

A STATIC AND DYNAMIC FINITE ELEMENT ANALYSIS OF UNREINFORCED MASONRY WALLS

BY

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Submitted in partial fulfillment of the requirements for the degree of

Master of Science in Engineering

School of Civil Engineering, Surveying and Construction

University of KwaZulu-Natal

Durban

2020

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EXAMINER'S COPY

Preface

The work presented in this dissertation was performed through the Civil Engineering department in the School of Engineering at the University of KwaZulu Natal from 2019 to 2020, under the supervision of Dr. Georgios A. Drosopoulos.

As the candidate's Supervisor I agree to the submission of this thesis.

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Acknowledgements

My heartfelt gratitude goes out to the following people for their help and support during the period of this study.

Firstly, I would like to thank my supervisor Dr Georgios A. Drosopoulos for his guidance, time, support and willingness to assist me in any regard. Dr Georgios A. Drosopoulos' supervision has played a major role in all aspects of this study. A further thank you goes out to my co-supervisor Dr Mayshree Singh for her valuable insight and input. It was indeed a great honor to work under such accomplished and vastly experienced academics.

A big thank you goes out to all members in the department of Civil Engineering at the University of KwaZulu Natal who offered unwavering support and encouragement during the period of this study. This includes lecturers, academics, admin staff, lab workers, technicians, cleaners, peers, friends and colleagues.

Finally, my sincere gratitude goes out to my family; Anand Moonsamy, Sushie Moonsamy and Dasendri Moonsamy. This accomplishment would not have been possible without their dedicated support and continuous motivation throughout my years of study.

Abstract

This dissertation presents a numerical study of the structural behavior of unreinforced masonry walls. The study involves creating and testing computational models that simulate the behavior of unreinforced masonry walls under static and dynamic loading. The finite element method together with ANSYS software are used to create and analyze the numerical models. A standard 1-meter square, three-dimensional unreinforced masonry wall and a full-scale wall representing that of low-cost houses found across South Africa are developed and evaluated under general static structural loading conditions as well as dynamic loading conditions. A heterogeneous finite element model is used so that each brick and layer of mortar are defined separately with a contact interface existing between the two layers. This approach allows for a detailed examination of the local failure behavior of masonry. The numerical models will be used to investigate structural parameters such as the stresses, deformations and the failure behavior of masonry walls under static and dynamic loads. The Drucker-Prager failure criteria is adopted to simulate the non-linear failure behavior of masonry. The dynamic analysis is conducted using both Modal and Response Spectrum analysis. The results from the study are compared with similar research using both numerical models and experimental tests.

Keywords: Drucker- Prager, Modal, Response Spectrum.

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List of Abbreviations

DEM	Discrete element method
DOF	Degrees of freedom
DP	Drucker-Prager
FEA	Finite element analysis
FEM	Finite element method
LCH	Low-cost housing
MC	Mohr-Coulomb
MPC	Multi Point Constraint
RDP	Reconstruction and development program
RVE	Representative volume element
URM	Unreinforced masonry
XFEM	Extended finite element method

Chapter 1 – Introduction

1.1 Background and Motivation

Masonry is a commonly used building material that has a wide variety of applications which go back thousands of years. Masonry is commonly used in the construction of highly populated domestic buildings, as such, failure of masonry structures can lead to casualties. Masonry, as a building material, consists of brick/block/stone units held together with mortar, each of these constituents have their own material properties, geometries and arrangements, which, in combination, form different masonry assemblages. As such, masonry is considered a heterogeneous and anisotropic material, making the structural analysis of masonry buildings complex.

In modern times, numerical modeling techniques have become increasingly popular to study the behavior of structures. Experimental research often requires expensive equipment and hard-to-source materials. Numerical simulations help in determining regions of large displacement, initiation of cracks, regions of large stress and weak points in a structure. Numerical methods such as the finite element method (FEM) and the discrete element method (DEM) have been widely used in the past to carry out numerical analysis of masonry structures. Even with modern numerical methods, the structural analysis of masonry remains challenging. This is due to several reasons which include.

- The non-linear behavior of masonry.
- The low-tension strength and fragile failure characteristics of masonry.
- The heterogeneous composition of masonry.
- The planes of failure which can occur along various bed joints.

With an estimated 2.8 billion people living in poverty worldwide, there is a significant need for low-cost housing. It is estimated that 35 million housing units will need to be produced each year in order to keep up with the housing demand in developing countries (Meyer, 2006). Masonry is an attractive choice of building material especially for high-volume, low-cost housing developments. Unreinforced masonry, which is commonly used to build low-cost housing, is strong in compression but weak to out-of-plane loading and tensile forces, thus making it extremely susceptible to failure when exposed to dynamic loads.

An ever-increasing population places an increasingly high demand on infrastructure development and the built environment, which often leads to poor construction and increased failures in structures. This study uses numerical modelling methods, to simulate the behavior of unreinforced masonry walls, often found in low-cost houses, under static and dynamic loading. The information gained from this study can contribute towards improving the design and construction of future masonry buildings as well as the maintenance of current masonry infrastructure.

1.2 Research Questions

- How does a 1-meter square unreinforced masonry wall behave under static loading, using non-linear and FEM methods?
- How does a full-scale, low-cost house, unreinforced masonry wall with door and window opening behave under static loading, using non-linear and FEM methods?
- How does a full-scale, low-cost house, unreinforced masonry wall with door and window opening behave under dynamic loading, using Modal, Response Spectrum and FEM methods?

1.3 Aims

- Understand the concerns around low-cost housing in South Africa and its vulnerability to static and dynamic loads.
- Understand the basic concept of numerical masonry modelling using the FEM.
- Investigate the structural behavior of a 1-meter square masonry wall under static loading, including non-linear effects.
- Investigate the structural behavior of a full-scale masonry wall with door and window opening, representing a low-cost house, under static and dynamic loading.
- Use engineering knowledge to evaluate results generated.

1.6 Objectives

- Carry out an extensive literature review to learn about low-cost housing, masonry modeling and the finite element method.
- Create geometries representing masonry walls found in low-cost housing across South Africa.

- Create numerical models and generate results using ANSYS software to investigate the behavior of masonry walls under static loading using the Drucker-Prager failure criteria, which enables a non-linear analysis.
- Perform a Modal and a Response Spectrum analysis to evaluate the behavior of masonry walls under dynamic loading.
- Investigate the stresses and deformations within the structure under static and dynamic loading.
- Compare results with existing numerical and experimental investigations.
- Evaluate the findings to draw meaningful conclusions from the study.
- Provide recommendations and scope for areas of future research.

1.6 Research Plan/Methodical Approach

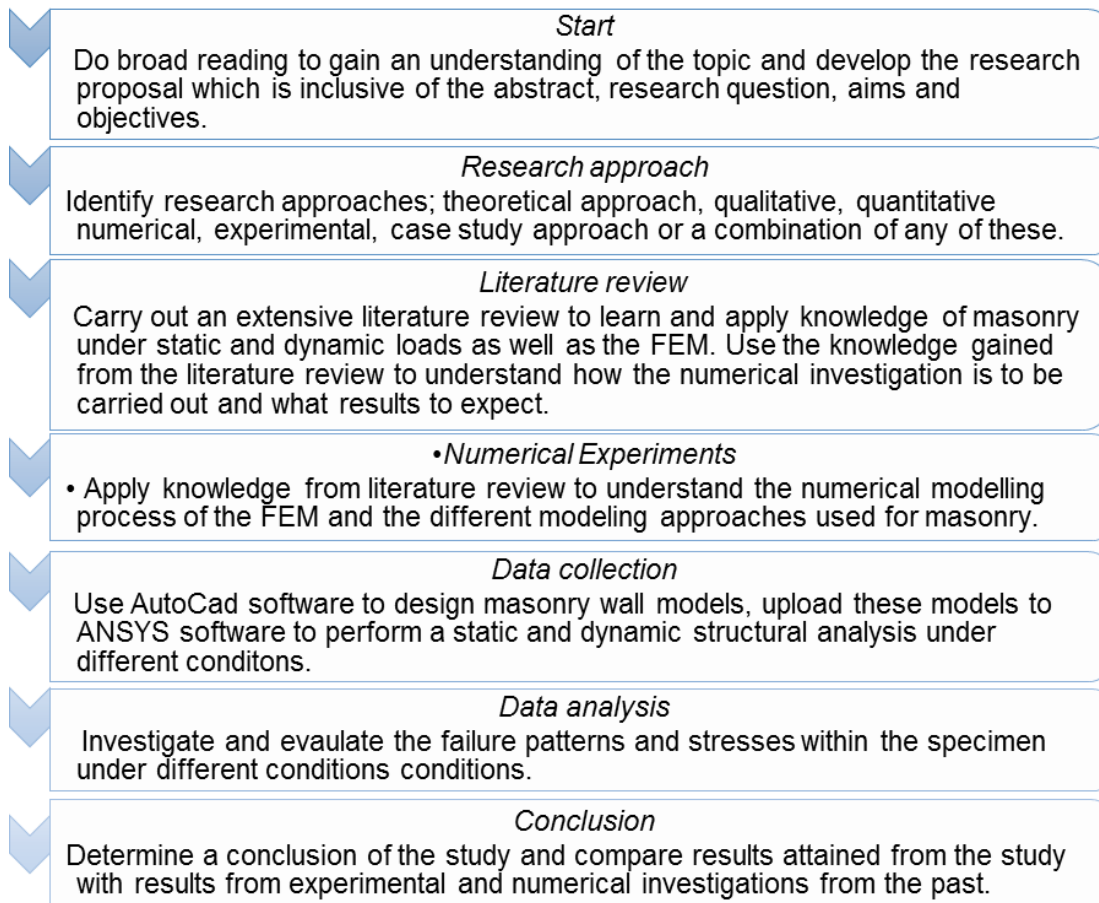


Figure 1-1: Breakdown of research plan for the study.

1.7 Structure of Dissertation

This dissertation is presented in 5 different chapters, excluding the reference list. The chapters are presented sequentially, with each chapter picking up from the previous chapter.

Chapter 1 - consists of the introduction, which provides the necessary background and motivation for the study. Chapter 1 also includes the research questions, the aims and objectives, research plan and structure of the dissertation.

Chapter 2 – contains the literature review which serves to provide a basis to gain background knowledge of the relevant areas of study. This consists of a review of various literature resources such as scientific journal articles, conference proceedings, patents, past theses and websites. The literature review focuses on the assessment of masonry, its material properties and behavior under static and dynamic loading. The literature review also includes a background to seismic activity and low-cost housing in South Africa as well as basic principles of finite element modelling and its applications to masonry.

Chapter 3 - provides a methodology which explains how the numerical experiments are carried out. The methodology presents the steps that are followed in creating the numerical models and performing the static and dynamic analysis. Tables, diagrams and flowcharts are used to illustrate how each numerical experiment is set up.

Chapter 4 - presents the results of the numerical investigations that are described in Chapter 3. This chapter presents the graphical outputs of the experiments illustrating stresses, strains, deformations as well as force-displacement diagrams. The results of each numerical experiment are also discussed in this chapter.

Chapter 5 - presents a concluding summary of the study. The aims, objectives and research questions outlined in Chapter 1 are discussed. This chapter also provides recommendations and scope for possible future work in a similar field of study.

Chapter 2 – Literature Review

2.1 Low cost housing in South Africa

2.1.1 Background to low-cost housing in South Africa

The South African government initiated the Reconstruction and Development Programme (RDP) to provide basic housing to disadvantaged citizens. However, there has been harsh criticism about the inferior building quality of these low-cost houses and the lack of basic services and amenities around these development projects (Ngxubaza, 2010). Statistics South Africa reported in 2016 that up to 18.6% of South Africans reside in informal dwellings. eThekweni is the third largest municipality in the country by population and the demand for low cost housing in the area is a concern. Over a million residents in eThekweni reside in informal dwellings (Statistics South Africa, 2016). According to eThekweni's Integrated Development Plan 2017/2018, the backlog of low-cost houses stands at over 387 000 units. To date eThekweni has delivered 186 000 RDP homes since 1994 (eThekweni Municipality, 2017). Although there are number of social and economic issues relating to the backlog, improved design and build quality will ensure a decrease in failure, saving the government time and money.

A Statistics South Africa census survey conducted between 2002 and 2010 revealed that 31% of people living in RDP houses regard them as substandard and uninhabitable. A further study conducted in 2011 found that 17% of RDP house owners complained about major cracking in walls (Statistics South Africa, 2016). According to the Ministry of Human Settlement, government spent well over two billion rand between 2014-2015 alone to fix poorly built RDP houses (South African Government, 2018)

Although the government has made attempts to provide low-cost housing at a good pace, the rate at which the need for housing is increasing is higher than the rate at which low-cost housing is being delivered. (Ghislaïne, 2015) mentions that the housing backlog of RDP housing in South Africa stood at 2.2 million in 2010. One of the main contributing factors to the backlog is the poor construction quality of the houses, resulting in government being liable for costly and time-consuming refurbishments.

Research by (Khoiratty, 2016) shows that common damages associated with low-cost housing in South Africa are cracking starting from window edges, door edges and intersection of walls and roofs. Analysis and research into masonry can lead to improved maintenance capabilities of existing infrastructure as well as improved design and construction for future masonry structures.

The South African government has embarked on a major housing project aimed at replacing informal shacks with low-cost housing for poor families across the country. RDP houses are generally built on the outskirts of main cities, thus making use of land which is vacant and less valuable (Greyling, 2009). New RDP houses consist of a simple single-story building, large enough to accommodate a single family. Initially low-cost houses consisted of a two-room block and mortar structure with corrugated iron sheeting used as the roofing material. In recent times however, low-cost housing can consist of a 5-room masonry structure with corrugated iron roof sheeting or clay roof tiles. The initial size of a RDP house was just 16m², however that has now been increased to 36m² (Greyling, 2009).

At the early stages, the development of low-cost houses was focused on quantity and cost. However, in recent times focus has shifted to key ideas such as sustainable development, environmentally friendly construction and social impact assessments. The National Home Builders Registration Council (NHBC) was introduced in 1997. Inspections carried out by the Department of Housing show that low-cost houses built pre-1997 are not up to standard. Many houses have serious issues which include severe cracking in walls and foundations, leaking roofs, doors, windows and sanitary fixtures not built in correctly. Sub-standard workmanship and low-quality materials are apparent to see in many low-cost houses. Repairs and maintenance to badly built RDP houses has costed the government billions (Greyling, 2009).

2.1.2 Design of low-cost housing

Size – The average approximate floor area for new low-cost housing is 36m², however this could go up to 40m². It should be noted that older RDP houses can be found to be as small as 16m².

Foundations - Strip foundations are generally used. However, the same designs are used throughout the country. Developers pay little attention to geotechnical reports which vastly vary across different regions and landscapes. Strip foundations are not suitable to use in regions with wet and clayey soil. Choosing the correct type of foundation has a drastic effect

on wall and foundation cracking. The strip foundations used for RDP dwellings form part of the shallow foundation type. This type of foundation requires strong sub-soil as it transfers the building load to the sub-soil. If the sub-soils are not strong the building becomes susceptible to cracking and even collapsing (Hlatshwayo, 2016).

Walls - Clay bricks, concrete bricks or concrete blocks together with mortar are used to construct the walls. The choice of brick or block is dependent on the region; however, majority of low-cost houses are made using concrete blocks and mortar as this is the most affordable option (Greyling, 2009). Walls are approximately 140 mm thick with the brickwork/blockwork held together with cement mortar joints. The masonry units are stacked on top of one another with mortar between them to bind them together and to hold them in place. Some houses are painted and plastered on external and internal walls, while some are only plastered and painted on the inside. (Ritchie, 2009) conducted a study to investigate the possible wall types which could be used in RDP houses. Masonry walls are used to construct almost all RDP houses, however possible alternatives such as prefabricated framed walled construction and polyblocks are an option. The study concluded that the prefabricated framed walls and the polyblocks permanent formwork were much faster than standard masonry methods, however, masonry remained the most cost-effective method of erecting walls for low-cost houses.

Roofs – Corrugated metal sheeting attached to timber purlins is used with some recent RDP houses making use of clay tile roofs. Low-cost houses do not include gutters and downpipes, thus leaving standing water at the base of walls, causing erosion. If gutters and downpipes are incorporated, rainwater can be stored and used as non-potable water, saving the government money in the long term.

Doors and windows - Steel door and window frames are currently used. In coastal areas, efforts have been made to use wooden frames to combat rusting. Prestressed steel door frames are usually 813 x 2032mm in dimension. Steel window frames come in varying sizes. Bathroom window – 533 x 654mm, bedroom window – 1511 x 949mm general window – 1511 x 1245mm (Department of Human Settlements, 2012)

2.1.3 The vulnerability of low-cost masonry housing to seismic activity

2.1.3.1 Seismic activity in South Africa

Seismic earthquake waves result in ground motion, when this motion reaches the earth's surface it can have devastating effects on human-built structures. Strong ground motion exposes buildings to forces which it might not be designed to withstand. As is the case with low-cost housing in South Africa, these houses are not designed to withstand the dynamic effects of an earthquake. South Africa lies in a stable continental region with low levels of seismic activity, however over 27,000 earthquakes have been recorded since 1650, ranging from *ML* 0.2 TO *ML* 6.3 (Khoiratty, 2016).

Table 2-1 lists all the seismic events that have occurred in South Africa with a magnitude of 5.0 or larger on the Richter Scale. According to (UPSeis, 2017), minor earthquakes above 2.5 *ML* can cause slight damage while earthquakes with an *ML* greater than 5.4 can cause major structural damage to buildings. According to the Geomatics Department at the University of KwaZulu Natal, minor earthquakes do not cause structural damage to large and well-built structures, however, small and hastily built structures such as low-cost housing have been affected by minor seismic activity.

Table 2-1: Earthquakes measuring magnitude 5.0 and larger in South Africa (1960-2005) (Saunders, 2005).

No	Date	Location	Magnitude on Richter scale
1	1963/08/27	Worcester Area	5.0
2	1964/06/09	Luckhoff Area	5.0
3	1966/06/18	Mokhotlong (Lesotho)	5.0
4	1969/09/11	Heidelberg Area (Cape Province)	5.2
5	1969/09/29	Tulbagh Area	6.3
6	1969/10/05	Tulbagh Area	5.1
7	1969/11/05	Tulbagh Area	5.4
8	1969/11/10	Tulbagh Area	5.1
9	1970/04/14	Cape Province	5.7
10	1976/07/01	Free State Province	5.9
11	1976/12/08	Free State Gold Mines	5.1
12	1977/03/02	Cape Province	5.3
13	1977/04/07	Klerksdorp Gold Mines	5.2
14	1977/06/07	Cape Province	5.4
15	1984/01/28	Klerksdorp Gold Mines	5.0
16	1985/05/08	Free State Province	5.2
17	1986/10/05	Lesotho	5.1
18	1989/09/29	Lesotho	5.0
19	1993/03/11	Bushmanland Area	5.2
20	1994/12/31	Bushmanland Area	5.1
21	1996/09/15	Bushmanland Area	5.7
22	1999/04/22	Free State Gold Mines	5.1
24	2001/07/31	Klerksdorp Gold Mines	5.0
25	2005/03/09	Klerksdorp Gold Mines	5.3

Chapter 2.1.3.2 to chapter 2.1.3.5 presents four case studies which show the hazardous effects an earthquake can have on unreinforced masonry buildings. Most of the buildings in these cases with serious damage are low-cost housing or other domestic masonry buildings.

2.1.3.2 Case 1 – Klerksdorp, South Africa 2014

In August 2014 the town of Klerksdorp in the North West Province of South Africa experienced a 5.5 magnitude earthquake. (Khoiratty, 2016) conducted a study to assess the effects of the earthquake on low-cost housing in the region. The effects of the earthquake were felt as far as Durban, Cape Town and even Mozambique. Towns close to the epicenter of the Earthquake; Orkney, Stilfontein and Khuma experienced serious damage. The township of Khuma was the worst effected with over 600 houses being damaged, a large majority of which were low-cost houses. According to the North West Government, one hundred million rands was used to repair infrastructure damage caused by the earthquake, in the town of Orkney alone (Khoiratty, 2016).

(Khoiratty, 2016) notes common damage patterns observed in the houses. Most of the RDP homes visited showed one or more of the following types of damage:

- Cracks along the intersections of adjoining walls.
- Cracks above windows and doors.
- Cracks on the wall at the intersection of the beam and the wall.
- Hairline cracks on the wall.
- Severe diagonal cracks along the wall.
- Horizontal crack under the roof.
- Falling of chunks of plaster.
- Damage to tiled roof.

Figure 2-1 illustrates common cracking and failure in low-cost houses which include diagonal cracks along the wall and failure above the door opening. (Khoiratty, 2016) mentions possible reasons for the damages found, these are listed below.

- Vertical cracks indicate lack of lateral binding.
- Cracks in the wall indicate poor workmanship and quality of materials.
- Cracking and large openings around doors and windows indicate poor contact between window and door frame and wall and lack of reinforcing lintels.

- Cracks at corners of door and window openings indicates high concentration of stresses developing at corners.



Figure 2-1: Common cracking and failure in low-cost houses (Khoyratty, 2016).

2.1.3.3 Case 2 - Kashmir, Pakistan 2005

A 7.6 magnitude earthquake struck the city of Kashmir killing over 80,000 people and injuring over 200,000 people. A significant portion of the casualties resided in single story unreinforced stone and mortar dwellings in the rural regions of northern Pakistan. The houses were found to be poorly constructed with weak, thin or excessively thick layers of mortar holding together the stones or bricks. The weak mortar was crushed during the earthquake resulting in the collapse of the houses. Out of plane failure was another contributing factor to the collapsing of the houses. (Meyer, 2006). Other reasons for the poor structural performance of the houses include:

- Wythes that were not interconnected.
- Large openings for windows and doors with uneven distribution, which increases the chances of crack initiation and propagation as well as weaknesses due to stiffness eccentricities.
- No structural bands tying up individual walls which result in the walls acting independent of each other, weakening its structural capacity.

2.1.3.4 Case 3 - Bam, Iran 2003

An earthquake which measured 6.6 on the Richter scale hit the Iranian city of Bam in 2003, killing approximately 30,000 people and leaving almost the entire city homeless. About 50,000 unreinforced masonry houses were destroyed by the earthquake (Meyer, 2006). It was found that most failure occurred due to out-of-plane loading and crumbling of walls as

well as the collapsing of heavy roofs. When performing a seismic analysis of masonry buildings, the vertical component of acceleration is often ignored. However, damages to the buildings in the region indicate high vertical forces up to 1G. As such, it is suggested to consider the effects of the combination of both horizontal and vertical forces (Meyer, 2006).

2.1.3.5 Case 4 - Bhuj, India 2001

A 7.6 magnitude earthquake occurred in the city of Bhuj in western India in 2001. The earthquake damaged over a million buildings and destroyed over 350,000 houses. A large majority of the buildings were unreinforced masonry houses. Both reinforced and unreinforced masonry buildings in the region performed very poorly under seismic loading indicating poor design and built quality. It is interesting to note that traditional hut-like dwellings in rural areas performed quite well due to their circular shape (Meyer, 2006).

2.2 Masonry

2.2.1 Introduction

The use of stone and brick units in buildings goes back to the earliest construction efforts of man. However, the use of molded clay and concrete brick/block units was developed much later in the nineteenth century (Orton, 1992). Lime mortar was used through the ages as a binding agent for masonry construction, however, the relatively recent development of Portland cement has greatly improved masonry construction. Masonry construction has changed drastically over time, with heavy solid stones and bricks being replaced with lighter, hollow clay and concrete brick/block units. Masonry can be used as the primary structural building material for domestic buildings, low-rise long-span buildings, crosswall construction and cellular construction (Orton, 1992). For the purpose of this dissertation, domestic buildings are focused on. Masonry has been overwhelmingly popular in the construction of domestic buildings. The relatively low cost of masonry units and simplicity of construction make masonry the preferred choice.

2.2.2 Masonry units

2.2.2.1 Clay masonry units

Clay is composed of silica, alumina and metallic oxides as well as an array of other ingredients in small amounts. For clay to be molded into a desired shape it must have plasticity when mixed with water. Clay should also have enough tensile strength to maintain

its shape after drying. Due to its clayey composition, clay masonry units have unique physical properties. Understanding the unique properties of clay brick units are important, as it helps in choosing the correct type of brick for the correct type of structure and exposure. The Clay Brick Organization of South Africa specifies the different type of clay masonry units available; this includes their geometry, composition and material properties. According to (Clay Brick Organisation, 2015), some important factors to consider are; rate of absorption, fire resistance, durability and moisture expansion.

2.2.2.2 Concrete masonry units

Concrete masonry units can be in the form of bricks, blocks or hollow blocks. According to (Smith, et al., 1979) more than two thirds of all masonry walls being constructed use concrete bricks or blocks. Concrete brick or block units are available in a wide range of sizes, shapes and finishes. Well recognized standards have been developed to document the physical properties of each type, these can be found in SANS 1215, Concrete Manufacturers Association (South Africa), British Standards, American Institute of Concrete and the National Concrete Masonry Association. Important properties to consider are compressive strength, density, moisture content, water absorption capacity, and shrinkage behavior. The main components of concrete bricks are Portland cement, water and aggregates. Aggregates could be sand, gravel, crushed stone or cinders. In addition to the main components there are air-trapping agents, coloring agents and siliceous materials. Concrete bricks are either developed as hollow or solid. Solid units are used for structures that had very high design stresses.

2.2.3 Mortar

Mortar is an essential component of masonry and is used to join masonry units together to form an integral structure and distribute pressures evenly throughout the individual masonry units. Mortar is applied to the edge of masonry units both vertically and horizontally. Typical mortar is made up of Portland cement, some cementitious material, sand and water. These ingredients are combined to produce a workable, plastic mixture. In addition to the basic mixture, certain admixtures can also be added for special purposes. In order to function properly mortar must possess a few important qualities. These include workability, compressive strength, water retentivity, rate of hardening and mortar bond.

2.2.4 Types of masonry walls

- *Load bearing wall* - A load bearing masonry wall is essentially a wall that can carry loads resting upon it as well as its own self-weight. The loads are conducted to the foundation or ground via the load bearing wall. A load bearing wall is structural integral to a building.
- *Confined masonry walls* - This wall is constructed by having a non-structurally integral external masonry layer in between reinforced concrete or structural steel beams. Confined masonry walls are often referred to as cladding and can be described as non-load bearing walls.
- *Unreinforced masonry walls* - Commonly used in the construction of domestic and low-cost housing. Unreinforced masonry walls consist of just masonry units held together with mortar and have low flexural resistance and high sensitivity to cracking.
- *Reinforced masonry walls* - Masonry is a quasi-brittle material that is quite susceptible to cracking. Reinforced masonry walls are strengthened with other materials to increase its resistance to stresses. Steel rods are used to reinforce masonry walls and increase resistance to tensile and shear stresses. This can be done by using reinforcement that is placed at specific intervals within the masonry walls, at both horizontal and vertical positioning.

2.2.5 Types of masonry bonds

Masonry bonds refer to the stacking arrangement of the masonry units. Bricks are typically laid to an offset pattern to maintain a decent lap between joints from one course of brick work to the next, and to ensure that vertical layers of mortar are not directly above one another on consecutive courses. There are several different bonding patterns, with the three most used bonds being the Stretcher bond, English bond and Flemish bond, these bonds are illustrated in Figure 2-2 (Orton, 1992). Bricks laid horizontally are referred to as stretchers and bricks laid out vertically are called soldiers while bricks laid 90 degrees to the face of the wall are referred to as headers. The orientation of stretcher, soldier and header masonry units can be seen in Figure 2-2a. The stretcher bond is the most used stacking arrangement and is made using stretchers exclusively, with the joints on each course centered above and below by half a brick, as shown in Figure 2-2b. The English bond is a type of masonry arrangement formed by laying courses of stretchers and headers alternatively, this is illustrated in Figure 2-2c. The English bond is one of the strongest

masonry bonds (Orton, 1992). The Flemish bond is often used for walls that are two-bricks thick and is constructed by placing headers and stretchers alternatively across each course as seen in Figure 2-2d. The headers of each course are centered on the stretchers of the course below.

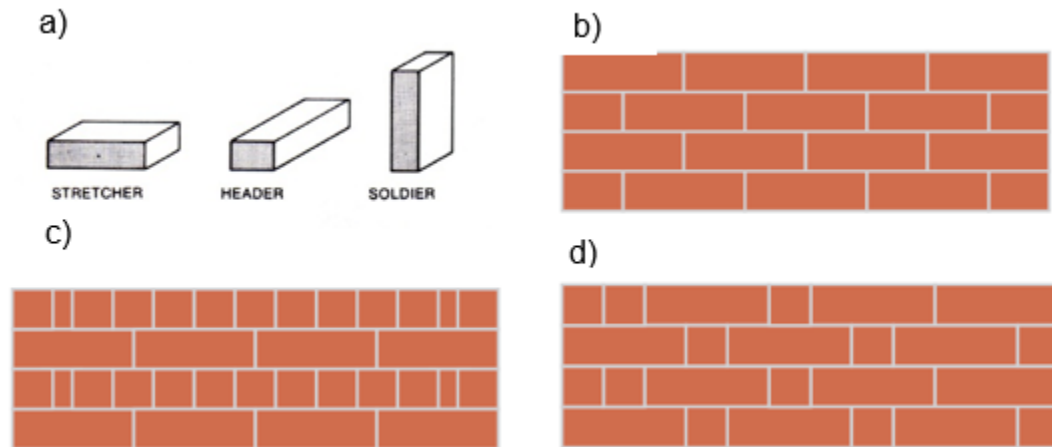


Figure 2-2: a) Brick orientation options. b) Stretcher bond. c) English bond. d) Flemish bond (Orton, 1992).

2.2.6 Material properties of masonry

From an engineering perspective, most building codes deal with masonry in compression. Masonry walls will support a greater axial load than eccentric loads. Unreinforced masonry is susceptible to tensile forces and out-of-plane forces.

2.2.6.1 Compressive strength

Compressive strength is the most fundamental material property of brittle or quasi-brittle materials like masonry. The greatest strength of masonry is its ability to withstand high compressive forces. Compressive failure of a brittle material is accomplished through softening. Softening can be described as a slow, continuous decrease of resistance under a continuous increase of applied deformation (Lourenco, 2009), or simply a deterioration of a material's strength with increasing strain. Several factors can influence the compressive strength of masonry such as; geometry and deformation characteristics of the masonry unit, water-cement ratio of the mortar, height-thickness ratio of the wall, bond between brick and mortar, thickness of mortar, direction of stressing and quality of workmanship.

It should be noted that the strength of masonry in compression is smaller than the compressive strength of the brick unit. However, the compressive strength of masonry is

much more than the compressive strength of cube crushing strength of the mortar used in it (Hendry, et al., 1997). Increasing the thickness of the mortar joint will decrease the strength of the masonry as shown in Figure 2-3b.

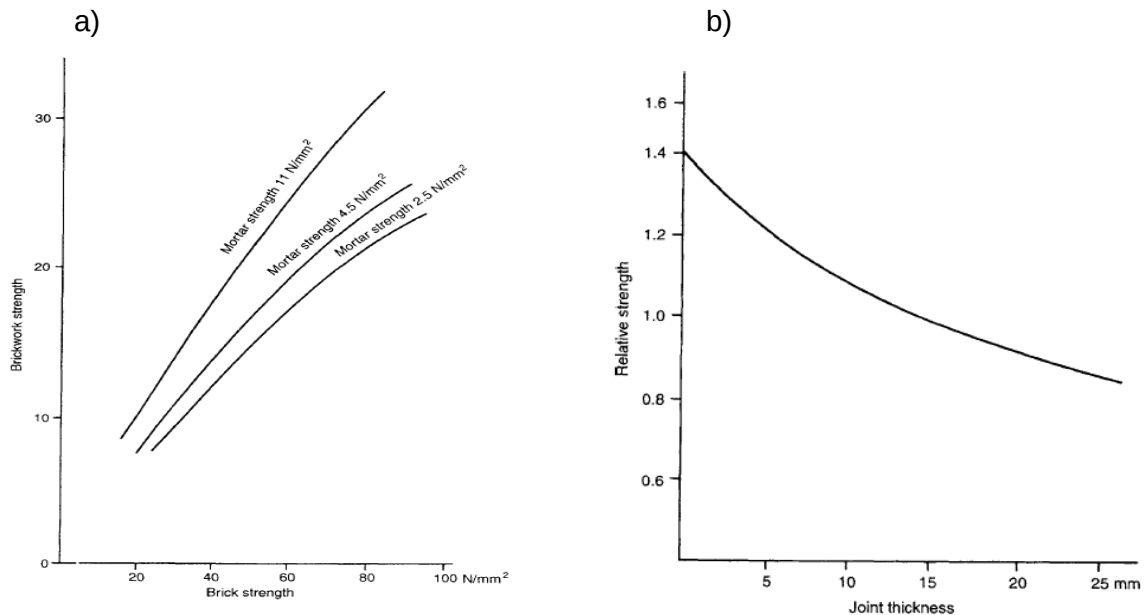


Figure 2-3: Effect of a) mortar strength and b) mortar thickness on strength of masonry (Hendry, et al., 1997).

2.2.6.2 Combined compression and shear

Combined compression and shear strength is an important factor to consider when assessing the resistance of a structure to lateral forces. The initial shear resistance of a masonry wall is dependent on the strength of adhesion between the brick units and the mortar.

2.2.6.3 Tensile strength of masonry

Direct tensile strength - In-plane loading such as eccentric gravity load, moisture movements or foundation movements can cause direct tensile stresses to develop in masonry. The tensile resistance of masonry is rather low, especially at the bed joints. Much of the tensile resistance of unreinforced masonry depends on the bond between the brick units and the mortar (Berndt, 1996). The relationship of the brick-mortar bond is complicated and not fully understood. It is known to be a physical-chemical interaction with the critical factor being the pore structure of both the brick and the mortar. Important material properties to consider are the fineness of the sand, as finer sand leads to better adhesion, and the moisture content of the brick.

Flexural tensile strength – this is caused mainly from wind loading and suction. A small amount of stability is derived from the self-weight of the wall, however, this is very minimal. The same factors that influence direct tensile strength also influence flexural tensile strength. If the brick-mortar adhesion is strong the flexural tensile strength will be limited by the tensile strength of the brick units. However, if the brick-mortar adhesion is weak the flexural tensile strength will be limited by the shear strength of the brick-mortar connection (Berndt, 1996).

2.2.6.4 The effects of workmanship on masonry

Unlike reinforced concrete and steel construction, masonry has a long history of craftsmanship without engineering supervision. As such, the validity of masonry as a structural material is often questioned. Masonry carries much higher safety factors than concrete for example. However, if the same level of engineering supervision is applied to masonry as concrete the structural integrity of masonry will be greatly improved. It should be noted that workmanship factors are important in developing the specified strength of masonry. Some common construction defects in masonry are listed below (Hendry, et al., 1997).

- Failure to fill bed joints. Gaps in the mortar result in a significant loss of strength in the masonry specimen. According to (Hendry, et al., 1997) masonry walls with incompletely filled bed joints can result in reduction of strength of up to 33%.
- Excessively thick bed joints will result in reduced masonry strength bed joints larger than 16mm are regarded as too thick and can result in a decrease of compressive strength of up to 30% when compared to 10 mm thick bed joints.
- Out of alignment geometry.
- Uneven geometry results in eccentric loading and consequent decrease in strength.
- Exposure to adverse weather
- The masonry specimen should be protected from extreme weather conditions, both heat and cold as well as rain, before the mortar has set. Exposure to extreme heat could result in increased evaporation, changing the composition of the mortar and resulting in reduced strength. Freezing can lead to displacement in both horizontal and vertical direction of the masonry wall.
- Incorrect mortar mixture.

2.2.6.5 Fire resistance

Fire is a major destructive force and can cause serious and irreparable damage to buildings which can lead to loss of human life and property. Masonry walls have the ability to resist the spread of fire and do not buckle under excessive heat (Orton, 1992). Masonry materials do not contribute to the combustibility of a building.

2.2.7 Failure criteria of masonry

2.2.7.1 Mohr-Coulomb Failure Criteria

Computer software packages such as ANSYS have failure criteria within them to test different parameters. Different materials have different failure criteria. The Mohr-Coulomb failure criteria describes a material's response to shear and normal stress and is commonly used for materials that have a greater compressive strength than tensile strength. The Mohr-Coulomb failure criterion defines a linear relationship between shear stresses and normal stress (or minimum and maximum principal stresses) at failure (Labuz & Zang, 2012). Most engineering materials follow this rule in some way. The Mohr-Coulomb failure criteria is used to determine the failure loads and angle of fracture particularly in granular materials. Mohr's circle (shown in Figure 2-4) helps determine which principal stresses will produce the combination of shear and normal stress. According to Coulomb's criteria, displacement at failure will form an angle equal to the angle of friction. Thus, by comparing external mechanical work and external load with the internal mechanical work introduced by the strain and stress at the line of failure, the material strength can be calculated (Labuz & Zang, 2012). Using the conservation of energy principle, the sum of these equals zero which enables the failure load to be calculated. The Mohr-Coulomb failure criteria is described numerically in Equation 2-1.

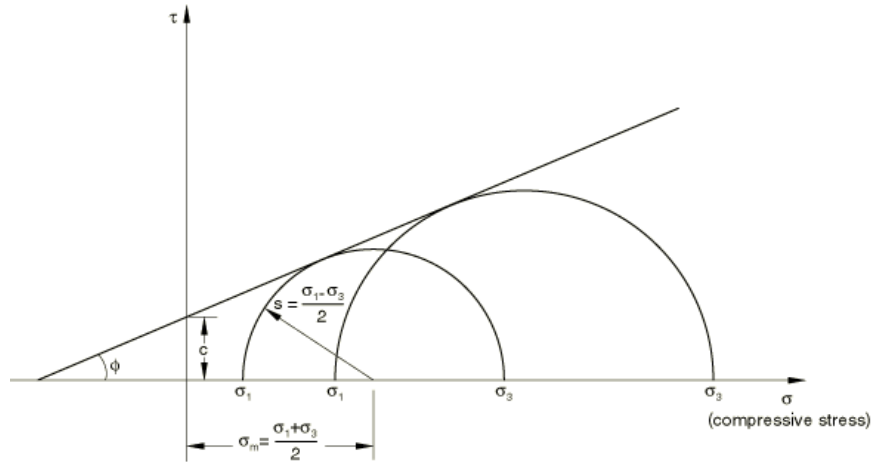


Figure 2-4: Mohr's circle (Labuz & Zang, 2012).

$$\tau = \sigma \cdot \tan(\varphi) + c \quad (2-1)$$

Where:

τ	=	shear strength
σ	=	normal stress
$\tan(\varphi)$	=	slope of failure
φ	=	angle of internal friction

Major advantages of the Mohr-Coulomb failure criteria are its mathematical simplicity and its general level of acceptance in the field of material science for geotechnical materials. Some limitations of the Mohr-Coulomb failure criteria include the criteria implies that a major shear stress occurs at peak strength and accuracy of results are questionable if the main failure criterion is not shear (Viswanathan, et al., 2014).

2.2.7.2 The Drucker Prager failure criteria

The Drucker-Prager model is used to define the failure criteria of materials whose behavior is dependent upon its equivalent compressive stress. Drucker-Prager is usually used to model frictional materials that have a greater compressive strength than tensile strength. Masonry, and both individual components brick and mortar, fall under this category. In the Drucker-Prager model, materials can be modelled to have isotropic hardening or softening, and the yield criteria is determined using the shape of the yield surface. The yield surface could be a linear, exponential or hyperbolic function. The Drucker-Prager failure criteria determines whether a material has failed or undergone plastic yielding and is used to estimate the stress state at which a rock-like material reaches its ultimate stress (Campbell & Durán, 2017). When plotted, the yield surface is cone shaped as seen in Figure 2-5. The

Drucker-Prager failure criteria can only be applied to brittle and not ductile materials and has many variants which have been applied to rock, soil, concrete, and other pressure-dependent materials (ANSYS, 2014).

$$F = 3\beta\sigma_m + \frac{\sigma_{eqv}}{\sqrt{3}} - \sigma_y \quad (2-2)$$

Where:

β	=	material constant
σ_m	=	hydrostatic pressure
σ_{eqv}	=	Von-Mises stress
σ_y	=	material yield parameter

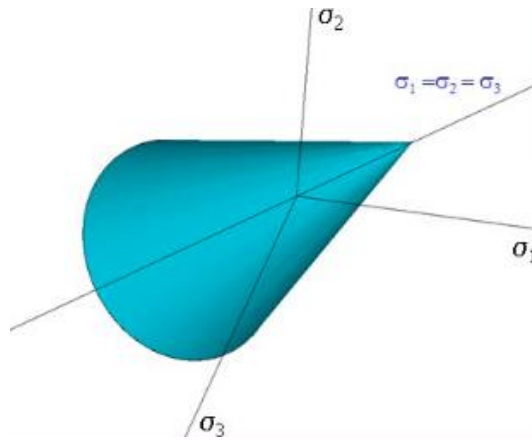


Figure 2-5: Conical yield surface of the Drucker-Prager model (ANSYS, 2014).

The Drucker-Prager failure criteria is a good option to use in finite element models as it requires the input of just three variables.

- f_c – uniaxial compressive strength
- f_t – uniaxial tensile strength
- f_{cc} – biaxial compressive strength

(Wojciechowski, 2018) conducts a study comparing the differences between the Drucker-Prager and Mohr-Coulomb failure criteria. The study mentions that the Drucker-Prager failure criteria is more suitable for cementitious materials such as masonry units and mortar while the Mohr-Coulomb method is better suited to soils. The Drucker-Prager failure criteria is also advantageous in that it requires minimal input variables.

2.2.8 Failure mode and behavior modes of masonry

2.2.8.1 General failure modes

There are three general failure modes common in all fracture mechanics problems. These are described below and illustrated in Figure 2-6.

- Mode I (traction mode) – Load is applied perpendicular/normal to the direction of the crack plane.
- Mode II (shear mode) – Refers to the in-plane shear loading.
- Mode III (torsion mode) – Corresponds to out of plane shear.

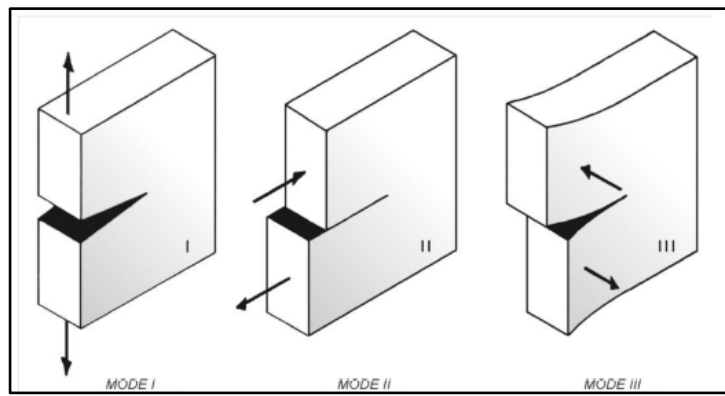


Figure 2-6: General failure modes in fracture mechanics (Oliveira, 2013).

2.2.8.2 Failure modes of masonry

In masonry walls, failure is usually achieved as a result of compressive or lateral loads or a combination of the two. Failure usually shows in the form of cracks in the mortar or cracks through the masonry units and the mortar. Failure resulting from axial loads tend to show up in the form of vertical splitting. The reason for this type of splitting is mainly due to the action of the mortar joint (Smith, et al., 1979). According to (Page, 1981), masonry exhibits distinct directional properties because the mortar joints act as planes of weakness. Typically, the mortar is less rigid compared to the masonry unit. Cracking in the mortar tends to spread laterally.

In (Huster, 2000), the cracking behavior of URM is studied. The mechanical and geometric properties of the masonry assemblage are investigated closely. A FEM model is used, incorporating the relevant mechanical and geometric properties of masonry. The brick is defined as a linear elastic material. The study finds that primary failure occurs as a result of one of three failure modes.

- i) Failure that occurred in the mortar.
- ii) Failure caused by high tensile stresses developing in the brick, near the brick-mortar interface.
- iii) A combination of i and ii.

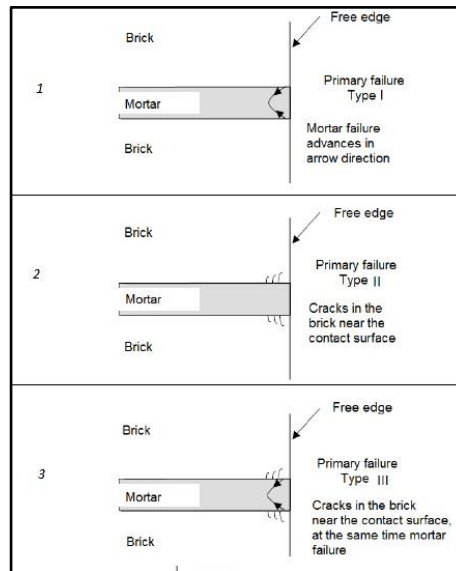


Figure 2-7: Three types of primary failure in masonry according to (Huster, 2000).

(Jamal, 2017) presents a study that describes the failure modes of masonry. Masonry is described as being relatively strong in compression but very weak in resisting bending and shear forces, which is often the reason for collapse. The different failure modes described are:

- i) Sliding shear failure, which is a horizontal splitting the mortar joints. This type of failure is caused by high compressive loads and poor mortar.
- ii) Diagonal cracking, which is a result of tensile stresses developing in the wall which exceed the tensile strength of the material. This is caused from a combination of vertical and horizontal loading.
- iii) Failure due to overturning, which is directly related to the geometry and the proportions of the masonry wall. Walls with a high height-to-thickness ratio are vulnerable to overturning.

(Lourenco & Rots, 1997) discusses five failure modes that are found in masonry, these failure modes are illustrated in Figure 2-8 and are described as:

- a) Joint tension cracking.

- b) Joint slip failure.
- c) Unit direction tension crack.
- d) Unit diagonal tension crack.
- e) Masonry crushing.

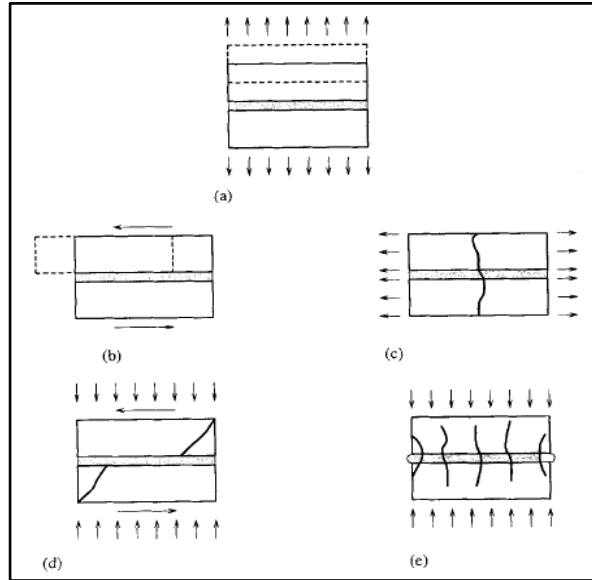


Figure 2-8: Five failure modes of masonry according to (Lourenco & Rots, 1997).

2.2.8.3 Uniaxial tensile behavior of masonry

For masonry assemblages, uniaxial tensile loading considers tensile loading perpendicular to the bed joints (Lourenco & Rots, 1997). This type of loading exploits the weak bonding point between the brick and mortar, resulting in failure and cracking in these regions. According to (Mohamad, et al., 2015), in masonry walls, tensile strength is approximately one tenth than that of compressive strength. (Lourenco & Rots, 1997) mentions that if low-strength masonry units are used in combination with high-strength mortar, failure could occur sooner due to stresses developing within the structure that exceed the tensile strength of the masonry unit. This indicates that in most masonry structures the tensile strength of the structure is directly proportional to the tensile strength of the masonry brick unit.

2.2.8.4 Uniaxial compressive behavior of masonry

Uniaxial compressive behavior in masonry considers compressive loading normal to the bed joints. (Zucchini & Lourenco, 2009) mentions that vertical cracks are a common sign of failure due to uniaxial compression, this is further illustrated in Figure 2-9. The compressive strength of the masonry structure is dependent on the compressive strength of the masonry

units and mortar as well as the bond strength between the two material. Since most masonry units have a high compressive strength, failure due to uniaxial compression usually occurs as a result of failure in the mortar. The strength of the bond is dependent on the type of mortar mix used and the thickness of the mortar, otherwise referred to as joint thickness. (Bilir & Çağatay, 2014) performed a study to determine a relationship between the overall uniaxial compressive resistance of a masonry panel and the mortar joint thickness. The study concluded that increasing the mortar joint thickness decreases the overall uniaxial compressive strength of the masonry panel. This indicates that a masonry structures' resistance to uniaxial compressive loading is highly affected by the mortar, in particular the mortar thickness, the quality of the brick-mortar joint and the quality of the mortar mix.

2.2.8.5 Biaxial compressive behavior of masonry

Biaxial compressive behavior in masonry arises when loading occurs along two different axes. Most masonry walls that undergo in-plane loads are in a state of biaxial stress (Page, 1981). (Lourenco & Rots, 1997) suggests that biaxial compressive behavior in masonry is usually induced from uniaxial stresses. (Page, 1981) uses a 3-D surface with two principal stresses acting upon it to define failure under biaxial stress. In the study, fracture came about due to splitting in a plane parallel to the free surface of the masonry panel at mid-thickness as shown in Figure 2-9. (Page, 1981) notes that the failure comes about abruptly, in a plastic manner. It should be noted that failure in the plane normal to the masonry face is an indication that one of the principal stresses is dominant. The failure as shown in (Page, 1981) initiated at the edge of the masonry specimen, this was unexpected as failure should have started at randomly distributed regions of the structure.

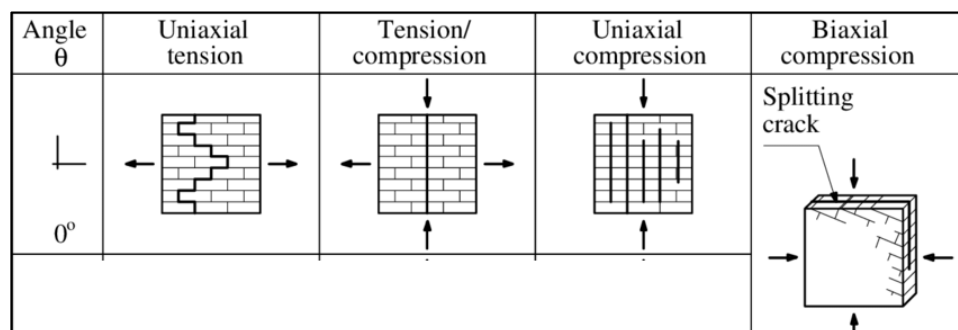


Figure 2-9: Modes of failure of masonry (Lourenco & Rots, 1997).

2.3 Numerical modeling of masonry

2.3.1 Finite element analysis

The finite element method is numerical method widely used across the field of engineering. The method is commonly applied to practical applications such as structural analysis, fluid flow and heat transfer. The FEM is well suited for cases that involve complex geometries, uneven load distribution and composite materials. For such cases, attaining analytical mathematical solutions, using ordinary partial differential equations, becomes extremely complicated and cumbersome. Therefore, computational numerical methods such as the FEM are required (Logan, 2007). The FEM incorporates a system of simultaneous algebraic equations which produce approximate values of the unknowns in an iterative process. To achieve this, the body under study needs to be divided into several small pieces which form an equivalent system of finite elements, this process is called discretization. Each finite element is interconnected to the finite elements around it, thus forming an interconnected system. It works by dividing the continuum into a finite number of elements connected by nodes present on the boundaries. Equations are formulated for each finite element and summed up to obtain the solutions for the entire body under study (Viswanathan, et al., 2014). The advantages of the finite element method are listed below.

- The finite element method allows geometries of irregular shapes to be easily modelled.
- Different loading conditions can be incorporated into the finite element method.
- The finite element method allows for modelling of composite material and bodies.
- The finite element method enables unlimited boundary conditions.
- Different finite elements can have different element sizes.
- Dynamic analysis can be performed using the finite element method.

2.3.2 Numerical modeling techniques used in masonry

Extra consideration is required with the modeling of masonry walls due to the different characteristics of the two components. Over the years, a large collection of research work involving masonry and the finite element method has been compiled, and as such, various modelling techniques have been developed. These various techniques, however, can be split into two broad categories:

- Micro or discrete modelling (heterogenous)
- Macro or continuous modelling (homogenous)

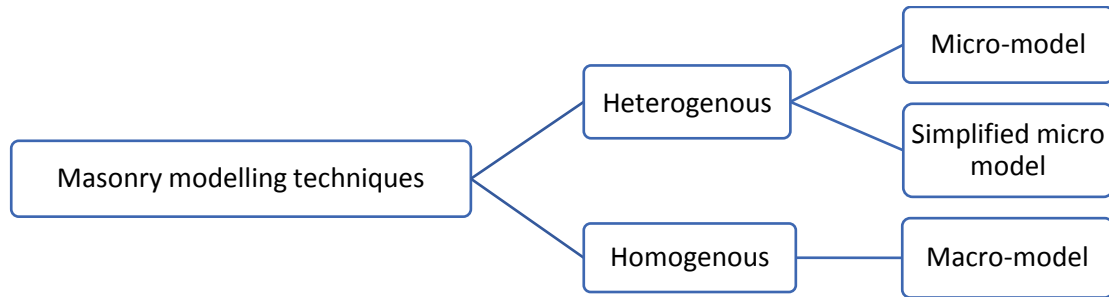


Figure 2-10: Numerical masonry modeling options.

In the micro-modelling approach, the two separate components of masonry; the brick units and the mortar are considered as two separate entities joined by a continuum brick-mortar interface layer. The micro-modelling approach regards the masonry as heterogeneous. Alternatively, the macro-modelling method models the brick units, the mortar and the interface between the brick and mortar as a single continuum element by using homogenization techniques (Viswanathan, et al., 2014). Micro-modelling is not the most common approach taken to model masonry as it poses a number of challenges, however many gaps do exist in the study of masonry structures which could be filled using micro-modelling. Most modern research papers relating to masonry modelling adopt homogenization techniques which eliminate the ability to study the contact between the brick and mortar. The homogenized macro-modelling method is efficient as it requires lesser computational capacity compared to the micro-modelling method and is better suited to large scale structural analysis. Micro-modelling can present more detailed models and localized solutions; however, this method requires much more time and computational effort.

A masonry micromodel consists of brick units and joints where positioning and dimensions of each unit corresponds to actual masonry unit parameters. Layers of mortar are modelled between the bricks. A micromodel is useful for generating localized, detailed results. When using the micro-modelling approach, Young's modulus of elasticity, Poisson's ratio, shear modulus and density for both materials need to be defined. The micromodel will illustrate the disposition of bricks however, this can be computationally taxing. This approach is suitable for academic use. When using the macro modelling technique, the masonry units

and mortar are modelled as a single, homogenous material. This process incorporates sophisticated homogenization techniques. Macro modelling techniques are well suited for large scale structures with repeated structural components.

(Lourenco, 2015) performed a study comparing micro and macro-modeling in masonry models. The study mentions that masonry is a complicated material to model numerically and requires advanced modelling techniques when compared to other materials such as steel. The author recommends implanting the micro-modeling approach for small geometries with few elements while homogenization techniques should be adopted when dealing with large and complicated geometries.

The different numerical masonry modelling techniques are illustrated in Figure 2-11 - Figure 2-13.

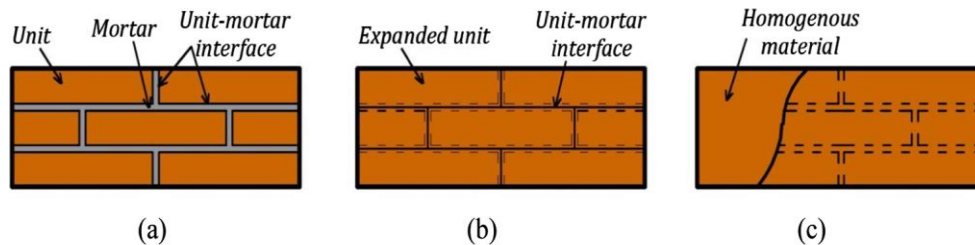


Figure 2-11: Detailed FEM micro-model; (b) simplified FEM micro-model; (c) FEM macro-model (homogenized) (Ahmad, et al., 2014).

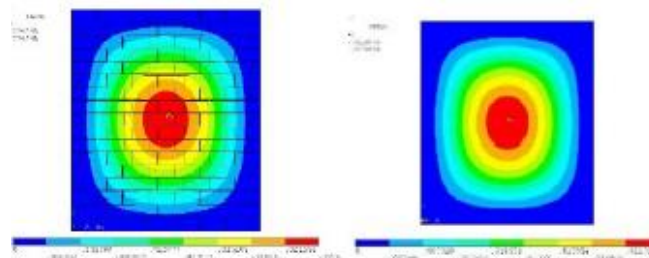


Figure 2-12: Micro(left) and macro(right) modelling illustrated using ANSYS software (Mynarz & Mynarzova, 2018)

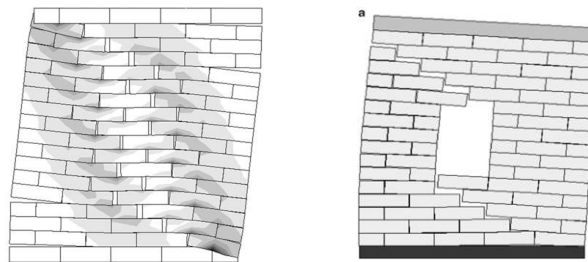


Figure 2-13: Examples of deformation at failure using structural micro-modeling, deformation of individual masonry units can be studied (Lourenco, 2015).

One of the major advantages of the micromodel approach is the ability to isolate and monitor the behavior of only one material. This can be seen in Figure 2-14 which shows the deformation and crack arrangement of the mortar only at several node points.

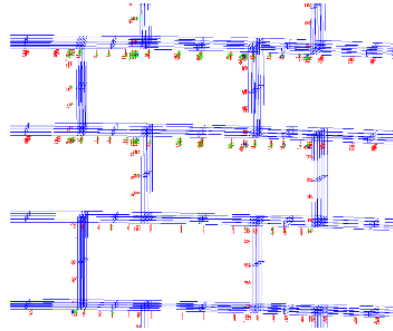


Figure 2-14: Isolation of mortar using micromodel (Mynarz & Mynarzova, 2018).

2.3.3 Linear vs Non-linear analysis

Linear analysis is considered when a material obeys Hooke's law and the response results in small displacements. When, either a non-linear material law (such as for instance the Drucker-Prager law used to depict failure of masonry) or large displacements are considered, the analysis is considered non-linear. Linear analysis requires less parameters and lower computational efficiency. On the other hand, the real behavior of most materials and structures is non-linear and non-linear analysis is needed to capture true failure. In a finite element linear analysis, the program assumes the body will undergo small deformations (Foley, 2018). If there is enough deformation to cause a change in the stiffness, then this is a non-linear situation. In a linear analysis the stiffness matrix $[k]$ is kept constant as shown in Equation 2-3. $[k]$ is dependent on the material, geometry and contact. If $[k]$ is varying, then the analysis cannot be linear.

$$[F] = [k] * [u] \quad (2-3)$$

Where:

$[F]$	=	load matrix
$[k]$	=	stiffness matrix
$[u]$	=	displacement matrix

In many real-world situations however, linear analysis is not valid and therefore, non-linear analysis may be required. With the advent of new software and hardware, non-linear analysis has become an integral part of structural analysis. Non-linear analysis overcomes

the shortcomings on linear analysis; however, it is difficult to model this complex behavior and it requires more computational time and capacity. In order to consider plastic deformation, non-linear analysis with non-linear materials need to be used. If a non-linear analysis is implemented with a body that behaves linearly, the same outcome will be attained but the analysis will take longer. Non-linear analysis should be considered under three general circumstances (Foley, 2018).

- i) *Geometric non-linearity* – If a model experiences large deformation, the change in the geometric configuration can lead to non-linear behavior. When large deformations exist, the stiffness matrix of the system needs to be updated after each iteration.
- ii) *Material non-linearity* – If the material being used has a non-linear stress vs strain graph, hence varying Young's modulus of elasticity and stiffness, a non-linear analysis is required.
- iii) *Contact non-linearity* – When the effects of contact are taken into consideration, it can lead to an abrupt change in stiffness when bodies come into contact with each other. In this situation a non-linear analysis is required.

In this case, two types of non-linearities arise: geometric and material non-linearity.

$$F = [k(u)] * u \quad (2-4)$$

Where:

F = Load matrix
 k = Stiffness matrix – dependent on u
 u = Displacement matrix

As seen in Equation 2-4, for a non-linear static analysis the stiffness matrix $[k]$ is variable and dependent on displacement (u). This will lead to a non-linear stress vs strain graph and becomes an iterative solution as the relationship between the load (F) and response (u) is not known at the start. The differences between linear and non-linear analysis are summarized in Table 2-2.

$$\frac{k}{\Delta u} = F_{ext} - F_{int} \quad (2-5)$$

Where:

k = tangent stiffness matrix
 Δu = incremental displacement vector
 F_{ext} = vector of external forces
 F_{int} = vector of internal forces (integral of stress on each finite element)

The Newton-Raphson method is used to make linear approximations in an iterative manner and is described in Equation 2-5. Figure 2-15 illustrates the iterative Newton-Raphson method and it can be seen that in iteration 1, the total load (F_a) is applied which results in corresponding displacement of x_1 . From this, the internal load (F_1) can be calculated. If $(F_1) \neq (F_a)$ then no equilibrium is reached and another stiffness matrix is determined, which initiates a new iteration. This iterative process is repeated until $(F_i) = (F_a)$. In Figure 2-15 it takes four iterations for equilibrium to be reached.

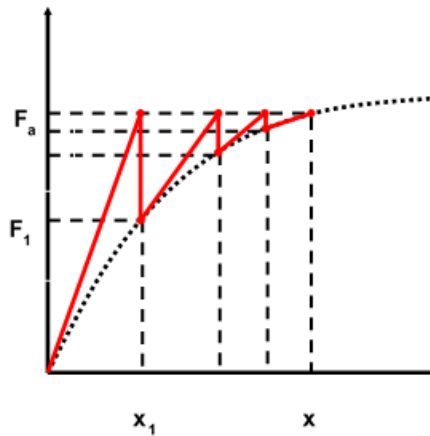


Figure 2-15: Stress vs strain plot using iterative Newton-Raphson method (ANSYS, 2014).

Therefore, a non-linear solution typically requires one or more load steps to apply the total external load to the boundary conditions and multiple sub steps thereafter. Each load step and sub step are represented by a value of time. This time value is not the actual time of the simulation but rather used as a counter. Δt is the time step and represents the time increment between each sub step. ΔF is the load increment applied over each sub step. ΔF is dependent on Δt .

Table 2-2: Difference between linear and non-linear analysis

	<i>Linear Analysis</i>	<i>Non-linear Analysis</i>
1	Displacement is proportional to applied load	Displacement can vary non-linearly with applied load
2	Stiffness matrix $[k]$ is constant	The stiffness matrix $[k]$ changes as the load changes
3	Assumes small changes in geometry	Displacement can be large
4	Original state is always used as reference state	Change in geometry cannot be ignored
5	Considers elastic failure only	Consider elastic and plastic failure
6	Straight line stress vs strain graph	Curved stress vs strain graph

2.3.4 Stresses, strains and deformations

2.3.4.1 Stress vs strain

Stress is the internal reaction or resistance force to external load and is defined as force per unit area. Strain is a measure of the degree of deformation and is defined as change in length over initial length (Hamid, et al., 2013). In a stress vs strain graph the gradient of the slope represents Young's modulus of elasticity (E). Young's modulus of elasticity (E) is a material property that indicates stiffness and rigidity and is defined as stress over strain. For the analysis of a linear material the stress vs strain graph is linear, hence the modulus of elasticity remains constant. For a non-linear material, the stress vs strain graph has curvature as illustrated in Figure 2-16. A linear stress vs strain graph with a steep gradient yields a high (E) and a graph with a gentle gradient yields a low (E). Generally, a larger (E) value means a high strength of material while a material with low (E) can exhibit large deformations at small loads. The difference between a linear and non-linear stress vs strain graph can be seen in Figure 2-16

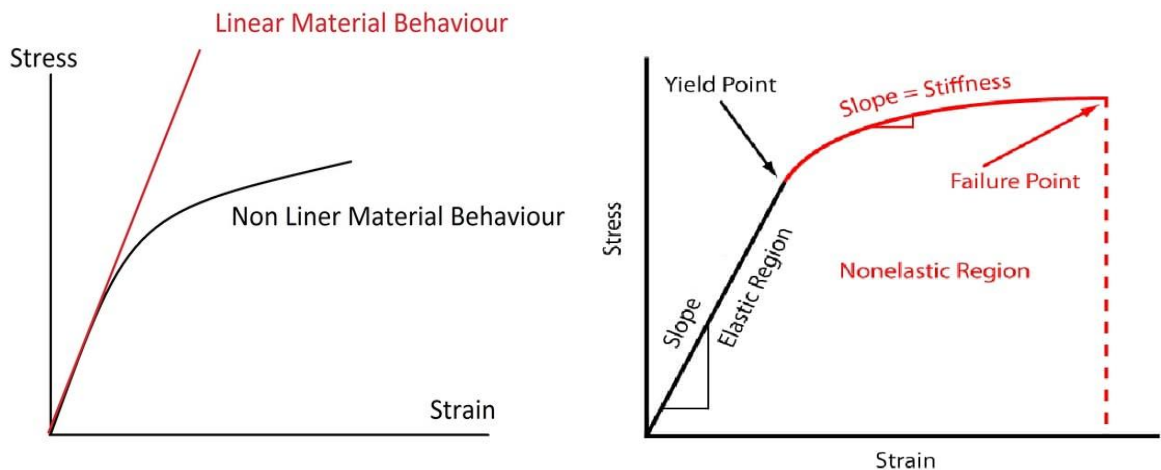


Figure 2-16: Stress vs strain graph of a linear and a non-linear material (ANSYS, 2014).

2.3.4.2 Ductile and brittle materials

Once yield stress is reached; the body will continue to deform or change its length (strain) without any increase in load. Ductile materials are materials that can resist large changes in length (strain) before reaching failure. Brittle materials exhibit very low tendencies to elongate and show little or no yielding before failure. Brittle materials break suddenly under stress after the elastic limit is exceeded and have a greater compressive strength compared

to tensile strength (Mynarz & Mynarzova, 2018). Masonry units and mortar are examples of brittle materials.

2.3.4.3 Elastic and Plastic deformation

Applying a force to a body results in a change of shape and size of the body. A force can be applied in several ways including compression, tension and torsion. A material is said to experience an elastic deformation if it returns to its original shape after a load is applied and removed. However, if the applied load is large enough, the body gets distorted beyond its elastic limit. As such, the body cannot return to its original shape and size. A material is said to experience a plastic deformation if it does not return to its original shape after a load is applied and removed (Andreaus, 1996). Plastic strain refers to the irreversible damage or dimensional changes caused by forces and displacements. The kind of damage occurs after internal stresses exceed the known strength of the material. This occurs after yield stress is reached. The fundamental issue with regard to plastic behavior lies in the formulation of the mathematical stress-strain relationship (Pöschel & Sabha, 1996). The mathematical formulation of plastic strain is done using plasticity models such as the Drucker-Prager model or Mohr-Column model.

2.4 Review of numerical and experimental structural masonry studies

Chapter 2.4 presents a review of numerous journal articles, theses and reports related to the analysis of masonry which includes both numerical and experimental studies. Chapter 2.4.1 looks at numerical and experimental studies dealing with the response of masonry to static loading while Chapter 2.4.2 details studies that investigate the dynamic behavior of masonry.

2.4.1 Review of studies on the static analysis of unreinforced masonry

(Drosopoulos & Stavroulakis, 2018) conduct a computational homogenization study focusing on the local failure of masonry walls. The study mentions that analytical methods produce more accurate results, however, such methods are complicated to apply to complex structures. The extended finite element method (XFEM) is applied to develop masonry models to simulate cohesive cracking in the macroscopic scale. Traction-separation laws are included in the model with contact conditions used to simulate the brick-mortar interface. A non-penetrable interface is created between the brick-mortar interface using averaging techniques. The model presented in (Drosopoulos & Stavroulakis, 2018)

allows for complicated failure patterns to be predicted and can be used for studies with complex macroscopic crack patterns as the results produced from this study are verified using existing experimental studies.

(Hilsdorf, 1965) studies the effects of varying modulus of elasticity values in both the brick and mortar. According to the study, high tensile stresses in the brick is the main contributor to failure. It is mentioned that the strength of the overall masonry wall will only be a fraction of the strength of the masonry unit if the mortar mixture is of a poor quality. The paper also mentions the significant increase in strength of the masonry wall samples with increased quality of mortar and workmanship. Alternatively, (Mann, 1983) states that it is usually a failure in the mortar that causes the overall failure of the masonry sample as the masonry brick units have a higher tensile strength.

(Stiglat, 1984) conducts an experimental study to investigate the failure load and deformation behavior of clay masonry brick units. It is noticed that masonry brick units with higher densities have increased compressive strength.

In (Berndt, 1996) a numerical model, using finite element analysis, for the failure of masonry walls is presented. The results show that the height of the masonry unit and the thickness of the mortar are crucial factors that influence the tensile strength of the masonry models. According to the study, the highest tensile stresses form at the corner of the brick units close to the brick-mortar interface.

(Pöschel & Sabha, 1996) carries out a numerical study to simulate the behavior of sandstone bricks and low strength lime mortar. The numerical model is developed using the FEM. The brick is modelled as a linear elastic material while it is assumed that the mortar exhibits elastic plastic behavior. The failure behavior of the models is described in full detail by the author.

(Page, 1998) presents a study investigating the in-plane analysis of masonry walls with clay brick units. The FEM is used to replicate the non-linear characteristics of masonry. The brick units are modeled as elastic continuum elements and the mortar is modeled as linkage elements. The brick-mortar interface is assumed to have high compressive strength, low tensile strength and low shear strength with non-linear deformation characteristics. The material properties used in the numerical model are obtained from uniaxial tests.

(Baraldi, 2017) presents a numerical study that investigates the non-linear behavior of masonry walls under in-plane loading. Models are created using both the FEM and the DEM and comparisons are drawn between the results attained from the two methods. The study adopts the Mohr-Coulomb failure criteria for all models. The study concludes that both the FEM and DEM are simple and effective mechanisms for modelling the non-linear behavior of masonry, however, the DEM is simpler and requires less computational effort but gives less accurate results compared to the FEM.

A paper published by (Vindhyashree, 2015) compares the results of a masonry prism test performed in a lab to the results of a computer model on ANSYS. Determining the compressive strength of a masonry requires a lot of time and effort if done experimentally in a lab. However, this can be overcome with the use of accurate computer models which simulate the behavior of masonry. The work done by (Vindhyashree, 2015) aims to simulate the masonry prism on ANSYS and compare the results to the experimental masonry prism test conducted in a lab. It is found that the ANSYS software produced results closest to the lab test. The crack propagation pattern produced by the ANSYS model resembles the results of the lab test to a good extent.

In (Boult, 1979), the relationship between the height of a masonry prism and its compressive strength is tested. The study found the compressive strength of the masonry prism decreased with increasing prism height. The rate of decrease changes with different masonry units but a decrease is common through all units. The study also detects that the decrease of compressive strength as a result of increased height is very low or even negligible between the 5-12 course high masonry prisms.

(Drysdale & Hamid, 1979) perform a study to test the behavior of hollow concrete blocks, held together with mortar, under axial compression. The study concludes that 3 course masonry prisms give more realistic results as opposed to 2 course masonry prisms. The study also observes that when under axial compressive loading only, large increases in the strength of the mortar result in small increase in the strength of the prism.

(Viswanathan, et al., 2014) conducted a study using the FEM and ANSYS software to generate numerical simulations of masonry under compression and shear loading. Unreinforced brick masonry is considered. Examining the results from the study, the FEM models are found to facilitate the behavior of unreinforced brick masonry. The models can also recognize regions of failure and crushing.

(Lourenco, 2009) describes the recent achievement in the field of numerical masonry modelling. According to this paper, computational modelling is necessary for investigating the structural behavior of complex masonry structures. Micro-modelling allows for an in-depth understanding of the behavioral phenomena that occurs within a masonry structure, however, macro-modelling, average continuum mechanics and homogenization techniques are better suited for large scale models. The paper mentions the importance of inputting reliable and accurate data while developing the model.

(Campbell & Durán, 2017) create a non-linear, numerical model for the analysis of masonry structures. The results attained from the model presented shows a good correlation with existing experimental studies. The study takes into account unreinforced masonry, reinforced masonry and confined masonry. Material properties are attained from relevant design codes and other studies.

In (Ghiassi, et al., 2010) a specialized micro-modelling method is presented to model the non-linear behavior of unreinforced masonry panels. The model considers all compressive failure modes. Shear and flexural failure are acknowledged as important considerations in the study of masonry and are considered in a simpler manner. The accuracy of the results produced are within acceptable range and the method is found to have a significantly reduced runtime as compared to the finite element method.

(Wang, 2014) performs an experimental and numerical study on unreinforced masonry. The numerical model is created using three-dimensional finite element analysis. ANSYS software is used to create the FEM model, using the built in solid 65 element. The model is subjected to shear and compressive loading with the results being compared to the experimental data as shown in Figure 2-17. At lower stress the model can accurately predict displacement and failure patterns.

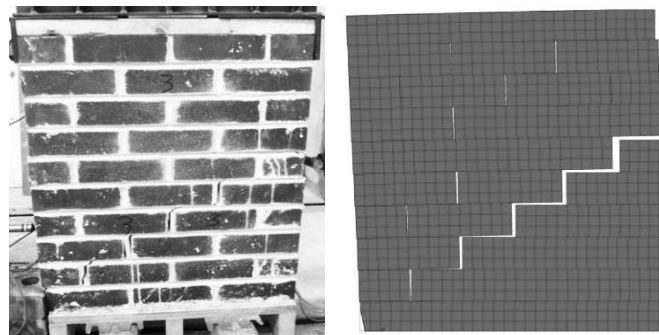


Figure 2-17: Shear failure pattern from experimental tests and corresponding numerical model (Wang, 1996).

(Jäger, et al., 2009) investigates the features and challenges of masonry simulations in both ANSYS and LS-DYNA. The paper highlights the challenges of numerically modelling masonry despite the major advances being made in the FEM. Mesoscopic modeling considers masonry as heterogeneous with separate parts for masonry units and mortar while macroscopic modeling is used in large scale structures and represent the masonry units and the mortar homogeneously. Macroscopic modeling is better suited for large scale structures, where all masonry components are smeared to a continuum. The study mentions that one of the main challenges modelling masonry using the FEM is the issues of large deformations during collapse. Large deformations often cause the finite element model or code to crash.

(Bolhassani, et al., 2015) developed a simplified micro model to gain further insight into the behavior of masonry assemblages. The authors use the results from experimental test data to incorporate yield criteria, failure criteria and stress-strain properties into the masonry model. Smearing techniques are used to model the brick units and mortar homogeneously. The traction-separation law is incorporated to model the behavior of the mortar joints.

(Lourenco, 2015) gives insight into the general approaches used to numerically model masonry structures. The two broad approaches are micro-modelling and macro-modeling with recent developments being made to combine the two using homogenization techniques. The brick-mortar interfaces act as planes of weakness, therefore, nonlinear behavior can be modelled in the joints. Another point of weakness in masonry are the center of brick units.

(Hamid, et al., 2013) studies the mechanical properties of ungrouted and grouted concrete masonry assemblages. Existing literature identifies a significant reduction in the shear strength and resistance to deformation of ungrouted concrete masonry compared to grouted concrete masonry. The study makes three broad conclusions which are:

- There is a significant difference in structural performance between ungrouted concrete and grouted concrete masonry. Structural performance parameters include failure mode, resistance strength and deformation capacity.
- The compressive strength, tensile strength and shear strength of the grouted samples are increased by 32%, 168% and 280% respectively, when compared to the ungrouted samples.
- Grouting reinforces weak mortar bonds and bed joints.

In (Berto, et al., 2004) micro and macro-modeling techniques are used to model masonry panels in cooperation with the finite element method. An isotropic model is used to simulate the behavior of the mortar and an orthographic model is used to simulate the non-linear behavior of the masonry. The results obtained demonstrate the advantages of macro modeling, which allows the global behavior of the masonry to be simulated. Such results are hardly seen using micro-modeling, mainly because of the high computational effort required.

(Mendola, et al., 2014) use the finite element method to study the of the out-of-plane behavior of reinforced masonry walls and unreinforced masonry walls. An experimental investigation using numerical finite element modelling is performed. A parametric analysis is carried out to investigate the effect on the brick-mortar interface. The numerical models for the unreinforced masonry walls are able to simulate the deformability of the bed joints. It is mentioned that in order to achieve the real behavior of the masonry walls, it is necessary to reduce the effective modulus of elasticity.

2.4.2 Review of experimental studies on the dynamic analysis of masonry.

According to (Tomaževič, 2016), earthquake ground motions are stochastic and the characteristics of an earthquake depend on the source of the earthquake and the ground conditions of the area. An earthquake with the same numerical data will never occur again. Therefore, when choosing the input data of a seismic analysis, consideration should be given to the similarity between the acceleration-time history and the design response spectrum from the codes. Masonry is aimed at holding compressive loads as opposed to tensile or shear forces. Tensile and shear forces, however, do develop on a masonry structure under seismic loading. When subjected to shear loads, masonry walls behave in a brittle manner. Thus, steel reinforcement is required in both vertical and horizontal directions. To obtain quantified data about the structural and material properties of masonry under seismic loading, specialized testing and experimental research is required. Such data can be used to build accurate mathematical models for design purposes. The study conducted by (Tomaževič, 2016) tests some of the considerations when simulating the behavior of masonry walls under seismic loading. Factors such as loading protocols, scaling effects and boundary conditions are considered and discussed. The paper describes masonry as a non-homogenous, anisotropic material lacking elasticity, therefore, making

the seismic assessment of masonry challenging. The study provides experimental results of typical examples which investigate the seismic behavior of masonry. A common uni-directional shaking table is used in the study. Response spectrum data of the 1979 Montenegro earthquake is used with the vertical component of the ground motion regarded as negligible.

(Galaso, et al., 2004) uses macro-modelling techniques to create 3-D models to assess the seismic performance of different masonry structures such as multistory buildings, historical masonry monuments and masonry arch bridges. The results attained from the models correlate well with existing data attained from experimental investigations.

(Kumar & Pallav, 2018) perform a static and dynamic analysis of an URM wall. The geometry used in the study is a senate hall building built in 1915 in Allahbad, India. A macro-modeling approach is used to create the model using ANSYS software. A modal analysis is performed using frequencies; 0.703, 0.844, 1.239, 1.855, 2.666, 3.017Hz. The on-site survey of the building shows severe cracking and damage, similar results are obtained in the finite element analysis, thus confirming the reliability of the model.

A numerical model to investigate the behavior of unreinforced masonry walls to in-plane dynamic loads is presented in (Zhuge, et al., 1998). The finite element model incorporates the non-linear effects of masonry. The model can produce joint sliding and cracking/crushing failure modes. The dynamic analysis is carried out using the iterative Newton-Rhapson method.

(Cakir, et al., 2015) carries out a numerical dynamic analysis of historical masonry structures, simulating the 7.2Mw Van earthquake of 2011 in Turkey. The study first examines the existing guidelines for seismic design on structures found in the relevant design codes. The numerical model is created using the FEM and ANSYS software. A response spectrum analysis is performed, however, the results from the response spectrum analysis are not sufficient to find the cause of failure for all buildings investigated. The study finds that a lot of failure occurred due to weakened column-arch joints caused by environmental and human effects. The expected failure modes in (Cakir, et al., 2015) can be seen in Figure 2-18.

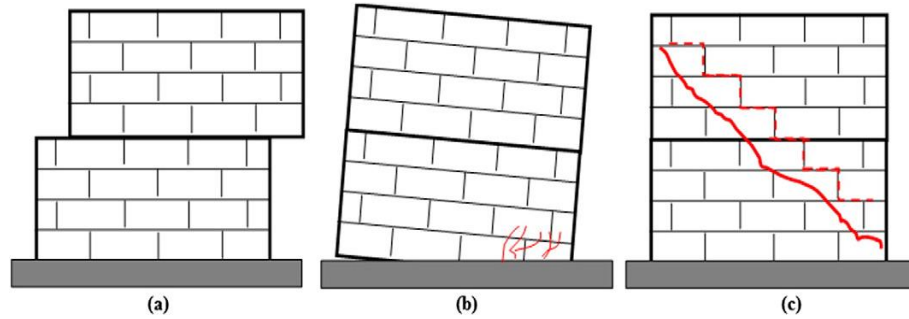


Figure 2-18: Failure modes in unreinforced masonry walls (a) bed-joint sliding, (b) rocking and toe compression, (c) stair-stepped diagonal and diagonal cracking (Cakir, et al., 2015).

(Betti & Vignoli, 2011) present a numerical assessment of the static and seismic behavior of the basilica of Santa Maria all'Impruneta. The FEM is used to conduct a numerical study on the historical structure in which non-linear effects are taken into consideration. A macro-modelling approach is used with homogenization techniques and smeared crack modelling. The pushover method was used to assess the seismic vulnerability of the structure. The paper aims to prove that sophisticated numerical models can provide advanced information and understanding of the structural behavior of historical buildings.

(Churilov, et al., 2016) identifies the dynamic properties of masonry buildings before formulating mathematical FEM models of existing masonry structures. The paper considers three unreinforced masonry buildings and one confined masonry building. Residential buildings, family houses and school buildings are used in the study. In-situ ambient vibration tests are used to identify the dynamic properties of the respective structures. The purpose of these tests is to obtain the properties of the masonry walls to determine their seismic resistance. Fixed boundary conditions are used with the soil-structure interaction being ignored. Walls and slabs are modeled using plane elements and all materials are assumed to be isotropic linear elastic. The basic material properties of Young's modulus = 3233 N/mm^2 , Poisson's ratio = 0.2, shear modulus 1293 N/mm^2 and unit weight = 16 kN/m^3 are obtained from Eurocode 6. All buildings are assumed to be made up of the same masonry material, bricks and cement mortar. At the end, natural frequencies and mode shapes are formulated experimentally and analytically for four masonry buildings with a relatively good correlation between experimental and numerical results.

(Elvin, 2009) conducts an experimental study on a full-scale plastered masonry wall under dynamic loading. The paper presents methods of testing full-scale structures under dynamic loading. The earthquake loading is simulated by servo-hydraulic test machines. Three

earthquakes are considered in the study: The El Centro, Llole and Northridge earthquakes which have a magnitude of 7.1, 7.8 and 6.7 ML respectively. Numerical data from these earthquakes are used to simulate the respective earthquake on a shake table. Displacement and acceleration values are measured off the table. When compared to the response spectrum plot from the original earthquake the response spectrum from the applied acceleration is almost the same.

(Ahmad, et al., 2014) investigates the seismic performance of a heritage masonry structure. With it being a brittle, heavy material with low tensile strength, unreinforced masonry structures are extremely vulnerable to seismic activity. Considering that most unreinforced masonry structures are domestic buildings, this poses a threat to human life. When an unreinforced masonry structure fails under seismic loading, it results in a dramatic and sudden collapse with little to no yielding. The structure investigated in this study is a 137-year-old heritage building in Aligarh. The structure is modeled and discretized using the FEM, incorporating homogenization and non-linear characteristics. The model is subjected to different levels of seismic loads and the results are useful in detecting failure zones which could be used to help maintain the structure.

(Koutromanos, et al., 2011) investigates the behavior of masonry-infilled reinforced concrete frames under dynamic loading. Numerical, non-linear finite element techniques are adopted to simulate the behavior of the structure. A smeared-crack continuum model describes the cracking and crushing damage in the masonry and concrete. The model uses an elasto-plastic formulation to determine the mixed-mode fracture of concrete and masonry. The models are validated using experimental data from existing work. The study illustrates the ability of non-linear finite element models to simulate the behavior of concrete frames with masonry infill under cyclic loads as it can predict the force–displacement diagrams, crack patterns, and failure mechanisms of the structure.

Chapter 3 – Methodology

3.1 Introduction

The first part of this study involves conducting a comprehensive literature review to gain insight into the structural behavior and the numerical modeling methods involved in masonry. The literature review presented in Chapter 2 reviews various sources of information including journal articles, theses, websites, government documents and technical reports. Understanding the behavior of masonry as a structural material and the modelling methods used for masonry, such as the FEM, is critical in order to proceed to creating the numerical models used in this study.

The FEM is adopted to create the numerical masonry models used in this study. The masonry models are analyzed under both static and dynamic loading. The static analysis uses the “Static Structural” option and the dynamic analysis uses “Modal Analysis” and “Response Spectrum Analysis” to generate results. This chapter details how the numerical experiments shown in Chapter 4 are set up.

ANSYS software is used to set up and analyze the numerical masonry models. The software uses the finite element method to carry out both a static and dynamic structural analysis. ANSYS is a commercially available numerical modeling computer software that can perform several types of analyses. This includes structural static analyses, dynamic analyses, fluid dynamics, heat transfer and basic vibration analysis. Two-dimensional and three-dimensional geometries can be created on or incorporated into ANSYS. The software generates visual outputs pertaining to the chosen type of analysis and results. Results are also presented numerically, which can be exported to other programs such as Microsoft Excel to formulate graphs

The ANSYS software incorporates the FEM to perform a structural analysis on a given geometry. The software does this using three broad steps, pre-processor, FEA solver and post processor. These steps are general to both static and dynamic analysis and are detailed further in Table 3-1. The methodology used to perform the static structural analysis is explained in Chapter 3.2 and the methodology for the structural dynamic analysis is found in Chapter 3.3.

Table 3-1: General steps for FEA in ANSYS.

<i>Pre-processing</i>	➤ Create solid geometry. ➤ Choose analysis type. ➤ Define material properties. ➤ Define contact. ➤ Generate meshing. ➤ Define loading and boundary conditions.
<i>FEA solver</i>	➤ Form stiffness matrix for each element. ➤ Assemble the global stiffness matrix. ➤ Calculate deformations, stresses and strain.
<i>Post-processing</i>	➤ Review results of analysis. ➤ Represent data graphically and numerically.

3.2 Methodology for static structural analysis

3.2.1 ANSYS Workbench – Static Structural Analysis.

The first step in creating the numerical model on ANSYS is selecting the analysis type. For static analysis the "Static Structural" option is used. The "Static Structural" option is used to analyze structures under static loading only. It allows for quick and accurate analysis of complex engineering problems which is helpful in optimizing designs and reducing the costs of physical testing. The "Static Structural" option on ANSYS allows for challenging non-linear adaptivity and progressive re-meshing. This study will make use of a uniformly distributed pressure loading as well as horizontal displacement loading. Figure 3-1 illustrates the steps to be undertaken to set up a static structural model on ANSYS.

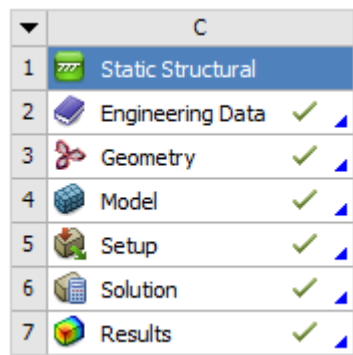


Figure 3-1: Static structural analysis process on ANSYS..

3.2.2 Engineering Data, material properties and failure criteria

ANSYS does have a database of common materials used in engineering, however, the materials used in this study have been created. Material properties of the two separate materials used in this study, concrete blocks and mortar, are inputted manually. For a basic, linear static structural analysis, only isotropic elastic material properties are needed. This consists of Young's modulus, Poisson's ratio, bulk modulus and shear modulus. It should be noted that the values of bulk modulus and shear modulus can be generated from the values of Young's modulus and Poisson's ratio. This study incorporates the Drucker-Prager failure criteria to simulate the non-linear behavior of masonry. The Drucker-Prager failure criteria requires an additional three material properties. This consists of the uniaxial compressive strength, uniaxial tensile strength and biaxial compressive strength. Furthermore, for the dynamic analysis, the density for both materials is needed. Uniform temperature is assumed for all numerical experiments in this study, therefore, material properties such as thermal expansion coefficient and temperature coefficient are not required. The material properties used in this study can be seen in Table 3-2 and are attained from similar studies relating to the failure behavior of masonry, found in (Drosopoulos & Stavroulakis, 2018), (Kömürcü & Gedikli, 2019) and (Lourenço & Pina-Henriques, 2006).

Table 3-2: Material properties used for the study.

Material Property	Masonry Unit	Mortar	Unit
<i>Isotropic Elasticity Properties</i>			
Modulus of elasticity (E)	4865	1180	MPa
Poisson's ratio (ν)	0.09	0.06	
Bulk modulus	1977.6	446.97	MPa
Shear modulus	2231.7	556.6	MPa
Density (ρ)	1800	2162	kg/m ³
<i>Geomechanical – Drucker Prager (non-linear) Properties</i>			
Uniaxial compressive strength (f_c),	26.9	3.2	MPa
Uniaxial tensile strength (f_t)	4.9	0.9	MPa
Biaxial compressive strength (f_{cc})	27	3.5	MPa

3.2.3 Geometry

ANSYS does have a built-in sketching option used for creating geometries, however, due to the geometrical complexity of masonry, all geometries are created on AutoCad software as the “Design Modeler” workspace on ANSYS is found to be tedious. The AutoCad drawings must be converted to an enclosed solid element in order to be read as a solid surface on ANSYS. This is done by adopting the “REGION” command on AutoCad. Before the geometry is uploaded to ANSYS it needs to be converted to a “SAT” file. The conversion can be done using the "Export">"Other format">"ACIS (*.sat)" option in AutoCad.

Two three-dimensional geometries are created. The walls depict concrete masonry blocks, held together with mortar. A 390x190x140 mm masonry unit is used as seen in Figure 3-3. A list of concrete block dimensions and specifications used in South Africa can be found in (Concrete Manufacturers Association, 2006). For ease of modelling, the holes in the concrete masonry unit as seen in Figure 3-3 are ignored and the masonry unit is assumed to be a solid block as shown in Figure 3-2. The Stretcher bond is used to lay the masonry units as illustrated in Figure 3-4, details of the Stretcher bond are explained thoroughly in Chapter 2.2.5. Geometry 1 is a 1-meter square masonry wall and Geometry 2 is a full-scale masonry wall with a window and door opening.

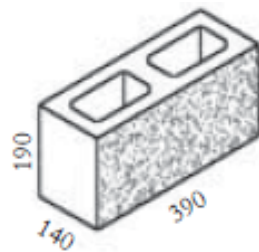


Figure 3-3: Concrete masonry unit.

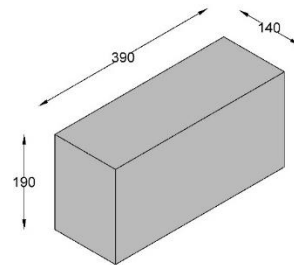


Figure 3-2: Dimensions of single masonry unit.

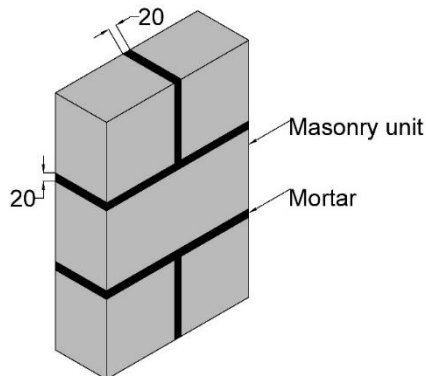


Figure 3-4: Masonry assembly done using "Stretcher Bond".

Geometry 1, as shown in Figure 3-5, is a standard 1-meter square masonry wall often used in numerical studies assessing the structural behaviour of masonry, as such, Geometry 1 is used to generate results that can be compared to previous studies. The mortar, which is represented in black in Figure 3-5, is modelled as one single body, with the masonry units inserted inbetween.

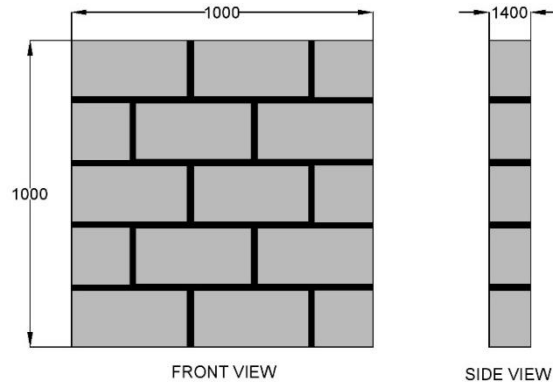


Figure 3-5: Geometry 1 – 1-meter square masonry wall.

Geometry 2 is created to represent one wall of a full scale low-cost masonry house. It includes a single door and a window opening. Dimensions are attained from the South African Department of Human Settlements (Department of Human Settlements, 2012). The wall depicted in Geometry 2 represents the front side of a 40m² low-cost masonry house. Door type 'D1' which indicates a 813x2032 mm door and window type "NC4" which represents a 1511x949 mm window according to (Department of Human Settlements, 2012) are employed. Just as in Geometry 1, the mortar is modelled as one single body, with the masonry units inserted inbetween. Geometry 2 is shown in Figure 3-6.

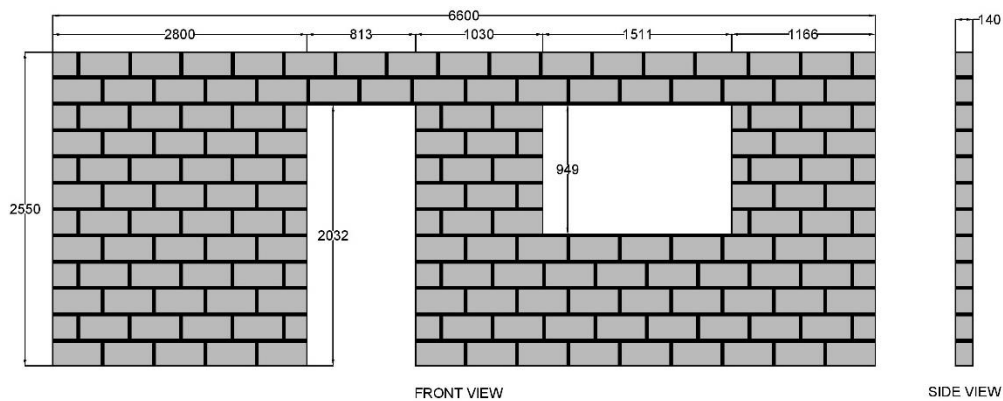


Figure 3-6: Geometry 2 - low-cost house wall.

3.2.4 Contact interfaces

Contact interfaces occur in ANSYS when two separate surfaces touch each other. The contact interface defines how tangential and normal forces are transmitted between the two surfaces. As such, certain types of contacts in a numerical model may introduce non-linearity into the model. Masonry, being a heterogeneous material, is highly influenced by the contact between the masonry units and the mortar. ANSYS provides five contact options which are described below

- *Bonded contact* – this is a linear contact that doesn't allow for sliding or separation between the two surfaces.
- *No separation contact* – this is also a linear contact that does not allow for separation between the two surfaces, however, sliding is allowed, be it without resistance.
- *Frictionless contact* – is a non-linear contact that allows the two surfaces to separate and slide without resistance.
- *Frictional contact* – is also a non-linear contact where both surfaces can separate from each other. The surfaces can also slide with resistance, defined with a frictional coefficient.
- *Rough contact* – this is also a non-linear contact that allows the two surfaces to separate but does not allow sliding.

It would be preferable to incorporate the non-linear “frictional” contact option into the model as this option best describes the contact found in masonry, however, non-linear contact options require extremely high computer capacity to execute. Computer capacity is limited in this study and as such, models incorporating non-linear contact do not converge and end up crashing. It is also noted that unilateral contact-friction is highly non-linear, indicating that when multiple interfaces (like in the models developed in this thesis) are introduced, convergence of the simulation to a solution is difficult to achieve (Drosopoulos & Stavroulakis, 2018). In this study, the bonded contact option is chosen in combination with a multipoint constraint (MPC). Multi-point constraint, bonded contact relates two surfaces in a simple manner which requires minimal computational effort, leading to good convergence and short run times.

When importing geometrical assemblages to ANSYS, contact regions are automatically created between each separate solid body within the geometry. The contact regions are defined using the concept of “*contact*” and “*target*” surfaces. At an interface point between

two bodies, one region is taken as the “contact” surface and the other region is taken as the “target” surface. An extract of the contact modelling process using Geometry 1 is discussed below.

Figure 3-7b shows the contact interface between two bodies, Part 7 which represents a masonry unit and Part 16 which represents the mortar. Part 7 is the contact body and is illustrated in red while part 16 is the target body and is illustrated in blue. Part 7, the contact body shown in Figure 3-7c, is in contact with 4 mortar target faces as shown in Figure 3-7d. Since the same contact settings are used throughout the geometry, the contact between Part 7 and Part 16 creates one contact interface between Part 7 and Part 16 but with 4 contact faces as seen in Figure 3-7a. Masonry units on the edges of the wall will be in contact with only 3 mortar faces while masonry units at the corner edges will be in contact with only 2 mortar faces.

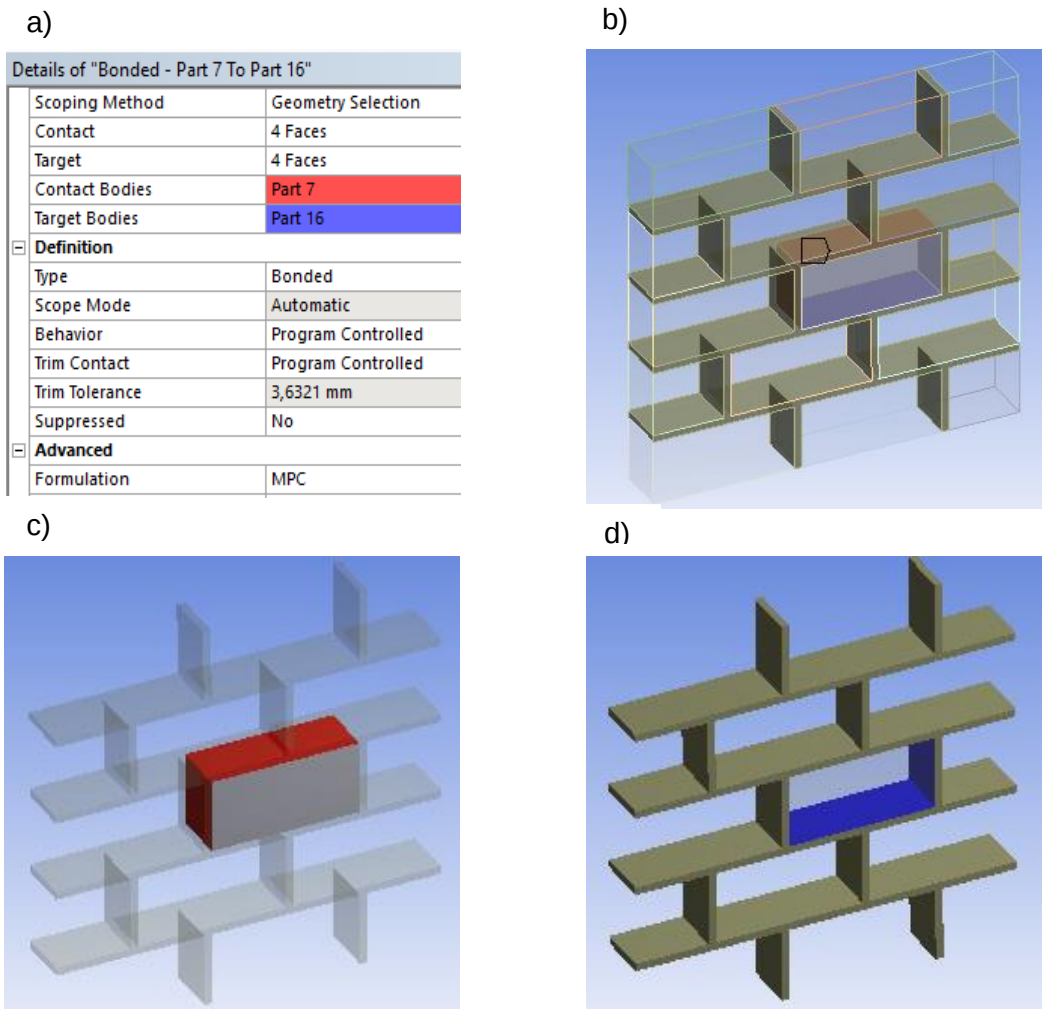


Figure 3-7: Extract of contact interface using Geometry 1.

3.2.5 Generate finite element mesh

The FEM divides a body into several tiny elements, this process is known as discretization. Discretization allows for each element to be analyzed individually and a stiffness matrix to be created for each finite element. The quality or density of the mesh has a direct impact on the accuracy of the results. A denser mesh means a greater number of finite elements and the general rule is, the denser the mesh, the more accurate the results. However, a greater number of finite elements increases computational time and effort. A model can often crash if there are too many finite elements and nodes due to lack of sufficient computer capacity. The ANSYS software allows for the finite element mesh to be automatically generated. However, in this study, the mesh sizing is manually generated. Different size mesh elements are chosen for different geometries.

The hexahedral element is used to define the shape of each finite element in this study. Each finite element is made up of surface faces and nodes. Nodes are defined points at the corner of each finite element. The hexahedral element has 8 nodes, one for each corner of the finite element. Each node is free to move according to how the body behaves. This is illustrated in Figure 3-8.

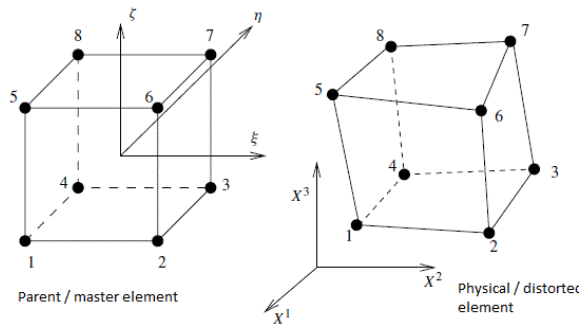


Figure 3-8: Movement of nodes on a hexahedral element (Campbell & Durán, 2017).

Figure 3-8 shows a node at each corner of the hexahedral element, leading to 8 nodes for each finite element, however more nodes can be added at midpoints between corner nodes, as seen in Figure 3-9. This results in a total of 20 nodes for each element. For the purpose of this study the midpoint nodes are ignored as they are viewed as being redundant while increasing the number of nodes in the mesh, which in turn increases computational time and effort.

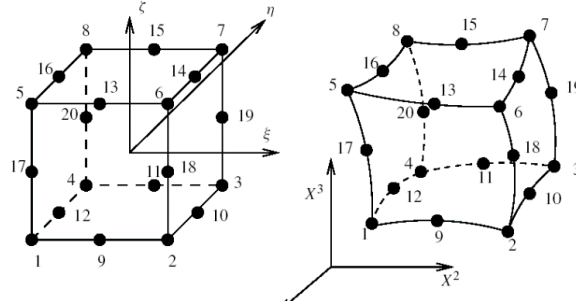


Figure 3-9: Midpoint nodes the hexahedral elements (Campbell & Durán, 2017)

Table 3-3: Number of elements and number of nodes related to mesh size.

Mesh Size (mm)	Elements in single masonry unit	Nodes in single masonry units
40	200	330
30	455	672
20	1400	1848
10	10374	12000

Table 3-3 represents the number of finite elements and nodes a single masonry unit is divided into using varying mesh sizes. Mesh sizes of 40, 20, 30 and 10 mm are depicted with their resulting number of finite elements and nodes.

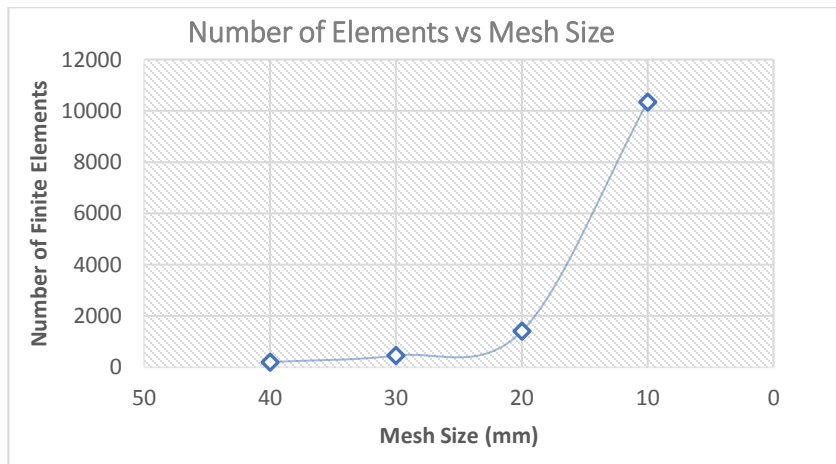


Figure 3-10: Number of finite elements vs mesh size.

Figure 3-10 is obtained by discretizing a single masonry unit, as seen in Figure 3-2, using different mesh sizes as seen in Table 3-3. Due to the three-dimensional nature of the geometry, changes in the mesh size will drastically affect the number of elements and nodes in the discretized geometry. Figure 3-10 shows that reducing the mesh size increases the number of finite elements in the model exponentially. Due to the midsize nodes being dropped the number of nodes remains relatively close to the number of elements.

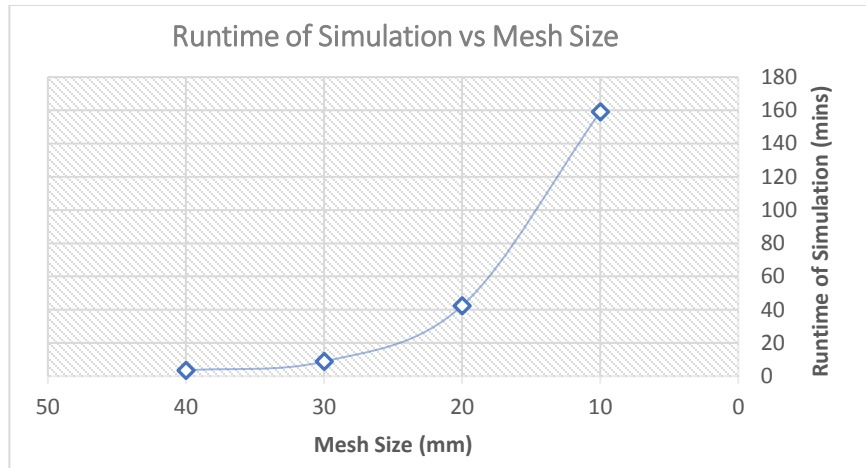


Figure 3-11: Runtime of simulation vs mesh size.

Figure 3-11 is obtained by noting the time taken to complete a structural analysis for different mesh densities. The four different mesh sizes considered are seen in Table 3-3. The analysis used to obtain the data is a basic static structural analysis using Geometry 1. To ensure an accurate comparison, all four simulations are run on the same computer with no other programs running in the background. Figure 3-11 clearly indicates that reducing the mesh size will increase the runtime of the simulation exponentially. It should be noted that when the simulation is run using a 5 mm mesh, the program crashes, thus defining the limit of the computer capacity at hand. Figure 3-11 allows for the correct mesh size to be chosen for the two geometries used in the study. While the runtime of the simulation is largely dependent of the processing capacity of the computer used, an exponential increase in runtime will always be expected with reducing the mesh size.

Considering that the 10 mm mesh pushed the limits of the computer capacity when performing a basic static structural analysis for Geometry 1, which is a small 1-meter wall, a larger 20 mm mesh size is chosen for the much larger Geometry 2. Figure 3-12a illustrates, on a single brick unit, the 10 mm mesh used in Geometry 1 and Figure 3-12b illustrates the 20 mm mesh used in Geometry 2.

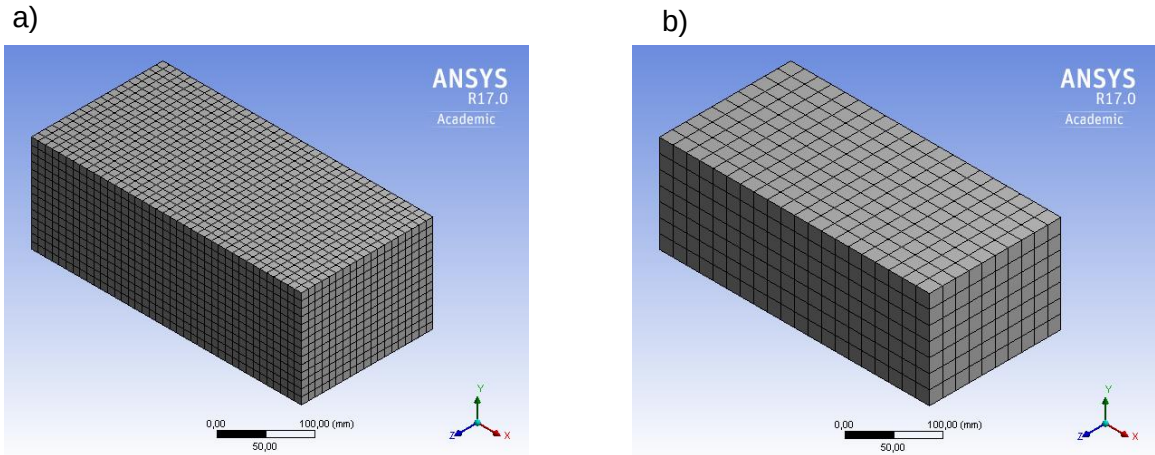


Figure 3-12: a) 10 mm mesh of discretized single masonry unit. b) 20 mm mesh of discretized single masonry unit.

Table 3-4: Total number of elements and nodes for Geometry 1 and 2

Geometry	Mesh Size	Number of elements	Number of nodes
1	10 mm	147,154	178,335
2	20 mm	245,434	353,568

Table 3-4 tabulates the total number of finite elements and nodes after discretization in Geometry 1 and Geometry 2 respectfully. The 245,434 elements used in Geometry 2 is a high number considering the limited computer capacity available. Any significant increase in the number of elements could lead to the simulation crashing. Figure 3-13 and Figure 3-14 illustrate the final mesh generated after discretization for Geometry 1 and Geometry 2 respectfully.

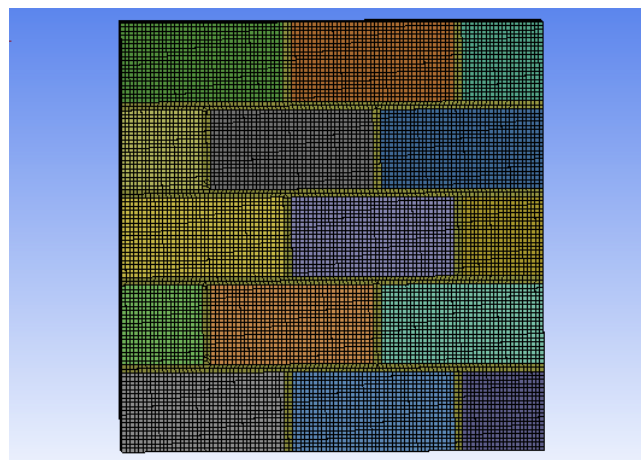


Figure 3-13: Fully discretized Geometry 1 using 10 mm mesh.

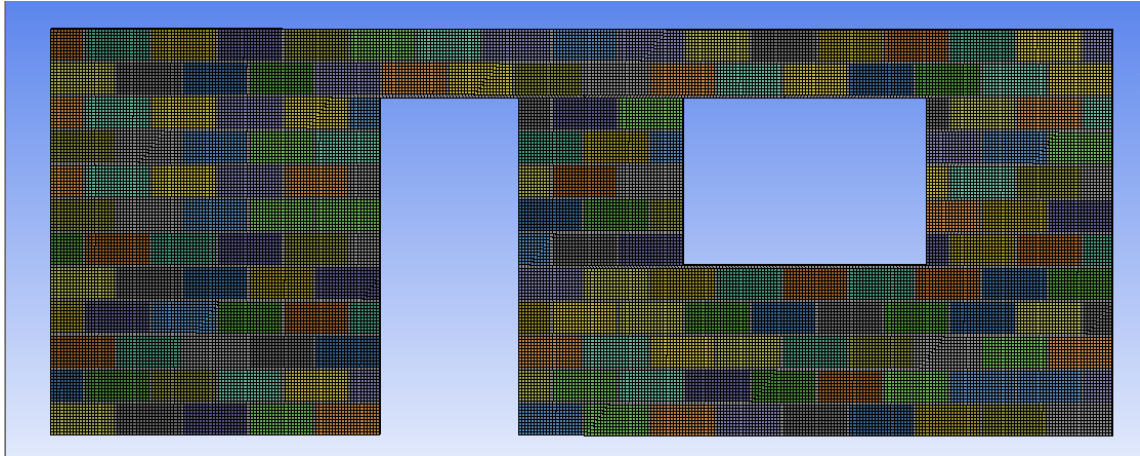


Figure 3-14: Fully discretized Geometry 2 using 20 mm mesh.

3.2.6 Define boundary conditions, static loading and load steps

3.2.6.1 *Boundary condition*

The boundary condition used for all models and geometries in the study is “Fixed”. Fixed supports can resist vertical forces, horizontal forces and moments which means all three equations of equilibrium can be satisfied. Fixed supports are also known as rigid supports and allow a structure to be stable with only one support. The “Fixed” boundary condition simulates walls found in domestic houses, with the bottom of the wall being fixed in place to the foundation. The “Fixed” boundary condition is applied to the bottom or under of both Geometry 1 and Geometry 2 as illustrated in Figure 3-15 and Figure 3-16.

3.2.6.2 *Static structural loading*

The static structural models are set up as shown in Figure 3-15 and Figure 3-16 for Geometry 1 and Geometry 2 respectively. Two types of loading are applied in the static models.

- A uniformly distributed vertical pressure loading along the top layer of the wall, acting downward and normal to the surface with a magnitude of 0.3 MPa . This load is applied at the start of the first load step.
- A horizontal displacement load acting from left to right along the top layer of the wall with a magnitude of 20 mm . This load is only applied at the second load step.

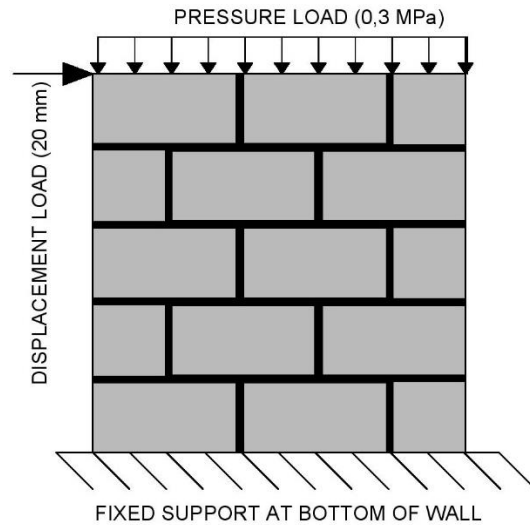


Figure 3-15: Applied static loads and boundary conditions for Geometry 1.

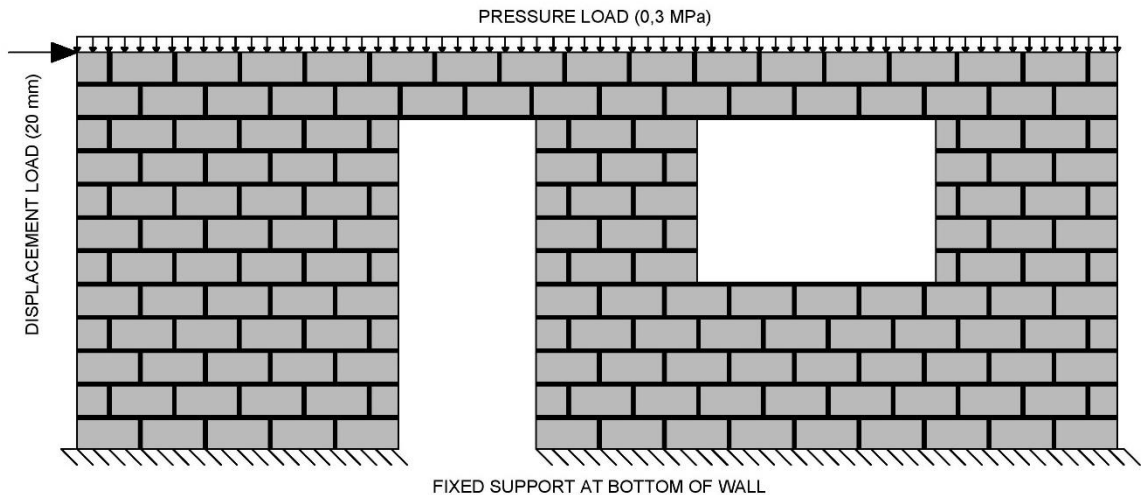


Figure 3-16: : Applied static loads and boundary conditions for Geometry 2.

3.2.6.3 Load steps

Two load steps are added for each model in the study. Each step has an auto time stepping function, for this study the initial time step is set at 0.1s with a minimum time step of 0.0001s and a maximum time step of 1s. This results in the load being applied in incremental iterations. Each load increment is equal to the time step multiplied by the overall loading. As the model approaches failure, smaller time steps are applied to get the most accurate failure load. These timesteps however, greatly increase computational time and effort, thus it is important to not choose too many time steps and too small minimum sub steps (ANSYS, 2011). The loads are applied in 2 load steps as shown in Table 3-5 which clearly illustrates

that the pressure loading is applied at the start while the horizontal displacement load is only applied at the start of the second load step.

Table 3-5: Load steps sued for static loads.

Load Step	Compressive pressure load	Horizontal displacement load
0	0.3 MPa	0
1	0,3 MPa	0
2	0	20mm

These load cases are consistent with those used in (Drosopoulos & Stavroulakis, 2018) which used the XFEM to simulate cracking failure in masonry walls. The uniformly distributed pressure load applies a compressive load to the model while the horizontal displacement load, in combination with the compressive load, generates shear and tensile forces in the model.

3.2.7 Choose output options

This is the final step to complete before executing the model. The visual outputs and results that are needed for the study are chosen here. The main outputs chosen are total deformation, maximum and minimum principal stress, equivalent (Von Mises) stress, shear stress and equivalent plastic strain. Each of these outputs detail how the masonry model behaves under the applied loading. The total deformation graphic illustrates points of high deformation in the structure which is important to find points of weakness and which areas need added support. The graphic illustrating distribution of stress can be used to identify areas where possible failure may occur. The equivalent plastic strain graphic is used to illustrate the non-linear behavior of the mortar. The Drucker-Prager failure criteria is incorporated into the mortar and hence will undergo plastic deformation and produce an equivalent plastic strain. A force probe and a deformation probe are included in the static analysis of both geometries. The force probe gives the horizontal force reaction at the point at which it is placed, and the deformation probe gives the corresponding horizontal structural deformation at that same point. These two sets of data are used to plot the force vs displacement diagram.

3.2.8 Summary of static structural methodology

The process of generating the static structural results for Numerical Experiment 1 and Numerical Experiment 2 is summarized in Figure 3-17.

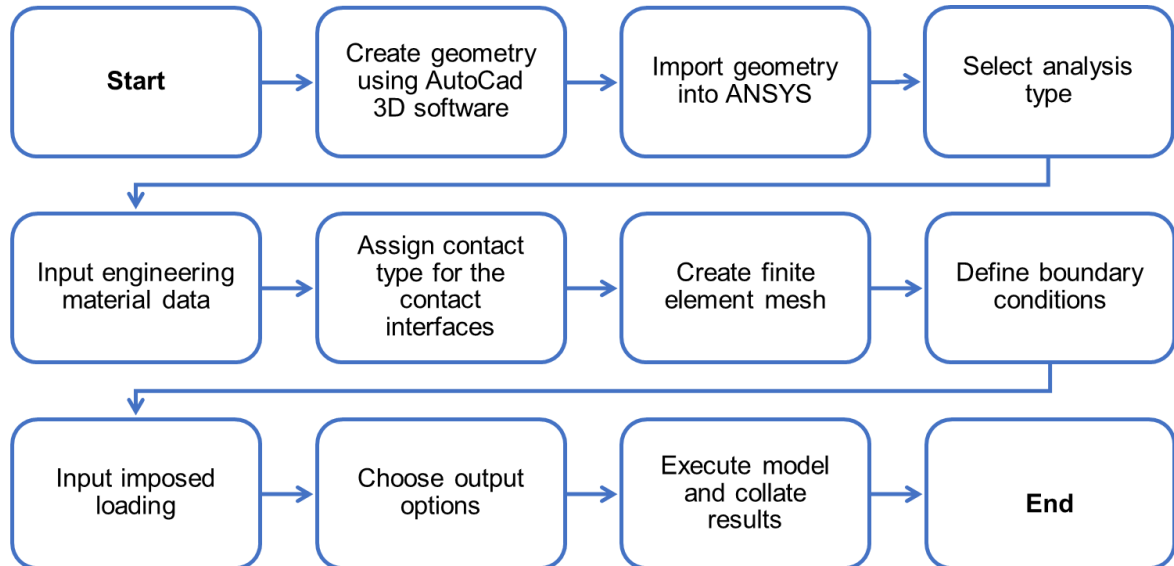


Figure 3-17: Summary of static structural modeling process.

3.3 Methodology for dynamic analysis

The dynamic analysis models are for the most part created using the same steps as the static analysis models, except for the load input. The geometry, selection of failure criteria, inputting of material properties, generating finite element mesh, defining boundary conditions and choosing outputs steps are all the same for the dynamic analysis as the static analysis.

3.3.1. Modal analysis

3.3.1.1 Introduction

Modal analysis is a method used to perform a basic dynamic analysis on structures. Modal analysis can be used to study the dynamic characteristics of a geometry experiencing vibrational excitation by determining the natural frequencies that the geometry oscillates at and generating the corresponding mode shapes. (ANSYS, 2014). A Modal analysis is often used as an initial step for more advanced dynamic analysis techniques like Response Spectrum analysis or Transient Structural analysis. It should be noted that the Modal analysis is a linear analysis, which ignores non-linear material properties, geometries and contacts. With the Modal analysis, the overall mass and stiffness of the structure under investigation is all that is needed to generate the various natural frequencies or periods that the structure will resonate at. This feature is useful as a building's natural frequency should not match that of frequencies of seismic activity in the region. If a structure oscillates at frequencies that match that of an earthquake for example, the structure may continue to resonate and experience structural damage. Modal analysis requires no loading input, rather the software depicts the model oscillating at different natural frequencies. When analyzing walls using modal analysis, the wall behaves like a bending slab after dynamic loads are applied. The dynamic response is dependent on the structures' stiffness and weight (Mynarz & Mynarzova, 2018). (Chopra, 2001) describes the process of attaining natural vibration frequencies and modes, these equations are presented in Equation 3-1 to Equation 3-8.

$$m\ddot{u} + c\dot{u} + ku = p(t) \quad (3-1)$$

Equation 3-1 describes the deformation $u(t)$ of the body, assuming it is idealized and linearly elastic, under external dynamic force $p(t)$ where c representants the damping factor and m and k are the mass and stiffness matrixes respectfully. For systems without damping and external loading, Equation 3-1 can be written as Equation 3-2.

$$m\ddot{u} + ku = 0 \quad (3-2)$$

T_n Represents the natural period of vibration for a single cycle of the simple harmonic motion of a natural mode in the system. ω_n is the natural circular frequency corresponding to T_n and f_n represents the natural cyclic frequency of the system which can be seen in Equation 3-3.

$$T_n = \frac{2\pi}{\omega_n} \text{ and } f_n = \frac{1}{T_n} \quad (3-3)$$

The free vibration of an undamped system in one of its natural vibration modes for a two-degrees of freedom system, is detailed in Equation 3-4.

$$u(t) = qn(t)\phi_n \quad (3-4)$$

In Equation 3-4, ϕ_n represents the deflected shape, which does not change with time in this case. The displacement's time variation is described by $qn(t)$, which is a simple harmonic function defined in Equation 3-5.

$$qn(t) = A_n \cos \omega_n t + B_n \sin \omega_n t \quad (3-5)$$

In Equation 3-5, A_n and B_n are constants derived from the initial conditions that generate the motion of the system at the start. Combining Equation 3-4 and Equation 3-5 gives Equation 3-6.

$$u(t) = \phi_n (A_n \cos \omega_n t + B_n \sin \omega_n t) \quad (3-6)$$

Combining Equation 3-6 with Equation 3-2 gives Equation 3-7.

$$[\omega^2 m \phi_n + k \phi_n] qn(t) = 0 \quad (3-7)$$

Equation 3.7 is used to derive Equation 3.8 which is referred to as the characteristic equation or frequency equation according to (Chopra, 2001). Equation 3.8 determines the N natural frequencies ω_n , where ($n = 1, 2, \dots, N$) of vibration and N is the DOF.

$$\det[k - \omega^2 m] = 0 \quad (3-8)$$

3.3.1.2 Modal analysis on ANSYS

In a modal analysis, Young's modulus, Poisson's ratio, and density are the only required inputs. All other material properties can be specified but are not used in a modal analysis. No loading input is required, the software generates the natural frequencies of the structure and shows results for each Mode. Figure 3-18a illustrates the Workbench setup for a Modal analysis on ANSYS. The Modal model is created using the same procedure as the static structural models, with the only exception being the loading input.

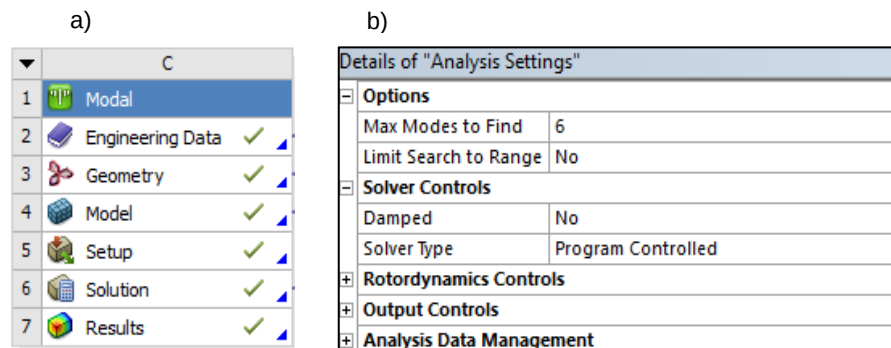


Figure 3-18: a) Modal analysis option on ANSYS Workbench and b) Analysis settings in Modal analysis.

In this study, the maximum number of Modes is set at 6, this can be seen in Figure 3-18b. Figure 3-19 illustrates the different natural frequencies corresponding to each Mode, as generated by ANSYS. The natural frequencies generated for the first 6 Modes using Geometry 2, range from approximately 6Hz to 40Hz.

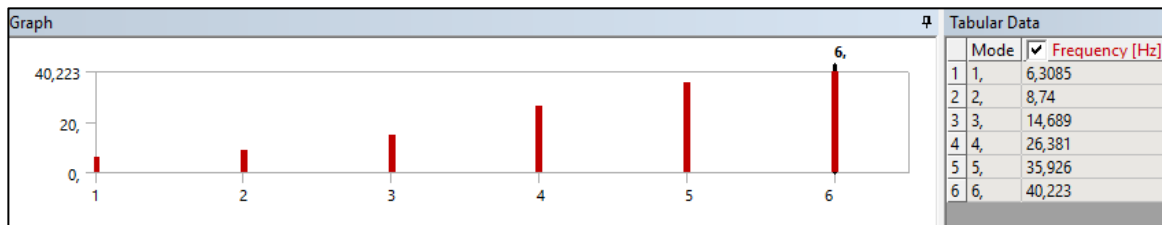


Figure 3-19: Frequencies corresponding to different Modes used in Modal analysis

3.3.2 Response spectrum analysis

3.3.2.1 Introduction

Calculating the behavior of a structure subjected to ground motion is highly challenging due to the complexity associated with earthquakes and the complicated structural set up and geometry of many structures. The response spectrum method has been introduced to help simplify the process. The method is used to estimate the structural response to dynamic events such as earthquakes. The response spectrum function shows the peak response of a simple oscillator that undergoes a certain dynamic transient event. The method, however, is not a direct representation of the frequency of excitation but rather the effect that the excitation has on a system with a single degree of freedom (Hudson, 1997).

It should be noted the earthquake response spectrum analysis is not a true dynamic load, but rather an approximation. It allows the seismic data to be incorporated into a model as equivalent static forces. The response spectrum method assumes the structure to have a single DOF and uses the highest structural response only as illustrated in Figure 3-20.

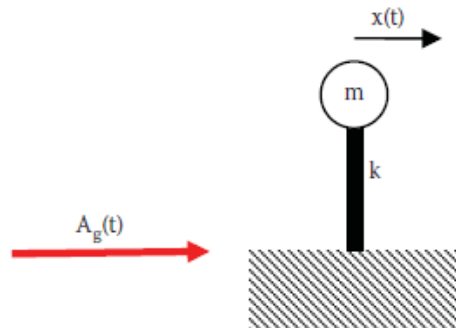
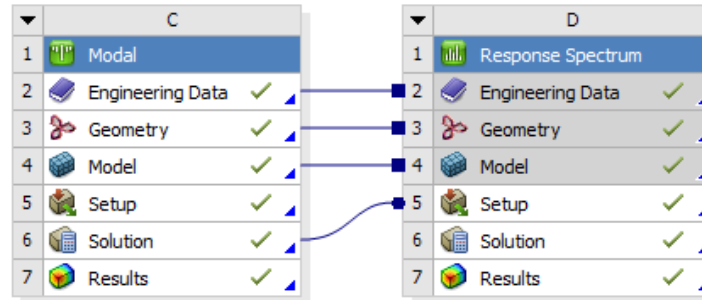


Figure 3-20: System representing a structure that is considered as a single degree of freedom (Elvin, 2009).

In order to apply response spectrum analysis for an event that has not yet happened a design spectrum needs to be created. A design response spectrum can be found in building codes and represents a convergence of all historic earthquakes in a certain geographical region. Design response spectrum graphs can be a plot of max acceleration, displacement or velocity vs period or frequency. The earthquake loading or seismic activity at a particular point in a body is represented by an elastic ground acceleration response spectrum called “*elastic response spectrum*”. The dynamic activity is described by orthogonal components, which are considered independent and represented by the same response spectrum (Elvin, 2009).

3.3.2.2 Response Spectrum analysis on ANSYS

To perform a Response Spectrum analysis on ANSYS, the model has to be connected to a Modal analysis model as depicted in Figure 3-20. The Response Spectrum analysis will share the same material properties, geometry, mesh, connections, support conditions and Modes of oscillation as the Modal setup.



3-21: Setup of Response Spectrum model connected to Modal analysis..

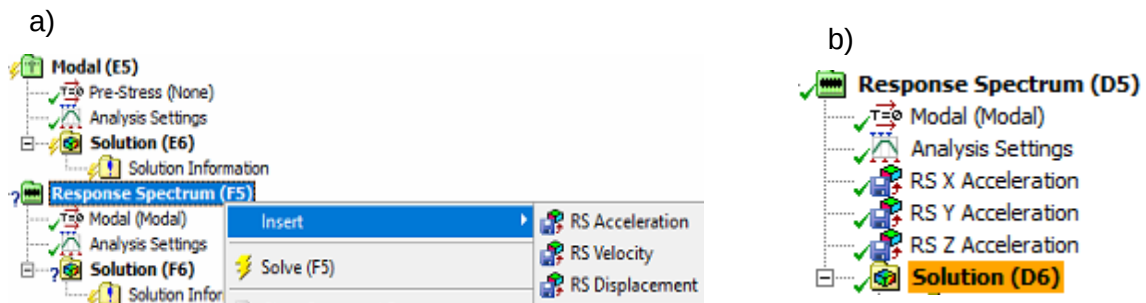


Figure 3-22: a) Load input options for Response Spectrum analysis. b) Acceleration applied in all three directions.

In the Response Spectrum analysis, loading can be added in the form of acceleration, velocity or displacement as seen in Figure 3-22a. Acceleration is used in this study. The response spectra acceleration values calculated are applied to the bottom fixed support of the wall, in all three directions as seen in Figure 3-22b. The ANSYS input requires the response spectra acceleration values to be added with corresponding frequency as opposed to period values.

3.3.2.3 Generating the design response spectra graph

The acceleration values are derived using (SANS 10160-4, 2009), “*South African National Standard. Basis of structural design and actions for buildings and industrial structures. Part 4: Seismic actions and general requirements for buildings*”. It should be noted that this study considers “Zone 1” seismic loading only according to (SANS 10160-4, 2009). “Zone 1”

considers natural seismic activity and not mining-induced seismic activity. The design response spectrum is plotted using the equations found in Figure 3-23. A design response spectrum graph is plotted for each ground type according to Table 3-6.

Ground type	Description of stratigraphic profile
1	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface
2	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanical properties with depth
3	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m
4	Deposits of loose-to-medium cohesion-less soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil

Table 3-6: Description of different ground types in South Africa (SANS 10160-4, 2009).

$$T_c \leq T \leq T_D : S_d(T) \left\{ = a_g \times S \frac{2,5}{q} \left[\frac{T_c}{T} \right] \right\} \text{ but } \geq \beta \times a_g$$

$$T_D \leq T : S_d(T) \left\{ = a_g \times S \frac{2,5}{q} \left[\frac{T_c \times T_D}{T^2} \right] \right\} \text{ but } \geq \beta \times a_g$$

$$0 \leq T \leq T_B : S_d(T) = a_g \times S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2,5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_c : S_d(T) = a_g \times S \frac{2,5}{q}$$

Figure 3-23: Equations to calculate the normalized design response spectra (SANS 10160-4, 2009).

Where:

A_g	=	Reference horizontal peak ground acceleration factor
T	=	The vibration period of a linear single DOF system (s)
T_B, T_c	=	Limits of the constant spectral acceleration branch (s)
$S_d(T)$	=	The design response spectrum for elastic analysis
q	=	Behavior factor depending on type of structure
β	=	The lower bound factor for horizontal design spectrum

As recommended in (SANS 10160-4, 2009), the reference horizontal peak ground acceleration factor (a_g) = 0,1 for Zone 1 areas, a behavior factor (q) of 1,5 is used for unreinforced masonry walls and the lower bound factor β has a value of 0,2. The other variables are dependent on specific ground types and are attained from Table 3-7.

Table 3-7: Values of the parameters describing the design response spectra (SANS 10160-4, 2009).

1	2	3	4	5
Ground type	Parameters*			
	S	T_B	T_C	T_D
1	1,0	0,15	0,4	2,0
2	1,2	0,15	0,5	2,0
3	1,15	0,20	0,6	2,0
4	1,35	0,20	0,8	2,0

The normalized design response spectrum is plotted for each ground type following the procedure in (SANS 10160-4, 2009) on Microsoft Excel. Figure 3-24 presents the calculated normalized design response spectra for all four ground types found in South Africa. The procedure to obtain S_d for various frequencies is shown in Figure 3-23. The S_d values are inserted into ANSYS alongside the corresponding frequency. As seen in Figure 3-24, ground type 4 produces the highest peak ground acceleration, therefore, the normalized design response spectra for ground type 4 is used in this study.

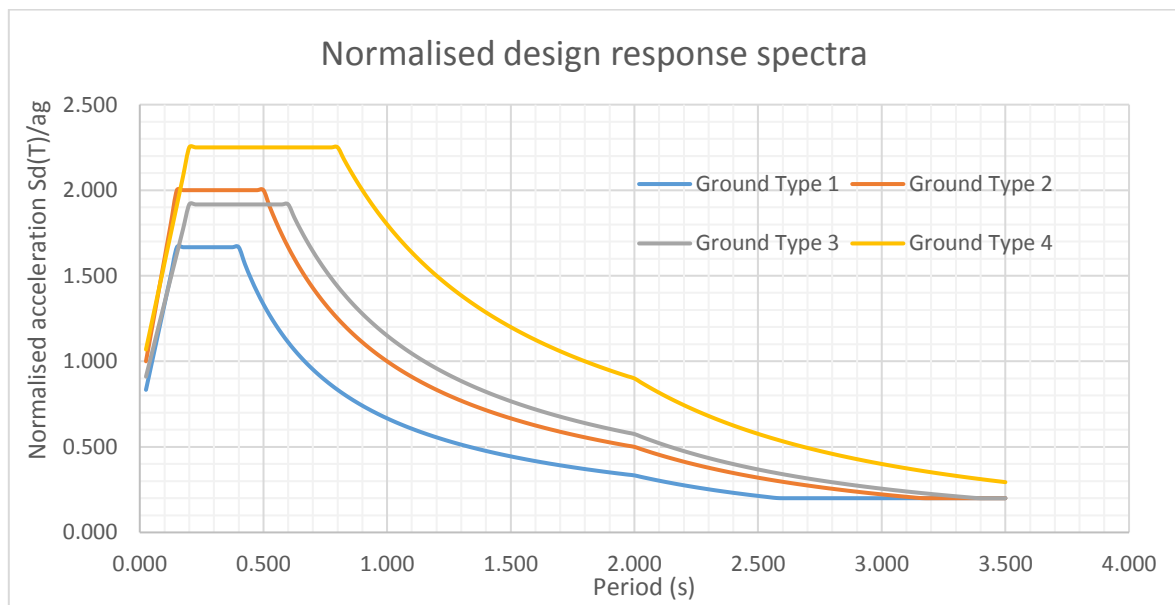


Figure 3-24: Normalized design response spectra.

3.4 Limitations to study

- The masonry units are modelled as solid blocks. The hollowness of the masonry units is ignored due to the high computational effort required to model such a unit. Taking into consideration the hollowness of a masonry unit will add several extra contact surfaces requiring increased computational capacity.
- Due to insufficient computer capacity the contact between the masonry units and mortar are modeled as bonded, which is a linear contact type. A more realistic representation of the contact would be adding a non-linear frictional contact. However, this requires much more computer processing capacity.
- A transient structural dynamic analysis is a more realistic representation of an earthquake load; however, it requires high computational capacity.
- For the Response Spectrum analysis, the number of data values that can be inputted is limited to 100, as such, every alternate value is inputted into the Response Spectrum acceleration table.
- An increased number of nodes gives more accurate results, however for trials done in the study, models with over approximately 300 000 finite elements do not converge and end up crashing.

Chapter 4 – Results and discussion

4.1 Introduction

Chapter 4 presents the results and outcomes of the three numerical experiments carried out in this study. The three numerical experiments carried out are as follows:

- Numerical Experiment 1 - static structural analysis of a 1-m square masonry wall.
- Numerical Experiment 2 - static structural analysis of a full-size, low-cost house masonry wall with door and window opening.
- Numerical Experiment 3 - dynamic structural analysis of a full-size, low-cost house masonry wall with door and window opening.

The numerical experiments are conducted in accordance with the aims and objectives as discussed in Chapter 1 and follows the methodologies that are presented in Chapter 3. The numerical experiments aim to depict the failure behavior of masonry walls under static and dynamic loading. The finite element method is used in conjunction with ANSYS software to create the numerical models. The outputs are presented as graphical illustrations, showing deformations, stresses and strains. Generally, the maximum values are shown in red while the minimum values are shown in blue with a spectrum of colors used to depict values in between. As well as the graphical outputs, the results also include force-displacement diagrams, which are used to illustrate the non-linear behavior of masonry when incorporating the Drucker-Prager failure criteria. The results attained are discussed and compared to existing literature from previous numerical and experimental studies.

(Agüera, et al., 2016) and (Kömürcü & Gedikli, 2019) conduct numerical studies on the structural behavior of unreinforced masonry walls by subjecting the wall to axial and horizontal loads and incorporating the Drucker-Prager failure criteria, similar to Numerical Experiment 1 and 2 in this dissertation. Some relevant results from this study are shown in Figure 4-1 and Figure 4-2. (Drosopoulos & Stavroulakis, 2018) uses the XFEM and non-linear methods to predict the failure pattern of masonry walls. The failure pattern discovered is shown in Figure 4-3. (Khoiratty, 2016) conducts a field assessment of low-cost houses in the North West Province of South Africa after an earthquake. The common damages and impact of the seismic events are presented, some of these damages can be seen in Figure 4-4a and Figure 4-4b. (Elvin, 2009) performs an experimental investigation of a full-scale masonry wall using a shake table to attain the wall's response. The damage from the shake table is shown in Figure 4-4c.

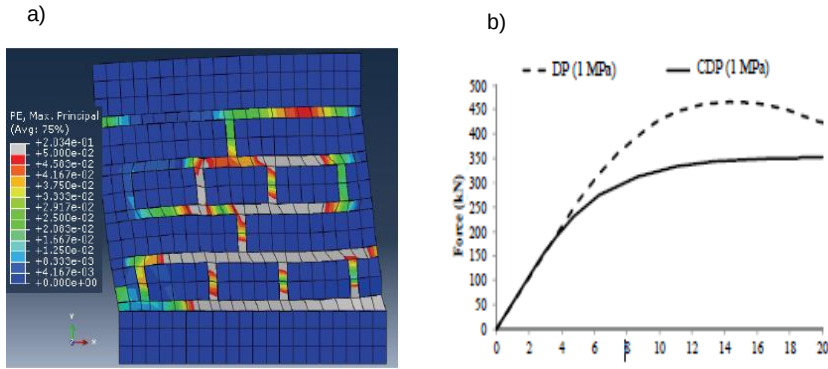


Figure 4-1: a) Plastic strain and b) force-displacement diagram with the DP failure criteria (Agüera, et al., 2016).

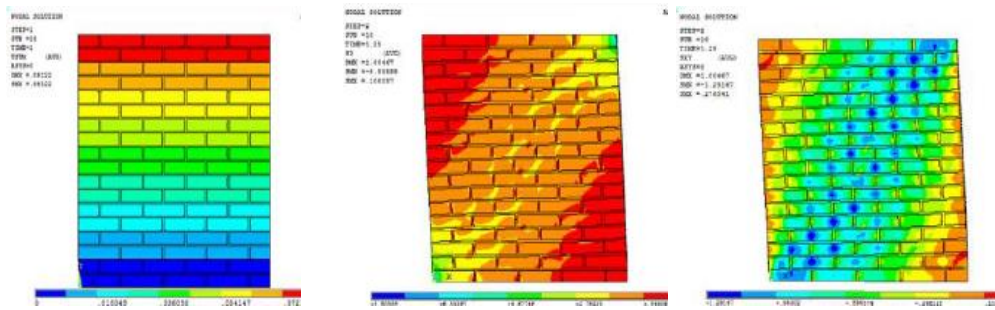


Figure 4-2: Total displacement, principal stress and shear stress distribution. (Kömürcü & Gedikli, 2019).

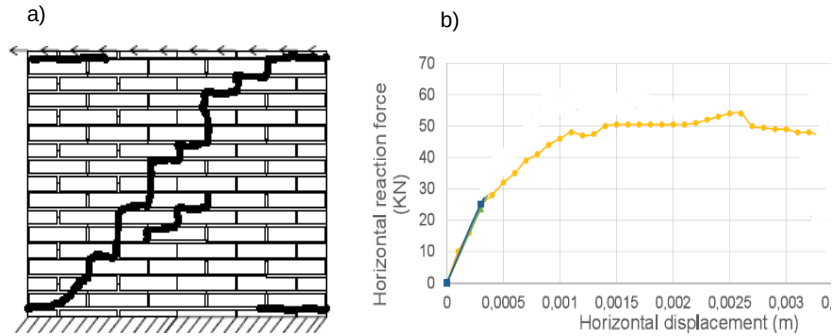


Figure 4-3:a) Failure pattern and b) force vs displacement graph of masonry, (Drosopoulos & Stavroulakis, 2018).

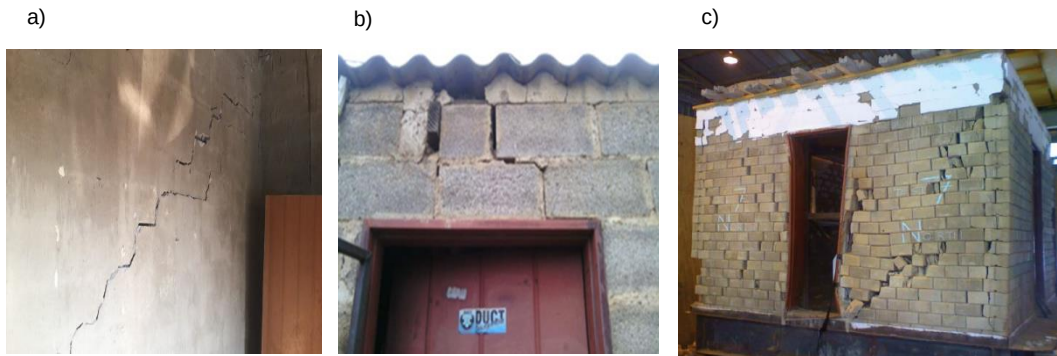


Figure 4-4: Damaged caused to low-cost masonry houses due to seismic activity. (Khojraty, 2016), (Elvin, 2009).

4.2 Numerical Experiment 1 – Static structural analysis of Geometry 1

Numerical Experiment 1 is a static structural analysis of Geometry 1. The static structural analysis is conducted in accordance with the methodologies presented in Chapter 3. The summarized input data for Numerical Experiment 1 can be seen in Table 4-1. The deformations, stress, strains and force vs displacement diagrams for Numerical Experiment 1 are presented and discussed in this chapter.

Table 4-1: Input table for Numerical Experiment 1.

Static structural analysis of 1-meter wall	
Geometry	1
Analysis type	Static structural
Contact	Bonded, MPC
Mesh size	10 mm
Mesh nodes	178335
Mesh elements	147154
Drucker-Prager	Yes
Analysis settings	2 load steps
Initial time step	0.1 s
Min time step	0.001 s
Max time step	1 s
Support	Fixed at bottom
Pressure load	0.3/0.3/0 MPa (@ time steps 0,1,2)
Displacement load	0/0/20 mm (@ time steps 0,1,2)

Table 4-2: Deformation results for Numerical experiment 1.

Load step	Time(s)	Total Deformation (mm)	X Deformation (mm)	Y Deformation (mm)	Z Deformation (mm)
1 End	1	0.052	0.003	0.051	0.035
2 End	2	22.385	20.000	9.829	1.179

Table 4-2 presents the deformation results attained from Numerical Experiment 1. The deformations are given in terms of x, y and z values according to the structures movement along the respective axis. As explained in Chapter 3.2.6, the static loads are imposed in two load steps. The deformation as a result of each load step can be seen in Table 4-2. It is clear to see that the vertical compressive load imposed in load step 1 causes little deformation to the structure, with only the y-deformation showing any significant change. The total deformation after load step 1 is illustrated in Figure 4-5a where the maximum

deformation occurs at the top-center part of the wall, this is to be expected as sagging would occur when the top of the wall is under a vertical compressive load.

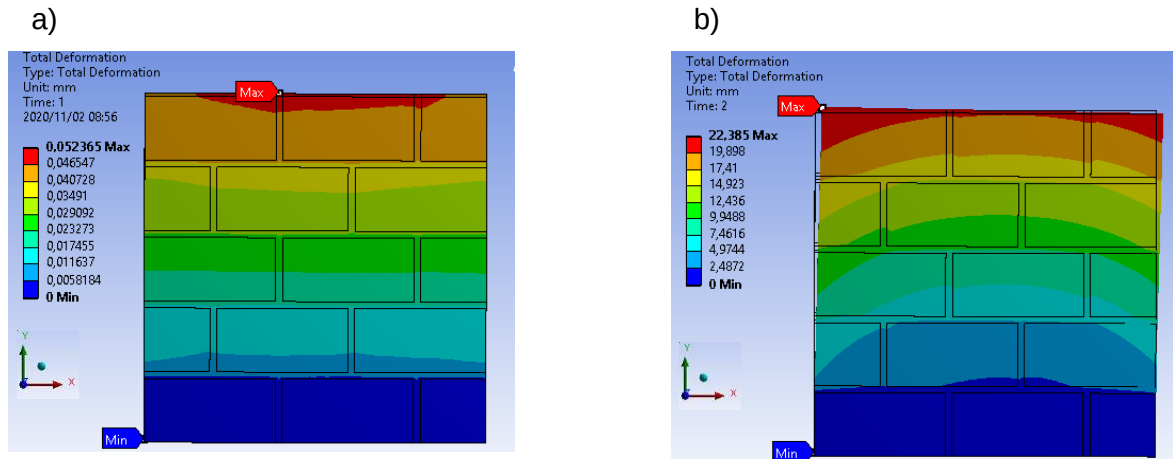


Figure 4-5: a) Total deformation after load step 1 and b) Total deformation after load step 2 (mm).

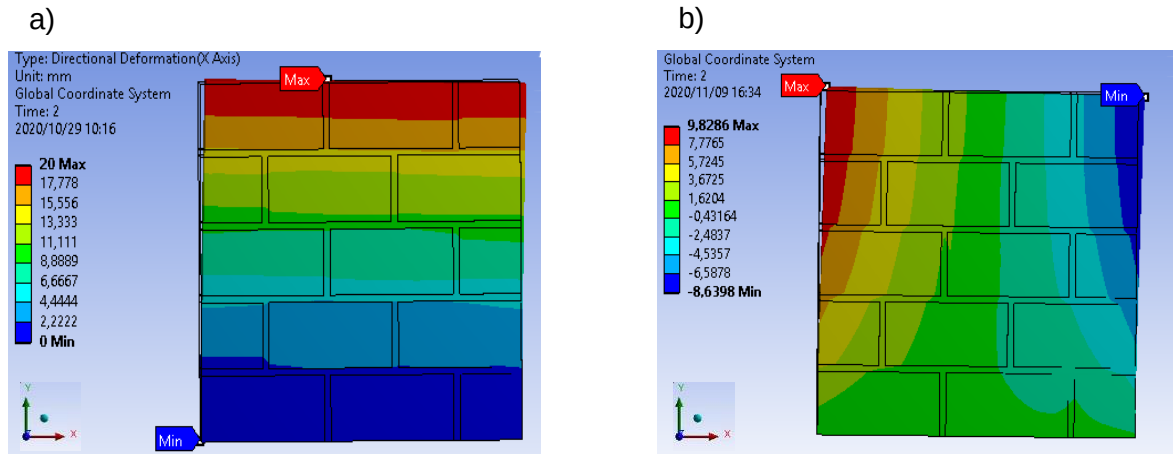


Figure 4-6: a) x-deformation and b) y-deformation after load step 2 (mm).

The second load step causes a significant and immediate increase in deformation as can be seen in Table 4-2. An immediate total deformation of 2.216 mm occurs as a result of the first sub-step of the horizontal displacement load. At the end of load step 2, a total deformation of 22.385 mm is attained, this can be seen in Figure 4-5b. The point of maximum total deformation occurs at the top left corner of the wall, this is to be expected and is qualitatively consistent with results from existing literature as shown in Figure 4-2. Much of the total deformation is as a result of the x-deformation, which can be seen in Figure 4-6a. The entire top layer of the wall undergoes a 20 mm movement in the x-direction at the end of load step 2. The total deformation at the bottom of the wall is close to zero,

this is expected as the wall is fixed at the bottom. Figure 4-6b illustrates the y-deformation, and it is noticed that the top left corner of the wall moves upwards in the y-direction with a significant maximum deformation of over 9 mm. A very small movement is noticed in the z-direction, this is because the static loads are imposed in the y and x direction respectfully.

Table 4-3: Stress and strain results for Numerical Experiment 1.

Load step	Time (s)	Equivalent Stress (MPa)	Shear Stress XZ (MPa)	Shear Stress XY (MPa)	Shear Stress ZY (MPa)	Equivalent plastic strain (mm/mm)
1 End	1	1,010	0.023	0.125	0.031	0.000
2 End	2	29.451	1.793	12.812	3.148	0.104

Table 4-3 presents the equivalent stress, shear stress and equivalent plastic strain results attained from Numerical Experiment 1. Just as in the case of deformation, load step 1 causes relatively small equivalent stress values to develop in the wall. The equivalent stress distribution after load step 1 is shown in Figure 4-7a, where an even distribution of stresses is noticed which could be attributed to the fact that the vertical load is a uniformly distributed load and the geometry is symmetrical to a degree. The highest stresses develop at the edges of the masonry units, this is also mentioned in (Berndt, 1996). It is interesting to note that relatively higher stresses develop in the horizontal layers of mortar as compared to the vertical layers of mortar.

The equivalent stress values drastically increase at load step 2 with the 20 mm horizontal displacement causing an immediate 22.915 MPa stress to develop. At the end of load step 2, a maximum equivalent stress of 29.45 MPa is attained. Figure 4-7b illustrates the equivalent stress distribution for Numerical Experiment 1 after load step 2. It is noticed that the highest stresses develop at the bottom right corner of the wall with high stress distribution forming across the wall in a diagonal manner. This is consistent with results found in (Drosopoulos & Stavroulakis, 2018) qualitatively, which shows that failure in masonry walls occurs at the bottom corner opposite to where the horizontal displacement load is applied and propagates diagonally upwards. As expected, the highest stresses develop in the masonry units and not the mortar.

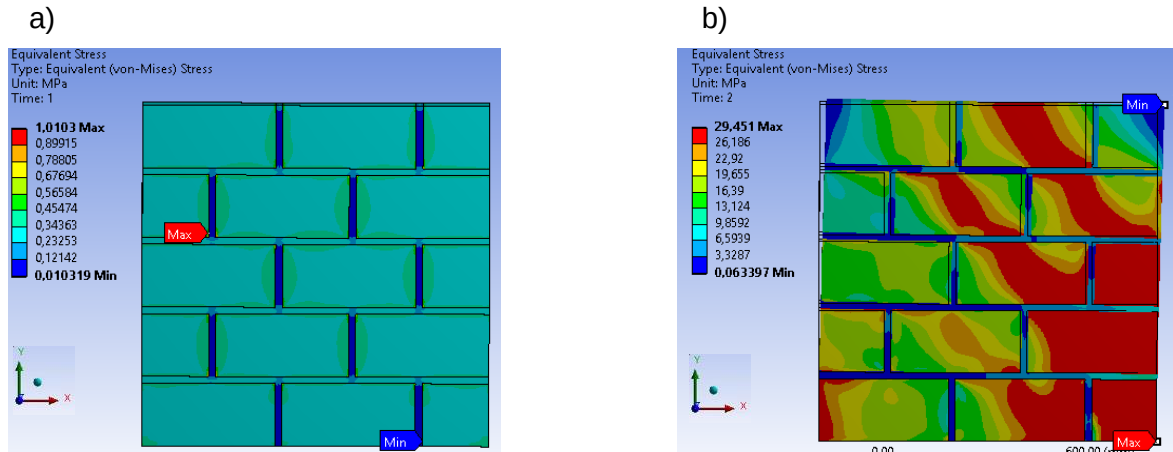


Figure 4-7: Equivalent stress (MPa) after a) load step 1 and b) load step 2.

The shear stresses are calculated across three different plane combinations as shown in Table 4-3. The plane that exhibits the dominant shear stress values is the XY plane. This is as a result of the applied static loads being applied in the y and x axis respectively. Figure 4-8a illustrates the shear stress distribution after load step 1. Relatively low shear stress values are seen after load step one, with the maximum shear stress values developing at the bottom course of brickwork. Shear stress values drastically increase at load step 2 as a results of the horizontal displacement load. At the end of load step 2 a maximum shear stress of 12.812 MPa is attained. Figure 4-8b shows the shear stress distribution after load step 2. The maximum shear stress after load step 2 is found at the same spot as the maximum shear stress after load step 1, this is a similar region to where the maximum equivalent stress also develops. It is noticed that higher shear stresses develop in the middle section of the wall, along the x-axis, with very low shear stresses at the outer edges.

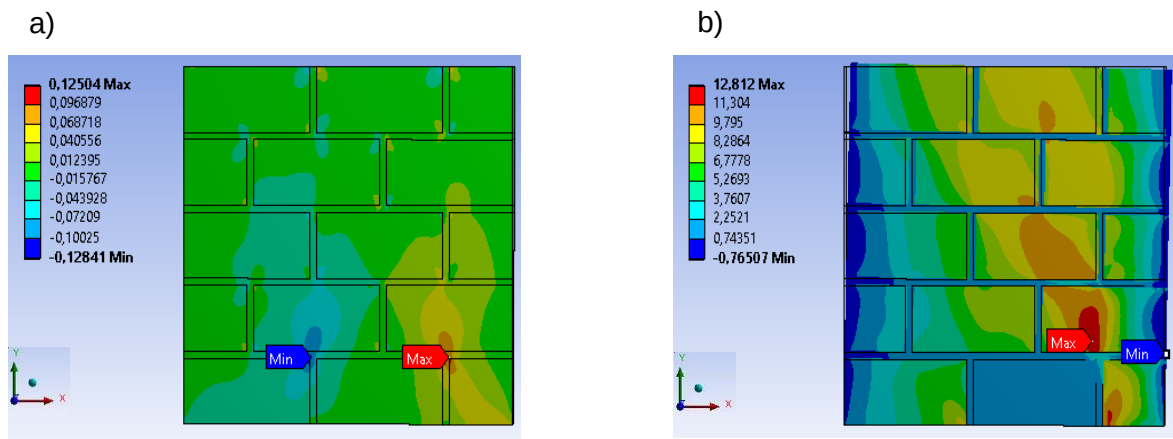


Figure 4-8: Shear stress (MPa) distribution in the XY plane after a) load step 1 and b) load step 2.

The equivalent plastic strain values attained for Numerical Experiment 1 are listed in Table 4-3 where it is evident to see that significant equivalent plastic strain values only develop after load step 2. Deformation is a measure of how much an object is stretched or moved, and strain is the ratio between the deformation and the original length (Kömürcü & Gedikli, 2019). Think of strain as percent elongation, how much bigger, or smaller, is the object upon loading. Plastic strain indicates that the structure will not return to its original shape after a certain yield load is reached. This is indicative of a non-linear analysis. The inclusion of the Drucker-Prager failure criteria accounts for the non-linear results attained for equivalent plastic strain. Therefore, plastic strain indicates permanent damage caused to the structure. Plastic strain damage is found predominantly in the mortar of the structure as shown in

Figure 4-9, this is as a result of the Drucker-Prager inputs of uniaxial compressive strength, uniaxial tensile strength and biaxial compressive strength being much lower for the mortar compared to that of the masonry unit. This is qualitatively consistent with results from previous research, as seen in Figure 4-1. The maximum equivalent plastic strain value is found in the bottom most layer of mortar on the same side that the horizontal displacement load is applied. It is interesting to note that relatively higher equivalent plastic strain values develop in the horizontal layers of mortar as compared to the vertical layers of mortar.

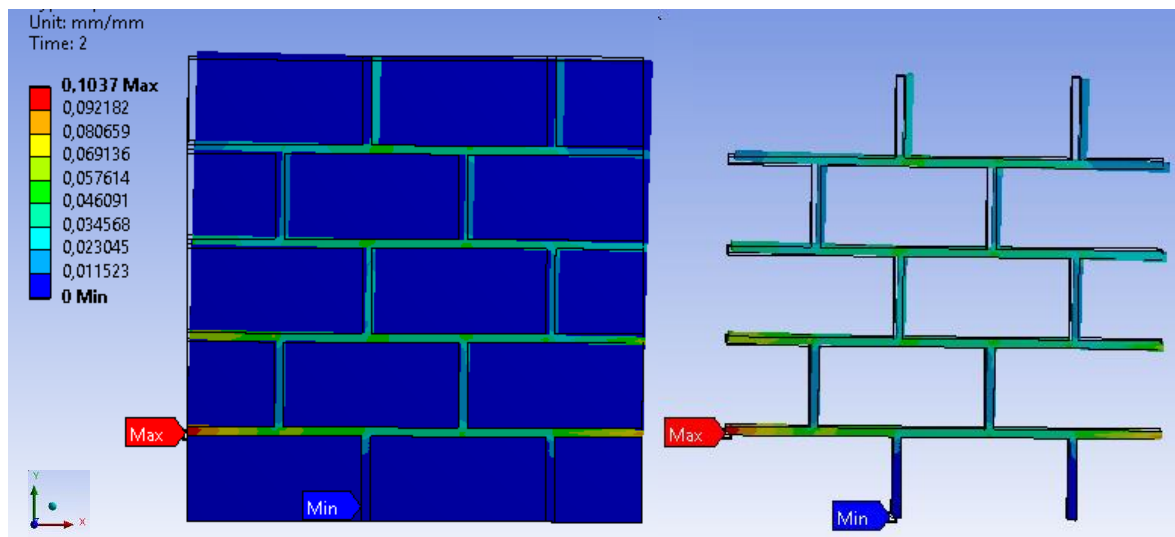


Figure 4-9: Equivalent plastic strain after load step 2 shown in full masonry wall and isolated mortar.

A horizontal reaction probe is created at the top left element of the masonry wall to get the horizontal reaction force. A structural deformation probe is also created at the same element in order to record the deformation and the movement of the wall along the X-axis corresponding to the horizontal reaction force. Using these two sets of data, a force vs displacement diagram is created, this can be seen in Figure 4-10.

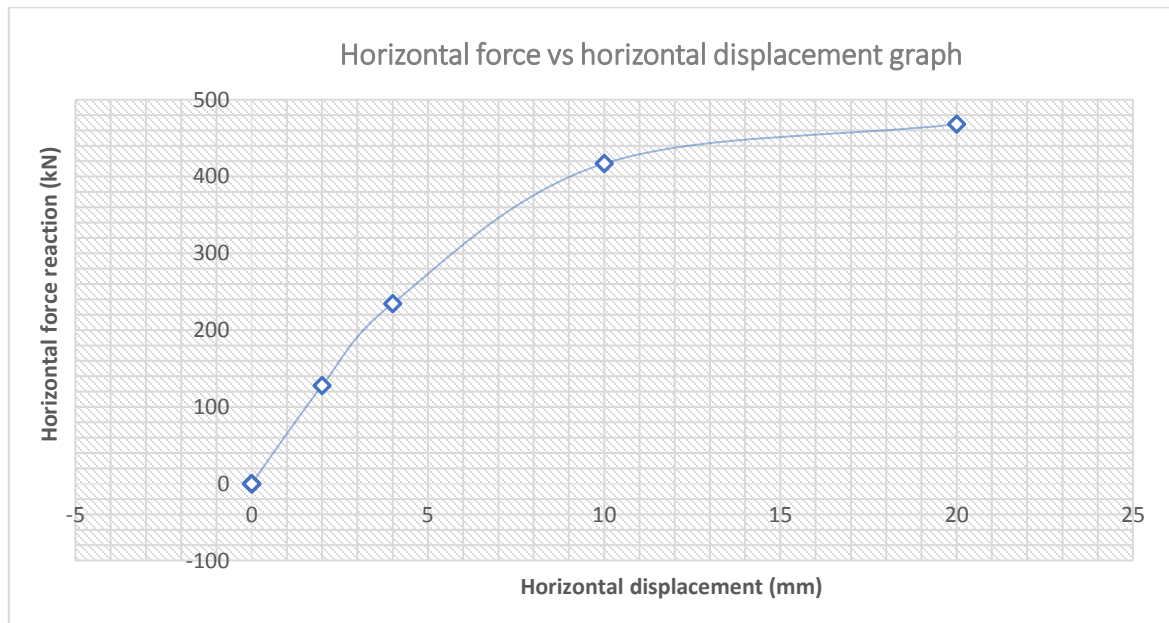


Figure 4-10: Force vs displacement diagram for Numerical Experiment 1.

The non-linear shape of the force-displacement curve depicts plastic failure. This shows the permanent damage caused to the structure. The Drucker-Prager failure criteria is added to the mortar and the masonry units to simulate this non-linear behavior. The shape of the force-displacement diagram is qualitatively consistent with previous studies, as shown in Figure 4-1b and Figure 4-3b. The failure of the structure occurs at around 200kN where the displacement is roughly 3 mm. Before 3 mm the graph is a straight line, indicating linear elastic behavior, once failure is reached, the displacement increases significantly while the reaction forces increase gradually. The maximum displacement is 20 mm and corresponds to a 468 kN force.

4.3 Numerical Experiment 2 - Static structural analysis of Geometry 2

Numerical Experiment 2 is a static structural analysis of Geometry 2. The static structural analysis is conducted in accordance with the methodologies presented in Chapter 3. The input data for Numerical Experiment 2 can be seen in Table 4-4. The deformations, stresses, strains and force vs displacement diagrams for Numerical Experiment 2 are presented and discussed in this chapter.

Table 4-4: Input table for Numerical Experiment 2.

Static structural analysis of low-cost house wall	
Geometry	2
Analysis type	Static structural
Contact	Bonded, MPC
Mesh size	20 mm
Mesh nodes	353568
Mesh elements	245434
Drucker-Prager	Yes
Analysis settings	2 load steps
Initial time step	0.1 s
Min time step	0.001 s
Max time step	1 s
Support	Fixed at bottom
Pressure load	0.3/0.3/0 MPa (@ time steps 0,1,2)
Displacement load	0/0/20 mm (@ time steps 0,1,2)

Table 4-5: Deformation results for Numerical Experiment 2.

Load step	Time (s)	Total Deformation (mm)	X Deformation (mm)	Y Deformation (mm)	Z Deformation (mm)
1 End	1	1.586	0.194	1.243	0.400
2 End	2	25.429	21.572	12.780	4.200

Table 4-5 presents the deformation results attained from Numerical Experiment 2. These are the maximum deformation values. Just as in Numerical Experiment 1, the deformations are given in terms of x, y and z values according to the structures' movement along the respective axis as well as the total deformation. The imposed loading and load steps for Numerical Experiment 2 can be seen in Chapter 3.2.6.2 and Chapter 3.2.6.3 respectively.

The deformations after load step 1 are quite similar to that found in Numerical Experiment 1, with a relatively small total deformation of 1.586 mm. Much of the deformation after load step 1 is found in the y-direction.

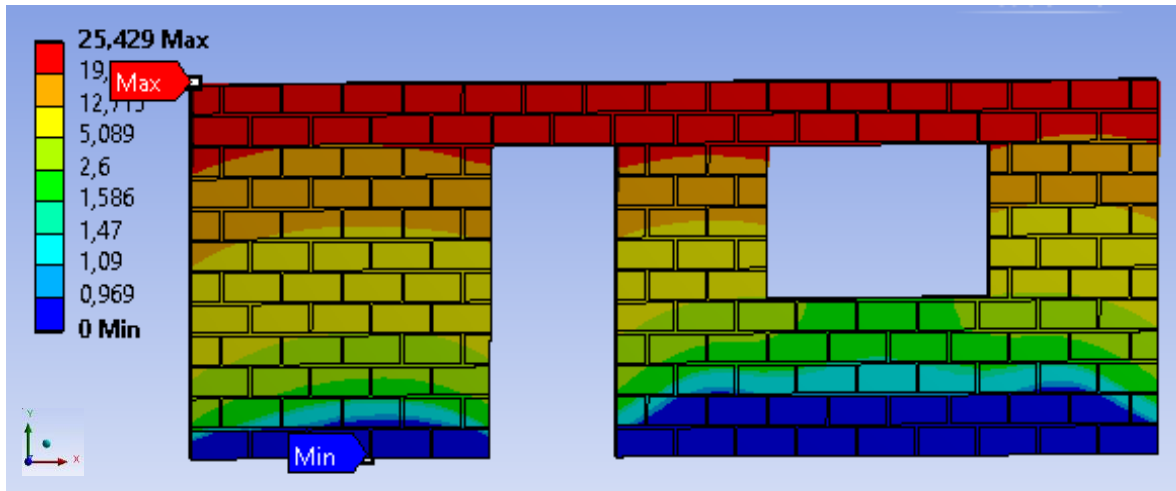


Figure 4-11: Total deformation for Numerical Experiment 2 after load step 2 (mm).

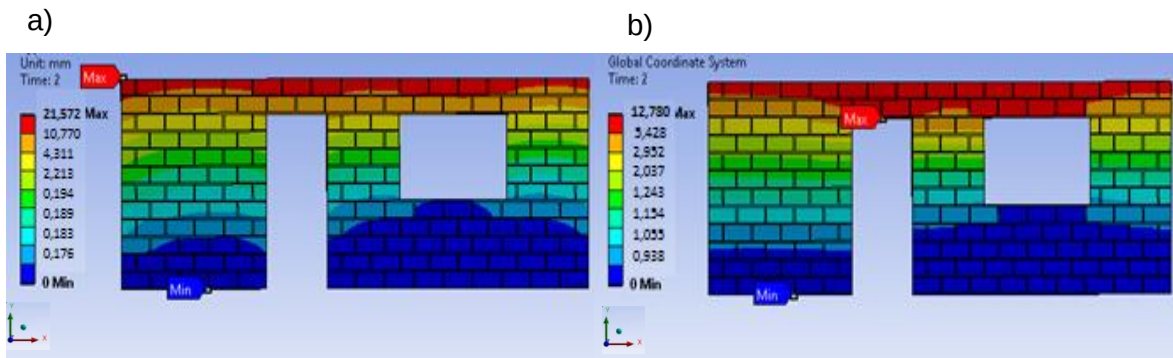


Figure 4-12: a) x-deformation and b) y-deformation after load step 2 (mm).

Figure 4-11 illustrates the distribution of total deformations after load step 2, where it is clear to see that the highest deformations occur at the top of the wall. The maximum total deformation of the wall is 25.429 mm and is found at the top left corner of the wall, the point at which the horizontal displacement load is applied. Other important points of deformation to note are regions above the door and window opening. These regions are often weak points in masonry buildings and require additional reinforcement and support which can be provided by lintel beams.

Figure 4-12a illustrates the x-deformation after load step 2. The top course of brickwork experiences the largest deformation in the x direction. Deformation in the x-direction decreases down the wall as it approaches the fixed support at the bottom. The y-deformation after load step 2 is illustrated in Figure 4-12b. The maximum y-deformation occurs just above the door opening. Large y-deformations can be seen across the top of the wall and especially above the window opening. The z-deformation, as listed in Table

4-5 has a maximum value of 4.200 mm, this makes up a much larger proportion of the total deformation as compared to Numerical Experiment 1.

Table 4-6: Stress and strain results for Numerical Experiment 1

Load step	Time (s)	Equivalent Stress (MPa)	Shear Stress XZ (MPa)	Shear Stress XY (MPa)	Shear Stress ZY (MPa)	Equivalent plastic strain (mm/mm)
1 End	1	2.442	0.63206	1.435	0.22647	0.004
2 End	2	27.595	0.81401	12.094	1.1768	0.054

Table 4-6 shows the equivalent stress, shear stress and plastic strain results from Numerical Experiment 2. It is interesting to see that the vertical compressive load applied in load step 1 results in an immediate 2,293 kN equivalent stress to develop in the wall. This stress does not increase by much for the rest of load step 1. The equivalent stress distribution after load step one can be seen in Figure 4-13. The maximum equivalent stress due to load step 1 occurs at the bottom corner of the window opening, this is a common region of weakness. Generally, higher stresses develop at the bottom of the wall as opposed to the top.

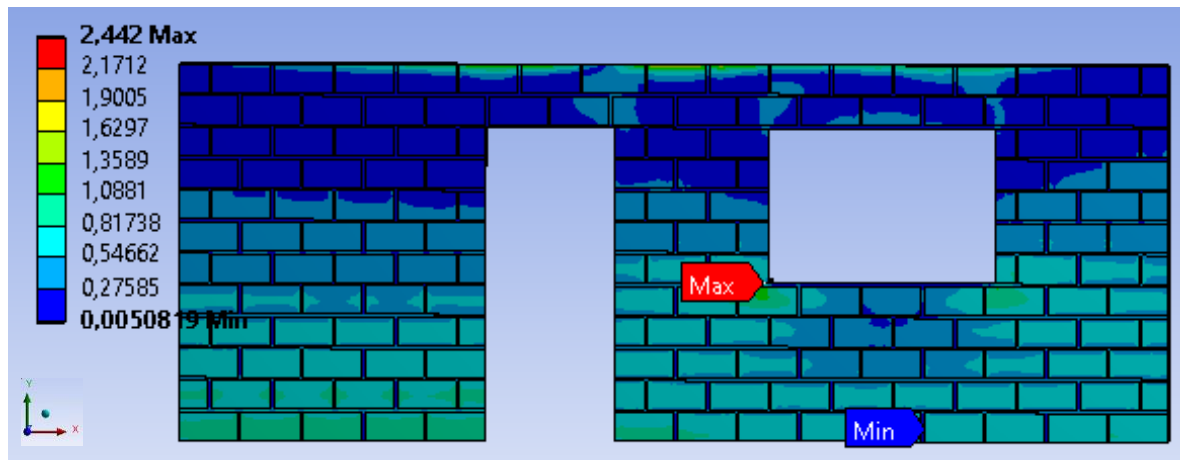


Figure 4-13: Equivalent stress (MPa) distribution after load step 1.

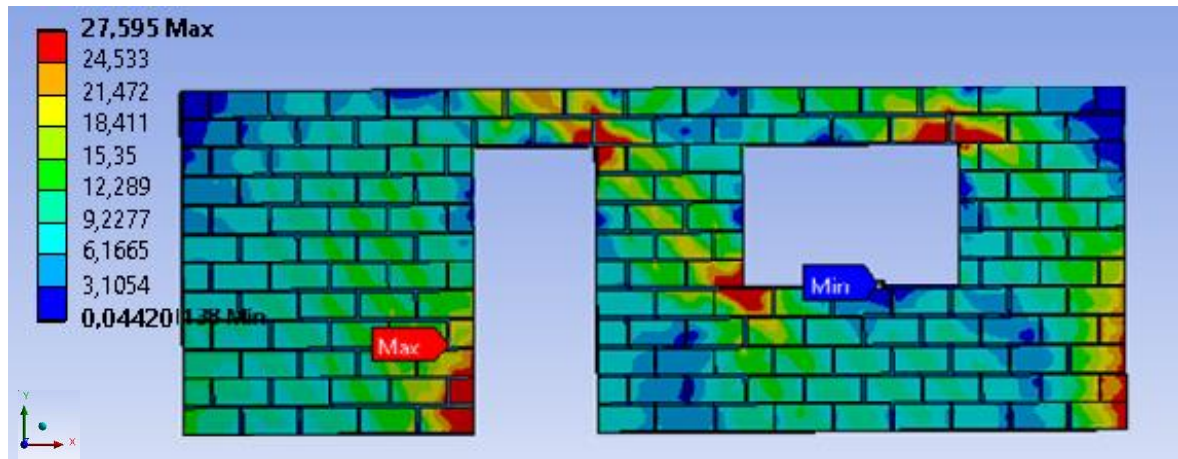


Figure 4-14: Equivalent stress (MPa) distribution after load step 2.

Load step 2 causes the equivalent stress values in the wall to drastically increase, resulting in a maximum equivalent stress value of 27.595 kN. The maximum equivalent stress occurs at the bottom left side of the door opening. Figure 4-14 shows that other regions of high stress include, the top right corner of the door opening, the bottom left and top right corner of the window opening and the bottom right corner of the wall. These are all common areas of failure in masonry walls. It is interesting to note the diagonal nature in which the stresses are distributed across the wall, the yellow and green shades illustrate this and is in line with results from Numerical Experiment 1 as well as results from previous studies, as seen in Figure 4-2 and Figure 4-3. The highest equivalent stress values develop in the the masonry units.

The shear stresses for Numerical Experiment 2 are calculated across three different plane combinations as shown Table 4-6. The plane that exhibits the greatest shear stresses is the XY plane, just as in Numerical Experiment 1, this is as a result of the applied static loads being applied in the y and x axis respectively. Minimal shear stresses develop as a result of the uniformly distributed pressure loading during load step 1. Much more significant shear stresses develop during load step 2. The shear stress distribution in the XY plane, after load step 2 is shown in Figure 4-15. The wall experiences a maximum shear stress of 12.094 kN which is found at the bottom left corner of the window opening. High shear stress values are also found above the door opening and the region between the door and window. Relatively higher shear stresses can be seen in the middle parts of the wall, with low shear stress values found at the sides.

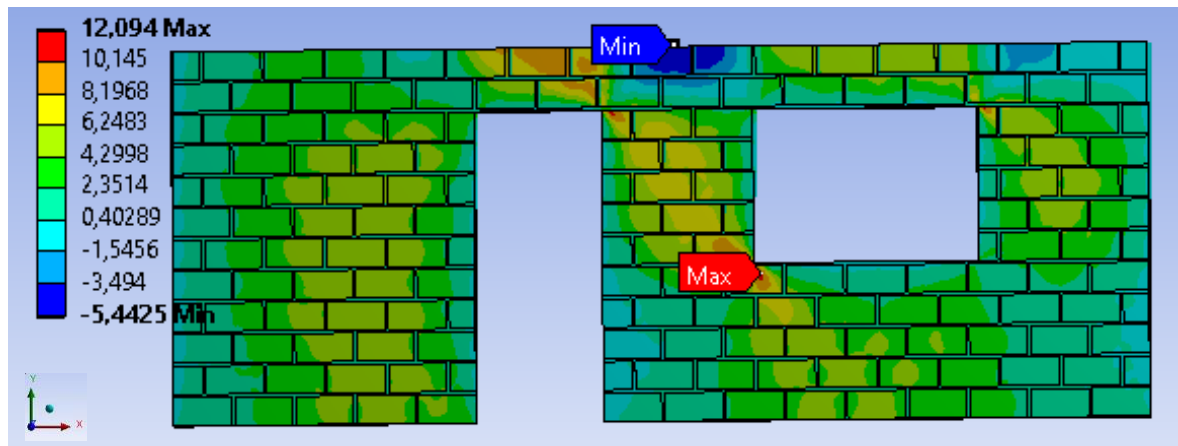


Figure 4-15: Shear stress (MPa) distribution in the XY plane after load step 2.

The equivalent plastic strain values attained for Numerical Experiment 2 can be seen in Table 4-6. Just as in Numerical Experiment 1, it is evident to see that significant equivalent plastic strain values only develop after load step 2. The occurrence of plastic strain indicates permanent damage, the structure will not return to its original shape thus illustrating a non-linear analysis. Just as in Numerical Experiment 1, the non-linear behavior of the wall can be attributed to the inclusion of the Drucker-Prager failure criteria. Plastic strain damage is found predominantly in the mortar of the structure as shown in Figure 4-16. This is qualitatively consistent with results from Numerical Experiment 1 and results from previous research, as seen in Figure 4 1a. Four zones, numbered 1-4, of importance are circled in Figure 4-16 which point out regions of high strain. Figure 4-17 shows the four zones of high strain in exploded views.

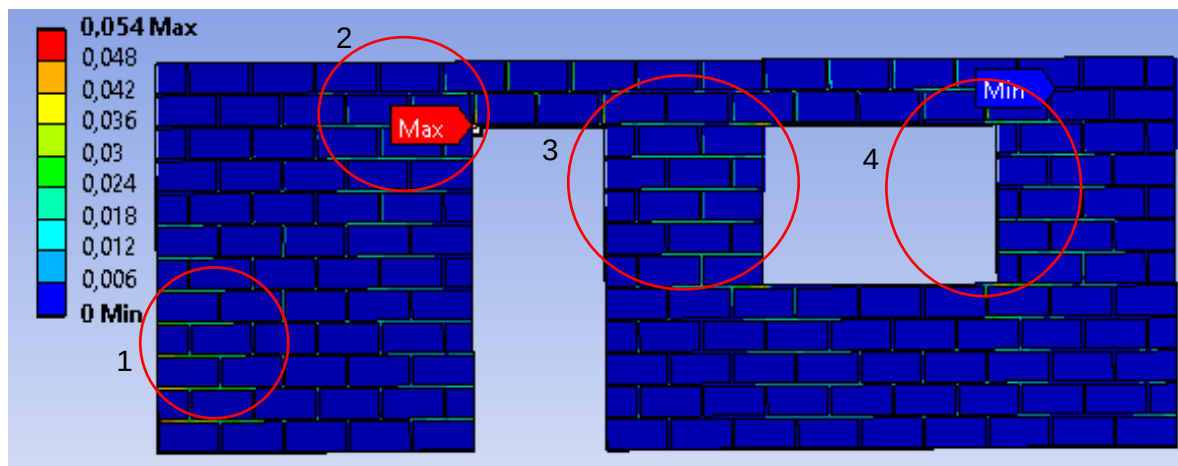


Figure 4-16: Equivalent plastic strain after load step 2.

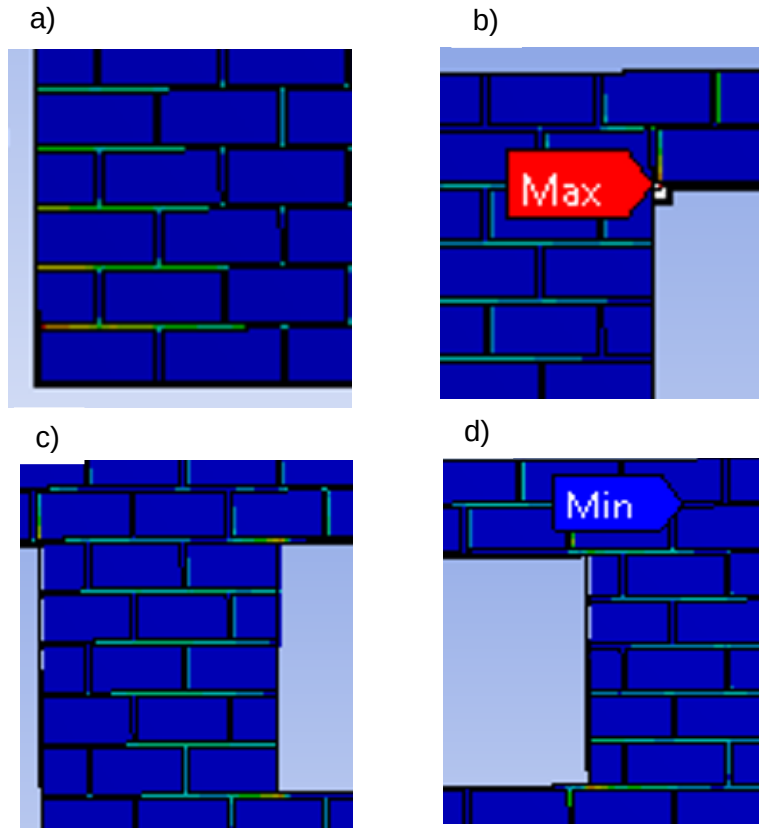


Figure 4-17: Exploded view of equivalent plastic strain from Figure 4-16. a) zone 1. b) zone 2. c) zone 3. d) zone 4

Zone 1 represents the bottom left corner of the wall; the exploded view can be seen in Figure 4-17a. The maximum plastic strain in zone 1 is found at the bottom most layer of mortar, high plastic strain values are found in this region for Numerical Experiment 1 as well. The overall maximum equivalent plastic strain for Numerical Experiment 2 has a value of 0,054 mm/mm and is found in zone 2, at the left corner on the door opening as seen in Figure 4-16 and Figure 4-17b. Another region of high strain is zone 3, which includes the section of wall between the door and window opening and the top left and bottom left corner of the window opening. The maximum plastic strain values for zone 3 are found at the top and bottom left corner of the window opening, this can be seen in Figure 4-17c. The last region of high strain is zone 4, which includes the top and bottom right corner of the window opening. The exploded view of zone 4 can be seen in Figure 4-17d. The maximum plastic strain in zone 4 occurs at the bottom right corner of the window opening. From this analysis it is clear to see the regions that undergo high strain. It is interesting to note that much higher plastic strain values occur in the horizontal layers of the mortar compared to the vertical layers, this is also observed Numerical Experiment 1.

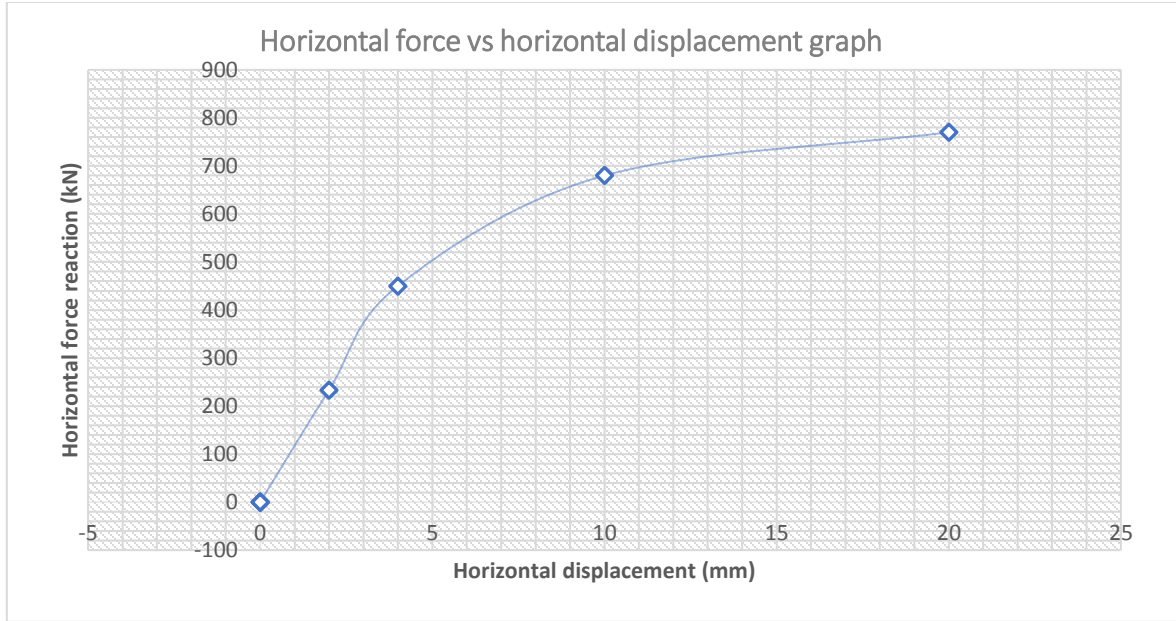


Figure 4-18: : Force vs displacement diagram for Numerical Experiment 2..

Just as in Numerical Experiment 1, a horizontal reaction probe and a structural deformation probe are created at the top left element of the masonry wall to get the horizontal reaction force and record the movement of the structure along the x-axis. Using these two sets of data, a horizontal reaction force vs displacement diagram is created for Numerical Experiment 2, which can be seen in Figure 4-18.

The non-linear shape of the force-displacement graph depicts plastic failure. This indicates permanent damage caused to the structure. The Drucker-Prager failure criteria is added to the mortar and masonry units to simulate this non-linear behavior. The shape of the force-displacement diagram is qualitatively consistent with that of Numerical Experiment 1 and those found in previous studies as shown in Figure 4-1b and Figure 4-3b. The initial damage of the structure occurs at around 350kN, this is where the force vs displacement diagram stops being linear. The corresponding displacement to this point is roughly 3 mm. Before 3 mm the graph is a straight line, indicating linear elastic behavior, once failure is reached, the displacement increases significantly while the reaction forces increase gradually. The maximum displacement is 20 mm and corresponds to a 770 kN horizontal reaction force. Although the shape of the force vs displacement diagram is similar for both Numerical Experiment 1 and 2, Numerical Experiment 2 shows higher horizontal reaction forces corresponding to the horizontal displacement.

4.4 Numerical Experiment 3 - Dynamic analysis of Geometry 2

The dynamic analysis consists of two parts: A Modal analysis and a Response Spectrum analysis which are presented in Chapter 4.4.1 and Chapter 4.4.2 respectfully. The Modal analysis is a linear analysis and ignores all non-linearity in the material properties and connections. The response spectrum analysis is connected to the Modal analysis and as such, also ignores all non-linearity.

4.4.1 Modal analysis Geometry 2

Table 4-7: Summary of results for Modal analysis.

Mode	Frequency (Hz)	Max Total Def (mm)	Max X Def (mm)	Max Y Def (mm)	Max Z Def (mm)	Equivalent Stress (MPa)
1	6.308	1.129	0.005	0.046	1.128	1.249
2	8.740	1.727	0.045	0.064	1.726	2.249
3	14.689	1.440	0.085	0.056	1.438	3.444
4	24.381	1.669	0.126	0.064	1.664	10.663
5	35.926	1.630	0.094	0.146	1.624	11.595
6	40.223	1.966	0.078	0.193	1.955	10.375

Table 4-7 summarizes the results attained from the Modal analysis performed on Geometry 2 by tabulating the frequency, maximum total deformation, maximum deformation in the x-direction, maximum deformation in the y-direction, maximum deformation in the z-direction and equivalent stress related to each Mode. It is evident from Table 4-7 that a large proportion of the total deformation is a result of the deformation in the z-direction, in fact the deformation in the x and y-direction can be considered negligible. A higher frequency does not necessarily result in a greater deformation, as each Mode oscillates in a different manner. When analyzing walls using Modal analysis, the wall behaves like a bending slab oscillating about a fixed point as shown in Figure 4-19. The dynamic response is dependent on the structures' stiffness and weight.

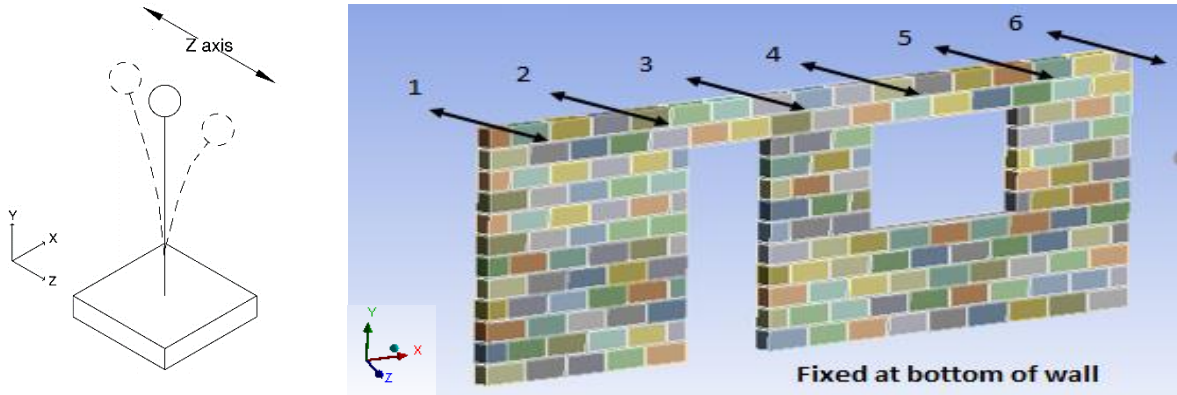


Figure 4-19: Modal oscillation in z-axis.

The deformation results shown in Figure 4-20 to Figure 4-25 have been exploded by a factor of 240 to illustrate the type of oscillation undergone by each Mode. The wall is fixed at the bottom as shown in Figure 4-19 and oscillates about the fixed support.

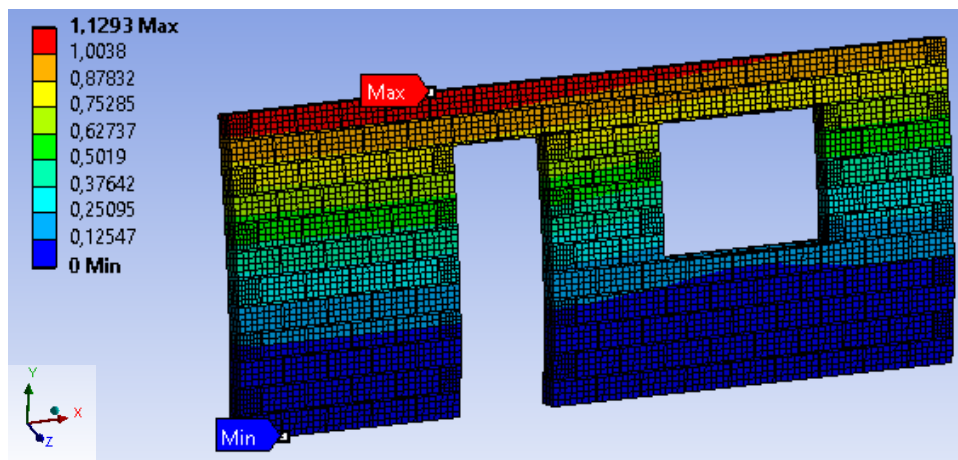


Figure 4-20: Total deformation for Mode 1 (mm).

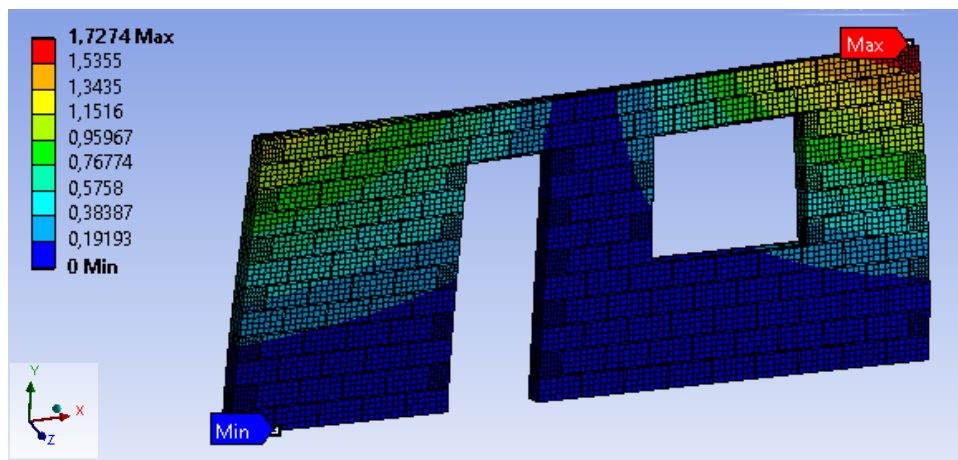


Figure 4-21: Total deformation for Mode 2 (mm).

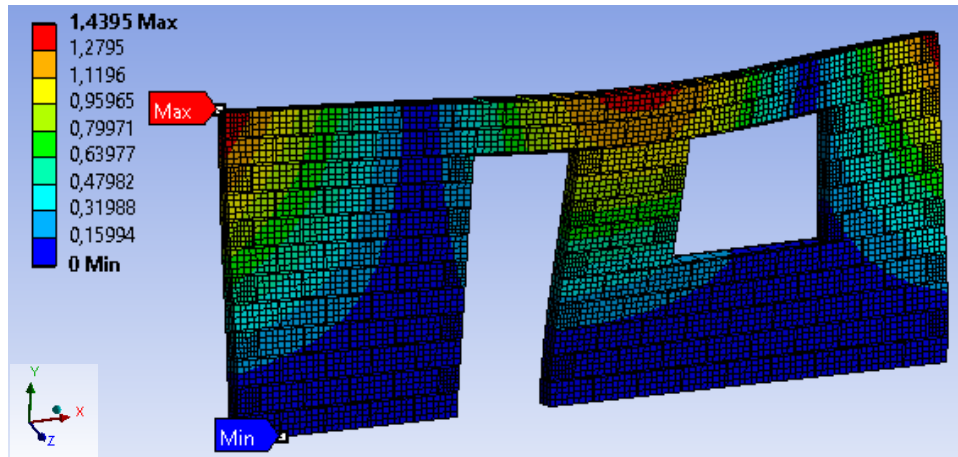


Figure 4-22: Total deformation for Mode 3 (mm).

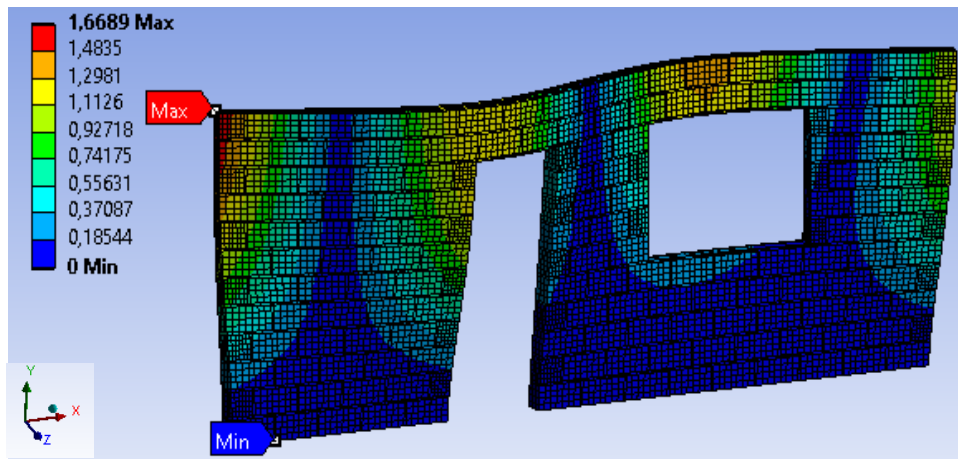


Figure 4-23: Total deformation for Mode 4 (mm)..

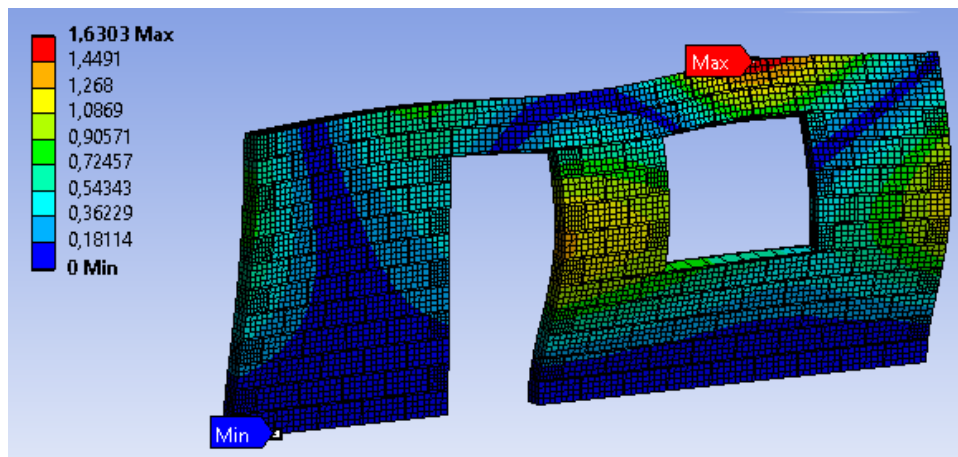


Figure 4-24: Total deformation for Mode 5 (mm).

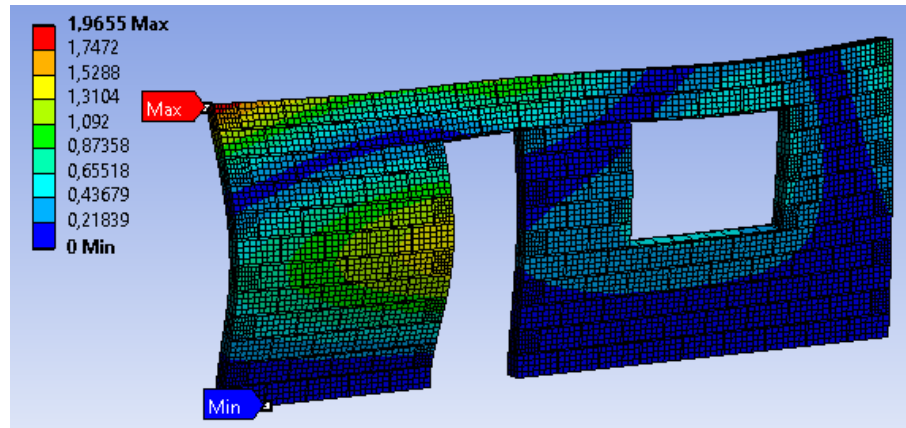


Figure 4-25: Total deformation Mode 6 (mm).

Figure 4-26 points out the regions of maximum stress and deformation of 6 different Modes after a Modal analysis. The frequency of each mode is given in Table 4-7 while the deformation pattern and the way in which each mode resonates can be seen in Figure 4-20 to Figure 4-25.

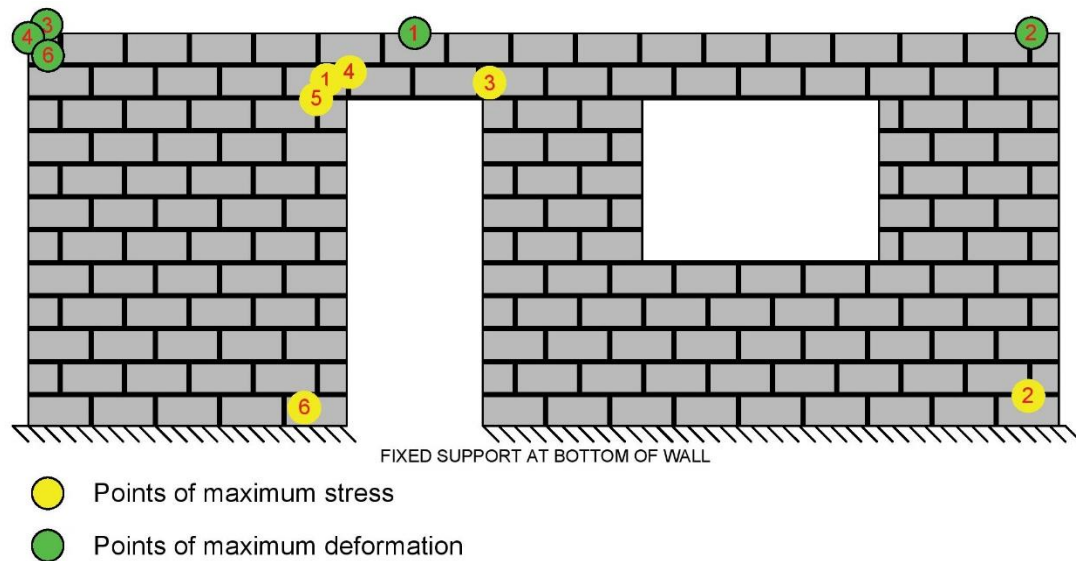


Figure 4-26: Points of high stress and deformation for each Mode 1-6 of Modal analysis.

In Figure 4-26 the points of maximum stress for each Mode are shown in yellow. Mode 1, 4 and 5 all show maximum equivalent stress values in the top left corner of the door opening, while for Mode 3 the maximum equivalent stress is found on the top right corner of the door opening. For Mode 2 and 6, the maximum equivalent stresses are found at the bottom of the wall. Mode 2 gives high stresses at the bottom right corner of the wall while Mode 6 indicates a build-up of high stresses at the bottom left corner of the door opening. On closer inspection it is found that the maximum equivalent stress always occurs on the edges of the

masonry units. This is in good agreement with results from previous experiments and damage found in actual low-cost housing in South Africa where damage is often found above door and window openings as well as the bottom corners of these openings. Damage is also expected to develop at the bottom corners of the masonry wall as seen in Figure 4-4. Considering that the wall is fixed at the bottom and oscillates at different frequencies around the fixed support, maximum deformation can be expected at the points furthest away from the fixed support. This is proven in Figure 4-26 with the maximum deformation for all Modes found at the top layer of the wall. The points of which maximum deformation in relation to each Mode are shown in green in Figure 4-26.

4.4.2 Response spectrum analysis of Geometry 2

The Response Spectrum analysis combines all six Modes from Chapter 4.4.1 together with response acceleration applied at the fixed support at the bottom of the wall in all three directions. The acceleration values are derived in Chapter 3.3.2. Table 4-8 and Table 4-9 contain the deformation and stress results from the Response Spectrum analysis respectively. Unlike the static analysis, there are no load steps for the dynamic analysis, only the final results are presented.

Table 4-8: Deformation results from Response Spectrum Analysis.

Max Total Deformation (mm)	Max X Deformation (mm)	Max Y Deformation (mm)	Max Z Deformation (mm)
9.878	0.049	0.399	9.870

Table 4-9: Stress results from Response Spectrum analysis.

Max Equivalent Stress (MPa)	Max Shear Stress XZ (MPa)	Max Shear Stress XY (MPa)	Max Shear Stress ZY (MPa)
3.455	1.642	1.698	1.232

Figure 4-27 illustrates the total deformation from the Response Spectrum analysis. It is clear to see that higher deformations occur at the top of the wall, and the deformations decrease closer to the fixed support at the bottom. The maximum total deformation is 9.877 mm and is found at the top layer of brickwork. Much of this deformation is due to movement in the z-direction and can be seen in Figure 4-8. Relatively high deformations to the value of 7-8 mm occur above the door and window opening.

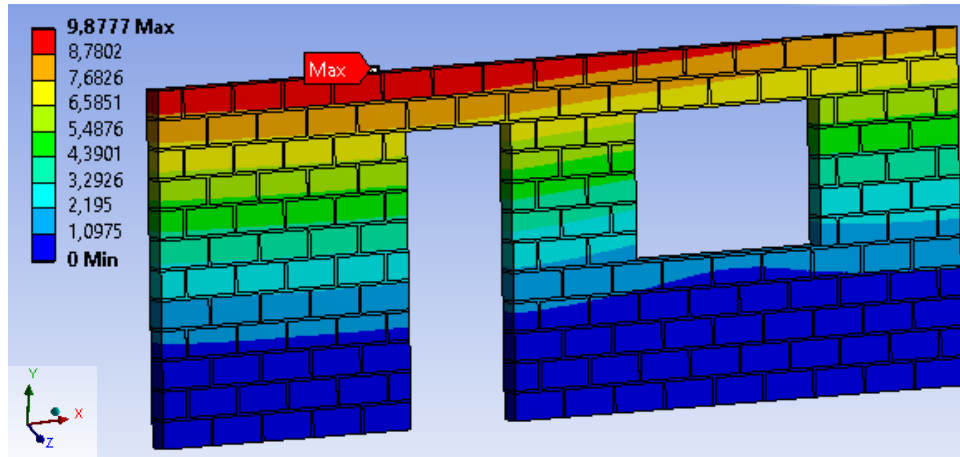


Figure 4-27: Total deformation (mm) after Response Spectrum analysis.

Figure 4-28 illustrates the equivalent stress distribution after the Response Spectrum analysis. It is noticed that high stresses are found at the bottom of the wall, with almost negligible equivalent stress values found at the top and near the top of the wall. The maximum equivalent stress has a value of 3.455 MPa and is found at the bottom of the wall, close to the door opening. Relatively high equivalent stress values are also found at the bottom corners of the window opening, both bottom corners of the door opening and the bottom corners of the wall. The left bottom side of the wall displays a large region of relatively high stresses. Unlike the static analysis, the top corners of the door and window opening do not exhibit high stresses.

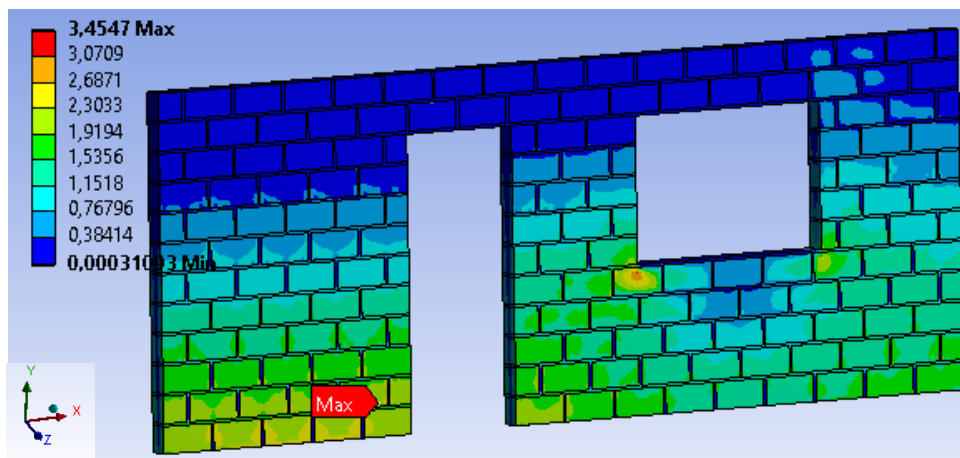


Figure 4-28: Equivalent stress (MPa) after Response Spectrum analysis.

Shear stresses are calculated across the XZ, XY and ZY axis as shown in Table 4-9. The XY axis produces the highest shear stresses, closely followed by the XZ axis. Figure 4-29

illustrates the shear stress distribution after the Response Spectrum analysis in the XY plane. Most of the wall exhibits minimal shear stresses in the XY plane, however large shear stresses develop at the bottom corners of the window opening. The maximum shear stress occurs at the bottom left corner of the window opening and has a value of 1.698 MPa

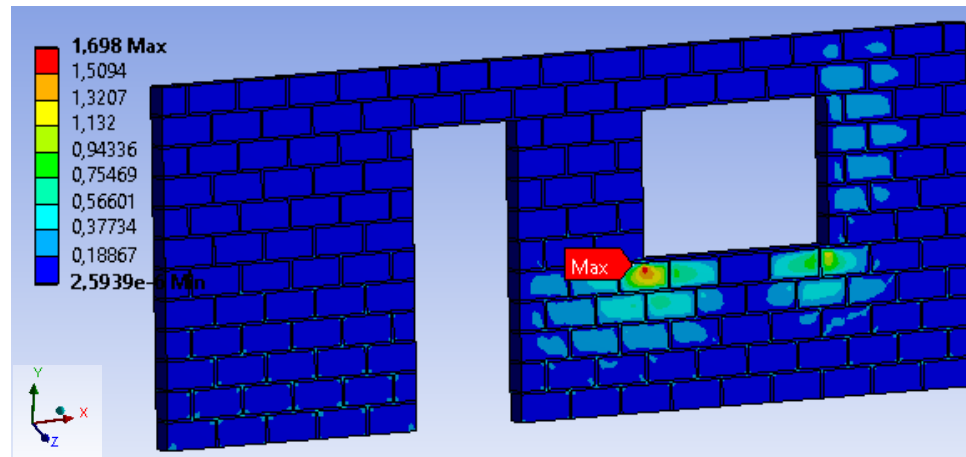


Figure 4-29: Shear stress (MPa) in XY plane after Response Spectrum analysis.

It is evident that the results generated from the dynamic analysis show significantly smaller, stresses and deformations as compared to the static structural analysis using the same geometry as seen in Numerical Experiment 2. The static loads used in Numerical Experiment 1 and 2 are exaggerated loads, especially the 20 mm horizontal displacement, which is aimed to bring a structure to the point of failure and beyond. The Response Spectrum analysis conducted in Numerical Experiment 3 uses historical data, specific to a region, to generate acceleration loads that are applied to the bottom of the wall. South Africa is not considered a region of high seismic activity, as such, the historical data used to create the response spectra loading consists of relatively small values. The Modal analysis is also limited to 6 Modes, a greater number of Modes would result in higher natural frequencies and increased impact on the structure.

Chapter 5 – Conclusion and Recommendations

5.1 Conclusion

Despite significant advances in numerical modeling techniques and the finite element method, numerical modeling involving masonry structures remains challenging. The heterogenous composition, the non-linear behavior, the various planes of failure which can occur along bed joints, the low-tension strength and the fragile failure characteristics of masonry make it a complicated structural material to model. This study involves creating and testing computational models that simulate the behavior of unreinforced masonry walls under static and dynamic loading. The finite element method together with ANSYS software are used to create and analyze the numerical models. A 1-meter square, unreinforced masonry wall (Geometry 1) and a full-scale wall with door and window openings, representing that of low-cost houses found across South Africa (Geometry 2) are developed and evaluated under general static and dynamic loading conditions. Material non-linearity is incorporated into the static analysis of both geometries with the inclusion of the Drucker-Prager failure criteria. Non-linear effects are ignored for the dynamic analysis. Two types of loads are applied in two load steps for the static analysis. Load step one is a uniformly distributed vertical pressure loading along the top layer of the wall, acting downward and normal to the surface with a magnitude of 0.3 MPa and load step 2 is a horizontal displacement load acting from left to right along the top layer of the wall with a magnitude of 20 mm. The dynamic analysis is performed using both Modal analysis and Response Spectrum analysis.

5.1.1 Numerical Experiment 1

The results attained from Numerical Experiment 1 show that most of the damage to the 1-meter square masonry wall is caused during load step 2, the horizontal displacement load. The vertical pressure load in load step 1 causes relatively small deformations, stresses and strains to develop. Most of the total deformation occurs in the x-axis. High stresses are shown to propagate diagonally, starting from the bottom corner of the wall. Relatively high shear stress values develop in the XY-plane. Equivalent plastic strain develops in the mortar. The non-linear behavior of masonry is proven through damage showing plastic strain and the shape of the force vs displacement diagram. Maximum plastic strain develops in the bottom right corner of the wall.

5.1.2 Numerical Experiment 2

Like Numerical Experiment 1, the results attained for Numerical Experiment 2 indicate that most of the damage caused by the static load occurs after load step 2, the horizontal displacement loading. Majority of the deformation occurs along the x-axis, though Numerical Experiment 2 shows a much higher proportion of deformation in the y-direction as compared to Numerical Experiment 1. Areas of high deformation include the entire top course of the wall and regions above the door and window opening. Areas of high equivalent stress include the bottom left and top right corner of the door and window opening and the bottom right corner of the wall. Relatively high shear stresses develop in the XY plane, above the door opening and across the middle of the wall. The non-linear behavior of masonry is proven through damage showing plastic strain and the shape of the force vs displacement diagram. Four regions of high plastic strain are identified.

5.1.3 Numerical Experiment 3

Numerical Experiment 3 is a dynamic analysis of Geometry 2, consisting of Modal and Response Spectrum analysis. The Modal analysis is limited to 6 Modes with natural frequencies ranging from approximately 6Hz to 40 Hz. Results from the Modal analysis identify areas of high stresses and deformations in the low-cost house wall. High deformations occur at the top of the wall, in the z-direction while relatively high stresses develop at the top corners of the door opening, the bottom left corner of the door opening and the bottom right corner of the wall. The Response Spectrum analysis is done by creating a design response spectra graph using (SANS 10160-4, 2009). The results from the Response Spectrum analysis show relatively high deformations at the top of the wall. The highest equivalent stresses and shear stresses are found at the bottom corner of the window opening. The results attained from the Response Spectrum analysis indicate that it is not the crucial load factor compared to the static structural loading.

5.1.4 Summary of results

The results compare well in a qualitative sense with existing literature found in (Agüera, et al., 2016), (Drosopoulos & Stavroulakis, 2018), (Elvin, 2009), (Khoyratty, 2016) and (Kömürcü & Gedikli, 2019). In the case of the 1-meter square wall, high stress concentrations are found at the bottom corner of the wall, indicating flexural failure as mentioned in (Tomažević, 2016). These high stresses are shown to propagate upwards in a diagonal manner, indicating possible shear failure which is also mentioned in (Tomažević,

2016). Considering the static structural analysis for Geometry 2 (low-cost house wall), high stress concentrations are found around the door and window openings, similar findings are shown in (Drosopoulos & Stavroulakis, 2018) numerically and by field inspection in (Khoiratty, 2016). High stresses are also found at the bottom corner of the wall and propagate diagonally upwards, this is also mentioned in (Drosopoulos & Stavroulakis, 2018). For both geometries, plastic strain develops predominately in the mortar under static structural loading, with high plastic strain developing in the bottom most layer of mortar for Geometry 1, similar results are found in (Agüera, et al., 2016) qualitatively. Four zones of high plastic strain are observed for Geometry 2. These zones include regions at the bottom of the wall, above and around window and door openings and sections of the wall between the door and window opening.

The dynamic analysis is performed using both Modal analysis and Response Spectrum analysis. Results from the modal analysis indicate that high deformations occur at the top of the wall while relatively high stresses develop at the top corners of the door opening, the bottom left corner of the door opening and the bottom right corner of the wall. These problem areas are consistent with evidence shown in (Khoiratty, 2016). The Response Spectrum analysis is connected to the Modal analysis. The results from the Response Spectrum analysis show relatively high deformations at the top of the wall. The highest equivalent stresses and shear stresses are found at the bottom corner of the window opening. The results generated from the dynamic analysis in Numerical Experiment 3 show smaller deformations and stresses when compared to the static analysis in Numerical Experiment 2. It should be noted that the static loads used in Numerical Experiment 2 are exaggerated loads used to bring the structure to failure, whereas the Response Spectrum analysis uses realistic values based upon historical seismic data, which is region dependent. Therefore, in this study, the dynamic load is not the crucial load factor.

6.1.5 Propositions for improvement of design

Major points of weakness in the static and dynamic analysis of Geometry 2 include regions above and around the door and window openings. Reinforcement lintel beams placed above door and window openings will counter large stresses and stop large deformations from developing around these openings. Steel lintel beams offer the greatest strength but are not of a suitable cost to be used in low-cost housing. Timber lintel beams offer good affordability but are susceptible to environmental conditions. Reinforced concrete lintels would be the ideal reinforcement to be used above door and window openings, considering

durability, strength and cost (Sattar, 2014). Bed-joint reinforcement could be incorporated to provide additional resistance to lateral loads. To reduce cost, bed-joint reinforcement can be added to every third or fourth course. Steel reinforcement can be added at key areas to increase ductility and resistance to flexural forces. These key areas include the bottom corners of the wall and the center panel between the door and window opening in Geometry 2.

6.2 Recommendations for future work

- Develop a larger, full scale model representing four walls, however this would require very large computer capacity.
- Reduce the size of the mesh to generate more accurate results.
- Incorporate non-linear frictional contact into the model. This will allow for the separation of the masonry units and the mortar to be seen.
- Perform a transient structural dynamic analysis. A non-linear dynamic analysis will give more accurate results as to how an unreinforced masonry wall will react to an earthquake loading.
- Include long-term effects on masonry such as creep.
- Cyclic loading could be incorporated into the model, which could be used to conduct a fatigue assessment of masonry walls. This requires advanced versions of structural software analysis programs.
- Include a reinforced concrete lintel above doors and windows in the model to tests its effectiveness in reducing failure above door and window openings.

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