complementary themselves, because the non-linearity allows to define the safety against pre-defined operational levels in serviceability conditions, while the linearity is aimed at identifying ultimate capacity of the structure (Tzamtzis and Asteris, 2003).

The main non-linearity has been defined by (Parisi, 2010) into the following types:

1. Mechanical non-linearity: where material can be expressed by non-linear constitutive law in form of the assumed stress/strain parameters of the structural material, so in this case stresses are not proportional to the relevant strains. Which means that shear or Young's modulus of the material can increase or decrease under the change of strain. Also it can be applied in case of creep and shrinkage where the time-dependent deformation increase.

2. Geometrical non-linearity: This kind of non-linearity can be induced in case of large strains or displacements. Where the mechanical non-linearity can not be applied, because the Taylor series expansion of strains shouldn't be truncated at the linear term, while in the large displacement theory, the equilibrium equations are considering the actual deformed shape of the structure through a second-order or upper-order theory. First-order theory includes the deformed shape just to evaluate displacements, while upper order theories assume that stiffness changes with deformations according to nonlinear relationships. Geometrical non-linearity can affect the behaviour of structures composed by materials with tensile strength much lower than compressive strength, such as in case of masonry.
3. Constraint non-linearity: linear analysis assumes two-way constrains, that's constrains are able to react in both directions of the applied loads. But if single constraints are considered, the boundary conditions of the structure change depending on the defined loading pattern. This can change as a result of cracking or yielding of constituent materials.

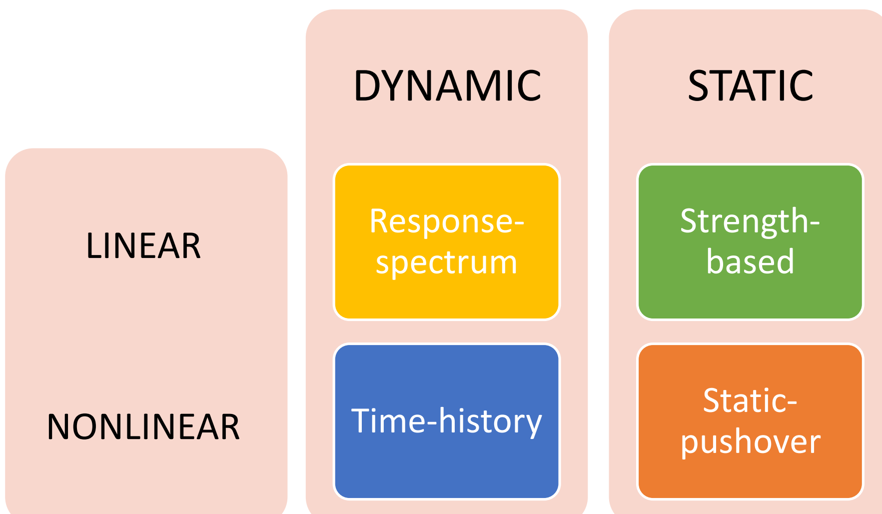


Figure 5.2: Approaches to structural analysis.

Linear analysis is preferable to non-linear, where non-linearity effects are considered after the analysis through different modification factors. For example mechanical non-linearity can be taken into account through the adoption of secant moduli, rather than those tangent. (Parisi, 2010).Iin linear analysis, only one solution of structural equilibrium since the current structural configuration does not depend on previous configurations. On the other hand, in full non-linear analysis the solution may not exist, because the current state of the structure is deformed with respect to the previous states. As proposed by (Augenti, 2004), masonry buildings have been extended to perform full non-linear analysis by spread plasticity modelling associated with the idealization of masonry walls with openings as systems of macro-elements. Therefore, the actual structural configuration under a given loading condition has memory of the previous states.

Non-linear stress/strain relations for masonry unit have been derived by (Dhanasekar, 1985) from the results of a large number of biaxial tests on square panels with various angles of the bed joint to the principal stress axes. It has been found that although the initial elastic behaviour is close to isotropic, the non-linear behaviour is strongly influenced by joint deformations and is best expressed in terms of stresses and strains referred to axes normal and parallel to the bed joint orientation.

5.2.3 Static and dynamic analysis

In order to have a complete model, it requires the elastic properties of masonry, a yield criterion, inelastic stress-strain relations, and a failure criterion. Most of the studies have concentrated on finding a failure criterion rather than studying the deformation characteristics of the material, and they are related to the tests under monotonic loading conditions (Tzamtzis and Asteris, 2003). Recently there were attempts to study the material properties of brick masonry subject to biaxial stress states produced by in-plane loading. The average elastic properties of brick masonry have been reported by (Dhanasekar, 1982) after a series of tests on masonry panels.

(Smith, 1970) has studied the failure of masonry under uniaxial compression, combined shear and compression, and tension. These failure mechanisms represent particular points on the general failure surface. The development of a general failure criterion for masonry is difficult because of the difficulties in developing a representative biaxial test. (Yokel, 1976) discussed the problem with reference to the failure of shear walls, comparing various failure hypotheses with the results of tests on single-leaf clay brick walls.

A failure surface for masonry unit subjected to biaxial tension-tension, using a non-linear finite element model accounting for joint failure has been proposed by (Page, 1980). He reported experimentally derived failure surfaces for half-scale masonry unit subjected to compression-compression and tension-compression stress states. A complete failure surface was developed later by (Dhanasekar, 1985) as an extension of the obtained experimental results. The shape of both these failure surfaces was found to be critically dependent on the bed-joint orientation and the relationship between the shear and tensile bond strengths of the mortar joints. The influence of the orientation of the applied stresses to the joints has also been investigated by (Hamid, 1981) and failure criteria were proposed as a generalized form for masonry taking into consideration its anisotropic nature as a composite material.

(Asteris, 2002) has developed a methodology for the nonlinear macroscopic analysis of unreinforced masonry walls under biaxial stress state using the finite element method. One of the advantages of the proposed material model is that average properties, which include the influence of both brick and joint, have been used. This means that a relatively coarse finite element mesh can be used, giving considerable computational advantages when analysing large wall panels. The methodology focuses on the definition / specification of a general anisotropic (orthotropic) failure surface of masonry under biaxial stress, using a cubic tensor polynomial, as well as on the numerical solution of this non-linear problem. The characteristics of the polynomial used, ensure the closed shape of the failure surface which is expressed in a unique mathematical form for all possible combinations of plane stress, making it easier to include it into existing software for the analysis of masonry structures.

In case of the dynamic analysis, it is important to study the behaviour of masonry walls under seismic forces for masonry structures in earthquake-prone areas, in order to recognize the weakness and to consider ways to prevent the collapse so that such damage can be reduced in the future. Analytical and experimental researches on the seismic behaviour of masonry structures have revealed that their earthquake performance can be improved by properly reinforcing against tensile stresses that cause brittle failure because of the low tensile strength of masonry (Tzamtzis and Asteris, 2003). The state of knowledge concerning shear strength and shear load-displacement behaviour of masonry is far less advanced than that concerning behaviour in compression, even though shear is the dominant mode of failure observed in many masonry buildings subjected to lateral loading due to earthquakes, wind or other causes. Dynamic effects are required in order to construct analytical models to simulate response under realistic seismic loading conditions. At present there appears to be very little knowledge about the dynamic mechanical properties of masonry (Kariotis, 1985).

(Atkinson, 1989) has studied the horizontal bed joint with respect to the shear failure mode and the shear load-displacement behaviour of unreinforced brick masonry during static and cyclic loading. Results of tests indicate the existence of two basic failure modes: shear failure, which is characterized by diagonal cracking of the masonry along lines of principal tensile stresses in the wall plane; and flexural failure, which is characterized by either yielding of tension steel, followed by crushing of the masonry, or by crushing of the masonry alone at the compression toe. The behaviour of masonry piers under in-plane cyclic loading has been studied in relation to the seismic design of buildings. These experimental studies are aimed at modifying and improving the existing codes for masonry structures.

Dynamic analysis by using shake-table testing of masonry walls performed by (Kariotis, 1985), indicated that the walls could sustain levels of excitation acceleration far greater than that predicted by elastic or ultimate strength calculations. The first attempt in developing a mathematical model to predict the response of masonry walls to dynamic excitation was made by (Sucuoglu, 1984). Two mathematical models were presented for predicting the linear in-plane dynamic behaviour of masonry walls. The first, called 'mixture model', recognizes the two-material composition of masonry and predicts its response accurately for a wide range of frequencies. The second, called the 'effective modulus' model, is simpler and accurate for a smaller frequency range.

Chapter 6

Comparison of FEM Softwares

6. 1 Introduction to FEM Softwares

The scope of this chapter is to compare a variety of different commercial software tools that employ linear and non-linear finite element analysis for masonry structures. The most important matter of this comparison involves the theoretical basis of the programs in which masonry structures are concerned. It includes materials libraries, brittle material models and the mechanical parameters needed for analysis. Following, this chapter covers a structural analysis of a simplified study case, modelled and analysed by different softwares.

The investigation of which FEA modules are integrated in each software aims to show what can be achieved by using different mechanical characteristics of the masonry elements implemented in the analysis by simulating a masonry wall. Typically, the users do not compare the analytical results given by the software with the theories behind the simulation. Instead, they are able to use the program and have a basic understanding of the simulations outcomes. The basic question is how significant the results are and whether or not they can be trusted.

6.2 Comparison between FEM softwares

Different FEM were selected with respect to their applications, availability, and specific analysis possibiltes for masonry element. In the following, a brief introduction to each software is followed by a general comparison in order to clarify the needs of each software with respect to the analysis requirements (Table 6.1), (Software's manual, 2014):

DIANA

- TNO DIANA BV was established in 2003 as a spin-off company from the Computational Mechanics department of TNO Building and Construction Research Institute in Delft, The Netherlands. Building on over 30 years of research and experience, the company provides world-class software products and services in the field of finite element solutions dedicated to civil, geotechnical, earthquake, and petroleum engineering.

ANSYS

- **ANSYS, Inc.** is an engineering simulation software (computer-aided engineering, or CAE) developer that is headquartered South of Pittsburgh in the Southpointe business park in Cecil Township, Pennsylvania, United States. ANSYS develops, markets and supports engineering simulation softwareused to foresee how product designs will behave and how manufacturing processes will operate in real-world environments.

SAP2000

- **SAP2000** is general-purpose civil-engineering software ideal for the analysis and design of any type of structural system. Basic and advanced systems, ranging from 2D to 3D, of simple geometry to complex, may be modeled, analyzed, designed, and optimized using a practical and intuitive object-based modeling environment that simplifies and streamlines the engineering process.

FEDRA

- **FEDRA** is an efficient tool for a complete design of masonry buildings, according to Eurocode 6. The masonry design is based on finite element analysis of each wall.

RISAMASONRY

- **RISAMasonry** offers a complete answer for analysis and design of masonry construction. **RISAMasonry**designs masonry walls, columns and beams (lintels) to the latest codes and includes a library of masonry units. The program addresses the working stress design of beams, columns and both in-plane and out-of-plane walls. The program designs slender walls using the iterative strength design method.

3MURI

- 3Muri is a program for analysis of structures in masonry and mixed materials through a non-linear (pushover) and static analysis. The strengths of 3Muri is its innovative computation method (FME) that is able to give more information on the structure's real behaviour to seismic actions, aside an extreme simplicity of use.

SOFISTIK

- FEA and CAD software for civil and structural engineering modeling, analysis, design, and detailing. SOFiSTiK is one of the most complete FE packages and covers almost all structural engineering disciplines: building, bridges, foundation, tunneling, membranes, lightweight, dynamics. Design code checks can be performed for 18 international codes.

AMQUAKE

- AmQuake program will help engineers to design safe masonry buildings in the seismic regions of Europe. AmQuake is using pushover analysis and the equivalent frame method to check the seismic safety of masonry buildings in a graphical user friendly environment.

STAADPRO

- **STAAD.Pro** is a structural analysis and design computer program originally developed by Research Engineers International in Yorba Linda, CA. It can make use of various forms of analysis from the traditional 1st order static analysis, 2nd order p-delta analysis, geometric non linear analysis or a buckling analysis. It can also make use of various forms of dynamic analysis from modal extraction to time history and response spectrum analysis.

PROKON

- ***ROKON*** **Structural Analysis and Design** is a suite of over forty structural analysis, design and detailing programs. The first ***PROKON*** programs were developed in 1989, and today ***PROKON*** is used worldwide in over eighty countries. The suite is modular in nature, but its true power lies in the tight integration between analysis, design and detailing programs. ***PROKON*** has been improving the lives of structural engineers everywhere for more than two decades.

SCIA ENGINEER

- Scia Engineer analyses, designs and details any type of structure. From the simplest to the most complex construction in concrete, steel, aluminium, plastic, timber or mixed, with integration of the local and international codes and with a link between the analysis and drawing components.

LUSAS

- LUSAS software consists of a Windows-based Modeller, used for model building and viewing of results, and a Solver for carrying out an analysis. Four commercial application products cater for Civil & Structural - for civil, structural, nuclear, seismic, geotechnical and offshore engineering.

STRAUS7

- The Strand computer software was first developed by a group of academics from the University of Sydney and the University of New South Wales. Strand7 is most commonly used for the construction and mechanical engineering sectors, but also has seen use in other areas of engineering including aeronautical, marine and mining.

ABAQUS

- **ABAQUS** is a software suite for finite element analysis and computer-aided engineering, originally released in 1978. The name and logo of this software are based on the abacus calculation tool. Abaqus is used in the automotive, aerospace, and industrial products industries. The product is popular with academic and research institutions due to the wide material modeling capability, and the program's ability to be customized.

Table 6.1: Comparison between FEM softwares.

	Masonry Modelling	Material Library	Input Parameters	Structural Analysis
3MURI	Designs masonry structures and analyze the existing one.	A library of Masonry Units and Mortars	Compressive strength. Modulus of elasticity. Shear modulus. Shear strength. Unit weight.	Static linear and nonlinear analysis
ABAQUS	------	------	general properties (mass density, material damping, thermal expansion); elastopastic mechanical properties (stress-strain data, Isotropic hardening, Linear kinematic hardening inelastic mechanical properties; thermal properties;	Linear static analysis Nonlinear static analysis frictional sliding Dynamic stress/displacement analysis Brittle cracking model Damaged Plasticity Model Evolution of failure controlled by hardening
AMQUAKE	Design of Masonry Buildings for Earthquake Resistance according to Eurocode 6 and 8.	Catalogue of materials	Characteristic compressive strength of masonry. Horizontal characteristic compressive strength of masonry. Characteristic initial shear strength. Modulus of elasticity of masonry. Shear modulus of masonry. Specific masonry weight. Flexural strength, parallel to bed joints. Flexural strength perpendicular to bed joints. Compressive strain of masonry at compressive strength.	Static linear and nonlinear analysis
ANSYS	Gives the ability to assess the influence of this range of variables in a virtual environment. Linear, nonlinear, static and dynamic analyses of masonry structures.	------	Linear materials: Density, Modulus of Elasticity, shear modulus. Nonlinear material: Elastoplastic stress-strain curves, creep data, tensile failure criteria, crack softening, flexural strength, Compressive strength Anisotropic Elastic Material: which are usually input in the form of a matrix, these properties are different from anisotropic plasticity, which requires different stress-strain curves in different directions.	Rate-independent plasticity Rate-dependent plasticity Swelling and creep Nonlinear elasticity Hyperelasticity and Viscoelasticity
DIANA	Offering the multi-directional fixed crack model and the plasticity models to simulate cracking and crushing respectively for masonry elements.	------	Properties for the masonry: Linear elasticity, mass density, Poisson ratio, Coefficients of Linear Thermal Expansion. Properties for the interface elements: linear stiffness moduli, normal and shear traction. Properties for discrete cracking in the interface elements: the stiffness moduli, the tensile strength, fracture energy. Properties of inelastic behaviour: tensile strength, Composite interface model, tension softening criterion, fracture energy, cohesion, friction angle, shear softening criterion, compressive strength Tension mode: bond tensile strength, bond fracture energy Shear mode: bond strength, friction angle Cap mode: compressive strength, compressive fracture energy, equivalent relative displacement	Elasticity and Viscoelasticity Nonlinear Elasticity and Linear elastic Plasticity and Cracking Creep and Shrinkage Interface analysis
FEDRA	FEDRA is a tool to design of masonry buildings, according to Eurocode 6, EN 1996	A library of Masonry Units and Mortars	Masonry unit: Compressive strength, Shear strength, Modulus of elasticity, Unit weight. Mortar: Compressive strength.	Linear static analysis Nonlinear seismic analysis Plane stress quadrilaterals with four nodes
LUSAS	------	------	Defining the behaviour of the element material, including linear, plasticity, creep and damage effects.	Linear analysis Nonlinear contact analysis Elasto-plastic analysis Dynamic analysis Natural frequency analysis Thermal analysis
MASTERKEY	Design reinforced and unreinforced masonry columns, stiffed and unstiffened, single and cavity wall panels with and without openings according to BS 5628.	Materials and workmanship can be selected from the in-built library	Compressive Strength. Flexural Strength. Units and Mortar Strength.	Static Linear and nonlinear analysis

	Masonry Modelling	Material Library	Input Parameters	Structural Analysis
PROKON	The masonry section design module, MasSec, is mainly used for the design of members such as lintels and masonry that span large openings in walls. MasWall, on the other hand is ideally suited for the design of wall panels and bearing walls. Characteristic compression strengths for masonry units are calculated based on unit geometry, nominal strengths and tables in the abovementioned code of practice.	A library of Masonry Units and Mortars	Nominal compression strength for the masonry units. Mortar class. load eccentricity The safety factors	Masonry beam at a critical section. It is assumed that the loads imposed on the beam causes uniaxial bending and a shear force only. In plane axial loading (Bearing walls) Out of plane loading, causing biaxial plate bending (Wall panels)
RISAMASONRY	Designs masonry walls, columns and beams (lintels) to the latest codes. UBC 97 and MSJC 02/05 (ACI/ASCE/TMS) codes	library of masonry units	Modulus of elasticity. Compressive strength. Density of masonry.	Cracked and uncracked analysis for in plane shear walls. Linear and nonlinear static analysis
SAP2000	------	yes (but not masonry)	The modulus of elasticity, for axial stiffness and bending stiffness. The shear modulus, for torsional stiffness and transverse shear stiffness, this is computed from the modulus of elasticity and the Poisson's ratio. The mass density (per unit of volume), for computing element mass. The weight density (per unit volume), for computing Self-Weight Load.	Linear static analysis Nonlinear Static Analysis Dynamic analysis Nonlinear Time-History Analysis
SCIA ENGINEER	------	Yes (but not for masonry)	Modulus of elasticity. Weight density. Poisson's ratio. Coefficient of thermal expansion.	Linear static analysis Non-linear static analysis (tension only members, pressure only supports), Geometrical non-linearity Advanced non-linear static analysis (springs and gaps for beams, pressure only slabs), Stability analysis, Dynamic analysis (eigenmodes, harmonic, seismic, general dynamic load) Advanced calculations: Soil interaction, Cables, Non-linear stability, Membranes, Sequential analysis, Friction springs Linear and non-linear construction stages Pre-stressed structures, Time Dependent Analysis
SOFISTIK	------	Yes (but not for masonry)	Compressive strength. Modulus of elasticity. Shear modulus. Shear strength. Unit weight.	Linear Analysis Non-linear Analysis
STAAD.PRO	------	Yes (but not for masonry)	Modulus of elasticity. Weight density. Poisson's ratio. Coefficient of thermal expansion. Composite Damping Ratio.	Linear Elastic Analysis P-Delta Analysis Nonlinear Cable Analysis Buckling Analysis Steady State and Harmonic Analysis Geometric Nonlinear Analysis
STRAUS7	------	Yes (but not for masonry)	Modulus of elasticity. Weight density. Poisson's ratio. Coefficient of thermal expansion. Composite Damping Ratio. Elasto-plastic materials. Nonlinear elastic materials.	Linear static solver Linear buckling solver Nonlinear static solver Linear transient dynamic solver Nonlinear transient dynamic solver Natural frequency and natural response solver.

Different FEM softwares are used for modelling and analysing a numerical example of masonry panel with respect to different loading conditions according to linear and nonlinear behaviour. The eight different softwares were selected with respect to different applications and uses each can perform, such as ANSYS, ABAQUS and DIANA, which are sfoftwares used for research and professional applications, where SAP2000, SRAUS7, and STAAD PRO are softwares apply for normal engineering applications. Detailed comparison between these softwares with respect to the theoretical basis and failure criteria adopted by each.

6.2.1 ANSYS

ANSYS is an engineering simulation software (computer-aided engineering CAE) developer that is headquartered south of Pittsburgh in the South Pointe business park in Cecil Township, Pennsylvania, United States of America. ANSYS offers engineering simulation solution sets in engineering simulation that a design process requires. The tools put a virtual product through a rigorous testing procedure (such as crashing a car into a brick wall, or running for several years on a tarmac road) before it becomes a physical object. It is capable of simulating problems in a wide range of engineering disciplines, such as Structural Analysis (deformation, stress, and strain fields, as well as reaction forces in a solid body can be simulated).

There are seven types of structural analyses available in the ANSYS family of products is explained below. The primary unknowns (nodal degrees of freedom) calculated in a structural analysis are displacements. Other quantities, such as strains, stresses, and reaction forces, are then derived from the nodal displacements. Structural analyses are available in the ANSYS/Multiphysics, ANSYS/Mechanical, ANSYS/Structural, and ANSYS/LinearPlus programs only. It is possible to perform the following types of structural analyses:

1. Static Analysis-Used to determine displacements, stresses, etc. under static loading conditions. Both linear and nonlinear static analyses. Nonlinearities can include plasticity, stress stiffening, large deflection, large strain, hyperelasticity, contact surfaces, and creep.
2. Modal Analysis-Used to calculate the natural frequencies and mode shapes of a structure. Different mode extraction methods are available.
3. Harmonic Analysis-Used to determine the response of a structure to harmonically time-varying loads.

4. Transient Dynamic Analysis-Used to determine the response of a structure to arbitrarily time-varying loads. All nonlinearities mentioned under Static Analysis above are allowed.
5. Spectrum Analysis-An extension of the modal analysis, used to calculate stresses and strains due to a response spectrum or a PSD input (random vibrations).
6. Buckling Analysis-Used to calculate the buckling loads and determine the buckling mode shape. Both linear (eigenvalue) buckling and nonlinear buckling analyses are possible.
7. Explicit Dynamics Analysis-ANSYS provides an interface to the LS-DYNA explicit finite element program and is used to calculate fast solutions for large deformation dynamics and complex contact problems.

Nonlinearity

Most real-world physical phenomena exhibit nonlinear behaviour. There are many situations in which assuming a linear behaviour for the physical system might provide satisfactory results. On the other hand, there are circumstances or phenomena that might require a nonlinear solution. A nonlinear structural behaviour may arise because of geometric and material nonlinearities, as well as a change in the boundary conditions and structural integrity.

These nonlinearities are discussed briefly in the following subsections:

1. Geometric Nonlinearity

There are two main types of geometric nonlinearity:

- Large deflection and rotation: if the structure undergoes large displacements compared to its smallest dimension and rotations to such an extent that its original dimensions and position, as well as the loading direction, change significantly, the large deflection and rotation analysis becomes necessary. For example, a fishing rod with a low lateral stiffness under a lateral load experiences large deflections and rotations.
- Stress stiffening: when the stress in one direction affects the stiffness in another direction, stress stiffening occurs. Typically, a structure that has little or no stiffness in compression while having considerable stiffness in tension exhibits this behaviour. Cables, membranes, or spinning structures exhibit stress stiffening.

2. Material Nonlinearity

A typical nonlinear stress-strain curve is given in (Fig.6.1). A linear material response is a good approximation if the material exhibits a nearly linear stress-strain curve up to a proportional limit and the loading is in a manner that does not create stresses higher than the yield stress anywhere in the body.

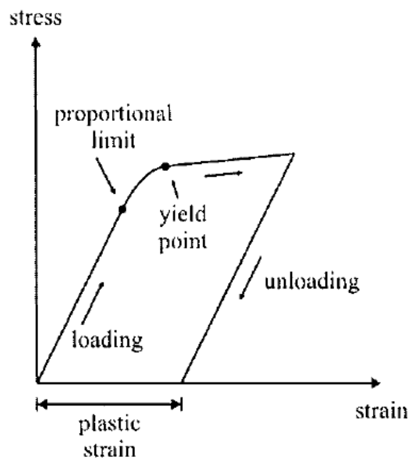


Figure 6.1: Non-linear material response (ANSYS, 2009).

Nonlinear material behaviour in ANSYS is characterized as:

- Plasticity: permanent, time-independent deformation.
- Creep: permanent, time-dependent deformation.
- Nonlinear Elastic: nonlinear stress-strain curve; upon unloading, the structure returns back to its original state—no permanent deformations.
- Viscoelasticity: time-dependent deformation under constant load. Full recovery upon unloading.
- Hyperelasticity: rubber-like materials.

Also ANSYS has the concrete material model which predicts the failure of brittle materials, both cracking and crushing failure modes, where the criterion for failure due to multiaxial stress state can be expressed by William and Warnke form:

$$\frac{F}{f_c} - S \geq 0$$

where;

F = a function of principle stress state$(\sigma_{xp}, \sigma_{yp}, \sigma_{zp})$.

S = failure surface expressed in terms of principle stresses and given by ultimate uniaxial tensile strength, ultimate uniaxial compressive strength, and ultimate biaxial compressive strength.

f_c = uniaxial crushing strength.

$\sigma_{xp}, \sigma_{yp}, \sigma_{zp}$ = principle stresses in principle directions.

The failure of brittle materials is characterized in four domains, which are:

1. (Compression – Compression – Compression) domain.
2. (Tension – Compression – Compression) domain.
3. (Tension – Tension – Compression) domain.
4. (Tension – Tension – Tension) domain.

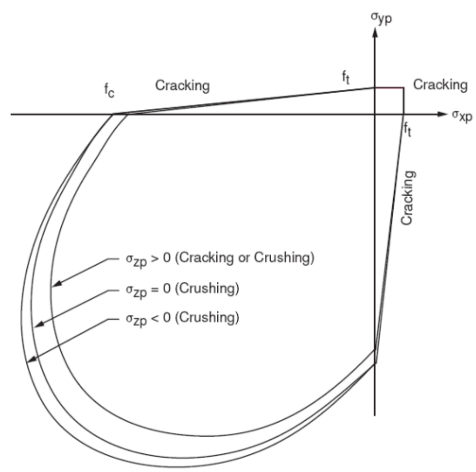


Figure 6.2: ANSYS failure surface in principal stress space with nearly biaxial stress (ANSYS, 2009).

Table 6.2: Most important features in ANSYS.

Masonry Modelling	Gives the ability to assess the influence of this range of variables in a virtual environment. Linear, nonlinear, static and dynamic analyses of masonry structures.
Material Library	------
Input Parameters	• Linear materials: density, modulus of elasticity, shear modulus. • Nonlinear material: elastoplastic stress-strain curves, creep data, tensile failure criteria, crack softening, flexural strength, compressive strength • Anisotropic elastic material: which are usually input in the form of a matrix, these properties are different from anisotropic plasticity, which requires different stress-strain curves in different directions.

Structural Analysis	• Rate-independent plasticity • Rate-dependent plasticity • Swelling and creep • Nonlinear elasticity • Hyperelasticity and Viscoelasticity

6.2.2 ABAQUS

ABAQUS is a general simulation tool based on the finite element analysis that can be used for a variety of applications ranging from the modelling of civil engineering structures to acoustics. To handle such a diversified range of applications, ABAQUS has two main analysis products; ABAQUS/Standard and ABAQUS/Explicit which is used to model dynamic events using an explicit dynamic finite element formulation, as well as four special-purpose products that are add-ons to ABAQUS/Standard. Analysis of a model is performed by one or more of this suite of products. Pre- and post-processing can be done using either another program or by ABAQUS/CAE, which represents a complete ABAQUS environment. ABAQUS/Standard employs the finite element method to implicitly solve a system of equations at each solution "increment" for the analysis of solid, shell, and framework models. Also it has an extensive library of elements that can be used to model masonry structures, In general, ABAQUS can be used to solve combinations of static and dynamic, linear and nonlinear problems.

There are three main models available in ABAQUS. Each can be used for modelling masonry structures at low confining pressures in all types of elements. These models are briefly discussed below. It should be noted that ABAQUS allows the user to define mechanical material behaviour.

Smeared Crack Concrete Model

This model is intended for applications in which the brittle material is subjected to essentially monotonic straining. In this model, linear elastic behaviour is used to define elastic properties and smeared cracking is used to describe the reversible part of the material's response after cracking failure. The model "consists of an isotropically hardening yield surface that is active when the stress is dominantly compressive and an independent "crack detection surface" that determines if a point fails by cracking".

The model is dominated by the cracking and post-cracking anisotropic behaviour and at each integration point, constitutive calculations are performed independently and the stress and stiffness are affected by the presence of cracking.

The uniaxial behaviour of the model can be seen in (Fig.6.3). Because the model assumes primarily monotonic straining and little or no unloading, the unload/reload response is elastic.

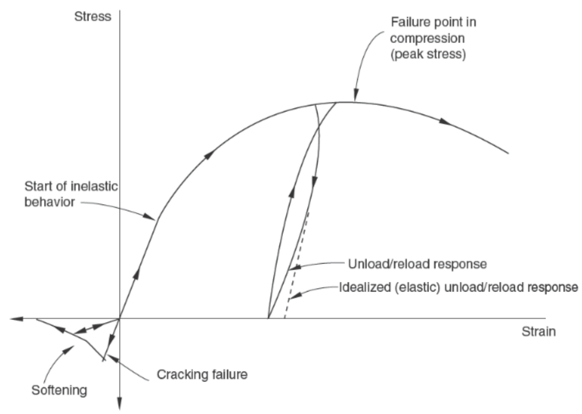


Figure 6.3: ABAQUS Uniaxial concrete behaviour (Abaqus 6.11, 2011).

Tension stiffening is accounted for by specifying a post-failure stress-strain relation or by applying a fracture energy cracking criterion. With the fracture energy criterion, the behaviour is specified by a stress-displacement response which requires the definition of a characteristic crack length. The two means are pictured in (Fig.6.4) and (Fig.6.5).

As previously mentioned, because the model assumes primarily monotonic straining and little or no unloading, the unload/reload response is elastic and includes plastic offset.

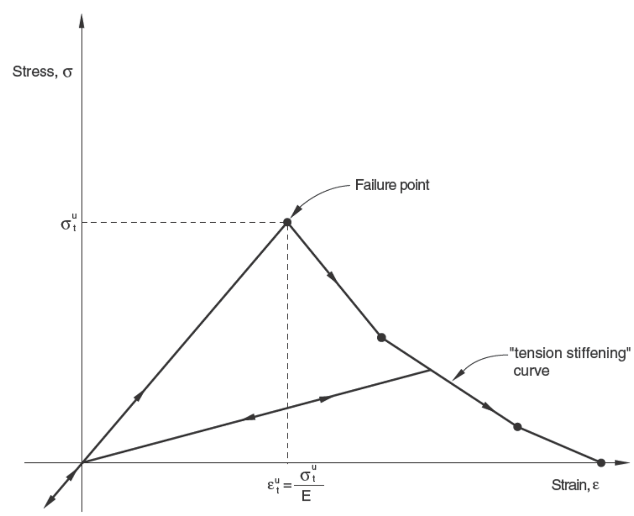


Figure 6.4: Tension Stiffening (Abaqus 6.11, 2011).

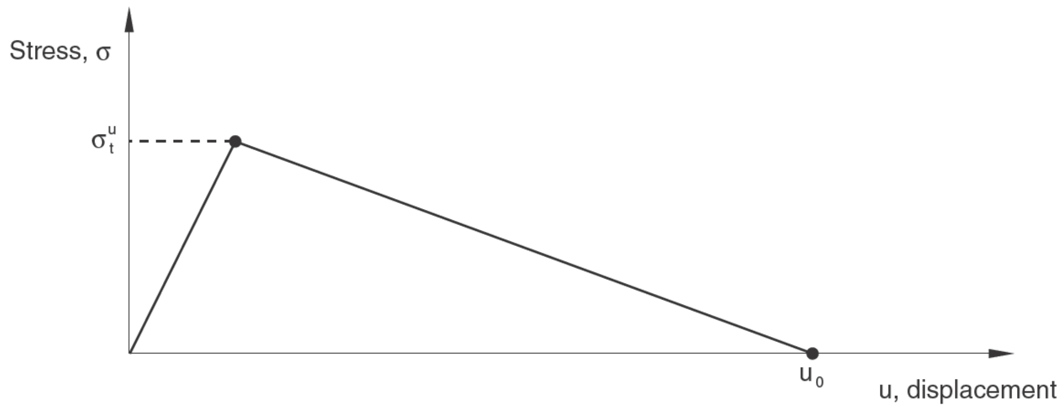


Figure 6.5: Fracture energy cracking models (Abaqus 6.11, 2011).

Brittle Cracking Model

This model is available only in ABAQUS/Explicit and is intended for applications in which "the brittle materials behaviour is dominated by tensile cracking and compressive failure is not important". The model assumes linear elastic compression behaviour. Unloading/reloading in the compression region is linear elastic just as is the assumed constitutive relationship for this model. This model also models cracks in a smeared manner and the tension stiffening post-cracking behaviour can be accounted for by specifying a post stress-strain relation or by applying fracture energy criterion. Additionally, a brittle failure criterion can be defined, in which the material point is considered to have failed once the number of cracks at that point reaches a user specified value (default is one). The associated element is then removed. If rebar is present, this failure does not remove the rebar's contribution to the element stress capacity, unless specified to do so by the user.

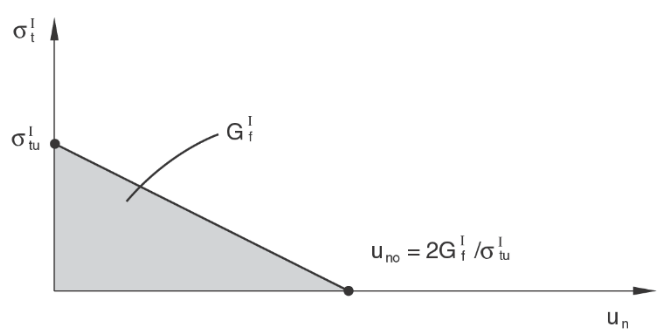


Figure 6.6: Postfailure stress-fracture energy curve (Abaqus 6.1,1 2011).

Concrete Damaged Plasticity Model

This model "takes into consideration the degradation of the elastic stiffness induced by plastic straining both in tension and compression. It also accounts for stiffness recovery effects under cyclic loading." The compressive stress-strain relation can be seen in (Fig.6.7/b). The compressive behaviour is elastic until initial yield and then is characterized by stress hardening followed by strain softening after the ultimate point.

After the onset of micro-cracking (failure stress) the response is softened, inducing strain localizations in the concrete structure. Like the previous two models, post-cracking behaviour can be accounted for by specifying a post stress-strain relation or by applying a fracture energy criterion. In both the tensile and compressive stress strain curves, the unloading response is characterized by a weakening of the material and a degradation of the elastic stiffness. These phenomena are defined by particular damage parameters. ABAQUS also allows the user to specify stiffness recovery factors.

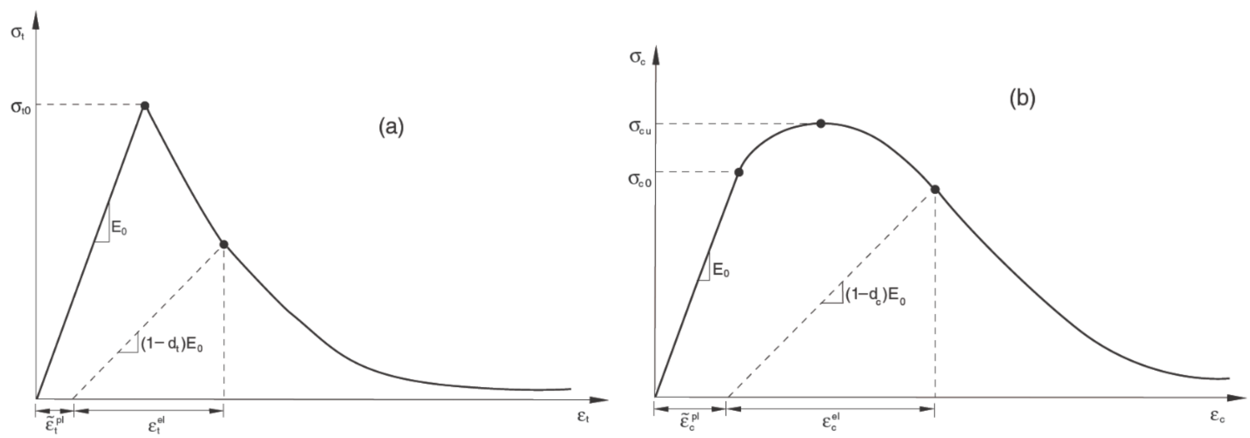


Figure 6.7: Response of concrete to uniaxial loading in tension (a) and compression (b) (Abaqus 6.11, 2011).

The way in which each of the three models in ABAQUS handles cracking is presented below:

1. Smeared Crack Concrete Model – This model does not track independent cracks, but rather cracking is assumed to occur when the stress reaches a "crack detection surface". The model is dominated by the cracking and post-cracking anisotropic behaviour. At each integration point, constitutive calculations are performed independently and the stress and stiffness are affected by the presence of cracking. Cracked shear retention is considered by either specifying the reduction in the shear modulus as a function of the opening strain across the crack or by reducing the shear modulus for closed cracks.

2. Brittle Cracking Model – As with the previous model, this model assumes cracking is smeared. This model assumes that the crack directions are fixed and orthogonal, and only a specified number

of cracks at a material point are allowed. Cracks are detected with a Rankine crack initiation criterion: the maximum principal tensile stress exceeds the tensile strength of the concrete. The cracked shear modulus is reduced as the crack opens.

3. Concrete Damage Plasticity Model – This model does not use the smeared crack approach of the other two models. However, certain criteria can be adopted to visualize graphically the effective crack direction.

Table 6.3: Most important features in ABAQUS.

Masonry Modelling	------
Material Library	------
Input Parameters	• general properties (mass density, material damping, thermal expansion); • elastopastic mechanical properties(stress-strain data, Isotropic hardening, linear kinematic hardening • inelastic mechanical properties; • thermal properties;
Structural Analysis	• Linear static analysis • Nonlinear static analysis • frictional sliding • Dynamic stress/displacement analysis • Brittle cracking model • Damaged Plasticity Model • Evolution of failure controlled by hardening

6.2.3 ADINA

ADINA stands for Automatic Dynamic Incremental Nonlinear Analysis. This software was developed by ADINA R & D, Inc., a company whose mission is the development of the ADINA analysis system. ADINA R & D was founded by Dr. K.J. Bathe, author of Finite Element Procedures. The ADINA system is composed of many modules which together can simulate nonlinear analysis of solids and structures, thermo-mechanical coupled analysis, compressible and incompressible flow, and fluid-structure interaction.

ADINA has an available “Brittle Material Model” that can be used with 2D or 3D solid elements which assumes small strains regardless of use with small or large displacement formulations. In compression, the brittle material model assumes a nonlinear stress-strain relation that includes softening behaviour. The multiaxial stresses train relations are formulated based on the uniaxial stress-strain relation as shown in (Fig.6.8). Tensile response is linear until tensile failure corresponding to the tensile capacity of the brittle material. The unloading response is linear back to the origin, however, the slope depends on how far along the monotonic loading regime the unloading begins. If unloading is initiated at a point prior to the ultimate uniaxial compressive strain, the slope is the initial tangent modulus. If unloading is initiated at a strain larger the ultimate uniaxial compressive strain, the unloading slope is a function of the stresses and strains corresponding to ultimate and maximum compressive stress.

The tensile and compressive crushing failures are governed by respective failure envelopes to "establish the uniaxial stress-strain law accounting for multiaxial stress conditions, and to identify whether tensile or crushing failure has occurred. Tensile failure occurs if the principle tensile stress exceeds the tensile failure stress. At this point, the normal and shear stiffness's across the failure plane are reduced. A smeared cracking approach is used in which tensile response can be based either on tension stiffening or fracture energy. Post- failure behaviour includes post tensile cracking, post compression crushing, and strain-softening.

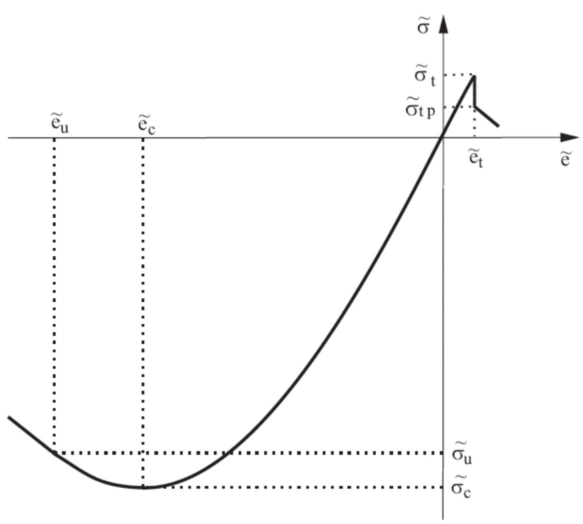


Figure 6.8: ADINA uniaxial stress-strain law for brittle materials (ADINA, 2012).

Crack propagation can be analysed in ADINA for 2-D solids using two techniques:

1) Node shift/release in which the propagation of the crack tip is modelled by shifting and releasing successive crack tip nodes through the mesh.

2) Node release-only in which the crack tip node is not shifted, but only released.

Table 6.4: Most important features in ADINA.

Masonry Modelling	------
Material Library	Yes (but not for masonry)
Input Parameters	• Compressive strength. • Modulus of elasticity. • Shear modulus. • Shear strength. • Unit weight.
Structural Analysis	• Linear analysis • Non-linear analysis evolution of failure controlled by hardening

6.2.4 DIANA

DIANA has been under development at the Department of Computational Mechanics at the TNO Building and Construction Research in the Netherlands since 1972. It is a multi-purpose finite element program based on the displacement method that is robust in the areas of concrete and soil. In addition to its nonlinear capabilities, it offers a variety of analysis types such as: linear static, dynamic, Euler stability, potential flow, and nonlinear dynamic analysis, to name a few. It is common in DIANA to combine a smeared cracking model for tension with a plasticity model for compression, such as Mohr-Coulomb or Drucker-Pragar, both of which consider strain hardening. As an alternative to specifying two separate models, the user can choose one of the following three special brittle material plasticity models which can handle both tension and compression:

1. Rankine.
2. Rankine/Von Mises.
3. Rankine/Drucker-Prager.

In each case, the Rankine criterion bounds the tensile stresses and the latter two cases, either the Von Mises or Drucker-Prager criterion is applicable in the compression region.

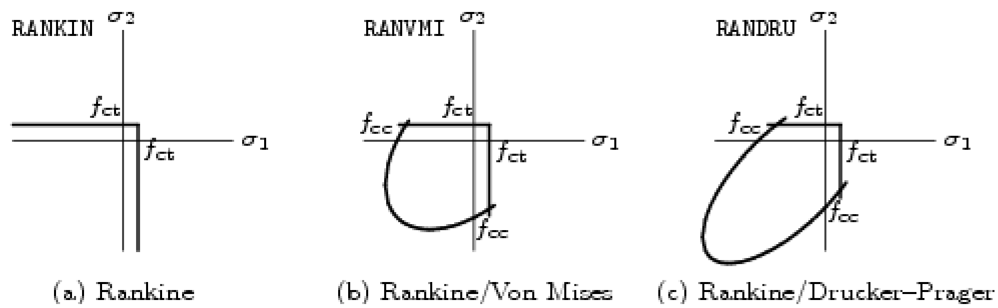


Figure 6.9: DIANA Rankine Plasticity Models (DIANA-9.4.2, 2010).

Each of these models can be combined with hardening/softening models to better predict response. The available hardening/softening models can be seen in (Fig.6.10):

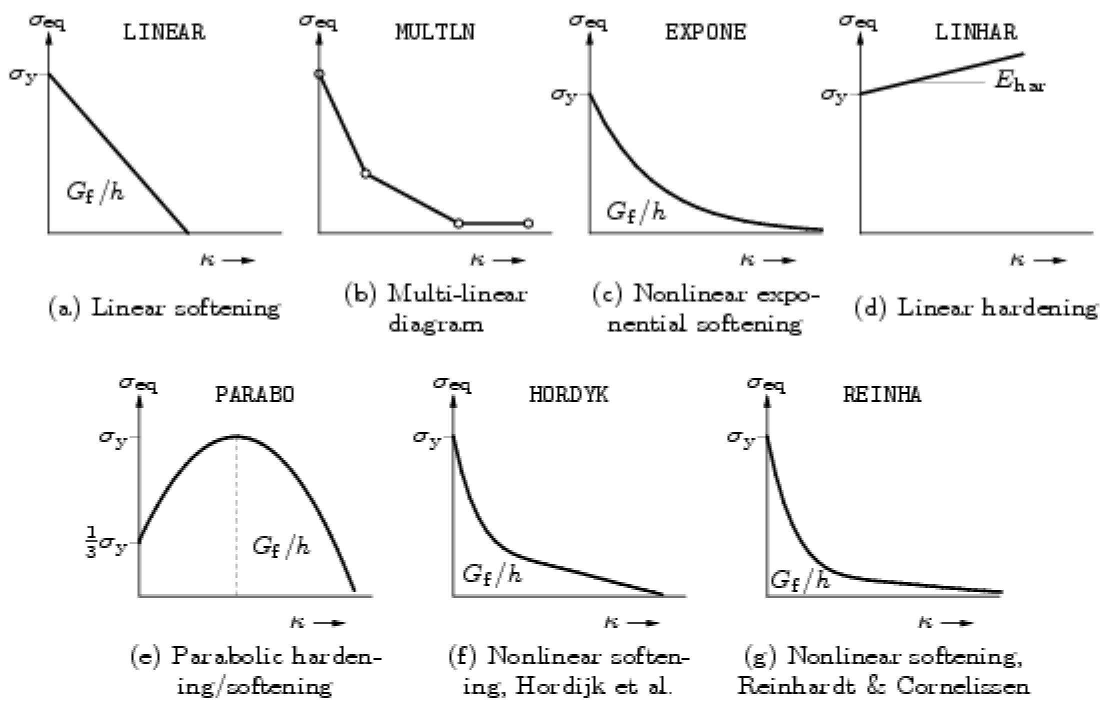


Figure 6.10: DIANA Hardening/softening models (DIANA-9.4.2, 2010).

The available cracking models in DIANA are: smeared cracking and total strain crack models based on fixed and rotating crack concepts.

Smeared Cracking

This model, also called multi-directional cracking, is fundamentally based on strain decomposition in which the total strain is decomposed into elastic strain and cracking strain as well as three parameters; tension cut-off, tension softening, and shear retention. There are two tension cut-off models for which crack initiation are defined, constant and linear as in (Fig.6.11):

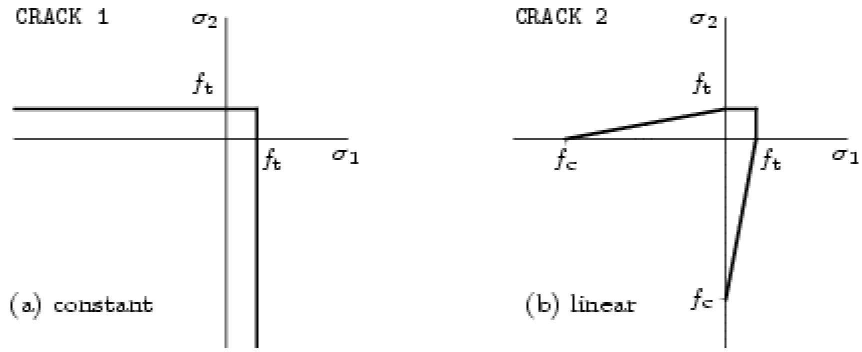


Figure 6.11: DIANA smeared cracking tension cut-off in twodimensional principal stress space (DIANA-9.4.2, 2010).

In the constant tension cut-off model, a crack arises if the major principal stress exceeds the tensile strength. In the linear tension cut-off model, a crack arises if the major principal tensile stress exceeds a minimum of two values, the tensile strength or, a formula that accounts for lateral principal stress. The available brittle, linear, multilinear, and nonlinear tension softening models for use with the smeared cracking are shown in (Fig.6.12):

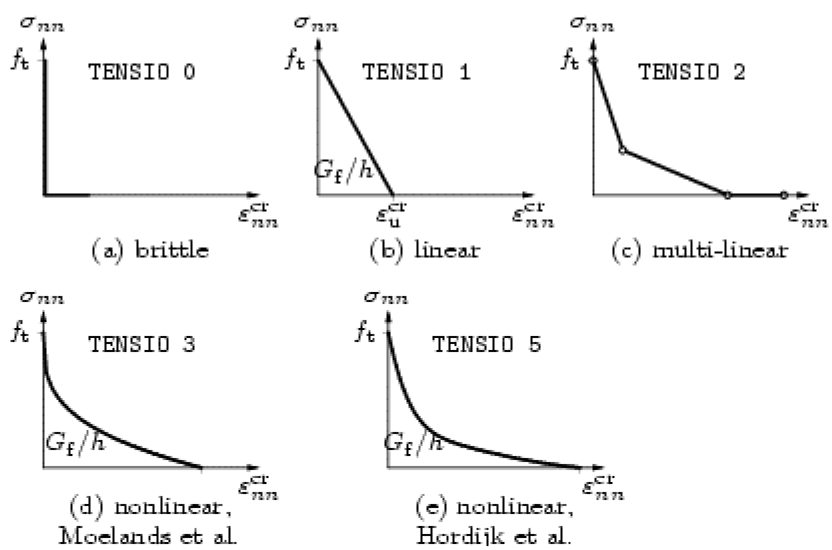


Figure 6.12: DIANA Smeared Cracking- Tension Softening (DIANA-9.4.2, 2010).

In order to account for the reduction in shear stiffness due to cracking, two shear retention relations are available for use with the smeared crack model: full and constant. In full shear retention, the shear modulus is not reduced, whereas, with constant shear retention, the cracked shear stiffness is reduced in relation to a shear retention factor β, which has a value less than one.

Total Strain Crack Models

The constitutive models based on total strain describe both the tensile and compressive response and are "developed along the lines of the Modified Compression Field Theory" (1999). There is also a three-dimensional version available that was proposed by Selby and Vecchio (1985). Three types of cracking models are available including:

1. Fixed crack model (constitutive relations are evaluated in a coordinate system that is fixed upon cracking).
2. Rotating crack model (constitutive relations are evaluated in the principal directions of the strain vector).
3. Non-orthogonal model (unlike the previous two, crack directions are not assumed to be orthogonal).

The available pre-defined compressive behavioural models for use with the total strain crack models are shown in (Fig.6.13). In addition, these models can be enhanced by adding an increase in compressive strength due to lateral confinement as proposed by Vecchio and Selby or a reduction duel to lateral cracking as proposed by Vecchio and Collins in 1993. Compression functions can also be customized by the user.

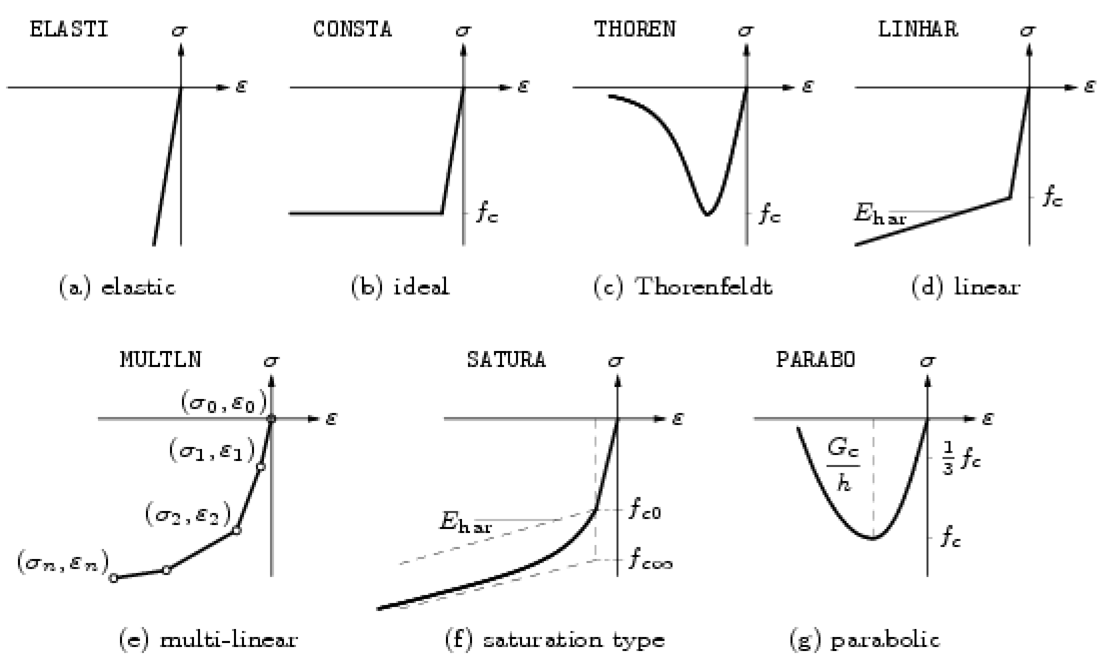


Figure 6.13: DIANA total strain crack model- Compressive behaviour (DIANA-9.4.2, 2010).

The available pre-defined tension softening behavioural models for use with the total strain crack models are shown in (Fig.6.14).

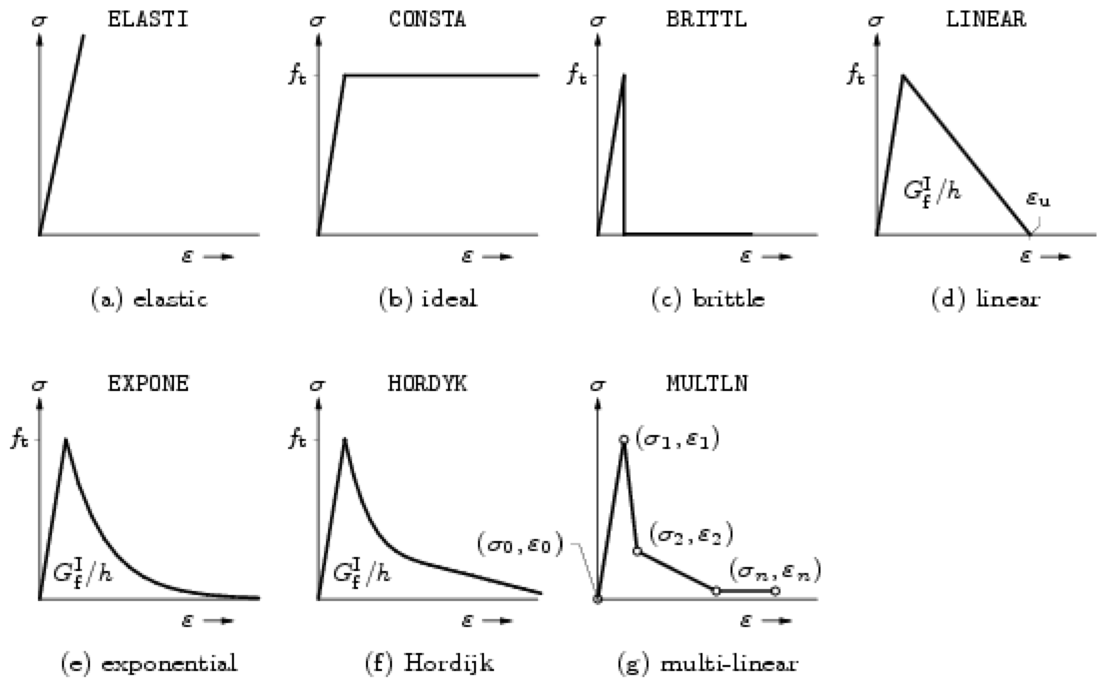


Figure 6.14: DIANA total strain crack model- Tension softening (DIANA-9.4.2, 2010).

The tension and compression stiffness degradation resulting from internal damage to the brittle materials are accounted for separately in the loading-unloading-reloading curves as shown in (Fig.6.15). Also, the user can define the hysteretic behaviour for use with the Non-orthogonal model.

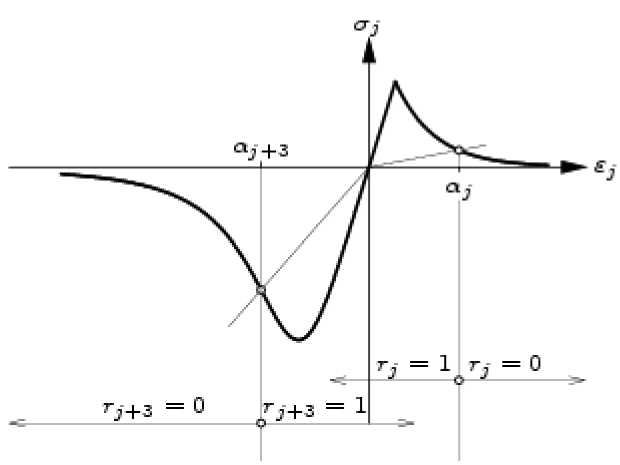


Figure 6.15: DIANA total strain crack models - Loading-unloading (DIANA-9.4.2, 2010).

Like in the smeared cracking models, a shear retention factor can be used to model the reduction in shear stiffness after cracking. This factor can range between zero and one with the fixed crack concept and is assumed to equal one in the rotating crack concept.

DIANA offers smeared and discrete crack modelling. The smeared crack models incorporate the full tensile response and therefore are described previously in the constitutive model section. Discrete cracks can be modelled with interface elements.

Table 6.5: Most important features in DIANA.

Masonry Modelling	Offering the multi-directional fixed crack model and the plasticity models to simulate cracking and crushing respectively for masonry elements.
Material Library	------
Input Parameters	• Properties for the masonry: linear elasticity, mass density, poisson ratio, coefficients of linear thermal expansion. • Properties for the interface elements: linear stiffness moduli, normal and shear traction. • Properties for discrete cracking in the interface elements: the stiffness moduli, the tensile strength, fracture energy. • Properties of inelastic behavior: tensile strength, composite interface model, tension softening criterion, fracture energy, cohesion, friction angle, shear softening criterion, compressive strength • Tension mode: bond tensile strength, bond fracture energy • Shear mode: bond strength, friction angle • Cap mode: compressive strength, compressive fracture energy, equivalent relative displacement
Structural Analysis	• Elasticity and viscoelasticity • Nonlinear elasticity and linear elastic • Plasticity and cracking • Creep and shrinkage • Interface analysis

6.2.5 SAP2000

SAP2000 is general-purpose civil engineering software ideal for the analysis and design of any type of structural system. Basic and advanced systems, ranging from 2D to 3D, of simple geometry to complex, may be modelled, analysed, designed, and optimized. It has been produced by Computers and Structures, Inc. (CSI) in United States of America.

The SAPFire ® Analysis Engine integral to SAP2000 drives a sophisticated finite-element analysis procedure. An additional suite of advanced analysis features are available to solve nonlinear and dynamic consideration. It is possible to implement advanced features for nonlinear and dynamic consideration. This versatility makes SAP2000 a practical tool for any analysis type ranging from simple static, linear-elastic to more complex dynamic, nonlinear-inelastic.

The SAPFire ® Analysis Engine drives analysis optimization with multiple 64-bit solvers. Options include Eigen analysis (with auto shifting for ill-conditioned relations) and Ritz analysis (for expedited convergence). P-delta effect captures geometric nonlinearity. Buckling analyses provide insight into structural stability through methods characterizing linear buckling (which considers multiple buckling modes under nonlinear-static or dynamic application), nonlinear buckling (which considers P-delta and large-deflection effects), snap-through buckling, and progressive collapse. Material nonlinearity capture inelastic and limit-state behaviour, along with such time-dependent phenomena as creep and shrinkage behaviour in reinforced-concrete systems. Plastic hinging may be specified in flexural members according to code-based standards or empirical data. Tension and compression-only springs may be assigned with limits and nonlinear attributes to simulate support plasticity.

Static and dynamic methods are available for earthquake simulation. Nonlinear-static-pushover analyses may consider modal, uniform, or user-defined lateral load patterns, plastic-hinging behaviour of slender elements, inelastic response of shear walls, floor slabs, and steel plates, and then formulate demand-capacity, damping, and performance-point calculations with customizable summary reports.

Dynamic methods include response-spectrum (for likely maximum seismic response given pseudo-spectral acceleration vs. structural period curve), power-spectral-density and steady-state (for fatigue behaviour with optional damping and complex-impedance properties), and time-history analyses. Time histories may follow modal or direct-integration methods, and they may be chained together and enveloped with such advanced analyses as P-delta and staged-construction procedures.

Staged-construction features are comprehensive. The construction sequence is scheduled with Gantt-chart options, enveloped with performance measures, and paired with analysis procedures. At each construction stage, evaluation may consider static or dynamic structural response, support reactions, geometric and material nonlinearity (including buckling, creep, and shrinkage), tendon and cable application with target-tensioning, etc. The Model Alive feature is available for small to medium-sized projects to analyse real or possible structural modifications.

Table 6.6: Most important features in SAP2000.

Masonry Modelling	------
Material Library	yes (but not for masonry)
Input Parameters	• The modulus of elasticity, for axial stiffness and bending stiffness. • The shear modulus, for torsional stiffness and transverse shear stiffness, this is computed from the modulus of elasticity and the poisson's ratio. • The mass density (per unit of volume), for computing element mass. • The weight density (per unit volume), for computing self-weight load.
Structural Analysis	• Linear static analysis • Nonlinear static analysis • Dynamic analysis • Nonlinear time-history analysis

6.2.6 STRAUS7

Strand7 is a Finite Element Analysis (FEA) software product developed by a group of academics from the University of Sydney and the University of New South Wales. Strand7 is most commonly used for the construction and mechanical engineering sectors, it includes the following solvers:

- Linear static
- Natural frequency
- Buckling
- Nonlinear static
- Linear and nonlinear transient dynamic
- Spectral and harmonic response
- Linear and nonlinear steady-state heat transfer
- Linear and nonlinear transient heat transfer

Table 6.7: Most important features in STRAUS7.

Masonry Modelling	------
Material Library	Yes (but not for masonry)
Input Parameters	• Modulus of elasticity. • Weight density. • Poisson's ratio. • Coefficient of thermal expansion. • Composite damping ratio. • Elasto-plastic materials. • Nonlinear elastic materials.
Structural Analysis	• Linear static solver • Linear buckling solver • Nonlinear static solver • Linear transient dynamic solver • Nonlinear transient dynamic solver • Natural frequency and natural response solver.

6.2.7 STAAD.Pro

STAAD.Pro is a structural analysis and design computer program originally developed by Research Engineers International in Yorba Linda, CA. In late 2005, Research Engineer International was bought by Bentley Systems. The commercial version STAAD.Pro is one of the most widely used structural analysis and design software. It supports several steel, concrete and timber design codes. It can make use of various forms of analysis from the traditional 1st order static analysis, 2nd order p-delta analysis, geometric nonlinear analysis or a buckling analysis. It can also make use of various forms of dynamic analysis from modal extraction to time history and response spectrum analysis. In recent years it has become part of integrated structural analysis and design solutions mainly using an exposed API called OpenSTAAD to access and drive the program using a VB macro system included in the application or other by including OpenSTAAD functionality in applications that themselves include suitable programmable macro systems.

Table 6.8: Most important features in STAADPRO.

Masonry Modelling	------
Material Library	Yes (but not for masonry)
Input Parameters	• Modulus of elasticity. • Weight density. • Poisson's ratio. • Coefficient of thermal expansion. • Composite Damping Ratio.
Structural Analysis	• Linear Elastic Analysis • P-Delta Analysis • Nonlinear Cable Analysis • Buckling Analysis • Steady State and Harmonic Analysis • Geometric Nonlinear Analysis

6.2.8 Scia Engineer

Scia Engineer is a finite element analyses, designs and details software for any type of structure. Produced by Nemetschek group (Nemetschek AG, Germany) is a vendor of software for architects, engineers and the construction industry. From the simplest to the most complex construction in concrete, steel, aluminum, plastic, timber or mixed, with integration of the local and international codes and with a link between the analysis and drawing components. Scia Engineer performs static, dynamic, stability, nonlinear and other special types of analysis. The results are directly used for design and checks according to appropriate technical building standards. Scia Engineer employs the displacement-based finite element method, but it does not work with finite elements directly. Instead, it exploits structural elements for which the finite element mesh is generated.

Table 6.9: Most important features in SCIA ENGINEERING.

Masonry Modelling	------
Material Library	Yes (but not for masonry)
Input Parameters	• Modulus of elasticity. • Weight density. • Poisson's ratio. • Coefficient of thermal expansion.

Structural Analysis	• Linear static analysis • Non-linear static analysis (tension only members, pressure only supports), Geometrical non-linearity • Advanced non-linear static analysis (springs and gaps for beams, pressure only slabs), Stability analysis, Dynamic analysis (eigenmodes, harmonic, seismic, general dynamic load) • Advanced calculations: Soil interaction, Cables, Non-linear stability, Membranes, Sequential analysis, Friction springs • Linear and non-linear construction stages • Pre-stressed structures, Time Dependent Analysis

6.3 A simple application: comparison between FEMs of a masonry panel

In this part several structural analyses were made using 6 different software solutions. While three of these software systems are professional finite element tools that can be used for multi and complex scientific fields, the others were stand alone structural simulation softwares. This comparison was done in order to determine if different results are obtained by different input parameters used for each software in linear analysis, besides to the general procedures of analysis adopted by each software with respect to building the model, meshing techniques and the outcomes representation . All simulations were done by modifying the standard settings proposed by the finite element analyses tools for each software in order to get an analysis in which the results can be comprised with the same finite element procedures and fixed meshing basics. The structural analyses were done with finite element meshes consisting of linear 4 node.

6.3.1 The analysis of masonry panel

This example illustrates the use of different softwares for the analysis of a masonry wall with a centred opening (Fig.6.17). A vertical load (V = 5kN/m) is applied on the top surface of the wall before shearing the wall with a horizontal force F = 2 kN and F = 20 kN in case of linear analysis. A shear force of F = 200 kN was applied with respect to nonlinear analysis, in order to study the difference in masonry panel performance along different loading conditions regarding each failure criterion adopted by the softwares. The opening (50x50 cm) in the centre of the ground fixed wall (200x150x25 cm) forces the compressive strut, which arises during loading, to spread around it. This leads to high diagonal stress distribution from two corners of the opening at the top and bottom. At collapse, four rigid blocks are formed. The chosen material and load parameters represent the elastic and inelastic behaviour of the historical masonry wall (Table 6.9) and (Fig. 6.16).

Table 6.10: Material parameters for macro analysis of homogenous-isotropic masonry wall.

Parameter	Unit weight	Modulus of elasticity	Poisson's ratio	Thermal expansion
value	20 kN/m^3	4988 MPa	0.15	5.58e-6 m/c

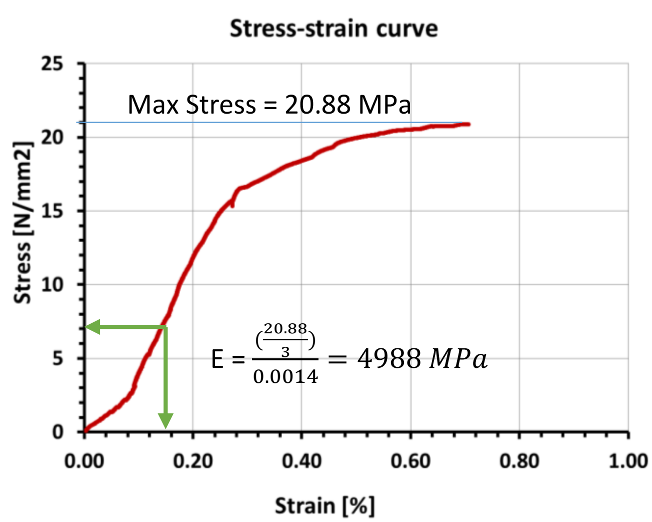


Figure 6.16: Experimental stress-strain curve of masonry in compression.

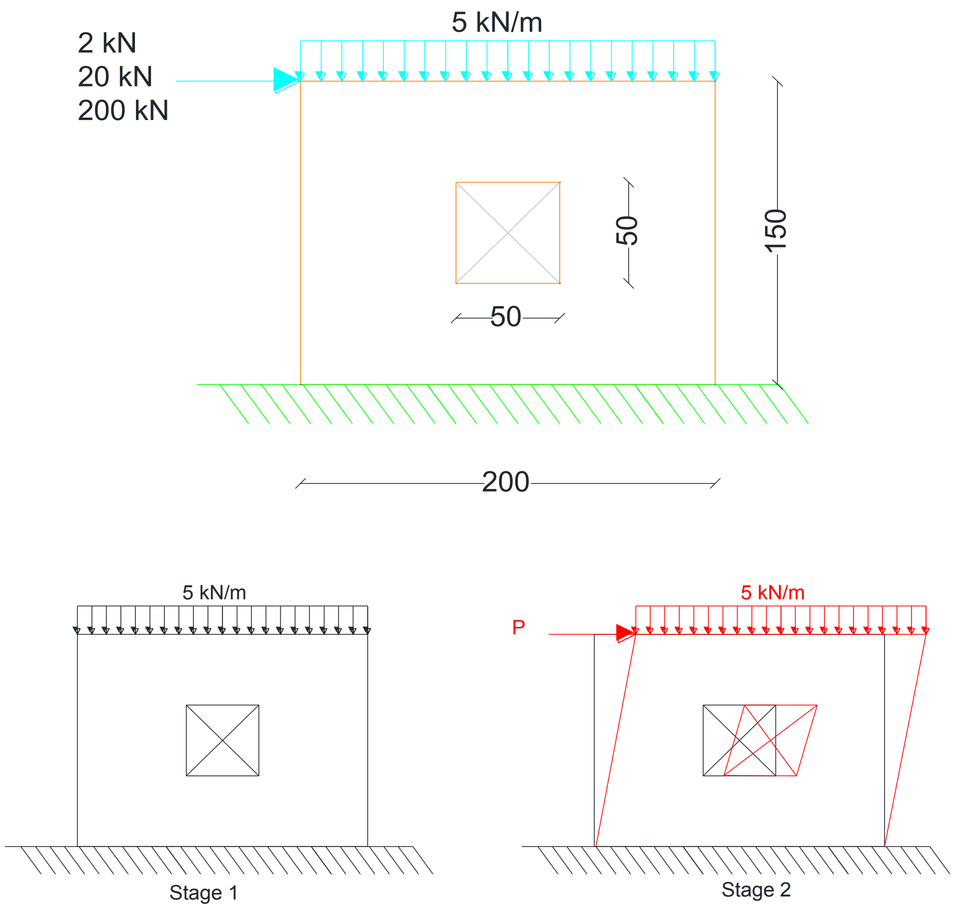


Figure 6.17: The dimensions and applied loads of the masonry wall model.

Different FEM softwares were used to perform linear and nonlinear analysis of the masonry wall under the assumption of a ground-fixed wall subjected to vertical and lateral in-plan forces (Fig.6.17). The FEM softwares graphic user interface is used to model, analyze, design, and display the structure geometry, properties and analysis results. The analysis procedure can be divided into three parts:

1. **Pre-processing:**

   The following information is needed in the preprocessing phase by FEM softwares:

   1. Choosing the units.
   2. Setting up geometry.
   3. Defining material and section properties of structural elements.

4. Assigning member section properties.
5. Defining load cases.
6. Meshing structural elements.
7. Assigning load magnitudes.
8. Assigning restraints.

2. **Solving:**

   In this part FEM softwares will assemble and solve the global matrix. The following steps are needed:

   1. Selecting analysis option.
   2. Checking the available DOF.
   3. Running the analysis.

3. **Post-processing:**

The main options in post-processing are:

1. Displaying the deformed shape.
2. Displaying the wall stress.
3. Obtaining the numerical results.
4. Modifying the structure.

Modelling strategies

It is important for modelling the masonry to understand the fundamental behaviour of its material which affects its global structural behaviour. For material characterisation, homogenised isotropic approach has been applied. For structural characterisation, simplified macro modelling has been performed.

The wall is subjected to normal loading conditions without time stepping, which gives an indication of stress distribution and deformation. Cracking is an important issue under loading conditions, but taking in consideration fracture mechanics approach would require a full mechanical characterization of mortar and brick units.

Geometrical modelling

Masonry walls are considered as an application of earthquake resisting shear walls, which can generally be represented as plane-stress bodies, adopting 2D meshed elements using plane stress, which saves time and computational efforts but it will reflect on the accuracy of the obtained results. However as macro modelling is used, the properties of the units, mortar, as well as the interface properties of interacting surfaces between units and mortar are neglected.

Failure Criteria

The failure of the masonry wall can be based on the mechanical behaviour at every loading step, so once the equivalent stress and strain in the masonry wall are calculated by modelling with respect to imposed constrains, the corresponding stress and strain of masonry which have been defined by the user is derived based on the structural relationship, and compared with the tensile and compressive strength defined before the analysis for each material. If the maximum principal stress exceeds the tensile strength at current step, the stiffness contribution of the whole structural element is to be negligible.

There are a number of criteria for the masonry model such as Mohr-Coulomb and Drucker Prager, in which the software determines the tensile failure referring to the input regarding each criteria.

- **SAP2000**

The nonlinear analysis has been done by using Drucker-Prager criterion (Friction angel = 37°) and stress-strain curve with respect to the values shown in (Fig.6.16), which represent the minimum requirement to perform the nonlinear analysis by the software.

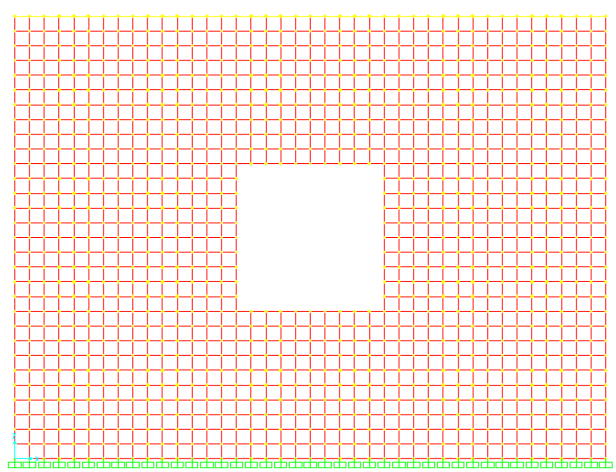


Figure 6.18: Finite Element Model by SAP2000.

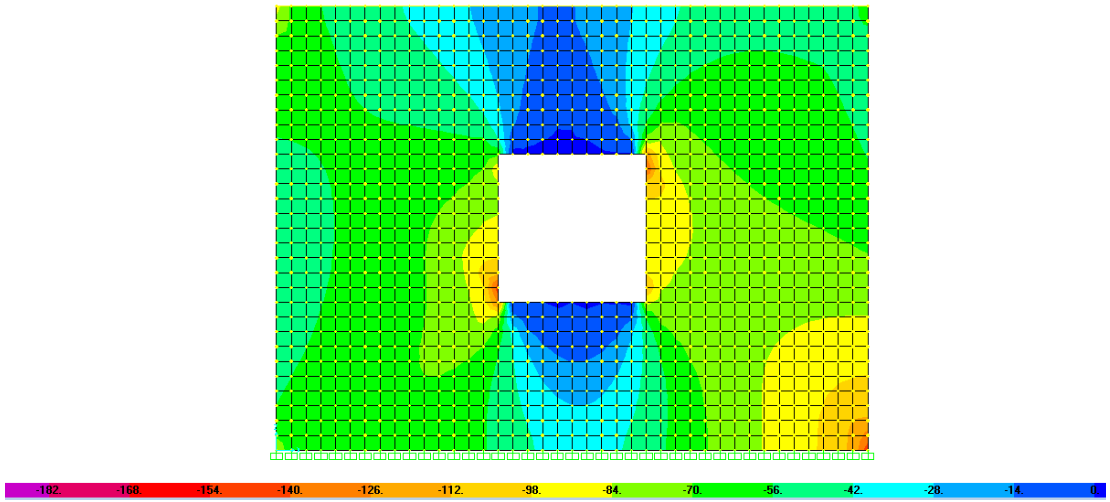


Figure 6.19: Linear analysis stress distribution y-y (2 kN) by SAP2000.

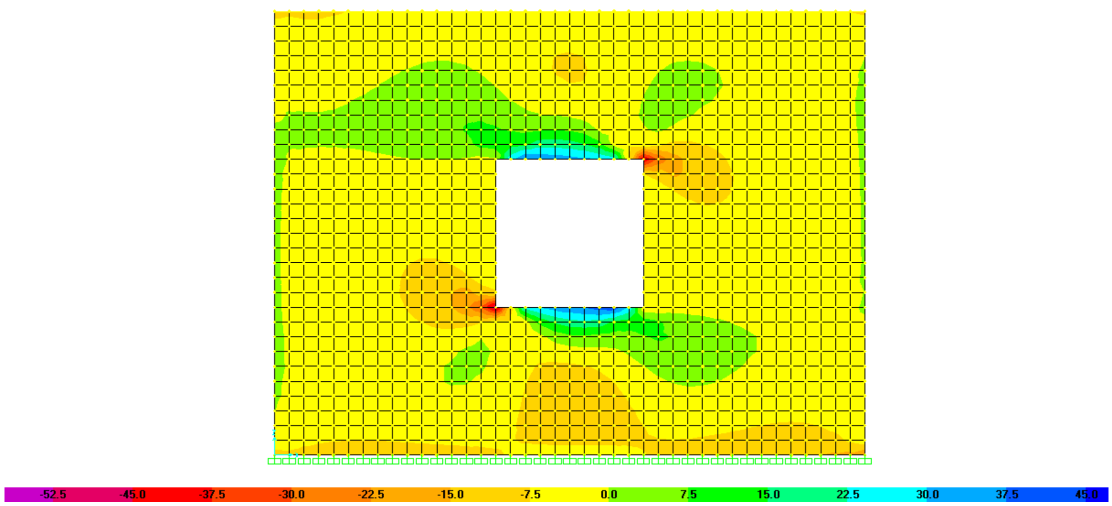


Figure 6.20: Linear analysis stress distribution x-x (2 kN) by SAP2000.

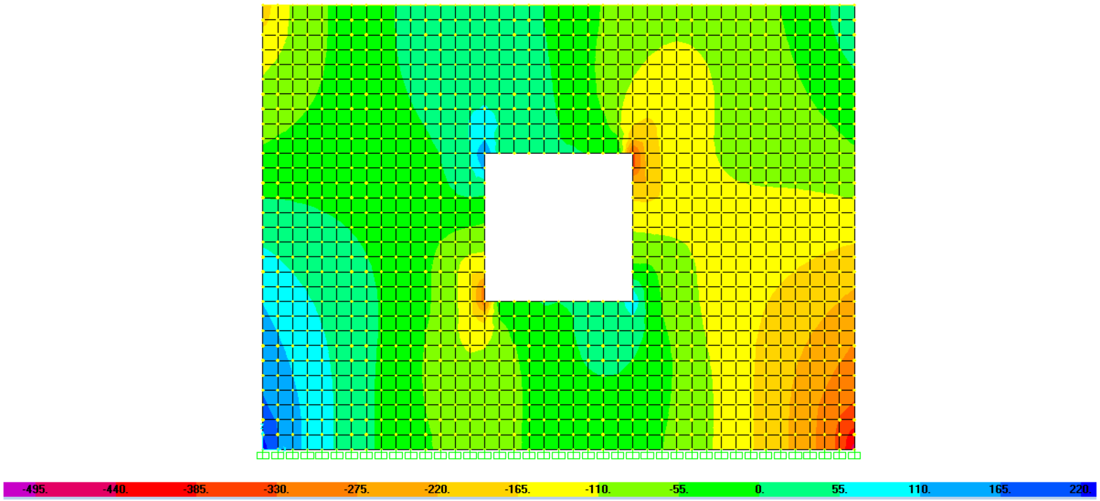


Figure 6.21: Linear analysis stress distribution y-y (20 kN) by SAP2000.

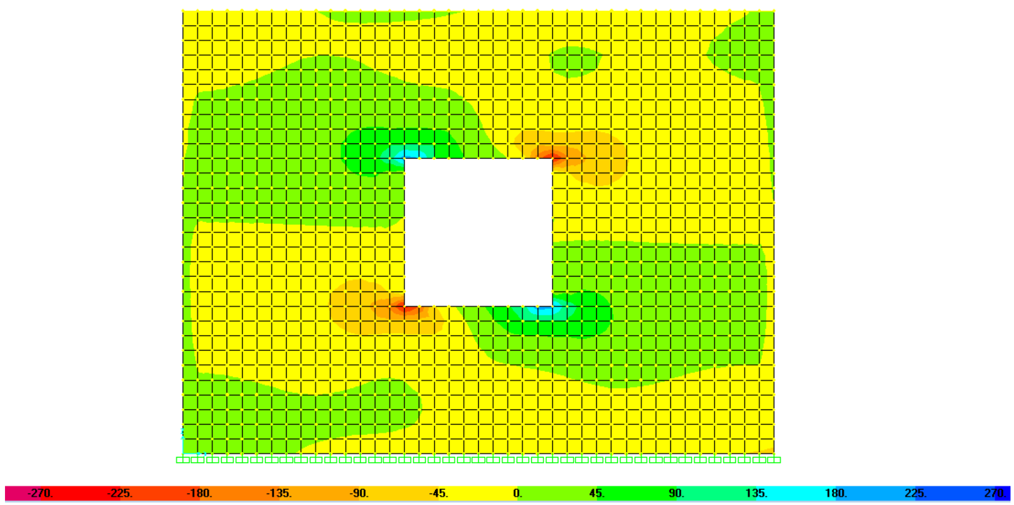


Figure 6.22: Linear analysis stress distribution x-x (20 kN) by SAP2000.

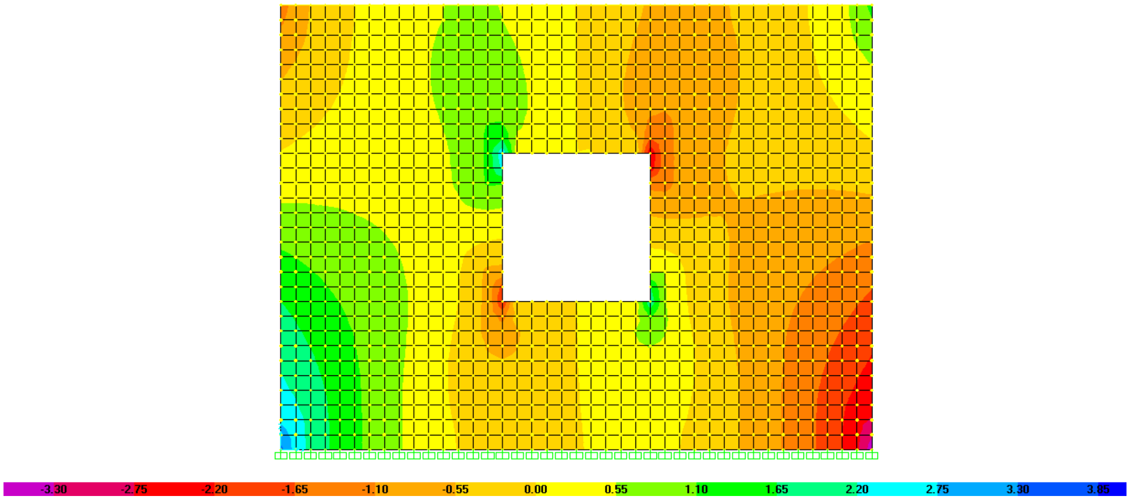
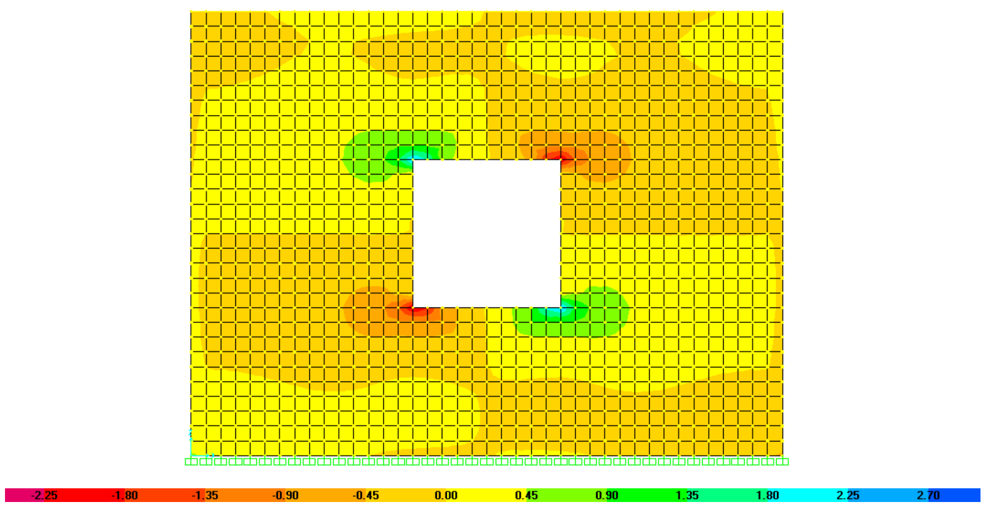


Figure 6.23: Nonlinear analysis stress distribution y-y (200kN) by SAP2000.



- *Figure 6.24: Nonlinear analysis stress distribution x-x (200kN) by SAP2000.*

- STRAUS7

The nonlinear analysis has been done by using Mohr Coulomb criterion (Friction angel = 37°, Cohesion = 0.3 MPa) and stress-strain curve with respect to the values shown in (Fig.6.16), which represent the minimum requirement to preform the nonlinear analysis by the software.

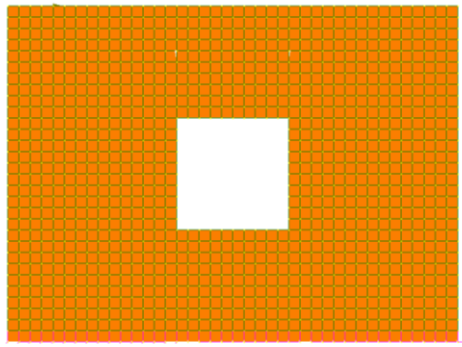


Figure 6.25: Finite Element Model by STRAUS7.

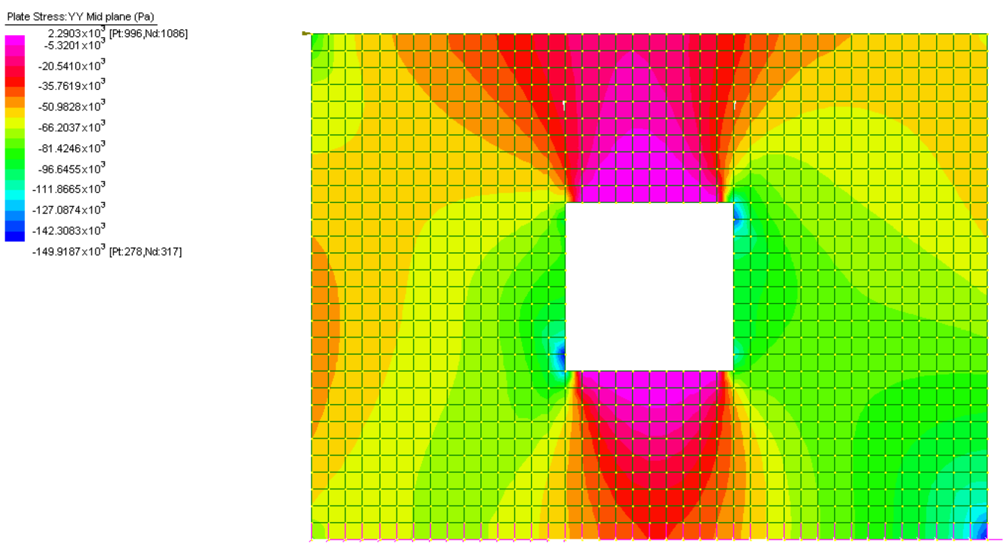


Figure 6.26: Linear analysis stress distribution y-y (2 kN) by STRAUS7.

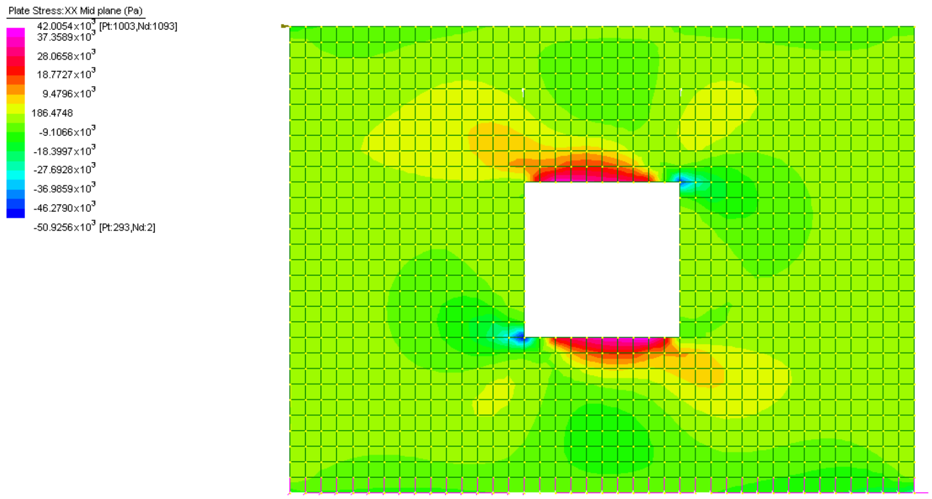


Figure 6.27: Linear analysis stress distribution x-x (2 kN) by STRAUS7.

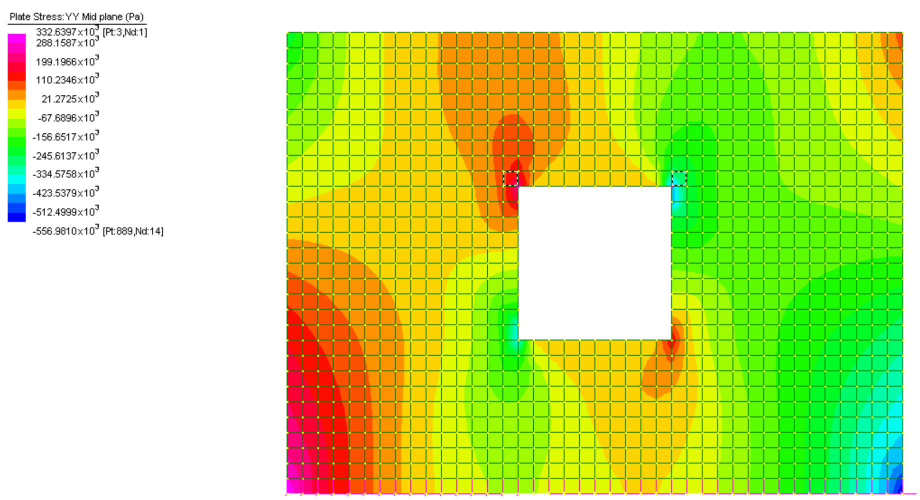


Figure 6.28: Linear analysis stress distribution y-y (20 kN) by STRAUS7.

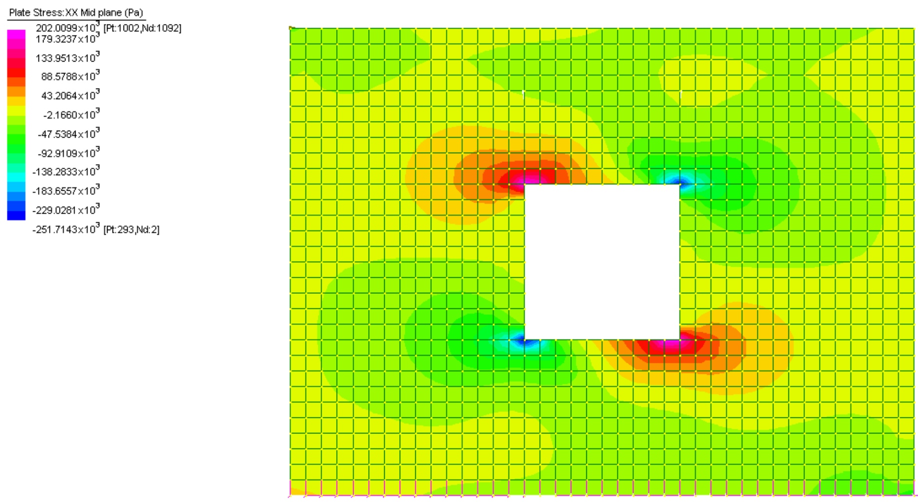


Figure 6.29: Linear analysis stress distribution x-x (20 kN) by STRAUS7.

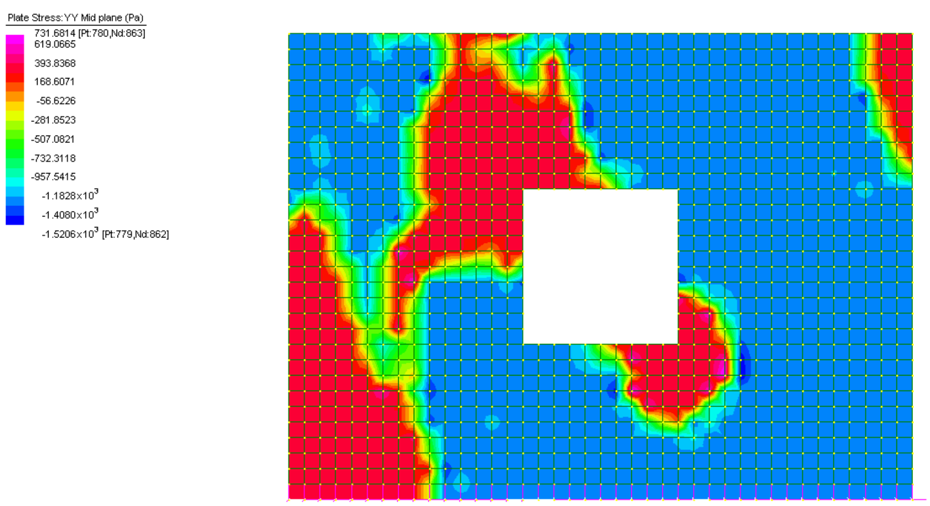


Figure 6.30: Nonlinear stress distribution y-y (200kN) by STRAUS7.

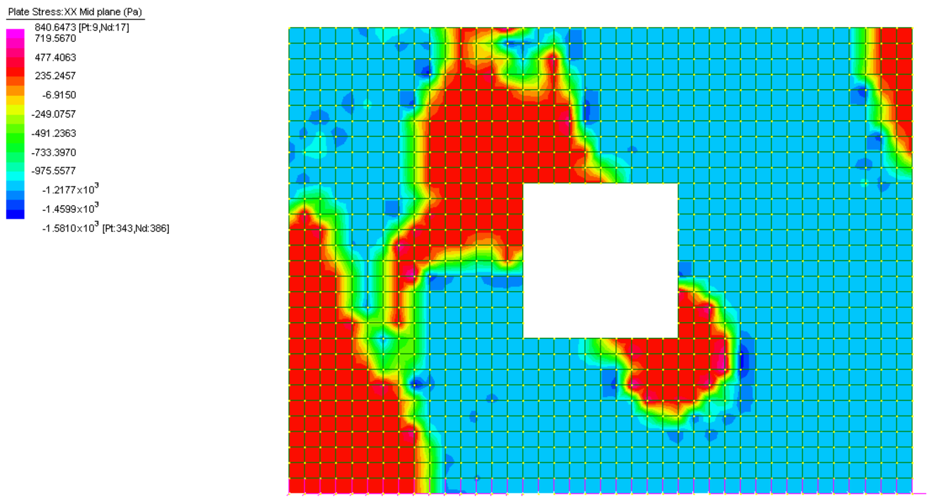


Figure 6.31: Nonlinear stress distribution x-x (200kN) by STRAUS7.

- STAAD Pro

The nonlinear analysis has been done by using stress-strain curve with respect to the values shown in (Fig.6.16).

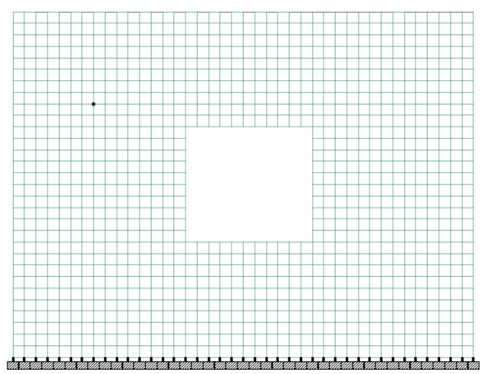


Figure 6.32: Finite Element Model by STAAD.

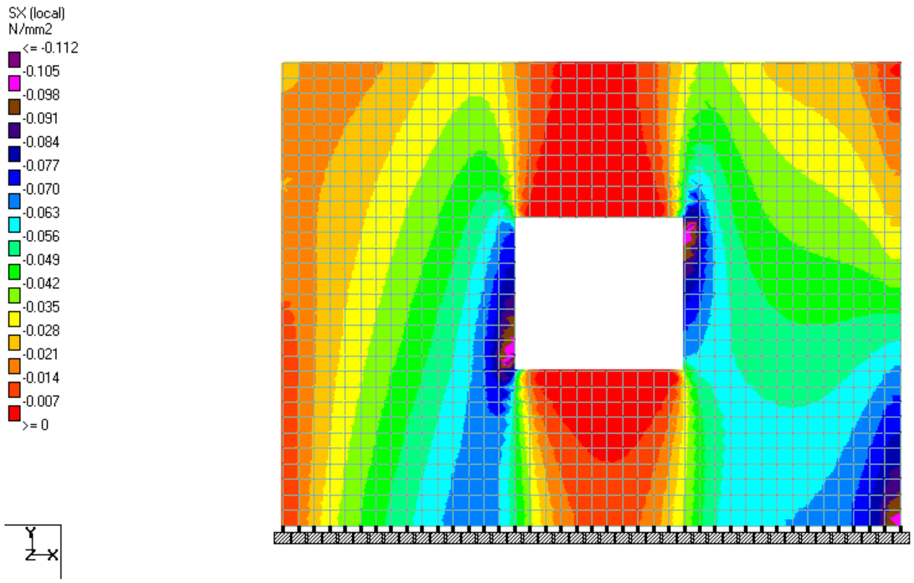


Figure 6.33: Linear analysis stress distribution y-y (2 kN) by STAAD.

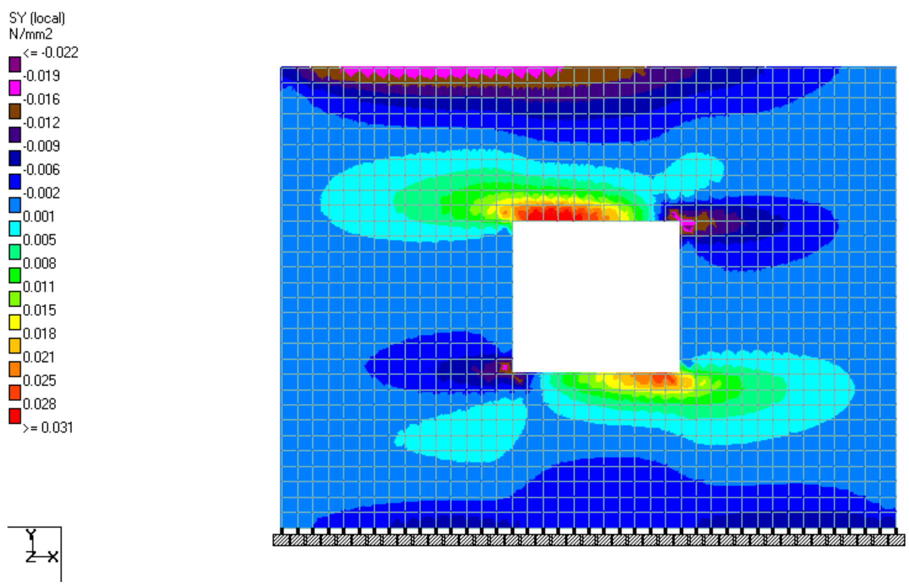


Figure 6.34: Linear analysis stress distribution x-x (2 kN) by STAAD.

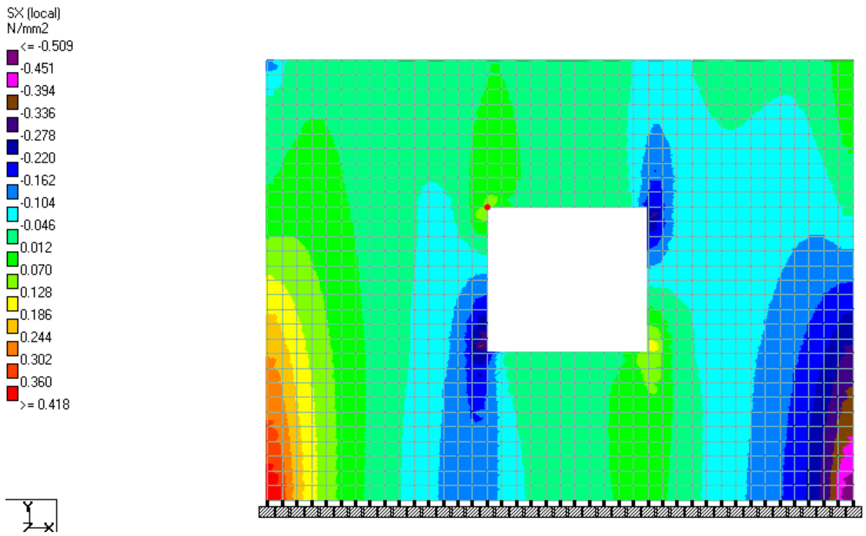


Figure 6.35: Linear analysis stress distribution y-y (20 kN) by STAAD.

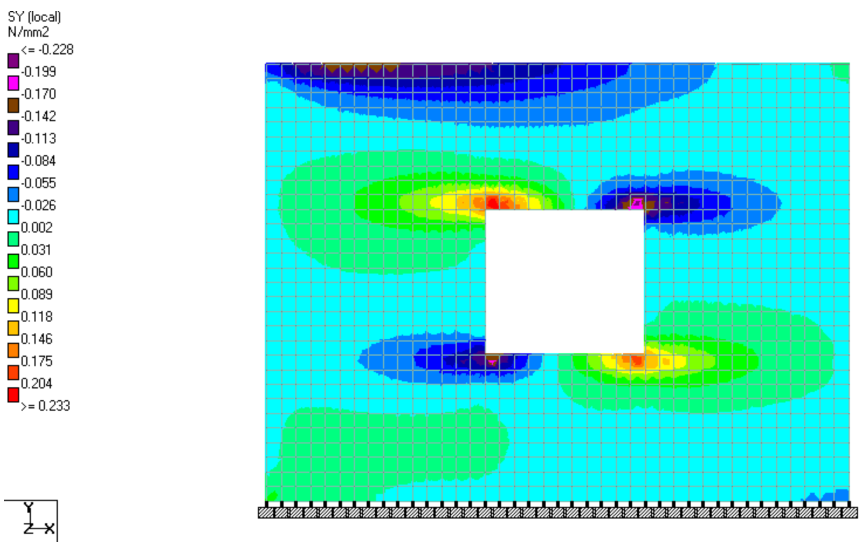


Figure 6.36: Linear analysis stress distribution x-x (20 kN) by STAAD.

- SCIA

The nonlinear analysis has been done by using stress-strain curve with respect to the values shown in (Fig.6.16).

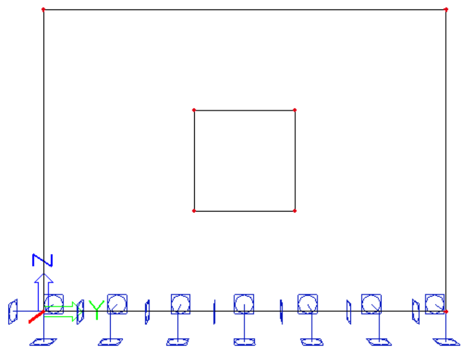


Figure 6.37: Finite Element Model by SCIA.

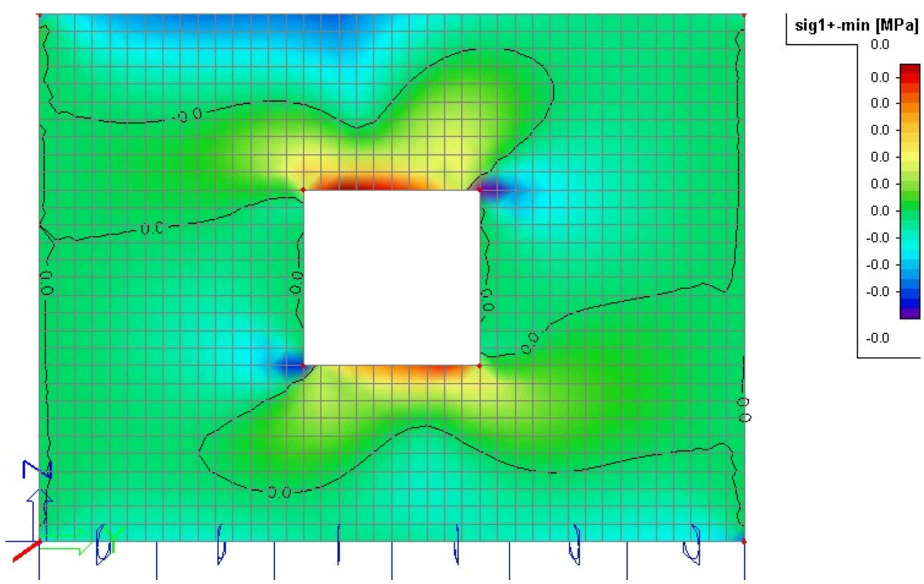


Figure 6.38: Linear analysis Stress Distribution y-y (2 kN) by SCIA.

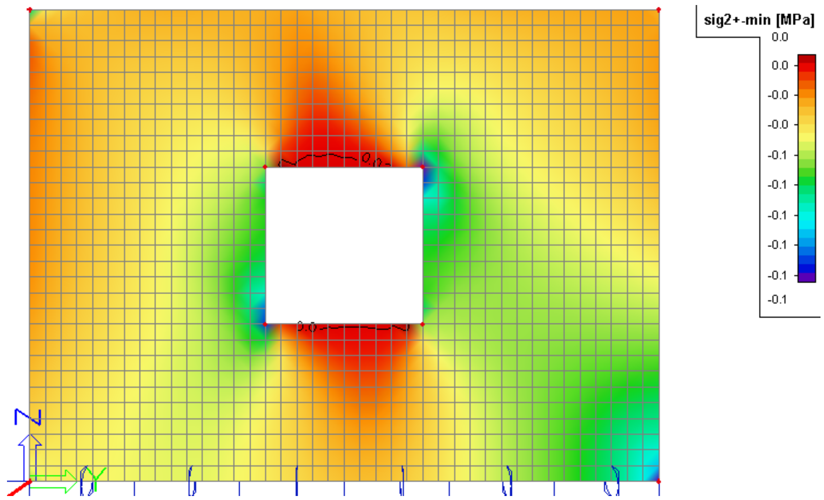


Figure 6.39: Linear analysis Stress Distribution x-x (2 kN) by SCIA.

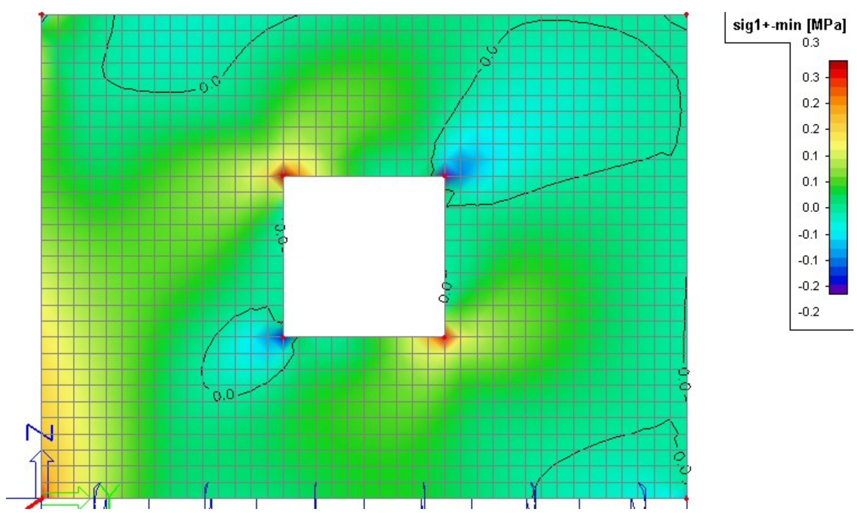


Figure 6.40: Linear analysis Stress Distribution y-y (20 kN) by SCIA.

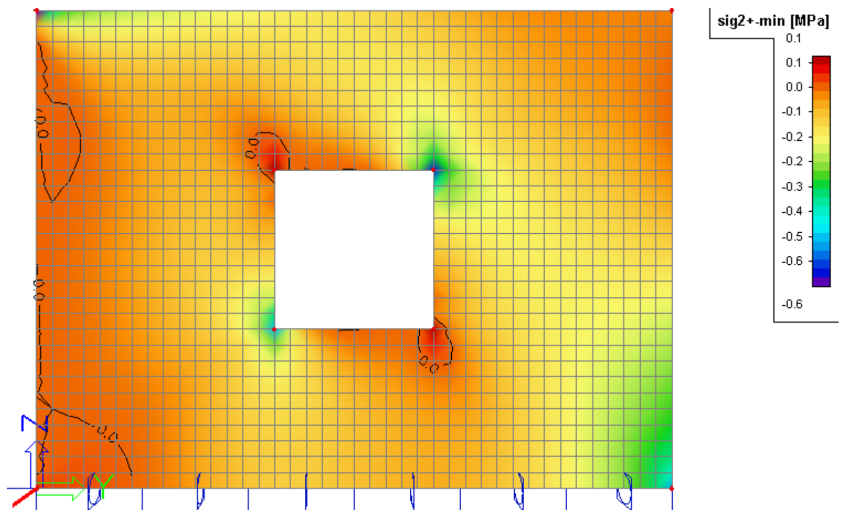


Figure 6.41: Linear analysis Stress Distribution x-x (20 kN) by SCIA.

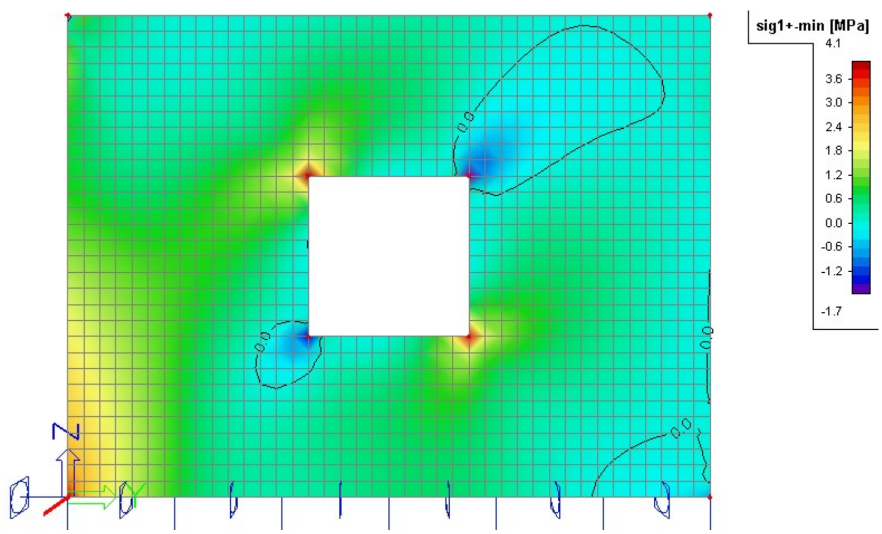


Figure 6.42: Nonlinear analysis Stress Distribution y-y (200 kN) by SCIA.

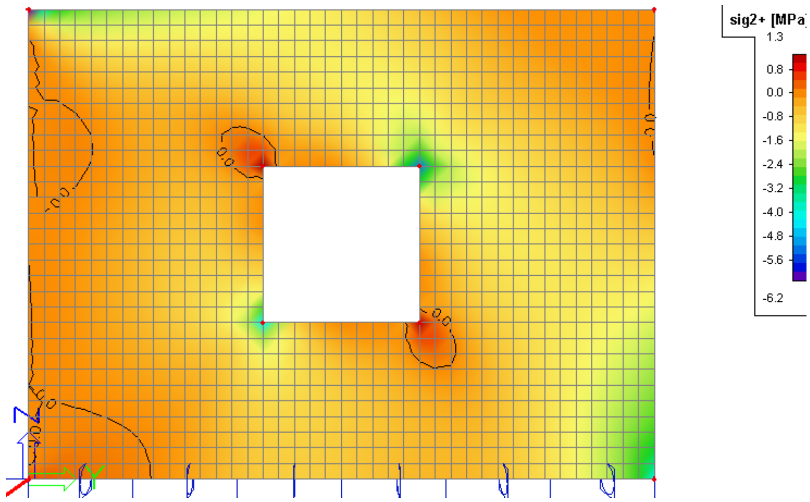


Figure 6.43: Nonlinear analysis Stress Distribution x-x (200 kN) by SCIA.

- ABAQUS

The nonlinear analysis has been done by using Mohr Coulomb criterion (Friction angel = 37°, Cohesion = 0.3 MPa) and stress-strain curve with respect to the values shown in (Fig.6.16), which represent the minimum requirement to perform the nonlinear analysis by the software.

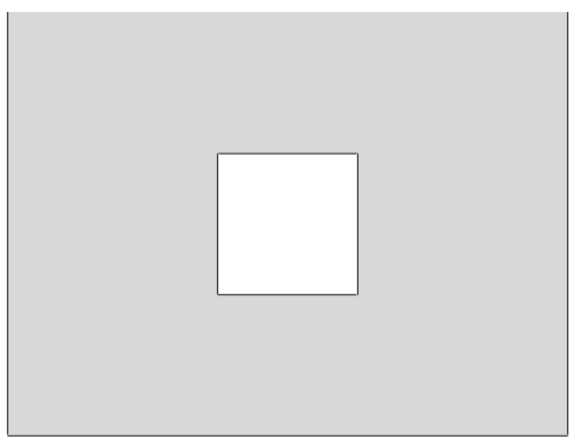


Figure 6.44: Finite Element Model by ABAQUS.

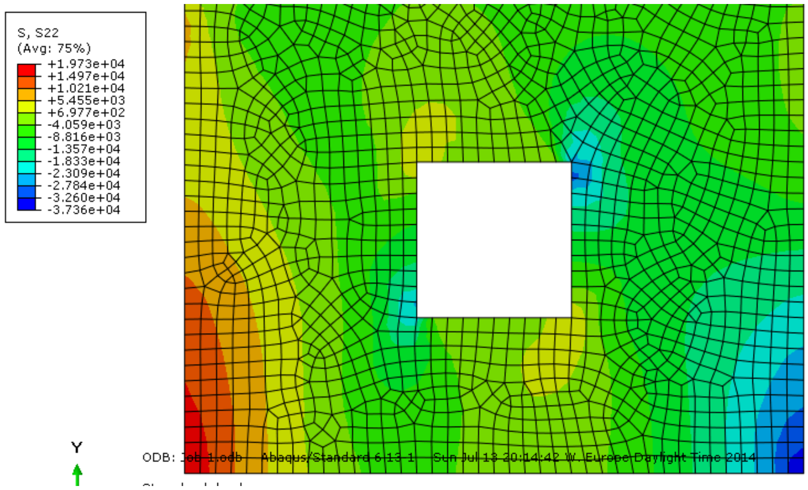


Figure 6.45: Linear analysis Stress Distribution y-y (2 kN) by ABAQUS.

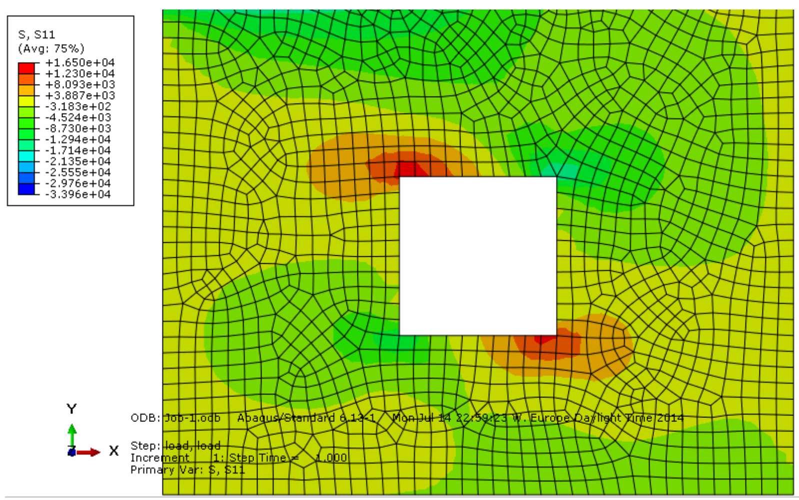


Figure 6.46: Linear analysis Stress Distribution x-x (2 kN) by ABAQUS.

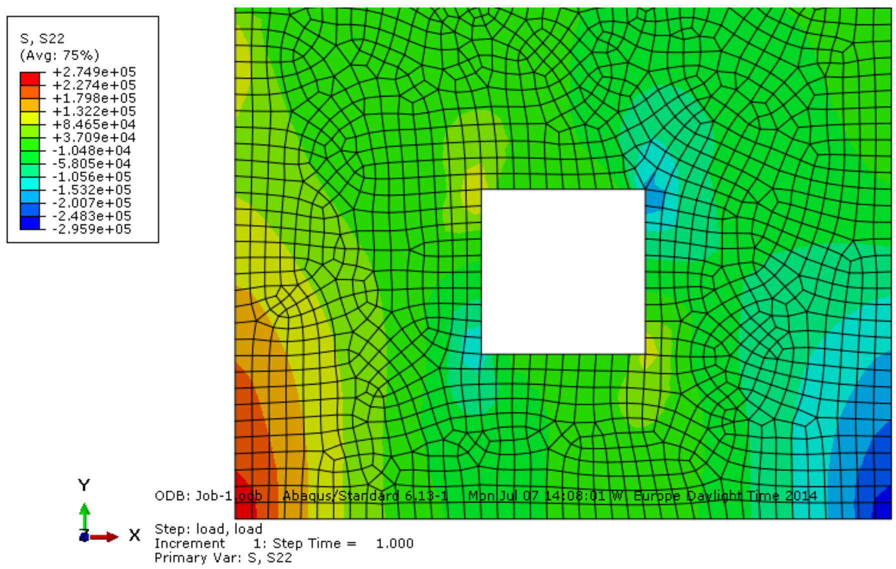


Figure 6.47: Linear analysis Stress Distribution y-y (20 kN) by ABAQUS.

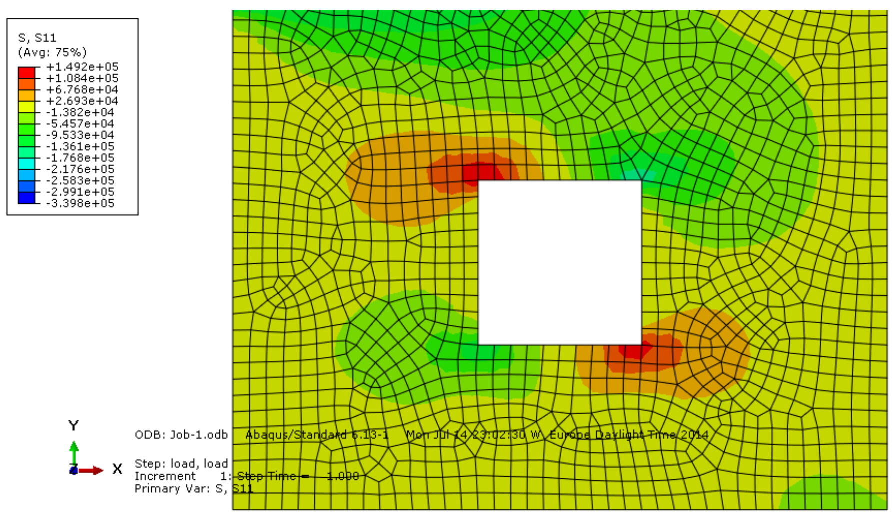


Figure 6.48: Linear analysis Stress Distribution x-x (20 kN) by ABAQUS.

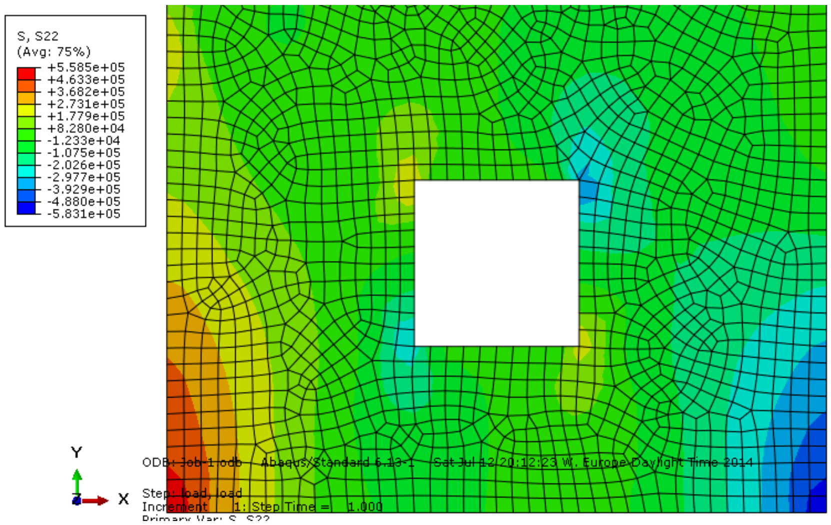


Figure 6.49: Nonlinear analysis Stress Distribution y-y (200 kN) by ABAQUS.

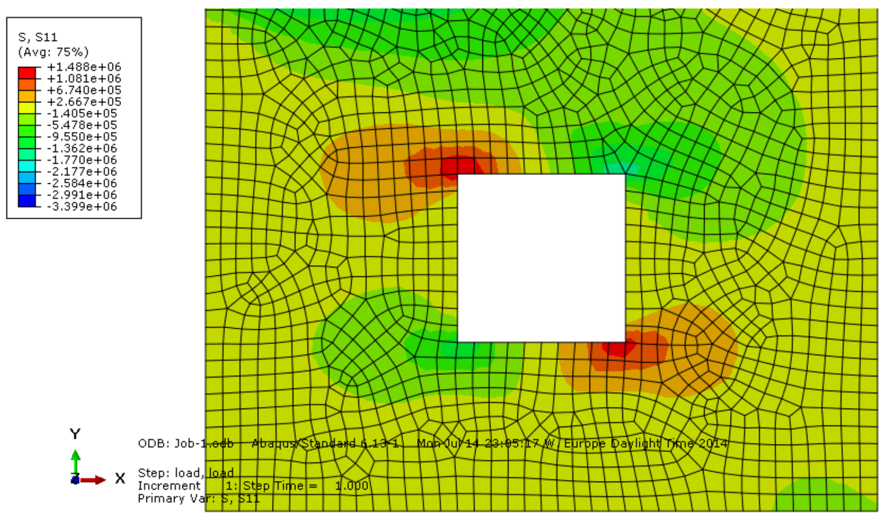


Figure 6.50: Nonlinear analysis Stress Distribution x-x (200 kN) by ABAQUS.

- ANSYS:

The nonlinear analysis has been done by using Mohr Coulomb criterion (Friction angel = 37°, Cohesion = 0.3 MPa) and stress-strain curve with respect to the values shown in (Fig.6.16).

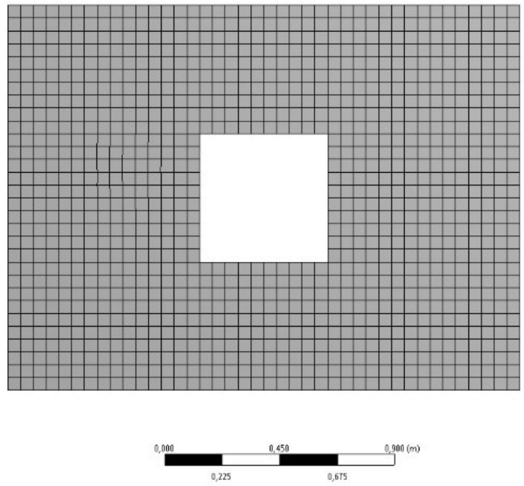


Figure 6.51: Finite Element Model by ANSYS.

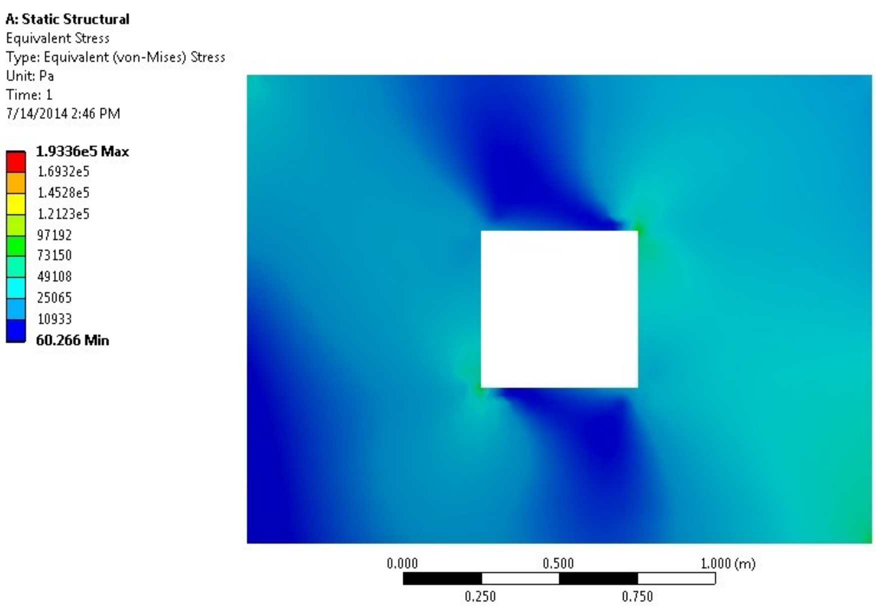


Figure 6.52: Linear analysis Stress Distribution y-y (2 kN) by ANSYS.

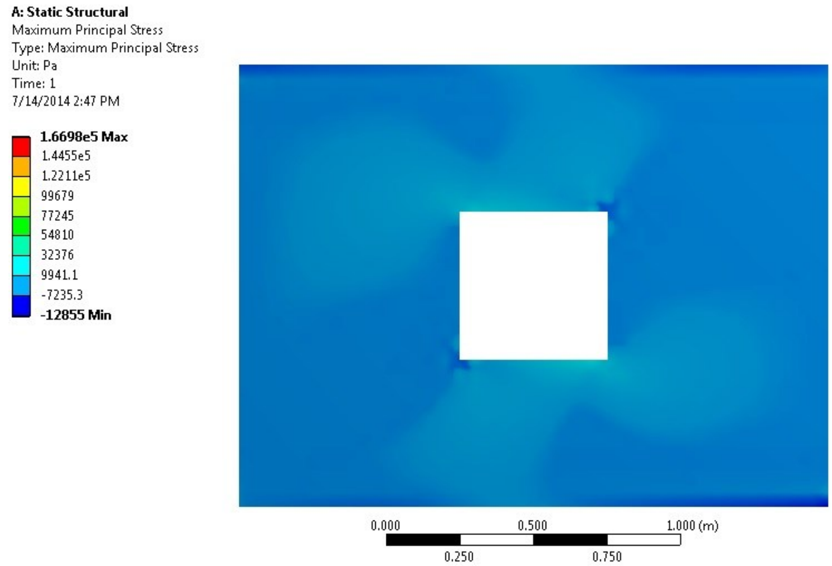


Figure 6.53: Linear analysis Stress Distribution x-x (2 kN) by ANSYS.

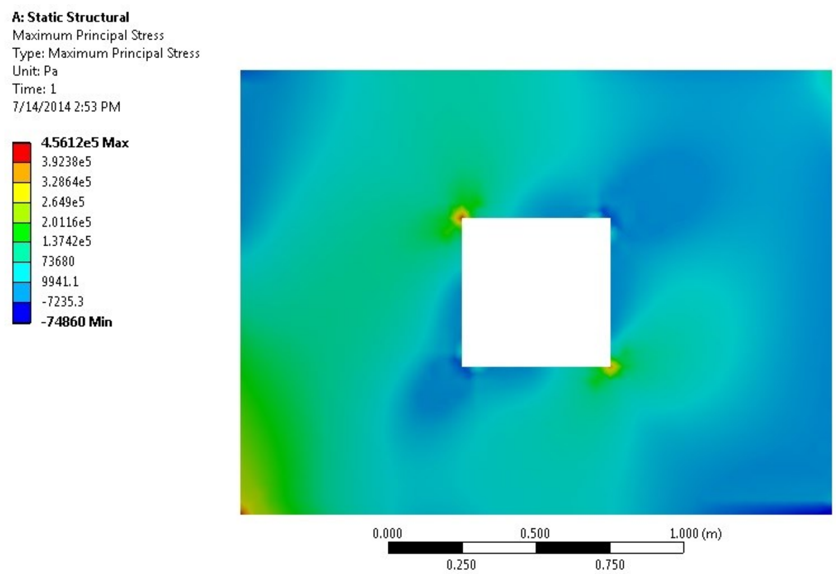


Figure 6.54: Linear analysis Stress Distribution y-y (20 kN) by ANSYS.

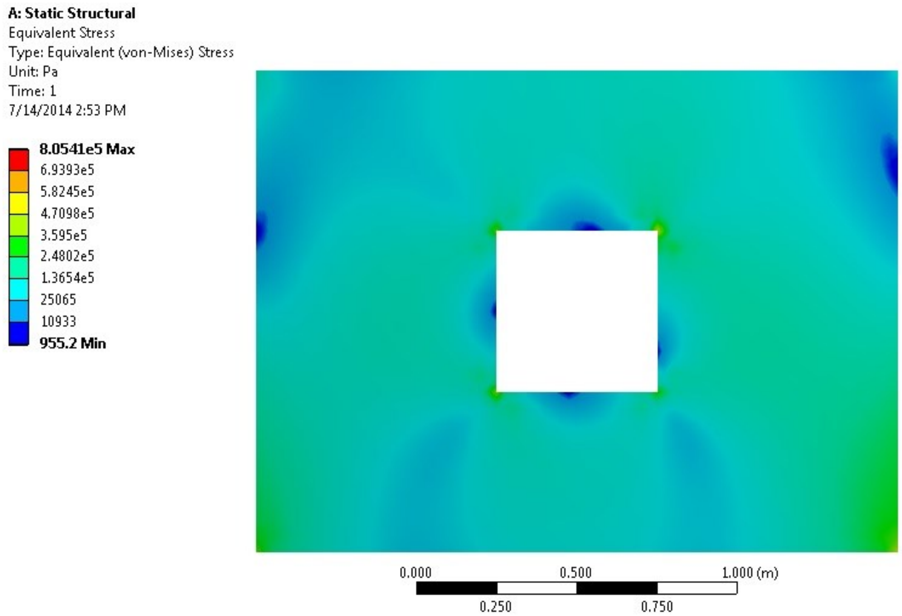


Figure 6.55: Linear analysis Stress Distribution x-x (20 kN) by ANSYS.

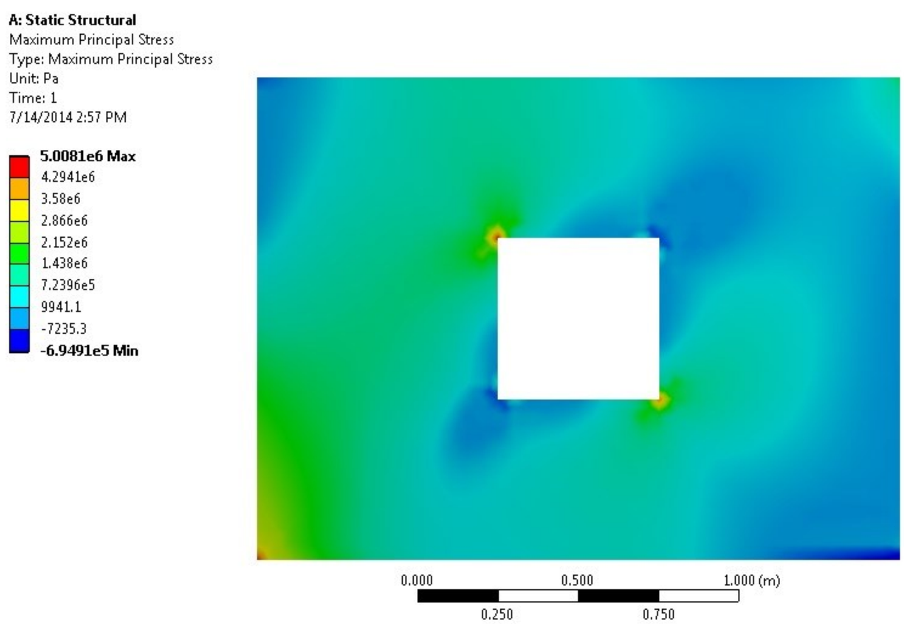


Figure 6.56: Nonlinear analysis Stress Distribution y-y (200 kN) by ANSYS.

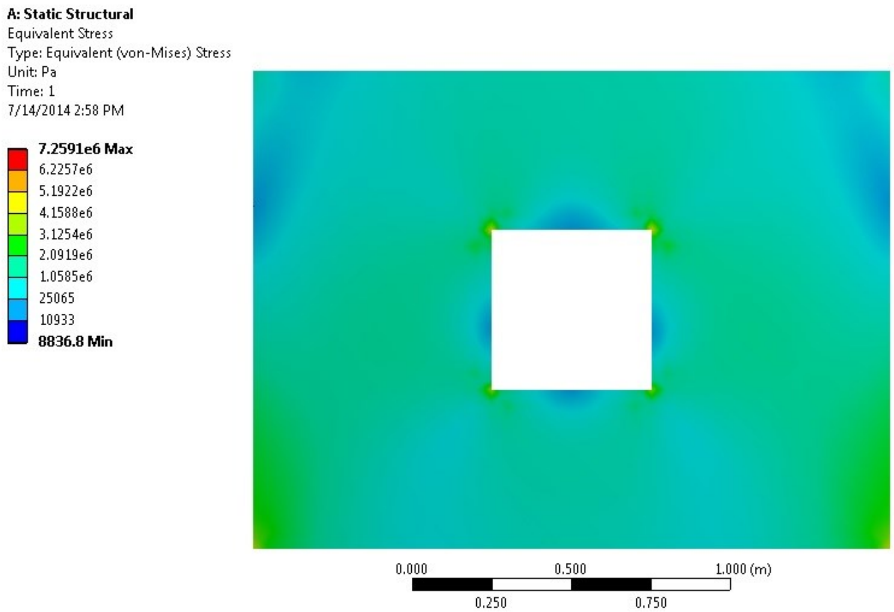


Figure 6.57: Nonlinear analysis Stress Distribution x-x (200 kN) by ANSYS.

Table 6.11: The maximum and minimum y-y stress values VS displacement (2 kN).

Softwares	Max. Stress (MPa)	Min. Stress (MPa)	Disp. at Max. Stress (mm)		Disp. at Min. Stress (mm)	
			dx	dy	dx	dy
SAP2000	0.0001	-0. 121	0.139	-0.047	0.206	-0.460
STRAUS7	0.0004	-0.142	0.570	-0.485	0.637	-0.411
SCIA	0.0442	-0.031	0.321	-0.021	0.332	-0.031
ABAQUS	0.0007	-0.037	0.210	-0.072	0.233	-0.039
ANSYS	0.0097	0.000	0.243	-0.058	0.224	-0.022
STAAD PRO	0.0004	-0.098	0.511	-0.032	0.553	-0.237

Table 6.12: The maximum and minimum y-y stress values VS displacement (20 kN).

Softwares	Max. Stress (MPa)	Min. Stress (MPa)	Disp. at Max. Stress (mm)		Disp. at Min. Stress (mm)	
			dx	dy	dx	dy
SAP2000	0.169	-0.360	1.69	-0.529	1.76	-0.404
STRAUS7	0.183	-0.325	2.19	-0.542	2.12	-0.384
SCIA	0.341	-0.193	3.16	-0.712	3.16	-0.433
ABAQUS	0.132	-0.211	2.29	-0.729	2.33	-0.447
ANSYS	0.451	-0.074	1.98	-0.642	2.21	-0.377
STAAD PRO	0.169	-0.228	6.67	-0.889	6.94	-0.434

Table 6.13: The maximum and minimum y-y stress values VS displacement (200kN).

Softwares	Max. Stress (MPa)	Min. Stress (MPa)	Disp. at Max. Stress (mm)		Disp. at Min. Stress (mm)	
			dx	dy	dx	dy
SAP2000	2.55	-2.74	17.2	-1.1	17.3	0.16
STRAUS7	0.0003	-0.00123	49.6	-5.8	50.5	-1.54
SCIA	4.2	-1.7	5.1	-1.7	5.1	1.1
ABAQUS	2.73	-2.91	6.7	-2.9	7.4	2.4
ANSYS	5.1	-0.78	16.1	-2.2	16.4	1.9

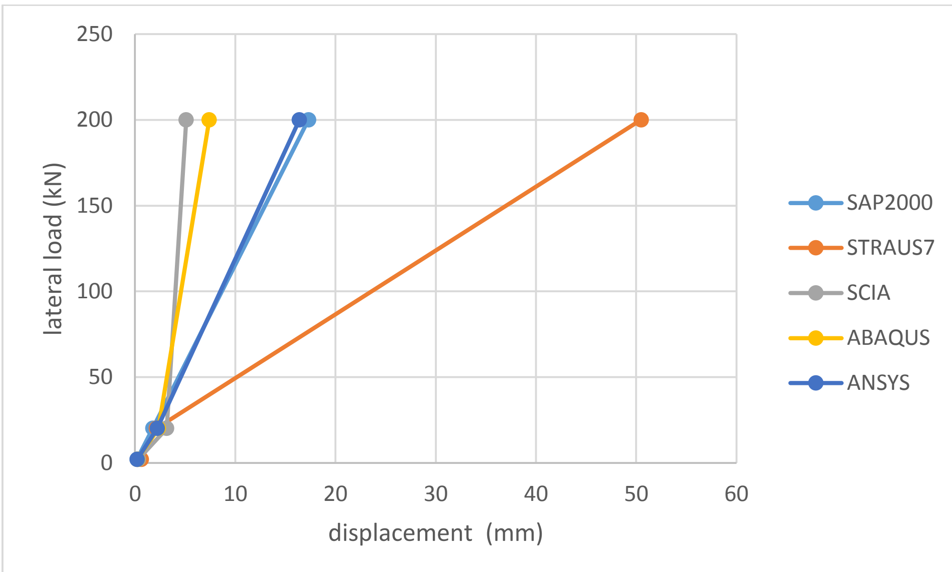


Figure 6.58: Load vs Displacement obtained by each software.

Discussion of the numerical results

The obtained results for the masonry wall consisted in the comparison between the displacement and the stresses for each model. Maximum and minimum stresses were occurring at the corners of the opening forming a linear distribution of the stress to the top corners of the wall, which leads to a higher displacement at top part of the wall.

Regardless of the type of modelling adopted in each software, the following structural behaviour has been noticed:

- High stress distribution along part between the opening corners to the wall corners.
- Sliding along top bed joints of the wall.

- High tension and compression stress in the bottom part of the wall.

Through linear analysis of the model, it is observed that with the increase of imposed horizontal load at top left corner 2 kN and 20 kN, both compression and tension stress level increase, compression occurs in two diagonal bands on either sides of the opening as shown in the stress distribution along the wall for all the FEM softwares, whereas tension takes place in opposite corner locations and middle diagonal band. The results show a min difference with respect to the obtained displacement. For linear analysis, the general behaviour of the masonry panel is the same in all softwares. The stresses calculated by nonlinear analysis with respect to the applied horizontal load 200 kN show significant differences in the global displacement through different softwares.

The most important and significant outcome is that it is not enough to know how to use a finite element software but it is necessary to understand the finite element method and the mathematic and theoretical background of structural mechanics. This knowledge can be used for interpreting the results. Another possible reasons for these results may be found in differences in the integration order of programs or in the internal routines utilized to compute stresses. These internals integrated modules cannot be analysed by the user nor can they be altered. As a result it is assumed that the predicted maximum stress can only be a useful indication for strength determination and that further comparative studies have to be done in order to verify the results.

The applications of linear elastic analysis for the masonry wall maybe inadequate, due to its very limited capacity in tension, as in reality masonry may show a complex non-linear response even at low levels of stress. Moreover, linear elastic analysis cannot be used to simulate masonry strength responses, which can typically be observed from the variation of obtained modelling results, which means that linear elastic analysis is not useful, in particular, to estimate the ultimate response of masonry structures and should not be used to conclude on their strength and structural safety. Given all the limitations of linear elastic analysis, nonlinear analysis is becoming more favourable. However, linear analysis is always performed, prior to the non-linear analysis to allow a quick and first assessment of the adequacy of the structural models regarding the definition of meshes, the values and distribution of loads and reactions, and the likelihood of the overall results. Along all the FEM softwares that have been used for performing the modelling, the advantages and disadvantages are represented in (Table 6.3)

Table 6.14: Modelling performance of each FEM software.

		SAP2000	STRAUS7	SCIA	ABAQUS	ANSYS	STAAD PRO
Pre-processing	Setting up geometry	✗	✓	✗	✓	✓	✓
	Defining material and section properties	✓	✓	✗	✓	✓	✓
	Meshing technique	✓	✗	✗	✓	✓	✗
	Failure criteria	✓	✓	✗	✓	✓	✗
	Assigning load and constrains	✓	✓	✓	✓	✓	✓
Analysis	Analysis options	✓	✓	✗	✓	✓	✗
	Time of running the analysis	✓	✓	✓	✗	✗	✓
Post-processing	Displaying the results	✓	✓	✗	✗	✗	✓
	Flexibility in modifying the structure	✓	✓	✗	✗	✗	✓

Chapter 7

Case Studies

The 2008 earthquake in Bologna produced extensive damage in historical structures as well as in current modern construction, which sustained a high degree of damage due to shear action, demonstrating the need for experimental investigation to study and improve the seismic response of those structures.

Three building have been selected to be a part of this study, in which the framework of the structural assessment contains both in-situ and laboratory tests in order to obtain the mechanical characterization which can be used as input parameters for conducting the numerical modelling and determine the ability to resist the seismic actions.

7.1. “Facoltà di Lingue e Letterature Straniere” Building

This work presents the results of an experimental program on a building in Via Fillipo Re 8 (Facoltà di Lingue e Letterature Straniere), which aims to measure the capacity of seismic resistance of the historical structures of Bologna university. The experimental investigation includes:

- Visual inspection.
- Verification of wall connections.
- In-situ shear strength test by applying hydraulic jack.
- Extraction of cores with mortar joint.
- Splitting test on masonry cores including a central mortar joint along a symmetry plane.
- Compressive strength test on cylindrical brick specimens.
- Double punching test on mortar specimens sampled from the site.

For each location 5 cores specimens were easily extracted by using a common core drill from two positions. Locations which were carefully selected with respect to the wall thickness, the structural importance (shear bearing wall), and the wall construction age.

Figure 7.1: The masonry walls, where the specimens have been execrated (L_PT_01), (L_P-1_01).

There are several critical factors which can affect the experimental results. The mechanical properties of the masonry structures do not depend only on brick and mortar properties as individual material, instead the confinement and interaction between the masonry structures parts have an important role in the way how masonry structures behave during the application of different kind of loads. A suitable way to prevent the damages is based on the use of laboratory destructive testing on small specimens. This activity can be pursued without inflicting severe damage to the construction.

In situ investigation

The in situ investigation contains shear test, corning masonry and checking masonry walls confections. Extracting masonry cores is important to understand the structural properties of the different types of masonry of which a building is composed, cores with 9.5 cm diameter have been taken as samples in the most representative points of the structure. Coring have been done with a rotary saw using a diamond cutting edge. This coring operation allows samples to be extracted from the material on which laboratory tests can be made such as the Brazilian splitting test.

Shear testing technique by hydraulic jack, has been used to determine the shear strength characteristics of the mortar between the brick layers. A brick is extracted from the centre of the selected masonry wall and a hydraulic jack of the same size is placed inserted of the brick for the application of a shear force.

In order to study the behaviour of masonry walls, it is important to know how the walls are connected together, which can be implemented in the numerical modelling, as shown in (Fig. 7.3).

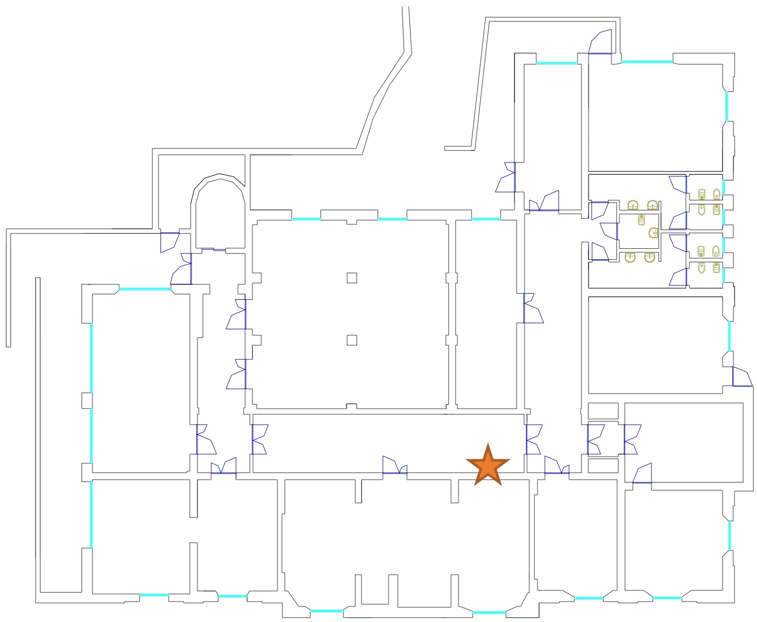


Figure 7.2/a: The basement level plan with position of in situ investigation tests (L_P-1_01).

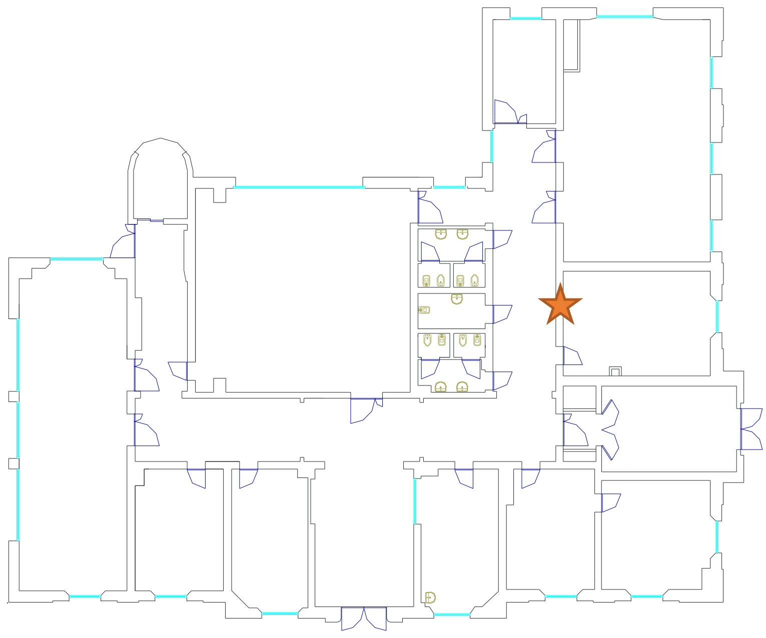


Figure 7.2/b: The ground level plan with position of in situ investigation tests (L_PT_01).

Figure 7.3: Visual inspection of the walls connection.

Shear test by using hydraulic jack

The tests took place in both locations in the building (L_PT, L_P1), by using manual hydraulic jack with oil pressurized pump (Fig.7.4). The masonry panel to be tested under shear load was prepared by taking out one masonry unit beside the tested unit, the hydraulic jack placed instead of the brick unit while the other one remains free for the specimen horizontal deformation. During the test the horizontal distances were recorded and the horizontal load applied by the hydraulic jack to the masonry. (ASTM C1531, 2009) has been followed for preforming this test.

Figure 7.4: Shear test setup (LPT).

Table 7.1: The results of shear strength test.

Location	Max Stress (bar)	Brick Dimension (mm)			shear strength (MPa)
		thickness	length	width	
L_PT	510	6	28	14	3.76
L_P1	460	6	13	14	8.92

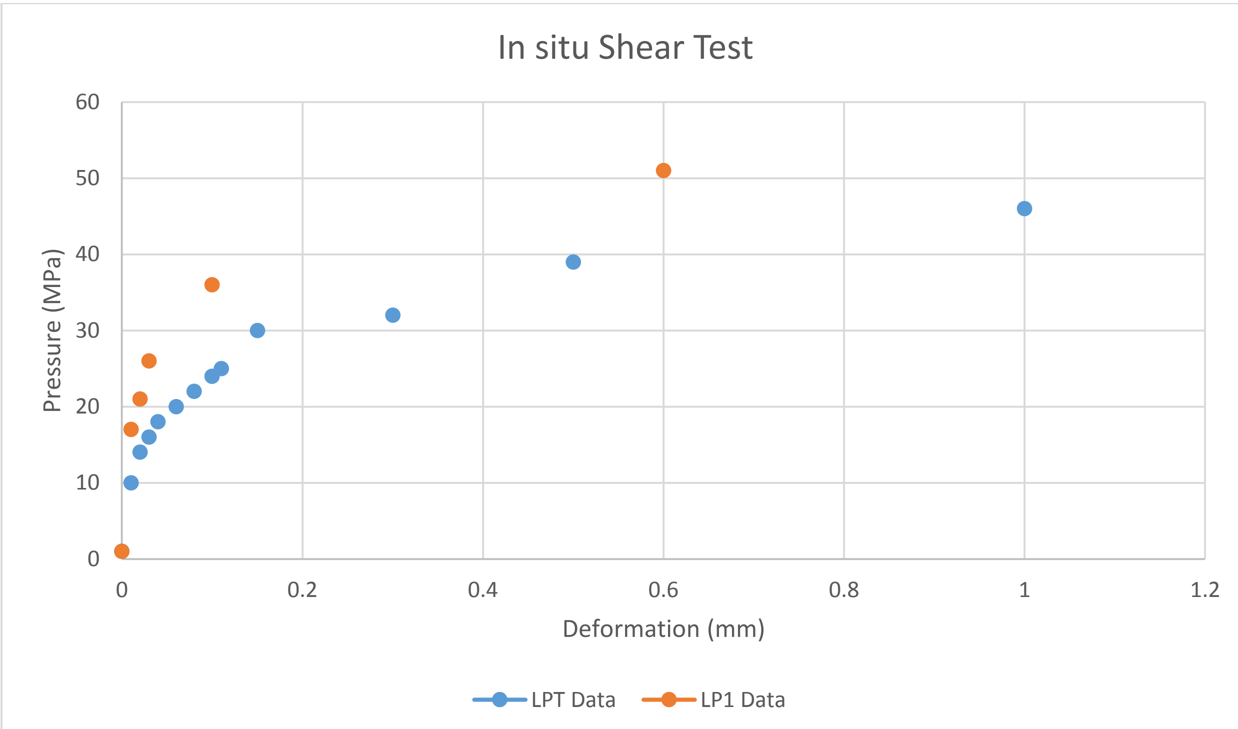


Figure 7.5: Pressure vs deformation results.

The splitting test

The obtained masonry cores have to be prepared before preforming any test, by cutting out the uneven edges and trying to obtain the same height along all samples.

The cores were subjected to splitting test providing 45°, 50° and 55° inclinations of the mortar joint with respect to the loading plane. This test induces a mixed compression–shear stress state in the central mortar joint. The experimental results have been interpreted using Mohr–Coulomb criteria in order to assess the mechanical properties of the mortar. In this case, the mortar is subject to shear-compression and it is possible to reproduce a brick-joint interface behaviour comparable to that observed in a wall diagonal test (Benedetti et al., 2008).

The masonry cores were subjected to splitting test using a universal machine (load capacity 120 kN, adopted loading rate 0.2 kN/s) (Fig.7.6), the test has been performed by following (ASTM C1006, 2007).

Figure 7.6: The application of splitting load.

Eight cores were tested in total (four for each position), in order to have a sample for each splitting angel. The cores were rotated so that the mortar joint was inclined either by 45°, 50° or 55° angle, with respect to the horizontal reference (Table 7.2).

Table 7.2: The tested specimen's details.

Sample Code	Diameter (mm)			length (mm)			Angle inclination
	D1	D2	D3	h1	h2	h3	
L_PT-1_A	93.64	93.71	93.67	130.04	130.73	130.93	50
L_PT-1_B	93.47	93.47	93.57	128.50	128.50	128.00	45
L_PT-1_C	93.73	93.67	93.70	129.90	129.73	128.59	55
L_PT-1_D	93.53	93.60	93.54	128.50	128.00	128.00	55
L_P-1_A	93.74	93.60	93.68	121.80	121.31	121.81	50
L_P-1_B	93.71	93.71	93.64	121.00	120.30	119.50	55
L_P-1_C	93.78	93.62	93.80	121.60	120.93	121.00	55
L_P-1_D	93.74	93.63	93.80	121.40	121.63	122.32	50

From the mean values of shear strength and compressive stress for the cores tested with the mentioned mortar joint inclinations (Fig.7.7). By linear regression, the masonry initial shear strength, cohesion (f_v) and the angle of internal friction ($\emptyset$) were obtained by plotting the corresponding compressive and shear strength for each angle for cores extracted from the same position.

Table 7.3: The compressive and shear strength values with respect to each specimen.

Sample Code	Angle inclination	Applied Load (kg)	Compressive strength (MPa)	Shear strength (MPa)
L_PT-1_A	50	650	0.34	0.40
L_PT-1_B	45	1180	0.68	0.68
L_PT-1_C	55	1980	0.92	1.31

L_PT-1_D	55	830	0.39	0.56
L_P-1_A	50	520	0.29	0.34
L_P-1_B	55	190	0.09	0.14
L_P-1_C	55	----	----	----
L_P-1_D	50	320	0.18	0.21

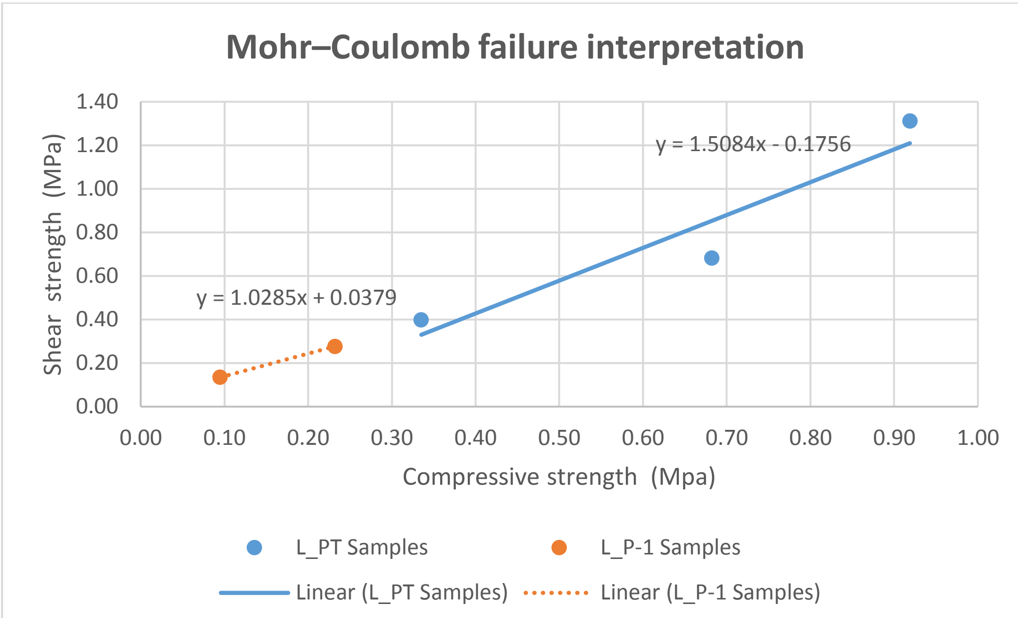


Figure 7.7: Shear-Compression graph of Brazilian splitting test.

The parameters of the Mohr–Coulomb failure criterion for each position.

Table 7.4: The Mohr-Coulomb parameters obtained by Brazilian splitting test.

Position	(f_v) MPa	(Ø)
L_PT	-0.1756	56.5
L_P-1	0.0379	45.8

The parameters shown in (Table 7.4) can be used as inputs for preforming a numerical analysis for those softwares adopting Mohr Coulomb as a failure criterion.

It is important to notice the failure behaviour along mortar joint in masonry cores, that the cracking goes through the mortar and the masonry interface, which means the obtained parameters reflect the bonding mechanical properties for mortar and masonry (Fig. 7.8).

Figure 7.8: Failure mechanism of cores after preforming Brazilian splitting test.

Double punch test

The compressive strength of the sampled mortar specimens from site is given by the double punch test. Eight specimens have been tested, four for each location, using punching device mounted on testing machine (load capacity 10 kN, adopted loading rate 0.05 kN/s) (Fig.7.9). The mortar has been sampled from masonry cores, then prepared by casting 2cm of the top and bottom to smooth the faces in order to obtain better distribution of the load. The failure mechanisms are different from sample to another, as some mortar samples have been totally destroyed, others were punched around the casting material, and these failure behaviours can be caused with respect to the mortar strength.

Figure 7.9: Double punch test of mortar specimens, Facoltà di Lingue building.

Table 7.5: The compressive strength values with respect to each specimen.

Sample Code	Height (mm)				Applied Load (kg)	Compressive strength (MPa)
	h1	h2	h3	Av.h		
LP-1_01_A01	10.30	10.50	10.60	10.47	40	1.25
LP-1_01_A02	10.10	10.20	10.40	10.23	35	1.09
LP-1_01_A03	10.40	10.10	10.30	10.27	45	1.41

LP-1_01_A04	11.10	10.80	10.60	10.83	35	1.09
LPT-01_B01	9.80	9.70	9.50	9.67	105	3.28
LPT-01_B02	10.10	10.20	9.90	10.07	85	2.65
LPT-01_B03	10.20	10.30	10.15	10.22	65	2.03
LPT-01_B04	9.90	10.00	10.10	10.00	80	2.50

The experimental data plotted as a function of the mortar thickness. (Fig.7.10) shows the dependency of the apparent strength from the thickness of the layer. The strength is inversely proportional to the layer thickness. This effect is very important in the masonry strength prediction. The average mortar compressive strength for LP-1 was (1.413 MPa) and for LPT was (2.615 MPa).

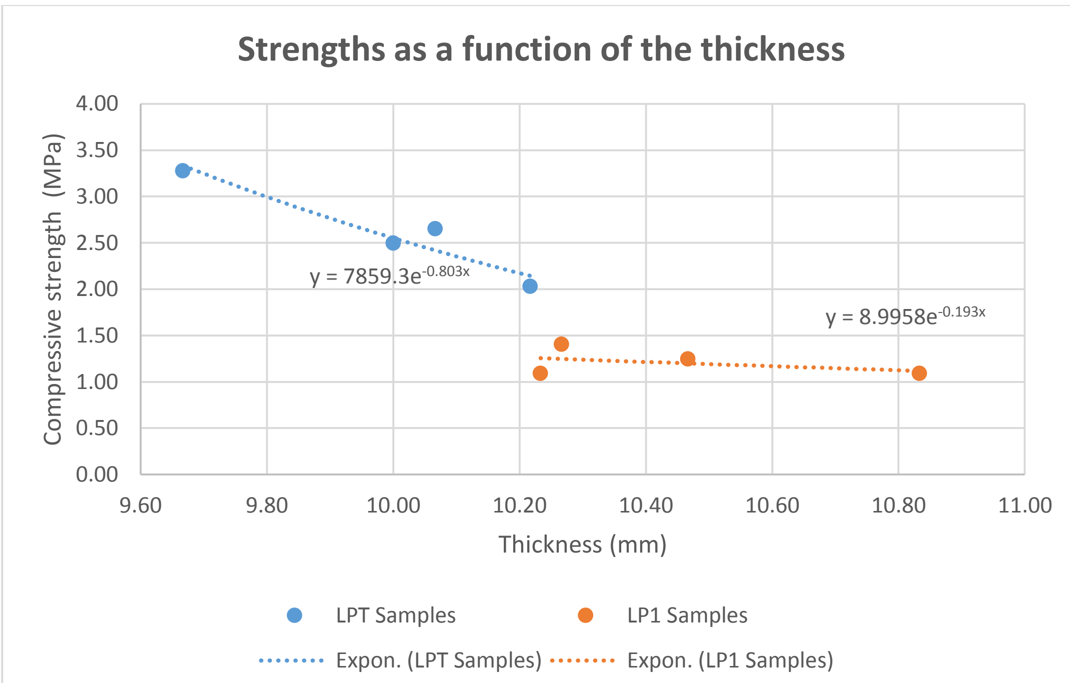


Figure 7.10: Strength as a function of the thickness, Facoltà di Lingue building.

Compression test on bricks cylinders

The mechanical properties of the bricks can be readily obtained by using standard compression test on solid brick cores which are subjected to axial loading. The brick cylinders were obtained from the brick unit taken out for the in situ shear test. For both locations compression tests were done on fifteen cylindrical specimens with a height to diameter ratio (5x5cm) equal to 1 in order to get the compressive strength, Using a universal machine (load capacity 120 kN, adopted loading rate 0.2 kN/s), the test lasts around 30 seconds for each specimen, the test has been performed by following (ASTM D7012, 2013).

Figure 7.11: Compression test on cylindrical brick samples.

Table 7.6: Compressive strength values with respect to each specimen.

Sample Code	Diameter (mm)			Height (mm)			Applied Load (kg)	Compressive strength (MPa)
	D1	D2	D3	h1	h2	h3		
L_PT-1_A	50.21	49.79	50.19	50.96	51.17	51.56	7450	37.13
L_PT-1_B	49.95	50.19	50.63	51.48	51.79	52.77	8600	42.53
L_PT-1_C	50.19	50.39	50.32	48.96	49.23	49.34	7850	38.75
L_PT-1_D	49.89	49.89	49.94	47.70	47.32	48.39	7300	36.61
L_PT-1_E	50.00	50.00	49.91	42.86	43.26	43.48	8500	42.52
L_PT-1_F	50.41	50.24	50.11	50.46	51.41	50.83	7000	34.62
L_PT-1_G	49.79	49.86	49.94	51.05	51.25	51.05	6850	34.41
L_PT-1_H	49.78	49.85	49.82	51.24	51.55	51.27	5800	29.19
L_PT-1_I	49.89	50.23	49.87	51.71	51.70	51.75	5750	28.73
L_P-1_A	49.93	49.88	49.90	49.84	49.90	50.38	8250	41.38
L_P-1_B	49.96	50.29	50.20	49.43	50.00	50.46	6150	30.54
L_P-1_C	49.94	49.89	49.90	50.29	50.00	50.41	7400	37.11
L_P-1_D	50.09	49.81	49.97	46.19	46.70	46.84	7550	37.79
L_P-1_E	49.84	50.15	49.90	50.23	50.24	50.70	7300	36.53
L_P-1_F	49.87	49.98	49.96	51.02	51.25	52.46	8350	41.82

The average compressive strength for masonry units in L_P-1 (37.53 MPa) and for L_PT (36.05 MPa).

The experimental characterization of mortar mechanical properties in existing masonry constructions is considerably complex. Whereas bricks parameters can be assessed with a sufficient precision, the mortar properties are very difficult to obtain and the results are highly dispersed. For instance, the in-situ techniques based on the measurement of the amount of energy required to drill a small cavity provide very scattered values that should be handled cautiously. Also, the characterization of existing mortar joints by means of surface testing may be difficult, since the surface decay or even the presence of new restoration mortar may spoil the results. On the other hand, tests on small mortar cubes or double punch tests usually lead to inaccurate estimates of mechanical characteristics, since the confining effect exercised by bricks on the mortar layer is completely disregarded. Another difficulty is the extraction of undisturbed specimens from the joints of existing brickwork.

7.2 Botanica Building

This work presents the results of an experimental program on Botanica building in Via Irnerio, which aims to measure the capacity of seismic resistance of the historical structures in Bologna, where the experimental investigation includes:

- Splitting test for masonry cores including a central mortar joint along a symmetry plane.
- Compressive strength test for cylindrical brick specimens.
- Double punching test for mortar specimens sampling from the sit.
- In-situ shear strength test by applying hydraulic jack.

Specimens were easily extracted by using a common core drill from two positions by four specimens for each location, which were carefully selected with respect to the wall thickness, the structural importance (shear bearing wall), and the wall construction age.

Figure 7.12: The positions of in situ investigation (T1), (T2).

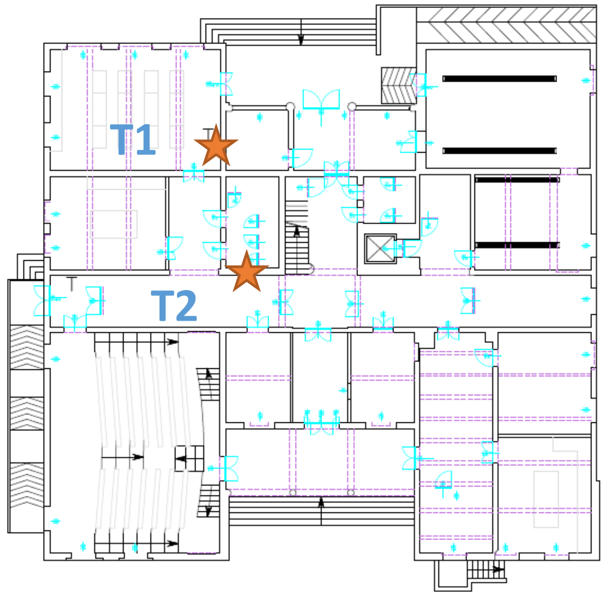


Figure 7.13: The position of T1 & T2 in the Ground level.

Compression test of brick

The mechanical properties of the bricks can be readily obtained by using standard compression test on solid brick cores which are subjected to axial loading. A compression test on fifteen cylinder specimens for both locations with a height to diameter ratio equal to 1 have been done in order to get the compressive strength, Using a universal machine (load capacity 120 kN, adopted loading rate 0.2 kN/s).

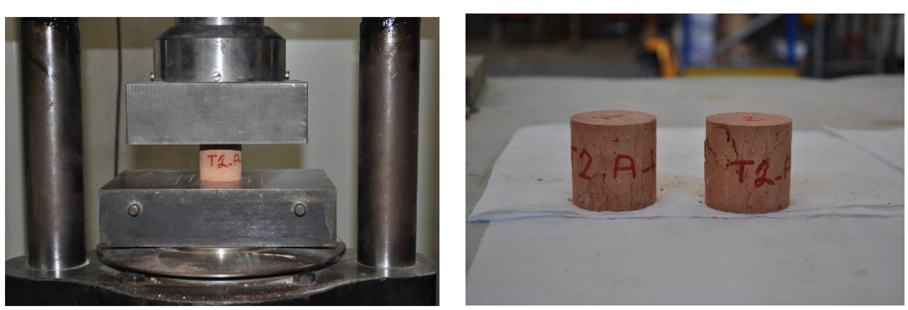


Figure 7.14: Compression test on cylindrical masonry unit.

Table 7.7: The compressive strength values with respect to each specimen.

Sample Code	Diameter (mm)			Height (mm)			Applied Load (kg)	Compressive strength (MPa)
	D1	D2	D3	h1	h2	h3		
T1-A-1	47.58	47.52	47.57	45.87	46.12	46.17	5900	32.58
T1-A-2	47.50	47.57	47.46	45.86	45.99	45.79	4850	26.84
T1-A-3	47.62	47.60	47.53	46.87	46.97	47.62	4300	23.72
T1-B-1	47.54	47.44	47.47	48.50	48.92	49.19	5750	31.85
T1-B-2	47.76	47.55	47.58	48.18	48.52	49.11	3350	18.44
T1-B-3	47.58	47.58	47.68	46.84	46.92	46.79	5100	28.10
T1-B-4	47.50	47.58	47.60	47.42	47.48	47.26	7800	43.07
T2-A(B)1	49.94	49.90	49.95	51.58	51.19	51.56	4000	20.04
T2-A(B)2	49.86	50.00	49.90	50.62	50.46	50.48	5300	26.56
T2-A-1	50.10	50.19	49.93	50.53	50.49	50.50	8100	40.35
T2-A-2	50.02	50.03	50.04	50.71	51.68	50.54	9000	44.91

As shown in the above table the average compressive strength for masonry units in T1-A (27.71 MPa), T1-B (29.87 MPa), T2-A (B) (23.30 MPa), and for T2-A (42.63 MPa).

Figure 7.15: The application of shear test by hydraulic jack (T2).

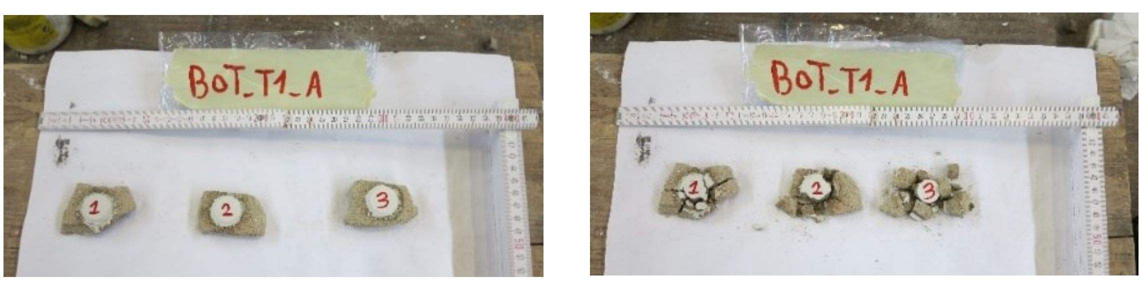


Figure 7.16: Mortar specimens before and after double punch test (BOT-T1-A Specimens).

Figure 7.17: Masonry core specimens with mortar joint.

Figure 7.18: Brazilian test for cylindrical masonry sample with mortar joint.

7.3 Palazzina della Viola

Palazzina Della Viola is a historical building in Bologna, located in the North part of the city center and it was built in the late of 15th century (1497), by using timber beams supported on masonry bearing walls. The palace was under restoration in the early 20th century, 1907 and 1928, followed by reconstruction of the North–Eastern corner of the building in 1946-1947 as a result of damage caused by the war (hit by an air-raid bombing in 1944).

Masonry is still one of the most popular construction materials in bologna. It is used to be weak against seismic actions, because of the low tensile strength of masonry comparing with its compressive strength. So it is important to improve the seismic performance of such buildings for any future seismic actions, by having a seismic assessment to its mechanical performance and evaluate the structural weakness.

Need for the study

Bologna lies in a seismically vulnerable region. In the 2012 earthquake many masonry building were destroyed in the area around Bologna (Modena and Ferrara). Most of these structures were built with little or no seismic consideration. The study focuses on the seismic performance of Palazzina Della Viola, which was constructed more than 500 years ago, and it has been used and occupied by different governmental and public offices. The palace was built with best available material during the time of construction with best available technology which has been noticed by the primary investigation. Hence, the need for the present study becomes important for the following reasons:

- The palace accommodates the international relation department of Bologna University, so it needs to be safe against the future earthquakes hazards.
- Since the palace is over 500 years, the heritage value must be preserved. Archeological conservation of the building is essential.
- No previous seismic evaluations of the palace were available.

Necessary strengthening or retrofitting of the palace requires the evaluation of seismic vulnerability of the structural system and components before establishing the mode of strengthening. Apart from these above mentioned points the building should be assessed to the following consideration and poor seismic performance may be expected due to the following reasons:

- Age and consequent degradation of structural materials leading to a decrease of local and global stiffness and strength.
- The high number and verity of structural changes that the palace suffered during service time, without considering the effect on the seismic performance;

Structural System

The structural system and components of the palace are essential for evaluating its seismic resistance. The building is nearly square shape with partial basement and two storeys above the basement, it has a large number of openings in its facade and has sloped roof covered with clay tiles supported by timber truss. The building is constituted of two long entrance wings. The main structural system for lateral load resisting is thin unreinforced masonry wall with about 40 cm thickness in the ground floor, reduced thickness in the upper floors but with 28 cm as a minimum thickness. The walls are made of clay brick with lime mortar. Sizes of opening are similar in all floors and placed in symmetrical position.

Figure 7.19: the East front side of Palazzina Della Viola.

Figure 7.20: Masonry units and mortar of Palazzina Della Viola.

Figure 7.21: Timber truss supporting the sloped roof.

Structural components of the building

Brick masonry wall

The main structural system for lateral load resisting system is thin brick masonry wall. Due to masonry wall the inertia forces are increased, which is adverse for earthquake to increases the in-plane stiffness of the building. The thickness of the wall ranges from 0.15 m to 0.5 m.

Timber Floor and Roof

The floor system consists of timber joist considered as rigid diaphragms. Despite the light dead load. Usually the connection between the timber elements of the floor and masonry walls is supposed to be done with iron element. So it supposed to provide the floor rigidity at their level and resistance the lateral load. The roof is covered with clay tiles and is supported by timber trusses and joints.

Figure 7.22: Timber beams supported by the walls.

Openings

There are large numbers of opening having same sizes for doors and windows. The openings are in the same horizontal and vertical alignment throughout the building. Most of the openings including doors and windows are of arch type. Due to the presence of opening, the seismic resistance capacity of the masonry wall is reduced with respect to in plane loading despite the thin masonry walls. When subjected to the seismic loading, stress concentration takes places in the opening zones, which may result in unexpected cracking of masonry wall and elements.

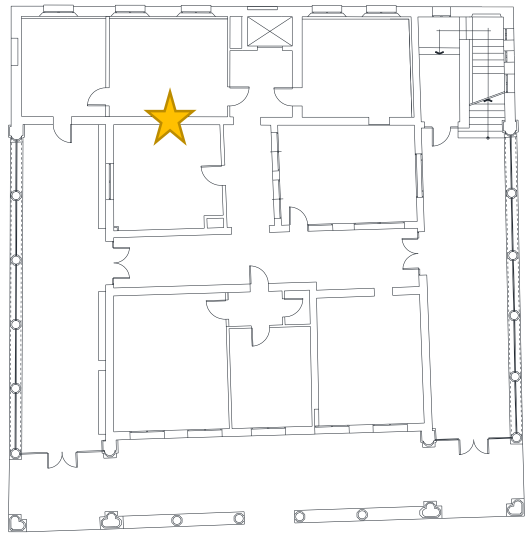


Figure 7.23: Ground floor of Palazzina Della Viola.

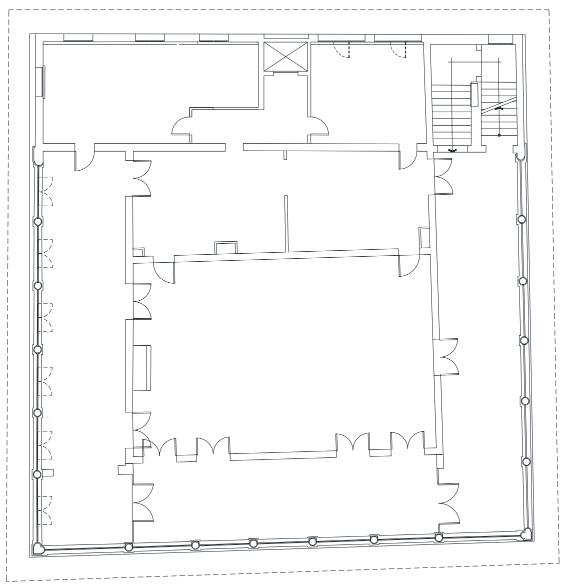


Figure 7.24: First floor of Palazzina Della Viola of the recent rehabilitations conducted in 2012.

The experimental tests which have taken place in the building, the investigation included:

- Splitting test for masonry cores including a central mortar joint along a symmetry plane.
- Compressive strength test for cylindrical brick specimens.
- Double punching test for mortar specimens sampling from the sit.
- In-situ shear strength test by applying hydraulic jack.

Figure 7.25: shear tests setup by using hydraulic jack.

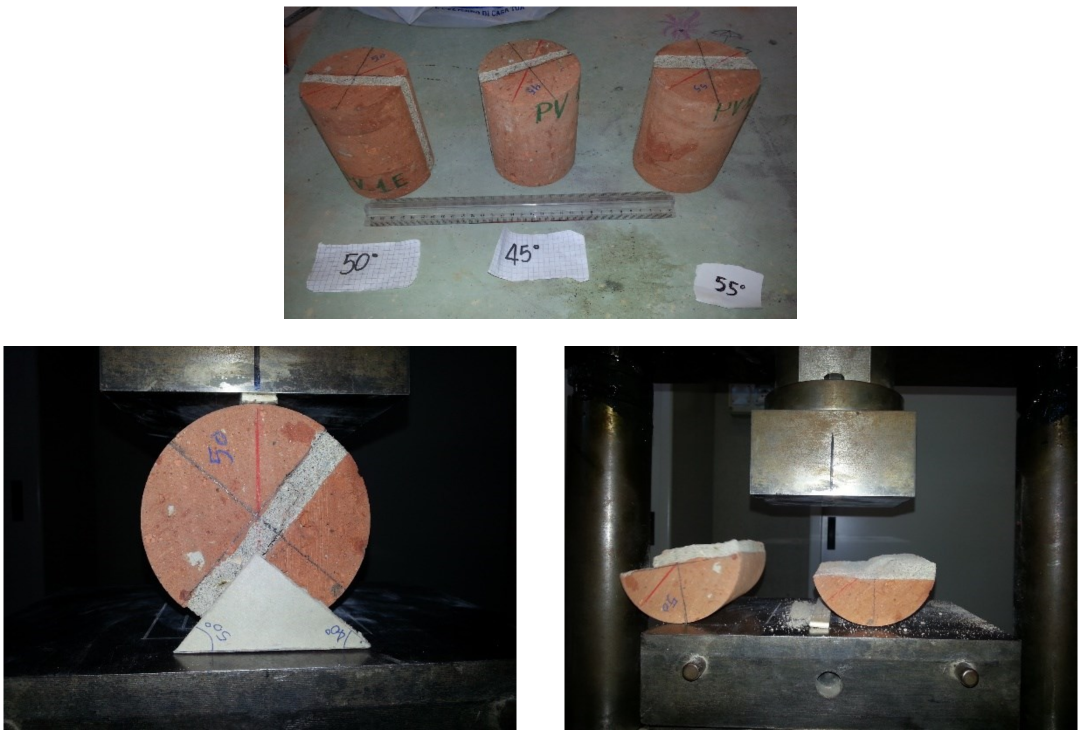


Figure 7.26: Brazilian splitting test of cylindrical masonry sample with mortar joint.

Figure 7.27: Masonry specimens for compression test.

Figure 7.28: Mortar samples for double punch test.

An in-situ survey of the building was made to accurately reproduce the geometry, the structural details and the irregularities of the building. This investigation was followed by GPR radar, IR thermography, Gucci penetrometer, and Helifix testing, aimed to characterize the masonry texture and to check the quality of masonry walls, and check the structural elements in the floors with respect to the loading direction.

Modelling and analysis

In order to build the model of the building, the as-built drawings have been used (Fig.7.23 and Fig.7.24).The length and width of the building are about 23 m and 22 m respectively. The building has two storeys and its ground storey height is around 4.0m while the first storey is around 3.7m for the surrounding rooms and 5.7m for the central hall. The mechanical properties of the masonry implemented in the analysis are obtained by experimental investigation. The sizes of the main windows are; (1.3 x2.3) m, (2.0 x2.6) m, (2.5 x2.5) m and door are (0.8 x2.3) m for single doors, (2.3 x1.6) m for double doors. The model considered the internal walls with 10cm, 35cm and 45cm thickness, in addition to the external columns.

For the sake of modelling, only the thick masonry walls are considered in the model with four-nodded solid elements. It is not realistic to use detailed models of walls and connections in large scale analysis, because of the geometric and material properties of the constituents/economy constraints. For most large-scale analysis, it is acceptable to model both regular masonry and rubble masonry walls assuming a continuum homogenous material.

The internal gypsum decorations, the timber roof and others architectural parts associated with the building are not considered in the modelling. However, the loads associates with these elements were included in the analysis. The foundation of building was considered fully restrained. The 3-D homogenous solid element model of the modelled building is shown in (Fig.7.29).

The analysis of the building is carried out by considering self-weight and earthquake loading and aimed to find essential information about overall stress distribution within the building and to provide insight to the response of the structure to vertical and lateral loads. The building is assumed to be supported by fixed foundation and has rigid floors, which are generally considered as a reasonable tool for structural investigation for basic understanding of response of ancient masonry buildings formed by complex components. In the modelling, the opening of windows and doors are included. However, it is believed that the number of the elements assumed is quite enough to identify the structural response of the building subjected to its own weight and the earthquake load. The structural walls of the building are composed of at least three major materials: brick, mortar and plaster. However, in the construction of the model the building is assumed to be of a single material regarding its modulus of elasticity and specific density.

The model is analysed by the lateral force method, in which seismic effect represents by horizontal force which is considered as a percentage of the total weight of the building. In this method, dynamic forces that act on the structure during the excitation are converted into equivalent horizontal force. The distribution of lateral load is different for different floor system. For simulations of the numerical model a commercial program STRAUS7 is used.

Two models have been conducted, the first one (Model_1) by using mechanical properties from the material library adopted by the software and without taking in consideration the NDT investigations with respect to non-uniformity of the walls. The second model (Model_2) uses the mechanical properties obtained by the experimental investigation, and takes in consideration the non-homogeneity of masonry walls deducted by visual inspection.

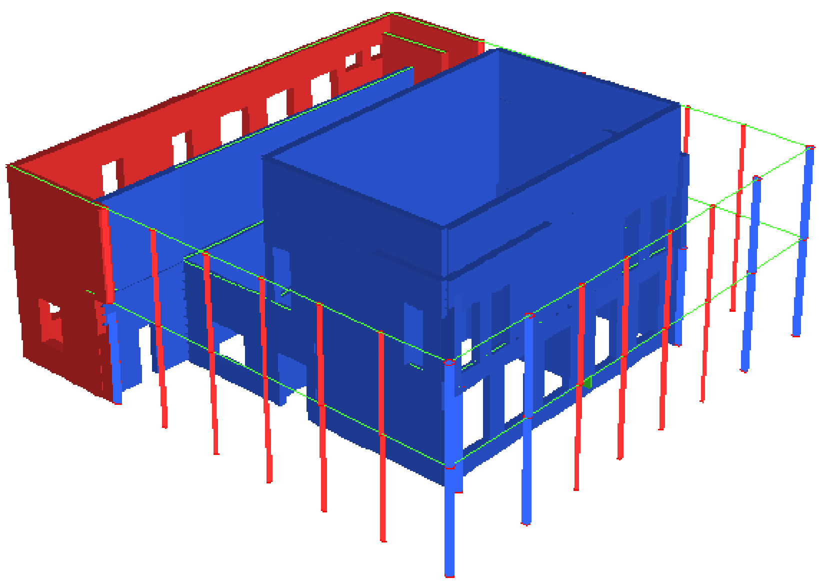


Figure 7.29: 3D Palazzina Della Viola numerical model.

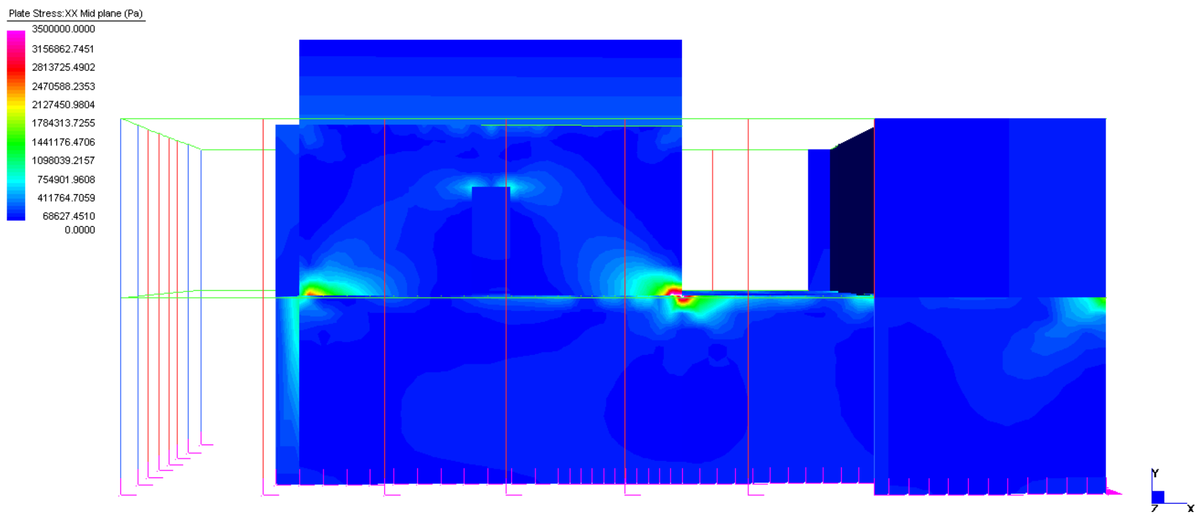


Figure 7.30/a: x-x stress distribution of the North side (Model_1). SXX: 0.0-3.5MPa

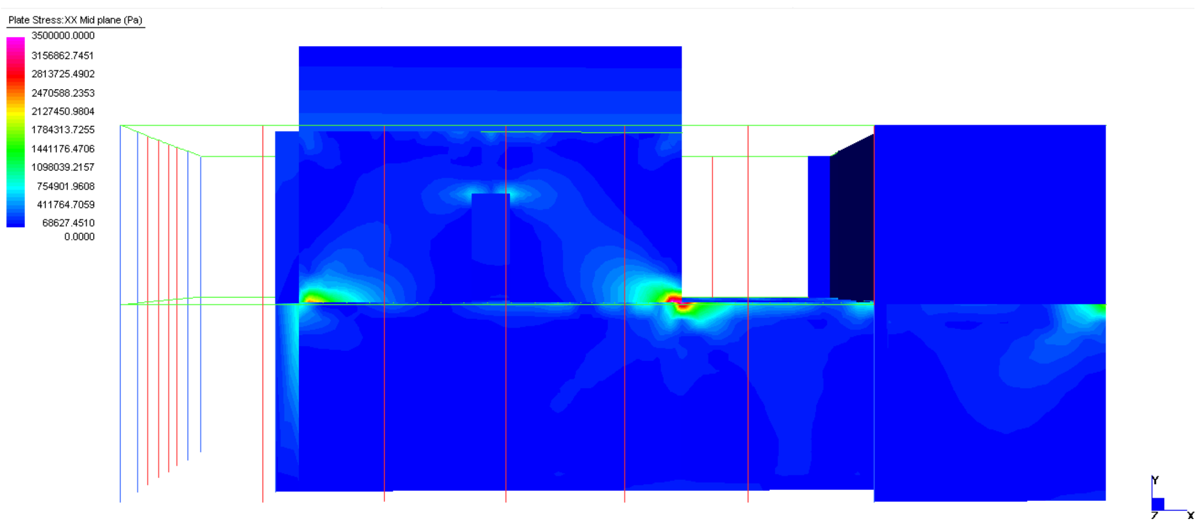


Figure 7.30/b: x-x stress distribution of the North side (Model_2). SXX: 0.0-3.5MPa

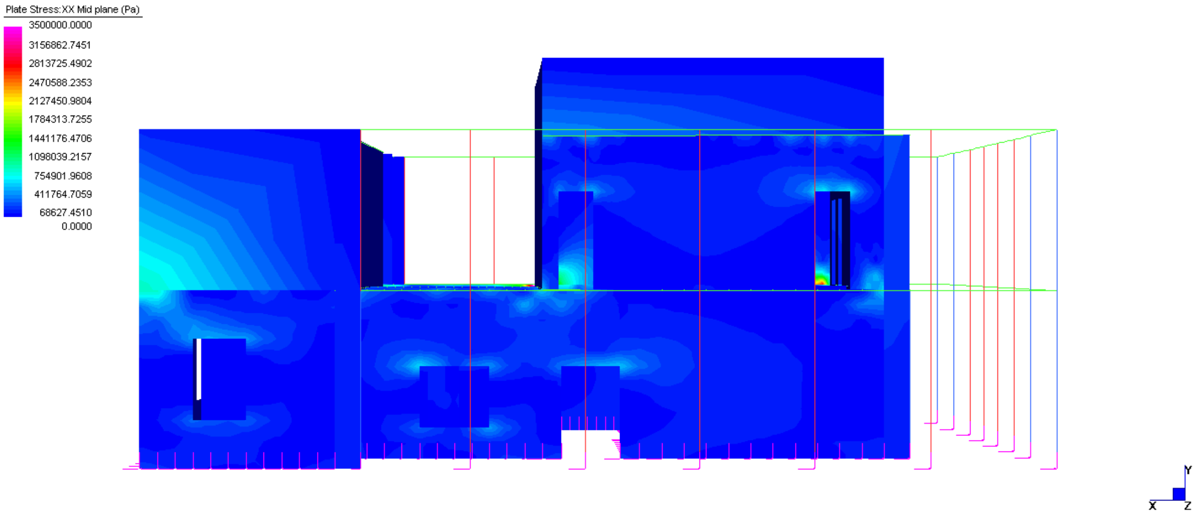


Figure 7.31/a: x-x stress distribution of the South side (Model_1). SXX: 0.0-3.5MPa

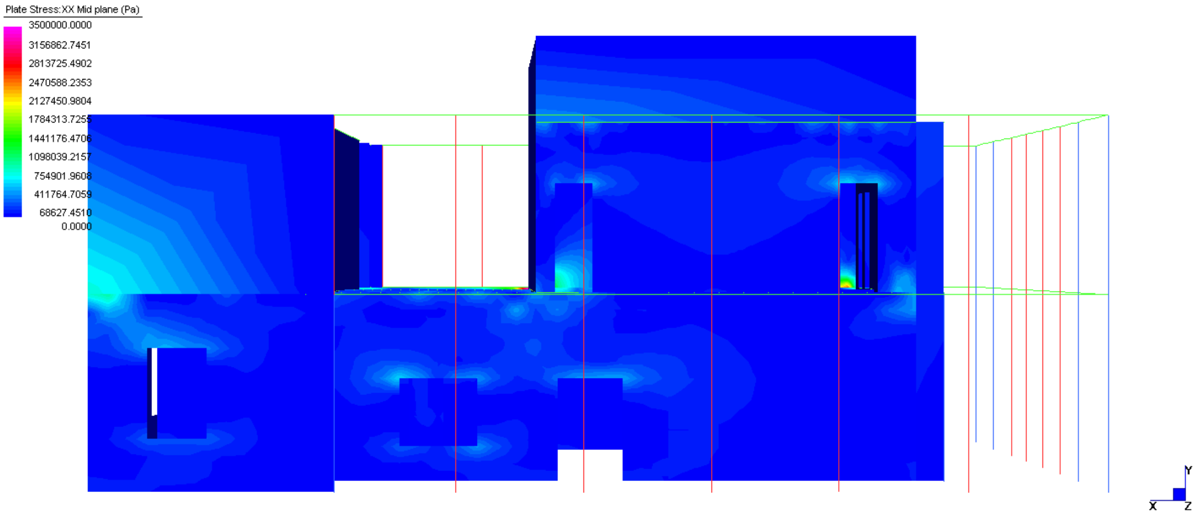


Figure 7.31/b: x-x stress distribution of the South side (Model_2). SXX: 0.0-3.5MPa

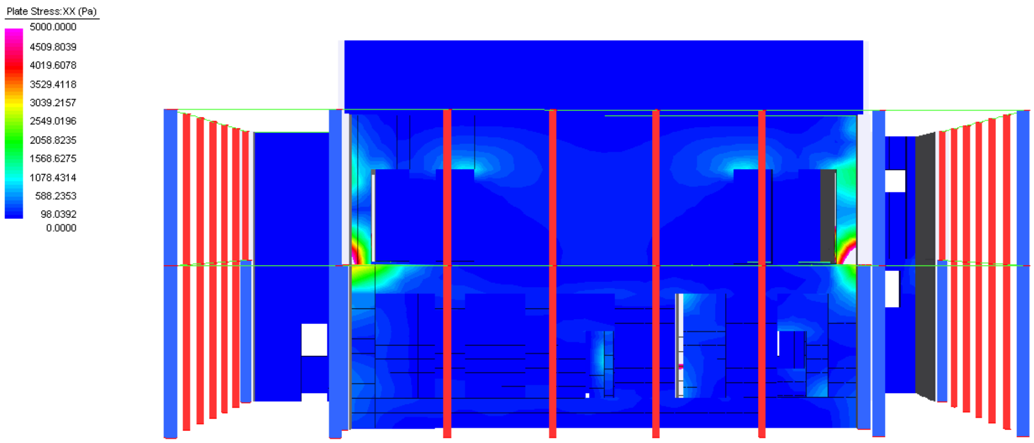


Figure 7.32/a: x-x stress distribution of the East side (Model_1). SXX: 0.0-5000Pa

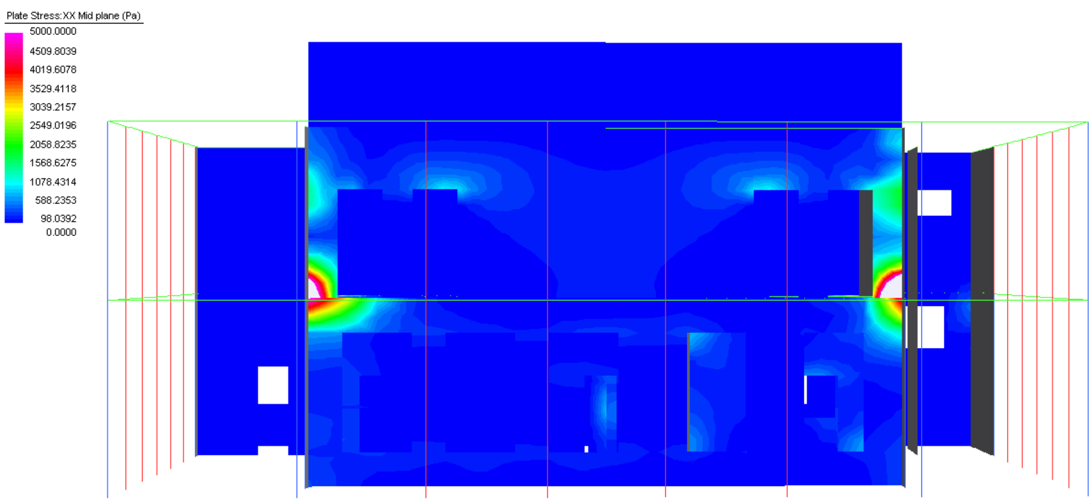


Figure 7.32/b: x-x stress distribution of the East side (Model_2). SXX: 0.0-5000Pa

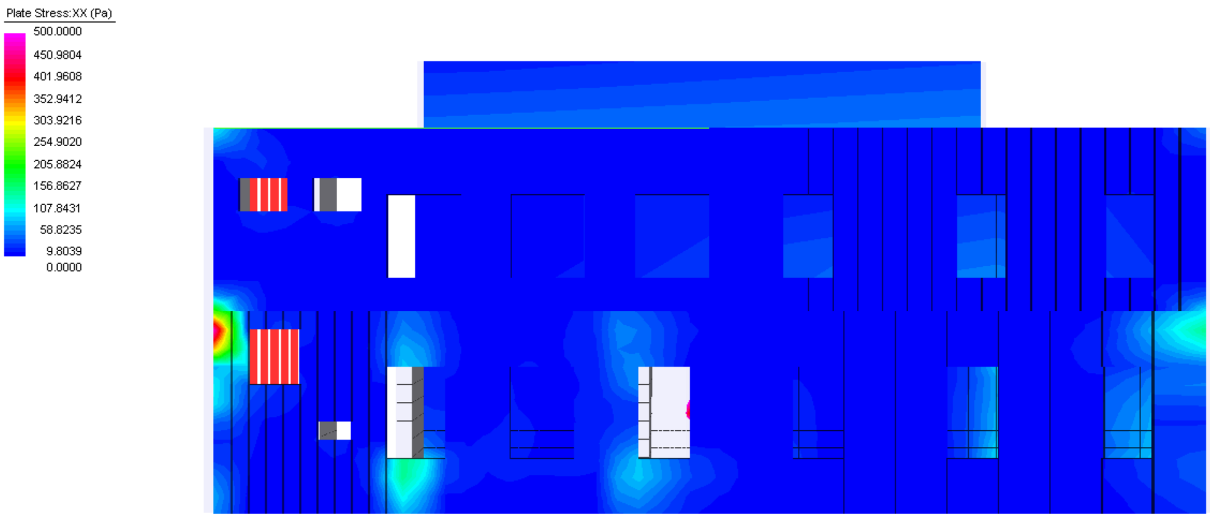


Figure 7.33/a: x-x stress distribution of the West side (Model_1). SXX: 0.0-500Pa

Figure 7.33/b: x-x stress distribution of the West side (Model_1). SXX: 0.0-500Pa

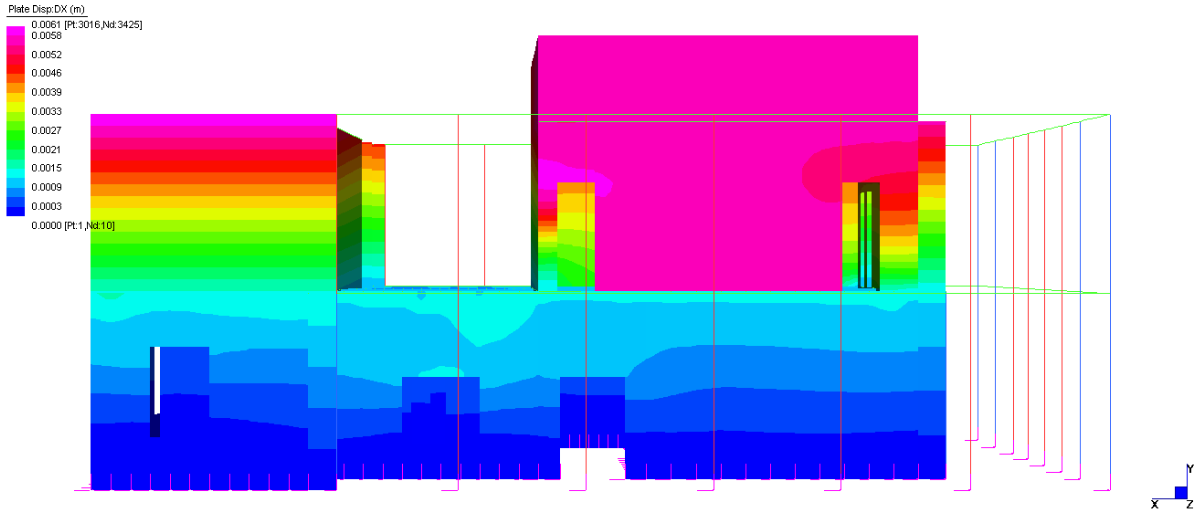


Figure 7.34/a: Displacement of the South side (Model_1). dx: 6.1mm

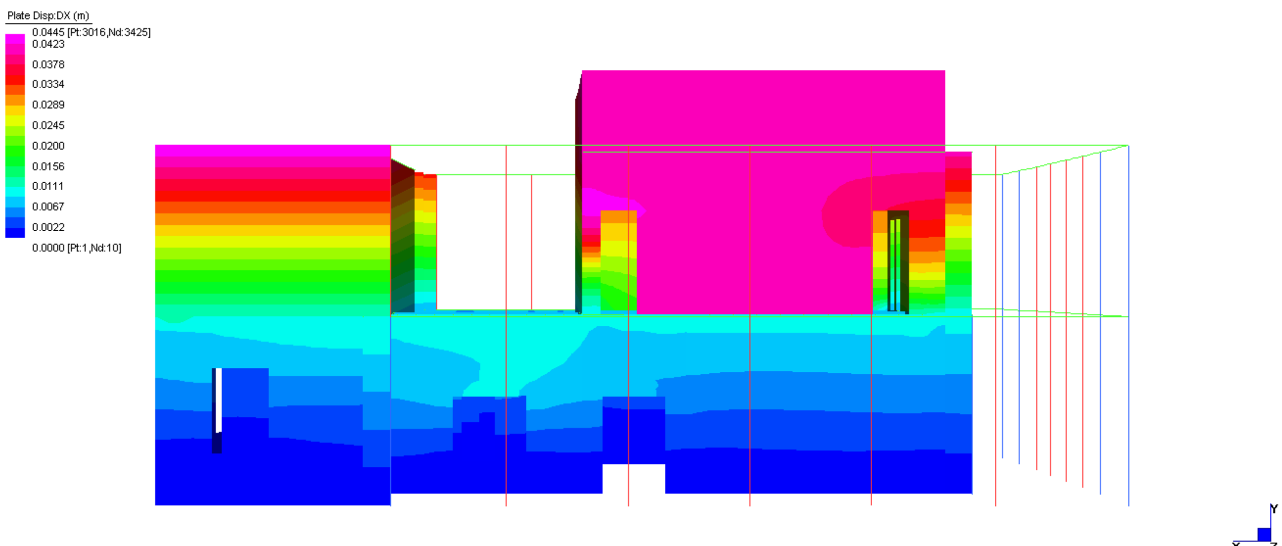


Figure 7.34/b: Displacement of the South side (Model_2). dx: 44.5 mm

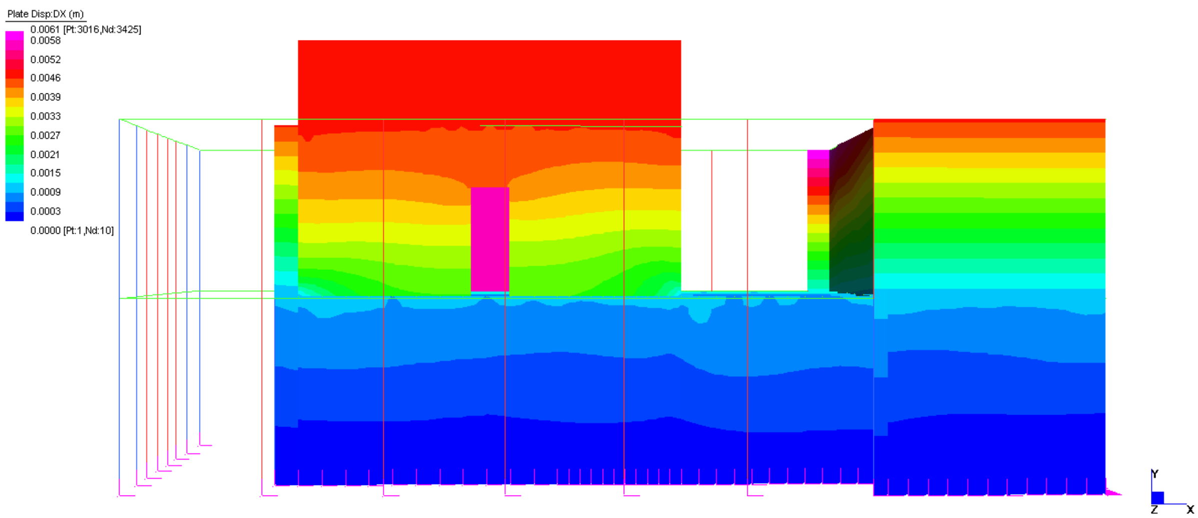


Figure 7.35/a: Displacement of the North side (Model_1). dx: 6.1mm

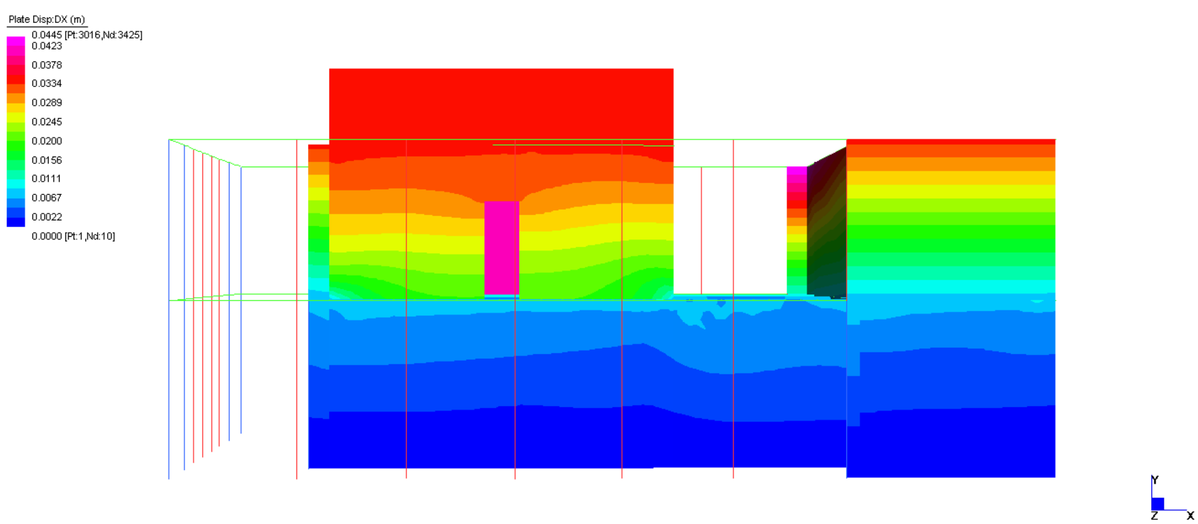


Figure 7.35/b: Displacement of the North side (Model_2). dx: 44.5 mm

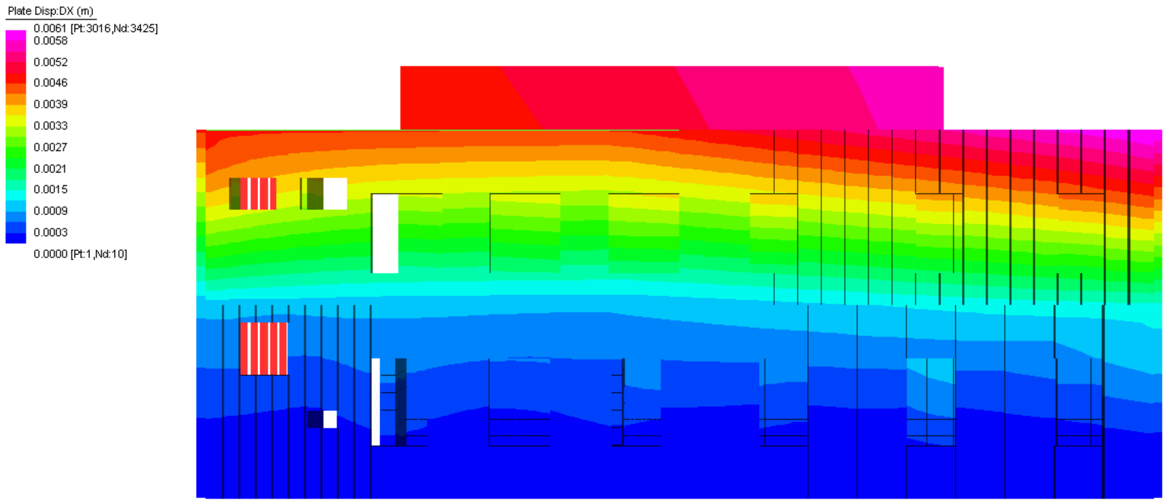


Figure 7.36/a: Displacement of the West side (Model_1). dx: 6.1mm

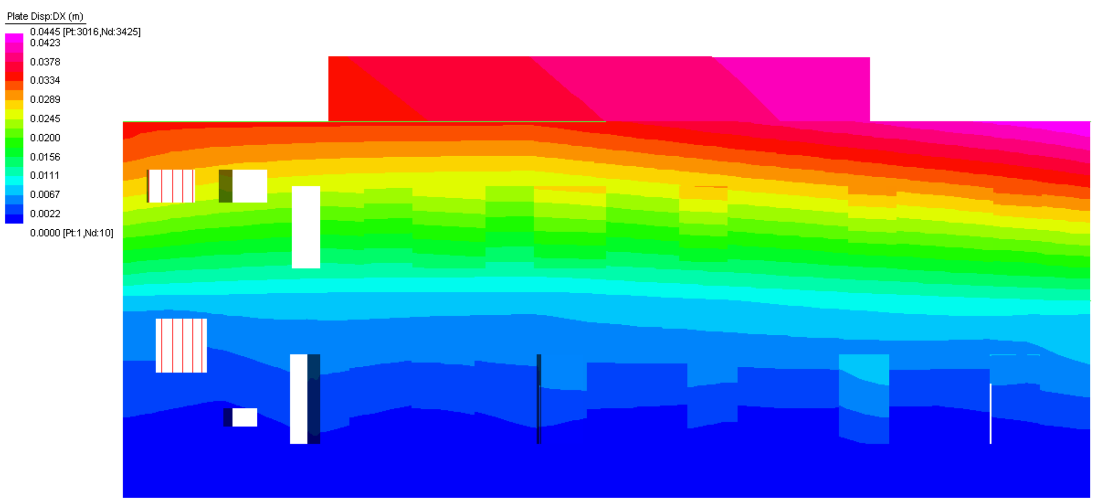


Figure 7.36/b: Displacement of the West side (Model_2). dx: 44.5 mm

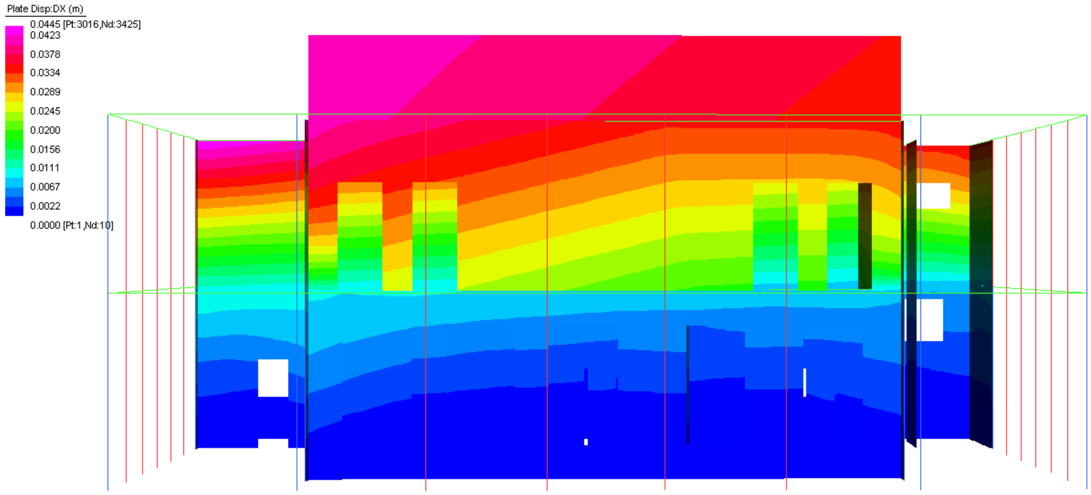


Figure 7.37/a: Displacement of the East side (Model_1). dx: 6.1mm

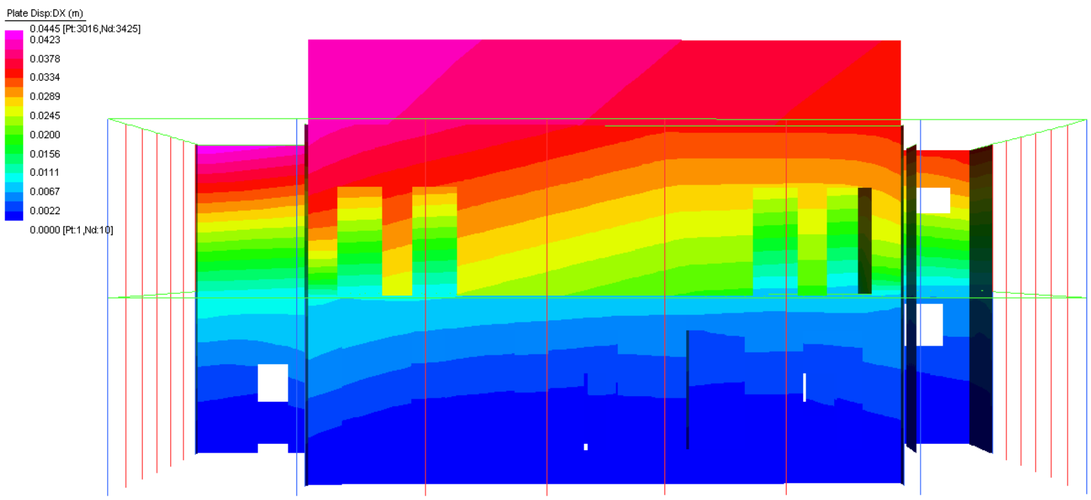


Figure 7.37/b: Displacement of the East side (Model_2). dx: 44.5 mm

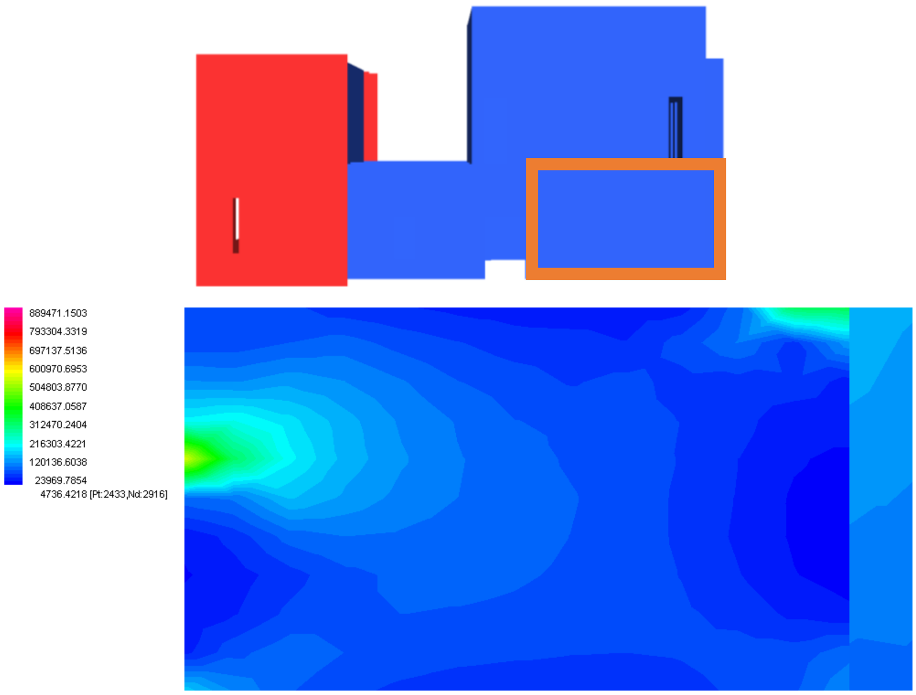


Figure 7.38/a: Stress distribution of the masonry wall at the left entrance (Model_1). SXX: 0,0047-0,89 MPa

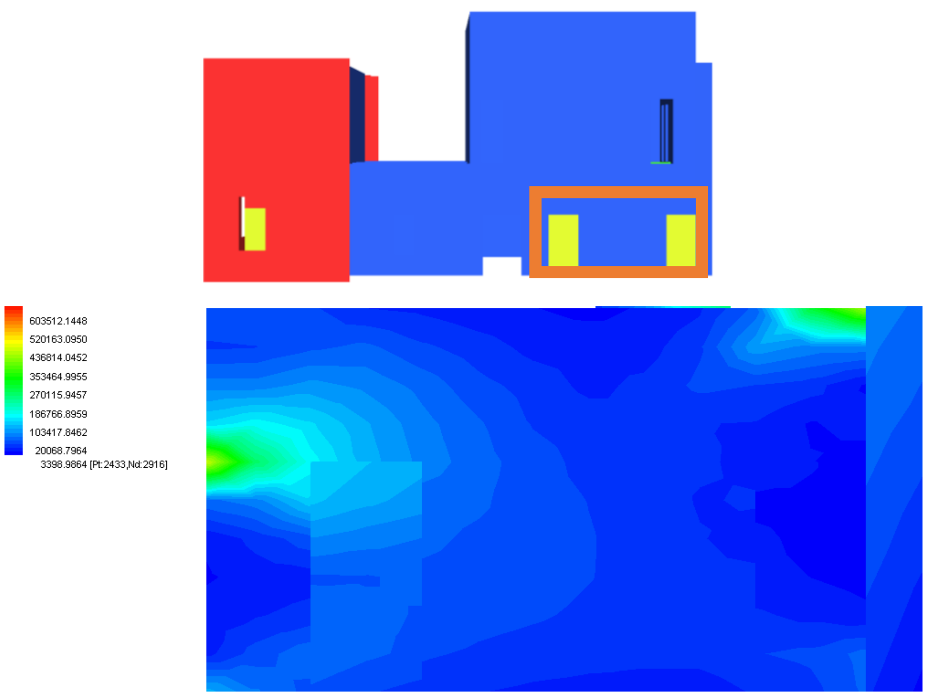


Figure 7.38/b: Stress distribution of the masonry wall at the left entrance (Model_2). SXX: 0,0033-0,6 MPa

Table 7.8: Comparison between the two models.

	Model_1	Model_2
Max. displacement	dx = 3.5 mm dy = 7.1 mm at node = 4053	dx = 25 mm dy = 53 mm at node = 4053
Max. stress	x-x = 2.64 MPa at plate = 4115 y-y = 2.66 MPa at plate = 3022	x-x = 2.47 MPa at plate = 4115 y-y = 2.61 MPa at plate = 2305

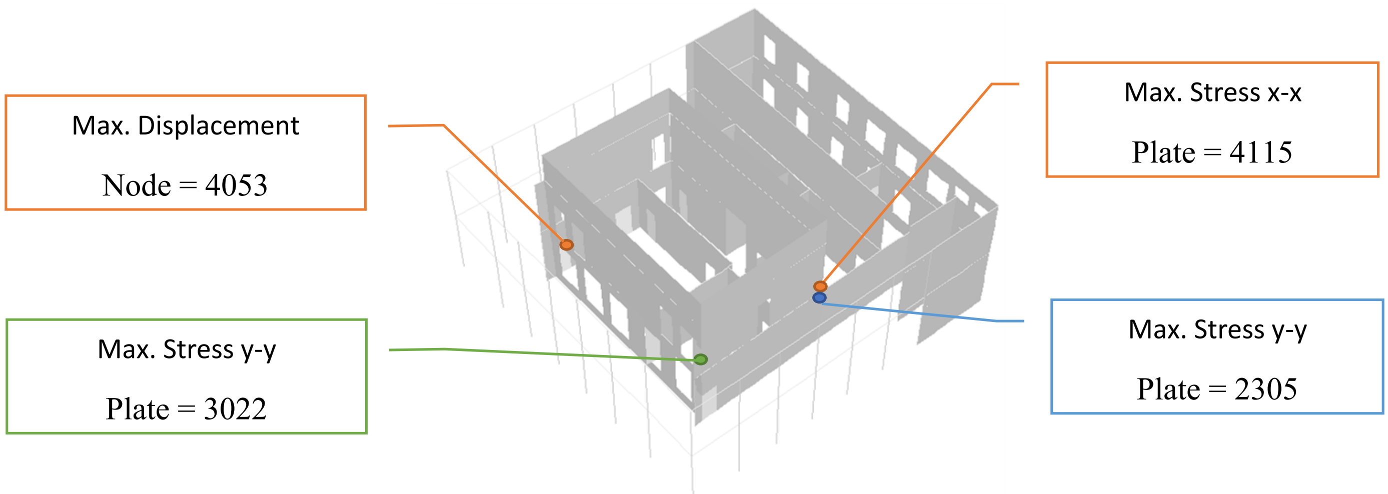


Figure 7.39: The location of Maximum Stress & displacement of the two models on the building.

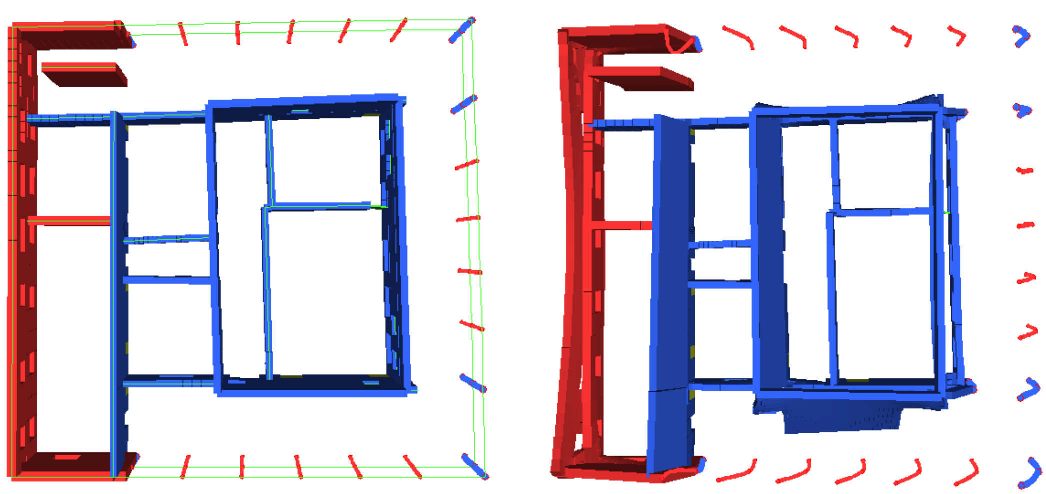


Figure 7.40: Undeformed and Deformed building (Model_2) (scaled x100).

Results and discussion

The prime concern is given by the difference in results by conducting, the two models: the first model takes in consideration the material library integrated in STRAUS7, while, the other model uses the qualitative and quantitative data obtained by the experimental investigations.

The studied models of the building are investigated for their performance in-terms of exhibited displacement for the existing floors and masonry walls. The maximum out-of-plane and in-plane displacement of masonry walls, and the maximum stress of the building are shown in (Table 7.8).

Due to the lower number of cross-walls and large size of rooms, the horizontal displacement of the building at the first floor level is not same as at the ground level. The displacement at the floor level is different in each span of the wall. The maximum displacement of (Model_1) and (Model_2) are at the same node (4053), but with higher value obtained by (Model_2).

It is clear that, following a knowledge path in structural assessment has a significant effect on the obtained results by the numerical modelling. The presence of the structural modifications along the walls in the model and the use of true mechanical parameters obtained by experimental techniques have a major effect in the displacement value which is drastically increased, due to loose connections between the old and new masonry walls which cannot act as an individual body and allow for the differential movement, leading to collapse of the walls, and due to a reduction in the modulus of elasticity obtained by the experimental testing values.

The global behaviour of the structure is similar for the two models, as shown in (Fig.7.30)-(Fig.7.37). In which the stress distribution along the masonry walls is similar even with respect to the value of max. stress as shown in (Table 7.8). Nevertheless it is clear that the max. stress is obtained in the zone of the large-size meeting room at the first floor. This hall separates the cross-walls apart greatly, making the walls more vulnerable to overturning during an earthquake.

The stress distributions obtained by (Model_1) nearly agree with those obtained by (Model_2). There were no vulnerability of the building behaviour due to the absence of adequate connections between the closed opening and the old masonry walls. This has been highlighted by (Fig.38). But the absence of effective connections between the new and old masonry leads to overturning and collapses of the closed opening under seismic loading due to the lack of interconnection and confinement behaviour. Results suggest a proper strategy to improve the structural assessment approach by taking in consideration both qualitative and quantitative data (structural homogeneity, material characterization) obtained by NDT and mechanical techniques.

The part of the study has presented a multiphase approach for structural analysis of historic buildings. Engineers are asked to cover a number of unknowns regarding the historical building, such as: materials properties, structural geometry, connections between structural and non-structural elements, stiffness of the masonry walls, and existing damages. The combination of simple computational model with the data obtained by the experimental investigation can offer proper indications for a subsequent behaviour of the structure.

The study on Palazzina della Viola leads to the following conclusions.

1. In order to simplify the analysis and interpreting the results, the masonry walls and columns are the only structural elements taken in consideration through the numerical modelling.

2. The seismic response of the building highly depends on the floor rigidity, geometry of the longitudinal and transverse masonry walls and their material homogeneity.

3. The large size of the meeting room in the first floor without cross walls is increasing the risk of having a collapse.

4. For the modelling and analysis of historical masonry structures, the effect of the non-homogeneity along masonry walls should be considered beside to the parameters obtained by the experimental techniques.

5. The global behaviours of the two models are similar.

6. The most significant difference between the two models appears due to the displacement, while the stress distributions have minor differences due to the changes in materials properties.

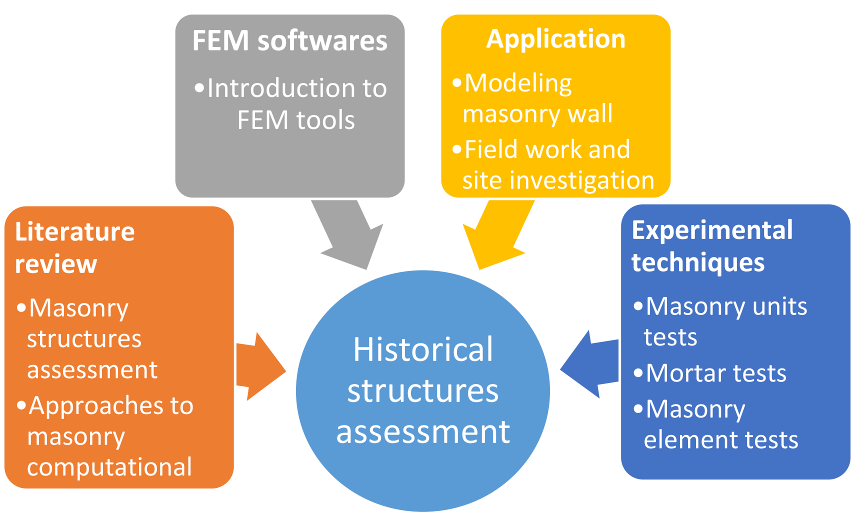
Chapter 8

Synthesis

8.1 Discussion of results

The structural assessment of historical structures is the evaluation of the collected data related to the safety of the building, with the scope of deciding whether the structures is safe or not. This study is aimed at investigating historical masonry structures at two different levels; experimental techniques and computational approaches. The objective is to review the experimental techniques that are used for determining mechanical characteristics of masonry, as shown in chapter 4. These experimental tests were divided into different categories depending on the tested masonry element; masonry unit, mortar, natural scale masonry element. Different techniques used to obtain the same mechanical parameters are compared by reviewing different international codes, such as ASTM and RILEM. In this study more attention was given to the structural assessment based on modelling the structural behaviour, through certain criterions. Where the second part of the study presented in chapter 6 is aimed at a comparison between different commercial FEM softwares with respect to the needed mechanical input parameters of each for linear and nonlinear analysis. The presented case study is a numerical model of a masonry wall with opening.

In order to assess the combined results of the two parts of this study, the research can be represented as shown below:



8.1.1 Literature review of masonry structures assessment and computational approaches:

Masonry is considered as the basic construction material that have been used in the historical structures. The literature review of the assessment methodologies of historical masonry structures, shows that structural assessment must follow basic principles, such as: material compatibility, conservation of overall lay-out, avoidance of the removal of any part, or additions to the structure elements. The understanding of structural behaviour and material characteristics is essential for any assessment related to the historical masonry structures. It is recommended that the work of analysis and evaluation should be done with the cooperation of the specialists from different disciplines. In addition, it is considered necessary for these specialists to have common knowledge on the subject of assessment and conservation of historical structures. The construction materials must be thoroughly investigated, by analysing the results obtained from experimental tests regarding geometrical data, internal structure, in situ strength of materials, structural properties of masonry walls, and in some cases the dynamic response of building elements.

On the other hand, the modelling of the historical structures is the set of structural elements used to represent the structural functioning of the building. The model should represent the structural behaviour of the building and the specific phenomena which are related to the study by using different numerical approaches. In fact modelling historical structures is more difficult and less reliable than in the case of new buildings. This is due to several factors, such as: the uncertainties related to the characteristics of the materials in the whole building and the influence of past events. The data obtained from the modelling may be useful, at least giving an indication of the structural behaviour and stresses distributions, which help to determine the critical zones. Besides being helpful in the design of strengthening phases, by comparing the results obtained from the modelling of the existing building with the results obtained for the same building on which the strengthening measures have been applied.

There are several approaches for modelling both static and dynamic analysis with different criterions with different models that can be used for simulating the behaviour of masonry structures. Micro-modelling and macro-modelling are considered as the main two approaches, the first approach represents a fully detailed analysis and modelling of masonry structures, which is appropriate for small structural masonry elements with specific research interest in heterogeneous states of stress-strain relation. On the other hand for the global structural behaviour where the interaction between units and mortar can be negligible, macro-modelling approach is applied, in case of considering the masonry structure as composite material characterized by average strains-stresses, where it is performed in large scale homogeneous states of stress with respect to the masonry experimental tests. Besides taking in consideration both micro and macro modelling, many researchers tried to produce more specific models, such as modelling bricks and mortar joints with respect to elastic analysis and apply the non-linear behaviour of masonry, considering masonry as a two-phase material.

8.1.2 Review of experimental techniques:

This part of the study shows the applications of experimental techniques, which can be used for obtaining mechanical parameters of numerical models. Three different categories have been considered for dividing the experimental techniques; Brick unit, mortar, and structural element. There is no single experimental techniques that is appropriate for all situations. To keep a high level of structural performance, the characterization of material properties and damage as a function of time and environmental influences should be taken in consideration in the structural assessment. The experimental techniques generally are divided into destructive, semi-destructive and non-destructive tests. Destructive tests can be applied to lab-made or site-extracted specimens of structural elements; both are completely destroyed during the tests. Semi-destructive tests can be applied also to lab-made or site-extracted specimens and in situ structural elements but they involve a small devastation into the structure and the need of repair is required. Afterwards this guides us to the non-destructive tests which can be applied to different structural elements without having any destructions. Most non-destructive tests are used to show a quality evaluation of the structure, beside it is fast, easy to use at site and relatively less expensive.

Strength determination of historical masonry structures is important because it represents its elastic behavior, in which it used to predict the structural performance by implementing the strength characteristics in numerical model. Non-destructive tests do not directly give the absolute values of strength, typically they evaluate the quality of masonry from which an estimate of its strength and other characteristics can be made. Beside they have the ability for establishing uniformity and detecting cracks or defects. When masonry structures have a variation in properties through one structural element, it affects the obtained results through the numerical model. So the use of one experimental technique alone will not be sufficient to study and evaluate the required mechanical characterization. Where the use of more than one method leads for more reliable results. By combining different non-destructive tests to assess the structures. For instance, the increase in humidity and water content of masonry increases the ultrasonic pulse velocity but at the same time it decreases the rebound number of Schmidt Hammer. But using both techniques together will reduce the errors in the obtained results. The different between actual results and predicted results may be controlled by extracting samples from existing structures as cores and obtain the compressive strength with respect to each position. Then constructing a plots to represent the comparison between the strength values and the quality index.

Experimental techniques are based on different principles, each with their advantages and limitations. It has been notice that non-destructive techniques play an important role in historical structures assessment, and there is a need to develop and perform techniques that have the capability to test in situ and giving a direct mechanical parameters.

8.1.3 Comparison of FEM softwares:

There are several commercial FEM softwares, available for modelling masonry structures with a variety of different tools that employ linear and non-linear analysis. The most important matter of the comparison involves the theoretical basis of the programs where masonry structures are concerned; including materials libraries, brittle material models and the mechanical parameters needed for analysis.

The investigation of the FEA tools which are integrated in each software aims to show what can be achieved by using different mechanical characteristics of the masonry elements implemented in the analysis by simulating a masonry wall. Typically, the users do not compare the analytical results given by the softwares with the theories behind the simulation. So the basic question was how significant the results are and whether or not they can be trusted.

From the comparison between FEM softwares with respect to the needed input parameters and the theoretical basis of numerical analysis, as have been shown in chapter 6, the difference in results between FEM softwares is relative to the quality of the input mechanical parameters. In simple cases, it is possible to have liner analysis by using modules of elasticity, material density and Poisson ratio, while for nonlinear analyses, many parameters can be required, depending on which failure criterion the FEM software is based. So the accuracy of the models depends on the amount of inputs used by the FEM softwares in order to restrict the mechanical behaviour of the structural model. This is also correspondingly related with the accuracy of the applied experimental techniques used to obtain the mechanical characteristics.

The model case deals with the analysis of simple masonry wall subjected to vertical and horizontal loading conditions. For this purpose, the models by different FEM software's are used to predict the amount of damage, by investigating the cracks patterns and stress distributions associated with each model

All simulations of the proposed masonry model were done by modifying the standard settings by the finite element analyses tools for each software in order to get an analysis in which the results can be comprised with the same finite element procedures and fixed meshing basics. The structural analyses were done with finite element meshes consisting of linear 4 nodes. The results showed a realistic structural behaviour with respect to the input parameters that have been used for the linear and nonlinear analysis, in which a comparison has been done with different study cases for the global behaviour of the model. The stresses calculated by the softwares showed significant differences with respect to the FEM tools. The possible reasons for these results may be found in differences in the integration order the programs used or in the internal routines utilized to compute stresses. These internal routines of the integrated modules cannot be analysed by the user nor can they be altered. The difference between the results with respect to the displacement were relatively high in both linear and nonlinear analyses, from one side it is acceptable for the basis of each analysis approach, on the other hand, homogenized isotropic masonry wall was the adopted approach in the model, which is considered as poor computational method for specific and detailed study cases. But most of the results showed the same mechanical behaviour by each software. As a result it is assumed that the predicted maximum stress can only be a useful indication for strength determination and further comparative studies have to be done in order to verify the results.

8.1.4 Case studies:

The experimental program presents case studies from different historical structures in Bologna. As the engineering propose to measure the capacity of their seismic resistance, different experimental investigations were included in-situ and laboratory. Through the experimental tests, several critical factors has been noticed which can affect the obtained results, such as the selection of the investigated masonry walls, the conditions of obtaining and preparing the samples, and the post-process of analysing the results with respect to the theoretical basis and number of samples.

The mechanical properties of the masonry structures do not depend only on brick and mortar properties as individual material, the interaction between the masonry structures parts have an important role in the way how masonry structures behave during the application of different kind of loads. Continues leaves walls have been investigated by limited removal of the plaster in the corners of masonry walls.

Chapter 9

Conclusions

9.1 Outcome of work

Historical masonry structures are difficult to study with respect to the large number of factors influencing the mechanical behaviour and stability of the structural elements. A review of different topics was performed in this study, starting with experimental techniques for obtaining the mechanical characterization of masonry structures, progressing with numerical modelling approaches, ending with practical applications of the structural assessment.

It emerged that both experimental techniques and computational approaches are essential to understand and control the behaviour of this kind of constructions. It is important to stress and reinforce the interactive role in structural assessment of the technical and analytical figures, by developing a strategy to extract all the needed parameters for FEM tools by proper experimental techniques. In function of the adopted strategy, the accuracy of outcome from numerical models is related to the material description, especially in describing the post-peak behaviour during nonlinear analysis.

In this study, a description of the experimental and computational approaches has been presented, explained and discussed. The main conclusions reached by the study are divided with respect to each part as follow.

9.1.1 Review of experimental techniques:

- Most of the techniques introduced by ASTM and RILEM standards have procedures suitable for typical and modern masonry structures. That is they do not take into consideration the particularity of historical structures like the complexity of geometry (thick walls, multiple-leaves wall sections, irregular masonries, …) and variability of the materials, which makes it difficult to apply the experimental techniques by following the standards. Hence, with respect to the used experimental techniques, it seems important to have a flexible methodology for diagnosing historical masonry structures.

- It would be important to undertake more experiments regarding definition of the material properties of masonry, especially if providing information on for nonlinear behaviour - such as the fracture energy, which, at the present state, cannot be determined through in situ experimental techniques -, as the fracture energy value is mostly based on assumptions. There is a need to develop new techniques to measure such mechanical parameters. Further it would be advisable to perform tests on the mortar and bricks to see what the influence of the components are on the properties of the masonry as composite.
- Experimental destructive techniques on brick and mortar specimens in the lab do not necessarily give direct and specific information about the masonry in the actual structural state. The tested specimens are not truly representative of the masonry in the structure, due to losing the true conditions of the surrounding state of stress. On the other hand non-destructive techniques are easy to apply, quick to perform and are less costly. Non-destructive techniques cause minor damage to the structure and do not affect their performance and general appearance.
- An important role of semi destructive and non-destructive techniques is that they permit retesting the same location so that changes can be monitored with respect to the time.
- In some of the non-destructive techniques the strength of masonry is evaluated as quality index and not measured. Nevertheless, the relation between the quality index being obtained and masonry strength (compressive, tension... etc.) can be determined in the lab and then used to convert the results obtained by non-destructive techniques into strength values.
- Experimental techniques are based on different principles, each has its own advantages and limitations. It has been noticed that non-destructive techniques play an important role in historical structures assessment, although not always providing direct quantitative information.
- Masonry structures may present wide variation in properties even across one individual structural element due to the aging effect. As long as this variation is not taken into account, it will affect the results of the numerical model. Thus, the use of one

experimental mechanical technique alone, or too few of them, will not be sufficient to study and evaluate the required mechanical characterization of the structure. While using more than one method leads to more reliable results, combining different non-destructive methods with mechanical tests would greatly improve the assessment of the structures.

- The splitting test of masonry cores with mortar joint is considered an efficient way to obtain the compression and shear capacity of the masonry element, but in case of having the possibility of testing only few core specimens (if the mortar is weak, the core may separate during extraction or transport to the lab, thus too few data may not be representative), the obtained results may not give true indication of the mechanical properties. Further, due to the need to use water during extraction and preparation of the samples, this may cause additional weakness and early failure of the cores.
- Masonry is typically a non-elastic, nonhomogeneous, and anisotropic material composed of two materials of different mechanical properties, in which the strength of mortar has not an accurate, easy and direct way to be obtained with respect to in-situ techniques. Even the helical steel screw and Gucci's penetrometer have limitations regarding the minimum possible strength can be measured and the width of the mortar joint, so they are used to give a quality index for the structural elements more than being reliable for giving a unique strength value .
- Historical masonry structures have various wall thickness but mainly large thickness and leaf-construction, which cause in some cases a difficulty to apply in-situ techniques, such as flat-jack test, as the slot does not cross the entire thickness of the wall, while the stress conditions are different between the internal and external masonry leaves. Repetition at greater depth or another technique is then used for the interior leaves of masonry walls, such as Dilatometer or Hole drilling tests.

9.1.2 Comparison of FEM softwares

- The differences in the numerical results given by the FEM softwares are caused by the theories adopted to perform the analysis. So it is significant to understand the computational and theoretical basis of the masonry structural mechanics used by each FEM software in order to control boundary conditions of the analysis.

- The difference in results between FEM softwares is related significantly to the number of mechanical parameters used in the models. In simple cases, it is possible to perform liner analysis by using modules of elasticity, material density, and Poisson's ratio, while for nonlinear analysis many parameters can be used, depending on which failure criterion the FEM software is applying, such as Mohr Coulomb and Drucker Prager. The accuracy of the model depends on the amount of input used in the FEM softwares in order to restrict the mechanical behaviour of the structural model, which also correspondingly is related to the accuracy of the applied experimental techniques to obtain the mechanical characteristics.
- From the results obtained by modelling the masonry wall, it can be said that the macro model did not provide an accurate comparison between the FEM softwares, of the representation of structural behaviour of the masonry wall. The models were not able to provide a fully detailed failure behaviour of the masonry wall with regards to the displacements, the stress and maximum crack width. Each model seems to work according to its own theoretical criteria, adopted by the FEM software. More detailed micro models are needed that can be used to determine more exact stress-strain behaviour. An anisotropic model might be the best option to model the material behaviour of masonry instead of the homogenized assumption.
- It becomes clear with this study that the anisotropy property of masonry components is important for the model. However, due to the limitations of the availability of information about the nonlinear behaviour of the materials, it would be interesting to have an expansion of the masonry wall model. For example, trying to implement and define a multi-linear stress-strain relation for tension and compression in different directions or defining anisotropic material parameters the nonlinearity can be defined more accurately.

9.1.3 The application of historical structures assessment

- From the results obtained by in-situ and laboratory tests in different historical masonry structures, it appears that many factors affect the general framework of the assessment, such as the cost, number of specimens, the use of the building and the unique geometric, material and construction technology characteristics of each building.

- By following and observing the experimental investigations carried out in historical structures, it has been noticed that many factors affect negatively the obtained results from the in-situ and laboratory tests. It appears difficult to follow and apply the experimental procedures stated in the standards. Some general notes can be summarized as follows:
    - Poor preliminary visual and instrumented investigations, that is testing positions selected by visual inspection with reference to as built shop drawings and no use of NDT techniques for quality control of the structural uniformity of the chosen testing elements may cause great delays and errors in chosen testing positions. These positions may need to be changed due to the interruption by new built masonry walls, closing of passages and windows or presence of service pipelines. This makes true that the lack of corporation between the analysts and technicians in selecting the tested positions leads to greatly affect the accuracy of the obtained results in both the experimental tests and the numerical modeling.
    - The number and position of the extracted specimens do not match the masonry variety with respect to the total area and number of floors in each structure. This may lead to an incorrect indication of the overall or global strength of materials, with repercussions on the obtained results by the numerical modelling of the structure and on the assessment of its ability to resist the external actions.
    - When standards exist, it is important to follow them at the time of applying and executing the test, as the wrong procedure by untrained technicians could lead to time and cost consuming. For example, simple observations of the effects of wrong application of the hydraulic jack in a shear test, or of the overuse of water in boring the cores, may make realise that these do not match the procedures stated in the standards. Thus, it is important for the analyst to take into consideration all the above factors in defining the material properties for the numerical model, by considering that a certain kind of probability distribution could be applied to obtain a safety factor of the material characterisation.
- The investigated historical structures were furnished and used by different parties, which makes it important to prepare a general framework for the experimental

investigation, by scheduling and controlling the process of extracting the specimens and in-situ testing within the defined time and cost.

- Historical structures have been through many kinds of restoration and structural modification along the years, thus it is possible to find different materials from different construction ages and techniques. Based on collected historical information about the monument, as well as on capacity to observe and discern and on non-destructive inspections, it is important to decide the right positions of in-situ tests and for extracting the specimens with respect to materials age and health state. In this, a real cooperation between the experimental and numerical researchers is needed to obtain a complete model by implementing the right parameters with the right numerical approach to have an accurate analysis of the historical structures.
- The numerical analysis of the historical structures requires the mechanical properties, which are influenced by different factors, such as; the construction technique and physical properties of the materials. So it is significant to take in consideration during the structural investigation many aspects, such as the building dimensions, the number of storeys, the role of the structural element with respect to the structure as a whole, and any significant environmental conditions.
- Historical masonry structures are more complex than modern structures. It is important to follow the knowledge path_stated in the national standards.
- Numerical modelling must take in consideration both qualitative and quantitative data (structural homogeneity, material characterization) obtained by NDT and mechanical techniques.
- The experimental investigations must be planned with respect to the numerical modelling scopes in order to get the necessary material characterization without need to overestimate the safety factor.

Generally, historical structures assessment needs to set a strategy before starting the assessment. This strategy, together with many other aspects, will contain both the experimental tests and the numerical model constrains related with the degree of accuracy needed for a certain study. Interaction rules between the two parties are needed in order to reach a result with ultimate degree of accuracy. This can be represented by a 2-way feedback relation between numerical modelling and experimental techniques. In which the experimental techniques is not just about obtaining the mechanical parameters for the numerical modelling, they are going further for quality control and geometric definition.

9.2 Recommendations for further studies

Although this study included the most used experimental techniques, and gave indications about commercial FEM software, there were some limitations and additional research is needed in the following areas:

- Study non-linear behaviour of a single masonry wall modelled by different FEM softwares, comparing the quality of the numerical model aginst the different results obtained in natural scale laboratory tests.
- Compare different FEM softwares with respect to a detailed anisotropic micro model.
- Develop in situ experimental techniques that can measure the nonlinear parameters of masonry such as evaluating Mohr Coulomb parameters and fracture energy.
- Compare different FEM softwares with respect to the behaviour under the action of explicitly dynamic loads and taking into account a structural model in global scale.
- Study the influence of Poisson's ratio with respect to the brick/mortar interface in a micro numerical approach.

Table of figures

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