

EARTH BRICK SHELL HOUSING IN SOUTH AFRICA: STRUCTURAL AND PERFORMANCE CONSIDERATIONS

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A thesis submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in fulfilment of the requirements for the degree of Doctor of Philosophy.

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DECLARATION

I declare that this thesis is my own unaided work. It is being submitted to the Degree of Doctor of Philosophy to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.


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30th day of March, 2021.

ABSTRACT

The economic and environmental benefits associated with earth brick shells strongly endorse their adoption as an affordable housing alternative. However, there is a dearth of information regarding their performance, durability and livability - particularly in South African conditions. Earth masonry is characteristically weak in tension and flexure; hence, there is a need to eliminate tension forces when designing and building with these materials. This can be accomplished by using optimal shell forms, such as catenary domes and vaults. In South Africa, numerous masonry shells have been built with profiles that do not engender structural efficiency, but many shells experienced cracking damage in their masonry and/or surface renders. Cracking is a concern because it provides a pathway for moisture ingress, which can lead to materials saturation, localised erosion, and strength degradation of earth-based building materials. Concerns also exist regarding the thermal performance of thin masonry shells built in South Africa, some of which comprise only a single-leaf brick section.

Several investigations regarding the design and performance of earth brick shell structures in South Africa are presented in this thesis. Design recommendations are presented, based on case study investigations, experimentation, experiences gained through the design and construction of masonry shells, and by comprehensive assessment of the literature. Measures to improve the durability performance of earth brick shells are reviewed and important design features highlighted. An original numerical method is presented for the shape optimisation of vaulted structures, i.e., to develop tensionless forms. Several loading considerations are discussed, and a significant finding being the criticalness of wind loading for the design of vaulted masonry shells with steep profiles. Wind design recommendations are also developed for catenary shells. The source of cracking in unreinforced shells is examined, and it is presented that elements such as gable walls and window frames tend to cause cracking if they restrain movement (contraction and expansion). Furthermore, shrinkage of mortar may also lead to cracking, if restraint is generated during construction. Finally, the long-term thermal monitoring of a compressed stabilised earth brick (CSEB) shell house is discussed highlighting some deficiencies with uninsulated masonry shells, and several passive design strategies are presented to improve the performance of such structures.

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CHAPTER 1. INTRODUCTION

1.1 Background to the problem and objectives

Housing in developing countries is one of the most significant needs of low-income groups. This is especially challenging because the costs of land, labour, materials, and skills are usually well beyond the economic resources of the poor. South Africa faces significant challenges in meeting the demand for affordable housing. The South African government, post-1994, has worked towards improving the situation by providing over 2.8 million opportunities through housing subsidies for poor households (National Department of Human Settlements, 2017). However, the proportion of informal housing, which typically lack basic services, remains troublingly high. The 2016 General Household Survey revealed that 13.9% of households lived in informal dwellings (Statistics South Africa, 2016), only 2% lower than that reported in 2005 (National Department of Human Settlements, 2017). The challenge is further emphasized when recognizing that it was predicted that informal settlements would be eradicated by 2014 (National Department of Human Settlements, 2017). Furthermore, population growth and urbanisation have led to increased demand for housing in the economic centres of South Africa. In Gauteng Province, informal structures comprise approximately 20% of all dwellings. The causes of poor delivery of affordable housing are complex; in part due to the ineffective subsidy provisions by the government as well as challenging regulations surrounding the planning, design and execution of projects.

Current low-cost housing solutions in South Africa predominantly focus on conventional methods of construction; i.e., rectangular brick structures. The typical floor area of a single storey low-cost house in South Africa ranges between 36 to 45 square metres. The structure normally comprises of strip foundations, concrete block or clay brick masonry walls, and a standard roof (i.e., corrugated metal sheeting or clay tile). Although the proportion of these state-subsidized formal houses has increased over the past few years (these structures comprised 13.5 % of dwellings as reported in the 2016 General Household Survey), there are issues with the quality and longevity of these structures. Some home-owners have reported dissatisfaction with the quality of their state-subsidized house, citing issues with the walls and roofs (Statistics South Africa, 2016).

Structural issues aside, poor thermal performance is another deficiency with state subsidized houses: especially those that lack ceilings and roof insulation (Naicker *et al.*, 2017), which is not uncommon. The problem is particularly acute in houses built with zinc or corrugated metal roofs, since temperatures get very hot during summer days and cold during winter nights (Bolton and Burroughs, 2001). There is certainly a need to improve the performance of these structures but also to develop inexpensive alternatives to make housing more affordable to the poor. In order to reduce costs, innovative systems should be investigated instead of simply searching for marginal savings with traditional systems, which may result in reduced performance and/or strength. Several cost-effective systems have entered the housing market in recent year, such as insulated prefabricated wall and roof panels. These systems replace more labour-intensive building materials, such as masonry, with components that are manufactured off-site.

Economic efficiency is not only accomplished by utilizing more affordable construction materials but is also dependant on the structural system chosen to resist the magnitude and type of stresses imposed on the structure. The structural benefit associated with reduced bending and shear forces is well understood, and shell structures are particularly effective in this regard. The principal advantage associated with shells is that forces are largely transferred through in-plane action, with limited bending moments and shears at supports (i.e., boundary effects). Further structural efficiency is achievable through the implementation of funicular shell forms (e.g., catenary domes and vaults), following strict design criteria to ensure that only compression exists within the structure. These shells generate strength and stability through structural form, rather than incorporating extra materials (e.g., steel reinforcement) that can resist tension. Possibly, the most well-known example of such a structure is the Pantheon in Rome, which was built over 1900 years ago. The walls of the Pantheon rotunda and concrete dome were constructed without any steel reinforcement, and the structure is still in excellent condition today. Amazingly, the Pantheon dome has a diameter of 43 m and the compression strength of the concrete used was only about 12 MPa (Brune *et al.*, 2010). Goshima (2011) inscribed a catenary shape on the cross-section of the Pantheon dome and walls and postulated that the structure is in pure compression. The pantheon is an example of a building designed without steel reinforcement, constructed at a large scale, and with relatively weak materials that has last almost two millennia.

Reducing building costs is challenging in the low-cost and affordable housing industry: the cost of materials, labour and construction are foremost in this regard. However, some other aspects, apart from initial cost, should be considered such as resilience, energy usage, materials efficiency, and the comfort and health of inhabitants. Furthermore, the materials used in housing construction should be sustainable and, preferably, locally sourced in order to reduce transportation and production costs. Earthen building materials are particularly appealing in this regard due to the wide availability of suitable soil in many regions worldwide. Earth-based building materials have a cultural heritage dating back over 10,000 years (Walker, 2004), and a large proportion of the world's population still inhabit some form of earth dwelling (Houben and Guillaud, 1994; Avrami *et al.*, 2008). The common forms of vernacular earth-based construction include cob, wattle and daub, adobe and rammed earth (Torgal and Jalali, 2011; Houben and Guillaud, 1994; Minke, 2009).

However, earth-based building materials have problems, which are related to their composition and mechanical behaviour, namely: poor-to-moderate durability characteristics and low load capacity. In the last few decades there has been significant research to improve earth-based materials by chemical and/or mechanical stabilisation (Mukerji, 1994; Houben and Guillaud, 1994; Walker and Stace, 1997; Walker, 1995). The compressed stabilised earth brick (CSEB) is one such development in which the soil material is stabilised with an additive, usually ordinary Portland cement (OPC) or lime, and formed by a hand or machine press. There is a significant interest in CSEB amongst researchers because it presents an affordable and environmentally friendly alternative to conventional forms of masonry construction, e.g., fired-clay bricks and concrete blocks (Zami and Lee, 2010; Kumar *et al.*, 2018; Maini, 2005). Considerable research has been conducted on implementing CSEB within engineered housing systems (Kumar *et al.*, 2018; Gohnert *et al.*, 2018, Holliday *et al.*, 2016; Matta, *et al.*, 2015).

The issues associated with the relative weakness, particularly tensile capacity, of earth-based materials may be moderated by utilising pure compression structures. The compressive stress level in these shells is significantly less than that of structures that are subjected to bending and shear, making them highly efficient regarding material usage (Gohnert, 2019). Several studies have highlighted the economic benefits of adopting earth brick shells to replace the standard roof of a house or the entire house. For example, Adam and Agib (2001) noted significant cost savings in the Al Haj Yousif experimental prototype

school built in Khartoum, Sudan. The walls and roof were constructed using CSEB, which enabled a 40% overall cost reduction when compared to the cost of a conventional school structure. Most of the cost saving came from replacing the traditional roof system with earth masonry vaults, and roofing elements usually contribute significantly toward the total cost of traditional housing structures. According to Minke (2009), the typical cost of a standard house roof in developing countries is between 25% and 30% of the total building cost. Several CSEB domes and vaults built in Southern Africa have also been shown to be economically competitive with other low-cost alternatives (Bulovic 2014; Gohnert and Magaia, 2004; Gohnert and Tallochino, 2011; Gohnert *et al.*, 2018).

Although, several authors highlight the economic benefit of adopting earth masonry shells for housing, there is limited information regarding their performance. Understanding the performance, and potential deficiencies, is critical to the safety and longevity of these shell structures. In South Africa, numerous shells have been built with profiles (i.e., shape and thickness) that do not engender structural efficiency, and many shells have experienced cracking damage in their masonry and/or surface renders (Bradley *et al.*, 2018a). Cracking is particularly concerning because it provides a pathway for moisture ingress, which can lead to materials saturation, localised erosion and degradation of strength (Bradley and Gohnert, 2018). Accumulation of water within the masonry may also lead to health concerns related to persistent damp. Finally, erosion via cracks has led to the partial or complete collapse of poorly maintained earth brick shell structures (Bradley *et al.*, 2018a). Thus, the prevention of tensile stress, which undoubtedly will cause cracking, is critically important in this regard.

In addition to strength and durability issues, questions are raised regarding the thermal performance of thin masonry (fired-clay, concrete and CSEB) shells. Several dozen shells built in inland South Africa, including a CSEB dome, comprise of only a single-leaf section roughly 105 mm thick. These structures contrast with vernacular masonry roofs, which have significant thermal inertia and insulation, due to their greater section thickness. The smaller sections observed in several shells, in combination with their inherent solar exposure, are concerning regarding their thermal performance.

Poor performance can hinder the adoption of alternative building materials, particularly earth-based materials because of its already low acceptability amongst most social groups

(Adam and Agib, 2001). In addition, structures that are cracked, poorly built and exhibit issues related to ill-considered design (e.g., poor thermal performance, persistent damp, and poor use of floor space) will undoubtedly impede acceptance of non-conventional systems. For example, Norton (1997) noted that several major housing projects built in the Sahel during the late 1970s did not result in further earth brick dome and vault constructions. Habitat (1992) reported that numerous domes built in the Sahel had issues with their external waterproofing as well as other problems related to poor construction practice. In South Africa, similar problems occurred in several fired-clay brick shells situated on the grounds of a school in the town of Bapong. These buildings sit derelict and uninhabited, and are now bordered by numerous inferior informal (ad-hoc) structures. All doors and windows have been removed from the buildings, and some of them are severely damaged (Bradley and Gohnert, 2018). Certainly, good quality structures are critical to instil confidence in the population.

The objectives of the research presented in this thesis were: (1) investigate the performance of CSEB shell structures, (2) to develop techniques and practises to improve structural efficiency, performance and longevity of CSEB shell structures, and (3) to summarise important design considerations for CSEB shell structures, particularly for the South African environment. In this thesis, structural (i.e., strength and durability) and liveability (i.e., thermal comfort) are considered in relation to building performance.

1.2 Outline of the present thesis

The topics considered in this thesis are broad, ranging from structural mechanics to thermal comfort of inhabitants. The relevant literature regarding the advantages and challenges associated with CSEB materials and shell structures are discussed in Chapter 2. Subsequent chapters incorporate and examine the literature relevant to the specific study/investigation in each chapter.

In Chapter 2, a review of the advantages and difficulties associated with adopting earth masonry shells for low-cost housing, particularly in the South African context, is presented. The relevant challenges are identified, and solutions are presented in the continuing chapters.

In Chapter 3, the issues related to the performance (e.g., strength and durability) of earth masonry shells are presented. Subsequently, considerations to improve the longevity of CSEB shells, which are based on a review of the literature, experimentation, as well as an evaluation of several full-scale masonry shells built in South Africa, are presented.

In Chapter 4, a simple numerical method is presented for the formulation of funicular arches and domes. The methodology is outlined, and several examples are considered to highlight its application and usefulness. The optimised shapes from two case-studies, namely an arch and dome, are compared to solutions derived using thrust-line analysis. Finally, the application of the method is highlighted, with emphasis on designing pure-compression shells appropriate to smaller low-cost housing structures.

In Chapter 5, the critical imposed loads that may cause tension, cracking or even collapse of free-standing catenary shells. The appropriate recognition of these loads is integral to the safety and performance of such unreinforced masonry structures. The significance of wind loading, particularly for the design of steep catenary shells, is presented based on the results from an extensive wind-tunnel study, which is presented in Appendix C. Finally, several design guidelines are provided at the end of the chapter.

In Chapter 6, the common causes of cracking in unreinforced shells are reported and, subsequently, the importance of self-straining stresses are highlighted. The information presented is largely based on several case study investigations. Finally, considerations are proposed to mitigate cracking, which are particularly applicable in a low-cost setting.

In Chapter 7, a review of conventional and contemporary techniques used to construct masonry shells is presented. Construction issues associated with several methodologies are identified based on case study investigations and lessons learnt from several construction studies of model and full-scale shells. Finally, a few considerations are presented to mitigate damage in earth brick shells, particularly when building with rigid formwork.

In Chapter 8, observations and in-situ experimental results from a long-term thermal evaluation of a prototype CSEB shell house are reported. Several passive design strategies were assessed following the CIBSE TM52: 2013 overheating guidelines. The efficacy of high thermal mass, natural ventilation, surface albedo and shading were examined. Based

on the aforementioned assessments, performance recommendations are presented for uninsulated earth-masonry shells built in South Africa.

In Chapter 9, a concise summary of the results, along with several proposed recommendations derived from the study are presented. A few suggestions for future work are also proposed.

CHAPTER 2. EARTH-BRICK SHELLS FOR LOW-COST HOUSING

2.1 Introduction

Earth as a building material has cultural heritage dating back over 10,000 years (Walker 2004); and according to Houben and Guillaud (1994) as much as 30% of the world's population still live in some form of earth constructed building. Having said so, the adoption of more modern materials and construction techniques is on the rise in developing countries. This is the focus of much construction work in South Africa, where there is considerable endeavour to implement novel housing types (and building elements) through bodies such as Agrément South Africa. However, many of the innovative systems entering the housing market are based on materials and practices which do not encompass sustainable design. Nevertheless, there is still a tradition of earthen construction in many developing countries. Several ancient vaulted masonry built millennia ago, still exist today. Vaults built over 3200 years ago are still standing within the temple precincts of Ramses II, near Luxor (Minke, 2009). Fathy (1973) describes several mud-brick vaults, over 2000 years old, situated in the town of Touna el Gabal, Egypt, and numerous mud-brick vaulted houses in towns such as Gharb Aswan, Fares, and Marg.

Egypt has a rich history of mud-brick domes and vaults used as roofing elements for housing structures. Fathy (1973) enhanced this ancient construction technique (i.e., the Nubian technique) through several buildings at the village of New Gournia, Egypt. Ramses Wissa Wasef was another Egyptian architect that embraced this traditional method of vaulting and dome building through several construction projects (Norton, 1997). One such development is the Ramses Wissa Wasef Arts Centre in Cairo, Egypt. The centre comprises numerous earthen masonry domes and vaulted buildings, some of which are shown in Figure 2.1a through 2.1d. Habitat (1992) describes woodless roofs using adobe vaults and domes in Niger, where several hundred adobe structures were successfully built during the 1970s and 80s.

More recently, the Nubian technique has been used and adapted at the Auroville Earth Institute, in India, to construct a variety of vaulted low-cost housing and recreational buildings (Auroville Earth Institute, nd). Vaults spanning up to 10 meters have been built. Similarly, the Adobe Alliance, based in Presidio Texas, have built several mud brick domes and vaults for housing (www.adobealliance.org). Walker (2004) noted that in developed

countries, due to demands for more sustainable forms of built development, there has also been renewed interest in earthen construction. By utilising cheap and sustainable materials (e.g., earth masonry) in combination with efficient structural forms (e.g., domes and vaults), additional cost saving may be realised. Several projects discussed in the literature highlight the economic benefit of adopting earth buildings for low-cost housing (e.g. Gohnert *et al.*, 2018; Gohnert and Talocchino, 2011; Zami and Lee, 2010; Gohnert and Magaia, 2004; Adam and Agib, 2001).



Figure 2.1 – Earth brick domes and vaults at the Ramses Wissa Wasef Arts Centre

However, the low tensile strength and durability problems associated with earth-based materials raise several design concerns that must also be recognized. Importantly, cracking facilitates deterioration through water erosion, which can ultimately lead to the collapse of earth masonry shells. This is particularly problematic for low-income home owners because repairs are typically only implemented upon availability of funds.

In this chapter, the advantages and difficulties associated with adopting earthen masonry shells for low-cost housing are reviewed, particularly in the South African context. The objective is to highlight the benefits of adopting earthen shells for low-cost housing and to identify the relevant challenges. These issues are then discussed further and addressed in the latter chapters in this thesis. The topics examined are multidisciplinary; consequently, each chapter will further examine the literature relevant to the area under consideration.

2.2 Advantages

2.2.1 Structural efficiency

2.2.1.1 Pure compression shells

Shell structures offer many structural benefits that promote their use when material and economic efficiency are required. Typically, shells are designed to function primarily as a membrane (i.e., to carry largely in-plane forces); however, often with the added consideration of boundary effects (i.e., localised bending moments and shear at the supports). In the case of reinforced concrete/masonry shell structures, these moments and shear are generally accounted for by thickening of the base and/or including additional reinforcing. An efficient low-cost CSEB dome was built at the Central Johannesburg College, which incorporated fibre reinforced plaster. Furthermore, the base was thickened to account for boundary effects and thermal stresses, as shown in Figure 2.2a (Gohnert and Talocchino, 2011). Figure 2.2b shows an earth brick dome, incorporating sisal fibre reinforcement, which was built near Maputo, Mozambique (Gohnert and Magaia, 2004).

Alternatively, efficient structural forms (e.g., catenary domes and vaults) may be utilized, with the adoption of strict design criteria (e.g. ‘the middle third’ principle), to ensure that only compression exists within the shell. These shells generate strength and stability through structural form, rather than through incorporating materials that can accommodate tension. Catenary forms are particularly efficient because they carry only compressive stresses under self-weight (Gohnert and Bradley, 2020a; Gohnert and Bradley, 2020b). Two steep catenary vaults (Gohnert *et al.*, 2018; Bradley *et al.*, 2017a), which were designed as pure compression forms, are shown in Figure 2.2c. To achieve a pure compression-state under combined dead and imposed loading, e.g., wind, designers must ensure that a thrust line is contained within the middle third of the shell’s cross-section for each load case/condition (Gohnert *et al.*, 2018; Allen and Zalewski, 2010). There are other

sources/actions which could cause tensile stress in a masonry shell, such as the restraint of movement, e.g., shrinkage and thermal movement, and differential settlement of foundations. These aspects require additional design consideration (Bradley *et al*, 2018a; Gohnert *et al.*, 2018).



(a) CSEB dome (Johannesburg)



(b) CSEB dome (Maputo)



(c) CSEB vaults (Wits University, Johannesburg)

Figure 2.2 – Low-cost prototype housing structures

2.2.1.2 Resistance to compressive stresses

Pure-compression shells used for housing and recreational buildings typically experience very low compressive stresses. For example, a catenary vault, which has a span of 6 m, height of 3 m, and a density of 2000 kg/m^3 , carries a maximum compressive stress, due to self-weight, of about 0.1 MPa. Because of these low magnitude compressive stresses within earthen masonry shells, the unit (i.e., brick or block) normally only needs to meet the

minimum compressive strength requirement, as stipulated in the relevant design standard, e.g., ARS 674:1996 (E) (Compressed earth blocks - Technical specifications for ordinary compressed earth blocks). According to Adam and Agib (2001), in the case of single-story structures, many building authorities around the world recommend block strengths between 1 and 4 MPa. Walker (2004) noted that a minimum requirement of 1.4 MPa characteristic unconfined dry compressive strength is commonly accepted for lightly loaded and non-structural applications. According to Walker and Stace (1997), the strength of earth bricks used for low-rise residential construction is dictated by handling requirements rather than load carrying needs. These authors recommended an unconfined characteristic saturated compressive strength of 1.0 MPa. Considering the typical block and mortar strengths recommended in the literature (e.g., Walker and Stace, 1997; Walker, 2004; Adam and Agib, 2001; Morton, 2008; Fratini *et al.*, 2011), blockwork compressive strengths will typically be much greater than compressive stress induced by loading.

2.2.1.3 Extremely low buckling risk

The brilliant Uruguayan engineer Eladio Dieste designed a number of large low-rise, reinforced (and pre-stressed) masonry catenary shells (Pedreschi and Theodossopoulos, 2007). Two of Dieste's most iconic innovations include free-standing barrel vaults and double-curvature vaults, also known as Gaussian vaults. The free-standing barrel vaults are so named because they often included large cantilevered spans. The design and construction of these possible cantilevered vaults were by using an innovative prestressing technique, which was developed by Dieste. Pedreschi and Theodossopoulos (2007) note that the span-to-rise ratios of these vaults is typically between 4 – 5 (i.e., height to base radius (H/R) between 0.4 to 0.5).

Many of Dieste's Gaussian vaults are exceptionally thin and span large distances, e.g. the J. Herrera y Obes (JHO) warehouse, at the Montevideo docks, built between 1977 and 1979. The shell's overall dimensions are 79 m long by 46 m wide, and consists of fourteen discontinuous double-curvature vaults, approximately 130 mm thick, with crown height of roughly 6.5 m. These geometric parameters give a H/R of roughly 0.28, and a radius to thickness ratio (R/t) of approximately 350. Because these vaults exhibit very large radius to thickness ratios, buckling becomes the governing criterion for design, particularly under wind loading. To account for this critical aspect, Dieste incorporated a wave-like corrugated form, which increases the effective depth (and moment of inertia) of the shell.

A truly remarkable aspect of these Gaussian shells is that the thickness is always comprised of only a single masonry unit (topped by a lightly reinforced thin cement/sand plaster). The reinforcement runs, longitudinally and transversely, through grouted hollow clay bricks.

Wilson (2005) reports the following established buckling design criteria for reinforced concrete shells, which are applicable to the reinforced masonry shells designed and constructed by Eladio Dieste;

- Generally, shells with a R/t ratio greater than 300 begin to exhibit buckling problems.
- The stresses in a shell should be kept between 30 to 40 percent of the maximum allowable stress value of the material.

Wilson (2005) noted that for reinforced concrete shells with small radii of curvature, buckling typically does not govern the design thickness, instead, the thickness is dictated by the construction technique or reinforcement cover requirements. The R/t of the earth vaults and domes considered in this thesis are significantly lower than the limit of 300 noted previously. A vault having $H/R = 1.0$, spanning 6m and with thickness of 150 mm, has an R/t of 20. Furthermore, applied compressive stresses are also typically an order of magnitude lower than the compressive strength of earthen masonry. Thus, the thickness of these unreinforced masonry earth shells is not governed by buckling but by non-uniform loads (e.g., wind loads or asymmetric distributed loads) that cause tensile stresses. As presented later, when uninsulated, shell thickness may need to be based on thermal performance.

2.2.2 Low-cost

A major advantage of using earth bricks, instead of conventional masonry units, is that production can often be on site, thus, significantly reducing transportation costs. This is particularly advantageous for construction in isolated rural areas. Maini (2005) noted that at the Auroville Earth Institute, one cubic meter of cement stabilized soil masonry brick is generally 48% cheaper than wire-cut bricks and 24% cheaper than country-fired bricks. Typically, the most significant cost is that associated with labour; e.g. digging, sieving, mixing, block production, stacking, and curing preparation (Auroville Earth Institute, n.d.; Adam and Agib, 2001). Auroville Earth Institute (n.d.) provides an approximate cost breakdown, dated July 2012, for earthen blocks stabilised with 5% cement and compacted

with a mechanical hand press (AURAM press 3000) – see Table 2.1. Although labour typically constitutes the majority of the brick production cost, the manufacturing process and common application of earthen masonry (i.e., in low-cost construction) offer an opportunity for local communities to become involved in appropriate projects, which facilitates job creation and skills development (Fitchett, 2009; Adam and Agib, 2001). According to Zami and Lee (2010), contemporary earth construction for low-cost urban housing can be cost effective when produced locally with natural resources, using semi-skilled labour and with limited transport needs.

Table 2.1 – Approximate cost breakdown for CSEB masonry – Auroville Earth Institute

Labour: 45 %	Raw materials: 27 %	Cement: 25 %	Equipment: 3 %
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Several building projects highlight the economic advantages associated with earthen materials and shell forms. Adam and Agib (2001) noted that the construction of an earth masonry prototype school in Khartoum was very cost effective by Sudanese standards. According to these authors, the total cost, per square meter, was reduced by approximately 40% from that of a traditional structure. Additionally, adopting earth shells, such as catenary vaults (Bulovic, 2014) and domes (Gohnert and Talocchino, 2011), to eliminate the need for traditional roofing, can significantly reduce costs, particularly when blocks are manufactured on site. The steep catenary vaults designed and built at the University of the Witwatersrand provided approximately double the floor area of a typical low-cost house (in South Africa) but only cost between 10% and 30% more (Bulovic, 2014).

The building of complex shell forms may engender additional cost implications, especially with regard to formwork. Increased cost is related to additional materials and labour required to manufacture, transport, and erect formwork. However, these costs may be reduced, or removed altogether, by adopting simple shell forms and/or free-hand construction techniques, e.g., segmental construction for singly-curved shells and free-hand techniques for doubly curved shells. There are several examples of modern earth masonry shells built with segmental formwork (Gohnert *et al.*, 2018) or free-hand (Ramage *et al.*, 2010). Gohnert and Magaia (2004) compared the cost of an earth brick dome, built free-hand, with several other low-cost alternatives, and showed that the dome was a significantly cheaper alternative.

2.2.3 Low energy requirement and carbon footprint

The major advantage of CSEB is that they do not need to be fired, like clay bricks, or incorporate large volumes of energy intensive materials, such as concrete blocks. Furthermore, if CSEB are produced on site, using locally sourced materials, there are no pollutant emissions or energy consumption associated with transportation. Additionally, when soil is appropriated from the construction site, it is usually extracted from relatively shallow levels; i.e., manually with limited plant/machinery. Zami and Lee (2010) noted that building a square meter of masonry with CSEB consumes five times less energy than a square meter of wire-cut brick masonry and 15 times less than locally-fired bricks. Morton *et al.* (2005) compared the ‘factory gate’ embodied energy, for different masonry materials, which illustrated the superior environmental performance of earth bricks, as shown in Figure 2.3. Lax (2010) conducted a life cycle assessment of rammed earth which shows that the stabilization with cement causes the embodied carbon to raise from 26 KgCO₂/ton to 70 KgCO₂/ton. These emission values are still well below the carbon dioxide emission associated with conventional building materials. For example, Waziri *et al.* (2013) reported that CSEB produces 9 and 7 times less CO₂ emissions than clay fired blocks and concrete blocks, respectively. Furthermore, according to Morton (2008), the long-term inputs, such as the processes of recycling or disposal of earth masonry materials, do not require high levels of energy.

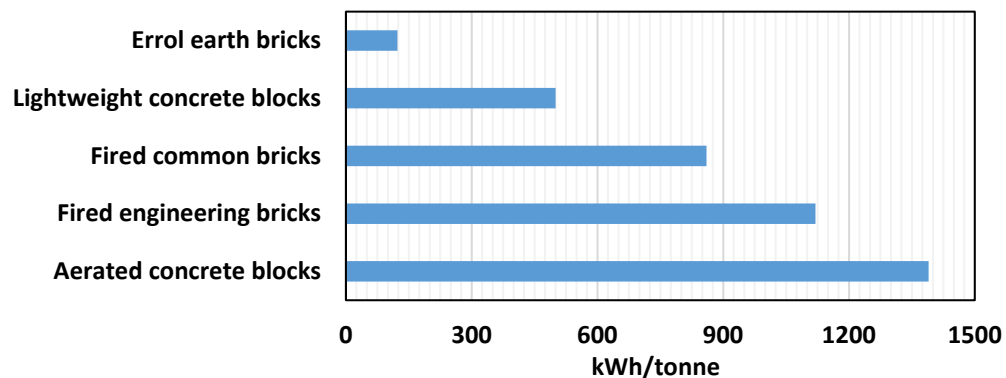


Figure 2.3 – Embodied energy in different materials (data from Morton *et al.*, 2005)

2.2.4 Sustainability

Walker (2004) noted that earth brick masonry enables a reduction of non-renewable materials. In particular, the production of earthen building materials does not exhaust

renewable and non-renewable energy resources, such as coal and natural gas (used to power kilns for clay brick production), like many other building materials. Natural resources (e.g., timber) could be conserved since it is not required in the production of earthen bricks, unlike locally-fired clay bricks produced in rural areas. “Earth materials commonly create extremely low levels of waste, all of which is benign and easily disposed of” (Morton, 2008). Moreover, according to Torgal and Jalali (2011), even when soil is stabilized with cement or lime, it can then be reused without generating waste. For example, the dry-stack blocks used to build the ground-floor and gable walls of the prototype structure at the University of the Witwatersrand, shown in Figure 2.2c, were manufactured from recycled building materials, including stabilised earth blocks.

2.2.5 Fire resistance

Morton (2008) noted that earth masonry generally performs well in fire. Various fire tests have been done on earth masonry systems, certified by Agrement South Africa, including masonry comprising of 140 mm wide bricks. Although earth is non-combustible, there can be certain components of the soil and some additives (e.g., stabilizers) which are flammable. These include organic matter (i.e., carbon), combustible stabilizers and inclusions such as straw fibres, wood shavings, and bitumen.

There are also risks associated with firefighting, as water will likely be sprayed onto a surface of the shell, which may cause rapid cooling, therefore large differences in temperature. The sudden temperature change may cause cracking. Furthermore, high-pressure water sprays can induce surface erosion (Morton, 2008). Nevertheless, according to Morton (2008), the risk of damage to earth masonry by fire and firefighting activities is low, but should always be considered on a case-by-case basis in relation to the need for fire protection and ensuring adequate means of escape.

2.2.6 Thermal and climatic performance

Adam and Agib (2001) noted that due to its high thermal capacity, low thermal conductivity and porosity, earthen blocks can moderate extreme outdoor temperatures and maintain a satisfactory internal temperature balance. Lamb (1998) conducted tests on Hydraform (Boksburg, South Africa) compressed earth blocks to assess their thermal conductivity and heat capacity. The tests were conducted on several types of earth blocks, and on standard fired clay bricks. The findings of the investigation confirmed that earth blocks had a lower

thermal conductivity and greater heat capacity than fired clay bricks, as presented in Table 2.2.

Table 2.2 – Thermal conductivity tests on earth blocks and fired clay bricks (Lamb, 1998)

Type of unit	Thermal conductivity [W/m °C]	Heat capacity [kJ/kg °C]
CSEB	0.58	0.85
Fired clay brick	0.82	0.80

Although earthen masonry possesses desirable thermal characteristics, the performance of a house also depends on several other aspects including the characteristics of the building envelope, fenestration, building orientation, and the climate in which the structure is built. Furthermore, the types of masonry shells discussed in this thesis are characteristically thin, sometimes a single imperial brick thick, which is dissimilar to vernacular earthen walling, which are traditionally between 250 mm and 500 mm thick. The thermal performance certainly differs between these types of elements. To better understand the performance of earth masonry thin-shell housing in South Africa, a long term in-situ thermal comfort study was carried out on a structure situated in Johannesburg. Several benefits, some challenges, as well as design recommendations are reported in Chapter 8.

Another advantage of earthen material is its ability to maintain the relative humidity of indoor air (Minke, 2009). Torgal and Jalali (2011) noted that some researchers have shown that earth blocks are capable of absorbing ten times more moisture (by weight) than fired-clay bricks. The authors stated that earthen structures act as a humidity flywheel, equalizing the relative humidity of the external environment with that of the pores within the walls. Morton (2008) noted that earth construction can keep the relative humidity of indoor air between 40% and 60%, which is the optimum range from a human health perspective.

2.3 Challenges

2.3.1 Tension

The development of tension in unreinforced earth shells can become problematic because of the low tensile capacity and moderate durability characteristics associated with the constituent materials. Consequently, unreinforced masonry shells are typically designed to resist compressive stress only. Design methodologies, whether graphical (i.e., equilibrium) or elastic, must be employed in combination with strict criteria (e.g., the middle third

technique) to ensure that no tension develops, for all anticipatable actions/loads. Nonetheless, even though unreinforced masonry shells may be designed for compression only, cracking is not uncommon. Numerous shells constructed inland South Africa show extensive cracking; a few of these are discussed below.

2.3.1.1 Mapungubwe Interpretative Centre, Limpopo Province

The Mapungubwe Interpretation Centre is located in the Limpopo Province; at the confluence of Limpopo and Shashe Rivers, where South Africa, Zimbabwe and Botswana meet. The Interpretative Centre is composed of numerous shell forms, striking in appearance, as shown in Figure 2.4. The beauty of these shells earned the 2009 World Building of the Year award at the World Architecture Festival. The shells are based on natural forms; designed as purely compressive structures. The material used in the construction of the shells was earthen tile, comprising of seven layers, with an average total thickness of 200 mm. The construction method was patterned after the celebrated architect Rafael Guastavino, renowned for his unique style of timbered vaulting (Milkovich, 1992). Earthen tiles exhibit similar strength characteristics to masonry and concrete; that is, the compressive strength is significantly larger than the tensile strength. Cracks, however, developed on several shells of the Mapungubwe Interpretation Centre, as shown in Figures 2.5a, 2.5b and 2.5c.



Figure 2.4 – Mapungubwe Interpretation Centre



(a) Large Internal crack



(b) Internal crack above a window frame



(c) Large Internal crack near shell connection/fixity

Figure 2.5 – Cracking in the Mapungubwe shells (cracks traced for clarity)

2.3.1.2 The Bapong catenary vaults, North West Province

Dozens of abandoned catenary structures exist on the premises of a local primary school in the town of Bapong (in the North-West province, South Africa). The structures can be seen along the R556 (on route to Sun City). Local community members indicated that these were erected a few decades ago, and the reason for their abandonment is unknown. These structures were built from standard fired clay bricks and cement-sand mortar. One of these vaults is shown in Figure 2.6a. Cracks are evident near the apex of several of the vaults, as shown in Figure 2.6b. These cracks are limited to areas close to the gable walls.



(a) Catenary Vault (Bapong)



(b) Internal apex cracks (traced for clarity)

Figure 2.6 – Unreinforced fired clay brick vaults: Bapong township, South Africa

2.3.1.3 *The Wits catenary vaults, Gauteng Province*

The Wits catenary vaults is a low-cost housing structure, comprising two-steep catenary vaults, built at the University of the Witwatersrand. The vaults enabled a construction study as well as a full-scale case study for the assessment of various aspects discussed within this thesis. Several months after construction had been completed, several vertical and transverse cracks began to appear in both vaults, as shown in Figure 2.7a and 2.7b.



(a) Internal cracks (traced for clarity)



(b) External cracks (traced for clarity)

Figure 2.7 – Unreinforced CSEB vaults: University of the Witwatersrand, South Africa

2.3.2 Bond strength

The flexural bond strength along bed joints is influenced significantly by the level of adhesion between mortar and masonry unit. Hendry *et al.* (1997) noted that unit-mortar adhesion is a physical-chemical process in which the pore structure of both materials is critical. According to Morton (2008), unstabilised earth mortars, consisting of earth similar to that of the bricks, will produce relatively weak bonds - around 0.0025 MPa, with variation subject to materials and construction technique. The bond strength can be improved by incorporating stabilisers, such as cement or lime, and additives within the mortar. Weak bond strength is typically not a problem in traditional earth masonry construction where loads are primarily compressive due to the mass, thickness and types of element (e.g., columns and walls). However, contemporary masonry walls are typically much slenderer than that used traditionally. Consequently, the stability derived from self-weight is insufficient, and the strength of the mortar bond must be relied upon, in order to counteract lateral loading. Akin to thin panel walls, modern masonry shells are typically very slender, often only one unit thick, when compared with traditional and vernacular masonry constructions. However, unlike in these slender vertical elements, the flexural capacity generated by adhesion between the mortar and brick is typically ignored in modern masonry shells. Rather, resistance is generated through optimal geometric forms, as mentioned in section 2.3.1.

The risk of sliding between units must also be considered due to the weakness of the mortar-unit interface. Traditionally, the risk of sliding between units of stone arches was mitigated by ensuring that the angle between the force resultant (i.e. thrust-line) and the normal to the horizontal unit face is less than the angle of friction of the stone, which is typically about 30° (von Ott, 1905). Similarly, in contemporary earth shell design, masonry joints should be approximately perpendicular to the calculated compressive forces to minimize the possibility of sliding wherever possible (Dahmen and Ochsendorf, 2012).

2.3.3 Durability

Moisture, particularly in its liquid state, can have significant negative effects on the strength and longevity of earthen masonry. Water influences the durability of earthen masonry through several distinct mechanisms including erosion due to the impact of wind-driven rain droplets, strength loss due to saturation (by infiltration through cracks, inundation due

to flooding, and capillary action), and gradual degradation due to repeated freeze/thaw cycles (which is not applicable in South Africa). These mechanisms are elaborated below.

2.3.3.1 Erosion due to the impact of wind-driven rain droplets

The impact of wind-driven rain droplets is a major source of surface erosion in earthen masonry. Evaluation of the effects of wind-driven rain is typically assessed with superficial tests (e.g., wet/dry abrasion) or by artificial simulation tests of wind-driven rain. A commonly used technique is the accelerated erosion test, which consists of spraying water onto an earth specimen for up to an hour or until the water completely penetrates the earth specimen (Heathcote and Moore, 2003). Water is sprayed at a pressure of 50 KPa (which aims to replicate rain with a velocity of 10 m/s). However, according to several researchers, such as Morton (2008) and Heathcote and Moore (2003), these tests do not reproduce real climatic conditions. Due to the accelerated exposure, in which holes are bored into the earth specimens under high pressure, used to simulate a long period wind driven rain. Morton (2008) noted that results from these tests should be treated with considerable caution.

The durability of earth building materials are also assessed indirectly through unconfined compressive strength, wet/dry abrasion and other established tests for masonry and concrete specimens. However, some simulation tests only offer a means of comparison, rather than quantifying the actual durability characteristics of specimens and/or structural components. Ipinge (2012) noted that the wet/dry abrasion tests ability to mimic even the most severe rainstorms has come into question, with experimental field data showing no correlation with laboratory test results. Nevertheless, Walker and Stace (1996) and Walker (2004) reported that erosion requirements of existing tests (e.g., wetting–drying test and accelerated erosion spray test), can be indirectly satisfied through the specification of minimum wet compressive strength.

Alternatively, long term duration tests, in which walls are exposed to the elements for several years, offer a more realistic evaluation of actual erosion resistance. Morton (2008), commenting on the results of several experimental sections of earth masonry evaluated over a 6-year period, noted the following.

- A typical pattern of erosion on a well-graded material will have fine particles washed down the surface, leaving a relatively stable surface of exposed, larger particles, protecting the finer particles behind.
- Poorly graded materials will result in loss of particles of all sizes from the surface in a progressive erosion pattern ending in complete decay.

Two examples of water deterioration in earth brick masonry walls are shown in Figures 2.8 and 2.9. In Figure 2.8a, the lower part of the earth masonry wall has lost the protective render, and consequently, the underlying masonry has begun to deteriorate, as shown by a close-up photo in Figure 2.8b. The lower parts of earthen walls and shells are extremely vulnerable to water damage caused by splashing. Damage of this nature can become problematic if maintenance is deficient. Figure 2.9 illustrates the effects of prolonged exposure of an unstabilised earth brick wall to wind driven rain. The mortar has better erosion resistance than the bricks, which can be problematic because as the bricks erode, water will stand on the exposed surface of the mortar causing additional erosion of the bricks (Houben and Guillaud, 1994). This could become a major issue in earth brick shells, particularly parts with shallow gradient, where significant amount of water would collect, stand and saturate the bricks and mortar.

Several adobe domes and vaults at the Swan House & Lab (situated in Presidio, Texas) were severely damaged by unusual and persistent rainstorms (www.adobealliance.org). The vaults were built, by the Adobe Alliance, following the Nubian free-hand construction technique. Nubian vaults are typically built with light and thin adobe bricks and sticky earth mortar; and are protected from moisture with several layers of earthen plaster. However, in wetter climates, unlike the deserts of Egypt from which these techniques originated, these types of structures need to be continuously monitored and maintained to ensure longevity and safety. De la Rosa (2013) noted that there had been no maintenance to any of the buildings at the Swan House between the spring of 2011 and the winter of 2013. The issues were (i) mounds of dirt and debris had amassed at the base of the walls due to erosion of the wall and shell, (ii) the inside rooms had leaks and showed erosion and dry dirt drippings, (iii) some mortar joints had been eroded over 25 mm in one of the vaults, (iv) some of the cracks in the vaults were approximately 75 mm deep, (v) the shape of one vault had been distorted by the erosion over several years. The consequence of the untimely maintenance of unstabilised earthen shells can be catastrophic for the inhabitants.



(a) Earth brick wall

(b) Exposed earth bricks and mortar

Figure 2.8 – Progressive erosion of the base of an earth brick wall caused by long-term splashing, El Kantara (The Red Village), Algeria



Figure 2.9 – Typical erosion pattern when mortar is stronger than the bricks, Algiers, Algeria

2.3.3.2 Water infiltration and material saturation

Theunynck (1994) reported that domes and vaults built in northern and sub-Saharan Africa with stabilized earth plasters had major waterproofing problems. The negative effects of water on unstabilized masonry has meant that these shells were typically built in dry regions (i.e. in regions such as North Africa and the Middle-East). However, in wetter climates these types of structures need to be continuously monitored and maintained to ensure longevity and safety throughout the building's service life.

According to Houben and Guillaud (1994), water striking wall surfaces is not serious if it evaporates thereafter; however, water accumulation within the masonry can cause serious problems. Water flow through cracks is particularly serious in earthen masonry. According to Houben and Guillaud (1994), three conditions must occur in parallel to facilitate material saturation; namely, 1) water must be present over the surface, 2) there must be an opening (i.e., a crack), and 3) a force (e.g., pressure, gravity, or capillary action), which facilitates water to enter. Grimm (1988) notes that cracks smaller than 0.1 mm are insignificant to water permeance because that is the least width through which wind-driven rain will enter. The author also noted, in reference to CMU masonry, that the maximum width of crack that will not cause distress is between 0.25 to 0.38 mm. Larger structural cracks create localised problems related to saturation as well as increased potential for erosion. De la Rosa (2013) discovered, during repairs to a long crack in the ‘kitchen’ vault at the Swan House and Lab, that a crack had widened from roughly 3 mm at the surface to about 38 mm at a depth of 50 mm. Erosion of this nature may eventually lead to collapse if cracks are left unrepaired.

Standing water and/or flooding will result in the flow of water under the action of a pressure difference across the masonry surface, causing the block to become saturated. Earthen masonry may also become saturated by capillary action, which is the ascension of water into, and through the pores of unsaturated material under cohesive and adhesive forces. Porous bricks and blocks, or those with expansive fibres, will certainly facilitate faster absorption. Several researchers have assessed the effects of saturation on the compressive and flexural strength of CSEBs (Ipinge, 2012; Walker, 1995; Walker, 2004; Guettala *et al.*, 2006; and Pave, 2007). The assessment is made by comparing strength values from compressive tests on wet and dry blocks, typically at 28 days. Figure 2.10a and 2.10b show the results from the aforementioned studies, expressed as a percentage, which demonstrate reduced mechanical strength of saturated bricks and blocks. In general, Figure 2.10a illustrates that increasing cement content results in lower strength losses, whereas the opposite is true of the clay content, as shown in Figure 2.10b.

The data shown in Figure 2.10a indicate the significant loss in compressive strength, between 20% and 60%, for units manufactured with typical cement content (i.e., between 3 and 10%). Similarly, significant strength losses are observed in bricks produced with soils containing high clay content. Walker (1995) noted that the uniaxial dry compressive

strength of clays is lost with saturation. Because clays are very susceptible to shrinkage and swell, particularly montmorillonite, the resulting contraction or expansion creates the potential for a loss of coherence, which can result in damage. According to Walker (2004), precaution is needed when active clay minerals are present in blocks because short-term wet strength can subsequently deteriorate further when exposed to cycles of shrinkage and swelling. Furthermore, water intake may be increased with increasing clay content due to the adsorptive nature of clay particles.

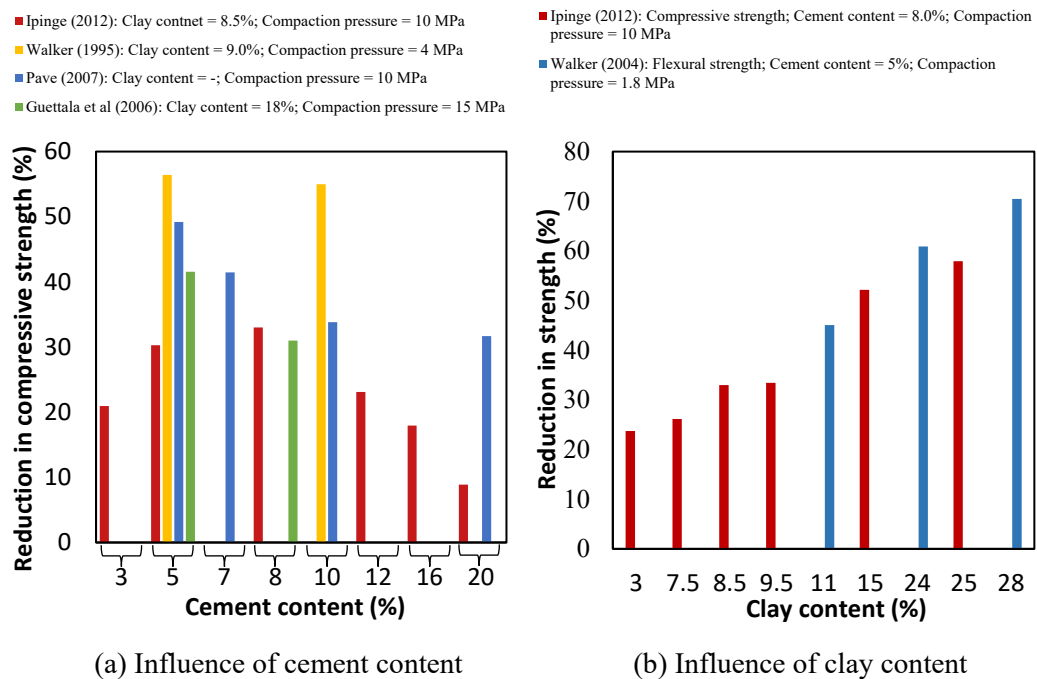


Figure 2.10 – Strength loss due to brick saturation

2.3.4 Resistance to abrasion and impact loading

Earth masonry is more vulnerable to damage from abrasion or accidental impact than common building materials due to lower surface hardness, although CSEB is more robust than adobe due to improved density (Morton, 2008). Nonetheless, according to Adam and Agib (2001), CSEB masonry generally have low resistance to abrasion and impact if not sufficiently reinforced or protected. Morton (2008) noted that damage caused by abrasion is typically superficial, and is easily remedied through localised protection with more robust materials in exposed areas (i.e., at the corners of building).

The susceptibility of unreinforced earth brick masonry to impact loading or actions is fundamentally related to limited ductility and low tensile strength. Morton (2008) noted that impact damage to thin sections can be more serious, given the relatively low bond strength of earth masonry. Many of the masonry shells investigated in this thesis are only a single leaf thick, which raises concerns regarding their resistance to large impact loads. Impacts from falling trees and wind-borne debris are examples of such loads, and Figures 2.11 and 2.12 show several unreinforced cement brick domes and fired clay brick vaults built in the vicinity of large trees. The consequence of impact actions, from such falling trees, could be catastrophic due to the mass of falling masonry, which could easily result in serious injury or even death.



Figure 2.11 – Large trees adjacent to several abandoned vaults in Bapong, North-West



Figure 2.12 – Trees and domes at the Sparrow Rainbow village (Google, 2017)

A recent study by Aymerich *et al.* (2016) shows that earthen materials have some favourable properties to dissipate the energy from an impact. In particular, the authors reported that much higher strength values and energy absorptions were observed under dynamic (impact) bending as compared to static bending on notched earth specimens. The interaction of high-strength projectiles with a low-strength building material, such as adobe, has also been investigated (Sauer *et al.*, 2017). The potential damage associated with large impact loads is concerning since impact actions from large objects could lead to significant damage or even collapse (Bradley *et al.*, 2017a).

2.3.5 Limited geographical applicability: poor seismic performance

It is widely acknowledged that unreinforced masonry shells are particularly vulnerable to seismic loading. The suitability of an unreinforced shell form may depend highly on the geographic location. Doubly-curved shells are intrinsically superior from a seismic resistance perspective, and they have been used in areas prone to seismicity such as Iran, Turkey, and India. Nevertheless, Houben and Guillaud (1994) recommend limiting the use of unreinforced masonry vaults in seismic regions, whereas Dahmen and Ochsendorf (2012) comment that vaults are not suitable in such areas.

2.3.6 Limited information on the liveability of earthen masonry shell housing

Several aforementioned studies address the design, construction, and benefits associated with earthen housing structures. However, there is little information on their liveability – specifically in South African climates. Although some authors (Adam and Agib, 2001; Morton *et al.*, 2005) have examined the liveability of earthen masonry structures in other regions, there is a dearth of information regarding thin masonry shell housing. What is particularly concerning is the, often, diminutive section thickness of masonry shells built in South Africa as well as the solar exposure of their envelopes.

2.4 Conclusions

Earth building materials offer many benefits such as sustainability, excellent climatic performance, low-cost, and low carbon footprint. Furthermore, earthen shells are typically designed as pure-compression structures, thus eliminating the need for expensive and energy intensive materials such as reinforcing steel. Although earth masonry shell structures offer these benefits, several issues associated with the negative impact of

moisture on strength and longevity must be recognised. Furthermore, cracking in the masonry and/or render is not uncommon in unreinforced masonry shells, and can become problematic as it facilitates the ingress of water into the underlying material. Because of these issues, the design, construction and materials specification of unreinforced earth shells are critical to ensure the safety and longevity of these structures.

CHAPTER 3. IMPROVING THE STRENGTH AND LONGEVITY OF EARTHEN MASONRY SHELLS

3.1 Introduction

Earthen masonry shells offer many economic, environmental, and aesthetic advantages. In particular, these structures may be an efficient solution to low-cost housing in some developing countries. However, the strength and durability characteristics associated with earthen building materials require attention throughout the implementation of an earth masonry shell project. The materials and design considerations, which should be implemented for the construction of durable stabilized earth brick shells for low-cost housing are presented in this chapter. Recommendations are derived from the literature as well as the monitoring of several masonry shells.

3.2 Improving traditional adobe masonry

3.2.1 Bricks and blocks

The traditional adobe block is made by placing moistened soil into a mould for setting and drying in the sun. Curing of these blocks requires them to be spread out and rotated, to ensure uniform exposure to the sun (Holliday *et al.*, 2016). However, these earthen building materials typically exhibit very low resistance to moisture deterioration. Thus, various techniques are employed to improve these building units and achieve acceptability as a modern construction material (i.e., to achieve adequate strength, durability and quality as specified in relevant regulatory standards).

Compressed earth bricks (CEBs) are one such improvement, in that units are made by compressing the soil, in a mechanical or hand press, into the desired form. This process enhances the strength by facilitating bonding of the soil constituents and densification of the unit. However, a major drawback of CEBs is their lack of durability, especially in places of moderate to high rainfall (Adam and Agib, 2001). Stabilizers (such as cement, building lime, bitumen, fly ash and pozzolans) are commonly added to the soil in order to improve strength and resistance to moisture deterioration. These units are referred to as compressed stabilized earth blocks (CSEBs). CSEBs are produced by mixing soil with an appropriate stabilizer and water, and then mechanically compressing the mixture to form a block. Furthermore, CSEBs can be stacked during the curing process, which eliminates

the need for large amounts of space and labour associated with the production of adobe bricks.

More recently, due to the high cost and environmental impacts associated with typical stabilizers, the application of sustainable alternatives, such as natural fibre reinforcing, have also been investigated. In particular, natural fibres are cheap and sustainable additives that may be used to improve the ductility and strength of earthen blocks.

3.2.2 CSEB masonry

According to Walker (2004), the behaviour of earth masonry is generally similar to that of fired clay and concrete block masonry (masonry compressive strength values are typically lower than the corresponding block values). In particular, masonry compressive strength of prisms, built with compressed stabilised blocks (measuring 295 x 140 x 120 mm) and soil-cement mortar, was 78% of the corresponding unconfined block strength (Walker, 2004). However, Walker reported several differences between standard and earthen masonry, based on small and full-scale masonry prisms tested in uniaxial compression; namely:

- The initial tangent elastic modulus values were typically only 25–50% of the value expected of similar strength clay or concrete masonry.
- Peak strains, corresponding to maximum stress, were two to four times greater than those of concrete blockwork.
- Masonry prism compressive strengths decreased as block size increased; i.e., prism strength increased as the relative area of mortar joints increased.

Walker (2004) ascribed the third outcome, noted above, to significant decreases in block strength when the block height is increased. CSEB masonry is typically built with either soil:cement or cement:lime:sand mortars. Soil:cement mortars are typically weaker than blocks made from the same soil because they are not mechanically pressed. There is, however, disagreement in the literature whether mortars should be weaker or have similar compressive strength to that of the units, due to issues associated with shrinkage. Interestingly, Walker (2004) observed that masonry prism strength was only marginally influenced by mortar type and strength.

Jayasinghe and Mallawaarachchi (2009) conducted out-of-plane load tests of single-leaf CSEB masonry walls. The CSEBs were stabilized with 5% cement, and bonded with a cement-sand mortar (1:5). These authors reported that flexural strength was between 0.24 – 0.36 MPa, which is similar to that of fired-clay brick masonry.

3.2.3 Soil selection for production of CSEBs

The manufacture of strong and durable earthen bricks and blocks requires soil containing fine gravel and sand to provide the ‘body’ of the block, as well as silt and clay to bind the sand particles together (Adam and Agib, 2001). However, not all soils exhibit these desirable characteristics. In such a case, soils are mixed together to obtain a final soil with a well-balanced composition. Alternatively, these ‘poor’ soils could be improved with the addition of sand, waste aggregates and/or fibres, and subsequently chemically stabilized with a low amount of a binder, if needed. Nevertheless, a wide range of soils are suitable for the production of good quality stabilized earth bricks. Burroughs (2008) conducted various tests on numerous soil types, which were compacted and stabilized with lime or cement, to assess their suitability for stabilization. The author considered the soil suitable (or that stabilization was successful) if the compressive strength exceeded 2 MPa. From these tests, the soils most appropriate for stabilization are those with clay and silt content between 20 to 35%, plasticity index <15%, and linear shrinkage < 6%, as shown in Figure 3.1.

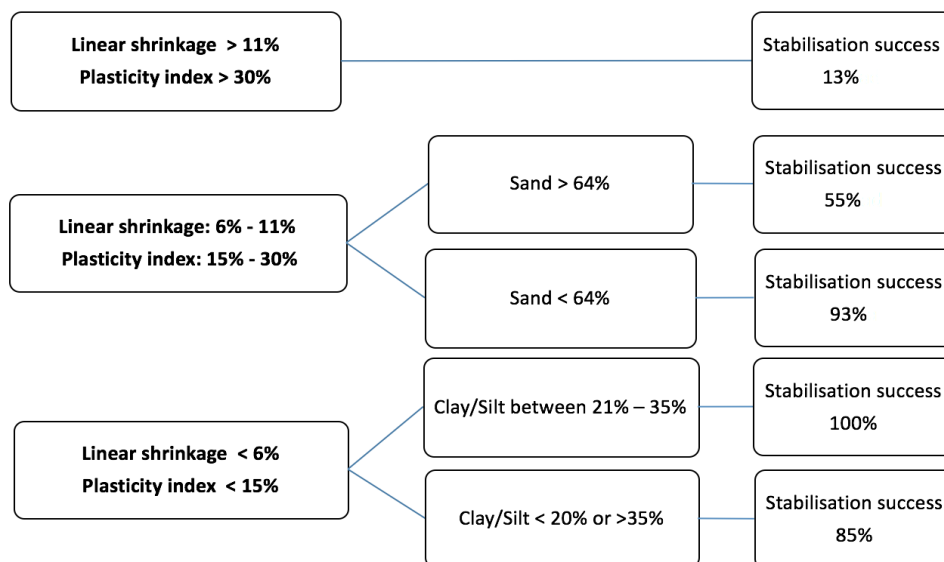
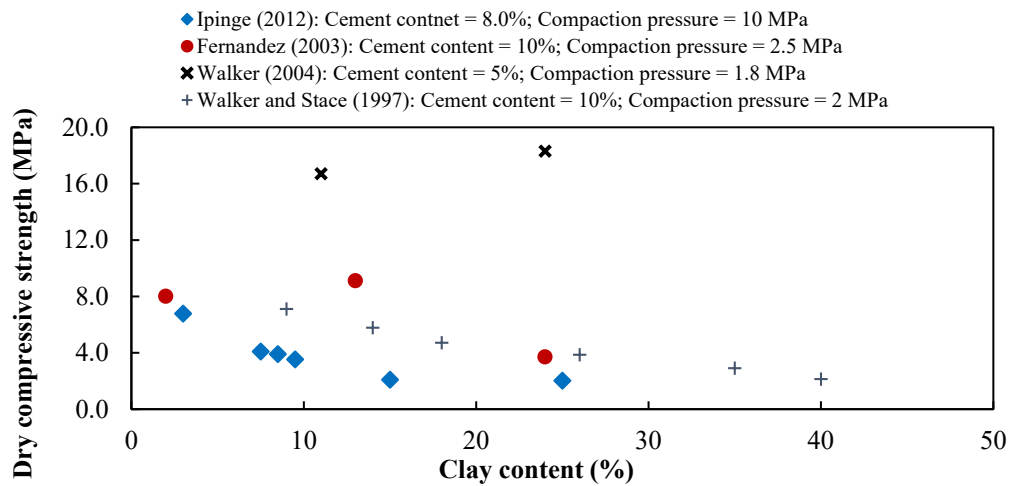
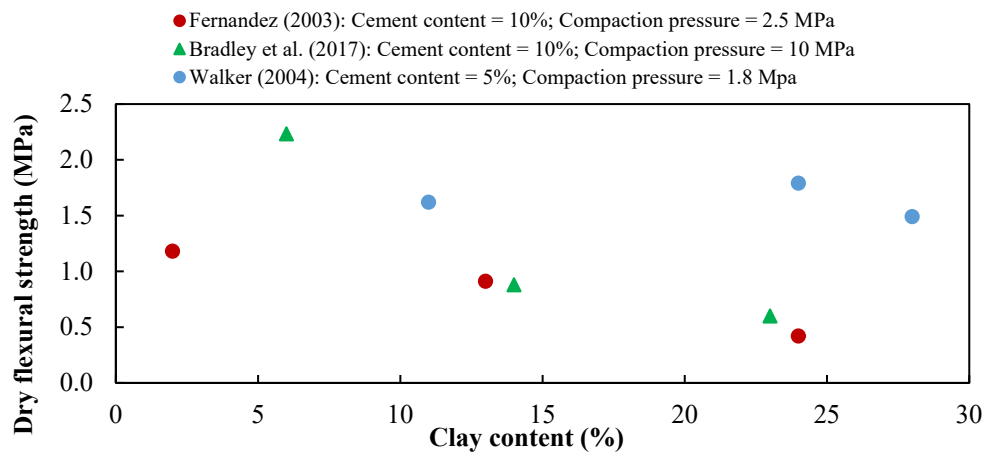


Figure 3.1 – Soil suitability for stabilisation (data from Burroughs (2008))

Ipinge (2012) and Walker and Stace (1997) showed that the strongest and most durable compressed cement stabilised blocks are typically those produced with minimal clay content. In fact, for a given quantity of cement and compaction, the compressive strength of CSEB decreases as the clay content increases. The reduction in strength is caused by disruption of cement binding due to the small clay minerals in the soil. According to Norton (1986), a soil with a 10% clay content (an effective minimum for the soil to be cohesive) may require only 5% cement, but the amount increases to 10% or more, when the clay content is 30%. Figures 3.2a and 3.2b show the results of confined compression and flexural tests on CSEBs reproduced from Walker and Stace (1997), Walker (2004), Fernandez (2003), Ipinge (2012), and Bradley *et al.*, (2017b).



(a) Compressive strength



(b) Flexural strength

Figure 3.2 –Strength of CESB vs. clay content

In contrast with the trends observed in these figures, Walker (2004) noted little or no deterioration in strength with increasing clay content. This author partly attributed this result to the relative increase in block density achieved with the higher clay content mixes considered in the study. In particular, blocks produced with soil having 11% clay content had an average density of 1914 kg/m³, whereas those made with soil containing 24% clay had density of 1980 kg/m³. The influence of density on the strength of earthen building units has been examined by Zak *et al.* (2016). The authors compared the compressive strength of earth blocks stabilized with cement, gypsum, flax, and hemp with their density. These authors noted that the compressive strength was highly dependent on the density of the blocks.

Ojo *et al.* (2016) analysed the effects of the mineralogical composition of soils on the performance characteristics of CSEBs. The results showed that soils with higher quartz and kaolinite content required lower levels of chemical stabilization. The authors noted significant correlation between quartz content and compressive strength and water absorption of the CSEBs.

Fernandez (2003) assessed the effect of particle size distribution on the compressive, flexural and tensile strength of compressed stabilized soil blocks. The author considered three soils; two were obtained from locations in Johannesburg, and the third was produced by mixing two soils to obtain a particle size distribution that matched, almost perfectly, the Fuller curve (with $n = 0.25$ for a clayey soil) for the soil with highest clay content. The results of the compressive and tensile tests indicated that the strongest blocks were those made with the mixed soil, even though it contained more clay and had lower density than the one produced with the soil having the highest sand content. According to the author, the results indicated that the particle size distribution influenced the strength of the blocks. In particular, the author postulates that the increase in strength, corresponding to the soil mix, was due to the optimization of the particle size distribution, which results in an increase in cohesion between soil particles.

Typically, the economic constraints, associated with a low-cost construction, govern the characteristics of the building unit selected for a particular project. In low-cost building projects, locally sourced soils should be used, when possible, to reduce costs. Preferably, soils should be assessed in a laboratory prior to block production; however, simple site

tests, such as the visual test, drop test, adhesion test or jar test, shrinkage test, can be used to establish the suitability of the material; i.e., clay content, soil composition, etc. (Houben and Guillaud, 1994; Bolton and Burroughs, 2001; Norton, 1997). These tests do not require special equipment and also provide valuable information such as minimum binder content, suitable binder material, and whether more clay or sand should be added to improve the soil. Bolton and Burroughs (2001) reported some important considerations for rural earth block production; namely:

- Never use topsoil. Suitable soils are found below the topsoil, usually starting at a depth from approximately 200 mm.
- Soil must be free from stones, plant matter (i.e., grass and roots), and rubbish (e.g., plastic and tins).
- Min. proportion of soil constituents: clay=5%; silt=10%; sand=25%; gravel=25%.
- Max. proportion of soil constituents: clay=20%; silt=25%; sand=50%; gravel=40%.

Earthen masonry units should also be evaluated prior to construction in order to ensure that performance requirements are met. Ideally, the strength of blocks should be tested throughout the project. This is certainly feasible in large low-cost housing projects. However, such quality control may not be possible in rural construction. According to Gohnert and Magaia (2004), more often than not, the strength of units produced in rural areas is not checked because equipment and laboratories are not accessible. There are, however, small block testers available, which incorporate a loading jack and gauge to easily assess the compressive strength (and quality) of bricks produced on-site. Simple compression testing systems, which are cheap, easy to construct and transport, are also available (Luker, 1999). Alternatively, flexural testing, which is easier to implement on-site, may be used as a substitute to compression testing as a means of assessing quality (Walker, 2004). According to Walker (2004), unconfined compressive strength may be approximated by $5 \times$ the bending strength. In Baglioni *et al.* (2016), the compressive strength was approximately $8 \times$ the bending strength, which was obtained from the three-point bending test. Norton (1997) and Bolton and Burroughs (2001) recommended a simple three-point bending test, in which a person stands on the middle of a dry block spanning 300 mm between two supports, to ensure the blocks are of good quality. If the block breaks, it is not strong enough and should be improved.

3.2.4 Chemical stabilization

Chemical stabilization is accomplished through inclusion of various materials such as cement, hydrated lime, gypsum, bitumen emulsions and resins to improve strength, durability and to reduce cracking. Water repelling agents may also be added to improve durability. The most widely used stabilizers in developing countries, are Portland cement and lime because they are relatively inexpensive and readily available (Adam and Agib, 2001). More recently, however, due to the increased cost and environmental impacts associated with Portland cement, bitumen and lime, the application of more sustainable alternatives has been investigated, e.g. waste materials (Gutiérrez-Orrego *et al.*, 2016), natural polymers (Achenza and Fenu, 2006), and geopolymer binders (Ojo *et al.*, 2017).

3.2.4.1 Cement

Cement is the most commonly used stabilizer for the production of compressed stabilized earth bricks. The reaction between cement and water (i.e., hydration) results in the formation of calcium silicate hydrate (C-S-H) and hydrated lime. The C-S-H particles form between, and link, the soil particles creating a cementitious gel structure. Furthermore, additional cementitious bonds are created due to the reaction between lime, released during hydration of the cement, and clay particles. Adam and Agib (2001) noted that the soil-cement mix should be compacted immediately after mixing so that the freshly created gel does not break down, thus reducing strengthening.

Typical cement content for compressed stabilized earth blocks ranges between 3% and 10%. According to Walker (2004), the increase in block compressive strength with cement addition is generally fairly linear. The influence of increasing cement content on the dry compressive strength of CSEBs, assimilated from Ipinge (2012), Pave (2007) and Blight (1994), is shown in Figure 3.3. Similarly, flexural strength also improves with cement addition (Walker, 2004).

The amount of cement required for stabilization is primarily governed by the soil type and characteristics. Cement acts most efficiently on coarse sandy soils and ineffectually on high clay content soils. According to Adam and Agib (2001), cement can generally be used with any soil type, but becomes uneconomical for ‘clayey’ soils because more cement is required to reach comparable strengths. Cement inclusion may not be applicable for excessively cohesive soils. Jayasinghe and Kamaladasa (2007) noted high

reductions in the compressive strength of lateritic soils stabilized with cement, when the percentage of clay and silt fines exceeded 40%. Zak *et al.* (2016) showed substantial reduction in block strength with the addition of cement (at 5% and 10% by mass). Specifically, the strength of compressed unstabilised blocks, with no additive or stabilizer, was an order of magnitude greater than that of blocks stabilized with cement. The blocks assessed by Zak *et al.* were produced by using a cohesive soil with very low sand and gravel content (i.e., PI = 15%, sand content = 5% and gravel content = 3%). Soil stabilization with cement is strongly affected by the presence of organic matter because it slows and/or inhibits the cementing action (Torgal and Jalali, 2011).

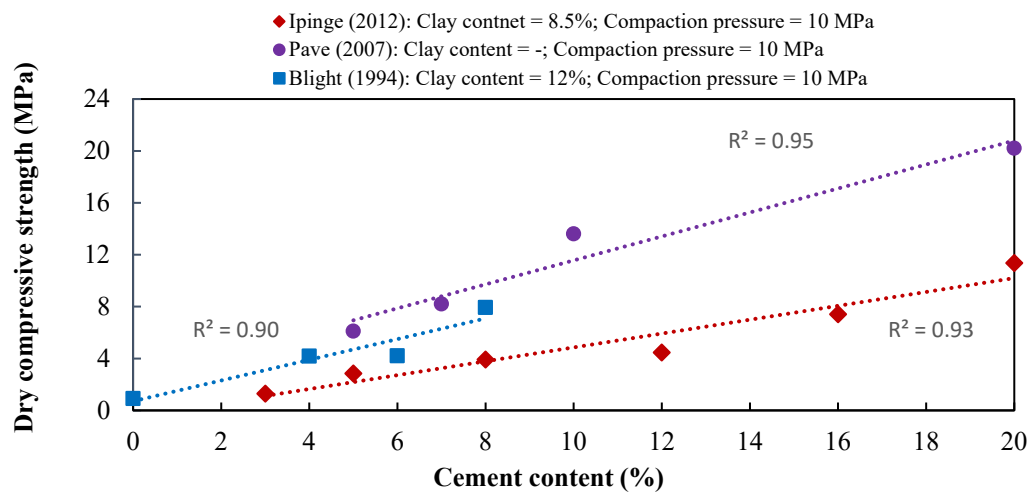


Figure 3.3 – Influence of cement content on unconfined compressive strength

3.2.4.2 Lime

Millogo *et al.* (2008) investigated the effects of lime addition on the material and absorptive properties of adobe bricks produced from clayey soils. The authors noted that the use of a lime content of 10% maximized compressive strength and minimized water absorption of the adobe bricks. According to Adam and Agib (2001), if a compacting effort between 8 – 14 MPa is applied to a soil with high clay, the required quantity of lime may be slightly reduced for the production of CSEBs. This is because the volume of air voids is reduced, bringing the lime and soil particles into closer contact enabling stabilizing reactions to take place more readily. The Auroville Earth Institute recommends lime proportion between 2 and 10%, with a typical quantity of 6%, for the production of compressed stabilized earth bricks. Typically, blocks stabilised with lime

require longer curing time, usually greater than 28 days, to achieve acceptable strength for construction. Furthermore, lime charlataneously does not yield blocks as strong as those produced with Portland cement. However, Burroughs (2008) and Gamrani *et al.* (2012) showed that blocks (and rammed earth) stabilised with lime may achieve strengths much higher than those stipulated in building regulations. Gamrani *et al.* (2012), examined the mineralogical, chemical and mechanical characteristics of the rammed earth of the Saadian sugar refinery of Chichaoua (XVIth century, Morocco). The results of uniaxial compression tests, on cubic samples, demonstrated that lime stabilised earth can attain compression strength as high as 3.5 MPa.

Lime is usually more expensive than cement and may be harder to acquire in some rural areas unlike cement, which is a common binder and available in all building supply shops and hardware stores (Bolton and Burroughs, 2001). Nevertheless, air lime, which reacts with carbon dioxide (CO₂) in the air to set, provides several environmental advantages over cement. Air lime is calcined (burnt) at much lower temperatures (900 – 1000°C) when compared with Portland cement (1400-1450 °C) and also captures CO₂ to form carbonates (e.g. CaCO₃), partially offsetting emissions during production.

3.2.5 Mechanical stabilization

3.2.5.1 General

Mechanical stabilization is accomplished by using manual or automated presses to reduce the air void volume, consequently increasing density, and improving the uniformity of the earthen building unit. The structure of mechanically stabilized units depends on the friction along the walls of the mould and compaction pressure and rate (Mesbah *et al.*, 2004). Adam and Agib (2001) noted that the main outcome of soil compaction is the increase of strength and reduction of permeability. The level of compaction possible, and consequently the quality of compressed earth bricks (CEBs), is affected by the soil characteristics, moisture content, and the applied compression. Typically, CEBs produced manually require more labour, and are more time-consuming to manufacture, than those made with automated presses. However, in many developing countries this may likely be a more cost-effective solution than large machine presses. Moreover, these blocks can be made on-site, thus reducing transportation costs. More recently, the development of small

mobile hydraulic machines enables for on-site production in remote areas and also reduction of transportation costs.

3.2.5.2 Strength and density variation

A concern with mechanically stabilised blocks relates to variable compaction achieved during production. In particular, the density of blocks produced with hand and single action machine presses are not consistent. The density decreases with distance away from the ram due to the effects of friction along the walls of the mould (Ipinge, 2012). Furthermore, greater density reduction can be expected as specimen height is increased because of greater side friction effects (Walker, 2004). Figure 3.4 shows the reduction in block density, and also masonry prism strength, as CSEB height is increased (Walker, 2004). The strength of CSEB masonry prisms decreased by 43% when the block height was increased from 23 mm to 113 mm; the blocks were mechanical hand-pressed along their height, which is typically the shortest dimension for brick, block and tile.

Some machines compress the soil, within the mould, along the length; consequently, blocks may show significant variation in density from one end to the other. Table 3.1 shows the confined compressive strength of cubes (or portions) cut from the top and bottom of dry-stack and normal blocks that were pressed along their length during manufacturing, as well as the strength of entire blocks. “Top cubes” and “bottom cubes” refer to a sample drawn from the top and the lower part of the block, respectively. The dry-stack blocks had dimensions of 110 mm x 220 mm x 270 mm long whereas the bricks, examined by Karam *et al.* (2017), were 90 mm x 140 mm x 185 mm. The units assessed by Ipinge (2012), Alwar *et al.* (2016) and Karam *et al.* (2017) were produced with the Hydraform M7E380V interlocking block-making press, which compresses the soil in the mould from the bottom upwards, whilst a secondary ram, at the top, keeps the soil in place. The individual cube strengths for blocks examined by Alwar *et al.* (2016) are shown in Figure 3.5. These were produced with a sandy silty clay; the soil characteristics are presented in Appendix A.

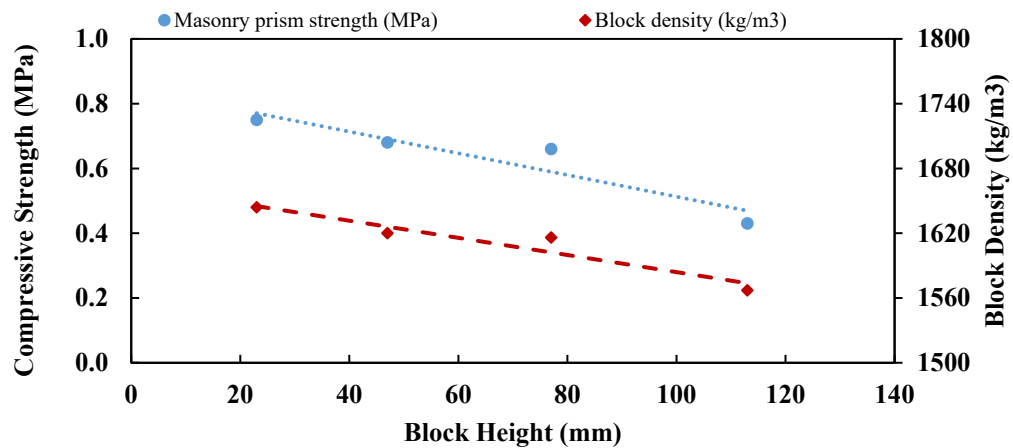


Figure 3.4 – Influence of block height on the strength and density of earth masonry (data from Walker, 2004)

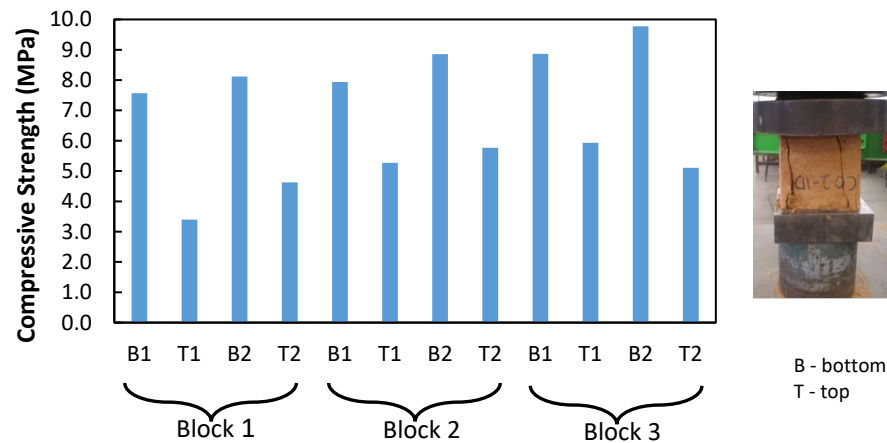


Figure 3.5 – Strength difference between the top and bottom of blocks (data from Alwar *et al.*, 2016)

The higher strength of entire blocks, observed by Pave (2007), Ipinge (2012), and Karam *et al.* (2017) can be attributed to the effects of platen restraint during testing. Walker (2004) investigated the influence of aspect ratio (i.e., specimen height to width) on block strength, and noted that as the block height reduces, the frictional restraint of the end platens on specimen deformation increases, thereby increasing apparent compressive strength. If empirical height/width correction factors, adopted from Walker (2004), are applied to the results from Pave (2007), the block strength falls between that of the top and bottom cubes, as shown in Table 3.2.

Table 3.1 – Influence of density on confined block strength

	Pave (2007)	Ipinge (2012)	Alwar <i>et al.</i> (2016)	Karam <i>et al.</i> (2017)
Strength (MPa) – top	3.2	2.3	5.1	1.9
Strength (MPa) – bottom	5.3	3.0	8.5	2.8
Strength (MPa) – entire block	6.7	4.1	-	4.8
Strength ratio (top/bottom)	0.60	0.77	0.60	0.67
Density (kg/m ³) – top	-	-	1867	1583
Density (kg/m ³) – bottom	-	-	1957	1670
Density ratio (top/bottom)	-	-	0.95	0.95

Table 3.2 – Adjusted block strengths to account for platen restraint (confined strengths from Pave (2007))

Specimen type	Strength (conf.)	Height/Width	Correction Factor	Strength (unconf.)
Top cube	3.2	1.00	0.70	2.2
Bottom cube	5.3	1.00	0.70	3.7
Block	6.7	0.52	0.53	3.5

The variation in strength between top and bottom cubes/portions is significant, as demonstrated by the strength ratios presented in Table 3.1. Considering the experimental data from Alwar *et al.* (2016) and Karam *et al.* (2017), there is a 5% reduction in overall density, between top and bottom portions, resulting in a 30 to 40% drop in compressive strength. The aforementioned variability is not a major issue for CSEB shells because the compressive stresses are typically an order of magnitude lower than that of the masonry. Furthermore, bricks will also have to meet minimum strength and durability requirements. However, it is important that the suitability of a brick be based on unconfined saturated compressive strength, as recommended by Walker (1995), Walker and Stace (1996) and Walker (2004). Walker and Stace (1996) advocate a minimum unconfined wet compressive strength of 1 MPa to satisfy strength and durability.

Because the durability of CSEBs is influenced significantly by density, meeting the aforementioned condition is particularly important for longer bricks/blocks, pressed along their length. Importantly, the expected durability characteristics may be overestimated when adopting tests on dry units alone. This is illustrated in the results from tests on saturated and dry bricks conducted by Karam *et al.*, (2017). The bricks used in that study were produced with a soil defined as a clayey sand with gravel (see Appendix A for soil

characteristics) and 5% ordinary Portland cement (by dry mass), and pressed in a block making machine. Table 3.3 shows the uniaxial confined saturated and dry compressive strengths corresponding to two different brick lengths denoted short (90 mm) and long (185 mm). The average strength values were calculated from four samples corresponding to each brick type, and the saturated specimens were immersed in water for 24 hours prior to testing, in line with standard practice (Walker, 1995).

Table 3.3 – Influence of density on strength (Karam *et al.* 2017)

Brick type	Confined dry comp. strength (MPa)	Confined saturated comp. strength (MPa)	Wet/Dry
Short (90 x 140 x 90)	3.1	1.5	~0.50
Long (90 x 140 x 185)	4.8	1.2	~0.25

The saturated strength of the short bricks was 50% of that of corresponding dry specimens, whereas the strength was only 25% for the long brick. When correction factors are applied to the confined saturated compressive strength of both bricks, only the longer unit fails to meet the recommendation of 1 MPa. These results further highlight the importance of saturated compressive strength when assessing the suitability of a particular compressed earth brick or block.

A preliminary investigation into the influence of variable block density on the strength of masonry prisms was also undertaken by Karam *et al.* (2017). The prisms were built from either the short or long bricks, mentioned previously, in a cement:lime:sand mortar (1:3:9). The bed- and head-joints were approximately 10 mm in size, but are larger near the surface due to the chamfered corners of the bricks shown in Figure 3.6a. The confined dry compressive strength of the bricks and the mortar cube strength are presented in Table 3.4. Only four prisms were examined, two for each brick type, and thus future work is required to better understand the influence of these variable material characteristics on masonry strength. Nevertheless, the observations were: (1) the compressive strength of the panels manufactured with the smaller bricks were not weaker than those built with the longer bricks, which is dissimilar to standard CMU and clay brick masonry (i.e. standard masonry strength increases as the size of the unit increases and number of joints decrease); (2) cracks in the panels built with longer blocks were generally straight, passing through brick and mortar, as shown in Figure 3.6b. In contrast, cracks in the panels built with the shorter bricks were generally stepped as shown in Figure 3.6c. This

is likely due to the larger number of head-joints in the panel shown in Figure 3.6c, i.e. 27 in Panel 2 versus 12 in Panel 1.



(a) Long brick (left) and short brick (right)



(b) Panel 1 (Long brick)



(c) Panel 2 (Short brick)

Figure 3.6 – Masonry panels after failure

Table 3.4 – Brick, mortar and prism strength (Karam *et al.*, 2017)

Brick type	Confined dry comp. strength (MPa)	Mortar cube strength (MPa)	Prism Strength (MPa)
Short (90 x 140 x 90)	3.1	0.9	1.1
Long (90 x 140 x 185)	4.8	0.9	1.0

3.2.5.3 Block production challenges

The strongest compressed stabilized blocks are typically those which incorporate minimal clay content. However, when using an automated press, it may be difficult to handle blocks made from soils with insufficient clay (i.e. non-cohesive soils), which acts as a binder. These blocks may disintegrate during removal from the mould, or whilst transporting the blocks to the curing area. Hydraform (2012) recommends a minimum clay and silt content of 10% for acceptable block removal from their machines. Delgado and Guerrero (2007) noted that soils should have minimum of 5% clay content. Though

blocks were successfully manufactured by Fernandez (2003) and Ipinge (2012) with clay content as low as 2% and 3%, respectively. On the other hand, CSEBs made from soils with high clay content are hard to remove from the mould due to adhesion between the block and rammer. Typically, part of the block remains stuck to the rammer during demoulding whilst the remainder breaks off. This issue can, in some cases, be overcome in the laboratory, where blocks can be slowly and carefully removed from the machine press. Understandably, the demand for rapid block production on-site necessitates lower clay content, but still a sufficient amount for the soil to be cohesive, so that blocks are not compromised during removal and stacking.

3.2.6 Fibre reinforcing

3.2.6.1 Improving strength and post-cracking characteristics

Due to the high cost and environmental impacts associated with the usage of typical binders (i.e. cement and bitumen), more sustainable alternatives have been investigated. In particular, natural fibres are a cheap and sustainable additive to improve the ductility, strength and durability of earthen blocks. These materials are typically less energy intensive and may be grown and processed locally. The use of natural fibres in earthen materials is certainly not a new development; straw, in particular, has been used in earth construction for thousands of years. The types of fibres incorporated into *fibre reinforced concrete* (FRP) (and also earthen composites) can be broadly classified into two types; namely:

- High strength, high elastic modulus fibres (e.g., steel, carbon, glass, and basalt) which improve strength and stiffness of the composite material.
- Low elastic modulus, high-elongation fibres (e.g., straw, polypropylene), which assist in controlling cracking in plastic concrete or cohesive soil (i.e., soils with high plasticity), but do not improve strength.

Most natural fibres, adopted in earthen construction, fall within the second category noted above. Mesbah *et al.* (2004) observed, from tests on blocks stabilised with low-stiffness fibres, that the following parameters influenced the tensile strength of fibre reinforced earth blocks: 1) Tensile strength of the plain earth matrix, 2) fibre tensile strength, 3) bond strength between the fibres and earth matrix, and 4) the fibre anchorage length.

Steel reinforcing fibres have been used to reinforce concrete for over a hundred years. The first steel fibres were simply cut from wire, and were, consequently, straight and smooth. However, modern steel fibres are typically “roughened”, through processes such as crimping and corrugating, to improve the bond between fibre and matrix. Normal steel fibres, however, are prone to corrosion, which could ultimately diminish the life of a masonry shell if not protected from oxidation. This shortcoming may be overcome by implementing more resistant varieties of steel, such as carbon steel, corrosion resistant alloys or stainless steel. Though the inclusion of such materials may be inconsistent with the principles of sustainability. It is largely accepted that steel should be minimized or removed entirely within earthen masonry due to the negative effects of corrosion.

Synthetic fibres used in concrete are typically incorporated in low-volume percentage (0.1% - 0.3%) or high-volume (0.4% - 0.8%). However, according to Perrie (2008), natural fibres need to be included at a minimum quantity of 3% (by volume) to improve the mechanical properties of concrete. The quantity of fibre included in earthen building materials is typically between 0.1% and 4% by dry mass. Many natural fibres, adopted in earthen construction, exhibit low elastic modulus; consequently, the primary purpose for their inclusion is to control shrinkage cracking in blocks made with clayey soil (i.e. soils with high plasticity) and improve post-cracking ductility.

Yetgin *et al.* (2006) conducted a series of shrinkage and uniaxial loading tests (compression and tension) on five different adobe mixtures. Each of them was tested with and without straw fibres, ranging between 0.8% and 3.8% by weight, obtained from wheat (*triticum*) stalk. The fibres investigated were hollow, 50 mm in length, and 3 mm in diameter. With an increase in fibre content, the amount of shrinkage decreased. However, the compressive and tensile strength of blocks decreased with increasing fibre content. According to Yetgin *et al.* (2006), the decrease in strength is because straw is a soft fibre (i.e., it has low stiffness) and adherence between the fibres and soil matrix is weak. Parisi *et al.* (2015) provided a comparison of existing experimental data on adobe reinforced with straw fibres. These authors noted that studies by Bouhicha *et al.* (2005) and Galán-Marín *et al.* (2010) report a contrasting relationship between unit compressive strength and fibre content. However, both studies reported that tensile capacity of adobe samples increased with increasing straw fibre content (Parisi *et al.*, 2015), which is dissimilar to data from Yetgin *et al.*, (2006).

Zak *et al.* (2016) assessed the compressive strength of earth blocks stabilized with several chemical and fibre additives. Two natural fibres were examined, namely hemp and flax (included at 1% and 3% by mass). The results from tests on blocks stabilized with hemp showed that fibre inclusion reduced the strength, similarly to the results from Yetgin *et al.* (2006). However, these authors attribute the loss of strength to increased porosity and reduced density generated by the addition of low-density fibres. Density is further reduced because of the additional water required in the production process of fibre reinforced blocks (i.e., higher water content was needed for the soil to reach a plastic state, when compared to soil without fibres). Flax fibres also reduced the density of blocks; however, their inclusion did not negatively affect the compressive strength of blocks due to the superior strength of flax, which is much stronger than hemp. Ductile failure of the fibre-reinforced blocks was observed.

Ghavami *et al.* (1999) investigated physical and mechanical (compressive strength) properties of soil reinforced with coconut and sisal fibres, with 4% fibre inclusion by mass. One of the significant effects of the inclusion of natural fibres in the soil matrix was the prevention of shrinkage cracks due to the drying process. Furthermore, the inclusion of 4% sisal imparted considerable ductility but only increased the compressive strength slightly.

Aymerich *et al.* (2012) and Aymerich *et al.* (2016) examined notched soil samples in flexure, which were reinforced by hemp and wool fibres at 2% and 3% by weight, respectively. In both studies, fibrous reinforcement greatly improved the peak load, ductility and energy absorption of samples after first cracking. Though the authors noted that the beneficial action of fibres only becomes apparent when the magnitude of crack opening was sufficient to facilitate strengthening through bridging and frictional fibre pull-out.

Mesbah *et al.* (2004) presented the technical aspects of a direct tensile test, developed for the assessment of compacted earth blocks reinforced with natural fibres. An objective of the study was to assess the tensile characteristics of compacted earth blocks reinforced with randomly distributed sisal fibres. The fibres were incorporated within the soil mixture at 0.5% by mass and at two different lengths, namely 20 mm and 50 mm. The authors conclude that fibre inclusion had no noticeable effect on material behaviour prior

to cracking. Furthermore, peak strength was not influenced by the inclusion of fibres. The authors attribute this result to the low elastic stiffness (i.e., 11.8 GPa) of the fibres, and also because the relative displacement between the fibres and matrix was insufficient to mobilize frictional resistance. Sisal fibres did, however, improve ductility in tension, in comparison with plain earth blocks, and also inhibited crack propagation.

Fernandez (2003) investigated various parameters that contribute to the strength and durability of sisal fibre reinforced cement stabilized earth blocks. Fibres were included at 0.5% and 1% by dry mass. One of the notable findings was that not all soil types are suitable for fibre-reinforcing; fibres enhanced the strength of bricks made with clay soils only, where enough friction was obtained in the interface with the soil matrix. Furthermore, by reducing the fibre content from 1% to 0.5%, an increase in compressive and flexural strength of the blocks was observed; flexural strength was increased by a maximum of 55% with the inclusion of sisal fibres at 0.5%. the author also noted a considerable drop in flexural strength from 7 to 28 days in fibre reinforced blocks. The author attributes this to either fibre degradation due to alkaline hydrolysis or drying shrinkage of the fibres, combined with volume changes in the vicinity of the fibres during drying, which may have induced micro-cracking.

Bradley *et al.* (2017b) investigated the effects of basalt fibre and soil type on the flexural strength of CSEBs. Only low concentrations of fibre were examined considering the relative high manufacturing costs of basalt fibres. In addition to fibres, most blocks were stabilised with 10% cement by mass. The details of these tests and the analysis of their results are presented in Appendix A. Conclusions from these tests endorse those of Fernandez (2003), in that not all soil types are appropriate for fibre reinforcement. In particular, the inclusion of low concentration of basalt fibres only marginally improved the flexural capacity of blocks produced with a soil classified, following the unified classification system (USCS), as a sandy silty clay. Blocks produced with two other soils classified as a clayey sand with gravel and a fat clay with sand appeared to show no influence of the fibre inclusion on the flexural strength. In fact, the soil characteristics and cement content had a far greater impact than did the low-concentration inclusion of basalt fibres.

Although most studies show only marginal strength improvement at low fibre concentrations, Gutiérrez-Orrego *et al.* (2016) noted that the inclusion of mineral wool residue (< 0.7 mm long at 4% by mass) and sisal fibres (approximately 70 mm long at 0.1% by mass) lead to a 10- and 20-fold increase in flexural strength, respectively. A much lower improvement in compressive strength was observed (i.e., an increase of under 20% with sisal fibre inclusion at 0.4% sisal). The authors noted that the sisal fibres exhibited surface grooves that could possibly improve their adhesion to the soil-cement matrix. Stiffness of the sisal fibres was 25 GPa, which was twice that of fibres used by Mesbah *et al.* (2004). The flexural strength of blocks reinforced with mineral wool increased with increasing fibre concentration, whereas the opposite was observed for the sisal reinforced blocks, similarly to the results in Fernandez (2003). Orrego *et al.* (2016) attributed the decrease in strength, with increased sisal fibre concentration, to the fibre, which absorbs water, expands in the matrix, and later contracts during drying, leading to a deficient fibre-matrix bond.

Significant variability exists in regard to the extent of strength improvement despite using similar fibre material and concentration (particularly sisal). Clearly, the characteristics of the fibre (e.g. strength, stiffness, length, water absorptivity, chemical composition, and surface roughness), and soil (e.g., clay content, sand content, soil plasticity, and linear shrinkage) vastly influence the success of a particular composite. Ghavami (1999) noted that numerous other factors influence mechanical behaviour of the natural fibres, such as soil condition, climate, time of harvest, process of fibre separation, and storage conditions. With such disparities in the data in the literature, further work is required to better understand the significance of these parameters. Practicality issues were raised by Fernandez (2003), particularly in low-cost projects, because earthen bricks stabilized with such materials may require considerable additional work for only marginal strength improvement. Bradley *et al.* (2017b) noted that if bricks are to be improved by incorporating fibres, substantial preliminary investigation (i.e. chemical and mechanical testing) may be required prior to manufacturing to ensure that the fibre and soil are compatible, and that the desired performance characteristics can be met.

3.2.6.2 Improving durability

Chandra (2003) and Houben and Guillaud (1994) provide information on the various methodologies to enhance the durability of earthen units and mortars, such as appropriate

soil selection and stabilization. Mesbah *et al.*, (2014) and Sharma *et al.*, (2016) noted significant improvements to the durability characteristics of earthen units with the addition of natural fibre reinforcement. Sharma *et al.* (2016) also reported that the addition of natural fibres (*Grewia Optiva* and *Pinus Roxburghii*) increased the “durability” of adobe samples by roughly 70% as compared with unstabilised samples.

3.3 Improving the longevity of earthen shells

Buildings constructed with good quality compressed stabilised earth blocks, following good block-laying practices, do not require external renderings to protect the masonry from rain penetration (Adam and Agib, 2001). The improved durability properties of CSEB motivated the adoption of earthen masonry as the final exterior finish; unlike traditional adobe masonry, which requires additional protection. Adopting earthen masonry as the exposed surface, in place of renders and paints, also offers long term economic benefits related to the reduced amount and frequency of maintenance operations. However, paints can be used, to great effect, as a means to improve the thermal performance of earth masonry shells. Like well-burnt clay brick masonry, good quality unrendered CSEB walls should be almost maintenance free. Numerous structures (residential, healthcare, school, and commercial) built with uncoated compressed stabilised earth brick masonry have been erected in many countries around the world (Hydraform, 2016; and Auroville Earth Institute, 2016).

However, in low-cost projects locally available soils should be used whenever possible. Consequently, in certain situations, the soils used to manufacture blocks, and possibly mortar, may need to be improved (e.g., if soils are excessively clayey then sand may be added to balance and improve the blocks). According to Morton (2008), earth materials must be well graded and not contain expansive materials (fibres and clay minerals). The trends shown in Figure 3.2a and 3.2b clearly endorse the use of soils with lower clay content (i.e., soils which contain just enough clay and silt to bind the block so that it does not disintegrate whilst handled on completion of production). Nevertheless, if the durability characteristics of the masonry are poor, rendering or other waterproofing elements must be used to protect the shell from erosion and water penetration.

3.3.1 Water repellents and waterproofing chemicals

Water repellents, such as bitumen, have been adopted in earthen masonry for decades to mitigate the negative effects of water. Typically, waterproofing chemicals and water repellents are either applied to external surfaces (normally by painting or spraying) or are added into the soil-cement mix during block manufacturing. Bulovic (2014) reported that the Wits shells were built with blocks, supplied by Hydraform, which incorporated a water repelling admixture called Pareflo (produced by Chrsyo). The admixture is primarily used in concrete applications where it reacts with the cement matrix, creating a polymer network that fills the capillary holes in concrete (Chyso, 2008).

Ipinge (2012) investigated multiple chemical admixtures produced by Badische Anilin und Soda Fabrik (BASF). In particular, the effects of Acronal[®] S400, Masterseal[®] 501 and Masterseal[®] 550 on the durability of earthen blocks were studied. All chemicals, whether included in the mix or painted over surfaces, enhanced the durability properties of the blocks. Though, according to Ipinge, increasing cement concentrations in blocks, instead of using chemical additives, was the most effective measure to resist abrasion and rain driven erosion. The best results, in terms of water sorptivity and uptake, related to bricks painted with the admixtures. However, painting is more time intensive than simple inclusion. Furthermore, shrinkage and structural cracking within blocks allows water to effectively bypass the superficial surface waterproofing, facilitating saturation and possible degradation of the untreated portions of masonry.

Ipinge (2012) did not compare the costs of the additives and the required additional labour as opposed to the addition of more cement to improve strength and durability. However, a material cost comparison is presented here, based on prices for CEM1 32.5 cement and MasterSeal[®] 501 that were acquired in July 2016. The prices for a 50-kg bag of cement, at R61.50 (\$4.40), and a 25-kg bag of MasterSeal[®] 501, at R800.00 (\$57.10), both excluding VAT, were obtained from a building supply retailer (Builders Warehouse) and BASF, respectively. These prices give a cost ratio, per kilogram of material, of approximately 26. At this cost ratio, none of the inclusions of MasterSeal[®] 501 were financially viable. Even at the lowest dosage investigated by Ipinge (2012), it would still have been cheaper to roughly double the original cement content, in order to achieve a similar saturated block strength. Thus, the inclusion of these additives appears far too costly for low-cost earthen constructions.

3.3.2 Renders

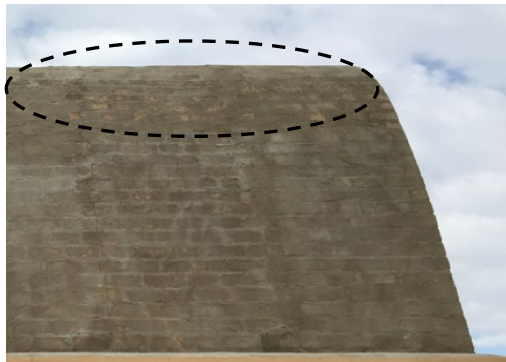
Typically, the most appropriate solution to improve longevity in low-cost developments is by applying rendering over external surfaces. Various methods of surface protection can be applied, but will often depend on the availability of materials and the budget. Membranes, chemical treatments (i.e. water repellents) and roof coverings (i.e. thatch, stones and tiles) have been applied to shelter earthen masonry from rain (Houben and Guillaud, 1994; Joffroy and Guillaud, 1994; and Adam and Agib, 2001).

The cheapest and often simplest form of render is earth; however, this layer requires consistent maintenance, particularly in areas prone to heavy rainfall. A sand-cement render presents difficulty because it is very rigid in comparison to the underlying earth material, as noted by Houben and Guillaud (1994). Joffroy and Guillaud (1994) note that a sand-cement mortar, over earthen material, causes cracking, which then enables water infiltration to take place. Joffroy and Guillaud (1994) recommended lime-sand or gypsum-lime-sand renders since they are better suited over earthen material because of their greater flexibility. Magaia (2003) noted that these renders act like blotting paper, absorbing and releasing moisture relatively freely. Although these renders are superior to a simple earthen render, some, albeit lesser, maintenance is still required.

Cheap renders are available, such as washes (lime, bitumen and cement), but most are only applicable to sheltered and/or internal surfaces. These are not appropriate for shells, particular in areas of moderate to high rainfall, due to their direct exposure to the elements (i.e., solar radiation and rain). Figure 3.7a shows the deterioration to a cement wash over the flatter apex of one of the steep catenary vaults built at the University of the Witwatersrand. This portion of the coating lasted less than six months. These shells were subsequently painted, as shown in Figure 3.7b, which offered a more durable solution. Furthermore, light coloured and infra-red (IR) reflecting paints can also be used to attenuate large surface temperatures and strains (Bradley *et al.*, 2018a), as well reduce heat gain and thermal discomfort in summer (Bradley, 2020).

Irrespective of the type of render applied over the surface of an earth masonry shell, it will require periodic maintenance to ensure longevity. An example of deterioration of a steel fibre reinforced cement-sand render over an abandoned semi-circular dome, which has not been maintained for several years, is shown in Figure 3.7c. An image of the dome

immediately after construction was presented in Figure 2.2a. Figure 3.7d shows the deterioration of a waterproofing membrane over an abandoned catenary vault in Bapong.



(a) Deterioration of a cement wash



(b) Painted shell



(c) CSEB dome (Johannesburg)



(d) Fired clay brick vaults (Bapong)

Figure 3.7 – Deterioration of renders and waterproofing membranes over CSEB and clay brick masonry shells

3.3.3 Prevention of water absorption

According to Morton (2008), earthen masonry should start at least 75 mm above floor level. Bolton and Burroughs (2001) recommend a more conservative value; the top of foundation walls should be at least 150 mm above ground level. This may be accomplished by extending foundations, using standard masonry materials (such as concrete blocks or fired clay bricks) in foundation walls, or by exploiting local stone built in cement-sand mortar in remote rural areas. Bolton and Burroughs (2001) advocated the use of strong cement-stabilised earth blocks for the construction of foundation walls. Such walls are built partly underground and, therefore, may become saturated. Consequently, the influence of moisture on the strength and durability on these units must

be established before a soil is adopted for construction. Gohnert *et al.* (2018) reported that several courses of traditional cement bricks, giving a total height of 230 mm above ground level, was adopted for the foundation wall in a low-cost prototype earthen shell built at the University of the Witwatersrand, as shown in Figure 3.8a and 3.8b. By using such an excessive height, an unbiased assessment of water penetration (from splashing and uptake) was ensured. Dry-stack blocks were used above these bricks for the construction of the remainder of the walls.

Earth masonry should also be isolated from ground moisture (i.e., rising damp) by including damp proofing. A damp-proof course was incorporated between the cement and dry-stack blocks in the Wits shells, as shown in Figure 3.8a. Figure 3.8c illustrates the benefit of adopting foundation walls and damp-proof course at the base of earth masonry buildings.



(a) Dry-stack blocks built above DPC



(b) Completed structure



(c) Standing water, adjacent to the south-facing wall, after a thunderstorm in late 2016

Figure 3.8 – Damp-proofing at the Wits shells

3.3.4 Channelling precipitation away from the structure

Although good quality compressed stabilised earth blocks do not require external renderings, proper design actions should be implemented to ensure water is not able to enter and saturate earthen masonry. Water should be channelled away from surfaces through spouts, guttering, and downpipes. The multiple shells in Bapong were designed with spouts, between adjacent catenary vaults, to ensure that water is channelled away from the masonry walls and vaults, as shown in Figure 3.9a. Figure 3.9b shows two damaged spouts, which no longer overhang from the masonry; whereas Figure 3.9c shows a crude attempt of a repair. It is unknown whether the remedial action shown in Figure 3.9c is a result of a lack of funds, damage or theft. In certain environments theft and/or damage to these types of elements may occur. These situations demonstrate the need for robust water conveying elements, such as spouts and pipes, which are resistant to abrasion and minor impacts.

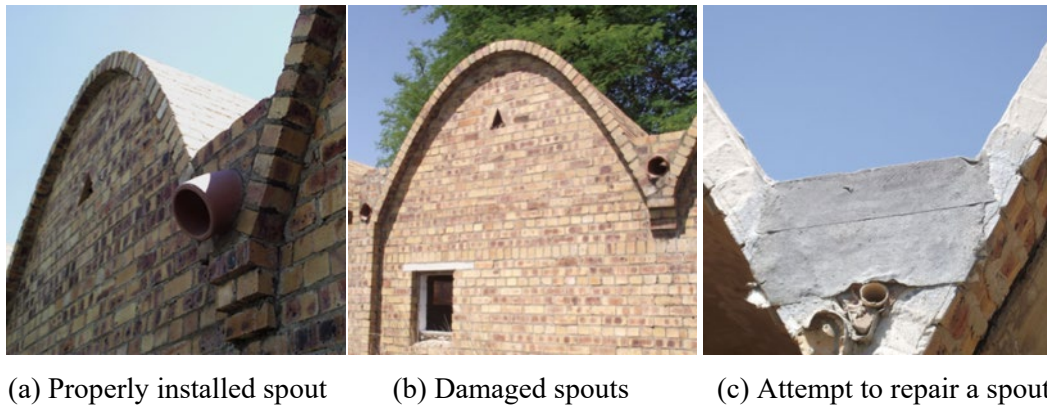


Figure 3.9 – Bapong vaults

To illustrate these concerns, it was observed that all of the Bapong structures have been stripped of doors and windows; and some even of lintels (above doors and windows), as shown in Figure 3.10. Fortunately, several structural measures including buttresses, ring beams, steel ties, and counteracting lateral thrusts (through juxtaposition of the vaults) were adopted to prevent lateral spreading of the supporting walls. This may well explain why a few of the shells still remain standing, even though several structural components have been damaged or removed.

Unlike the Bapong vaults, the Wits shells were not designed with spouts between the vaults; rather, the gap between the vaults was grouted, at a slight slope, to enable water to channel towards the east-facing side of the structure (Bulovic, 2014). This design detail caused rainwater to cascade off the edge and, consequently, run down the dry-stack walls below. This enabled water to leak through the dry-stack wall, as shown in Figures 3.12a and 3.12b. Forghani *et al.* (2016) noted that there is some concern about the water penetration performance of mortarless dry-stack masonry construction. It is not advisable to plaster dry-stack masonry, as is often done to improve the water tightness of traditional masonry, due to the possibility of reflection cracking over the joints. Rather, a solution to this problem could be to incorporate gap-fillers between the dry-stack blocks. Forghani *et al.* (2016) investigated the water penetration through *topographical* and *mechanical* semi-interlocking masonry (SIM) walls, with and without gap-fillers (i.e., putty and film tape). The authors noted that the water penetration through SIM walls without gap-fillers was excessively high for most practical purposes. The inclusion of a putty gap-filler, however, significantly reduced the volume of leaked water through the SIM wall. The water tightness of the wall built from topological dry-stack bricks with putty was comparable to that of a traditional CMU wall.



Figure 3.10 – Lintels removed from a vault in Bapong

Standard low-cost earth masonry housing structures also typically incorporate roof overhangs to prevent water from running and splashing onto the walls. Bolton and Burroughs (2001) recommend that roof overhangs, at eaves and gable ends, should

extend at least 200 mm beyond the outside walls. Burnt clay brick masonry vaults have been built in Bapong and Johannesburg (Vantillard Architects, 2014), as shown in Figures 3.10 and 3.11, with relatively short (<150 mm) gable-end overhangs. Some earth masonry vaults built in Auroville, India, have overhangs which appear to be longer than 500 mm (Auroville Earth Institute, 2016).



Figure 3.11– Burnt clay brick vault, Melville, Johannesburg

For several years of monitoring the Wits shells (Johannesburg), which do not incorporate any gable-end overhangs, no water penetration through the first-floor gable walls was observed (Bradley and Gohnert, 2018). However, significant water ingress was observed through gaps between dry-stack blocks during extreme rainfall in March 2018, as shown in Figures 3.12c and 3.12d. The South African Weather Service (SAWS) reported that Johannesburg recorded 117 mm of rain during a 24-hour period between 22 and 23 March 2018 (Floodlist News, 2018). This is about 16% of the average yearly rainfall, for the period 1960 – 1991, in Johannesburg. The local weather conditions clearly influence the extent and types of measures to channel water away from the surface of these structures. The observations in early 2018 highlight that overhangs should be employed to shield vertical walls, especially in dry-stack, adobe, or CEB applications. Additionally, overhangs toward the apex can limit solar heat-gain in the peak summer months.



(a) East-facing wall: Outside



(b) East-facing wall: Inside



(c) March 23 2018 (Outside)



(d) March 23 2018 (Inside)

Figure 3.12 – Water penetration between dry-stack blocks of the east-facing wall

3.3.5 Mitigating water penetration through cracks

3.3.5.1 Surface cracks

Vaults and domes are typically far more exposed to the elements (e.g., wind-driven rain, running water, and solar radiation) than walls. Furthermore, because the surfaces of shells are not vertical, cracks in the masonry are more accommodating to water penetration under the action of gravity. Water infiltration through surface cracks, created by thermally induced stresses, shrinkage, and repeated wetting and drying cycles, can become problematic if not repaired. Surface cracks within the external and internal render has been observed over numerous shells built in the inland provinces of South Africa, as shown in Figures 3.13 and 3.14.



Figure 3.13 – External Surface cracking in two CMU domes



Figure 3.14 – Internal surface cracking in a CMU dome (cracks traced for clarity)

At Sparrow Rainbow Village, these shrinkage cracks enabled water to pass through the underlying CMU masonry and drip onto furniture and appliances below. Several of the domes were repaired by repainting over the entire external surface, as shown in Figures 3.15a and 3.15b. However, one year later, the cracks were again visible over the repainted domes, as shown in Figure 3.16. Applying a few coats of paint over the surface is a relatively easy solution; however, thermally induced movement inevitably causes reflection cracking in the re-painted surfaces. This is clearly not a sustainable long-term solution because of the high costs associated with continuous repainting over these fine cracks.



(a) Cracked dome (November, 2012)



(b) Repainted dome (January, 2016)

Figure 3.15 – CMU dome: Sparrow Rainbow Village



(a) November, 2012



(b) February, 2017

Figure 3.16 – External surface cracking in a cement brick dome

A bituminous waterproofing solution was also used in an attempt to protect some of the dome. However, this was unsuccessful due to the steepness of the shells. Consequently, several of the domes were covered with layers of waterproofing membrane, which are somewhat expensive in a low-cost setting. A dome repaired several years ago is shown in Figure 3.17a (These repairs were not implemented by the author of this thesis). These membranes provide good protection from water; however, they too require maintenance due to inevitable impacts and abrasion. Damage to part of a membrane is shown in Figure 3.17b. Due to the relative high cost of such membranes, many domes at the Sparrow Rainbow Village were still unrepaired approximately five years later. Unfortunately, these repairs are carried out in stages, when the situation is considered serious and funds become available.

Economic constraints may well delay remedial action. In the absence of maintenance, the damage may become so severe, as was the case with the Swan House and Lab (De La

Rosa, 2013), that structures may require full restoration instead of just re-plastering and/or painting. Timely maintenance will be simpler and cheaper, in the long run, than extensive restorations. Additionally, the saturation of the masonry may result in persistent damp, which can become an acute problem if not remedied. In particular, damp encourages the growth of mould, which can negatively affect the health of residents.



(a) Dome in good condition

(b) Damage to a portion of the membrane

Figure 3.17 – Waterproof membrane over a CMU masonry dome

3.3.5.2 *Structural cracking*

Structural cracking in earth masonry shells can become problematic due to the increased potential for erosion. Two earth brick vaults built at the Swan House and Lab collapsed due to deficient maintenance and repair of large structural cracks. These types of cracks, which permit moisture into the masonry, should be repaired as soon as possible to safeguard the service life and safety of the structure.

Repairs to structural cracks, which expand and contract due to heating and cooling (i.e. dynamic cracks), can become an excessive burden due to the inevitable maintenance and associated costs. Standard rigid fillers are typically insufficient for these types of cracks. Figure 3.18a shows a repaired crack, at the Mapungubwe Interpretation Centre, identified as a continuous white line at the interface and highlighted with black arrows in the figure (These repairs were not implemented by the author of this thesis). Figure 3.18b shows this same crack over a year later, which indicates the failure of the adopted filler. Flexible fillers, which accommodate thermal movement between the adjacent sections, are more appropriate, especially in the South African climate. Results from Gohnert (2011) show

that temperature induced strains in the external plaster over CSEB dome were as high as $1500\mu\epsilon$ (measured between 7:00 AM and 3:30 PM).

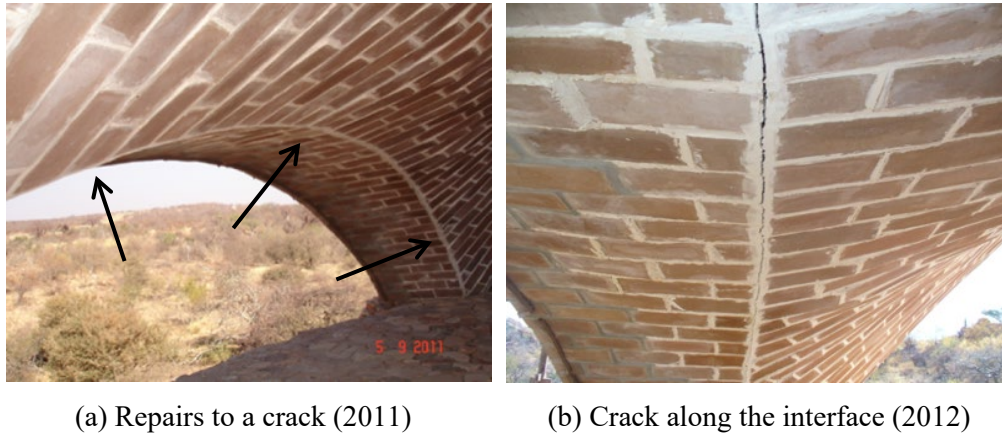


Figure 3.18 – Repairs to a crack at Mapungubwe

Extensive repairs were necessary to prevent moisture ingress through several vertical cracks in the Wits shells, as shown in Figure 3.19. The maximum measured width of these internal and external cracks were approximately 2.0 mm and 6.5 mm, respectively. These cracks could be classified, following Kaminetzky (1985), as light (0.8 to 3.2 mm) and moderate (3.2 to 12.4 mm). The extent of moisture movement through the vertical and horizontal cracks in the Wits shells is shown in Figure 3.20 (darker regions are noticeable in the image).



Figure 3.19 – Vertical cracks in the north-facing catenary vault (cracks traced for clarity)

The external cracks were cleaned of debris and then filled with silicone (applied with a caulking gun) to bridge the opening. Silicone was used due to its relative low-cost, broad

availability and ability to accommodate movement. The vaults were then painted with a plaster primer, followed by three coats of acrylic PVA to mitigate any further moisture ingress. A repaired north-facing vault at Wits is shown in Figures 3.21 and 3.22b.



Figure 3.20 – Moisture movement through the cracks in the Wits shells



Figure 3.21 – Repaired steep catenary shells at Wits (June 2015)

These actions prevented further moisture ingress for approximately 18 months; however, once the summer thunderstorms began in late 2016, water penetration became evident along one of these cracks. Figure 3.22c shows the extent of water penetration after a brief thunderstorm, in December 2016, whereas the photograph in Figure 3.20d was taken following several hours of light rainfall in January 2017. Thermal imaging was used to further highlight the issue of moisture movement through these cracks. Figure 3.21 shows a thermographic image of the north facing vault, which was generated with a FLIR ONE

PRO thermal camera. The image was taken on a clear summer's day at approximately 2:00 PM, a few days after a period of intermittent rainfall. Two distinct vertical bands of lower surface temperature, shown in Figure 3.23, coincide with the actual cracks shown in Figures 3.19 and 3.22a, highlighting the presence of moisture below the surface.

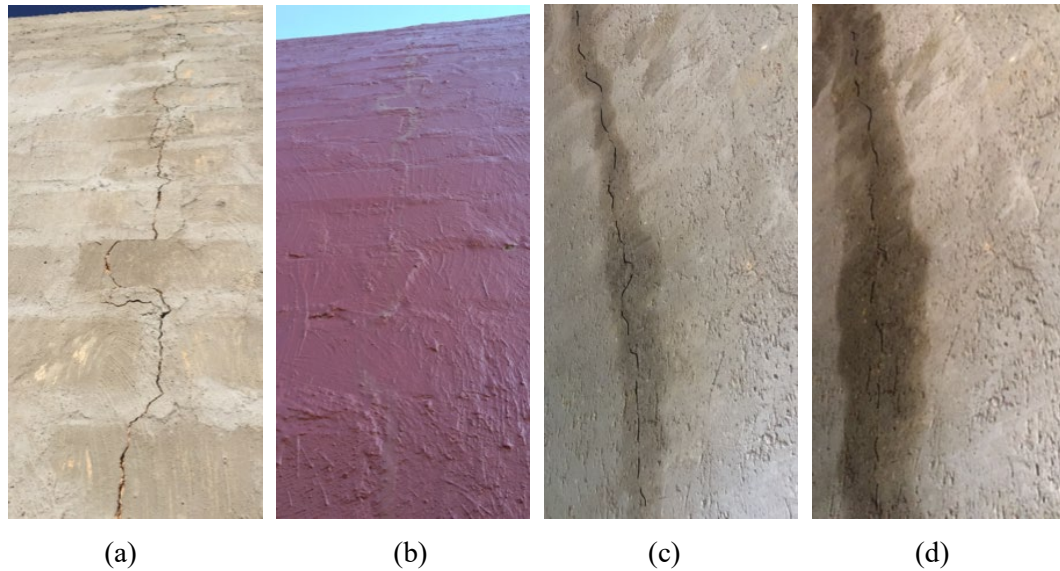


Figure 3.22 – Water penetration through a repaired vertical crack in the Wits shells (Cracks have been traced for clarity): (a) Large vertical shrinkage crack; (b) Repaired crack; (c) Moisture ingress after a brief thunderstorm; (d) Moisture ingress after several hours of rain

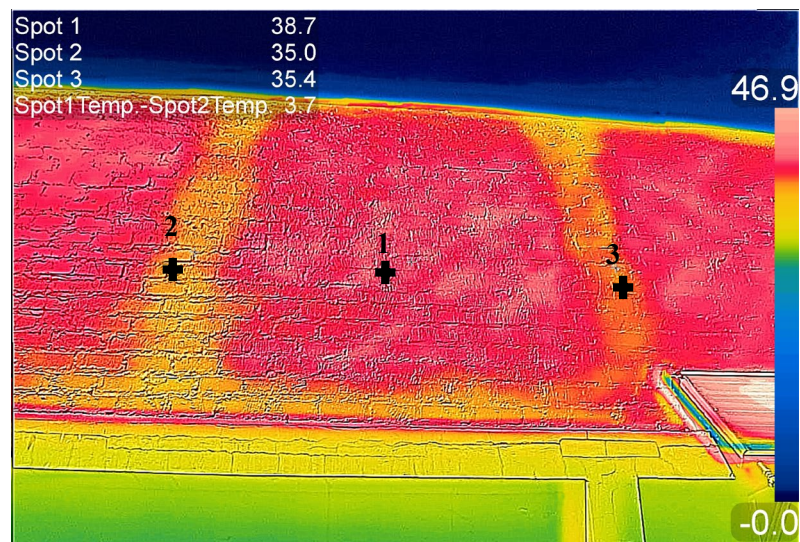


Figure 3.23 – Thermographic image of the north facing vault (image processed using FLIR Tools software application)

An alternative repair technique is engineered composite materials, which incorporate high strength cementitious mortars and reinforcing (e.g., textiles and mesh) to bridge and waterproof cracks. However, such systems may be costly, and require specialized tools and training for proper application. Figure 3.24 shows a repaired structural crack in one of the domes at Sparrow Rainbow Village (These repairs were not implemented by the author of this thesis). These cracks were caused by an impact of a falling tree in November 2015. The cracks were repaired with strips of waterproofing membrane mesh. The membrane strip, although unsightly, was able to accommodate the inevitable stresses developed through the restraint of diurnal and seasonal thermal movements, which may be significant in Johannesburg (Gohnert, 2012). Furthermore, no evidence of water penetration was apparent approximately one year later. However, over time water may enter at the edges or seams of these elements, if not regularly maintained as shown in Figure 3.25.



Figure 3.24 – Repairs to structural cracks in a CMU dome



(a) External

(b) Internal

Figure 3.25 – Water penetration through large cracks in a CMU dome

If shells are extensively cracked, and funds are available, waterproofing of the entire structure should be considered, e.g., several layers of membrane impregnated with waterproofing compounds. Alternatively, seams may be eliminated by using sprayed waterproofing, e.g., EPDM rubber. Several shells at the Sparrow Rainbow Village were covered with membranes, which have successfully prevented moisture from entering through the underlying cracks. A dome repaired several years ago, which is still in good condition, is shown in Figure 3.17a. Due to the relative high cost of such membranes, many domes at the Sparrow Rainbow Village were still unrepaired in 2017. Unfortunately, these repairs are carried out in stages, when the situation is serious and funds become available. It is apparent from this case study that economic constraints may well delay remedial action, which can become hugely problematic (e.g., the collapse of two vaults at the Swan House and Lab).

3.4 Summary and recommendations

The economic constraints associated with a low-cost construction typically govern the characteristics of the building unit selected for a project. The price of compressed stabilized earth bricks depends on the cost of labour, soil, binders, transportation, and machinery. In low-cost building projects, locally sourced soils should be used, whenever possible, to reduce costs. However, soil characteristics have a significant influence on the strength and durability of earthen blocks. The properties of clayey soils can negatively influence the durability of bricks and blocks. Thus, the use of excessively clayey soils should be avoided. However, in certain regions, this may be unavoidable due to either local geology and/or severe economic constraints. Several options are typically employed to improve the strength and durability of CSEB made with clayey soils, such as combining the soil with sand or other aggregates, adding higher concentrations of the binder material (e.g., cement and lime), or using natural fibre reinforcement. The most appropriate solution for a low-cost building typically depends on the availability of materials, and the associated cost of its purchasing and transportation to the site.

The amount of cement used in low-cost projects should also be kept to a minimum for economic and environmental reasons. However, at low cement concentrations, the strength of CSEB masonry may be significantly reduced when exposed to moisture. If the durability characteristics of the masonry are poor, then an appropriate render should be applied over external surfaces of shells. However, the rigidity created by shell curvature,

in combination with stiff plasters (i.e., cement:sand) often lead to cracking over the surface of these masonry structures. Many earth-block and cement-brick masonry domes, built inland South Africa, show extensive surface cracking. An alternative to rendering is the adoption of good quality CSEBs, which can provide a secondary function as the final finish.

Chemical additives can be used to improve CSEB durability. Although many are successful, some of these additives may introduce significant additional expenses (materials and labour) and may also not be readily available in remote areas. Usage of expensive materials, that need to be transported long distances is inconsistent with a low-cost, sustainable, design. Poor durability characteristics of some blocks may be more effectively improved by including more cement. Shells should incorporate damp-proofing as well as water channelling elements to ensure that the masonry does not become saturated. The service-life of these elements should be carefully considered, noting that adequate maintenance and repair are sometimes lacking.

Because of the low tensile strength of earthen masonry, some aspects that are overlooked (or not realised) during analysis, design and/or construction may become an acute problem. Cracking damage within earth masonry shells, whether built with adobe or CSEB, should be repaired immediately. The best solution to avoid water penetration through cracks is to mitigate the possibility of cracking through appropriate design, construction, and material specification. This is a primary focus of several following chapters, which examine the causes of performance failures and subsequently highlight design and construction measures to mitigate tension (and damage) within low-cost earth masonry shells.

CHAPTER 4. DESIGNING PURE-COMPRESSION EARTH MASONRY SHELLS

4.1 Introduction

The design of unreinforced shells characteristically encompasses form-finding and/or other analytical techniques to ensure a state of pure-compression under load, e.g., self-weight, imposed and wind loads. The fundamental aim being the elimination of tensile stresses. To achieve this objective, design methodologies must ensure that the calculated thrust-line (or resultant load path) remains within the middle-third of the cross-section at all locations within the shell. The concept of the middle-third comes directly from the combined action of bending and axial stress calculated using Equations 4.1 through 4.3 and is demonstrated for an arch in Figure 4.1. The first term of Equation 4.1 is an even distribution of stress due to the axial force, whereas the second term is the stress due to bending (i.e., the offset of the axial forces from the centreline of the arch).

$$\sigma = \frac{P}{bt} \pm \frac{6Pe}{bt^2} \quad (4.1)$$

where,

P = the axial force;

b = the width of the arch/segment;

t = the thickness of the arch/segment;

e = the eccentricity of the thrust-line from the centreline of the arch.

The positive/negative sign indicates an eccentricity, from the centreline, toward either the extrados (outside surface) or intrados (inside surface) of the arch. The maximum eccentricity for no tension is determined by setting Equation 4.1 to zero, as shown in Equation 4.2.

$$0 = \frac{P}{bh} - \frac{6Pe}{bt^2} \quad (4.2)$$

Equation 4.2 is then rearranged to solve for the eccentricity e , as shown in Equation 4.3.

$$e = \frac{t}{6} \quad (4.3)$$

Thus, the thrust line must remain within a band confined by $t/6$ on either side of the centreline, i.e., the inner third of the cross-section.

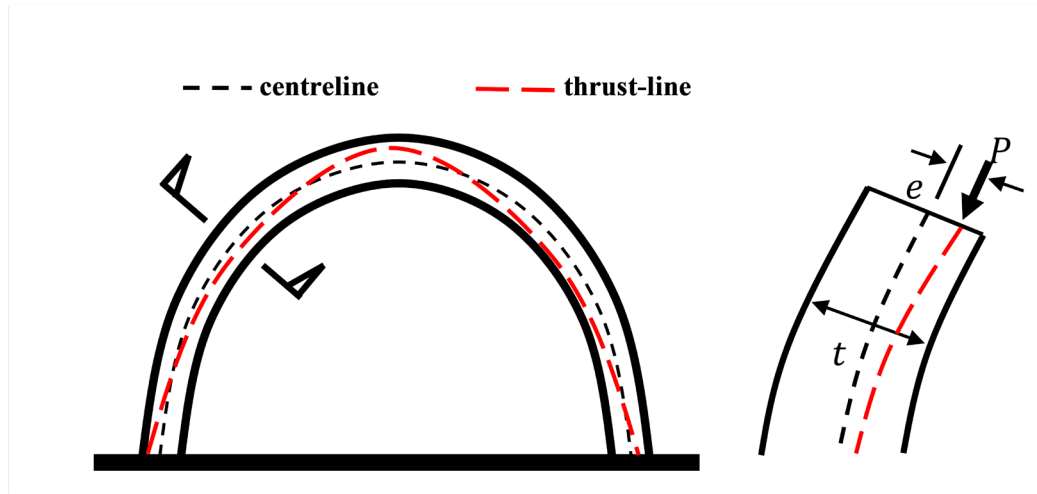


Figure 4.1 – Thrust lines within a shell segment

Several techniques have been employed to design and analyse masonry arches and shells including graphical methods (Wolfe, 1921; Galassi *et al.*, 2017), thrust-line analysis (Allen and Zalewski, 2010; Minke, 2009; O'Dwyer, 1999), thrust-network analysis (Block and Ochsendorf, 2007; Adriaenssens *et al.*, 2014; Marmo and Rosati, 2017), upper bound limit analyses (Heyman, 1969), analytical solutions (Farshad, 1977; Gohnert and Bradley, 2020b), and other numerical solutions (Gohnert and Bradley, 2020a; Tempesta, and Galassi, 2019). Most of these traditional and contemporary methodologies are largely based on lower-bound procedures. For example, the award winning Mapungubwe Interpretative Centre was designed using lower-bound thrust-line techniques (Ramage *et al.*, 2010) and the recently constructed steep catenary vaults at the University of the Witwatersrand were designed with a segmental equilibrium method (Gohnert *et al.*, 2018; Gohnert and Bradley, 2020a). Physical models, incorporating hanging chains or fabric, have also been used as a form-finding technique to 1) ensure pure-compression structures, 2) optimise material usage, and 3) achieve a desired geometrical shape/form. For example, several vaults and domes situated at the Auroville Earth Institute, India, were designed using hanging chain/string models (Auroville Earth Institute, 2016c).

In this chapter, a unique numerical form-finding formulation (the Moment-Step method) is presented for the determination of funicular forms. It is applicable to simple two- and three-dimensional structures such as vaults and domes. The method involves the minimization

of bending moments, which are highly inefficient forces, through an iterative numerical technique. The methodology is simple, and classical structural analysis techniques guide the approach. It is particularly appealing for the design of pure-compression vaults and domes having either uniform or varying section thickness. Initially, an arbitrary starting shape is discretised into a predefined number of elements, and the reactions and internal bending forces are determined. Subsequently, the position of each element is adjusted by a vertical geometrical step, which is directly dependent on the eccentricity of the thrust-line at the relevant point. The base reactions and internal bending forces are determined for the new shape, which is then adjusted iteratively until there is *zero* bending moment and the desired span-height-ratio is obtained. Optimised shapes of a vault and dome derived using the proposed method are compared with those determined using iterative thrust-line analysis techniques. The pure-compression shapes obtained using both methodologies are practically identical. The appeal of the new methodology are the simplicity of its formulations and ease of application. Several examples are presented to illustrate the method as well as highlight its usefulness to the design smaller free-standing shell structures.

4.2 Design techniques

4.2.1 Graphic Statics and the ‘true’ line of thrust

Wolfe (1921) explained the solution of thrust-lines within masonry arches and vaults using graphic statics and the method of least work. These techniques depend on the geometric and material properties of the arch for the determination of the “true” line of thrust. Similarly, modern and conventional structural analysis techniques (e.g., classical methods or finite-element analysis) are based on these same principles and assumptions. However, according to Heyman (1969), elastic design methodologies applied to a structure which is imperfectly elastic will give results that are safe in the sense of the plastic theorems, but which are not as exact as the designer thinks. Heyman (1969), reflecting on earlier elastic methodologies, noted that elastic methods are lengthy and rely heavily on a range of conventional assumptions. Moreover, even minor yielding at the abutments (i.e., support movement) may lead to cracking and a change in the internal stress distribution within the structure, making the calculated load path rather meaningless.

4.2.2 Limit-analyses

Limit analyses enable the assessment of safety (or collapse load) of masonry structures based on equilibrium and geometry rather than the strength of the material (Heyman, 1995). The fundamental reason limit analyses are preferred over elastic methodologies for the design and analysis of masonry arches (and shells) is that they do not rely on assumptions associated with the material properties. According to Heyman (1969), commenting on masonry arches, plastic methods are simpler than elastic methods, and yield results which are at least as reasonable as those determined using elastic methods.

The theory of engineering plasticity incorporates three theorems: a lower bound (or equilibrium) theorem, an upper bound (or mechanism) theorem and the uniqueness theorem. Romano (2005) identified two points on the applicability and reliability of limit analysis with respect to masonry structures.

- Infinite compression strength is assumed. Experience has shown how structures made of materials with poor mechanical properties often do not develop mechanism-like collapse, but rather large portions of the masonry crush.
- Though limit analysis reveals the weak points of the structure and provides a bound of the admissible horizontal action, it neglects, due to material assumptions, a few structural inelastic capacity issues, so that the safety assessment turns out to be pessimistic.

Heyman (1969) confirmed that the upper bound theorem of plasticity may be applied to masonry arches and vaults. Heymans's method is applicable to arches and vaults which collapse in a 'rocking' type failure. The adoption of the methodology suggests that masonry has a ductile moment capacity such as steel or reinforced concrete. Gilbert (2007) reported that experimental evidence largely supports such a premise. The author conducted tests on two bare masonry arch ribs (of different construction) to collapse and remarked "... the overall response is reasonably ductile, in that the fall off in capacity is relatively modest providing that the displacements are small".

Heyman's methodology incorporates three fundamental conditions, which he termed "the principles of ultimate load analysis of masonry constructions" (Kurrer, 2008). These are summarised below:

- The material has infinite compressive strength. This is equivalent to the assumption that stresses are very low in masonry; and consequently, there is little danger of crushing of the material.
- Sliding failure of the stones (or bricks) is ignored. The assumption is that friction is high enough or that the stones are effectively interlocked so that sliding will not occur. According to Kurrer (2008), the coefficient of friction for masonry is between 0.6 to 0.7 and according to O'Dwyer (1999), the assumption is correct in practice.
- The material has zero tensile strength. This is a conservative assumption for CSEB masonry because it has some tensile capacity and adhesion/bond exists between the mortar and bricks; i.e., CSEB masonry has shear and flexural capacity (Jayasinghe and Mallawaarachchi, 2009; Holliday *et al.*, 2016).

Heyman's 'safe' theorem of plasticity states that an arch will be stable (or safe) if a thrust line can be constructed in equilibrium with all the loads acting on it and is contained wholly within the masonry. Several researchers have utilised Heyman's safe theorem in conjunction with funicular analysis to develop numerical and analytical procedures for the determination of the collapse load or minimum shell thickness. O'Dwyer (1999) developed the force network model for the abovementioned purpose. Block and Ochsendorf (2007) developed the thrust network analysis, which allows the determination of funicular solutions for three-dimensional shell structures. A shell structure will be safe if a compression-only thrust network can be found that lies within the dimensional envelopes of the shell (i.e., a lower-bound solution). The lower bound techniques are preferred over upper-bound solutions due to the inherent difficulty in predicting the complex nature of possible collapse mechanisms. Lau (2006) noted that the mechanism procedures for the determination of the upper bound limits of complex three-dimensional shells remain largely unproven. However, such analyses have been employed to assess the safety of several historical masonry shell structures (Foti *et al.*, 2019). Recently, discrete-element modelling has also been applied to the analysis of masonry arches and vaulted structures (Pulatsu *et al.*, 2018).

4.2.3 Physical models

The method of using suspended chains to determine the shape of pure tension structures has been an architect's/engineer's tool for centuries (Kurrer, 2008). Robert Hooke first identified the relationship between a hanging chain and a rigid arch, about 400 years ago,

in the margins of his scientific works (Heyman, 1998). A hanging chain is in pure tension, and unable to develop bending moment and shear forces. If the links of the chain are locked, and the shape inverted, the tensile stresses are reversed to compression stresses, as depicted in Figure 4.2. The catenary arch derived from a hanging chain engenders a highly efficient structure because the location of the thrust-line and axis of the member coincide.

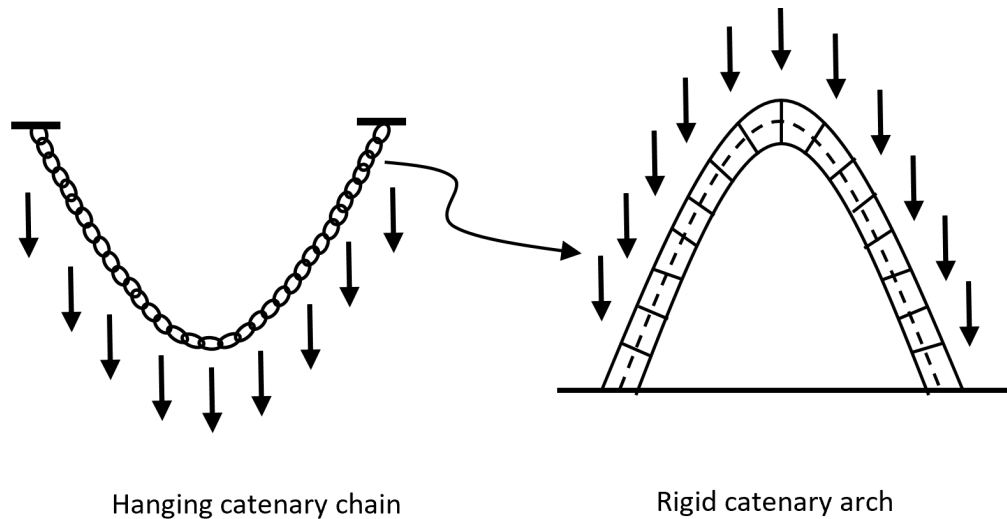


Figure 4.2 – A hanging chain forming the catenary shape (Gohnert and Bradley, 2020c)

Due to the low tensile capacity of masonry, and concrete, catenary forms are of interest when building unreinforced arches, vaults and domes. Several of the most celebrated structures incorporate catenary shapes such as Saint Paul's Cathedral in London, Saint Peter's Basilica in Rome and Sagrada Familia in Barcelona (Gohnert and Bradley, 2020c; Goshima, 2011). Possibly the most famous designer of catenary structures is Antoni Gaudí, who utilized hanging models with weights to find the shapes of large spires at the entrance of Sagrada Familia (Huerta, 2006). Gaudí also designed steep vaults for residential and recreational structures, such as the Chalet Catllaràs built in the Pyrenees in 1905 (Ciriot *et al.*, 2012). More recently, numerous CSEB shells built at the Auroville Earth Institute, India, were designed using 2-dimesional hanging string/chain models (Auroville Earth Institute, 2016c).

The celebrated swiss engineer Heinz Isler used hanging fabric/cloth models as a form-finding tool to determine the shape of complex thin concrete shell structures (Chuang and Chilton, 2016). The profiles of Isler's shells were determined by accurately measuring the surface coordinates of fabric models rigidified with resin, frozen water or plaster (Gohnert

and Bradley, 2020c). Like chain/string models, a hanging cloth model presents an efficient structural form by matching the thrust-network with the shape of the structure. The suspended cloth is in pure tension: unable to carry bending moments, shear or compression. However, the hanging cloth technique is limited to shallow shells (Chilton, 2012), due to the influence of the weave of the fabric, which distorts when modelling deeper profiles. These distortions or folds are discernible in the cloth model shown in Figure 4.3. The model was constructed by hanging medical gauze impregnated with quickset plaster-of-paris, which was then sprayed with water and allowed to set (Gohnert and Bradley, 2020c). The depth of cloth membranes is related to the number of supports—as the number of supports increase, the height of the model will decrease (Gohnert and Bradley, 2020c).



Figure 4.3 – Cloth model supported in the corners (Image courtesy of Mitchell Gohnert)

An additional issue with fabric models is the distortions that naturally occur along the edges when draped under gravity. Bradley and Gohnert (2016) refer to these distortions as lips or eyebrows, and these are clearly discernible in the classical *handkerchief* shape formed when a rectangular or square piece of cloth is supported at the corners. Figure 4.3 shows a cloth model with these edge distortions. In distinction to fabric forms, these edge lips are absent in hanging chain models, as shown in Figure 4.4. It is mooted that the formation of the lips is related to the flow of stress; the regions droop because they carry minimal force induced by the gravity loading; i.e., they are not integral to the load path in the shell.

Thermoformed modelling using acrylic (Poly (methyl methacrylate)) plates is an improvement over cloth modelling because the surface is smooth and unimpeded by the weave of the fabric, i.e., the folds present in deeper fabric forms do not occur in acrylic models. However, lips still appear in thermoformed acrylic models. Figures 4.5a and 4.5b show the edge lips in a handkerchief and barrel vault shell models, respectively; a magnified image of an edge lip is shown in Figure 4.6. These models were constructed by placing an acrylic sheet (200 mm x 200 mm x 3 mm), supported by a frame that accommodated free rotation at the supports, in an oven at a temperature of 150°C for about 2.5 hours (Bradley and Gohnert, 2016). When heated, the acrylic material reaches a semi-fluid state and sags to form a tension structure. The material rapidly hardens when removed from the oven, and when flipped upright, the tensile stresses are reversed to compression stresses.

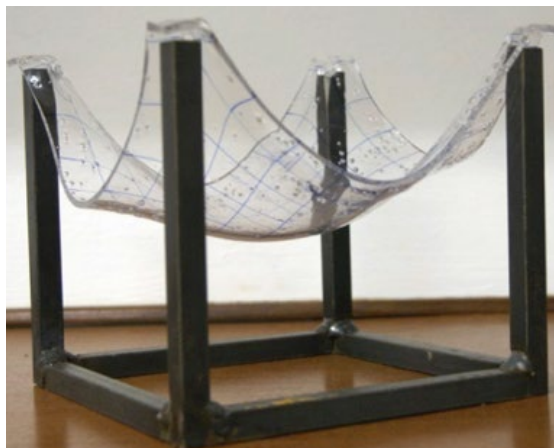


Figure 4.4 – Hanging chain model supported in the corners

By measuring the changes in thickness over the thermoformed acrylic handkerchief shells, Bradley and Gohnert (2016) demonstrated that stresses were not uniform but varied and concentrated toward the supports. The stress magnitude was proportional to the thickness of the shell - reduced thickness resulting from greater tensile stress. The contour plot, shown in Figure 4.7, indicates the change in thickness for one model. The contour colour scale in Figure 4.7 denotes the thickness contraction in millimetres. An important finding derived from this study was that the edge regions (lips) are redundant and only carry a minor proportion of the load. If these regions are cut away, so the edges lean inward, the

resulting shape more closely matches that derived from a hanging chain model, as shown in Figure 4.8.

Although lips/eyebrows are not integral to the structural resistance of the shell, they have nevertheless been incorporated in some building designs; e.g., the shells of Mapungubwe Interpretative Centre. The inclusion of lips may also generate an abrupt change in geometry; i.e., at the inflection between the shell and lip, which may cause stress concentrations and possible cracking damage (Bradley and Gohnert, 2016). Several of the shells which comprise the Mapungubwe Interpretative Centre have cracked along the interface between shell and lip (Bradley and Gohnert, 2016).



(a) Handkerchief model



(a) Catenary vault model

Figure 4.5 – Acrylic models used to determine the flow of stress

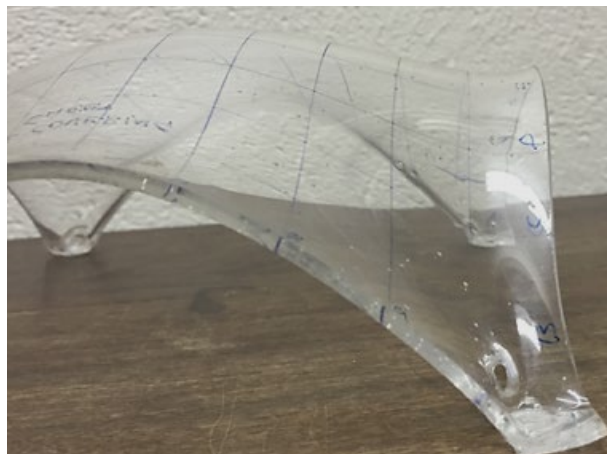


Figure 4.6 – Lip along the edge

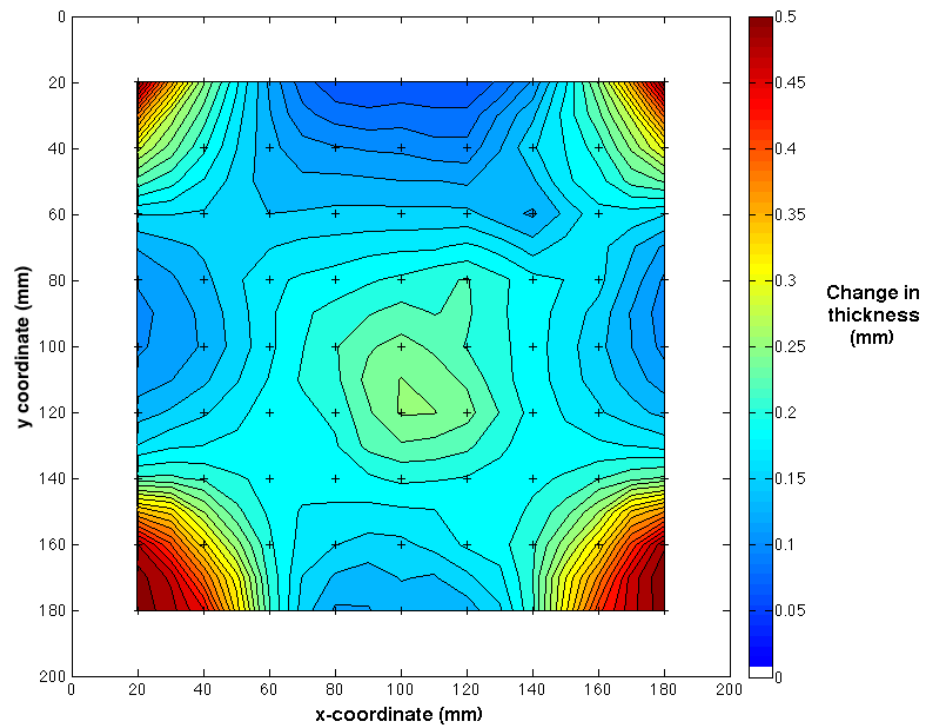


Figure 4.7 – Contour plots of changes in thickness reflecting the stress pattern

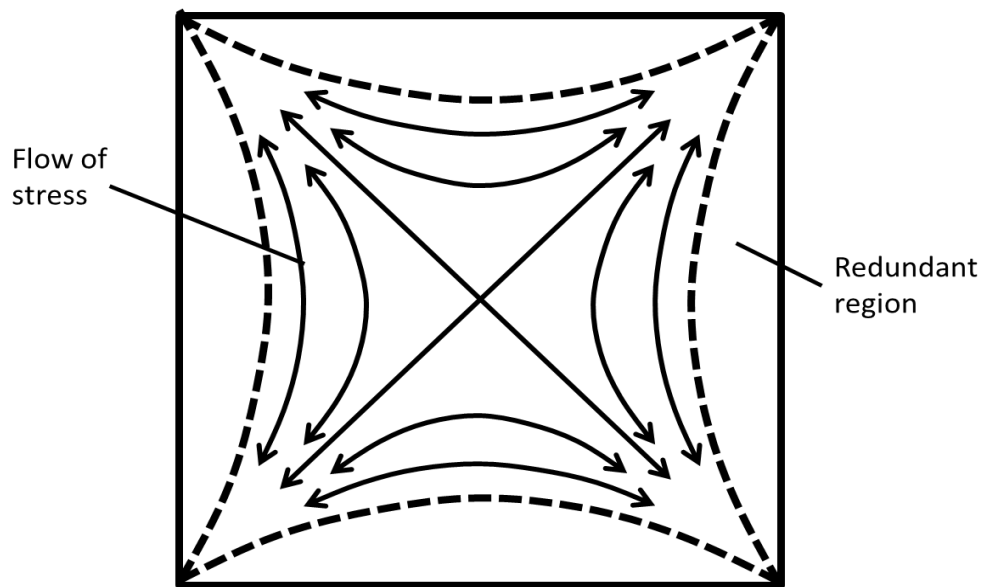


Figure 4.8 – Redundant regions and flow of stress

4.2.4 Analytical and numerical methods

Analytical (shell theory) and numerical (finite element analysis) methods have been used to design CSEB masonry shells; e.g., Gohnert and Magaia (2004) and Gohnert and Talocchino (2011). These methodologies depend on the material characteristics of the masonry, unlike the safe theorems of plasticity, but provide valuable information on the locations where tension may occur in a shell or element. These methods are able to predict zones of tension, e.g., in the vicinity of boundaries and structural discontinuities, allowing suitable measures to avoid cracking to be incorporated in the design. For example, a CSEB dome built in Johannesburg was designed with a thickened base section as well as steel wire stitched around openings to accommodate the tension forces determined from numerical and analytical calculations (Talocchino, 2005; Gohnert *et al.*, 2006).

4.3 Numerical form-finding procedure: moment-step method

4.3.1 Basis and formulation of the proposed methodology

Circular arches, vaults and domes may be architecturally appealing and are relatively easy to construct, but bending is an inherent characteristic of these profiles. From a stress efficiency point of view, a circular shape is not ideal since the axis of the arch does not match the natural flow of stress. In fact, the Auroville Earth Institute (2018) indicate that the minimum thickness of a semi-circular arch under self-weight must exceed one-fifth of its span to ensure the thrust-line remains within the middle-third of the cross section (i.e., for a state of pure compression). The thickness of a semi-circular arch must be varied in order to match the line of thrust with the axis of the member. An example of such a shell is the semi-circular vault designed, using hanging masses, and built at the Auroville Earth Institute, India. The vault's thickness was varied from a maximum of 365 mm at the base to only 70 mm at the apex (Auroville Earth Institute, 2016c). The vault spans 6 m and has a rise and length of 3 m and 11m, respectively. It was built using CSEB, and a combination of horizontal and vertical brick courses were adopted to achieve the variable profile.

To achieve the most efficient structure, in terms of material consumption and mitigation of tension, the thrust-line should match the axis of the member. This is the foremost aim of form-finding procedures, which match the load path (thrust-line) with the axis of the member so that bending moments are avoided. The form-finding procedure described herein begins with the adoption of an arbitrary starting shape (the preliminary form), which

is discretized into segments. Although the starting shape may be of any form, a triangular arch is favoured due to the ease by which the geometry may be defined using straight line functions (i.e., $y = mx + c$). The accuracy of method is dependent on the number of segments (n).

The basis of the method is the minimization of bending moments, iteratively, to an acceptable value. The moment minimization is accomplished through adjustment of the vertical co-ordinate of each discretized element; i.e., points along the axis of the arch, by the appropriate vertical eccentricity of the thrust-line (force-resultant). The new vertical position of the centreline, at point i , is given by Equation 4.4.

$$y_{i(j+1)} = y_{i(j)} + z_{i(j)} \quad (4.4)$$

where $y_{i(j)}$ and $z_{i(j)}$ are the height of the arch's centreline and the vertical eccentricity of the thrust line at point i , corresponding to the j^{th} iteration, respectively. The vertical eccentricity of the thrust-line is expressed mathematically by the division of the bending moment by the horizontal thrust, as given by Equation 4.5.

$$z_{i(j)} = \frac{M_{i(j)}}{H_{i(j)}} \quad (4.5)$$

Through each iteration, the profile is adjusted by adding $z_{i(j)}$ to the y -coordinate. This is followed by the solution of a new bending moment diagram, reaction forces, etc. The procedure is repeated until bending moments become appreciably small. Although the initial span remains constant, the vertical adjustment to each element creates a final shape with an arbitrary height. To resolve this limitation, the apex height of the initial shape is designated as an input variable, which is back calculated to generate the desired output, i.e., a funicular form with chosen height-to-span ratio. The calculations were setup in Excel (Microsoft, Redmond, Washington), and back-solving was done with a subroutine in Visual Basic for Applications (Microsoft).

4.3.2 Arch/vault

An arch under the action of self-weight is chosen to illustrate the proposed methodology. For arches, when only self-weight is considered, the ideal form is an inverted catenary. The catenary shape may be explained by considering a chain, which, when suspended from its

two ends, will hang in pure tension. When this shape is inverted, the meridian stresses will be of pure compression, and the line of thrust will coincide with the centre of the cross section.

In this example, the criteria for convergence was that the stresses at the intrados (inner surface) and extrados (outer surfaces) at any section may differ by no more than 1%. It was also required that the apex height of the final funicular arch be 3m; providing a height-to-span ratio of 0.5. The material and geometric properties of the initial arch were as follows: span = 6 m; thickness = 0.25 m; and unit weight = 19.5 kN/m^3 . The initial apex height of the preliminary triangular arch was assumed. The calculations were setup for pin supports, as shown in Figure 4.9, so that no vertical profile adjustments were made at the base. These support conditions correspond to an arch that is indeterminate by 1 degree. Reaction forces, and internal bending moments, were obtained using the flexibility and unit load methods. The horizontal reaction at the right support, denoted as H_B , was selected as the redundant reaction. Figures 4.10 and 4.11 illustrate the primary and virtual force systems. Equations 4.6 and 4.7 define the primary and virtual horizontal deflections at point 'B'.

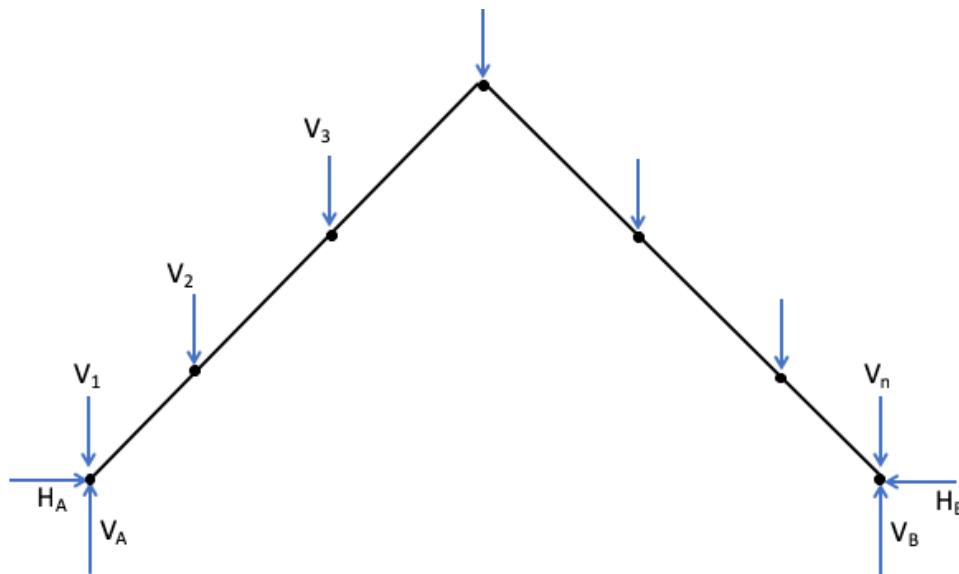


Figure 4.9 – Triangular arch with pin support conditions

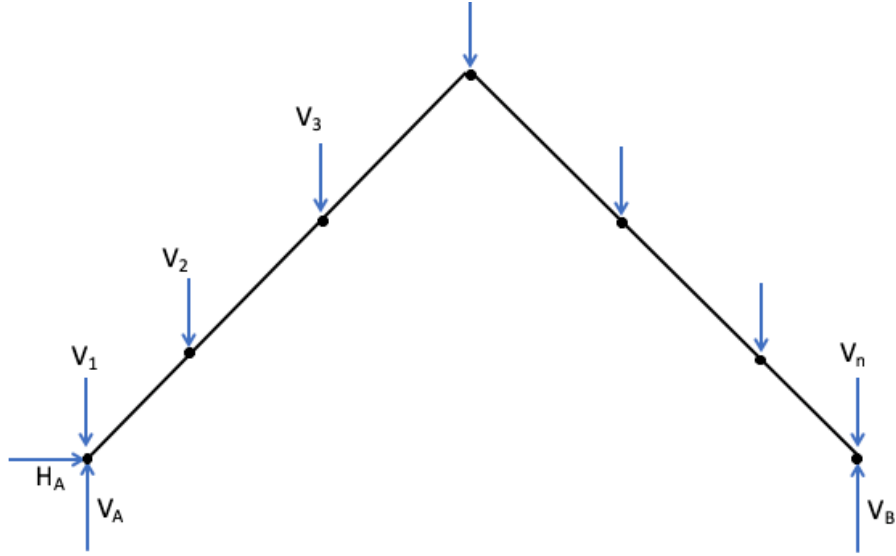


Figure 4.10 – Primary force system (P-system)

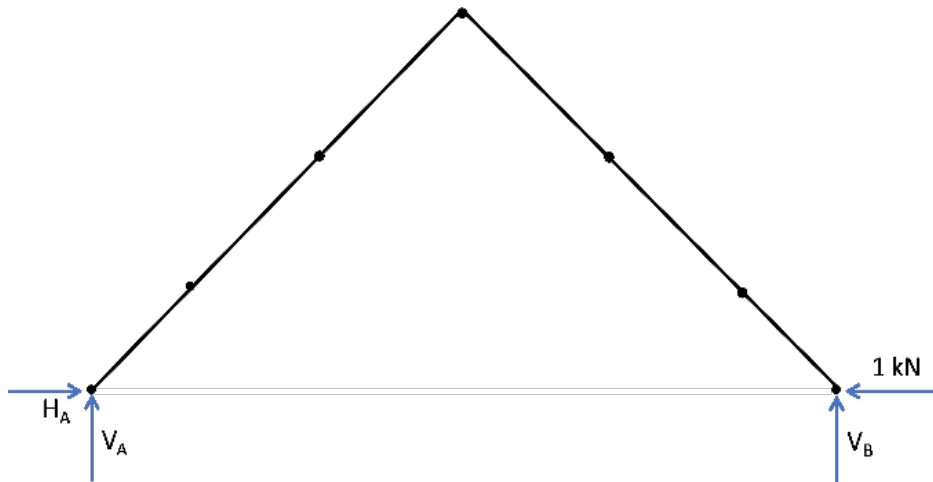


Figure 4.11 – Virtual force system (V-system)

$$\delta_{H(P \text{ system})} = \sum_{i=1}^{i=n} M_{(P)i} \cdot y_{(P)i} \cdot \Delta s_{(P)i} \cdot \frac{1}{EI} \quad (4.6)$$

$$\delta_{H(V \text{ system})} = \sum_{i=1}^{i=n} M_{(V)i} \cdot y_{(V)i} \cdot \Delta s_{(V)i} \cdot \frac{1}{EI} \quad (4.7)$$

where $M_{(P)i}$ and $M_{(V)i}$ are the bending moments at point i , $y_{(P)i}$ and $y_{(V)i}$ are the vertical co-ordinates of the centre of element i , $\Delta s_{(P)i}$ and $\Delta s_{(V)i}$ are the segment lengths of element i , and EI is the bending stiffness.

The aforementioned deflections could have been determined by integrating the bending moment equations corresponding to the P and V systems. Because the method presented in this thesis was setup in Excel, it was convenient to calculate H_B by summing the contributions from each discretized segment. Having calculated the horizontal displacements using Equations 4.6 and 4.7, the equation of condition is given by Equation 4.8.

$$\delta_{H(P \text{ system})} + H_B \cdot \delta_{H(V \text{ system})} = 0 \quad (4.8)$$

Accordingly, the horizontal reaction at the right support is given by Equation 4.9.

$$H_B = \frac{\sum_{i=1}^{i=n} M_{(P)i} \cdot Y_{(P)i} \cdot \Delta s_{(P)i} \cdot \frac{1}{EI}}{\sum_{i=1}^{i=n} M_{(V)i} \cdot Y_{(V)i} \cdot \Delta s_{(V)i} \cdot \frac{1}{EI}} \quad (4.9)$$

This equation may be simplified for a constant section thickness. For this case, the horizontal reaction at point B is given by Equation 4.10.

$$H_B = \frac{\sum_{i=1}^{i=n} M_{(P)i} \cdot Y_{(P)i} \cdot \Delta s_{(P)i}}{\sum_{i=1}^{i=n} M_{(V)i} \cdot Y_{(V)i} \cdot \Delta s_{(V)i}} \quad (4.10)$$

The accuracy of the calculated reaction H_B is directly related the number of elements (n) adopted. Furthermore, the element density effects the *smoothness* of the final funicular form. The moment forces ($M_{(P)i}$ and $M_{(V)i}$) in Equation 4.10 were solved by cutting the arch and applying the equations of equilibrium to the left section. The resulting bending moment equations, for the primary and virtual force systems, are given by equations 4.11 and 4.12, respectively.

$$M_{(P)i} = V_A \cdot x_i + H_A \cdot y_i + \sum_{k=1}^{k=i-1} V_k \cdot (x_i - x_k) + \sum_{k=1}^{k=i-1} H_k \cdot (y_i - y_k) \quad (4.11)$$

$$M_{(V)i} = V_A \cdot x_i + H_A \cdot y_i \quad (4.12)$$

V_k and H_k are the vertical and horizontal applied loads acting on elements of the primary arch, and x_k and y_k are the corresponding moment arms. Figures 4.12a and 4.12b show the bending moment diagrams correspond to the back-calculated starting shape. Once the horizontal reaction is calculated, the remaining support reactions (i.e., V_A , H_A , and V_B) are determined using the equations of equilibrium. The total bending moment at any point

along the arch can then be calculated by summing the contributions from the primary and virtual systems, as given by Equation 4.13.

$$M_i = M_{(P)i} + H_B \cdot M_{(V)i} \quad (4.13)$$

Figure 4.13 shows the output bending moment diagrams corresponding to the back-calculated triangular arch. The arch forms corresponding to these bending moment diagrams are shown in Figure 4.14. Figure 4.15 shows the final stress distributions along the intrados and extrados of the catenary arch. The aforementioned convergence requirement was achieved after 4 iterations. A pure compression profile, having the same span and height, was also derived using graphic statics following the procedures outlined by Allen and Zalewski (2010). The final shape, shown in Figure 4.14, matched exactly that determined using graphic statics (Figure 4.16).

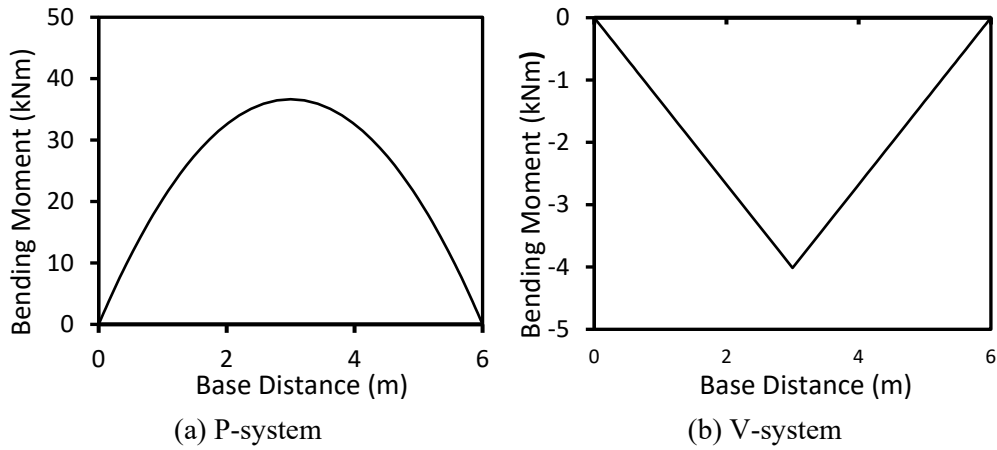


Figure 4.12 – Bending moment diagram for back calculated starting shape

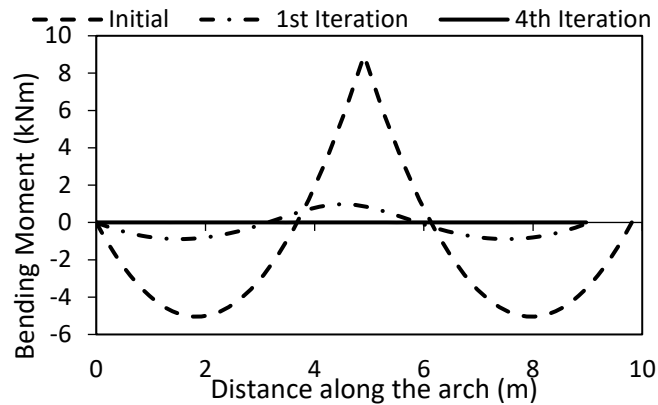


Figure 4.13 – Bending moment diagrams: arch

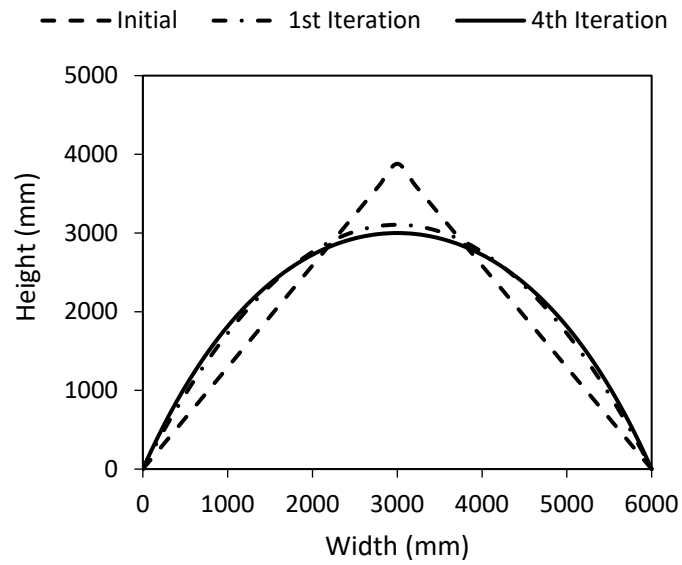


Figure 4.14 – Arch shapes

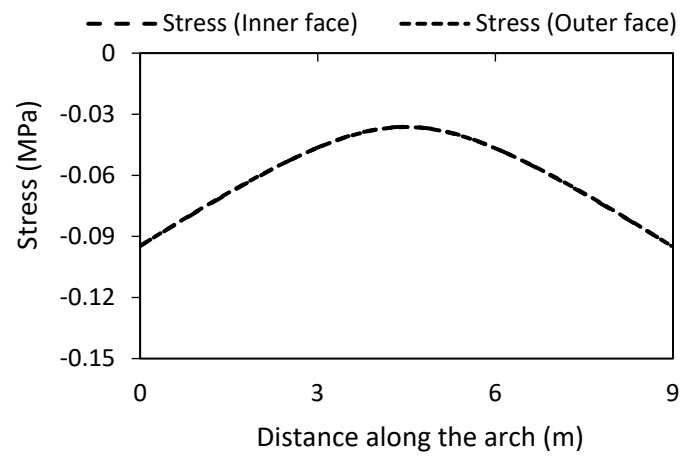
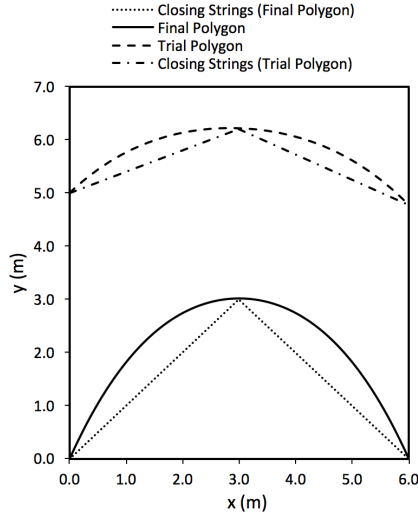
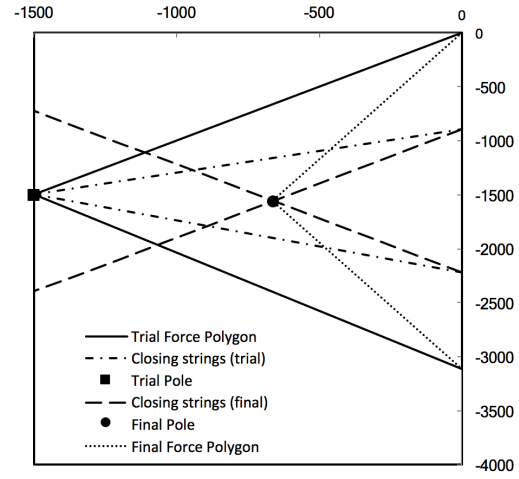


Figure 4.15 – Stresses along the intrados and extrados of the funicular arch: 4th iteration



(a) Arch shapes

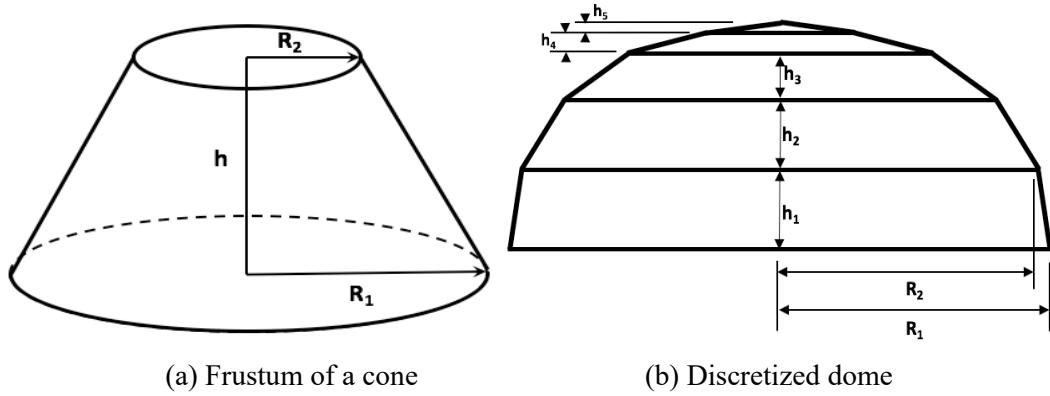


(b) Force polygons

Figure 4.16 – Results using iterative graphic statics

4.3.3 Dome

In this section, the funicular form for a dome is solved with the proposed methodology. The solution is compared with that obtained using iterative thrust-line analysis (Minke, 2009). The problem was solved by discretizing the dome into a series of frusta, as shown in Figures 4.17a and 4.17b); the three-dimensional preliminary shape being a cone. This setup is ‘analogous’ to the triangular shape adopted for the two-dimensional arch.



(a) Frustum of a cone

(b) Discretized dome

Figure 4.17 – Frustum segments

The surface area of a frustum segment i is given by Equation 4.14.

$$A_i = \pi(R_i + R_{i+1})\sqrt{(R_i - R_{i+1})^2 + h_i^2} \quad (4.14)$$

where R_i and R_{i+1} are the radii of the lower and upper bases, respectively, and h_i is the height of the frustum. For the purposes of the present comparison, it was assumed that all segments have the same density (p) and thickness (t). Instead of determining the actual weight of each segment, the problem was setup by assigning a self-weight of 1 kN to the base segment and then relating all other segment forces to it through a surface area ratio as given by equation 4.15.

$$P_i = 1\text{kN} \left(\frac{A_i}{A_1} \right) \quad (4.15)$$

where A_1 is the surface area of the base segment. By substituting equations for A_1 and A_i into Equation 4.15, the load for segment i is given by Equation 4.16 and shown in Figure 4.18.

$$P_i = \frac{(R_i + R_{i+1}) \sqrt{(R_i - R_{i+1})^2 + h_i^2}}{(R_1 + R_2) \sqrt{(R_1 - R_2)^2 + h_1^2}} \quad (4.16)$$

The simplified approach, Equation 4.16, is valid because the shell thickness and density are constant. The element forces are then applied to a two-dimensional line model of the dome and are solved in the same manner as aforementioned for the arch/vault example. The bending moment diagrams, corresponding to this line model, are shown in Figure 4.19a. Figure 4.19b illustrates the corresponding arch shapes, and, as before, the desired solution, i.e., $H/S = 0.5$ and base diameter = 6 m, was obtained after four iterations. The shape corresponding to the 4th iteration was compared with that from Minke (2009), and the shapes derived from both methodologies are practically identical (Figure 4.20).

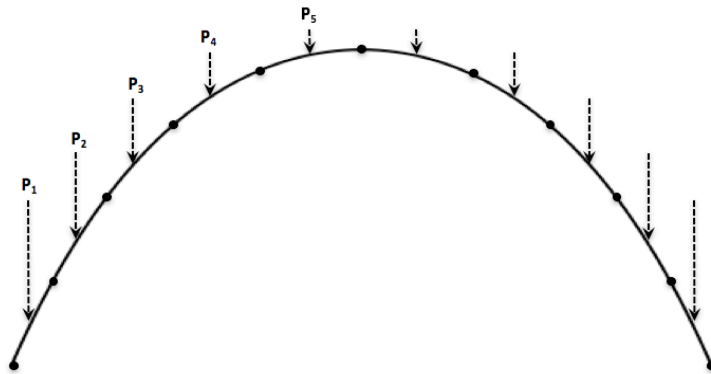


Figure 4.18 – Varying load magnitude along the profile

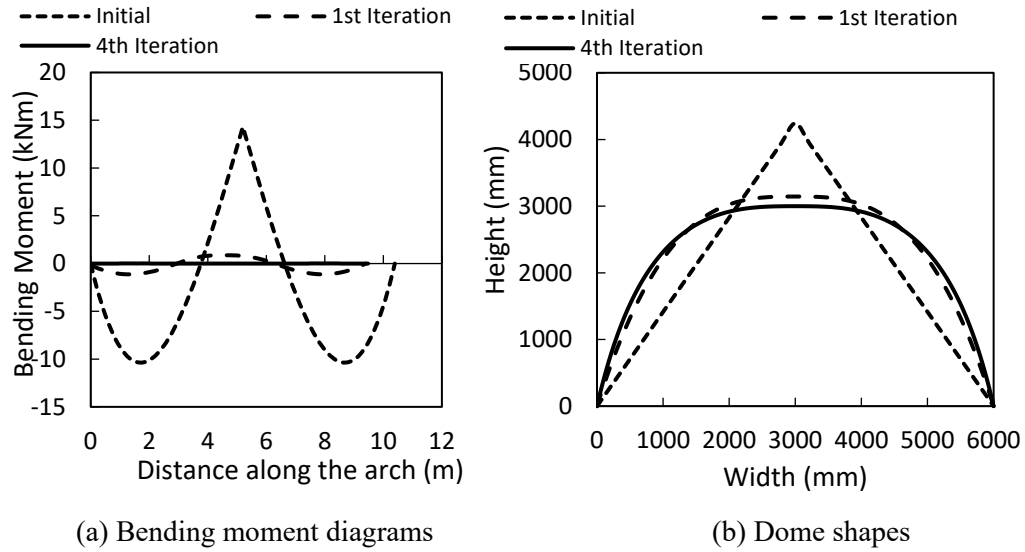


Figure 4.19 – Bending moment diagrams: dome

Although the method matches the form derived by Minke (2009), it must be highlighted that the moment-step method presented herein is a two-dimensional analysis: the method does not take into account hoop effects in shells of non-zero Gaussian curvature (i.e., doubly-curved surfaces). Further to this, because of the hoop effect, an infinite number of mid-surfaces could satisfy the condition of zero bending moment under gravity load, unlike the unique solution of a catenary shape for an arch under self-weight.

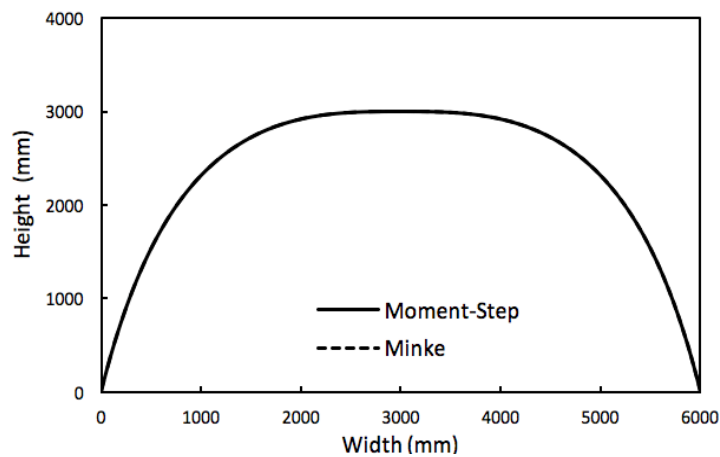


Figure 4.20 – Funicular dome shapes: Moment-step and Minke

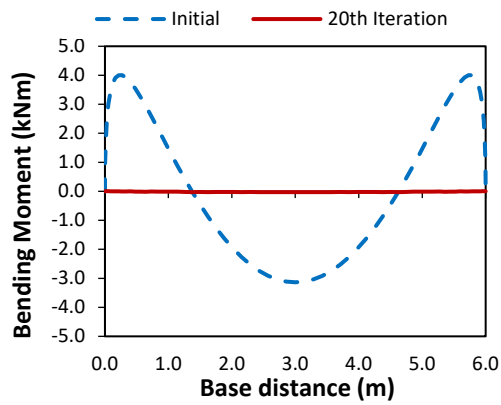
4.3.4 Complex three-dimensional forms

The applicability of the proposed moment-step method to form-finding of three-dimensional shells was further examined using shell elements in a finite element package. Finite element modelling was carried out using LISA FEA v8.0.0. An assembly of straight edged elements was implemented to represent the geometry and internal force distributions for the doubly-curved dome structures. This physical approximation was considered through an appropriate choice of element and adequate mesh refinement (element subdivision) to ensure convergence.

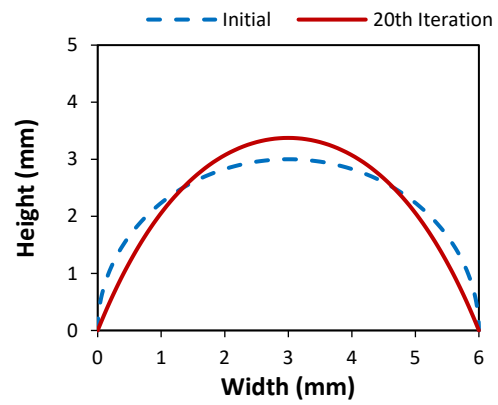
A single-curved vault was initially checked by adjusting the geometric profile based on the bending moments along the centreline, from left to right base. A semi-circular vault was chosen as the starting shape, and no constraint was set on the final height of the shell. Material and geometric parameters are shown in Table 4.1. After several iterations, a catenary profile was obtained, and the results are presented in Figure 4.21. However, there were some problems when doubly-curved shells were modelled using shell elements. Unlike the singly curved vault, solutions would diverge due to large moment inconsistencies at nodes caused by the distortion of shell elements (i.e., skewed elements with large aspect ratios) during the iterative procedure. A path for future study may be the adoption of bar elements to represent the geometry of the shell with nodal loads to account for self-weight and applied actions.

Table 4.1 - Geometric and material properties

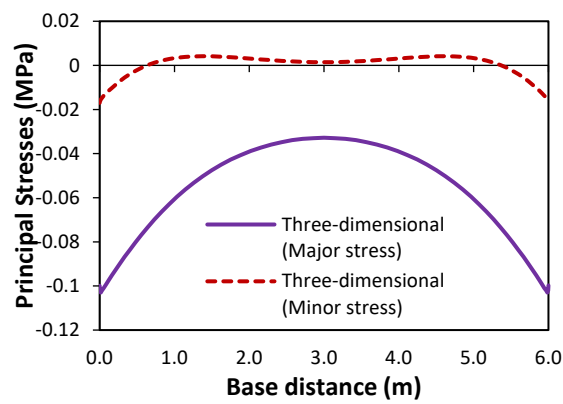
Thickness	Unit weight	Young's Modulus	Poisson's Ratio	Span	Height
0.1 m	19.5 KN/m ³	3500 MPa	0.15	6 m	3 m



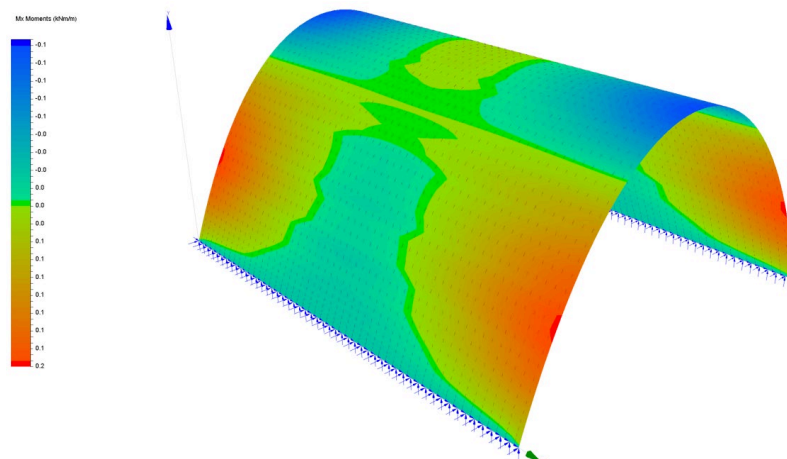
(a) Bending moment diagrams



(b) Vault geometry



(c) Longitudinal and lateral stress distributions along the vault's centreline



(d) Major principal moment

Figure 4.21– Form-finding results using shell elements

4.4 Application

4.4.1 Varying profile thickness

Varying the thickness of an arch, or dome, may be appealing to designers when a specific pure-compression profile is desired. One such example is the pure-compression vault built at the Auroville Earth Institute, which was designed with variable thickness to achieve a semi-circular shape. The form was determined by hanging variable weights from a string to mimic the mass of masonry segments. Another architectural aspect may be gained by varying the thickness, from a maximum at the base to a minimum at the apex, to obtain more usable floor space. This is particularly appealing for low-cost residential applications, where the amount of floor space may be constrained by the limited availability of funds.

Figures 4.22 and 4.23 show two arches, with variable thickness, which were obtained using the proposed method, i.e., the moment-step method. The centreline of the first arch, shown in Figure 4.22, closely matches that of a semi-circular shape. The dashed lines correspond to a uniform catenary arch of equivalent mass/material. Figure 4.23 illustrates how additional usable space can be gained by adopting a steeper profile in combination with varying thickness. Although the arch/vault profiles shown in Figures 4.22 and 4.23 are in compression under self-weight, they would need to be assessed for non-uniform loading (i.e., asymmetric distributed loads and wind loading). Lateral loads become particularly significant when profiles are steep (Bradley *et al.*, 2017a), and such shells are certainly not suitable in areas prone to seismicity. Additionally, the adoption of ultra-thin sections, such as those at the apex of the vaults shown in Figures 4.22 and 4.23, may be susceptible to punching damage (Bradley *et al.*, 2018a), and unfavourable solar heat gain when uninsulated (Bradley, 2020).

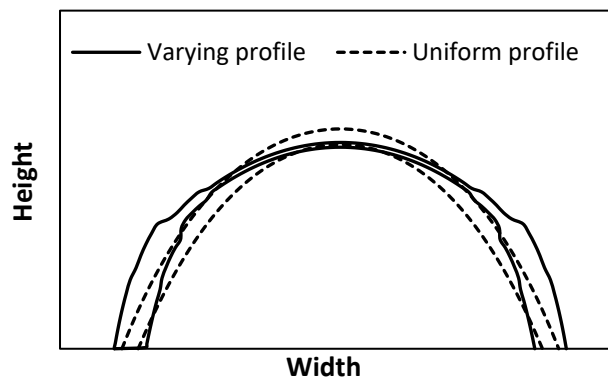


Figure 4.22 – Equivalent height to span ratio

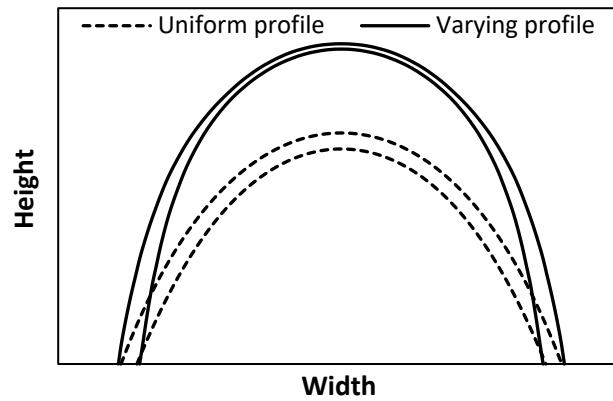
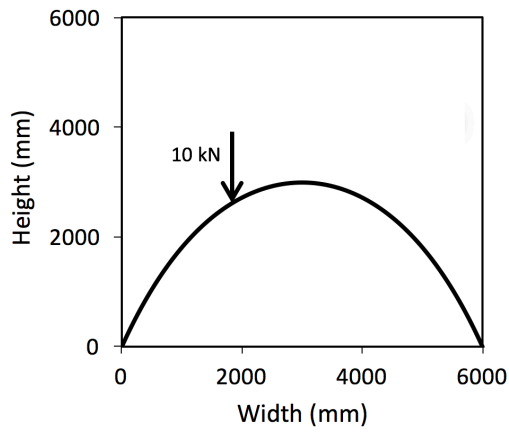


Figure 4.23 – Steep arch with varying profile

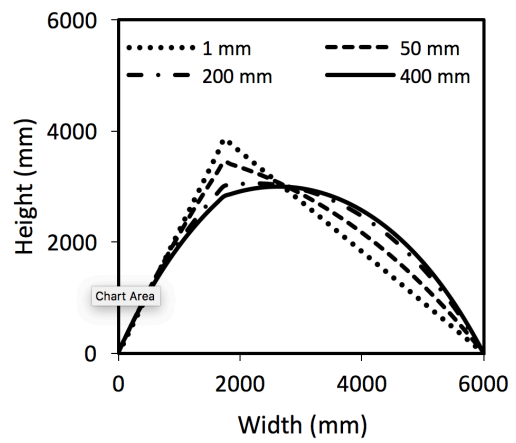
4.4.2 Non-uniform loads

The numerical procedure presented in this thesis offers simplicity and efficiency in the formulation of pure-compression shells such as arches, vaults and domes. It is particularly effective at optimizing the shape and thickness under the action of self-weight, which is typically the dominant load case. Thrust-lines corresponding to non-uniform loading (e.g., distributed and wind) may subsequently be checked, to ensure they remain within the middle-third of the cross-section. For example, the thickness of the shell must be increased until a thrust-lines for each imposed load, or load case, is confined within the middle third. Such an analysis could be done with several simple techniques, such as: graphic statics, thrust-line analysis, or the segmental equilibrium method (Gohnert and Bradley, 2020b).

The moment-step method can also be used to find the funicular shape of an arch under non-uniform loading. This point is illustrated by considering a catenary arch, with various thicknesses, subjected to self-weight (assumed at a value of 19 kN/m^3) and either a vertical or horizontal point load of magnitude 10 kN . The results from these analyses are shown in Figures 4.24 and 4.25. No limiting height-to-span ratio was imposed in these analyses, which allowed the funicular shapes to form ‘freely’. A feasible funicular shape was obtained for all vault thicknesses. Although, some of the shapes are impractical in terms of functionality and constructability. Other structural aspects could become critical, e.g., buckling instability, second-order effects, and increased influence of geometrical imperfections and they need to be checked. Nevertheless, if these shapes mimic those that could be produced with a hanging chain, as shown in Figure 4.26.

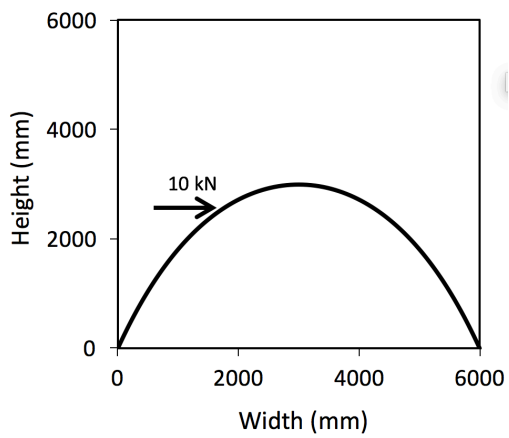


(a) Load application

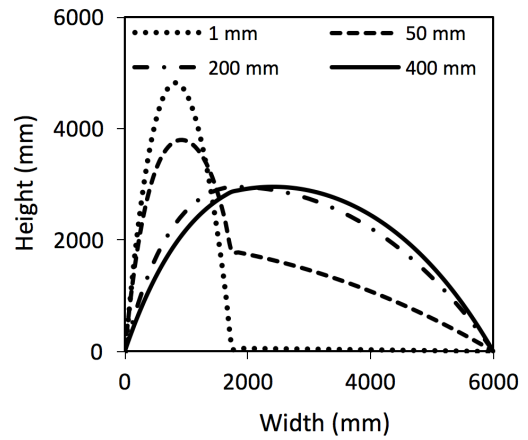


(b) Funicular shapes

Figure 4.24 – Vertical load



(a) Load application



(b) Funicular shapes

Figure 4.25 – Horizontal load

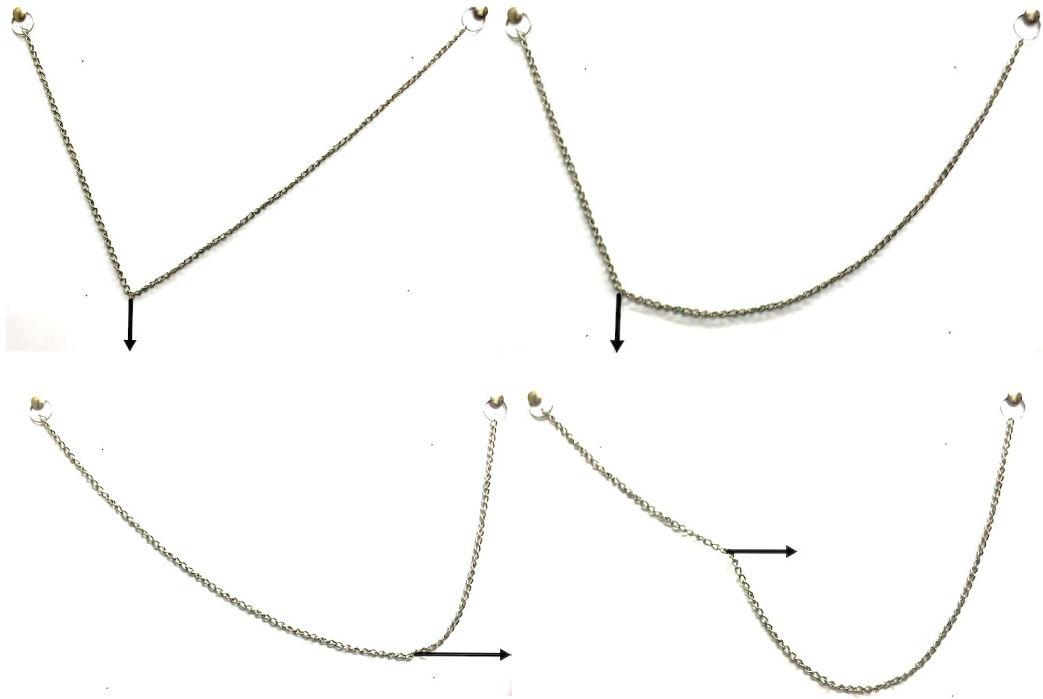


Figure 4.26 – Hanging chain models

4.5 Conclusions

This chapter presented a description of a new methodology, the moment-step method, to obtain funicular forms of shell structures. The method was implemented to find pure compression forms for an arch/vault and dome under the action of self-weight. These forms were then compared with those obtained using thrust-line analysis techniques. The profile of the arch/vault matched that of inverted catenary. The shape of the dome was practically identical to that presented by Minke (2009). The method was also employed to determine two pure-compression arch profiles of varying thickness. These shapes were chosen to either generate a specific profile and/or obtain more usable space. Design implications associated with these profiles were also discussed, with emphasis to smaller shells appropriate to low-cost housing.

CHAPTER 5. LOADING CONSIDERATIONS FOR CATENARY EARTH SHELLS: ENVIRONMENTAL AND IMPOSED ACTIONS

5.1 Introduction

Catenary structures are structurally efficient alternatives to traditional circular domes and vaults. In particular, singly curved unreinforced catenary thin-shells have been used in several developing countries for housing and recreational buildings. Houben and Guillaud (1994) noted that stabilised earth brick vaults have been built with spans up to 6 m. Even larger unreinforced vaults, spanning up to 10 m, have been built at the Auroville Earth Institute in India. The fundamental advantage of catenary shells is that they are subjected to compressive stress only (i.e., neither bending, shear, nor tensile forces) under the action of self-weight alone. Furthermore, materials and construction costs associated with these structures may be less expensive than typical masonry and reinforced concrete structures of a similar nature. Further cost savings may be realised by adopting steeper shells, which would replace a standard roof and wall.

Although earthen masonry is strong in compression, it is very weak in tension and flexure. The flexural strength is influenced significantly by the amount of adhesion between mortar and brick; consequently, earthen masonry shells crack easily when exposed to relatively small tensile forces. There are also durability problems that must be addressed during design. These issues highlight the importance considering non-uniform loading (in regard to both magnitude and distribution) during design and construction of earthen masonry shells. The engineer/designer of unreinforced earthen shells must ensure that the structure will remain in compression for all anticipatable loading conditions. If the thrust line is not maintained within the middle third of the cross section, tension will develop, which may ultimately lead to cracking.

In this chapter the critical actions (i.e., applied loading), and situations, that may cause tension, cracking or even the collapse, of free-standing catenary shells are presented. The appropriate recognition of these loads is integral to the appropriate design thickness, shape, and finishes of the shell. Many of the conclusions presented in this chapter are made in reference to the catenary form; however, some of the loading considerations should be considered in the design of unreinforced earth shells in general.

5.2 Basis for adopting steeper singly-curved shells

5.2.1 Architectural

The layout of housing structures is critical to ensure that all of the basic needs of the occupants are met. In particular, the usability of each room and its relation to the remaining indoor spaces is important when attempting to create an ordinary living environment (i.e., so that homeowners can enjoy the functional definition versus flexibility within the living spaces). However, there is limited flexibility in this regard when designing low-cost structures because of the financial and spatial limitations (i.e., any wastage of space can become an acute problem of the design). In terms of shell structures, the wastage of space can be caused by some geometrical configurations/shapes, i.e., by creating volumes of unusable space close to the base where furniture will typically be situated. The adoption of a steeper catenary profile, such as those constructed at the University of the Witwatersrand (Wits), can minimize this wasted space. For example, the Wits shells can easily accommodate low furniture, such as small cabinets, desks, and chairs, with only negligible dead space behind. Furthermore, these vaults enable placement of higher furniture items and appliances, such as cupboards, more efficiently than some other shell forms. This is dissimilar to conventional catenary vaults, where the rise (H) is usually 50 cm plus half the span (denoted as R in this thesis) (Houben and Guillaud, 1994). Accordingly, these vaults are constructed on vertical walls to allow for reasonable amounts of usable space below, but they require elements such as buttresses, ring beams and ties to accommodate the resultant lateral thrusts.

Another positive architectural aspect of steep catenary vaults is that they create a sense of space, even though the actual floor area may be relatively small. This is particularly apparent when the gable walls include large windows that enable natural light to enter the house.

5.2.2 Ease of construction

The level of geometrical complexity adopted for low-cost housing shells will typically be governed by structural efficiency, internal space created, liveability and/or construction complexity rather than aesthetics. The construction of geometrically complicated unreinforced shells is by no means a trivial activity; in particular, poor construction practice can be hazardous during and after building activity. Typically, simpler shapes such as

vaults, domes and singly-curved cross-vaults are preferred in low-cost applications. The singly curved vault, in particular, appears to be a favoured form in developing areas such as India, the Middle-East and parts of Africa (Adam and Agib, 2001; Norton, 1997; Habitat, 1992; Auroville Earth Institute, 2016; Fathy, 1973). Steeper profiles can be built with lighter formwork because the self-weight of the masonry is largely transferred within the section itself (Bradley *et al.*, 2017a; and Gohnert *et al.*, 2018).

5.2.3 Structural

5.2.3.1 General

The extent of structural damage depends on the risk associated with the hazards/loads and the resistance (i.e., vulnerability) of structures. Goliger (2002) noted that damage potential is, broadly speaking, the product of risk and vulnerability. The resistance of CSEB masonry can be enhanced by adopting thick sections and/or by incorporating reinforcing mesh or bars. These mechanisms are largely employed in areas exposed to seismicity (Holliday *et al.*, 2016) and wind speeds associated with extreme events (Matta *et al.*, 2015; Holliday *et al.*, 2016). The risk associated extreme environmental loads is largely dictated by climate and location but can, in some cases, be mitigated through architectural actions, e.g., adopting steeper roofs to shed snow.

5.2.3.2 Reduced lateral base thrusts

The structural failure of earthen masonry shells is primarily attributed to excessive support movement, and several collapses are reported in the literature (Habitat, 1992). The source of support movement is typically associated with deficient or non-existent design of lateral force resisting elements (i.e., buttresses, ring-beams and ties). Theunynck (1994), De La Rosa (2013) and Habitat (1992) provide an evaluation of the causes of damage to earth masonry vaults and domes. Theunynck (1994) attributed the collapse of two dome roofs in the Sahel, in 1988, to missing lateral force resisting elements, low quality plaster, and poorly bonded brickwork. Habitat (1992) reported that structural problems in several shells built in the Sahel were due to missing foundations, inadequate wall thickness, incorrect bonding of blocks, too rapid construction, and poor drainage of rainwater. More recently, two adobe vaults, which make up part of the Swan House & Lab (Texas), collapsed due to excessive material deterioration at cracks, which appear to have been caused by lateral support movement (De la Rosa, 2013).

By adopting steeper vaults, instead of shallower profiles atop vertical walls, the lateral base thrust can be greatly reduced. This is illustrated with a comparison of base reactions for several catenary vaults of varying height. The span (6m), thickness (0.2 m) and unit weight (19.5 kN/m^3) were kept constant for the analysis. Figure 5.1a and 5.1b show the axial compressive stress and base reactions for several profiles of catenary vault. These results indicate that using a steeper profile generates larger vertical reactions and compressive stresses toward the base, which should generally not be a problem considering their low magnitude. Obviously problem soils, e.g., compressible or collapsible, or those with low bearing capacity, may require remedial action (i.e., densification or soil compaction) and/or appropriate selection/sizing of foundations. Although there is increased vertical load, the horizontal base reaction reduces in magnitude; potentially decreasing costs associated with lateral forces resisting elements.

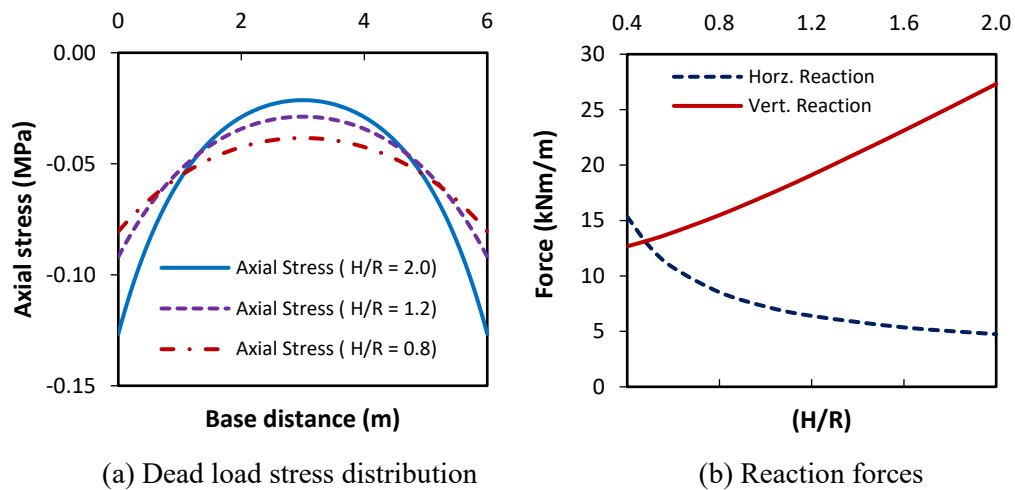


Figure 5.1 – Stress and base reactions: catenary vaults

5.2.3.3 Low Seismic Risk in South Africa

Unreinforced masonry arches and vaults are especially vulnerable to seismic loading. The brittle nature of earthen materials is particularly concerning because buildings may collapse without warning under load (i.e., the large inertial forces caused by an earthquake). Nonetheless, earthen masonry shells have been constructed in areas prone to moderate earthquakes. According to Houben and Guillaud (1994), the rise of singly-curved catenary vaults should be restricted to 20 to 30% of the span, i.e., H/R between 0.4 and 0.6, in earthquake areas. These shallower vaults are typically limited to roofing, replacing traditional wooden trusses and metal sheeting. Steeper shells are particularly susceptible to

seismic loading; and hence, their use is naturally avoided in these regions. According to Norton (1997), special precautions are needed for earthen shell structures, such as strengthening of walls and ring beams and ties at the base. Furthermore, failure of supporting walls in earthquake could lead to roof collapse. Although this also applies to traditional roofing, the hazards presented by a relatively heavy earthen masonry roof must be recognised.

In an earthquake event, the resultant inertial forces will cause the internal load path (i.e., the thrust line) to shift which, in turn, could cause tension in the masonry. Excessive loading could also generate a sufficient number of hinges to trigger collapse. For unreinforced masonry arches and vaults, two rocking-type modes of failure are addressed in the literature. Mode 1 is defined by failure that occurs after multiple impacts (i.e., rocking), whereas mode 2 is defined by the non-impact failure caused by a base acceleration of a sufficient magnitude and duration (Zhang and Makris, 2001). More complex means of analysis are required for modelling mode 1 failure (Oppenheim, 1992; DeJong, 2009). Herein, only mode 2 failure is considered, since it provides a simple method to assess the relationship between the height-to-span ratios of the vaults and the critical lateral acceleration.

The principle of virtual work was adopted in the current investigation to calculate the minimum lateral component of gravitational acceleration ($\lambda_{\min}$) required to cause collapse of an unreinforced masonry vault. This methodology was implemented by Ochsendorf (2002) to analyse circular arches. The calculation is iterative because all possible hinge combinations must be considered in order to identify the critical lateral acceleration. In this study, computation was setup in Microsoft Excel, and the critical hinge locations were solved using subroutines in Visual Basic for Applications (VBA). These subroutines were adopted from Surat (2016). The inertial lateral loads and vertical gravity loads acted at the centroid of each bar. The arch / vault was assessed as a three-bar (or 4-hinge) mechanism, as shown in Figure 5.2. The three-bar arch mechanism is a single degree of freedom system; therefore, if a virtual displacement is applied to one bar, the relative rotations of the other bars may be calculated. Using the calculated rotations, the virtual horizontal and vertical displacement of all forces may be calculated. This method assesses the arch at the stage of collapse, where each bar in the mechanism will undergo rigid-body motion. For a more

detailed discussion of the methodology, the reader is referred to Ochsendorf (2002). The work equation is given by Equation 5.1.

$$\sum_{i=1}^n W_i \cdot d_{vi} + \lambda \cdot \sum_{i=1}^n W_i \cdot d_{hi} = 0 \quad (5.1)$$

where: W_i = the weight of the respective bars; d_{hi} and d_{vi} are the horizontal and vertical displacements of the respective forces. The formula may be rearranged to solve for λ as given by Equation 5.2.

$$\lambda = \frac{-\sum_{i=1}^n W_i \cdot d_{vi}}{\sum_{i=1}^n W_i \cdot d_{hi}} \quad (5.2)$$

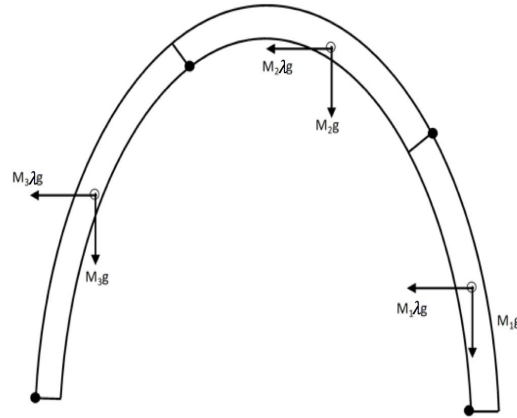


Figure 5.2 – Gravitational and inertial loads for the three-bar system

For the purposes of these analyses, the span of the arch/vault was set at 6m and the thickness was varied between 0.15m and 0.35m. The relationship between the critical lateral acceleration (λ_{min}), as a function of the height, to half span ratios (H/R) is shown in Figure 5.3a. The graph indicates that λ_{min} is initially very sensitive to changes in the height to width ratio; however, for steeper profiles, λ_{min} converges towards zero. This is coupled with the internal hinges, as shown in Figure 5.3b, moving toward the apex as the height of the vault increases. The plot shown in Figure 5.3 demonstrates the suitability of lower rise catenary profiles over steeper profiles, from a seismic perspective. Furthermore, the trends shown in the plot indicate the design limitations suggested by Houben and Guillaud (1994) with respect to the rise of earthen catenary vaults built in seismic areas.

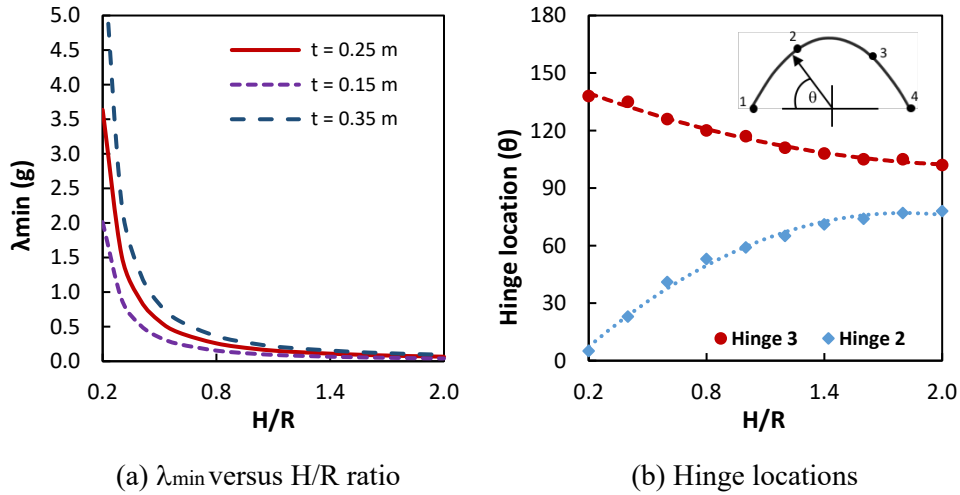


Figure 5.3 – Four-hinge analysis of catenary vaults under base motion

Although all locations are exposed to some level of seismicity; many areas, including most of South Africa, exhibit very low seismic risk, which is favourable for the construction of steeper unreinforced earthen shells for low-cost housing. According to Brandt (2011), the seismicity of Southern Africa is, by world standards, very moderate and of shallow character. Considering the peak ground acceleration (PGA) map, with a 10% probability of being exceeded at least once in a period of 50 years, for South Africa (as presented by Fernandez and Du Plessis, 1991), the majority of the country falls below a PGA of 0.05g (i.e., very low seismic risk).

Due to extremely low seismicity throughout much of South Africa, it is wind which comprises the principal lateral action for low-rise buildings and housing structures. For example, the sizing and proportioning of housing systems submitted for Agreement South Africa certification is based on wind loading - either simulated full-scale testing or rational design following the relevant South African National Standards. In particular, lateral wind loading on walls and uplift on roofing elements are considered, and this is consistent with the usual types of collapse induced by extreme environmental events in South Africa (Mahachi, *et al.*, 2018; Goliger, 2002).

5.2.3.4 Reduced vertical distributed imposed loads

Vertical distributed loads over roofs arise from several sources, including occupancy; presence of workmen or stacked materials (during maintenance, repair, and construction); and the accumulation of snow, hail, or rainwater. Since the roof of unreinforced earth-shell

housing structures are designed as inaccessible, the critical vertical loading would either be generated during construction/maintenance operations or by environmental loads (i.e., snow, hail, and rain). These loading actions, as stipulated in several loading codes, are considered to evaluate their significance in relation to the design of steep catenary shells.

Imposed loads: inaccessible/ordinary roofs

In the South African and European loading codes [i.e. SANS 10160 (SABS, 2011) and EN1991-1-1 (CEN, 2002a)], imposed loads on roofs are categorized according to their accessibility (i.e., being either accessible or inaccessible). Live loading on inaccessible roofs, as specified in SANS 10160-2, would account for the following situations:

- maintenance or construction loads intended to represent the effects of workmen or stacked materials, etc., and
- the limited accumulation of snow (<250 mm), hail (<60 mm), or rainwater (<50 mm) on roofs.

These roof loads are specified for horizontal areas $>3 \text{ m}^2$ and $<15 \text{ m}^2$, with linear interpretation implemented in between. For an area that is larger than 15 m^2 , an imposed load (q_k) of 0.25 kN/m^2 is stipulated for maintenance and construction. Loading codes, such as EN1991-1-1 (CEN, 2002a) and ASCE/SEI 7-10 (ASCE, 2010), also consider this type of load along the horizontal projection of the roof. Imposed loading, which is not occupancy related, over ordinary roofs are specified as 0.96 kN/m^2 in ASCE/SEI 7-10 (ASCE, 2010). This uniform load reflects the action of workers, equipment, and materials applied over the roof during maintenance, as well as miscellaneous moveable objects located over the roof in its lifetime. Ordinary flat, pitched, and curved roofs are permitted to be designed for a reduced roof live load, as specified in Equation 5.3.

$$L_r = L_o R_1 R_2 \quad (5.3)$$

where $0.58 \text{ kN/m}^2 \leq L_r \leq 0.96 \text{ kN/m}^2$; L_o is the minimum uniformly distributed load; and R_1 and R_2 are reduction terms: one for the tributary area (R_1) and the other for the slope of the roof surface (R_2). R_1 and R_2 are determined using Equations 5.4 and 5.5, respectively.

$$R_1 = \begin{cases} 1, & \text{for } A_t \leq 18.58 \text{ m}^2 \\ 1.2 - 0.011A_t, & \text{for } 18.58 \text{ m}^2 < A_t < 55.74 \text{ m}^2 \\ 0.6, & \text{for } A_t \geq 55.74 \text{ m}^2 \end{cases} \quad (5.4)$$

where A_t is the tributary area.

$$R_2 = \begin{cases} 1, & \text{for } F \leq 4 \\ 1.2 - 0.05F, & \text{for } 4 < F < 12 \\ 0.6, & \text{for } F \geq 12 \end{cases} \quad (5.5)$$

where F is the rise-to-span ratio multiplied by 32 for a vault or dome. For all the catenary roof shapes considered in this thesis, the value of F exceeds 12; consequently, the minimum reduced uniform load of 0.58 kN/m^2 is appropriate.

EN1991-1-1 recommends an imposed surface load (q_k) over roofs that are not accessible, except for routine maintenance and repair, of 0.4 kN/m^2 . However, q_k may be varied in National Annex, subject to the roof slope/geometry. The British loading code [BS EN 1991-1-1 (BSI, 2002)] is one such standard that considers the load variation over the roof. The magnitude of the imposed load is 0.6 kN/m^2 for roof slopes below 30° and 0 kN/m^2 for roof slopes greater than 60° , whereas the load corresponding to intermediate slopes is given by Equation 5.6.

$$q_k = 0.6 \left(\frac{60-\alpha}{30} \right) \text{ kN/m}^2 \quad (5.6)$$

The BS EN 1991-1-1 (BSI, 2002) standard stipulates that, for curved surfaces, the roof should be subdivided into not less than five equal segments and the mean slope of each segment considered to be equivalent to the roof slope (a). Figure 5.4a shows the imposed loads for several profiles of a catenary vault with $0.8 \leq H/R \leq 2.0$, corresponding to routine construction and maintenance, derived from the stipulations of EN1991-1-1 (CEN, 2002a). The distribution was formulated with 12 segments of equal base length of 0.5 m . Vaults that span 6 m , and with a length of 8 m , were adopted for the purposes of this analysis. By integrating the surface load distributions obtained in Figure 5.4a, the total load (in kN) may be derived. The steepest catenary vault ($H/R = 2.0$) carries approximately 40% of the load over a flat roof covering the same plan area, as shown in Figure 5.4b.

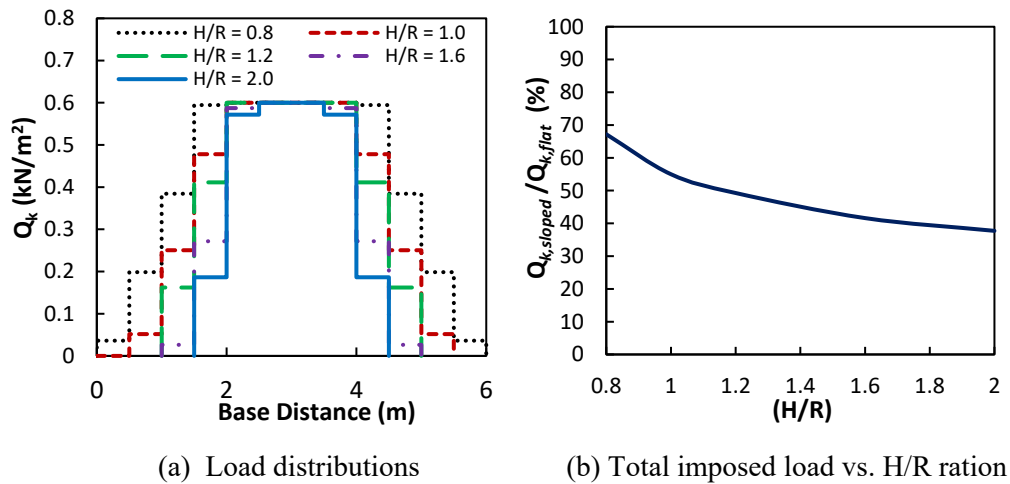


Figure 5.4 – Imposed loading: inaccessible catenary roofs (following BS EN 1991-1-1)

Figure 5.5 shows the variation of the total imposed load (in kN) acting over catenary roofs, adopting the aforementioned codes, with aspect ratio (H/R). ASCE/SEI 7-10 and SANS 10160-1: 2011 correspond, in the most part, to the highest and lowest loads, respectively. The lowest total load for the H/R = 2.0 vault is that specified by BS EN 1991-1-1, which is the only standard that considers the effect of the roof slope on the magnitude and distribution of vertical imposed loading.

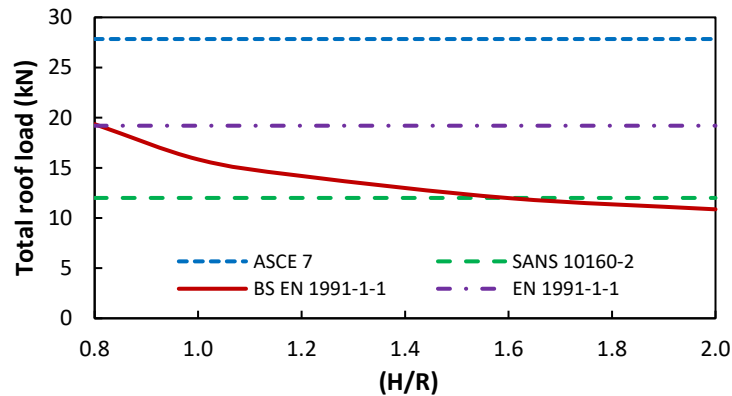


Figure 5.5 – Comparison of imposed loading derived from codes of practice

The specifications given in BS EN 1991-1-1 best reflects the actual types of loading that will be generated over curved roofs and structures, i.e., that loads associated with maintenance would realistically be localized to flatter regions of small to medium sized shells, where building materials could be stacked. For instance, if the stipulations presented BS EN 1991-1-1, for distributed roof loads, are adopted for the design of a shallow vaulted

roof, a hemi-spherical dome, and a steep catenary vault, with $H/R = 2.0$, then the loaded areas would correspond to 100%, approximately 75% and 50% of the plan area under the respective roof. The locations of these zones are illustrated graphically in Figure 5.6, which have been observed through actual maintenance work done on a dome and vault situated in Johannesburg.

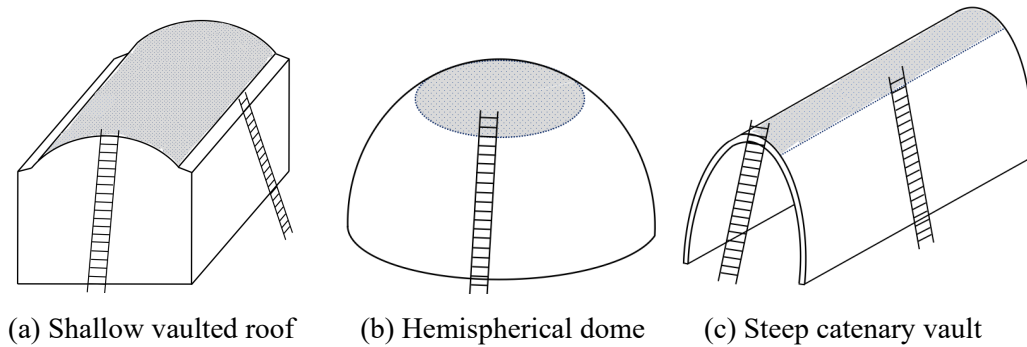


Figure 5.6 – Regions most likely to experience maintenance induced loads

These analyses, in combination with the experiences gained during the construction and maintenance of two catenary vaults at Wits, indicate that even the relatively low surface load given in SANS 10160-2 is likely conservative for steeper profiles. The construction process of the steep Wits shells, with an average roof pitch of 65° , required the use of scaffolding and long ladders for workers to access all parts of the roof. Furthermore, during routine maintenance and repair work shown in Figure 5.7, workers were only able to stand, without the assistance of a ladder, over the apex of the shells. Consequently, one would reasonably expect that materials placed during maintenance work would most likely be positioned on scaffolding, at ground level, or over a limited area near the roof apex.



Figure 5.7 – Access onto steep catenary vaults at Wits

Some codes stipulate that a minimum roof live load must still be considered, even when the structure requires scaffolding to create a work surface for labourers and materials during maintenance and repair processes. For example, the International Building Code (IBC) recommends that structures such as greenhouses, which typically require special scaffolding, be designed for a minimum roof load of 0.58 kN/m^2 .

Snow, rain and hail.

It is unrealistic to assume that water or hail would accumulate over the steeper vaults examined in this research; however, in cold climates, significant depth of snow may accumulate over roofs. This is marginally applicable to South Africa, and the national loading standards are limited in this regard. Therefore, this topic is discussed on the basis of ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures. Several aspects influence the magnitude and distribution of snow over roofs, such as the physical and climatic conditions, topography, inclination of the roof, roughness of the roof surface and internal building temperature. The roof slope is a particularly important factor in determining design snow loads. The snow shedding capabilities of pitched roofs are such that if they have a slope steeper than 60° they need not be designed for snow loading, whereas flat surfaces situated in the same location may need to be designed for significant accumulations (Sandaker *et al.*, 2013). According to Sandaker *et al.* (2013), typical snow loads may range from 1 kN/m^2 to 8.0 kN/m^2 or more. Furthermore, wind may cause snow to heap on certain parts of a roof, whilst it may be blown / swept off completely in other areas (i.e. snow from the windward side of a gable becomes the source for the drift accumulation on the leeward side). This can lead to complex loading patterns, which are simplified for common geometric shapes.

Roof snow loads are computed using the ground snow load (p_g) as the basis for calculation. The load is then multiplied by several factors, which account for building exposure, internal building temperature, building importance, roof geometry and surface roughness, to determine the curved roof snow load (p_s). Unbalanced loads, for duo-pitch roofs are only applicable for roof slopes between 1/2 in 12 (approximately 2.5°) and 33 in 12 (70°). Beyond 70° , balanced and unbalanced loads need not to be considered. For curved roofs, sections having a slope exceeding 70° are considered free of snow loading. The influence of roof geometry and surface condition are accounted for by the roof slope factor (C_s) in

snow load calculations. The factor is constant for a particular slope of duo-pitch roof; however, the magnitude varies from eave to apex for curved roofs.

Equations 5.7 and 5.8 are used to calculate the snow load on flat and sloped roofs, respectively.

$$p_f = 0.7C_e C_t I_s p_g \quad (5.7)$$

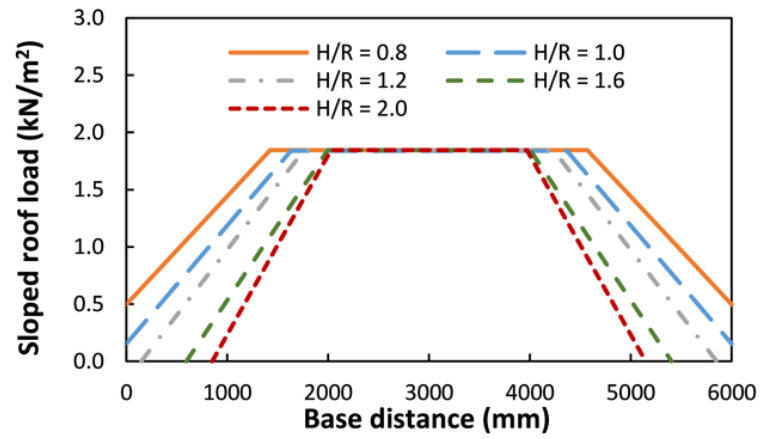
$$p_s = C_s p_f \quad (5.8)$$

where p_s and p_f are the sloped roof (balanced) snow load and the flat roof snow load, respectively.

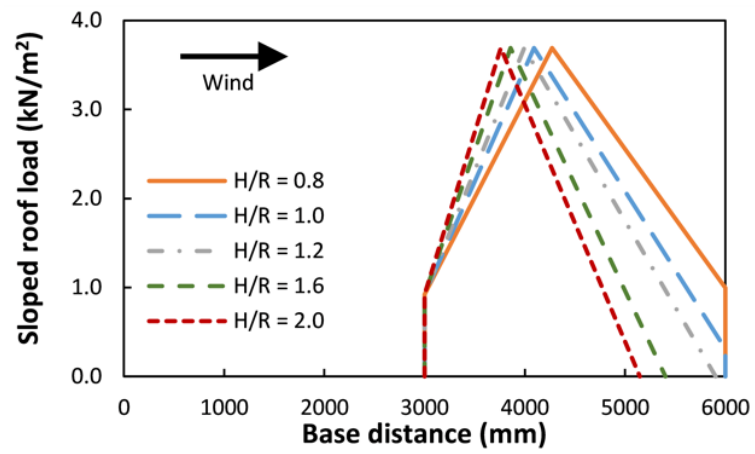
Figures 5.8a and 5.8b show the balanced and unbalanced snow load distributions derived for catenary vaults having $0.8 \leq H/R \leq 2.0$ and span of 6 m. A ground snow load (p_g) of 2.4 kN/m^2 , specified for US cities such as Portland and Anchorage, exposure factor $C_e = 1.0$ (partially exposed roof, open terrain (Category C)), importance factor $I_s = 1.0$ (for Risk Category II), and thermal factor $C_t = 1.1$ (i.e. a cold roof), were assumed herein. Figure 5.8c shows the relationship between the total snow load (kN), expressed as a percentage of the total balanced snow load over a flat roof, and the height to base radius ratio (H/R). Shallower catenary roofs have larger magnitude snow loading (balanced and unbalanced) than steeper shells. The total loads (balanced and unbalanced) on the steepest catenary vault, $H/R = 2.0$, are approximately 60% of that on the shallowest catenary vault, $H/R = 0.8$.

5.2.3.5 Reduced likelihood of concentrated loads developing over roofs

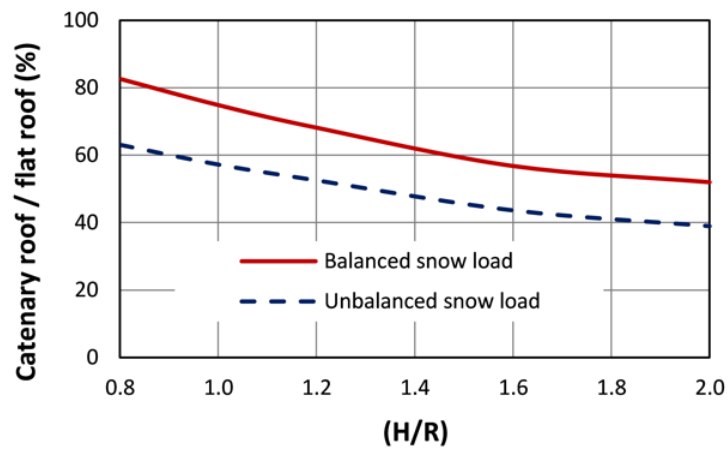
Limiting the possibility of concentrated loads is particularly important as they may cause large localised bending and shear forces, which could cause cracking, local punching failure or even collapse. According to Dahmen and Ochsendorf (2012), point loads on earthen shells should be avoided to prevent local failures. However, the complete prevention of point loading over low-cost housing shells is practically impossible since point loads are generated by people walking over shell roofs during construction. Thus, maintenance and repair operations must be considered during design, as stipulated in building regulations across the world.



(a) Balanced snow loads derived from ASCE/SEI 7-10



(b) Unbalanced snow loads derived from ASCE/SEI 7-10



(c) Relationship between total snow load and H/R ratio

Figure 5.8 – Balanced and unbalanced snow loads

However, systems should still be employed to mitigate the risk of concentrated loads on roofs. For example, by adopting steeper profiles, which inhibit people from gaining access onto the structure. Furthermore, a steep slope reduces the area over which objects may be placed during maintenance and repair work. Such a measure was considered during the design, construction and maintenance of the two steep vaults built at the University of the Witwatersrand. Workers required ladders and scaffolding to gain access to the top of these shells, as shown in Figure 5.7. The steep nature of these vaults also impedes people from climbing onto and walking over much of the structure. In fact, most maintenance and repair work had to be done whilst workers stood on ladders. Consequently, concentrated loads created by workers would be limited to areas where they are able to access, stand and walk on the structure, e.g., over relatively small areas at the apex of hemispherical domes and steep catenary vaults (Figure 5.6). Figure 5.9 shows a semi-circular earth brick vault built at the Auroville Earth Institute. Several people are standing or sitting at the apex during the casting of an earthen plaster to waterproof the surface. Figure 5.10 illustrates this point for a hemispherical CSEB dome at Central Johannesburg College (CJC).

5.3 Loading considerations for catenary shells

5.3.1 Concentrated loads

5.3.1.1 Construction, maintenance as well as occupancy

Many loading codes stipulate minimum concentrated loads for construction, maintenance as well as occupancy. As mentioned previously, earthen shells for housing should be designed as inaccessible, without access provided onto the roof. The International Building Code (IBC) stipulates that roofs must be designed for either uniformly distributed surface loads or concentrated loading, whichever produces the most severe load effect. Other codes follow a similar approach (SANS 10160 – 2 (SANS, 2011) and EN 1991-1-1 (CEN, 2002a)). The relevant point load magnitudes and application areas derived from these standards are shown in Table 5.1. Furthermore, minimum concentrated loads should be positioned so as to produce the maximum stresses within the roof system. For example, ASCE/SEI 7-10 stipulates that concentrated loads, must be assumed to be uniformly distributed over an area of 762 mm by 762 mm and be located so as to produce the maximum load effects in the members.



Figure 5.9 – Semi-circular vault at the Auroville Earth Institute, India

* These images were taken from the Auroville Earth Institute website (http://www.earth-auroville.com/semicircular_vault_en.php) - no rights reserved



(a) Access onto a dome



(b) Person standing over the apex

Figure 5.10 – Access onto a hemispherical CSEB dome at CJC

Table 5.1 – Minimum stipulated concentrated loads

Standard	Load (kN)	Area (m ²)
EN 1991 - 1- 1	1.00	0.0025
BS EN 1991 - 1 - 1	0.90	0.0025
SANS 10160 - 2	1.00	0.01
ASCE/SEI 7-10	1.33	0.5806

López-Almansa *et al.* (2010) examined the load capacity of reinforced masonry circular vaults and noted that the curvature was not considered significant for the resistance to

punching, since it is a localized phenomenon. However, there are several factors that influence the susceptibility of shells to such damage; e.g., the proximity of concentrated loads to shell boundaries as well as section thickness. Regions along shell edges that are free (e.g., not supported by walls) will characteristically have the lowest resistance to these local actions; similar to a point load positioned at a free-edge of a reinforced concrete slab.

A simple analysis, using finite elements, was adopted to determine the influence of the aforementioned factors, with reference to free-spanning catenary vaults. The minimum stipulated point load, as per SANS 10160 – 2, was applied as a vertical pressure over an area of 0.1m x 0.1m (i.e., a pressure of 100000 N/m² acting over 0.01m²). A load combination of 0.9D_L + 1.6L_L was considered in the numerical investigations, and partial factors were acquired from SANS 10160 – 1. Several FE model, with different H/R ratios and thicknesses, were considered for the analyses discussed here. The material and element properties are presented in Appendix C. Two of these FE models, which correspond to H/R = 1.0 and H/R = 1.6, with span 6m, length 8 m, and thickness 0.2 m are shown in Figures 5.11a and 5.12a. In both models, the point load is shown positioned at the apex edge. The major principal stress contour plots, corresponding to the aforementioned loading, are shown in Figure 5.11.b and 5.12b. Tensile stresses with magnitude of 0.02 MPa, as well as all compressive stresses, are shown in black on the plots.

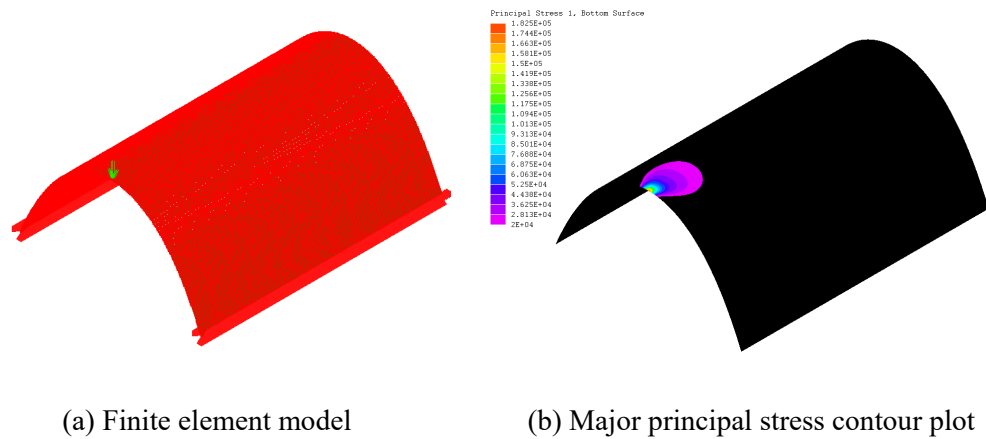


Figure 5.11 – Point load: H/R = 1.0

The location of the concentrated load was varied over the catenary roof so as to generate maximum tensile stress. Figure 5.13.a shows maximum principal tensile stress, for load variation between 65° and 90°, along the centre and edge of the H/R = 1.0 catenary vault.

Placing the point load at the edge led to stresses much larger than those observed for the load at the centreline: roughly four times more. Figure 5.13b shows the relationship between the maximum tensile stress, for the point load positioned at the apex edge, and vault thickness. The results confirm that shell thickness plays a significant role in the magnitude of stresses induced by concentrated loads, especially when profiles become ultra-thin.

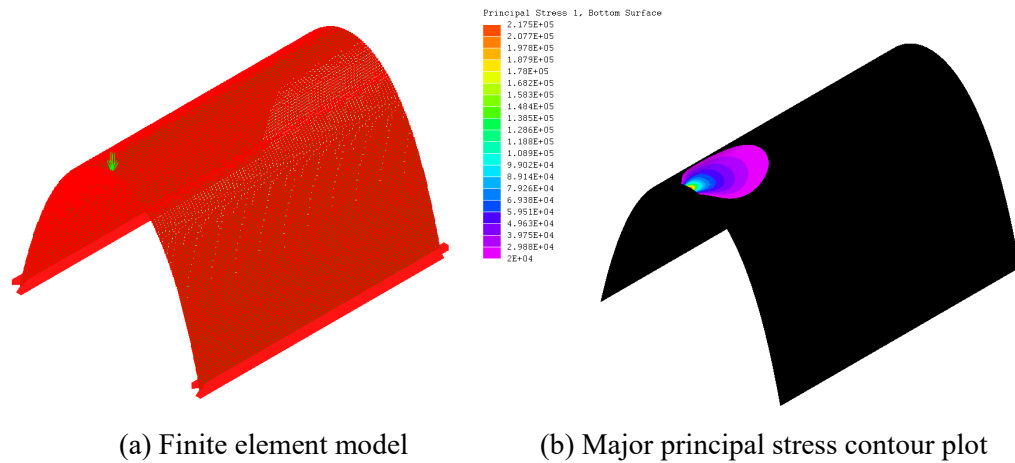
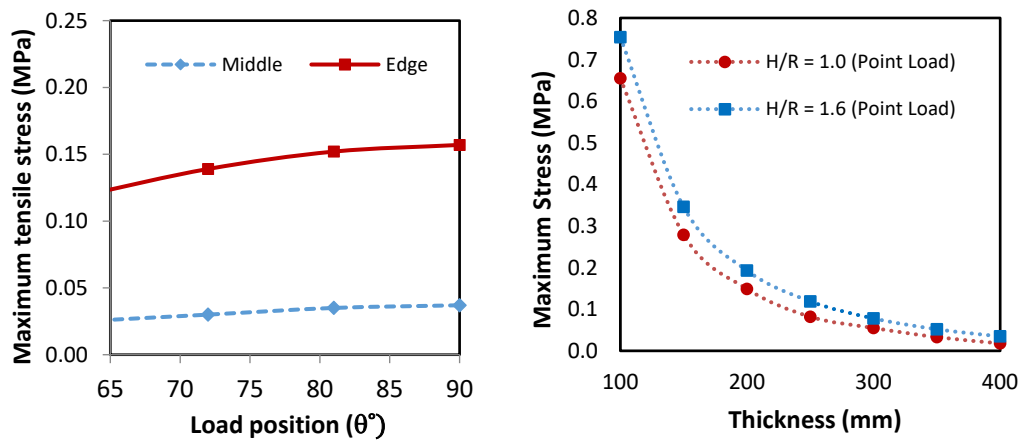


Figure 5.12 – Point load: $H/R = 1.6$



(a) Major principal stress vs. load position (b) Major principal stress vs. vault thickness

Figure 5.13 – Results from FE models

The susceptibility of extremely thin sections to concentrated loads is illustrated by localised damage observed in a two compressed stabilised earth tile (CSET) cross vaults built at the University of the Witwatersrand. Figures 5.14a and 5.15a show the two prototype shells, which are between 50 to 60 mm thick, comprising two layers of CSET separated by 10-20

mm of cement-sand mortar. Although these shells are exceptionally thin, they are capable of resisting the load of several individuals sitting, standing and walking over the apex. However, Figs 5.14b and 5.15b-5.15d show localized punching damage in the two shells, which were caused by a person attempting to step from the apex-edge of one shell onto the other. Figure 5.14d shows cracks extending towards the centre of the shell, which collapsed several weeks later after a heavy rainstorm. Thin shells, such as these, will inherently have lower resistance to concentrated loads, particularly near a free edge. Housing shell structures would be situated sufficiently far apart to discount a scenario where people could step or jump between adjacent structures; however, concentrated loads could develop at a free-edge as a person steps off a ladder onto the shell.



(a) Three people on the shell apex



(b) Local punching damage at the free edge



(c) Enlarged image of the punching damage



(d) Adjacent cracks (traced for clarity)

Figure 5.14 – Domed cross-vault at Wits



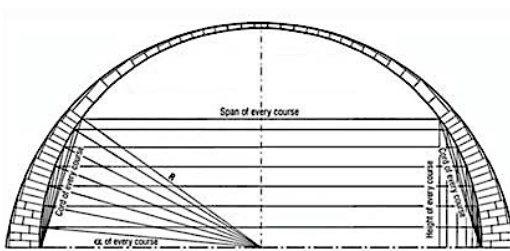
(a) Two people standing on the shell



(b) Local damage at the free-edge

Figure 5.15 – Catenary cross-vault at Wits

Some shells are designed and built with ultra-thin sections toward the apex, where compressive stresses due to gravity are the lowest. Several vaults built at the Auroville Earth Institute were designed using a combination of hanging chain models and graphic statics to optimise the shape under gravity load. For the shells, the theoretical thrust-line position lies along the central axis of the profile. A schematic and photograph, obtained from the Auroville Earth Institute website, of one such shell is shown in Figures 5.16a and 5.16b, respectively. To achieve pure-compression within this semi-circular shell, the thickness was varied from 365 mm at the base to only 70 mm at the apex. The vault has span of 6 m, rise of 3 m, length of 11m, and was built with a combination of horizontal and vertical courses to accommodate the variable thickness as well as the adopted free-spanning construction technique.



(a) Drawing



(b) Building structure

Figure 5.16 – Semi-circular vault at the Auroville Earth Institute

* These images were taken from the Auroville Earth Institute website (http://www.earth-auroville.com/stability_calculations_en.php) - no rights reserved

The shape of a shell considered by Allen and Zalewski (2010) was optimised by passing a thrust line through three points. An envelope of all thrust lines was generated (for dead and imposed loads) by forcing them to pass through the same point at mid-span; effectively minimising the thickness at the apex. Varying the thickness from a maximum at the base to a minimum at the apex may also be adopted to increase usable floor space. The optimization technique presented in Chapter 4 was utilised to illustrate the technique. Figure 5.17a shows two pure-compression vault profiles, with the same height-span ratio, corresponding to a uniform and varying section. Figure 5.17b illustrates how additional usable space can generated by adopting a steeper profile in combination with varying thickness. The varying profiles (solid black lines) shown in Figures 5.17a and 5.17b were selected purely because they provide vaults of equivalent material/mass to the catenary vault with uniform cross-section (dashed red lines). Based on the authors experience as well as supplementary evidence offered by Figures 5.7, 5.9, 5.10, 5.15, 5.14, and 5.16, the region of least thickness will coincide with zones having the highest potential of exposure to vertical concentrated loads.

5.3.1.2 Solar water heaters and photovoltaic panels

In South Africa, due its high solar irradiance, it is reasonable to expect that solar panels and/or solar water heaters might be placed on the roof of housing structures. Solar geysers are particularly popular in South Africa, as shown in Figure 5.18. The International Building Code (IBC) stipulates the design and loading requirements of roof structures that provide support for photovoltaic panel systems. The IBC specifies that roofs supporting photovoltaic (PV) systems must accommodate the full solar photovoltaic panels or modules, ballast dead load, roof live load, concentrated loads from support frames, as well as snow drift loads created by panels where applicable.

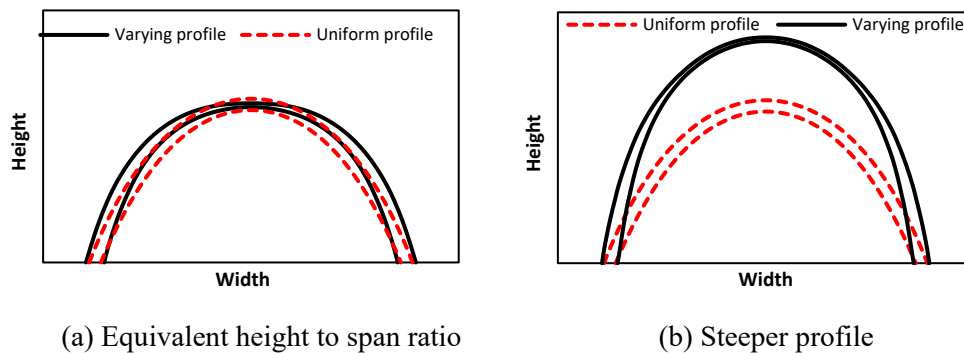


Figure 5.17 – Pure compression shells with varying profile

Of major concern is the potential loading created by the attachment and placement of solar water heaters over unreinforced earth shells, which are typically thin, brittle and exhibit limited tensile capacity. Solar water heaters, particularly those which incorporate a geyser on the roof, could lead to significant concentrated loading during placement, operation, and repair. The combined load of a full 150 litre geyser (~200 kg), supporting steel frame, collector, and potential person undertaking repairs may well control the design. Figure 5.19 illustrates how an evacuated-tube solar water heater support frame creates a concentrated load over a small portion of a roof. The potential magnitude of these concentrated loads in combination with the susceptibility of thin earthen shells to punching failure are concerning, noting that these elements, are in some cases, positioned on roofs without any engineering consideration.

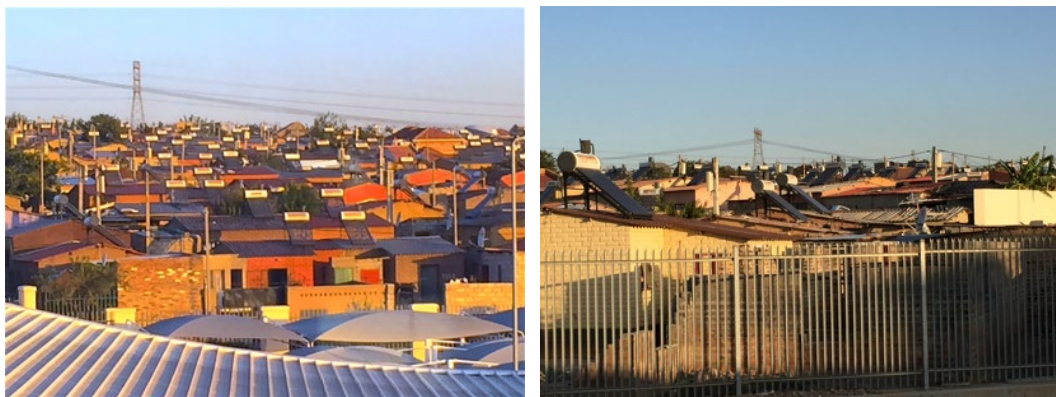


Figure 5.18 – Solar water heaters in Alexandra Township, Johannesburg



Figure 5.19 – Point loads created by a solar water heater

The current trend in photovoltaic panel installations is to avoid the need for roof penetrations, which may be prone to water leakages. Penetrating systems in concrete typically incorporate fasteners (i.e., wedge and sleeve anchors, lag bolt and shield, or screws) embedded into the roof to hold down panels against wind uplift. Ballasted photovoltaic panel systems are a common alternative to penetrating systems for flat roofs. Several possible photovoltaic panel configurations for the prototype house at Wits are illustrated in Figures 5.20a through 5.20d. In principle, multiple solar panels could be attached to the north facing vault; bolted into the masonry and/or screed above the edge beams (Figure 5.20a). Ballasted systems could potentially be adopted over flatter portions towards the apex of some shells (Figure 5.20b).

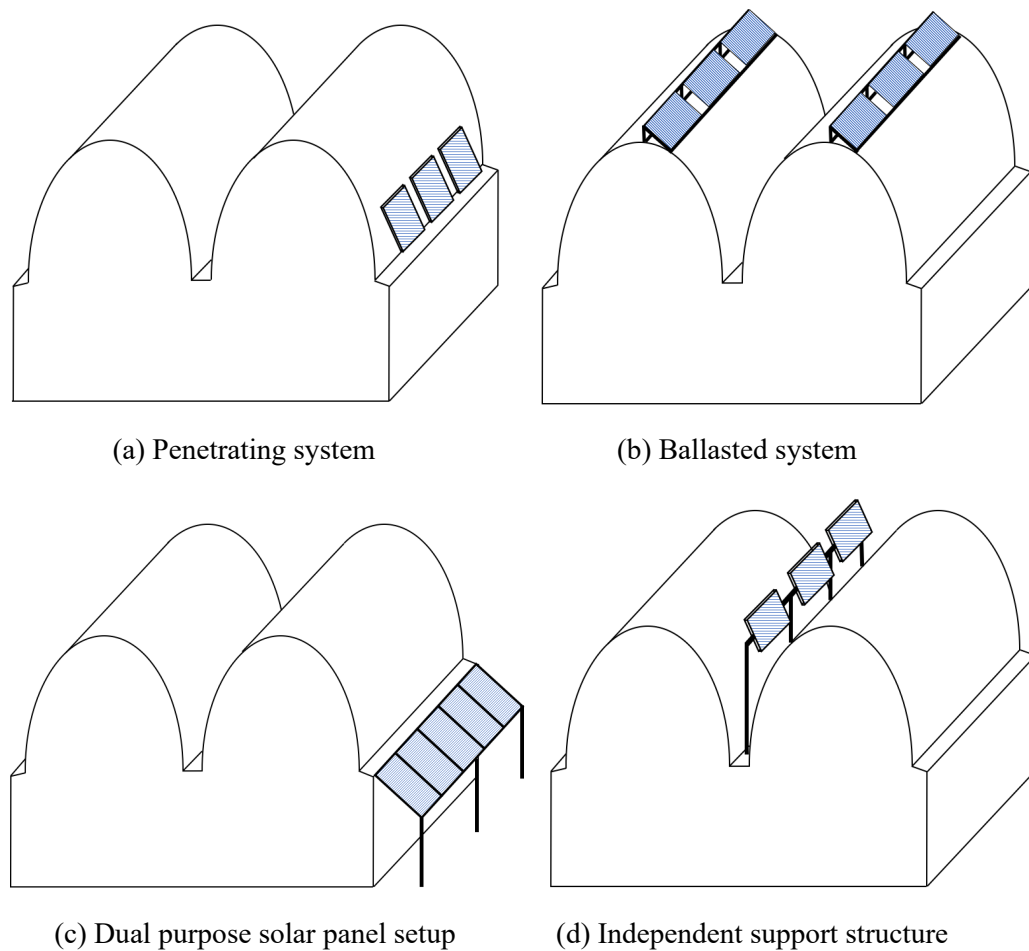


Figure 5.20 – Possible solar panel configurations

There are several other systems that are particularly appealing for use on unreinforced earthen masonry shells, such as film collectors. These require no supporting framework,

reducing costs and mass, and penetrations are eliminated. If standard solar panels are to be used in low-cost environments, then it would be beneficial to adopt systems which offer dual-functionality. One such solution is illustrated in Figure 5.20c, in which solar panels are used to generate similar thermal benefits associated with eaves in conventional housing structures (i.e., assisting in seasonally regulating solar heat gain). Alternatively, solar panels could be supported by independent structures or elements so that the roof is not required to carry the associated dead and live loading, as shown in Figure 5.20d. This is certainly a cautious solution but may be economically and/or technically practicable in some situations.

Solar water heaters should be supported by a separate structural system, which does not load the shell or roof. Two such methods were considered for supporting solar panels and heaters at the Wits shells. A photovoltaic solar panel, shown in Figure 5.21a, was incorporated into the prototype house to power instrumentation situated indoors. This panel had to be elevated in order to receive sunlight throughout the year. Expanding shield anchors were used to attach the supporting steel frame, housing the solar panel, to the concrete screed between the two catenary vaults; thus, applying no load to either unreinforced shell. Similarly, a solar water heater was fixed to the structure, and supported by two steel brackets attached to the wall, as shown in Figure 5.21b.



(a) Solar panel



(b) Solar water heater

Figure 5.21 – Panel supported by an independent structure between the vaults at Wits

5.3.1.3 Measures to mitigate damage associated with concentrated loads

Steel reinforcing bars have been used in earth construction to enhance tensile strength. One such example, discussed by Morton (2008), was the use of stainless-steel reinforcement to improve the strength of late medieval clay cob being repaired in Moorhouse, Cumbria. Common steel, however, is prone to corrosion, and its inclusion could ultimately diminish the life-span of a masonry shell if not safeguarded from oxidation unless more resistant types of steel such as corrosion resistant alloys or stainless steel is used. According to Dahmen and Ochsendorf (2012), steel should be minimized or removed entirely within earthen masonry because it is difficult to get an effective bond between earth and steel. Furthermore, the inclusion of such materials may be inconsistent with employing less expensive and sustainable materials.

Several studies in the literature present alternate methodologies for the reinforcement and repair of masonry vaults and conventional housing structures using fibre-reinforced polymer (FRP), geotextiles, and geogrid mesh (Valuzzi *et al.*, 2001; Holliday *et al.*, 2016; Surat, 2016). Garmendia *et al.* (2011) performed static load testing on circular masonry arches reinforced with basalt fiber geogrid. The mesh was placed over the outside (extrados) of the arch, and was secured with anchorage spikes. Failure of the system was characterized by masonry crushing and debonding of the reinforcement, which is different from the typical instability collapse observed for unreinforced arches. Ductility of the structure was, however, improved, and the load capacity was increased 13 times. Aymerich *et al.*, (2016) examined the influence of discrete fibre reinforcement on the fracture resistance and energy absorption capabilities of earthen materials under dynamic loading (impact) conditions. The authors reported that hemp fibre inclusion greatly improved the peak load, post-crack strength, ductility and the energy dissipation of earth specimens under impact loading. However, there are additional considerations in masonry construction, which incorporate joints between blocks; the quality of which may significantly influence the response of masonry constructions under dynamic/impact loading (Aymerich *et al.*, 2016). Understandably, much of the effort to improve the strength (i.e., in-plane flexural, out-of-plane flexural and in-plane shear) and seismic response of masonry has focused on reinforcing strategies. These include reinforcing mesh, fibre reinforced polymer (FRP) and fabric embedded in cement-based mortars (Valuzzi *et al.*, 2001; Blondet *et al.*, 2006; Garmendia *et al.*, 2011; Bove *et al.*, 2016; Holliday *et al.*, 2016;

Alecci *et al.*, 2016). The physical-chemical compatibility of some FRP composites with the masonry may be a problem, and innovative composites made of a fabric embedded in a cement-based mortar fabric reinforced cementitious matrix (FRCM) have been proposed as an alternative to FRP composites (Alecci *et al.*, 2016a; Alecci *et al.*, 2016b). Natural composites have also been examined to offer more sustainable reinforcing solutions. Bove *et al.* (2016) assessed the anti-seismic effectiveness of jute and coconut nets embedded in mortar for earthen buildings.

If the objective is the adoption of unreinforced shells, then alternative measures need to be considered, i.e., rather than reinforcing to improve capacity, design actions must be implemented to mitigate the risk and/or extent of large vertical loads being applied over these structures. Several appropriate solutions for low-cost unreinforced structures are listed below.

- Access onto the roof, such as stairways, access ladders or walkways should not be provided. Shells would still need to be designed to accommodate the minimum stipulated concentrated loads, for inaccessible/ordinary roofs, presented in codes of practice.
- Adopt shell forms which inhibit people from climbing, walking and placing objects onto the structure (e.g., steeper shell profiles).
- Support solar panels using independent structures or elements so that the roof is not required to carry the associated dead and maintenance induced live loading. When costs need to be kept to a minimum that solar panels would either be attached to the shell or supported by some other structure that provides additional functionality/purpose.
- Avoid ultra-thin sections toward the apex of singly-curved shells. These relatively flat, thin regions are more susceptible to punching damage, and have the highest likelihood of being exposed to vertical concentrated loads, e.g., stacked building materials, people, ballasted solar panels, etc.

5.3.2 Impact loads

5.3.2.1 Accidental impact loads

Accidental actions are typically categorized as short duration loads, of substantial magnitude, that are unlikely to occur during the design life of the structure. These usually

include impact loads, explosions and fires. In most categories of structures, an impact action is an unusual event that affects a specific structural component at a random moment in time. Typically, such hazards have low probability of coincidence with other actions; although, in some cases, accidental actions may be associated with other actions; e.g., trees falling as a result of strong winds and wind-borne debris.

Impacts caused by wind borne debris can cause catastrophic damage to building infrastructure. This is particularly true in extreme wind events, such as tornadoes, where projectiles have the potential to travel at speeds in excess of 100 m/s (Goliger, 2002). McDonald (1990) reported that conventional wall construction systems used in the United States (e.g., wooden frame with Masonite, plywood, or stucco sheeting) are not able to stop to tornado-generated missiles, irrespective of the shape of the tip inflicting the damage. In reference to tornado-generated debris in South Africa, Goliger (2002) noted that traditional brick walls typically sustain cracks or partial damage but not total collapse; however, these exterior walls are typically at least 200 mm thick.

Matta *et al.*, (2015) recommended steel wire mesh or geogrid mesh plaster as a solution to enhance the resistance of CSEB against impactors such as flying debris from tornadoes and hurricanes. Such a reinforcing system leads to improved ductility and the system's ability to absorb significantly more strain energy before collapsing. The design capacity of reinforced concrete (RC) shells, of the scale considered in this thesis, will typically be governed by minimum steel requirements. This will, in some instances, offer additional safety against unexpected loading or design errors. Examples of the structural performance of RC shells, under the action of a concentrated impact loads, are provided by Ulrich (2017) and South (2018). Ulrich (2017) reported that a large tree, which fell onto a RC dome house situated in Lakemont, Georgia, caused only minor damage to the paint and insulating foam. South (2018) described minor damage to a caterpillar shaped dome house situated in Florida - a small hole in the shell wall - which was caused by a wind-borne residential transformer (estimated to weigh about 500 lbs) during the category 5 Hurricane Michael.

Damage to unreinforced earthen masonry shells, however, may be significant. Figure 5.22 shows damage to a dome at Sparrow Rainbow Village caused by an impact from a falling tree, which occurred during a thunderstorm in late November 2015 (Bradley *et al.*, 2017a). Another dome was struck by a tree but suffered with only minor damage. A third tree was

also uprooted but fell on an unoccupied space within the village. The dome shown in Figure 5.22, used as a bakery, partially collapsed and was eventually demolished. The other dome, used as an office, cracked extensively in the vicinity of the impact. These cracks passed through plaster as well as masonry; consequently, they were repaired with strips of waterproofing membrane to prevent moisture ingress. The office shell includes several CMU partitions, unlike the free-spanning bakery shell, which certainly stiffened the structure as well as offered additional loads paths. This may explain the minor damage suffered by the office shell compared to that of the bakery shell, although the characteristics of the impact (i.e., mass, velocity, impact location, etc.) are also possible explanations for the differences observed. Impact loading of this nature may result in falling masonry, of significant mass, which may result in serious injury or even death of occupants.



(a) CMU masonry dome



(b) Dome collapse (outside)



(c) Dome collapse (inside)



(d) Large uprooted tree

Figure 5.22 – Partial collapse of unreinforced masonry dome

5.3.2.2 Measures to mitigate damage associated with impact loads

In certain environments, earthen shells used for housing would be exposed to the risk of falling trees. In such situations, the detrimental effects of these impact loads must be recognized. In general, several alternatives may be implemented to ensure structures have enough robustness to accommodate impact loads. These are, somewhat, limited in reference to free-spanning earthen shells. A few options are listed below.

- Design the shell to sustain the action; however, increasing the thickness of the masonry to accommodate accidental loads may result in excessive technical and/or economic consequences.
- Limit the extent of failure by ensuring alternate load paths within the shell. Doubly-curved shells, such as domes, are intrinsically superior to singly-curved shells in this regard. Pier-stiffening or corrugating are also potential tools to provide additional load paths within a shell. Although load redistribution may mitigate complete collapse, the hazards associated with partial failure, as shown in Figure 5.22, are still significant.
- Incorporate reinforcing to enhance capacity and improve ductility, which is desirable when enough time is necessary for people to evacuate a structure in situations that may result in collapse. These solutions are particularly appealing in areas prone to moderate-high seismicity and/or extreme wind events, where reinforcing may be necessary to meet minimum strength demands prescribed in building codes. However, in areas that not exposed to the aforementioned loadings, the added cost associated with integrating such reinforcement may be inconsistent with the dual objectives of materials optimization and sustainability.
- Select a construction site away from large trees and/or transport infrastructure. Alternatively, minimize the possibilities of accidental events through control and prevention, e.g., removal of large trees and/or elimination of the transport infrastructure from the vicinity of the shells.

Since the objective is to use unreinforced earthen masonry, then the most appropriate solution is to reduce the risk of accidental actions through control and prevention. For example, the large trees adjacent to several CMU domes, shown in Figure 5.23a, at Sparrow Rainbow Village were removed to prevent any further incidents, as shown in Figure 5.23b. Because of funding issues, the trees were only removed several months after the dome had

collapsed. The inability to mitigate such risks because of insufficient finances is a major difficulty in low-cost environments.



(a) Large trees adjacent to several domes (b) Trees removed after the dome collapse

Figure 5.23 – Large trees adjacent to several domes at the Sparrow Rainbow village

5.3.3 Lateral loading

5.3.3.1 General

Significant lateral loading may develop on buildings due to earthquakes, flooding and strong wind events. Several authors have examined the design considerations, associated with natural hazards, for traditional CSEB masonry housing structures, i.e., a one-story masonry dwelling with shallow roof. Holliday *et al.* (2016) noted that unreinforced CSEB masonry is only acceptable for use in limited areas in the United States due to the mass of standard walls and the inertial loads they generate during earthquakes. Matta *et al.* (2015) provided a range of CSEB walls thicknesses, between 203 mm to 508 mm, for different wind speeds associated with events up to Category 4 hurricanes and EF3 tornadoes. These authors noted that it may be uneconomical to improve the in-plane and out-of-plane flexural capacity of CSEB masonry by increasing wall thickness, and that grouted steel reinforcement or reinforcing mesh is more appropriate to accomplish these improvements. Varum *et al.* (2014) provided information on the structural behaviour and seismic vulnerability of earthen masonry structures, as well as rehabilitation and retrofitting techniques (e.g., reinforcing) for existing earthen masonry structures. Adopting mesh reinforcing and/or specifying thick walls may be required in regions prone to seismicity and extreme wind events. If steeper catenary vaults are to be built in such areas, then additional design measures would need to be introduced, such as reinforcement to improve structural response.

In areas not prone to seismic activity, steeper unreinforced vaults may be appealing to owners. However, wind loading becomes more prominent as building height increases. Furthermore, the inclusion of expansion joints between the shell and internal walls (e.g., gable and partition) to mitigate thermal and/or shrinkage cracking removes the wall from the load-path within the structure; the consequence of this is three-fold:

- Bending and shear forces within the shell will increase;
- The shell must transfer all lateral loads along its length to the foundations or supports;
- The stiffness of the vault, parallel to the non-load bearing gable-walls, is significantly reduced.

5.3.3.2 Wind climate and action in South Africa

In South Africa, and many other regions throughout the world, because of very low seismic risk and absence of snow, wind constitutes one of the most critical environmental actions affecting the design of built environment. According to Goliger (2002), wind speed is the most critical characteristic of a strong wind event in relation to the extent of damage. The highest wind speeds affecting the built environment are typically related to severe thunderstorm activity, i.e., outflow from a thunderstorm's supercell (often as short but powerful frontal bursts), or severe downdrafts (microbursts) and tornadoes generated within the supercell (Goliger, 2002). South Africa regularly experiences tornadic events, with an historic average of about 4 events per year (Goliger *et al.*, 1997). The majority of tornadoes occur during the summer months, and events were recorded as recently as 12th, 14th, 21st and 22nd of November 2019 in KwaZulu-Natal (News24, 2018). Tornadoes can be devastating to infrastructure, sometimes resulting in loss of human life (Goliger *et al.*, 1997); but they are an extreme environmental event of low probability of occurrence. As such, tornadoes are not considered in conventional design of building structures across the world (i.e., considering a 50-year return period and 3s-gust wind speeds per SANS 10160-3). In fact, in South Africa, for return periods less than 500 years, the wind climate is dominated by thunderstorm winds and large-scale storms (Goliger *et al.*, 1997).

At the time of conducting calculations and analyses presented in this thesis, the design peak wind speed (3s-gust) stipulated in SANS 10160-3, for the majority of South Africa, was approximately 40 m/s (50-year return period). Instruments have captured speeds associated with such strong wind events; e.g., a peak wind speed of 39,4 m/s was recorded in February

2000 in Fort Beaufort (Goliger, 2002). Due to the higher probability associated with extreme wind events in some other parts of the world, buildings in these regions must be designed to withstand markedly higher wind speeds. For example, the Florida Building Code (2010) prescribes a nominal design 3s-gust wind speed of 175 mph for Risk Category II buildings (e.g. one- and two-family dwellings) in Miami-Dade County. The above factored design wind speed corresponds to an equivalent nominal wind speed (3s-gust) of 136 mph (61 m/s).

Extreme wind events typically cause severe damage to both engineered and non-engineered infrastructure in South Africa. A recent thunderstorm in Johannesburg, with at least one observed tornadic funnel, caused damage to roof coverings of 855 houses, and 7 houses experienced major structural damage (Mahachi *et al.*, 2018). The storm was classified by the South African Weather Bureau as a Tornadic Supercell Thunderstorm with an EF2 damage rating on the Fujita/Pearson scale. This rating represents damage caused by wind speeds of 50 - 60 m/s, which is significantly larger than the basic wind speeds prescribed by SANS 10160:3 (2018). According to Goliger and Bradley (2018), the magnitude of winds at low levels above the ground, could have likely reached about 45 m/s, with a margin of +/-15%. Studies of damage to housing infrastructure in South Africa (e.g., Goliger, 2002; Mahachi *et al.*, 2018) indicate that other factors lead to the extent and nature of observed damage, including non-engineered structural systems, issues with construction practice, and shortcomings of some stipulations, e.g., the NHBRC Home Building Manual (2015) is not aligned with recent wind loading specifications (SANS 10160-3) and other international best practices.

Formal or engineered housing structures characteristically include structural and fixing elements (e.g., hurricane clips), between roof and walls, to prevent roofs being blown off in high winds. In reference to conventional earth masonry housing, Bolton and Burroughs (2001) recommended ring beams, atop walls, which provide a strong base onto which the roof may be tied. Ring beams also enhance the strength and 3-dimensional rigidity of the system. These authors also recommended an unreinforced earth block wall thickness of 200 mm or 250 mm, throughout South Africa, based on local conditions. Figure 5.24a shows a typical formal low-cost housing structure, which was severely damaged by a downburst in Matsulu (Mpumalanga, South Africa) in December 1991. The failure was triggered by collapse of a large unsupported gable wall (Goliger, 2002). Figure 5.24b shows

a similar housing unit, but with a large stiffening pier providing additional support, which withstood the same extreme wind event (Goliger, 2002).



Figure 5.24 – Formal housing, Matsulu, July 1992 (images courtesy of Adam Goliger)

The susceptibility of masonry walls to damage can be amplified in informal or non-engineered housing due to inadequate section thickness, poor brick bonding, etc. Figure 5.25 shows the wind damage to an informal housing structure in Bethany in December 1997. Figure 5.26 shows a piece of an exterior wall removed from a low-cost house damaged by strong winds in Protea Glen, Soweto, on 30 December 2017 (Goliger and Bradley, 2018). The ‘piece’ is a solid concrete brick, 90 mm thick, with 10 mm plaster finishes. The use of such thin sections in unsupported walls is not according to code - 90 mm is much less than that prescribed in SANS 10400-XA: 2011 to meet minimum thermal requirements for walls enclosing habitable rooms. For a single leaf masonry wall, the building regulations prescribe a nominal wall thickness greater than or equal to 140 mm (excluding plastering and rendering), plastered internally and rendered externally.

Most informal housing structures in South Africa typically comprise metal sheeting typically of single pitch and nearly flat (Goliger, 2002). These roofs are especially susceptible being blown off in strong winds due to its inadequate fixing to walls (Goliger, 2002). Conversely, Norton (1997) reported that earth shell roofs are good in high wind areas, where metal sheet roofs are frequently blown off. The performance advantage of earth shell roofs can be attributed to several factors; the foremost being that domed or vaulted masonry roofs provide considerable mass to counteract uplift forces. The benefit of mass, or ballast, is recognised by the occupants of informal housing, who often opt to

place heavy objects atop the roof to offset poor or non-existent fixing, e.g. rocks bricks, sand-bags, and other waste materials, as shown in Figure 5.25 and 5.27.



Figure 5.25 – Collapse of unsupported adobe wall of informal house (Bethany-Drakensberg, Dec. 1997) (image courtesy of Adam Goliger)

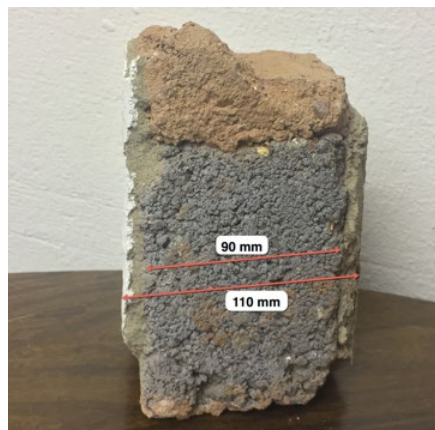


Figure 5.26 – Part of a wall damaged during a thunderstorm in Protea Glen, 30 Dec. 2017



Figure 5.27 – Traditional method of preventing roof uplift (informal house adjacent to the N14 highway between Pretoria and Roodepoort)

Although the mass associated with an earthen shell roof provides additional resistance to wind uplift, lateral forces should not be overlooked. Goliger (2002) noted that a typical display of wind damage to housing in South Africa is the collapse of large sections of unsupported walls. The form of free-spanning unreinforced catenary shells, particularly those having steep profiles, may be a problem due to the magnitude and distribution of loading that may develop over such structures in strong wind events (i.e., loading generated by design wind speeds given in SANS 10160 – 3). An initial analysis was conducted to determine the critical nature of lateral loading on catenary vaults; the analysis is presented and discussed herein.

5.3.3.3 Significance of lateral load on catenary vaults

To illustrate the significance of lateral loading, plastic analysis was adopted to assess the stability of free-standing catenary arches under unit load configurations. The full details of the numerical procedure are given in Appendix B. For the purpose of the discussions presented here, the fracture lines (or hinges) were assumed along the centreline of the section and moment capacity was considered equal at all hinges.

Two load situations were considered, namely; a vertical and horizontal unit load. A catenary profile having $H/R = 1$ was selected for this assessment, and the geometric properties are given in Table 5.2. An illustration of the vault, spanning between fixed supports, is shown in Figure 5.28. The load was varied from its starting point at the left support to the apex. The catenary shape and collapse mechanism for a vertical or horizontal load is shown in Figure 5.29a. Figures 5.29b illustrates the relationship between the plastic moment capacity and load position corresponding to fixed support conditions for both vertical and horizontal loads. A larger plastic moment capacity is required for the horizontal load situation. The difference is rather significant as indicated by the ratio of the maximum moment capacity for the horizontal load to that of the vertical load, which is approximately 1.7. These maxima moment capacity values correspond to approximate point load location of one quarter and one sixth of the span for the vertical and horizontal load, respectively.

Table 5.2 - Geometric properties of the catenary vault

Span (mm)	Height (mm)	Width (mm)	Thickness (mm)
6000	3000	1000	200

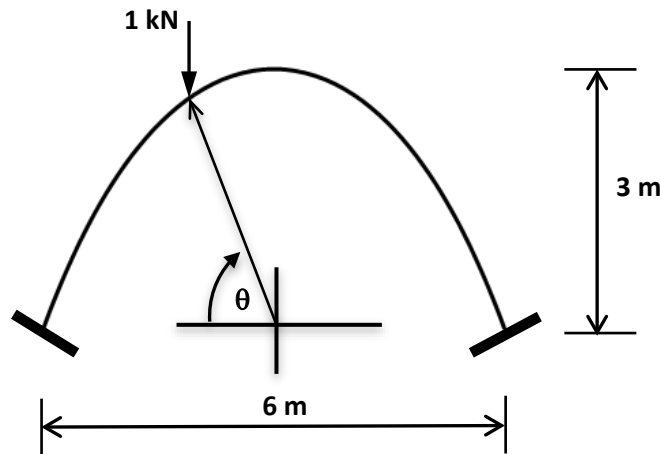


Figure 5.28 - Catenary vault: vertical unit load

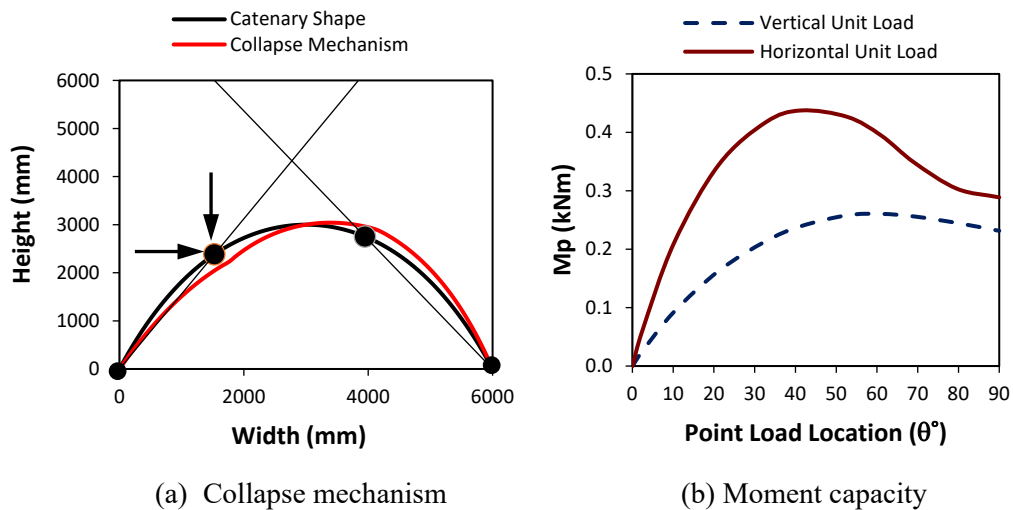


Figure 5.29 – Vault geometry and collapse mechanism

If a second load were positioned on the arch, then additional critical scenarios would develop. These are inferred by inspection from the collapse mechanism shown in Figure 5.29. These scenarios include two vertical loads acting at the same location, as shown in Figure 5.30a, two opposing vertical loads on opposite sides of the arch, as shown in Figure 5.30b, two horizontal loads acting at the same location, as shown in Figure 5.31a, and two horizontal loads acting in the same direction but on opposite sides of the arch, as shown in Figure 5.31b. An illustrative load distribution was formed by “combining” the vertical and lateral load scenarios presented in Figure 5.30b and 5.31b. Point loads have been illustrated as pressure distributions to better facilitate comparisons regarding wind loading over three-dimensional bluff body structures. In reference to wind loading, the distribution shown in

Figure 5.32 would engender positive and negative (suction) pressures on either side of the vault. This pattern bears a striking resemblance to wind pressure coefficient (C_{pe}) distributions over steep duo-pitch roofs presented in design standards. See, for example, Table C.7 and Figure C.35 in Appendix C for the C_{pe} distribution as per SANS 10160 – 3. There is some similarity between the wind load distributions over steep vaults and that over duo-pitch roofs (Bradley *et al.*, 2016).

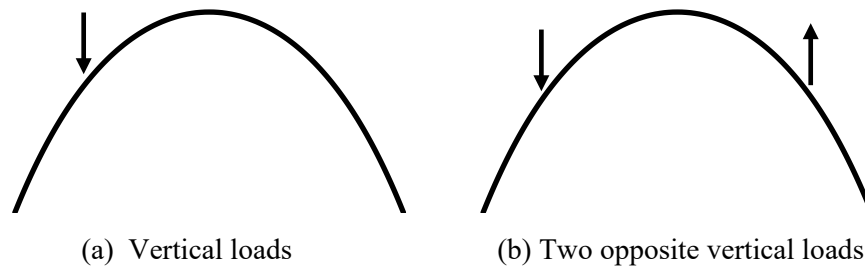


Figure 5.30 – Vertical load

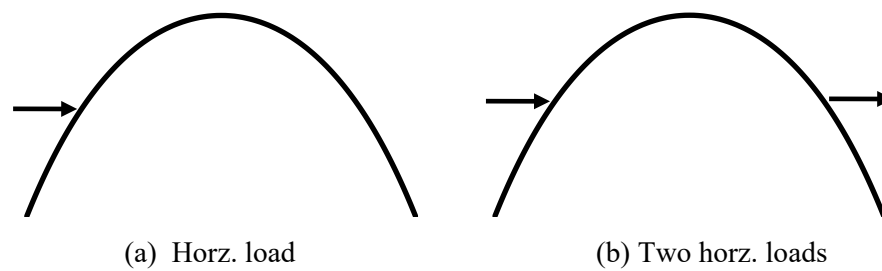


Figure 5.31 – Lateral load

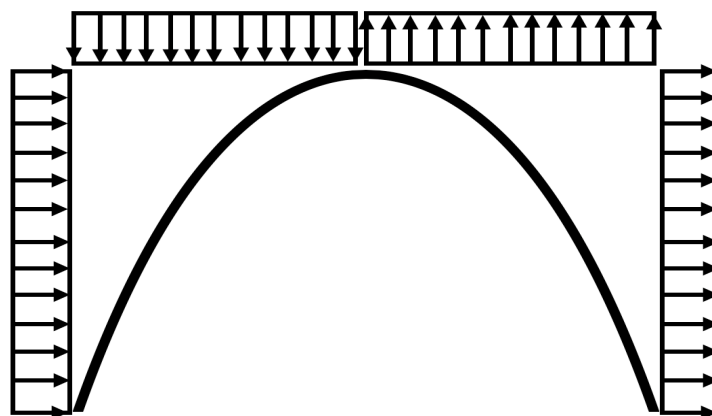


Figure 5.32 – Schematic illustration of critical distributed loading for catenary vaults

5.3.3.4 Wind loading on catenary vaults

General

Shell structures offer substantial benefits with regard to aerodynamic efficiency. The relatively substantial mass of masonry roofs also provides improved resistance to uplift. Although masonry shells have these benefits, most of the relevant wind loading research and design information correspond to commonly utilized shapes (e.g., circular) and structures (e.g., silos, circular elements, circular vaults and domes). However, the scale and form of the housing structures examined in this study necessitates the use of steeper profiles, which in some cases, differ noticeably from circular shapes considered in loading codes. Thus, a set of wind tunnel tests on catenary vault models was undertaken at the Council for Scientific and Industrial Research (CSIR) to determine the appropriateness of loading information corresponding to circular vaults for the design of catenary vaults. The full details of these tests (experimentation, analysis, interpretation of results, and formulation of design information) are presented in Appendix C. The major finding of the tests is that codified data for circular profiles is unsuitable for the design of steep catenary vaults (Bradley *et al.*, 2016). A similar observation is also reported in a study, co-supervised by the author of this thesis, on catenary domes (van Gool *et al.*, 2019).

Significance of wind loading for steep catenary profiles

Figure 5.33 illustrates the mean pressure coefficient distribution along the centre of a steep catenary vault, with $H/R = 1.6$, and a semi-circular vault, with $H/R = 1.0$, for wind blowing parallel to the gable-walls. Although the negative (suction) pressure coefficients are considerably larger for the semi-circular vault, the loading effect result is a markedly more critical situation for the steep catenary vault – based on the calculations presented in section 5.3.3.3. This critical situation develops due to the large positive pressure (applied over a larger area) on the windward face in combination with negative pressure over the leeward half.

To appropriately evaluate the wind loading effects on the shells, the pressure coefficients derived from wind-tunnel investigations are implemented as loading inputs for the relevant comparisons discussed herein. These pressure coefficient distributions correspond to $C_{pe,10}$, i.e., largest overall loaded areas of 10 m^2 , as specified in SANS 10160-3 (which follows the EN 1991-1-4 approach). The critical wind orientations correspond to $\alpha = 0^\circ$ and α

between 30° and 45° , where α is the wind orientation, as shown in Figure 5.34. Only the design data corresponding to these critical orientations are implemented in the present study. The external pressure coefficients for H/R between 0.8 and 1.6, corresponding to $\alpha = 0^\circ$, are shown in Figure 5.35.

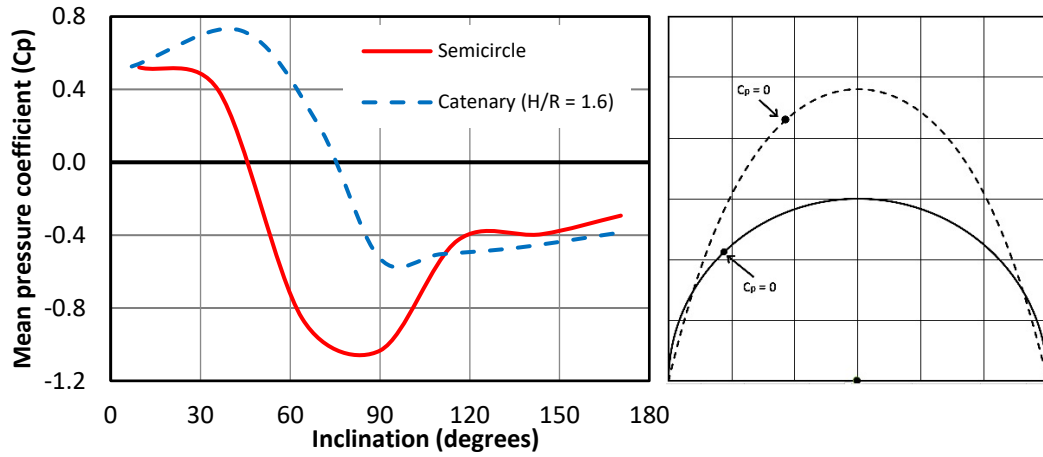


Figure 5.33 – Mean centreline pressure coefficient distributions (Bradley *et al.*, 2017a)

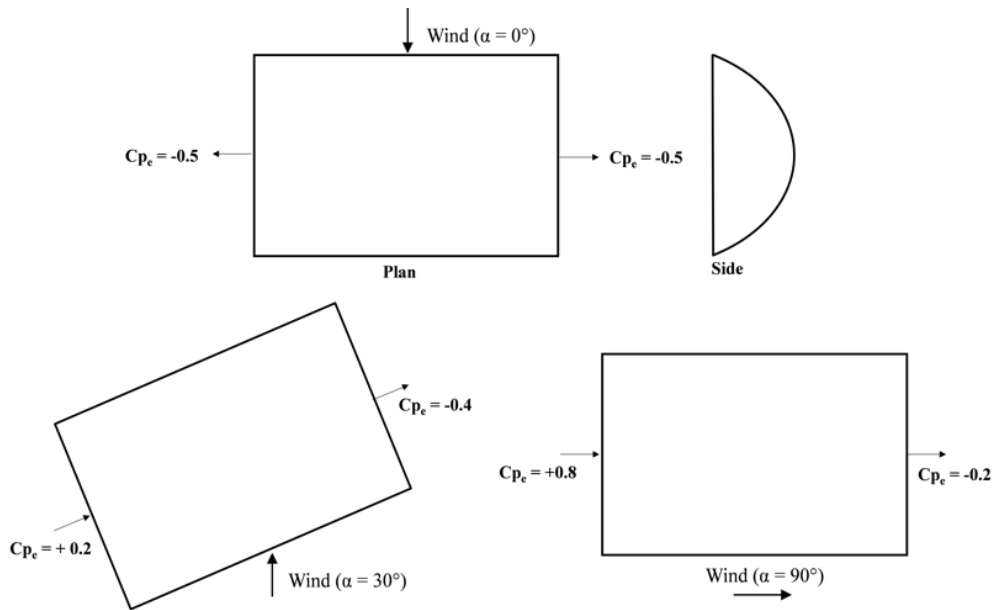


Figure 5.34 – External pressure coefficient: mid-height of sidewall (Bradley *et al.*, 2017a)

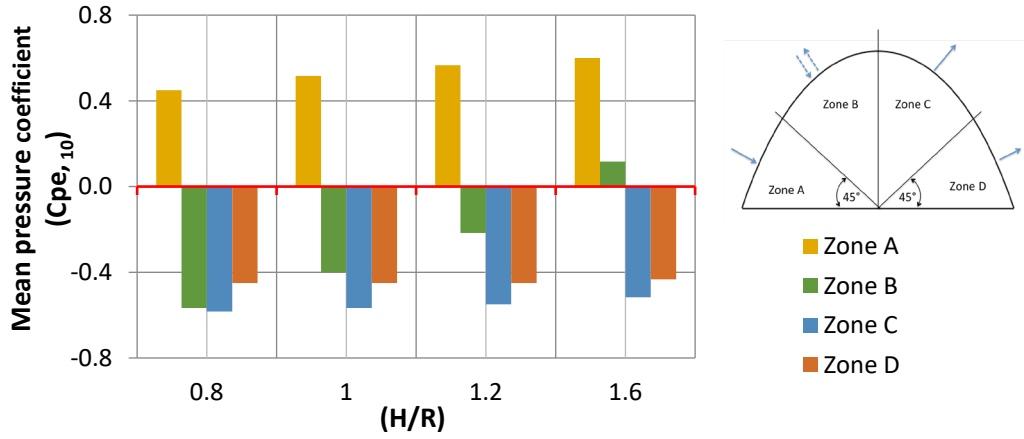


Figure 5.35 – Mean wind pressure coefficients ($\alpha = 0^\circ$) (Bradley *et al.*, 2017a)

The mean pressure coefficients at mid-height of the gable-walls, for the 0° , 30° and 90° wind orientations, are shown in Figure 5.34; these data correspond to a vault with height to base radius of 1.0. In principle, the full-scale gable walls will accommodate openings (i.e., doors and windows). This, in turn, implies a risk of generating the situation of *dominant* openings in the walls. A wall is considered as having a dominant opening, if the area of the openings in that wall is at least twice the area of openings and leakages in the remaining surfaces of the building (SANS 10160-3). In a building with walls having dominant openings, the internal pressure is taken as a fraction of the external pressure at the openings of the dominant wall, as given by Equation 5.9.

$$C_{p_i} = \begin{cases} 0.75C_{p_e} & \text{(if the area of openings in the dominant wall is twice the area of the openings in the remaining facades)} \\ 0.90C_{p_e} & \text{(if the area of openings in the dominant wall is at least three times the area of the openings in the remaining facades)} \end{cases} \quad (5.9)$$

where C_{p_e} is the external pressure coefficient at the opening in the dominant wall. If Equation 5.9 is applied to the experimental data from Bradley *et al.* (2016), substantial internal pressure may develop for certain wind orientations, as shown in Table 5.3. Figure 5.36 illustrates the net pressure distribution, for wind flow parallel to the gable walls, corresponding to an internal pressure of -0.45 (e.g., gable windows open). The plot indicates that a large net pressure may develop over the windward part of steeper shells. For buildings without a dominant opening, the internal pressure coefficient is calculated

using the height-to-depth (h/d) ratio of the building and the opening ratio (μ). However, if it is not possible (or deemed to be justified) to estimate μ , Cp_i should be taken as the more onerous of $+0.2$ and -0.3 (SANS 10160-3).

Table 5.3 – Internal wind pressure coefficients for *dominant* openings

Wind direction (α)	Wall	Cp_e^*	$Cp_i = 0.75Cp_e$	$Cp_i = 0.9Cp_e$
0°	both	-0.5	-0.38	-0.45
30°	windward	$+0.2$	$+0.15$	$+0.18$
	leeward	-0.4	-0.30	-0.36
90°	windward	$+0.8$	$+0.60$	$+0.72$
	leeward	-0.2	-0.15	-0.18

* Cp_e values taken from Bradley *et al.* (2016)

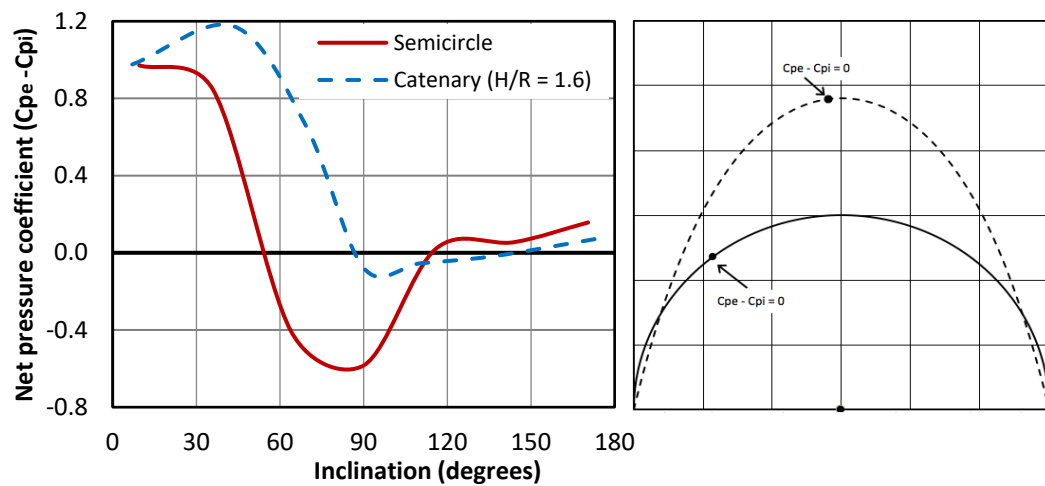


Figure 5.36 – Net centreline pressure coefficient distributions ($Cp_i = -0.45$)

Several load scenarios were considered to emphasize the importance of the correct consideration of wind loading for the design of unreinforced masonry catenary shells. Two load cases incorporating dead, live and wind loading, are used. Imposed distributed loads are derived from SANS 10160-2 for inaccessible roofs (i.e., Category H1 and H2). The surface loading (q_k), for a plan area greater than 15 m^2 , of 0.25 kN/m^2 is adopted. The most severe case for this distributed vertical load corresponds to its action over one half of the structure, specifically the windward half-section when wind and live loads are considered in combination. However, it is unreasonable to assume that extreme wind loads would act simultaneously with loads due to maintenance activities (i.e., very low probability of coincidence). This is addressed in EN 1990: 2002, which permits wind action not to be considered simultaneously with loads arising from maintenance activities. This is similarly

addressed in SANS 10160-1 (2018) through the load combination factors being set to zero.

Partial safety factors are taken from SANS 10160-1. Since dead loading is a favourable action for all load cases considered, a value of 0.9 is implemented in the current study. The wind pressures acting over the structures were determined using the coefficients derived in Bradley *et al.* (2016) in combination with the stipulations of SANS 10160-3 (2011). The structures were assumed to be in Cape Town; the design and site parameters are summarized in Table 5.4. For these analyses, the internal pressure coefficients of -0.3 and +0.2 were adopted because the various opening possibilities (with respect to position, size, and number) were not considered. The magnitude of wind loading depends on several site-specific conditions (e.g., terrain roughness, topography, and location / altitude). Wind loads could also conceivably be increased for structures built in coastal areas exposed to open sea, or large open water basins, where the smoothest category (i.e., Category A, as specified in SANS 10160-3) may be the *influencing* terrain category.

An elastic analysis, assuming pinned end-conditions, was selected to facilitate the comparison of the critical vertical and lateral design loadings over catenary roofs in the coastal regions of South Africa. The material and geometric properties for these FE models were as follows: Young's modulus = 6200 MPa, Poisson's ratio = 0.2, density = 1950 kg/m³, thickness = 250 mm, span = 6m, and length = 8 m. The height of the vaults varied between 2.4 and 6 meters (i.e., H/R between 0.8 and 2.0). Furthermore, the vaults were analysed without the support and restraint created by the gable-walls.

Table 5.4 – Design and site parameters derived from SANS 10160 (2011)

Design parameter	Units
Fundamental basic wind speed ($v_{b,0}$) =	28 m/s
Probability factor (C_{prob}) =	1
Topography factor ($c_o(z)$) =	1
Peak wind speed ($v_{b, peak}$) =	39.2 m/s
Air density (Cape Town) =	1.20 kg/m ³
Terrain category =	B

Figure 5.37 shows the relationship between aspect ratio (H/R) and the maxima bending moments, which were derived from all load combinations and critical wind orientations, corresponding to the vault size adopted in the present analysis (i.e. plan area of 48 m²). The

pressure distribution corresponding to $H/R = 2.0$, used in this comparison, was obtained through an extrapolation of the data given in Figure 5.35. The bending moments calculated for the vertical imposed loads do not vary with changes in aspect ratio; however, an exponential relationship for the bending moments is obtained for the wind load combination. In fact, considering the $H/R = 2.0$ vault, the bending moment calculated for the wind load case is approximately 27 times greater than that for the imposed vertical live load stipulated in SANS 10160. This outcome is attributed to the critical nature of the pressure distribution over vaults with high aspect ratios (i.e., H/R). The results presented in Figure 5.37 clearly indicate the significance of wind loading in the design of steep vaults. Furthermore, in order to reduce the amount of material, whilst still ensuring the thrust line remains within the middle-third of the section (i.e., no tensile stress), larger steep vaults may need to be pier-stiffened, corrugated, or have their cross-section varied along the shell.

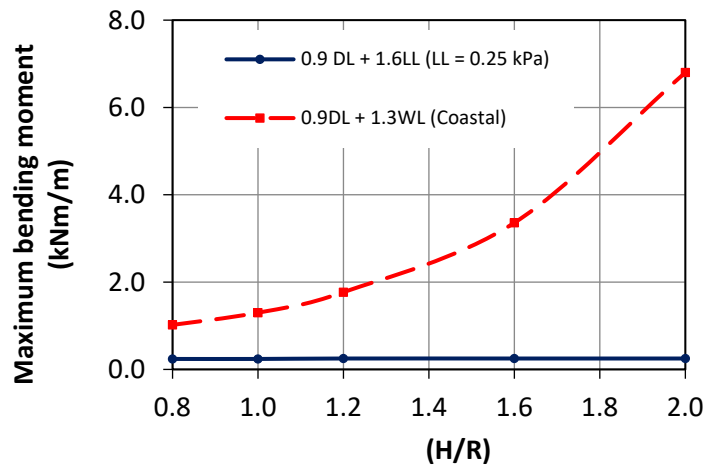


Figure 5.37 – Wind loads versus asymmetric distributed loads

5.3.3.5 Gable walls

An additional aspect is the design of the upwind gable wall under the action of wind pressure, as well as the leeward wall, if the situation of a dominant opening is generated. These walls must be designed as free-standing if control joints are included between the wall and shell. Furthermore, the inclusion of large windows and doors may significantly reduce the self-weight and consequently the resultant stabilizing effect of the gable wall. Whilst there are several methods of design of free-standing walls, if a dry-stack construction is to be implemented, then the wall could be designed for the rotation about one of the faces (at the base) or for pure compression only. However, ensuring a state of

pure compression for walls with flexural capacity will lead to an overly conservative design.

In order to improve the speed of construction and reduce the material costs, the gable walls of the Wits shells were built using dry-stack blocks. Furthermore, large windows were incorporated into several of the walls, as shown in Figures 5.38a and 5.38b. Consequently, pier stiffening was used to ensure stability, eliminating the need for an unreasonably thick wall. To illustrate the need for piers, a design established for rotation about the center of a compressive stress block, with offset width of 10 % of the thickness, at the base of the wall, is presented. The stabilizing and destabilizing moment forces are given by Equations 5.10 and 5.11, respectively.

$$\begin{aligned} M_S &= F_g x \\ &= (\gamma_g \cdot \delta_s \cdot t \cdot A) \cdot (0.45t) \end{aligned} \quad (5.10)$$

$$\begin{aligned} M_{DS} &= F_w y \\ &= (\gamma_w \cdot w_K \cdot A) \cdot (y) \end{aligned} \quad (5.11)$$

where x and y are the moment arm corresponding to the self-weight (F_g) and wind load (F_w) respectively; γ_g and γ_w are the partial load factors for dead and wind loads, respectively; δ_s is the unit weight of the masonry; t is the wall thickness; A is the projected surface area (against the wind) of the wall; and w_K is the wind pressure. The wind pressures were calculated using the procedures outlined in SANS 10160-3 in combination with the wind pressure coefficients from Figure 5.34. The term $0.45t$ corresponds to an “offset” width of 10% at the base. By equating Equations 5.10 and 5.11, the wall thickness (t) may be expressed as follows:

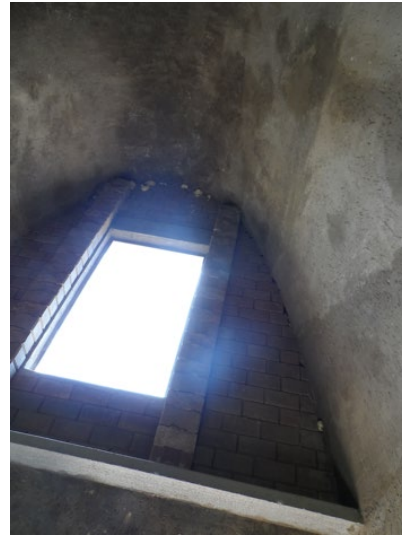
$$t = \sqrt{\frac{\gamma_w w_K y}{0.45 \gamma_g \delta_s}} \quad (5.12)$$

Assuming the earthen brickwork has unit weight of 19 kN/m^3 , $\gamma_g = 0.9$, and $\gamma_w = 1.2$, the required wall thickness is given by Equation 5.13.

$$t = 0.0125 \sqrt{w_K y} \quad (5.13)$$



(a) west-facing wall



(b) east-facing wall

Figure 5.38 – Pier-stiffened freestanding gable walls

The results of the analysis are shown in Figure 5.39. The geometry of the gable walls corresponds to the vaults adopted throughout this chapter; i.e., a plan area of 48m^2 . Substantial mass (or thickness) is required to ensure stability of the walls. However, instead of requiring unreasonably thick walls, the walls could be designed to resist flexure or stiffening elements, such as piers, could be used. The end walls could also be built with other materials and/or systems; e.g., timber, steel or aluminum framed windows. An introduction of such elements can increase the costs, which may be considered as being inconsistent with the objective of utilizing inexpensive and sustainable materials.

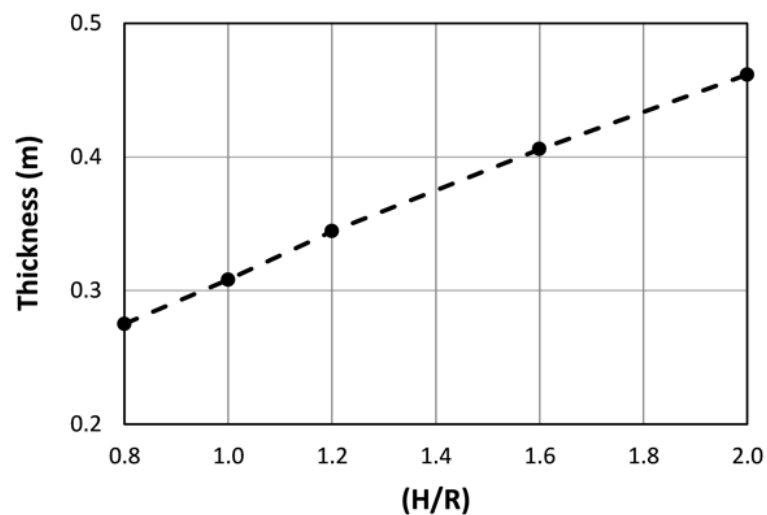


Figure 5.39 – Required gable wall thickness

5.4 Conclusions

Several loading considerations and principles of good practice were presented in this chapter. These are primarily based on physical investigations, experimentation as well as through the assessment of the related loading stipulations presented in the literature. These recommendations are mainly made in reference to the free-spanning catenary vaults; although many may be extended to unreinforced masonry shells in general. The main conclusions are as follows:

- The minimum loading (UDL) stipulations for *inaccessible* (or *ordinary*) roofs will not govern the design of steep catenary vaults. These vertical loads, which are usually assumed uniformly distributed, are likely overly conservative. Of the loading codes considered in this thesis, only the BS EN1991-1-1 considers the influence of the roof slope on load magnitude and feasible load distribution;
- Earthen masonry roofs should be designed as *inaccessible*, which may be achieved by limiting access onto the shell and adopting steeper shell geometries that inhibit the potential placement of large vertical loads.
- The possibility of designers adopting ultra-thin sections toward the apex of shells, which inherently have the highest probability of being exposed to concentrated vertical loads (e.g. during maintenance work or additions to a roof structure, such as solar water heaters), must be considered during design.
- Impact loading (such as trees or vehicles) may have a significant effect on the types of low-rise unreinforced thin-shells. Earth materials are inherently brittle and exhibit limited ductility (when compared to reinforced concrete/masonry shells); consequently, if these types of loads are not appropriately recognized (and accounted for in the design where applicable), the risk of damage, or even total collapse of the structure, is increased. In certain regions, such as those exposed to hurricanes, designers may have to incorporate reinforcement to minimise damage associated with wind-borne debris;
- As unreinforced shells become steeper, internal bending forces, due to wind loading, increase exponentially. Moreover, the nature of the wind pressure distribution over steep catenary shells, for wind blowing parallel to the gable-walls, leads to a critical loading situation (i.e., pressure over the windward face and suction over the leeward half), which may govern design;

- The 90° wind orientation (i.e., wind blowing onto the gable wall) is, in general, not critical for catenary roofs. This wind orientation does, however, affect the stability of the upwind gable wall and the leeward wall if the situation of a dominant opening can exist. This aspect requires special attention if a control joint, to prevent cracking damage, is incorporated between the walls and shell.

CHAPTER 6. LOADING CONSIDERATIONS FOR EARTH MASONRY SHELLS: SELF-STRAINING LOADS

6.1 Introduction

Unreinforced masonry shells are normally designed with lower-bound (equilibrium) methodologies such as graphic statics, thrust line analysis, thrust network analysis, etc., to ensure that internal force resultants remain within the middle-third of the cross-section (i.e., no tensile stress). Actions such as self-weight, wind and asymmetric distributed loads are typically considered in design. Although a pure-compression design technique offers considerable safety against collapse, most structural damage (e.g., cracking) can be attributed to loads that appear to be regularly overlooked. In particular, extensive cracking caused by self-straining stresses, such as the restraint volumetric changes, is observed in several unreinforced masonry shells built in South Africa. The mitigation of cracks is important as they are the most common cause of a masonry structure's failure to perform as envisaged. This is especially significant in adobe and CSEB shells because of the modest durability characteristics of these building materials. In this chapter, the common causes of cracking in unreinforced shells are reported and, subsequently, the importance of self-straining stress is highlighted. The study demonstrates that unreinforced CSEB shells should be designed for a range of cracking factors. Several guidelines, applicable in a low-cost setting, are proposed to prevent and mitigate cracks on CSEB shells.

6.2 Cracking in earthen masonry shells

6.2.1 Characteristics of cracking

Structural cracks in earth masonry tend to go through both bricks and mortar, directly following lines of stress, because the strengths of these constituents are typically similar (Morton 2008). Morton (2008), Mukerji (1994) and Joffroy and Guillaud (1994) noted the benefit of adopting mortars that have similar compressive strength as the units, because such an approach gives a reasonably even strength to the masonry element. Such an approach is in contrast to general masonry practice, where the compressive strength of mortar is typically smaller than that of the units (Walker and Stace, 1997). For this reason, cracks typically occur within the mortar joints. Cracks also appear along the unit-mortar interface, and formation therein is dependent on bond rather than the relative strength of

mortar and brick. Stress concentrations may also align with the joints, resulting in large, relatively straight, cracks. This is illustrated in several fired clay brick catenary vaults in Bapong, as shown in Figure 6.1, where longitudinal cracks appear to coincide with the joints and/or along the unit-mortar interface. Although cracks are only visible in the overlying plaster, the location of cracking in the joints is supported by the appearance of lime run, as shown in Figure 6.1. Rain water entering the masonry through these cracks dissolves water-soluble calcium compounds (e.g., free lime) within the mortar, which subsequently runs down to the surface and reacts with carbon dioxide in the atmosphere to form white calcium carbonate deposit.



Figure 6.1 – Straight-line cracks in two of the Bapong shells (cracks traced for clarity)

If earth blocks are designed to meet the minimum design requirements, as would be expected, the strengths of masonry unit and mortar may be similar. Consequently, cracks will likely follow a direct path, possibly influenced by local weaknesses within the masonry and will not appear stepped (diagonal) or cogged (vertical) along joints. Figures 6.2a and 6.2b illustrate cracking through unit, mortar as well as along the unit-mortar interface in the Wits and Mapungubwe shells. Crack deviations shown in Figure 6.2a could possibly be attributed to local inconsistencies in block strength. Significant disparities in density and strength, within individual compressed stabilised blocks, is not uncommon due to uneven compaction during manufacturing. Furthermore, when soils are appropriated from the construction site, they may show variability in composition. This inconsistency could lead to large variance in strength, from one brick to another, which is supported by the large scatter reported in some experimental studies (Zak *et al.*, 2016).



(a) External crack (blocks)

(b) Internal crack (tiles)

Figure 6.2 – Cracking in earth masonry shells

6.2.2 Self-straining stresses

6.2.2.1 General

Self-straining loads arise from several sources, such as volumetric changes (associated with temperature change, shrinkage, moisture change and creep) and differential settlement. These loads, and the internal stresses that develop, are related to the characteristics of the structure and materials. This is dissimilar to applied actions, which are completely independent from the characteristics of the structure itself (fib, 2014). Self-straining stresses created by the restraint of thermal and/or shrinkage movement can be of considerable magnitude in certain types of structures and should be considered in a rational analysis. For example, reinforced concrete (RC) shells designed according to ACI 318-95 (ACI, 1995) must have a minimum steel reinforcement ratio of 0.0018 (A_s/A_c) to resist shrinkage and temperature changes in regions where compressive forces exist (e.g., in the upper region of semi-circular domes). In these regions, the shell may be only 50 mm thick, in shotcrete applications (Wilson, 2005). Ultra-thin RC domes, used for housing, will routinely incorporate insulation to reduce heat loss and gain, in winter and summer, respectively. Thus, thermal loading is justifiably ignored when RC domes are insulated. By adopting the two aforementioned design actions, i.e., minimum reinforcing and external insulation, temperature and shrinkage induced stresses are generally not of concern for the designers of RC shells used for housing.

By contrast, CSEB shells characteristically are not reinforced. Furthermore, the numerous unreinforced masonry, including CSEB, shells considered in this thesis lack insulation, which is partly attributed to:

- CSEB masonry having much lower thermal conductivity than standard concrete. Lamb (1998) determined the thermal conductivity of CSEB to be between 0.31 and 0.58 W/m.K; whereas, that for concrete is typically between 1.20 and 3.50 W/m.K (Ballim, 2008). For standard fired clay bricks the value is approximately 0.85 W/m.K (Lamb, 1998). Thus, CSEBs conduct heat more slowly than concrete and burnt clay bricks.
- Earth brick masonry shells are characteristically thicker than their ultra-thin RC counterparts, which provides additional insulation beyond that associated with the material characteristic noted above.
- Thermal mass. Good quality CSEBs have a high heat capacity, which is beneficial in regions that experience high diurnal temperature fluctuations because the shell will absorb heat during the day, keeping the indoor temperature relatively low, and then release it at night, by convection, when the temperature of the immediate air drops below that of the masonry.

Unreinforced and uninsulated masonry shells, especially those situated in the Highveld of South Africa, have experienced substantial cracking damage associated with self-straining stresses. Cracks are most pronounced within the external plaster coating and in the masonry near areas of restraint, e.g., near stiffened window openings, window frames, sudden geometrical changes, and near gable ends.

6.2.2.2 Support movement

Differential movement, due to settlement or heave, may cause the foundations of the structure to bend and possibly cracking of the supported masonry. All foundations will settle, which is not deemed a problem if the foundations move as a single unit. The appropriate choice of foundation and its structural design are critical in this regard. In addition to differential vertical settlement, lateral support movement is well documented as a serious contributor to cracking in masonry shell structures (Habitat, 1992; Norton, 1997; Theunynck, 1994; De la Rosa S, 2013). The effects of support movement on masonry vaulted structures are observed in historical constructions such as cathedrals, bridges and arches (Heyman, 1969; Heyman, 1995). In the case of arches and vaults, lateral support

movement inevitably results in cracking at the apex and abutments as depicted in Figure 6.3. Heyman (1969) noted, in reference to arches and vaults, that this ‘cracked state’ is safe, and may, in some cases, have a higher factor of safety against collapse than the original or undeformed structure. The arch then becomes statically determinate (i.e., the position of the thrust line is known at three locations/hinges).

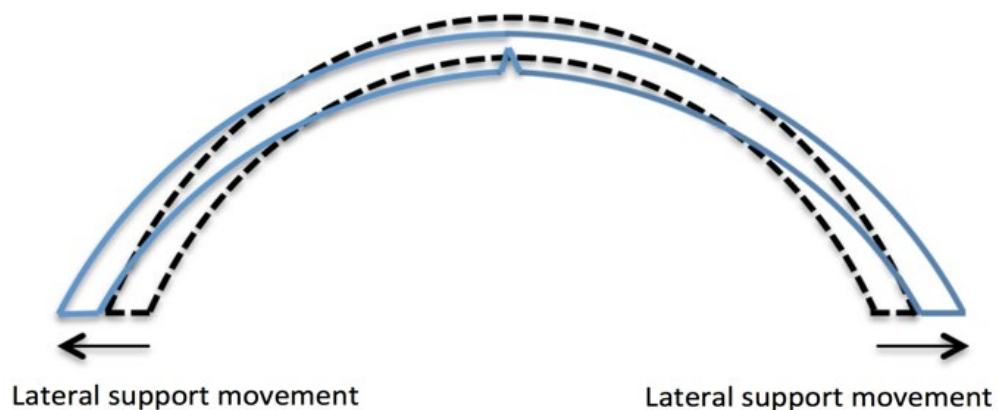


Figure 6.3 – Lateral support movement

However, the vault is only ‘safe’ for small deformations, and if the support displacements become excessive, collapse may result. Two adobe vaults at the Swan House & Lab collapsed due to excessive material deterioration at cracks that may have been caused by lateral spreading of the supports (De la Rosa, 2013). De la Rosa (2013) also noted that a third vault was at the brink of collapse, and the remaining vaults had a similar foreboding if left unrepaired. A vault, covering the kitchen, had several internal cracks as well as an external crack (about 3.2 mm wide and 50 mm deep) running along the apex (De la Rosa, 2013). A long internal crack, situated along the apex, is a clear sign of lateral spreading at the supports. This type of cracking is also observed in a long-vaulted structure at Sparrow Rainbow Village, as shown in Figure 6.4. De la Rosa (2013) reported repairs to the numerous cracks of the Swan House & Lab as well as remedial actions to prevent any further lateral spreading, such as adding two new buttresses to the kitchen’s vault and a metal strap ring around the base of a dome to accommodate the lateral kick-out force.



Figure 6.4 – Long apex crack in a semi-circular vault at the Sparrow Rainbow Village

6.2.2.3 Exposure of external rendering

Renders are commonly used to prevent water penetration into the underlying material. When a render or plaster is applied to the exterior and/or interior surface of a masonry shell, it will begin to dry, and consequently shrink, which introduces stresses. The effects of these stresses are visible through the various forms of deterioration at the surface. Surface cracking is one form of deterioration, and especially common when stiff renders are applied over earthen masonry. The difference in stiffness between mortar and render is presented in the literature as the primary cause of surface cracking (Joffroy and Guillaud, 1994; Talocchino, 2005). The nature of deterioration depends on the type of render adopted and the surface condition of the shell. Houben and Guillaud (1994) mentioned two typical forms of deterioration:

- Loss of adhesion or loosening of the render at the interface with the supporting surface due to it being too stiff and the supporting surface being too smooth.
- Cracking in a render that is too stiff if applied over a rough surface. Such cracking may be numerous and fine (i.e., crazing) or clearly visible and few, depending on the level of adhesion between the plaster and supporting masonry surface.

Beyond the quality and durability of the surface material, cracking of the render may cause the earthen masonry to deteriorate due to infiltration of water. The inherent exposure to wind, sun, and rain of masonry shell structures also contributes to the extent of cracking. For example, many of the CMU domes at Sparrow Rainbow village have extensive surface cracking on much of their external cement-sand renders, as shown in Figures 6.5a and 6.5b; whereas, far less cracking is observed on the internal renders even though both renders have the same constituent materials. Such observations are not limited to shells, i.e., cracking is more predominant when the plaster over standard masonry walling is exposed to the elements. Figures 6.6a and 6.6b depict the difference between the extent of cracking in an exposed plastered wall and a sheltered plastered wall.

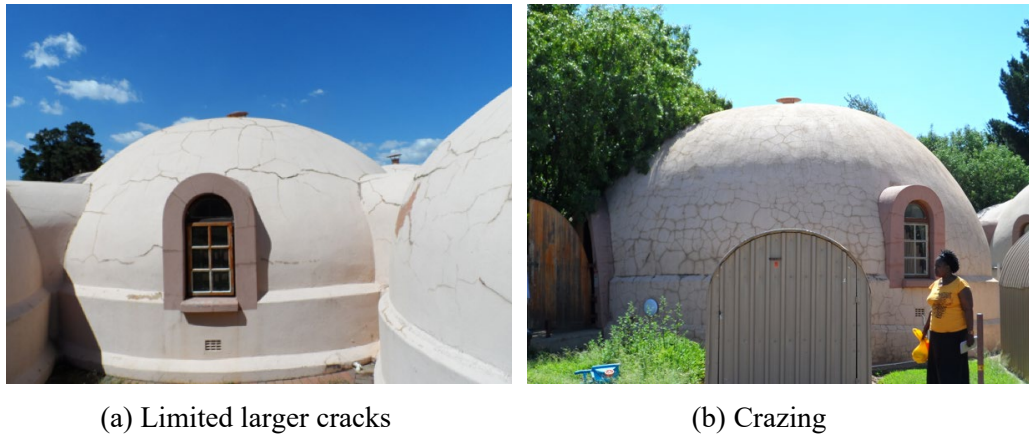


Figure 6.5 – Several CMU masonry domes at the Sparrow Rainbow Village

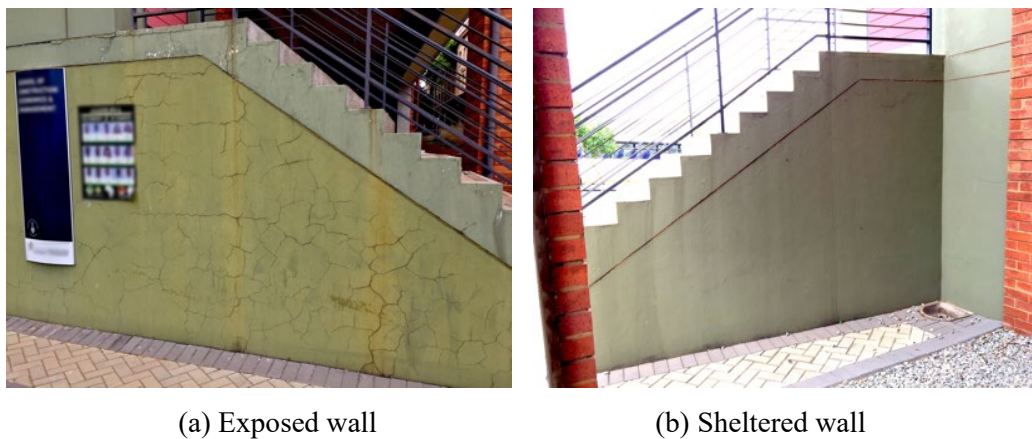


Figure 6.6 – Crazeing in exposed and sheltered plastered walls

Large surface temperature changes also contribute to the development of external tensile stress. Gohnert (2012) measured the external temperature induced strains over the CSEB dome shown in Figure 6.7. Measurements were made at ten locations along a north-south line at 7:00 AM and 3:30 PM over a five-day period in the winter. The measurements indicated that daily temperature strains were as high as $1500\mu\epsilon$ towards the apex on the north-facing portion of the shell. Gohnert (2012) reported that these measured strains exceeded the capacity of a standard cement-sand plaster, which is typically less than $1000\mu\epsilon$. Figure 6.7b shows the extensive surface cracking of the cement-sand plaster on the dome several years after construction (Bradley *et al.*, 2018a).



(a) After construction (Talocchino, 2005)



(b) Several years after construction

Figure 6.7 – CSEB masonry dome at Central Johannesburg College

Bradley *et al.* (2018a) also evaluated the surface temperature distributions over domed forms to establish the extent of diurnal and seasonal surface temperature fluctuations in Johannesburg. Several small models built with 25mm thick CSET were instrumented with 25 J-type thermocouples embedded into the masonry, and covered with a layer of cement-sand plaster. The surface temperature measurements indicated that peak diurnal variations frequently exceeded 30°C during winter and summer. Seasonal external surface temperature changes, between winter and summer, were in the order of 50°C . Like the full-scale dome examined by Gohnert (2012), extensive surface cracking was observed several years after construction, as shown in Figure 6.8.



(a) Model in 2012

(b) Model in 2016

Figure 6.8 – Small CSET dome

6.2.2.4 *Restraint associated with bond and friction*

The traditional methods adopted to manage self-straining stresses in masonry walls is through the inclusion of control joints and/or reinforcing. CMU masonry walls are typically built with bed joint reinforcement and/or vertical control joints. In longer CMU walls, bond beams are used in lieu of bed joint reinforcement (Grimm, 1988). Burnt clay brick masonry walls usually incorporate regularly spaced expansion joints to accommodate thermal and moisture-induced movement. Such joints are not regularly used in CMU masonry walls, because shrinkage is greater than thermal expansion. By contrast, earthen shells are typically constructed with no control measures to prevent or limit shrinkage cracking.

The typical types of mortar used in CSEB masonry construction are soil-cement, cement-sand and cement-lime-sand. Characteristically, the magnitude of drying shrinkage in these mortars is amplified when the amount of water and/or the amount of cement is increased. High-water content mortar, in particular, should be avoided or the masonry may experience excessive shrinkage (Houben and Guillaud, 1994). Furthermore, the type of soil used in soil-cement and unstabilized mortars has a significant effect on shrinkage. Shrinkage increases as clay content increases (Walker, 1995). Bricks manufactured using soils with high clay content may also experience excessive shrinkage resulting in large cracks whilst curing. Some studies have shown that drying shrinkage in earth blocks may exceed typical limiting values, such as 0.1% adopted for concrete masonry units (Walker, 1995; Walker and Stace, 1996).

In long CMU walls, vertical shrinkage cracks occur at mid-length or at fairly regular intervals (Grimm, 1988). The cause of these cracks is the prevention of shrinkage at the base of the wall because of bond and friction. Vertical shrinkage cracks are more likely to occur in regions of greater exposure, such as wing walls and parapets (Rainer, 1983). Vertical shrinkage cracks in CMU walls have also been attributed to the restraint created by vertical elements, such as piers, and are especially prevalent near re-entrant corners (Grimm, 1988). Similar to wing walls and parapets, masonry shells are inherently exposed to wind, wind-driven rain, and solar radiation. Several vertical shrinkage cracks were observed in the two steep catenary vaults built at the University of the Witwatersrand, as shown in Figure 6.9.



Figure 6.9 – Vertical shrinkage cracks in a CSEB vault (Bradley *et al.*, 2017a)

The masonry comprises CSEBs, produced with a sandy soil and 10% cement, in a standard cement-sand mortar (1:4). The designers adopted the sliding formwork technique (Gohnert *et al.*, 2018) in which the vault is constructed by building a series of arches connected by bonding the brickwork. Vertical cracks occurred where segments were keyed together because the preceding arch segment shrunk prior to construction of the subsequent segment, generating restraint against the shrinkage of the new segment (Bradley *et al.*, 2017a).

6.2.2.5 Restraint at locations of localised shell stiffening

Concentrated stresses associated with restraint typically occur near foundations; stiffened openings, such as arches around windows and doors; and other connections. Beyond the in-situ shrinkage of bricks and mortar, temperature variation is also responsible for volumetric changes and self-straining loads when shells are restrained. Due to their exposure, thin-shell structures act like a living lung: expanding and contracting, depending on the thermal exposure. If the materials used in the construction of a shell have similar coefficients of thermal expansion, cracking should generally not occur. However, complex temperature distributions and restrictions of thermal movement (i.e., fixity) may lead to stress concentrations. Furthermore, the differences in thermal conductivity and coefficient of linear expansion and physical characteristics of different components may lead to differential temperature and movement. Volume changes due to temperature differences between a thin reinforced concrete shell and ring-beam can result in large tensile hoop stresses at the connection between these elements (Wilson, 2005).

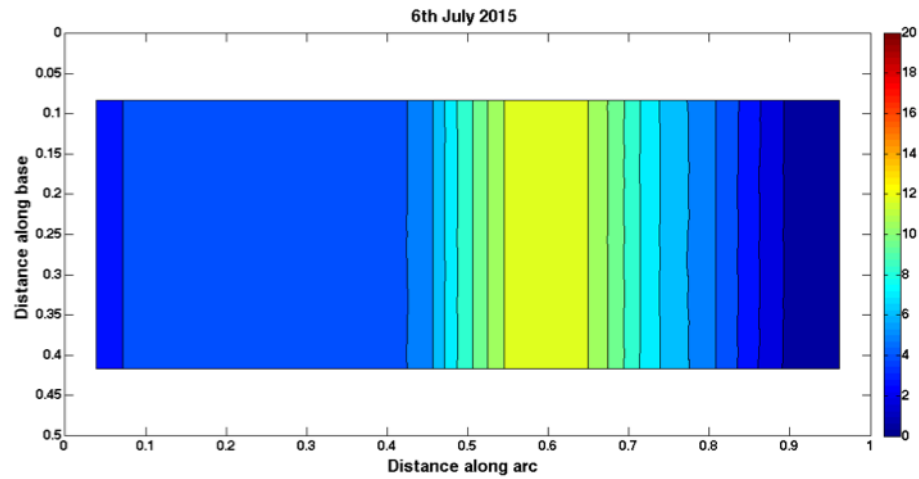
Wilson (2005) noted that a 6°C temperature drop will produce a large hoop tension and bending moment more than three times the gravity-load moment on a fixed base RC dome. The author noted that actual temperature drops in some locations can be as high as 40°C. Consequently, the stresses associated with such temperature drops may well control the design of a shell. In fact, large and sudden temperature changes, associated with thunderstorms, have caused the collapse of one RC dome just after construction and another dome after several years of service (Wilson, 2005). In both cases, collapse was preceded by a rapid and large temperature drop due to heavy rainstorms. The uninsulated dome that collapsed prior to being placed into service experienced an internal temperature change of at least 25°C, in just a 30-minute period, which was deemed as the main reason for the failure (Wilson, 2005).

Shells built in South Africa are characteristically uninsulated, and potentially exposed to large rises and falls in temperature over only a few hours. Solar radiation is absorbed at the surface, which generates heat that is transferred by thermal conduction through the masonry. The sun's position in the sky, as well as the inclination of the structure's surfaces, affects the intensity of radiation received. During the winter in South Africa, north facing surfaces will experience the highest level of solar radiation while the south facing portion receives the least. In summer, when the sun's elevation is much higher, the entire shell will

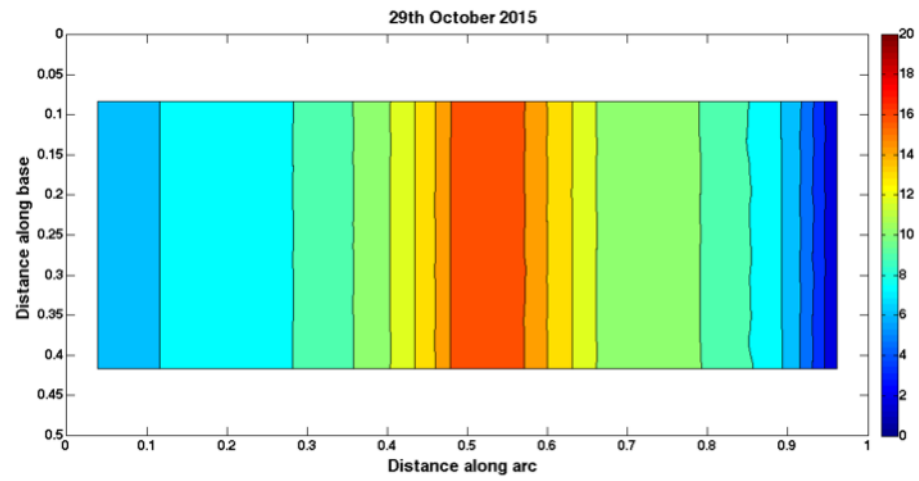
be heated by direct solar radiation for a significant part of the day. Bradley *et al.*, (2018a) assessed the magnitude of surface temperature fluctuations for one of the steep catenary earthen shells built at the University of the Witwatersrand. Measurements were made with thermocouples, which were embedded into the rendering on the internal and external surfaces of the shell. The steep geometry and orientation of these shells caused unique temperature distributions and gradients across the shell. Figures 6.10a through 6.10c show the diurnal temperature range for an instrumented section of the internal surface of a vault on a winter, spring and summer day. The left and right edges of these contour plots correspond to the south and north base of the vault, and the distance values along the horizontal and vertical axes have been normalised by the total arclength and width of the vault, respectively.

In the winter, the north-facing vault shades part of the south-facing vault, as shown in Figure 6.11; the south-facing portions of both vaults remain largely shaded throughout the day, apart from a small region near the apex. Figure 6.10a shows the maximum diurnal temperature changes at the internal measurement points on the 6 July 2015 (mid-winter). The effects of shading are clearly apparent, where diurnal internal surface temperature changes are less than 5°C in these parts of the shell. These diurnal variations approach 15°C towards the sunlit apex on the north-facing portion of the vault. Measurements made with a hand-held infrared (IR) thermometer on the northern vault show that its north-facing surface, which is exposed to the sun through the day, experiences fairly uniform temperature changes.

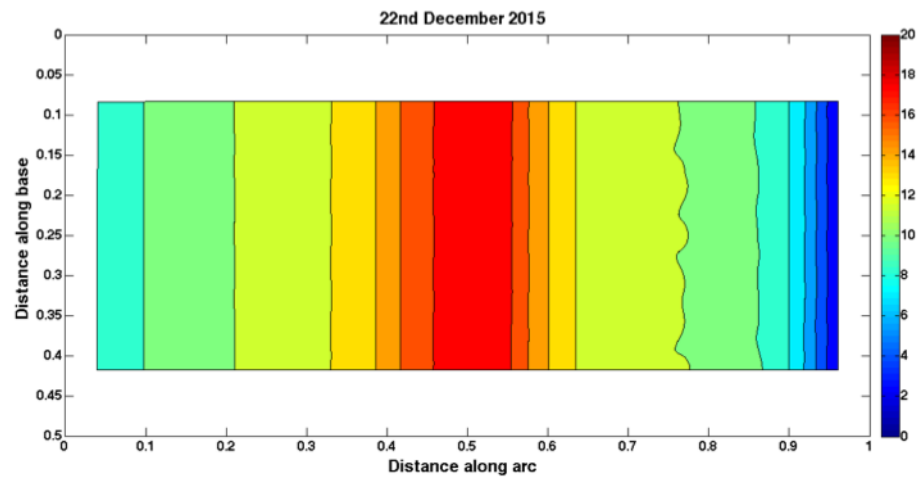
Throughout much of the day in the summer, both vaults are completely bathed in sunshine, as shown in Figure 6.12a and 6.12b. Figure 6.10b and 6.10c show the maximum diurnal temperature changes of the south-facing vault on 29 October (early summer) and 22 December 2015 (summer solstice). Maximum internal temperature variations of approximately 20°C occurred in the apex in mid-summer, which is the flattest portion of the shell. The lowest diurnal fluctuations were observed at the base, where the shell is steeper than 70° to the horizontal. The temperature changes are particularly small at the north-facing base because of the concrete infill between the base of the vaults, as shown in Figure 6.11.



(a) Mid-winter



(b) Late spring – early summer



(c) Mid-summer

Figure 6.10 – Diurnal temperature range on several days in 2015



(a) House from the north-west



(b) House from the south-west

Figure 6.11 – CSEB shell house: Winter 2015



(a) House from the north-west



(b) House from the south-west

Figure 6.12 – CSEB shell house: Summer 2015

The diurnal temperature changes for the sunlit external surfaces were in the order of 30°C in winter and summer (Bradley *et al.*, 2018a). Temperature differences were considerably larger between summer and winter, when they approach 30°C for the interior and 50°C for the exterior. These variations are significant and attributable to the local climate, intensity of solar radiation received at the external surface, and thickness of the exposed masonry section.

Internal air temperature and surface temperatures were also recorded during several thunderstorms. In Johannesburg, thunderstorms can be intense but brief and are usually followed by the rapid dissipation of clouds. Figures 6.13a and 6.13b show the internal and external surface temperatures, measured over two separate 24hr periods in which a thunderstorm was observed (29th Oct. 2015 and 21st Dec. 2015). The measurements correspond to a position near the apex of the shell, as shown in Figure 6.14. The surface

temperatures corresponding to a comparable day of the year with no thunderstorm (21st Oct. 2015 and 30th Dec. 2015), have also been included in Figures 6.14a and 6.14b. Although the external surface temperature dropped rapidly by approximately 25°C during these brief thunderstorms, there is a lag-time between 1 and 2hrs before the internal surface temperature begins to decrease. Furthermore, the thermal gradient across the shell is reduced, not increased, after the storm. These observations can be attributed to the thickness of shell, which is approximately 170 mm with plaster finish, and low thermal conductivity of the masonry material. Thus, these sudden temperature drops during thunderstorms smaller than typical diurnal and seasonal temperature fluctuations.

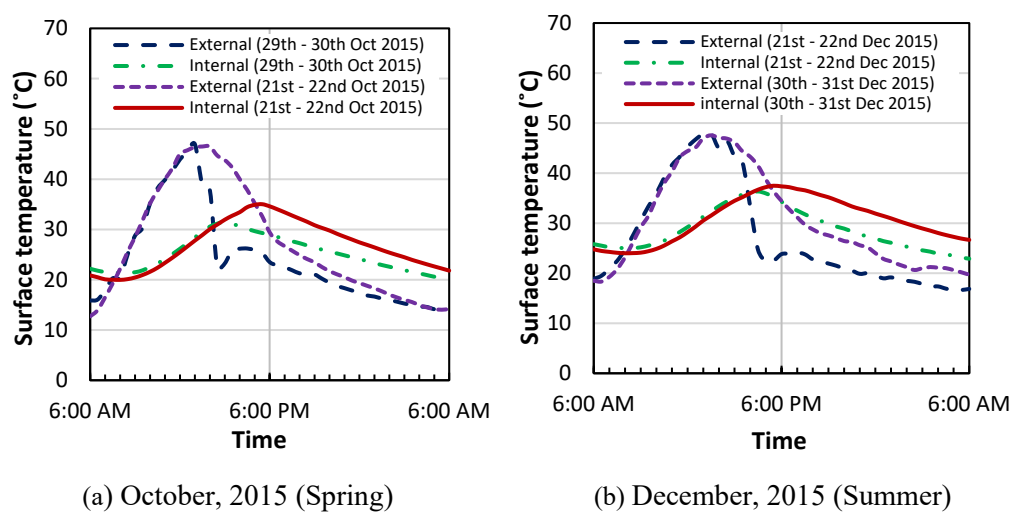


Figure 6.13 – Diurnal surface temperature progression



Figure 6.14 – Thermocouples attached to the internal surface of the south-facing shell

Establishing realistic temperature load parameters can be challenging due to the complexity of time-dependent temperature changes through the shell thickness and over the envelope. Consequently, conservative temperature distributions and fluctuations are normally considered in design. Such load combinations are hypothetically carried out on structures at an ambient temperature, say 20°C, about which the determined positive or negative temperatures are applied to the element or structure (fib, 2014). Bulovic (2014) and Gohnert (2012) considered finite element (FE) analysis to determine the magnitude and locations of thermally induced stresses in the CSEB vaults and dome built at Wits and Central Johannesburg College, respectively. The authors adopted a uniform temperature change (rise and fall) of 20°C through the shell thickness, which was applied to half and to the entire shell structure to model conditions in winter and summer. The temperature changes shown in Figure 6.10 indicate that the value of 20°C is not unrealistic for Johannesburg. This temperature change was applied to the shells without consideration of the actual stress-state of the shells, which is influenced by environmental conditions during construction; e.g., humidity and temperature, shrinkage, support movement, etc.

The results of the FE analyses showed that significant tensile stresses, occasionally beyond the capacity of typical CSEB masonry, were concentrated at the base of each shell. In the models, thermal contraction and expansion were restrained due to the fixity associated with either a pinned, ring-beam or fixed base connection. Gohnert and Talocchino (2011) used a damp-roof course (DPC) at the base of the CSEB dome not only to prevent moisture ingress but also to facilitate free expansion and contraction, which is beneficial in relieving the thermal stresses.

Errors and mistakes during either design or construction have led to cracking at several of these previously identified critical regions in the CSEB dome built in Johannesburg. Internal horizontal cracks exist in the CSEB dome, as well as in several CMU domes at Sparrow Rainbow Village, at positions where there is a sudden change in shell thickness (and stiffness) near the base. Figures 6.15a and 6.16a show these abrupt changes in thickness, which are circled for clarity, and Figures 6.15b and 6.16b show the associated internal cracks. Limited shrinkage cracks within the plaster are also visible in Figure 6.16b.

Sudden change in stiffness also causes cracks to occur. Figures 6.17a and 6.17b show a large horizontal crack in a masonry half-dome in the town of Paternoster in the Western

Cape, South Africa. The crack occurred where an internal element is attached to the shell. Cracks have also been observed at rigid shell connections, as shown in Figure 6.18.



(a) Sudden change in shell thickness (b) Internal horizontal crack (traced for clarity)

Figure 6.15 – CSEB dome (Central Johannesburg College)



(a) Sudden change in shell thickness (b) Internal horizontal crack (traced for clarity)

Figure 6.16 – CMU dome (Sparrow Rainbow Village)



(a) From the north

(b) From the east

Figure 6.17 – Horizontal crack in a masonry dome, Paternoster, South Africa (crack traced for clarity)



(a) Rigid shell connection

(b) Internal crack at the rigid connection

Figure 6.18 – Mapungubwe shells (Internal crack traced for clarity)

6.2.2.5 *Restraint generated by secondary elements: internal walls and window frames*

There are numerous instances of cracking near gable walls and window frames in several shells built in the inland provinces of South Africa. The earth-tile shells that comprise the Mapungubwe Interpretative Centre in Limpopo Province, South Africa, have cracked near several regions of restraint. The cracks, shown in Figure 6.19, are in the vicinity of almost all windows and are consistently at the highest point of the opening. It is postulated that these cracks have formed due to the restraint created by the frames of the windows since these elements prevent contraction (thermal and shrinkage) of the shell. Figure 6.20a shows an abandoned burnt clay brick catenary vaults in the town of Bapong (North-West Province, South Africa). The end walls are connected firmly against the shell, preventing contraction. Similar to the Mapungubwe shells, cracking in these shells also occurred at the highest point where the gable wall meets the shell. Although, these apex cracks may be due to lateral spreading of the supports; they are limited to areas close to walls or window frames. Furthermore, no cracks exist near the base of these shells. Such cracks have occurred in multiple shells at Bapong and Mapungubwe. These case studies, as well as the literature on the design of unreinforced shells (Auroville Earth Institute, n.d.; Joffroy and Guillaud, 1994; Allen and Zalewski, 2010; Dahmen and Ochsendorf, 2012; Gohnert *et al.*, 2018), indicate that the designers of unreinforced masonry shells rarely incorporate control joints and sliding connections in areas that restrain volumetric changes.



(a) Window frames fixed against the earthen shells



(b) Cracking above window openings

Figure 6.19 – Cracking in the Mapungubwe shells (cracks are traced for clarity)



(a) Solid wall-shell connection

(b) Cracking at the high point

Figure 6.20– Cracks in the Bapong shells (crack is traced for clarity)

A schematic strain plot, shown in Figure 6.21, is postulated in order to explain the observed cracking at the high-points in these masonry shells. The left and right boundaries of the strain plot correspond to the supports, whereas the mid-point represents the apex of the

shell. The black solid line 2 represents the variation in compressive strains in the shell due to gravity loads. If the shell is restrained against contraction along its entire length due to an internal wall, the shell will be subjected to tension strains as indicated by line 1. The resulting overall strains (compressive and tensile strains combined) are shown by the dotted line (labelled as 1+2). If the tensile strains are greater than the cracking strain of the shell material, cracking most likely will occur in the apex, which is the location of the cracks above the window frames in the Mapungubwe shells and above the end walls of Bapong.

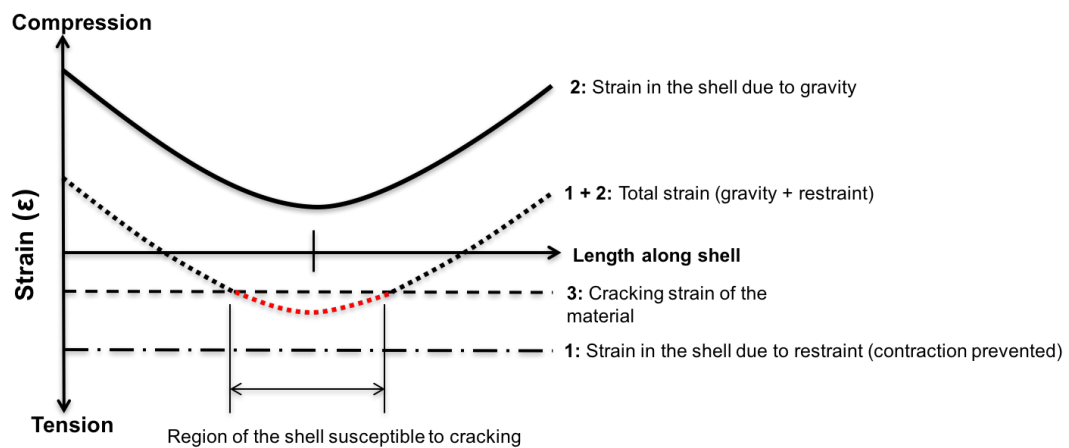


Figure 6.21 – Strains in the shell due to gravity and restraint

The designers of the Wits vaults were aware of the potential cracking associated with restraint created by rigidly attached gable walls; thus, during construction an expandable foam was sprayed between the shell and wall to facilitate differential movement (Gohnert *et al.*, 2018). The designers of the CSEB dome built at Central Johannesburg College, determined that tensile stresses were also concentrated around the stiffened window and door openings. Consequently, steel wire was stitched within the masonry, above the openings in the CSEB shell, to accommodate these stresses (Gohnert and Talocchino, 2011).

Although control joints were incorporated between the vaults and non-load bearing gable-walls at Wits, the required tolerance was not monitored throughout construction. Also, because one of the gable-walls was built up against the shell and the foam proved to be fairly stiff after setting the same tell-tale crack occurred as shown in Figure 6.22a. The other three gable walls were built with a sufficient gap between the wall and shell, and no cracking has occurred as shown in Figure 6.22b. The Wits shells were built during the

summer of 2013 and the apex crack only became visible several months later in the winter of 2014. Most likely the restraint of shrinkage and thermal contraction also contributed to the observed cracking. Cracking was also observed above a few of the stiffened openings in the CJC dome because the steel window frames were rigidly attached to the shell, as shown in Figure 6.23. These case studies clearly highlight the importance of correctly designed and installed control joints in preventing cracking in unreinforced CSEB shells.



(a) Wall built up against the shell

(b) Large gap between wall and shell

Figure 6.22 – Gable ends of the Wits shells



Figure 6.23 – Crack above window frame in a CSEB dome at CJC

6.2.2.7 Restraint generated by rigid formwork

Certain mortars, e.g. unstabilised and soil-cement, in combination with lightweight bricks or tiles may facilitate free-hand construction of masonry shells. However, when building CSEB shells with traditional mortars, formwork is usually required to give time for the mortar to gain strength and set. Rigid formwork, however, can restrain contraction as the mortar sets. Cracking damage associated with the prevention of shrinkage by rigid formwork was observed in several models as well as in one of the full-scale CSEB masonry vaults built at the University of the Witwatersrand (Bradley *et al.*, 2017a). These cracks consistently appeared toward the apex (or high point) of the shell.

6.3 Measures to mitigate cracking

6.3.1 Foundations and lateral force resisting elements

Typically, masonry walls are supported by strip foundations. Gohnert and Talocchino (2011) and Bradley *et al.* (2017a) recommend the adoption of vertically orientated foundations for CSEB shells. These deeper foundations are stiffer and better resist movement associated with differential settlement. The cost associated with using a vertically orientated foundation can be offset by making it narrower than standard strip foundations. Because CSEB shells are relatively thin, the magnitude of forces transferred to foundations, and supporting soil, are typically smaller than that for a load-bearing wall supporting a standard roof; typical loadbearing CMU walls may be twice as thick as a masonry vault or dome. The foundation width, however, must be sufficient for the soil bearing strength.

The correct design and construction of foundations and lateral force resisting elements are critical to mitigate structural cracking and collapse. Even minor support movement can cause cracking and subsequent redistribution of forces in a masonry shell. If cracking is to be prevented, the additional supporting structural system (i.e., buttress, ring beam, steel ties or piers) must be designed properly to ensure the base of the vault will not spread laterally. Lateral thrusts at the base of one vault may be counteracted by the base of another if they are built one adjacent to the other. Numerous vaults erected at Bapong utilise this juxtaposition to resist these kick-out forces at the base, as shown in Figure 6.24.



Figure 6.24 – Juxtaposition of vaults at Bapong, South Africa

6.3.2 Materials control

6.3.2.1 *Reducing drying shrinkage*

Masonry shells used for housing typically include minimal control measures to limit shrinkage and/or thermally induced stresses. CSEB shells, however, characteristically incorporate no steel reinforcement; thus, shrinkage of bricks and mortar can be minimised by considering the; 1) use of appropriate soils to produce bricks and mortar, 2) duration of brick curing, 3) shrinkage characteristics of the mortar, and 4) fiber addition.

Bricks, blocks and tiles should be cured to ensure that they do not shrink excessively after the shell is constructed. CSEB units should be cured for at least 28 days. However, blocks with excessive clay content have a higher water demand during manufacturing, and may experience significant shrinkage well after 28 days. The Auroville Earth Institute (n.d.) recommends curing CSEB blocks for four months before use to mitigate the potential of shrinkage cracking. In South Africa, CSEBs are sometimes used after only 14 days, when sufficient strength has developed. If CSEB units are used prematurely, without adequate control measures, shrinkage may be much greater than that observed in standard CMU masonry. In these situations, the combined shrinkage of units and mortar may cause cracking, especially near regions of restraint.

If the clay content in the soil used to manufacture bricks is limited and curing time is adequate, then, nearly all of the shrinkage can be limited to the joints. According to Morton

(2008), linear shrinkage of earthen mortar should be between 1% and 5%. Some national standards provide limits on linear shrinkage, for example, the New Zealand Standard 4298 (1998) stipulates a linear shrinkage limit of 3% for earthen mortars. The shrinkage characteristics of mortar and unit can be improved by adding coarse sand to reduce the percentage of clay (Walker, 1995). According to Joffroy and Guillaud (1994), a minimum amount of binder component should be used to avoid the risk of shrinkage during drying out after building work. The Auroville Earth Institute (nd) recommend using a thin layer of stabilized mortar (soil-cement or soil-lime), which is more like a glue, between CSEB units to limit shrinkage. When using standard CMU masonry, it is best practice to use mortar that is weaker and less stiff than the brick in order to better accommodate drying shrinkage. The National Building Research Institute (NBRI, 1987) recommends a similar approach to earthen masonry; namely, the mortar adopted should be weaker than the earth bricks to prevent the formation of large unattractive cracks, such as those in the Wits shells, as shown in Figure 6.9. The discrepancies between the reported mortar-brick strength ratios, i.e., a weaker mortar versus mortar with similar strength to the brick, reported in the literature highlight the importance of the correct consideration of possible issues with a particular mortar type.

Fiber reinforcing, such as straw, has been used extensively in vernacular earthen masonry to reduce shrinkage and micro-cracking in units and mortar. Parisi *et al.* (2015), Galán-Marín *et al.* (2010), Yetgin *et al.* (2006), and Bouhicha *et al.* (2005) discussed the benefits of straw fibre reinforcement in earthen building units; however, fibers need to be very fine when used in mortars (Morton, 2008).

6.3.2.2 Improving strength

There are various methodologies to enhance the strength and durability of earthen building materials, such as appropriate soil selection and stabilization (Chandra S, 2003; Houben and Guillaud, 1994). Typically, CSEBs are produced by incorporating cement and lime due to their relatively low cost (Adam and Agib, 2001). More recently, stabilizers with lower embodied energy have been investigated, such as natural polymers (Achenza *et al.*, 2006), geopolymer cements (Ojo *et al.*, 2017), waste materials (Gutiérrez-Orrego *et al.*, 2016), and natural fiber reinforcement (Sharma *et al.*, 2016).

The inclusion of natural fibers within CSEB shells is a low-cost option to mitigate and control cracking associated with self-straining stresses. Gohnert and Magaia (2003) used sisal fiber reinforced plaster and masonry in the construction of a low-cost CSEB dome house in Maputo, Mozambique. Natural fiber inclusion, however, does not significantly influence the elastic response of adobe and CSEB (Aymerich *et al.*, 2012; Aymerich *et al.*, 2016), since most natural fibers have low stiffness. Several studies have shown that fibrous reinforcement can increase the flexural/tensile capacity of earthen building units (Gutiérrez-Orrego *et al.*, 2016; Bouhicha *et al.*, 2005; Galán-Marín *et al.*, 2010; Aymerich *et al.*, 2016; Aymerich *et al.*, 2012). Furthermore, fiber inclusion improves ductility and post-cracking characteristics (e.g., energy absorption); these benefits only become apparent when the crack width is large enough to facilitate strengthening through bridging and frictional fiber pull-out (Aymerich *et al.*, 2016).

Several studies report either marginal or no influence of fiber addition on tensile and flexural capacity (Mesbah *et al.*, 2004; Fernandez, 2003). Furthermore, the dimensional changes of many natural fibers due to moisture, may significantly influence the adhesion between matrix and fiber (Fernandez, 2003; Ghavami *et al.*, 1999; Gutiérrez-Orrego *et al.*, 2016). The reduced bond can negatively influence the mechanical properties of the earth-fiber composite. Yetgin *et al.* (2006) reported that the tensile strength of earth reinforced with straw decreased because the fibers were soft and bond between the fibers and soil matrix was weak. The inclusion of some natural fibers may also reduce the density of earthen blocks, due to higher water demand during block production, causing a reduction in strength (Zak *et al.*, 2016; Yetgin *et al.*, 2006).

The stiffness, composition, length, and aspect ratio of fibers have an influence on the effectiveness of a fiber-reinforced composite. The fiber-matrix bond, which depends, in part, on the surface roughness of the fiber, is also of significance (Aymerich *et al.*, 2016; Gutiérrez-Orrego *et al.*, 2016). Several authors also reported that the material properties (plasticity index, linear shrinkage, clay content, particle grading, etc.) of soils also influence the successful use of a fiber (Fernandez, 2003; Bradley *et al.*, 2017b). Fernandez suggested that soils need to contain sufficient fines (i.e., silt and clay) to act as binders for fibers. Thus, it is likely that chemical and mechanical testing, prior to use, would be required to ensure that the fiber and soil are compatible, especially noting the variability in performance of certain earth-fiber composites.

6.3.3 Renders and membranes

Cracking of the cement-sand render have been observed on several masonry domes located at the Sparrow Rainbow Village and Central Johannesburg College (Johannesburg, South Africa). If such renders are used over earthen shells, then additional elements such as fiber-reinforcing or wire mesh should be used within the render to accommodate shrinkage and/or tensile stress. Steel reinforcing mesh and sisal reinforcing fibers were employed in the earthen shells discussed in Gohnert and Talocchino (2011) and Gohnert and Magaia (2004). An alternative to incorporating wire mesh is the use of waterproofing fabric membranes; although, these may be too expensive for the use in low-cost construction. A waterproofing membrane was applied on several of the domes at the Sparrow Rainbow Village, with great success, to prevent moisture ingress, as shown in Figure 6.25.



Figure 6.25 – Waterproofing membrane over a hemispherical dome

6.3.4 Control joints

Thin masonry shells can experience cracking when inadvertently restrained against small movements resulting from variations in temperature and shrinkage. Self-straining stresses in masonry walls are typically mitigated through the inclusion of control joints that allow for expansion and contraction. Steel, or other reinforcing materials, can also be integrated to resist tensile stresses (e.g., bed-joint reinforcement and bond beams). These elements and/or materials are characteristically absent within CSEB shells. However, considering the extent of thermal and/or shrinkage induced cracking in numerous unreinforced CMU, CSEB and fired clay brick shells, it is clear that control joints should be implemented when applicable. This is an important design consideration in regions with large diurnal and

seasonal temperature fluctuations, such as on the Highveld in South Africa. Bradley *et al.* (2017a) recommend the use of 25 mm control joints above secondary elements such as window frames, walls and suspended slabs, to allow the free movement of the shell. However, such joints preclude the gable walls from supporting and stiffening the shell, and this has implications on the load path within these structures.

6.3.5 Reducing solar gain

6.3.5.1 Insulation

RC domes built using inflated forms have built-in insulation. The air form has an external membrane over sprayed-in-place polyurethane foam insulation (Wilson, 2005). Shotcrete to form the dome, is then applied from inside the inflated, insulated form. Wilson (2005) notes that urethane, extruded polystyrene, or moulded or extruded EPS (expanded polystyrene), 50 to 75 mm thick, can be applied to the exterior of shells to improve insulation, which can be protected from solar radiation and hail by an external coating of concrete with mesh reinforcement or a layer of ethylene propylene diene (EPDM) rubber sprayed over the surface. However, the implementation of insulation and protection would increase costs, potentially negating any savings made by adopting an unreinforced CSEB shell.

6.3.5.2 Paints

The surface condition and colour of walls and roofs have significant influence on heat gain. Adam and Agib (2001) noted that a whitewash applied directly to CSEB masonry or render is a cheap solution to reduce solar gain. However, the application of washes (e.g., cement and lime) is typically limited to sheltered walls. Because shells used for roofing are exposed to direct rainfall, wind, hail and solar radiation, washes do not last long. The shells at Wits were coated with a cement wash, for purely aesthetic reasons, which was quickly eroded over the apex during the first summer after application, as shown in Figure 14a. Unlike washes, paints offer a long-lasting solution to moderate large heat accumulation in building envelopes. A common methodology adopted in housing and commercial structures is to use pastel coloured or white paints over walls and/or roofs to reflect solar energy. The ability of white paint to reflect visible and infrared (IR) light makes it particularly good at limiting heat gain. The opposite is true of darker colours, which absorb light within the near-infrared band, resulting in substantial heat build-up in the surface. Paints

characteristically have rather poor emissivity, which means that heat build-up in these outer coats will be transferred into the substrate and masonry below. A solution to attenuate heat build-up in the coat, substrate and masonry is through the adoption of infrared reflective paints, which is sometimes referred to as *cool paints*.

Cool paints include infra-red reflecting pigments, which offer thermal improvements over standard paint. The primary purpose of these paints in building applications is to reduce costs by reducing air-conditioning demand as well as improving the longevity of the paint and substrate. Durability is increased by reducing surface temperatures and, subsequently, lowering temperature strain in the coating layers and substrate (BASF, 2017). Furthermore, these pigments allow the use of darker paints. The ability of a painted surface to reflect visible, infra-red and ultra-violet light, is in recent times being quantified by the Total Solar Reflectance (TSR). TSR is specified as the percentage of irradiated energy reflected back into the atmosphere and depends on the type and quantity of pigment, the colour of the paint and substrate as well as the paint film thickness.

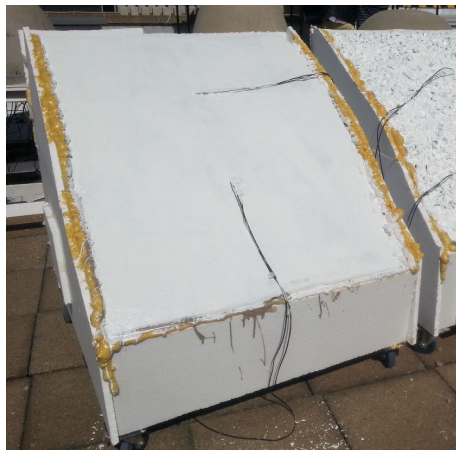
To examine the extent by which several paints could potentially attenuate large surface temperatures as well as reduce thermal gradients across CSEB shells, a series of tests was undertaken on curved panels on the roof of the Hillman Building, University of the Witwatersrand. The panels were built with 120 mm thick CSEBs, a standard cement-sand mortar and plaster approximately 15 mm thick over both faces of the panel. Four conditions were considered, namely; A) unpainted cement-sand plaster, B) standard acrylic light-brown paint, C) standard enamel white paint, and D) acrylic light-brown paint with infra-red reflecting pigments. The panels are shown in Figures 6.26a through 6.26d. White paints typically have a TSR greater than 80%, whereas a black coating pigmented with carbon black have a TSR less than 5% (BASF, 2017). The manufacturer's quoted TSR for the paint used on Panel D, over a white substrate, was approximately 50%. The paint offered the highest TSR from the manufacturer's range of colours, apart from white. The panels were positioned facing north, instrumented at two locations with J-type thermocouples. Five thermocouples were embedded at each location, and spaced at equidistant intervals between the external and internal surfaces. Data was recorded at 15-minute intervals with an Agilent 34972A logger. Polystyrene was fixed around the panel to ensure that heating was due to direct solar radiation over the exposed surface. Testing was conducted in the winter between 17 July and 4 August 2013.



(a) Condition A



(b) Condition B



(ac) Condition C



(d) Condition D

Figure 6.26 – Instrumented CSEB panel

Figure 6.27 shows the maximum temperatures measured just below the exposed and non-exposed surfaces, as well as at mid-thickness. These peak temperatures generally occurred around 13:30 PM, 15:30 PM and 16:30 PM for the exposed, middle and non-exposed points respectively. Surface temperatures were significantly reduced when using white paint. The IR reflecting light-brown paint performed better than its standard acrylic equivalent but less than the white paint. Furthermore, the thermal gradient across the panel painted white was also reduced considerably; the maximum difference in temperature between non-exposed and exposed faces was consistently less than 6°C. For the unpainted plaster, the difference was as much as 16°C. Thus, using white paint can significantly attenuate large surface temperatures and reduce heat build within an earthen shell.

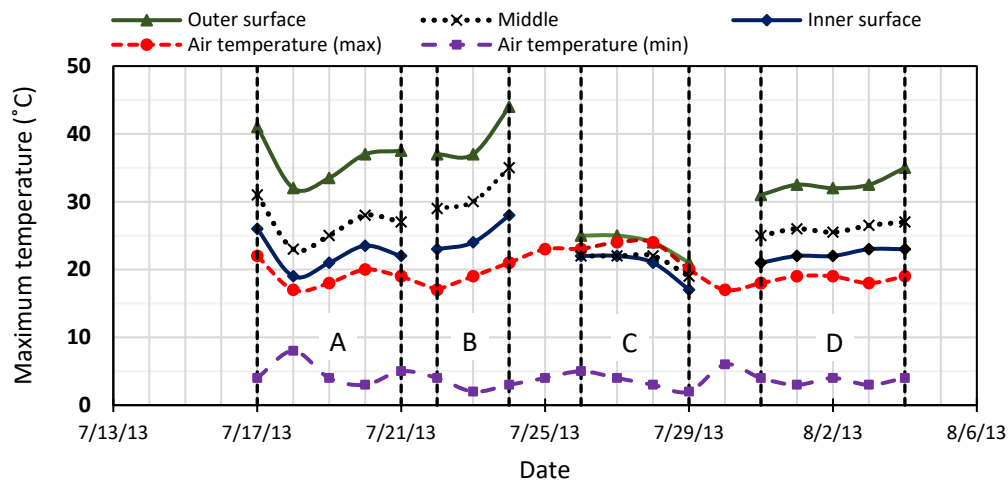


Figure 6.27 – Panel temperatures: Winter 2013

6.4 Conclusions

Designers should be aware that unreinforced masonry shells are susceptible to cracking, and that these cracks can become an acute problem, particularly in a low-cost setting where maintenance and repairs may be deficient. Consequently, the potential for cracking should be mitigated through simple low-cost design actions, as proposed below:

- Surface cracks are particularly common in cement-sand renders over masonry domes and vaults. An inexpensive solution is to cover such a plaster with a light coloured or IR reflective paint; white paint is particularly effective at decreasing peak temperatures, and, consequently, large diurnal surface strains.
- The shrinkage characteristics of mortar and brick should be identified before they are used in an unreinforced CSEB shell. Based on recommendations in the literature and field observations, the following measures should also be adhered to: 1) Bricks, blocks or tiles should be cured for an adequate amount of time to ensure that they do not shrink excessively after construction (Auroville Earth Institute, n.d.; Houben and Guillaud, 1994); 2) Mortar which better accommodates shrinkage, such as soil:cement or cement:lime:sand, should be considered instead of stiffer cement:sand mortars (NBRI, 1987); and 3) The volume of mortar should also be minimized by using small joints and/or large units (Auroville Earth Institute, n.d.). In addition, the amount of water used in mortar should be kept to a minimum, but still sufficient for hydration reactions

and strength gain, noting the common practice of adding water to partially set mortar to re-establish workability.

- Secondary elements, such as suspended slabs, beams, walls, and window frames should not be rigidly connected to unreinforced masonry shells. These elements have the potential to cause large structural cracks if they restrain expansion or contraction. This is also applicable to rigid connections between adjacent shells. To avoid cracking, expansion and contraction of CSEB shells should be facilitated through the inclusion of control joints between the structure and all secondary elements (window and door frames, partition walls, gable walls, etc).
- Abrupt variations in shell thickness and/or geometry should be avoided. These thicker, and stiffer, sections can attract moment forces under the action of gravity loading alone. Furthermore, volume changes due to temperature between the thinner and thicker portions may result in restraint, stress concentration and cracking at the juncture – analogous to cracking observed between ring-beam and shell in uninsulated RC domes (Wilson, 2005).

CHAPTER 7. CONSTRUCTION CONSIDERATIONS FOR LOW-COST EARTH SHELLS

7.1 Introduction

Earthen shells are typically designed as pure-compression structures; however, some issues associated with construction practices may cause the shells to experience other types of stress. In particular, the construction methodology and quality of the workmanship may have a significant influence on the stress distribution and the potential for cracking within these structures. In this chapter, conventional and contemporary techniques utilised for the construction of earthen shells are reviewed. Furthermore, construction issues are identified based on case study investigations and lessons learnt from the construction and monitoring of some model and full-scale earthen shells. Finally, several construction considerations are presented to possibly mitigate damage in low-cost earthen shells.

7.2 Construction methodologies

7.2.1 General

There are a variety of methods for the construction of earthen masonry vaults, domes and complex shells. Some of the methods can be dated back thousands of years when they were used in regions such as Egypt and Middle-East (Minke, 2009). There are primarily two methodologies adopted for the construction of earthen shells; namely, free-hand (typically with guide-work) and with formwork. Both methods require careful consideration during planning and design to ensure that the shells are not damaged during construction. The advantages, concerns and design considerations associated with several classic and contemporary construction methodologies are discussed in the following sections.

7.2.2 Excluding formwork

7.2.2.1 The Nubian technique

Nubian technique is one of the oldest construction methods, which was developed in Upper-Egypt thousands of years ago, for the construction of masonry arches and domes. The method exploits the surface friction of the blocks and adhesion of the mortar to construct earth masonry vaults without the use of formwork. These vaults were built-up

using consecutive arches, inclined between 65° to 70° from the horizontal (Minke, 2009). Deviation such a range would result in collapse, if the angle was too low, or blocks would fall/slide off, if the angle was too high. Furthermore, to prevent sliding during construction, the bricks were light weight (such as adobe rich in straw) and not too thick (typically between 5 and 6 cm) (Houben and Guillaud, 1994). Nubian domes were built free-hand with minimal equipment requirements. Traditionally, the domes were built in horizontal courses of adobe blocks; only aided by a marking line attached to the ‘focus’ of the dome and the mason’s wrist (Houben and Guillaud, 1994). Normally, a sticky earthen mortar (i.e., a high clay percentage) was used to prevent the bricks from sliding off from the preceding course. Usually the light weight bricks, which were raked or keyed to improve adherence, were used.

Many of the vaulted adobe structures built using the Nubian technique are attached to the gable walls, as it can be seen in vaults built at the Auroville Earth Institute (India), Swan House and Lab (Texas, USA) and New Gournia (Egypt). Because these vaults were built free-hand, using sticky mortar, one of the gable walls was initially erected to create a ‘starting point’ to which the first vertical course of bricks was stuck, as shown in Figure 7.1

7.2.2.2 Construction using a cupola tracer

A range of simple to fairly complex tools has been developed to assist in precise placement (i.e., at the correct position and inclination) of masonry units. The ‘cupola tracer’ is one such tool, which enables the rapid laying of brick courses, as shown in Figure 7.2. Gohnert and Magaia (2004) discussed the design and construction of a low-cost CSEB masonry dome built near Maputo, Mozambique, using the ‘cupola-tracer’ tool, while Magaia (2003) mentioned the difficulties encountered during the construction of that dome during the top closure. Near the top of the dome, the ‘cupola tracer’ is no longer useful because the adhesion between CSEB and mortar is insufficient to prevent the bricks from sliding off. The top section of the dome had to be built over shaped earth, which required horizontal support through propped formwork. Subsidence of the soil occurred near the apex of the dome during the removal of the formwork, which led to the geometrical imperfections shown in Figure 7.2b.



Figure 7.1 – Vaults built against the gable walls, Auroville Earth Institute

* These images were taken from the Auroville Earth Institute website (www.earth-auroville.com) - no rights reserved



(a) Construction with cupola tracer

(b) Completed dome

Figure 7.2 – Freehand construction of an earth brick masonry dome (images courtesy of Mitchell Gohnert)

7.2.2.3 *Timbrel vaulting*

Timbrel (or Catalan) vaulting is another traditional construction technique which gained recent revival in many projects, including the Mapungubwe Interpretation Centre (Ramage *et al.*, 2010). Timbrel vaulting is a form of construction initiated in the fourteenth century, but not until the sixteenth century did it become a common practice (Huerta, 2003). The construction technique comprises laying several layers of overlapping tiles, or bricks, which are bonded with rapid-setting mortar. Light-weight framework is employed during construction; Ochsendorf (2010) noted that lightweight framework was used in North America between 1880 and 1960, where over 1 000 buildings were constructed by the Guastavino Company. Rafael Guastavino refined the materials and construction practices of Catalan vaulting; his best-known contribution is that of the “Cohesive Construction Technique”. Guastavino replaced the traditional gypsum mortar with rapidly hardening Portland cement, which enabled construction of vaults spanning three to five times the span of traditional Catalan vaulting (Tarragó, 2002).

The Mapungubwe shells were built without formwork or centring; construction was done by sighting between completed portions and curved plywood guides (Fitchett, 2009). The first layer of compressed stabilized earth tiles was placed by hand, using rapid setting gypsum (Crystacal). This layer then provided the ‘formwork’ for successive layers, which were laid with typical cement-sand mortar, until the required thickness was achieved. Chothia (2010) and Gohnert *et al.* (2013) described the construction of earth-tile cross-vaults, built at the University of the Witwatersrand, using similar techniques to those used for the construction of the Mapungubwe shells. Several images of the construction process of the two cross-vaults are shown in Figure 7.3a through 7.3d.

When building shells free-hand, it is extremely important that strict attention be given to the construction accuracy of the structure. Since earthen materials cannot accommodate flexural or tensile stress, overall strength and stability are achieved through geometrical form. Divergence from the required shape may impact the capacity of the shell, possibly leading to cracking or even collapse. Thus, when utilizing free-hand construction techniques it is critical that accurate guiding and framework be used. Furthermore, skilled bricklayers should be employed to ensure quality construction. However, in many developing countries, there is potential to create employment and develop skills during

the construction of low-cost housing and recreational structures. In some projects, employment of community members may be a requirement of the client; thus, unskilled labour may make up the bulk of the workforce. The work of Eladio Dieste and Felix Candela in Uruguay and Mexico, respectively, exemplify the use of a largely unskilled labour force to construct complex shell forms (Fitchett, 2009; Pedreschi, 2000; and Melaragno, 2012). According to Fitchett (2009), Dieste's use of sophisticated engineering, to allow for simplicity of site operations, and supervision of the work by highly trained managers, enabled the majority of the construction work to be done by unskilled labour, often farm workers. Melaragno (2012) noted that Candela successfully attained complex geometric shapes using relatively unskilled labour by having a clear perception of the construction methodology throughout the project. The approaches used by Dieste and Candela allowed for the safe and accurate construction of earthen masonry shells, even those that were built free hand by unskilled labour. Issues with the accuracy of the free-hand construction process are apparent in the Mapungubwe shells; in particular, undulations are shown in Figure 7.4.



(a) Freehand construction (cross-vault)



(b) Completed cross-vault



(c) Freehand construction (domed vault)



(d) Partially completed domed vault

Figure 7.3 – Construction of two earth-tile cross-vaults (image courtesy of Anne Fitchett)



(a) Large free-spanning shell

(b) Shell undulations

Figure 7.4 – Large earth tile structure: Mapungubwe Interpretation Centre

7.2.2.4 Additional considerations

When building shells without formwork, the construction sequence should be carefully considered so that additional loading is not generated which the partially complete shell cannot sustain. In particular, the loading incurred during construction, when the shell is incomplete, may be much greater than that experienced by the completed structure, i.e., during the construction process the partially complete shell may be unstable, and the erection precarious, until the form is completed.

7.2.3 Construction with formwork

7.2.3.1 Earth formwork

Because the production and erection of typical formwork (i.e., timber boards, steel sheets, etc.) is labour, material and energy intensive, earth formwork was historically used for the construction of many masonry shells. The use of earth for formwork dates back millennia when it was used by the Egyptians for pyramid construction. Even in modern times, the earth form method offers an economic and versatile solution for the construction of thin shells in certain situations. Earth formwork has been employed for the construction of masonry domes (Minke, 2009) as well as several large thin shell concrete structures (Wilson, 2005). An ice-skating rink built in Provo, Utah, was a well-known large reinforced concrete shell structure built using earth formwork (Wilson, 2005); unfortunately, the shell has recently been demolished. The triaxial elliptical dome was 73 metres long, 49 metres wide and 12 metres high. The form was made by using approximately 30,000 cubic metres of earth, moulded into the required shape (Wilson,

2005). This soil was appropriated from the foundation excavation of a nearby structure which, significantly reduced transportation costs. However, in many situations, the amount of soil required to make an earth formwork may incur significant transportation cost, and, consequently, cheaper techniques such as inflated forms may be needed for large shells (Wilson, 2005).

The application of the earth formwork technique to construct a masonry shell construction is relatively simple - moist soil is shaped into a form defining the intrados (interior) of the shell and the masonry is subsequently built over this form. Bricks or some other supporting structure can also be used to occupy part of the internal volume in order to limit the amount of soil, and consequently work required. Figures 7.7a and 7.7b show model catenary domes built to examine this technique. Moist sand was placed over and around an inverted bucket and shaped with a wooden catenary cut-out to attain the desired shape. The removal of the earth mould requires careful and considered excavation. According to Wilson (2005), soil should be removed from the centre of the shell to its outer edges to ensure shell action and prevent the structure's collapse during excavation.



(a) Construction

(b) A completed dome

Figure 7.5– Small dome built with earthen formwork

7.2.3.2 Inflatable formwork

Inflatable formwork has become very popular in recent years for the construction of a variety of thin shell structures. Wilson (2005) noted that during the past few years, more than two thousand structures have been built throughout the world using the inflatable formwork method. Inflated forms can also be used to create shells with unusual shapes.

The forming system incorporates an inflatable fabric structure, typically made of PVC coated nylon or polyester fabrics. The primary application of this system is for sprayed concrete (i.e., shotcrete). Shell construction can either be carried out from inside or outside of the inflatable form. However, most of the shells built using the inflatable formwork technique have been built from the inside (Wilson, 2005). This option is preferred because construction can be accomplished irrespective of weather conditions. Furthermore, the internal atmosphere is controlled so that the curing conditions are optimal. An airlock, however, is required for workers to enter the inflated form which may not be economical for smaller structures.

Although this method has been widely adopted for the construction from small to very large concrete shell structures, it has only recently been utilized for the construction of fired or concrete brick and earth masonry structures. Numerous concrete brick masonry domes have been successfully built by Dome Space (South Africa) using inflatable formwork at the Sparrow Rainbow Village, Johannesburg (Talocchino, 2005). Understandably, due to the different building requirements for masonry, when compared with shotcrete, construction of masonry domes takes place from the outside. Gohnert and Talocchino (2011) discuss the use of this method to construct a small low-cost earth-brick dome at the Central Johannesburg College (CJC), as shown in Figures 7.6a and 7.6b.



(a) Construction nearing completion



(b) Completed structure

Figure 7.6 – Construction of a masonry dome with inflatable formwork (images courtesy of Mitchell Gohnert)

A major advantage of this type of formwork is that it can be deflated and re-used, making it a very economical and efficient option if many similar structures are to be built. Such formwork also facilitates rapid construction - the dome at CJC was built in only two days.

Furthermore, the form is flexible, making it more accommodating to shrinkage when compared with rigid formwork.

7.2.3.3 Full formwork

In the case of domes and even complex earthen shell forms, construction without formwork is preferred since curved formwork is complicated and expensive to manufacture. However, when formwork is utilised it should be light, mobile and easily dismantled to be effective (Houben and Guillaud, 1994). Understandably, formwork also needs to be sufficiently rigid to ensure that it does not warp during construction. Typically, formwork is built from timber boards or metal sheeting, supported by a truss system.

Although formwork may increase the costs associated with a one-off construction, the effectiveness of formwork becomes apparent when multiple shells forms of similar geometry need to be built. Felix Candela's work, in particular, demonstrates the effectiveness of reusing formwork (Melaragno, 2012). The success of his construction company came largely through designing reinforced concrete thin-shells that allowed for the re-use of formwork from previous projects, which expedited construction and greatly reduced materials costs. According to Melaragno (2012), the re-use of formwork has produced extraordinary cost reductions in the construction of concrete thin shells.

7.2.3.4 Shifting/sliding formwork technique

When adopting the sliding technique, only a small segment of formwork is used instead of continuous formwork. Construction takes place sequentially, as the formwork is shifted to create the barrel vault. The technique is relatively cheap because only a short segment of singly curved formwork needs to be manufactured. This construction technique was used to build two steep catenary vaults at the University of the Witwatersrand, shown in Figure 7.7.

Another advantage of adopting prefabricated formwork is that its use may greatly improve the accuracy of construction; although, formwork alone does not guarantee high quality of work. Figures 7.8a and 7.8b show the initial segments of a vault built with formwork at the University of the Witwatersrand. The vault was built by bricklayers accustomed to typical vertical wall construction, and consequently this 'unusual'

technique required some practice in order to reduce the chance of mistakes. One such imperfection is shown in Figure 7.8a. At times, several layers of bricks had to be removed and rebuilt to rectify these mistakes. The initial segments came to be a learning experience for the bricklayers, and the remaining work, in the most part, was completed quickly and accurately, as shown in Figure 7.8b. These lessons highlighted the importance of experience (and skill), as well as the need for continuous monitoring to ensure that any inaccuracies in the form are identified immediately, before significant remedial action or rework is required.



Figure 7.7 – Construction sequence of the Wits catenary vaults (images courtesy of Ivanka Bulovic)



(a) Inaccuracies

(b) Improved construction accuracy

Figure 7.8 – Construction of the first segment of the Wits shells

7.3 Prevention of excessive shrinkage during construction

7.3.1 Shrinkage of the mortar

The common types of mortar used in CSEB masonry construction are soil:cement, cement:sand and cement:lime:sand. The magnitude of drying shrinkage of these mortars is amplified when the amount of water, lime and/or cement is increased. Furthermore, the type of soil used in soil:cement and unstabilised mortars has a significant effect on drying shrinkage (e.g., shrinkage increases as clay content increases). Typical linear shrinkage of earthen mortar should be between 1 and 5% (Morton, 2008). Some national standards provide limits on linear shrinkage. The New Zealand Standard 4298 (1998) stipulates a linear shrinkage limit of 3% for earthen mortars. According to Morton (2008), an earth-brick wall built with 10-mm clay mortar joints will typically experience linear shrinkage of 0.0004–0.002%, which is evenly distributed among the joints. This is significantly lower than the drying shrinkage of cement:sand mortars. Vertical shrinkage, due to gravity load, is typically 0.01–0.05% (Morton, 2008), which results in lowering of the head of the wall. Vertical shrinkage is only problematic if fixity, which restrains shrinkage, is generated. According to Morton (2008), vertical shrinkage is reduced if the

mortar dries as the wall is built. Joffroy and Guillaud (1994) and Houben and Guillaud (1994) noted the following considerations for mortar used in earthen construction;

- Use components of similar strength;
- The use of mortar with a high-water content should be avoided because of the excessive shrinkage;
- Stabilized bricks and the mortar laying surface (bed) should be pre-soaked thoroughly before placement;
- Thin joints should be used to avoid the risk of shrinkage during the subsequent drying out.

Morton (2008) and Mukerji (1994) also noted the benefit of adopting mortars that have the same compressive strength as the units, which results in the masonry element having a reasonably even strength. Walker and Stace (1996) noted that such an approach is in contrast to general masonry practice, where weaker mortars are used; thus, generally, ensuring cracks occur along the joints. A report compiled by the National Building Research Institute (NBRI, 1987) recommended using mortars having strengths smaller than that of the earth bricks because they will distribute shrinkage cracks and mitigate the formation of large unattractive cracks. Similarly, Grimm (1988) noted that weaker mortars have greater extensibility (i.e., lower modulus of elasticity and greater creep in tension), which accommodates shrinkage better than stronger mortars.

7.3.2 Shrinkage of the units (bricks, blocks and tiles)

Drying shrinkage of CSEBs increases rapidly above 0.1%, which is a typical limiting value for drying shrinkage of concrete masonry units, as the clay content increases beyond 25% (Walker, 1995; Walker and Stace, 1996). Walker and Stace (1996) noted that the shrinkage of blocks comprising soils with low clay content was magnified as the cement content increased. However, shrinkage was controlled better in blocks having clay content above 25% by including additional cement. Thus, clay content should be limited to control drying shrinkage. Furthermore, soils with high clay content and plasticity index may experience excessive shrinkage resulting in large cracks whilst curing. Figure 7.9 shows large shrinkage cracks within a dry-stack block after 28 days of curing; the block had a clay content of 23%; plasticity index = 25%; and included 10% cement by mass.



Figure 7.9 – Typical shrinkage cracking in a CSEB

7.4 Case studies: restraint generated during construction

7.4.1 General

The majority of the shrinkage occurs in the early stages, after the masonry has been laid. This is observed in standard wall construction, which will result in a lower wall head. However, overall shrinkage is reduced if the mortar dries as the wall is being built (Morton, 2008). Nevertheless, cracking may still occur if the shrinkage movement is restrained. Such restraint can be generated by a rigid formwork, if it is used in the construction of earthen shells. When masonry vaults are built using rigid formwork they may be vulnerable to cracking after the final bricks are laid. The shell will try to contract, due to drying and/or vertical shrinkage of the mortar, but since it is restrained by the formwork it will crack. In the case of vaulted structures, cracks typically occur towards the apex of the shell if shrinkage is restrained, similar to restraint created by gable walls. Such cracking was observed during the construction of a tile cross-vault and the steep catenary vault built at the University of the Witwatersrand.

7.4.2 Cracking at the apex during construction of an earth tile cross-vault, Wits

The earth tile shells built at Wits and Mapungubwe incorporate edge arches (i.e., lips), which were built over rigid formwork to create a starting point for the free-hand construction, and the formwork was left in place until the shell construction was

complete, as shown in Figures 7.3 and 7.10. The rigid formwork prevented shrinkage, which ultimately led to cracking at the apex as shown in Figure 7.10b.



(a) Edge formwork left in place

(b) Apex crack

Figure 7.10 – Cracking caused by the restraint of shrinkage (image courtesy of Anne Fitchett)

Several small-scale earth-brick vaults were built to explore the cause of cracking observed in the Wits shells. The tiles used to build the models were the same as those used to build the cross-vault shells, and a class II mortar, as specified in BS 5628 (2005), was used to bond the units. The models were built using a rigid steel frame to prevent any movement at the base, as shown in Figure 7.11. The model shell dimensions were as follows: length = 1000 mm, span = 750 mm, height = 300 mm, and thickness = 20 mm. Fairly large cement-sand mortar joints, roughly 5 mm, were used to increase the overall length experiencing drying shrinkage. Several models were built for the purpose of the analysis and two distinct setups were adopted; namely, 1) formwork left in place overnight, and 2) formwork dropped after the initial set of the mortar, which was typically about 90 minutes after construction had been completed. The following day, longitudinal cracks, through the entire thickness and along the apex, were present only in the vaults that the formwork was left in place. The cracking in one of these models is shown in Figure 7.11c (external surface) and 7.11d (internal surface). The cracks occurred irrespective of the construction methodology; i.e., simultaneous construction from either support, as illustrated in Figure 7.7, or one side followed by the other, as shown in Figure 7.11a.

Thus, formwork should be removed immediately after the setting of the mortar, but the mortar must have set sufficiently to accommodate load transfer between adjacent bricks. Initially, a few of the models collapsed because the formwork was removed prematurely. In a full-scale structure, if the formwork is removed too early distortions or even collapse will occur.



(a) Small scale catenary vault construction (b) Shrinkage prevented by rigid formwork



(c) Apex crack (external)

(d) Apex crack (internal)

Figure 7.11 – Model catenary vaults built over rigid formwork

7.4.3 Cracking at the apex during construction of the Wits catenary vaults

The same type of cracking that was observed during the construction of the Wits cross-vault was observed during construction of first segment of one of the Wits steep vaults. Extensive cracking at the apex of this segment is shown in Figure 7.12. Cracking was due to the restraint created by a segment of the formwork that was mistakenly left in place overnight, which further confirmed the recommendation that the rigid formwork should be removed after the setting of the mortar. The cracks necessitated significant repairs, and

the bricks and mortar near the apex were removed and relayed. To eliminate cracking in the remaining segments, the formwork was removed after the mortar had set - allowing it to dry unrestrained. No further cracking was observed during the remainder of the construction work.



Figure 7.12 – Shrinkage cracks at the apex of the Wits vault

7.4.4 Vertical shrinkage cracking in the Wits catenary vaults

Vertical cracks occurred in the Wits vaults several months after their construction was completed. These cracks, which extend along the entire curvature of the shells, are shown for two interface locations on the north facing vault in Figure 7.13. The formation of these cracks has little influence on the stability of the one-way-spanning structures.

Cracks like those observed in the Wits shells are not uncommon in CMU walls with excessive length to height ratios (L/H). For long CMU walls, vertical shrinkage cracks occur at the mid-length or at rather even intervals (Grimm, 1988). The cause of these cracks is the restraint of shrinkage at the base of the wall due to bond and friction. These cracks are more likely to occur in regions of greater exposure to wind, rain and solar radiation, such as wing walls and parapets (Rainer, 1983). Bond beams, bed reinforcement and/or control joints are typically used to prevent shrinkage cracking in CMU walls. Typically, CSEB masonry shells incorporate none of these control measures even though shrinkage in CSEB masonry shells may be higher than the limits prescribed for

CMU masonry; i.e., as high as 0.1%. This is particularly problematic if CSEB, made with clayey soils, are employed prematurely.

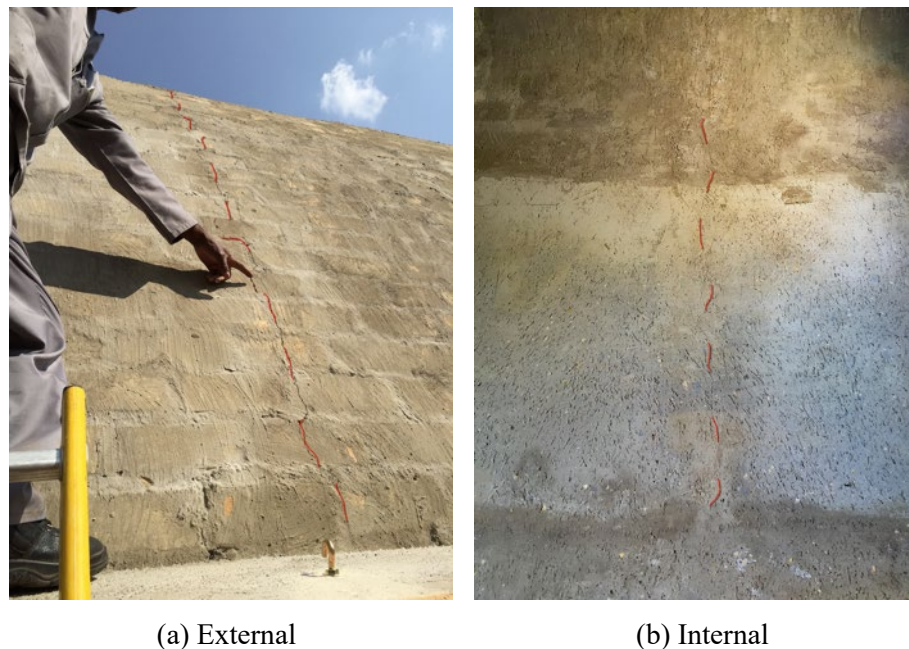


Figure 7.13 – Vertical cracks in the Wits shells (cracks traced in red for clarity)

The Wits shells were built using blocks manufactured from a sandy soil with low clay content, and were cured for at least 28 days. Consequently, most of the shrinkage should have been limited to the joints. However, the mortar joints at segment interface had to be fairly large, approximately 25 mm, so as to facilitate bonding to the subsequent segment. Larger joints increased the total shrinkage during drying out after building work, and magnified stress concentrations. Since the shells were restrained, cracks developed and propagated vertically along the interface of the arch segments.

Another reason for the formation of the cracks at the segment interface is that fixity was generated during construction of the vaults due to differential shrinkage in adjacent segments. The vaults were built sequentially as the formwork was shifted to create the barrel vault, as shown in Figure 7.7. Only a single 1.2 m wide segment of formwork, instead of a continuous formwork over the entire length, was implemented for each vault construction. It is postulated that the preceding arch segment had shrunk prior to the construction (and attachment) of the subsequent segment. Thus, shrinkage of the latter was restrained by the former. Vertical shrinkage cracks in CMU walls have also been

attributed to the restraint created by vertical elements; i.e., at wall stiffeners such as pilasters, piers and are especially prevalent near re-entrant corners (Grimm, 1988).

The shifting formwork technique is commonly used (Joffroy and Guillaud, 1994; Minke, 2009); however, no information about cracking along the segments could be found in the literature. The use of this technique in the construction of the Wits shells was somewhat unique because they are much steeper than typical vaults. Steeper vaults experience greater vertical shrinkage than shallower vaults under gravity load, like a vertical wall, which is fundamentally due to the slope of the shell; i.e., a steep vault carries much of the load internally, and therefore transfers less load to the supporting formwork. This is illustrated by the fact that the formwork used at Wits worked effectively, without the need for excessive stiffening; the sparsity of the internal truss system is shown in Figure 7.7a and 7.7b.

Several small-scale masonry models were built to support the proposed postulate for cracking in the Wits shells. These models were like those used to assess the influence of rigid formwork; however, the height of the models was increased to 550 mm. A fundamental limit of the small-scale model used in this assessment was associated with the scaling of stiffness and strength. Thus, to promote the formation of vertical cracks in the small-scale model, two actions were implemented, namely:

- Use a stack bond so that all vertical and horizontal joints were continuous and aligned. Such an approach would ensure that vertical cracking would be isolated to zones of weaknesses within the shell (i.e., along the mortar-brick interface).
- Place the models in a ‘solar oven’ to promote rapid curing.

Construction of the models began on the 24th July 2015 and was completed on the 28th July 2015. The models were constructed in three segments, as illustrated in Figure 7.14, and each comprised two columns of bricks. A class II mortar was used. To ensure that apex cracks did not occur due to restraint, the formwork was dropped approximately one and a half hours after construction. Segments were built roughly two days apart, which is comparable to the time required for a full-scale vault built in this manner. The ‘solar oven’ was fixed in place after the formwork had been dropped. The models were positioned so that the curved length paralleled the North-South orientation, identical to the Wits shells. Testing was conducted in July, 2015; the typical maximum temperature

within the Perspex chamber was approximately 60°C, whereas the corresponding external temperature was 25°C. The temperature was measured using two Pt100 RTDs attached to a Huato data logger.

A small crack, running from the base to the apex, on the northern face, appeared between two adjacent segments in one of the models, as shown in Figure 7.15. These model tests, however, were not conclusive since cracking occurred in only one of the models. To investigate further the cracking pattern, a simple analysis using finite elements (FE) was considered. A finite element software was used to create a model of a steep free-standing catenary vault, as shown Figure 7.16a. Eight-node quadrilateral shell elements and a regular/structured mesh were utilised in the analysis. The material and geometric properties implemented are shown in Table 7.1. Material isotropy was assumed and the properties for CSEB used were adopted from Magaia (2003).

Table 7.1 – FE model properties

Span (m)	Height (m)	Thickness (m)	Density (kg/m ³)	E (MPa)	Poisson's Ratio
6	4.8	0.15	1950	3500	0.15



Figure 7.14 – Construction sequence of a vault model



Figure 7.15 – Vertical crack in a catenary vault model

The FE model mesh was refined, for self-weight loading, until the model converged to the closed-form solution for a catenary vault (Gohnert and Bradley, 2020). Subsequently, thermal contraction, created by applying a temperature differential to the shell, was used to simulate shrinkage. A temperature drop was applied to one half of the shell only, as shown by the black region in Figure 7.16a, to simulate the effects of different rates of shrinkage in adjacent sections of a vault. The calculation of the temperature differential (ΔT) and coefficient of linear expansion (α), to roughly simulate shrinkage, are discussed below.

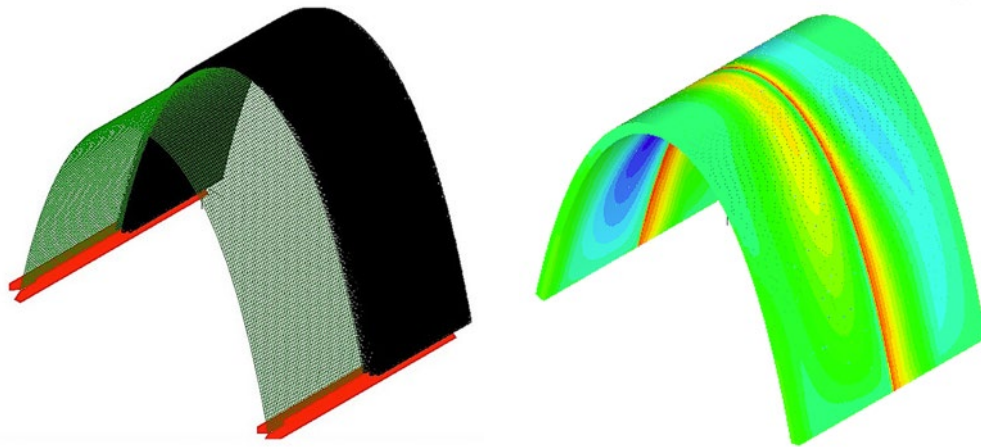
A vertical shrinkage of 0.03%, which is within the characteristic range for 10 mm joints as noted by Morton (2008), was selected for the analysis. By adopting an arbitrary linear element of masonry, the corresponding shrinkage strain can be expressed by Equation 7.1.

$$\begin{aligned}\text{Strain} &= 0.03 \div 100 = 0.0003 \text{ m/m} \\ &= 300 \mu\epsilon\end{aligned}\tag{7.1}$$

For the purpose of the simulation, a 20°C temperature drop (ΔT) and a coefficient of linear expansion of 0.015 mm/m°C (α) provide a thermal strain which matched that given by Equation 7.1, as indicated by Equation 7.2

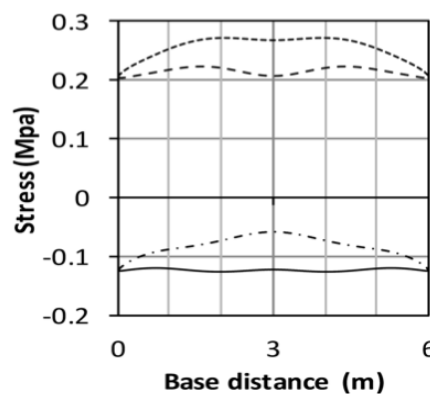
$$\begin{aligned}\text{Strain} &= \alpha \Delta T = 20^\circ\text{C} \times 0.000015 \text{ m/m}^\circ\text{C} = 0.0003 \text{ m/m} \\ &= 300 \mu\epsilon\end{aligned}\tag{7.2}$$

Figure 7.16b shows the internal and external minor principal stress contour plots corresponding to pinned boundary conditions, since a damp-roof course (DPC), situated at the base of the shell, engendered a simple support. Maximum tensile stresses, identified by warm colours (i.e., red), are localized along the interface (i.e., along the centreline). The small arrows in black show the direction of these stresses at the interface. The major and minor principal stresses along the centreline of the vault were plotted and are shown in Figure 7.16c. The plot shows that the major principal stresses are purely compressive while the minor principal stresses are entirely tensile at the inner and outer faces. Tensile stresses also exist when the support condition was changed to fixed (i.e., an infinitely stiff support). This simplified FE indicates that the maximum tensile stresses coincide with the observed cracking in the Wits shells, giving support to the aforementioned postulate for the observed cracking.



(a) 20°C temperature drop over half of the structure

(b) Minor principal stresses



(c) Principal stresses: pinned boundary conditions

Figure 7.16 – FE model and results

7.4.5 Shrinkage considerations

The vertical cracking observed in the Wits shells signifies a shortcoming of the shifting-formwork technique. It further highlights the importance of the mortar joint characteristics and the construction scheme (or progression). If the shifting method is to be used for steep profiles, such as those built at Wits, the following should be considered:

- Any fixity created during construction will likely result in cracking damage due to the inevitable shrinkage of the mortar (and possibly bricks). Thus, the design should accommodate any differential movement between dissimilar materials or parts of the structure (i.e., the shell should be designed so that shrinkage and the resulting stresses are not focused in a region). Control joints may need to be used if CSEB masonry shells are designed without reinforcement.
- Thinner joints, which are less susceptible to shrinkage, should be used when possible. Very thin joints should, however, be avoided as capillary suction can induce rapid drying and possible weakening of the brick-mortar bond. Furthermore, acceptable jointing of segments becomes problematic with thin joints when using the conventional shifting technique.
- Material and shrinkage properties of the mortar mix must be considered in the design of the shell. Typical linear shrinkage of earthen mortar should be between 1 to 5% (Morton, 2008).
- Bricks, blocks and tiles should be adequately cured before being used.

7.5 Conclusions

Several construction methodologies for low-cost earth shells were reviewed and recommendations, particularly in relation to the prevention / mitigation of cracking, were given in this chapter. The main findings are presented below.

7.5.1 Free-hand construction

When adopting free-hand construction techniques, there should be constant monitoring of the in-situ shell geometry to ensure that it meets the design specifications. Because pure-compression shells strength and stability come from their geometry, even small variations in shape can cause unintended bending or tensile stresses that may result in cracking. Thus, skilled bricklayers should be used when building free-spanning earthen shells

without formwork. Free hand construction is typically used with certain building unit types such as multiple layers of earthen tiles or small light-weight earthen blocks. Free-hand construction using CSEB is far more challenging, particularly when building shallow portions of shells.

7.5.2 Construction with formwork

The use of formwork will typically increase the cost of a one-off earth masonry shell construction. Formwork is typically made from materials that are either expensive and/or energy intensive (i.e., steel sheeting over a timber/steel supporting trusses). Due to these considerations, free-hand techniques are often more appealing for the construction of complex shells, such as the shells of the Mapungubwe Interpretation Centre. Formwork, however, may need to be fabricated to accommodate complicated shell forms, such as complex double-curvature shells.

Although formwork is an added cost, its use may, in some cases, greatly improve economy, especially when building multiple shells of the same form such as in low-cost housing developments. The use of reusable formwork may greatly reduce labour costs due to improved speed of construction, and may also lessen the likelihood of construction errors, possible damage and the consequent increased time and costs associated with repairs and re-work.

Formwork itself can create problems due to the restraint it might generate whilst the mortar between units shrinks. The fixity generated during construction with rigid formwork can lead to significant cracking in earthen shells, particularly in shells which experience vertical shrinkage, which is much larger than drying shrinkage. The size and shrinkage characteristics of the mortar joints are critically important in mitigating cracking damage and large mortar joints made with high water content should be avoided. Bricks, blocks and tiles must also be given adequate time to cure prior to construction or shrinkage will be increased, especially in units made from clayey soils. Clay content and plasticity index of the soil should be limited to control drying shrinkage. If blocks are used prematurely for the construction of earthen masonry shells, the combined shrinkage of blocks and mortar may cause cracking near areas of restraint.

CHAPTER 8. IMPROVING THE THERMAL PERFORMANCE OF EARTH-MASONRY SHELL HOUSING: CASE STUDY OF A HOUSE LOCATED IN JOHANNESBURG, SOUTH AFRICA

8.1 Introduction

Recently, there has been renewed interest in earthen building materials (e.g., rammed earth and earth block masonry) because they offer several benefits, such as sustainability, low embodied carbon and energy, good thermal and climatic performance, and affordability (Morton *et al.*, 2005; Morton, 2008; Zami and Lee, 2010). The excellent thermal performance associated with vernacular earthen buildings is attributed to the material properties, specifically its heat capacity and thermal conductivity, as well as the considerable thickness of earthen walling, sometimes greater than 400 mm. Nevertheless, contemporary earthen masonry also possesses high thermal capacity and low thermal conductivity, which may assist in moderating extreme outdoor temperatures and maintain a satisfactory internal temperature balance (Adam and Agib, 2001). Furthermore, like other heavy building materials, e.g., concrete and standard brickwork, earthen masonry possesses high thermal mass, and numerous sources advocate this feature throughout much of South Africa (NBRI, 1987; Conradie, 2012).

The benefits of earthen masonry have also promoted its adoption for roofing in the form of singly- and doubly-curved shells (Habitat, 1992; Theunynck, 1994; Norton, 1997; Adam and Agib, 2001; Auroville Earth Institute, 2016). Adam and Agib (2001) have also reported significant cost reductions achieved by replacing the standard roof with an earthen masonry shell. The reduction is achieved by eliminating expensive and/or energy-intensive materials as well as transportation costs if the soil is sourced from, or near, the building site. Greater economic efficiency is possible by replacing the walls and roof with a shell, such as a dome (Gohnert and Magaia, 2004; Gohnert and Talocchino, 2011) or vault (Gohnert *et al.*, 2018). Several examples of shell houses and recreational buildings, situated in South Africa, are shown in Figure 8.1 (inland South Africa) and Figure 8.2 (Western Cape coast). These shells were built with either traditional (burnt clay and cement) bricks, compressed stabilized earth blocks, or reinforced concrete. All of these shells are, seemingly, uninsulated - irrespective of their location and the local climate.



(a) Earth brick dome, Johannesburg



(b) Cement brick dome, Johannesburg



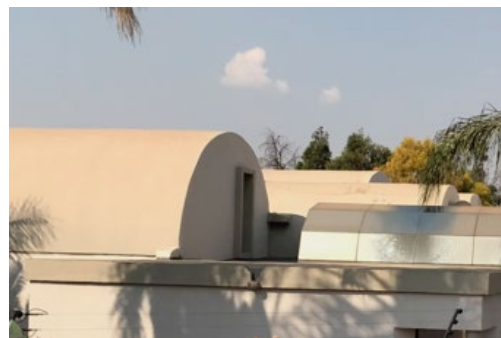
(c) Burnt clay brick vault, Johannesburg



(d) Burnt clay brick vaults, Bapong



(e) Masonry dome, Clarens



(f) Vaulted circular roofs, Pretoria



(g) Conical dome, Farm Rooikraal 454



(h) Earth-tile shells, Mapungubwe

Figure 8.1 – Masonry shell buildings situated inland South Africa



(a) Shallow vaulted roofs: Hermanus



(b) Shallow vaulted roof: Pearly beach



(c) Masonry half-dome: Paternoster

Figure 8.2 – Several shells situated on the Western Cape coast, South Africa

Masonry domed and arched roofs have been widely used for centuries in regions such as Egypt, the Middle East and Iran (Dabaeih *et al.*, 2015; Hadavand *et al.*, 2008; Fathy, 1973). The thermal performance of curved roofs in these hot arid regions has been examined and results indicate that vaulted roofs are superior to flat roofs in terms of thermal performance (Dabaeih *et al.*, 2015; Hadavand *et al.*, 2008). Dabaeih *et al.* (2015) compared flat and domed and vaulted roofs in a numerical study, and reported that spaces covered by a vault produced lower heat gains and average indoor temperatures. The source of the advantage associated with vaulted over flat roofs in hot and dry climates are;

1. the reduced area exposed to direct solar radiation due to self-shading for part of the day. However, sloped roofs having greater surface area may absorb more radiation during certain times of the day, and year, than flat roofs covering the same plan

area (NBRI, 1987). The geometry of the roof and its orientation are of principal importance in this regard. Tang *et al.* (2006) recommend that vaults be orientated north-south facing, and Runsheng *et al.* (2003) showed that heat gain through shallower east-west facing vaulted roofs is similar to that absorbed by a corresponding flat roof.

2. the curved roof generally has a larger convection heat-transfer surface than a flat roof covering the same floor area (Dabaeih *et al.*, 2015). Hadavand *et al.* (2008) calculated reduced daily heat flow through vaulted roofs due to superior cooling at night and early morning in the presence of wind.
3. the air heated within the building, gathers under the vaulted roof, resulting in thermal stratification of indoor air (Hadavand *et al.*, 2008)

Such heavyweight shells are, however, fundamentally different from a conventional housing structure, which incorporates several components to meet structural, performance and thermal insulation requirements. Nevertheless, several of the masonry roofs and buildings shown in Figure 8.1 are somewhat thinner than walling implemented in conventional engineered housing in South Africa. In fact, the shells shown in Figures 8.1a – 8.1d comprise of only a single-leaf brick section roughly 105 mm thick. These thin sections, in combination with the solar exposure of the envelopes, may impare or harm the performance of several inhabited masonry shells inland South Africa. Some of the shells at the Sparrow rainbow village were also coated with dark paints, as shown in Figures 8.3a – 8.3c, which would increase the heat gain when the shells are sunlit. In contrast, other shells in this same village were covered with paints having much higher albedo, as shown in Figure 8.3a.

Although several studies have been conducted on the design, construction, and benefits associated with earth masonry shell structures, there is little information on their liveability, specifically in South African climates. To elucidate the salient thermal performance considerations of such thin shells, a long-term in-situ assessment was undertaken for a CSEB house built in Johannesburg, South Africa. The shell roofs comprised of a single-leaf section, which was initially coated with a dark maroon paint. This chapter presents the results from the study, as well as several recommendations for the design of these structures.



(a) Dark brown and light tan roof colours



(b) Grey roof colour



(c) Maroon roof colour

Figure 8.3 – Different roof colours at the Sparrow Rainbow Village, Johannesburg, RSA

8.2 Thermal performance of affordable housing in South Africa

8.2.1 Standard low-cost housing

Conventional housing, throughout much of South Africa, features either cavity burnt clay brick, cement brick or hollow concrete block walling, concrete floor slabs, galvanized steel roof sheeting or tiles, under tile membranes, trusses, purlins, rafters, ceiling board and occasionally roof insulation. The extent of heat flow, whether up or down, through roofing has a major influence on thermal comfort within internal spaces. Morton *et al.* (2005) attributed occasional overheating in an insulated house incorporating earthen masonry walling (situated in Dalguise, Scotland) to the greater amount of solar radiation received by the roof as well as its low thermal mass. The significance of heat gain and/or loss through roofing is addressed in building standards through specification of minimum total R-values for different standard roof assemblies (SANS 10400-XA: 2011).

Much of the informal, as well formal low-cost, housing infrastructure in South Africa does not comply with the performance requirements of SANS 10400-XA: 2011. Naicker *et al.* (2017) assessed indoor temperatures and humidity for 100 low-cost houses situated in five suburbs of Johannesburg, and none of the houses evaluated had any form of roof insulation, and the majority of informal and post-1994 government-built houses (i.e., Reconstruction and Development Programme [RDP] houses) did not have ceilings. Figure 8.4 illustrates several low-cost housing structures in the cities of Johannesburg and Port Elizabeth, South Africa.



(a) Formal housing, Johannesburg



(b) Formal housing, Port Elizabeth



(c) Informal housing, Port Elizabeth



(d) Semi-formal housing, Port Elizabeth

Figure 8.4 – Examples of low-cost formal and informal housing structures

Another aspect contributing to poor thermal performance in low-cost dwellings is that single-leaf exterior brick walls are used for habitable portions (Naicker *et al.*, 2017; Makaka and Meyer, 2006), and these are sometimes less than a standard imperial brick thick (106 mm). Although such single-leaf walls may meet structural requirements following SANS 10400-K: 2015, the approximately 100 mm thickness typically used is much less than that advocated in SANS 10400-XA: 2011 to meet minimum thermal

requirements for walls enclosing habitable rooms, which is that a single leaf masonry wall with a nominal wall thickness greater than or equal to 140 mm (excluding plastering and rendering), plastered internally and rendered externally should be used.

The maximum and minimum indoor temperatures for several types of low-cost housing in Johannesburg are presented in Table 8.1 (Naicker *et al.*, 2017). These recorded extremes correspond to a monitoring period between 18 Feb 2014 and 12 May 2014 (i.e., late summer through to late autumn), which is characterized as a more comfortable period of the year in Johannesburg. The hottest months are January and December, whereas the coldest are June and July (Meteotest AG, Bern, Switzerland). The measurements were made in living room spaces, with an average of 7-8 occupants per house (Naicker *et al.*, 2017). Noting the poor compliance of informal, and some low-cost formal housing, it is unsurprising that the surveyed structures experienced such large indoor temperature fluctuations. Naicker *et al.* (2017) reported that houses lacking ceilings performed significantly worse, in terms of extreme cold and hot temperatures, than the other low-cost structures investigated. Informal structures experienced the most severe extremes; these semi-permanent structures are ad-hoc in nature, often comprising a variety of materials such as corrugated metal, earth, wood, brick, and plastic. These structures provide the barest of protection from the elements because they lack sufficient insulation and thermal mass, which are two characteristics that are indispensable throughout the majority of South Africa.

Table 8.1 – Minimum and maximum temperatures (°C) recorded in several types of low-cost houses between 18 Feb 2014 and 12 May 2014 in Johannesburg.

	Informal	Formal	Post-1994, RDP	Pre-1994
Minimum	6.1	12.7	6.6	10.9
Maximum	45.4	34.9	39.0	33.3
Range	39.3	22.2	32.4	22.4

Data from Naicker *et al.* (2017)

Added to the aforementioned deficiencies are the large numbers of occupants in such smaller low-cost structures - an average of 8 persons occupy low-cost houses in South Africa (Naicker *et al.*, 2017; Govender *et al.*, 2010), sometimes with only one designated

bedroom. Unlike the design deficiencies, the number of people who occupy a structure is beyond the control of the designer.

8.2.2 Heavy versus light-weight buildings

Figures 8.5a and 8.5b illustrate the thermal performance of two conventional affordable housing systems implemented in an inland (Pretoria) and coastal (Durban) region of South Africa (NBRI, 1987). The first is a heavy structure comprising 230 mm burnt clay brick cavity walls and standard roof with insulation (mineral wool), whereas the latter is an insulated light-weight structure. Pretoria's climate is classified as Cwa (Warm temperate, winter dry, hot summer) whereas Durban's is Cfa (Warm temperate, fully humid, hot summer) according to the Köppen-Geiger classification system (Conradie, 2012). Figures 8.5a and 8.5b indicate that there is considerable difference between the typical diurnal temperature ranges in these two regions, which can be attributed to their location, i.e., Pretoria is several hundred kilometres inland and approximately 1300 m above sea level whereas Durban is situated on the eastern coast. The NBRI reported that the heavier structure performed best in Pretoria, whereas the light-weight timber structure was better suited for Durban's hot and humid climate (NBRI, 1987). In particular, there is increased risk of discomfort in such high-mass structures in continually hot regions (Bhikhoo *et al.*, 2017), and naturally ventilated lightweight alternatives are better at producing cooler night conditions (i.e., materials with low thermal storage capacities are better suited for improving the thermal comfort at night due to the rapid rate of cool down (Eyre *et al.*, 2016). Bhikhoo *et al.* (2017) reported that insulating the roof and using natural ventilation were particularly effective measures to reduce indoor temperatures (and overheating) in continually hot and humid climates. Much of the populated parts of South Africa have temperate climates, characterized by high diurnal temperature fluctuations as well as plentiful solar radiation, so it is the combined effect of the thermal resistance (insulation) of the structure as well as its ability to store heat that determines thermal performance (NBRI, 1987). This is exhibited by the abundance of heavy brick masonry residential buildings throughout South Africa.

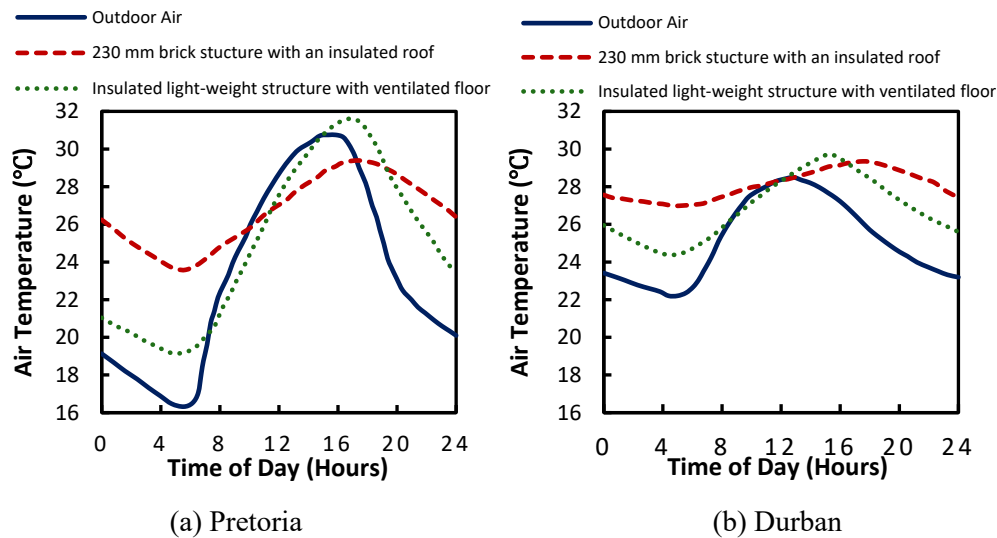


Figure 8.5 – Comparison of the indoor air temperature in high-mass and light-weight structures under typical summer conditions in Pretoria and Durban (reproduced from NBRI, 1987)

8.3 Passive design potential and features

The designers of low-cost/affordable residential buildings should incorporate passive techniques to improve the thermal comfort of occupants. Passive design incorporates strategies such as solar heat gain, high thermal mass, vertical and horizontal shades, and natural ventilation. The impact of these passive techniques is not only governed by design (e.g., fenestration, shading, building orientation, materials specification, etc.) but also by the climate. Conradie (2012) identified passive design strategies applicable in South Africa by computational building performance (CBP) modelling of buildings in different climatic zones. These were classified according to the Köppen-Geiger system (Conradie, 2012). The results of the analyses showed that natural ventilation and passive solar heating were applicable throughout South Africa (Conradie, 2012). Furthermore, high thermal mass was appropriate in approximately 96% of the land area, apart from those classified as Aw (Equatorial, winter dry) and Cfa (Warm temperate, fully humid, hot summer). Regions classified using the Köppen-Geiger system, however, do not always correlate well with the estimated energy demand in buildings due to the fact that it is based on functions of precipitation and monthly mean dry-bulb temperature and it does not incorporate humidity. Due to significant diurnal temperature variations throughout much of South Africa, particularly in the arid and semi-arid regions, energy demand calculations should be based

on hourly weather information, as opposed to daily or monthly mean temperature and relative humidity.

The software program Climate Consultant 6.0 (University of California, Los Angeles) was utilized to assess the potential of several passive design techniques at the site of the prototype house built at Wits, based on hourly weather data. Additional locations, at which earthen masonry shells have been built, were also considered for comparison (Norton, 1997; Adam and Agib, 2001; Gohnert and Magaia, 2004; Minke, 2009; Ramage *et al.*, 2010; Swan, 2016; Auroville Earth Institute, 2016). Simplified assumptions about a building design, to relate outdoor conditions to indoor comfort and ultimately provide passive design guidelines for a particular location, are incorporated into the software. The default settings were adopted herein; namely, the California Energy Code Comfort Model, 2013, and climate data necessary to perform these analyses must be in the annual 8760-hour EPW (Energy Plus Weather) format. These weather files were generated for typical years using the Meteoronorm (Meteotest AG, Bern, Switzerland) software package. The results of these analyses are shown in Table 8.2; namely, the percent of yearly hours which were classified as comfortable. It must be indicated that the numerical values are representative, and not those for the prototype house. However, of relevance to the present study are the improvements in comfortable hours related to high thermal mass and passive solar gain (high mass) in Johannesburg, especially when compared with the other locations.

In Johannesburg, passive solar gain is particularly effective in the winter due to clear and sunny conditions as shown in the data presented in Table 8.3, in which the percentage cloud cover for daylight hours for a representative winter and summer month, obtained with Meteoronorm, are presented. Roughly, two-thirds of daylight hours in the peak winter month of June are completely clear. Thermal improvements through passive solar gain are normally achieved by facing the majority of windows north but with suitably proportioned roof overhangs to shade the high-angled summer sun. North-facing windows transmit approximately twice as much solar energy than east and west-facing during the winter in South Africa (NBRI, 1987). Passive solar heating is especially beneficial when heavy-weight flooring, such as concrete, and masonry walls are adopted because the materials heat up during the day and then act as a heat source through the night (NBRI, 1987). NBRI (1987) reported that in two otherwise similar houses, the air temperature in the house with a concrete floor was 3°C higher than that in the house with a suspended timber floor.

Internal spaces were also heated by direct gain through sunlit exterior walls and/or roofs, which was the approach utilized in the prototype house of the present study. In particular, the masonry envelope absorbs and retains heat, which slows the rate at which the room/space is heated in the summer.

Table 8.2 – Comfortable hours assuming high thermal mass, passive solar gain, internal heat gain, and natural ventilation

Location	Köppen-Geiger Zone	Improvement : High thermal mass (%)	Improvement: Passive gain - high mass (%)	Comfortable hours (%)
Johannesburg, RSA	Cwb	10	30	88
Mapungubwe, RSA	BWh	20	22	84
Presidio, Texas, USA	BWh	20	24	76
Cairo, Egypt	BWh	12	14	73
Maputo, Mozambique	Aw	12	11	70
Khartoum, Sudan	BWh	10	3	34
Filingue, Niger	BSh	9	2	31
Auroville, India	Aw	0	0	10

Table 8.3 – Cloud cover % (Daylight hours only)

Cloud cover (CC)	100%	90%<CC<50%	50%<CC<10%	0%
% of hours in January	30	16	17	37
% of hours in June	9	10	16	65

8.4 Experimental arrangement and performance monitoring

8.4.1 Performance criteria

Thermal comfort of occupants in smaller residential buildings, such as free-standing houses, is dominated by the design of the building envelope (i.e., walls, fenestration, roofing, etc.). The South African building standard SANS 10400-XA: Energy Usage in

Buildings: 2011 prescribes a deemed-to-satisfy route, in which all external walls of habitable parts of the building, roofs, floors with under-floor heating and fenestration must adhere to satisfy minimum thermal requirements. In particular, SANS 10400-XA: 2011 specifies minimum total R-values ranging between $2.7 \text{ m}^2\cdot\text{K}/\text{W}$ and $3.7 \text{ m}^2\cdot\text{K}/\text{W}$, depending on the climate zone in which the structure is to be built. Thermal insulation is necessary for typical roof assemblies, i.e., those incorporating metal sheeting and clay tile, to achieve the prescribed R-values. SANS 10400-XA: 2011 specifies minimum total R-values of $0.35 \text{ m}^2\cdot\text{K}/\text{W}$ for masonry and either $1.9 \text{ m}^2\cdot\text{K}/\text{W}$ or $2.2 \text{ m}^2\cdot\text{K}/\text{W}$, depending on climate zone, for light-weight walling. Restrictions on fenestration and building layout are also specified.

For practical reasons, these deemed-to-satisfy regulations address established building types and components, e.g., traditional roof assemblies and walls. When the design falls outside these conditions/criteria it must be shown, using thermal calculation software certified by the Board of Agrément South Africa, that the building has a theoretical energy usage performance less than or equal to that of a reference building in accordance with SANS 10400-XA. Agrément South Africa is a technical assessment agency that certifies innovative and non-standard construction products, and is recognized in the National Building Regulations, i.e., Agrément certificates may be used to demonstrate compliance with the building regulations. For example, Agrément South Africa make such comparisons against a standard light-weight and/or masonry building, referred to as the Standard Agrément South Africa Comparative House (SANS 10400-K:2015). The masonry Agrément Comparative House has 53 m^2 floor area, 230 mm wide masonry walls, 75 mm thick concrete floor slab, 6 mm gypsum plasterboard ceiling, 40 mm thick mineral wool roof insulation, galvanized steel sheet roof, and window area 15% of floor area (Agrément Board of South Africa, 1986).

The expected extreme indoor temperatures within the aforementioned building, for a design summer and winter day in Johannesburg, without heating or cooling, are 5°C and 34°C . Peak temperatures may, however, be significantly higher than 34°C in low to moderate mass structures, without artificial cooling, even though the yearly energy demand may be lower than that of the Comparative House. The occupants of low-cost housing in South Africa are, however, unlikely to afford much heating or any mechanical cooling due to socio-economic factors (NBRI, 1987), which may also be a problem in developed

countries. The Building Research Establishment (BRE), in the United Kingdom, noted that air conditioning may not be an option for those on a low-income because of increased running costs of the dwelling (BRE, 2016). Centralized heating, ventilation, and air conditioning (HVAC) systems are very rare in residential buildings in South Africa. The penetration rate of air conditioning (AC) is just 4% in households, but as high as 40% in the highest income residences (Covary *et al.*, 2015). Due to the tremendous economic constraints under which much of the population lives, it is clear that the chief mechanisms to improve personal comfort are by opening or closing windows and through occupants adapting their clothing to the thermal conditions. ANSI/ASHRAE Standard 55-2010 (ASHRAE, 2010) and BS EN 15251 (BSI, 2007) provide information on the acceptable range of temperatures in naturally ventilated buildings. Adaptive comfort models used in these standards express the comfort temperature through its relationship with the outdoor temperature, which makes it more likely that these buildings (which tend to have more variable indoor temperatures) will conform to the relevant standard (CIBSE TM52: 2013). Several codes are applicable for the assessment and design of naturally ventilated structures; however, BS EN 15251 (BSI, 2007) was adopted in the present study. Equation 8.1 was used to estimate the comfortable temperature (T_{comf}) in naturally ventilated buildings:

$$T_{\text{comf}} = 0.33 T_{\text{rm}} + 18.8 \text{ (where } T_{\text{rm}} > 10 \text{ }^{\circ}\text{C}), \quad (8.1)$$

where T_{rm} is the exponentially weighted running mean temperature ($^{\circ}\text{C}$) for the day under consideration. The calculation puts higher importance on the most recent days. Equation 8.2 was used for the determination of the running mean temperature.

$$T_{\text{rm}} = [1-\alpha]\{T_{\text{od-1}} + \alpha T_{\text{od-2}} + \alpha^2 T_{\text{od-3}} \dots\} \quad (8.2)$$

where α is a constant and $T_{\text{od-1}}$, $T_{\text{od-2}}$, $T_{\text{od-3}}$, etc. are the daily mean temperatures for yesterday, the day before, and so on. A value of 0.8 for the parameter ' α ' is recommended by Tuohy *et al.* (2009). The value is based on comfort surveys conducted throughout Europe and provides the best correlation between T_{rm} and T_{comf} (Chartered Institution of Building Services Engineers London (CIBSE), 2013). CIBSE TM52: 2013 suggest that designers should aim to remain within the Category II (Normal expectation) limits prescribed in BS EN 15251 (BSI, 2007), which sets an acceptable temperature range of $\pm 3^{\circ}\text{C}$ about the comfort temperature (T_{comf}) for naturally ventilated buildings. Although the

building evaluated herein was not inhabited during testing, the evaluations were still valuable in identifying potential performance issues and/or design improvements. On a few occasions, however, the building was occupied by several people, and conservative heat gains from electrical sources were considered to estimate their influence on indoor operative temperature whilst the space was naturally ventilated.

In addition to limits prescribed in BS EN 15251 (BSI, 2007), CIBSE publishes a comprehensive set of guidelines for assessing the risk of overheating in naturally ventilated buildings. Although the guidelines were developed for buildings in the UK and Europe, they have been applied to assess buildings in tropical, subtropical and temperate climates (e.g., Bhikhoo *et al.*, 2017). The CIBSE guidelines assess the risk of overheating against three criteria, which provide limits on the severity and frequency of overheating. The building, or internal space, is said to overheat if it fails any two of the three criteria outlined in Table 8.4. The criteria are defined in terms of ΔT , which is the difference between the indoor operative temperature (T_{op}) and the maximum acceptable temperature (T_{max}), as shown in Equation 8.3. The specifics of the criteria are summarized in Table 8.4; further information is provided in CIBSE TM52:2013.

$$\Delta T = T_{op} - T_{max} \text{ (rounded to the nearest whole degree)} \quad (8.3)$$

Table 8.4 – Overheating Assessment Criteria

Assessment criteria		Limits
Criterion 1	Percentage of occupied hours during which $\Delta T \geq 1^\circ\text{C}$	Up to 3% of occupied hours
Criterion 2	Daily weighted exceedance (W_e) in any one day $> 6^\circ\text{C.h}$ (degree.hours)	0 days
Criterion 3	Maximum temperature level (T_{upp}) $\Delta T > 4^\circ\text{C}$	0 hours

When evaluating a building the monitoring period should be long enough to be representative of the cold and warm season (BSI, 2007). Furthermore, for the winter (heating season) measurements at or below mean outside temperatures for the three coldest months of the year, and for the summer (cooling season) measurements at or above statistically average outside temperatures for the three warmest months of the year with clear sky should be considered (BS EN 15251, 2007).

8.4.2 Prototype house

The prototype affordable house assessed herein was built in open area on the main campus of the University of the Witwatersrand, Johannesburg, South Africa (situated at an elevation of approximately 1700 m above sea level, with coordinates of 26°11'09"S 28°01'30"E). The house is somewhat unique, incorporating several materials and components to achieve improved cost and structural efficiency (Bulovic, 2014). The roof of the structure comprises two steep catenary vaults, which eliminated the need of roofing materials and doubles the living space. Figure 8.6a shows the completed structure; and the design details are presented by Gohnert *et al.* (2018). Walls, at ground and first floor levels, were built with dry-stack interlocking blocks, manufactured from recycled building rubble. The vaults were constructed from compressed stabilized earth blocks (CSEBs) in a cement:sand mortar. The CSEB and dry-stack blocks were 150 mm and 220 mm thick, respectively. Internal surfaces of the vaults were plastered (cement-sand), whereas the external surfaces were painted with a cement-wash, and then dark acrylic paint was applied to the roof several months later. The total thickness of the shell is roughly 170 mm. Moreover, the load and non-load bearing walls are 220 mm thick. A 170 mm deep rib and block slab was included above the dry-stack ground floor walls, as shown in Figures 8.6c and 8.6d. The dry-stack walls and rib and block slab were adopted to further reduce material costs and enable rapid construction (Gohnert *et al.*, 2018)

Openings were incorporated in the east and west-facing dry-stack walls. The windows integrate aluminum framing and McCoys's McLam® clear low-E glass (6.38 mm nominal thickness; R-value = 0.28 m².K/W). The single-glazed glass transmits 83% of visible light and 70% solar energy (McCoy's Glass, 2018), which still facilitates passive gain through the east and west facing windows in the morning and afternoon, respectively. In the summer, vertical or lateral shades could be used to prevent unwanted solar gain, particularly in west-facing windows in the late afternoon, but these mechanisms were not considered in the present study. Instead, windows were shielded on a few days to facilitate several assessments. Heavy curtains or blinds were not installed to reduce night-time winter heat loss.



(a) Completed prototype house



(b) Earth brick vault



(c) Dry-stack walls



(d) Rib and block slab

Figure 8.6 – Construction of the prototype house at the University of the Witwatersrand

8.4.3 Setup

The operative temperature (T_{op}) was measured 1.1 m above the floor in several spaces within the building, away from any direct solar radiation (ASHRAE Standard 55, 2010). Several black globe thermometers were also used to assess the spatial (plan and elevation) temperature variation in the south vault. Operative temperature represents the combined effect of air temperature, mean radiant temperature as well as air movement, as a single quantity and approximates closely to the temperature at the centre of a painted globe of ~40 mm diameter (CIBSE, 2013). Globe thermometers, appropriate for indoor spaces, may be constructed with standard table tennis balls (CIBSE, 2013), and temperature measurements can be made with either liquid-in-glass or electronic instruments positioned at the centre of the black or grey globe (CIBSE, 2006).

Several black globe thermometers were constructed by mounting J-type thermocouples inside table tennis balls, which were sprayed with black paint. The exposed end of the wires/sensors were positioned at the centre of the globes (CIBSE, 2006). Data were recorded every 15 minutes with an Agilent data-acquisition unit (Agilent Technologies Inc., Santa Clara, California). The Agilent module (model 34970A) included a built-in thermocouple reference junction, and the software applied conversion routines (compatible with Internal Temperature Scale of 1990) to calculate temperature from measured voltage (Agilent Technologies, 2003). Temperature output was compared with that of a calibrated liquid-in-glass thermometer (mercury thermometer) and three-wire Pt100 RTD (Resistance temperature detector) with a manufacturer cited accuracy of $\pm 0.5^{\circ}\text{C}$. Measurements were made in a water bath, and all thermocouples showed excellent correlation ($R^2 = 0.99$) with these instruments over a temperature range between 10°C to 40°C . Furthermore, temperatures measured with thermocouples were within 0.5°C of the corresponding mercury thermometer and RTD readings. The maximum difference between any two thermocouples was approximately 0.2°C . The observed precision and consistency were considered adequate for the analyses presented in this study since measurement accuracy is in the order of $\pm 1^{\circ}\text{C}$ for J-type thermocouple (Agilent Technologies, 2003).

The consistency observed in the water bath assessment was maintained when thermocouples were positioned within the globes. For example, the correlation between globe thermometers positioned at the same location, in the south vault, for 2nd April 2018 is shown Figure 8.7. The difference between daily mean temperatures, calculated using 96 daily measurements, was typically less than 0.1°C . Such analyses were conducted throughout the study to recheck the consistency of instruments.

The location of globes, relevant to the discussion presented in this thesis, are illustrated in Figures 8.8a and 8.8b. Figures 8.8c and 8.8d show globe thermometers positioned in the south vault and north ground-floor spaces, respectively. Outdoor air temperature was measured hourly, in a continually shaded region of the southerly side of the building. Measurements were made with an Oregon Scientific weather station with accuracy of $\pm 1^{\circ}\text{C}$.

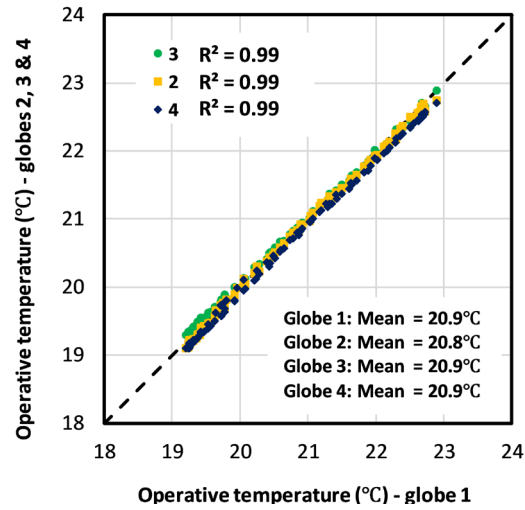
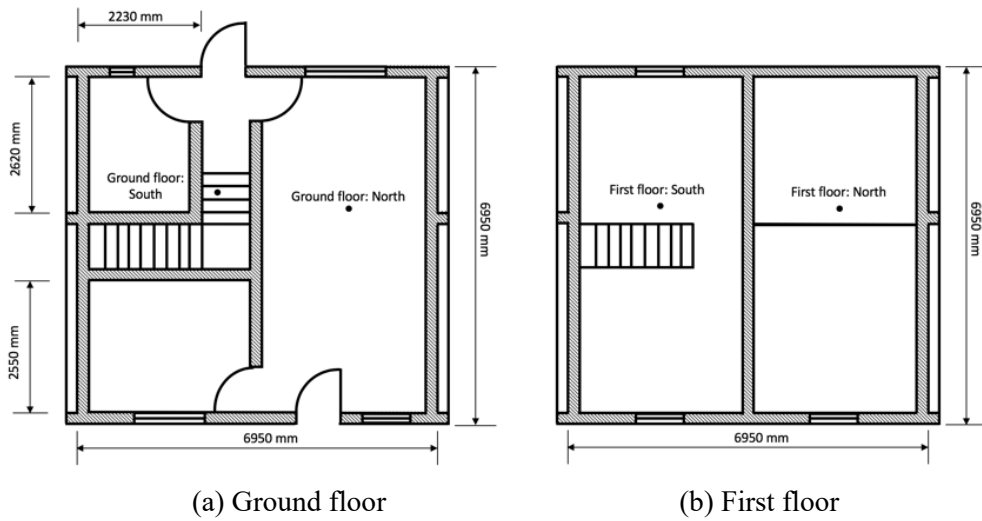


Figure 8.7 – Correlation of black globe 1 with all other globes (2,3, and 4)



(c) First floor, south vault



(d) Ground floor, north

Figure 8.8 – Black-globe thermometer positions

8.4.4 Monitoring period

The building was monitored from January, 2018, to November, 2019, in order to record characteristic data for a prolonged period. Summer in Johannesburg is between October through to March (Dyson, 2009). The Agilent logger ran for the majority of days; however, several days were missed due to interruptions in power supply as well as being switched off whilst renovations were made in the building. The mean monthly outdoor temperature corresponding to the measurement period is shown in Figure 8.9a (denoted prototype): the mean monthly temperature in Johannesburg from 1961 – 1990 is also shown (National Oceanic and Atmospheric Administration, 2018). Daily mean temperatures measured at the location of the house are shown in Figure 8.9b.

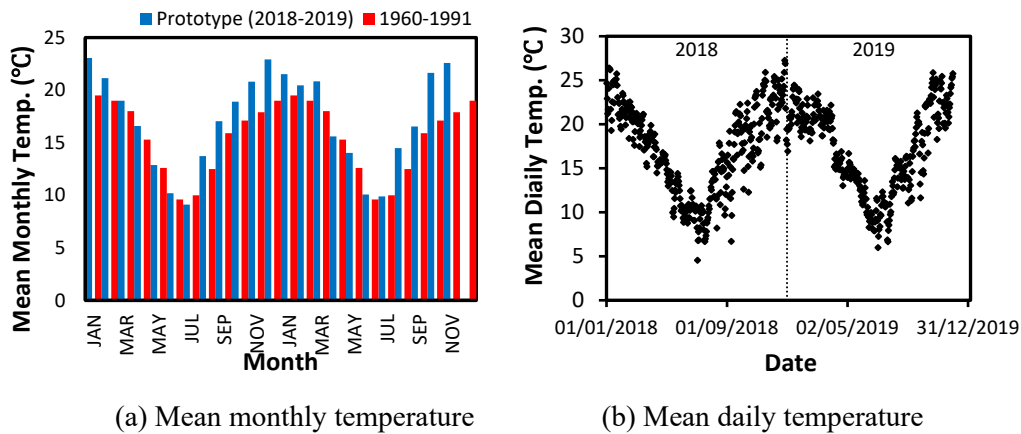


Figure 8.9 – Outside temperatures for the monitoring period

The summer months were hot, with many days having mean temperatures above statistical historical records, as shown in Figures 8.9a and 8.9b. Two heat wave warnings were issued in January, 2018, by the South African Weather Service (SAWS) (Mapenzauswa, 2018; Patel, 2018). A heat wave warning was also issued in late December (Maphanga, 2018), mid-November (Cox, 2018), and even as early as September (Mahamba, 2018). In 2019, a heat wave warning was issued in October (Grobelaar, 2019). SAWS declare a heat wave if the maximum temperature is expected to meet or exceed the average maximum temperature of the hottest month by 5°C, for that particular location, as well as persisting in that mode for 3 or more days (SAWS, 2018). Presently, the threshold is 32°C for Johannesburg (SAWS, 2018), and outdoor temperatures in excess of this temperature were measured, on at least three consecutive days, for the periods 6th – 8th January 2018, 13th – 18th November 2018, 4th – 6th December 2018, 25th – 27th December 2018, and 18th – 21st October 2019 at

the prototype house. Peak temperatures during several other hot periods did not exceed 32°C over three successive days. The hottest outdoor temperature was 34.3°C, measured on the 26th of December 2018. The coldest days were recorded in July, with a minimum temperature of -0.2°C measured on 3rd July 2018, following the arrival of a cold front (Khumalo S and Masilela B, 2018).

8.5 Results and discussion: summer

8.5.1 Building envelope

The indoor operative temperature in the south rooms and outdoor air temperatures, for a hot day in 2018, is shown in Figure 8.10a. The peak outside temperature exceeded the applicable monthly average maximum by several degrees Celsius, using the 1960 – 1991 historical data as reference. The benefit of high thermal mass is evident, especially for the ground floor room, as illustrated by the slower rate of indoor temperature development compared with that at the outdoors (Figure 8.10a). This is particularly evident at ground floor level where the heavy materials absorb heat during the day, keeping the indoor temperature relatively low, and then releasing it at night by convection, when the temperature of the immediate air drops below that of the masonry. Furthermore, the ground floor space (GS) performed especially well on this day, evidenced by the majority of measurements falling within the comfort band (i.e., $T_{\text{comf}} \pm 3^\circ\text{C}$).

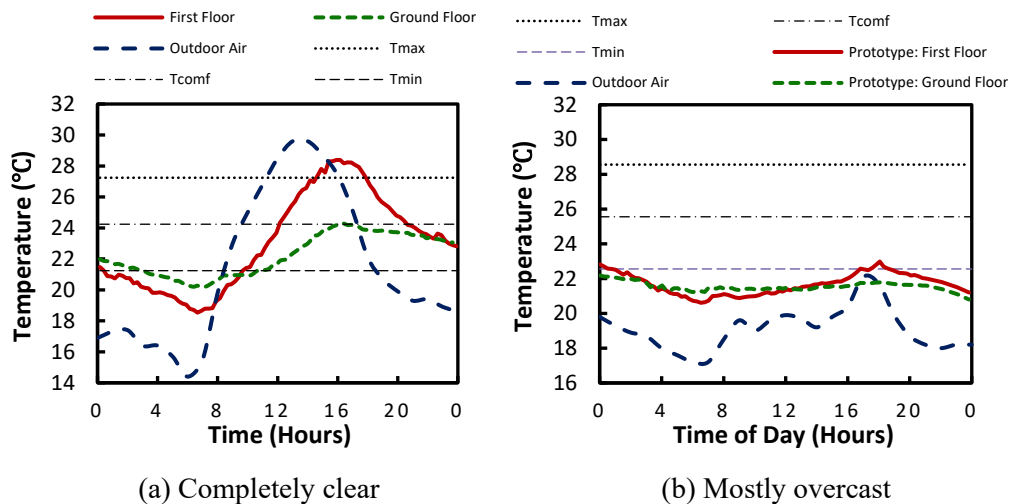


Figure 8.10 – Exterior and interior temperatures on two summer days, 2018

In contrast, operative temperature in the first-floor vault (FS) exceeded the maximum acceptable temperature ($T_{\max}$) between 15:00h and 18:00h. Although high indoor temperatures are less concerning in the late afternoon, when inhabitants are likely to be outdoors or away from home (Agrément Board of South Africa, 1986), they should ideally not continue into the evening. In fact, the elevated temperatures were not sustained into the late evening and night because the outdoor temperatures dropped sufficiently to induce cooling of the naturally ventilated space. The ‘flywheel effect’, associated with high thermal mass, is most effective in regions which experience large daily variations in temperature, such as Johannesburg. The superior performance of the ground floor space may be attributed to several factors. The principal of these are listed below:

1. *Greater thermal mass.* The ground floor comprised 220 mm thick exterior and partition walls, as well as heavy concrete slabs.
2. *Superior insulation.* The exterior walls comprising the ground floor envelope were approximately 50 mm thicker than the shells. The insulating property of masonry walling improves by using greater thickness; i.e., thicker building envelopes have higher thermal resistance (lower U-value/ higher R-value).
3. *Lesser exposure to direct solar radiation* (intensity and duration). During the peak-summer monitoring period, the edge beams and piers acted as vertical and horizontal shades, which reduced heat gain through the north and south-facing walls. The north-facing wall was shielded from direct sunlight for the majority of the day, apart from some mid-morning sun; whereas the south-facing wall had limited sun exposure in the early morning and late afternoon (Sunrise and sunset occur south of east and west in Johannesburg in the summer months). In contrast, due to the orientation of the structure and summer sun trajectory, both first-floor shells were directly exposed to solar radiation for the majority of daylight hours. This was particularly evident at the flatter portions, toward the apex of these shells, which received significant direct solar radiation (in early January the sun’s elevation is approximately 87° at midday). Peak internal surface temperatures in excess of 40°C have been recorded in this region (Bradley *et al.*, 2018a). The influence of direct solar radiation on the indoor operative temperatures within the building, and especially the first-floor spaces in the summer, is illustrated in the plots of Figures 8.10a and 8.10b. Figure 10a is for a completely clear day, whereas Figure 8.10b is for a mostly overcast late-summer day.

4. *Lower surface albedo*. Surface albedo has a significant impact on the extent of heat gain through the masonry envelope, as well as indoor temperatures, which is partly due to the extent of surface area exposed to direct solar radiation.

8.5.2 Surface albedo

High albedo lime-wash has been traditionally used on earthen masonry to limit solar gain through in hot and dry climates (Adam and Agib, 2001), but such finishes are unsuitable in Johannesburg's climate (Bradley and Gohnert, 2018). Using light coloured paints, or those incorporating infrared reflecting pigments, is a durable alternative for moderating heat gain through CSEB masonry (Bradley *et al.*, 2018a). Several surface colours were examined in the present study, as shown in Figure 8.11.



Figure 8.11 – Examined surface colour conditions

The influence of surface albedo, under hot and clear weather conditions, on the indoor air temperature in two conventional housing types situated in the neighbouring city of

Pretoria, South Africa (NBRI, 1987), is shown in Figure 8.12. The temperature inside the high-mass house was reduced by 3°C when exterior surfaces were painted white (NBRI, 1987). The effect of surface albedo on the light-weight structure was negligible, which the authors attributed to the well-insulated building envelope. The extent to which the indoor temperature was reduced within the prototype house at Wits by means of painting external surfaces white, is also illustrated in Figure 8.12. A significant reduction in peak temperature, approximately 5°C, was induced by increasing surface albedo as illustrated in Figures 8.12 and 8.13.

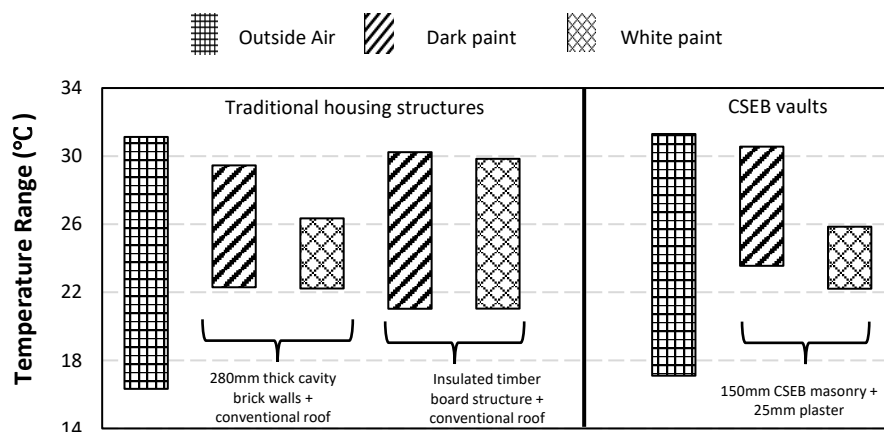


Figure 8.12 – Temperature range in traditional houses (NBRI, 1987) and the Wits vaults

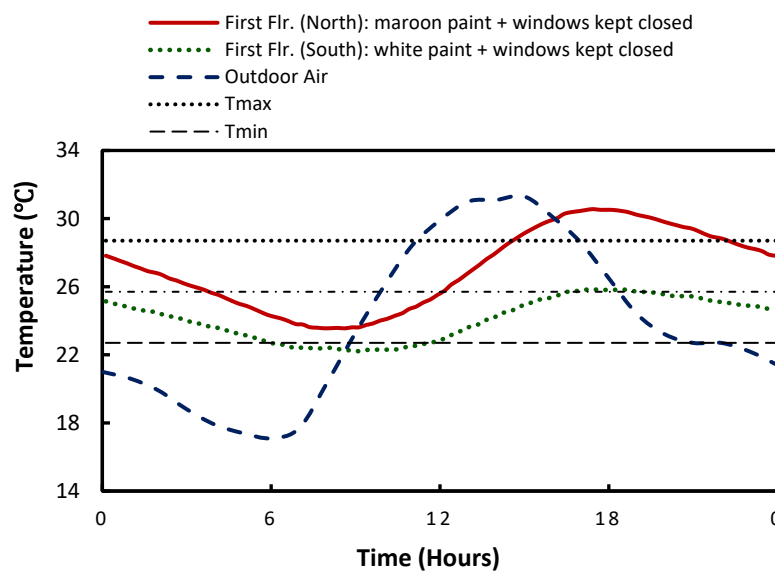


Figure 8.13 – Diurnal temperature range on 30th October 2018

Two additional black globe thermometers were positioned at 2 m and 3 m above the floor of the south vault to assess the variation of temperature with height. The diurnal temperature range at 1.1 m, 2 m, and 3 m meters for the white and maroon coloured surfaces with windows closed is shown in Figure 8.14. Peak temperatures, corresponding to the maroon surface, increased with height above the floor; the peak operative temperature, measured at 3 m, was typically approximately 3°C higher than that at 1.1 m within the maroon coloured vault. Several factors likely contributed to the observed disparity, such as warm air rising and collecting under the apex of the vault and the closer proximity of the uppermost black globe to the masonry envelope. Additionally, internal and external surface temperatures increased from the base to the apex of the maroon coloured shell in spring and summer (Bradley *et al.*, 2018a), which indicates higher heat gain through the upper portion of the shell. The minima and maxima internal surface temperatures, from southern base to apex, on a hot day in January, 2016, are illustrated in Figure 8.15. In contrast to the dark surface condition, there was practically no difference between operative temperatures measured within the vault painted white, as shown in Figure 8.14, and only minor variation between surface temperatures were recorded for those vaults. The dual effect of a shallow inclination and low surface albedo on the high rate of heat gain through the envelope in Johannesburg and their combination are, most likely, the cause of the vertical temperature variation.

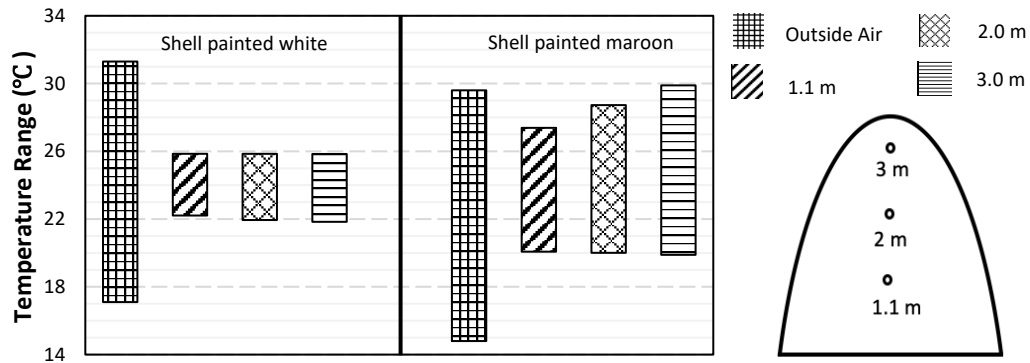


Figure 8.14 – Temperature range at 1.1 m, 2.0 m and 3.0 m above the floor

To examine the aforesaid postulate, the software package OPAQUE 3 (University of California, Los Angeles) was used to determine the influence of surface inclination on heat gain/loss through the shell envelope. Several inclinations of a CSEB masonry section, having either low or high surface albedo, were considered. The materials parameters

assumed in the analysis are shown in Table 8.5; thermal properties for soil:cement blocks were provided by the manufacturer (Hydraform). The internal temperature was fixed at 20°C and Sol-air temperature (an empirically derived temperature of a surface in sunlight) was automatically calculated by OPAQUE 3 using data from the weather-file for Johannesburg. The weather-file was created using the Meteonorm software package (Meteotest AG, Bern, Switzerland). The surface absorptivity, or the proportion of the incident solar radiation absorbed by the surface, corresponding to a high and low albedo finish was assumed to be 0.26 for white paint and 0.70 for dark paint. Several assumptions were made in these analyses, such as a constant indoor temperature of 20°C, but the exercise was still valuable to elucidate the experimental observations.

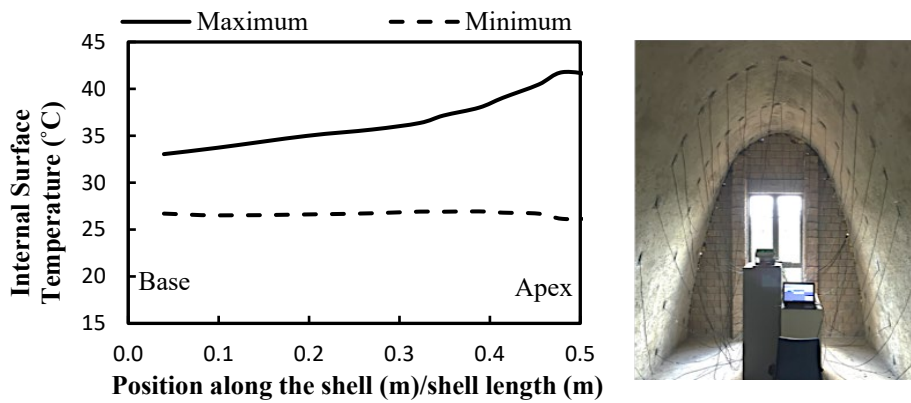


Figure 8.15–Maxima and minima internal surface temperatures on 30th January 2016

Table 8.5 – OPAQUE 3 R-value calculations

Material	Thickness (mm)	Density (kg/m ³)	Thermal cond. (W/m.K)	Specific heat (kJ/kg.K)	R-value (m ² .K/W)
Outside air film					0.04
CSEB masonry	150	1900	0.72	0.85	0.21
Cement:sand plaster	25	1500	0.50	1.00	0.05
Inside air film					0.12
Total	175				0.42

The range of heat loss and gain for several surface inclinations of CSEB masonry envelope, with R-value as per Table 8.5, for a typical summer's day is shown in Figure 8.16. When the exterior surface had high absorptivity; i.e., the maroon paint, significant disparity

between peak heat gain values corresponding to the vertical and horizontal inclinations was observed. In contrast, the difference was negligible for the low surface absorptivity coating.

These analyses indicate that the proportion of incident solar radiation absorbed by the shell, and consequent heating of the indoor space, is affected more by inclination when the surface albedo is low.

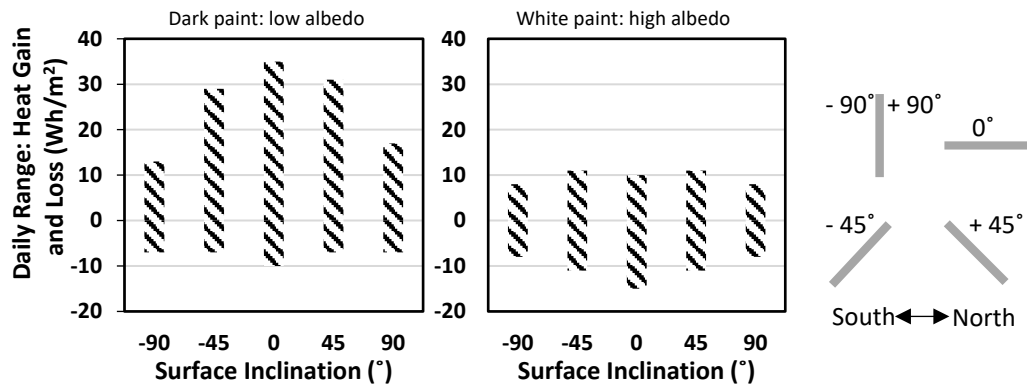


Figure 8.16 – Comparison of diurnal heat gain/loss for low and high albedo surfaces: Jan.

8.5.3 Natural ventilation

8.5.3.1 General

A primary mechanism of improving thermal comfort in housing, and especially low-cost structures, in South Africa is by opening and closing windows to naturally ventilate the space. Improvements can be achieved through air circulation to cool, and more rarely warm, the living space. Furthermore, air movement acts to strip away the warm air from the surface of the body and is therefore important for convective heat loss (CIBSE TM52, 2013). In a similar way, air movement also promotes evaporation of perspiration from the skin (NBRI, 1987). Bhikhoo *et al.* (2017) reported that air movement of up to 1 m/s can reduce indoor temperatures by as much as 3.5°C in hot climates. The effectiveness of natural ventilation is however dependent on the climate, as well as several design factors such as the internal layout of the building and the distribution, size and number of openings. A critical factor being that there are adequate inlets and outlets for the ventilation air (NBRI, 1987).

The windows in the first-floor vaults were positioned in the east and west-facing gable walls, which was ideal for natural ventilation during the summer. Hourly wind direction

data for a typical year, acquired from Meteonorm, shows that the prevailing wind comes from the east during the summer months in Johannesburg. Specifically, an average of 70% of the hourly wind direction data falls between north-east and south-east. In addition, the gable walls in the south vault (SV) had equally sized windows, on opposite sides of the buildings, which was best for the cross-ventilation. Furthermore, the long narrow floor plan assisted with cross-ventilation in the temperate climate of Johannesburg (Climate Consultant 6.0, University of California, Los Angeles).

8.5.3.2 Maroon surface

When the windows were kept closed during early summer, there was negligible temperature disparity between the vaults as shown in Figure 8.17a. This was expected because neither vault shaded the other for most of the day, as the sun rises and sets very close to east and west in early to mid-October, as well as has much higher altitude angle than that in the winter. The abovementioned similarity (Figure 8.17a) enabled a direct assessment of ventilation, and several window opening/closing scenarios were considered. The results of several of these scenarios are presented in Figures 8.17b through 8.17f, for the maroon surface condition.

The opening of windows in the late afternoon, or evening, when the outside air temperature was less than that of the indoors, was very effective at reducing operative temperatures. The temperatures corresponding to a set-up where windows were continuously open are shown in Figure 8.17c, which was similarly effective to night ventilation as indicated by the temperatures shown in Figure 8.17d, despite the windows being unshaded (i.e., with solar gain through the west facing window). A 2°C to 3°C reduction in the indoor operative temperature was induced in the first-floor vault if the windows remained open at night as indicated by the temperatures shown in Figures 8.17b through 8.17d. The temperature disparity occasionally exceeded 5°C when the windows were open during afternoon thunderstorms. Opening windows when the outside temperature was greater than that of the indoors offered no benefit in reducing indoor temperatures but may be necessary for indoor air quality. In some cases, a rapid increase in temperature was observed, as shown in Figures 8.17e and 8.17f. When windows were closed, whilst outdoor temperatures exceeded those of the indoors, the operative temperature typically fell sharply before gradually increasing again.

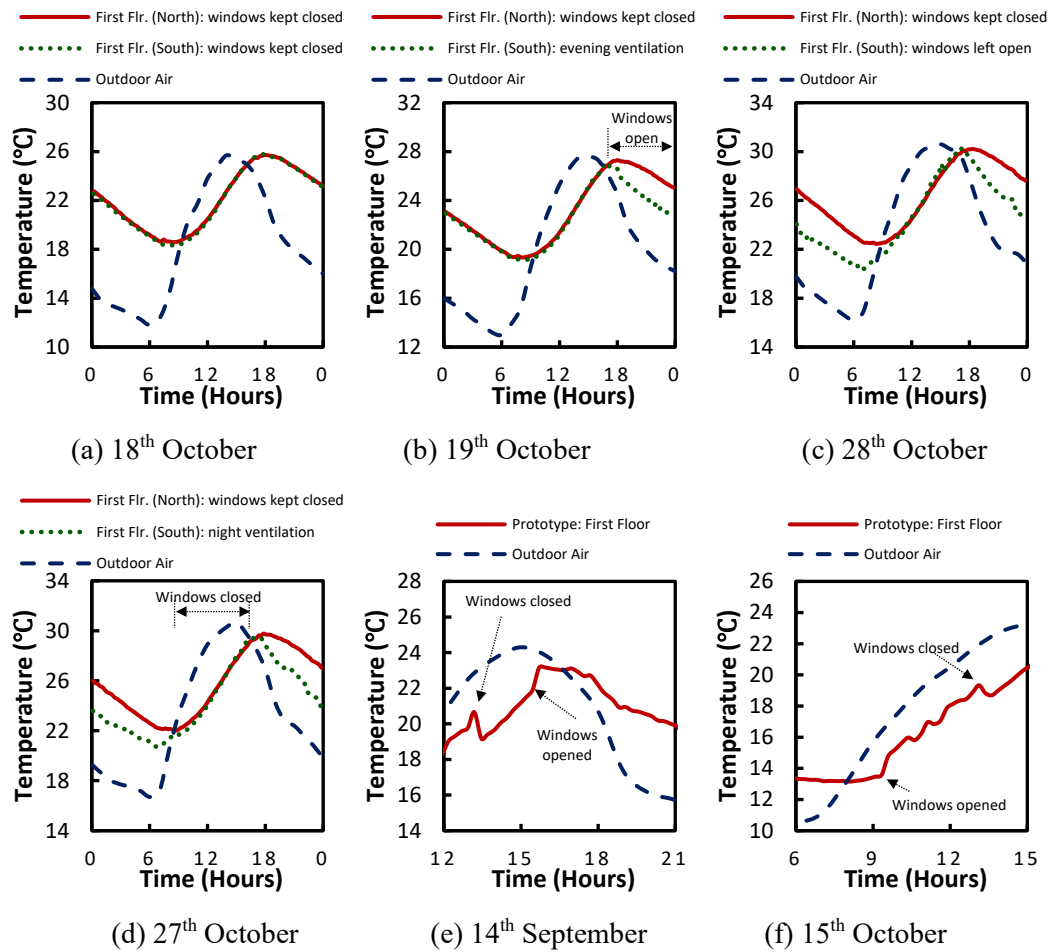


Figure 8.17 – Exterior and interior temperatures on several summer and spring days, 2018

The results shown in Figures 8.17 highlight the benefit of natural ventilation with respect to modifying indoor operative temperatures. However, the maximum temperature recorded when windows were open in the late afternoon/evening was similar to that for the scenario with windows kept closed.

8.5.3.3 Dark grey surface

The results of the analysis with a dark grey surface are shown in Figures 8.18a – 8.18c. Ventilation resulted in lower indoor temperatures during the late afternoon and evening in the vaults. Indoor temperatures corresponding to the dark grey surface were generally a few degrees higher than those for the maroon surface. Ventilation resulted in an even greater reduction in operative temperature in the afternoon and evening, usually between 3°C to 4°C. Unlike the maroon coloured vaults, peak temperatures in the ventilated dark grey coloured vaults were typically several degrees cooler than the unventilated case. In

fact, keeping windows continually open on a hot summer's day resulted in more comfortable hours during the afternoon, evening and at night.

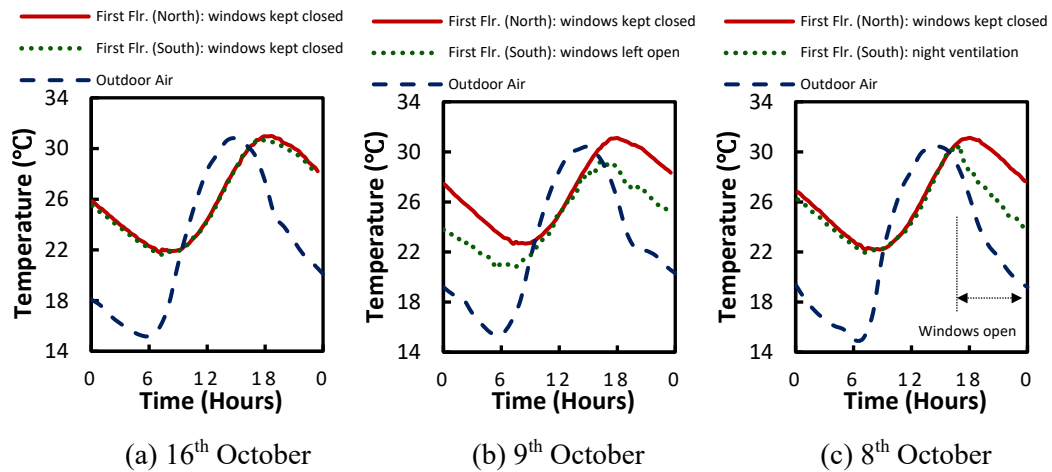


Figure 8.18 – Exterior and interior temperatures on several summer and spring days, 2019

8.5.3.4 White surface

The 24h temperature distributions, corresponding to either windows open or closed for both vaults painted white on several summer days are presented in Figure 8.19. Afternoon temperatures were slightly higher in the south vault when windows were kept closed, which can be ascribed to passive solar gain through the west-facing window of the south vault as well as partial shading of the north vault's surface (in mid-summer the sun rises and sets south of east and west). Unlike the previous roof conditions, the indoor temperatures were several degrees higher when the windows were opened on hot summer days (Figure 8.20). The maximum difference between the ventilated and unventilated spaces was typically between 2°C and 4°C.

Similar to the dark surface conditions, the indoor temperature reduced rapidly when the vaults were naturally ventilated in late afternoon through early evening due to the cooler outdoor air entering the space. The temperature was however still lower during this period when windows were kept closed. The temperature in the ventilated space generally dropped below that of the unventilated space after 20:00hrs. After 20:00hrs, the disparity between first-floor spaces (ventilated and unventilated) was usually negligible as shown in Figures 8.20a – 8.20c.

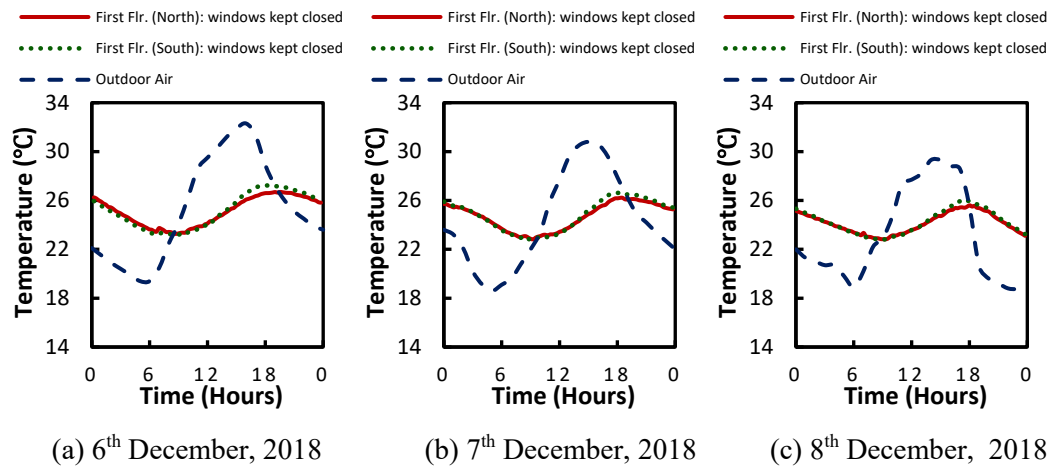


Figure 8.19 – Passive solar gain through the west-facing window

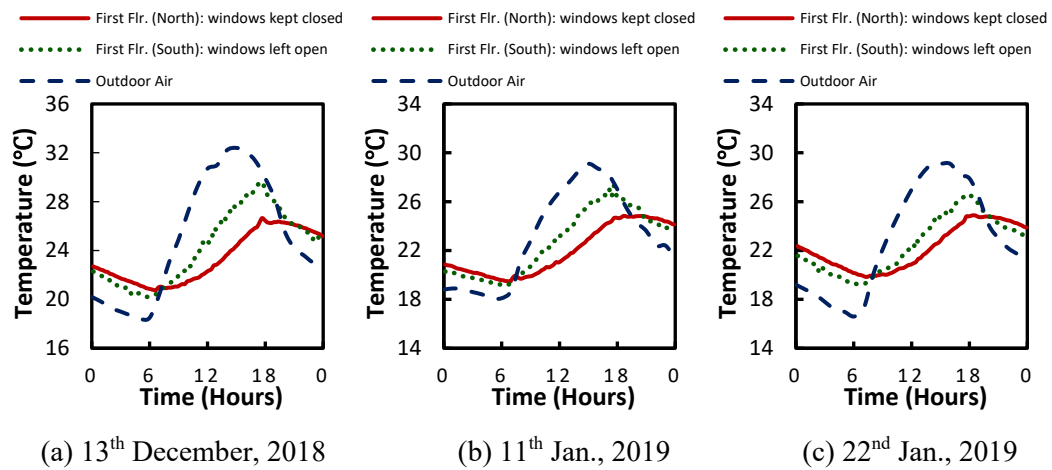
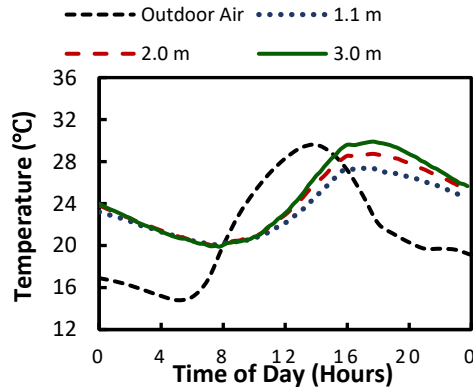


Figure 8.20 – Exterior and interior temperatures on several summer and spring days

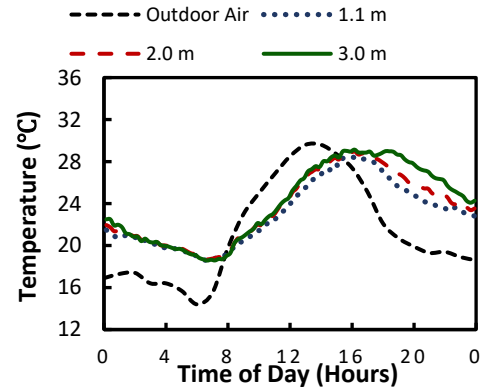
8.5.3.5 Temperature variation with height

The temperature distributions at 1.1 m, 2 m, and 3 m metres for a day when the windows were kept closed, for the maroon paint condition, are shown in Figure 8.21a. As discussed previously, the peak operative temperature measured at 3 m was approximately 3°C higher than that at 1.1 m. When windows were left open, the maximum difference between the upper and lower black globes was similarly about 3°C, as shown in Figure 8.21b. However, unlike the gradual divergence observed when windows were kept closed, the temperature disparity changed abruptly in the early evening. Prior to such sudden change, the temperature difference between upper and lower measurement points was typically less than 1°C. The abrupt change can be attributed to an increase and decrease in temperature measured at 3 m and 1.1 m, respectively. A similar trend, albeit with larger temperature

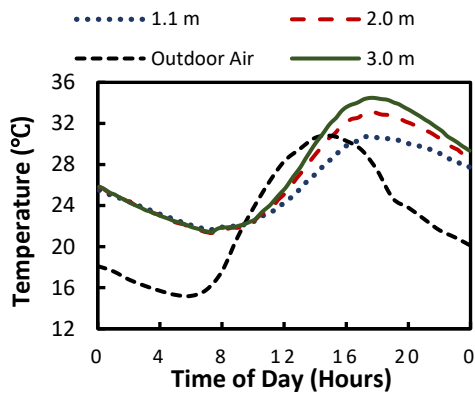
disparity between 1.1 m and 3.0 m, was observed for the dark grey surface, as shown in Figures 8.21c and 8.21d. There was only marginal temperature variation with height within the shell painted white, irrespective of whether windows were open or closed, as shown in Figures 8.21e and 8.21f.



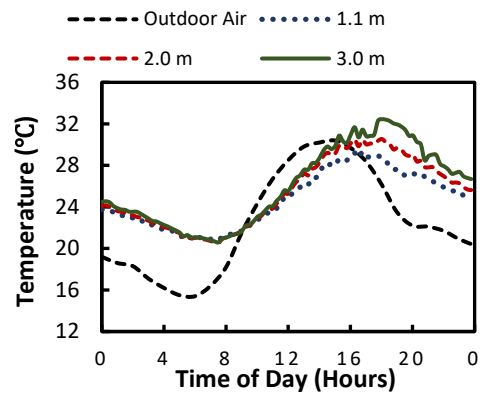
(a) Windows closed: maroon paint



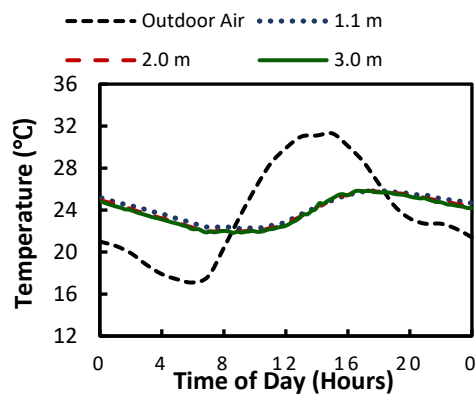
(b) Windows open: maroon paint



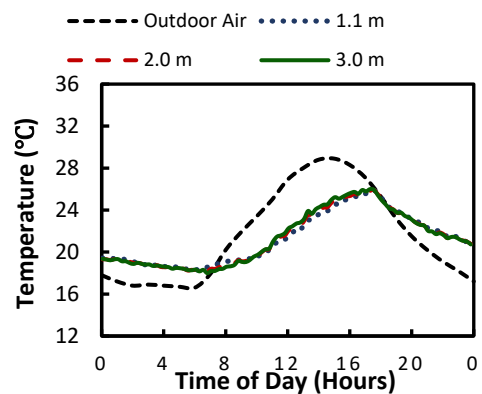
(c) Windows closed: dark grey paint



(d) Windows open: dark grey paint



(e) Windows closed: white paint



(f) Windows open: white paint

Figure 8.21– Exterior and interior temperatures on several summer and spring days

In the case of the maroon and dark grey surface conditions, the operative temperature recorded at 1.1 m was typically a few degrees cooler than those at 2 and 3m during the evening, when occupants were more likely to be at home.

8.5.4 Assessing the effectiveness of natural ventilation and surface albedo with the CIBSE TM52:2013 overheating criteria

8.5.4.1 General

The potential for overheating within the prototype house was assessed without considering the influence of internal heat gains from people and appliances. Some spaces were occupied on several occasions during the assessment period, and heat gains from electrical sources were considered to estimate their influence on indoor operative temperature. Realistic heat gains from people, lighting, etc., had little influence when the space was naturally ventilated. Also, no attempt was made, on the majority of summer days, to inhibit direct solar radiation through the east and west-facing windows. The resultant solar heat gains certainly contributed to higher measured indoor temperatures. In South Africa, maximum solar radiation is transmitted through east and west-facing windows in the summer (NBRI, 1987). Furthermore, the penetration of solar radiation through west-facing windows occurs in the afternoon, when spaces are usually most susceptible to overheating. Due to the uncertainty in internal and solar heat gain through the windows, a conclusive deduction on overheating is not made. Rather, the criteria are implemented to highlight potential problems with certain spaces, and examine the effectiveness of several passive strategies to improve comfort.

8.5.4.2 Overheating assessment

The ground floor spaces were not assessed on the basis of overheating criteria because the indoor operative temperature did not exceed $T_{\max}$ during the monitoring period, which were Jan. 2018 to Mar. 2018, Oct. 2018 to Feb. 2019, and Oct. 2019 to Dec. 2019. In fact, the majority of peak daily operative temperatures were consistently lower than T_{comf} ; apart from those measured during several periods of prolonged heat, as indicated in Figure 8.22. Except from these spaces, $T_{\max}$ was exceeded on a number of days inside the dark coloured vaults during summer, as indicated in Figure 8.22.

The influence of surface albedo and natural ventilation on indoor temperatures have been discussed previously; but their effectiveness is evaluated herein using the CIBSE overheating criteria. These criteria are assessed based on when a space is occupied; by varying a room's occupied hours or classification (e.g., bedroom versus living room) dissimilar conclusions from the overheating assessment can be obtained, i.e., either overheating or not overheating. In this study two occupancy patterns were used; namely, continually occupied and only occupied between 8:00 PM and 6:00 AM (i.e., a night-time space).

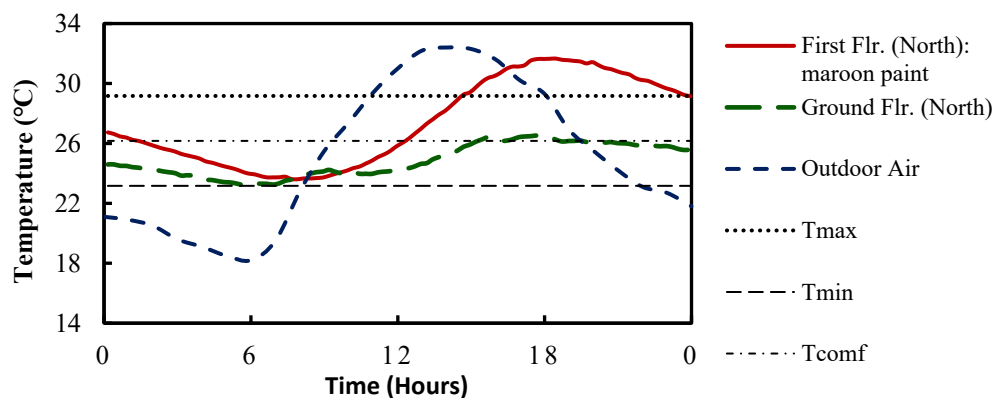


Figure 8.22 – Temperatures on 16 Nov. 2018: Maroon roof coating; windows closed

The results of the overheating assessment for the aforesaid occupancy periods are presented in Tables 8.6 through 8.9. The maximum operative temperatures were measured at a height of 1.1 m above the floor. Each setup was surveyed for at least 50 summer days, including a minimum of one heat wave. The evaluations of Criteria 2 and 3 during the aforesaid periods of extreme heat are illustrated in Figures 8.23 and 8.24.

8.5.4.2 Discussion: maroon surface

The continually occupied vaults overheated when the windows were kept closed during the afternoon and evening, as shown in Table 8.6. If these rooms were designated as night-time spaces (i.e., bedrooms), occupied between 8:00 PM and 6:00 AM, then only Criterion 2 was exceeded on 17th of November 2018. When the spaces were naturally ventilated, and assumed occupied at all times, then Criterion 2 failed on several occasions in January 2018. In contrast, in the naturally ventilated first-floor spaces neither criterion was exceeded when these spaces were designated as bedrooms. Criterion 2 was the most likely to be violated for these maroon-coloured vaults. Opening the windows in the late afternoon and

evening greatly reduced the risk of spaces failing Criteria 1 and 2, as indicated in Figures 8.25a and 8.25b, because natural ventilation reduced the period that T_{op} was greater than T_{max} by as much as 3.5 hours. Furthermore, the temperatures did not exceed T_{max} after 8:00 PM, and this same result was obtained for all days during which windows were open during late afternoon and early evening.

Table 8.6 – Overheating assessment: assuming continually occupied and windows closed

	Max T_{op}	Exceed T_{max}	Criterion 1	Criterion 2	Criterion 3	Overheat
Maroon surface (101 days)	32.4°C (90.3°F)	Yes	Fail (3.8%)	Fail (We=19.25°C.h)	Pass ($\Delta T=3^{\circ}\text{C}$)	<u>Yes</u>
White surface (82 days)	28.6°C (83.5°F)	No	<i>NA</i>	<i>NA</i>	<i>NA</i>	No
Dark grey surface (70 days)	34.8°C (94.6 °F)	Yes	Fail (14.9%)	Fail (We=29.75°C.h)	Fail ($\Delta T=5^{\circ}\text{C}$)	<u>Yes</u>

Table 8.7 – Overheating assessment: assuming continually occupied and windows open

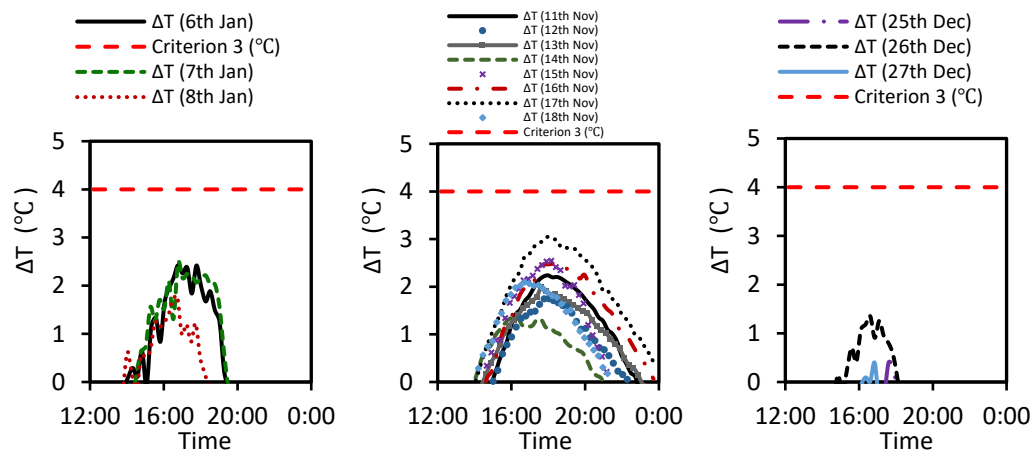
	Max T_{op}	Exceed T_{max}	Criterion 1	Criterion 2	Criterion 3	Overheat
Maroon surface (56 days)	32.0°C (89.6°F)	Yes	Pass (1.4%)	Fail (We=8.25°C.h)	Pass ($\Delta T=3^{\circ}\text{C}$)	No
White surface (61 days)	31.1°C (88.0°F)	Yes	Pass (0.2%)	Pass (We=2.25°C.h)	Pass ($\Delta T=1^{\circ}\text{C}$)	No
Dark grey surface (62 days)	33.9°C (93.0°F)	Yes	Fail (6.6%)	Fail (We=15.00°C.h)	Fail ($\Delta T=4^{\circ}\text{C}$)	<u>Yes</u>

Table 8.8 – Overheating assessment: assuming occupied between 8 PM and 6 AM and windows closed

	Max T_{op}	Exceed T_{max}	Criterion 1	Criterion 2	Criterion 3	Overheat
Maroon surface (101 days)	31.9°C (89.4°F)	Yes	Pass (2.2%)	Fail (We=6.25°C.h)	Pass ($\Delta T=3^{\circ}\text{C}$)	No
White surface (82 days)	28.5°C (83.3°F)	No	<i>NA</i>	<i>NA</i>	<i>NA</i>	No
Dark grey surface (70 days)	34.1°C (93.4°F)	Yes	Fail (11.7%)	Fail (We=7.75°C.h)	Fail ($\Delta T=4^{\circ}\text{C}$)	<u>Yes</u>

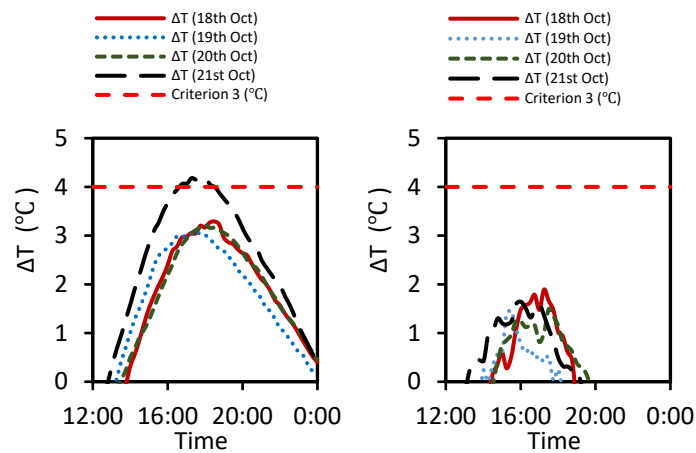
Table 8.9 – Overheating assessment: assuming occupied between 8 PM and 6 AM and windows open

	Max T _{op}	Exceed T _{max}	Criterion 1	Criterion 2	Criterion 3	Overheat
Maroon surface (56 days)	29.1°C (84.4°F)	No	NA	NA	NA	No
White surface (61 days)	27.8°C (82.1°F)	No	NA	NA	NA	No
Dark grey surface (62 days)	31.1°C (88.0°F)	Yes	Pass (0.6%)	Pass (We=1. 5°C.h)	Pass (ΔT=1°C)	No



(a) Maroon, windows open (b) Maroon, windows closed (c) White, windows open

Figure 8.23 – Overheating assessment during heat waves in Jan., Nov., and Dec. 2018



(a) Dark grey, windows closed (b) Dark grey, windows open

Figure 8.24 – Overheating assessment during a heat wave in Oct. 2019

The effectiveness of cross-ventilation can be attributed to a large outdoor diurnal temperature range, building layout and the positioning of fenestrations. Windows were equally sized, situated on opposite sides of the buildings, and faced the direction of the prevailing wind (wind largely comes from the east during the summer months in Johannesburg). Furthermore, the vaults had a long narrow floor plan, without any internal partitions, which assisted with cross-ventilation, particularly in temperate and hot-humid climates (Climate Consult. 6.0).

The results of the third overheating assessment, unlike the first and second, were not markedly affected by ventilation. For example, the peak temperatures had similar magnitude in the unventilated and naturally ventilated spaces, as illustrated in Figures 8.25a and 8.25b. Although Criterion 3 was not violated, the measured peak operative temperatures were only within 1° to 2°C of exceeding $\Delta T = 4^\circ$, as indicated in Figures 8.23a and 8.23b. The 1° to 2°C excludes the potential increase of temperature associated with internal heat gains.

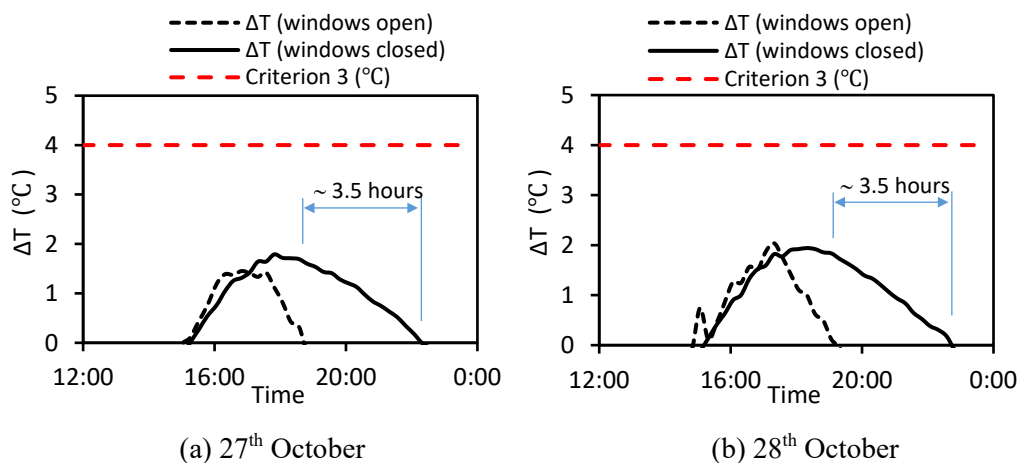


Figure 8.25 – Assessment of Criteria 2 and 3 in Oct. 2018: Maroon surface

The close “pass” of Criterion 3 is especially concerning because it is an upper limit covering extreme weather conditions (e.g., heat waves) and future climate situations. The authors of the South African Risk and Vulnerability Atlas (Department of Science and Technology, 2010) reported, based on projections obtained from a dynamic regional climate model (Engelbrecht *et al.*, 2009), that future warming is expected to be the greatest over inland South Africa. The same model, assuming moderate to high growth in greenhouse gas concentrations, showed that temperatures may increase inland South Africa

by approximately 3°C by mid-century. Based on these considerations, as well as the overheating assessment presented in Tables 8.6 and 8.7, a masonry shell thickness of 175 mm in combination with a low emissivity surface are unsuitable in regard to overheating if the space is occupied during daylight hours.

8.5.4.3 Discussion: dark grey surface

The roof of the structure was coated with a dark grey paint on 9th September 2019 (early spring). This surface condition performed the worst during the summer months, and the north vault failed the second overheating criterion ($W_e = 8^\circ\text{C.h} > 6^\circ\text{C.h}$) shortly after painting on 11th September 2019. More significant than the amount by which the criterion was exceeded, was that September falls outside the peak summer period. For comparison, the vaults coated with maroon paint first failed Criterion 2 on 28th October 2018 (excluding the summer period between 1st January 2018 and 31st March 2018). Having said this, October 2019 was significantly hotter than normal, as indicated in Figure 8.9, due to the late arrival of summer rainfall in 2019. Nevertheless, the dark grey surface eventually caused the first-floor spaces to fail Criteria 2 and 3 in early summer when unventilated, as shown in Tables 8.6 and 8.8 and Figure 8.24a. When the first-floor space was assumed continually occupied, the limit of 3% corresponding to Criterion 1 was greatly exceeded during the summer period. Natural ventilation was very effective at reducing ΔT and resulted in improvements regarding all overheating criteria. However, the first-floor space still overheated when assumed continually occupied and ventilated.

8.5.4.3 Discussion: white surface

Painting the shell white and ensuring windows were closed from mid-morning through to the late afternoon kept the operative temperature below T_{max} . Such a strategy has been used extensively in places with hot-dry climates, such as the Middle East, to keep indoor spaces cool and/or reduce cooling demand. The performance of the white coating was also far superior to that of the maroon coating when the space was ventilated during the day, and improvements were observed regarding all three overheating criteria, including a reduction of ΔT . However, the indoor operative temperature exceeded T_{max} on several occasions in late 2018, but these ventilated spaces still passed all three overheating criteria, as shown in Table 8.8 and Figure 8.23c.

8.5.5 Benefits associated with the summer climate in Johannesburg

The summer climate on the Highveld contributed to lessening the potential of overheating within the prototype house. In the summer months, there is significantly more cloud cover than in the winter, as indicated in Table 8.3, which assists in reducing the amount of direct solar heat gain through exposed parts of the building envelope. Furthermore, there are about 100 days with rain per annum in Johannesburg (Kruger, 2004), most of which occur in the summer months (Dyson, 2009). According to Kruger (2004), the maximum rainfall over Johannesburg occurs during December and January. These summer conditions are coupled with cyclical temperature patterns, where prolonged periods of summer heat do not persist: apart from occasional extreme weather events such as heat waves and drought.

Heat induced thunderstorms also occur regularly on the Highveld during summer (Engelbrecht and Landman, 2018), which can moderate high temperatures in the late afternoon and evening. For example, 28% of peak summer days (1st January 2018 to 28th February 2018 and 1st November 2018 to 28th February 2019) monitored at the prototype house had some afternoon and/or evening rain. The rain accompanying these short duration storms cool the masonry envelope (Bradley *et al.*, 2018a), leading to lower indoor temperatures, as can be observed by comparing Figure 8.26a (no thunderstorm) and Figure 8.26b (with an afternoon thunderstorm). An even greater reduction of indoor temperature was observed when windows were open during these storms, due to reduced outdoor air temperature and wind; e.g., the indoor temperature in the first-floor vault (south) fell by approximately 6°C during a brief thunderstorm on 28th September 2018, as shown in Figure 8.26c. The indoor temperature also increased rapidly, after an earlier decline, when the windows were closed directly after a brief afternoon storm on 15th March, as shown in Figure 8.26d.

8.6 Results and discussion: winter

8.6.1 Performance: maroon roof

The indoor operative temperature plotted against outdoor running mean temperature, between 15 May 2018 (early winter) and 15 August 2018 (late winter), for the ground and first-floor rooms are shown in Figure 8.27. Additionally, the indoor operative temperature and outside air temperature distributions for two typical winter days are shown in Figure 8.28a, when windows kept closed, and Figure 8.28b, when windows kept open during the

day. Note that the majority of winter days in Johannesburg are sunny and clear. The results show that the first-floor spaces experienced greater diurnal temperature variation than those at ground floor. For the peak winter monitoring period (15 May to 15 August), indoor diurnal temperature ranges were largest for the northern vault (Mean = 6.2°C; Stdev = 1.2°C), followed by the southern vault (Mean = 4.9°C; Stdev = 1.0°C), then the northern ground floor (Mean = 3.2°C; Stdev = 0.8°C), and finally the southern ground floor (Mean = 1.8°C; Stdev = 0.5°C). The larger diurnal range for the first-floor spaces can be attributed to greater heat gain during the day and heat loss at night.

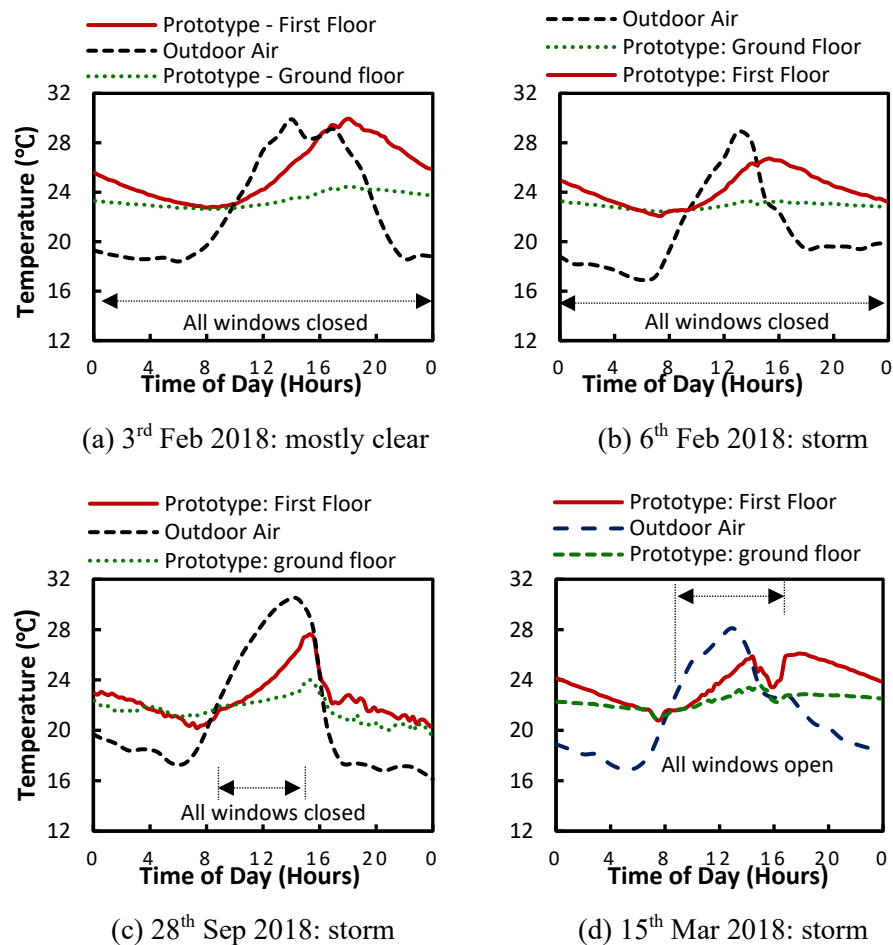


Figure 8.26 – Operative temperatures on days with/without thunderstorms in 2018

Temperatures measured in the northern side of the building were also, mostly, greater than those in the southern rooms. In fact, the mean operative temperature for a typical winter's day in the northern vault and ground-floor room were roughly 3°C and 1.5°C greater than their southern equivalents, respectively. The temperature disparity between rooms on the

north and south sides of a building was anticipated, and associated with the quantity of direct solar radiation (i.e., the extent of solar exposure of the building envelope). Of particular importance are the indoor temperatures in the house during weekday evenings, between 17:00 and 21:00, when occupants are most likely to be at home, and energy demand is the greatest (Guedes, 2015; Sigauke, 2014). It should be noted that space heating is one of the largest sources of electricity consumption in the home during winter in some regions of South Africa (Eskom, n.d.). Operative temperature measured within in the northern vault was significantly higher than those in other internal spaces between 17:00 and 21:00, as illustrated in Figures 8.28a and 8.28b. The significant discrepancy between the temperature in first-floor spaces is attributed to surface shading.

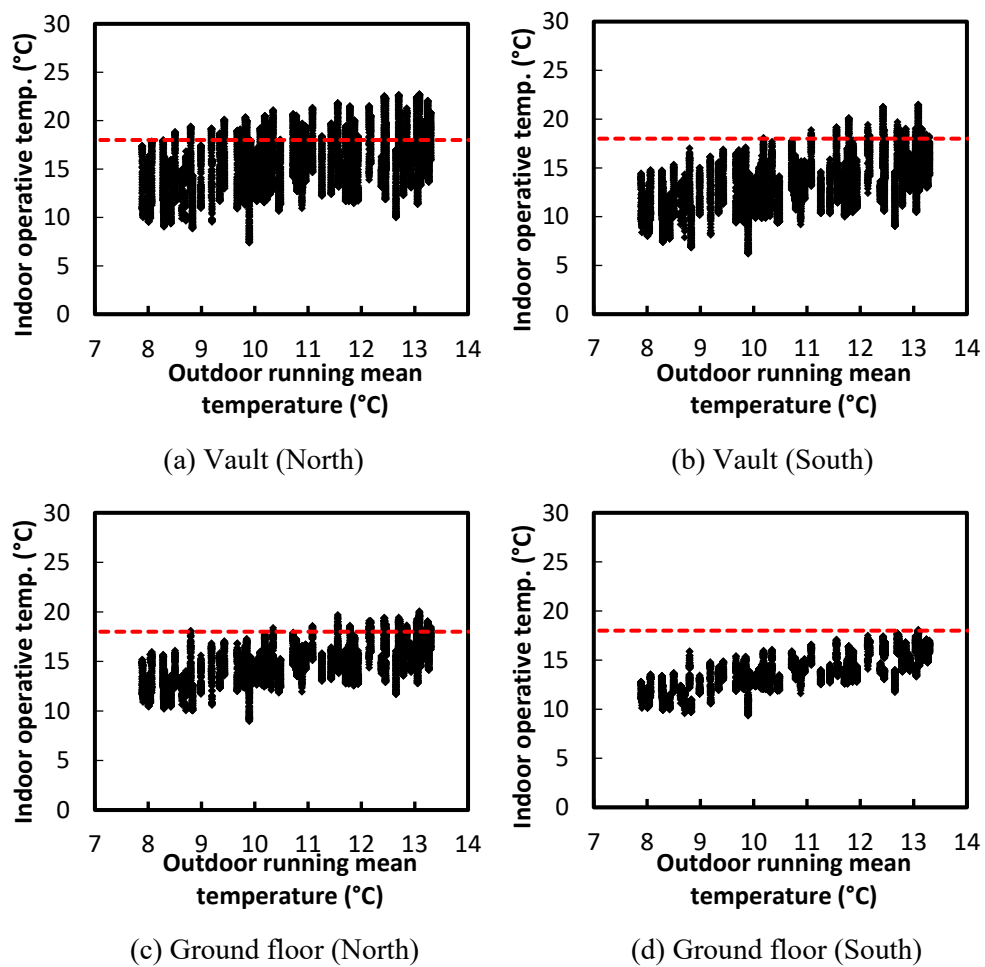


Figure 8.27 – Outdoor running mean temperature versus indoor operative temperature

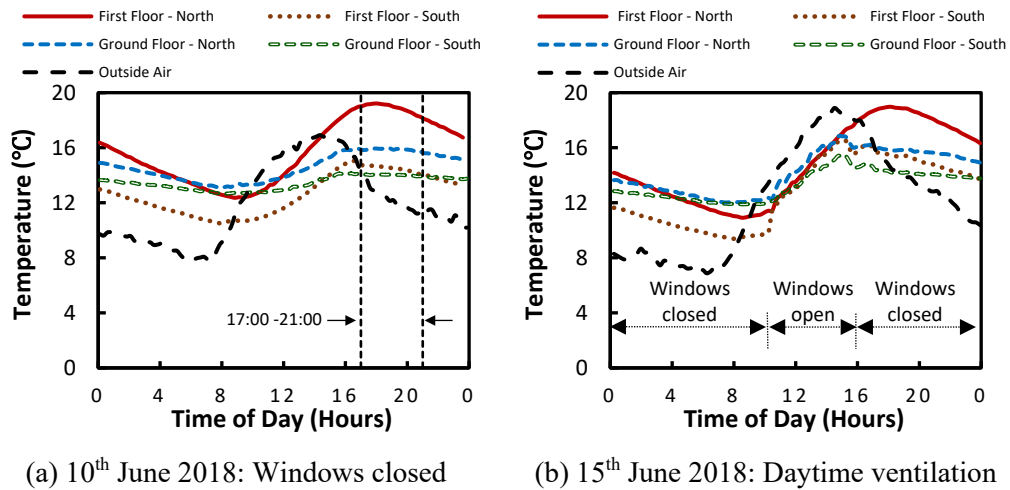


Figure 8.28 – Diurnal temperature range on two days in winter, 2018

8.6.2 Performance: white roof

Incident solar radiation is absorbed at the surface of the shells, which generates heat that is transferred by thermal conduction through the masonry. The rate of heat transfer through sunlit elements is governed by their interior surface and exterior surface temperatures rather than the air-to-air temperature difference across the element (NBRI, 1987). High albedo paints act to reflect solar radiation, resulting in cooler surface temperatures and, ultimately, reduced heat gain. The benefit observed for the maroon-coloured vaults, between 17:00 and 21:00, was no longer present when the shells were painted white, as shown in Figures 8.29a and 8.29b. Furthermore, minima temperatures recorded in the early morning were also several degrees cooler than those corresponding to the maroon coloured vaults.

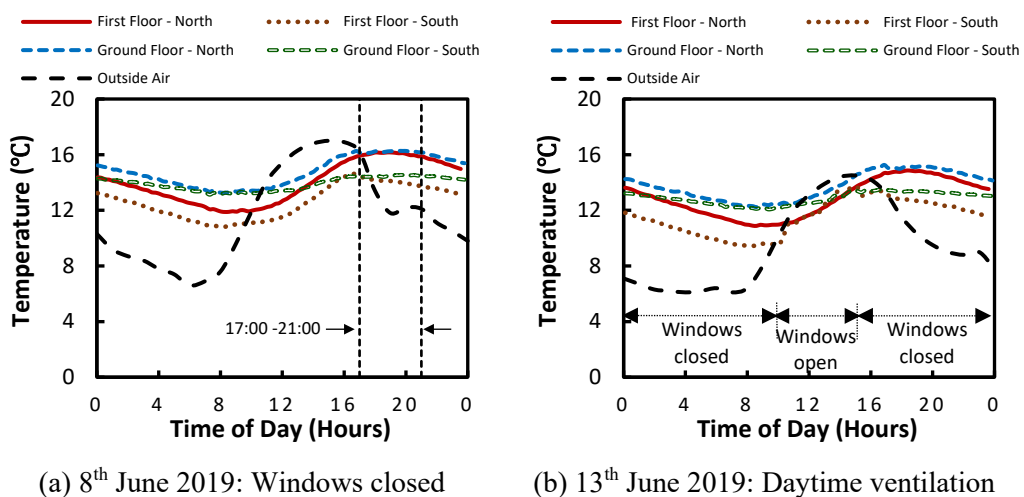


Figure 8.29 – Diurnal temperature range on two days in winter, 2019

Based on the performance of the white surface condition in the winter, it is unlikely that adopting high albedo paints over uninsulated masonry shells is beneficial in Johannesburg; although Johannesburg is only a few hundred kilometres from the Tropic of Capricorn, its high elevation means temperatures are several degrees cooler than neighbouring cities. However, the results presented herein indicate the role surface albedo can play in affecting the thermal performance of spaces enclosed by thin masonry shells.

8.6.2 Shading

Passive solar heating is tremendously beneficial inland South Africa where winters are predominately sunny and clear. Shading of the building envelope, in a similar manner to high albedo paint, acts to negate the observed positive effects of passive solar gain. The sunlit and shaded parts of the prototype's envelope at midday on the 6th June 2018 are shown in Figure 8.30. The majority of the south facing portions of both vaults, apart from a small region near the apex, were shaded throughout the day (i.e., auto-shading). At midday, when the sun was at its zenith, about 45% of the north vault's surface was shaded. The south vault was partly shielded by the north vault, which resulted in approximately 70% of its surface to be shaded at midday. The majority of the south shell surface was shaded in the early morning and late afternoon.



(a) Structure from the north-west

(b) Structure from the south-east

Figure 8.30 – Sunlit and shaded regions in winter (12:00PM, 9th June 2018)

The surface shading resulted in significantly lower temperatures in the south vault during the mid-winter period, as shown in Figures 8.31a and 8.31b. Operative temperature was frequently between 4°C to 5°C warmer in the maroon-coloured north vault during the evening on a typical winter's day. The variance between the first-floor spaces was only 2°C

to 3°C in magnitude for the white surface condition, which can be attributed to the effect of surface albedo on the rate of heat gain through the sunlit parts of the masonry envelope. This trend for a winter month is illustrated in Figure 8.34. The plot was developed using OPAQUE 3, with all assumptions and materials characteristics outlined in section 8.5.2. For comparison with the results presented for mid-winter, the temperature differences corresponding to other periods throughout 2018 are shown in Figures 8.32a through 8.32d. The maximum temperature difference between vaults was regularly less than 0.5 °C during the peak summer period as indicated in Figure 8.32c. For the ground floor spaces, the disparity was routinely between 1.5 °C and 2°C during winter, irrespective of the vault's surface colour/condition, as shown in Figure 8.33.

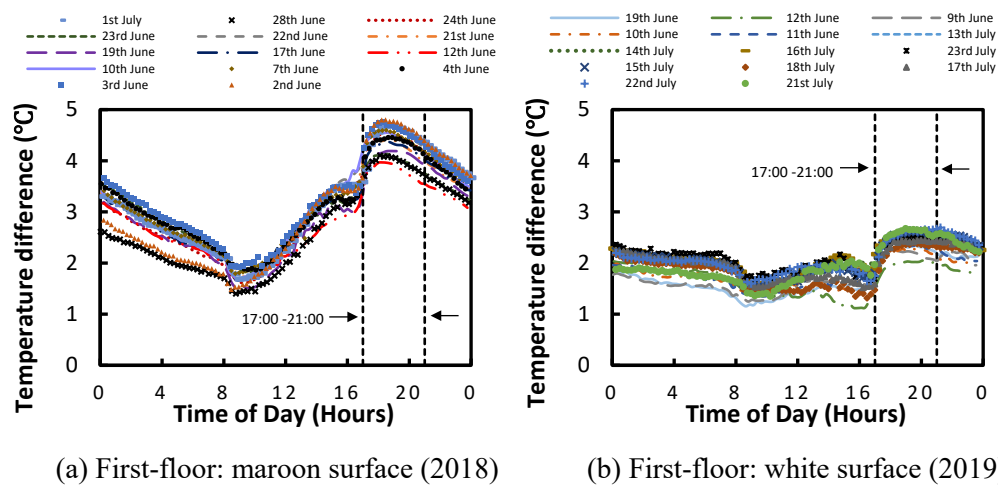


Figure 8.31 – Temperature difference between the northern and southern portions of the building in mid-winter (windows closed)

The large difference between the north and south spaces for the maroon-coloured masonry vaults can be largely attributed to the locations of surface shading and temperature measurement (i.e., at 1.1 m above the floor). More specifically, the zone of shading over north facing part of the south vault, as shown in Figure 8.3, would experience some of the highest solar gain if it were sunlit in mid-winter. This is illustrated in Figure 8.35, derived using OPAQUE 3, and further supported through internal surface temperature measurements inside the north and south parts of the structure, as shown in Figure 8.36. These observations highlight that shading of external surfaces, particularly the lower north facing portion, is detrimental to the thermal performance of the structure during winter.

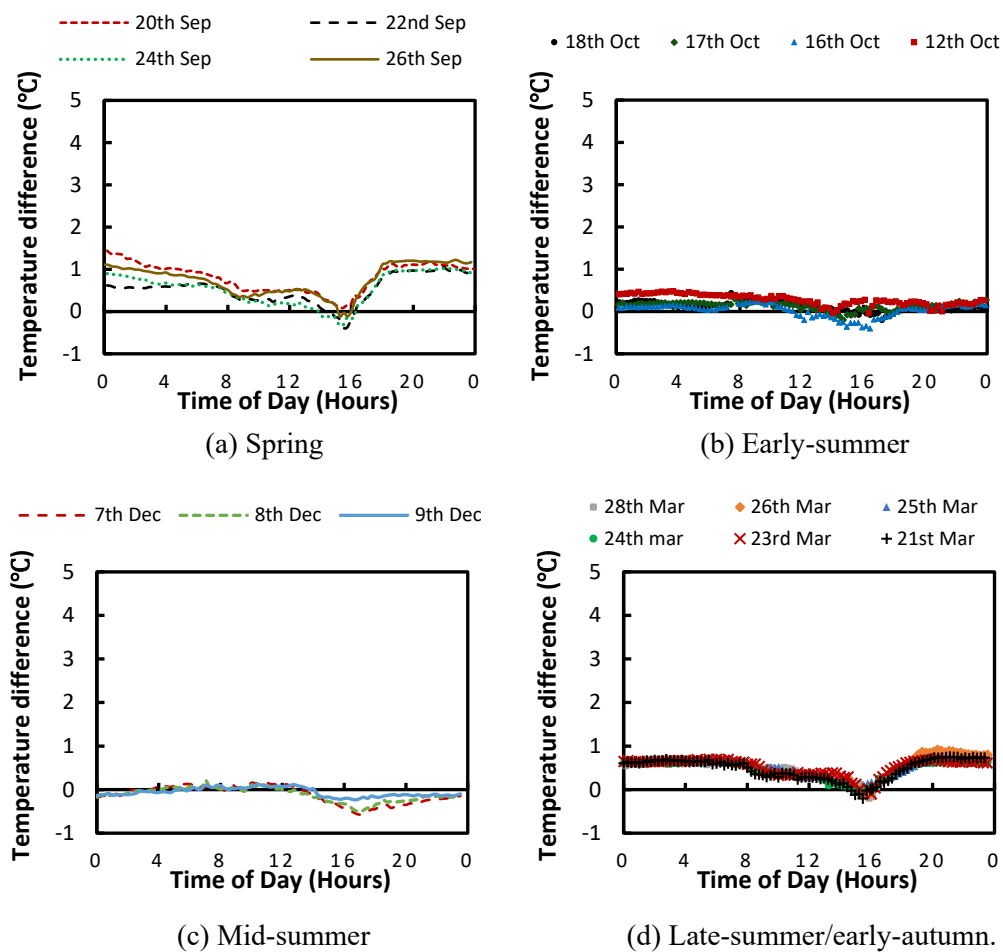
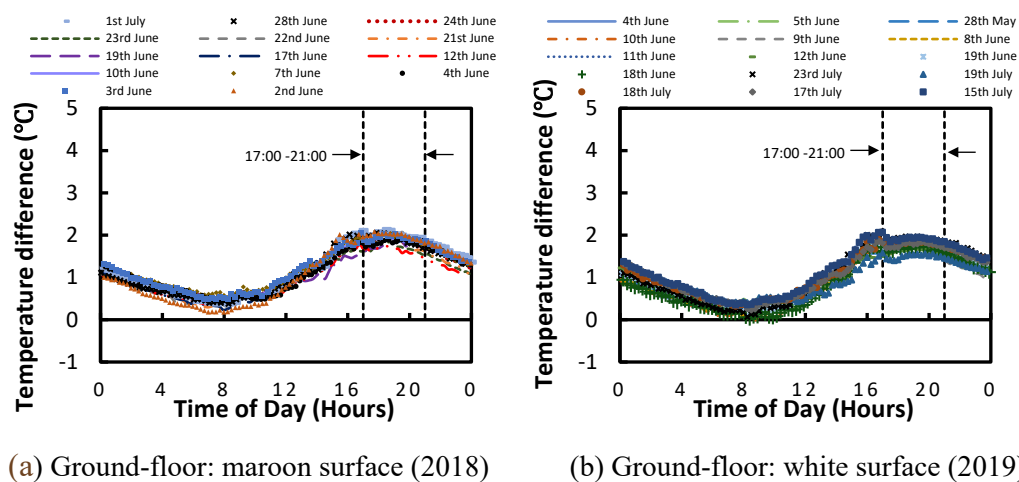


Figure 8.32 – Temperature difference between the northern and southern vaults during autumn, summer and spring (windows closed and maroon surface)



(a) Ground-floor: maroon surface (2018) (b) Ground-floor: white surface (2019)

Figure 8.33 – Temperature difference between the northern and southern portions of the building in mid-winter (windows closed)

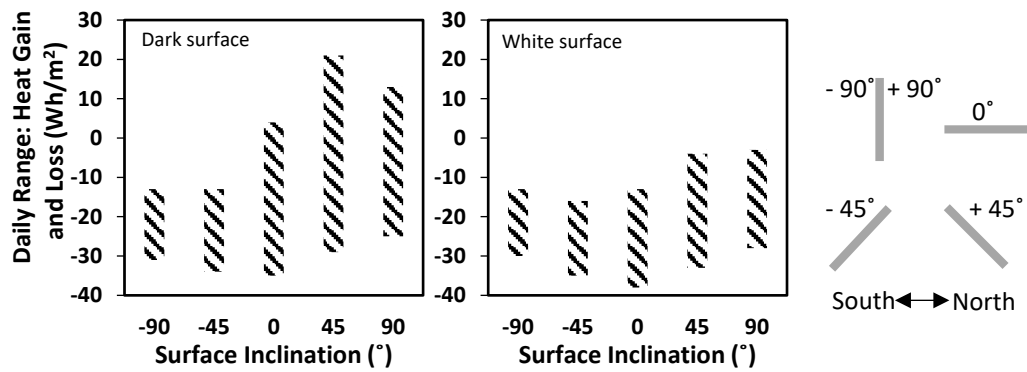
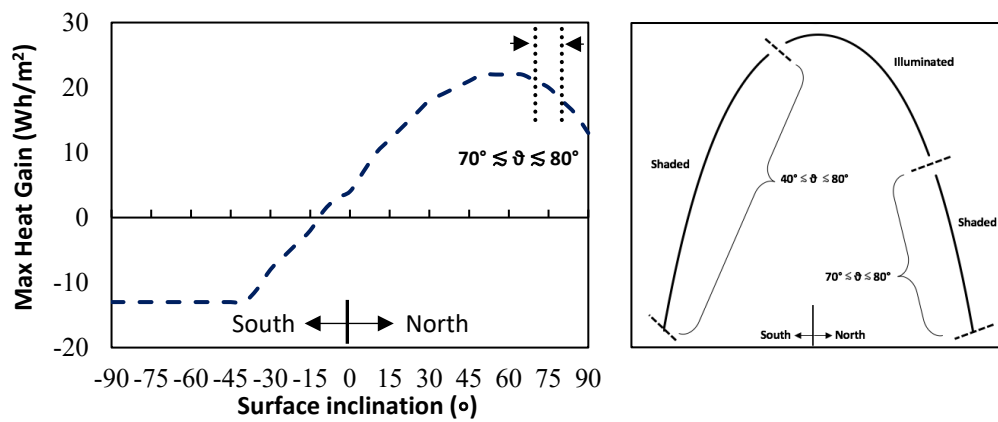
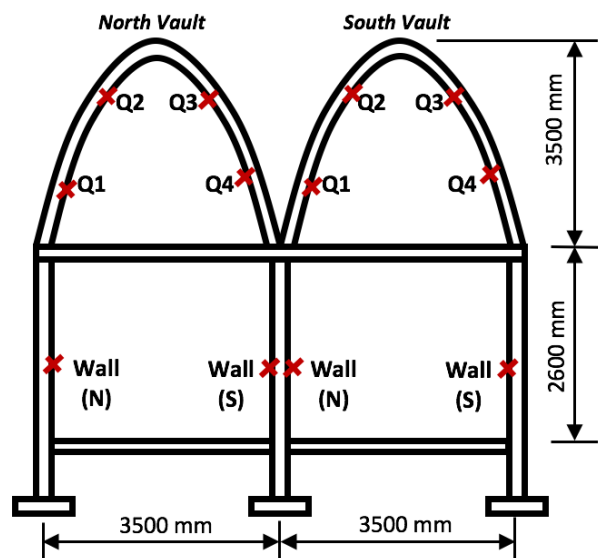


Figure 8.34 – Diurnal heat gain/loss for low and high albedo surfaces: June

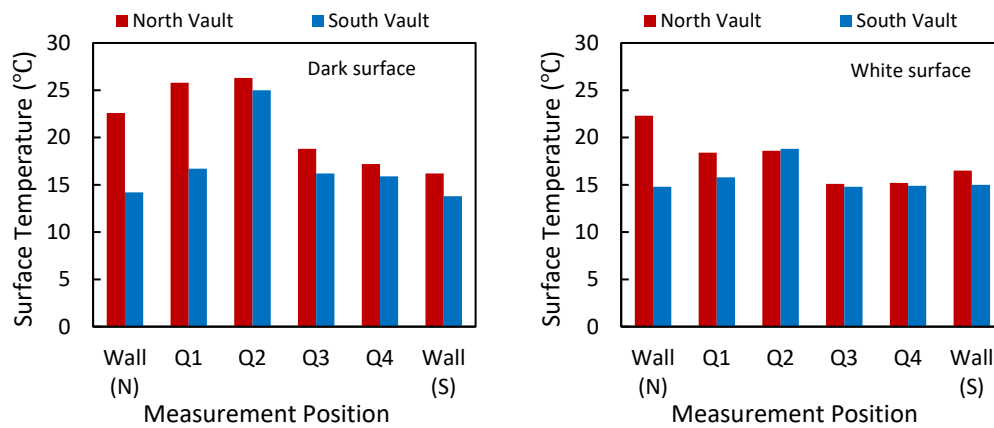


(a) Peak heat gain (June) (b) Surface inclinations and shading at midday

Figure 8.35 – Analysis of surface shading in mid-winter



(a) Measurement positions



(b) 06:00 PM, 22nd May 2018

(c) 06:00 PM, 1st August 2019

Figure 8.36 – Internal surface temperatures in the prototype structure

8.7 Design considerations

8.7.1 Thermal resistance

The maximum and minimum temperatures measured in the prototype house occurred during a heat-wave and a cold-front, respectively. These extreme temperatures were lower and higher than those expected on a design summer and winter day in the Standard Agrément South Africa Comparative House. Nevertheless, the indoor temperatures in the vaults occasionally exceeded the maximum acceptable temperature ($T_{\max}$), following BS EN 15251 (BSI, 2007), during the summer months. A thicker shell could have reduced the occasionally high indoor temperatures in the summer, as well as night-time heat loss through the envelope in the winter. The results from an analysis using OPAQUE 3, are presented in Figure 8.37, which illustrates the influence of the masonry thickness on heat gain through CSEB masonry in a peak summer month in Johannesburg. The same assumptions outlined in section 8.5.2 were considered in this analysis; the section thickness of the shells was selected based on structural considerations alone (Gohnert *et al.*, 2018).

Better thermal insulation can be achieved by using lower density earth blocks, which is particularly advantageous in hot dry climates exposed to extreme temperatures (Adam and Agib, 2001). Moreover, thermal resistance is of greater significance than heat capacity in climates exposed to prolonged hot or cold weather, with small diurnal temperature changes. Indoor summer temperatures could also be reduced by adding insulation such as EIFS (Exterior Insulation and Finish System) cladding. Such systems incorporate urethane, extruded polystyrene, or moulded or extruded expanded polystyrene (EPS) applied to the

exterior of the shell (Wilson, 2005). Such insulation must also be protected from solar radiation and hail. The implementation of insulation and protection may be a necessity in some climates, such as those which experience small diurnal temperature changes in combination with prolonged periods of either high or low temperatures (e.g., tropical summers and Northern European winters). Teixeira and Rodger (2019) indicated that adding a 50 mm layer of expanded polystyrene and structural plaster over enclosed CSEB sections resulted in considerably reduced diurnal temperatures ranges. However, the implementation of such an EIFS system would add significant costs, and may not be affordable for low-income households.

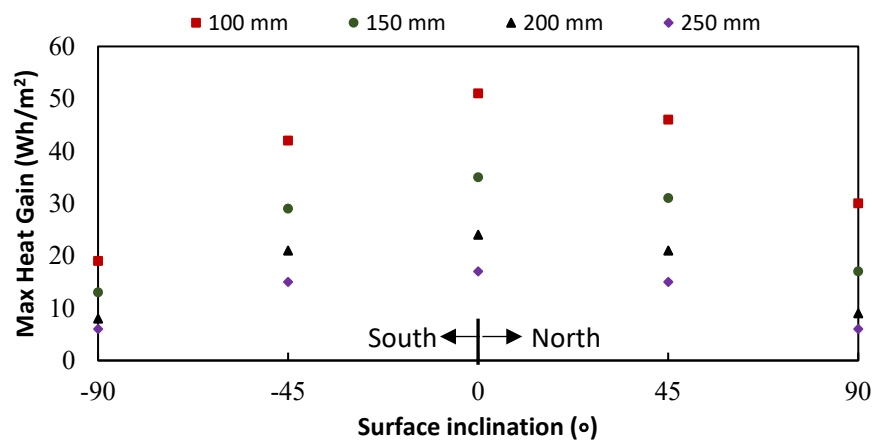


Figure 8.37 – Influence of inclination and thickness on peak heat-gain during a summer month

8.7.2 Ultrathin masonry sections

It has been discussed in this chapter that overheating occurred within two catenary vaults comprising CSEB masonry coated with dark roof paints. Several shells built inland South Africa are considerably thinner than the vaults, comprising only a single-leaf imperial brick approximately 100 mm thick and external plaster. Some shells are also coated with dark paints, which would most likely cause these structures to overheat without the implementation of measures to improve thermal comfort such as air-conditioning. Even thinner sections than a standard brick have been used to enclose habitable spaces, especially toward the apex of singly curved shells. Designers may want to vary the thickness of an arch or dome so as to achieve specific pure-compression profile and/or develop more usable floor space. In both cases, the thickness is varied from a maximum at the base to a minimum at the apex.

The adoption of ultrathin sections, especially at the apex, should be cautiously considered due to structural issues (Bradley *et al.*, 2018a) but also due to the thermal performance of the enclosed space. Specifically, the analyses and in-situ results presented in this chapter highlight that solar heat-gain is greatest through the flatter portions of the shell (i.e., at the apex) during the summer months, except when the shell was painted white. The influence surface inclination, for several thickness of maroon coloured vault, is highlighted in Figure 8.37. Ultimately, spaces enclosed by thinner sections (having lower R-values) will experience higher heat gains and losses in summer and winter, respectively.

8.7.3 Surface albedo

Painting external surfaces white and keeping windows closed during the afternoon were the most effective passive measures for mitigating the potential of overheating. High albedo paints also reduced heat gain through the shell during Johannesburg's sunny winter, resulting in lower indoor temperatures. On several occasions in mid-winter, the indoor temperatures within the maroon coloured vaults were sufficiently high during the evening that artificial heating was not necessary (Bradley *et al.*, 2020). In contrast, the worst performing space was the partially shaded white coloured vault. The indoor diurnal temperature range, in the room of the analysed structure, on several winter days is shown Figure 8.38. This room would, ultimately, require higher heating demand in the winter months.

This postulate was examined by warming the space enclosed by the south vault, which had a white surface, with an oil heater. A heater having sufficient capacity for the air volume was placed 2 m away from the thermometer, and was either left on continuously, which is economically impracticable, or on between 5 PM and 9 PM. A maximum and minimum heating power of 3000 W and 1500 W were considered. Of significance to the present discussion is that indoor temperatures measured during the evening period in the maroon-coloured north vault were higher than those in the heated white-coloured vault, apart from the case of the 3000W heater running continuously. Considering the temperature range shown in Figure 8.38, it is unlikely that adopting white coatings over uninsulated masonry shells is advantageous in Johannesburg. In contrast, overly dark paints may cause thin masonry shells to overheat, even when naturally ventilated, in the summer months.

8.7.4 Surface shading

During late spring and early summer, shading reduced high temperatures, during the late afternoon, by several degrees Celsius, as shown in Figure 8.39a. By mid-October, the sun's diurnal path resulted in minimal shading (auto-shading and shading of the south shell by the north), resulting in minimal variation between the spaces, as shown in Figure 8.39b, which indicates the potential of surface shading for the improvement of thermal comfort in the summer months. However, shading of the south vault, by the north, resulted in significantly lower temperatures in the winter, as shown in Figure 8.38, which illustrates the need to consider the orientation of the building, especially those comprising multiple shells, during design.

Shading may be achieved through several mechanisms including overhanging the roof to the north or south to maximise shading of windows and walls when the sun is at its peak, and placing vertical shading elements on the east and west facing windows to obstruct direct sunlight in the morning and late afternoon. Edge beams provided such shading of the north- and south-facing ground-floor walls of the prototype house. Surface shading can also be accomplished with vegetation. However, trees should not be planted in front of the north facing façade of the building, as this would inhibit solar gain in the winter months. Large trees, if they fall, can cause catastrophic damage when impacting unreinforced masonry shells (Bradley *et al.*, 2017a); they should therefore be removed from the vicinity of such buildings.

8.7.5 Natural ventilation

The observed effectiveness of natural ventilation can be attributed to a large outdoor diurnal temperature range, building layout and the positioning of fenestration. Pressure differences on opposite sides of the building facilitated internal air movement; the effectiveness of which was amplified because the vaults had a long narrow floor plan, as shown in Figure 8.40, without any internal partitions, which assisted with cross-ventilation.

Surface colour also influenced the efficacy of natural ventilation to cool or warm the living space, as shown Figures 8.17 through 8.20. Keeping windows closed throughout the day in hot weather was the most and least effective means to cool the living space for the white and dark surfaces, respectively.

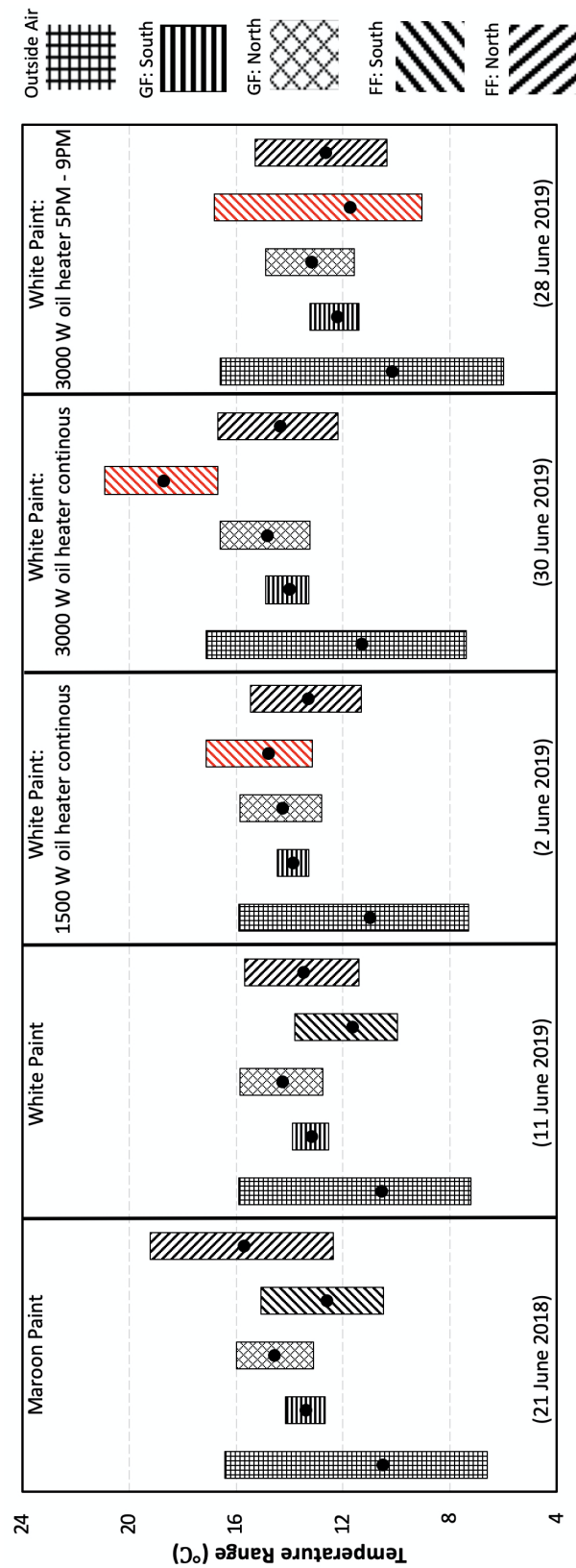


Figure 8.38 – Diurnal temperature ranges on several winter days in 2018 and 2019

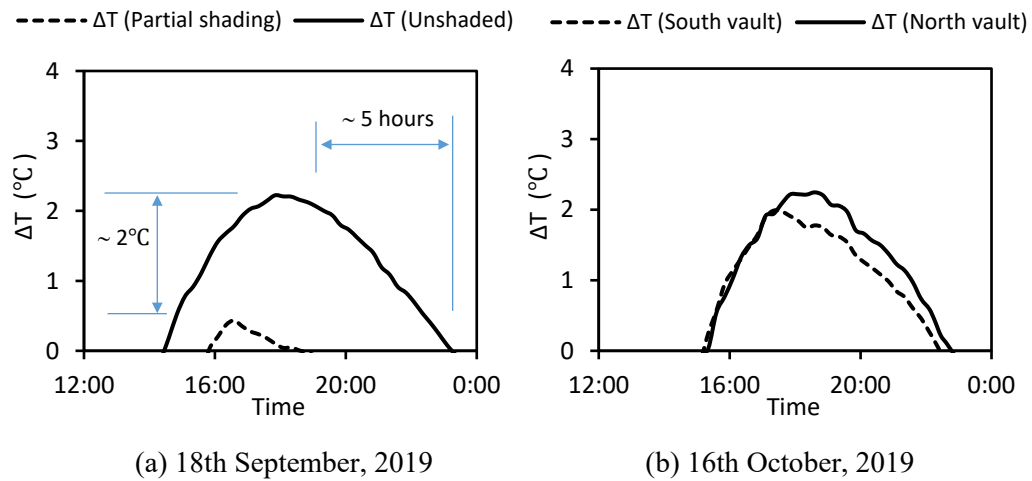


Figure 8.39 – ΔT for north and south vaults in September and October (dark grey surface)

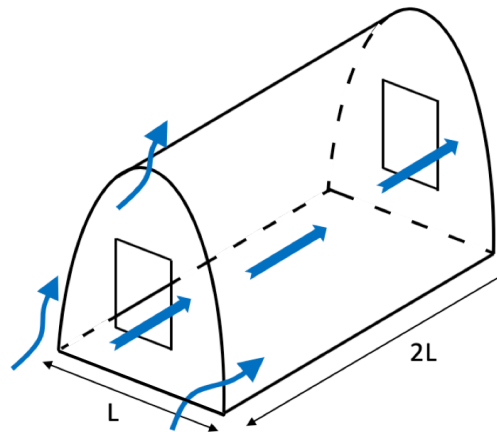


Figure 8.40 – Simple illustration of cross-ventilation

The success of natural ventilation is also contingent on occupants opening and/or closing windows at appropriate times. Johnson (1981) reported that the occupants of households, considered in a Canadian survey, opened the windows on arriving home in the afternoon. Fabi *et al.* (2012) determined however that it was difficult influencing occupants' window opening behaviour. Several factors may also limit the potential of opening windows to ventilate indoor spaces, such as noise and security concerns (BRE, 2016). Another consideration in the region around Johannesburg, is that the dust from mine dumps can accumulate in nearby houses through windows that are kept open for ventilation (Moolla *et al.*, 2011). The dust also poses health concerns (Nkosi *et al.*, 2015), and may dissuade residents from opening windows to improve indoor conditions. About 1.6 million people

live in informal and formal settlements on, or directly next to, mine dumps in South Africa (Nkosi, 2018), and most likely those living in these poorer communities don't have the means to improve thermal comfort through mechanical ventilation.

8.7.6 Space heating

Air warmed by a space heater rose and collected toward the apex of the vault during mid-winter, as illustrated in Figure 8.41a (Figure 8.41a shows the diurnal temperature variation, at heights ranging between 1.1m and 3.0m above the floor, in the white coloured south vault). The temperature at a height of 3.0 m was almost 4°C greater than that at 1.1 m whilst the space was heated; even though the lowest and uppermost black globe thermometers were closest and furthest away from the heater, respectively. For comparison, the difference on a day without heating was considerably less, about 1°C in the white coloured shell, as shown in Figure 8.41b, which can be attributed to shading near the base.

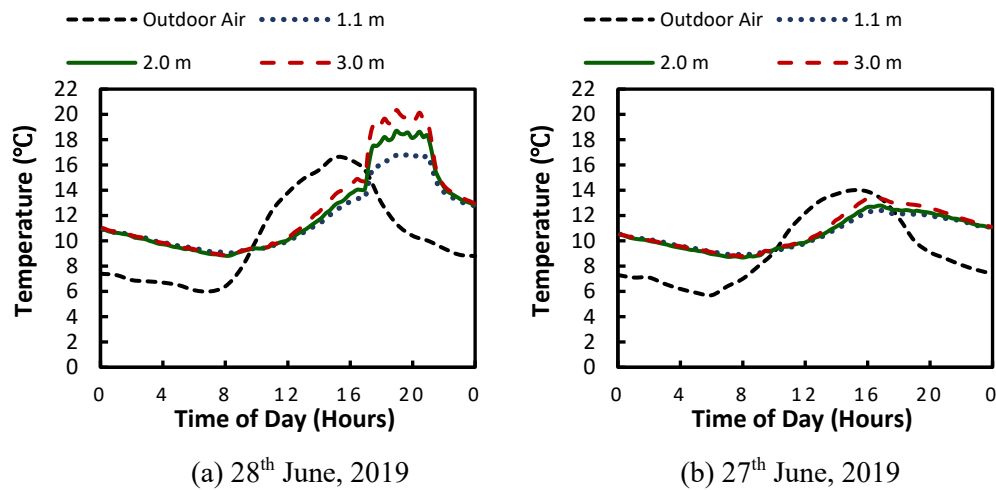


Figure 8.41 – Temperatures in the south vault (white surface condition)

Nevertheless, the height of these shells, and large resultant internal air-volume, mean that the effectiveness of space heating was diminished; i.e., more energy is needed to achieve a certain temperature at a height where persons will inhabit the space. The height from the top of slab to the internal plaster at vault the apex was roughly 3.2 m in the prototype house, and the internal air volume of the space enclosed by one of these shells was approximately 48 m³, which is comparable to that for a rectangular room of the same plan dimensions but having a standard height of 2.4 m. The traditional height to ceiling in residential living spaces is between 2.1 m and 2.4 m according to the National Building Regulations, though

ceilings can be higher than 2.4 m. Although not usual in masonry shell structures, the data presented in Figure 8.41 may support the inclusion of a suspended ceiling to reduce the enclosed air-volume and diminish the extent of stratification of indoor air when the space is heated.

8.8 Conclusions

This chapter presented the results from an in-situ thermal assessment of a house incorporating CSEB shell roofing. The shells were initially coated with a dark maroon paint, which led to the first-floor spaces to overheat when unventilated due to the high transmittal of heat through the envelope. More specifically, high temperatures were attributed to the large surface area exposed to direct solar radiation and low surface albedo. Subsequently, natural ventilation and high albedo paint were investigated as a means to improve the thermal performance of the spaces.

Painting external surfaces white was extremely successful at mitigating overheating. Reducing solar gains through shading or coating with white paint each resulted in a roughly 5°C drop in peak operative temperature within the vaults, indicating the significance of surface albedo and shading on the thermal performance of CSEB shells built inland South Africa. These techniques may be an effective means of improving thermal comfort in the warmer parts of South Africa; however, solar exposure was tremendously beneficial during winter in Johannesburg. Of specific significance to energy consumption, and the associated costs of space heating, was that operative temperatures, between 5pm and 9pm during the winter period, were typically 3°C to 5°C warmer in the north vault. On many occasions in the winter, the operative temperature within the north vault was sufficiently high during the evening that artificial heating was not necessary.

Naturally ventilating the first-floor spaces in the late afternoon and evening greatly reduced the period that indoor operative temperatures exceeded the maximum acceptable temperature (i.e., better performance regarding CIBSE Criteria 1 and 2). Designating the spaces as bedrooms was particularly effective because temperatures did not exceed $T_{\max}$ after 20:00 hrs. Thus, none of the overheating criteria were exceeded, which is particularly important because the occupants of low-cost and affordable housing may not be able to afford mechanical ventilation. Such results were not obtained if the spaces were assumed

occupied in the afternoon, since natural ventilation, per se, had only marginal effect on the magnitude of indoor temperature during this period for dark surfaces.

The study presented herein clearly indicates the significant effects of surface albedo and shading on the thermal performance of masonry shell housing inland South Africa. Analysis showed that natural ventilation resulted in more comfortable hours during the late afternoon and evening. The results showed the critical need to consider the thermal characteristics of the masonry shell as well as the orientation of the building during design.

CHAPTER 9. CONCLUSIONS

9.1 Summary and Recommendations

Earthen masonry shells have many economic, environmental and aesthetic benefits. These structures may also be an efficient solution to low-cost housing in some developing areas. However, their adoption requires careful attention by designers and builders to ensure safe and durable structures. A fundamental consideration when designing unreinforced earthen shells is that the thrust-line or compressive force resultant remains within the middle-third of the shell due to the low tensile strength and durability issues associated with earthen building materials. Several case-studies examined in this thesis indicate that masonry shells are susceptible to cracking.

A shell may, however, be entirely stable even if cracked, which can be understood by considering the lower bound theorem of plasticity. Specifically, a shell is safe if a thrust surface (or line) can be found which is in equilibrium with the applied loads and contained within the masonry section (i.e., the yield condition is not violated). However, cracking of surface renders and/or the earthen masonry provide pathways for moisture ingress, which can lead to localised erosion, degradation of strength, and the partial or complete collapse of earthen masonry shells. Maintenance and repairs are unfortunately often deficient or absent in low-cost environments and may only be implemented when resources are available, long after damage has occurred.

In the absence of regular maintenance and repair, damage may become so severe that structures may require full restoration instead of regular maintenance. Furthermore, cracks may need expensive remedial action, such as the application of sealants and waterproofing membranes. Unfortunately, the most affordable perceived solution to repair and maintain buildings is often sought; e.g., simply re-painting over cracks. Re-painting a cracked shell may be adequate for a very short period time, but thermally induced strains inevitably cause reflection cracking. Larger cracks are particularly problematic, and should be repaired immediately with an appropriate method, such as filling them with an expandable crack filler that accommodate thermal movements or reinforced mortars that accommodate tensile stress. Although there are a variety of repair methods, the best solution to avoid water penetration is to mitigate the possibility of cracking in low-cost applications through proper design, construction and materials specification.

In this thesis, the basis for adopting steeper shell forms for smaller housing structures in South Africa was presented. These profiles are appealing because of the reduced magnitude of lateral base thrusts and the lower probability of concentrated and distributed vertical loads developing over the surface. These profiles also provide more usable floor space, which is critical in small, low-cost structures where space is restricted by financial constraints. A unique method was presented for the determination of catenary and funicular forms, including those with variable and steep profiles. The method facilitates form-finding for structural and architectural (i.e., floor space) efficiency.

Although steep earth masonry vaults have many structural and architectural benefits, the materials are inherently brittle and lack ductility. In some regions, designers may have to incorporate reinforcing to minimise damage associated with seismicity and wind-borne debris, as well as wind loads caused by hurricane-force winds. In South Africa, wind loading characteristically governs the design of low-rise structures (roof components, walling, etc). Although wind uplift is resisted by the mass of a masonry shell roof, the stability of free-spanning vaults is critically affected by lateral loading (e.g., wind and seismic). Several contributions were presented regarding wind loading on catenary shells, founded on data from a comprehensive wind-tunnel study:

- wind design recommendations and tables for catenary vaults.
- the inadequacy of wind design data (pressure coefficients) corresponding to circular profiles for the design of steep catenary shells.
- that wind blowing parallel to gable-walls causes a critical situation for steeper profiles due to positive pressure over the windward face and suction over the leeward half.
- and that wind loading dictates the design of smaller steep catenary shells.

Several other loading considerations and principles of good practice were recommended. The appropriate consideration of concentrated and impact loading on low-rise unreinforced thin shells was highlighted. The objective is the adoption of unreinforced shells. Therefore, alternative measures to reinforcing to improve capacity need to be considered (i.e., mitigating the risk and/or extent of these loads developing over unreinforced shells). Several options were examined, including limiting access onto the roof, using steeper shell forms and supporting heavy fixings with independent structures or elements so that the roof is not required to carry dead or maintenance induced point loads. In terms of impact loads,

control and prevention measures should be implemented to minimise the possibilities of accidental events.

Noting the extent of cracking observed in the numerous unreinforced masonry shells considered, one might assume that the complete prevention of cracking in these structures is not technically or economically feasible. However, it is apparent that most of the observed cracking can be attributed to errors or oversights made during the design phase. In only a few cases, were cracks attributed to poor construction practice. Design errors are particularly problematic because even the best construction cannot offset such deficiencies. Self-straining stresses are a particularly important issue in the design of CSEB shells, because cracks are common in areas where expansion and contraction are restrained. Many shells experienced substantial cracking damage near stiffened openings, abrupt changes in geometry (e.g., lips/eyebrows) and at wall-shell connections. Vertical shrinkage cracks, associated with bond and friction, have also been reported, highlighting the importance of the shrinkage characteristics of the mortar joints and units. Furthermore, large mortar joints made with high water content should be avoided, and CSEBs should be cured for an adequate period prior to use in construction. Several techniques were identified to mitigate the potential of cracking, which include:

- use of control joints between the shell and secondary elements.
- avoiding abrupt changes in section geometry and/or thickness.
- avoiding rigid connections between adjacent shells.
- using high albedo paints to attenuate large diurnal temperature fluctuations in plasters covering CSEB masonry.

Several construction techniques, either with or without formwork, were examined. Reusable formwork may greatly improve the speed of construction when building multiple shells. Precise formwork can also reduce the possibility of errors, damage and the subsequent increased time and costs associated with repairs and re-work. However, studies on a model and full-scale structure, indicate that rigid formwork may lead to significant cracking if it restrains shrinkage. The following observations, and proposed solutions, are reported based on several construction studies:

- cracks typically manifest toward the apex of vaulted structures, and formwork should be dropped after the initial set of the mortar to prevent crack development.
- an issue related to restraint associated with segmental construction approaches (e.g., the shifting formwork technique) was also reported. To allay possible cracking, CSEB shells should be constructed in a continuous, rather than segmental, manner.

The thermal assessment of a house incorporating CSEB roofing indicated that indoor operative temperature frequently exceeded the maximum acceptable temperature following BS EN 15251. Spaces enclosed by the CSEB shells overheated (i.e., failing at least two of the CIBSE overheating criteria) when coated with dark paints. The following strategies were highly effective at reducing overheating:

- naturally ventilating spaces in the late afternoon and evening. The effectiveness of ventilation was due to the positioning of fenestration, as well the long and narrow floor plan created by the vaulted roof.
- reducing solar gains through shading or painting external surfaces white. White paint reduced the operative temperature by more than 5°C, and indoor spaces failed none of the overheating criteria. By changing the surface colour (albedo) of a smaller CSEB shell (such as those applicable to low-cost housing) contrasting outcomes from the CIBSE overheating assessments could be obtained (i.e., either failing none or all the overheating criteria).

Although painting shells white was extremely effective at mitigating overheating, additional heating would be required, during Johannesburg's sunny and clear winter, due to reduced passive heat gain associated with a high albedo surface (i.e., white paint) and/or shading. These observations show that the thermal performance of masonry shells built inland South Africa are considerably influenced by surface albedo and shading. Conventional roofing in South Africa must meet minima insulation targets, per *SANS 10400-XA: 2011*, and shading and surface albedo are not as effective on well insulated buildings/components. Shading and surface albedo are important considerations for the design of CSEB shell structures even though neither mechanism is addressed within the National Building Regulations (or stipulated in the procedures to check compliance).

9.2 Future Work

- 1 External membranes and/or paints are an effective means of waterproofing masonry shells. However, the use of membranes over earthen masonry is not conventional. It is traditional practice to use protective earth plasters (e.g., soil-cement and soil-lime), and several recent studies have focused on the application of local materials to improve the durability of modern CSEB walling. However, the environmental exposure of shell roofs necessitates frequent maintenance of such renders. When deficient, the consequences may be catastrophic. Renders over the flatter portions of shells are more susceptible to erosion caused by heavy rainfall associated with intense thunderstorms that occur in inland South Africa. Considering the South African climate, future work is needed to improve the performance of such renders: especially if used in low-income housing where maintenance and/or repair may be lacking.
- 2 The numerical form-finding method presented in Chapter 4 was largely implemented for two-dimensional and simple three-dimensional shapes. Its application to more complex geometries is unproven, and could be examined further using finite elements. Some issues were reported in Chapter 4 regarding the distortion of shell elements after each iteration, which resulted in divergence; i.e., the solution did not converge to a tensionless form. This might be overcome by remeshing the structure after each iteration, but this would add additional computation. Alternatively, an arrangement of linear bar elements, to represent the geometry of the shell, may be an avenue for further investigation.
- 3 The performance of unreinforced earth masonry shells under dynamic loading should be further investigated, especially regarding suitable shell forms/shapes, span to depth ratios, etc.
- 4 The long-term thermal assessment of a CSEB shell house indicated that there was a juxtaposition between unfavourable and beneficial solar heat gain in summer and winter respectively. The critical importance of surface albedo and shading were reported, but the aforesaid observation also emphasises the significance of section thickness on the thermal performance of CSEB shells. In fact, the minima shell thickness required to meet structural considerations may not be adequate to ensure good thermal performance of the building. Further work could be done based on energy demand using building

simulation modelling. Furthermore, other passive design measures could be examined, such as improving the thermal resistance of earth masonry shells by adding insulation and adopting greater shell thickness.

- 5 The impact of performance deficiencies, e.g., cracking and poor thermal comfort, on the long-term maintenance and operational costs associated with CSEB shell housing should be further examined. Enhancements to certain material properties of CSEB masonry could be considered, which are not boxed in by the requirement of a low-cost material, if long-term financial benefits could be achieved.

APPENDIX A. IMPROVING THE STRENGTH OF CSEBS USING BASALT FIBRES

A.1 Introduction

Sustainable alternatives to traditional binders such as cement and bitumen are increasingly being investigated in order to reduce the costs and environmental impacts associated with the production of earth bricks. Furthermore, relatively cheap and sustainable alternatives, such as natural fibres and waste materials, may also improve the ductility and strength of earthen building materials. However, some natural fibres are susceptible to degradation due to alkaline hydrolysis or drying shrinkage of the fibres, which in combination with volume changes in the vicinity of the fibres during drying, may reduce the fibre-matrix bond. Additionally, it is generally accepted that steel should not be incorporated within earthen masonry due to the high cost and problems associated with corrosion. As an alternative to natural and steel fibres, basalt is investigated in the present study. In particular, the results from three-point bending tests on earth blocks reinforced with basalt fibres are discussed.

Basalt is a naturally occurring material found in extrusive igneous rock. Fibres are produced by melting basalt rock and drawing it through dies. The resultant continuous fibres are then spun to create a twisted yarn, or wound as an individual filament. Basalt fibres have similar chemical composition to glass fibre; however, it has been shown that basalt fibres are far more resistant to alkali (van de Velde *et al.*, 2003). Furthermore, basalt does not corrode like steel. Consequently, the material is very versatile, and particularly appealing for inclusion within cementitious composites. However, the cost of basalt fibres, when compared with natural fibres such as sisal, is still high. Consequently, to be feasible in low-cost earth masonry shell constructions, basalt fibres would need to enhance strength at lower concentrations than natural fibres.

A.2 Literature

Several authors have investigated the fracture characteristics of earthen building materials (e.g., Clementi *et al.*, 2008; Aymerich *et al.*, 2012; Aymerich *et al.*, 2016; Mesbah *et al.*, 2004). It is apparent from the literature that the inclusion of natural fibres (between 0.1% and 4% by mass) improves the ductility and post-cracking characteristics (i.e., fracture energy) of earthen bricks/blocks (e.g., Zak *et al.*, 2016; Ghavami *et al.*, 1999; Fernandez, 2003; and Mesbah *et al.*, 2004). At low fibre concentrations, blocks show tension-softening

behaviour (Mesbah *et al.*, 2004), whereas at higher concentrations strain-hardening is apparent (Fernandez, 2003; Zak *et al.*, 2016). Furthermore, fibre inclusion does not significantly influence the elastic response (e.g., initial stiffness) and first crack strength (Aymerich *et al.*, 2012; Aymerich *et al.*, 2016). This is readily explained when recognizing that most natural fibres have low stiffness.

The results from several studies show that at practicable fibre concentrations (i.e., at quantities which do not obstruct workability, reduce density, and hinder economy), only marginal improvement of tensile/flexural capacity is achievable (e.g., Mesbah *et al.*, 2004; Ghavami *et al.*, 1999; Fernandez, 2003). Furthermore, the dimensional changes of many natural fibres due to moisture, may significantly influence the adhesion between matrix and fibre (Fernandez, 2003; Ghavami *et al.*, 1999; Gutiérrez-Orrego *et al.*, 2016). This reduced bond can negatively influence the mechanical properties of the composite. Yetgin *et al.* (2006) report that the strength (compressive and tensile) of earth reinforced with straw decreased because the fibres were soft and adherence between the fibres and soil matrix was weak. In contrast, the flexural capacity of adobe samples increased with increasing fibre content in studies by Bouhicha *et al.* (2005) and Galán-Marín *et al.* (2010). It is also relevant to mention that the inclusion of some natural fibres may reduce the density of earthen blocks, due to higher water demand during block production, causing a reduction in strength (Zak *et al.*, 2016; Yetgin *et al.*, 2006).

Although several studies report marginal strength improvement with fibre addition, others report significant peak load improvement. Aymerich *et al.* (2012) and Aymerich *et al.* (2016) examined notched soil samples in flexure, which were reinforced by hemp and wool fibres, respectively. In both studies, fibrous reinforcement greatly improved the peak load, ductility and energy absorption of samples after first cracking. Though, these authors note that the beneficial action of fibres only becomes apparent when the magnitude of crack opening is sufficient to facilitate strengthening through bridging and frictional fibre pull-out. A study by Gutiérrez-Orrego *et al.* (2016) showed that the low-concentration inclusion of sisal fibres ($\gg$ 70 mm long at 0.1% by mass) lead to a 20-fold increase in flexural strength of blocks. It is well known that the stiffness, content, length, and aspect ratio of fibres have an influence on the effectiveness of a fibre-reinforced composite. Furthermore, the fibre-matrix bond, which depends, in part, on the surface roughness of the fibre, is also of significance (Aymerich *et al.*, 2016). The fibres used by Gutiérrez-Orrego *et al.* had high

aspect ratio (i.e., were long and thin), a rough surface, and relatively high elastic stiffness (25 GPa), which may explain the significant improvement in flexural strength of blocks.

A.3 Soil selection

Adam and Agib (2001) note that the production of good quality, durable compressed stabilised earth blocks requires the use of soil containing fine gravel and sand for the body of the block, together with silt and clay to bind the sand particles together. However, not all soils will exhibit these desirable characteristics; thus, mixing of different soils may be required. Nevertheless, a wide range of soils are suitable for the production of strong and durable stabilised earth bricks. Burroughs (2008) conducted numerous tests on various soil types, which were compacted and stabilized with lime or cement, to assess their suitability for stabilisation. The author considered the soil suitable (or that stabilisation was successful) if the compressive strength exceeded 2 MPa. Figure A.11, reproduced from Burroughs (2008), demonstrates the important parameters (i.e., linear shrinkage, plasticity index, clay content and sand content) for the assessment of the soils' suitability for stabilisation. The zone, demarcated with red dashed lines in Figure A.1, illustrates the wide range of soils appropriate for stabilisation. Though, the strength and longevity of CSEBs are negatively influenced by increasing fines (i.e., clay and silt) content. The prescribed limit on fines is typically 35%.

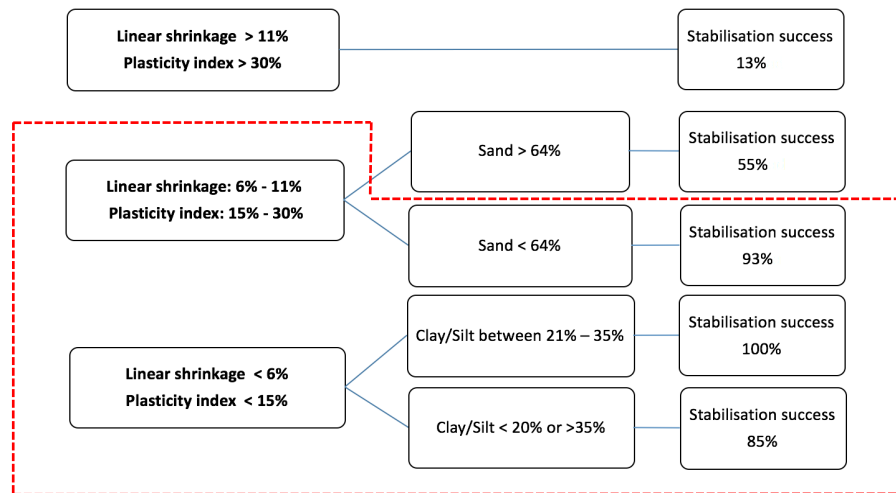


Figure A.1 – Soil suitability for stabilisation

Three soil types (denoted A, B and C) were selected for the production of fibre reinforced blocks assessed in this study. Soil A and B were obtained from a local building supplier (in

Johannesburg), whereas soil C was obtained from a construction site in Ekurhuleni, South Africa. A combination of the sieve (TMH1 method A1a) and hydrometer method (ASTM method D442) were used to determine the particle size distributions for the soils. These results are shown in Figure A.2. The linear shrinkage, plasticity index (following TMH1) and the percentage of gravel, sand, silt and clay are shown in Table A.1. These characteristics were used to classify the soils following the unified soil classification system (USCS) - specified in ASTM D2487. According to the unified classification system soil A was a clayey sand with gravel, soil B was a sandy silty clay and soil C was a fat clay with sand.

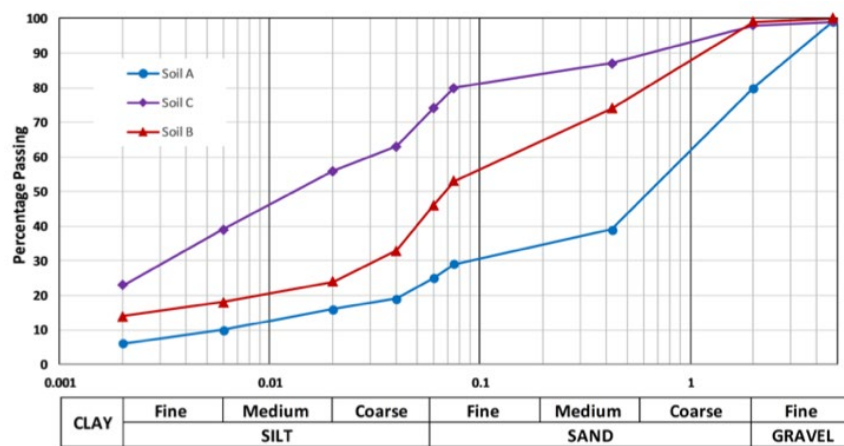


Figure A.2 – Particle size distributions of soils tested

Table A.1 – Soil characteristics

Soil	LS (%)	PI (%)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	USCS
A	4	9	6	19	55	20	SC
B	4	7	14	31	54	1	CL-ML
C	10	25	23	51	24	2	CH

PI (plasticity index), LS (linear shrinkage)

Fernandez (2003) suggested that soils used in fibre-reinforced blocks need to contain sufficient fines (silt and clay) to act as binders for the fibres (The adoption of soils with high fines content for use in reinforced CSEB is illustrated in Table A.2.) This is in contrast to soils used in standard CSEBs where the strongest blocks are those produced with minimal clay content). On the other hand, expansive or highly plastic fines can result in damaging volume changes during curing. Soil A and C are therefore potentially less

suitable for reinforced CSEBs as soil A has a low fines content and soil C has highly plastic fines. Soil B is potentially the most suitable as it has high fines content, low plasticity and low linear shrinkage.

Table A.2 – Comparison of several studies on fibre-reinforced earth blocks

	Mesbah <i>et al.</i> (2004)	Fernandez (2003)			Zak <i>et al.</i> (2016)		Gutiérrez-Orrego <i>et al.</i> (2016)	
Fibre material	Sisal	Sisal			Hemp	Flax	Sisal	Rock wool
Fines content (%)	54	16	39	62	92		47	
Plasticity index (%)	10	10 to 19			15		6	
Fibre % (by mass)	0.5	0.5 & 1			1 & 3		0.1 to 0.4	1 to 4
Cement % (by mass)	0	10			0	0	9	9

A.4 Methodology

A.4.1 Materials: basalt fibres and cement

A series of tests were undertaken on CSEBs produced with basalt fibres up to 0.2% (by dry mass) of the soil-cement mix. This upper limit (i.e., 0.2%) was selected because the material cost (excluding those of transportation and block manufacturing) of the fibres was equivalent to the cost of doubling the cement content from 10% to 20% (by dry mass). Thus, only low concentrations of fibre were tested. The price of cement and basalt fibres (per kilogram, and excluding VAT) considered in this study were R1.61(≈\$0.1) and R84.67(≈\$6), respectively. It is relevant to mention that data from various studies (i.e., Pave, 2007; Ipinge, 2011; Walker, 1995; and Blight, 1994) show that doubling the cement content within CSEBs led to an increase in mechanical strength of between 50% and 120%. Thus, strength (in particular flexural strength) would need to show similar improvement to warrant inclusion at current prices. However, it is reasonable to expect that manufacturing costs would reduce if worldwide demand and production increase.

Basalt fibres used in these tests were 36mm long, ~ 1mm wide and ~ 0.1mm thick. The stiffness of the basalt fibres was ~ 70 GPa. Like glass fibre, basalt exhibits brittle stress-strain characteristics (The basalt material experienced approximately 1.5% elongation at break). Fibres were added to all soils up to a maximum concentration of 0.2% by dry mass. More fibre concentrations were investigated for soil B, due to its better fines characteristics. In addition to fibres, most blocks were stabilised with 10% cement (CEM II 42.5 R) by dry mass.

A.4.2 Block manufacturing

The soils were initially passed through a 5 mm sieve to remove any large particles. This was followed by mixing of the constituent materials within a pan-mixer. Initially, the sand, cement and fibres were mixed (Figure A.3). After this process, the water was gradually added over a period of several minutes in an attempt to avoid ‘balling’ within the mix. The mixing continued until the material appeared homogeneous. Typically, the mixer had to be stopped during this process because the fibres would agglomerate around the blades. These fibres had to be removed by hand and dispersed back into the mixer. Each batch was approximately 40 kg in mass; enough for the production of 3 blocks (For reliability and reproducibility of these provisional tests, 3 blocks were produced for each sample). Blocks were formed with a Hydraform M7E380V block-making machine (at a compression of 10MPa), as shown in Figure A.3e. It should be mentioned that it was difficult to remove blocks made with soil B and soil C from the bottom rammer of the machine, due to the high fines content within these soils. This process required two people to carefully and slowly remove the blocks from the machine. After removal, blocks were cured in the laboratory, under plastic sheets, for 28 days. Average 28-day fibre-reinforced block densities were 2020kg/m³, 2071kg/m³, and 1822kg/m³ for soils A, B and C, respectively. All of the compressed stabilised blocks satisfied a minimum suggested density of 1700 kg/m³ (Fernandez, 2003).



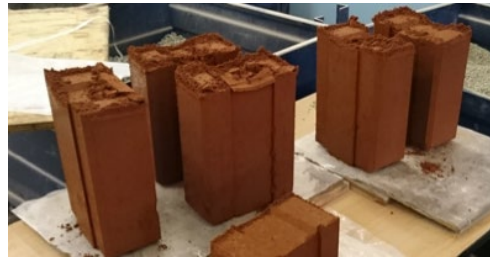
(a) Soil B in the pan-mixer



(b) Basalt fibres



(c) Mixing the cement, soil and water



(d) Several blocks after manufacturing



(e) 5 mm sieve



(f) Hydraform M7E380V machine

Figure A.3 – Block production

A.4.3 Flexural testing

Samples were examined in compression and flexure with a Tinnius-Ohlsen press. Only the results from the flexural tests are presented and discussed in this dissertation. It must be noted that the direct tension test (on actual blocks) will provide the most accurate assessment of tensile strength; however, this test is appreciably more difficult to setup than the three- or four-point bending test - as discussed by Mesbah *et al.* (2004) and Fernandez (2003). The nature of these experimental assessments did not justify such as setup; therefore, the three-point bending test was selected for its simplicity, as most commonly adopted to assess the effect of fibre inclusions in earth blocks (e.g., Fernandez, 2003; Gutiérrez-Orrego *et al.*, 2016).

A.5 Results and discussion

Results from the flexural tests are shown in Figure A.4 (Error bars show maximum and minimum strengths obtained from 3 tests). The following conclusions are drawn from these preliminary tests:

- After failure, visual examination revealed that fibres did not fail in tension but pulled out from the cement-earth matrix, as shown in Figure A.5;
- The inclusion of basalt fibres, at a concentration of 0.2% by dry mass, appeared to have no influence on the flexural strength of blocks produced with soils A and C;

- Blocks produced with soil B appeared to show marginal improvement with the addition of fibres at 0.1 % and 0.2 %. This improvement is possibly associated with the material properties of the soil (i.e., good fines content, low plasticity and low linear shrinkage);
- The soil characteristics and cement concentration had a greater influence on the flexural strength than did the low-concentration inclusion of basalt fibres.
 - The inclusion of 10% cement, within blocks produced with soil B, resulted in an almost 5-fold increase in flexural strength;
 - Blocks manufactured with soil A were four times stronger than those made with soil C, and roughly twice as strong as those made with soil B.

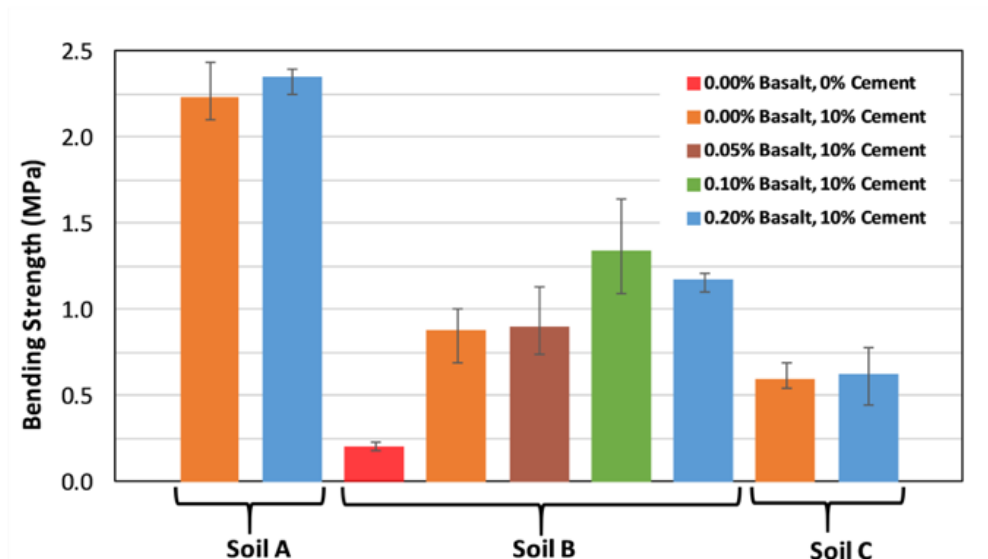


Figure A.4 – Average flexural strength of blocks in three-point bending



(a) Three-point bending

(b) Fibre pull-out

Figure A.5 – Three-point bending tests

The significantly higher strength of blocks made with soil A is due to a combination of the following factors.

- *High block density.* However, blocks made with soil B were generally denser than those made with soil A, indicating this is not the principal mechanism contributing to the significant difference in block strength.
- *Good soil grading.* The soil grading curve for soil A falls within the typical Fuller envelopes (i.e., maximum density gradings) for clayey and sandy soils (i.e., n between 0.25 and 0.45). The grading and Fuller curves are shown in Figure A.6.
- *Low fines content.* The influence of fines content on the flexural strength of blocks reinforced with 0.2% fibres is shown in Figure A.7. It is widely accepted that clay content should be minimised in CSEB because clay minerals disrupt the creation of cementitious bonds (between the larger sand and gravel particles), increase shrinkage, and expand and contract during wetting and drying cycles.

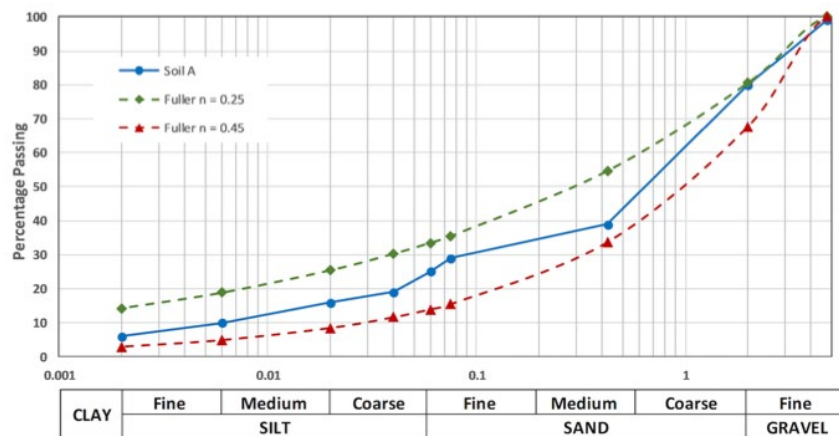


Figure A.6 – Fuller curves for soil A

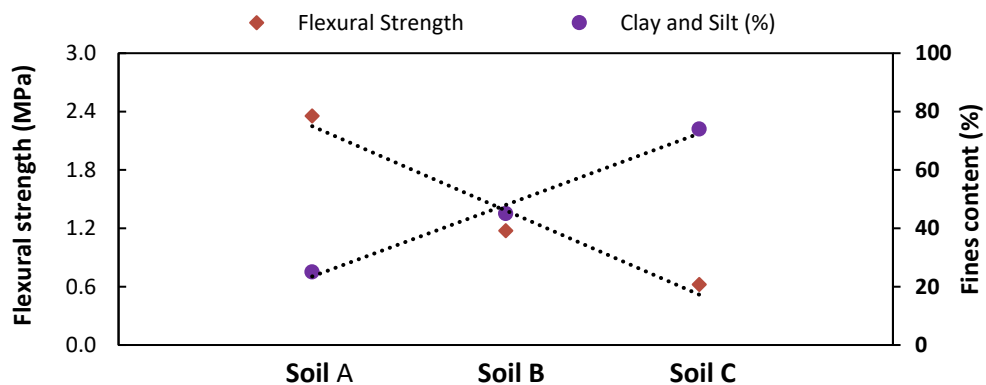


Figure A.7 – Influence of fines content on the flexural strength of fibre reinforced CSEBs

A.6 Summary findings

A.6.1 Conclusions

The low-concentration inclusion of basalt fibres marginally improved the flexural strength of blocks produced with soil B only. This result does not endorse basalt fibre inclusion to improve the tensile strength of low-cost earth bricks; particularly because of the increased cost and the work associated with such additions. Furthermore, bricks incorporating fibres may require substantial preliminary work (i.e., chemical and mechanical testing) prior to manufacturing to ensure that the fibre and soil are compatible, and that the desired performance characteristics can be met. Rather, it appears that strength may be more efficiently increased by incorporating additional binder material (i.e., cement) or by adding sand to reduce the fines content. However, the results presented by several authors (e.g., Gutiérrez-Orrego *et al.*, 2016) show substantial flexural strength improvement with the addition of even very low concentration of fibre. Clearly, further work is needed to quantify the effects of several parameters and their influence in varying soils types.

A.6.2 Limitations and future work

The tests discussed in this appendix were limited in extent. Only three soils, low-concentration fibre inclusions, and a single fibre size were assessed. Nevertheless, the soils adopted in this study showed good variety in material characteristics. Increasing the fibre length and effective diameter could be considered in future tests, noting their influence on composite strength - as identified in the literature. Finally, the influence of basalt fibres on post-cracking ductility (which is particularly attractive in earthquake regions) and shrinkage were not examined in this study.

A.7 Acknowledgements

The author gratefully acknowledges Basalt Technology and Pan Mixers South Africa for providing the fibres used in the study. Messrs. C. Alwar, S. Ramsunder, R. Cassim, B.P. Sing, N.O.G. Mahomed and Y. Bapeekee are thanked for their assistance with block production and the execution of experimental work.

APPENDIX B. COLLAPSE LOAD ANALYSIS

B.1 Introduction

Collapse load analysis is an upper bound method that requires all possible collapse mechanisms be examined to ensure that load-carrying capacity is not overestimated. When a collapse mechanism has formed, the ultimate resistance moments at these locations must be in equilibrium with the ultimate loading on the structure/element. Thus, a minute increase in load will cause the structure to deflect. If this increase in load is infinitesimal, the work done on the structure while the hinges are rotating must be equal to the loss in work due to the load displacement. Therefore, if a point is given a virtual displacement, compatible rotations take place along the fracture lines (or hinges). The internal work is given by the sum of the rotations of hinges multiplied by the resisting moment capacities. The external work is given by the sum of the external loads multiplied by their respective deflections.

For voussoir arches, Heyman (1969) gives the moment capacity as $M = \pm N(t/2)$, where N is the horizontal thrust and t is the vertical section thickness. The term $t/2$ is the maximum load eccentricity at which the bricks will remain in contact (i.e., a hinge will occur for $M = \pm N(t/2)$). In other words, four hinges will form (creating a mechanism) when the vault thickness is just sufficient to contain the thrust line. Clearly, variation in vertical thickness and/or horizontal thrust will affect the magnitude of the moment capacity. In the case of a reinforced concrete arch/vault, the capacity is dictated by the characteristics of the section (e.g., depth to steel and section thickness) and material properties (e.g., bar size, spacing, yield strength of steel, and concrete cube strength). In this analysis, the purpose of which is to highlight the significance of lateral loading, the plastic moment capacity was considered/assumed equal at all hinges. Furthermore, fracture lines were assumed to coincide with the centre-line of the section.

B.2 Methodology

B.2.1 Development of the numerical model

The critical collapse mechanism and corresponding moment capacity were determined numerically through a non-linear optimisation technique. In particular, the Evolutionary Solver available in Microsoft Excel. The formulations of the external and internal work equations are discussed in the following sections. Both energy equations were expressed

in terms of four variables, namely; x_1, x_2, x_3 and x_4 (which are the x-coordinates of the hinges). Point loads were applied to the structure and the plastic moment capacity was maximised to determine the true collapse mechanism. It should be noted that only the applied loads contribute to the external work equation since the self-weight of the vault does not induce any bending moment in the structure.

B.2.2 Internal work

The internal work done along each fracture line (or hinge) is calculated with the following expression;

$$W_i = M_p D \theta \quad (B.1)$$

where M_p is the plastic moment capacity, θ is the fracture line rotation, and D is the length of the fracture line parallel to the axis of rotation. To be noted, a 1 metre strip of vault was considered in all analyses; i.e., $D = 1$ m. For a vault with rigid supports, four plastic hinges are required to form a collapse mechanism, and the total internal work is given by Equation B.2 below.

$$W_i = 2M_p D (\theta_1 + \theta_2 + \theta_3) \quad (B.2)$$

Rotations were calculated using the collapse mechanism and related geometry shown in Figures B.1 and B.2. All rotation terms were expressed in terms of θ_1 (the rotation of segment 1) using the instantaneous centre of rotation (Figure B.2). These are given by equations B.3 and B.4.

$$\theta_2 = \theta_1 (L'' / L') \quad (B.3)$$

$$\theta_3 = \theta_1 (L'' / L') (B' / B'') \quad (B.4)$$

where L' , L'' , B' and B'' are length terms (Figure B.2). These lengths were expressed by the variable co-ordinates corresponding to the four plastic hinges ($x_1, x_2, x_3, x_4, y_1, y_2, y_3$ and y_4) and the coordinates of the instantaneous centre of rotation (x', y'). See Equations B.5 – B.8).

$$L'' = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2} \quad (B.5)$$

$$B'' = \sqrt{(x_4 - x_3)^2 + (y_3 - y_4)^2} \quad (B.6)$$

$$L' = \sqrt{(x' - x_2)^2 + (y' - y_2)^2} \quad (B.7)$$

$$B' = \sqrt{(x_3 - x')^2 + (y' - y_3)^2} \quad (B.8)$$

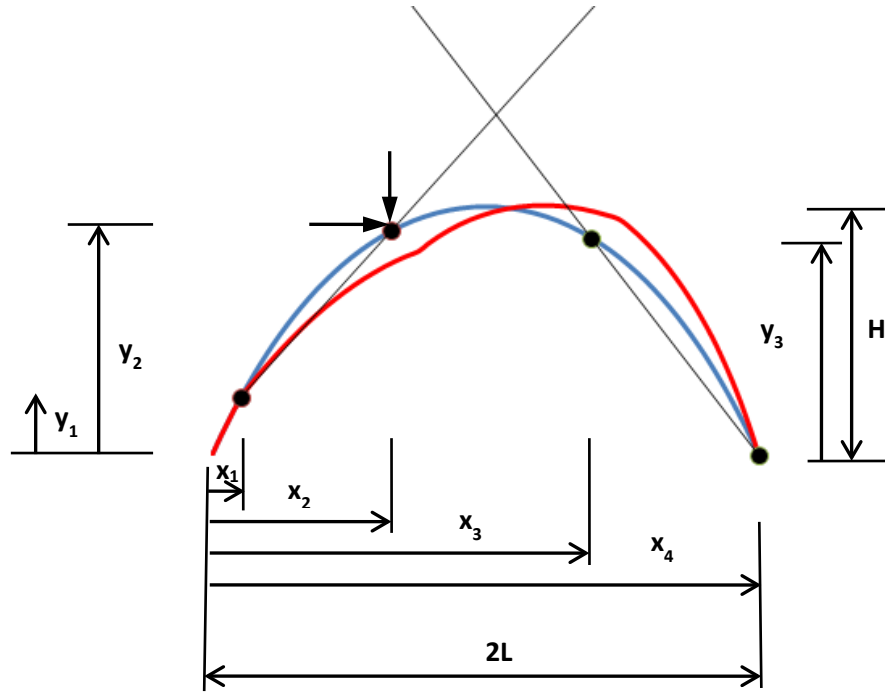


Figure B.1 – Collapse mechanism and hinge variables

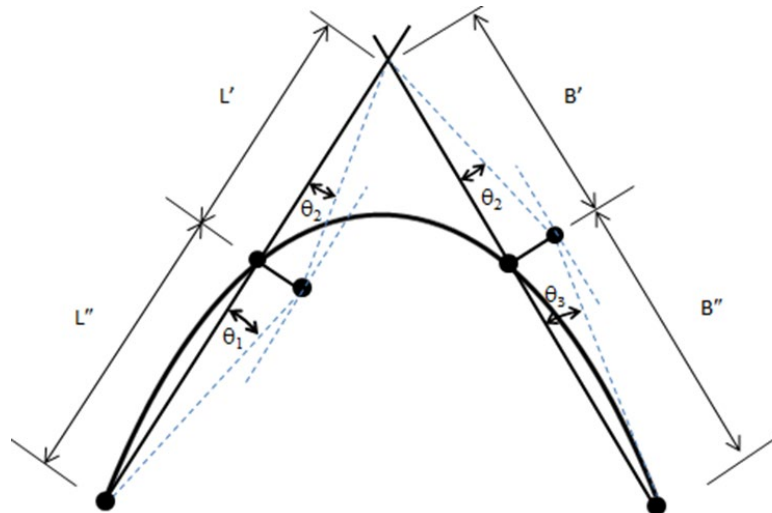


Figure B.2 – Segment rotations

The height of each hinge (y_1, y_2, y_3 and y_4) is now expressed in terms of the corresponding x co-ordinates (x_1, x_2, x_3 and x_4) by way of the catenary equation of the following form;

$$y_i = -a \cosh(x_i/a) + c \quad (\text{B.9})$$

where a and c are terms which define the geometry of the catenary shape (Gohnert and Bradley, 2020). In addition, the instantaneous centre of rotation (x', y') , shown in Figure B.3, was also expressed by the four x -coordinate variables as follows:

$$x' = (c_R - c_L)/(m_L - m_R) \quad (\text{B.10})$$

$$y' = m_L x' + c_L \quad (\text{B.11})$$

$$m_L = (y_2 - y_1)/(x_2 - x_1) \quad (\text{B.12})$$

$$c_L = y_1 - m_L x_1 \quad (\text{B.13})$$

$$m_R = (y_4 - y_3)/(x_4 - x_3) \quad (\text{B.14})$$

$$c_R = y_4 - m_R x_4 \quad (\text{B.15})$$

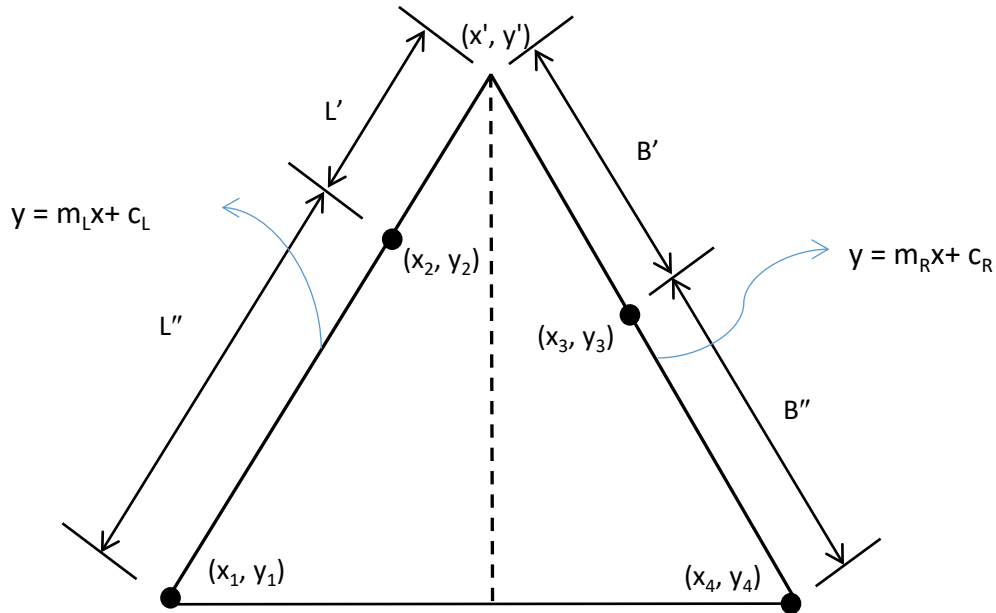


Figure B.3 – Point of intersection (x', y')

By substituting Equations B.12 through B.15 into Equations B.10 and B.11, the following equations defining the coordinates of the instantaneous centre of rotation are derived:

$$x' = \frac{\left(y_4 - \left(\frac{y_4 - y_3}{x_4 - x_3}\right)x_4 - y_1 + \left(\frac{y_2 - y_1}{x_2 - x_1}\right)x_1\right)}{\left(\frac{y_2 - y_1}{x_2 - x_1}\right) - \left(\frac{y_4 - y_3}{x_4 - x_3}\right)} \quad (\text{B.16})$$

$$y' = \left(\frac{y_2 - y_1}{x_2 - x_1}\right)x' + y_1 - \left(\frac{y_2 - y_1}{x_2 - x_1}\right)x_1 \quad (\text{B.17})$$

Substitution of θ_2 and θ_3 into Equation B.2 gives the following expression for the total internal work.

$$W_i = 2M_p\theta_1 D (1 + (L''/L') + (L''/L')(B'/B'')) \quad (\text{B.18})$$

By substituting Equations B.5 through B.8 into Equations B.18, the following equation is derived.

$$W_i = 2M_p\theta_1 D \left[1 + \left(\frac{\sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2}}{\sqrt{(x' - x_2)^2 + (y' - y_2)^2}} \right) + \left(\frac{\sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2}}{\sqrt{(x' - x_2)^2 + (y' - y_2)^2}} \right) \left(\frac{\sqrt{(x_3 - x')^2 + (y' - y_3)^2}}{\sqrt{(x_4 - x_3)^2 + (y_3 - y_4)^2}} \right) \right] \quad (\text{B.19})$$

B.2.3 External work

The external work is calculated by the summing all work terms, which encompasses the discrete applied load multiplied by their virtual displacement. It is dependent on the magnitude and location of the applied loads (i.e., V_i , H_i , x_i , and y_i) and the hinge locations (x_1 , x_2 , x_3 and x_4). The mode of collapse necessitates three separate segments for which the external work is calculated as follows.

B.2.3.1 Segment 1

For a small rotation θ_1 , the displacement at the second hinge is given by:

$$\delta_1 = L''\theta_1 \quad (\text{B.20})$$

The vertical and horizontal displacements at the second hinge are given in Equations B.21 and B.22 (Figure B.4).

$$\delta_{v,1} = \delta_1 \cos \phi = L'' \theta_1 \cos \phi = (x_2 - x_1) \theta_1 \quad (B.21)$$

$$\delta_{h,1} = \delta_1 \sin \phi = L'' \theta_1 \sin \phi = (y_2 - y_1) \theta_1 \quad (B.22)$$

The displacement of any load along the segment (i.e., V_i and/or H_i) is calculated in a similar manner. The total external work for the first segment is given by the summation of the loading multiplied by their virtual displacement as follows:

$$W_{e,1} = \theta_1 [\sum_{x_1}^{x_2} V_i x_i + \sum_{y_1}^{y_2} H_i y_i] \quad (B.23)$$

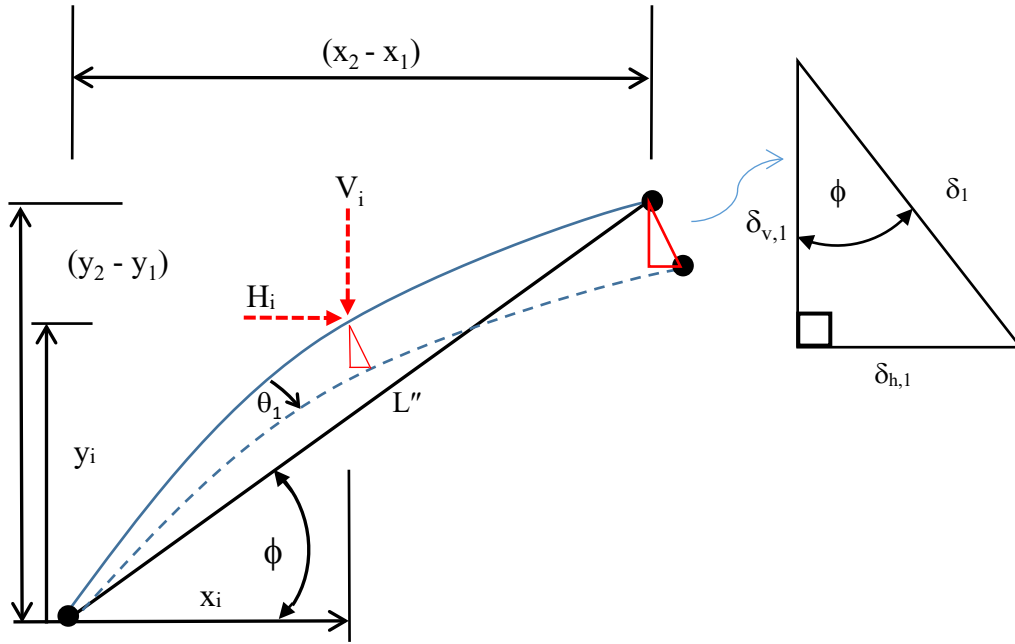


Figure B.4 – External work: Segment 1

B.2.3.2 Segment 2

The displacements at the second and third hinges corresponding to *segment 2* are calculated from the geometry shown in Figure B.5 as follows:

$$\delta_1 = L' \theta_2 \quad (B.24)$$

$$\delta_{v,1} = (x' - x_2) \theta_2 \quad (B.25)$$

$$\delta_{h,1} = (y' - y_2) \theta_2 \quad (B.26)$$

$$\delta_2 = B'\theta_2 \quad (B.27)$$

$$\delta_{v,2} = (x_3 - x')\theta_2 \quad (B.28)$$

$$\delta_{h,2} = (y' - y_3)\theta_2 \quad (B.29)$$

The total external work for the second segment is illustrated by equation B.30.

$$W_{e,2} = \theta_2 [\sum_{x'}^{x_2} V_i x_i + \sum_{x'}^{x_2} H_i y_i] + \theta_2 [\sum_{x'}^{x_3} V_i x_i + \sum_{x'}^{x_3} H_i y_i] \quad (B.30)$$

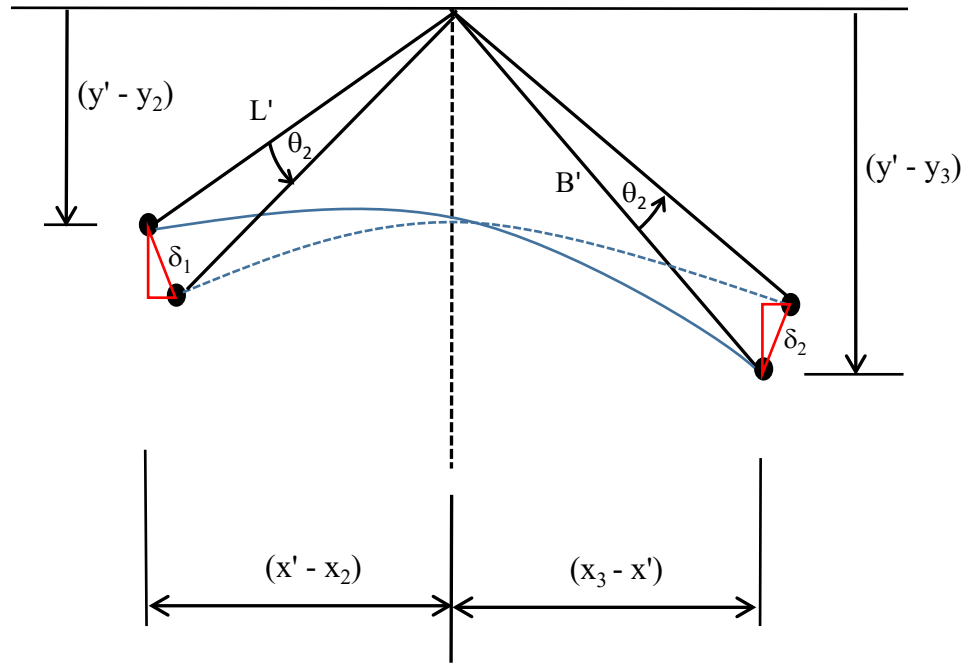


Figure B.5 – External work: Segment 2

B.2.3.3 Segment 3

The displacements at the third hinge are calculated as follows:

$$\delta_2 = B''\theta_3 \quad (B.31)$$

$$\delta_{v,2} = \delta_2 \cos \beta = B''\theta_3 \cos \beta = (x_4 - x_3)\theta_3 \quad (B.32)$$

$$\delta_{h,2} = \delta_2 \sin \beta = B''\theta_3 \sin \beta = (y_3 - y_4)\theta_3 \quad (B.33)$$

The total external work for the third segment (Figure B.6) is given in Equation B.34.

$$W_{e,3} = \theta_3 [\sum_{x_4}^{x_3} V_i x_i + \sum_{x_4}^{x_3} H_i y_i] \quad (B.34)$$

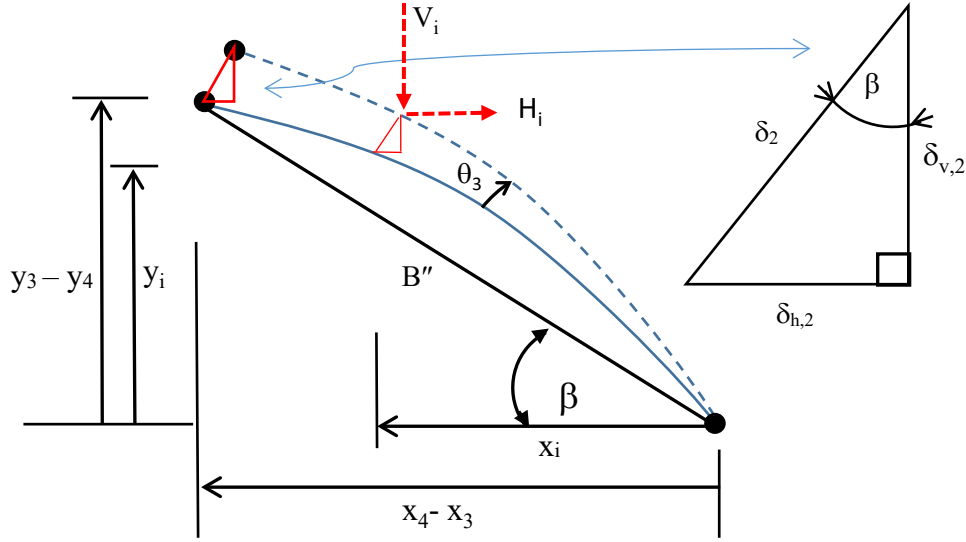


Figure B.6 – External work: Segment 3

B.2.3.4 Total external work

The total external work is the sum of

$$W_e = W_{e(\text{Segment 1})} + W_{e(\text{Segment 2})} + W_{e(\text{Segment 3})} \quad (B.35)$$

$$\begin{aligned} &= \theta_1 [\sum_{x_1}^{x_2} V_i x_i + \sum_{x_1}^{x_2} H_i y_i] + \theta_2 [\sum_{x'}^{x_2} V_i x_i + \sum_{x'}^{x_2} H_i y_i] + \theta_2 [\sum_{x'}^{x_3} V_i x_i + \sum_{x'}^{x_3} H_i y_i] + \theta_3 [\sum_{x_4}^{x_3} V_i x_i + \sum_{x_4}^{x_3} H_i y_i] \\ &= \theta_1 \{ [\sum_{x_1}^{x_2} V_i x_i + \sum_{x_1}^{x_2} H_i y_i] + (L''/L') [\sum_{x'}^{x_2} V_i x_i + \sum_{x'}^{x_2} H_i y_i + \sum_{x'}^{x_3} V_i x_i + \sum_{x'}^{x_3} H_i y_i] + (L''/L') (B'/B'') [\sum_{x_4}^{x_3} V_i x_i + \sum_{x_4}^{x_3} H_i y_i] \} \end{aligned}$$

B.4 Summary

By equating the external and internal work, the plastic moment capacity was determined. The critical collapse mechanism and moment capacity were obtained by maximising the expression for M_p through non-linear optimisation. The Evolutionary solver included in Microsoft's Excel 2011 package was used in the present study. The relevant constraints on the hinge locations are given in Equations B.36 – B.39.

$$0 \leq x_1 < x_2 \tag{B.36}$$

$$x_1 < x_2 < x_3 \tag{B.37}$$

$$x_2 < x_3 < x_4 \tag{B.38}$$

$$x_3 < x_4 \leq 2L \tag{B.39}$$

APPENDIX C: WIND LOADING ON CATENARY VAULT STRUCTURES

C.1 Introduction

From a structural point of view, the catenary is a highly efficient one-way spanning shell structure. However, the work published pertaining to wind loading on curved surfaces generally refers to circular vaults and domes, and no research was found in relation to catenary structures - apart from a study on catenary cross-vaults by Bradley *et al.* (2011). There is, however, a wealth of information on aspects relating to similar bluff body structures and wind-tunnel testing procedures, which are applicable to the current investigation. Several papers, including Paluch *et al.* (2003), Blessmann (1996), Taylor (1991), Cheung and Melbourne (1983), which discuss experimental investigations on single vaults, domes and curved roof structures, were considered. Some studies have also been conducted to assess the sensitivity of surface pressures (over curved surfaces) to specific parameters, namely: aspect ratio, geometry, surface roughness and the related Reynolds number. Of interest are the contributions of Blackmore and Tsokri (2006) and Toy and Tahouri (1988) (in regard to circular vaults) from which selected data is evaluated against the current results.

Blackmore and Tsokri (2006) conducted a series of wind tunnel tests on circular vaulted roofs with different length to diameter ratios (L/d), height to diameter ratios (f/d) and with different sidewall heights (h). From these tests the authors noted that the EN 1991-1-4 values for Zone B (i.e., over the central zone) appear to be most appropriate for long vaulted buildings (i.e., $L/d > 10$) with two-dimensional flow rather than shorter buildings that generate three-dimensional flow. The authors also proposed a procedure using EN 1991-1-4 duo-pitch roof data for the design of circular vaults for wind normal to the eaves. Toy and Tahouri (1988) conducted tests on two and three-dimensional semi-cylindrical models with different cross-sectional geometries and length/height (L/H) ratios in a turbulent boundary layer. Of particular interest are the mean surface pressure distributions over a pointed circular vault, which is similar in shape to a steep catenary vault, in three-dimensional flow.

C.2 Aims and Objectives

The aims of the investigations described in this appendix are as follows:

- To establish the applicability of the design pressure coefficients for circular and duo-pitch roofs stipulated in SANS 10160-3: 2011, to catenary roofs (therefore, in principle, the pressure coefficients that are considered in the current study correspond to the $C_{pe,10}$; i.e., largest *overall* loaded areas of 10m^2 , as specified in SANS 10160-3).
- If necessary, to develop specific external pressure coefficient data for catenary vault structures.

In order to achieve these objectives, a series of wind-tunnel tests were undertaken at the boundary-layer wind tunnel (BLWT) of the Council for Scientific and Industrial Research (CSIR), Pretoria, South Africa. Five vault roof forms were tested in a turbulent boundary layer at multiple wind orientations. Only the mean and root-mean-square (rms) pressure coefficients are considered in this investigation.

C.3 Wind Tunnel Measurements

C.3.1 Wind-tunnel facility

The boundary-layer wind tunnel used in this study is an open circuit type, with a centrifugal fan, powered by a 75 kW thyristor-controlled motor. The cross-section is 1 m high x 2 m wide, with an 18 m long boundary-layer development length. The facility can produce a maximum wind speed of approximately 22 m/s. Variation in wind orientation is accomplished by a ‘turn table’ situated in the test section. Over the test section, the wind-tunnel is equipped with an adjustable roof in order to minimise the effects of blockage. The wind tunnel facility and test section are shown in Figures C.1a and C.1b respectively. An overview of this facility and its capabilities are presented in Milford *et al.* (1989).

C.3.2 Boundary layer modelling

A standard ‘open terrain’ boundary-layer profile, as specified in the literature (Davenport *et al.* 2000) was achieved using a honeycomb mesh screen at the entrance of the tunnel (to introduce the initial small-scale turbulence), four 250 mm high and 100 mm wide triangular spires, followed by a 50 mm high tripping wall (to achieve an initial speed deficit in the lower section of the profile). The boundary-layer was developed by a 16 m length of corrugated cardboard with a uniform roughness height of approximately 2.5 mm. Figures C.2a and C.2b show the mean wind speed and turbulence intensity profiles together with the Davenport’s reference data. The longitudinal turbulence spectrum, corresponding to a

50 mm elevation above the wind-tunnel floor, is shown in Figure C.2c. It can be seen that the mean wind speed profile achieved in the wind-tunnel compares well with the reference data, but the turbulence characteristics do not match exactly with those stipulated for open terrain. This mismatch constitutes a typical limitation of boundary layer modelling and larger differences are also reported in full-scale experimentation (Goliger and Milford, 1985).

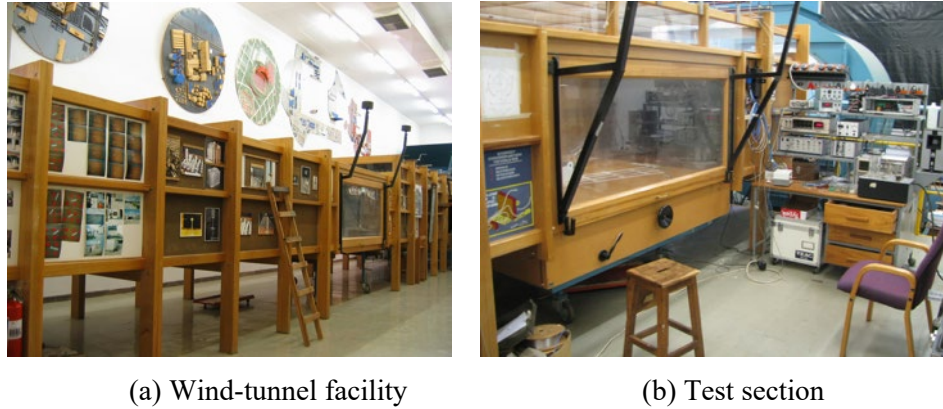
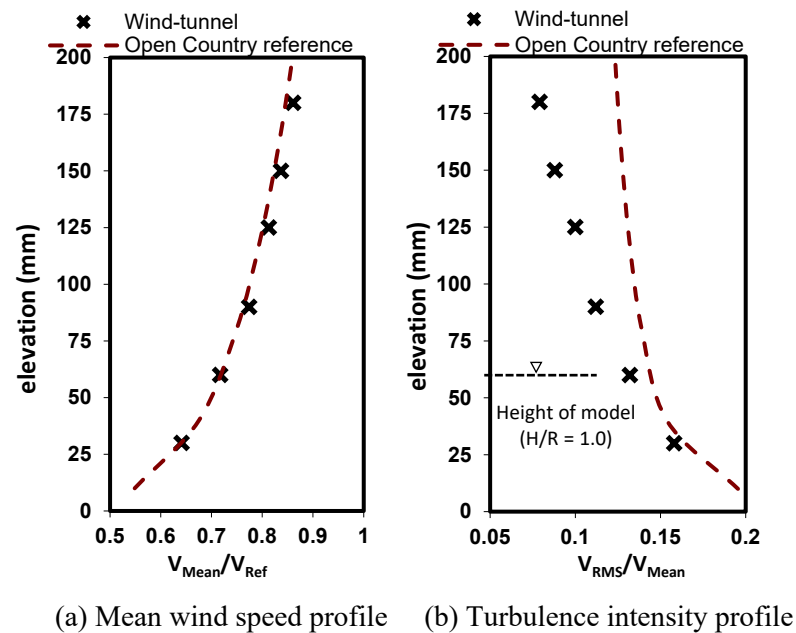
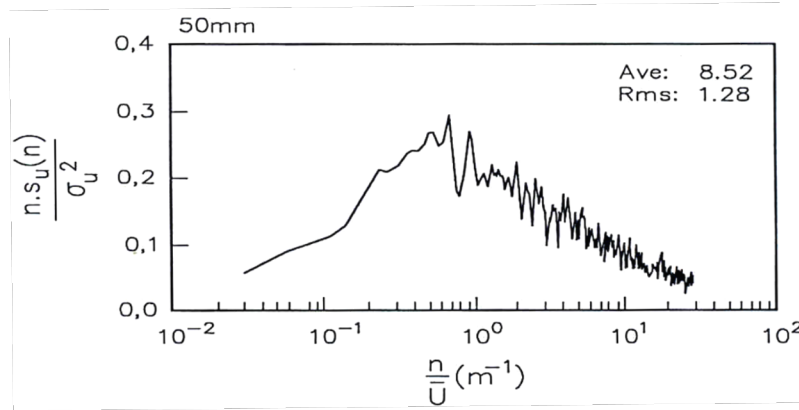


Figure C.1 – Boundary-layer wind-tunnel





(c) Longitudinal turbulence spectra

Figure C.2 – Boundary layer characteristics

C.3.3 Pressure and velocity measurements

Point pressures were measured for wind directions between 0° and 360° , with an interval of 15° . These measurements were obtained using a Synchronous Multi Pressure Scanning System (SM-PSS)(based on RAD3200 manufactured by the Scanivalve Corporation). The scanner type is the ZOC23B/8px, which contains 8 individual piezo-electric pressure gauges. Two ZOC23B/8px units, capable of instantaneous measurements at 16 pressure taps, were used in this investigation. The pressure scanners, stainless steel tubing connectors (mounted to the circular model) and flexible plastic tubing are shown in Figure C.3.

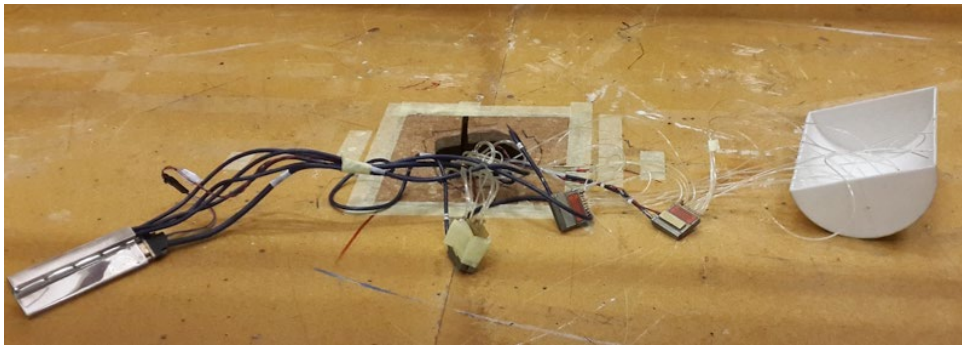


Figure C.3 – ZOC23B/8px scanners attached to the circular model

Pressure taps were connected to the system using tubing of 1.0 mm internal diameter (The nature of pressure measurement system enabled the use of short tube lengths and consequently flow restrictors, which account for the effects of aerodynamic resonance, were not required). The reference static pressure was obtained from the wind-tunnel roof,

above the model, and all pressure coefficients were normalised by the free-stream dynamic pressure at apex height of the relevant vault model. A sampling frequency of 625Hz was used in this study.

Pressure signals were recorded in the form of a time series. A data processing program, developed in excel, was used to calculate the mean and rms pressure coefficients. The equations used for the calculation of the mean and rms coefficients are defined as follows:

$$\text{Mean - } Cp_{mean} = \frac{1}{N} \sum_{i=1}^N \frac{Pi}{Q_{Ref}} \quad (C.1)$$

$$\text{Root-mean-square - } Cp_{RMS} = \frac{\sqrt{\frac{1}{N} \sum_{i=1}^N (Pi - P_{mean})^2}}{Q_{Ref}} \quad (C.2)$$

where;

p_i	i^{th} pressure measured at a tap on the roof
p_{mean}	mean pressure
Q_{Ref}	free-stream dynamic reference pressure at roof apex level
N	number of samples

Testing was conducted at a wind speed of approximately 11.5 m/s, measured by a Pitot tube located at 750 mm above the wind-tunnel floor. Additional wind speed measurements were taken using a hot-wire anemometer system. The blockage introduced by the model was less than 1.0% of the tunnel cross-section, which is well below the generally accepted limit of 5% (beyond which blockage effects need be considered) noted in the literature (Stathopoulos and Baniotopoulos, 2007). Therefore, blockage effects were considered insignificant in these investigations.

C.3.4 Models

A geometrical comparison between catenary and circular profiles, corresponding to H/R ratios of 0.2, 0.6 and 1.0, are shown in Figure C.4 (R refers to half of the base width and H is the vault height). It is apparent that for low H/R ratios (e.g., 0.2), the profiles are identical, for 0.6 they are fairly similar, whereas for 1.0 the difference is pronounced. Four catenary vault models (and a single semi-circular model), at a scale of 1 in 500, with different ratios of height to base width, were manufactured using a three-dimensional printer. The models

were initially drawn using Autodesk's AutoCAD software and then printed at the CSIR. The printer utilises colour inkjet technology in which successive layers of material are printed to create a three-dimensional shape. This method offered high precision (e.g., surface finish) and significantly improved the speed of construction - all models were printed simultaneously (the CAD arrangement is shown Figure C.5a). A comparison of the catenary and semi-circular models (with attached sandpaper), corresponding to $H/R = 1.0$, are shown in Figure C.5b. The catenary model profiles are illustrated in Figure C.5c and all relevant geometric parameters are given in Table C.1.

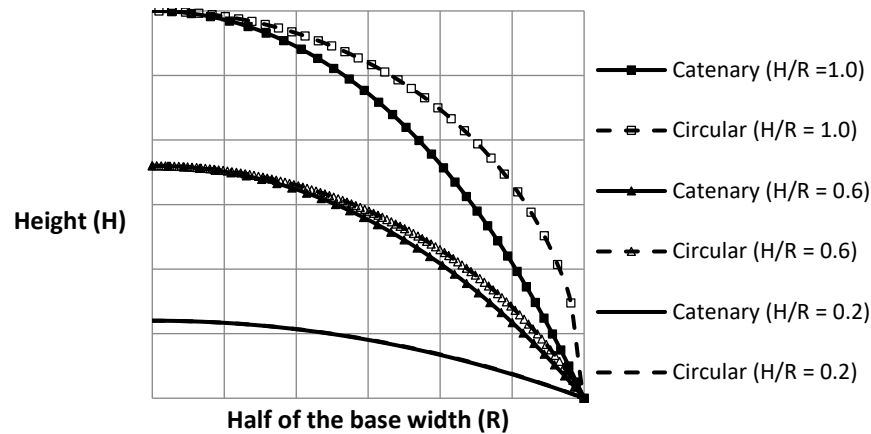
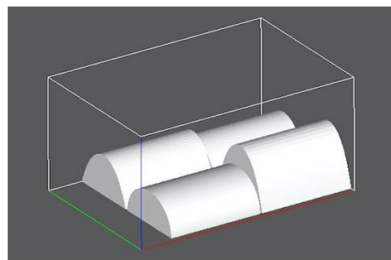
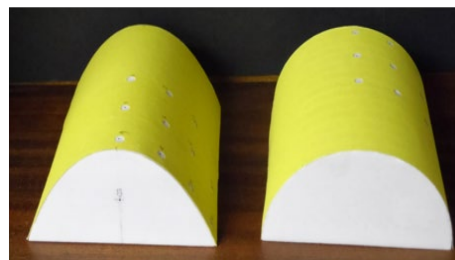


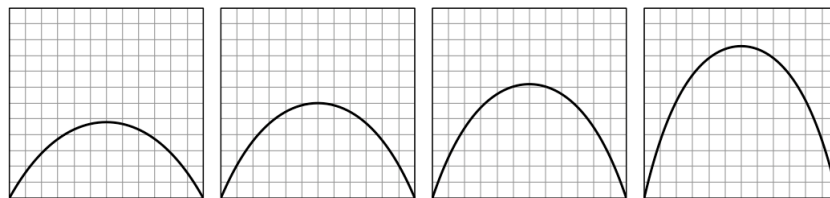
Figure C.4 – Half profiles for catenary and circular vaults



(a) CAD setup



(b) $H/R = 1.0$ models



(c) Catenary profiles (From left: $H/R = 0.8$; 1.0; 1.2; and 1.6)

Figure C.5 – Profiles for catenary and circular vaults

Table C.1 – Model dimensions

Shape	Apex Height (H) (mm)	(H/R)
Catenary	48	0.8
Catenary	60	1.0
Catenary	72	1.2
Catenary	96	1.6
Semi-Circular	60	1.0

*Note: All models have width (D) = 120mm, length (L) = 160mm, and footprint aspect ratio (L/R) = 1.33

In all models, only a quarter of the roof was instrumented with twelve pressure taps, as shown in plan and elevation, in Figure C.6. A single pressure tap was also located at the centre of a sidewall for each model. Due to the geometric symmetry of the models, and by applying a range of wind directions from 0° to 360° , effectively 35 roof pressure locations, distributed over the entire roof, were investigated. In this study, the wind direction is denoted by α , whereas the geometric inclination angle is represented by θ , as illustrated in Figure C.6.

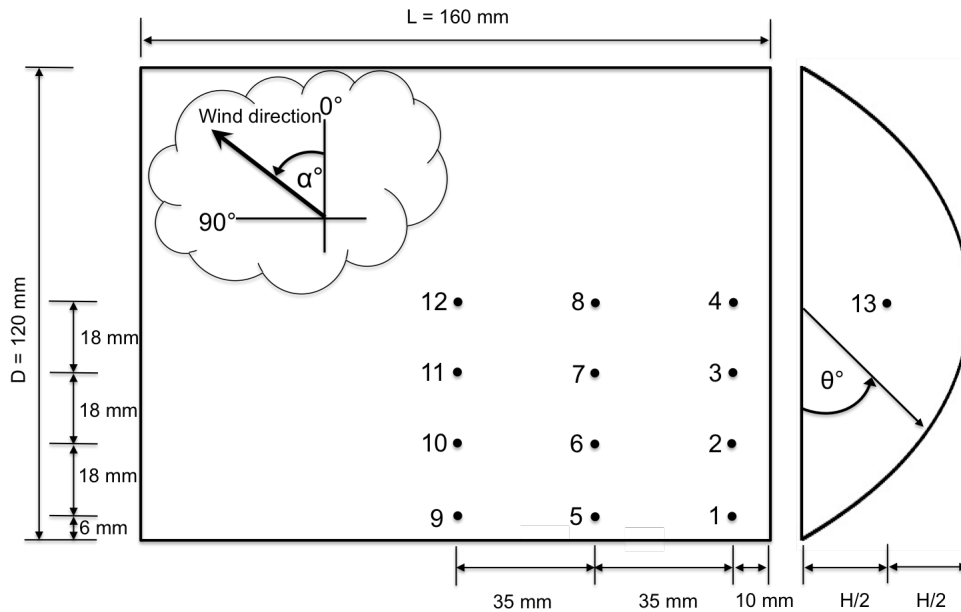


Figure C.6 – Pressure tap locations

C.3.5 Reynolds number modelling

For all structures, flow separation points determine the distribution of external pressures. In the case of elements with sharp edges, this separation takes place (and is controlled) by

these edges. However, this is not the case for rounded surfaces. Numerous studies on the pressure distributions over rounded surfaces (Taylor, 1992 and Cheung and Melbourne, 1983) have shown that the points of flow separation, wake characteristics and consequently the pressure distribution are dependent on the viscous forces in the boundary layer. These are characterised by Reynold's number, which defines the relationship between the characteristic wind speed (V), size of the structure (D) and air viscosity (ν), as follows $Re = VD/\nu$. The Reynolds number corresponding to a typical vault, with a size comparable to a small house, is about 10^7 , and in the model scale of about 10^5 . These correspond to the *supercritical* and *subcritical* Reynolds number regimes respectively, and the loading implications are reflected in SANS 10160-3. In order to achieve similitude between model and full-scale flow regime, various artificial means may be implemented, which include:

- manipulating viscosity of the flow (e.g., introducing gases of different characteristics),
- roughening of the model surface (e.g., sand grains or sandpaper), and
- increasing the level of turbulence in the approaching flow

The procedure given in the Engineering Science Data Unit (ESDU) - 80025 (1980) for simulating the pressure distribution over a circular cylinder in a high Reynolds number flow, was adopted in this study. Essentially, the model surfaces were roughened to achieve Reynolds number regime similitude. Roughening was accomplished by attaching sandpaper to the model surface, which included 4 mm diameter holes cut out around pressure taps to prevent the influence of the grains in the immediate vicinity of the taps, as shown in Figure C.7. The standard used to classify the sandpaper grain sizes was that of the Federation of European Producers of Abrasives (FEPA). The ESDU-80025 (1980) report suggests, from experimental evidence, that the mean quoted equivalent uniform sand-grain roughness size should be increased by 50% due to variation of grit size about the manufacturers' mean and the uneven distribution of grains over the surface.

The required size of the grain (k) was determined based on ESDU-80025 (1980). The relevant procedure was used to calculate the mean centreline pressure distribution over a circular cylinder with different surface roughness sizes and geometric scales. The results of these calculations, for several differing model roughness sizes, and the most appropriate full-scale surface finish are shown in Figure C.8. The model (with P100 sandpaper), and a full-scale structure (i.e., rough concrete finish) generate similar pressure distributions.

Consequently, this roughness of paper was adopted (giving k/H ratios from 2.5×10^{-3} to 5×10^{-3}).

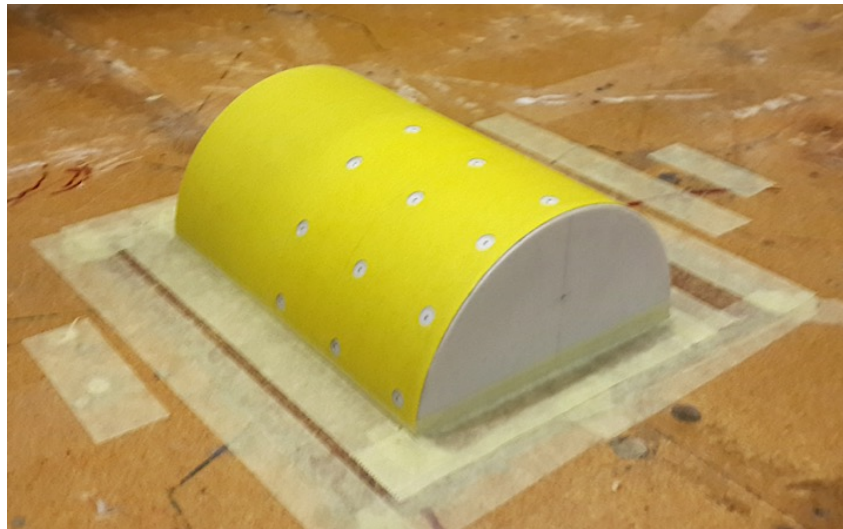


Figure C.7 – Tap surroundings (Circular Vault)

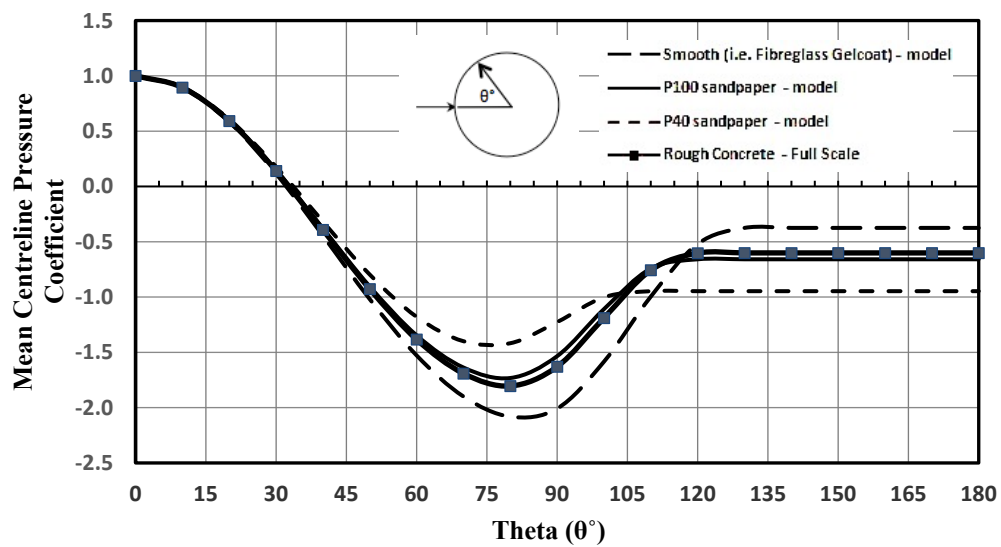


Figure C.8 – Mean centreline pressure coefficient distribution: ESDU procedure

C.3.6 Repeatability

In order to assess the accuracy of measurements, the mean pressure distribution over the semi-circular vault was measured approximately a month apart. These tests revealed acceptable levels of consistency. Furthermore, three experimental runs (across all wind orientations) were carried out to ascertain the extent of experimental scatter. Generally

good correlation was achieved, as demonstrated in Figure C.9 (for the catenary vault with $H/R = 1.2$).

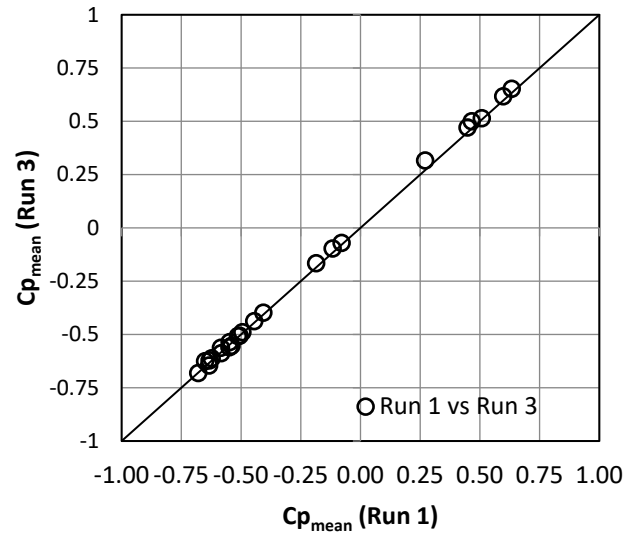


Figure C.9 – Experimental Scatter

C.4 Results and Discussion

C.4.1 Effect of wind orientation (α)

Initially, the variation of surface pressure coefficient (at selected positions) with wind direction is discussed. In general, similar trends were observed for all models investigated; however, significant variation is evident at certain tap locations and wind orientations. The mean pressure coefficient plots for four selected tap locations, namely: base edge (tap 1), base middle (tap 9), apex edge (tap 4), and apex middle (tap 12) are shown in Figures C.10a to C.10d (Note that these figures are plotted to two different scales, for clarity purposes).

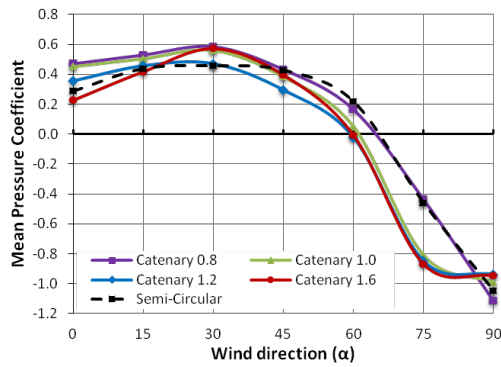
It is evident, in Figures C.10a and C.10b, that the pressure coefficients corresponding to the base taps (1 and 9) show a relationship in which they initially experience positive pressures until the presence of the sharp edge begins to control the location of flow separation. At wind directions (α) between 0° and 60° pressures are positive, and the switch from positive to suction pressures roughly occurs between 60° and 70° for all models. It can also be seen in Figures C.10a and C.10c, that the highest negative pressure coefficients for the ‘edge’ taps (1 and 4) occur between 75° to 90° . The reason for these larger suctions is the close proximity of these taps to the position of flow separation at these wind orientations.

Considering tap 12 (Figure C.10d), a substantial shift in the magnitude of pressure coefficients is apparent, at a wind orientation between the 15° and 45° , for all models. In fact, for the orientation of about 30° , the highest magnitude suction in the entire study were measured. For H/R ratios between 0.8 and 1.2, pressure coefficients range between $C_{pe} \approx -1.6$ and $C_{pe} \approx -1.3$ respectively. For the highest aspect ratio (H/R = 1.6), pressure coefficient was less negative ($C_{pe} > -1.4$) and took place at $\alpha = 45^\circ$. This observation indicates the trend in which shallower catenary vaults generate higher negative pressures, and furthermore, the importance of three-dimensional flow for structures with a low footprint aspect ratio; i.e., L/D.

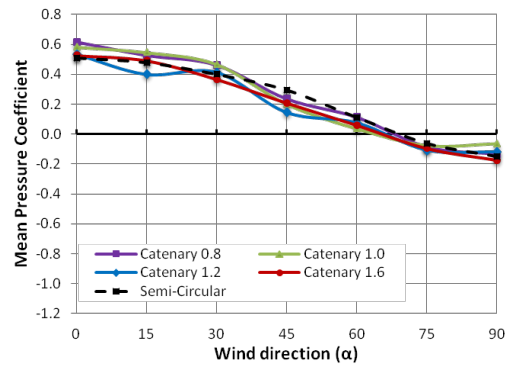
Figures C.11a to C.11g and Figures C.12a to C.12g present contour lines of mean and rms pressures, respectively (These plots were produced using MATLAB[®] subroutines, with the adoption of cubic interpolation to approximate contour lines). The plots are presented in terms of length (x-axis) and inclination angle θ° (y-axis). Note that these figures do not depict the actual footprint of the roof but are distorted geometrically for brevity reasons. These graphs, for H/R=1.0 only, confirm the observation made above in which most negative pressures (i.e., dark blue) correspond to $\alpha = 30^\circ$. Furthermore, the largest positive pressures (i.e., intense red) correspond to the 0° orientation for all models. Both negative and positive pressures are relevant, as they can lead to critical loading situations and are thus the focus of further analysis later in this thesis.

The distribution of mean pressure coefficients measured at mid-height of the gable wall is shown in Figure C.13. As expected, the maximum positive pressure, $C_{pe} > +0.8$, occurs for wind blowing onto the wall (90° orientation), and its magnitude is similar to that which could be derived for vertical walls of rectangular plan buildings (SANS 10160-3). For a wind orientation of $0^\circ/180^\circ$ (flow parallel to the gable walls), the negative mean pressure coefficient is $C_{pe} \approx -0.5$. For a wind orientation of 270° (pressure tap located at the leeside of the structure), the negative mean pressure coefficient is $C_{pe} \approx -0.2$. Both are visibly smaller than the respective values that could be established from the loading code. Intuitively, such relationships seem to be correct, due to the lack of a vertical edge (i.e. walls) along which separation of the flow takes place. In the case of the curved edge, part of the air volume is directed upwards, thus reducing the volume of the flow subjected to flow separation. Having said this, for both cases (i.e., $\alpha = 0^\circ$ and $\alpha = 270^\circ$), the adjacent orientations ($\pm 15^\circ$) generate higher magnitude negative pressures, like those stipulated in

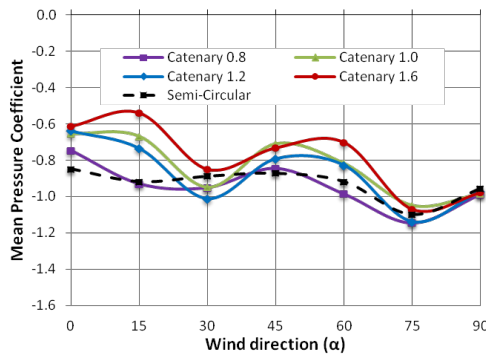
the loading standard (i.e., for $\alpha = 195^\circ$ and $= 345^\circ$). For these orientations, the presence of the separation edge becomes more prominent, which explains larger magnitude suctions. To be noted the apparent lack of perfect symmetry of the graph (i.e., small differences in the magnitude of pressure coefficients) serves as an indication of the levels of experimental inconsistencies related to models, geometrical setting up and the data capturing system.



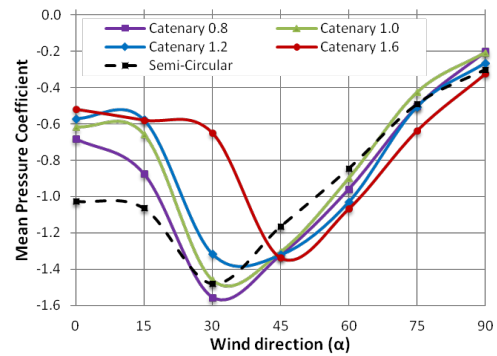
(a) Base Edge Tap (1)



(b) Base Centre Tap (9)

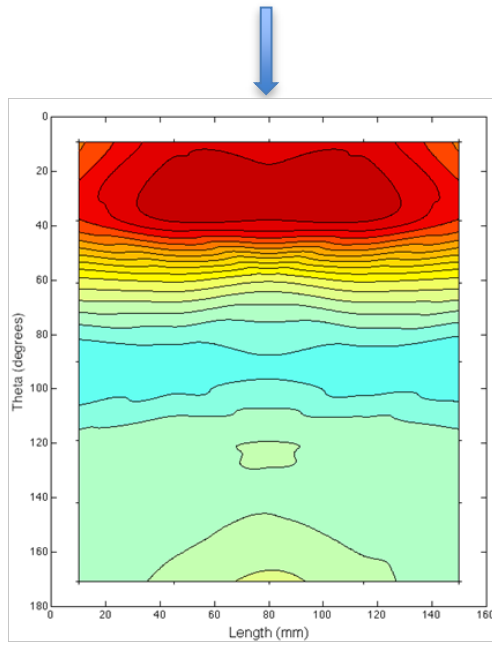


(c) Apex Edge Tap (4)

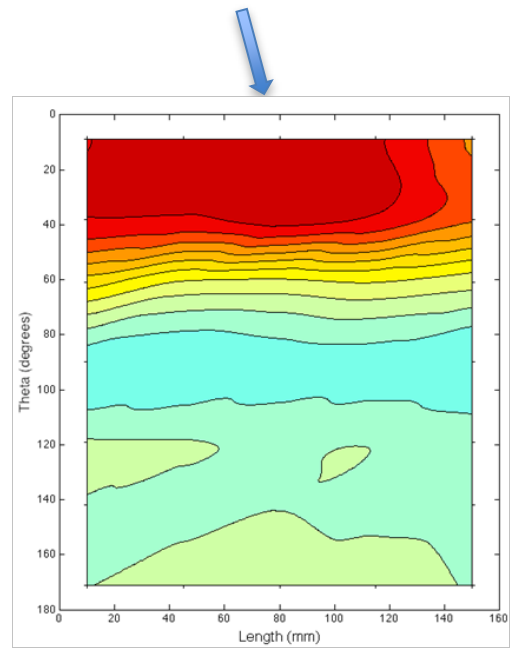


(d) Apex Centre Tap (12)

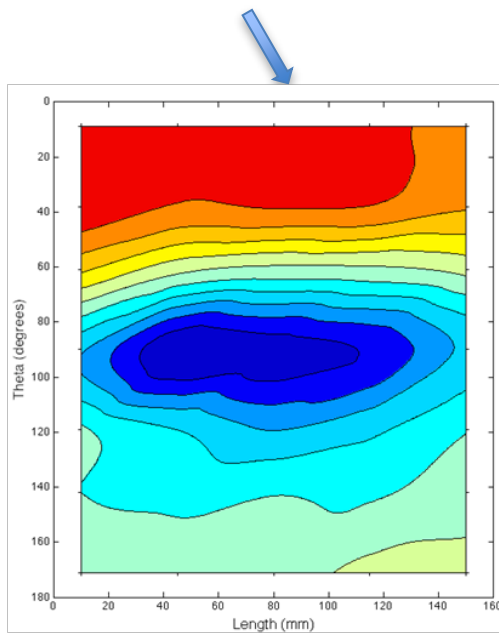
Figure C.10 – Mean pressure coefficients at selected locations



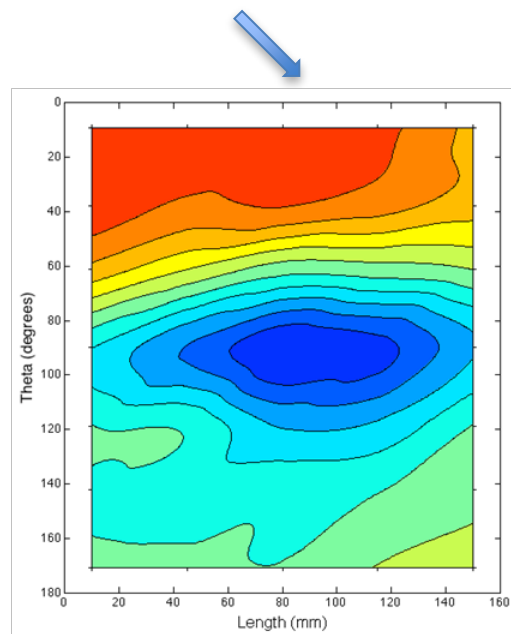
(a) $\alpha = 0^\circ$



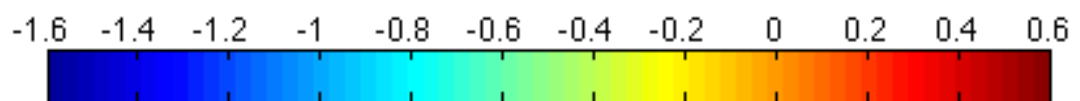
(b) $\alpha = 15^\circ$



(c) $\alpha = 30^\circ$



(d) $\alpha = 45^\circ$



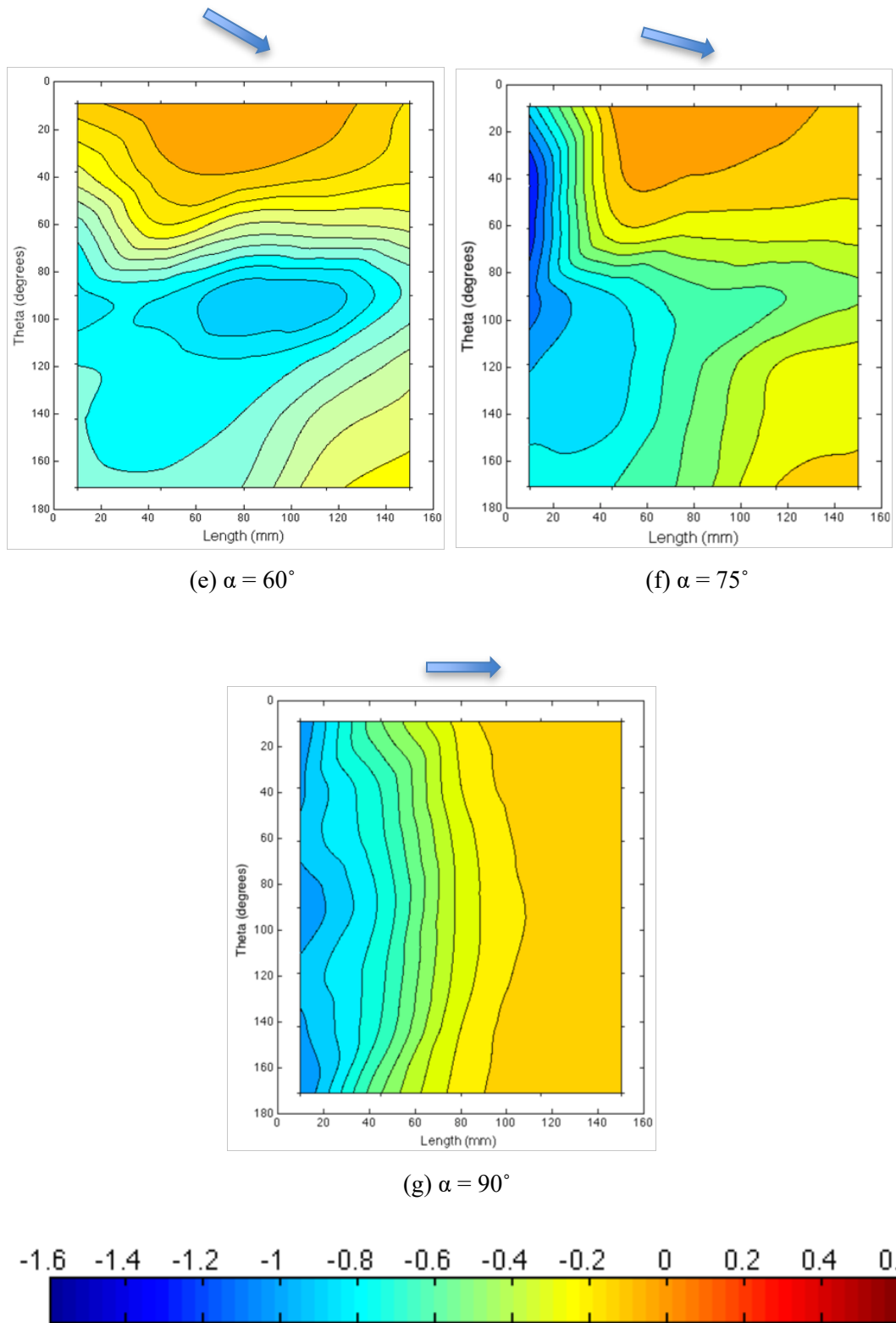
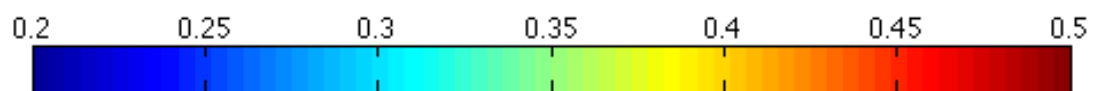
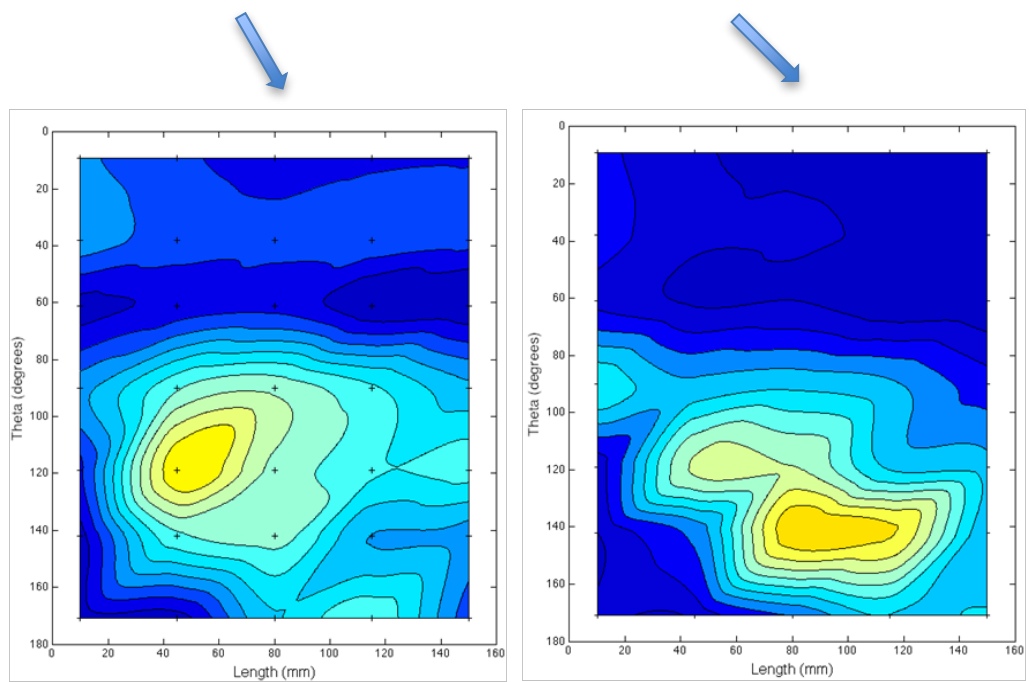
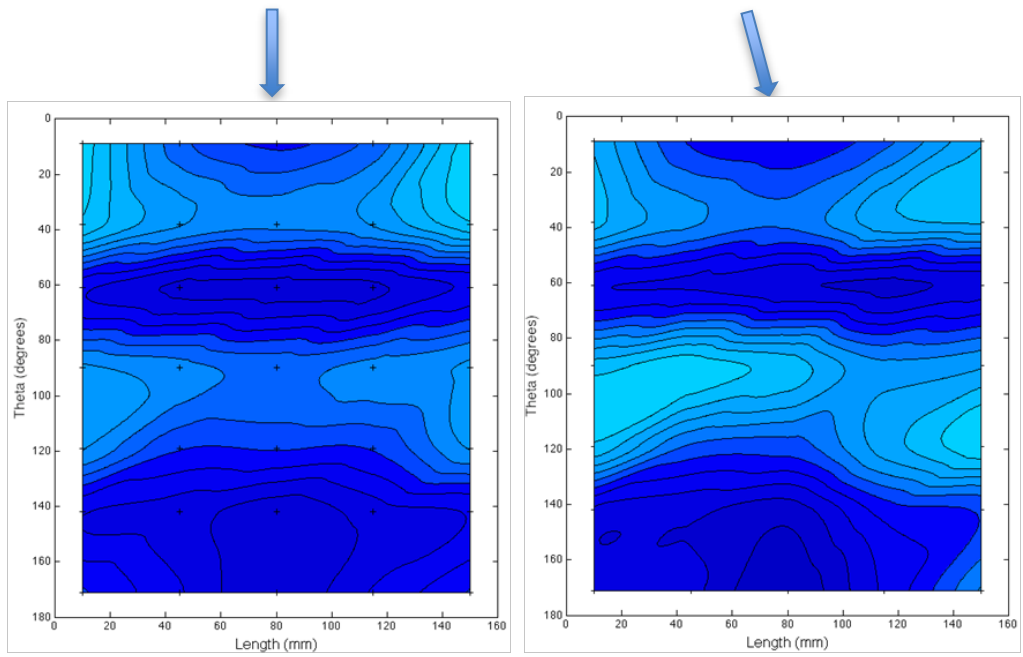
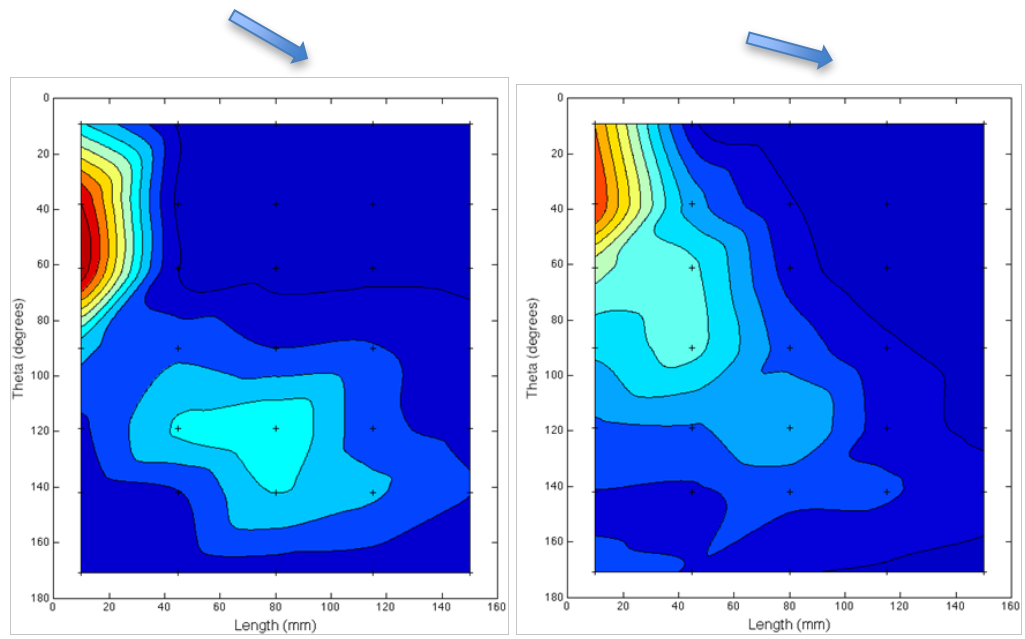


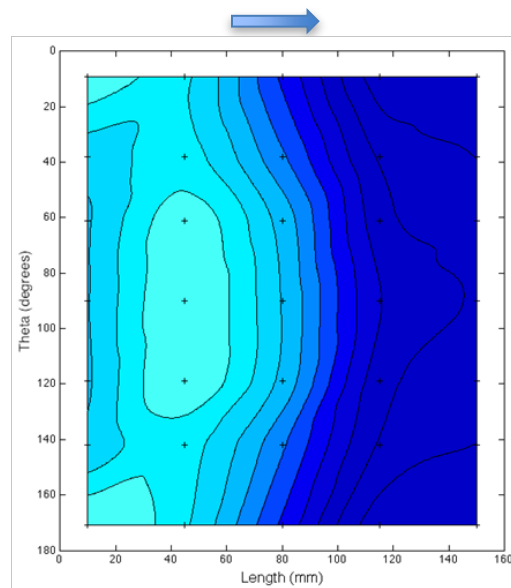
Figure C.11 – Mean pressure coefficient contour plots: $H/R = 1.0$





(e) $\alpha = 60^\circ$

(f) $\alpha = 75^\circ$



(g) $\alpha = 90^\circ$

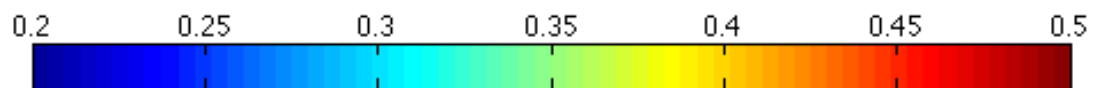


Figure C.12 – Root-mean-square pressure coefficient contour plots: $H/R = 1.0$

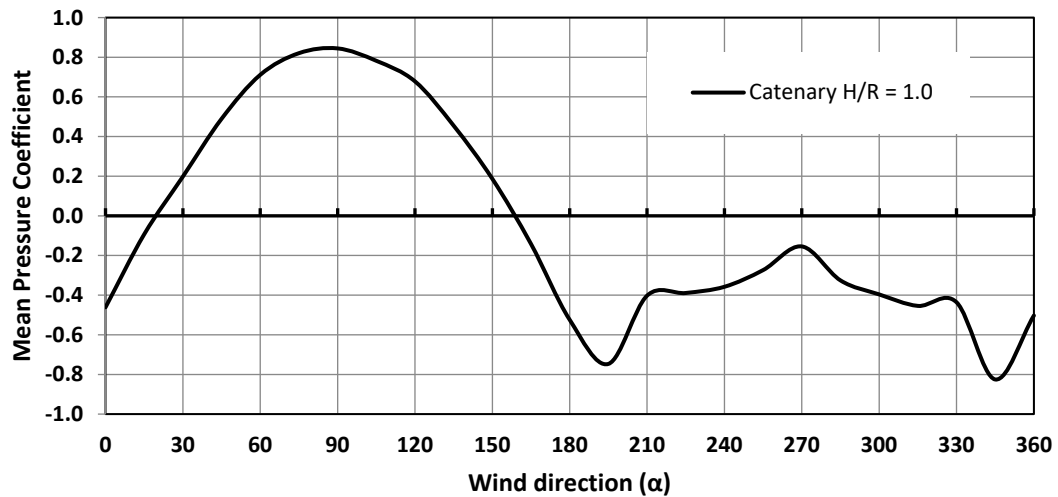


Figure C.13 – Mean pressure coefficients: mid height sidewall

C.4.2 Geometry of catenary (H/R)

The current section summarises outcomes of the investigation into the influence of (H/R) ratio on wind pressure distribution, which was aimed at establishing the relevant design data information for critical wind orientations.

C.4.2.1 Wind parallel to the gable walls ($\alpha = 0^\circ$)

Mean and root-mean-square pressures measured along the centreline, for the four catenary vault models, are compared in Figures C.14a and C.14b respectively. The results of these tests reveal that there is a substantial mean pressure coefficient difference between all vaults on the windward side. The contour plots in Figure C.15 – in which warm colours indicate positive pressure and cooler colours indicate suction, further illustrate this point. It is apparent in Figure C.15a through C.15d that by increasing the H/R ratio, and effectively the average slope of the roof, the point at which positive pressure transitions to negative pressure (i.e., suction) moves upward along the windward face. This trend has been observed on other similar bluff body structures (i.e., circular vaults and duo-pitched roofs). This shift (when H/R is increased) generates a larger area subjected to positive pressures on the windward surface, which has important design implications that are discussed later in this appendix.

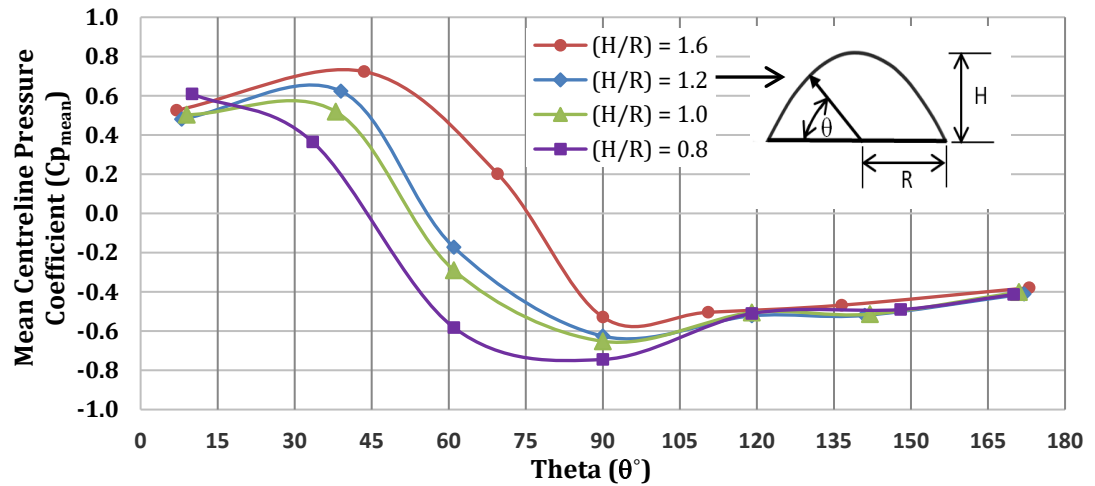


Figure C.14(a) – Mean centreline pressure coefficient distribution ($\alpha = 0^\circ$)

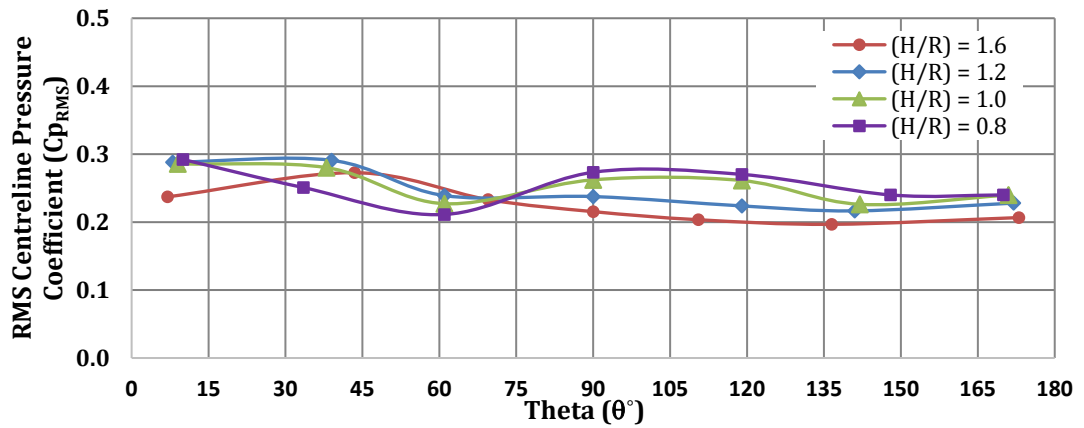


Figure C.14 (b) – RMS centreline pressure coefficient distribution ($\alpha = 0^\circ$)

The opposite can be observed in respect of the mean suction pressures. The minimum, or most negative, mean pressures occur near the apex of all four models. Furthermore, if to compare the contour plots (Figure C.15a – C.15d) it is evident that the extent of these minimum pressures (shown in dark blue) decreases with an increase in height to radius ratio of the vaults. This also illustrates the variation in the position of flow separation between the steepest and shallowest vaults. It is observed that the pressures on the leeward side of the $H/R = 1.6$ catenary vault are near constant, from the roof apex, i.e., from $\theta = 90^\circ$; however, the distribution for the $H/R = 0.8$ model reveal suctions become fairly constant at an elevation angle of roughly 120° , which is similar to that noted for the semi-circular vault. Though, for the most part, only negligible variation between all four models is evident over the leeward half for this wind orientation.

An interesting trend can be noted in respect of the rms values (Figure C.14b) in which the levels of fluctuations are practically identical at a location determined by an inclination $\theta \approx 70^\circ$. Furthermore, generally lower magnitude of rms is observed for shallow vaults at $\theta < 70^\circ$ and opposite for $\theta > 70^\circ$. This ‘switch’ in the turbulence contents of the flow, indicates complex relationships governing changes in flow regimes (which may justify more in-depth investigation involving flow visualisation).

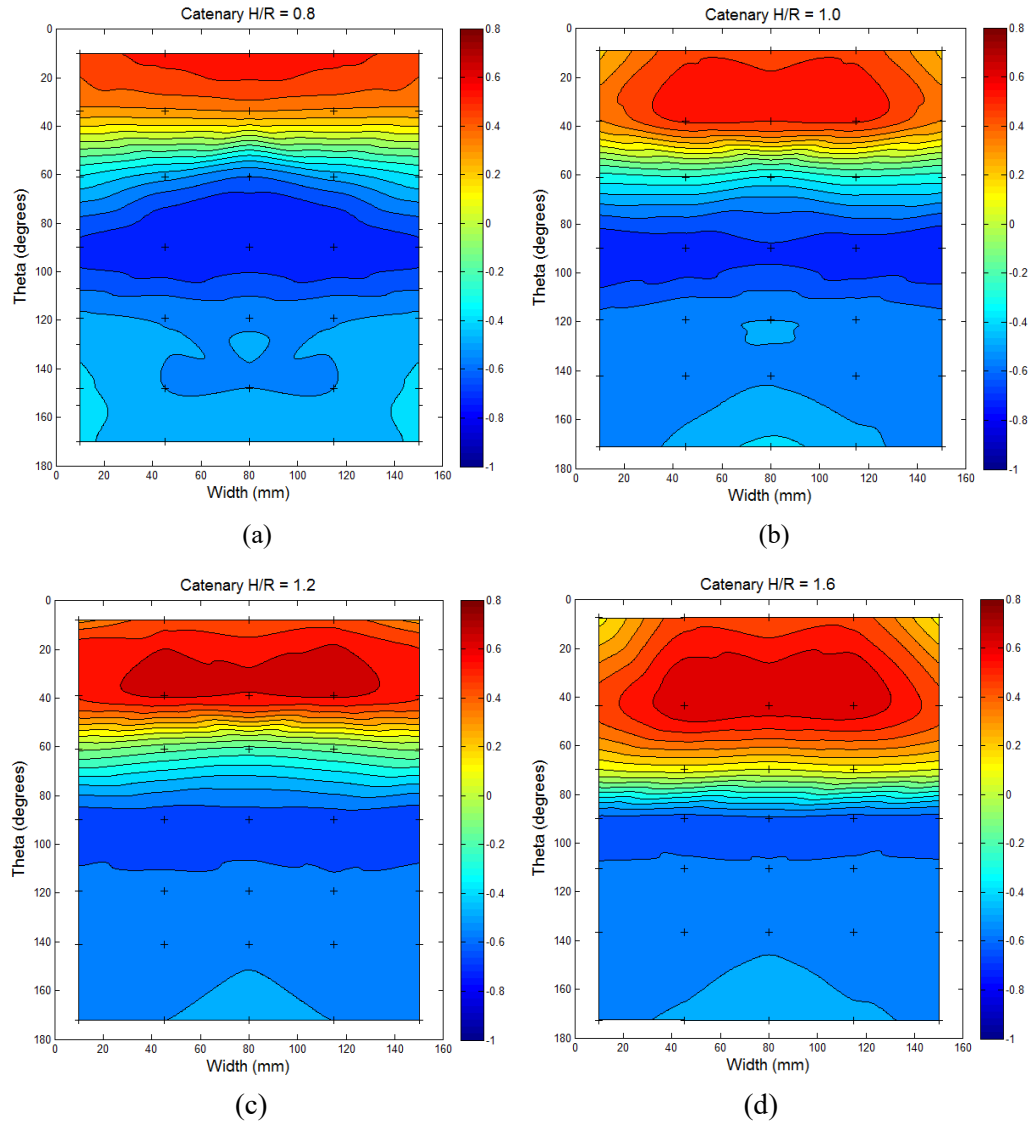


Figure C.15 – Mean pressure contour plots ($\alpha = 0^\circ$)

It should be noted that pneumatic averaging is regularly adopted to calculate mean pressure or force coefficients for surfaces and components. Goliger and Milford (1990) compared

numerically and pneumatically averaged surface pressures over a hangar. These authors noted very good mean pressure consistency, even though point pressures measured at each tap varied considerably. However, Szepessy and Bearman (1993) and Bradley (2011) have observed errors when using pneumatic averaging devices for curved geometries. Szepessy and Bearman note that using pressure taps with different angles to the oncoming flow (i.e., on curved elements) caused, in conjunction with large pressure gradients, a misreading of the average pressure. These authors postulate that the inclination of the taps to the oncoming flow affects the pressure losses at the tap surface connections. Data from Bradley (2011) shows that pneumatically averaged pressures were consistently lower than those averaged numerically (which were calculated for an area comprising 4 taps of equivalent tributary area), with a maximum error of approximately 30%. Bradley (2011) also observed that the magnitude of these discrepancies typically increased with increasing pressure gradient.

Due to these observations, numerical averaging was adopted in this study. Thus, the magnitude of the design pressure coefficient corresponding to each zone was determined through numerical averaging of the point pressure distributions over the respective area. Each zone was divided into six equal segments and the ‘actual’ distribution (derived through cubic interpolation) was approximated and replaced by uniform values. The coefficient for each zone was approximated as given in Equation C.3 below:

$$Cp_{e,Zone\ i} = \frac{\sum_{j=1}^{j=6} Cp_{ej}}{6} \quad (C.3)$$

The variation of pressure coefficients with H/R ratio, for $\alpha = 0^\circ$, are given in Figure C.16. An attempt was made to interpret these pressures in a similar a manner as those given in SANS 10160-3 for circular vaults. However, due to the nature of the wind pressure distributions over the steeper catenary vaults, it was considered necessary to subdivide the central portion of the roof (Zone B – in SANS 10160). Consequently, four loading zones A-D, are stipulated. The distribution / extent of these zones is illustrated in Figure C.16. It is interesting to note that the numerically averaged pressure coefficients are linearly related to the H/R ratio.

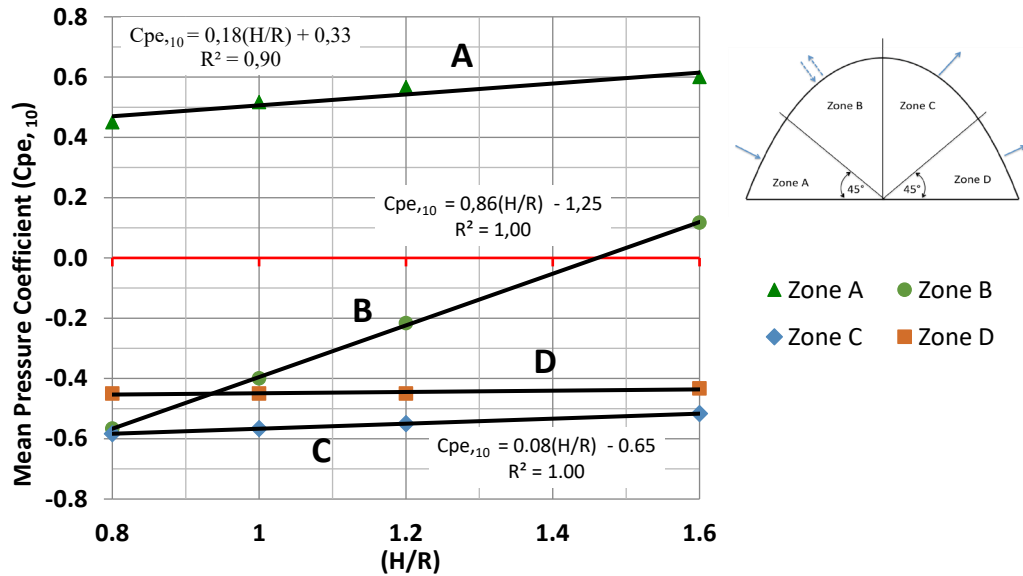


Figure C.16 – Design pressure coefficients ($\alpha = 0^\circ$)

C.4.2.2 Oblique wind orientations ($\alpha = 30^\circ; =45^\circ$)

As discussed in the previous section, the largest negative pressures develop for wind orientations (α) between 30° and / or 45° for all catenary vaults, and the respective pressure coefficient design data is considered in this section. The data corresponding to most negative suctions at both orientations, and for each catenary profile, is plotted in Figure C.17a. For aspect ratios $H/R = 0.8, 1.0$ and 1.2 , the graphs correspond to $\alpha = 30^\circ$; whereas for $H/R = 1.6$ the graph corresponds to $\alpha = 45^\circ$. This may explain the difference in these distributions. It also reconfirms the previous observations that the most negative mean pressures correspond to shallowest catenary vault. It can also be observed that at lower elevations, over the lee-side of all vault types (for $\theta > 140^\circ$), the magnitude of pressure coefficients is similar ($-0.5 > C_{pe} > -0.6$). Figure C.17b shows the rms centreline pressure distribution corresponding to the 30° wind orientation. Large variation between all catenary vaults, over the leeward half, is observed. The largest rms pressures were measured over the lowest catenary vault; this result is similar, but more prominent, than that noted for the 0° orientation.

Figure C.18 presents the design plot, derived from numerical averaging, in a similar manner to that discussed in reference to Figure C.16. (Note that Figures C.16 and C.18 are plotted to different scales.)

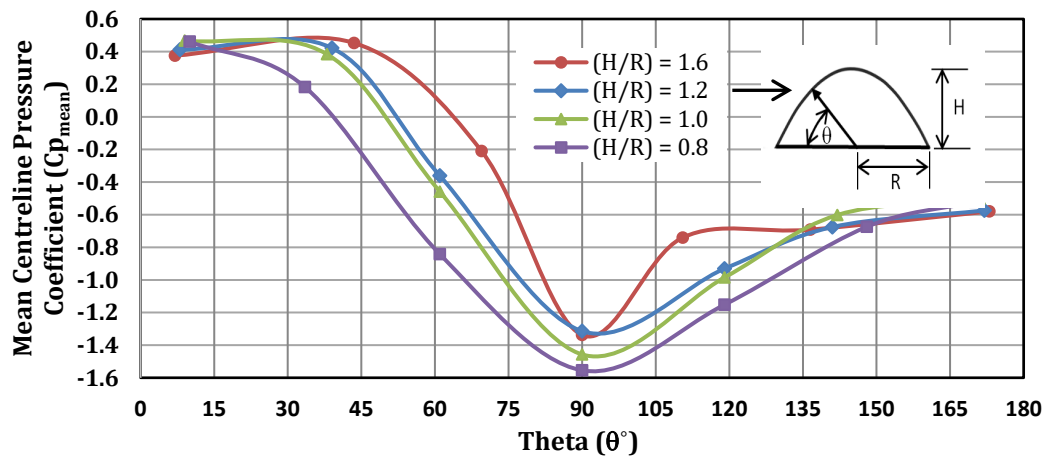


Figure C.17a – Mean centreline pressure coefficient distribution ($\alpha = 30^\circ/45^\circ$)

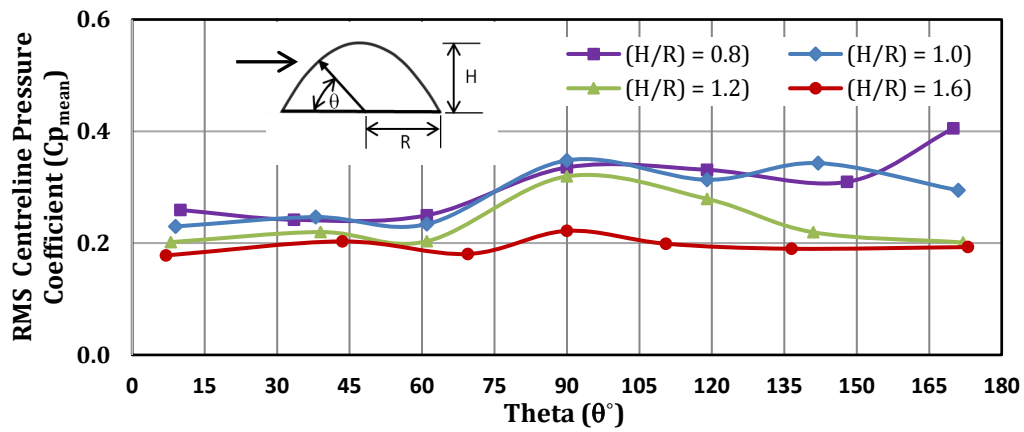


Figure C.17b – RMS centreline pressure coefficient distribution ($\alpha = 30^\circ$)

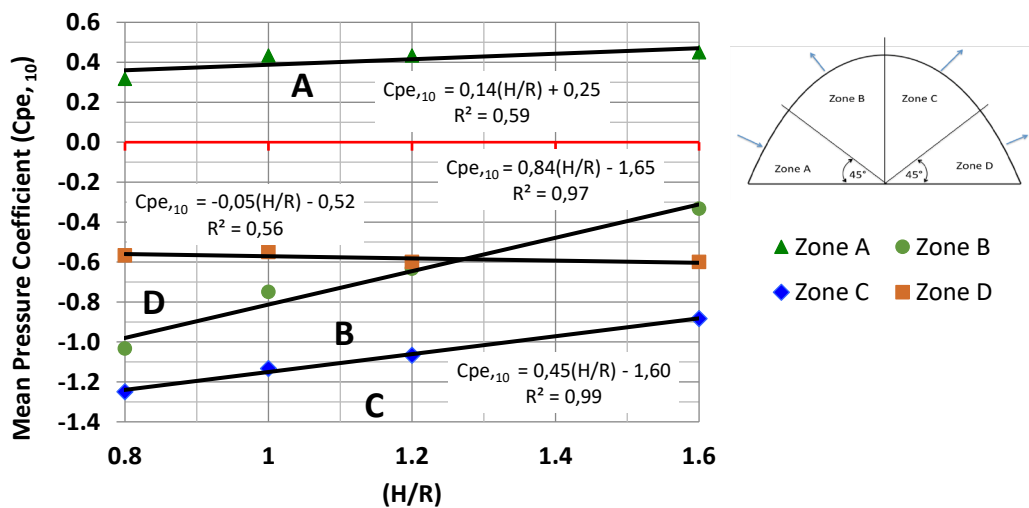


Figure C.18 – Design pressure coefficients ($\alpha = 30^\circ/45^\circ$)

C.4.2.3 Wind blowing onto a gable wall ($\alpha = 90^\circ$)

Typically, in loading manuals, design pressure coefficients are given for the two most critical wind orientations. These, in many cases, comprise either wind blowing normal or parallel to the eaves. For wind normal to eaves of a circular roof there is no information given in EN 1991-1-4 (and also SANS 10160-3). However, Blackmore and Tsokri (2006) conducted tests on curved roofs and compared their results with EN 1991-1-4 regarding duo-pitch roof data. Their findings generally revealed satisfactory agreement for design purposes. In the current study, a similar approach has been investigated and proposed (in regard of the extent of loading zones and magnitude of the design pressures).

The distribution of mean and rms pressure coefficients measured along the apex of the catenary vaults is shown in Figure C.19a and C.19b receptively. Flow separation for this wind orientation is controlled by the wall roof interface, thus variation observed between the roofs is likely due to differences in edge geometry. The contour plots of mean surface pressure coefficients over the catenary vaults investigated are shown in Figure C.19c. The corresponding distribution of design coefficients for this orientation is given in Figure C.20. Along the edge, largest suctions correspond to the lowest vault. However, suctions over zones D, C and E (as shown in Figure C.20) are largest over the steepest vault, and gradually reduce with height. (A similar trend is observed in data due to Blackmore and Tsokri (2006) for circular vaults).

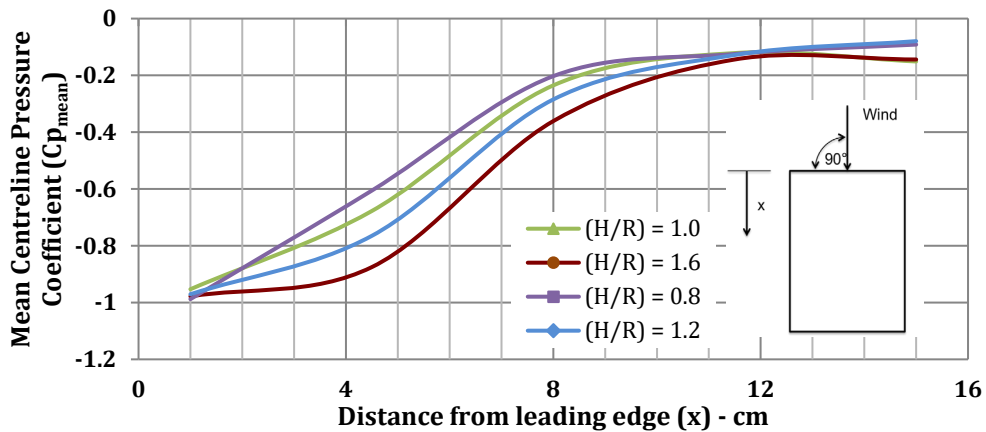


Figure C.19a – Mean apex pressure coefficient distribution ($\alpha = 90^\circ$)

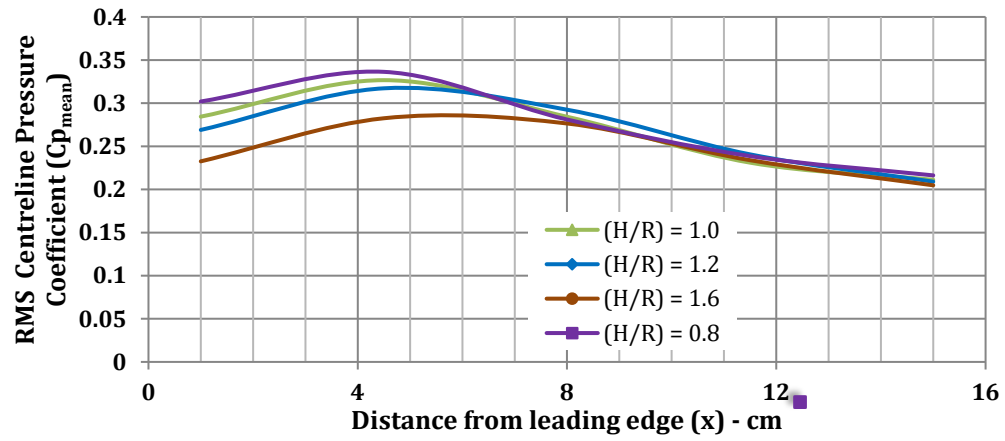


Figure C.19b – Rms apex pressure coefficient distribution ($\alpha = 90^\circ$)

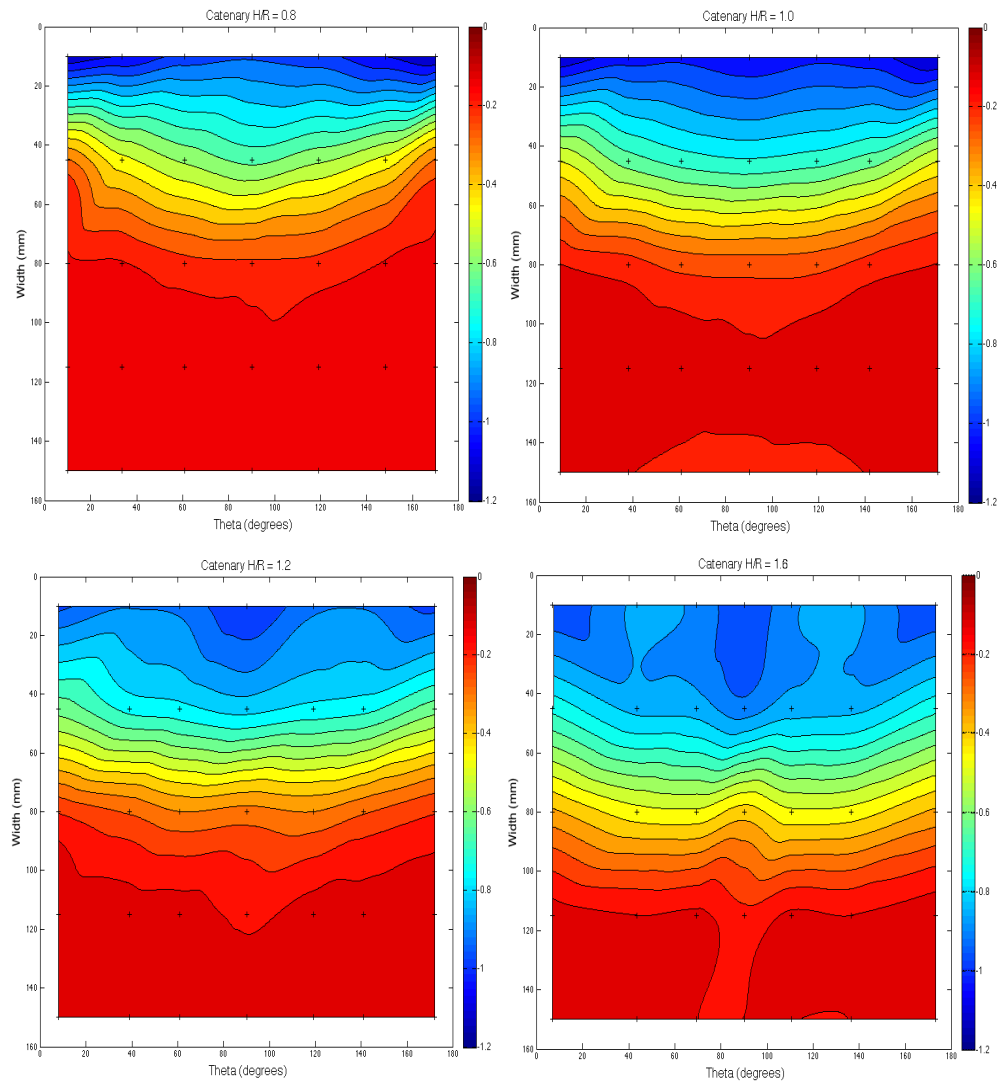


Figure C.19c – Contour plots of mean pressure coefficients ($\alpha = 90^\circ$)

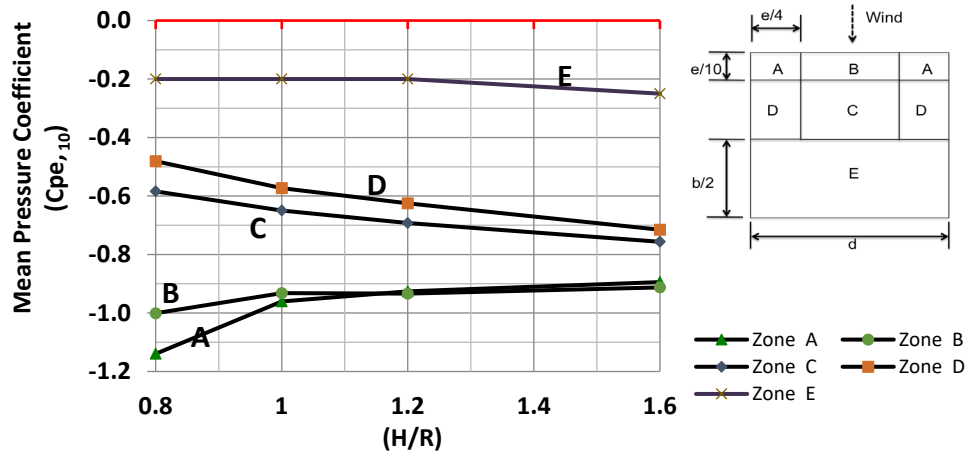


Figure C.20 – Design pressure coefficients ($\alpha = 90^\circ$)

Although, the overall design coefficients (corresponding to $C_{pe,10}$) are given for the catenary vaults, it should be noted that this is not a critical wind orientation for the profiles investigated. This is fundamentally due to the nature of the critical load patterns on these freestanding vaults (which is discussed later in this Appendix). Local coefficients (i.e., $C_{pe,1}$) may be significant for this orientation; however, this issue is not considered in this thesis.

C.5 Comparisons

C.5.1 Catenary vs. circular (current data)

The distributions of mean and root-mean-square centreline pressure coefficients corresponding to the catenary and semi-circular vault models (with the same height and base width; i.e., $H/R = 1.0$) are compared in Figures C.21a and C.21b. The mean pressure coefficients indicate a similar trend, with considerable difference in the magnitude of about 40% over the apex. The overall form compares well with the standard textbook distribution for a circular element. Furthermore, the rms pressure coefficients are largely similar.

The contour lines of the mean pressure coefficients (for $\alpha = 0^\circ$) corresponding to both geometries are also shown in Figures C.22a and C.22b. It can be noted that the semi-circular vault generates a larger magnitude of negative pressures and the extent of the negative pressure zone is also larger. This disparity is likely to be caused by a steeper (over most of the roof) slope corresponding to the catenary shape. In Figure C.23 a comparison of both geometries, in terms of an ‘inclination’ angle β , was made. The line corresponding to the semi-circle is straight (i.e., constant angle), whereas the line of the catenary is curved

and either above or below the 45° line. It can be seen that the semi-circle is steeper up to about $\theta = 30^\circ$, after which the catenary becomes steeper until the apex (i.e., $\theta = 90^\circ$). It can also be observed that the suctions are slightly larger beyond $\theta \approx 110^\circ$ over the catenary vault.

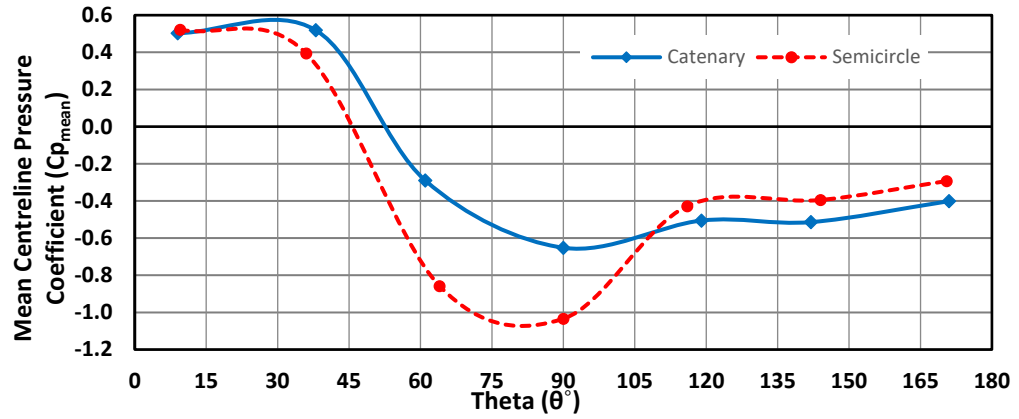


Figure C.21(a) – Mean centreline pressure coefficient distribution ($\alpha = 0^\circ$)

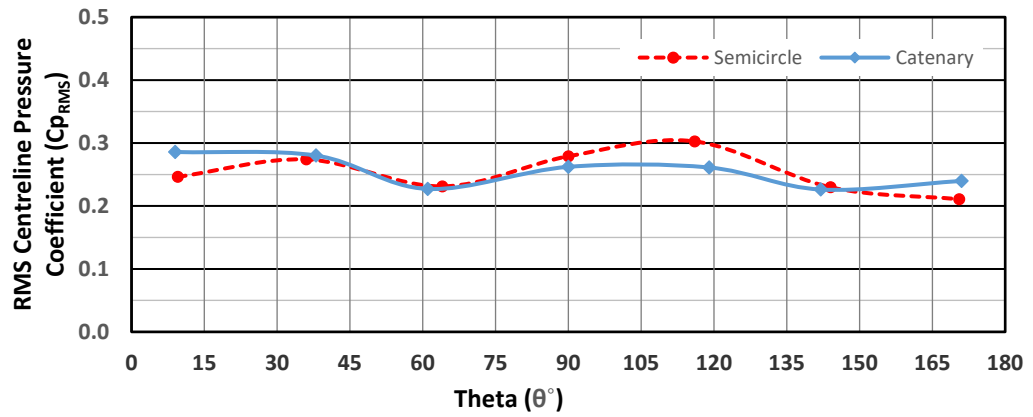


Figure C.21(b) – RMS centreline pressure coefficient distribution ($\alpha = 0^\circ$)

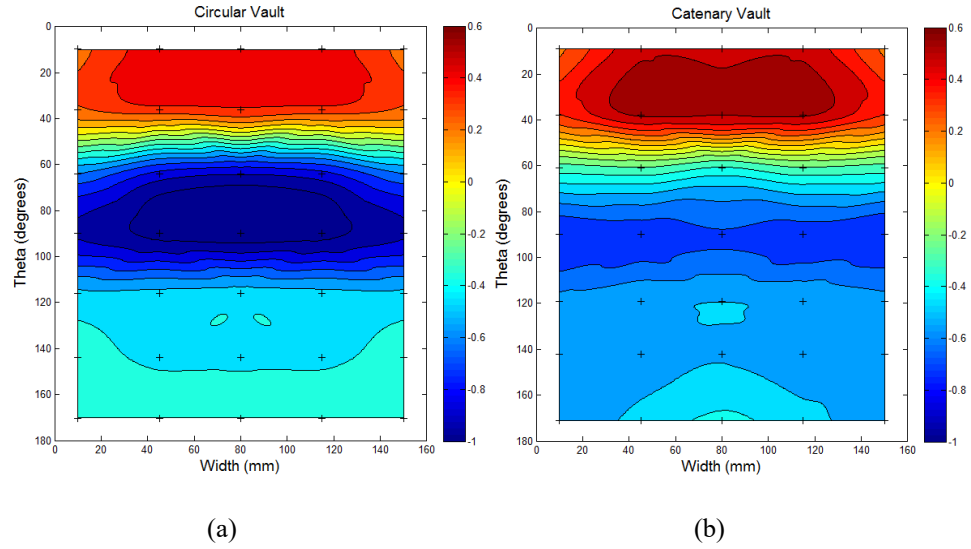


Figure C.22 – Mean pressure contour plots ($\alpha = 0^\circ$)

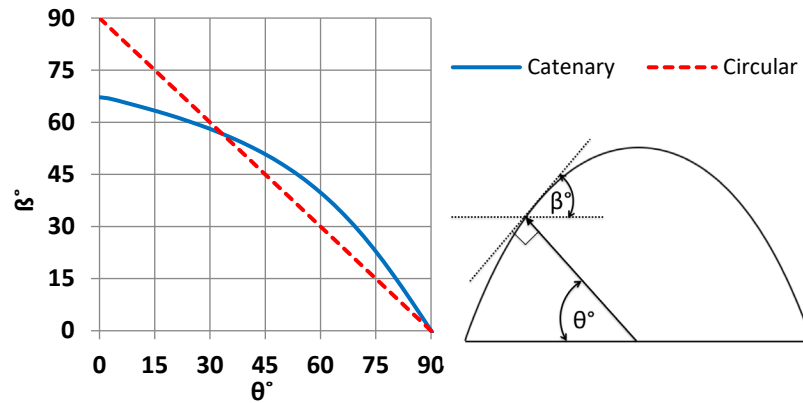


Figure C.23 – Variation of gradient (β) for the catenary and circular geometries

C.5.2 Blackmore and Tsokri (2006)

Figure C.24 evaluates the current results (obtained from numerical averaging for $H/R = 1.0$) with the corresponding data from SANS 10160-3 (which follows EN 1991-1-4) as well as the experimental data due to Blackmore and Tsokri (2006). It is evident that the current experimental data, as well as that due to Blackmore and Tsokri, for the semi-circular vault differs from that prescribed by SANS 10160-3 - particularly over the central portion of the roof (i.e., zone B in Figure C.24). The current data does, however, generally match that of Blackmore and Tsokri (2006) for a vault with a length to diameter ratio of 1.0 (i.e., small footprint aspect ratio). The vaults considered in the current investigation are relatively short, with $L/D = 1.3$. Furthermore, the pressure distributions shown in Figure C.22

emphasise the three-dimensional nature of the flow. The difference observed in Figure C.24 (over zone B) can be attributed to the disparity between two- and three-dimensional flow regimes over circular vaults. (These differences indicate that the values stipulated in the loading standard were based on studies conducted in two-dimensional flow conditions.)

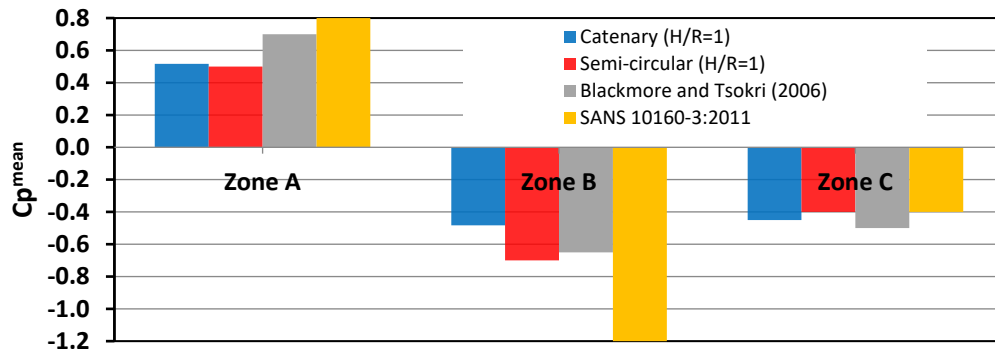


Figure C.24 – Pressure coefficients ($\alpha = 0^\circ$)

C.5.3 Toy and Tahouri (1988)

Toy and Tahouri (1998) investigated the influence of the model aspect ratio (in the horizontal plane) reflecting the effects of two - vs three - dimensionality of the flow. (Three situations were considered, namely: $L/H = 2$; $L/H = 4$ and a 2-dimensional situation; i.e., a model across the entire width of the test-section. Furthermore, differences in cross-sectional geometry of the vaults were also tested. Figure C.25 presents only two of these semi-circular cross-sectional geometries.

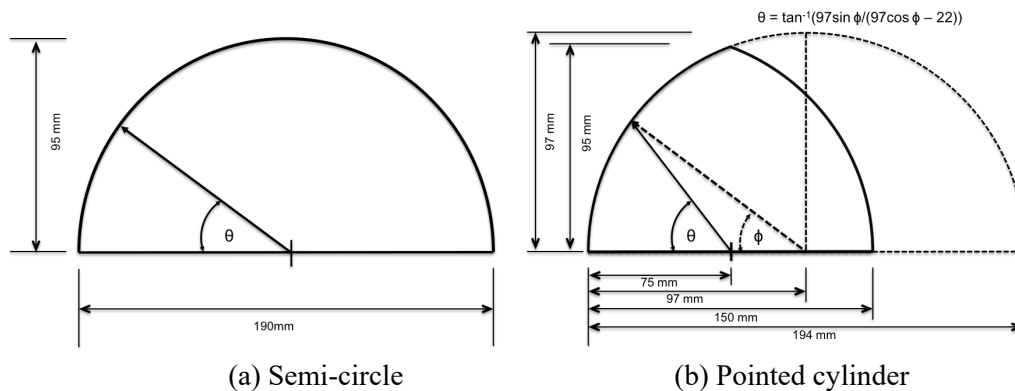


Figure C.25 – Two semi-cylindrical vaults investigated by Toy and Tahouri (1988)

Figure C.26 compares three semi-circular distributions (at different aspect ratios) and the current data. Data corresponding to Toy and Tahouri (for ratio $L/H = 4$) compares well

with the present data. A similar observation can be made to that noted by Blackmore and Tsokri, regarding the significant effect of two- vs three-dimensionality of the flow.

In Figure C.27, a comparison is also made between the ‘pointed’ cylinder (as illustrated in Figure C.25b) and the most similar geometry of catenary vault (i.e., $H/R = 1.2$). (To be noted, data published by Toy and Tahouri has been extracted and re-plotted in a different manner to suit the current comparison; i.e., a mathematical conversion was developed in order to express the vault geometry in terms of angle θ and not ϕ .) Interestingly, the distributions show some similarity, despite the fairly large difference in curvature between the pointed semi-cylinder and catenary. For the pointed cylinder, its sharp apex controls flow separation, but this is not the case for the catenary vault.

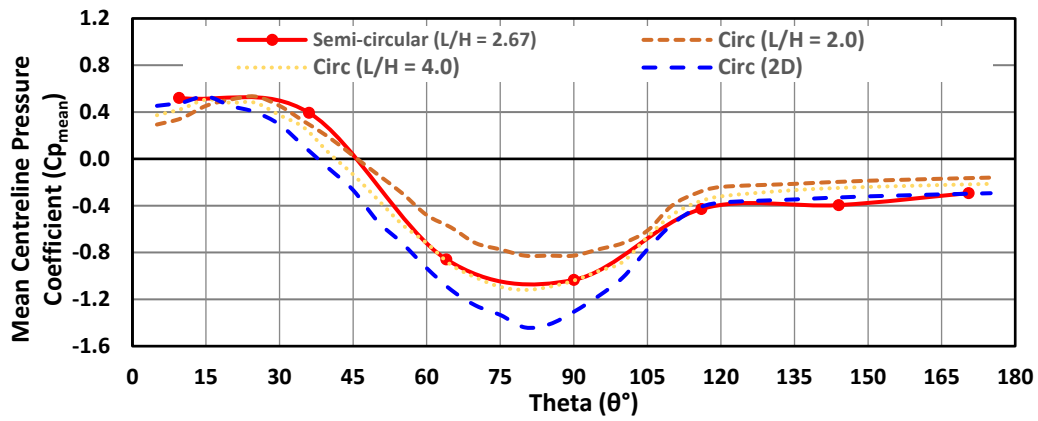


Figure C.26 – Mean pressure distribution (Semi-circular comparison)

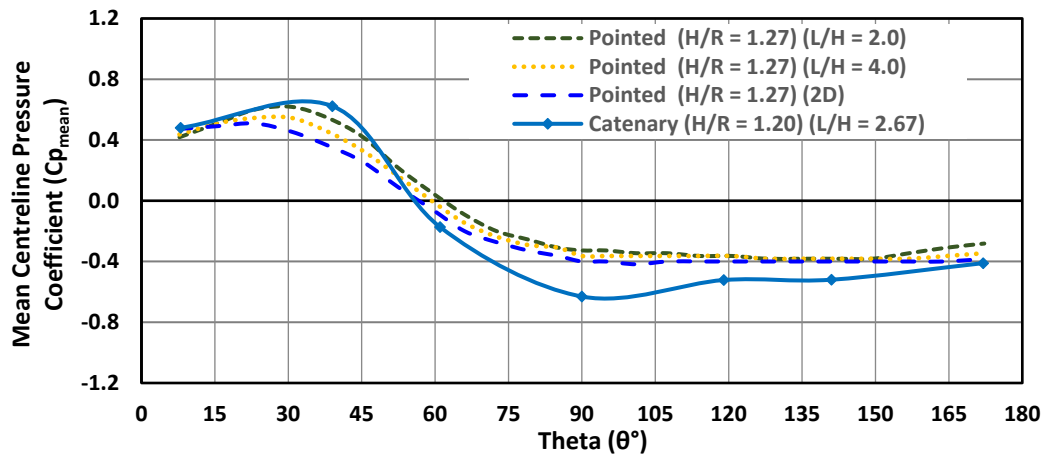


Figure C.27 – Mean pressure distribution (Catenary vs. pointed cylinder)

Also of interest to note is that Toy and Tahouri observed the largest negative pressures over the pointed cylinder at a wind orientation $\alpha = 45^\circ$. A similar trend was identified in the present study for all catenary vaults investigated (i.e. for $\alpha = 30^\circ / 45^\circ$).

C.6 Application

This section describes a series of finite element models (FE), which were implemented using the data developed in the current study as well as the corresponding information derived from SANS 10160-3, as loading inputs. This was done so to establish the applicability of the standardised loading data to the design of catenary vaults.

C.6.1 Finite element models

For ease of calculation, the catenary vaults were modelled as a 1-metre wide arch / strip. This simple analysis is considered adequate due to the nature of the loading, support conditions and the need for a critical expansion joint between the vault and sidewalls. An expansion joint eliminates the restraint associated with a wall-roof interconnection - effectively transforming the vault into a freestanding (and one-way spanning) structure. At the supports, shells are restrained always in translation, and sometimes in rotation. The analyses discussed in this appendix are primarily undertaken for comparative purposes; thus, for ease, translational restraint only is assumed for the FE models. It should, however, be noted that by adopting pinned end supports, the calculated bending moments and required uniform vault thickness will be overly conservative. It is more appropriate, when designing masonry vaults to use graphic statics in combination with ensuring that the thrust line remains within the middle third everywhere in the section.

The material and geometric properties of the masonry vaults are given in Tables C.2 and C.3 respectively. To be noted, although in many cases simple two-dimensional FE models are assumed, all stress and bending analyses reflect vaults having plan area of 48 m^2 (i.e., a width of 6m and length of 8m); however, the height and vault thickness are adjusted to accommodate the various comparisons considered in this appendix.

A standard clay brick and class II mortar (as specified in BS 5628) were assumed for these comparisons. Although masonry is orthotropic, with variable stiffness and strength, material isotropy was assumed due to the one-way span and load distribution. Comparison of the simple isotropic analysis with a full three-dimensional shell model (with orthotropic

material properties $E_x = 6200$ MPa and $E_y = 8800$ MPa) revealed only negligible differences in the stress distributions, as shown in Figure C.28. Note that the $H/R = 1.0$ catenary vault with constant radial thickness of 150 mm was considered in this comparison. Consequently, a two-dimensional analysis using bar-elements to create the curved catenary shape was employed in the current investigation. The use of linear bar-elements for a curved vault requires a discretised model to approach the exact solution. The number of bar-elements required for these analyses (i.e., the element density) was determined through convergence of the resulting bending moment diagram with that obtained from a closed-form solution (i.e., moment-area method). The surface pressures were applied to the models as a series of horizontal and vertical point loads. Full three-dimensional FE models are implemented, for several load scenarios, later in this appendix.

Table C.2 – Masonry properties

Brick Type	Mortar	Density (kg/m ³)	E_x (MPa)	ν	f_{xb} (MPa)	f_{xp} (MPa)
Standard clay brick	Class II (1:1:6)	1950	6200	0.2	0.5	1.5

f_{xb} – flexural strength parallel to bed joint, f_{xp} – flexural strength perpendicular to bed joint

Table C.3 – Geometric properties of full-scale catenary vaults

H/R	Apex height (m)
0.8	2.4
1.0	3.0
1.2	3.6
1.6	4.8

* all structures span 6m and have length 8 m.

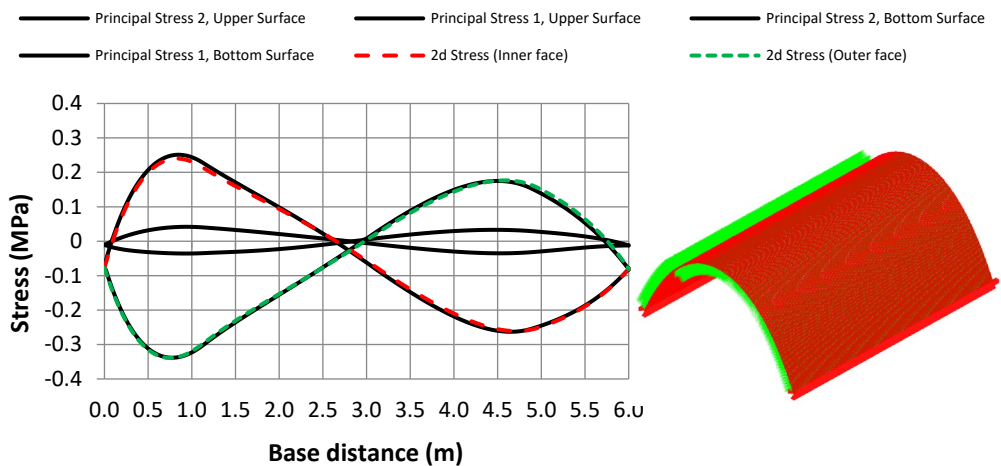


Figure C.28 – FE model and stress distributions (2d and 3d): $\alpha = 30^\circ$

Three load cases associated with the two critical wind orientations (0° and $30^\circ/45^\circ$) are summarised in Table C.4. These relate to wind blowing parallel to the gable walls; the 90° wind orientation is not critical for the vaults considered in these comparisons. For these load cases the internal pressure coefficients (C_{pi}) of -0.3, 0 and +0.2 were adopted. In principle, it has to be considered that, the full-scale gable walls will accommodate openings (i.e., doors and windows). This, in turn, implies a risk of generating the situation of *dominant* openings; i.e., higher magnitude of positive internal pressures ($\approx +0.7$), corresponding to the wind blowing onto these walls ($\alpha = 90^\circ$). However, this situation results in a symmetrical load distribution, which is not critical for the catenary vaults – confirmed by a three-dimensional FE model discussed later in this appendix.

Table C.4 – Load cases (load factors from SANS 10160 – 1)

Load Case	Variation 1	Variation 2	Variation 3
0.9 DL + 1.3 WL ($\alpha = 0^\circ$) 0.9 DL + 1.3 WL ($\alpha = 30^\circ/45^\circ$)	$C_{pi} = 0.0$	$C_{pi} = -0.3$	$C_{pi} = 0.2$

DL – dead load, WL– wind load

The wind pressures acting over the structures were determined using the coefficients derived in Figures 7.16 and 7.18 in combination with the stipulations of SANS 10160 – 3. The structures were assumed to be in Johannesburg; the design and site parameters are summarised in Table C.5.

Table C.5 – Design and site parameters (SANS 10160 – 3)

Fundamental Basic wind speed ($v_{b,0}$) =	28 m/s
Probability factor (C_{prob}) =	1
Topography factor ($c_o(z)$) =	1
Peak wind speed ($v_{b, peak}$) =	39.2 m/s
Air density (at 1700m) =	0.98 kg/m ³
Terrain category =	B

C.6.2 Finite element results

The bending moment diagrams for the 0° and 30° wind orientations (corresponding to load variation 2, as shown in Table C.4) are shown in Figures C.29 and C.30 respectively. As expected, the bending moments are largest for the ‘steepest’ vault (i.e., $H/R = 1.6$) and lowest for the shallowest vault (i.e., $H/R = 0.8$). This observation is attributed to the critical

nature of the pressure distribution over vaults with high aspect ratios (H/R). In particular, this is due to the development of larger positive pressure (applied over a larger area) on the windward face, in combination with negative pressure over the leeward half. However, it is also partially attributed to the large difference in total surface area between the various vaults (The total surface area for the ‘steepest’ vault is roughly 1.5 times larger than the ‘shallowest’). Interestingly, for H/R between 0.8 and 1.2 the maximum bending moments correspond to the load case with wind blowing at 30° . However, for the ‘steepest’ vault (i.e., $H/R=1.6$), maximum bending is observed for $\alpha = 0^\circ$.

The major principal stress results from a three-dimensional FE analysis for the $\alpha = 90^\circ$ wind orientation are shown in Figure C.31. (This plot corresponds to an arc-line that is 0.5 m from the leading edge – as illustrated by the dotted blue line in Figure C.32). The material properties for the 150 mm thick catenary vault ($H/R = 1.0$) are stipulated in Table C.2. An internal pressure coefficient of +0.7 was implemented in line with SANS 10160-3; and external pressure coefficients were obtained from the design plot shown in Figure C.20.

When comparing the principal stress distributions shown in Figures C.28 and C.31, for the 30° and 90° orientations, it can be seen that the maximum tensile stress corresponds to the oblique wind orientation. (The maximum tensile stress at $\alpha = 30^\circ$ is roughly twice as large as that for the 90° orientation). Furthermore, the highest magnitude stresses (due to the action of wind) are limited to a relatively small area, in close vicinity of the leading edge (where separation of flow takes place). These localised maximum tensile stresses are illustrated in Figure C.32. The black zone seen in the contour plot represents all stresses below a value of 0.08 MPa (tensile).

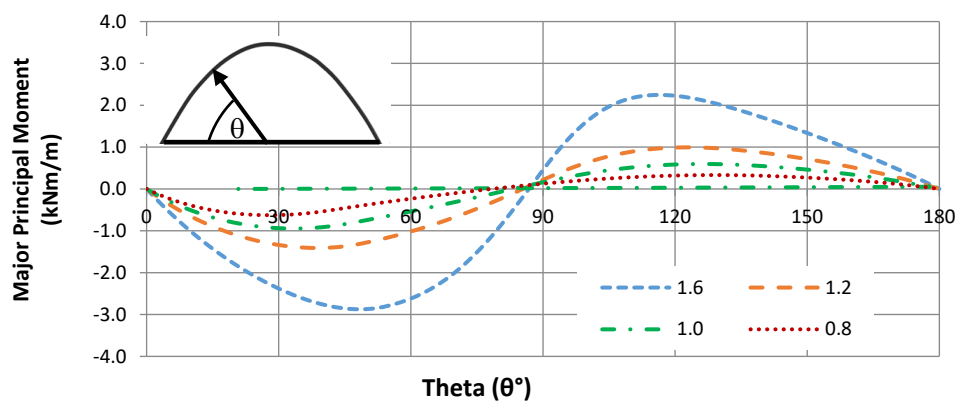


Figure C.29 – Bending moment diagrams ($\alpha = 0^\circ$)

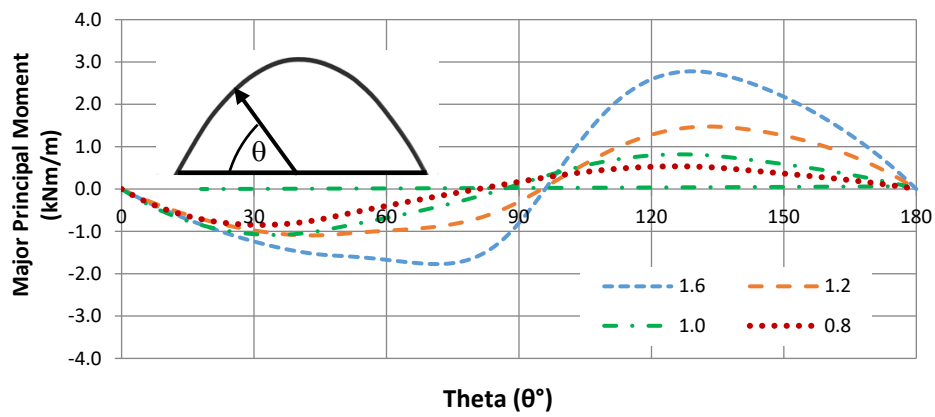


Figure C.30 – Bending moment diagrams ($\alpha = 30^\circ/45^\circ$)

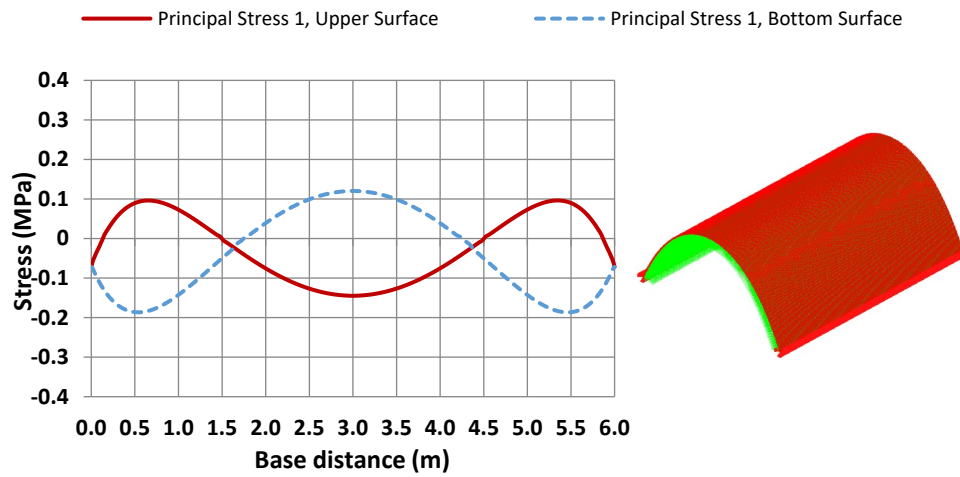


Figure C.31 – FE model and stress distributions: $\alpha = 90^\circ$

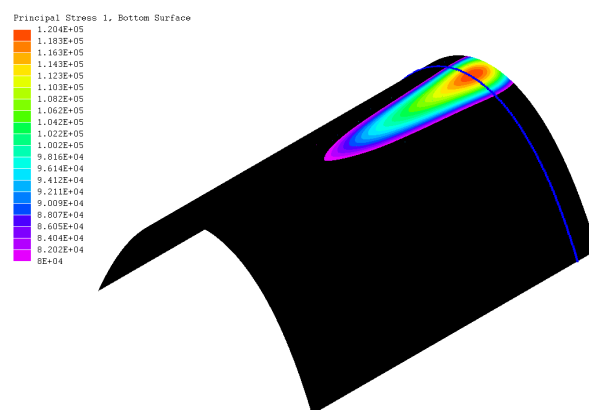


Figure C.32 – Principal tensile stress distribution (bottom surface) (3d): $\alpha = 90^\circ$

C.6.3 Comparisons of experimental data and SANS 10160-3

The above observations, in combination with the form of the pressure coefficient distributions, shown in Figures C.14a and C.17a, prompted a comparison of the experimental data with the design coefficients stipulated for traditional structural forms (i.e., circular and duo-pitch roofs) in SANS 10160-3.

C.6.3.1 Circular roofs

Pressure coefficients, from SANS 10160-3, corresponding to circular vaults are given in Table C.6. Figure C.33 shows a comparison between the experimental pressure coefficients for the catenary vault with $H/R = 0.8$ and the abovementioned data corresponding to a circular vault with $f/d = 0.4$. Both distributions are represented by four zones (i.e., A, B, C and D), as discussed previously. A good degree of similarity is observed between these pressure coefficients. Figure C.34 illustrates the bending moment diagrams obtained by applying both distributions to the catenary vault with $H/R = 0.8$. This figure demonstrates the general applicability of the codified data for circular roofs to shallow catenary vaults.

Table C.6 – Pressure coefficient data for circular roofs derived from SANS 10160-3

Zone	H/R = 0.8	H/R = 1.0	H/R = 1.2	H/R = 1.6
A	+0.65	+0.80	+0.80	+0.80
B	-1.10	-1.20	-1.20	-1.20
C	-1.10	-1.20	-1.20	-1.20
D	-0.40	-0.40	-0.40	-0.40

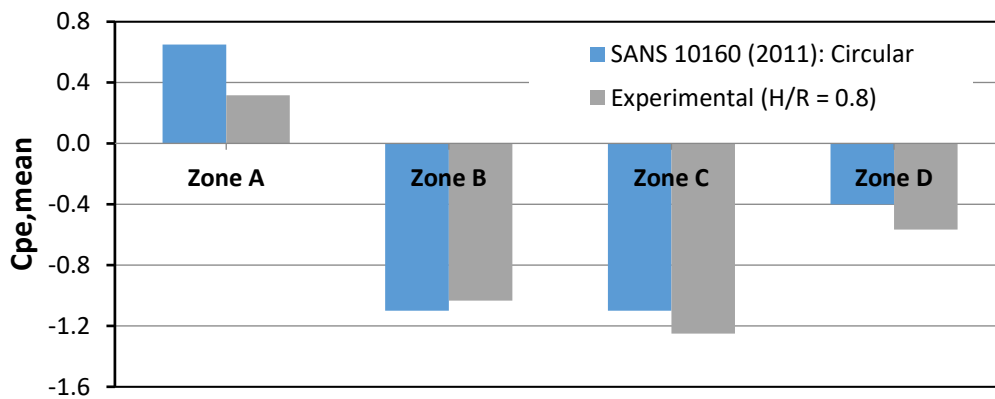


Figure C.33 – Design pressure coefficients: H/R = 0.8

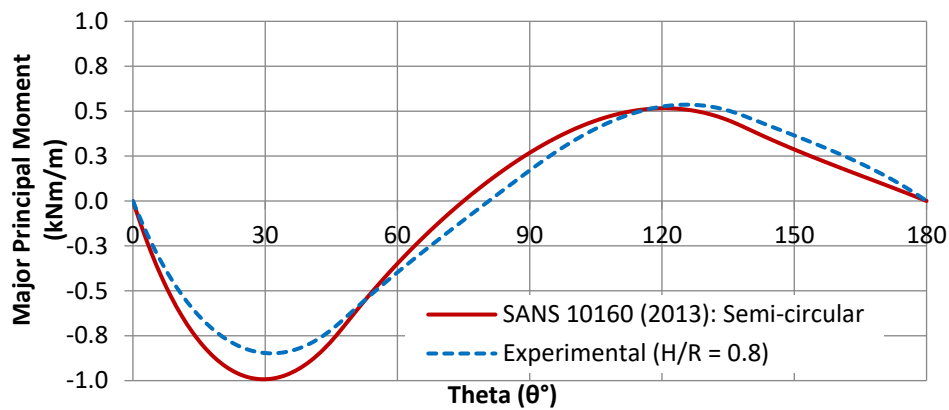


Figure C.34 – Bending moment diagrams: $(0.9D_n + 1.3W_n)$: $H/R = 0.8$

C.6.3.2 Duo-pitch roofs

Divergence of the bending moment diagrams occurs when the semi-circular vault design coefficients (from the loading code) are evaluated against the experimental data for the ‘steepest’ vault. Consequently, the applicability of data for duo-pitch roofs is investigated, as an alternative. The relevant pressure coefficients, derived from SANS 10160-3, and the distribution of related zones: F, G, H, J and I, are given in Table C.7 and Figure C.35 respectively. It should be noted that the suctions in zones F, G and J over duo-pitch roofs reflect the risk of vortices generated over the corners (i.e., wing tip vortices), and flow separation at the eaves and ridge (This, however, is not the case for the curved roofs investigated in this study, which do not contain raised corners, a sharp ridge or eaves). Consequently, the negative pressures stipulated for zones F and G in SANS 10160-3 are disregarded; and only positive pressures are considered over the windward areas. However, the suction over zone J is adopted.

Table C.7 – Selected duo-pitch design data taken from SANS 10160-3

Roof –Pitch (α)	Zone F	Zone G	Zone H	Zone J	Zone I
30°	+0.7	+0.7	+0.4	-0.5	-0.4
45°	+0.7	+0.7	+0.6	-0.3	-0.2
60°	+0.7	+0.7	+0.7	-0.3	-0.2

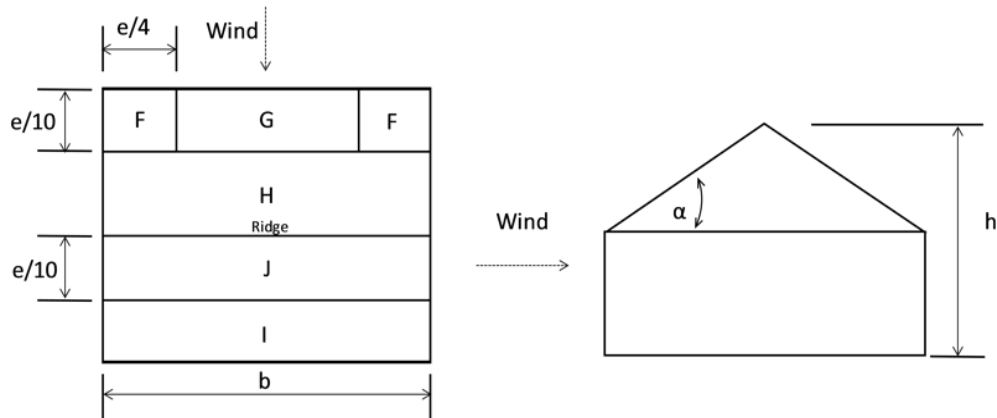


Figure C.35 – Wind pressure zones defined in SANS 10160-3 (2011): duo-pitch roof

The average ‘pitch’ angle of the curved roofs is determined by inscribing straight lines from its base to apex; i.e., $\psi = \tan^{-1}(H/R)$ – see illustration in Table C.8. The parameter ‘e’, which defines the extent of zones F, G and J, is also incorporated in Table C.8. Each catenary geometry gives rise to a different average roof slope (not necessarily the same as per loading code). Therefore, in order to accommodate a fair comparison of both datasets, linear interpolation between the 30°, 45° and 60° roof slopes were implemented. The adopted coefficients are included in Table C. 9.

Table C.8 – Roof-pitch and pressure zone geometry

H/R	h (m)	b (m)	e (m)	e/10 (m)	Roof-pitch (ψ)
0.8	2.4	8.0	4.8	0.48	≈ 40
1.0	3.0	8.0	6.0	0.60	45
1.2	3.6	8.0	7.2	0.72	≈ 50
1.6	4.8	8.0	8.0	0.80	≈ 60

e = minimum (2h or b)

Table C.9 – Relevant duo-pitch pressure coefficients for catenary vaults investigated

H/R	Roof-pitch (ψ)	Zone F	Zone G	Zone H	Zone J	Zone I
0.8	$\approx 40^\circ$	+0.70	+0.70	+0.53	-0.37	-0.27
1.0	45°	+0.70	+0.70	+0.60	-0.30	-0.20
1.2	$\approx 50^\circ$	+0.70	+0.70	+0.63	-0.30	-0.20
1.6	$\approx 60^\circ$	+0.70	+0.70	+0.70	-0.30	-0.20

In this section, only the pressure distributions corresponding to the ‘steepest’ catenary vault are compared. The pressure coefficients for both data sets are grouped into six zones to clearly illustrate the differences, as shown in Figure C.36. Some of the zones corresponding to the duo-pitch data have been sub-divided in half (i.e., zone H and I) to accommodate this

comparison. It can be seen in Figure C.36 that there is good agreement between both distributions for the windward zones 1 and 2. However, significant discrepancy between the coefficients over zone 3 is apparent. Also, the coefficients corresponding to the experimental data are more negative over the leeward surface. Although these coefficients show somewhat large variation on some parts of the roof, they ultimately result in a similar bending moment diagram as shown in Figure C.37. If Figures C.36 and C.37 are considered in combination, the effect of the lower pressure on the windward face (in the experimental data) is countered by larger suctions on the leeward face, leading to a similar net loading effect for both distributions.

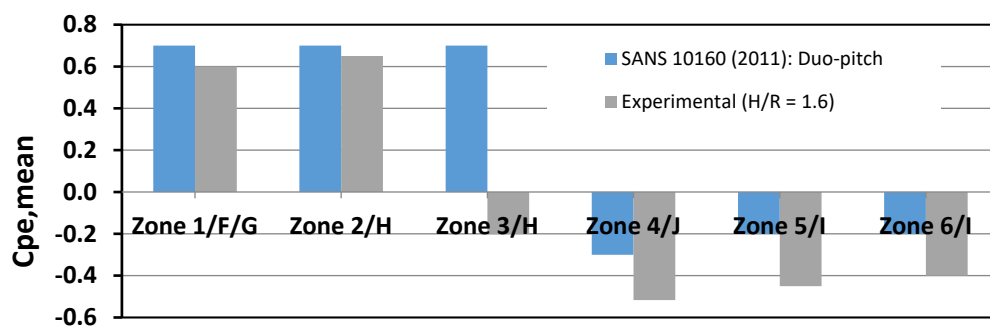


Figure C.36 – Design pressure coefficients: H/R = 1.6

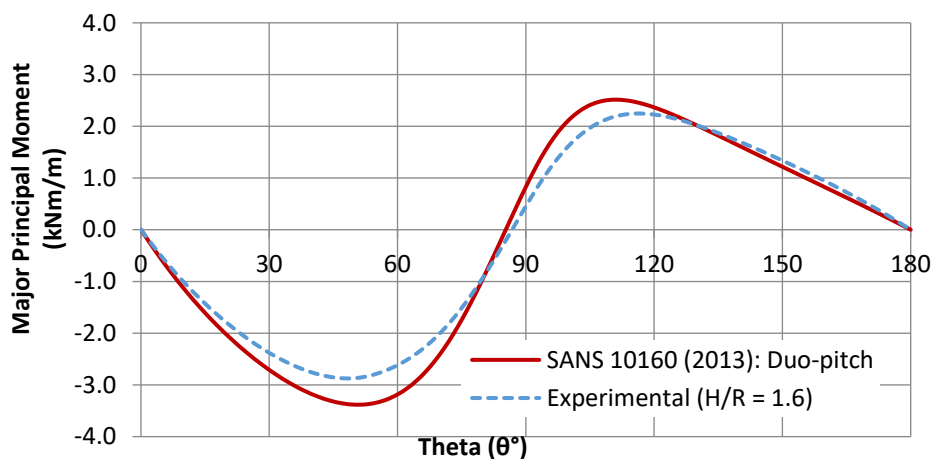


Figure C.37 – Bending moment diagrams ($0.9D_n + 1.3W_n$): H/R = 1.6

C.6.4 Design recommendations

In order to determine the point at which the circular design coefficients become inadequate, and the duo-pitch data should be adopted, a comparison of the minima required design vault thickness was carried out. An elastic design, adopting ‘the middle third’ technique

and pinned end connections, was selected as a simple methodology to facilitate the relevant comparisons. To be noted, graphic statics (and limit analyses) are preferred design methodologies, since elastic analyses require several assumptions about the material, which is anisotropic and imperfectly elastic. Furthermore, elastic methods may lead to overly conservative designs. However, a simplified elastic analysis is reasonable for the comparisons made in the present paper. The concept of “the middle third” is derived directly from bending theory as follows;

$$\sigma_{C/T} = \frac{-P}{A} \mp \frac{My}{I} = \frac{P}{bt} \mp \frac{6Pe}{bt^2} \quad (C.4)$$

$$\sigma_T = \frac{-P}{bt} + \frac{6Pe}{bt^2} = 0 \quad (C.5)$$

$$\text{Required thickness } [t] = 6e \quad (C.6)$$

The results of the stress design processes are shown in Figure 31. It can be seen that the use of the duo-pitch data typically leads to a conservative design. However, the minima shell thicknesses, derived from ‘duo-pitch’ and experimental coefficients, converge at an H/R value of approximately 2. However, the circular data is less conservative than the duo-pitch data up to an H/R value of roughly 1.2. The significance of these differences is clearly emphasised in the percentage error plot shown in Figure 32. Based on these comparisons it is proposed, for design purposes, that catenary vaults with an $H/R \leq 1.2$ could be designed using the coefficients given in SANS 10160-3 for circular vaults, whereas duo-pitch data may be adopted for profiles greater than $H/R = 1.2$. This finding demonstrates that no specific design data on wind loading of catenary vaults needs to be derived.

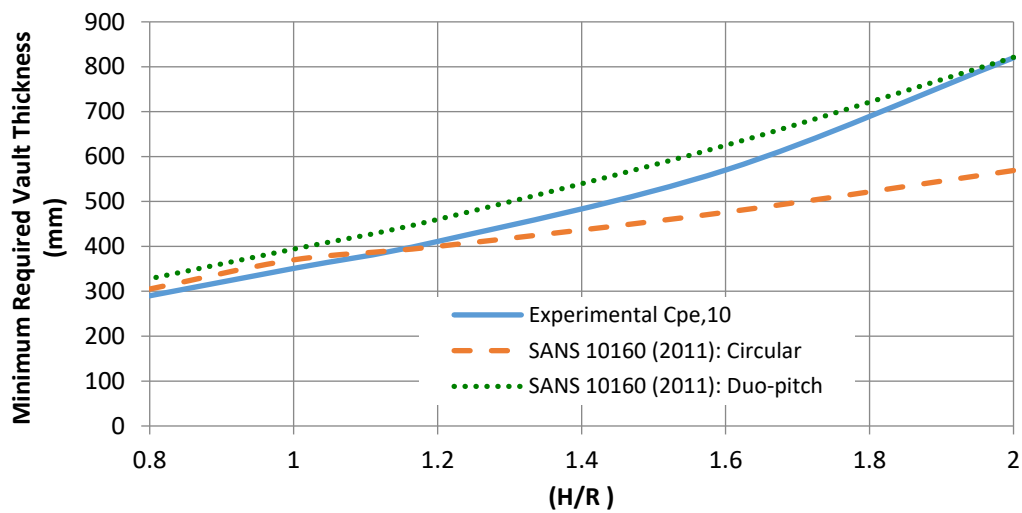


Figure C.38 – Required vault thickness (zero tensile stress)

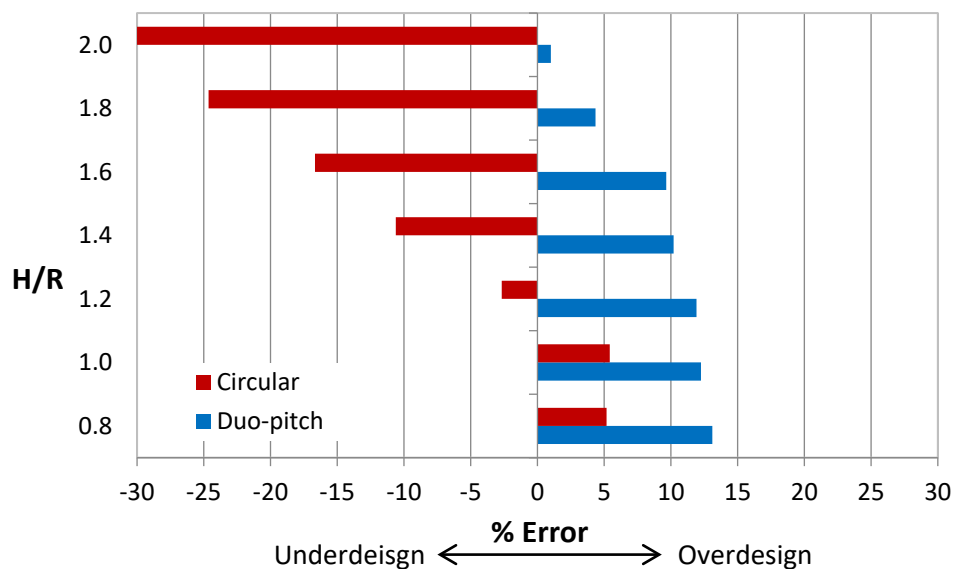


Figure C.39 – Percentage error when using codified data: zero tensile stress

C.7 Summary Findings

A series of wind-tunnel tests have been carried out to measure wind pressure coefficients on catenary vault structures. Four catenary vault roof geometries and a semi-circular shape were modelled and tested in a nominal boundary layer corresponding to an open terrain category. The primary purpose of these measurements was to assess applicability of the design data included in SANS 10160-3 and to establish the need for development of data related to catenary vaults. These design coefficient values were also formulated for the

purpose of the analyses and discussions presented in Chapter 5. The following supplementary conclusions can also be made.

- A set of design wind pressure coefficients, measured in three-dimensional flow conditions, was derived for catenary vaults with H/R between 0.8 and 1.6.
- The critical orientation for the shallow (i.e., $H/R \leq 1.2$) catenary vaults, corresponds to $\alpha = 30^\circ$, whereas for the steepest vault (i.e., $H/R = 1.6$) this corresponds to $\alpha = 0^\circ$.
- The distribution of pressure coefficients over the catenary vault with lowest H/R ratio shows the greatest degree of similarity with the circular form (as per SANS 10160-3). For the steepest vault, the experimental data exhibits the closest resemblance to the duo-pitch data from the abovementioned standard.
- It was established that an H/R ratio of 1.2 constitutes ‘the border case’ for the application of both approximations (i.e., for $H/R \leq 1.2$, the ‘circular data’ may be adopted, whereas for $H/R > 1.2$ the ‘duo-pitch data’ is more adequate).
- When adopting the ‘duo-pitch data’ (for $H/R > 1.2$), the average roof pitch (ψ) for the catenary profiles can be approximated as $\psi = \tan^{-1}(H/R)$.
- Using the pressure coefficient distributions for semi-circular vaults in two-dimensional flow (as per SANS 10160-3) is inadequate for the design steep catenary shells.

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