Susceptibility of low cost housing to seismic activity in South Africa

by
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ABSTRACT

In South Africa, certain regions are susceptible to low to moderate intensity seismicity, i.e. the South Western region of the Western Cape Province and areas around Johannesburg.

Unreinforced masonry (URM) structures within these regions are regarded as the most vulnerable types of infrastructure when subjected to seismicity. Based on past experience, these types of infrastructure have performed poorly during past earthquake of low to moderate seismicity.

The low cost housing developments provided by the South African government is of highest concern, since URM structures built at minimal cost are extremely susceptible to seismicity. The structural integrity of these types of infrastructure is further questionable since they experience structural damage due to normal loading conditions. This is due to poor material quality and poor supervision.

In this study, a Finite Element (FE) model was developed of a typical single story URM low cost housing unit built to minimum standards which is commonly found in the South Western region of the Western Cape Province. This FE model was used to investigate the behaviour of the URM when subjected to seismic activity.

By using the meso-modeling approach in this investigation it was possible to capture the three dimensional behaviour of the URM structure. This allows the FE model to be a more accurate representation of the physical structure as it allows earthquake records to be applied in any direction, which results in both in and out of plane seismic loading of the walls.

The results obtained from this investigation illustrate that a large percentage of the single story URM low cost housing units in South Africa will be able to withstand the maximum seismic loading that they could possibly experience in their lifetime. The findings of this investigation are reinforced with experimental tests.

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I would like to thank my study leader, Dr. Trevor Haas, for the support over the two years and time spent reviewing my work, especially the time over the festive season. To all my parents for all the support during my many years of studies, from my first year to the proof reading of my thesis at the very end.

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1 INTRODUCTION

1.1 BACKGROUND TO STUDY

The task with the greatest priority to a Structural Engineer is to design structures that are safe and economical. Over the past decades, structural engineers have designed structures to withstand natural and human induced loadings. However, when structures are subjected to natural hazards, the anticipated structural analysis and design resulting from the hazard becomes complex. This has resulted in many of the structures around us being ill-equipped to withstand natural hazards. Table 1 illustrates the effect that natural disasters have had throughout the world, both in terms of fatalities and economic loss (Food and Agriculture Organization of the United Nations, 2015). These statistics indicate that significant research is however required to allow engineers to understand how these hazards behave and thus design safer infrastructure.

Table 1: Global impact of natural disasters between 2003 and 2013 (Food and Agriculture Organization of the United Nations, 2015).

Year	Damages (US billion)	Fatalities
2003	70	113 518
2004	136	244 880
2005	214	93 115
2006	34	29 893
2007	74	22 422
2008	190	242 489
2009	46	16 016
2010	132	329 998
2011	364	34 143
2012	156	11 526
2013	119	22 225
Total	**1.535 trillion**	1 159 925

An earthquake is one type of natural hazard which can result in catastrophic consequences. Since 1990 50.2 % of all globally reported fatalities due to natural disasters occurred as a result of earthquakes (Max Roser, 2016).

Two factors determine the risk a country is exposed to when considering the effects of seismicity. The first factor that requires consideration is how prone a particular area is to seismicity. The field of seismology has assisted engineers in obtaining seismic hazard maps. These maps illustrate the expected maximum peak ground acceleration that may occur in an area for a specific return period.

In South Africa, only certain regions of the country are susceptible to moderate intensity seismicity, i.e. the South Western region of the Western Cape Province and surrounding areas near Johannesburg. The majority of the seismicity occurring in the South Western region of the Western Cape Province occurs naturally, whereas the areas near to Johannesburg are mining induced.

The second factor that determines the potential risk of a country against earthquakes is the ability of its infrastructure to withstand seismic action. This quality is determined by the design and construction standards during the time of construction. Well established/reliable building codes are common in developed countries and are strongly enforced, whereas in developing countries there are typically not well-established or reliable codes of practice and/or not properly enforced. This can be validated since, four of every five fatalities caused by earthquakes occurred in developing countries between 1900 and 2000. Since 2000, this number increased to nine of every ten fatalities (Geohazards international, 2014).

1.2 PROBLEM DEFINITION

Since South Africa's first democratic elections in 1994, 5.6 million low cost housing units were constructed (Housing, 2015). The majority of these structures were constructed using unreinforced masonry (URM) due to its relatively inexpensive material costs and quick construction.

URM is however regarded as the most vulnerable type of building technique or construction material when subjected to seismicity. These types of structures have performed poorly during past earthquakes when subjected to low to moderate seismicity (Bruneau, 1994; Tomaˇzeviˇc, 1999). Due to the complex behaviour of URM buildings, which is characterized by strong brick elements bonded together by a weaker material called mortar, it is extremely difficult to evaluate the seismic performance capabilities of these structures.

The majority of the 5.6 million low cost housing units were suppose to be or should have been designed using the seismic guidelines which were included in South African loading code of practice, SABS 0160 – 1989 (South African Bureau of Standards). Practicing engineers considered these guidelines to be vague and/or over conservative. This resulted in many practicing engineers simply dismissing the proposed guidelines and allowing for seismicity in their structural designs as they deem fit. A concern now expressed is whether these units can resist a moderate intensity earthquake.

With the uncertainties regarding the seismic capabilities of URM structures, it is common knowledge that the low cost housing developments in South Africa have been built at minimal cost and as a result, many of these structures are experiencing structural damages due to normal loading conditions (De Nobrega, 2007).

It is therefore hypothesized that if a maximum probable earthquake occurs in South Africa the effect could be catastrophic as a large percentage of the population reside in low cost housing developments.

Due to the aforementioned reasons, a study was commissioned to investigate the sensitivity of low cost housing units in the south western region of the Western Cape Province of South Africa to seismicity in an attempt to determine the risk that these structures pose to its occupants.

1.3 OBJECTIVES

The aim of this investigation is to determine the sensitivity of low cost housing units in South Africa to seismic events in an attempt to determine the risk that these structures pose to its occupants. In the process of achieving the aim of this investigation, the current knowledge of the seismic behaviour of URM buildings will be expanded and the tools currently available to analyze these structures will be further developed.

The use of finite element modeling (FE modeling) is viewed as a possible approach of gaining a better understanding of the complex behaviour of masonry. Due to the complex behaviour of URM structures, this approach has not been developed to a point where it is possible to study the behaviour of large URM structures in detail. Existing FE modeling approaches found in literature focus predominately on modeling the behaviour of small masonry assemblages and shear walls. The most promising FE modeling approach found in literature will be investigated in detail and then used to model the entire masonry structure of a low cost house.

By using FE modeling in this investigation, significant information regarding the seismic behaviour of URM structures will be available. The information includes the stresses, strains and deformations which occur in a typical Reconstruction and Development Program (RDP) structure throughout the seismic simulation. This information will be used to expand the current knowledge on the subject.

1.4 METHODOLOGY

Firstly, it is necessary to investigate the low cost housing developments found in South Africa. In this section the standard at which these buildings are being constructed will be discussed, as well as the typical design specifications of these buildings.

The next topic of interest is the occurrence of earthquakes in South Africa and what approaches are available in literature to test the seismic resistance of URM structures. With this knowledge, the method best suited is selected to explore seismic sensitivity of the chosen structure.

Various failure mechanisms which define the behaviour of URM structures will be investigated, as it is crucial that the developed FE model is capable of capturing the entire behavioural range of URM structures. Various FE modeling approaches from literature will be examined, the approach which is most aligned with the nature of this investigation will be chosen to create the FE model of the URM structure.

The anticipated capabilities of the chosen FE modeling approach will be validated against the data of experimental tests carried out in literature.

With the groundwork complete, it will be determined which low cost house should be modeled as well as the seismic loading procedure which would be best suited for the nature of this investigation. The results of these simulations will then be analyzed and discussed.

2 LITERATURE REVIEW

2.1 LOW COST HOUSING IN SOUTH AFRICA

2.1.1 Background

Due to the lack of housing in the low income bracket, the South African (SA) government has attempted to solve the problem through the construction of low cost housing units. Since 1994, the SA government has built 5.6 million low cost housing units which amounts to approximately 250 000 houses constructed per year (Housing, 2015).

In order to build such a large number of low cost housing units, the provincial governments used a tendering approach. This entails the government specifying the number of units to be built in a particular area. The company with the lowest price is usually awarded the contract. The architects, engineers and the contractors of the project are determined in this manner. The disadvantage of this approach is that in order to be competitive in tendering for one of these government projects architects, engineers and contractors are inclined to offer rates with very small margins. This has resulted in the houses being built with a lack of site supervision, lack of detailed engineering design and decrease in building material quality.

2.1.2 Materials used to construct low cost housing

To date three types of materials are used in the construction of low cost housing units in South Africa; i.e. clay bricks, cement bricks and hollow core concrete (HCC) blocks. Clay and cement bricks were the material of choice until approximately 2000, thereafter HCC blocks became more popular as a result of their size, cost and fast construction.

Since the majority of low cost housing units in South Africa were constructed using HCC blocks, only HCC masonry structures will be considered in this investigation. Figure 1 illustrates an example of a HCC structure. For this type of structure there are two critical materials which need to be investigated, i.e. the mortar and the HCC blocks.

Figure 1: HCC masonry structure

2.1.2.1 Mortar and HCC block properties in South Africa

The Cement & Concrete Institute specifies that mortar could be either class 1 or class 2. A class 1 mortar is required for highly stressed masonry (which is commonly used in multi-story buildings) whereas class 2 mortar is required for normally loaded masonry (housing environment). The compressive strength of class 1 and 2 mortar is specified as 14.5 MPa and 7 MPa, respectively (Mortar mixes for masonry, 2009).

For low cost housing, HCC blocks of 390x190x190mm are typically used as they allow for quick construction as illustrated in Figure 2. The factored compressive strength for these types of concrete blocks in South African is 7 MPa (Blocks, 2015).

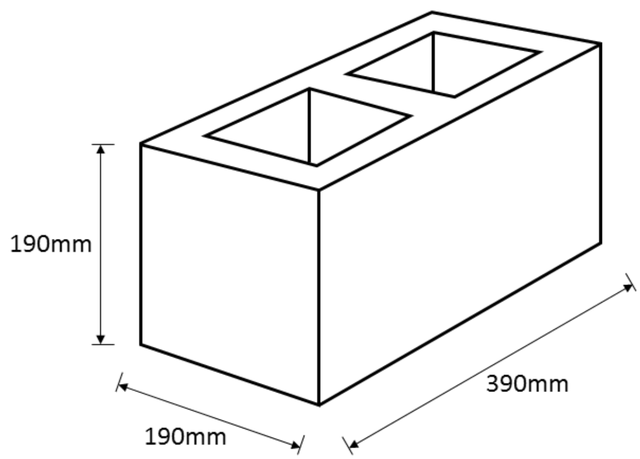


Figure 2: South African HCC block dimensions

2.1.2.1.1 The quality of mortar and HCC blocks used in practice

Despite the performance specifications stated in section *"2.1.2.1 Mortar and HCC block properties in South Africa"*, the mortar and HCC blocks used to construct low cost housing units can differ significantly.

In order to mix a batch of class 2 mortar, a ratio of three wheelbarrows of sand to one bag of cement is needed. Without adequate quality control, it is possible that the contractor could mix mortar batches with higher sand quantities, as it would allow him to make a larger profit.

With the large number of HCC blocks that is required to construct a low cost housing development it could be potentially cheaper for the contractor to manufacture the HCC blocks himself rather than purchasing them. Without adequate quality control, it is possible that the contractor could produce HCC blocks of inferior quality compared to factory manufactured quality controlled HCC blocks. Like the mixing of mortar, the inferior blocks may be due to the contractor trying to maximize profit or it could as a result of pure negligence.

2.1.3 Low cost housing layouts

The tendering processes used for the low cost housing developments have resulted in many different designs. In this investigation, a representative conceptual single story low cost housing layout is shown in Figure 3. The representative layout incorporates many of the structural elements found in other building layouts. The results of this investigation can therefore be applied to other low cost housing units with minimal differences in design.

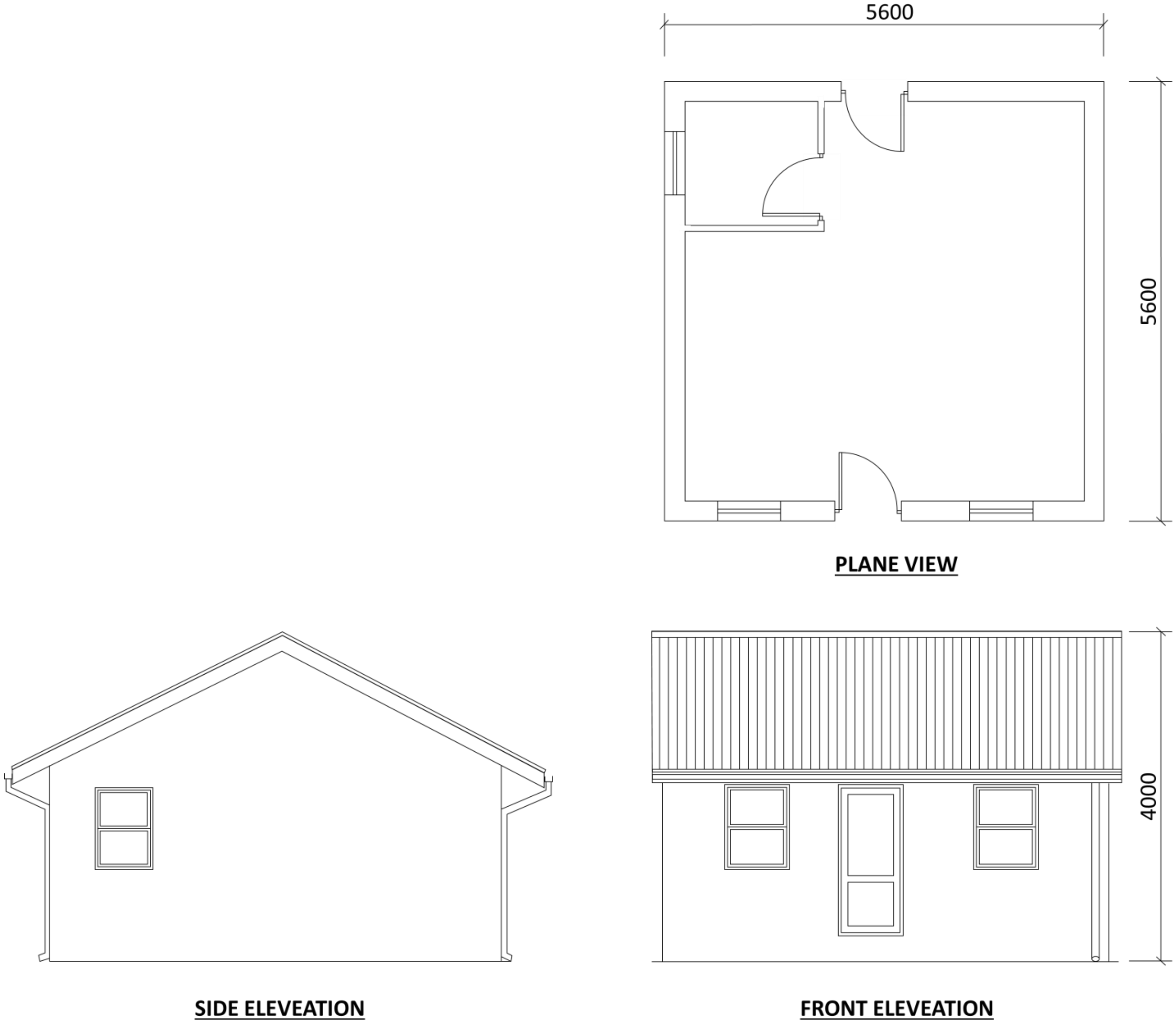


Figure 3: Architectural drawing of conceptual low cost housing design (not to scale)

2.1.4 Current condition of low cost housing in South Africa

2.1.4.1 Background

The quality of materials are an important factor when assessing the robustness of low cost housing units.

2.1.4.2 Observations throughout South Africa

An investigation conducted by Eastern Cape's Human Settlements Department, found that 5 461 low cost housing units had structural defects as a result of poor workmanship and inadequate quality assurance. Many of these houses were reported to have developed significant structural cracks in the masonry, probably as a result of these units being built without proper foundations. R 500 million was set aside by the department to repair the existing houses (Fuzile, 2013).

Another study undertaken in 2007 by the Public Service Accountability Monitor (PSAM) of Rhodes University, investigated the structural quality of state-subsidized housing in Ngqushwa local municipality (De Nobrega, 2007). This investigation again showed the unacceptable design and construction quality of subsidized housing, as cracks formed around the windows, doors and corners as shown in Figure 4. It was reported that these cracks appeared soon after tenants took occupation of these units. Similar observations were found by Mehlomakulu & Marais (1999) and Khumalo (2010) when investigating the low cost housing developments in the Mangaung and Port Elizabeth municipalities, respectively.

(a) (b)

Figure 4: Observed cracking in state-subsidized housing of the Ngqushwa local municipality

2.1.5 Comments and observations

Literature indicates that low cost housing structures vary significantly throughout South Africa, both in terms of structural layout and structural quality of the finished units. To provide insight into the behaviour of a typical South African low cost housing unit when subjected to the maximum probable earthquake loadings, a structure with the same layout illustated in section *"2.1.3 Low cost housing layouts"* will be analysed. It will also be assumed that the structure is constructed using building materials and construction quality that meets the minimum South African building standards

As mentioned in *"2.1.4 Current condition of low cost housing in South Africa"* a large number of the low cost housing units in South Africa do not comply with these standards. Due to the complex behaviour of URM structures, it would not be possible to use a blanket approach of decreasing all the material properties by a certain percentage to simulate a structure constructed of materials with same percentage of strength reduction.

The flaw in using this approach is that in reality, a decrease in compressive strength of the mortar and masonry units will not result in a proportional decrease in the strength of the other material properties which are required to describe the behaviour of URM structures. These material properties are the cohesive stiffness (describes the stiffness of the bond between the brick elements), plastic displacement (determines when cracking between the brick elements occurs) and the stress–strain curves of the masonry units during compression and tension.

In order to investigate the effect that poor material properties and poor construction quality would have on the behaviour of low cost housing would require a large number of experimental material tests. This course of action would not be possible in the time allowed for this investigation. The proposed approach will illustrate how a structure built to regulatory standards will resist a maximum probable earthquake and through doing so, insight will be gained on what level of risk low cost housing units pose to its occupants.

2.2 DESIGNING STRUCTURES IN SOUTH AFRICA FOR SEISMIC ACTIVITY

2.2.1 Introduction

Simply defined, an earthquake is the geological event in which ground movement or shaking is caused by a wave of energy traveling through the earth's crust. This wave of energy may originate from naturally or human induced causes. The majority of naturally induced earthquakes is due to the movement of tectonic plates. Man-made causes may originate from activities such as mining and fracking.

There are two methods available to describe the severity of an earthquake; i.e. magnitude and intensity. The magnitude quantifies the amount of energy that is released by a particular earthquake (Scawthorn, 1999). There are various methods available to measure this energy of which the Richter scale is the most well-known. The intensity measures the effect that an earthquake has on a specific location. For the same earthquake the intensity will vary at different locations. This is due to the type of soil that the earthquake's waves pass through having a significant influence on how the earthquake is felt on the surface. The intensity is normally based on the observed behaviour of the earthquake or the resulting structural damages. Recorded ground acceleration can also give an indication of the intensity of an earthquake.

2.2.2 Seismic activity in South Africa

Based on SANS 10160-4 (South African National Standard), Figure 5 shows the areas in South Africa that are prone to natural and human induced seismicity. Certain areas in the Western Cape Province are susceptible to natural seismicity, whereas the areas surrounding Johannesburg are susceptible to human induced seismicity cause by mining activities.

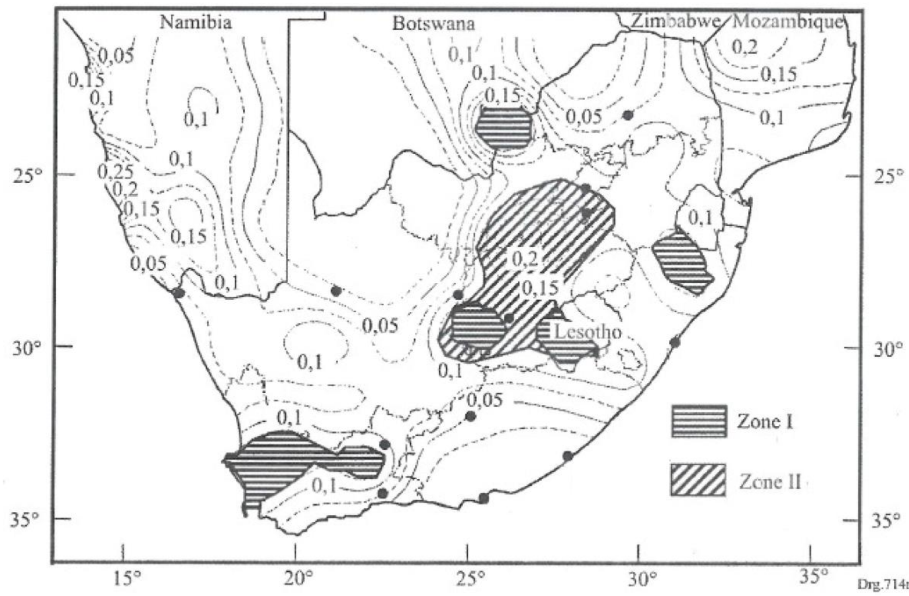


Figure 5: Seismic hazard zones showing peak ground acceleration and earthquake zones (SANS 10160-4, 2011).

According to research, certain areas in the Western Cape Province can expect an earthquake with a maximum magnitude of 6.7, whereas the maximum magnitude earthquake predicted for Johannesburg is 5.6 (Visser& Kijko, 2010).

Table 2 presents some of the earthquakes that have occurred in South Africa, accompanied with their magnitude.

Table 2: Recent significant seismic events in South Africa (Brandt et al., 2005).

Date	Region	Magnitude (Richter)
31/12/1932	St. Lucia, KwaZulu- Natal	6.0-6.5
29/09/1969	Tulbagh, Western Cape	6.3
14/04/1970	Tulbagh, Western Cape (Aftershock)	5.7
08/12/1976	Welkom, Free State	5.2
07/03/1992	Carletonville, Gauteng	4.7
02/08/2014	Orkney, North West	5.5

2.2.3 South African design recommendations

Although all regions of South Africa experience seismicity, only certain parts are considered to be at risk. For these regions, it is apparent that a seismic design code is required. To allow engineers to design structures in these areas capable of resisting probable seismic events, various guidelines where included in the first South African loading code of practice, SABS 0160 – 1980. Practicing engineers at the time, considered these guidelines too vague and conservative. This resulted in many practicing engineers simply dismissing the proposed guidelines and allowing for seismicity in their structural design as they deem fit or simply ignoring this requirement (Wium, 2010).

In 2009, South Africa adopted a new seismic design code, SANS 10160 Part 4, which is based on Eurocode 8 and other international codes. The new code is more comprehensive than its predecessor.

In the case of simple masonry structures, SANS 10160-4 provides design engineers with geometrical criteria which must be adhered to by the proposed structure. If these geometrical constraints are adhered to, the designer is not required to perform any additional design checks to the seismic capabilities of the structure. Through observations from recent earthquakes the criteria have shown to produce structures capable of resisting seismic forces (Magnes, 2006). These criteria are however very general and do not provide the design engineer with insight into how the proposed structure will resist a probable seismic event.

2.2.4 Seismic performance of low cost housing in South Africa

Van der Kolf (2014) found that a three-story clay brick low cost residential apartment located in the Stellenbosch has a high probability of significant damage/failure when subjected to an earthquake with a magnitude of 0.15g, despite being deemed structurally acceptable by the criteria specified in SANS 10160-4.

To date no research was conducted to investigate the seismic performance of single story low cost housing in South Africa. This is mainly due to the uniqueness of the low cost housing buildings.

After the Coalinga Earthquake of 1983, it was observed that the gables of masonry buildings constructed using clay bricks, were prone to collapse, as shown in Figure 6 (Reitherman, 2009). These structural elements (gables) have been incorporated into most low cost housing designs and could pose as an additional area of concern.

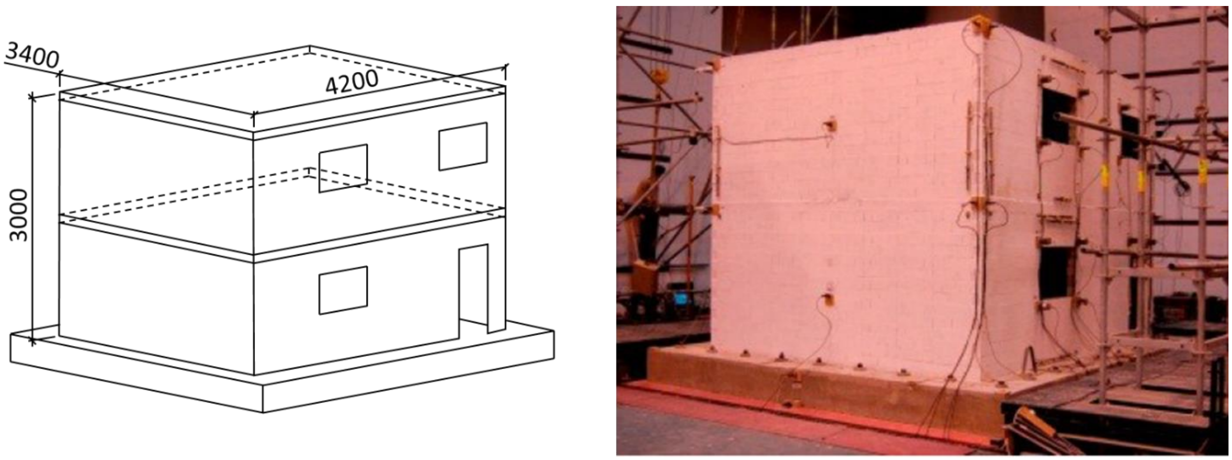
Figure 6: URM building with gable failure during Coalinga Earthquake of 1983 (Reitherman, 2009)

2.2.5 Experimental tests investigating the seismic performance of URM buildings

Throughout literature, many experimental tests were conducted in an attempt to understand how unreinforced masonry structures behave during seismic events. Unfortunately, due to the vast range of building materials available to construct URM structures, only a few experimental tests found in literature are relevant, as they too investigated URM structures constructed using HCC blocks.

2.2.5.1 Two-story building constructed using HCC blocks

Experimental tests conducted by Lourenço et al (2012) at University of Minho in Portugal could provide some insight into the seismic behaviour of low cost housing in South Africa. The investigation determined the ability of a two story building constructed HCC blocks to resist seismic activity, as shown in Figure 7(a) & 7(b). Fortunately, the HCC blocks and mortar used in this investigation are similar to that found in South Africa.



(a) (b)

Figure 7: (a) Schematic drawing of test structure, (b) physically tested structure

The two-story URM structure was subjected to a series seismic records with increasing PGA's (Peak Ground Acceleration). Each seismic record was applied along both of its horizontal axes simultaneously. From this investigation it was concluded that the two story URM structure could sustain seismic loadings with a PGA of up to 0.25g without incurring any visible damage. Only when subjected to seismic loadings with a PGA of 0.5g, could the presence of cracks be identified, in which repair of most of them could be easily be carried out. Despite occurring serious damages the structure did not collapse when exposed to the final stage of the experiment, which was a seismic load with a PGA of 1.33g.

2.2.6 Observations and comments

Literature indicates that structures in certain areas of South Africa must be designed to resist the seismic forces induced by moderate intensity earthquakes. If structures in these areas are not designed to accommodate the anticipated seismicity, the results could be catastrophic.

In order to be deemed structurally safe for seismic activity, a masonry building is required to meet the criteria specified in SANS 10160-4. Research shows that these criteria are not always capable of preventing damage/collapse when subjected to seismic activity. This approach also does not indicate the level of damage that could occur in masonry structures during seismic activity. This results in the seismic risk that the structure poses to its occupants to be unknown.

In order to better understand the seismic performance of low cost housing in South Africa, our focus is diverted to the experimental investigations published in literature. The findings from these investigations indicated that the proposed hypothesis for this investigation could possibly be incorrect and that South African low cost housing units could comfortably withstand the maximumn anticipated earthquake.

Apart from the experimental investigations found in literature, there is no clear information that indicates that single story South African low cost housing units can resist the maximumn anticipated earthquake loadings. The investigation will however continue to proceed with the original stated hypothesis and later compare its finding to the experimental investigations found in literature. The aim of this investigation is to illustrate how these structures will behave during an expected seismic activity and in doing so the risk that it poses to its occupants will also be determined.

2.3 MODELING MASONRY USING FINITE ELEMENT SOFTWARE

2.3.1 Introduction

Due to the poor performance of masonry structures during past earthquakes, engineers have realised that there is a need for a better understanding of the complex behaviour of masonry structures. One method of obtaining such an understanding is through the use of finite element modeling (FE modeling). FE modeling allows researchers to model masonry assemblages and complete virtual simulations on these assemblages. The detail at which these models are developed are dependent on the requirements of the investigation. The benefit of using the FE modeling for studying the seismic behaviour of masonry assemblages is that it is more practical than performing full-scale experiments, which are expensive to set up, require specialized testing apparatus and experienced personnel to perform the tests.

There are three approaches in which masonry can be modeled using the FE modeling namely, micro modeling, macro modeling and meso modeling. Micro modelling allows the bricks and mortar within the masonry to be modelled separately, i.e. both are modeled using solid elements. The macro modelling approach combines the properties of the bricks and mortar into one homogenous material, i.e. shell elements are commonly used for this approach. The meso modelling approach is a compromise between the two approaches. It allows the user to define the bricks as solid elements and the mortar as an interaction interface between the brick elements (Lourenco, 1996).

2.3.2 Three FE modeling approaches which can be used to represent URM structures

2.3.2.1 Macro modelling approach

The macro modelling approach allows the bricks and mortar of masonry assemblages to be modeled as an isotropic or anisotropic continuum material. The anisotropy is thought to arise from the geometrical layout of the bricks and mortar within the masonry. Such a model was developed by Lourenco (1996). The model utilized a Hill type yield criterion for compression and a Rankine type yield criterion for tension. These yield surfaces allow different behaviour along each material axis and thus produces an anisotropic material. The model presented by Lourenco (1996) replicated the behaviour of physically tested shear walls. However, there is concern on how accurately the macro model would replicate the behaviour of more irregularly shaped URM structures.

2.3.2.2 Micro modelling approach

The micro modeling approach allows the user to define the bricks and mortar separately. This makes it possible for the FE model to capture all the failure mechanisms that characterize the behaviour of masonry. Figure 8(a) to 8(e) illustrate these failure mechanisms, i.e. (a) direct tensile failure in the bricks, (b) tensile failure in the mortar, (c) shear failure in the mortar, (d) diagonal tensile failure in the bricks and (e) bricks failing in compression.

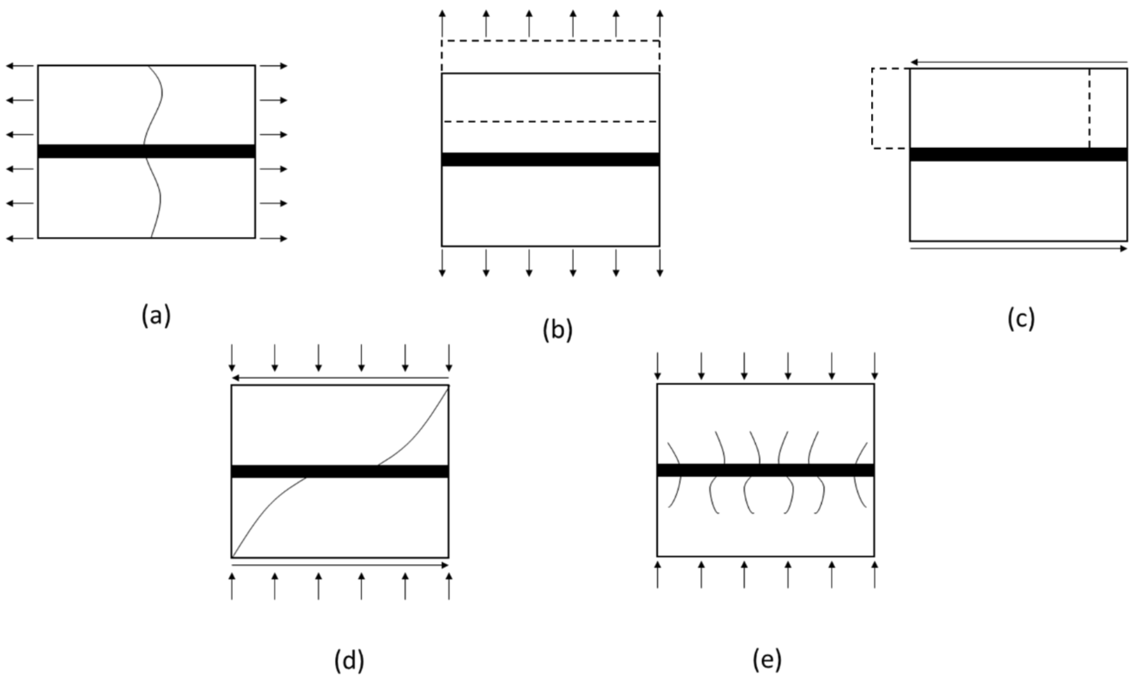


Figure 8: Masonry failure mechanisms (Lourenco, 1996).

2.3.2.1 Meso modelling approach

Like the micro modeling approach, the meso modelling approach also allows the user to define the bricks and mortar separately, however in this instance the mortar is not defined as a solid element but rather as an interface between the solid brick elements.

Lourenco (1996) published an influential papers on the subject. He investigated using interface elements opposed to solid elements to represent the mortar between the bricks. The interface elements have a zero thickness and for all practical purposes, could be viewed as an interaction property. The benefit of using the interface elements is that it drastically decreased the computational time of the simulations without compromising the accuracy. As reported by Lourenco (1996), the meso model with interface elements was 12.5 times faster than the micro model with solid elements. Lourenco (1996) also illustrated the meso model was able to replicate the results of experimentally tested walls.

The detailed formulation of the interface model which Lourenco (1996) developed in his paper paved the way for future development on the subject. Two investigations which followed a similar trend as Lourenco, are presented in the papers of Dolatshahi (2013) and Bolhassani (2015).

The paper presented by Dolatshahi (2013) develops a model which is very sophisticated, as the behaviour of the mortar in the model is specified by a user defined input file which can be read by Abaqus. This allows all the behavioural characteristics of the mortar interface between the bricks to be accurately defined. The second approach of interest developed by Bolhassani (2015) approaches the problem from a more practical perspective. Instead of coding the mortar's behaviour in a user-defined input file, the author used the available material properties found in Abaqus library. The paper shows that this simplified model was able to provide accurate results despite it only requiring simple material parameters.

2.3.3 The use of the FE method to model the seismic behaviour of URM buildings

Most of the research reported in literature predominantly focuses on modelling of shear walls. Unfortunately, no published work could be found that utilized the meso-modeling approach to investigate the behaviour of an entire URM structure when subjected to seismic loading.

Through recent work conducted by Lourenco (2012) the macro modeling approach was able to show the degree to which a structure can resist seismicity. It was however not capable of illustrating when and where failure occurs within a structure.

2.3.4 Model validation

Another aspect that needs to be considered when choosing to model a structure is how the accuracy of the FE model will be validated. The trend in literature is that physically tested shears walls should be modeled using a proposed modeling approach. If the results produced by FE model are within 5-10% of the experimental results, the accuracy of the FE model's approach can be regarded as accurate and therefore validated. The shear walls tested by Raijmakers and Vermelfoort (1992) and Raijmakers and Vermelfoort (1993) are very popular for performing such validation processes.

2.3.5 Comments and observations

When deciding what modeling approach to use in this investigation, it was important that the chosen modeling approach would be capable of accurately replicating the behaviour of the masonry structure throughout the simulation. The hypothesis expects the structure investigated will encounter severe damages during the seismic loading. Therefore the macro modeling approach is not suitable for this investigation as it is not capable of capturing the nonlinear behaviour of masonry assemblages.

The meso and micro modeling approaches are the more preferred option as they have both been proven to accurately replicate the linear and non-linear behaviour of masonry assemblages. It is important to note that neither one of these two approaches have been used to model a large structure. It is believed that this is due to these approaches being extremely computational expensive.

Due to the small size of the of the low cost housing structure chosen for evaluation in this investigation, it is anticipated that it will be possible to use either of the two modeling. The meso-modeling approach is however the more preferred option as it has been proven to replicate the behaviour of masonry assemblages to the same degree of accuracy as that of the micro modeling approach but is 12.5 times more computational efficient than the micro modeling approach (Lourenco, 1996).

3 THEORY

3.1 METHODS OF EVALUATING THE ABILITY OF A FE MODEL TO RESIST SEISMIC ACTIVITY

3.1.1 Equivalent lateral force method defined in SANS 10160-4

SANS 10160-4 allows the design engineer to evaluate the structural integrity of a building when subjected to seismicity using the equivalent lateral force method. This method specifies the lateral base shear force, which the structure must withstand. The uniqueness of a specific design problem is accommodated through the construction of the normalized design spectra.

In order construct the normalized design spectra, the following parameters must be known:

1. Type of building (i.e. Unreinforced masonry, steel, concrete etc.)
2. The location of the building
3. The type of soil on which the structure is supported.

3.1.1.1 The effect of different types of buildings

When the type of building is known, the design engineer is allowed to increase or decrease the design load. This decision is based on the ability that the known type of building has to resist dynamic forces. For example, when dealing with unreinforced masonry structures it is well know that these structures exhibit brittle failure and therefore must be designed to remain within the elastic region (SANS 10160-4, 2011). However, when dealing with framed reinforced concrete structures we can expect some degree of plasticity as a result of the reinforcement. This allows these structures to be designed for higher loads as they no longer have to remain within the elastic region.

3.1.1.2 The effect of where the building is located

Figure 9 specifies the peak ground acceleration (PGA) which a structure must be designed for in South Africa. Based on Figure 9 the south western region of the Western Cape Province and the areas near to Johannesburg must be designed for a PGA of 0.1g and 0.2g, respectively.

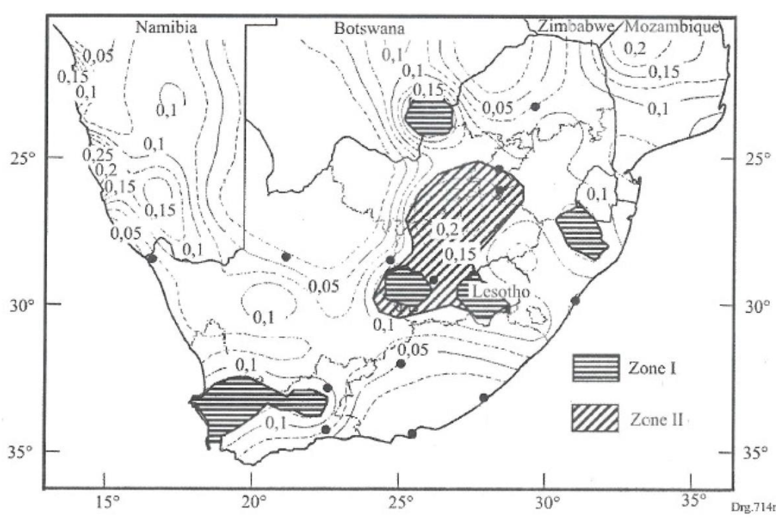


Figure 9: Seismic hazard zones showing peak ground acceleration and earthquake zones (SANS 10160-4, 2011).

3.1.1.3 The effect of different soil types

The type of soil that surrounds a structure has a significant influence on the magnitude of the loading the structure experiences during an earthquake. In SANS 10160-4 (2011) this phenomenon is incorporated into the design process. The design engineer is allowed to select a ground type from Table 3 which best matches that found on site/surrounding area. Each one of the four possible ground types have their own specific parameters as shown in Table 4. These parameters are then used to determine the normalized design spectra.

Table 3: Ground types (SANS 10160-4, 2011).

1	2	3		
Ground type	**Description of stratigraphic profile**	**Parameters**		
		$V_{s,30}$ *m/s*	N_{SPT} blows/30cm	c_u kPa
1	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	-	-
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	360 - 800	>50	>250
3	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180 - 360	15 – 50	70 - 250
4	Deposits of loose-to-medium cohesion-less soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	<180	<15	<70

where

$v_{s,30}$ = average value of propagation of S-waves in the upper 30 m of the soil profile at shear strains of 10^{-5} or less

N_{SPT} = standard Penetration Test blow-count

c_u = un -drained shear strength of soil (kPa)

Table 4: Values of the parameters describing the design response spectra (SANS 10160-4, 2011).

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	1,12	0,20	0,6	2,0
4	1,35	0,20	0,8	2,0

Figure 10 illustrates the effect that different ground types have on the normalized design spectra of a structure. It is evident that as the soil reduces in firmness, the normalized design spectra increases. Since the normalized design spectra is directly proportional to the base shear force, it can therefore be concluded that softer soils result in greater design base shear forces. Lawson and Reid (1908) made a similar observation when they showed that structures situated on softer soil underwent more damage when compared to structures situated on firm soil.

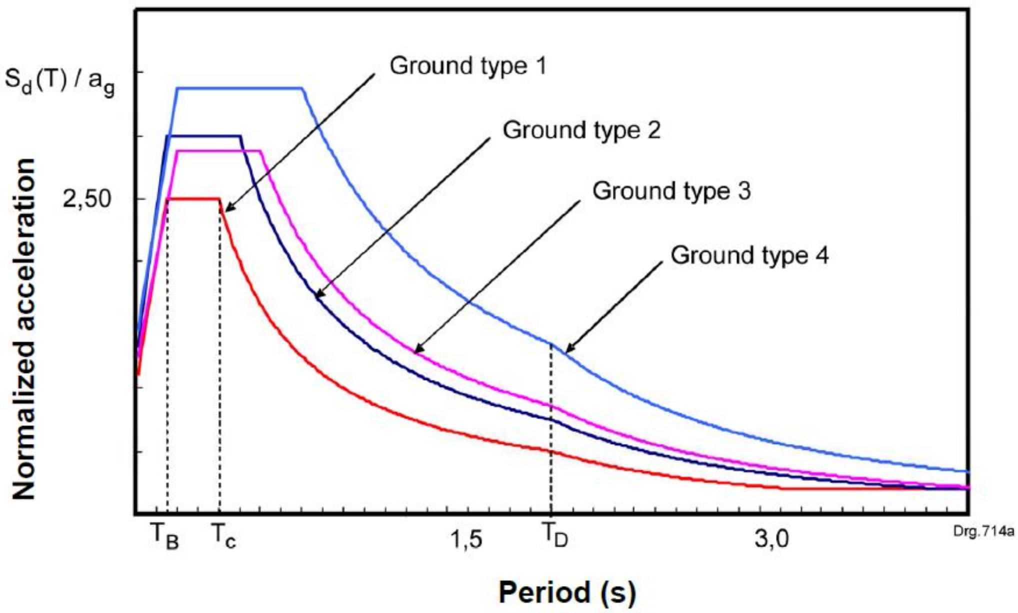


Figure 10: Normalized design response spectra, $S_d(T)/a_g$ for 5% damping and q = 1,0 (SANS 10160-4, 2011).

3.1.2 Time history analysis defined in EN 1998-1:2004

A time history analysis entails subjecting a FE model of a structure to the ground acceleration records of an earthquake. This ground acceleration data used to subject the FE model can be obtained from various earthquake records.

3.1.2.1 Method of obtaining ground acceleration

EN 1998-1:2004 allows the design engineer to perform a time history analysis using artificial accelerograms or recorded accelograms from past earthquakes. In order to determine what atificial/recorded accelograms are appropriate to use in an investigation, the spectra of the accelograms must provide a good match with the elastic reponse spectra of the structure under investigation.

3.1.3 Approach chosen for this investigation

The goal of this investigation is to determine whether single story low cost housing units in South Africa are capable of resisting the largest earthquake that is expected to occur durring its life time. To provide insight into the overall behaviour of the building, the method defined in EN 1998-1:2004 will be adopted for the analyses.

This method provides a more realistic evaluation of the proposed structure than the lateral force method proposed in SANS 10160-4. This is due to the the time history analysis being able to take account of the non-linearity caused by the materials and the loading.

Another reason that this method was chosen was because it allows the designer to simulate the actual building with earthquake records which are capable of representing the largest earthquake that is expected to occur durring its life time. This is accomplished by the designer being able to construct an elastic response spectra for the worst possible case that could occur and then selecting earthquake records with similair spectra.

3.2 MODELING MASONRY USING FINITE ELEMENT ANALYSIS.

3.2.1 Overview

When analyzing masonry using finite element analysis, there are three approaches which could be used; micro, macro and the meso modeling approach (Lourenco, 1996). The macro model assumes that the material is isotropic and that the properties of the mortar and cement blocks can be lumped together. This can be very effective when large masonry structures are investigated. Unfortunately, because the material is seen as a homogenous material the exact failure mechanism of the structure cannot be illustrated.

On the other hand, the micro modeling approach allows the user to define the material properties of the mortar and the cement blocks separately. This model has shown to replicate the failure mechanisms of masonry structures. Due to the high computationally effort required to execute the simulations, it can only be used to investigate small masonry assemblages.

The idea of using a meso modeling approach is a result of the macro and micro models. The meso-model is able to capture the failure mechanisms of masonry structures as the micro model does but instead of modeling the mortar as a solid material, it is instead models it as an interface between unit elements, thus significantly decreasing the computational effort required to analyze such a model.

The meso-modeling approach was chosen to create a FE model of the structure to be evaluated in this investigation. The reader is referred to section *“2.3 Modeling masonry using finite element software”* for a more descriptive explanation for the selection of this modeling approach.

Two meso-modeling approaches will be discussed in terms of how the behavioural characteristics of masonry assemblages are implemented in Abaqus. If a more theoretical understanding of the modeling approaches is required, the reader is referred to Bolhassani (2015) and Dolatshahi (2013). The first method that will be discussed is a simplified modeling approach which was developed by Bolhassani (2015). The second method is a complex modeling approach developed by Dolatshahi (2013). The simplified approach will be discussed in more detail as it was adopted in this investigation.

The reason the simplified modeling approach was used in this investigation is purely due to the original authors of the complex modeling approach being unwilling to publish the VMAT material file, which is essential to implement the approach. Dolatshahi (2013) explained the theory used to develop the VMAT material file but instead of spending large amount of time and effort on developing the VMAT material file, the author chose to complete the investigation using the simplified method which makes use of available material properties found in Abaqus.

3.2.2 Simplified model presented by Bolhassani (2015)

3.2.2.1 Introduction

The proposed model consists of unit elements connected by a cohesive interface. The unit elements represent the bricks and the cohesive interface represents the mortar bond between the bricks. As shown in Figure 11, the unit elements have the dimensions of a standard brick with the addition of half the mortars thickness on each of its four sides.

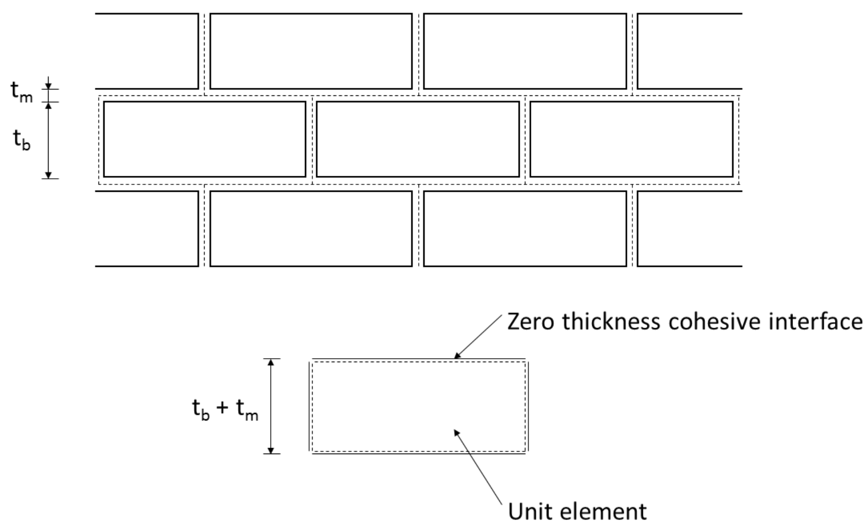


Figure 11: Model layout (Bolhassani, 2015)

3.2.2.2 Unit elements

The unit elements are modeled using of Abaqus's concrete damage plasticity (CDP) material model. Abaqus's CDP material model is specified for modeling concrete and other quasi-brittle materials under monolithic and cyclic loading (Simula, 2013).

The CDP material model assumes that tensile and compression are the two dominate types of failure, illustrated in Figure 12. Both the compressive and tensile behaviour can be defined by uniaxial compression and tensile tests.

Abaqus additionally allows the user to define compressive and tensile damage parameters, i.e. d_c and d_t, respectively. These parameters allow the model to decrease the materials stiffness during cyclic load.

The CDP material model requires that the plasticity of the material be defined. The tests required to determine this information are very specialized and are extremely sensitive. The plasticity properties used in this paper have been extracted from literature where similar experiments were conducted (Bolhassani, 2015).

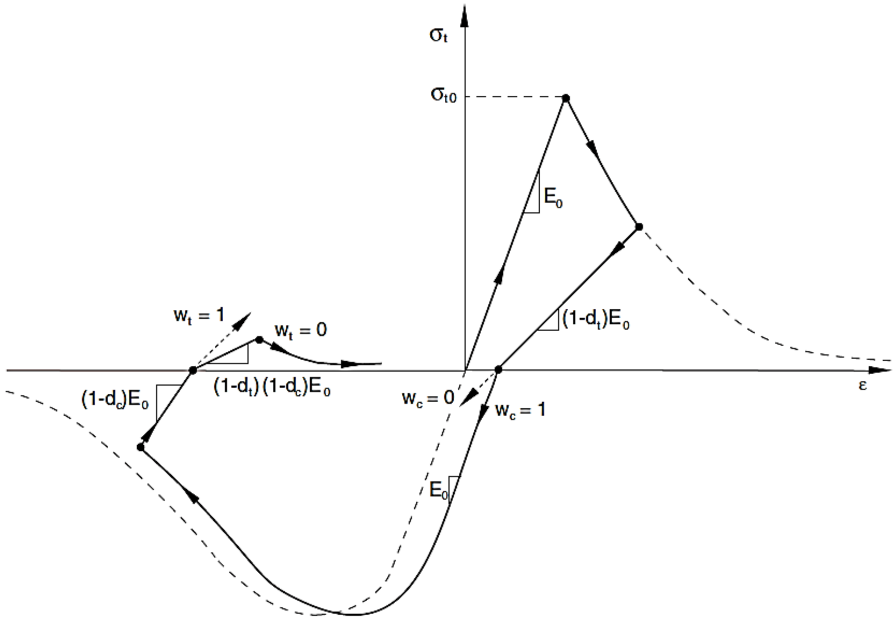


Figure 12: Concrete damage material model under uniaxial tension and compression test (Simula, 2013).

3.2.2.3 Cohesive interface

The cohesive interface is modeled as a surface based interaction in Abaqus. The cohesive behaviour is defined in terms of a traction-separation law. The traction-separation law allows linear behaviour prior to yielding, there after the material experiences progressive degradation (Simula, 2013).

3.2.2.3.1 The elastic behaviour

Prior to damage initiation the cohesive interface acts linearly. This linear behaviour is defined in terms of an elastic stiffness matrix **k**. Equation 1 illustrates the elastic stiffness matrix **k** and is composed of the elastic stiffness's in the normal and two shear directions; $\mathbf{k_n}$, $\mathbf{k_s}$ and $\mathbf{k_t}$, respectively. Equation 2 & 3 illustrate how these three components of the elastic stiffness matrix **k** are calculated (Bolhassani, 2015). The thickness of the mortar between the HCC blocks is represented by $\mathbf{t_m}$, whereas the elastic and shear modulus's of the HCC blocks and the mortar are represented by $\mathbf{E_b}$, $\mathbf{G_b}$, $\mathbf{E_m}$ and $\mathbf{G_m}$, respectively.

$$k = \begin{bmatrix} k_n & 0 & 0 \\ 0 & k_s & 0 \\ 0 & 0 & k_t \end{bmatrix} \quad (1)$$

$$k_n = \frac{1}{t_m}\left(\frac{E_m E_b}{E_b - E_m}\right) \quad (2)$$

$$k_{s,t} = \frac{1}{t_m}\left(\frac{G_m G_b}{G_b - G_m}\right) \quad (3)$$

3.2.2.3.2 Plastic behaviour

To capture the nonlinearity of the cohesive interface, Abaqus applies a concept called damage-modeling. It is able to handle both shear and the tensile failure mechanisms. Compression failure cannot occur within the cohesive interface, this failure mechanism is accommodated by the CDP model of the unit elements. Using the damage-modeling concept both the shear and the tensile failure mechanisms are divided into three steps namely; a damage initiation criterion, a damage evolution law and a choice of element removal once complete failure has occurred.

3.2.2.3.2.1 Damage initiation criterion

Abaqus allows the user to define at which stresses and strains the material starts to degrade. In Figure 13, the point $\mathbf{t^0_n}$, $\mathbf{t^0_s}$ and $\mathbf{t^0_t}$ illustrate the peak stresses in the normal and two shear directions respectively, coupled with their displacements $\boldsymbol{\delta^0_n}$, $\boldsymbol{\delta^0_s}$ and $\boldsymbol{\delta^0_t}$ at which they occur (Simula, 2013).

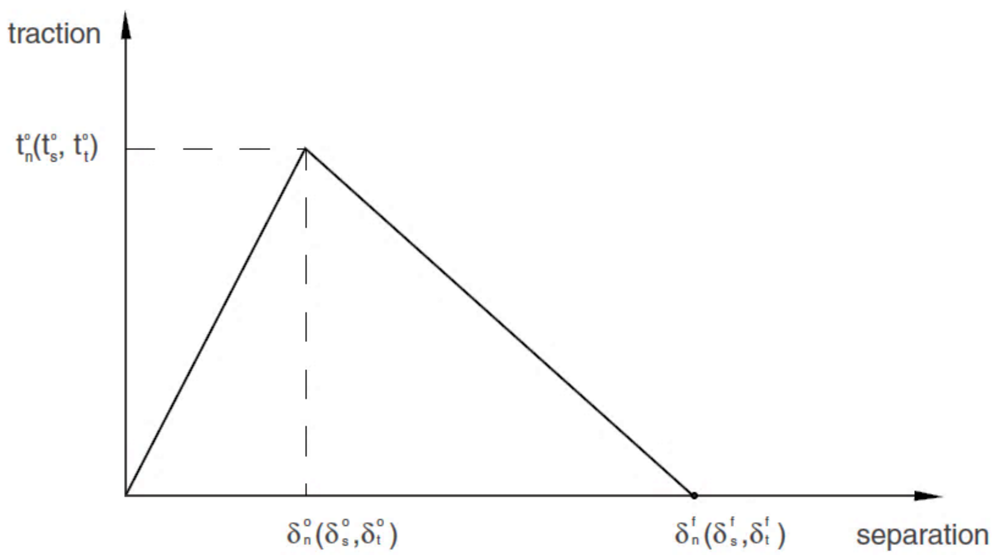


Figure 13: Traction-separation law defined in Abaqus (Simula, 2013).

Abaqus offers various criterion statements, which is constantly checked to determine if the damage evolution should commence. For the purpose of this investigation, the “Maximum stress criterion” was chosen. It initiates damage evolution if either one of the three stresses within the cohesive interface reaches their corresponding peak values, which is believed to be a good reflection of the actual behaviour of the material.

Equation 4 illustrates the "Maximum stress criterion" statement which must be met before damage evolution can begin. The variables $\mathbf{t_n}$, $\mathbf{t_s}$ and $\mathbf{t_t}$ represent the stresses at a particular time within the cohesive interface instance in the normal and two shear directions, respectively.

$$max\left\{\frac{t_n}{t^0{}_n},\frac{t_s}{t^0{}_s},\frac{t_t}{t^0{}_t}\right\} = 1 \qquad (4)$$

3.2.2.3.2.2 Damage evolution

In this module Abaqus allows the user to specify to what degree the material can deform plastically and in what manner it must deform. For this investigation, it was chosen that the material should degrade linearly and the plastic displacement should be specified. The plastic displacement $\boldsymbol{\delta_p}$ is defined in equation 5, where it specifies the displacement that may occur after the peak stresses have been reached within the cohesive interface. The variables $\boldsymbol{\delta^0{}_{n,s,t}}$ and $\boldsymbol{\delta^f{}_{n,s,t}}$ represent the magnitude of separation which occurs at the point of peak stresses and at complete bond failure, respectively.

$$\delta_p = \delta^f{}_{n,s,t} - \delta^0{}_{n,s,t} \qquad (5)$$

3.2.2.3.2.3 Element removal

This step allows the user to choose to delete the interface elements or let them remain active after failure. Choosing to delete the interface elements is the preferred option. By doing this the cohesive interface will not reactivate when the unit elements come back into contact.

3.2.2.3.3 Post failure behaviour

Unfortunately, the cohesive interaction model in Abaqus does not allow the user to specify all the interaction properties related to this particular problem. However, by incorporating other interaction models it is possible to specify all the properties required to recreate the mechanics of the bond created by the mortar between the HCC blocks (Simula, 2013).

3.2.2.3.3.1 Normal constraints

After the cohesive interface has failed between the unit elements, the unit elements may come into contact again. It is for this reason that "Hard contact" must be specified in the normal direction. This is to prevent the unit elements from penetrating into one another and causing convergence issues.

3.2.2.3.3.2 Additional shear behaviour considerations

An additional friction model must be created in order to capture the true shear behaviour of the bond between the bricks. If the cohesive shear stiffness is undamaged, the friction model can be viewed as being dormant. It is only once the cohesive shear stiffness has reached yielding strain that the friction model start to contribute to the stiffness of the interaction. This friction model is defined by the confining pressure between the bricks and a user defined frictional constant (Simula, 2013).

3.2.3 Complex model presented by Dolatshahi (2013)

3.2.3.1 Introduction

Instead of utilizing the material options predefined in Abaqus, Dolatshahi (2013) used the approach of developing his own interface model, which allowed him to define all behaviour characteristics that compose the bond between the unit elements. The physical layout of the complex FE model is the same as the simplified model discussed in section *"3.2.2 Simplified model presented by Bolhassani (2015)".*

3.2.3.2 Elastic and plastic behaviour of brick elements

Similarly to that of the simplified model, the complex model utilizes the concrete damage plasticity model in Abaqus to model the behaviour of the HCC blocks.

3.2.3.3 Elastic behaviour of the joints

The elastic behaviour of the joints is defined using the same approach as with the simplified method.

3.2.3.4 Plastic behaviour of the joints

In the material model, Dolatshahi (2013) was able to define the yield surface along which the interface element will behave. The yield surfaces represent the various modes of failure, i.e. tension, shear, tension combined with shear and compression failure.

The model runs through an algorithm that checks if the anticipated stresses for the next time step lie within the yield surface. If it does, the anticipated stresses are correct as they follow elastic behaviour. If not the stresses must be modified according to the yield surface it is overstepping (Dolatshahi, 2013).

4 Development of the FE model

4.1 Introduction

In this chapter, the steps followed to create a FE model capable of simulating a single storey low cost housing unit in South Africa is developed when subjected to a moderate intensity earthquake.

The first section validates that the simplified modeling approach using the FE method is capable of accurately capturing the nonlinear behaviour of URM walls. By comparing the results of the FE models to that of experimental test data, the author will comment on the capabilities of using the simplified modeling approach to model URM structures.

Section *"4.3 Material properties validation of HCC blocks"* validates the material properties of HCC blocks found in literature. In order to do this a FE model of a HCC block wall was simulated, the results derived from the FE model was then compared to experimental test data found in literature.

With the groundwork completed, sections *"4.4 Finite element model"* and *"4.5 Loading scheme"* illustrates how the FE model of the single story URM structure was developed in Abaqus, as well as what earthquake records were imposed on the FE model.

4.2 Modelling approach validation

4.2.1 Introduction

The simplified modelling approach developed by Bolhassani (2015) was only validated against small HCC block assemblages in axial compression, diagonal tension and bed joint shear tests as illustrated in Figure 14. Before the simplified masonry model was used to model the low income house, the simplified masonry model was first calibrated and validated for simple shear walls obtained in literature. This approach of model validation is very popular in this field of using the finite element method to model URM.

(a) (b) (c)

Figure 14: (a) Axial compression, (b) Diagonal tension and (c) Bed joint shear tests performed by Bolhassani (2015)

In order to validate the modeling approach, the comparison between experimental and numerical results must illustrate that the proposed modeling approach is capable of replicating the elastic and nonlinear behaviour of unreinforced masonry structures when subjected to various loading. Pushover curves and crack pattern comparisons will be used to address these two criteria.

The two types of experimentally tested shear walls that will be used in this investigation where conducted by Raijmakers and Vermelfoort (1992) and Raijmakers and Vermelfoort (1993). The first shear wall is constructed without an opening while the second shear wall with an opening, as illustrated in Figure 15(a) & 15(b). The shear walls are 990mm wide and 3500mm high, which was constructed using 210×52×100mm clay bricks with a 10mm thick mortar. The walls have a fixed support to their base and have a roller support at its apex which prevents vertical and out of plan movement. A pressure load of 0.3 MPa was also applied at the top of the wall.

In order to develop the push over curves for these walls, the base shear force of the walls was recorded while an in-plane lateral displacement was applied to the top of the wall.

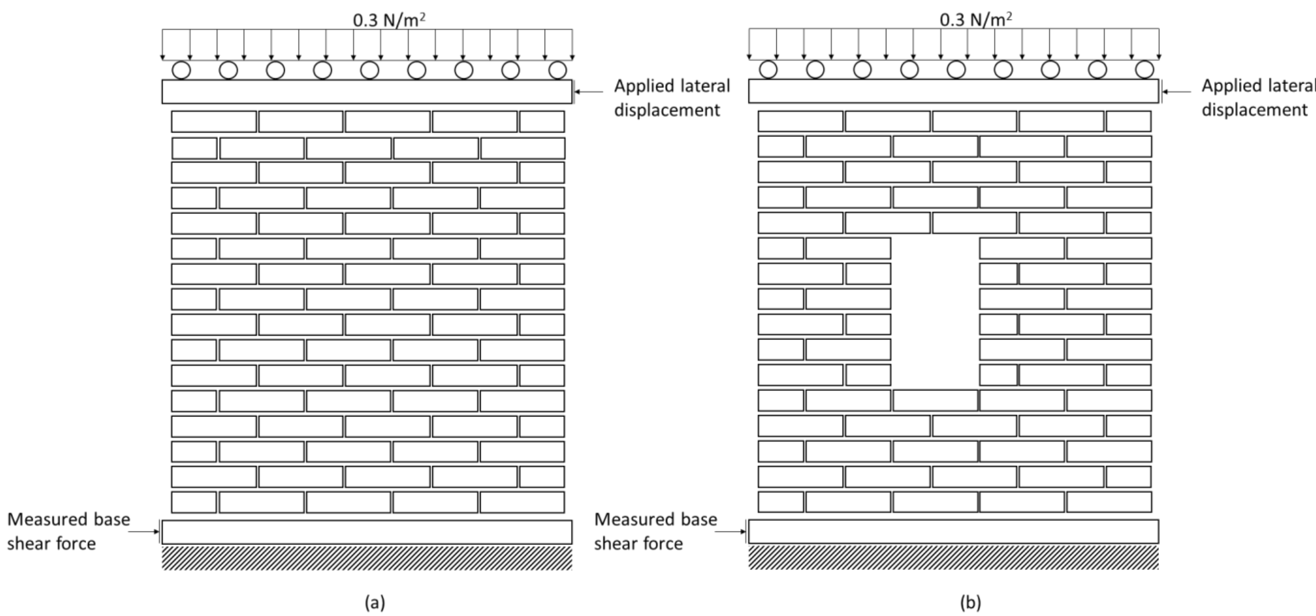


Figure 15: (a) Shear wall without an opening, (b) Shear wall with opening

4.2.2 Material input properties

In order to develop the FE models of the shear walls the material properties of the clay bricks and the cohesive interface needed to be established.

While developing the material models for this investigation, it became apparent that not all the material properties required to develop the material models in Abaqus were available from the material tests conducted by Raijmakers and Vermelfoort (1992 & 1993). It was therefore decided to obtain the material properties from similar experiments found in literature.

Table 5 & 6 present all the material properties which were required to develop the FE model in by Abaqus, which were obtained from various sources.

4.2.2.1 Brick properties

In order to use the concrete damage plasticity (CDP) model in Abaqus, the plastic properties of the clay bricks where required. Unfortunately, these material properties where not provided by Raijmakers and Vermelfoort (1992 & 1993), instead these properties where extracted from Bolhassani (2015).

The CDP model also required the stress vs. strain relationship for the clay bricks in tension and compression. Unfortunately, only the ultimate tensile and compression stresses of the clay bricks were provided by Raijmakers and Vermelfoort (1992 & 1993). In order to determine the stress vs. strain relationship for the clay bricks, the results of compression and tensile tests completed by Bolhassani (2015) where scaled to match the ultimate stresses provided by Raijmakers and Vermelfoort (1992 & 1993).

4.2.2.2 Cohesive interface properties

The plastic displacement ($\boldsymbol{\delta_p}$) which determines when cracking occurs after peak stresses are reached, were not provided by Raijmakers and Vermelfoort (1992 & 1993). This material property was also extracted from Bolhassani (2015).

Table 5(a): Clay brick material properties (Raijmakers and Vermelfoort, 1992-3)

Clay brick properties

Mass	Elastic		Plastic				
Density (kg/m3)	Young's Modulus (GPa)	Poisson's ratio	Dilation angle	Eccentricity	f_{bo}/f_{co}	K	Viscosity parameter
2000	16.7	0.15	32	0.1	1.16	0.67	0.001

Table 5(b): Clay brick material properties (Raijmakers and Vermelfoort, 1992-3 & Bolhassani, 2015)

Additional clay brick properties

Compressive behaviour		Tensile behaviour	
Stress (MPa)	Strain	Stress (MPa)	Strain
8.5	0.0000	1.05	0.0000
10.1	0.0030	0.81	0.0003
10.5	0.0005	0.58	0.0004
10.2	0.0010	0.21	0.0009
9.5	0.0015	0.07	0.0030
7.3	0.0029	0.04	0.0045
5.0	0.0045	0.03	0.0053
1.0	0.0099		

Table 6: Cohesive interface properties (Raijmakers and Vermelfoort, 1992-3 & Bolhassani, 2015)

Cohesive interface properties

Tangential behaviour	Normal behaviour	Cohesive behaviour (N/mm^3)			Damage				
Frictional coefficient	Type				Initiation (MPa)				Evolution
		K_{nn}	K_{ss}	K_{tt}	t^0_n	t^0_s	t^0_t	Criterion	δ_p (mm)
0.78	"Hard"	82	36	36	0.25	0.35	0.35	MAXS	2.3

4.2.3 Modeling procedure

In order to replicate the boundary conditions and loading scheme of the shear walls used by Raijmakers and Vermelfoort (1992 & 1993), the testing procedure for FE models of the two types of walls had to be divided into three steps. This testing procedure is shown in Figure 16.

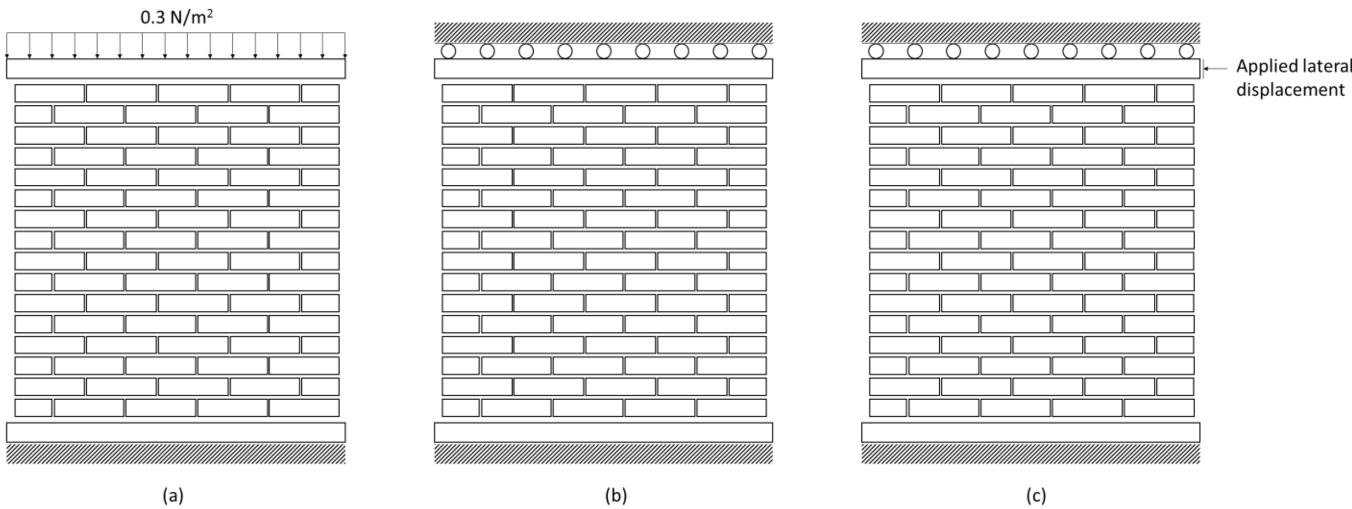


Figure 16: Testing procedure

The first step (a) which was defined as a "Static-General" step, applies the boundary conditions to the base of the wall as well as the gravity and pressure loads. In step (b) which was also defined as a "Static-General" step, the top of the wall is constrained in the vertical and out of plane direction. This was done to replicate the testing procedure carried out by Raijmakers and Vermelfoort (1992 & 1993). In the last step (c) which was defined as a "Dynamic - Implicit" step, the lateral displacement is applied to the wall.

4.2.4 Results

Firstly, the crack patterns of the experimentally tested walls and the FE model were compared, followed by a comparison of their load vs. displacements profiles.

During the investigation various mesh sizes were used to find a modeling approach that is computational efficient and yet accurate. It was found that a 50mm mesh provided accurate results without drastically increasing the computational effort of the simulation. Mesh sizes greater than 50mm were not able to accurately capture the behaviour of the shear walls.

4.2.4.1 Crack pattern comparison

Figures 17(a) & 17(b) & Figures 19(a) & 19(b) represent the crack patterns observed from the experimentally tested walls without an opening and with an opening, respectively. Figure 18 & 20 illustrate the final crack patterns observed in the FE models for the same types of walls.

The final crack patterns represent the areas within the FE model where either the HCC blocks or the bond between the HCC blocks failed. In order to develop these final crack patterns, Abaqus compares the stresses at each point within FE model to the allowable stresses. If the stresses at a point is larger than the allowable stresses, a crack starts to develop. This comparison is completed for each time frame of the simulation.

On inspection, it is clear that the numerical model is able to capture the two failure mechanisms of each wall type with insignificant differences.

The first and most important mechanism being the diagonal cracking that forms through the centre of the shear walls. Another failure mechanism that all the walls share is the lifting of the corners that do not lie on the main diagonal crack.

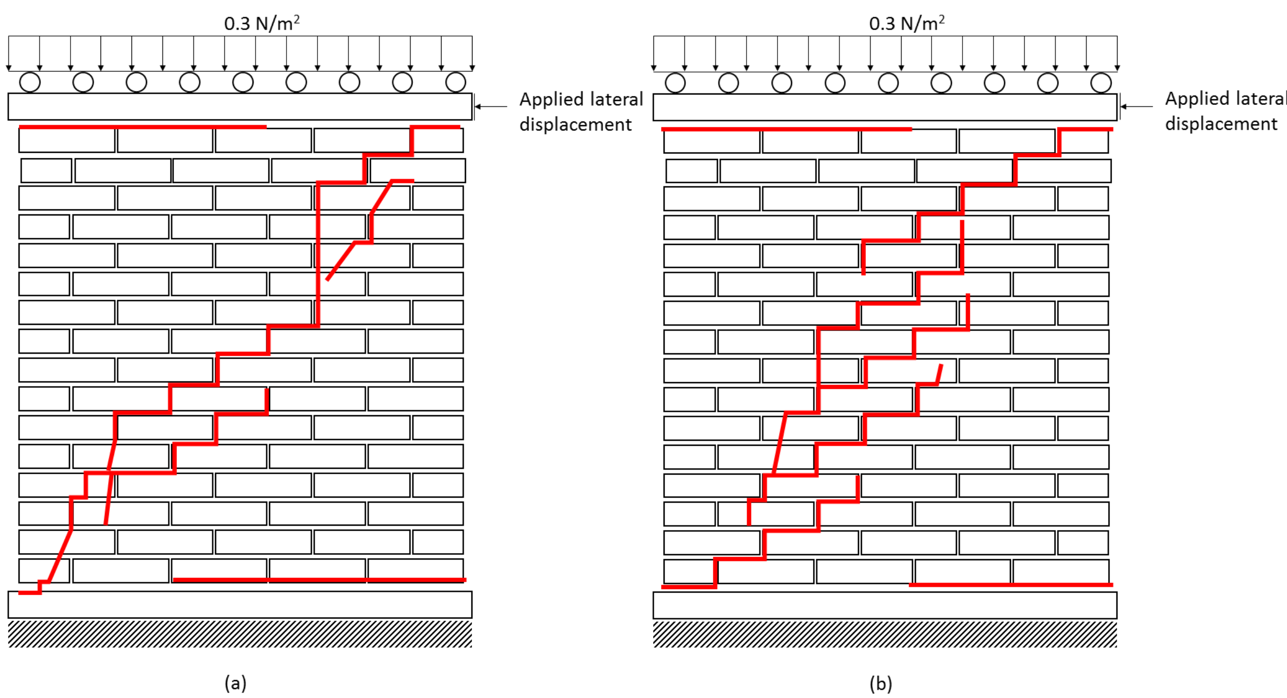


Figure 17: Crack patterns at 4mm of laterally applied displacement, for experimentally tested shear walls: (a) test specimen 1 (b) test specimen 2 (Lourenco, 1996)

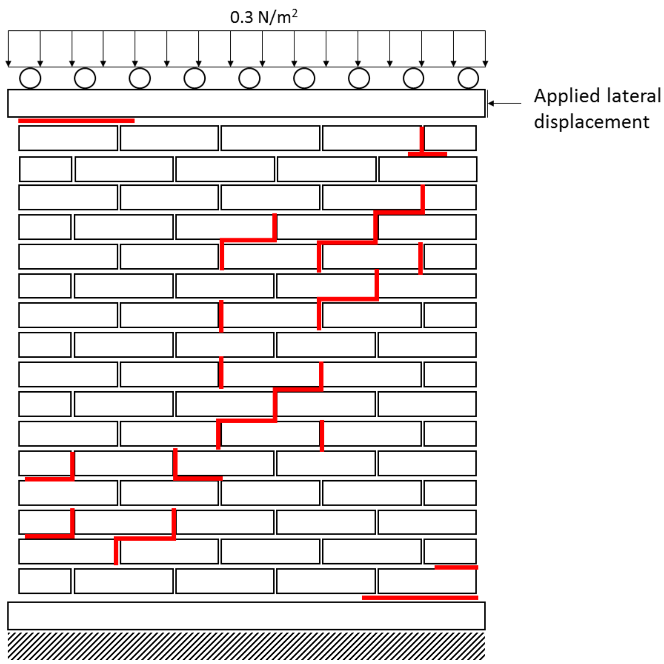


Figure 18: Crack patterns at 4mm of laterally applied displacement for FE model of shear wall without an opening

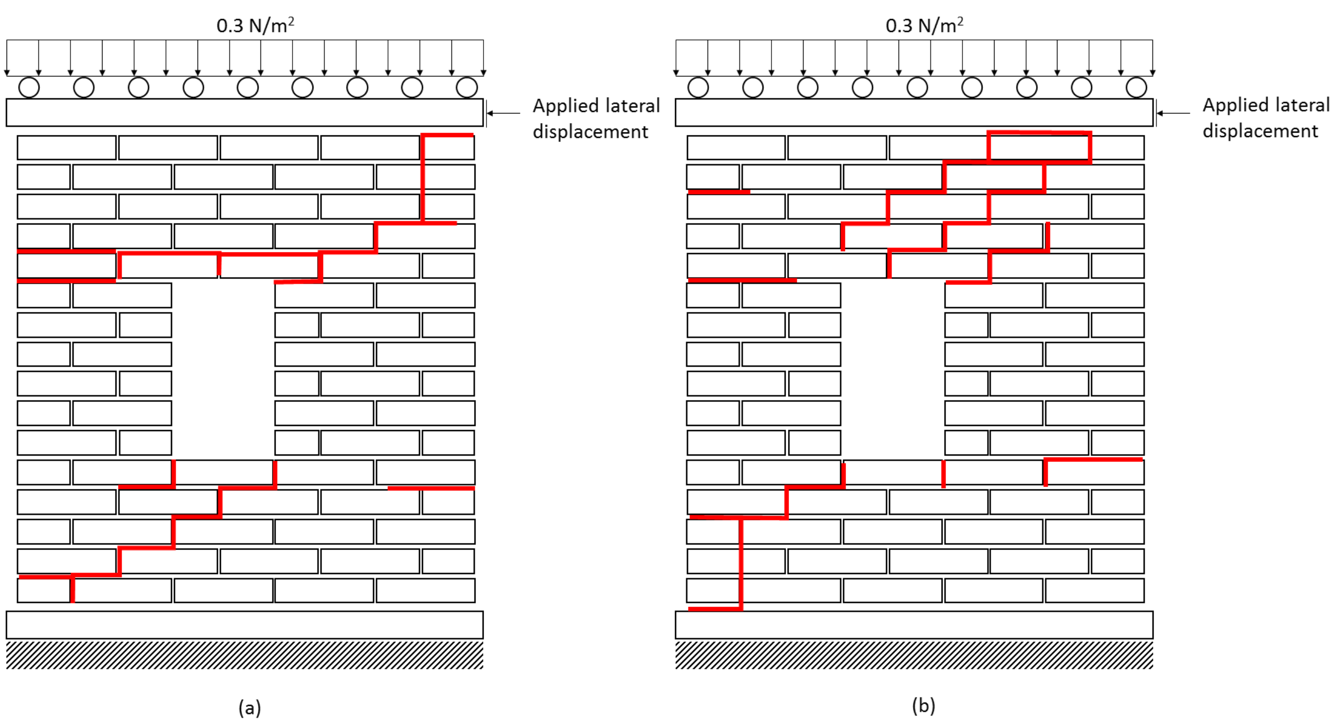


Figure 19: Crack patterns at 4mm of laterally applied displacement, for physically tested shear walls with an opening: (a) test specimen 1 (b) test specimen 2 (Lourenco, 1996)

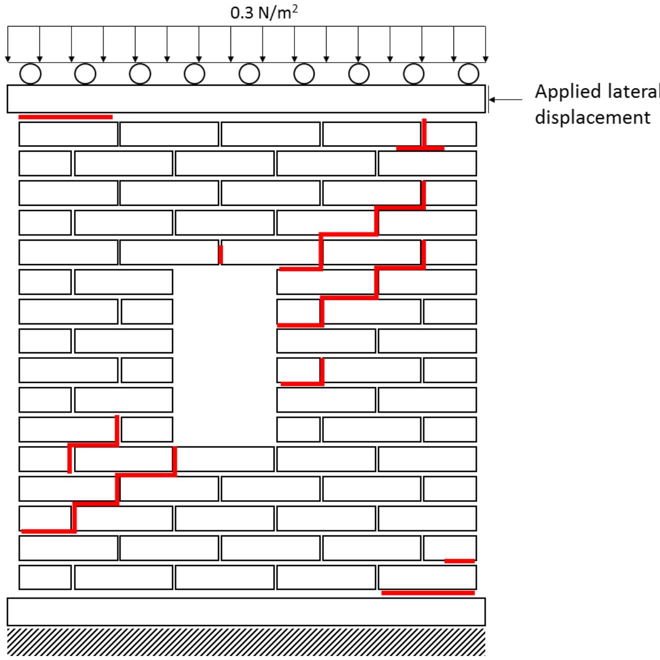


Figure 20: Crack patterns at 4mm of laterally applied displacement for FE model of shear wall with an opening

4.2.4.2 Load vs. displacements graphs

When comparing the results from the simplified model with that of the experimentally tested shear walls and the complex model developed by Dolatshahi (2013) it was evident that the simplified model is capable of replicating the behaviour of masonry assemblages to a level of acceptable accuracy.

The same material properties were used for both walls with and without opening. The walls with openings yielded FE results between the 2 experimental results. The FE results for the walls with no openings yielded good results up to the elastic limit and deviated slightly thereafter. This can be attributed to the plastic behaviour of the materials in Tables 5(a), 5(b) and 6.

From the results illustrated in Figure 21 and Figure 22 it is clear that the walls are very ductile and can undergo large post-peak deformation at a reduced strength. This type of behaviour will significantly help with the ability masonry structures have to withstand seismic activity.

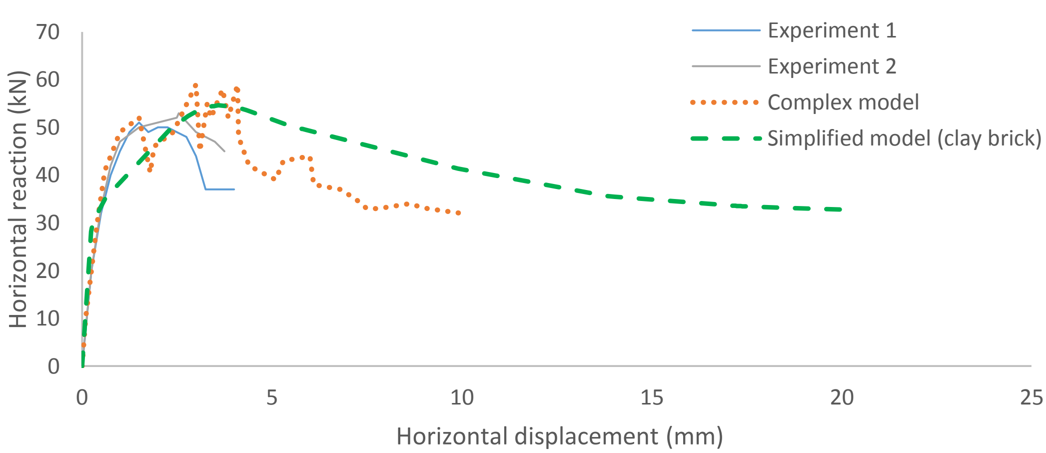


Figure 21: Results for shear wall without an opening

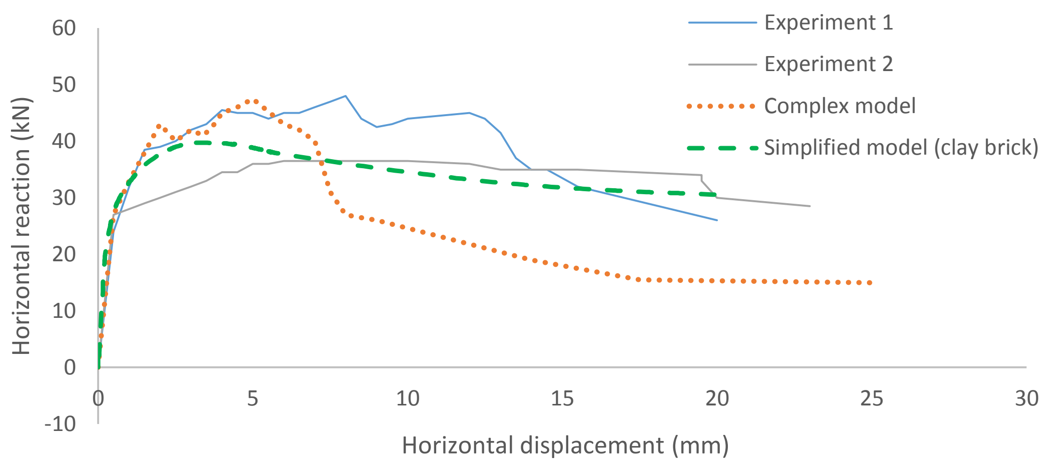


Figure 22: Results for shear wall with an opening

4.3 Material properties validation of HCC blocks

4.3.1 Introduction

Due to the uncertainties that surround this investigation it was decided to use the HCC block material properties from literature in the FE models. This option was chosen due to the time consuming nature of the experimental tests and the limited time to complete this thesis as well as the significant cost associated in obtaining these parameters experimentally. A description of the experimental tests required to obtain all the properties of HCC blocks are include in Appendix A. Determining the material properties through experimental testing would only be necessary if all the other uncertainties were accurately known. The uncertainties surrounding this investigation are attributed to the following factors:

1. The magnitude of the expected maximum PGA
2. The properties of the soil on which the structure is built as well as the surrounding soil conditions
3. The distance the structure under consideration is from the epicenter of the earthquake
4. The quality of construction
5. The quality of the building materials.

In order to confirm that the chosen material properties are capable producing realistic results it was decided to model an experimentally tested shear wall constructed of HCC blocks and compare results. Unfortunately no experimental tests on HCC blocks walls were obtained in literature. However, HCC block properties where determined by Bolhassani (2015).

In an attempt to circumvent this short coming, an HCC block wall with the same dimensions as the clay brick shear walls which were experimentally tested by Raijmakers and Vermelfoort (1992 & 3) was modeled in Abaqus. It was hypothesized that the HCC block wall should yield an ultimate lateral force within 25% of that of the clay brick wall's ultimate lateral force. If this is proven correct, the material properties used from literature will prove realistic and therefore sufficient to use in this investigation.

4.3.2 Implications of using material properties from literature

The material properties of the HCC blocks were experimentally determined by Bolhassani (2015). Although the HCC blocks tested by Bolhassani (2015) originate from USA, they are similar to the HCC blocks found in South Africa.

The American HCC block has dimensions of 397x194x194mm, whereas the South African standard size for a HCC block is 390x190x190mm. The factored compressive strength of South African HCC blocks is 7MPa whereas the American blocks showed a maximum compressive strength of 8.95MPa. This comparison illustrates how similar the two types of blocks are.

The mortar used by Bolhassani (2015) had a compressive strength of 12.6MPa. This value falls between the two classes specified for South African mortar, namely class one and class two mortars with an un-factored strengths of 14.5MPa and 7MPa, respectively (Mortar mixes for masonry, 2009). Class one mortar is specified for lightly stressed load bearing masonry whereas class two mortar is specified for normally stressed masonry. Because of small compression loads which low cost housing units are required to resist, it is safe to assume that a class two mortar is used in practice. This slight difference in material properties is deemed acceptable as South African mortar is weaker than the mortar used by Bolhassani (2015) and thus will result in a slightly more conservative representation of a South African low cost house.

4.3.3 Layout of HCC block shear wall

Figure 23 illustrates the HCC block shear wall, which was modeled in Abaqus V6.14. Both the dimensions of the wall and the modeling procedures are the same as that of the models simulated using clay bricks in section *"4.2 Modelling approach validation".*

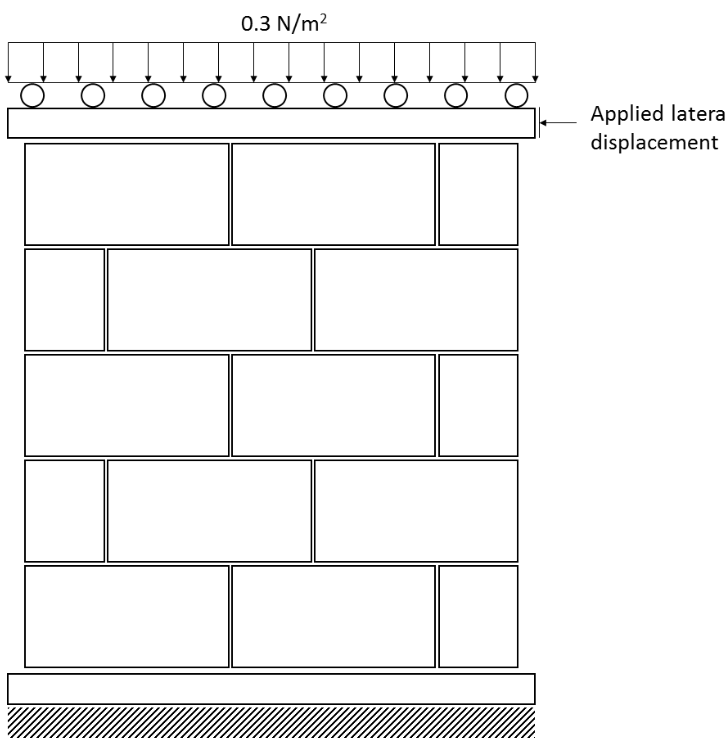


Figure 23: Concrete block shear wall

4.3.4 Material input properties

Table 7 & 8 present all the material properties which were required to develop the FE model in by Abaqus, Bolhassani (2015).

Table 7(a): HCC block material properties (Bolhassani, 2015)

HCC block properties

Mass	Elastic		Plastic				
Density (kg/m3)	Young's Modulus (GPa)	Poisson's ratio	Dilation angle	Eccentricity	f_{bo}/f_{co}	K	Viscosity parameter
1000	13.1	0.2	32	0.1	1.16	0.67	0.001

Table 7(b): HCC block material properties (Bolhassani, 2015)

Additional HCC block properties

Compressive behaviour	
Stress (MPa)	Strain
6.9	0.00000
8.6	0.00012
8.95	0.00032
6.9	0.00057
3.8	0.00131
2.2	0.00191
1.1	0.00245
0.5	0.00295

Tensile behaviour	
Stress (MPa)	Strain
0.69	0.00000
0.67	0.00006
0.36	0.00028
0.23	0.00045
0.13	0.00079
0.06	0.00139
0.03	0.00299
0.02	0.00349

Table 8: Cohesive interface properties (Bolhassani, 2015)

Cohesive interface properties

Tangential behaviour	Normal behaviour	Cohesive behaviour (N/mm^3)			Damage				
Frictional coefficient	Type				Initiation (MPa)				Evolution
		K_{nn}	K_{ss}	K_{tt}	t^0_n	t^0_s	t^0_t	Criterion	δ_p (mm)
0.39	"Hard"	1.875	2.625	2.625	0.075	0.105	0.105	MAXS	1.96

4.3.5 Results

It is apparent from the load vs. displacement graphs illustrated in Figure 24, that the HCC block wall behaves similarly to that of the clay brick walls, however it does differ in strength and in its dominate failure mechanism.

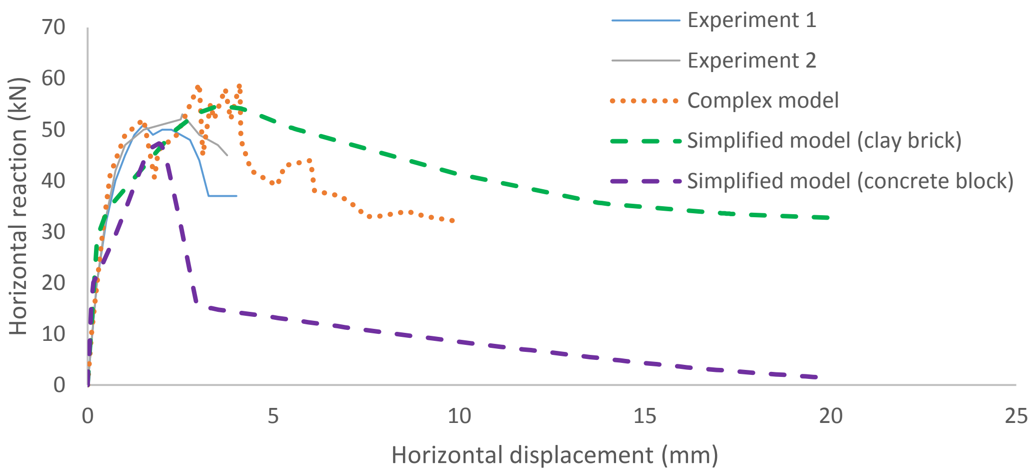


Figure 24: Results of concrete block wall alongside previously calculated results

4.3.5.1 Reason for lower strength and different dominant failure mechanism

The HCC block walls can resist a lateral force of 88.5 % of the tested clay walls. This decrease in strength is believed to be due to the layout of the HCC block wall as well as the large size of the concrete blocks that do not allow for a clear diagonal to form from the top right corner to the bottom left corner. This leads to the shear sliding of the top coarse of bricks to be the dominant failure mechanisim. Without diagonal cracks forming it is believed that the capacity of the bricks do not reach their full potential.

Figure 25 illustrates the min principal stresses in the concrete block wall at various stages of its loading. At the point of ultimate lateral force, the compression struts that form within the wall is visible in Figure 25(a). However, because no clear diagonal crack path is able to form, the top coarse of the wall fails in sliding shear, as seen in Figure 25(b).

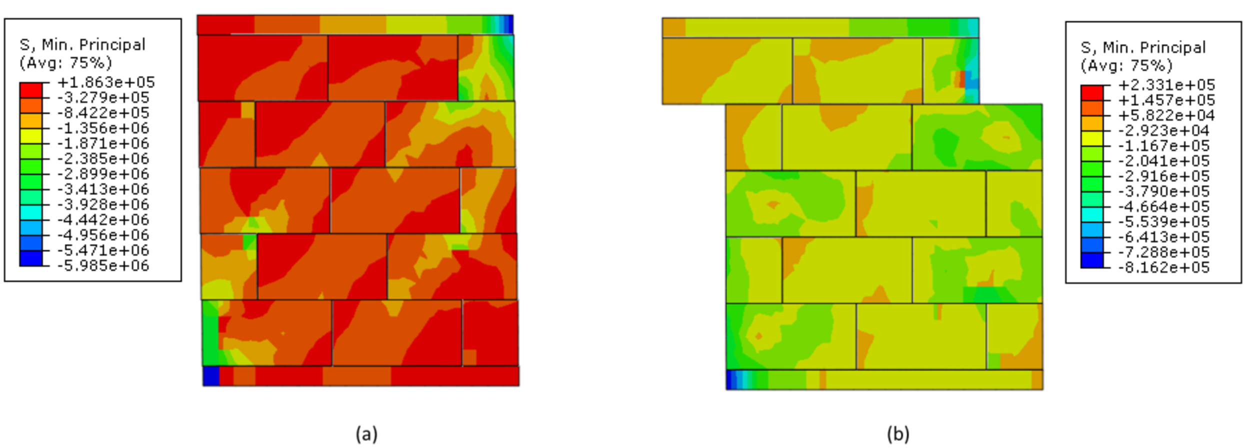


Figure 25: (a) Min principal stresses at ultimate lateral force and (b) at maximum applied displacement

The author finds that these results prove the hypothesis correct as the model with the concrete block wall produce realistic results. The material properties which where experimentally determined by Bolhassani (2015) will thus be used for this investigation in simulation a low income house.

4.4 FINITE ELEMENT MODEL

4.4.1 Introduction

In this section, the FE model developed to represent the unreinforced masonry structure proposed in *"2.1.3 Low cost housing layouts"* is presented. As stated in section *"3.2 Modeling masonry using Finite Element Analysis"* the simplified modelling approach developed by Bolhassani (2015) will be used to create the FE model that will be used in this investigation. Only the application of Bolhassani's (2015) approach will be discussed in this section.

The description of the model is split into four sections. The first section illustrates what types of elements were used to construct the FE model. This is followed by a description of the boundary conditions which were imposed on the FE model in order to allow it to function as a physical structure would. The third section explains how the ground acceleration records of the various earthquakes were obtained and applied to the FE model. Lastly, the fourth section explains the steps used to incrementally execute the simulation are discussed.

4.4.2 Elements

4.4.2.1 Concrete blocks & lintels

Figure 26(a) illustrates that 3D solid elements were used to represent the concrete blocks, the foundation slab and the reinforced concrete lintels. Because the RC lintels have a significantly higher strength than the concrete blocks, it was assumed that they would not fail in tension or compression. To incorporate this assumption into the FE model, the RC lintels where modeled as a purely elastic material with a high modulus of elasticity.

The purpose of incorporating the foundation slab into the FE model was to allow the seismic loading to be easily applied to the base of the structure. The purpose of the foundation slab is purely to distribute the seismic loading and thus it was modeled as a stiff and purely elastic material with a high modulus of elasticity.

To ensure that the proposed FE model was capable of producing accurate results, a mesh size of 100mm was used in this investigation. This resulted in sixteen 8-node linear brick elements per concrete block as opposed to the clay bricks used in section *"4.2 Modelling approach validation"* which only had eight elements per brick.

4.4.2.2 Cohesive interaction

To model the interaction between the bricks, cohesive interfaces where created for all surface pairs that experience contact. A portion of these surface pairs that experience contact is illustrated Figure 26(b).

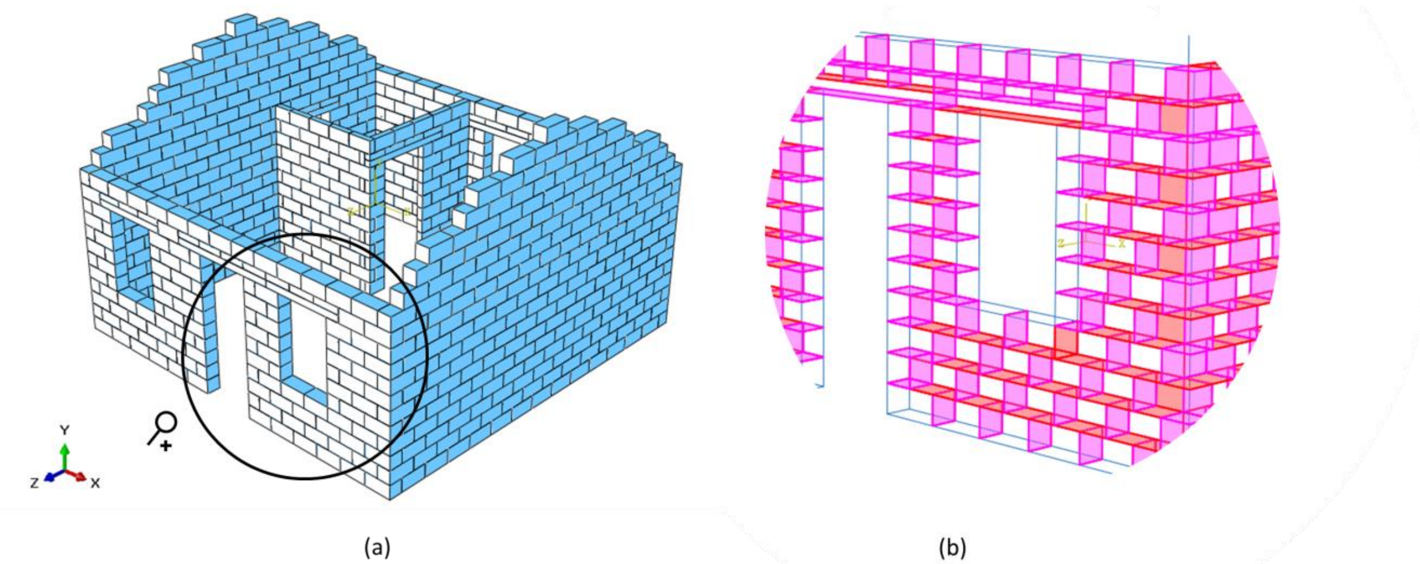


Figure 26: (a) 3D solid elements (b) Portion of surface pairs experiencing contact

4.4.2.3 Roof structure

A lumped mass, which represented the entire roofing structure's mass, was located at the centroid of the roof's structure. This lumped mass was then connected to the apex of the walls where the trusses are located through rigid links, as presented in Figure 26.

The roof structure consisted of two main components, i.e. the solar geyser of 200kg and the roof truss and sheeting system of 1000kg. The combined centroid of these two components was calculated and positioned in the center of the building at a height of 0.8m above apex of the interior walls.

4.4.3 Boundary conditions

Two types of boundary conditions where required in the FE model. Firstly, the translation of the base of the model was restrained in all directions, as presented in Figure 28. The second boundary condition is illustrated Figure 27, the proposed boundary condition provides a rigid link connection between the lumped mass and the points where the trusses of the roof structure rests on the walls of the building.

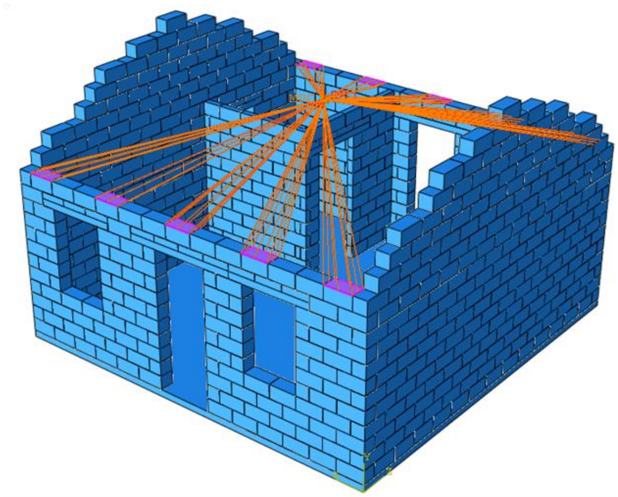


Figure 27: Rigid links connecting the lumped mass to the rest of the building

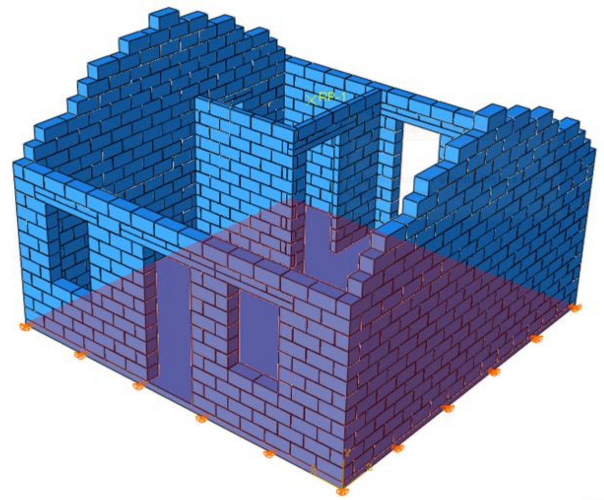


Figure 28: Base fixities

4.4.4 Loading

A gravity load was incorporated to the model to incorporate the mass of each component in the model.

The seismic loading of each earthquake was applied to the structure by prescribing the acceleration to the base of the model.

4.4.5 Analysis steps

In order to simulate an earthquake on the model, the analysis was divided into two steps. The first step “Static-General”, three translation restraints were applied to the base of the model as well as the gravity load. The second step “Dynamic - Implicit”, simultaneously modifies the restraints of the base of the model so that it is only restrained in the vertical direction, as well as applying the acceleration record of the earthquake history to the base of the model in the two horizontal directions.

4.5 LOADING SCHEME

4.5.1 Introduction

The goal of this investigation is to determine the affect of moderate intensity seiemic activity on low cost housing units in South Africa. To provide insight to this question, it was decided to perform a time history analysis on the developed FE model. This entails subjecting the FE model to the maximumn probable earthquake loadings that the region could experience. The FE model would then illustrate the degree of damage/collapse of the housing unit during the simulation.

To determine what earthquake records should be used in this time history analyses, guidelines defined in EN 1998-1:2004 section 3 were used. The first step of this process entails constructing an elastic response spectrum for the worst possible case that could occur in the Southern Western region of the Western Cape Province of South Africa. From Figure 29, ground type D, which represents loose and soft soil, results in the largest seismic load. It must also be noted that when constructing these elastic reponse spectrum a PGA of 0.23g recommended by Kijko (2003) was used, as apposed to 0.15g specified in SANS 10160-4 (2011).

Using the PEER Ground Motion Database, the author was able to select earthquake records that matched the elastic response spectrum for a ground type D soil. The comparison between the elastic spectrum for a ground type D soil and the spectrum of the chosen earthquakes is presented in Figure 30. EN 1998-1:2004 stipulates that a minimum of three earthquake records be used when performing a time history analysis.

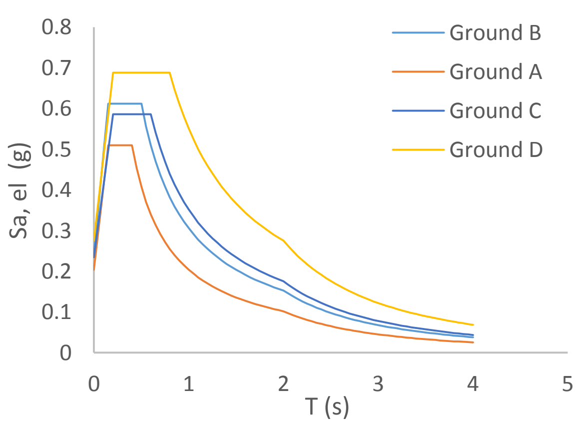


Figure 29: Elastic response spectrums of various ground types (SANS 10160-4, 2011).

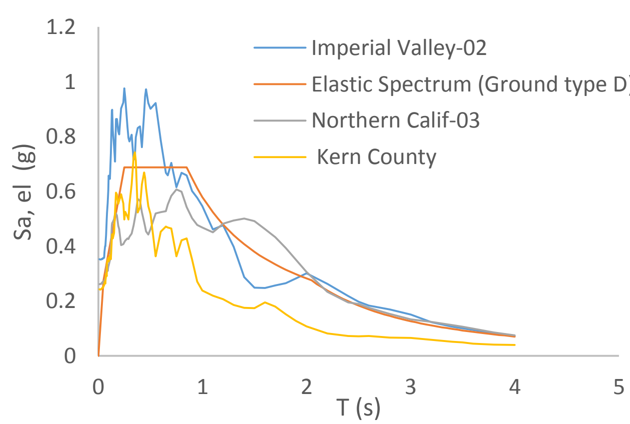


Figure 30: Elastic response spectrum of ground type D and chosen earthquake records

4.5.2 Chosen earthquake records

The selected portions of the horizontal ground acceleration recordings in the North-South and East-West directions for the three chosen earthquake are presented in Figure 31-33.

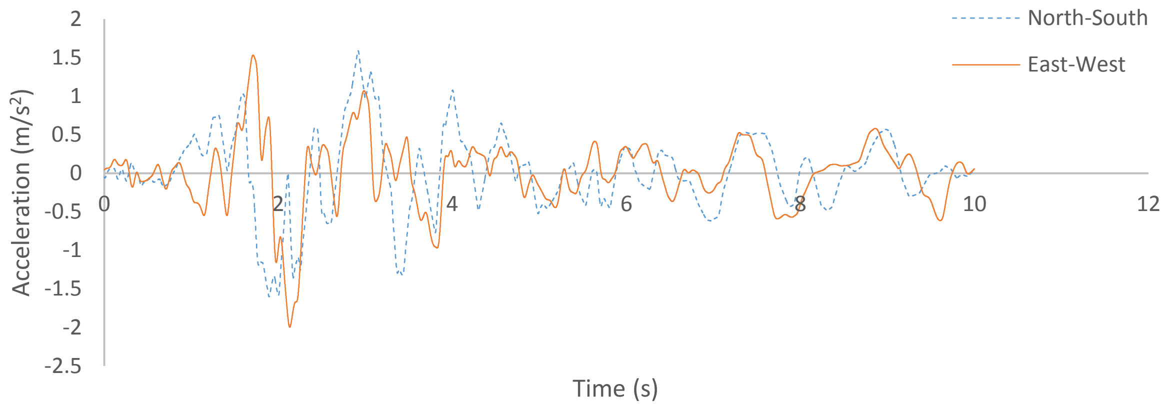


Figure 31: Horizontal ground acceleration records for Northern Calif-03 earthquake, maximum PGA = 0.20g (Station name: "Ferndale City Hall")

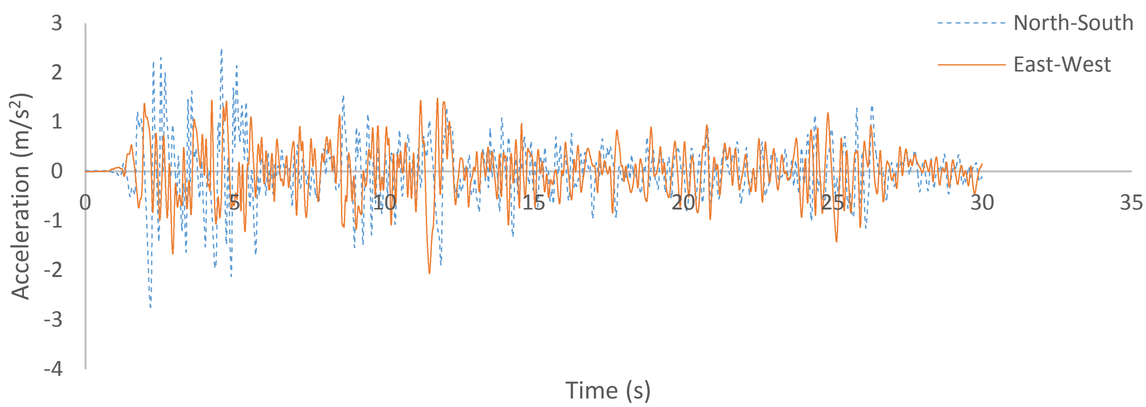


Figure 32: Horizontal ground acceleration records for Imperial Valley-02 earthquake, maximum PGA = 0.27g (Station name: "El Centro Array #9")

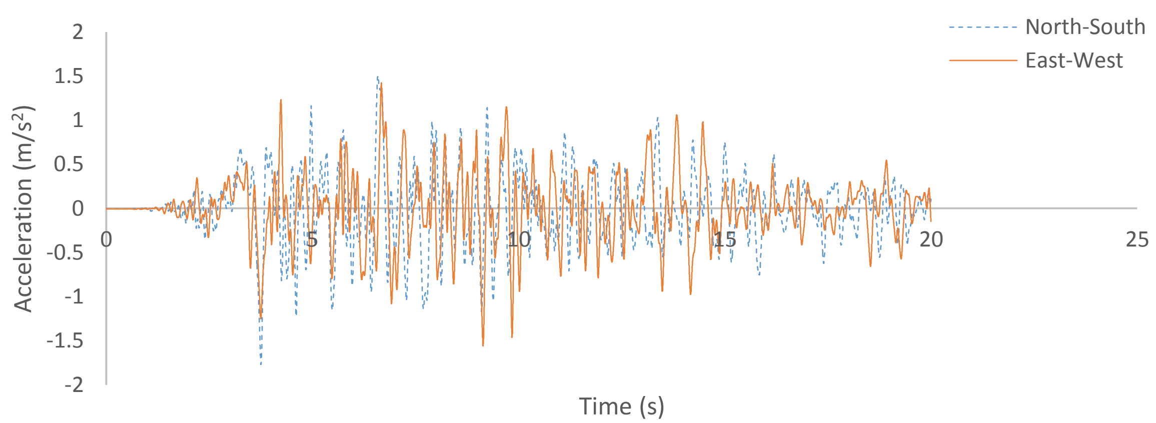


Figure 33: Horizontal ground acceleration records for Kern County earthquake, maximum PGA = 0.175g (Station name: "Taft Lincoln School")

4.5.3 Loading procedure

Each of the earthquake recordings will be applied to the structure in four directions. The first analysis was completed with the earthquake accelerations applied parallel with the axes of the walls. The applied accelerations was then incrementally rotated through an angle of 45 degrees for each of the remaining directions, as shown in Figure 34. The reason that these four directions where chosen was that it results in all walls being tested in both their in and out of plane directions as well as a combination of the two as well as to determine the maximum forces (i.e. maximum shear stresses) for a given earthquake loading.

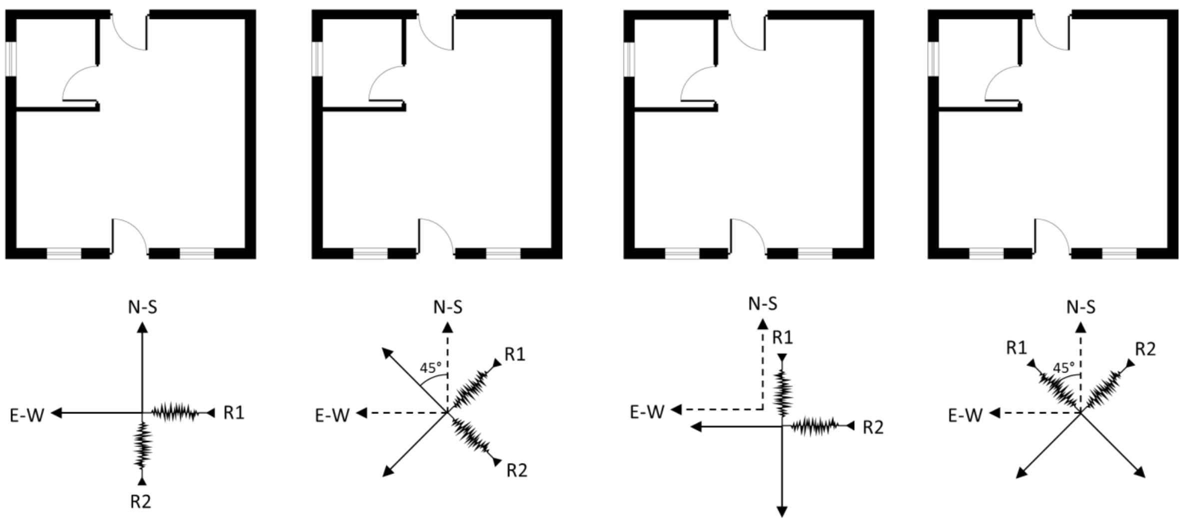


Figure 34: Directions of applied seismic loading

5 RESULTS

5.1 INTRODUCTION

In this chapter, the results from the FE model will be reviewed to provide insight into the behaviour of a typical low cost income house subjected to seismic loadings.

5.2 VALIDATION THAT THE SEISMIC LOAD WAS APPLIED CORRECTLY

Before the results of the simulations can be analyzed, it must first be confirmed that the earthquake records were applied correctly and that the FE model yield the correct results.

To verify that the acceleration time histories which are applied to the FE model yield the correct ground displacements at the base of the structure, the ground displacements obtained from the FE model will be compared to the displacement records of the earthquake. Figure 35 illustrates that the displacements obtained from the FE model are an exact match to the displacement records of Imperial Valley-02 earthquake. This shows that Abaqus correctly applies the acceleration time history profile to the model to obtain the correct displacement time history.

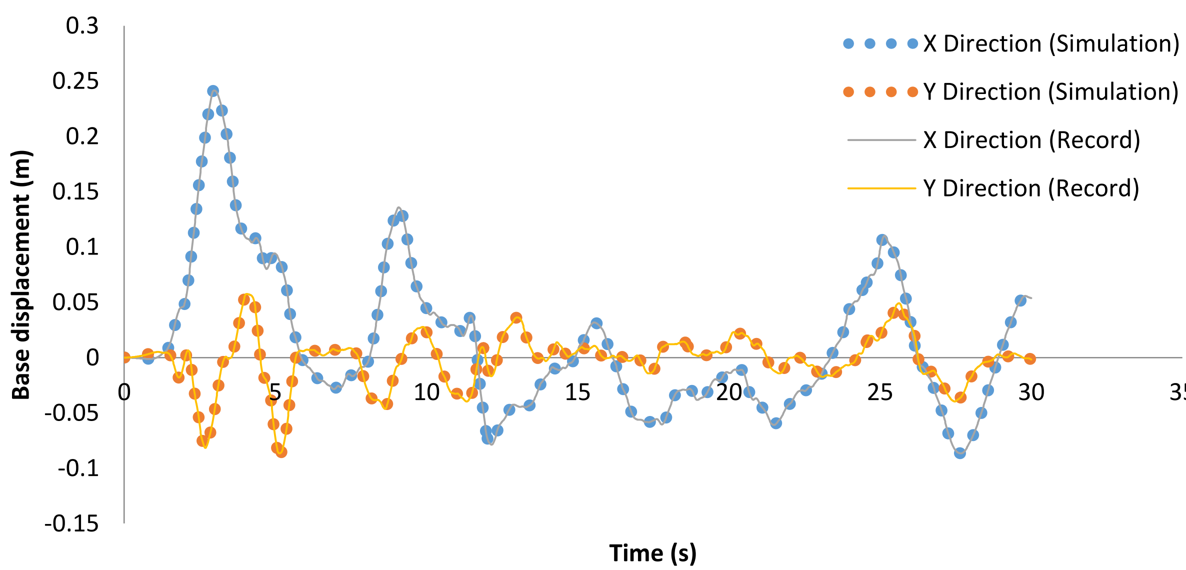


Figure 35: Base displacement comparison

5.3 POST SIMULATION ANALYSIS OF THE FE MODEL

5.3.1 Introduction

The aim of analyzing the post simulation analysis of the FE model was to determine whether damage would occur, the extent of the damage and whether the damage is sufficient to cause failure.

The detail of the meso-modeling approach used in this investigation allows the user to analyze the post simulation condition of the FE model in the same manner as for a physically tested structure, which is through the formation of cracks. The cracks are formed when the stress within the FE model exceed the allowable stresses. In section *"4.2 Modelling approach validation"* it was illustrated that the modeling approach used in this investigation was capable of showing the dominant crack patterns which would appear in a masonry structure when it exceeded the maximum lateral force.

5.3.2 Damage sustained by FE models

After completing the simulations for each of the three applied earthquake, it was observed that cracking did not occur in any of the FE models. This can be interpreted that FE models are still structurally stable and that the allowable stresses where not exceeded.

To illustrate (in the simulated FE models) where cracks would start developing, the maximum stresses which occured in each of the simulations was compared to the yield stresses for each type of failure that could occur within a masonry structure. Figures 36-38 illustrate the comparitive stress for the "Northern Calif-03", "Imperial Valley-02" and "Kern County" earthquake simulations, respectively.

When deriving Figures 36-38, the allowable compression and tensile stresses of the HCC blocks were compared to the minimum and maximum principal stresses within the FE models, respectively. The maximum shear and tensile stresses that can be resisted by the bond between the HCC blocks were compared to the maximum shear and the normal stresses within the FE models, respectively. S12, S23 and S13 refer to the shear stress variables in Abaqus, whereas S11, S22 and S33 refer to the normal stress variables in Abaqus.

The maximum stress results illustrate the effect that changing the direction at which the earthquake histories were applied to the FE models had on the degree of stress which were experienced by the FE models. It was observed that changing the direction along which an earthquake record was applied could result in a maximum stress variation of 4-66 %. This illustrates the importance of applying earthquake records in multiple directions.

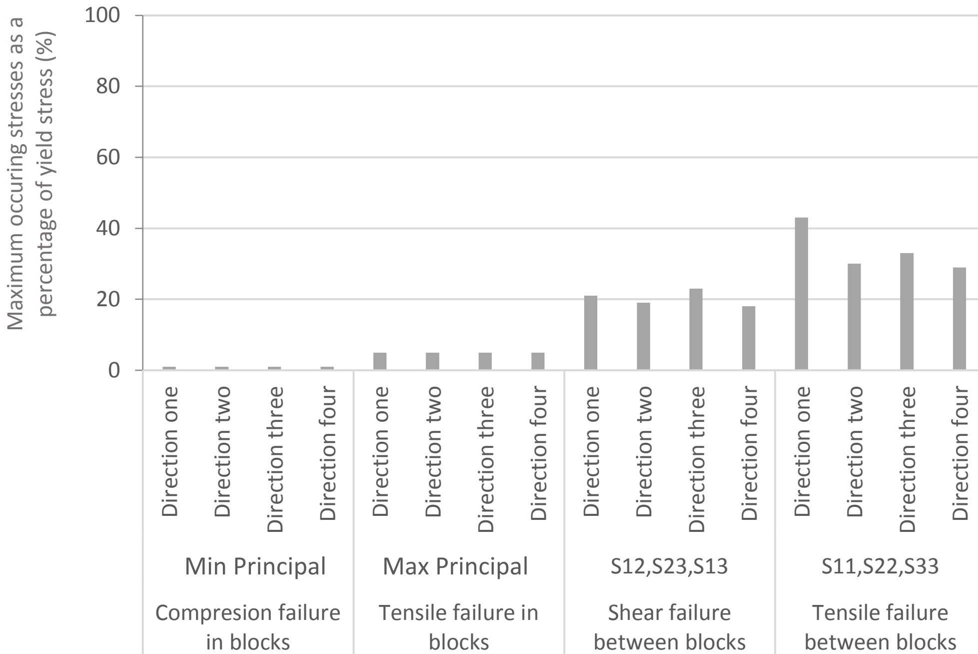


Figure 36: Maximum stresses incurred during Northern Calif-03 earthquake simulation (maximum PGA = 0.20g)

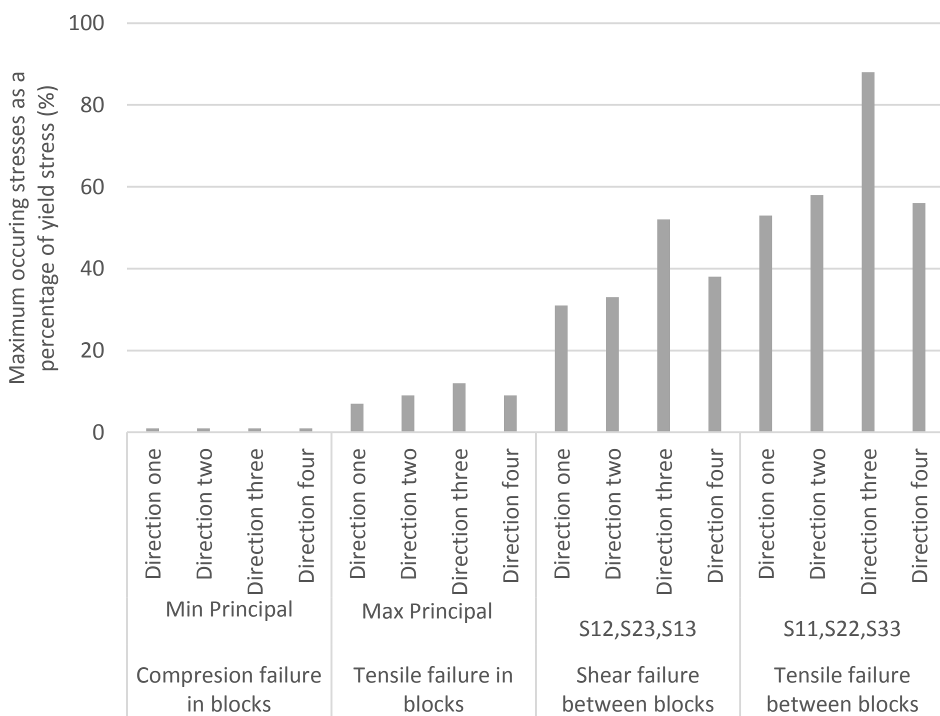


Figure 37: Maximum stresses incurred during Imperial Valley-02 earthquake simulation (maximum PGA = 0.27g)

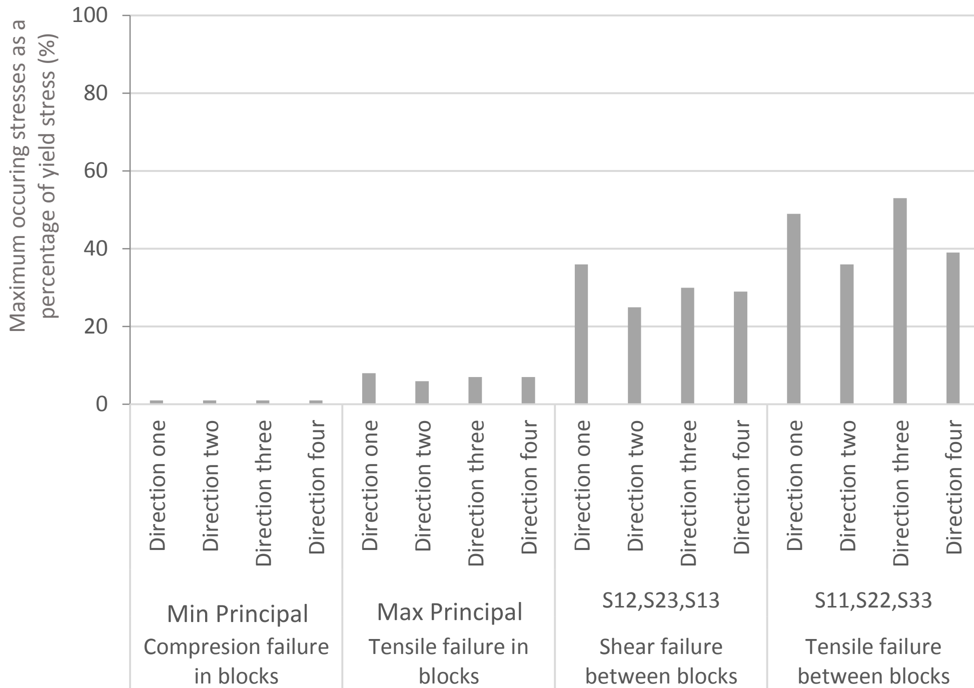


Figure 38: Maximum stresses incurred during Kern County earthquake simulation (maximum PGA = 0.175g)

5.3.3 Observations made from maximum stress results

Compression and tensile failure in the blocks is insignificant as their maximum stresses as a percentage of their yield stress is less than 2 % and 12 %, respectively. Shear and tensile failure between the blocks are the critical failure mechanisims as they reach maximum stresses of 52 % and 88 % of their yield stresses, respectively.

This confirms why cracks did not occur in any of the FE models.

5.4 BEHAVIOUR OF FE MODELS DURING SEISMIC LOADING

Table 9 illustrates a time series of the deformations which occurred during the first Northern Calif-03 earthquake loading. Only one simulation is shown as the illustration is intended for general observation purposes.

Table 9: Time-lapse illustration of deformations occurring in the FE model during the first Northern Calif-03 earthquake loading

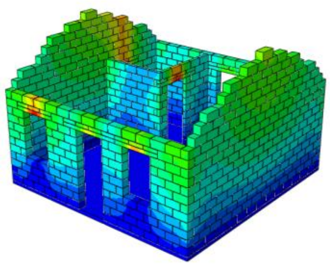
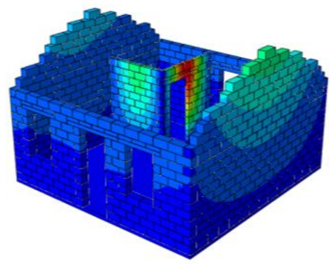
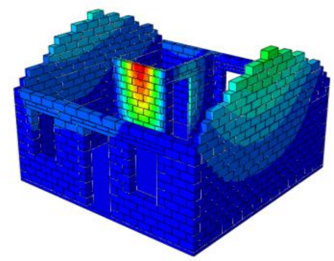
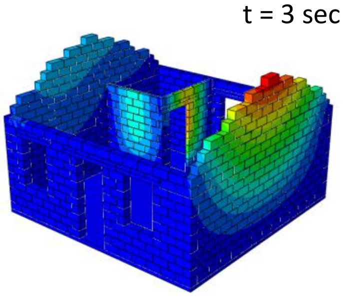
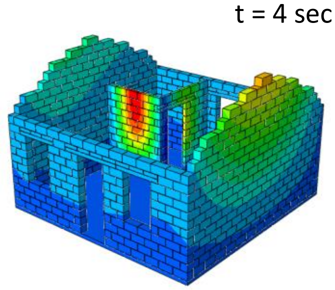
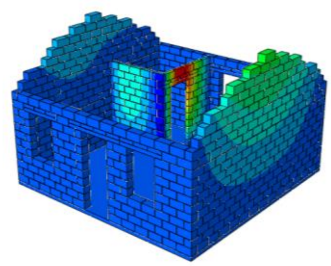
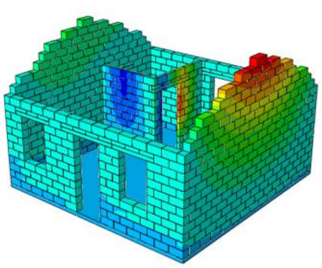
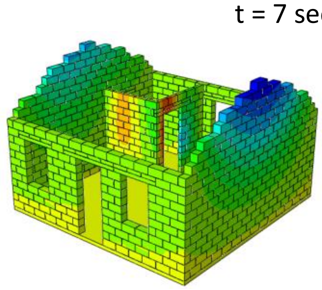
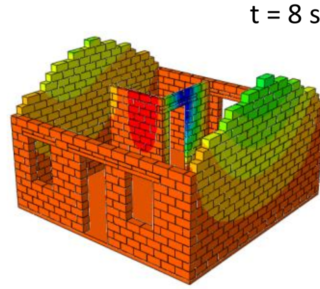
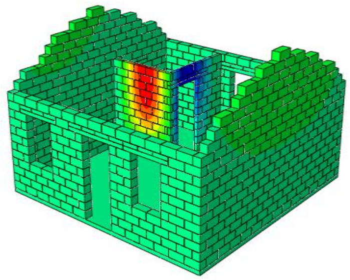
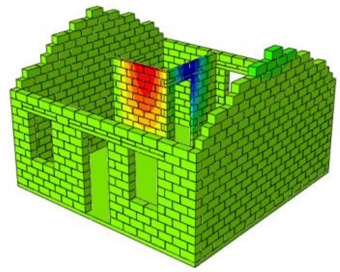
t = 0 sec  **Key** 0.000 mm 0.007 mm 0.010 mm	t = 1 sec  **Key** 0.000 mm 0.171 mm 0.466 mm	t = 2 sec  **Key** 0.000 mm 0.313 mm 0.694 mm
t = 3 sec  **Key** 0.000 mm 0.350 mm 0.484 mm	t = 4 sec  **Key** 0.000 mm 0.463 mm 0.477 mm	t = 5 sec  **Key** 0.000 mm 0.167 mm 0.293 mm
t = 6 sec  **Key** 0.000 mm 0.151 mm 0.036 mm	t = 7 sec  **Key** 0.000 mm 0.035 mm 0.109 mm	t = 8 sec  **Key** 0.000 mm 0.056 mm 0.371 mm

Table 9: Time-lapse illustration of deformations occurring in the FE model during the first Northern Calif-03 earthquake loading (cont.)

t = 9 sec	t = 10 sec
	
Key	**Key**
0.000 mm	0.000 mm
0.117 mm	0.118 mm
0.054 mm	0.216 mm

5.4.1 Observations made from presented deformations

The first observation encountered was that the majority of the deformations occur in the gable walls and the interior walls. The insignificant deformation in the front exterior wall which contains the a door and two window openings were unexpected.

The effect of the interior walls on the exterior walls is seen by comparing the deformations of the gable walls which is supported by an interior wall and the other gable wall which is unsupported. This comparison illustrates that if an exterior wall is connected to an interior wall it will undergo significantly less deformation and thus be less prone to failure.

Another unexpected behavioural aspect was that the largest observed deformations were more prone to occur in the interior walls than in the exterior walls. This observation illustrates that failure may first occur in the interior walls. It is however not possible to confirm this statement based on the presented deformation results. The stress distributions within the structure provides better insight when determining where failure would occur first.

5.5 MAIN AREAS OF POTENTIAL FAILURE WITHIN THE EVALUATED FE MODELS

The deformation results of the FE models were reviewed to examine which part of the evaluated FE models would possibly exhibit cracks. As illustrated in section *"5.4.1 Observations made from presented deformations"*, the interior walls of the evaluated FE models experienced greater deformation than the exterior walls in most cases. This observation led the author to believe that the maximum stresses developed during the simulations would also occur within the interior walls and thus proving that the interior walls would be the first to experience cracking and possible failure.

In this section, the stress results obtained from the simulation of the Northern Calif-03 earthquake loading will be used to better understand how the FE model behaved during the simulations.

5.5.1 Where do the maximum stresses occur?

In order to validate the observations made in section *"5.4.1 Observations made from presented deformations"* the maximum stresses which occurred in the interior and exterior walls will be compared against one another.

Figure 39 illustrates that the interior walls do not experience larger stresses than that of the exterior walls, which would imply that cracking develops in the exterior walls first. The reason that the interior walls undergo larger deformations than the exterior walls and remain at a lower stress state is as a result of the interior walls being less stiff than the exterior walls. The stiffness of both walls is directly proportional to their thicknesses. Since the interior walls are half the thickness of the exterior walls.

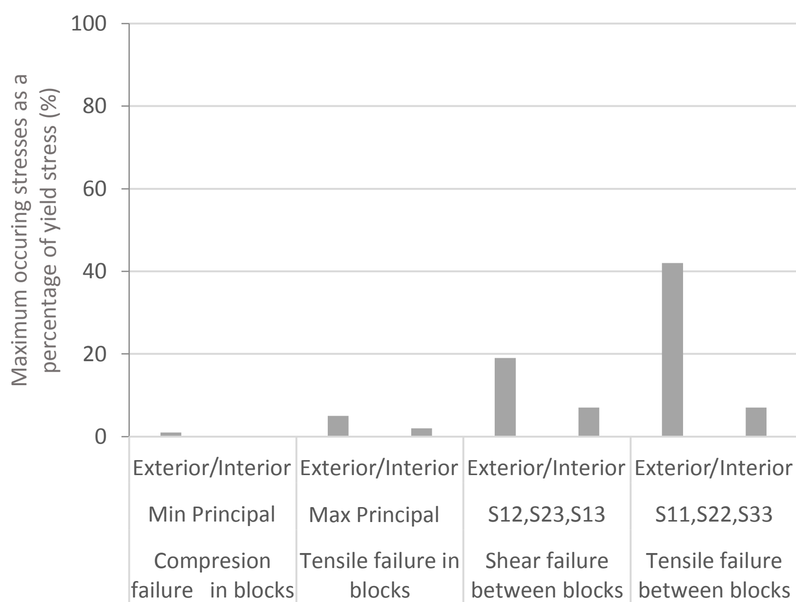


Figure 39: The maximum stresses which occurred in the exterior and interior walls during the Northern Calif-03 earthquake loading

On inspection of the FE models, it was apparent that the gable wall which was not connected to an interior wall experienced the highest stress concentrations. The mechanism that resulted in these high stress concentrations was the oscillating in and out of plane movement of the wall.

To confirm that the gable wall which was not connected to an interior wall did indeed experience the greatest stress concentrations, the maximum stresses which occurred in the unsupported gable wall was compared to the maximum stresses which occurred in the rest of the FE model. As illustrated in Figure 40, it is observed that the statement is correct.

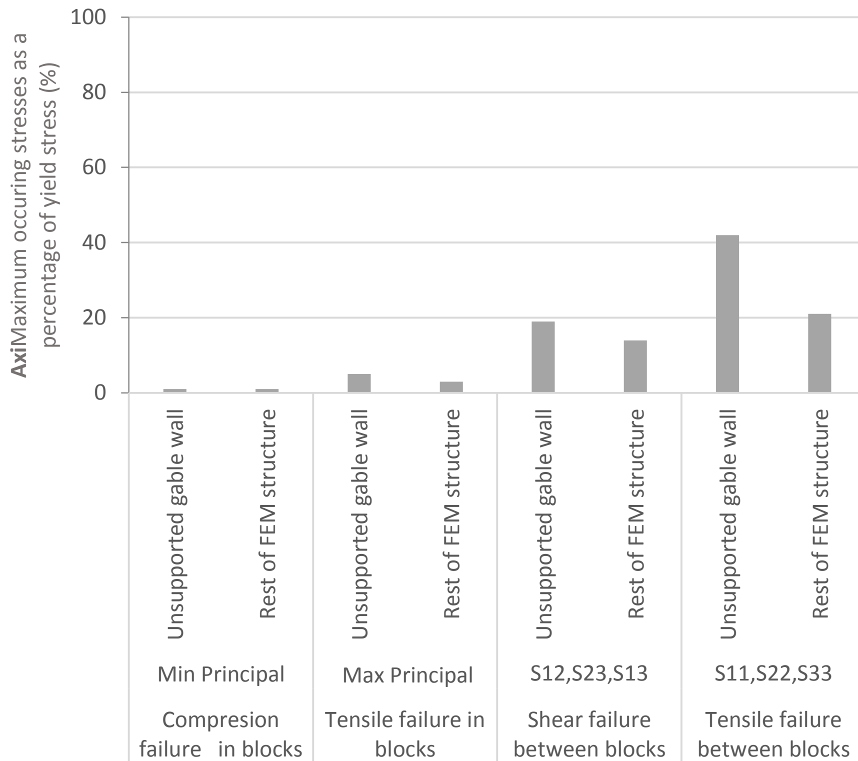


Figure 40: The maximum stresses which occurred during the Northern Calif-03 earthquake loading

5.5.2 What will be the dominate failure mechanism

In order to understand the dominate failure mechanism in the structure, the FE model was analyzed at a point in time when the maximum stresses occurred. The point in time when the maximum stresses occur correlates directly to the moment when the applied earthquake loading reached its maximum PGA of 0.2g, at a point of 2.14 seconds into the simulation.

To obtain a better indication of the performance of the wall at this point, the deformation and the stress distributions within the wall were investigated. The stresses leading to compression and tensile failure of the HCC blocks will not be included in this investigation as section *"5.3.2 Damage sustained by FE models"* illustrated that these types of failure have the least potential of occurring.

Figure 41 presents exaggerated illustrations of the deformations in the unsupported gable wall when the maximum stresses occur. On inspection, it is clear that the wall behaves similarly to that of a concrete slab which has three of its edges restrained while the remaining edge is unrestrained.

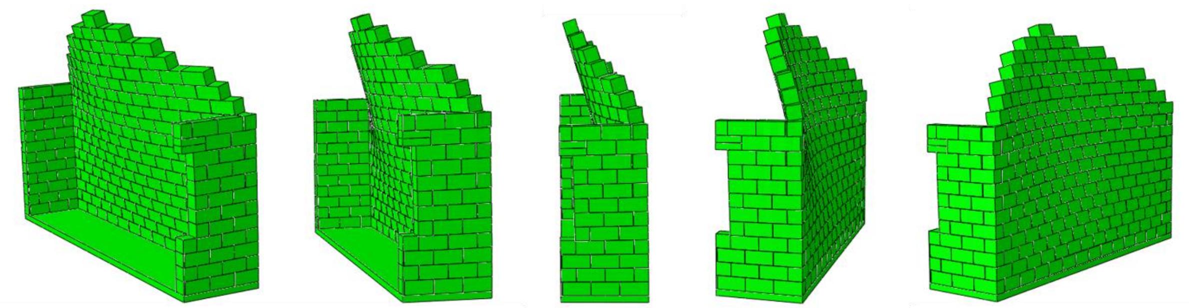


Figure 41: Exaggerated deformation of unsupported gable wall

Table 10 illustrates the in-plane shear stresses which is present in the unsupported gable wall. These stresses are responsible for the shear failure between the HCC blocks. The blue and red areas represent the region of the highest shear stresses. Both regions are similar in magnitude however act in opposite directions.

Table 10: Shear stresses

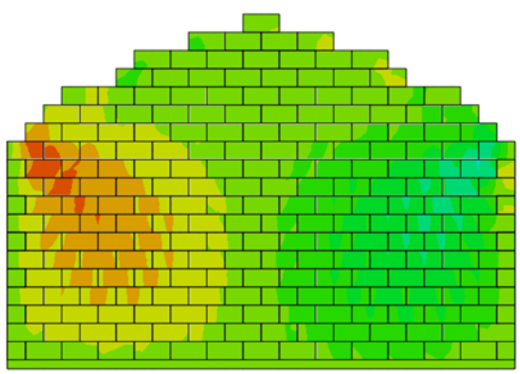
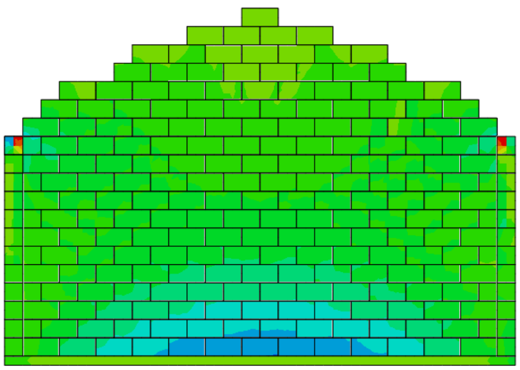
In-plane shear stresses
 (a)
Comments: Figure (a) above represents the in-plane shear stresses which occur on the outer surface of the unsupported gable wall. The shear stresses for the inner surface have not been illustrated as they are the same as that of the outer surface.

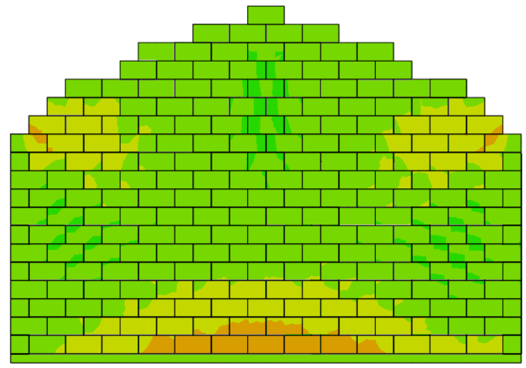
Table 11 illustrates the vertical and horizontal normal stresses which are present in the unsupported gable wall. These stresses are responsible for the tensile failure between the HCC blocks. The red and blue areas represent the regions experiencing the highest tensile and compressive stresses respectively.

Table 11: Normal stresses

Vertical stresses



(a)

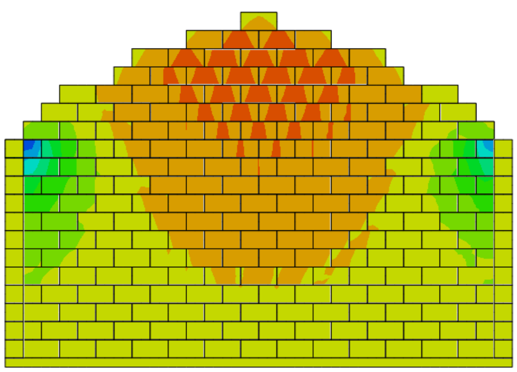


(b)

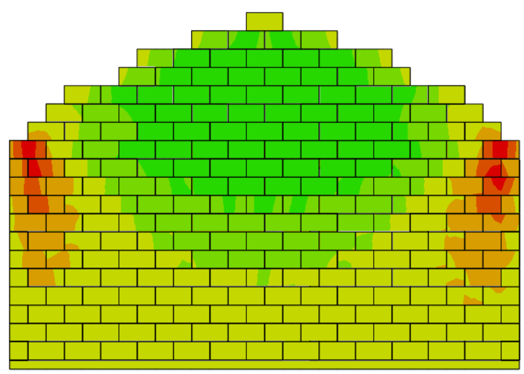
Comments:

Figure (a) and (b) represent the vertical stresses which occur on the inner and outer surfaces of the unsupported gable wall respectively. The connection between the wall and the foundation acts as a fixed connection and induce tensile stresses on the outer surface of the wall and compression on the inner surface of the wall. A similar phenomenon can be seen at the bottom of the gable, the stresses at the bottom of the gable are not as server as at the connection between the wall and the foundation, this is thought to be as a result of the geometrical ratios of the gable in relation to the wall.

Horizontal stresses



(a)



(b)

Comments:

Figure (a) and (b) represent the horizontal stresses which occur on the inner and outer surfaces of the unsupported gable wall respectively. The connection between the unsupported gable wall and the two side walls acts as a fixed connection and induce tensile stresses on the outer surface of the wall and compression stresses on the inner surface of the wall.

5.5.3 Predicted initial crack patterns

The traction-separation law which defines how the interface interaction between the brick elements behave stipulate that some separation occurs between the brick elements as the shear and tensile stresses between the bricks approach yielding stresses, see section *"3.2.2.3.2.1 Damage initiation criterion"* for more information. In order to determine the predicted crack patterns that will occur in the unsupported gable wall the author used a function in Abaqus which allows the user to exaggerate the scale at which the deformation sustained by a FE model is displayed. Figure 42 illustrates the predicted initial crack pattern for the unsupported gable wall.

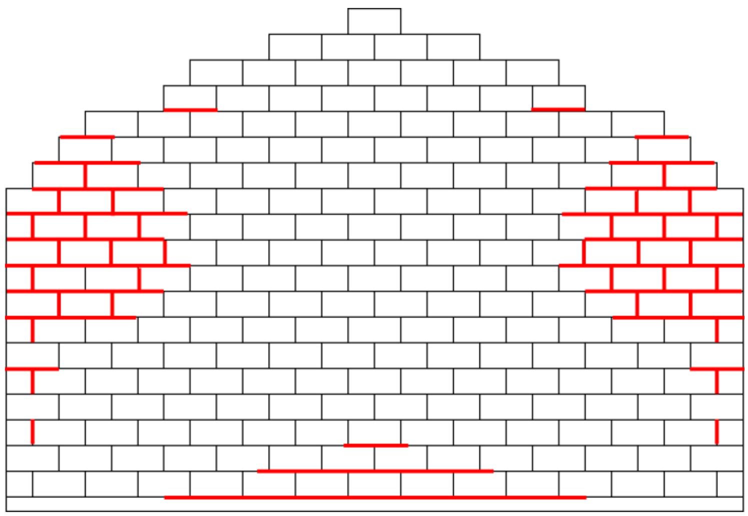


Figure 42: Predicted initial crack pattern for the unsupported gable wall

6 CONCLUSIONS

6.1 INTRODUCTION

In order to present the findings of this investigation in a clear fashion, conclusions will be presented for three sections;

- Conclusions regarding the FE modelling approach used in this investigation
- Results of the FE models
- Risk that the low cost housing units pose to its occupants

After the conclusions of this investigation has been presented, recommendations for future research will be discussed.

6.2 CONCLUSION REGARDING THE FE MODELLING APPROACH USED IN THIS INVESTIGATION

To provide a review of the performance of the FE modelling approach used in this investigation, the advantages and disadvantages which were experienced by the author will be discussed. This will provide additional insight to fellow researchers which could assist them in choosing an appropriate FE modelling technique.

6.2.1 Advantages

The main advantage of using the chosen meso modeling approach was due to its ability to capture the three dimensional behaviour of a masonry structure. This quality allows the FE model to yield a more accurate representation of the physical structure as it allows earthquake records to be applied in any direction, which results in both in and out of plane seismic loading.

Using a FE modelling approach that takes into account each of the failure mechanisms which can occur between the bricks, yields a FE model which is capable of capturing the complex behaviour of URM structures. The uniqueness of the specific URM structure's layout is taken into account by the geometry of the physical model and not through the use of factors. This allows the user to investigate structures without being concerned with how the geometry of the structure needs to be accommodated.

Another advantage of using the meso-modeling approach is the detailed results of the simulation. Information such as the stresses and strains that occur within each brick (element) is accessible to the user throughout each increment of the simulation. The meso-modeling approach also allows users a more time efficient method of evaluating the structural integrity of the FE model, through visual inspection of the crack patterns.

6.2.2 Disadvantages

The modeling approach used in this investigation is however surrounded by number of issues that made it difficult to use.

6.2.2.1 Significant time & computational effort required to complete the FE simulation.

To the run the simulations in this investigation, a specialized computing system was used, as it was not possible to run the simulations on a standard computer. The main reason that it was not possible to run the simulations on a standard computer is due to the large memory requirements (16 GB) of each simulation. Another advantage of using a more powerful computing system was the number of CPU's which were available to use during a simulation. By increasing the number of CPU's in the computing system drastically decreases the time which is required to run a simulation. Despite having access to a specialized computing system, simulations with a 10 & 30 second long earthquake record took 3 and 20 hours, respectively to complete.

6.2.2.2 Post-processing of data

The most inconvenient aspect of using such a detailed FE modeling approach in this investigation was working through the large output file which was created. The output file which can be displayed graphically, allows the user to easily extract information of interest relating to the simulation.

In this investigation, the output file for a simulation with a 10 & 30 second duration earthquake record was 3GB & 7GB, respectively. Due to the large size of this file, the speed at which Abaqus displayed the results of the completed simulation was incredibly slow, even with a high end computer.

Another issue which became apparent was that it was not possible to extract the maximum stresses which occurred in the simulations using Abaqus's post processing tools. This is due to Abaqus needing to compile all the stresses (9 in total) at each nodes, resulting in a total of 30 000 magnitudes at each of the iterations. The process of extracting this data is extremely slow and due to the large computational effort required to complete this data extraction, Abaqus warns the user that the output may be inaccurate. In order to circumvent this obstacle, it was chosen to rather write all the stress data to the "dat" file during the execution of the simulation. This resulted in no additional post processing time being required to obtain the stress data. The "dat" file could easily be imported into Microsoft Excel, from which the maximum stresses could easily be determined.

6.3 Conclusions relating to the evaluated FE models

6.3.1 Background behind evaluated FE models

The material properties, layout and the seismic loading used in the development of the FE models were all selected to be a representation of a physical problem. The physical problem being "How a single story low cost housing unit built to minimal standards in the Western Cape of South Africa would behave when subjected to the maximum earthquake loading that it could experience in its lifetime".

6.3.2 Steps used to create FE models

To insure that the developed FE models were an accurate representation of the physical problem, the process of developing the FE models was broken up into various steps. Firstly, the most commonly used building materials and building layouts of the South African single story low cost housing units were determined. The second step involved finding a modeling approach which is capable of modeling the complex behaviour URM structures, once chosen its capabilities were validated.

The third step, involved determining the material properties which were to represent the South African low cost housing units. The required material properties were found in literature, to ensure their correctness they were validated. The last step of the FE models development entailed determining earthquake records which were to be used in the FE simulations. In order to ensure that the maximum probable earthquake was represented, the guidelines defined in the Eurocode 8 were used to determine the earthquake records which would be used in the simulations.

6.3.3 Conclusions

After completing all the simulations, it was observed that cracking had not occurred in any of the FE models. In order to confirm this observation, the maximum stresses which occurred in each of the FE models were compared to the yield stresses for each type of failure that could occur within a masonry structure. This investigation illustrated that the majority of the simulations resulted in maximum stresses significantly less than the stresses required to initiate cracking and in no simulation were the yield stresses exceeded.

When processing the results of the simulations it was identified that the greatest stresses occurred within the unsupported gable wall and thus this wall would be the first to crack and eventually collapse. This indicates that the chosen layout of the low cost housing unit did not represent the worst-case scenario. The worst-case scenario would have been if the unsupported gable wall had also been designed as the front wall of the structure, as this would have resulted in the wall typically having a door and two windows within it.

The effect that these openings have on the seismic behaviour of the structure is unknown as both the maximum shear and tensile stresses which are developed in the unsupported gable wall run through the areas of these potential openings. It can be said that there is a small chance that this scenario occurring as the front and back walls are typically supported by interior walls.

It can be said that the vast majority of single story low cost housing unit built in the Western Cape of South Africa to minimal standards would be able to sustain the largest probable earthquake that it could experience in its lifetime without sustaining any visible damage. Only a small percentage of these structures may however experience cracking and possible collapse.

6.4 Conclusion relating to the risk that low cost housing poses to its occupants

6.4.1 Approach used to determine the risk that low cost housing poses to its occupants

The problem of determining the risk that low cost housing structures in South Africa pose to its occupants as a result of seismic activity, is a very broad topic which is still in the process of being fully understood. The aim of this investigation is to provide additional insight into this broad topic as well as to develop tools which can be used by future researchers.

To provide additional insight into this broad topic, this investigation focuses on a simplified problem statement which was not as broad, “How a single story low cost housing unit built to minimal standards in the Western Cape of South Africa would behave when subjected to the maximum earthquake loading that it could experience in its lifetime”.

The aim of this section is to provide insight into the original broad problem statement using the results obtained from investigating the simplified problem statement, which was discussed in section "Conclusions relating to the evaluated FE model FE models"

6.4.2 What insight has been gained in determining the risk that low cost housing poses to its occupants?

Before conclusions can be drawn, it must first be stated how the evaluated FE models discussed in section "Conclusions relating to the evaluated FE model FE models" relate to the original broad problem statement. The evaluated FE models can be seen as being an accurate representation of the original broad problem statement with regards to the earthquake records which were used in the simulations. Both problem statements required the structure to be investigated when subjected to the maximumn probable earthquake loadings that the region could experience.

The evaluated FE models are however un-conservative with regards to the material properties which were used in their development. The evaluated FE models assume that low cost housing units are constructed of building materials and construction quality which meets the minimum South African standards. As illustrated in the literature review this is not the case, a vast amount of the low cost housing unit have been built using substandard building materials and construction quality.

The results from the simulations illustrate that a single story low cost housing unit built to the minimum South African standards will be able to comfortably withstand the maximum seismic loading that it could experience in its life time. The ease at which the evaluated FE models were able to sustain the subjected earthquake activity suggests that a large range of weaker construction materials could also produce structures which would be able to sustain such seismic behaviour. Although this observation is not concise it is however promising.

When choosing the structural layout of the evaluated FE model, it was decided to use a conceptual layout of a single story unit, as the majority of SA low cost housing structure are single story. This allowed for many of the structural elements found in other building layouts to be incorporated into the evaluated FE models layout. In section "Conclusions relating to the evaluated FE model FE models" it was illustrated that the evaluated FE models did not represent the worst-case scenario in terms of the structural layout of the building. If this worst-case scenario was taken into account it may result in the evaluated FE models experiencing some cracking and possible collapse. It can however be said that there is a small chance that this worst-case scenario will occur in practice.

In closing, from the results obtained from this investigation it can be said that a large percentage of the single story low cost housing units in South Africa will be able to withstand the maximum seismic loading that they could experience in their lifetime. The outcome of this investigation does not agree with the original hypothesis of this investigation which stated that if the maximum probable earthquake was to occur in South Africa the effect could be catastrophic as such a large percentage of the population reside in sub-standard low cost housing developments. The findings of this investigation are however reinforced with the experimental tests carried out by Lourenço (2012).

6.5 RECOMMENDATIONS FOR FUTURE RESEARCH

6.5.1 Introduction

This section aims to highlight critical areas of knowledge which are not of yet fully understood and which future research can help to understand.

6.5.2 Future research relating to the FE modeling approach used in this investigation

The results of this investigation illustrate that the use of the meso-modeling approach can provide valuable insight it to the seismic behaviour of URM structures. In order to further develop this approach and to make it possible for it to be applied to bigger and different types of URM structures continued research is required.

Future research is required to investigate if it is possible to refine the manner in which the meso modeling approach is modeled in Abaqus in order to make the output data more manageable and that the simulations are not as computationally expensive. Another route that could be investigated is if there are other FE modelling packages which are better suited for these types of simulations.

Another factor that makes the meso-modeling approach difficult to apply is the scarcity of material properties which are required to develop the FE model of a specific URM structure. In order to determine these material properties experimental testing is required (i.e. triplet, compression and tensile tests of URM assemblages). Future research is required to determine these material properties for the various types of bricks/blocks and mortars commonly used in practice. Various testing procedures have been included in Appendix A. This section hopes to assist future research in determining these material properties as no clear guidelines could be found in literature.

6.5.3 Future research relating to assessing the seismic risk of low cost housing in SA

In order to provide a concise seismic assessment of the low cost housing in South Africa, future research is required to assess how substandard building materials and construction quality effect the ability of URM structures to withstand seismic activity.

Future research will also be required to investigate the seismic behaviour of low cost housing structures with different structural layouts, as well as low cost housing structures with more than one story.

7 APPENDIX A

7.1 MATERIAL PROPERTIES AND EXPERIMENTAL TESTING PROCEDURES

Upon completing this investigation, it became apparent that in literature the experimental testing procedures required to determine the material properties of the bricks and mortar are not clearly defined. This section aims to discuss which experimental testing procedures are best suited for determining the material properties for the bricks and mortar.

7.1.1 Unit elements

The compressive behavior can be defined from the compression tests of three bricks joined by mortar, as shown in Figure 43(b). It is important that the mortar is included in this test as the unit elements are solely responsible for the compressive behavior of the model and thus it must take into account the both the brick and mortar properties.

In literature, it is common practice to take the tensile strength of the bricks as 10% of its compressive strength. Another approach could be to complete a tensile test on the bricks. It is important to note that this tensile strength is that of only the bricks, the tensile strength of the bond between the mortar and the bricks will be incorporated into the material properties of the cohesive interface.

7.1.2 Cohesive interface

In order to define the normal and shear stiffness of the cohesive interface, the normal stiffness and poisons ratio for both the mortar and the bricks must be determined. These can be derived from compressive cube tests of the respective materials.

Figures 43(a) & 43(c) illustrate the triplet and standard tensile test setups which can be used to determine the shear and tensile material properties of the cohesive interface. Investigations conducted in literature have used beam tests to determine the tensile material properties of the cohesive interface, it is the authors opinion that a standard tensile test correlates better with the manner in which tensile failure occurs in URM structures.

By varying the compressive force on the outer bricks of the triplet test it will be possible to derive the kinetic frictional coefficient.

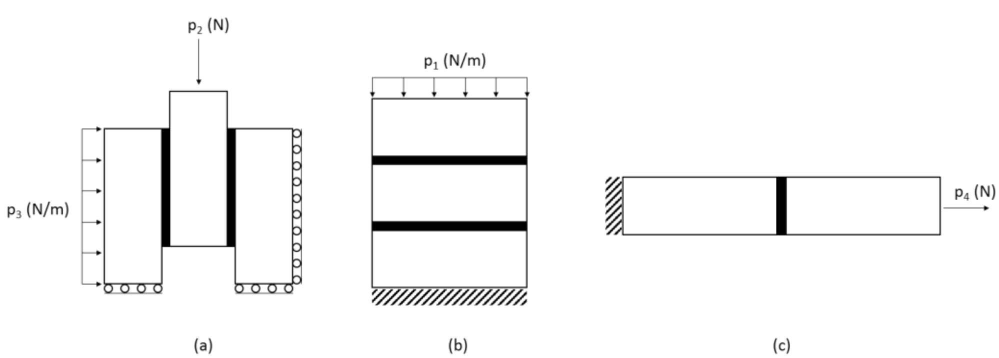


Figure 43: (a) Triplet test (b) Compression test (b) Standard tensile test

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