SHRINKAGE AND FLEXIBILITY BEHAVIOUR OF BITUMEN STABILISED MATERIALS

STELLENBOSCH UNIVERSITY
DEPARTMENT OF CIVIL ENGINEERING

Lizemari Campher – 15780406

Supervisor: Professor K.J. Jenkins

January 2015

FAKULTEIT INGENIEURSWESE
FACULTY OF ENGINEERING

The shrinkage behaviour as well as the flexible behaviour of BSMs (Bitumen Stabilised Materials) was investigated to obtain an improved understanding of the benefits and performance mechanisms of these materials. This knowledge can improve BSM mixture designs, which has an impact on in service pavement performance.

UNIVERSITEIT
STELLENBOSCH
UNIVERSITY

SHRINKAGE AND FLEXIBILITY BEHAVIOUR OF BITUMEN STABILISE MATERIALS

by

Lizemari Campher

Thesis presented in fulfilment of the requirements for the degree of Master of Science in the Faculty of Engineering at Stellenbosch University

Supervisor:

Professor Kim Jonathan Jenkins

SANRAL Chair in Pavement Engineering

Faculty of Engineering

Department of Civil Engineering

March 2015

Declaration

I declare that all research, tests and results obtained during this Thesis-project is my own, original work. In addition, references have been used to give recognition to all persons who's work have been sited during this project. I declare that the publication of this Thesis by the University of Stellenbosch will not infringe any third party rights and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

Name and surname: Lizemari Campher

US Student number: _____

Signature: _____

Date: 09 /01 /2015

Acknowledgements

To God almighty, for my health and knowing that I can always rely on Him for any help.

The University of Stellenbosch, for providing the opportunity to obtain my degree of Master of Science and providing the facilities needed to complete my project.

Professor Kim Jenkins, my promoter and supervisor for this project, for his confidence in me and his constant assistance and guidance during the entire duration of my project.

Gavin, for all the help and assistance with my laboratory work, as well as the workshop personnel for making the equipment needed for my laboratory work.

Lastly, my friends and family for their love, encouragement and constant support during times of stress.

Abstract

The increasing awareness of climate change causes a growing interest in pavement rehabilitation. Pavement rehabilitation by in-situ stabilisation with bitumen reduces the extraction of natural aggregate resources while enhancing flexibility and durability properties, which lowers maintenance costs over the design-life of the pavement structure. Incorporating Bitumen Stabilised Materials (BSMs) into a pavement structure can therefore have economic and environmental benefits, but more research is needed to fully understand the behaviour and potential of these materials.

Stabilising materials with bitumen provides useful properties to pavement layers. The "TG2 2nd Edition, Bitumen Stabilised Materials" was published by the Academy of South Africa in May 2009, which provides a good understanding of the usage of Bitumen Stabilised Materials (BSMs). However, the shrinkage and flexible behaviour of these materials are still not fully understood and therefore more research on these materials is needed.

The aim of this project is thus to determine the shrinkage and flexible behaviour of BSMs to incorporate these behavioural types in the revised design method for flexible pavements in the SAPDM. In addition, the influence of several additives on the shrinkage and flexible behaviour of BSMs have been evaluated to get an improved understanding of these properties. The additives included cement (1% and 2% content) and both bitumen emulsion (0.9% and 2.4% content) and foamed bitumen (only 2.4% content).

Two linear shrinkage testing methods have been designed to test the shrinkage potential of BSMs, including a beam testing method and a cylindrical testing method. Based on the usage of the shrinkage measurements the applicable method can be used to determine the shrinkage potential of a BSM. The flexibility is a more complex property and was tested using a simple monotonic beam test. The strain-at-break parameter obtained from this test provided an indication of the material flexibility.

Slight differences in the trends were observed between beam and cylindrical shrinkage due to specimen geometry, exposed surface area and shrinkage testing direction. Beam specimens initially show swelling when bitumen is added to the mixture and shrinks thereafter. Cylindrical specimens on the other hand show initial shrinkage followed by a slight length increase, where after shrinkage continues. The additives had the same influence on the shrinkage for both the beam and cylindrical specimens. Although all shrinkage measurements were small, an increase in bitumen reduced shrinkage and an increase in cement increased shrinkage. Stabilisation with

foamed bitumen rather than bitumen emulsion proved to show less shrinkage, but only in combination with 2% cement.

The strain-at-break, dissipated energy and material stiffness calculated from the monotonic beam tests provided a good indication of the flexibility behaviour of BSMs. Higher bitumen content increased the flexibility potential and an increase in cement decreased the flexibility potential of BSMs.

This project has provided good insight on both the shrinkage and flexibility behaviour of BSMs, which can be used in the revised copy of the SAPDM. Increased bitumen contents decreases the shrinkage potential and increases the flexibility of a BSM. Increased cement contents on the other hand, increases shrinkage and decreases flexibility of BSMs. The correct combination of cement and bitumen in a BSM can thus provide a material with the wanted flexibility while keeping the shrinkage to a minimum.

Opsomming

Die toenemende bewustheid van klimaatsverandering veroorsaak toenemende belangstelling in die rehabilitasie van plaveisels. Plaveisel rehabilitasie deur in-veld stabilisasie met bitumen verminder die ontginning van natuurlike hulpbronne, terwyl die verbetering van buigsaamheid en duursaamheid eienskappe die onderhoudskoste verlaag oor die ontwerp-lewe van die plaveiselstruktuur. Die inkorporasie van Bitumen Gestabiliseerde Materiale (BGM) in ‘n plaveisel struktuur kan dus omgewings en ekonomiese voordele inhou. Meer navorsing word wel benodig om die gedrag van hierdie materiale beter te verstaan.

Die stabilisering van materiale met bitumen verskaf nuttige eienskappe aan ‘n plaveisellaag. Die "TG2 2de Uitgawe, Bitumen Gestabiliseerde Materiale" is gepubliseer deur die Akademie van Suid-Afrika in Mei 2009 en verskaf ‘n goeie begrip van Bitumen Gestabiliseerde Materiale (BGM). Die krimpings en buigsaamheid gedrag van die materiaal word wel nog nie ten volle verstaan nie en daarom word meer navorsing oor hierdie materiaal benodig.

Die doel van hierdie projek is dus om die krimpings gedrag sowel as die buigsaamheid gedrag van ‘n BGM te bepaal en sodoende die kennis te gebruik in die hersiende ontwerp metode vir buigsame plaveisels in die SAPDM. Die invloed van verskeie bymiddels op die krimpings en buigsaamheid gedrag van ‘n BGM is ook geëvalueer om ‘n beter begrip van hierdie eienskappe te verkry. Die bymiddels sluit sement in (1% en 2% inhoud) asook beide emulsie bitumen (0,9% en 2,4% inhoud) en skuim bitumen (slegs 2.4% inhoud).

Twee lineêre krimpings toets metodes was ontwerp om die krimping potensiaal van BGM’s te bepaal, wat 'n balk toets metode en 'n silindriese toets metode insluit. Die metode wat gebruik sal word om die krimping van ‘n BGM te bepaal moet baseer word op die toepassing waarvoor die krimpings resultate gebruik gaan word. Die buigsaamheid is 'n meer komplekse eienskap en was getoets met behulp van 'n eenvoudige monotoniese balk toets. Die spanning-by-breekpunt waardes wat verkry was vanuit die balktoetse, het ‘n goeie aanduiding van die buigsaamheid van die materiaal verskaf.

Klein verskille in krimpingstendense tussen balk en silindriese proefstukke is opgemerk tydens die projek en is veroorsaak deur die geometrie van die proefstuk, die blotgestelde oppervlakte asook die rigting van kimp toetsing. Balk proefstukke toon aanvanklike swelling wanneer bitumen bygevoeg is, gevolg deur krimping. Silindriese proefstukke aan die ander kant toon aanvanklike krimping gevolg deur 'n effense toename in lengte, waarna krimping weer plaasvind. Die bymiddles het dieselfde invloed op die krimping van beide die balk en silindriese proefstukke. Alhoewel al die krimpingswaardes baie klein was, het ‘n toename in bitumen ‘n

vermindering in krimping voortgebring en 'n toename in sement het ‘n toename in krimping voortgebring. Stabilisasie met skuim bitumen in plaas van emulsie bitumen toon verlaagde krimping, maar slegs in kombinasie met 2% sement.

Die spanning-by-breekpunt, verkose energie en materiaal styfheid wat bereken is vanaf die monotoniese balk toets resultate, het 'n goeie aanduiding van die buigsaamheid gedrag van BGM’s verskaf. 'n Hoër bitumen inhoud verhoog die buigsaamheid potensiaal van BGM’s terwyl 'n toename in sement die buigsaamheid potensiaal van BGM’s verlaag.

Hierdie projek bied goeie insigte vir beide die krimpings en buigsaamheid gedrag van BGM’s, wat in die hersiende ontwerp metode van die SAPDM gebruik kan word. Verhoogde bitumen inhoud verminder die krimping potensiaal en verhoog die buigsaamheid van 'n BGM. Verhoogde sement inhoud aan die ander kant, verhoog krimping en verminder buigsaamheid van BGM’s. Die korrekte kombinasie van sement en bitumen in 'n BGM kan dus 'n materiaal produseer met die gewenste buigsaamheidseienskappe en terselfde tyd die krimping tot ‘n minimum beperk.

Table of Contents

List of Figures

List of Tables:

Abbreviations

AASHTO	– American Association of State Highway and Transportation Officials
BSM	– Bitumen Stabilised Materials
CSIR	– Council for Scientific and Industrial Research
$E_{modulus}$	– Elastic modulus
LVDT	– Linear variable displacement transformer
MDD	– Maximum Dry Density
MOD	– Material Optimum Density
Mod. AASHTO	– Modified AASHTO compaction
MPa	– MegaPascals
M_r	– Resilient Modulus
OMC	– Optimum Moisture Content
SAPDM	– South African Pavement Design Method
TG2	– Technical Guideline second edition: Bitumen stabilised materials

CHAPTER 1 - Introduction

1.1. Problem statement

Stabilising materials with bitumen provides useful properties to the base layer of pavement structures, causing an increasing interest in BSM's. The "TG2 2nd Edition, Bitumen Stabilised Materials" was published by the Academy of South Africa in May 2009, to give a better understanding of Bitumen Stabilised Materials (BSMs). However, the shrinkage and flexible behaviour of these materials are still not fully understood.

1.2. Motivation for research

The earth's natural resources are limited and should not be wasted. The increasing awareness of climate change causes an unquestionable interest in pavement rehabilitation by in-situ stabilisation with bitumen. BSMs have been used in South Africa for more than 30 years and is used to an increasing extend in the construction and rehabilitation of flexible pavements. Not only does pavement rehabilitation by stabilisation provide environmental benefits, but economic benefits as well.

The materials used for BSMs are either fresh aggregate materials or materials recovered from an existing pavement. Seals or asphalt surfacing from existing pavements are recycled, mixed with the underlying layer and treated to form the new base or sub-base layer. This will help minimize the consumption of fresh aggregate resources.

The environmental impact caused by fresh aggregate extraction and greenhouse emissions from the heating process of the bitumen, can be reduced by minimizing aggregate extraction and using aged binders more sufficiently. Bitumen stabilising agents will enhance the performance of recycled materials, providing flexibility and durability, which lowers maintenance and rehabilitation costs over the design-life of the pavement.

Using Bitumen Stabilised Materials (BSMs) in a pavement structure will thus have economic and environmental benefits, but more research is needed regarding the behaviour and potential of these materials. A better understanding of these materials will increase their potential usage.

1.3. Research aim

The shrinkage behaviour as well as the flexible behaviour of BSMs has to be determined to obtain an improved understanding of the benefits and performance mechanisms of these materials. This knowledge will improve BSM mixture designs, which has an impact on in service pavement performance. In this manner, these behavioural types of BSMs could then be incorporated in the revised design method for flexible pavements in the SAPDM (South African Pavement Design Method).

1.4. Research objectives

1.4.1. Shrinkage

Shrinkage of pavement materials can cause stresses to develop in a pavement layer, which can damage the pavement layer by cracking. These cracks will reduce the water susceptibility, which leads to a shortened pavement life.

One objective of this study is to design practical shrinkage methods to adequately determine the linear shrinkage potential of a material used in a pavement structure. Shrinkage of BSM mixtures can be determined using one of these designed methods.

In addition, the influence of several variables on the shrinkage behaviour of a BSM will be evaluated to get an improved understanding of this property. These variables are:

1.4.1.a) The influence on the shrinkage of a BSM mixture using different binder types, i.e. emulsion and foamed bitumen, will be evaluated for each designed shrinkage method.

1.4.1.b) The influence of different amounts of bitumen and cement contents on the shrinkage of BSM mixtures will be tested and evaluated for each method.

Comparisons between the different designed shrinkage methods will be made.

1.4.2. Flexibility

Flexibility is a complex characteristic to define. Flexibility can be defined as a material's ability to withstand bending (flexure) with only minimal damage, which can take place in the form of fatigue. One of the objectives of this study is to investigate the potential of a simple monotonic beam test, to reliably characterise flexibility. From the beam test a strain-at-break parameter can be measured that provides an indication of the material flexibility.

The influence of different variables on the strain-at-break value of BSMs will be evaluated to get a better understanding of this property. These variables are:

1.4.2.a) The influence on the strain-at-break of BSMs using different binder types, i.e. emulsion and foamed bitumen will be determined.

1.4.2.b) The influence of different bitumen and cement contents on the strain-at-break value of BSMs will be determined.

1.5. Limitations of research

Complete routine tests for this project were done on a bulk material sample recovered from the R35 experimental road section in Johannesburg in December of 2011. Multiple different types of laboratory tests were also conducted using these materials in many other projects, causing a shortage of R35-materials for the intended tests of this project. Due to this shortage of material, a limit had to be put on the amount of specimens tested for each specimen type. It should also be noted that only Bitumen Stabilised R35-materials were tested in this project (not cement stabilised materials), but with different variables to better understand the properties of this one material when stabilised with bitumen.

The addition of binder was limited to amounts as defined in the broader research programme, where emulsion mixtures are limited to 0.9% and 2.4% bitumen emulsion and limited to 2.4% for the foamed bitumen mixtures. Additionally, cement content added to all mixtures where limited to 1% and 2%.

All shrinkage tests were conducted at a temperature of 40°C during this project and all strain-at-break tests were conducted at ambient temperature (25°C).

Research about shrinkage and strain-at-break of BSMs are limited and therefore comparisons between results from this project and previous findings will be challenging.

1.6. Report layout

Key elements regarding shrinkage and strain-at-break behaviour of BSMs have been summarized in the Literature Review (Chapter 2).

Major findings from the Literature Review is summarised in Chapter 3 and a hypotheses on the expected outcome of this project is formed. Chapter 3 also contains the material properties of the materials used during this project.

The methodology of the testing procedure that will be followed to test these materials is described in Chapter 4. The laboratory testing program is thoroughly described in Chapter 5.

Results obtained from the laboratory shrinkage tests (Chapter 6) and laboratory strain-at-break tests (Chapter 7) were processed and presented in the simplest manner. Thereafter the results were interpreted and findings are discussed (Chapter 8 Chapter 9). A conclusion is made and recommendations for further research are presented (Chapter 10).

Lastly, recognition is given to all authors whose work has been sited during this project.

CHAPTER 2 - Literature review

2.1. Pavements

Pavement structures generally consist of several layers of materials (as illustrated in Figure 2.1), with each material layer having different strength and stiffness characteristics. The purpose of each pavement layer is to spread the load it receives (at the top) to a wider area, at the bottom of the layer. Since the layers in the upper part of the layer are subjected to higher stress levels than the lower layers (Figure 2.1), the top layers need to be constructed from stronger and stiffer materials. Road pavements are made up out of the surfacing layer, structural layers and the subgrade (Figure 2.1). Each layer varies in composition, thickness and function.

There are two fundamental types of pavements, being either rigid pavements or flexible pavements (Wirtgen GmbH, 2012, p. 16).

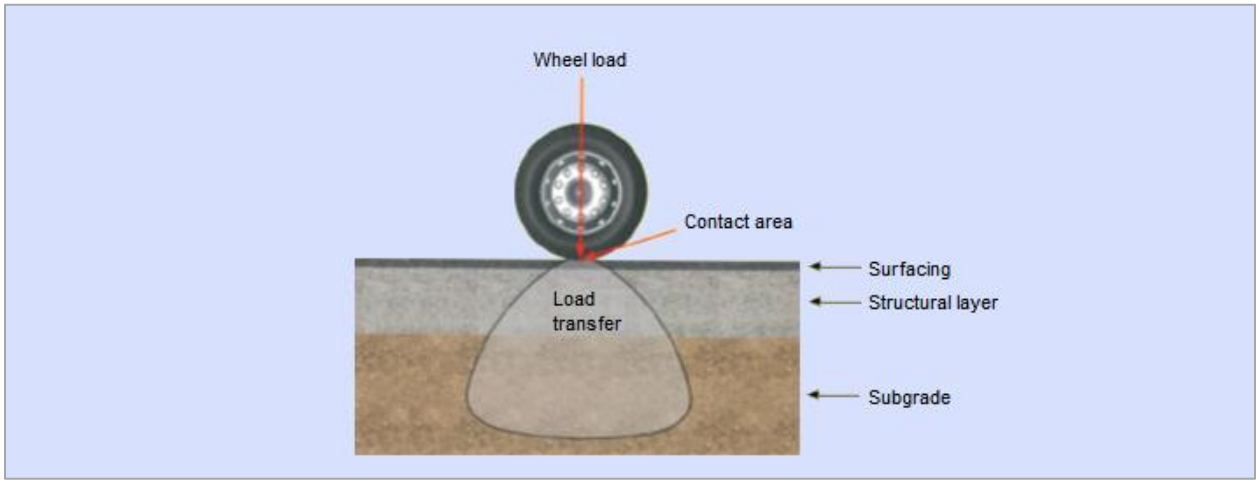


Figure 2.1 Load transfer through the pavement structure

2.1.1. Rigid pavements

A rigid pavement has a thick layer of high strength concrete overlaying a bound layer, which is usually demolished at the end of its pavement live (Wirtgen GmbH, 2012).

2.1.2. Flexible pavements

Flexible pavements are characterized by bituminous surfacing (Ebels, 2008) and the upper layers of the pavement are sometimes bound to achieve higher strength requirements. In contrast to rigid pavements, flexible pavements can economically be recycled in situ at the end of its pavement life (Wirtgen GmbH, 2012, p. 16).

Flexible pavements are constructed from three types of materials:

- **Unbound materials** (granular materials), including crushed stone and gravel, transfer applied loads through the individual particles that forms the matrix or skeleton (Figure 2.2) of the unbound materials. (Wirtgen GmbH, 2012, p. 18).

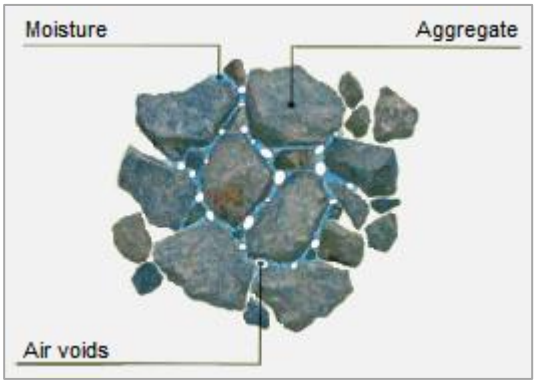


Figure 2.2 Skeleton of unbound (granular) materials

- **Bound materials**, including cement stabilised materials and asphalt, acts like a wide beam because of the bound nature of the skeleton structure (Figure 2.3). If a vertical load is applied to the surface of the bound layer (or beam), horizontal compressive stresses is generated within the upper half of the layer and horizontal tensile stresses generated in the lower half. These horizontal tensile strains will developed at the bottom of the layer due to the maximum horizontal stresses at the bottom of the layer, which ultimately leads to a fatigue type of failure after multiple load repetitions. Cracks will form at the bottom of the layer which will propagate upwards as more load repetitions is applied.

- **Non-continuously bound** materials (including BSMs), behave similar to granular materials. The bitumen in these materials is non-continuously dispersed (as illustrated in Figure 2.4) and therefore fatigue is not a design consideration. In BSMs, permanent deformation is the main mode of distress.

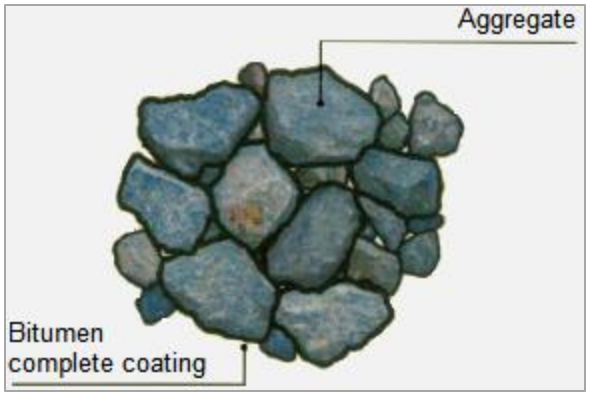


Figure 2.3 Bound material (Hot mix asphalt) (Wirtgen GmbH, 2012)

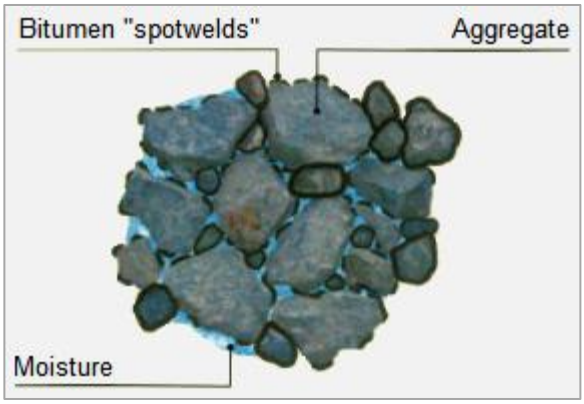


Figure 2.4 Non-continuously bound materials (BSM)

A large variety of materials can thus be used for flexible pavements, where BSMs are used often. BSMs can be used for either base or sub base layers. The two main components of BSMs are granular and bituminous materials, with bituminous materials being either bitumen emulsion or foamed bitumen (Ebels, 2008, p. 45).

2.2. Stresses in a pavement structure

Effective stresses are present in granular structural layers which are caused by residual compaction and overburden stress, internal suction stress and external stresses (Figure 2.5). A change in any of these stresses will have an effect on the effective stress in a material.

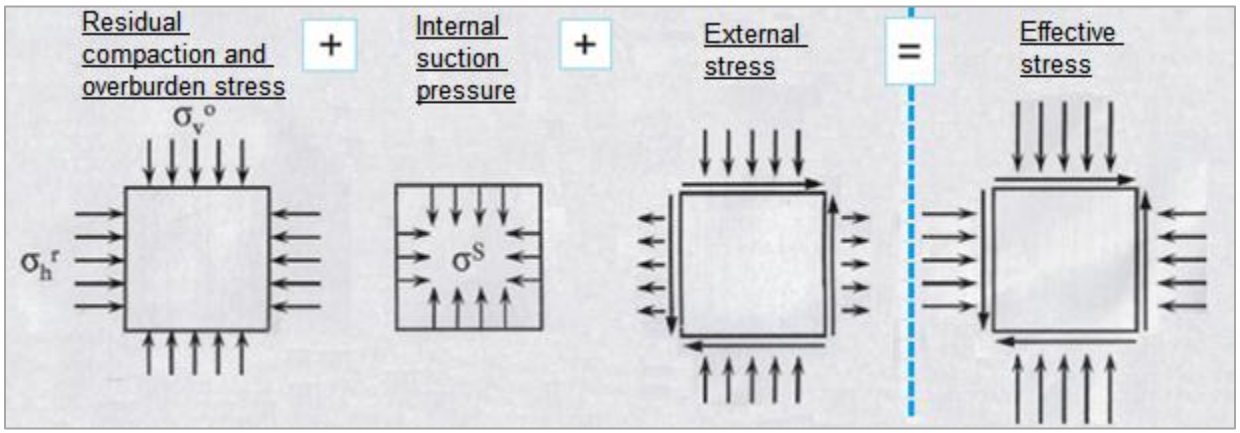


Figure 2.5 Components of effective stress inside a granular structural layer (Theyse, 2010).

2.2.1. Residual compaction

Soil compaction is a critical component of road construction (2008, p. 1). By applying a moving load on a soil (Lowery, 2008) the soil particles are pressed together, air is ejected from the soil (2008, p. 1) and the pore space between the particles are reduced, thus increasing the bulk density (Lowery, 2008, p. 3). Therefore, compaction is the process of increasing the density of a soil while improving particle contact and reducing voids between particles (Wirtgen GmbH, 2012). Maximum density should always be aimed for in compaction, since it improves the response of all pavement materials.

Compaction is achieved by applying heavy loads to a granular layer, using heavy compaction equipment. Pavement layers will thus experience larger stresses during the initial stages (construction phase) than the rest of its lifetime. The residual internal stresses induced during compaction will be locked into the granular layer after compaction is done (after all loads have been removed). Confinement and aggregate interlock will both form part of these residual stresses. During laboratory compaction, compaction moulds will provide the confinement. Laboratory tests show that an increase in grading modulus will increase the residual compaction stress (Figure 2.6).

The residual stresses developed during compaction will decrease during unloading. If the stresses reach equilibrium, the horizontal stresses will reduce and the vertical stresses will reduce to the overburden stress.

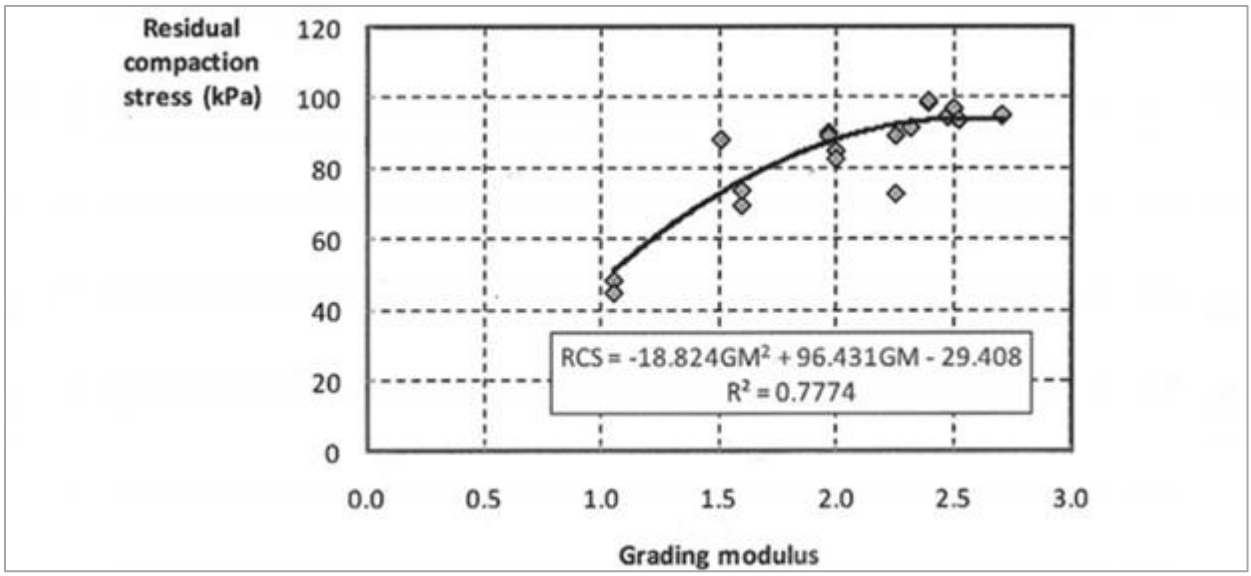


Figure 2.6 Effect of Grading Modulus on residual compaction stress (Theyse, 2010).

2.2.2. Internal suction

Porous materials have the ability to attract and retain water (Fattah, 2013, p. 231). The force exerted by the material to take up water is known as soil suction or negative pore water pressure. In saturated unbound materials the suction will contribute to the stiffness and strength of materials.

Total suction is the amount of energy required to remove a water molecule from a soil matrix through evaporation (Sweere, 1990). Total suction is the combined effect of matric suction and osmotic suction.

$$\Psi = (u_a - u_w) + \pi \qquad \text{[Equation 1]}$$

Where: $(u_a - u_w)$ = Matric suction

$\boldsymbol{\pi}$ = Osmotic suction

These two soil suction types are independent and have no effect on each other. A change in either or both of these components will change the soil suction (Fattah, 2013, p. 232).

- **Matric suction:** Matric suction ($u_a - u_w$) is the measure of energy (or vapour pressure of the soil moisture/relative humidity) required to remove a water molecule from a soil matrix without the water changing state (Sweere, 1990). In unbound materials, matric suction is generated by surface tension.

  The surface of liquids possesses a property known as surface tension, which allows a tensile pull to exist on the liquid surface. This tensile pull is the result of intermolecular forces at the air-liquid interface between liquid and particle. A liquid exerts equal forces in all directions on a molecule (Figure 2.7.a), causing the molecule to experience a resultant force towards the interior of the liquid. In order to keep all forces in equilibrium, a tensile pull is generated along the surface of the liquid (Figure 2.7.b) (Sweere, 1990). The surface tension will cause meniscus to form at the soil-air interface which will result in a reduction in vapour pressure in the pore-water. As the saturation decreases, so does the vapour pressure. The radius of curvature (Figure 2.7.c) of the meniscus

decreases causing matric suction (vapour pressure in the soil) to increase (University of Tolendo).

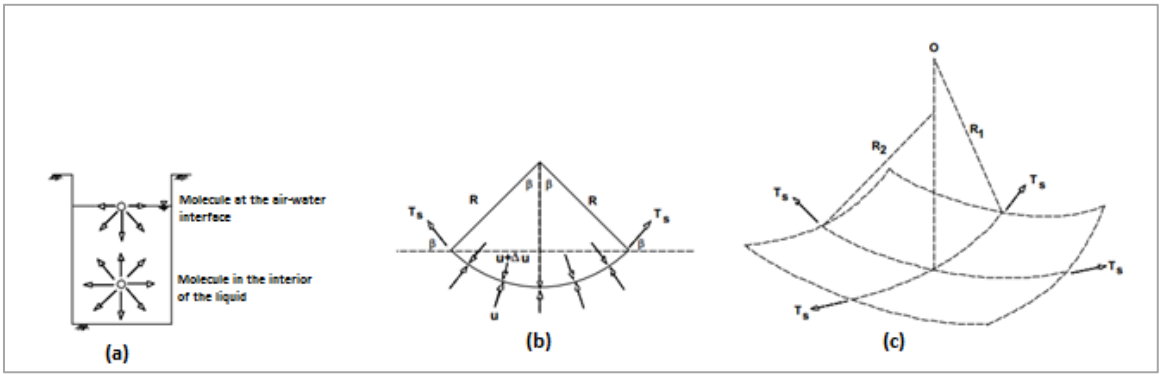


Figure 2.7 Surface tension: a) Intermolecular forces acting in a liquid; b) & c) Surface tension acting on a membrane surface (University of Tolendo).

An example of the existence of surface tension causing matric suction is the capillary effects in unbound materials (Sweere, 1990, p. 54). Water is retained in the soils through capillary forces arising from the curved air-water interfaces, in the voids of un-saturated soils.

- **Osmotic suction:** The energy needed to remove molecules from the water phase is known as the osmotic potential (Sweere, 1990). Osmotic suction (difference between total suction and metric suction) will arise from the presence of dissolved ions (salts) in the pore-water (University of Tolendo). The ion concentration in the water will reduce the tendency of water evaporation (Sweere, 1990) and decrease the soil vapour pressure (University of Tolendo). A decrease in soil vapour pressure will increase the total suction, which is an increase in the energy required to remove a water molecule from water phase in a soil (Sweere, 1990).

A change in soil suction is mostly accompanied by considerable volume changes (Murillo, Caicedo, Hoyos, Colmenares, & Berdugo, 2013). These changes will occur due to a change in moisture content (and is also dependent on the soil characteristics). In the case where clay is present in a soil, high levels of suction may be reached, but material behaviour is significantly affected by suction even if no clay is present in the soil (Sweere, 1990). These volume changes,

either swelling or shrinkage, will each cause a change in stresses in different ways and is discussed below.

2.2.2.a) Swelling

During wetting of expansive soil, the clay minerals absorb water molecules and expand (J. Rogers).

During particle hydration the water is “forced” into the interparticle and interlayer voids, causing an increase in pore water pressure, a decrease in effective stress and volumetric expansion (De Wet, 2014). A volume increase will thus be accompanied by a decrease in effective stress.

2.2.2.b) Shrinkage

Drying of soil shows a decrease in volume (shrinkage) and an increase in effective stress (De Wet, 2014). As these soils dry out they shrink and could leave large voids in the soil (J. Rogers), as shown by the cracks (voids) in Figure 2.8.

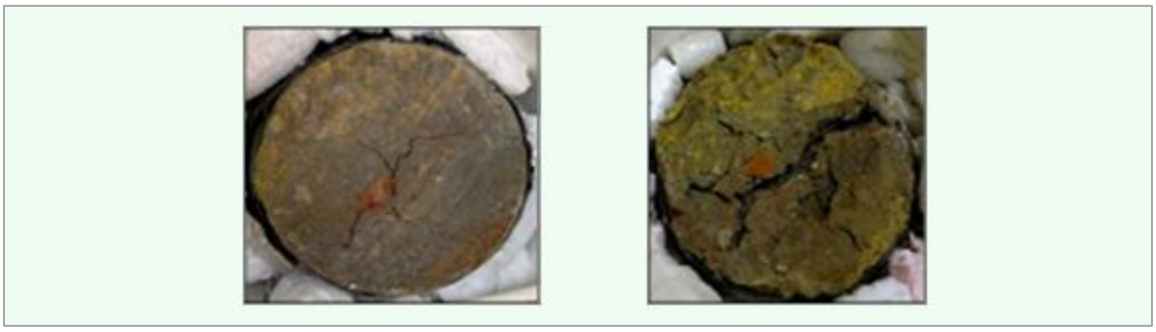


Figure 2.8 Development of shrinkage cracks in High PI clay during laboratory tests (Amer, Mantri, & Chen).

During drying of materials, water leaves the pores of fine-grained clayey soils and develops menisci at the pore entrances causing a decrease in pore water pressure. The total stress in the soil will not be affected which means the effective stress will increase. With an increase in effective stress, the volume will decrease as well as the void spaces.

The decreasing void spaces will force water to the pore entrances, keeping menisci intact and the process of shrinkage continues.

As more and more water is lost, air starts to penetrate the pore spaces, driving the menisci to smaller pore entrances which results in smaller menisci with further decrease in pore water pressure to negative values, increase in effective stress and volume change (De Wet, 2014).

The process cannot proceed indefinitely, because the resistance to compression of the soil increases with decreasing volume, so that an equilibrium condition is reached and maintained between the moisture content (amount of water lost) and the volume (void ratio).

Finally the shrinkage limit of the soil is reached and no further volume change is possible. Air enters the voids and water is concentrated only in the smaller pores (De Wet, 2014).

2.2.3. External stresses

Roads are designed and constructed to withstand loads imposed by the traffic for that specific road over the expected lifetime (Wirtgen GmbH, 2012, p. 24).

The load imposed on the road surface by the traffic is dependent on the load carried by the tyre (or axle load), the tyre pressure (kPa) which will determine the contact area of the load to the surface, as well as the vehicle speed which provides the load frequency (Wirtgen GmbH, 2012). The tyre-pavement contact stress is applied to a small area (area of tyre footprint) and then distributed over an area that increases with an increase in pavement depth (SANRAL, 2013). Applying these traffic loads to a homogeneous material will produce normal and shear stresses at any element in the material. Figure 2.9 depicts the stress distribution in a multi-layered pavement structure when loaded by a dual-wheel, half-axle load (The Constructor, 2012). These stresses depend on the magnitude of the applied wheel load, radius of contact area and the distance to the centre of the load.

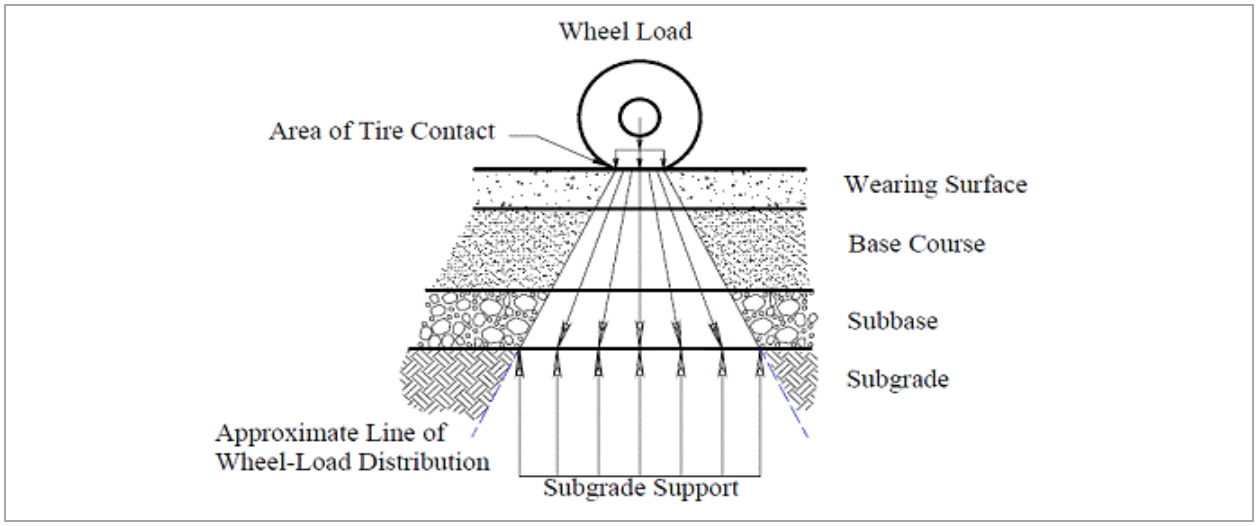


Figure 2.9 Load distribution in a flexible pavement structure (The Constructor, 2012).

Considering the pavement layer as a beam, applying a load will cause bending, forming tensile stresses to develop at the bottom part of the beam (layer) and compression stresses at the upper part. These stresses (σ) will act perpendicular to the plane, but stresses can also act along a plane, known as shear stresses (τ). The occurring stresses can be calculated using Boussinesq's theory and can be represented with the use of a Mohr circle. A change in shape or volume may occur in response to these stresses, called strain (National Giographic).

2.3. Material models

2.3.1. Material behaviour models

A stress applied to a material (or material layer) will allow a change in size, shape or volume of the material, known as deformation (Nelson, 2012). This deformation will either be permanent or temporary, determined by the behaviour of the material, being elastic, viscous or plastic behaviour.

Elastic behaviour

In a material showing elastic behaviour, the strain in the material caused by a load will only be temporary and will thus return to its original shape and size when the stress is removed (Kailas). The elastic behaviour of a material will thus resemble that of a spring model. Plotting the stress

versus strain of this material behaviour will yield a straight line (Hooke's law). The deformation of an elastic material is given by Equation 2.

Stress strain is linear, thus: $\sigma(t) = E.\varepsilon(t)$

$$\varepsilon_s = \frac{\sigma_s}{E} \qquad \text{[Equation 2]}$$

Where: ε_s = Elastic deformation

σ_s= Stress applied to a material (spring)

E = Elastic Modulus of the material (spring) (Kelly, 2010)

Plastic behaviour

A material showing plastic behaviour will accumulate non-recoverable deformation (plastic strain – $\mathcal{E}_p$) when a load is applied to the material. Some materials will only show plastic behaviour after a certain amount of stresses (loads) have been applied (for example elasto-plastic behaviour) (Jenkins, 2012).

Viscous behaviour

If a viscous material is subjected to stresses, the material will start to flow and continue to do so with constant stress (Jenkins, 2012). Deformation of a viscous material will occur slowly over time and the material will not recover to its original position, causing the deformation to be permanent (Kelly, 2010). The linear viscous behaviour can be modelled by a dash-pot and the deformation caused by a viscous material is given by Equation 3.

The shear stress is given by: $\tau = \eta \frac{d\gamma}{dt}$

Using integration, the strain is:

$$\varepsilon(t) = \left(\frac{\sigma_0}{\eta}\right) t + c \qquad \text{[Equation 3]}$$

Viscous materials can either be Newtonian (having a constant η) or non-Newtonian (where the η varies with stress levels) (Jenkins, 2012).

Visco-elastic behaviour

Some materials do not behave purely elastic or purely viscous but rather a combination of the two known as visco-elastic behaviour, such as BSMs (Jenkins, 2012).

The response of a visco-elastic material due to applied stresses and strains can be better understood by visco-elastic models helping to visualise the combination of the viscous and elastic behaviours (spring and dash-pot behaviours) in different combinations. Figure 2.10 provides three of the models used to predict the behaviour of visco-elastic materials, where the Burger Model is the best representation of the three. Figure 2.11 illustrates the behaviour of a visco-elastic material when subjected to constant stress.

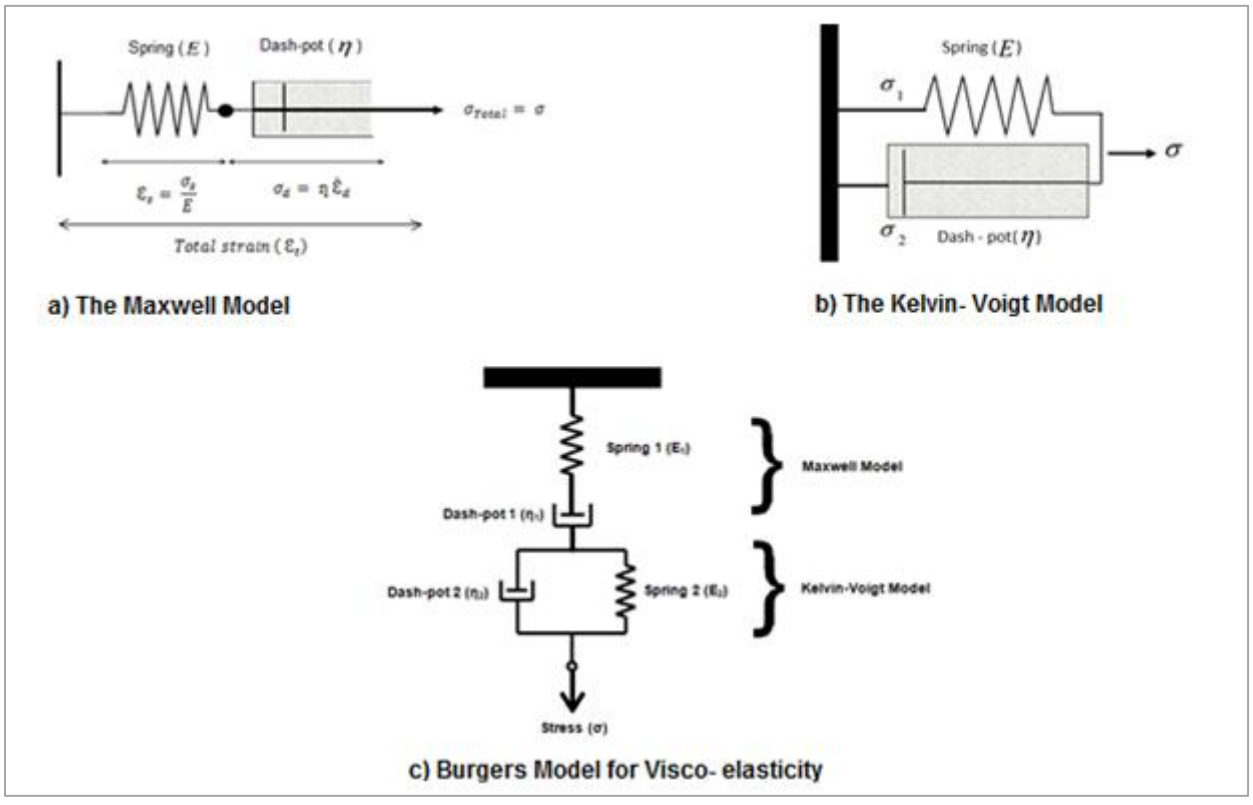


Figure 2.10 Visco-elastic material models – a) Maxwell; b) Kelvin Voigt and c) Burger model. (Kelly, 2010).

A visco-elastic material (or Burger model) will react immediately after a load is applied to the material and deformation (elastic deformation) equal to the spring deformation of the Maxwell model will occur. Creep deformation will occur with time due to the viscous deformation of the viscous property, or dash-pot deformation over time caused by the Kelvin-Voigt Model and dash-pot which is part of the Maxwell Model. When the applied stresses are removed, the elastic deformation (spring deformation of the Maxwell Model) will recover immediately. The deformation caused by the Kelvin-Voigt Model will recover with time, since the spring element will cause the displacement of the dash-pot to be recovered over time. Deformation from the dash-pot part of the Maxwell model will not recover and is thus the permanent deformation experienced by a visco-elastic material.

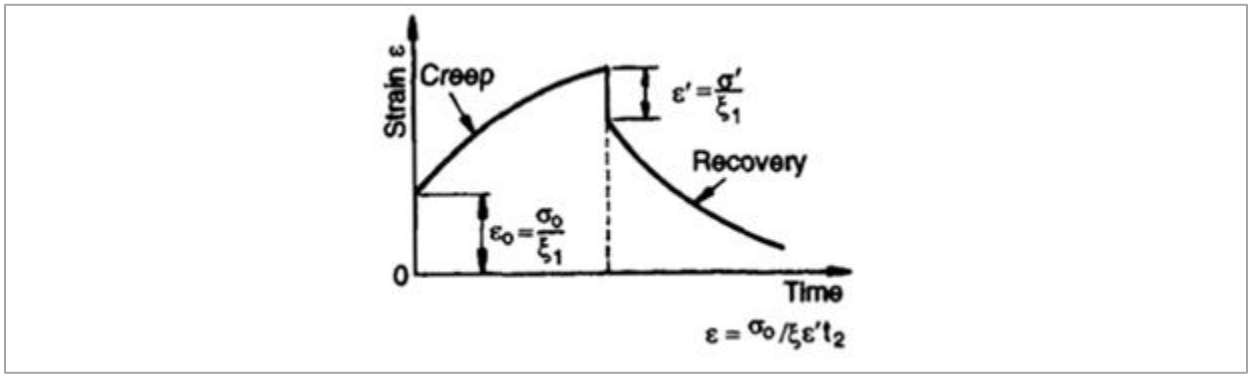


Figure 2.11 Behaviour of the Burger Model when subjected to constant stress over a time (Crawford, 1998).

2.3.2. Material fracture models

If a material is subjected to greater stresses than it can withstand, it will fracture by either breaking (behaving brittle) or bending (behaving ductile) (Selverstone), which depends on the plastic behaviour (deformation) of the material.

Ductile behaviour

Ductile deformation is a permanent deformation caused by flowing or bending of the material where the material gradually changes shape or volume (National Giographic). Materials with a ductile behaviour will undergo a small region of elastic behaviour when low levels of stress is applied (Nelson, 2012), followed by a large region of ductile behaviour (ductile flow) when the elastic properties cease (Kailas), as indicated in Figure 2.12. The ductile behaviour is

accompanied by significant plastic deformation (National Giographic) prior to and during crack propagation (Kailas), causing fracture of the material and providing warning signs before failure (Nelson, 2012). A visco-elastic material (such as a BSM) will show behaviour closer to ductile behaviour (Wirtgen GmbH, 2012).

Brittle behaviour

Brittle deformation is a permanent deformation in a material, in which the material will break or crack (National Giographic). Brittle materials will show little or no elastic behaviour (Figure 2.12) followed by a small region of ductile behaviour (showing plastic deformation) before fracturing by breaking into pieces without warning (Selverstone).

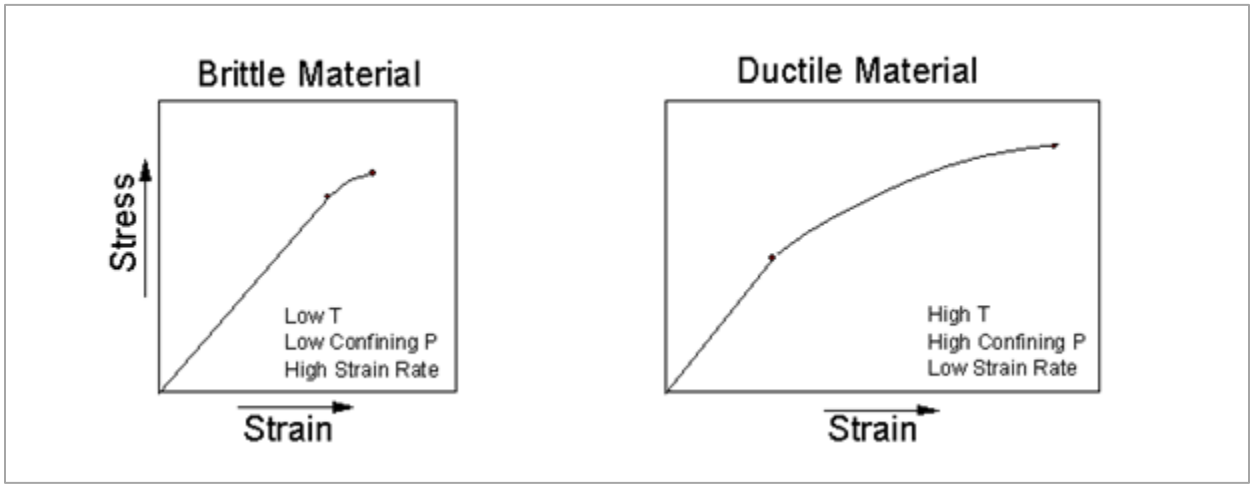


Figure 2.12 Ductile and brittle behaviour of materials (Selverstone).

2.4. Pavement material behaviour

The behaviour of pavement materials due to pavement material characteristics can be divided into either a response behaviour or damage behaviour, as shown by the flow chart in Figure 2.13.

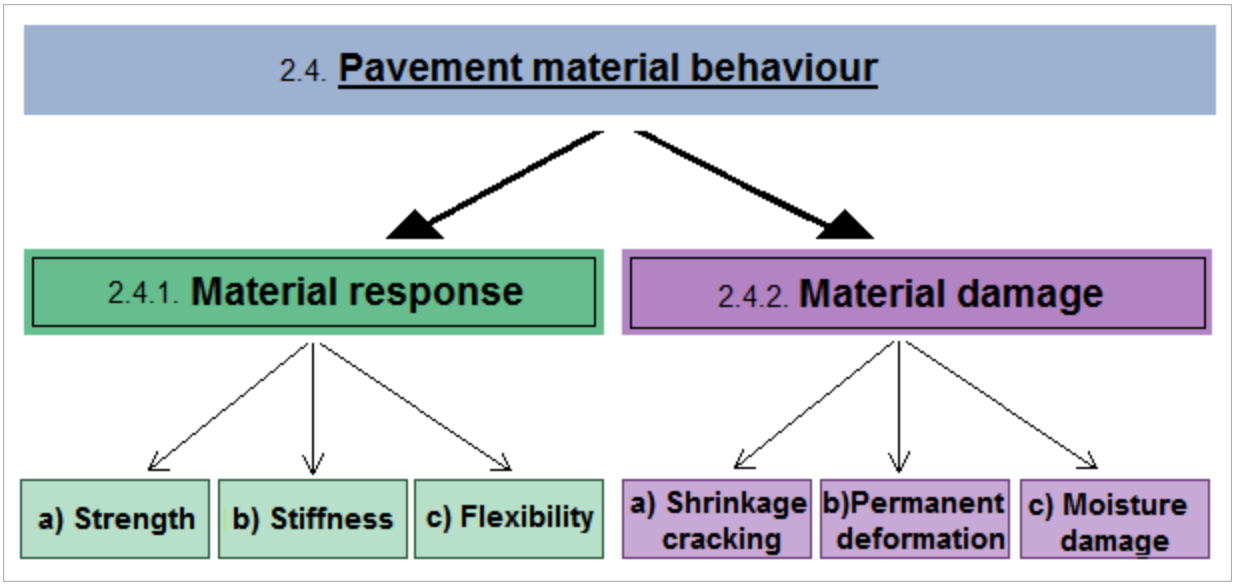


Figure 2.13 Flow chart of pavement materials behaviour.

2.4.1. Material response

2.4.1.a) Strength

The strength (shear strength) of granular materials varies from material to material and can be determined by obtaining the failure line/envelope, through triaxial testing.

By plotting a Mohr circle (Figure 2.14) of the monotonic triaxial test results, the cohesion (C; y interception) and the friction angle (ϕ; tangent line slope) of the material can be determined. The line tangent to the Mohr circle is known as the failure envelope above which stress states cannot exist (failure occurs). The shear strength can then be determined by using Equation 4.

$$\tau = C + \sigma . \tan \varphi \qquad \text{[Equation 4]}$$

Where: τ = Shear stress (kPa)

C = Cohesion (kPa)

Φ = Friction angle (°)

σ = Normal stresses from triaxial results (MPa)

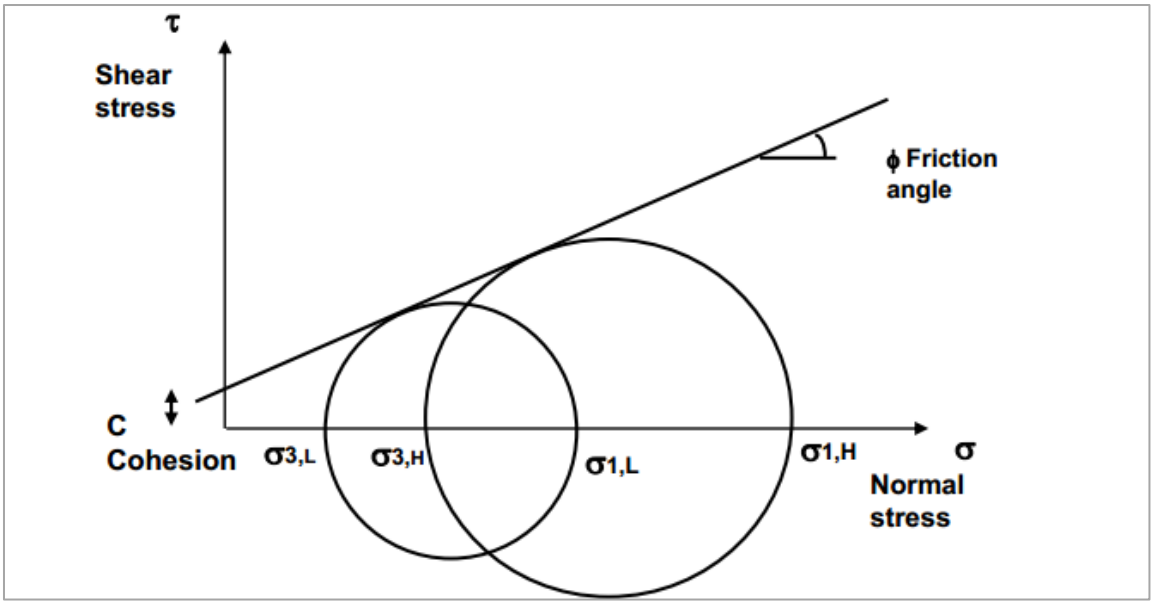


Figure 2.14 Mohr coulomb plot of Monotonic triaxial test results (Jenkins, 2012).

2.4.1.b) Stiffness

As described previously, the behaviour of bitumen stabilised materials can be assumed to be similar to that of granular materials due to their non-continuously bound nature.

The load spreading ability of a granular layer is dependent on the stiffness of the layer (Jenkins, 2012). The stiffness (resilient response) of a granular layer is characterized by the Resilient Modulus (M_r) under dynamic loading and is stress dependent. The elastic modulus of a material is defined as the slope of the tangent line to the stress-strain relationship, but because it is not a material constant, it is termed resilient modulus instead of elastic modulus (Molenaar, 2013). The M_r value of a material is thus provided for a given stress state and is an important material input for pavement design.

A stress pulse is applied to a pavement layer when a vehicle passes over the pavement structure (Li, Liu, & Zhang, 2010), similar to the dynamic loading at applied vertical stresses and at different deviator stresses during Dynamic Triaxial testing (Jenkins, 2012), indicated by Figure 2.15. The unbound granular layer (base layer) will exhibit a combination of resilient strains and permanent strains, where the resilient strains are recoverable after each load and the permanent strains will accumulate with application of load cycles (Li, Liu, & Zhang, 2010). As illustrated by Figure 2.16, the stress-strain relationship for the granular material is non-linear.

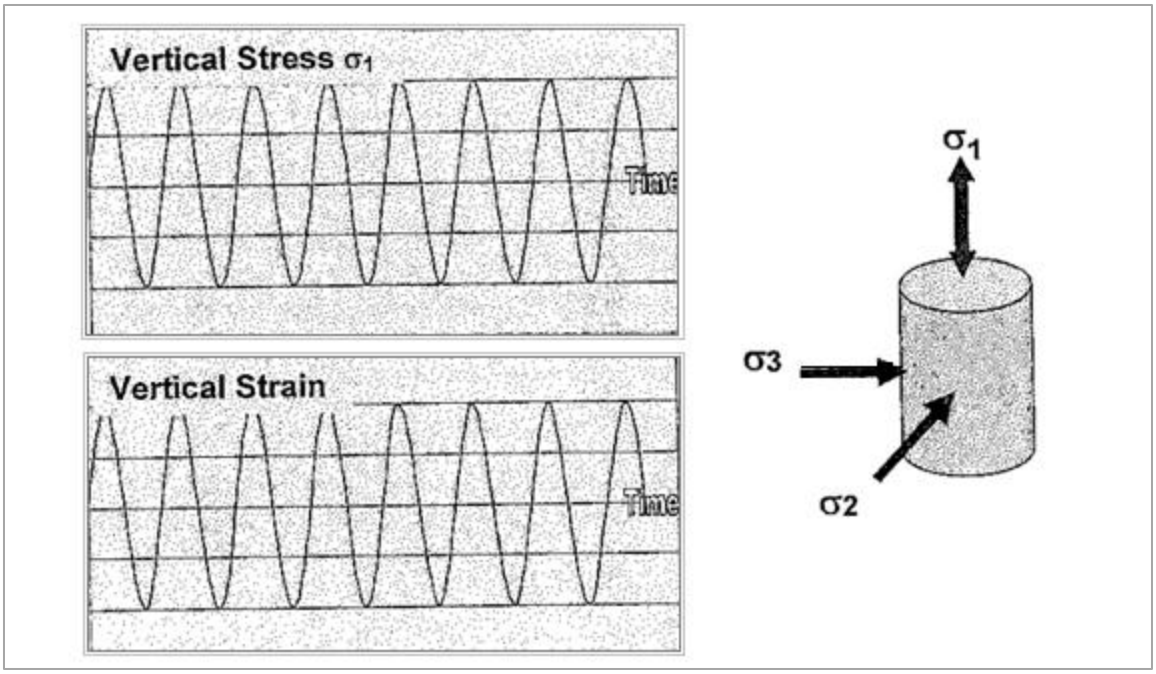


Figure 2.15 Loading configuration for dynamic testing (Jenkins, 2012).

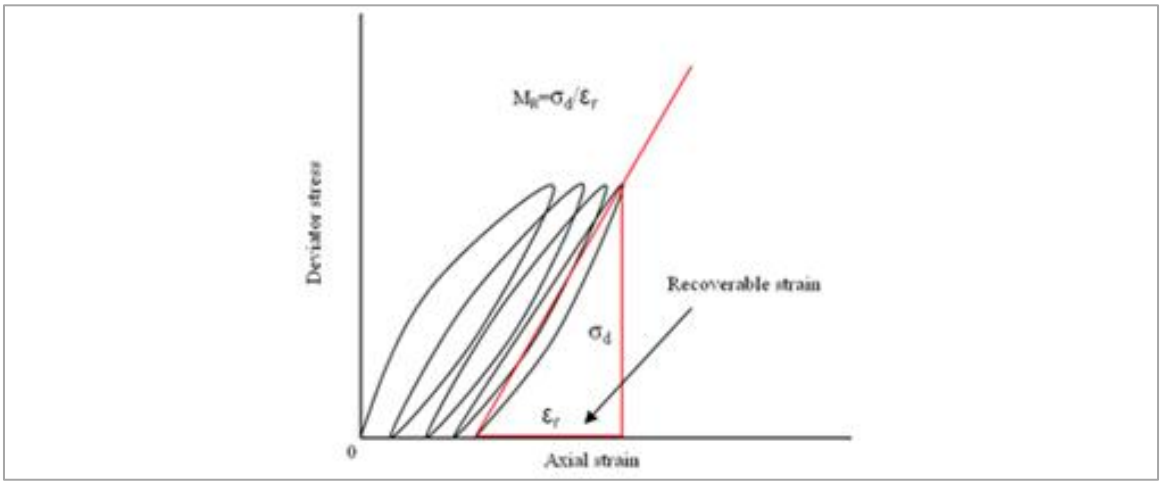


Figure 2.16 Definition of Resilient Modulus (Li, Liu, & Zhang, 2010).

Stress dependency implies that the stiffness of the material layer will increase with an increase in applied stresses (Jenkins, 2012). Figure 2.17 provides results of repeated load triaxial tests to obtain the M_r of an unbound base material.

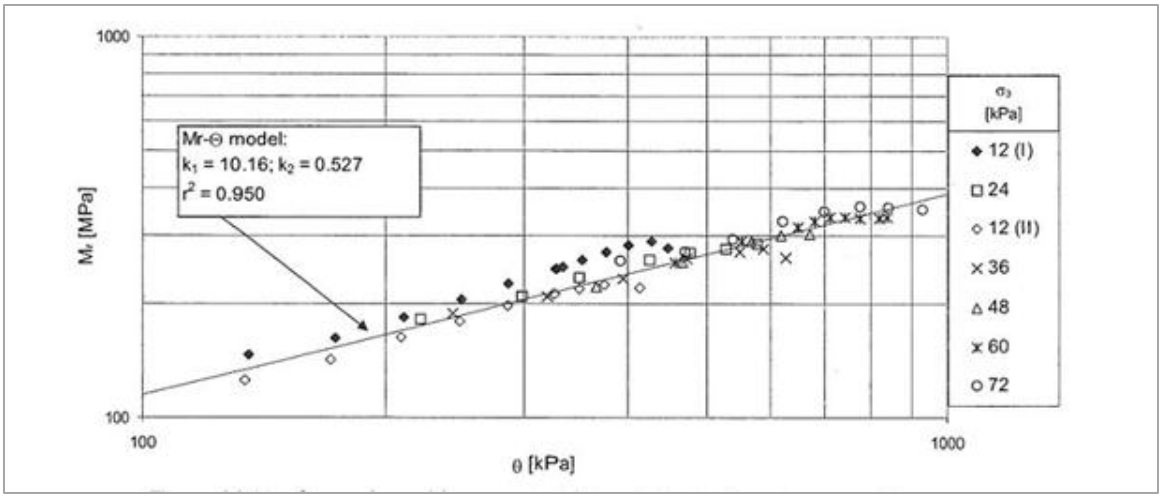


Figure 2.17 Resilient Modulus versus stress conditions of an unbound base material (Jenkins, 2012).

One of the most important factors that have an influence on the resilient properties of a material is the principle stresses. This influence of the confining stress and major principal stress is evident in the example in Figure 2.17, where an increase in confining stresses and an increase in the sum of principle stresses, increases the M_r. The M_r value of a material is also dependent on other factors, such as the aggregate shape and texture of coarse aggregates (Li, Liu, & Zhang, 2010). As the density of a granular material increases, the layer stiffness increases and permanent deformation caused by repeated loading will be reduced. It is thus also suggested by some researchers that increased densities will increase the M_r value of a material layer.

The Resilient Modulus (M_r) can be defined mathematically as the deviator stress divided by the "recoverable" strain when a material is un-loaded (Li, Liu, & Zhang, 2010). The M_r is calculated using Equation 5, which is sugested by AASHTO T307.

$$M_r = \frac{\sigma_d}{\varepsilon_r} \quad \text{[Equation 5]}$$

Where:

σ_d = deviator stress

ε_r = recoverable or resilient strain

Due to the complexity of this problem, it is difficult for researchers to model the relationship between the stress states with the material stiffness. Most models have been developed based on simple curve-fitting procedures by using the data obtained from laboratory triaxial testing (Li, Liu, & Zhang, 2010).

The results from triaxial tests can best be analysed by plotting the Resilient Modulus against the bulk stress on logarithmic scale, as illustrated in Figure 2.18. A simple model used to describe the stress dependent nature of the Resilient Modulus is depicted by Equation 6 (Molenaar, 2013).

$$M_r = k_1.\theta^{k2} \qquad \text{[Equation 6]}$$

Where:

M_r = Resilient Modulus (MPa)

k_1 and k_2 = material coefficients (-)

Θ = bulk stres = $\sigma_1 + \sigma_2 + \sigma_3$ (kPa)

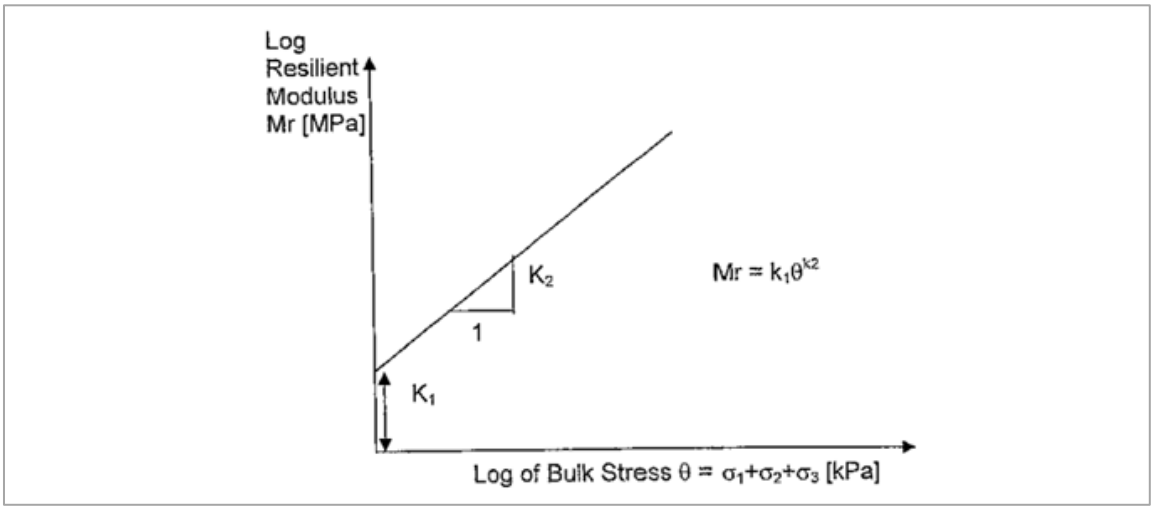


Figure 2.18 Mr – Θ Model of the Resilient Modulus of a course grained granular material (Jenkins, 2012).

Although this relationship seems to be adequate for granular materials, this model is not accurate to characterize material stiffness to stress conditions for materials, since it predicts that the modulus will increase when the material will come closer to failure thus leaving out the fact that softening occurs at higher stress levels (Molenaar, 2013). Equation 7 was thus developed by van Niekerk and Huurman to take into account the decrease in resilient stiffness as the vertical stress ratio approaches a critical value. This model predicts a lower modulus with an increase in deviator stress, providing a more accurate relationship between the stress states with the material stiffness for all materials (including granular materials).

$$M_r = k_1 \left(\sigma_3/\sigma_0\right)^{k2}.1 - k_3\left(\sigma_1/\sigma_{1,f}\right)^{k4} \quad \text{[Equation 7]}$$

Where:
- σ_3 = confining stress [kPa]
- σ_0 = reference stress = 1 kPa
- σ_1 = applied total vertical stress [kPa]
- $\sigma_{1.f}$ = total vertical stress at failure at the given confining stress [kPa]
- k_1 = model parameter [MPa]
- $k_2\ to\ k_4$ = confining stress [kPa]

2.4.1.c) Flexibility

Different materials respond differently to applied stresses. The addition of bitumen to granular materials provides a BSM with the ability to behave as a visco-elastic material. This visco-elastic behaviour provides flexibility to a pavement layer when responding to applied loads.

Flexibility is a material property that will provide a material with the ability to bend without breaking into pieces (fracturing) when subjected to stresses and therefore lies somewhere between ductile and brittle behaviour.

The area under a stress-strain curve is known as the dissipated energy of that material, which provides some indication of the toughness of the material. Figure 2.19 compares the behaviour of a brittle and ductile material (Ebels, 2008) and it can be concluded that the dissipated energy of a ductile material will be higher than that of a brittle material, indicating that a ductile material is tougher than a brittle material. The critical parameter

that is used to analyse the fatigue of a material is the strain-at-break (Ignacio Pérez, 2013). The strain-at-break test is used to measure the flexibility of a material by monotonically loading a beam specimen (Long & Theyse, 2004).

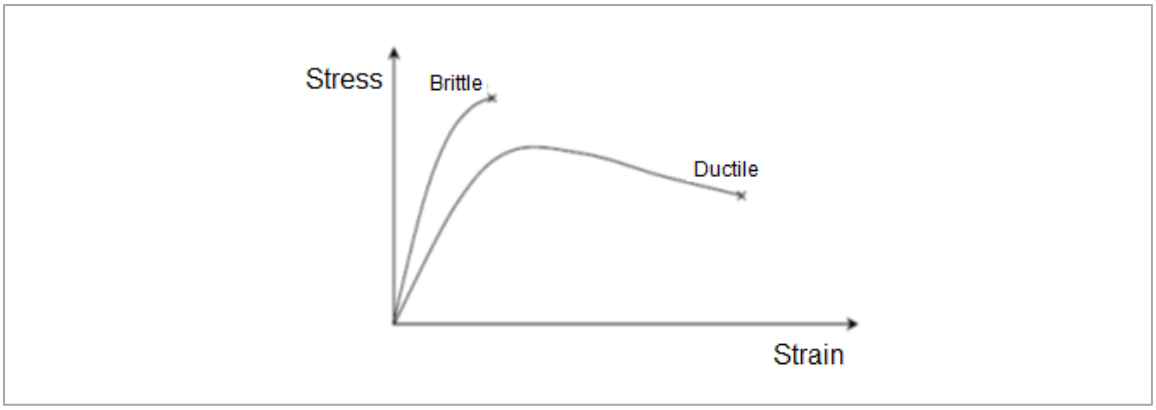


Figure 2.19 Brittle vs. ductile behaviour of materials (Ebels, 2008).

2.4.2. Material damage

2.4.2.a) Cracking

Cement-bound pavement materials will shrink due to material hardening caused by the reduction in moisture as well as a reduction in temperature.

The shrinkage is greatly obstructed by friction from the underlying layer, which will cause tensile stresses to develop within the layer. Too high values of these tensile stresses will cause cracks to develop, known as shrinkage cracks as seen in Figure 2.20 (Asphalt Pavements). All cement treated materials (only cement) will exhibit shrinkage cracks (Wirtgen GmbH, 2012). Any crack in a pavement layer is a weak point in the pavement, since bending moments (formed in the layer due to traffic loading) can not be transmitted by the cracked point (Asphalt Academy, 2009).

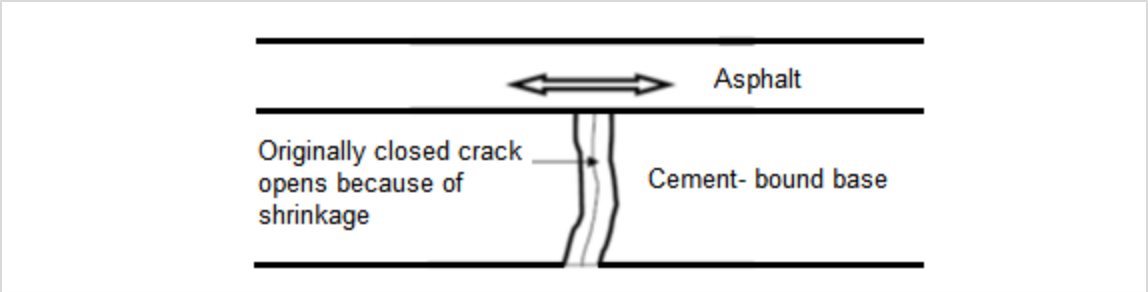


Figure 2.20 Cracks in cement-bound base will propagate to the asphalt layer (DFID).

An increase in load repetitions after cracking failure can cause these cracks to be reflected through the asphalt surfacing (DFID), creating a path for moisture to enter the pavement which will reduce the pavement life.

2.4.2.b) Permanent deformation

Permanent deformation in pavement layers can provide dangerous driving conditions, in both wet and dry weather.

A flexible layer with a low stiffness (such as asphalt) is more prone to permanent deformation than a less flexible pavement layer with a higher stiffness (such as a granular layer). Materials with higher stiffnesses will therefore be more resistant to permanent deformation.

The high stiffness of concrete pavements will thus prevent rutting (permanent deformation) to take place in the layer. These layers may however crack due to permanent deformation of the layers under the cement layer (Jenkins, 2012).

2.4.2.c) Moisture damage

Cracking of a stabilised material is caused by shrinkage (usually a cement treated layer) or due to repetitive loading which causes cracks to develop and propagate (R.Luhr, 2004). With additional load application, shrinkage cracks in the stabilised granular layer can reflect through the asphalt surfacing (DFID, p. 17). These cracks will allow water to filter into the pavement structure.

Water entering through the cracks can cause the materials to soften and deteriorate. The water will also provide inter-particle lubrication when a load is applied, reducing the bearing capacity of the pavement layer (Wirtgen GmbH, 2012).

The voids in a mixture can increase the ability of water movement through materials, making adequate compaction necessary (Lent, 2008).

2.5. Pavement material characteristics

All of the components added to a pavement material will have an influence on the characteristics and behaviour of a pavement material. The addition of different components to a pavement material will be discussed below.

2.5.1. Aggregate material

The aggregates used to construct a pavement layer provide the structure (skeleton) of the pavement layer that will spread the applied loads (TECHNICAL RECOMMENDATIONS FOR HIGHWAYS DRAFT TMH8, 1987).

Important aggregate characteristics that will have a major influence on the behaviour of a pavement layer are:

- Material quality:

  A good quality aggregate material can produce a strong, good quality pavement layer. The lower the quality of the material the weaker the pavement layer.

- Particle shape:

  Angular aggregates will have higher interlock between particles than rounded particles. Angular aggregates will thus increase the stiffness and strength of the material.

- Grading:

  A sufficient material grading should be used for the pavement layer, ensuring adequate voids in the pavement layer while not using too many fine or too few fine graded particles (Jenkins, 2012).

2.5.2. Cement (active filler)

Different types of active filler can be added to granular materials, i.e. cement, lime and fly ash (Wirtgen GmbH, 2012, p. 140). The effects of cement will be discussed since that is the active filler used for this project. Cement can be used to modify the properties of a granular material even if small amounts (even 1% cement) are applied (Jenkins, 2012).

Immediately after water has been added to cement, a short period of intense chemical reaction takes place (Kaspar & Potgieter, 1999), where crystalline bonds start forming between particles promoting adhesion between the particles (Wirtgen GmbH, 2012). This reaction is known as cement hydration. This chemical reaction changes the structure of the plastic mass, forming a hard and rigid paste (Kaspar & Potgieter, 1999).

2.5.2.a) Strength

The cementitious bonds formed during cement hydration will increase the internal friction, causing an increase in material strength. Triaxial tests conducted at CSIR and the University of Stellenbosch indicates an increase in friction angle with an increase of active filler (K. J. Jenkins, 2007), shown in Figure 2.21.

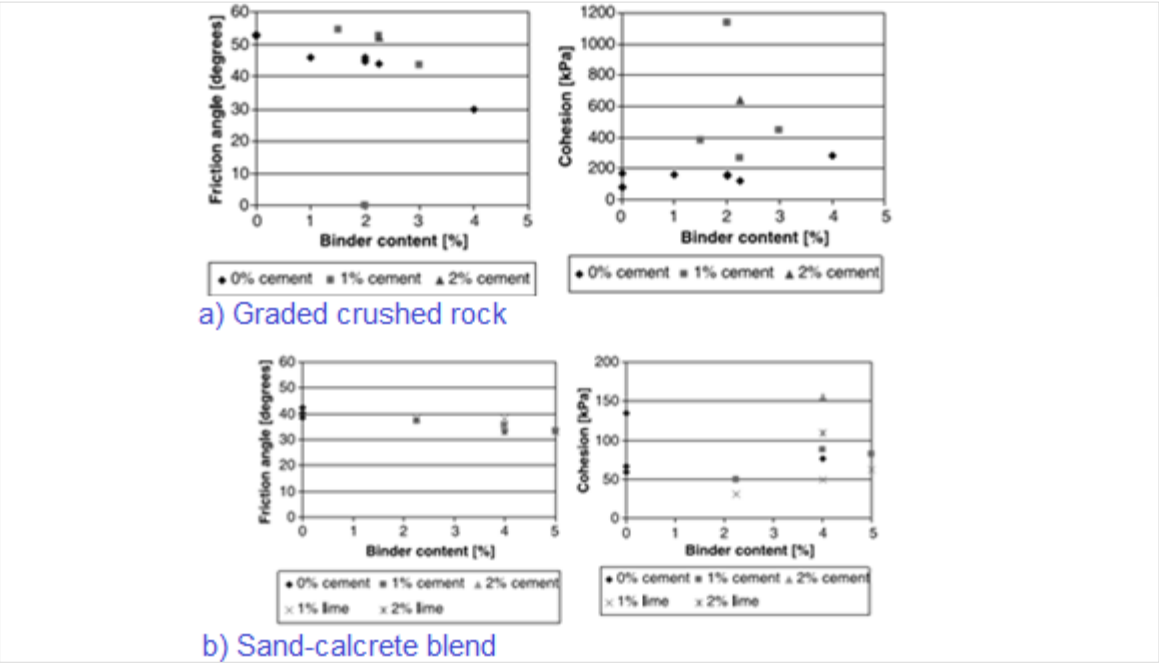


Figure 2.21 Effect on friction angle with change in bitumen and cement content for different materials.

Addition of cement to materials will also cause an increase in compressive strength and tensile strength (Wirtgen GmbH, 2012, p. 110). Both tensile and compressive stress will increase with increase in cement content (R.Luhr, 2004) and will be able to resist more load repetitions before failure.

As time passes, the strength of a material stabilised by cement will increase, as shown by the graph in Figure 2.22 (DFID).

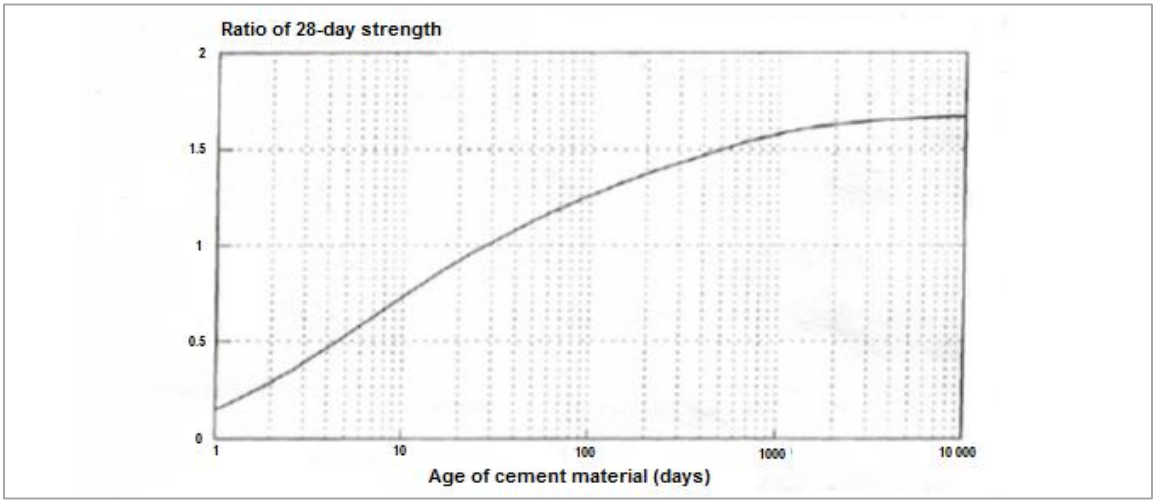


Figure 2.22 Increase in strength of a cemented material with time (DFID).

2.5.2.b) Stiffness

As stated previously, cement causes a mixture to become hard and rigid, increasing the material stiffness. The flexibility of the mixture will thus be reduced with the addition of cement (also see Section 2.6) and an increase in cement thus also results in an increase in resistance against permanent deformation (Jenkins, 2012).

2.6.2.c) Shrinkage

As bonds form during cement hydration, a volume change is experienced by the materials in the form of shrinkage (Wirtgen GmbH, 2012, p. 111). The hydration shrinkage contributes to only a small amount of the shrinkage, other factors (such as moisture reduction) contributes to the shrinkage in a greater extend. Adding cement to materials that exhibit volume change without cement will actually decrease the total

shrinkage. However, to high amounts of cement will cause more water consumption during cement hydration, causing an increase in drying shrinkage (R.Luhr, 2004). The amount of shrinkage will thus be determined by the amount of cement added to the materials (Wirtgen GmbH, 2012).

2.5.3. Bitumen (binder)

Bitumen is a visco-elastic material which can enhance the performance of either fresh or recycled aggregate materials. Granular aggregates can either be treated by bitumen emulsion or foamed bitumen (Asphalt Academy, 2009).

- ***Bitumen emulsion*** comprises of bitumen emulsified in water (Wirtgen GmbH, 2012). In this oil-in-water type bitumen emulsion, the bitumen is hold in suspension by an emulsifying agent. Once the bitumen emulsion has been mixed with the aggregates the emulsion needs to break to allow the bitumen to act as binder (Asphalt Academy, 2009). Both the moisture and aggregate type play an important role in the breaking of the emulsion as well as with the dispersion of the bitumen during mixing (Wirtgen GmbH, 2012, p. 106). The bitumen act as lubricant and therefore breaking should occur only after compaction (Asphalt Academy, 2009).

- ***Foamed bitumen*** is a mixture of air, water and bitumen (Martin Kendall). The foam is produced by injecting water into hot bitumen, forming foam (Asphalt Academy, 2009). The bubbles of the foam will burst during the mixing process, producing bitumen “splinters” that disperse throughout the aggregates, reacting with the finer particles to form a mastic (Wirtgen GmbH, 2012).

Adhesion from bitumen binds particles together and is one of the most important properties of bitumen. The adhesion property is influenced by the characteristics of bitumen itself, the aggregates in the mixture as well as the properties of the mixture such as the void content and permeability. High surface porosity of aggregate surfacing absorbs high bitumen quantities and keeps it locked in, causing a strong adhesive bond to form between particles (Lent, 2008).

2.5.3.a) Strength

Stabilising a material with bitumen provides strength to the pavement layer. As the bitumen is dispersed through the materials during mixing, the shear properties change with an increase in cohesion values and with a small decrease in the friction angle (Collings, 2009). The reduction in friction angle can be due to the added bitumen which acts as a lubricant (K. J. Jenkins, 2007).

2.5.3.b) Stiffness

The visco-elastic property of the bitumen will reduce the material stiffness, while providing flexibility and durability to the materials (Collings, 2009). An increase in bitumen will provide an increase in flexibility (Ignacio Pérez, 2013), which can lead to an increase in permanent deformation.

2.5.3.c) Moisture susceptibility

A layer stabilised with bitumen will experience a reduction in moisture susceptibility as a result of the manner in which the bitumen is dispersed through the particles (Collings, 2009). The bitumen will act like a glue that sticks the particles together, creating a layer that prevents the ingress of water (DFID).

2.6. Behaviour of Bitumen Stabilised Materials

Stabilisation improves the performance of a material by increasing its strength, stiffness and durability (DFID). The performance of the stabilised material should be at least equal to, or better than a good quality natural material.

A number of different types of possible ingredients can be added to a BSM. The amount and type of component added to the mixture should be optimized to produce an adequate material for the specific needs and requirements of the application thereof. The basic components incorporated in a BSM mixture is the virgin aggregate material, water, bitumen and in many situations active filler, with each having its own variability (Wirtgen GmbH, 2012). These components can be added in different amounts, but if the cement content exceeds the bitumen

content the material will be considered as cement stabilised material (Collings, 2009). Even the addition of small amounts of these ingredients will modify the properties of the material (DFID).

Until now not much research have been done to gain knowledge on the mechanical behaviour of BSMs in order to improve the structural design of this material (Ignacio Pérez, 2013). As illustrated by Figure 2.23, BSMs can vary from stress dependent, to brittle or flexible (K. J. Jenkins, 2007). Each of the materials added to BSMs, will have an effect on the material performance, with cement and bitumen having the most important influence (Figure 2.23) on the structural role of these materials (Ignacio Pérez, 2013). BSM behaviour depends on all the added components, as well as the degree of compaction, moisture content and the success of mixing the material with the stabiliser (DFID).

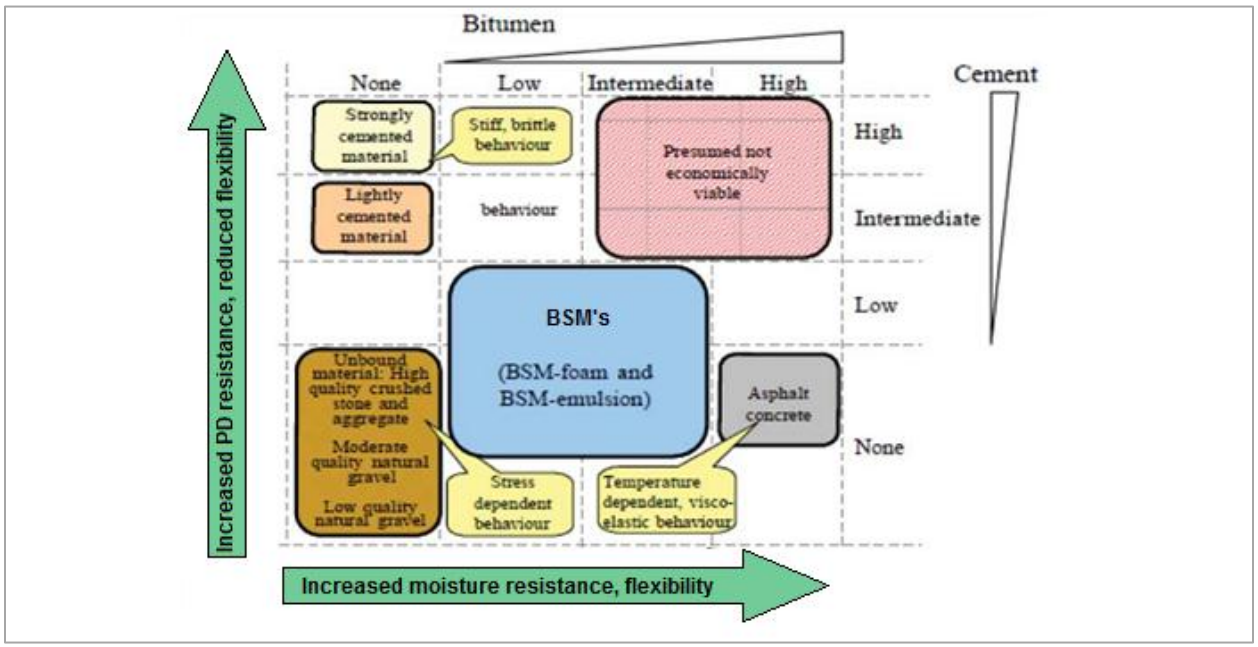


Figure 2.23 Influence of cement and bituminous binder on the behaviour of BSMs (Asphalt Academy, 2009).

2.6.1. Response behaviour

2.6.1.a) Strength/durability

Stabilisation with bitumen will increase the strength of a material (Wirtgen GmbH, 2012) and improve moisture sensitivity and durability (Asphalt Academy, 2009).

The water and cement in a BSM mixture will influence the “breaking” (separating the bitumen from the water in the emulsified state) of the bitumen emulsion. The bitumen emulsion is dispersed amongst the finer particles, with some “painting” of the larger particles (Asphalt Academy, 2009). In BSM-foam, the bitumen bubbles burst and tiny bitumen splinters are dispersed throughout the aggregate by sticking to the finer particles to form a mastic (Wirtgen GmbH, 2012). Compaction presses the mastic against the larger particles, resulting in “spot-welding”. BSMs will thus not have a continuum of bitumen (discussed in Section 2.2) and will seldom be homogeneous (Jenkins, 2012), making BSMs different from all other materials.

Due to the non-continuously bound nature of the BSM, the material will resemble an unbound granular material with stress dependent behaviour in the first phase after stabilisation, whose properties have been improved with the addition of bitumen (Ignacio Pérez, 2013). In the second phase the BSM shows a tendency to visco-elastic behaviour with some temperature and time dependent behaviour. The active filler and the moisture in the BSM will assist in dispersing the bitumen through the mixture (Asphalt Academy, 2009). Since hydration is such a rapid process, bitumen should be added as soon as possible to a mixture after water has been added to the aggregate and cement mixture, ensuring better dispersion of the bitumen through the mixture (Wirtgen GmbH, 2012).

The “spot-welding” bonding of BSMs will significantly increase the cohesion in comparison to that of the untreated material and will have no significant reduction in friction angle. The cohesion reduces the tendency of the material to ravel under the action of traffic, thus increasing the strength of the material (Wirtgen GmbH, 2012, p. 107). Cement (or active filler) is used in a BSM to improve the adhesion of bitumen to the aggregates and since the fines are covered and immobilized during BSM mixing, the moisture sensitivity and durability of the treated material is also improved (Asphalt Academy, 2009).

2.6.1.b) Stiffness

The stiffness of a BSM layer will provide a good indication of the quality of the pavement material as well as the pavement performance. A material layer with a higher stiffness is able to reduce the load-induced stresses that are spread to the less stiff underlying layers (Epps, Harvey, Kim, & Roque).

The stiffness of BSMs is dependent on the inherit stiffness of the aggregates, density of the material (achieved by compaction) and the added quantities of binder and active filler. The dispersion of binder and active filler throughout the mixed material will also contribute to the stiffness of the BSM layer. Due to the high cohesive strength of a BSM, the stiffness of the material under the load will be higher than the unbound parent material (Asphalt Academy, 2009). Another factor that will influence the stiffness of a BSM layer in a pavement structure is the stiffness of the underlying layers (Araya, 2011).

Addition of cement to a BSM will modify the characteristics of the aggregates by reducing the plasticity of the material. The cement will accelerate the curing proses of the compacted materials and strengthens and stiffens the mixture (Wirtgen GmbH, 2012).

Ebels (Ebels, 2008) proposed a lifetime stiffness hypothesis of BSMs based on in-place gathered data. He proposed two phases as illustrated in Figure 2.24; the curing phase (first phase) and a stiffness reduction phase (second phase). The initial stiffness will increase in the so called “curing phase” due to the densification of the layer and the reduction of moisture in the layer. During the second phase, or stiffness reduction phase, the stiffness of the BSM layer will decrease with time (Ignacio Pérez, 2013).

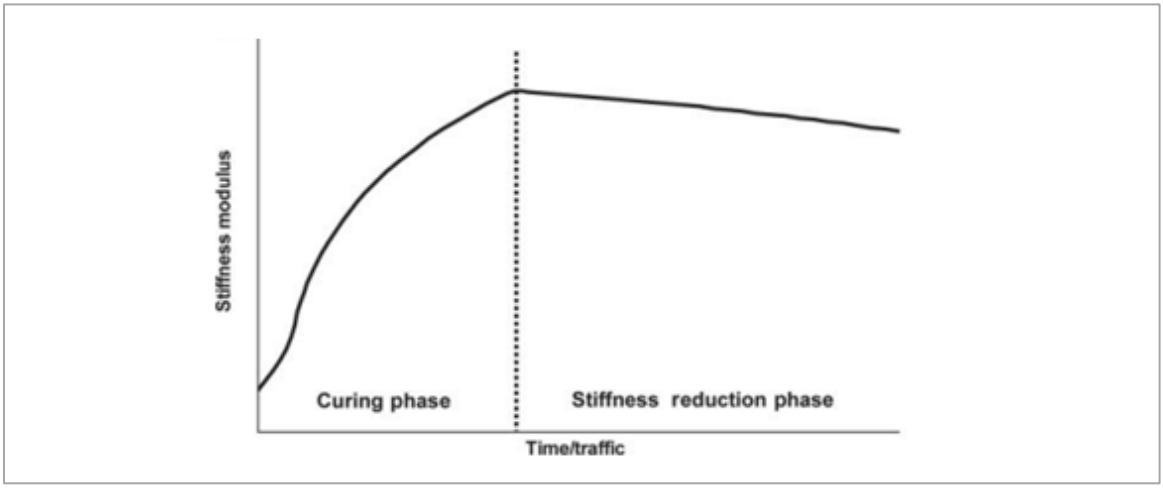


Figure 2.24 Hypothesis of the behaviour of BSMs - curing phase followed by the stiffness reduction phase (Ignacio Pérez, 2013).

2.6.1.c) Flexibility

The addition of bitumen to a BSM will cause the material (BSM) to behave like a visco-elastic material during the second phase (see stiffness in Section 2.6.1) of the lifetime of a BSM. The visco-elastic properties of the BSMs will provide flexibility to the fresh materials (Jenkins, 2012), where an increase in bitumen will increase the flexibility of the BSM. Since the addition of cement increases stiffness, the flexibility will be reduced if high cement contents are added (Asphalt Academy, 2009).

F. Long and H. Theyse conducted four point strain-at-break tests on various beam specimens using both foamed and emulsion stabilised materials. The results showed that with an increase in binder content the strain-at-break increases. It can thus be concluded that an increase in binder provides more flexibility to the mixture (Long & Theyse, 2004). These tests also showed a decrease in flexibility with the addition of cement, as illustrated in Figure 2.25.

The optimum ratio between cement and bitumen will thus have to be used to achieve the desired flexibility.

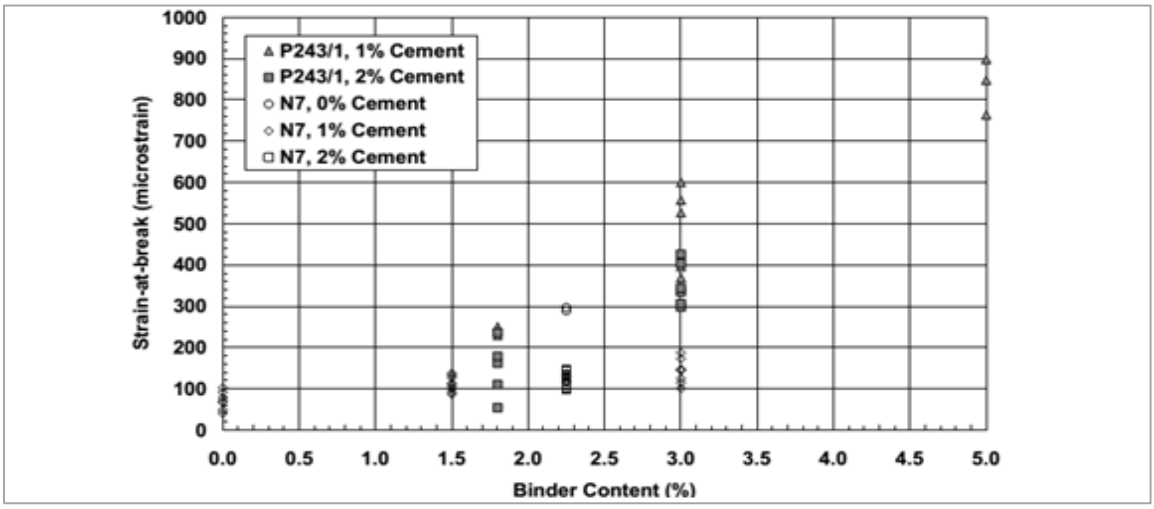


Figure 2.25 Strain-at-break test results (Long & Theyse, 2004) for emulsified bitumen treated materials.

2.6.2. Damage behaviour

2.6.2.a) Permanent deformation

Addition of bitumen to a BSM will provide a layer with visco-elastic behaviour. The visco-elastic properties increase the flexibility of a layer stabilised with bitumen, avoiding a stiff layer which can develop cracks due to fatigue (Jones, 2007). Since a BSM is non-continuously bound, the BSM will retain the granular characteristics of the parent material and is therefore stress dependent and not prone to cracking when subjected to tensile stresses during loading (Asphalt Academy, 2009).

The primary failure mechanism will thus not be the conventional fatigue cracking, but rather permanent deformation due to the flexibility of the layer (Collings, 2009). If a spot-weld breaks, particles will re-orientate (micro-shear) which will result in shear deformations (Jenkins, 2012).

2.6.2.b) Water susceptibility

A BSM will have improved moisture resistance and durability compared to the parent material (Wirtgen GmbH, 2012).

The moisture resistance is due to the bitumen dispersed through the materials (Collings, 2009) and the added cement to the mixture will improve the bitumen dispersion. The moisture susceptibility can further be reduced by stabilising good graded aggregates and with good compaction (Wirtgen GmbH, 2012). The decreased moisture susceptibility will decrease the damage caused by exposure to high moisture contents, ensuring that the strength is maintained and therefore increasing the durability.

2.7. Construction of BSMs and behaviour over time

2.7.1. Moisture

The amount of moisture (water) that should be added to a mixture to achieve the best possible mixture is known as the optimum moisture content (Wirtgen GmbH, 2012, p. 147). The moisture added during compaction causes changes in material strength and volume change.

Strength

One of the factors influencing the achievable density during compaction is the moisture content during compaction (TSHIVHASE, 2008). A higher density can be achieved with a lower compaction moisture content. Better compaction provides a stronger pavement layer.

If a BSM layer gains moisture, the resistance of the layer to shear failure will be reduced. The moisture increase will have a small effect on the friction angle, but will reduce the cohesion (K. J. Jenkins, 2007). Increased moisture after compaction causes the rate of deterioration in a pavement layer to greatly increase, leading to premature failure of the pavement layers (TSHIVHASE, 2008).

Volume change

If the percentage of swelling clays is more than 5% (by weight) in any type of soil, the clay will control the behaviour of that soil (J. Rogers). The primary characteristics of expansive soils are their potential to change volume due to a change in moisture content (Wankhade, 2014). This volume change, either swelling or shrinkage, is accompanied by a change in effective stress (De Wet, 2014) caused by a change in soil suction.

Wetting and drying of expansive soils, wetting of collapsible soils, as well as drying of slurry materials (Murillo, Caicedo, Hoyos, Colmenares, & Berdugo, 2013), will cause volume changes due to soil suction changes.

Metric suction tests were conducted by A.S. Udjianto (and others) in accordance with ASTMD 5298-2003, to determine the relationship between water content and metric suction of a specific expansive soil (Sudjianto, Suryolelono, Rifa'i, & Mochtar). An increase in water content showed a decrease in matric suction, as illustrated in Figure 2.26. Similar results were achieved by Rubulasa (2011), at the University of Stellenbosch, by comparing the suction pressure to the degree of saturation from different material types, as shown in Figure 2.27.

Furthermore, results of volumetric swelling tests with variation in matric suction (Figure 2.28) showed a decrease in swelling with an increase in matric suction (Sudjianto, Suryolelono, Rifa'i, & Mochtar). During the drying proses of an initially saturated soil, the suction will thus increase and could cause extremely large volume changes (Murillo, Caicedo, Hoyos, Colmenares, & Berdugo, 2013).

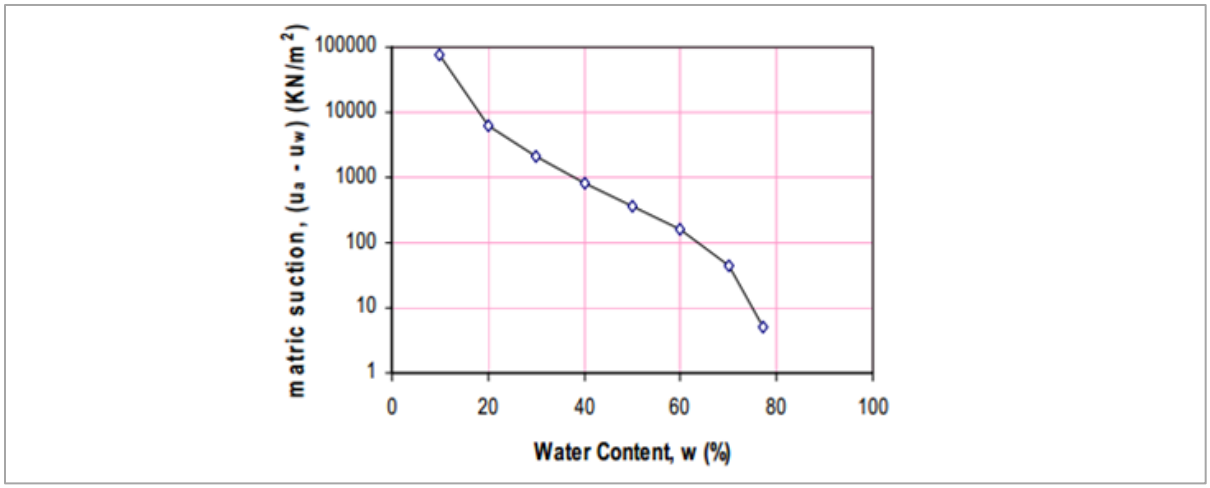


Figure 2.26 Curve indicating decrease in matric suction with increase in water content (Sudjianto, Suryolelono, Rifa'i, & Mochtar, p. 15).

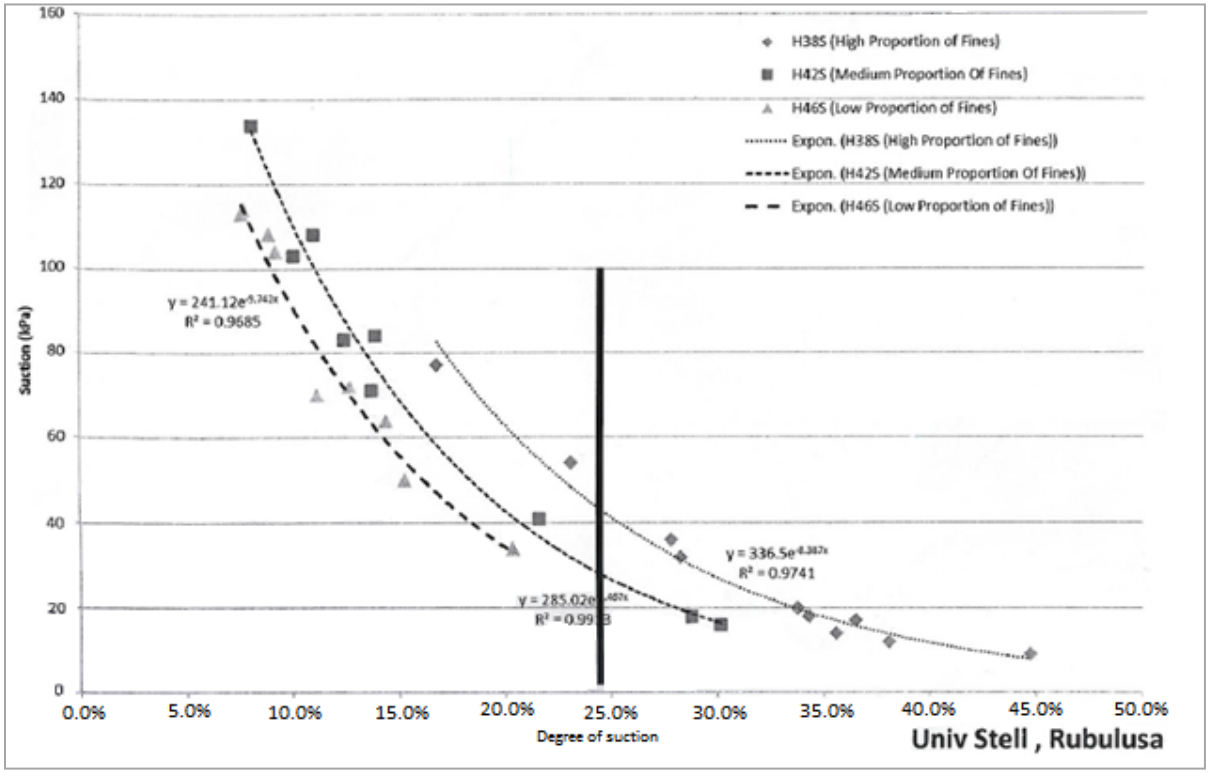


Figure 2.27 Relationship between suction and degree of saturation (Rubulusa, 2011).

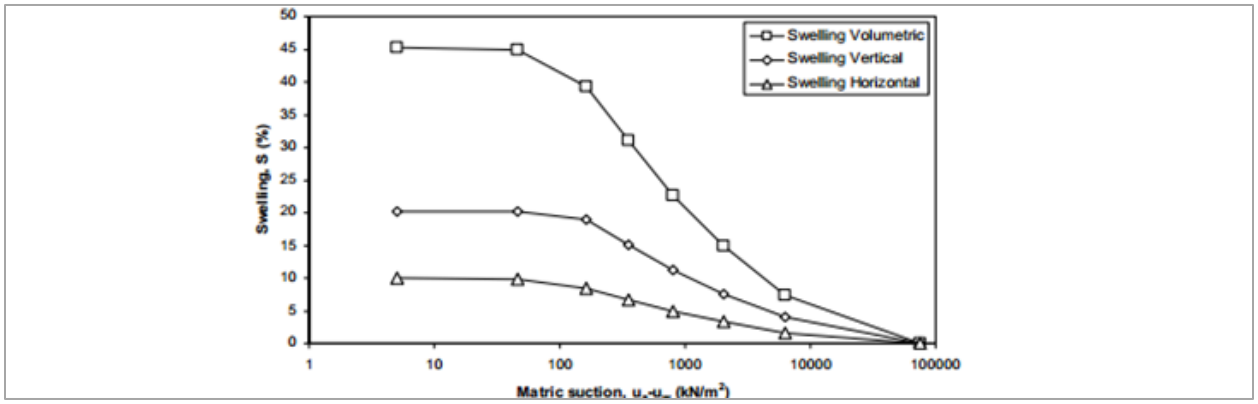


Figure 2.28 Relationship between swelling and metric suction of clay soil (Sudjianto, Suryolelono, Rifa'i, & Mochtar)

2.7.2. Compaction

Compaction is one of the most important factors influencing the mechanical behaviour of unbound materials. The response of pavements incorporating BSMs will improve when the density of the BSMs are increased (Ebels, 2008). The degree of compaction is influenced by the aggregate grading, aggregate form, plasticity, moisture content and compaction energy (Jenkins, 2012).

Strength

Monotonic triaxial tests has shown an increase in cohesion values and friction angles of BSMs due to the particle rearrangement, providing better stability of the pavement layer.

A study done by Long and Ventura on different specimen compaction levels proved that an increase in density will reduce the risk of shear failure (K. J. Jenkins, 2007). Not only will better compaction improve shear strength but the resilient modulus as well (Ebels, 2008), which will lead to a longer pavement life.

Stiffness

Compaction has a major influence on the permanent deformation behaviour of materials stabilised with bitumen (Ebels, 2008).

The degree of compaction will influence the stiffness of a pavement layer and thus the load spreading ability of that layer (Jenkins, 2012). An increase in compaction provides an increase

in stiffness. Better compaction will thus improve the load spreading ability of a material which causes a significant reduction in vertical deflections. Damaging settlement of pavement layers (including BSM layers) can be prevented by adequate compaction (Ebels, 2008).

Volume changes

An increase in compaction energy increases the dry density and reduces the optimum moisture content (Jenkins, 2012). Therefore good compaction will aid in moisture reduction (R.Luhr, 2004) and lower shrinkage will occur due to the decreased amounts of moisture in the mixture.

During compaction, soil particles are packed closely together leading to a lower void content. Lower amounts of void spaces will reduce the shrinkage potential and therefore compaction can influence volume change.

The least shrinkage (volume change) of a material will thus be obtained at higher densities and lower moisture content (R.Luhr, 2004). This response can be seen in Figure 2.29.

Compaction has a marked influence on materials that undergo great amounts of volume change, such as clayey soils. The greater the density of these soils, the greater the volume change potential due to swelling, unless the soil is restrained. These materials should thus be compacted to a degree where the shrinkage and swelling is kept to a minimum (Chapter 8, Failure).

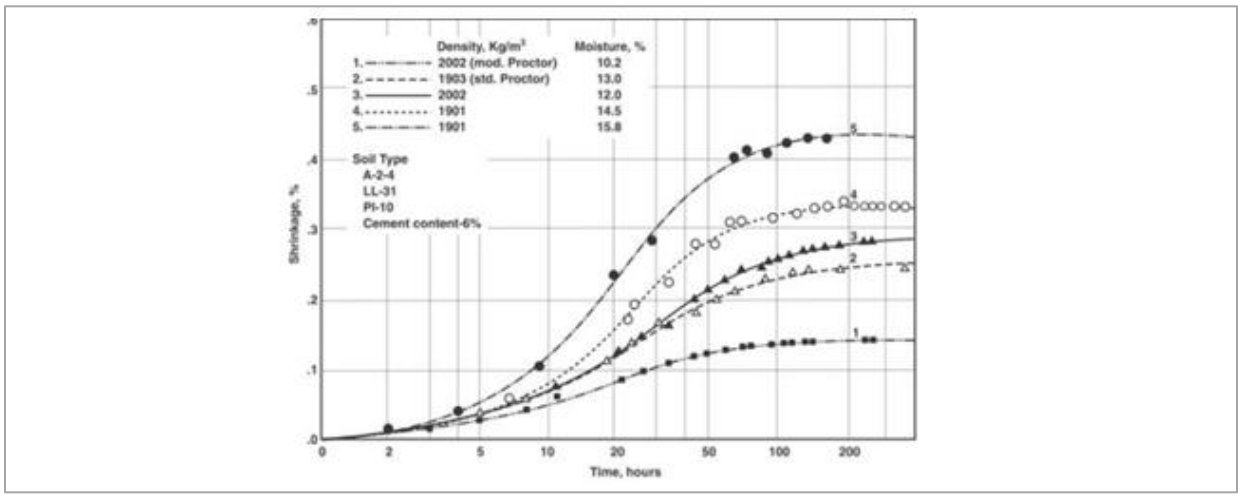


Figure 2.29 Effect of density and moisture on shrinkage (R.Luhr, 2004).

Moisture susceptibility

As mentioned earlier, increased density through compaction decreases the void spaces between particles (Jenkins, 2012). The decreased amount of voids will restrict seepage of water through the aggregates (Lent, 2008) and therefore adequate compaction can thus decrease permeability (Jenkins, 2012). This is very important for pavement layers since moisture ingress into layers will cause a lower resistance to failure of the material, resulting in a shorter pavement life (K. J. Jenkins, 2007).

2.7.3. Curing

A reduction in moisture content of a pavement layer after compaction is known as curing (Ebels, 2008). The rate of moisture loss in a pavement layer (BSM) plays a significant role in the performance of the layer.

Strength

Reduction of moisture during curing will lead to an increase in material stiffness (Ignacio Pérez, 2013). An increase in both tensile and compressive strength (and overall strength) will also be a result of moisture loss via evaporation (Wirtgen GmbH, 2012).

Shrinkage

Curing- or drying shrinkage is a result of moisture loss during curing. During the curing process, the capillary water is lost through evaporation, causing drying shrinkage in the material. This type of shrinkage is common in cement-bound materials (Asphalt Pavements) during the hardening process of the hydrated cement. The moisture content during mixing will affect the drying shrinkage effects. If the drying shrinkage is restricted, great tensile stresses can develop in the material which can lead to cracks. In a pavement structure, the shrinkage is restricted due to the friction between the relevant layer and underlying layer (Asphalt Pavements). With traffic loading, these cracks will propagate through the top layers and will enable water to seep through and cause premature failure of the pavement structure.

2.8. Main findings from literature

Material properties determine the behaviour of pavement layers and can be studied using laboratory tests. These tests provide an indication of the response behaviour of pavement materials as well as some insight into the material damage caused by repetitive traffic loading.

Figure 2.30 show how certain tested material properties can be related to the response behaviour of a pavement layer (such as BSM layers).

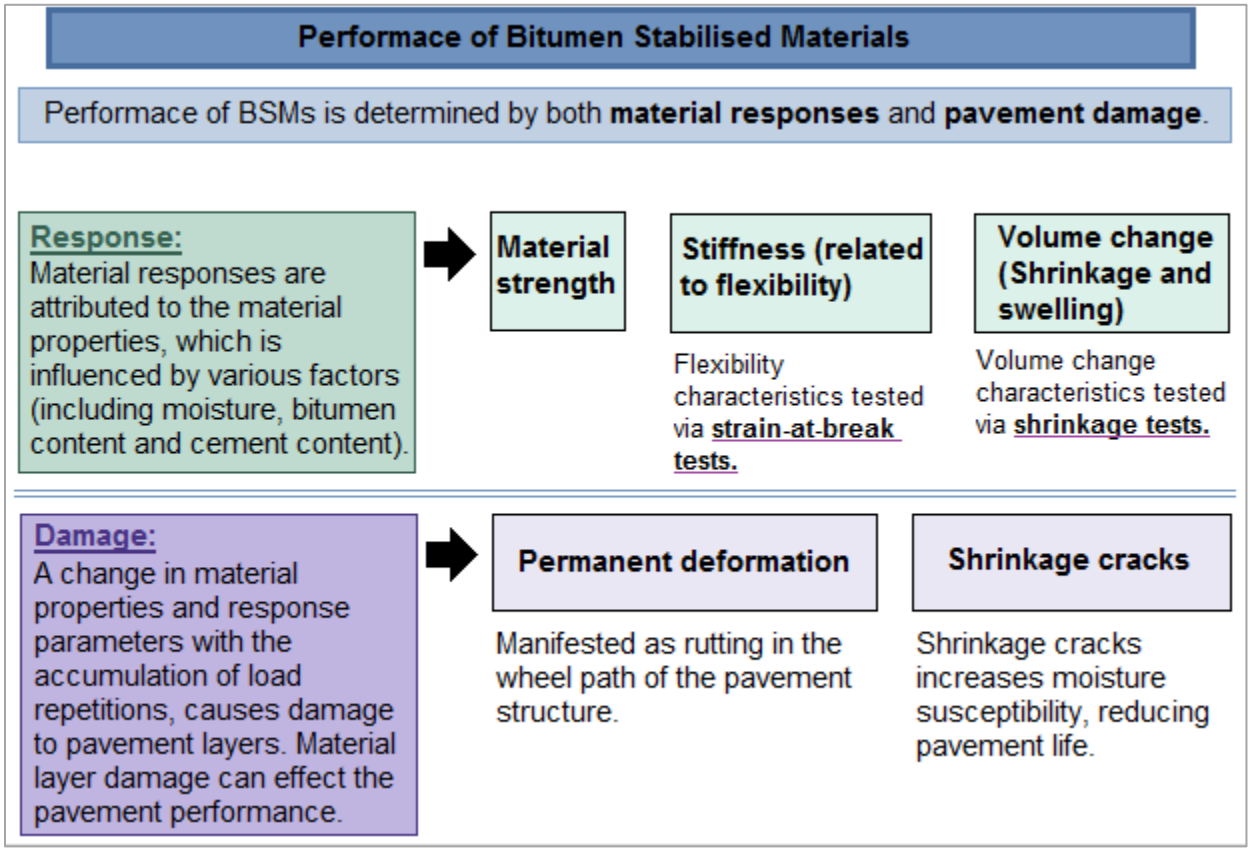


Figure 2.30 Pavement performance of BSMs.

The material strength is mostly attributed to the aggregate material but can be increased with the addition of cement. The dissipated energy of a material provides an indication of its behavioural properties, such as the material toughness. An increase in dissipated energy represents a tougher material.

During material curing, moisture evaporates from the material causing the stiffness of the material to increase. Adding cement to a mixture causes the material to become rigid, increasing the stiffness of the material, while reducing the flexibility and increasing the resistance of the material to permanent deformation. An addition of bitumen on the other hand, will reduce the stiffness and increase the flexibility (due to the visco-elastic properties), which reduces the resistance to permanent deformation in the material. Higher material flexibility will provide higher strain-at-break values.

Changes of the effective stresses in pavement layers cause volume changes to occur. Initially effective stresses are formed and locked into the pavements through compaction. Moisture changes over time, changes the effective stresses, leading to either shrinkage or swelling of the pavement materials. An increase in effective stresses causes shrinkage while a decrease in effective stresses causes swelling. Suction forces present during curing causes an increase in effective stresses (shrinkage), which can be reduced with lower initial moisture contents. The addition of cement to a mixture will also cause shrinkage, due to cement hydration.

CHAPTER 3 - Theoretical analysis

3.1. Introduction

Based on the research, hypotheses have been formed and an experimental design has been set for this project.

3.2. Hypotheses

3.2.1. Shrinkage

Moisture evaporation plays a major role in material shrinkage. As moisture evaporates from specimens during the curing process, internal suction will increase causing shrinkage. Shrinkage will thus start at the beginning of the testing period and will increase to the end of the testing period.

The chemical reaction that takes place during cement hydration will consume water, provide particle adhesion and increase drying shrinkage. An increase in cement content of the BSM will increase the shrinkage due to increased cement hydration.

The bitumen added to a BSM will cover the material particles when absorbed by the material while reducing the air voids. The bitumen will therefore lubricate the material and provide adhesion between particles. The reduction in air voids due to the absorbed bitumen will reduce the shrinkage tendency of the material. In increase in bitumen will thus provide a reduction in shrinkage. Stabilisation with foamed bitumen provides a less continues BSM in comparison to stabilisation with bitumen emulsion. This will allow the foamed bitumen stabilised materials to shrink more, since the cement will have a greater effect.

3.2.2. Flexibility

An increase in cement content in a BSM will increase the stiffness of the material, causing less flexibility. The higher the cement content the lower the flexibility or strain-at-break.

Higher bitumen contents will provide more flexibility and an increase in strain-at-break value. Specimens stabilised with bitumen emulsion will provide higher strain-at-break values in comparison with specimens stabilised with foamed bitumen.

3.3. Known R35-material properties

In December 2011, a bulk sample of material was collected from the experimental road-section in Mpumalanga (R35 road near Bethal). As part of a greater project, the CSIR processed the bulk sample of rehabilitated milled material and completed routine tests to determine the material properties.

In addition, CSIR set up the experimental design for cemented materials to be tested during the bigger project. Stellenbosch University set up the experimental design for the bitumen stabilised materials.

Based on those mix designs, the following treatments were considered for field construction on the southbound lane of the R35 (Theyse, July 2012).

- **Cement treatment** – 2% cement in combination with 1% lime;
- **Emulsion treatment** – 2.4% residual bitumen with 1 and 2% cement respectively; an additional 1% cement and 0.9% residual bitumen is considered for construction;
- **Foamed-bitumen treatment** – 2.4% bitumen with 1 and 2% cement respectively.

Material processing and routine testing was done during part one of the larger experimental testing project. Part two of the larger project comprises of non-standard laboratory tests, which this specific project is part of.

During part one the optimum moisture content of the R35-material was determined as 11.2 % and the maximum dry density determined as 2098kg/m^3 (Theyse, July 2012). The milled material obtained from the rehabilitated R35 consists of a mixture of 80 to 85% of granular materials (dolerite from the R35-pavement base and subbase) and 15 to 20% Reclaimed Asphalt (milled from the R35-pavement surfacing). The R35-material was separated into sample sizes of 25kg to 30kg each and sent to Stellenbosch University for laboratory testing. Achille Nwando (2014) from the University of Stellenbosch conducted a sieve analysis on three different R35-samples to compare the grading curve of the R35-material with that of the material grading recommended by the TG2 (Asphalt Academy, 2009). A. Nwando proved that each R35-sample has a grading that falls within the grading range suggested by the TG2 of materials to be stabilised with bitumen (indicated in Figure 3.1). Therefore the R35-material has a grading that will produce a good quality BSM mixture (Nwando, 2014).

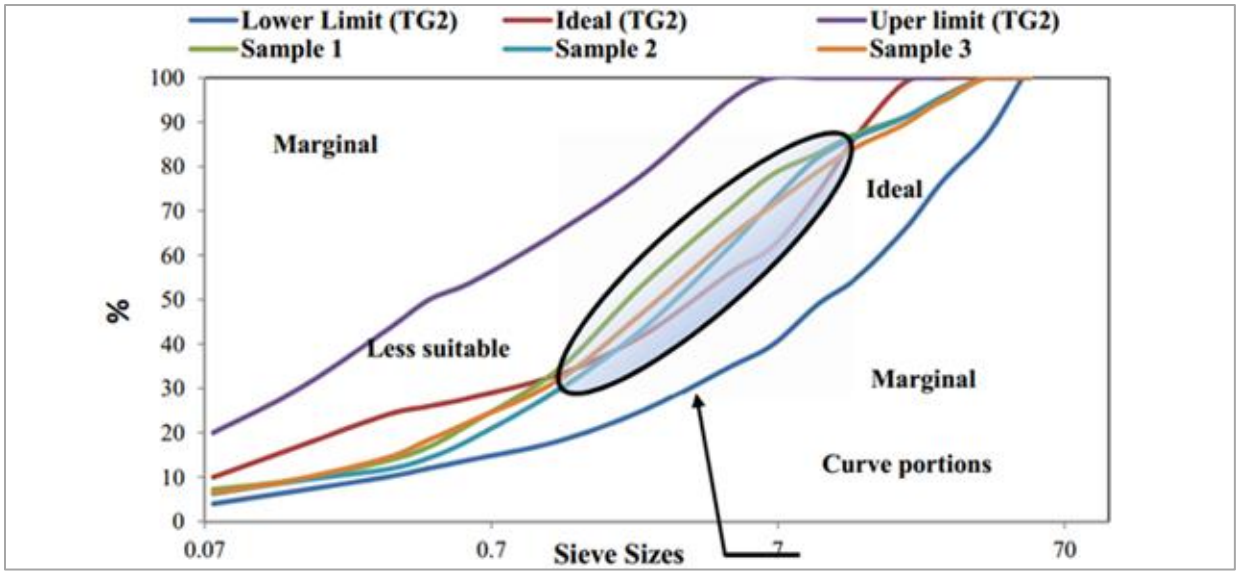


Figure 3.1 R35-material grading compared to TG2 guideline grading envelopes (Nwando, 2014).

3.4. Experimental design

The experimental design for BSMs that was set up by the University of Stellenbosch, as part of the bigger project, was used to set the experimental design for both the shrinkage and strain-at-break tests.

3.4.1. Shrinkage

The shrinkage behaviour of the Bitumen Stabilised R35-Materials will be determined by conducting linear shrinkage tests using two different testing methods. One method will consist of shrinkage tests conducted on beam specimens and the other will consist of shrinkage tests on cylindrical specimens. The influence of different variables on these specimens will be tested to better understand the shrinkage of BSMs. Each specimen type will be repeated three times to increase result accuracy. These variables are discussed below and the full experimental design for shrinkage testing is provided in Table 3.1.

3.4.1.a) Temperature

All shrinkage tests will be conducted at a temperature of 40 °C. This temperature has been selected as a realistic value that BSMs can reach in service and at the same time provide a higher order of shrinkage for more reliable measurements.

3.4.1.b) Aggregate material

The shrinkage of the R35-material itself will be evaluated during this project by testing specimens containing no additives (no bitumen or cement).

3.4.1.c) Bitumen

Specimens containing different bitumen emulsion quantities (0.9% and 2.4%) will be tested during this project to determine the influence of an increase in bitumen on the shrinkage of BSMs. Bitumen emulsion quantities (0.9% and 2.4%) are net bitumen percentages.

The influence of the type of bitumen on the shrinkage will also be measured by testing samples stabilised with either bitumen emulsion or foamed bitumen (see Table 3.1).

3.4.1.d) Cement

Specimens containing 1% cement and 2% cement will be tested during this project to determine the influence of increased cement quantities on the shrinkage of a BSM.

Table 3.1 Experimental design for **shrinkage testing** (Allocation of specimen identification was done to keep track of specimens during the testing procedure).

Test	Bitumen content		Cement content	Specimens type	Amount of specimens	Specimen identification		
	Bitumen type	Bitumen content				Specimen 1	Specimen 2	Specimen 3
Shrinkage - Tested at 40°C	(No bitumen)	0%	0%	Beam	3	B6	C6	P6
				Cylinder	3	B12	C12	C12
	Emulsion	0.90%	1%	Beam	3	B9	C9	P9
				Cylinder	3	B15	C15	P15
		2.40%	1%	Beam	3	B7	C7	P7
				Cylinder	3	B13	C13	P13
		2.40%	2%	Beam	3	B8	C8	P8
				Cylinder	3	B14	C14	P14
	Foam	2.40%	1%	Beam	3	B 10	C 10	P 10
				Cylinder	3	B16	C16	P16
		2.40%	2%	Beam	3	B11	C11	P11
				Cylinder	3	B17	C17	P17

3.4.2. Flexibility

Monotonic strain-at-break tests will be conducted on specimens with different variables, to determine the flexibility characteristics of Bitumen Stabilised Materials (R35-materials). Three specimens of each type will be tested to improve the accuracy of the results. These variables are discussed below and the full experimental design for strain-at-break testing is provided in Table 3.2.

3.4.2.a) Temperature

All strain-at-break tests will be conducted at an ambient temperature of 25 °C, to keep testing conditions consistent.

3.4.2.b) Bitumen

To determine the influence of bitumen emulsion content on the flexibility of BSMs (R35-material), specimens containing 0.9% and 2.4% bitumen emulsion will be tested during this project (0.9% and 2.4% net bitumen).

The influence of the type of bitumen on the flexibility will also be evaluated by testing specimens stabilised with both bitumen emulsion and foamed bitumen (see Table 3.2).

3.4.2.c) Cement

Specimens containing 1% cement and 2% cement will be tested during this project to determine the influence of increased cement quantities on the flexibility of a BSM.

Table 3.2 Experimental design for **strain-at-break testing** (Naming of specimens are done to keep track of specimens during testing procedure).

Test	Bitumen content		Cement content	Specimens type	Amount of specimens	Specimen identification		
	Bitumen type	Bitumen content				Specimen 1	Specimen 2	Specimen 3
Strain-at-break - Tested at 25°C	Emulsion	0.90%	1%	Beam	3	B3	C3	P3
		2.40%	1%	Beam	3	B1	C1	P1
		2.40%	2%	Beam	3	B2	C2	P2
	Foam	2.40%	1%	Beam	3	B 4	C 4	P 4
		2.40%	2%	Beam	3	B5	C5	P5

Chapter 4 - Methodology

4.1. Aim

The shrinkage and flexible behaviour of BSMs will be investigated to improve the understanding of the benefits and performance mechanisms of these materials. An improved understanding of both shrinkage and flexibility will enable more judicious mix designs to be carried out, with the selection of the correct components, which will have an impact on in service pavement performance. In this manner, the behaviour of these materials could be incorporated into the revised design method for flexible pavements in the SAPDM (South African Pavement Design Method).

4.2. Research

Research has been done on the factors that may influence the behaviour of BSMs to improve the understanding of the behaviour that will be obtained from laboratory tests. Extra attention was given to shrinkage and flexibility behaviour.

The possible damages associated with these behaviour types has also been researched to gain knowledge about the performance of pavement layers within a pavement structure.

4.3. Theoretical analysis

The main findings from the research have been summarized and used to form a hypothesis for both the shrinkage and flexibility behaviour of BSMs. These hypotheses will then be used as the theoretical model against which the outcome of the project will be compared.

An experimental design has been formulated to incorporate all the aspects that have to be tested in the laboratory that can change the performance of the material.

4.4. Moulds and testing apparatus

Shrinkage test methods together with the adequate testing equipment need to be designed to test specimens. A method will have to be designed for testing both beam and cylindrical specimens. One of the aims of the new developments of test configurations is to take shrinkage

beyond the realms of research and into the sphere of mix design, including simplicity, accuracy and robustness.

Moulds need to be designed enabling the researcher to quickly remove specimens from them, without disturbing and damaging the test specimens. Both beam and cylindrical moulds have to be designed for the purpose of this research project.

4.5. Specimen preparation

Material properties of the R35-material have already been determined during the first stages of the bigger research project of which this project is part of. Therefore determining the material properties is not part of the scope of this project.

The aggregate material needed for specimen compactions will be prepared prior to compaction and will be heated to the optimum ambient temperature for BSM mixing, which is 30°C according to the TG2 (Asphalt Academy, 2009).

All testing equipment has to be conditioned to the appropriate testing temperatures before initiating tests. Shrinkage tests will be conducted at 40°C and strain-at-break at 25°C (as discussed in Chapter 5).

4.6. Specimens types

Specimens produced during the project correlates with the experimental design that has been set (Table 3.1 and 3.2). Beam specimens as well as cylindrical specimens will be compacted.

- Three specimens of each variable type will be compacted and tested.
- Emulsion bitumen as well as foamed bitumen will be used to prepare specimens. Bitumen contents will vary between 0.9% and 2.4%.
- Specimens containing active filler of 1% and 2% per mass will be produced and tested.

4.7. Specimen compaction

Beam specimens with a height of 75mm, width of 75mm and length of 470mm will be compacted using a handheld MOD compaction hammer. Depending on the test that will be conducted on the specimens, they will either immediately be tested (shrinkage) or cured for a 28 day period (strain-at-break).

Cylindrical specimens with a diameter of 100mm and a height of 300 mm will be compacted using a vibratory hammer. Since all cylindrical specimens will be tested for shrinkage, these samples will all be tested immediately after de-moulding.

4.8. Specimen testing

4.8.1. Shrinkage testing

Shrinkage testing will start immediately after specimen compaction and de-moulding for both the beam and cylindrical specimens. Testing will be conducted according to the designed testing procedure, for a 72 hour period at 40°C. As mentioned earlier, this temperature has been selected as a realistic value that BSMs can reach in service and at the same time provide a higher order of shrinkage for more reliable measurements.

4.8.2. Strain-at-break testing

Specimens compacted for monotonic strain-at-break testing were removed from moulds after compaction and cured for a 28 day period, where after they were tested at a temperature of 25°C. These strain-at-break tests will give an indication of the flexibility of the BSM.

4.9. Results and conclusions

Results obtained from the laboratory tests will be processed and represented graphically for better analysis.

These laboratory results will then be analysed to obtain an understanding of the shrinkage and flexible behaviour of BSMs. Comparisons between the laboratory test results and hypothesis will also be discussed.

A conclusion will be drawn from all findings obtained during the project and suggestions will be made on any possible improvements for further research.

CHAPTER 5 - Laboratory testing

5.1. Introduction

Shrinkage and strain-at-break tests were conducted on specimens made from R35-materials, which were stabilised with either bitumen emulsion or foamed bitumen.

The designed shrinkage testing procedures of these materials as well as the strain-at-break testing procedure will be described thoroughly.

5.2. Testing methods

5.2.1. Shrinkage testing methods

Beam and cylindrical shrinkage testing methods along with compaction moulds and testing apparatus have been designed for the purpose of this project.

These two methods will be investigated in parallel to determine whether new specimen configurations could overcome some of the difficulties with friction and crack development during material shrinkage.

The shrinkage methods itself were designed to be simple and repeatable. The moulds have been designed for easy and fast removal of the specimens without causing damage to the specimens. All drawings of the test apparatus and compaction moulds are provided in Appendix A.

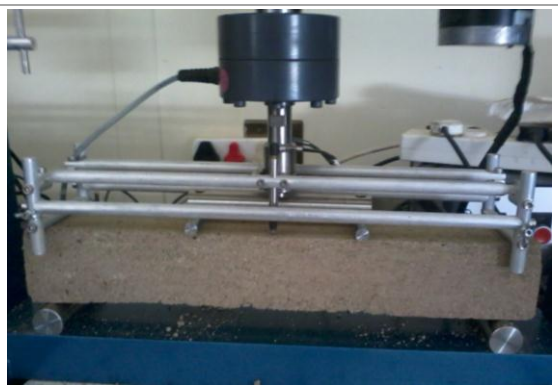
5.2.2. Monotonic strain-at-break testing method

The strain-at-break parameter of materials is usually determined using a 4-point-beam test, but for this project the new monotonic equipment developed by the CSIR will be used (see section 5.6).

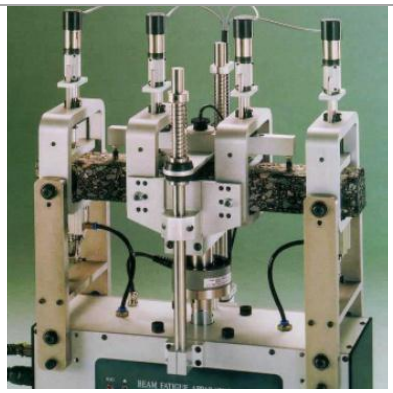
The differences between these two testing methods are as follows:

- The equipment developed by the CSIR is monotonic and the 4-point-beam test is dynamic (allowing movement up and down).
- The monotonic test is simply supported whilst the 4-point-beam test is partially fixed (with clamps) with clamps, indicated in Figure 5.1.

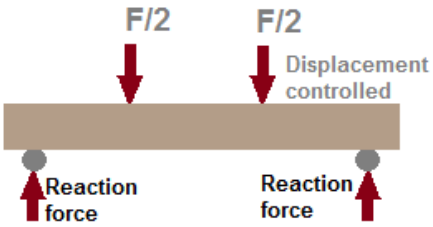
- Two LVDTs is mounted on the CSIR equipment for monotonic displacement measurements. The 4-point-beam apparatus on the other hand has LVDTs mounted on a mini beam for displacement measurements.
- The displacement rate is controlled through a mechanical worm gear in the monotonic beam test, whereas the strain rate is controlled pneumatically in the 4-point-beam test. The monotonic test is thus a displacement controlled test and the 4-point-beam test is a constant strain test.



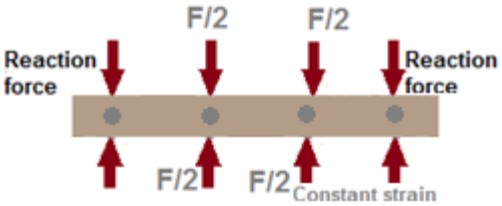
5.1.a) Monotonic equipment



5.1.b) Four-point-beam equipment



5.1.c) Schematic setup of monotonic equipment (Specimen is simply supported).



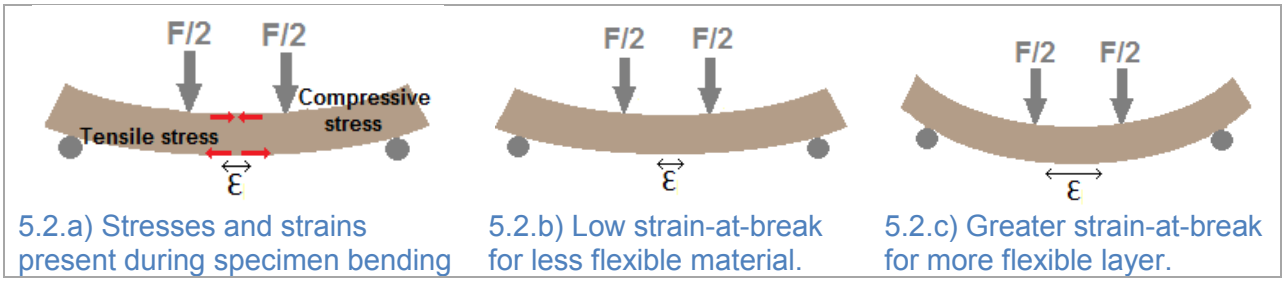
5.1.d) Schematic setup of four-point-beam equipment (Specimen is partially fixed).

Figure 5.1 Monotonic and four-point-beam equipment setup.

During the monotonic beam test, small increments of displacements are applied to the specimen in one direction by the mechanical worm gear and the displacement experienced with time is measured. This allows the horizontal, mid-span strain in the bottom of the beam, to be determined. In contrast thereto, during the 4-point-beam test, the strain during each dynamic load is kept constant requiring the applied stresses to be reduced with time by the controller as the material fatigues. The 4-point-beam test can also apply monotonic loading, but the clamped end-supports are not simple-supports; so some shear stresses are imposed by the clamping effects. The monotonic testing apparatus has “frictionless” simple-supports and can therefore

generate a more accurate strain-at-break evaluation than the 4-point-beam apparatus, making it the better choice for this specific project. If the fatigue of the material needs to be investigated the 4-point- beam test must be used, since this property cannot be determined with the one-directional monotonic beam test.

As the force is applied during the monotonic flexural test, the specimen bends, causing compressive stresses to develop at the top of the beam and tensile stresses to develop at the bottom of the specimen (Figure 5.2). The tensile stresses cause strains to develop at the bottom of the beam. The strain a material can withstand during bending, before ultimate breaking point, is known as the strain-at-break and is determined with the monotonic beam test. The strain-at-break parameter will therefore give an indication of the flexibility of a material. A greater strain-at-break value, indicates a material with greater flexibility as illustrated in Figure 5.2.b and 5.2.c.



5.2.a) Stresses and strains present during specimen bending

5.2.b) Low strain-at-break for less flexible material.

5.2.c) Greater strain-at-break for more flexible layer.

Figure 5.2 Strains and stresses caused during specimen flexure.

5.3. Apparatus

The apparatus needed for the entire testing procedure is provided below:

1) **Specimen production:**
    a. Measuring apparatus (bowls, scales, ext.)
    b. Riffel – apparatus
    c. Draught oven with a temperature range from 25° C to 100° C.
    d. Foam – plant with pug mill
    e. Vibratory hammer
    f. Mould oil
    g. Cylindrical moulds (**specimen dimensions:** $\emptyset = \mathbf{100}\ \boldsymbol{mm},\ \boldsymbol{h} = \mathbf{300}\ \boldsymbol{mm}$)
    h. Rectangular moulds (**specimen dimensions:** *Length* = ***470mm***, *Width* = ***75mm***, *Height* = ***75mm***) [Appendix A provides more information on moulds]
    i. Room with regulated ambient temperature (25 °C).

2) **Shrinkage testing:**
    a. Testing frame for both beam and cylindrical specimens (refer to Appendix A).
    b. Draught oven with a temperature range from 25° C to 100° C and fitted with a temperature gauge for temperature measurements during the experimentation.
    c. 6 LVDTs (LVDTs with -/+ 0.1 mm sensitivity and preferably 1 mm measurement)
    d. 3 Dial gauges
    e. Teflon sheet
    f. Epoxy glue with setting time less than two minutes.
    g. Spider 8, along with a computer and data capturing program (Catman was used in this project) to capture test results.

3) **Strain-at-break testing**
    a. Strain-at-break testing apparatus (gear driven for constant displacement). See Section 5.2.2.
    b. 2 LVDTs (LVDTs with -/+ 0.1 mm sensitivity and preferably 1 mm measurement)
    c. Computer with a data capturing program to obtain test results.

5.4. Apparatus preparation

5.4.1. Testing equipment

The entire shrinkage testing frame (testing apparatus) was conditioned at the test temperature of 40°C, in a draught oven, for at least 4 hours before specimens were placed in the testing position.

The strain-at-break testing apparatus was placed in a room with a controlled ambient temperature (25°C) to ensure that the apparatus was conditioned at the testing temperature of 25°C when tests were conducted.

5.4.2. Moulds

Prior to the compaction of beam and cylindrical specimens, mould oil (or non-stick spray) was spread in the moulds to ensure that the specimen would not stick to the mould during de-moulding.

5.5. Specimen production

5.5.1. Material preparation

The material gradation, OMC and density of the R35-material have been determined during the first phase of the greater R35 project, as described in Chapter 3.

Three specimens were tested simultaneously (Chapter 5.6) and for this reason the materials needed for all three specimens were mixed together in one batch. Mixture component amounts added to each batch were calculated by mass.

5.5.1.a) Aggregates

The amount of aggregates based on the maximum dry density (MDD) of the material as determined during the preparation stadium (first phase), was calculated for each batch mixture. A good aggregate packing was obtained by only adding material passing P19.00mm for specimen production. To obtain a good representation of the determined gradation (Chapter 3) for each batch, a riffle apparatus was used to divide the material into calculated amounts needed for each batch. Keep in mind that an extra amount of material should be added to each batch for spillage during mixing.

Each weighted batch material was heated at 30°C (the ambient mixing temperature (Asphalt Academy, 2009) for BSMs) in a draught oven, for at least 4 hours to ensure sufficient mixing of bitumen through the aggregates.

5.5.1.b) Other mixture components

The required cement content is calculated by mass of the aggregates, for both the bitumen emulsion and foamed bitumen mixtures.

Bitumen emulsion mixtures (BSM-emulsion)

Bitumen emulsion contains both bitumen and water. For the emulsion mixtures, the amount of water needed was thus calculated based on the OMC of the natural aggregates (by mass) taking into account the material's hydroscopic moisture content as well as the water in the emulsion (60/40 bitumen emulsion was used for this project). The amount of binder added to the mixture was also calculated by mass of the aggregates, whilst taking into account the binder to water ratio of the bitumen emulsion.

Foamed bitumen mixtures (BSM-foam)

The amount of water needed for the foamed bitumen mixtures was determined based on the OMC, taking into account only the hygroscopic moisture content. Since the bitumen (70/100 pen bitumen was used for this project) used for foaming is pure bitumen, the amount of bitumen needed was calculated by mass of aggregates.

5.5.2. Mixing

Since both bitumen emulsion and foamed bitumen were used for the specimen production, two different mixing methods were used.

5.5.3.a) Bitumen emulsion

The amount of aggregate materials needed, depending on the specimen type (beam or cylinder), were added to a mixer. Mixing can be done either by hand or by using a mixer.

The required cement content was added to the dry aggregates first and mixed to ensure even distribution of cement within the material before the water was added. After the cement was distributed in the aggregate material, the required water was added and mixed until the water was uniformly distributed within the mixture. Lastly the measured amount of bitumen emulsion was added to the mixture and thoroughly mixed.

In the case of cylindrical specimens, the mixed batch of BSMs was divided into 15 bags of equal amounts, 5 for each specimen. The bags were then closed to ensure no moisture evaporation before compaction. The batch mixed for beam specimens was divided into 9 bags of equal amounts, 3 for each specimen.

5.5.3.b) Foamed bitumen

The aggregate material was added to a pug-mill mixer, which is used along with the foam plant.

The required cement content was added to the dry aggregate first and mixed to ensure the cement is evenly distributed within the material before adding the water. After the cement was distributed in the aggregate material, 80% of the required water was added and mixed. The required amount of foamed bitumen was then sprayed into the mixture by the foam plant while being mixed. The additional 20% of water was then added to the mixture and mixed thoroughly.

In the case of cylindrical specimens, the mixed batch of BSMs were divided into 15 bags of equal amounts, 5 for each specimen and closed to ensure no moisture evaporation before compaction. The batch mixed for beam specimens was divided into 9 bags of equal amounts, 3 for each specimen.

NOTE: *Since shrinkage starts immediately after mixing, the time delay between mixing and placing the compacted specimens in the rigid frame (testing apparatus), should be kept to a minimum.*

Time delay between mixing and compaction of all three specimens is recommended as 15 minutes for bitumen stabilised materials and 5 minutes for cement stabilised materials.

5.5.3. Compaction

Beam specimens as well as cylindrical specimens were compacted for testing. Both these specimens were compacted using split moulds to ensure that they were removed as fast and easy as possible without causing any damage.

5.5.3.a) Beam compaction

The dynamic compaction of MOD AASHTO was used to compact the **beam specimens**. The specimens (**dimensions:** *Length* = ***470mm***, *Width* = ***75mm***, *Height* = ***75mm*****)** were compacted by hand, in a split mould (Figure 5.3) by using a 5kg MOD AASHTO hammer (Appendix A provides more detail on the designed beam split mould).

Figure 5.3 Beam split mould, designed for fast and easy specimen removal.

The compaction procedure was as follows:

1) The marked compaction split mould (Figure 5.4) was placed on a concrete floor and mould oil was spread in the mould to ensure the specimen would not stick to the sides.

NOTE: On the compaction mould, 15 positions each 30 mm apart, was marked to indicate where hammer blows should be applied.

2) Each beam specimen has three layers. The material from one bag (material for first layer), was evenly spread into the beam mould. To minimize the corner effect during compaction ensure that the material is spread towards the corners.

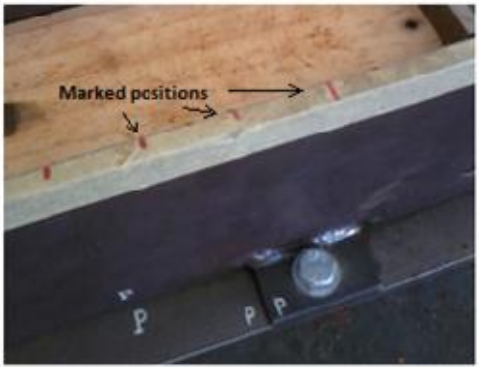


Figure 5.4 Marked positions on the beam split mould.

3) The hand MOD hammer was positioned at the first reference point against one side of the mould and allowed to fall freely from its maximum height. The hammer was then moved to the next adjacent reference point and allowed to fall again. This process was repeated for all points (total 15) along one side and repeated along the opposite side, to give a total amount of 30 blows on the first layer. A repeat of this process was done on this layer to give a total of 60 blows on the first layer.

   The same process was followed for the second and third layer.

4) Lastly a plate was placed on top of the specimen (that fits perfectly in the mould) and one last compaction round (of 30 blows) was given to even the surface and ensure MOD AASHTO compaction.

5.5.3.b) Cylindrical compaction

A cylindrical split mould (Figure 5.5) was used to compact a cylindrical specimen (**dimensions:** $\emptyset = \mathbf{100}\ \boldsymbol{mm}$, $\boldsymbol{h} = \mathbf{300}\ \boldsymbol{mm}$) with a vibratory compaction hammer. (Appendix A provides more detail on the designed cylindrical split mould).

Each cylindrical specimen was compacted in 5 layers as follows:

1) The material from one bag (material for first layer) was poured into the mould and compacted with the vibratory compactor for a predetermined compaction time, to achieve 100% Modified AASHTO density.

2) A scarifier tool was used to roughen the interface between each compacted layer for good layer bonding.

3) The remaining four layers were compacted in the same manner. A total of three specimens were made from each mixture.

Figure 5.5 Cylindrical split mould, designed for fast and easy specimen removal.

Specimens tested for shrinkage were de-moulded and tested immediately after compaction. As mentioned earlier, specimens start shrinking immediately after compaction and therefore the time between compaction and testing should be kept to a minimum.

All specimens that were tested for strain-at-break were de-moulded and cured for 28 days (Asphalt Academy, 2009, p. 38) before testing.

5.6. Shrinkage testing

All shrinkage test procedures were designed together with Professor Kim Jenkins and Alex Mbagara at the University of Stellenbosch. All shrinkage tests have been conducted at a realistic field temperature of 40°C.

5.6.1. Beam shrinkage

The apparatus designed for testing beam shrinkage can only test one beam specimen at a time. Three of these rigid frames were produced to be able to test three specimens simultaneously, which shortens testing times.

The compacted specimens were de-moulded in a controlled environment where after the weight and dimensions were recorded. Thereafter each beam specimen was placed on a Teflon sheet to provide a frictionless surface during shrinkage testing. Perspex squares (with springs) were glued to the rear ends of each beam specimen and the position of the LVDTs were adjusted to allow for the full measurement range during shrinkage (Figure 5.6).

Each specimen was then immediately placed into their individual conditioned testing apparatus (rigid frame) inside of the draught oven. They were placed in a manner that leaves the specimens undisturbed (illustrated in Figure 5.7).

Caution: *Ensure that the entire frame is conditioned beforehand at the correct testing temperature (40°C).*

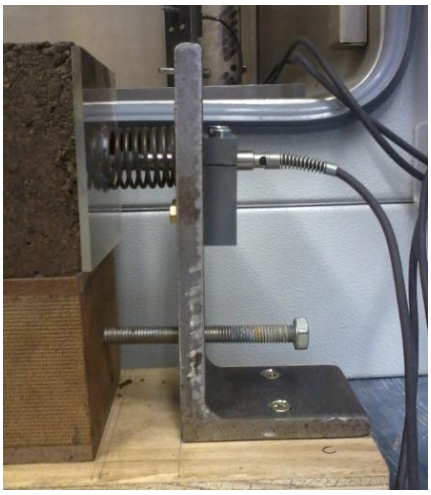


Figure 5.6 Positioning of Perspex squares and LVDTs before testing.

Figure 5.7 Three rigid frames used to test three beam specimens simultaneously.

Ensure that the actual temperature of the draught oven is the desired temperature of 40°C and start-up Catman (or a similar program) to record the LVDT measurements. Testing started immediately after all of the specimens have been placed in their testing frames and the initial measurement was recorded at time = 0 hrs.

Shrinkage measurements were recorded at intervals of 0 hrs, 0.5 hrs, 1.5 hrs, 3 hrs, 6 hrs, 12 hrs, 24 hrs, 48 hrs and 72 hrs. The analysis period can be extended up to the point where no further shrinkage occurs if the research project requires it. After 72 hours the final LVDT measurement was recorded and the Catman results were saved in excel.

5.6.2. Cylindrical shrinkage

The frame used to test cylindrical specimens was designed to measure the shrinkage of three specimens simultaneously, which shortens testing time. This was done to ensure consistent shrinkage results for all the repetitions of the specific specimen type. Shrinkage measurements of each specimen were taken with a LVDT as well as a dial gauge for comparative purposes.

The compacted cylindrical specimens were de-moulded in a controlled environment where after the weight and dimensions were recorded. Thereafter, the three specimens were immediately placed in the testing apparatus (rigid frame) in a manner that leaves the specimens undisturbed (Figure 5.8).

Caution: *Ensure entire frame is conditioned beforehand at the correct temperature (40°C) for at least 4 hours.*

Figure 5.8 Rigid frame used for testing three cylindrical specimens simultaneously.

A cylindrical Perspex disk (conditioned at 40°C) was glued to the top of each specimen. This ensured that the LVDT and dial gauge measured shrinkage on an even surface to provide consistent measurements. After these disks were in place, the position of the LVDT's and dial gauges were adjusted to allow for the full measurement range during shrinkage. Testing started immediately after all three specimens have been placed in the testing frame and adjustments made to all of the LVDT's and dial gauges.

Ensure that the actual temperature is at the desired temperature of 40°C and start-up Catman (or a similar program) to record the LVDT measurements. Physical measurements will be required, using the dial gauges. Initial LVDT and dial gauge measurement readings (at time = 0h) were recorded after the specimens had been placed in position.

Shrinkage measurements were recorded at intervals of 0hrs, 0.5 hrs, 1.5 hrs, 3 hrs, 6 hrs, 12 hrs, 24 hrs, 48 hrs and 72hrs for both the LVDTs and dial gauges. The analysis period can be extended up to the point where no further shrinkage occurs, if the research project requires it. After 72 hours the final LVDT measurement was recorded and the Catman results were saved in excel together with the dial gauge measurements.

Note: *At a later stage during this study, a circumferential LVDT apparatus was also used to test the shrinkage in the circumferential direction. Since this was only introduced later on in the project, only a few of these tests were conducted.*

Figure 5.9 indicates the placement of the circumferential LVDT around the cylindrical specimen. The LVDT should be in place before measurements start.

Figure 5.9 Placement of cylindrical LVDT on cylindrical specimen.

5.7. Strain-at-break testing

Specimens were cured for 28 days, then weighed and measured before testing. All strain-at-break tests have been conducted at a temperature of 25°C.

The data capturing computer program was started and the details of that specific specimen were recorded to keep track of the test results. The specimen was then carefully placed in the monotonic strain-at-break testing apparatus, making sure that it was centered with no end hanging over more than the other (Figure 5.10). Adjustments were made before testing to ensure the force was at the zero position (at the top of the beam with no displacement) and that both LVDTs touched the beam's surface.

The strain-at-break tests were then conducted and tests results saved to excel files.

Figure 5.10 Placement of specimen in strain-at-break testing apparatus.

CHAPTER 6 - Shrinkage results

6.1. Introduction

Beam and cylindrical tests were investigated in parallel whilst researching the shrinkage behaviour of BSMs. The objective of this approach was to determine whether new specimen configurations could overcome some of the difficulties with friction and crack development during material shrinkage. Results from both the beam and cylindrical shrinkage tests were collected from the computer software and presented using graphical representations. Differences and similarities between the acquired data are also pointed out.

6.2. Beam shrinkage results

The total shrinkage of the beam specimens were measured using a LVDT on each end of the specimen, as indicated in Figure 6.1. All measurements started at a zero position on the x-axis (indicated by M_1 and M_2 in Figure 6.1). A positive reading in the x-direction (in the relevant direction indicated by an arrow for both x1- and x2-axis in Figure 6.1) indicates swelling and a negative reading indicates shrinkage. Both swelling and shrinkage took place in the axial direction during shrinkage testing of this investigation and was therefore termed "**axial length change**" on all graphs during this project as appose to shrinkage. The beam shrinkage or axial length change were calculated using Equation 6.1.

Figure 6.1 LVDT placement during beam shrinkage testing.

Axial length change = (LVDT 1 measurement) + (LVDT 2 measurement) [Equation 6.1]

= mm

All of the presented graphs show total axial length change (change in x-direction) of each beam specimen with the elapse of time. Shrinkage (or axial length decrease) is indicated by a negative measurement on all graphs and swelling (or axial length increase) is indicated by a positive measurement.

6.2.1. Results

The shrinkage measurements of each specimen type (all 3 specimens tested with the same composition) are compared on one graph to evaluate the consistency of the test results. The consistency of the results can be improved by increasing the amount of each specimen type tested.

6.2.1.a) Shrinkage of un-stabilised R35-material

Shrinkage results from the beam specimens with no added binder or cement indicate the shrinkage trend for the R35-material itself (Figure 6.2). Initially shrinkage is observed and continues to shrink until the end of the testing period (behaviour consistent between similar types of specimens). Note that the axial length change observed (shrinkage or swelling) is small in comparison to the specimen size.

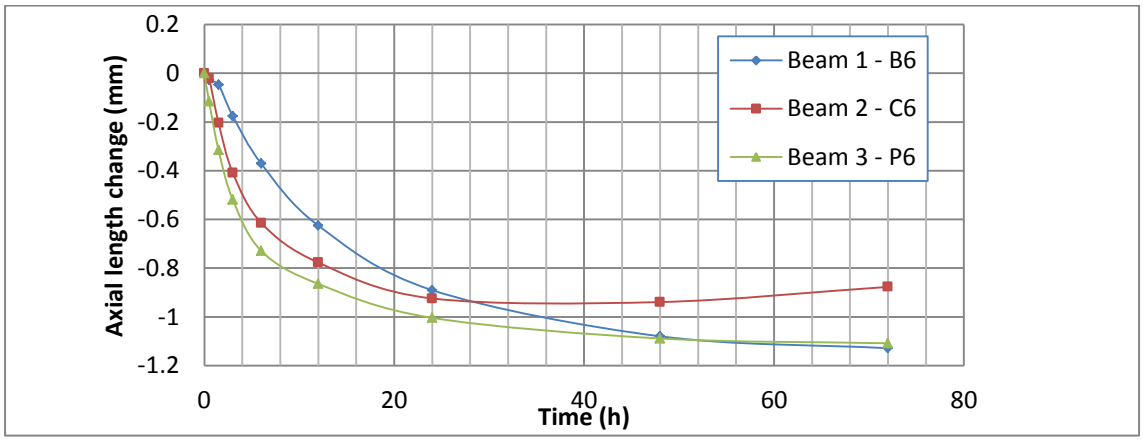


Figure 6.2 Shrinkage of beam specimens with 0% bitumen emulsion and 0% cement.

6.2.1.b) Shrinkage of bitumen stabilised R35-material

The typical shrinkage behaviour of all the bitumen stabilised beam specimens are similar, as illustrated by both Figure 6.3 (specimens containing 2.4% net bitumen emulsion and

2% cement) and Figure 6.4 (2.4% foamed bitumen and 2% cement). Shrinkage test results of all other tested specimens are provided in Appendix B.

Even though the tests results presented in Figure 6.3 and Figure 6.4 show good consistency between trends of similar types of specimens, not all tests results show such good consistency (see Appendix B). Nevertheless the results still provide a good idea of the behavioural trends of the R35-material.

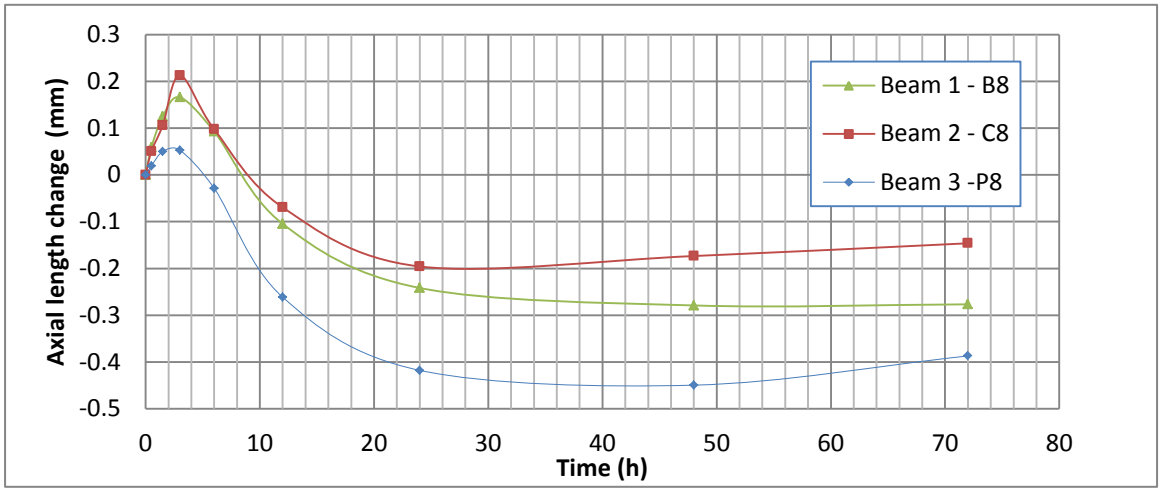


Figure 6.3 Shrinkage of beam specimens with 2.4% net bitumen emulsion and 2% cement.

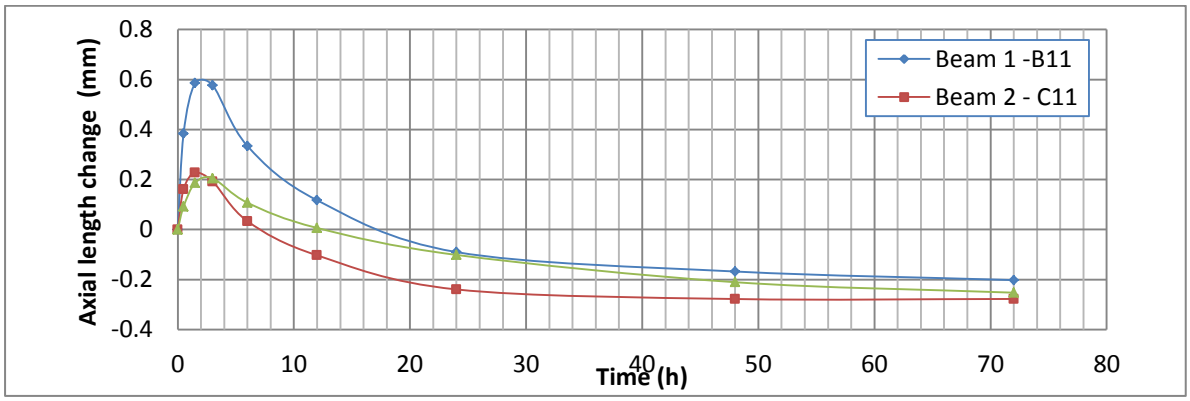


Figure 6.4 Shrinkage of beam specimens with 2.4% foamed bitumen and 2% cement.

It should be noted that the amount of axial length change (shrinkage or swelling) is extremely small, but a trend can still be seen in the behaviour of the materials (Figure 6.3 and Figure 6.4). Initially a slight length increase (swelling) for a short time period is

observed followed by a reduction in length (shrinkage) for both emulsion and foam stabilised specimens. Thereafter shrinkage continues until the length change stabilises to the end of the testing period.

6.2.2. Influence of additives

The average axial length change (of the three specimens with the same type) at three different stages (times) during the shrinkage tests was studied closely to better understand the influence of certain factors on the shrinkage of bitumen stabilised beam specimens. The time periods investigated were chosen where the length change has shown to be the most significant for all of the tested beam specimens.

6.2.2.a) Bitumen (emulsion and foamed)

Containing 1% Cement:

Specimens containing 1% cement were stabilised with either 0.9% net bitumen emulsion, 2.4% net bitumen emulsion or 2.4% foamed bitumen.

- An increase in bitumen emulsion provides an increase in swelling during the initial stages of the testing period, as indicated at the time period of 1.5 h in Figure 6.5.
- Increased bitumen emulsion quantities reduces shrinkage (as seen in the time period after initial swelling; Figure 6.5).
- Specimens stabilised with 2.4% foamed bitumen show higher swelling and shrinkage than 2.4% net bitumen emulsion stabilised specimens (Figure 6.5).

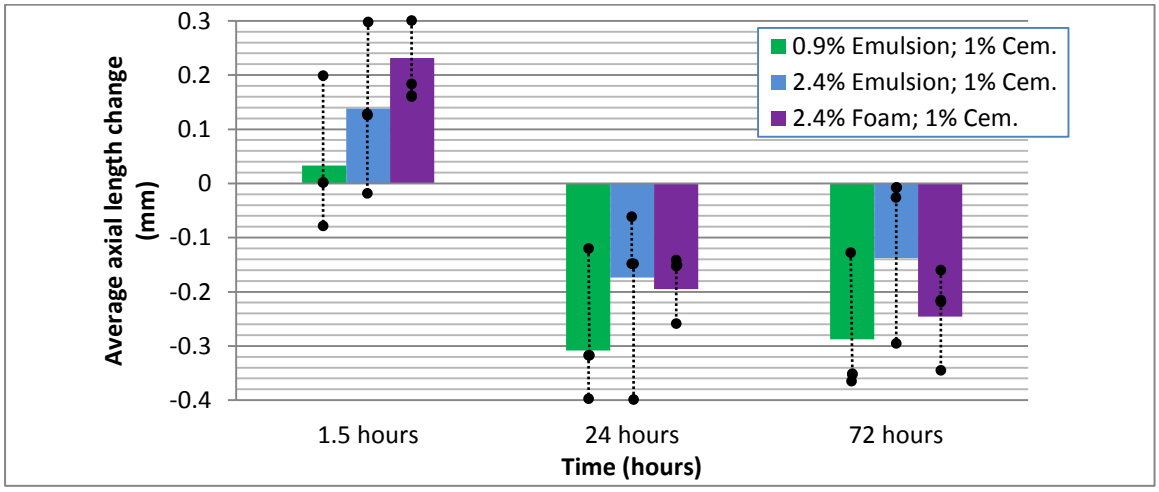


Figure 6.5 Shrinkage of beam specimens containing 1% cement.

Containing 2% Cement:

Beam specimens containing 2% cement, all contain 2.4% net bitumen, but was stabilised with either bitumen emulsion or foamed bitumen.

- Behavioural trends between specimens of the same type that is stabilised with foamed bitumen are more consistent than that of specimens stabilised with bitumen emulsion (Appendix B).
- Foamed bitumen stabilised specimens show higher swelling and lower shrinkage values than specimens stabilised with bitumen emulsion (Figure 6.6).

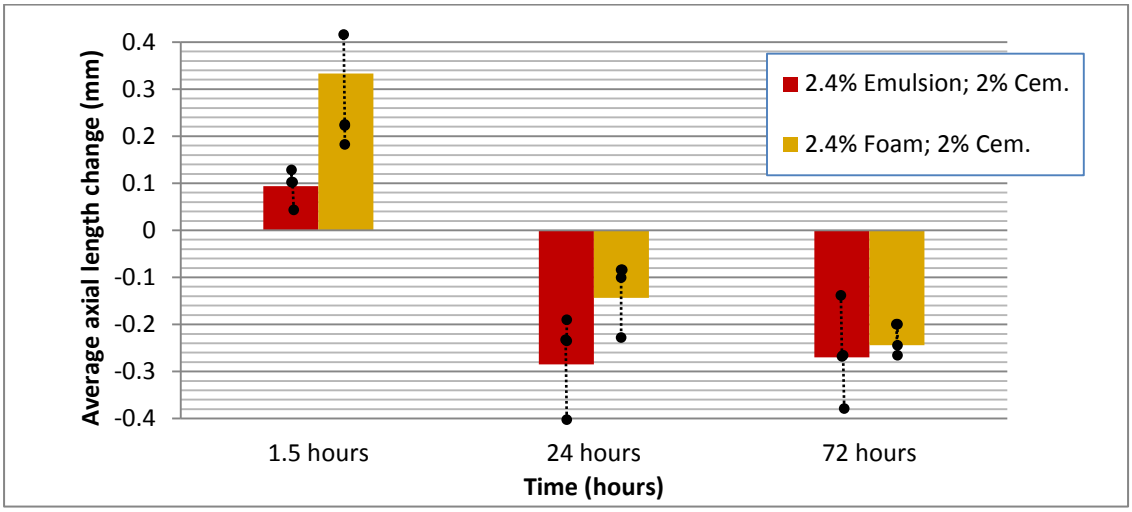


Figure 6.6 Shrinkage of specimens containing 2% cement stabilised with different bitumen types (2.4%).

6.2.2.b) Cement

Stabilised with bitumen emulsion:

The influence of the cement content on the shrinkage of the bitumen emulsion stabilised R35-materials have been tested by increasing the cement content from 1% to 2%.

- An increase in cement content, indicate a decrease in swelling (at 1.5 h) and an increase in shrinkage, as illustrated in Figure 6.7.

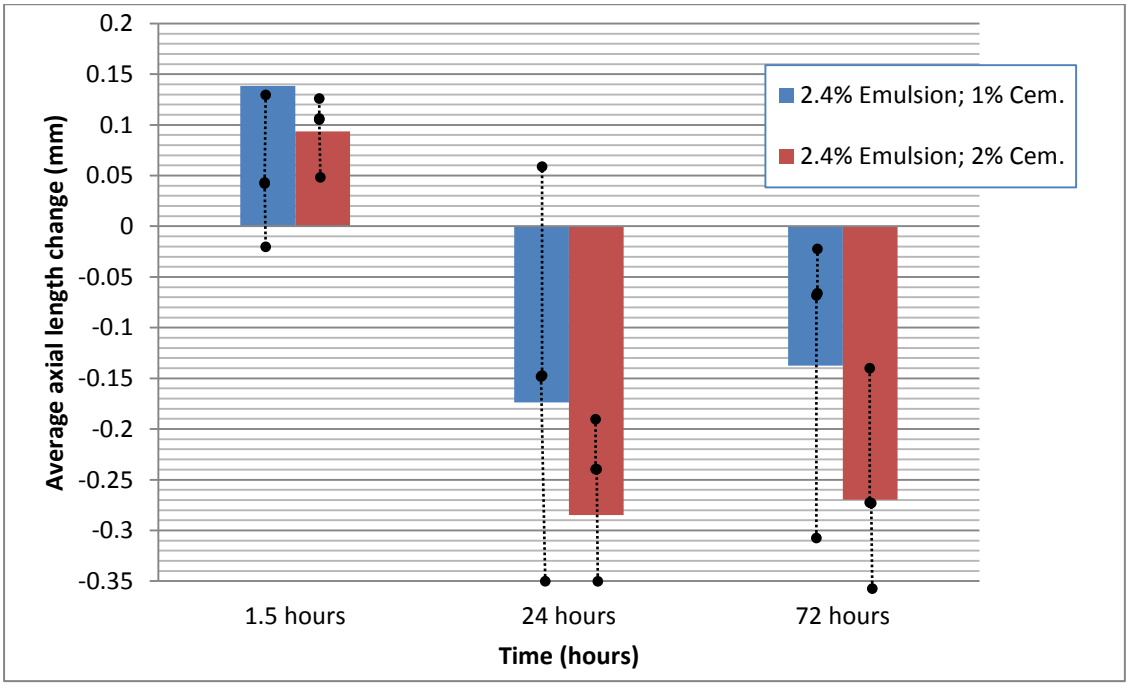


Figure 6.7 Influence on shrinkage of emulsion stabilised materials with an increase in cement.

Stabilised with foamed bitumen

The influence of the cement content on the shrinkage of the foamed bitumen stabilised R35-materials have been tested by increasing the cement content from 1% to 2%.

- An increase in cement content indicates an increase in swelling (at the start of the testing period) and a decrease in shrinkage, as illustrated by Figure 6.8.

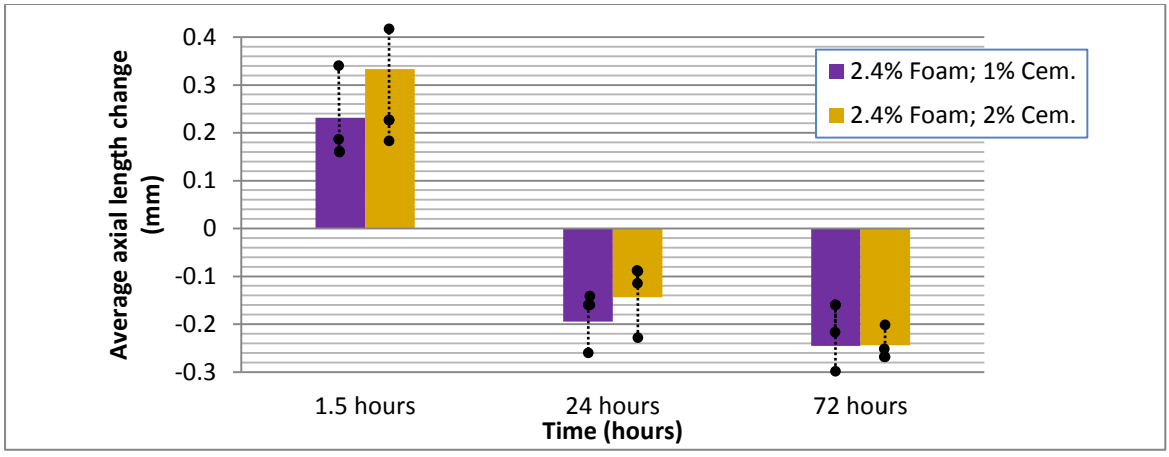
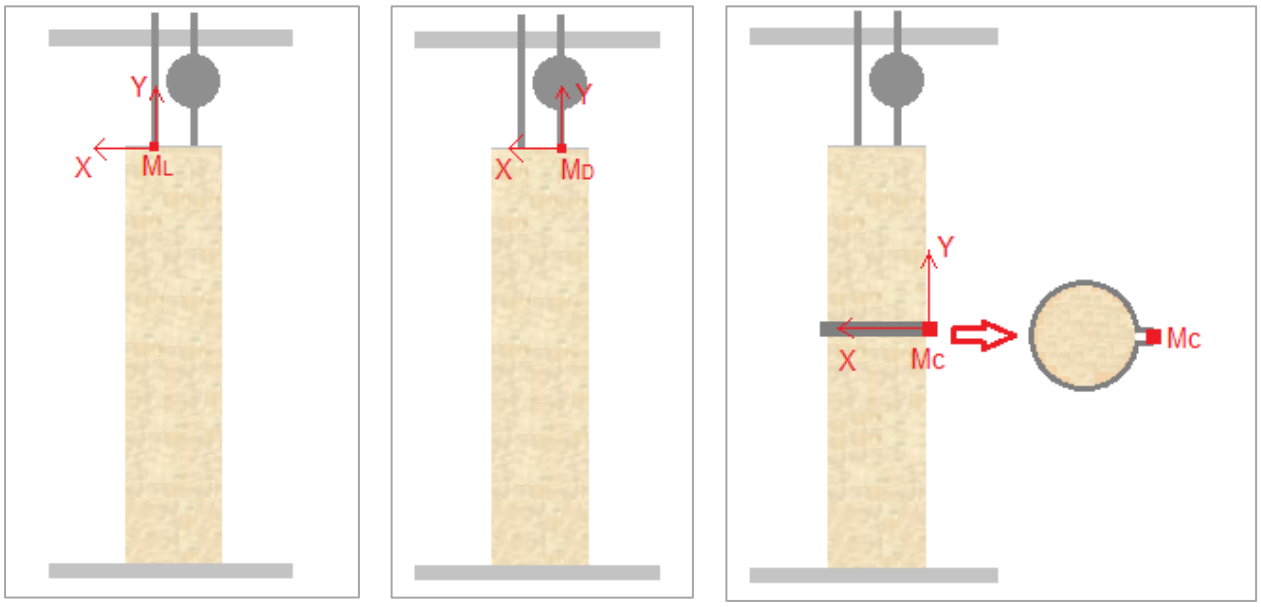


Figure 6.8 Shrinkage of foam stabilised materials with an increase in cement content.

6.3. Cylindrical shrinkage results

The shrinkage of the cylindrical specimens was measured from above using both a LVDT and a dial gauge as indicated in Figure 6.9 (only measuring form one side). All measurements started at a zero position on the y-axis (indicated by M_L and M_D in Figure 6.9). A positive reading in the y-direction (in the relevant direction indicated by an arrow for both y1- and y2-axis in Figure 6.9) indicates swelling and a negative reading indicates shrinkage. Both swelling and shrinkage took place in the axial direction (y-axis) during shrinkage testing of this investigation and was therefore termed "**axial length change**" during this project as appose to shrinkage on all graphs. The cylindrical shrinkage or axial length change are equal to the measurements obtained during testing.

The circumferential LVDT on the other hand is placed around the specimen and measures the change in circumference of the specimen (measurement takes place at position marked Mc in Figure 6.9.c). An increase in measurements once again indicates swelling and a decrease indicates shrinkage for the "**circumferential change**".



6.9.a) LVDT 6.9.b) Dial gauge 6.9.c) Circumferential LVDT

Figure 6.9 LVDT, dial gauge and circumferential LVDT placement during cylindrical shrinkage testing.

Data from both dial gauge measurements and LVDT measurements of the axial length change (measured vertically) for each tested cylindrical specimen is graphically represented for

evaluation. Only a few specimens were tested using the circumferential LVDTs and those results were also represented graphically. Shrinkage (or axial length decrease) is indicated by a negative measurement on all graphs and swelling (or axial length increase) is indicated by a positive measurement.

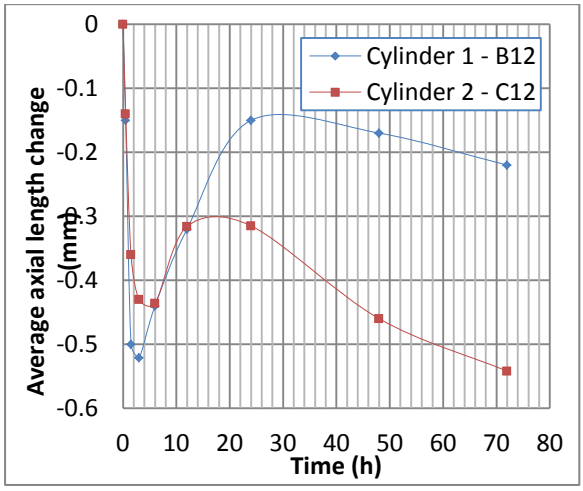
6.3.1. Results

Shrinkage measurements of each specimen type (all 3 specimens tested with the same composition) are compared on one graph to evaluate the consistency of the test results. The consistency of the results can be improved by increasing the amount of each specimen type tested.

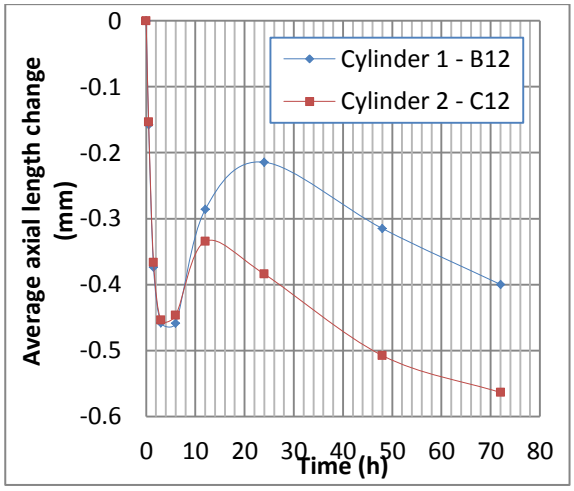
6.3.1.a) Shrinkage of un-stabilised R35-material

Cylindrical specimens containing no cement or bitumen show an initial decrease in length. Thereafter the length increases after approximately 4 hours of testing and decreases once again after 24 hours (Figure 6.10). Although slight differences in trends are observed between the different specimens (with the same composition), the shrinkage trends prove to be consistent between the different types of measuring equipment (dial gauge and LVDT).

Note that the axial length change observed during the testing period is extremely small compared to the specimen height.



6.10.a) Dial gauge shrinkage measurements



6.10.b) LVDT shrinkage measurements

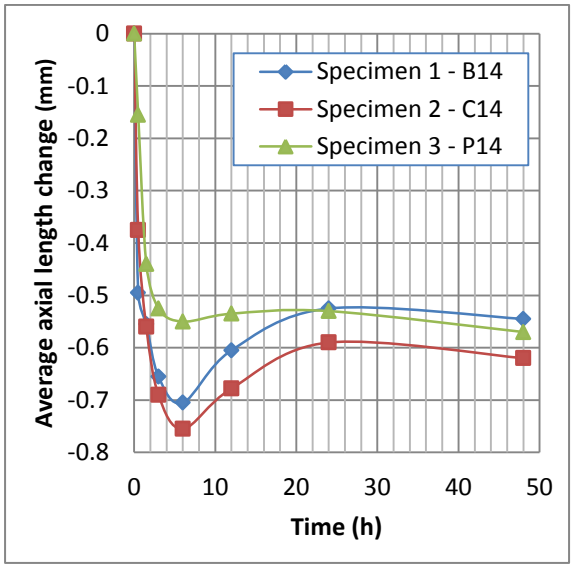
Figure 6.10 Shrinkage of cylindrical specimens containing 0% bitumen and 0% cement.

6.3.1. b) Shrinkage of bitumen emulsion stabilised R35-material

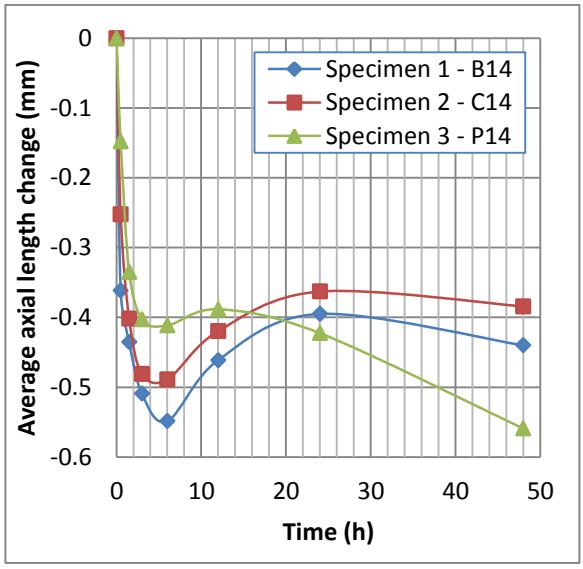
A consistent shrinkage trend is observed in the results of all the tested bitumen stabilised cylindrical specimens. The typical behavioural trend of all the emulsion treated specimens is shown through the results of the cylindrical specimens containing 2.4% net bitumen emulsion and 2% cement (Figure 6.11). All other test results for emulsion treated cylindrical specimen are provided in Appendix B.

An initial length decrease (shrinkage) is observed followed by an increase in length after a short time period. After approximately 24 hours the length slightly decreases (shrinkage) where after the length change eventually stabilises.

The testing period for these specific test specimens (Figure 6.11) was only 48 hours. After testing these specimens the testing period was lengthened to 72 hours, but unfortunately these tests could not be repeated due to the material shortage.



6.11.a) Dial gauge shrinkage measurements



6.11.b) LVDT shrinkage measurements

Figure 6.11 Dial gauge and LVDT shrinkage measurements of cylindrical specimens containing 2.4% bitumen emulsion (net bitumen) and 2% cement.

Circumferential shrinkage of the cylindrical specimens shows different shrinkage behaviour to that of the axial shrinkage. The shrinkage trend observed for the circumferential shrinkage (Figure 6.12) starts with initial shrinkage and continues to shrink until the end of the testing period.

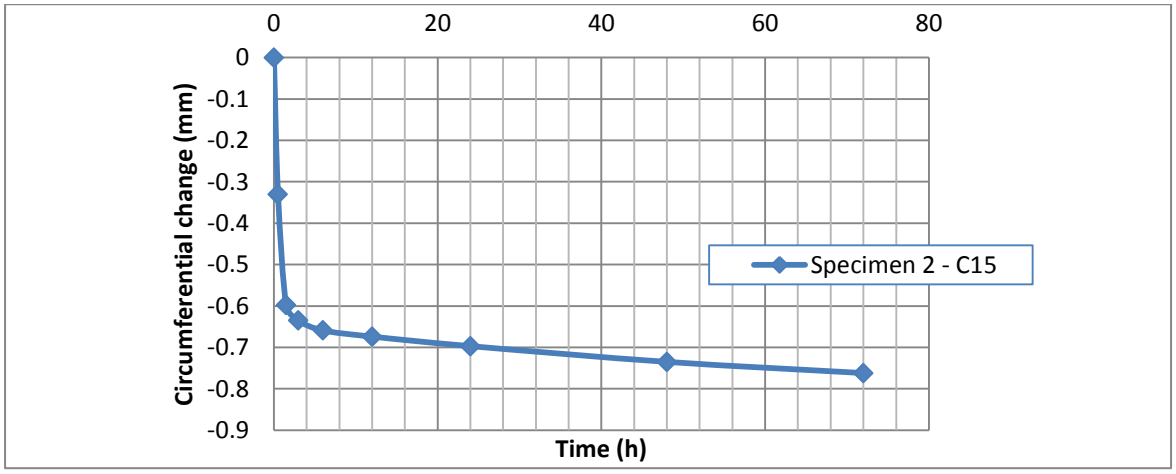
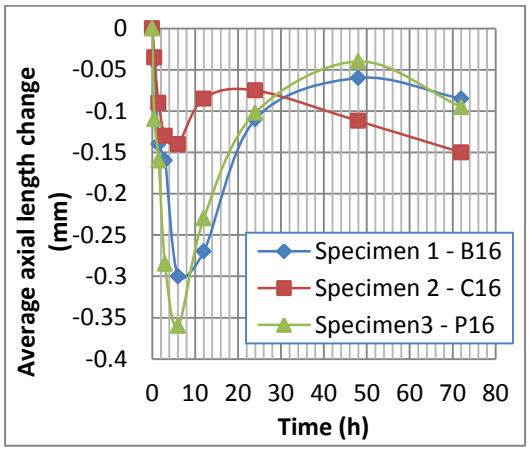


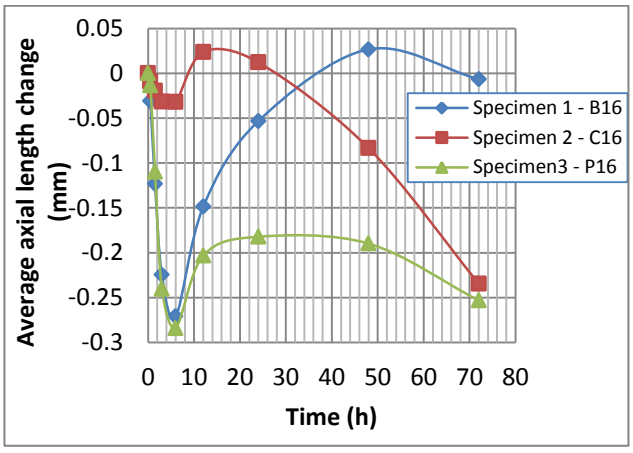
Figure 6.12 Circumferential LVDT shrinkage of specimens containing 0.9% net bitumen emulsion and 1% cement.

6.2.1. c) Shrinkage of foamed bitumen stabilised R35-material

The behaviour of specimens containing 2.4% foamed bitumen and 1% cement (in Figure 6.13) show inconsistent test results between the specimens as well as the shrinkage measuring types (apparatus). Considering specimen 2 as an outlier, the behavioural trend for the remaining specimens (specimen 1 and 2; with similar trends) will by some means be consistent. At the start of the testing period a decrease in length is observed, followed by an increase in length after 6 hours and once again the axial length gradually decreases (± 50 hours of the testing period), for both dial gauge and LVDT test results.



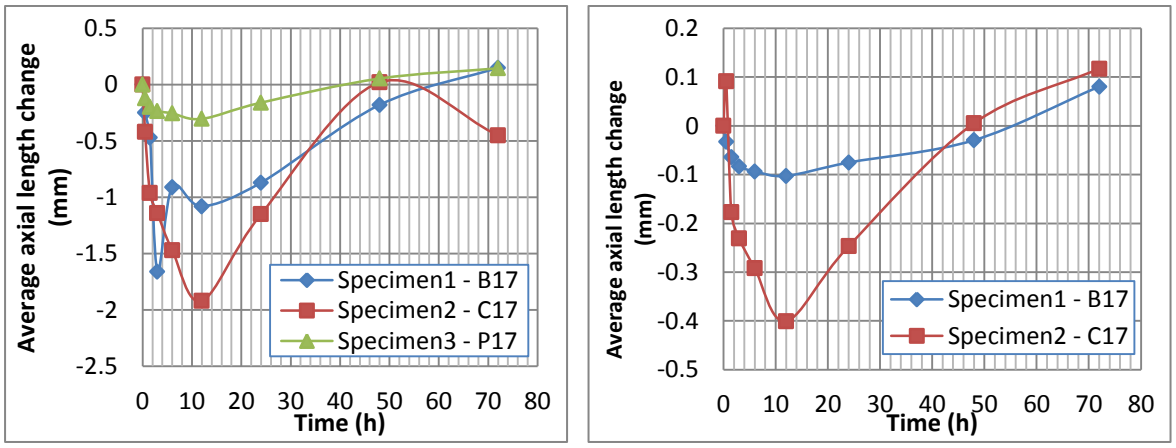
6.13.a) Dial gauge shrinkage measurements



6.13.b) LVDT shrinkage measurements

Figure 6.13 Dial gauge and LVDT shrinkage measurements of cylindrical specimens containing 2.4% foamed bitumen and 1% cement.

Figure 6.14 provide shrinkage test results of specimens containing 2.4% foamed bitumen and 2% cement. Observed trends are slightly different than that of the specimens containing 1% cement (with 2.4% foam). The overall trend observed is a decrease in length followed by an increase in length (after 12 hours).



6.14.a) Dial gauge shrinkage measurements 6.14.b) LVDT shrinkage measurements

Figure 6.14 Dial gauge and LVDT shrinkage measurements of cylindrical specimens containing 2.4% foamed bitumen and 2% cement.

Circumferential shrinkage of the cylindrical specimens once again shows different shrinkage behaviour to that of the axial shrinkage. The shrinkage trend observed (Figure 6.15) for the circumferential shrinkage starts with initial shrinkage and keeps on shrinking to the end of the testing period.

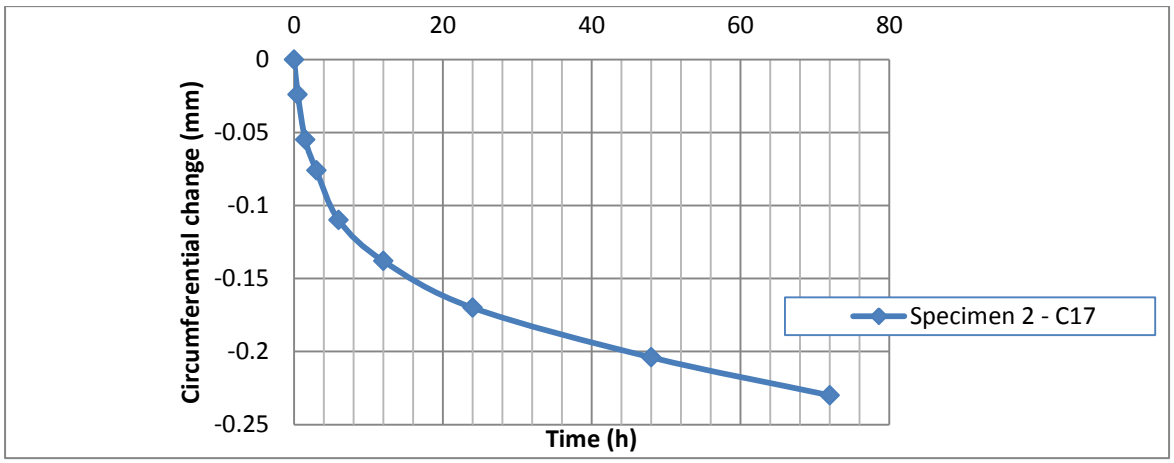


Figure 6.15 Circumferential LVDT shrinkage of specimens containing 2.4% foamed bitumen and 2% cement.

6.3.2. Influence of different components

The average axial length change (of the three specimens with the same type) at different stages (times) during the shrinkage tests was studied closely to better understand the influence of certain factors on the shrinkage of bitumen stabilised cylindrical specimens. These stages are not identical for all of the test specimens and therefore four stages were chosen to incorporate the most significant axial length change for most specimens.

6.3.2.a) Measurement apparatus

Dial gauge and LVDT measurements:

Dial gauge measurements of bitumen emulsion treated specimens are overall consistent with LVDT measurements (typical difference indicated by Figure 6.16). However, measurements from foamed bitumen treated specimens differ significantly between measurement apparatus, especially in the results provided in Figure 6.17.

NOTE: All further comparisons will be made considering only the test results from the LVDT apparatus, since LVDT measurements was taken for the beam specimens. This will provide more accurate comparisons between beam and cylindrical specimens.

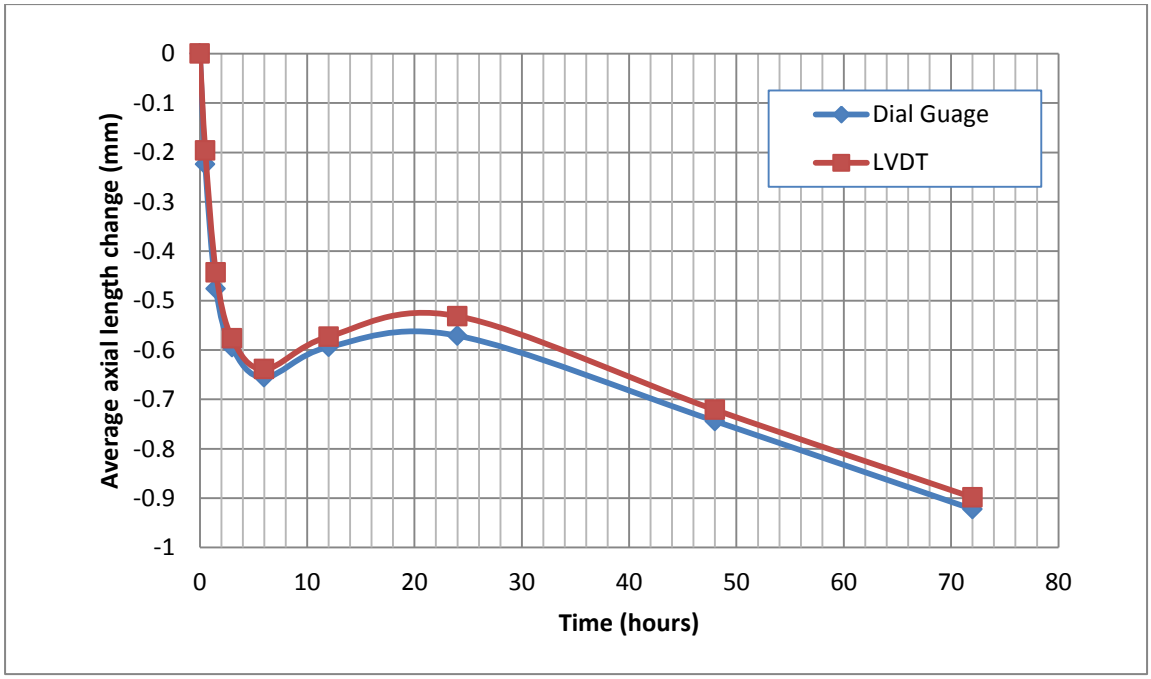


Figure 6.16 Comparison between shrinkage instruments for specimens containing 0.9% net bitumen emulsion and 1% cement.

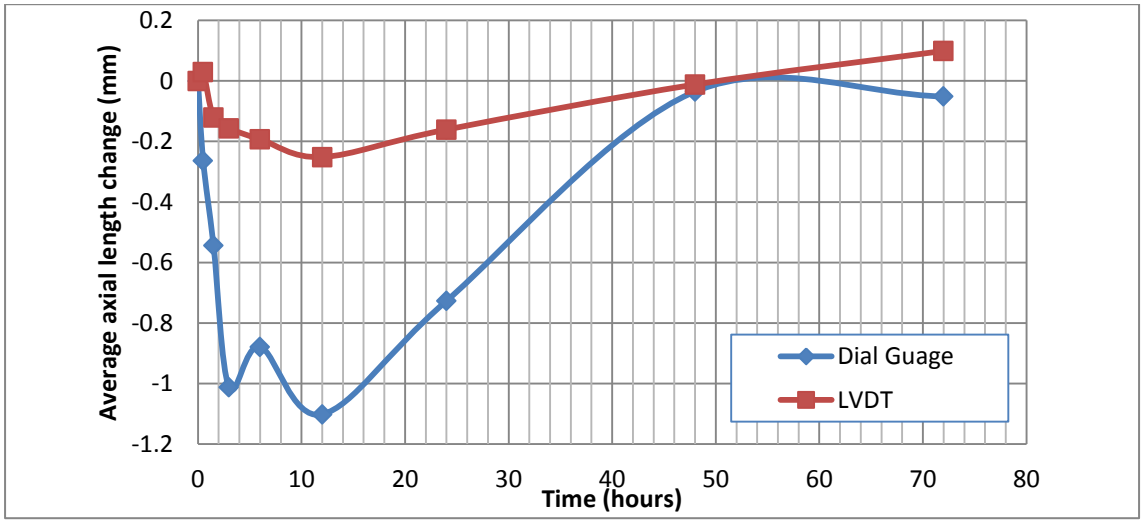


Figure 6.17 Comparison between shrinkage instruments for specimens containing 2.4% foamed bitumen and 2% cement.

Circumferential measurements:

Unfortunately only two circumferential shrinkage tests were conducted and are not truly comparable. Figure 6.18 shows that specimens stabilised with foamed bitumen (and higher foam and cement content) shrink more than specimens stabilised with emulsion (with lower bitumen and cement content). These results are subjected to both the bitumen and cement content of each specimen.

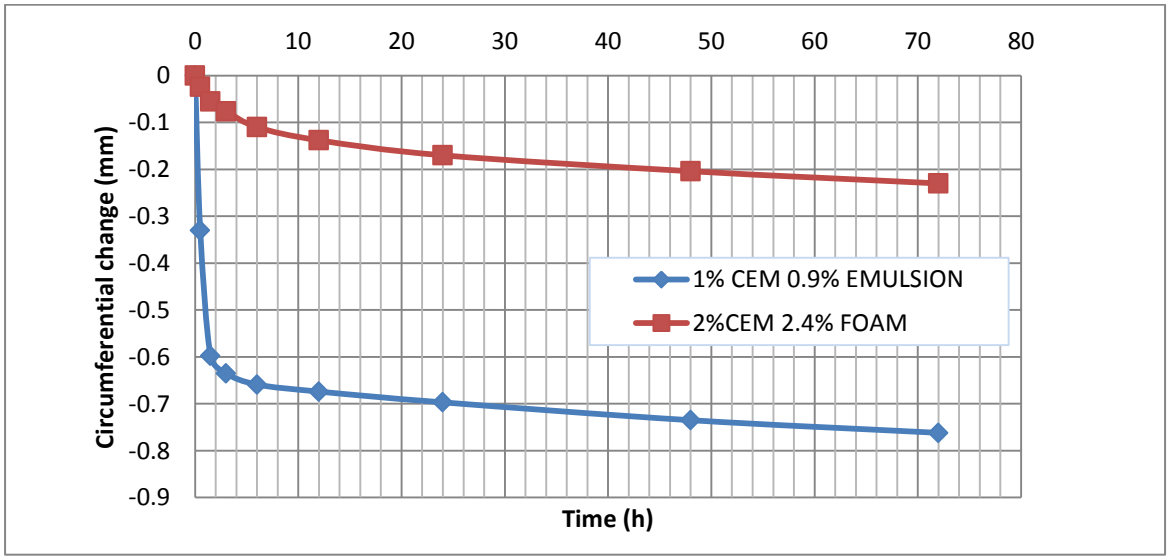


Figure 6.18 Circumferential shrinkage of different tested specimens.

6.3.2.b) Bitumen (emulsion and foamed)

Containing 1% Cement:

Specimens containing 1% cement were stabilised with either 0.9% net bitumen emulsion, 2.4% net bitumen emulsion or 2.4% foamed bitumen.

- An increase in bitumen emulsion provides a decrease in shrinkage and increased swelling (length change) as illustrated in Figure 6.19.
- Specimens stabilised with 2.4% foamed bitumen show less shrinkage and swelling than specimens stabilised with 2.4% bitumen emulsion.

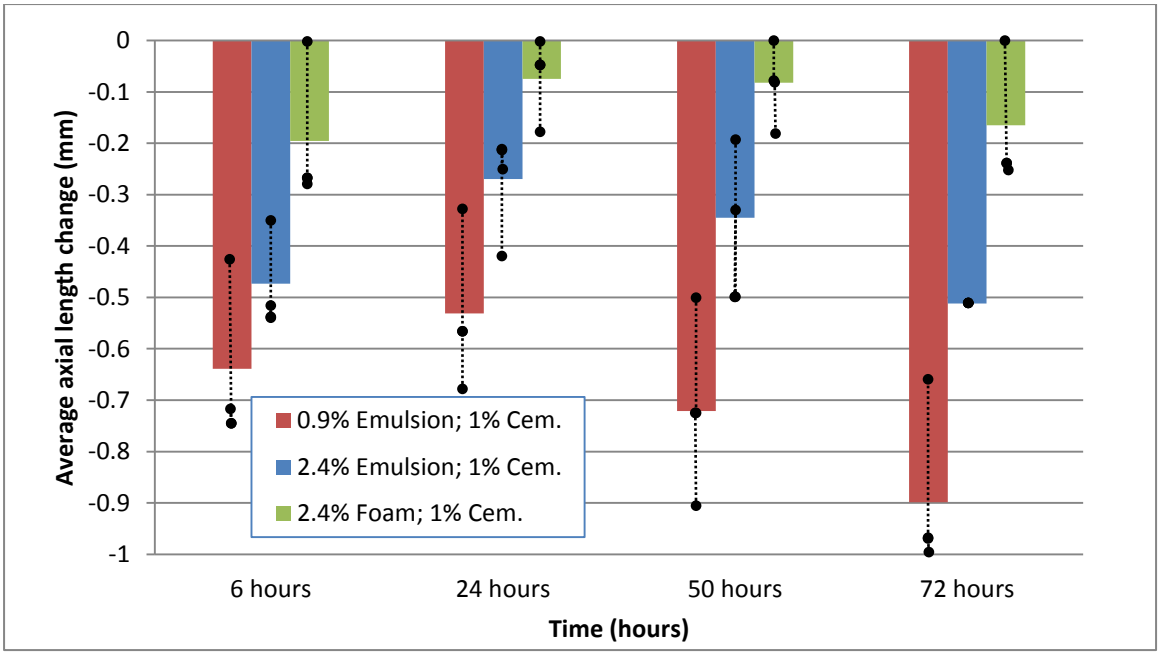


Figure 6.19 Shrinkage of cylindrical specimens containing **1% cement**.

Containing 2% Cement:

Cylindrical specimens containing 2% cement, all contain 2.4% bitumen, but was stabilised with either bitumen emulsion or foamed bitumen.

- Emulsion stabilised materials show greater shrinkage than specimens stabilised with foamed bitumen (Figure 6.20).
- The shrinkage behaviour between the bitumen emulsion and foamed bitumen stabilised specimens show some variation in trend to the end of the testing period.

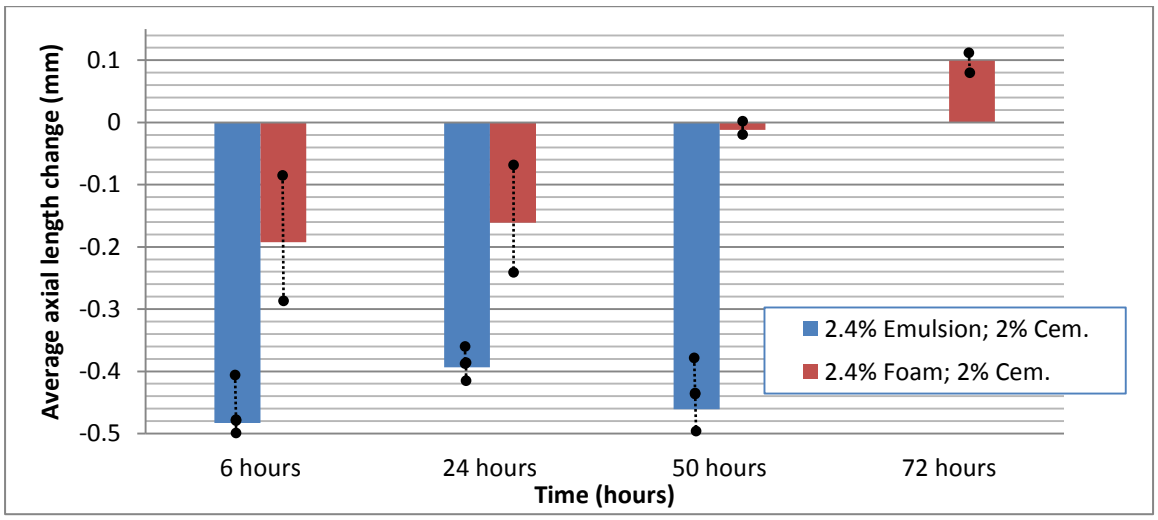


Figure 6.20 Shrinkage of cylindrical specimens containing 2% cement.

6.3.2.c) Cement

Stabilised with bitumen emulsion:

The influence of the cement content on the shrinkage of the bitumen emulsion stabilised R35-materials have been tested by increasing the cement content from 1% to 2%.

- An increase in cement content, in emulsion stabilised materials, indicate an increase in shrinkage and reduced swelling as illustrated in Figure 6.21.

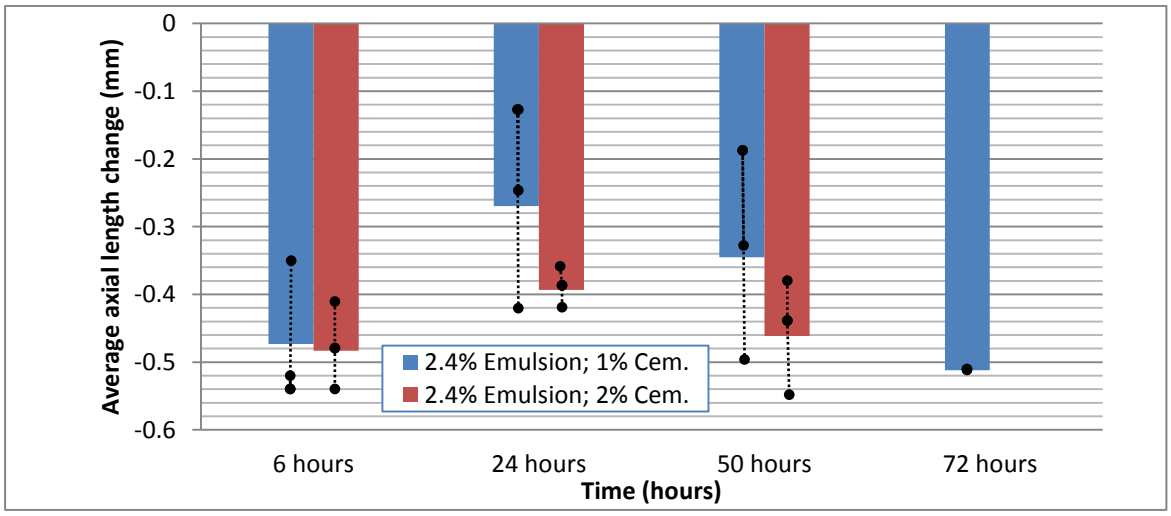


Figure 6.21 Shrinkage of bitumen emulsion (2.4%) treated cylindrical specimens containing different cement quantities.

Stabilised with foamed bitumen:

The influence of the cement content on the shrinkage of the bitumen emulsion stabilised R35-materials have been tested by increasing the cement content from 1% to 2%.

- Foamed bitumen specimens (Figure 6.22) show varying trend results and therefore no credible findings could be made considering the effect of the cement content on foamed bitumen stabilised specimens. An increased number of test specimens could provide more insight on the influence of cement.

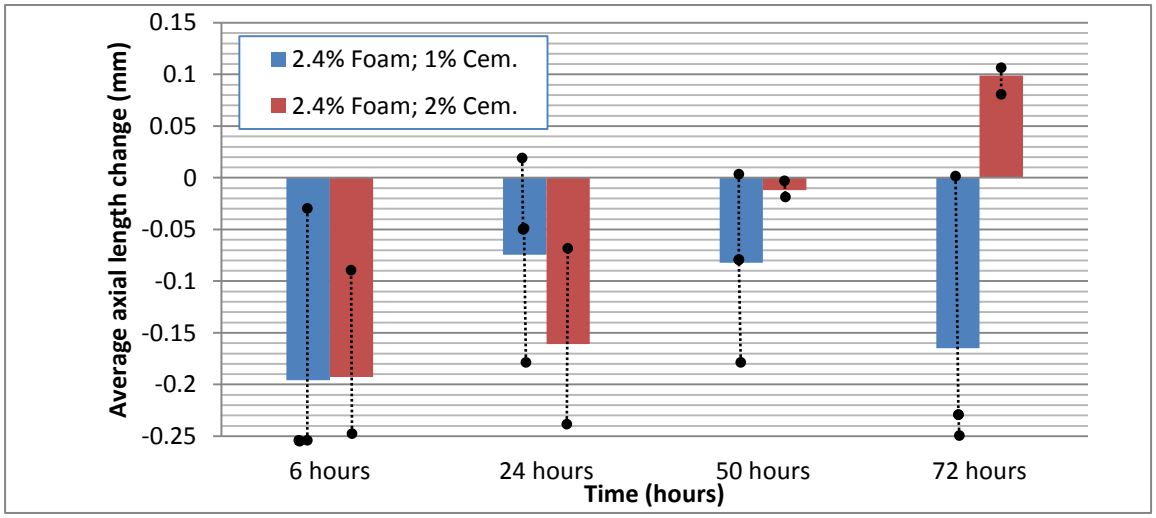


Figure 6.22 Shrinkage of foamed bitumen (2.4%) stabilised cylindrical specimens, containing different cement quantities.

6.4. Comparison between beam shrinkage and cylindrical shrinkage

6.4.1. Shrinkage trends

Shrinkage data of specimens containing 0.9% net bitumen emulsion and 1% cement, provide a good representation of the shrinkage trends for the different testing methods that was recognized for all specimen types during this project (as illustrated in Figure 6.23).

- Beam specimens initially show a slight increase in length (swelling) for a short time period before shrinkage occurs. Cylindrical specimens show initial shrinkage in both the axial direction ("cylinder shrinkage" in Figure 6.23) and the circumferential shrinkage.

- Beam specimens experience shrinkage after the initial swelling and later show a slight increase in axial length to the end of the testing period. Cylindrical specimens show swelling in die axial direction ("cylinder shrinkage") after 6 hours of testing, but shrinkage once again takes place after 24 hours. Circumferential shrinkage shows no swelling during the entire testing period (Figure 6.23).

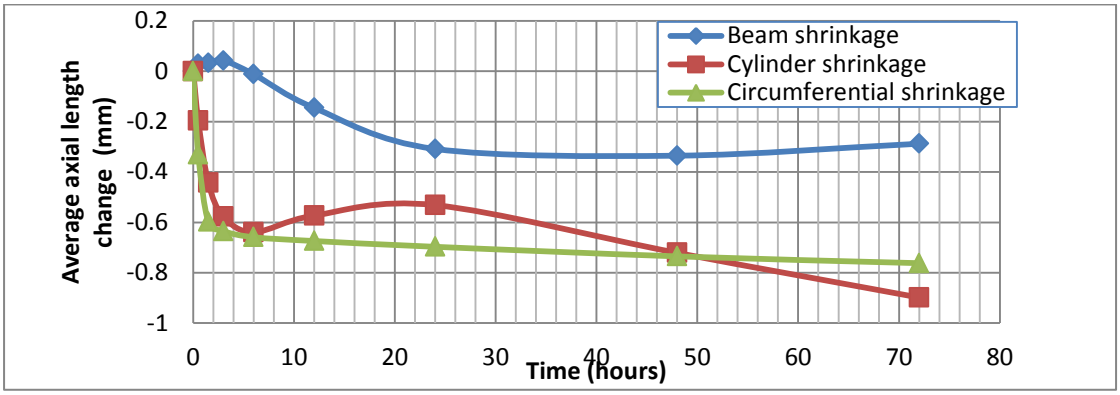


Figure 6.23 Shrinkage of specimens containing 0.9% bitumen emulsion and 1% cement.

6.4.2. Shrinkage magnitude

Figure 6.24 best indicates the difference between the magnitude of shrinkage measured during the beam and cylindrical specimen tests.

- As mentioned earlier, the amount of shrinkage measured for all specimens are extremely small in comparison to the specimen size.
- The cylindrical specimens show higher shrinkage than beam specimens.

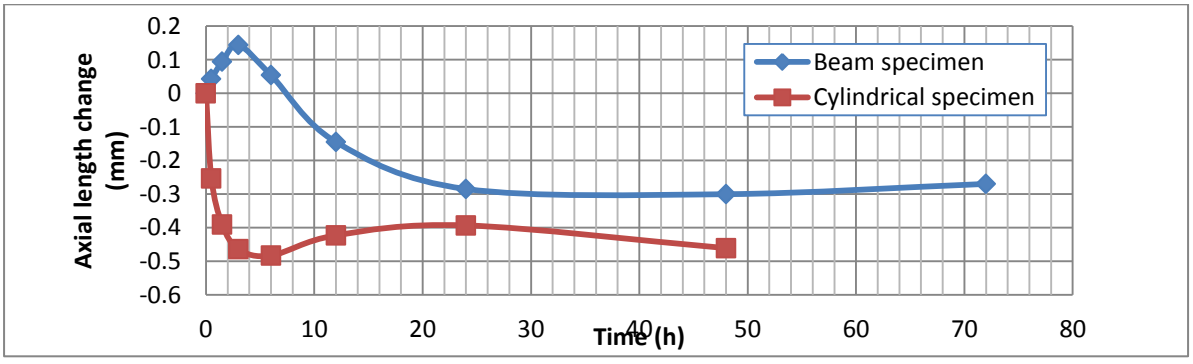


Figure 6.24 Shrinkage of beam and cylindrical specimens containing 2.4% net bitumen emulsion and 2% cement.

6.5. Closing remarks

The research done during this project has expanded the knowledge of the shrinkage behaviour of BSMs. Useful findings were made on the shrinkage behaviour based on the shrinkage testing method as well as the BSM composition.

The two different testing methods showed slight differences in the shrinkage trend of the BSMs. The beam specimens showed an initial length increase followed by a length decrease (shrinkage). The cylindrical specimens on the other hand showed an initial length decrease, followed by a slight length increase and thereafter shrank towards the end of the testing period. A new specimen configuration showed that the difficulties with friction and crack forming could be overcome to some extent.

Overall, higher cement content provided greater shrinkage and higher bitumen emulsion content provided less shrinkage. All findings and the principles causing these behaviours will be discussed thoroughly in Chapter 8.

CHAPTER 7 - Strain-at-break test results

7.1. Introduction

The flexibility behaviour of BSMs was investigated by determining the strain-at-break parameter using new monotonic equipment developed by CSIR. This equipment allows for very accurate and slow displacement rates due to a worm gear configuration. The displacement is measured with accurate LVDTs, which is specifically configured for monotonic loading. Test results are then stored on a data capturing computer program called Catman.

Strain-at-break results were collected from the Catman computer program, processed and presented using graphical representation. Similarities and differences between the collected data are also highlighted.

7.2. Laboratory test results

Results obtained from the monotonic beam test of each specimen type (all three specimens with the similar composition), have been plotted on one graph to determine the consistency of the test results. Graphical results provided in Figure 7.1 show the highest variability in test results between specimens with the same composition.

Figures 7.1 – 7.5 provide the tests results for all of the strain-at-break tests conducted during the project. Refer to Appendix C for more detailed test results.

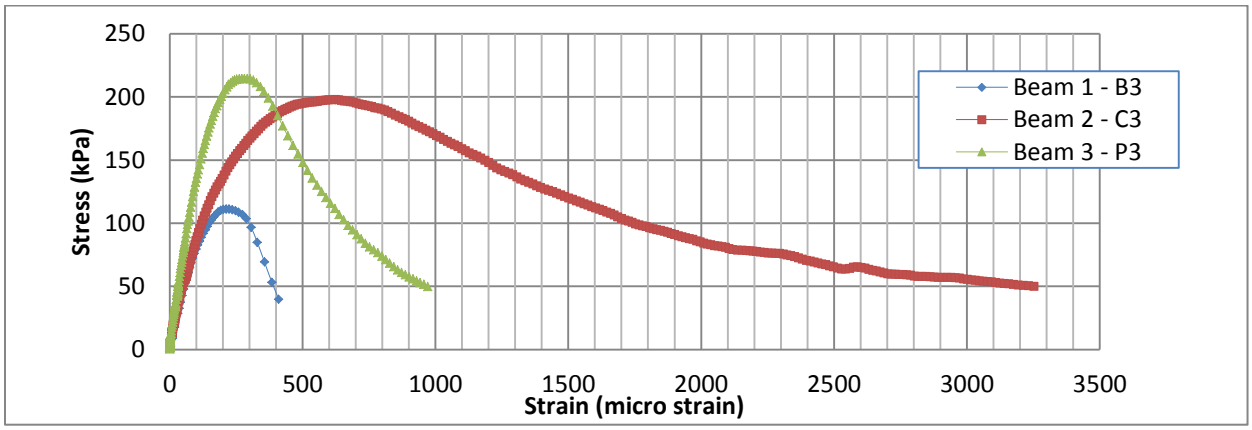


Figure 7.1 Stress vs. strain for specimens containing 0.9% bitumen emulsion and 1% cement.

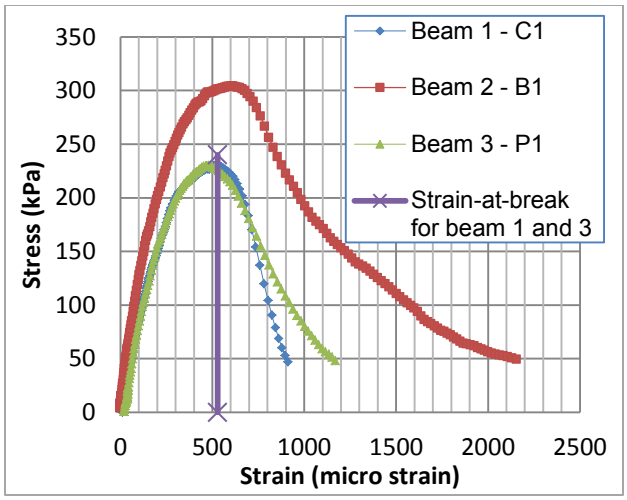


Figure 7.2 Stress vs. strain for specimens containing 2.4% emulsion bitumen and 1% cement.

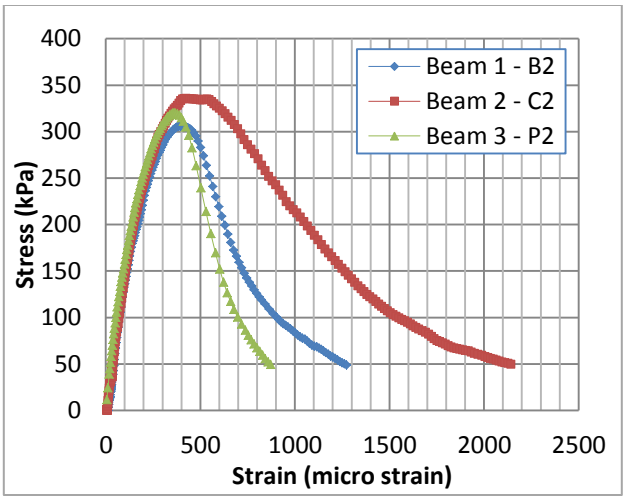


Figure 7.3 Stress vs. strain for specimens containing 2.4% emulsion bitumen and 2% cement

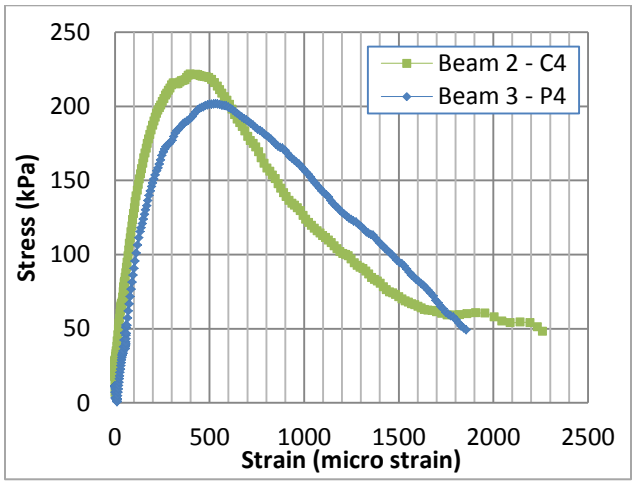


Figure 7.4 Stress vs. strain for specimens containing 2.4% foamed bitumen and 1% cement.

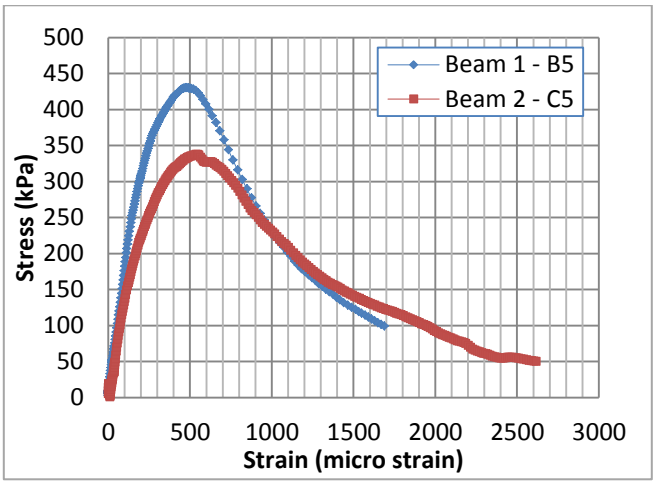


Figure 7.5 Stress vs. strain for specimens containing 2.4% foamed bitumen and 2% cement.

7.3. Strain-at-break

The strain-at-break value of a material is known as the strain at the moment where the stress applied to the specimen is at its maximum (as indicated in Figure 7.2), which provides an indication of the flexibility of the material.

7.3.1. Strain-at-break results

The stress-at-break and strain-at-break results collected form the strain tests conducted on the R35-material is summarized in Table 7.1. The values provided in this table are the average value of the three specimens with the same composition.

Table 7.1 Average stress- and strain-at-break values for all tested specimens.

Specimen specification	Beam property	
	Average Maximum Stress (kPa)/ Stress-at-break	Average Strain-at-break (με)
0.9% Emulsion; 1% Cement	174.4	376.5
2.4% Emulsion; 1% Cement	254.9	537.2
2.4% Emulsion; 2% Cement	320.4	391.1
2.4% Foamed; 1% Cement	211.6	480.8
2.4% Foamed; 2% Cement	383.8	508.7

7.3.2. Influence of additives

7.3.2.a) Bitumen (emulsion and foamed)

Containing 1% cement:

R35-material containing 1% cement were stabilised with either 0.9% bitumen emulsion, 2.4% bitumen emulsion (net bitumen) or 2.4% foamed bitumen.

- Specimens containing higher bitumen contents (emulsion stabilised materials) prove to achieve a higher strain-at-break value during the monotonic beam tests (Figure 7.6).
- Stabilisation with 2.4% foamed bitumen shows a lower strain-at-break value than stabilisation with 2.4% bitumen emulsion (Figure 7.6).

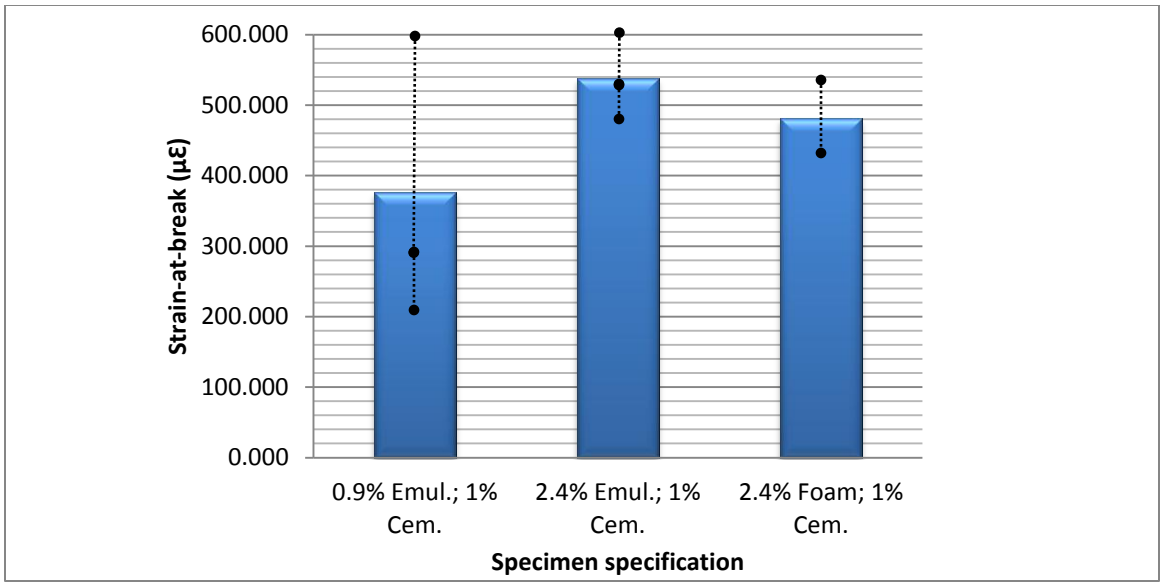


Figure 7.6 Strain-at-break of specimens containing 1 % cement.

Containing 2% cement:

R35-material specimens containing 2% cement were stabilised with either 2.4% bitumen emulsion or 2.4% foamed bitumen.

- Foamed bitumen treated specimens containing 2% cement (Figure 7.7) show higher stain-at-break values than specimens treated with bitumen emulsion.

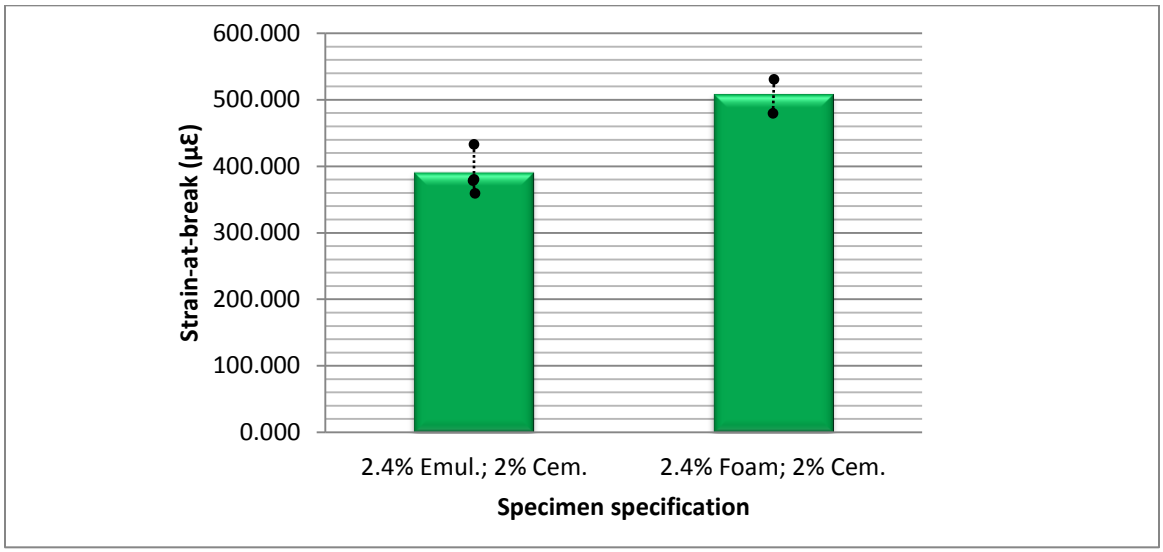


Figure 7.7 Strain-at-break of specimens containing 2% cement.

7.3.2.b) Cement

Stabilised with bitumen emulsion:

The influence of the cement content on the strain-at-break value of the bitumen emulsion stabilised R35-materials have been tested by increasing the cement content from 1% to 2%.

- An increase in cement shows a decrease in strain-at-break value for the BSM R35-material, as indicated in Figure 7.8.

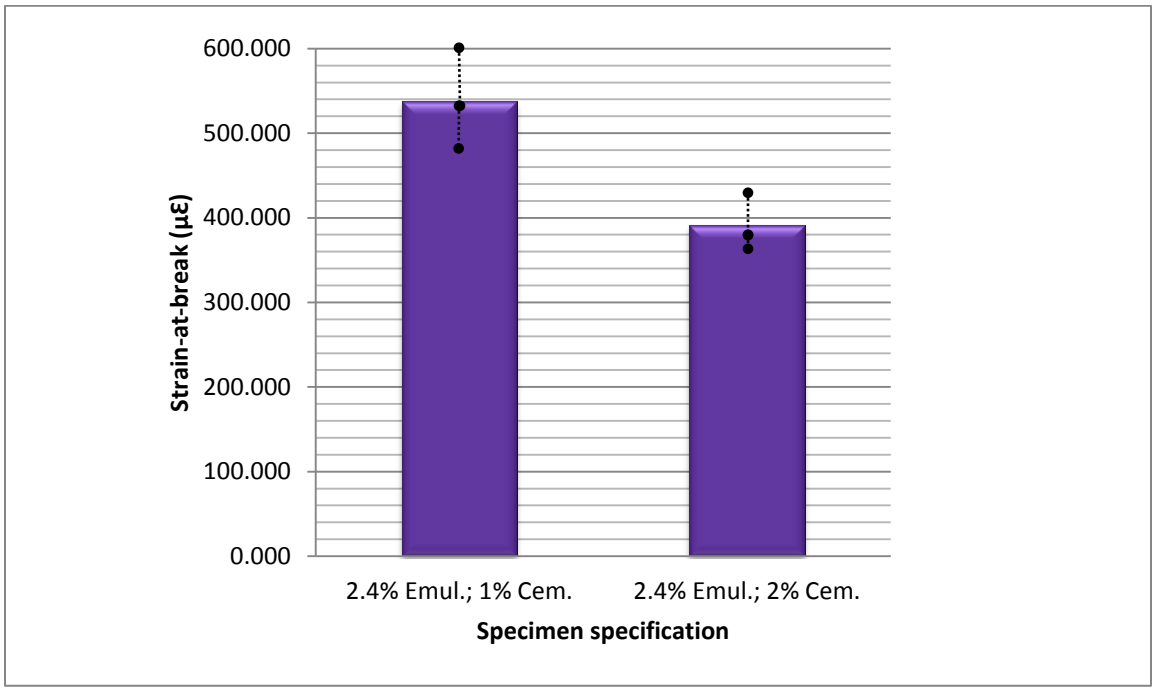


Figure 7.8 Strain-at-break values for specimens containing 2.4 % bitumen emulsion, with an increase in cement.

Stabilised with foamed bitumen:

The influence of the cement content on the strain-at-break value of the foamed bitumen stabilised R35-materials have been tested by increasing the cement content from 1% to 2%.

- Increased cement quantities in foamed bitumen stabilised R35 materials provide a higher strain-at-break value (Figure 7.9).

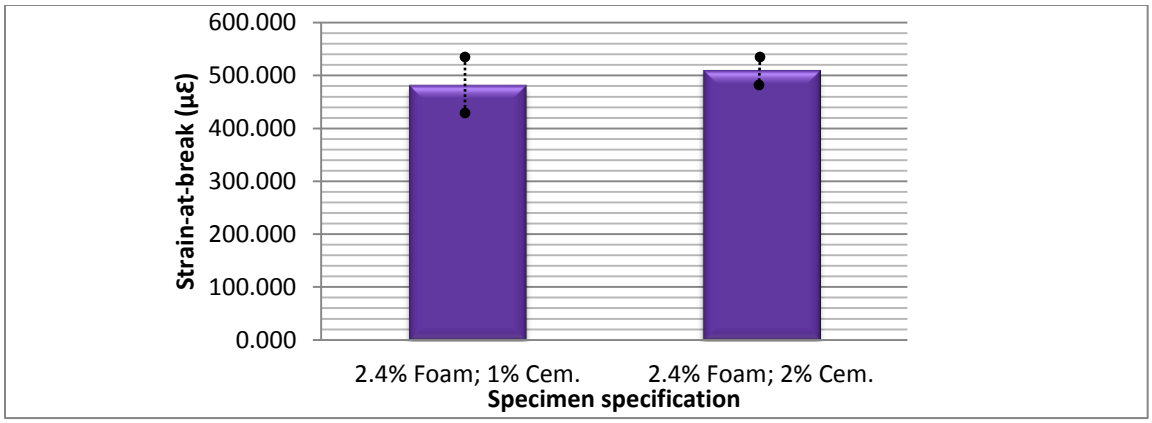


Figure 7.9 Strain-at-break values for specimens containing 2.4% foamed bitumen, with increased cement quantities.

7.4. Dissipated energy

Dissipated energy is the amount of strain energy per unit volume absorbed by the material (Roylance, 2001) until the point of breaking. This can be determined by calculating the area under the stress- strain cure of a material up to the breaking point (of the specimen).

The Simpson method of areas was used to determine the dissipated energy during this project (Appendix C for more detailed calculations). The integrated area under the stress-strain curve until the breaking point can be determined using Equation 7.1. As example, Figure 7.9 provides the stress-strain curve until breaking point for beam specimen 1, containing 0.9% bitumen emulsion and 1% cement.

$$\int_a^b f(x)dx \quad — \quad f(x_0) \qquad (x_1) \qquad f(x_2) \quad 4f(x_3)+ \qquad 4f(x \quad) \quad f(x_n)] \qquad \text{[Equation 7.1]}$$

Where: f(x) = Polynomial function of stress-strain curve until breaking point (for example f(x) = 2E-06x^3 - 0.0031x^2 + 1.1071x - 1.9545 for LVDT 1 from Figure 7.10).

a = Minimum stress

b = Maximum stress

n= Subintervals (between strain values)

$\Delta x =$ $(b \quad a)/n$

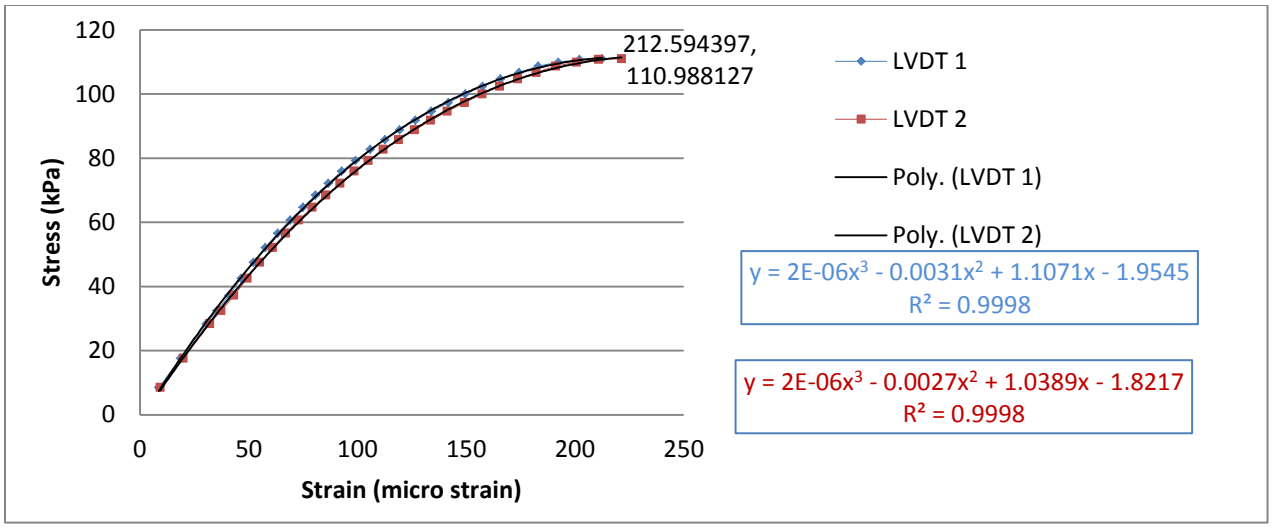


Figure 7.10 Stress vs. strain for beam 1 (0.9% emulsion and 1% cement) until break point.

7.4.1. Dissipated energy results

The dissipated energy for all tested R35-material specimens is summarized in Table 7.2. The values provided in this table are the average value of the three specimens with the same composition.

Table 7.2 Average dissipated energy for all tested beam specimens.

Specimen specification	Beam property	
	Average Maximum Stress (kPa)/ Stress-at-break	Average Dissipated energy (Pa)
0.9% Emulsion; 1% Cement	174.4	39.1
2.4% Emulsion; 1% Cement	254.9	89.8
2.4% Emulsion; 2% Cement	320.4	78.8
2.4% Foamed; 1% Cement	211.6	68.7
2.4% Foamed; 2% Cement	383.8	151.3

7.4.2. Influence of additives

The influence on addetives on the dissipated energy is similar to the influence on the strain-at-break values discussed earlier in this chapter.

7.4.2.a) Bitumen (emulsion and foamed)

Containing 1% cement:

R35-materials were either stabilised with 0.9% bitumen emulsion, 2.4% bitumen emulsion or 2.4% foamed bitumen.

- An increase in bitumen emulsion content shows an increase in dissipated energy (Table 7.2).
- The energy dissipated by foam stabilised specimens are less than emulsion stabilised specimens.

Containing 2% cement:

R35-material specimens containing 2% cement were stabilised with either 2.4% bitumen emulsion (net bitumen) or 2.4% foamed bitumen.

- Specimens stabilised with foamed bitumen indicate higher dissipated energy than specimens stabilised with bitumen emulsion (Table 7.2).

7.4.2.b) Cement

The influence of cement on the dissipated energy of the R35-materials was tested by increasing the cement content from 1% to 2%.

Stabilised with bitumen emulsion:

- An increase in cement content show an decrease in dissipated energy (Table 7.2).

Stabilised with foamed bitumen:

- Increased cement quantities show an increase in dissipated energy (Table 7.2).

7.5. Stiffness

A relationship exists between the stress and strain for linear elastic behaviour of materials, known as Hooke's Law (Jenkins, 2012). Hooke's Law states that the slope of a stress-strain

curve indicates the stiffness of the material, which is also known as the Elastic modulus (Figure 7.10). The stiffness or Elastic Modulus is thus denoted by Equation 7.2.

$$E = \frac{\sigma}{\varepsilon}\ (MPa) \qquad \text{[Equation 7.2]}$$

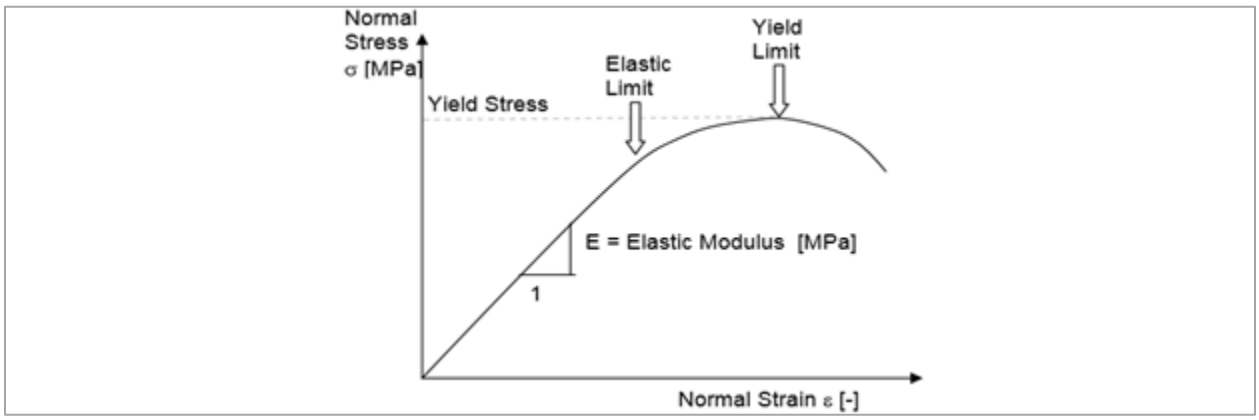


Figure 7.11 Elastic modulus as a function of stress and strain (Jenkins, 2012).

7.5.1. Stiffness results

Hooke's law will only be applicable during the elastic behaviour of the test specimens. Using this Law the stiffness of the material was determined and is tabulated in Table 7.3. The values provided in this table are the average value of the three specimens with the same composition.

Table 7.3 Stiffness values of all tested beam specimens.

Specimen specification	Beam property		
	Average Maximum Stress (kPa)/ Stress-at-break	Average Strain-at-break (με)	Average stiffness (MPa)
0.9% Emulsion; 1% Cement	174.4	376.5	524.2
2.4% Emulsion; 1% Cement	254.9	537.2	473.1
2.4% Emulsion; 2% Cement	320.4	391.1	821.6
2.4% Foamed; 1% Cement	211.6	480.8	447.4
2.4% Foamed; 2% Cement	383.8	508.7	761.9

7.5.2. Influence of additives

7.5.2.a) Bitumen (emulsion and foamed)

Containing 1% cement:

R35-materials were either stabilised with 0.9% bitumen emulsion, 2.4% bitumen emulsion or 2.4% foamed bitumen.

- Material (beam) stiffness showed a decrease with an increase in bitumen emulsion content (Figure 7.12).
- Specimens stabilised with 2.4% foamed bitumen has a lower stiffness than specimens stabilised with 2.4% bitumen emulsion (Figure 7.12).

Containing 2% cement:

R35-material specimens containing 2% cement were stabilised with either 2.4% bitumen emulsion (net bitumen) or 2.4% foamed bitumen.

- Bitumen emulsion stabilised materials with a cement content of 2% show higher stiffnesses than specimens stabilised with foamed bitumen (Figure 7.13).

7.5.2.b) Cement

The influence of cement content on the R35-material was tested by increasing the cement content from 1% to 2%.

Stabilised with bitumen emulsion:

- An increase in cement content provides higher stiffness values, seen in Figure 7.14.

Stabilised with foamed bitumen:

- An increase in cement content provides higher stiffness values, seen in Figure 7.15.

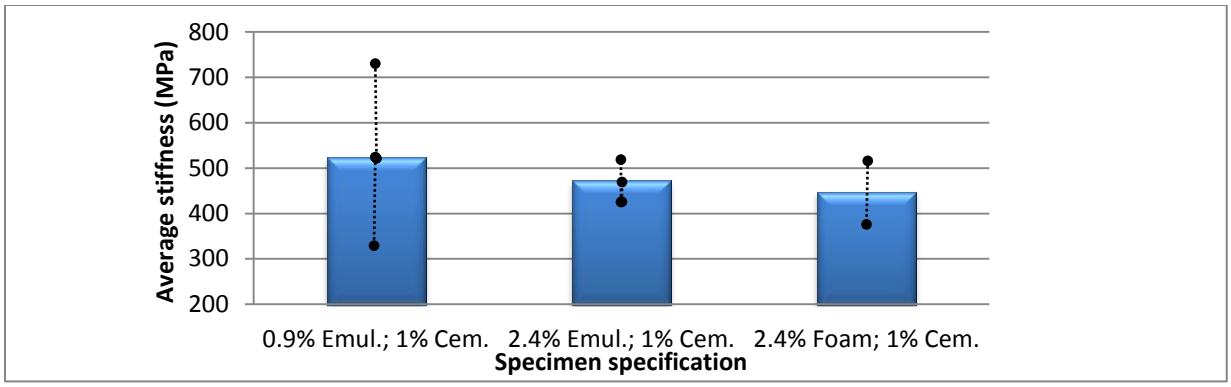


Figure 7.12 Stiffness of materials with 1% cement.

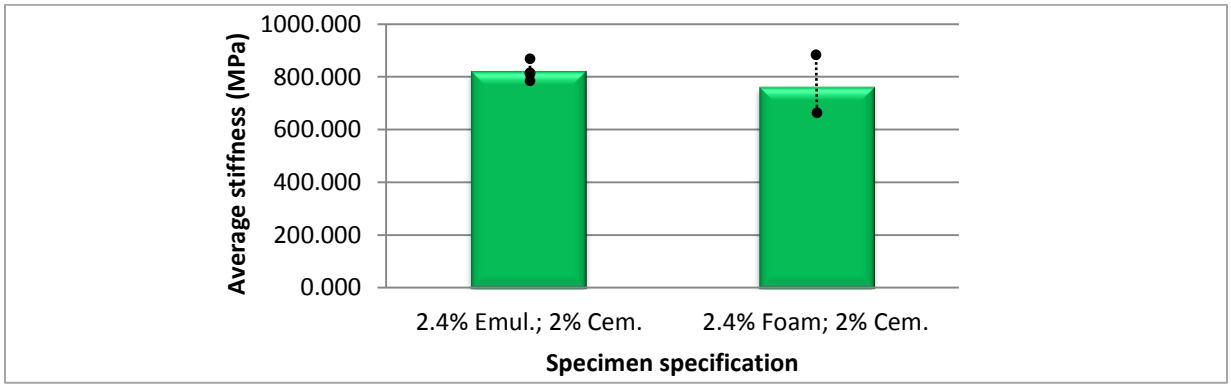


Figure 7.13 Stiffness of specimens containing 2% cement

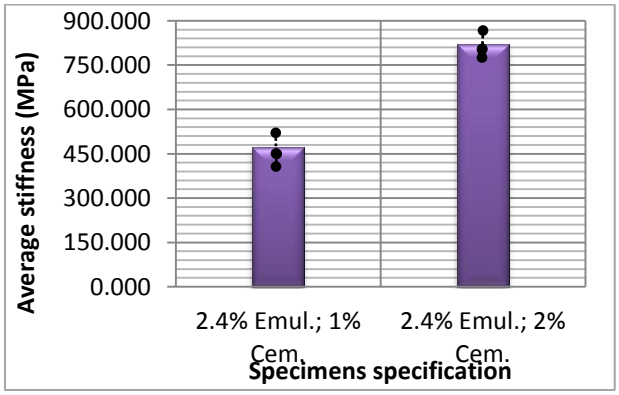


Figure 7.14 Stiffness of specimens stabilised with 2.4% emulsion bitumen, with increased cement quantities.

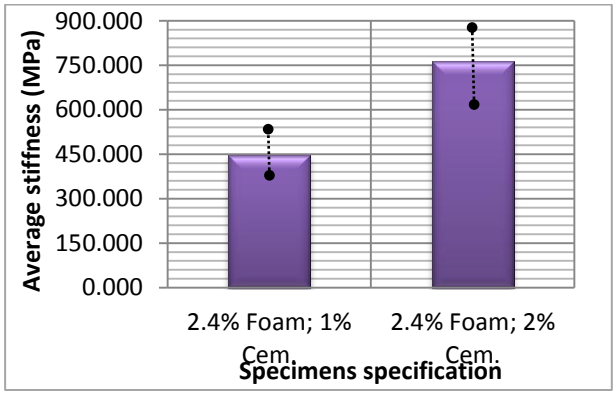


Figure 7.15 Stiffness of specimens stabilised with 2.4% foamed bitumen, with increased cement quantities.

7.6. Closing remarks

The monotonic equipment designed by the CSIR was successfully used to determine the strain-at-break parameter of BSMs. The research also provided useful insights into the flexibility behaviour of BSMs with a change in different additives.

The test results showed consistent results between similar specimen types, thus providing acceptable repeatability. Higher cement contents increase material stiffness and higher bitumen contents increases flexibility. Therefore both cement and bitumen have a great influence on the flexibility behaviour of BSMs. Greater flexibility indicated higher energy absorption before breaking point during the project.

All findings made from the monotonic beam tests and the principles causing this behaviour are discussed in more detail in Chapter 9.

CHAPTER 8 - Shrinkage interpretations

8.1. Introduction

Important findings were made during the shrinkage tests. Differences were noted, evaluated and reconciled between beam and cylindrical shrinkage evaluations in the laboratory. Even though some tests provided inconsistent results the cause of variability was investigated and analysed. These findings are highlighted and discussed.

8.2. Specimens

The difference in geometry between specimen types influences the orientation of compaction and testing, aggregate size, aggregate packing and exposed surface area. These factors all have a big influence on both the shrinkage trends and shrinkage magnitude of BSMs. This was kept in mind during the interpretation of the shrinkage test results.

8.2.1. Specimen geometry

The following factors differ between specimen types, which contribute to the difference in shrinkage trends of the tested specimens:

- Particle size compared to specimen dimension
- Shape: beam versus cylindrical
- Surface area: beam surface area < cylindrical surface area
- Compaction orientation
- Shrinkage testing direction

8.2.2. Aggregate packing

Each ***beam specimen*** consists of three layers each with a thickness of 25mm. The ***cylindrical specimen*** on the other hand is made up out of five layers each with a thickness of 60mm.

Since the largest particle size of the material is 19mm, the cylindrical specimens have a more realistic aggregate packing in the mixture than the beam specimens.

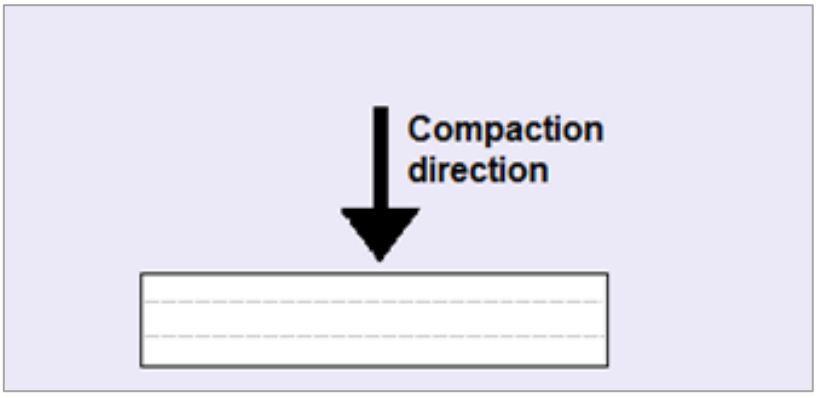
8.3. Testing procedures

The testing direction, exposed surface and forces acting on the specimens during testing have an influence on the shrinkage trends and measurements. Each test method has their advantages and disadvantages.

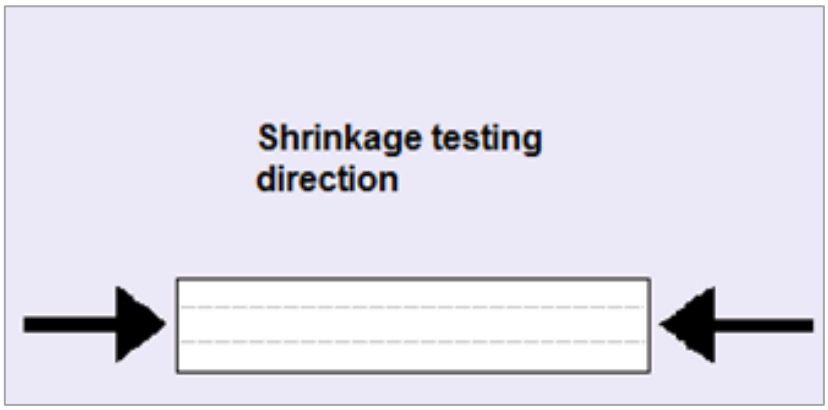
8.3.1. Testing direction

Beam specimens are compacted from the top (vertically downwards) as illustrated in Figure 8.1.a, but are tested in the horizontal direction as illustrated in Figure 8.1.b.

Cylindrical specimens are also compacted from the top (vertically downwards – Figure 8.2.a), but is tested in the same direction as compaction (vertically - Figure 8.2.b).

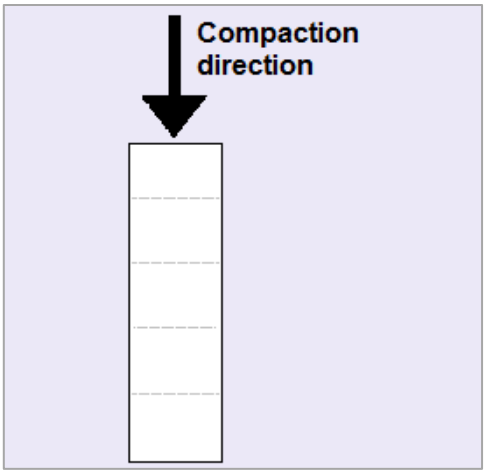


8.1.a) Direction of compaction for beam specimens.

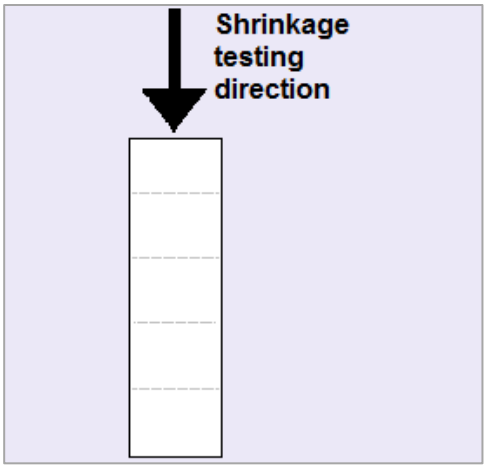


8.1.b) Direction of shrinkage testing for beam specimens.

Figure 8.1 Compaction and shrinkage testing direction of beam specimens.



8.2.a) Direction of compaction for cylindrical specimens.



8.2.b) Axial shrinkage testing direction for cylindrical specimens.

Figure 8.2 Compaction and shrinkage testing direction of cylindrical specimens

8.3.2. Exposed surface

The surface of the beam specimen that is in contact with the Teflon sheet during shrinkage testing (area in the direction of testing), causes the shrinkage to be non-continuous. In contrast to the beam specimen, the cylindrical specimen has no covered area in the direction of testing. This allows for continues shrinkage (even shrinkage) during testing.

8.3.3. Forces acting on specimens

Friction forces are present between the beam and the surface area the beam is resting on during shrinkage testing. These friction forces cause the shrinkage to be restricted and non-continuous even if mould oil is used. Possible discontinuities caused by shrinkage cracks in the beam can make it difficult to use overall specimen measurements to evaluate shrinkage, especially if cemented materials are tested using this method. This obstacle was overcome by applying an external compressive force equal to the friction force between the beam and the Teflon at either ends of the beam using springs. This may have induced unwanted stresses during testing. Gravitational forces also act on the beam, perpendicular to the testing direction.

The only forces acting on the cylindrical specimens are the gravitational forces acting in the direction of testing. This contributes to the shrinkage continuity and produces consistent measurements.

8.3.4. Comparison between different testing methods

Both the beam shrinkage method and cylindrical shrinkage method have advantages and disadvantages. These have been tabulated in Table 8.3.

Table 8.1 Advantages and disadvantages of shrinkage testing methods.

	Shrinkage method type	
	Beam shrinkage testing	**Cylindrical shrinkage testing**
Advantages	- Shrinkage measurements are taken in the direction that matters the most in a pavement layer. In practice shrinkage causes horizontal tensile stresses in adjacent layers, which could cause shrinkage cracks. Therefore beam shrinkage measurements are realistic.	- Thicker compaction layers insure a more realistic aggregate packing. - Greater shrinkage is observed in cylindrical specimens than beam specimens, providing a better idea of the material shrinkage behaviour. Cylindrical specimens will thus provide the most conservative shrinkage results. - Large exposed surface area, keeping shrinkage continuous. - Shrinkage method is easy and repeatable.
Disadvantages	- One surface covered during testing, causing possible discontinuities in shrinkage measurements. - Thin compaction layers cause less realistic aggregate packing than cylindrical specimens. - Testing method is less repeatable than cylindrical specimens, especially if cemented materials are tested (due to shrinkage cracks).	- Shrinkage measurements are taken perpendicular to the shrinkage (in practice) that causes damaging tensile forces.

8.4. Shrinkage results

The shrinkage results will undoubtedly differ between cylindrical and beam specimens due to the geometry of the specimens and the direction of testing.

8.4.1. Shrinkage trends

Results show that not only shrinkage takes place during the testing period as predicted by the hypothesis, but swelling as well. Shrinkage remains the most important volume change, since shrinkage cracks can shorten the pavement lifetime.

It is also important to note that all length changes during testing are extremely small, but may be larger in practice where shrinkage occurs over the total width or length of the pavement. A less dense material layer will show higher shrinkage values due to high void contents.

Table 8.2 provides a summary of the trends observed during shrinkage tests for both the beam and cylindrical specimens.

Table 8.2 Summary of shrinkage trends for beam and cylindrical specimens (Table continues on next page)

Time period	Beam		Cylinder	
	Trend	Discussion	Trend	Discussion
1.5 h	**Specimen swelling** (Higher bitumen content show higher swelling): Inconsistent with hypothesis	- Specimens are removed from moulds after compaction, effectively removing resultant forces equal to that of the residual stresses applied during compaction. High lubrication provided by the viscous bitumen content presumably allowed the material to relax (no mould providing reaction forces), with too little cement to have an immediate shrinkage effect caused by cement hydration. - Drying shrinkage has not taken place yet.	**Initial shrinkage:** Consistent with hypothesis	- Gravitational forces acting on the specimens are presumably greater than the swelling forces caused by the bitumen (as seen in the beam specimens) and therefore shrinkage from cement hydration is experienced. - Circumferential shrinkage show immediate shrinkage as well, caused by cement hydration.

.

Time period	Beam		Cylinder	
	Trend	Discussion	Trend	Discussion
6 h	**Shrinkage** (1.5h -24 h): Consistent with hypothesis	- Hydration of cement and water evaporation causes shrinkage.	**Shrinkage:** Consistent with hypothesis	- Hydration of cement and water evaporation further increases shrinkage.
24h	**Shrinkage** (maximum shrinkage): Consistent with hypothesis	- High moisture evaporation from the specimens takes place, increasing suction forces, which causes the beam specimens to shrink. - Although the shrinkage is predominately caused by drying, the cement hydration process will also contribute to the shrinkage of the beam specimens. This may somewhat be counteracted by the elasticity of the added bitumen. -The elasticity and binding properties of the bitumen will prevent any shrinkage cracks from forming during shrinkage in the BSM.	**Swelling** (for a short time period): Inconsistent with hypothesis	- Presumably swelling takes place due to the effect of the material itself (plastic properties). - Circumferential tests do not show any volume increase during the entire testing period. If any swelling took place it would be too small to be tested, due to the small section area of the cylindrical specimen.
72 h	**Shrinkage** (some specimens show slight swelling): Consistent with hypothesis	- Further shrinkage occurs due to curing and cement hydration. - Shrinkage continues until stabilising at the end of the testing period. Shrinkage stabilises due to total moisture evaporation.	**Shrinkage:** Consistent with hypothesis	- Shrinkage occurs once again as more moisture evaporates from the specimen. - Further cement hydration also takes place.

8.4.2. Influence of additives on shrinkage

The combination of the added cement and bitumen to a BSM will affect the shrinkage of the material. Not only will one additive influence the other, but the amount of additives as well.

8.4.2.a) Combination of additives

Shrinkage

Approximately equal but low amounts of cement and bitumen have proven almost identical results as specimens containing high but approximately equal amounts of bitumen and cement. This is shown in Figures 8.3 and 8.4. These figures illustrates that the influence of the additives on each other is significant in itself. Bitumen percentages are net bitumen percentages.

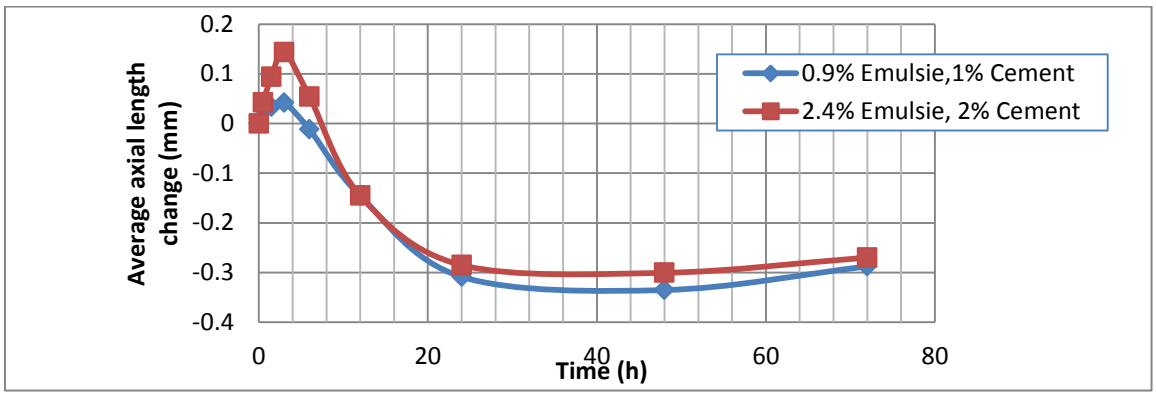


Figure 8.3 Shrinkage of beam specimens containing approximately equal amounts of additives.

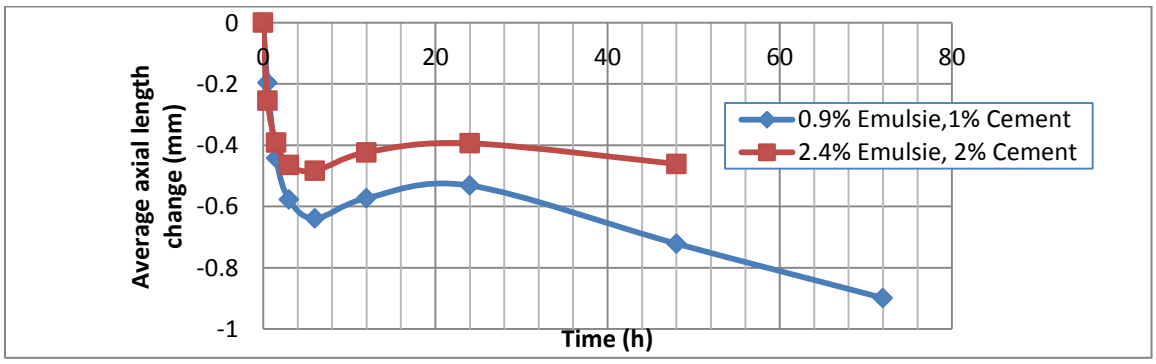


Figure 8.4 Shrinkage of cylindrical specimens containing approximately equal amounts of additives.

BSMs with cement : bitumen ratios that are close to 1 i.e. similar percentages of each additive, provide a relative consistent amount of shrinkage. This is due to the fact that the combination of additives provides a balance between strength and flexibility. If however, the cement : bitumen ratios is significantly less than 1, then significantly less shrinkage will result. The converse is also true for cement : bitumen ratios significantly greater than 1.

Cracks

Cement stabilised materials experience shrinkage cracks during curing, due to the hydration of the added cement. Although BSMs contain cement in limited amounts, shrinkage in a BSM will usually occur without cracking. This is due to the added viscous bitumen in a BSM that bonds the aggregate particles together, eliminating cracking during shrinkage. This is seen throughout the shrinkage testing where no cracks occurred in any specimen.

8.4.2.b) Additives

As mentioned previously, one additive has an effect on the behaviour of the other. However some findings could still be made on the influence of each additive on a BSM. The influence of additive quantities on the shrinkage of the BSMs (R35-materials) is summarised in Table 8.3.

Bitumen (emulsion or foamed):

- *Containing 1 % cement:*

  An increase in bitumen emulsion showed an increase in swelling as well as a decrease in shrinkage, for both beam and cylindrical specimens (as stated in Table 8.3). This occurrence is as expected by the project hypothesis.

  Bitumen is dispersed through the aggregates during material mixing. The bitumen is then absorbed by the fine aggregate particles or the pores of the aggregates. The bitumen is therefore locked into the mixture (or aggregates) while coating the surface area of the aggregate materials. The bitumen coating forms a strong cohesive bond (known as adhesion) binding the aggregate materials to each other. In the case of a BSM this bonding is non-continuous. As the bitumen is absorbed by the material it is captured in the voids, reducing the void content.

Table 8.3 Influence of additives on the shrinkage behaviour of both specimen types.

Additive	Bitumen type	BEAM Specimen		Cylindrical Specimen	
		Effect	Discussion	Effect	Discussion
Bitumen INCREASE (0.9% - 2.4%)	Bitumen emulsion	1% Cem: **Increased swelling, decreased shrinkage:** Consistent with hypothesis	-Increased swelling with increased emulsion, since bitumen provides adhesion, lubrication and visco-elastic properties. Cement hydration has not yet had an effect (beginning stages). -Cement content low enough for emulsion properties to have a greater effect.	1% Cem: **Increased swelling, decreased shrinkage:** Consistent with hypothesis	- Cement content low enough for emulsion properties to have a greater effect. Bitumen emulsion provides better adhesion between particles, visco-elastic properties and thus decreasing shrinkage.
		2% Cem: No tests conducted.	-	2% Cem: No tests conducted.	-
	Foamed bitumen	(No tests conducted)	-	(No tests conducted)	-
Cement INCREASE (1%-2%)	Bitumen emulsion	**Decreased swelling, increased shrinkage:** Consistent with hypothesis	-The non-continuous nature of BSMs allow high cement quantities to have a greater effect, causing more shrinkage and a reduction in swelling.	**Increased shrinkage:** Consistent with hypothesis	-The non-continuous nature of BSMs allow the high cement content to have a greater effect on the material, increasing shrinkage.
	Foamed bitumen	**Increased swelling, decreased shrinkage:** In-consistent with hypothesis	Cement: a) Provides better adhesion between bitumen and aggregates. b) High cement content improves dispersion of bitumen through mixture, increasing the swelling and reducing shrinkage.	Shrinkage trends inconsistent	-

Shrinkage occurs when water is forced out of the voids in a material thus increasing the effective stresses in the material. An increase in effective stress will thus reduce the void spaces in the material, decreasing the material volume (shrinkage). The amount of possible shrinkage is in effect limited by the amount of voids in the material.

A material containing bitumen (BSM) will thus have fewer voids than the same material without bitumen (granular materials). The higher the bitumen content the lower the void content and the lower the ability of a material to shrink, which in effect "reduces" shrinkage.

Replacing bitumen emulsion with foamed bitumen also has an effect on the shrinkage behaviour of BSMs. Foamed bitumen provides more swelling (initially) and greater shrinkage than bitumen emulsion in beam specimens. The increased swelling presumably takes place due to increased bitumen lubrication, but during curing it is apparent that the low cement content (1%) does not provide enough dispersion of bitumen through the material in the beam specimens. The low bitumen dispersion and adhesion causes the cement hydration to have a greater effect, which increases shrinkage. Cylindrical specimens on the other hand show less shrinkage and swelling when stabilised with foamed bitumen. The gravitational forces acting on the specimen is presumably greater than the swelling forces of the material causing a reduction in swelling. Moisture in the cylindrical specimens can evaporate laterally and does not contribute to axial shrinkage, which reduces shrinkage measurements. Inconsistent results between testing types are therefore observed when stabilising with foamed bitumen instead of bitumen emulsion. More tests should be done on these specimen types to improve the accuracy of the results.

- *Containing 2 % cement:*

  Results show that foamed bitumen provides greater swelling and less shrinkage than bitumen emulsion, when the cement content is 2%.

High cement content (2%) provides better dispersion of the viscous bitumen through the mixture. This provides better adhesion that causes a decrease in shrinkage. These results are as predicted by the hypothesis.

Cement:

An increase in cement for specimens stabilised with bitumen emulsion show a decrease in swelling and an increase in shrinkage, which is consistent with the hypothesis (Table 8.3).

Chemical bonds form immediately after water has been added to cement, called cement hydration. During cement hydration a reduction in volume is experienced. Higher cement contents thus increased the shrinkage due to higher cement hydration.

8.5. Closing remarks

The two shrinkage testing methods both have advantages and disadvantages. It is recommended that the method used to determine the material shrinkage should be based on the usage of the shrinkage measurements.

Shrinkage cracks could develop in a pavement material, especially cemented materials, during the material curing period. BSMs usually do not show shrinkage cracks, as seen during this investigation. These shrinkage cracks or material shrinkages in a pavement layer can cause stresses to develop in the adjacent layers, which could then cause cracks to form in these layers. These cracks could lead to pavement aging and water damage, shortening the pavement lifetime.

The shrinkage measured for the bitumen stabilised R35-materials was very small, but other materials may show greater shrinkage due to the nature of the aggregate material.

Valuable findings were made during this investigation, but could be improved by doing more research on the shrinkage behaviour of BSMs.

CHAPTER 9 - Strain-at-break interpretations

9.1. Flexibility

A flexible pavement material has the ability to bend without experiencing excessive damage when subjected to stresses. The flexibility behaviour of a BSM can be characterized by different parameters e.g. the strain-at-break value, dissipated energy and the stiffness of the material.

The addition of bitumen to a granular material will provide a BSM with visco-elastic material properties. A BSM will therefore display visco-elastic behaviour, offering material flexibility when the pavement layer is subjected to applied loads. In contrast to this, the cement content in a BSM will cause the material to become more rigid, which reduces the flexibility properties provided by the bitumen. The cement will also increase the material strength, which is a valuable property that could increase the lifetime of a pavement layer.

A material with high flexibility properties usually has a significant visco-elastic component. The visco-elastic properties of a BSM will allow the material to react immediately after a load is applied with elastic deformation, which will be recovered immediately after the load has been removed. During sustained loading, delayed elastic deformation will occur, as a result of the viscous component. This is creep deformation (viscous deformation), part of which will not totally recover when the applied load is removed i.e. the viscous component. This deformation is manifested in a pavement as permanent deformation.

A layer with higher flexibility usually has a lower stiffness and will therefore be prone to greater permanent deformation than a less flexible, rigid pavement layer with a higher stiffness. Permanent deformation of pavement layers should be minimized since it could provide dangerous driving conditions i.e. rutting that causes aquaplaning in wet conditions.

The influence of additives (bitumen and cement) on the parameters related to flexibility has been summarized in Table 9.1. The percentages that each parameter changes with a change in additives are also graphically shown in Figure 9.1, showing clearly if an increase or decrease in parameter was observed. The relation of each of these parameters to flexibility as well as the changes caused by the additives, seen in both Table 9.1 and Figure 9.1, will be thoroughly discussed.

Table 9.1 Influence of additives on flexibility related parameters.

Additive	Bitumen type	Parameters related to flexibility		
		Strain-at-break	Dissipated energy	Stiffness
Bitumen INCREASE (0.9% - 2.4%)	Bitumen emulsion	Containing 1% Cem: Increase ↑↑	Containing 1% Cem: Increase ↑↑↑↑	Containing 1% Cem: Decrease ↓
	Foamed bitumen	(No tests conducted)	(No tests conducted)	(No tests conducted)
Cement INCREASE (1% -2%)	Bitumen emulsion	Decrease ↓↓	Decrease ↓	Increase ↑↑↑
	Foamed bitumen	Increase ↑	Increase ↑↑↑↑	Increase ↑↑↑

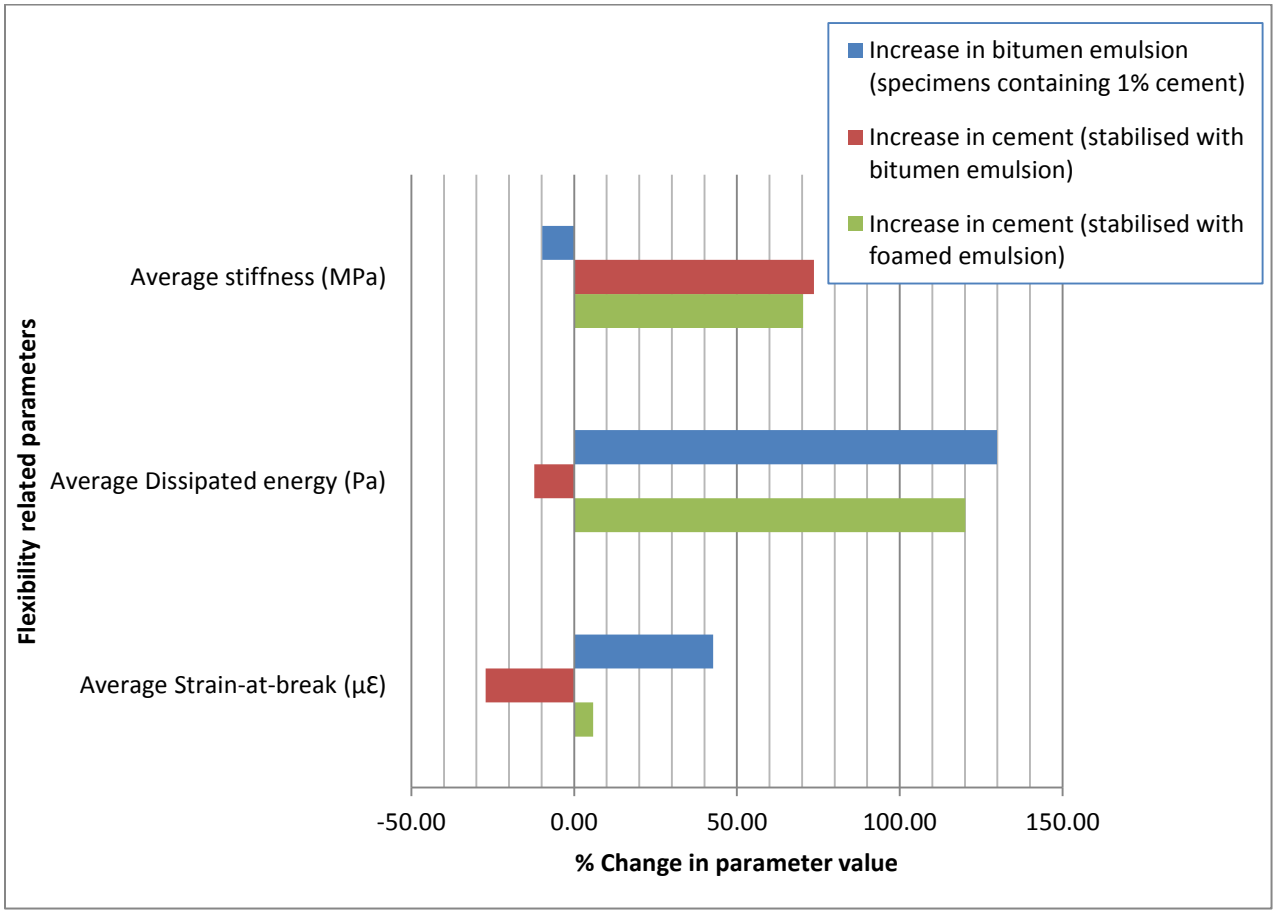


Figure 9.1 Change in flexibility related parameters with different additive quantities.

9.1.1. Strain-at-break

The greater the flexibility of a material the greater the strain the material can withstand before failure. Strain is manifested in visco-elastic materials in a pavement structure as permanent deformation and will reduce the riding quality for road users. A BSM will thus have to be flexible enough to bend without cracking, but stiff enough to show only minimal pavement deformation.

9.1.1.a) Bitumen (emulsion and foamed)

Containing 1% cement:

As stated previously, the added bitumen in a BSM will change the natural granular material to a more visco-elastic material, providing flexible properties. Higher bitumen content in a BSM will thus provide greater flexibility, enabling more flexure in the material before reaching breaking point. The amount of strain a material can withstand without breaking is therefore the strain-at-break value of the material, which provides an indication of the flexibility of the material.

Results from this project indicated that an increase of bitumen emulsion from 0.9% to 2.4% (increase of 1.5% bitumen) increased the strain-at-break value by 42% (Figure 9.1). The addition of higher bitumen emulsion therefore increases the strain-at-break value (Table 9.1) of the material and therefore provides greater flexibility to the material. This behaviour is consistent with that of the hypothesis of this project as well as previous strain-at-break test results, illustrated in Figure 9.2. The higher flexibility provided by an increase in bitumen emulsion will therefore allow greater permanent deformation to take place when a load is applied.

Foamed stabilised materials containing 1% cement, provides lower strain-at-break values than when stabilised with bitumen emulsion. The low cement content (1%) in the foamed stabilised BSM causes insufficient dispersion of the viscous bitumen during mixing. In addition, the cement forms part of the filler fraction in the foamed bitumen mix, which is the most important grading fraction in these BSMs i.e. the cement plays a vital role in the “spot-welds” of the mastic. This causes less flexibility and a decrease in the strain-at-break values.

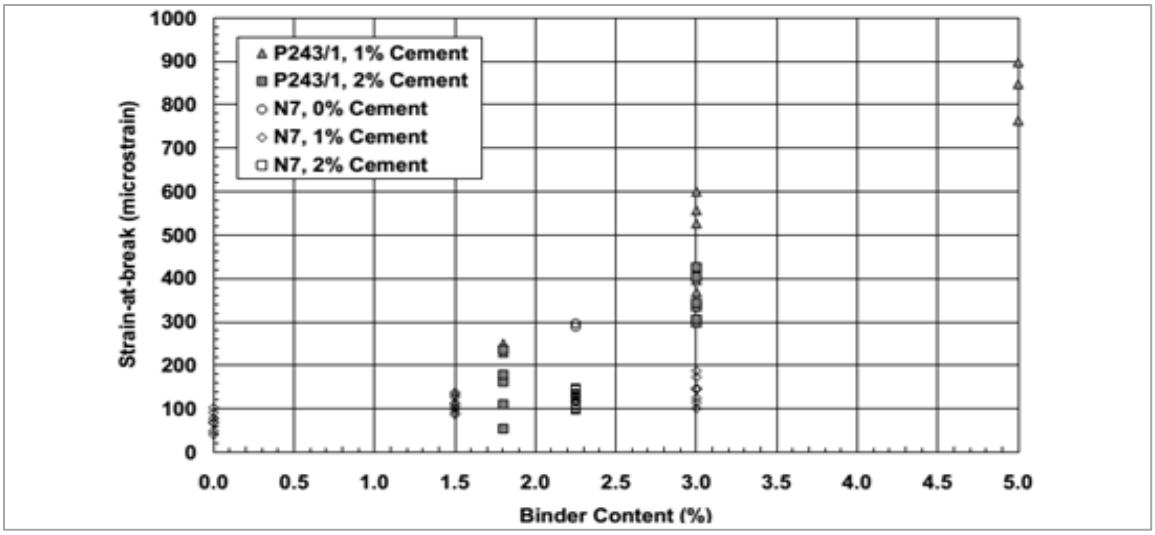


Figure 9.2 Strain-at-break test results (Long & Theyse, 2004) for emulsified bitumen treated materials.

Containing 2% cement:

BSMs containing 2% cement that are stabilised with foamed bitumen, provide higher strain-at-break values than when stabilised with bitumen emulsion. This is due to the high cement content, which will increase the dispersion of bitumen through the mixture. Better bitumen dispersion will increase the contribution of this visco-elastic component and therefore increase the strain-at-break value as well as the flexibility.

9.1.1.b) Cement

Cement provides hardness and rigidity to materials causing resistance to permanent deformation. But, at the same time, the cement can cause brittleness and lack of flexibility.

Stabilised with bitumen emulsion:

At a higher cement content the emulsion treated materials are less flexible (more rigid) and can therefore withstand less strain before breaking (Table 9.1 and Figure 9.1).

Stabilised with foamed bitumen:

The addition of cement to a BSM will also improve the dispersion of bitumen through the mixture. This phenomenon was represented during this project through the results obtained from the foam treated specimens. An increase in cement content spread the foamed bitumen particles better through the BSM. The improved bitumen dispersion caused the properties of the bitumen to be more pronounced and therefore provided greater strain-at-break values and therefore greater flexibility.

9.1.2. Dissipated energy

Dissipated energy is the energy absorbed by a material until breaking point when a load is applied. Based on the force exerted on the material while undergoing displacement (or permanent deformation), it will therefore provide an indication of the energy that the material can process during flexure. The flexibility behaviour of a BSM can therefore be understood better by indicating the displacement experienced by the material while maintaining its stiffness.

The amount of energy absorbed is influenced by both the maximum stress value and the strain-at-break. The dissipated energy is defined as the area under the stress-strain curve until the strain-at-break is reached, in this project. Hence, the dissipated energy coincides with the strain-at-break values throughout this project.

An increase in bitumen emulsion proved to absorb more energy during flexure (Figure 9.1). Comparing R35-BSMs containing 0.9% and 2.4% bitumen emulsion (both with 1% cement), it is clear that greater dissipated energy provides greater flexibility (Figure 9.3). Specimens containing 2.4 % were able to withstand greater forces while at the same time undergoing greater displacements, therefore processing more energy during flexure. Specimens containing 2.4% bitumen emulsion can therefore flex more while maintaining its strength and will therefore have a greater flexibility (and dissipated energy) than specimens containing 0.9% bitumen emulsion. The greater dissipated energy is represented with a greater area under the stress-strain curve provided in Figure 9.3.

When comparing the dissipated energy of the foamed bitumen specimens with the bitumen emulsion specimens, the foam stabilised specimens showed less flexibility than the emulsion stabilised specimens (Figure 9.3). The forces that the foam stabilised specimens were able to withstand during deformation were less than that of the emulsion stabilised specimens.

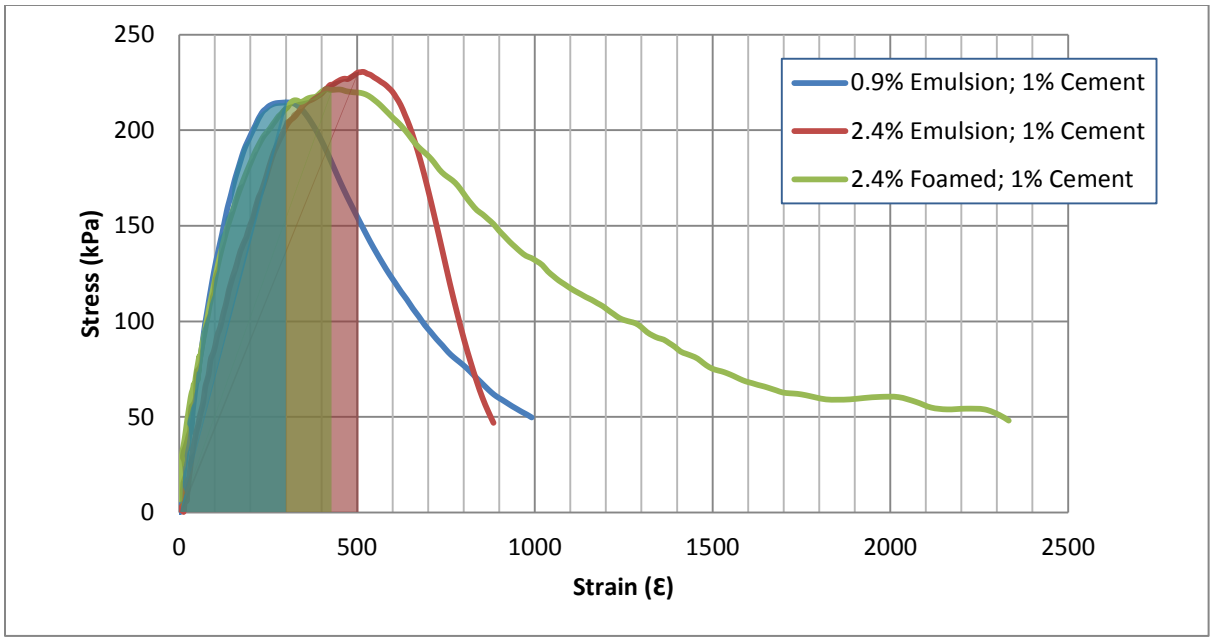


Figure 9.3 Dissipated energy of specimens containing 1% cement (values are from specimens which gives the best indication of the behaviour of that specific specimen type).

Results from this project indicate that an increase in cement for emulsion stabilised materials show a decrease in flexibility (Table 9.1). Specimens containing 2% cement are able to withstand greater forces than specimens containing 1% cement, but less strain occurs before failure (Figure 9.4). Specimens containing 1% cement can therefore withstand forces with greater displacement and are more flexible.

Increasing the cement content in a foam stabilised BSM has proved an increase in dissipated energy. Due to the increased bitumen dispersion caused by the 2% cement, the foam stabilised BSM will achieve greater flexibility than a specimen with 1% cement.

A BSM that shows greater flexibility will have a longer fatigue life than that of a specimen with lower flexibility.

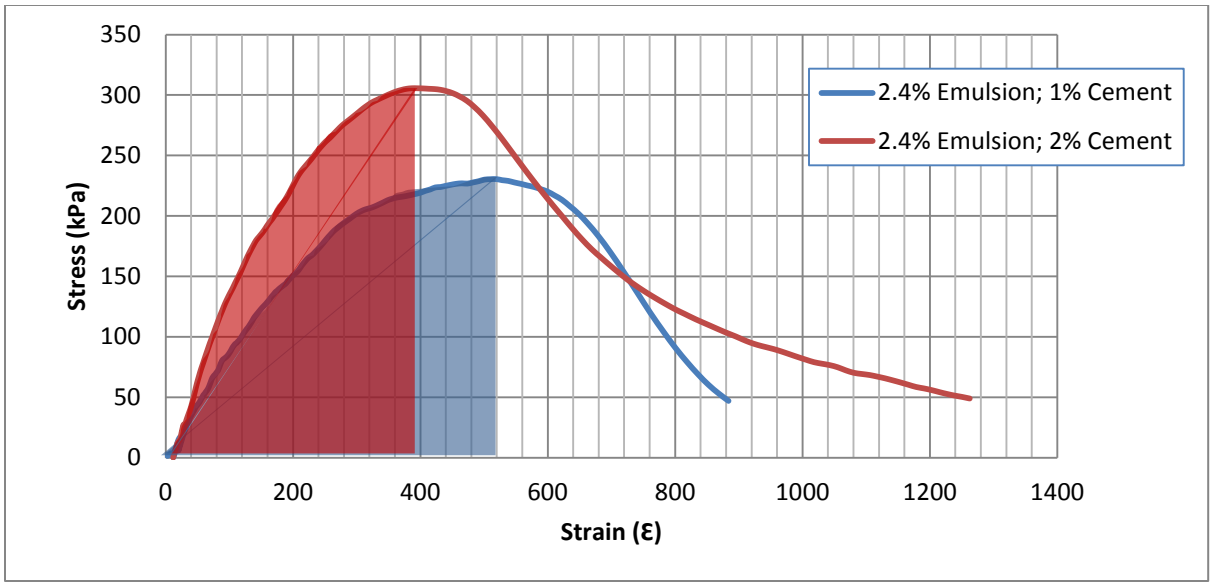


Figure 9.4 Dissipated energy of specimens containing 2.4% bitumen emulsion (values are from specimens which gives the best indication of the behaviour of that specific specimen type).

9.1.3. Stiffness

The aggregate material used for a BSM, density of the material, thickness of the material layer and the added quantities of bitumen and cement will all have an effect on the stiffness the material. Designing a pavement with upper layers having higher stiffnesses and lower layers with lower stiffnesses will produce a balanced pavement structure that will sufficiently spread the applied stresses during its lifetime.

The load spreading ability of a pavement layer is dependent on the stiffness of the material. It is important to design each layer in such a way that the stress levels remains low, keeping permanent deformation to a minimum. Since BSMs are widely used for base layers in flexible pavements, the stiffness should be high to be able to spread the large stresses it receives from the surfacing layer during repeated loading, to less stiff underlying layers. An increased stiffness will therefore improve the load spreading ability, causing a significant reduction in vertical deflections.

As seen during this investigation, different stiffnesses can be achieved by changing either the cement content or the bitumen content in a BSM.

- **Influence of cement**

  Cement modifies the characteristics of a material and provides hardness, rigidity and stiffness to materials. Figure 9.1 and Table 9.1 indicate that an increase in cement for both foam stabilised and emulsion stabilised materials will increase the stiffness of a BSM. A material with a higher stiffness as a result of increased cement content, will have lower flexibility and an increased resistance to permanent deformation. Repeated loading could cause cracks to develop in a pavement layer with a high stiffness, allowing damaging water to enter the pavement and thereby decreasing the durability of the pavement. However, cracks do not usually form in BSMs due to the non-continuously bound nature of the material.

- **Influence of bitumen**

  As seen in Figure 9.1, the visco-elastic properties of the added bitumen in a BSM will generally reduce the stiffness of the material allowing greater flexibility. The material is therefore able to withstand repeated applied stresses without excessive damage (such as crack forming), thus increasing the durability of the material.

9.1.4. Influence of variables on stiffness and strain-at-break combined

The results for BSMs in this project are illustrated in Figure 9.5 as cement and binder content values separately.

Interestingly, looking at these variables (cement and bitumen) separately, higher cement quantities provide increased stiffness in emulsion stabilised specimens (only at 2.4 % emulsion) but a slight drop in strain-at-break values was experienced (pointed out by the dark blue arrow in Figure 9.5). Looking at it from another perspective, for the same cement content an increase in emulsion percentage (from 0.9% to 2.4%) resulted in a general increase in strain-at-break values.

For the foamed bitumen stabilised specimens, an increase in the cement % indicate an increase in stiffness and a slight increase in the strain-at-break value (pointed out by the light blue arrow in Figure 9.5).

These values seem counterintuitive and are possibly because the cement and bitumen should be seen as a combination and it shows that a balance between the binder and cement must be maintained to obtain the optimum values for both parameters: stiffness and strain-at-break.

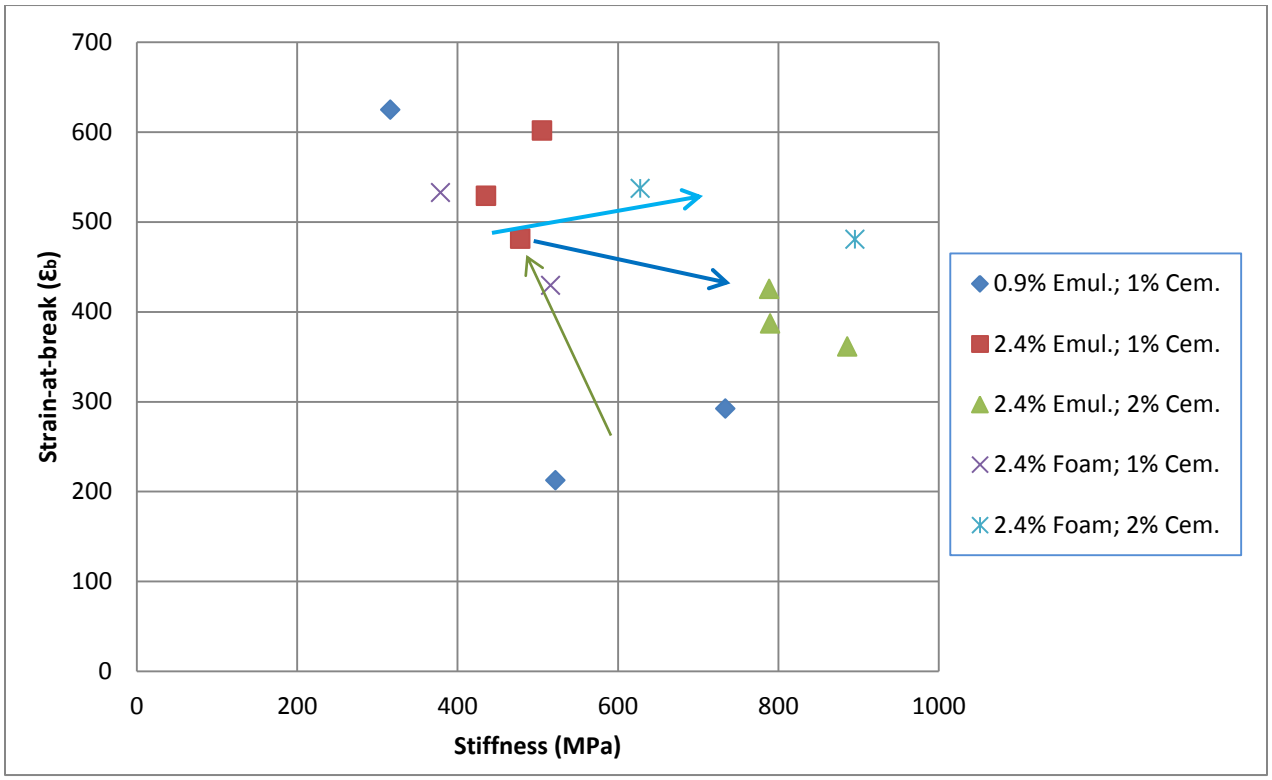


Figure 9.5 Strain-at-break in relation to the stiffness of all specimen types tested during this project.

In order to understand the influence of the variables on both parameters, data of the test results have been plotted alternatively (including a cement : bitumen ratio) as shown in Figure 9.6.

It is interesting to note that a larger disparity between results for greater cement : bitumen ratios is observed than for these same results sets at lower cement : bitumen ratios (at a ratio of 1.1 as apposed to the other ratios e.g. 0.42 and 0.83). This is an indication that a lower cement : bitumen ratio have a specific influence on the material, which will be discussed in the following paragraphs.

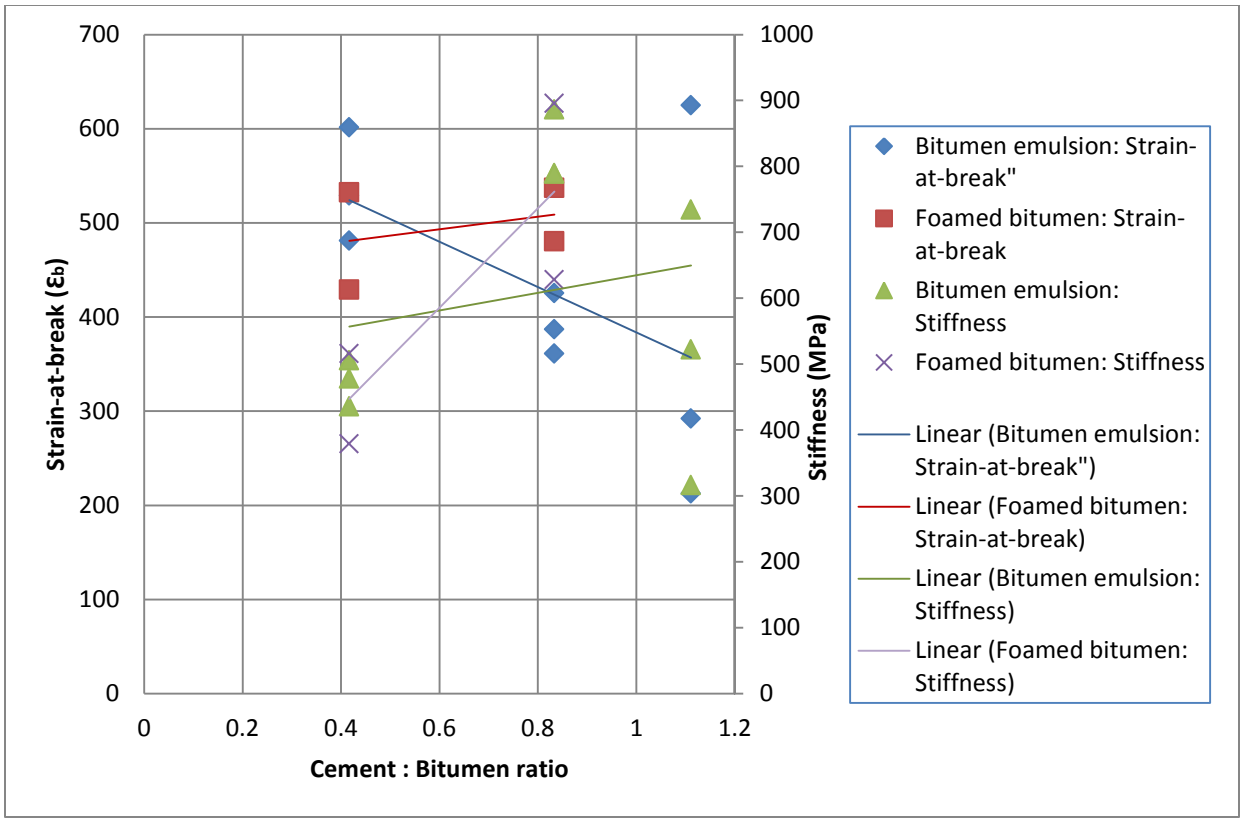


Figure 9.6 Effect of the cement : bitumen ratio on the strain-at-break and stiffness of the materials testes during this project.

Figure 9.6 indicate that for the emulsion stabilised specimens, with an increase in cement : bitumen ratio, result in an increase in stiffness and decreasing strain-at-break values when a general linear trend line is applied to all results within the same variable group, as expected. However, a drop is observed in stiffness for emulsion specimens at a cement : bitumen ratio of 0.83 to 1.1. This potentially indicates that there is an optimum cement : bitumen ratio for a higher stiffness and indicates that adding increased amounts of cement does not necessarily increase stiffness values.

The foamed bitumen stiffness results show an increase in stiffness with an increase in the cement : bitumen ratio but interestingly, also show a slight increase in the strain-at-break with the cement : bitumen ratio increase. The greater cement content improved the bitumen dispersion in the foamed bitumen specimens and increased the filler fraction in the material gradation, which affects the “spot-welds” in the mastic and increased the stiffness as well as the strain-at-break values.

9.2. Application of flexibility and stiffness requirements of BSMs

A BSM is usually used as a base layer in a pavement structure, providing flexibility and stiffnesses that can prolong the lifetime of a pavement.

Case studies in Figure 9.7 illustrate how a BSM can be designed to fulfil the requirements of a pavement structure for a specific load class in different situations. Each case consists of a pavement with a seal surface, BSM base and either a poor or good subgrade support. It is assumed that moderate traffic is applied to all the pavement structures in the case studies.

- **Rural roads:**

  Considering the case of a rural approach (Figure 9.7), the thick BSM layer provides good load spreading.

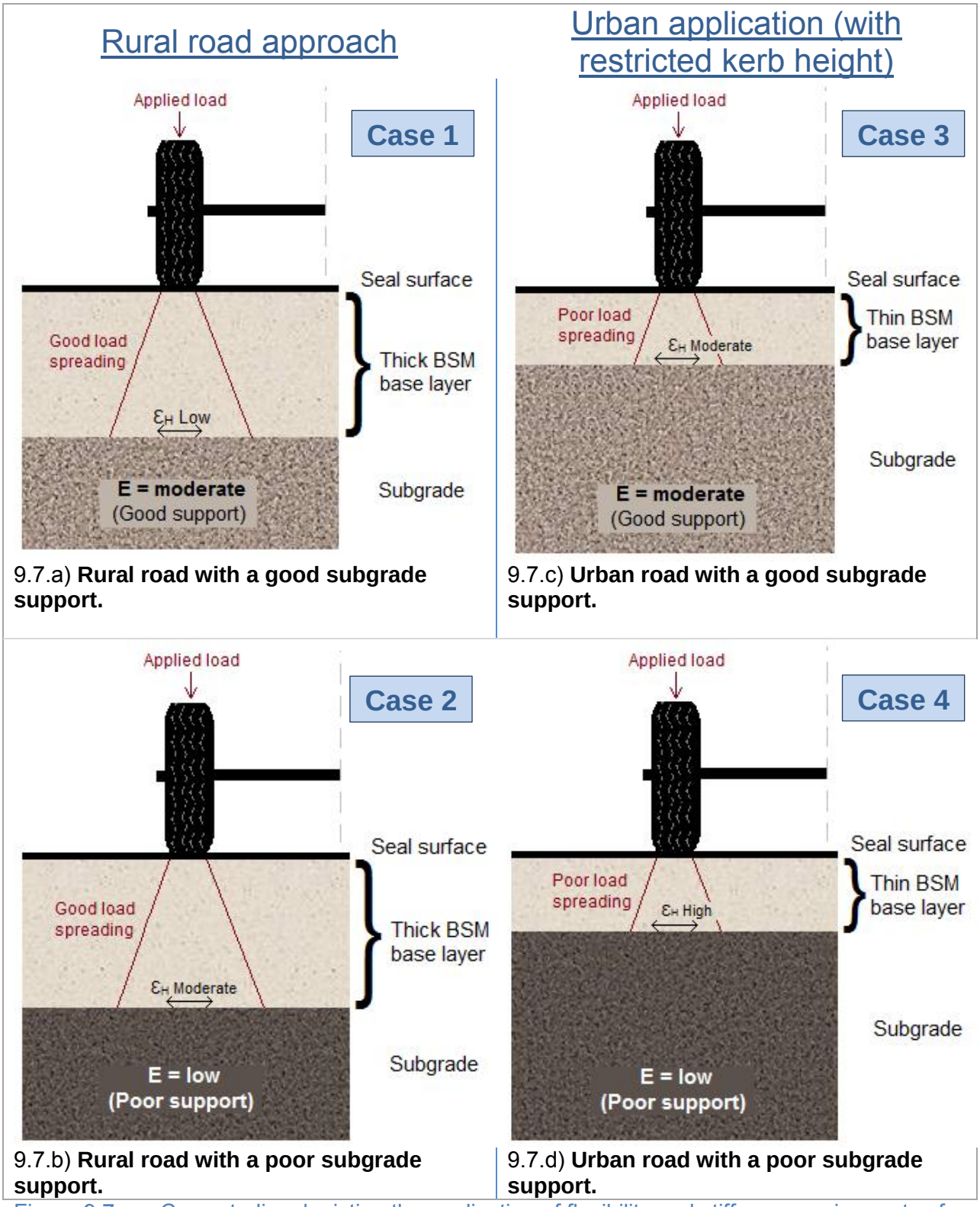
  Case 1

  If the thick BSM-layer overlays a good support (with a moderate $E_{modulus}$), low horizontal strains (ε_H) will develop at the bottom of the BSM layer (Figure 9.7.a). Due to the BSM layer thickness and good support, a less flexible BSM-layer can thus be designed. This is achieved by adding low bitumen quantities to the mixture, which will lower the construction costs. The stiffness of the layer will still provide good load spreading.

  Case 2

  A rural road with poor subgrade support (subgrade with low $E_{modulus}$) will be able to provide good load spreading due to the BSM layer thickness, but will experience more horizontal strain at the bottom of the BSM-layer than that of a good supported BSM layer (Figure 9.7.b). A BSM layer with greater flexibility will thus be required for a rural pavement structure with a poor subgrade support, to be able to withstand more flexure before breakpoint during load application. Higher bitumen quantities will provide a BSM layer with greater flexibility and a balanced % cement content should be added to maintain good load spreading.

  The results from this project indicated that an increase in bitumen will increase the strain-at-break value and dissipated energy value, indicating an increase in material flexibility. Specimens containing high bitumen emulsion contents (2.4%) and low cement contents (1% cement) were able to withstand forces with greater displacement than specimens containing 2% cement (Table 9.1).



9.7.a) **Rural road with a good subgrade support.**

9.7.c) **Urban road with a good subgrade support.**

9.7.b) **Rural road with a poor subgrade support.**

9.7.d) **Urban road with a poor subgrade support.**

Figure 9.7 Case studies depicting the application of flexibility and stiffness requirements of BSMs.

- **Urban roads:**

  In an urban road application the BSM layer could be restricted to a thin layer due to kerb heights and hidden services, providing poor load spreading.

Case 3

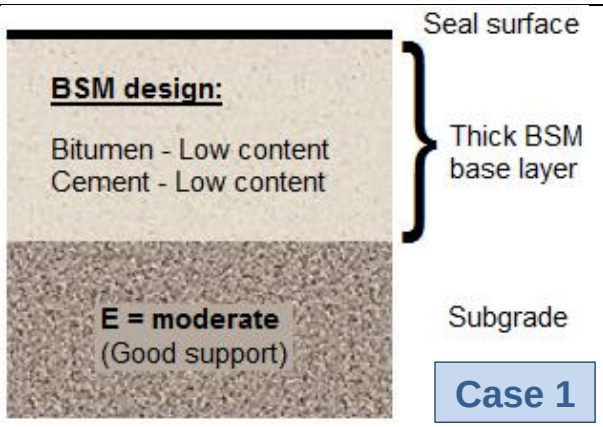
An urban road with a good subgrade support will allow moderate horizontal strain to develop at the bottom of the BSM layer and will also have poor load spreading due to the BSM layer thickness (Figure 9.7.c). A BSM with a higher cement content and higher bitumen content should therefore be designed. This will produce a BSM with a higher material stiffness to improve load spreading and at the same time allow greater flexure to take place before experiencing excessive damage.

Case 4

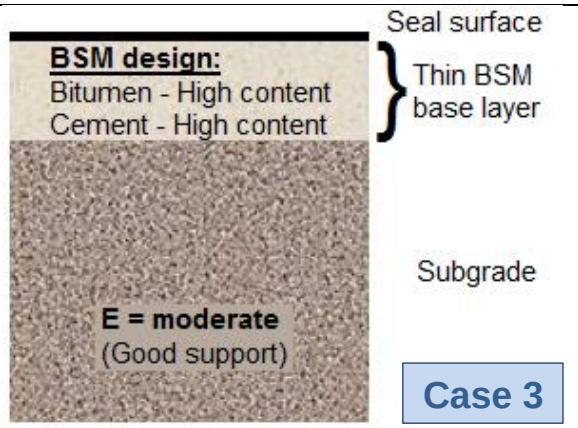
An urban road with a poor subgrade support will have a poor load spreading ability (due to the layer thickness) and will allow high horizontal strains to develop at the bottom of the BSM layer due to the poor support (Figure 9.7.d). The BSM should therefore be designed with greater flexibility and a high stiffness, to improve the load spreading and at the same time allow flexure during load application. Significantly high amounts of both bitumen and cement should therefore be added to the BSM, but should be done with caution, since a pavement with a short lifetime can easily be designed.

Results from this project show an increase in material stiffness with an increase in cement content and an increase in strain-at-break value with an increase in bitumen content, which indicates greater flexibility (Figure 9.1). R35-BSM specimens containing high amounts of foamed bitumen (2.4%) with high cement contents (2%) indicated that more energy is processed during flexure (greater dissipated energy), which indicated that the material will provide a longer pavement lifetime.

The examples clearly show that a base layer should in some cases have flexibility as well as sufficient stiffnesses to produce a durable pavement structure. These properties are provided by using a BSM. Both the cement and bitumen are important components in a BSM layer. A summary of mix design selections for BSMs based on flexibility and stiffness requirements of the discussed examples is provided in Figure 9.8.

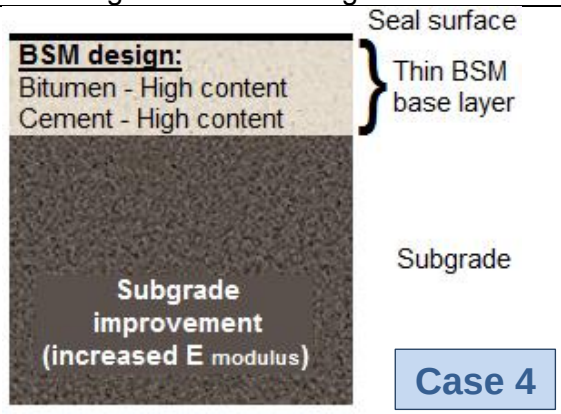


9.8.a) The BSM is designed to provide low flexibility to withstand low flexure induced during loading. Low cement content will balance the stiffness lost with the addition of bitumen.



9.8.c) The BSM is designed to provide high stiffness to improve load spreading. High bitumen content is also added to allow the stiff layer to undergo greater flexure before experiencing excessive damage.

9.8.b) The BSM is designed to provide good flexibility to withstand high flexure that will occur due to poor support. Moderate cement contents is added to balance with the bitumen and maintain material stiffness (maintaining load spreading).



9.8.d) The BSM is designed to provide good stiffness and flexibility, improving the load spreading and at the same time allow high flexure during load application. By improving the subgrade, the load spreading will be increased and the flexure experienced by the BSM during loading will be reduced.

Figure 9.8 Summary of mix design selections for BSMs based on flexibility and stiffness.

9.3. Closing remarks

A good understanding of the flexibility of BSMs is provided by the combined information of the strain-at-break value, stiffness and dissipated energy of the material.

An increase in flexibility is indicated by an increase in strain-at-break. This is achieved by increasing the bitumen content and decreasing the cement content. Higher foamed bitumen

content (2.4%) will provide higher strain-at-break values when in combination with higher cement contents (2%).

An emulsion stabilised specimen with low cement content (1%) or a foam stabilised specimen with high cement content (2%) will be able to absorb the most energy before breakpoint. The higher the energy a material can absorb the greater the flexibility of the material. A BSM that shows greater flexibility will have a longer fatigue life than that of a less flexible BSM.

A higher stiffness is produced when increasing the cement content of a BSM, which could cause cracks to develop with fatigue. Addition of higher amounts of bitumen content will decrease the stiffness of a BSM, providing higher flexibility and therefore allowing permanent deformation to take place with time, rather than fatigue cracking. The combination of cement and bitumen is therefore important in a BSM and should be designed according to the pavement structure requirements.

CHAPTER 10 - Conclusions and Recommendations

10.1. Introduction

This project has provided good insight on both the shrinkage and flexibility behaviour of BSMs.

An increase in bitumen content decreases shrinkage and increases the flexibility of a BSM. An increase in cement on the other hand, increases shrinkage and decreases flexibility of BSMs. The correct combination of cement and bitumen in a BSM can thus provide a material with a flexibility that will increase the fatigue life of a material, while minimizing the shrinkage which could have damaging effects on pavement layers.

Using BSMs in a pavement structure has both environmental and economical benefits. Lower quality materials and recycled materials can successfully be used for BSMs, thereby reducing environmental impact. BSMs will also lengthen the fatigue lifetime of a pavement, which reduces maintenance costs.

10.2. Shrinkage behaviour

10.2.1. Conclusions

The methods designed during this project to determine the shrinkage of BSMs provide meaningful results, each for different reasons. Beam shrinkage provides a good indication of the shrinkage in the damaging shrinkage direction in practice, whereas cylindrical shrinkage provides a better indication of the shrinkage potential of the BSM itself. Even though the testing methods could be improved, the findings on the shrinkage behaviour of BSM can still be used and incorporated into the revised copy of the SAPDM.

The shrinkage experienced by BSMs is very small and no cracks occur during this shrinkage. The following useful findings were made in terms of the influence of additives on the shrinkage behaviour of BSMs:

- Increased bitumen contents reduce shrinkage.
- Increased cement contents increases shrinkage.
- Stabilising with foamed bitumen rather than bitumen emulsion reduces shrinkage, but only in combination with higher cement contents (2%).

- BSMs with cement : bitumen ratios that are close to 1 i.e. similar percentages of each additive, provide a relative consistent amount of shrinkage.

10.2.2. Recommendations

10.2.2.a) Testing procedure

The shrinkage behaviour of BSMs have successfully been obtained during this project, but more research could be done to improve the understanding of this material behaviour. It is therefore suggested that the method selected to test the shrinkage behaviour of a BSM is based on the usage of these measurements.

Shrinkage results obtained from both tests methods are insightful, but it is suggested that circumferential measurements is added to the shrinkage testing method for cylindrical specimens. These measurements could then be compared to the shrinkage of the beam specimens, which could possibly provide values closer to the actual shrinkage conditions in practice. If circumferential measurements are taken the volumetric shrinkage can be determined, providing a better understanding of the material shrinkage in comparison to the one-directional shrinkage.

If one of these methods is selected to test cemented materials, it is suggested that the cylindrical shrinkage method is used due to the fact that cemented materials can cause shrinkage cracks. If the beam method is used non-continuous shrinkage could be tested making measurements less accurate. Cylindrical testing would thus provide more continuous shrinkage measurements.

10.2.2.b) Further research

Recommendations to improve the shrinkage testing are as follows:

- Thoroughly investigate the material that will be stabilised with bitumen, to provide a better understanding of the influence of the material itself of the shrinkage behaviour.
- Conduct shrinkage tests on more than one material type.
- Test at least 6 specimens to obtain reliable results (test more specimens for improved accuracy).
- Test specimens with different amounts of foamed bitumen (not only 2.4%).

- A larger range of bitumen contents should be tested (lower and higher than bitumen contents used in this project).
- Test specimens with equal amounts of cement and bitumen content.

10.3. Flexibility behaviour

10.3.1. Conclusions

The monotonic beam tests provide a good indication of the flexibility behaviour of BSMs. The findings can be used and incorporated into the revised copy of the SAPDM.

The flexibility behaviour is dependant on both the cement and bitumen content and the best combination is needed to obtain the wanted material flexibility. The following findings were made on the influence of additives on the flexibility behaviour:

- Higher bitumen content provides lower stiffnesses, higher stain-at-break values and higher dissipated energy, therefore **increasing flexibility**.
- Higher cement content provides higher stiffnesses, lower strain-at-break values and lower dissipated energy, therefore **decreasing flexibility**.
- Stabilising with foamed bitumen rather than bitumen emulsion provides higher flexibility, but only when combined with high cement contents (2%).

10.3.2. Recommendations

10.3.2.a) Testing procedure

Four-point-beam tests should be run parallel to monotonic beam tests to compare the strain-at-break results for the specific BSM.

10.3.2.b) Further research

Recommendations to improve the strain-at-break research are as follows:

- Test a greater number of beam specimens.
- Conduct strain-at-break tests on more than one material type.
- Calculate the dissipated energy using both LVDT readings and not just the one LVDT (as done during this project).

References

(1987). *Technical recommendations for highways draft TMH8*. Pretoria: Department of Transport.

(2007, January). Retrieved September 19, 2012, from http://eprints.ums.ac.id/612/1/(3)A.pdf

Araya, A. A. (2011). *Characterization of Unbound Granular Materials for Pavements*. Netherlands: TU Delft.

Asphalt Academy. (2009). Technical Guideline: Bitumen Stabilised Materials (TG 2). Asphalt Academy.

(n.d.). Asphalt Pavements .

Chapter 8, Failure. (n.d.). *Introduction to Materials Science.*

Collings, D. (2009). Understanding Bitumen Stabilised Materials. *13th International Flexible Pavements Conference: "Pavements for Today"* (pp. 5-15). South Africa: Loudon International.

Collings, D., & Jenkins, K. (n.d.). The long-term behaviour of Bitumen Stabilised Materials (BSMs). *10th Conference on Asphalt Pavements for Southern Africa*, (pp. 1-14).

Crawford, R. (1998). Mechanical behaviour of plastics. In R.J.Crawford, *Plastics Engineering* (pp. 84-103). Burlington.

De Wet, D. (2014). *Soil composition and engineering properties*.

DFID. (n.d.). *Literature Review- Stabilised Sub-Bases for Heavily Trafficked Roads*. (Department For International Development): Transport Research Laboratory, UK.

Ebels, L.-J. (2008). *Characterisation of material properties and behaviour of could bituminous mixtures for road pavements.* Stellenbosch: Stellenbosch University.

Epps, A., Harvey, J. T., Kim, Y. R., & Roque, R. (n.d.). Structural Requirements of Bituminous Paving Mixtures.

Fattah, M. Y. (2013). Effect of salt content on total and matric suction of unsaturated soils. *European Scientific Journal (Vol 9).*

Ignacio Pérez, L. M. (2013). Mechanical properties and behaviour of in-situ materials which are stabilised with bitumen emulsion. *Road Materials and Pavement Design, Volume 14, 221 - 238.*

J. Rogers, R. O. (n.d.).

Jenkins, K. (2012). *Pavement engineering.* Stellenbosch: Stellenbosch University.

Jones, J. M. (2007). Characterization of foamed-bitumen stabilisation, Volume 8. *International Journal of Pavement Engineering,111-122.*

K. J. Jenkins, F. M. (2007). Foamed bitumen mixes = shear performance. *International Journal of Pavement Engineering, 87 - 98.*

Kailas, P. V. (n.d.). Material Science(Chapter 8. Failure). India: Dept. of Mechanical Engineering, Indian Institute of Science.

Kaspar, H., & Potgieter, J. H. (1999). Hydration of cement. *S. Afr. Tydskr. Chem., 104 - 109.*

Kelly. (2010). Rheological Models. In *Solid Mechanics Part I.*

Lee, Y. K. (2011). *Influence of Reclaimed Asphalt Pavement Temperature.* Retrieved August 27, 2012, from ascelibrary.org by University of Stellenbosch-Period

Lent, D. v. (2008). *Aggregate characterisation in relation to bitumen-aggregate adhesion*. Delft University of Technology.

Li, L., Liu, J., & Zhang, X. (2010). Resilient Modulus Characterization of Alaskan Granular Base Materials. Alaska: Alaska University Transportation Center.

Long, F., & Theyse, H. (2004). Mechanistic- empirical structural design models for foamed and emulsified bitumen treated materials. *8th Conference on Asphalt Pavements for Southern Africa (CAPSA'04).* Document Transformation Technologies cc.

Lowery, R. W. (2008). *Soil Compaction*. Madison: University of Wisconsin.

Martin Kendall, B. B. (n.d.). *Foamed Bitumen Stabilisation.*

Metso. (2012). *Metso*. Retrieved September 20, 2012, from Roading aggregates: http://www.metso.com/miningandconstruction/mm_crush.nsf/WebWID/WTB-050222-2256F-894BC?OpenDocument

Molenaar, A. (2013). *Cohesive and non-cohesive soils and unbound granular materials for bases and sub-bases in roads.* The institute for transport technology.

Murillo, C., Caicedo, B., Hoyos, L., Colmenares, J. E., & Berdugo, I. R. (2013). *Advances in Unsaturated Soils.* CRC Press.

National Geographic. (n.d.). Folds, faults and geological maps.

Nelson, P. S. (2012, February 10). Deformation of Rock. Tulane.

Nwando, A. T. (2014). *Flexibility and Performance Properties of Bitumen Stabilised Materials.* University of Stellenbosch.

R.Luhr, W. S. (2004). Control of reflective cracking in cement stabilised pavements. *5th International RILEM Conference.* France.

Roylance, D. (2001). *Stress-strain curves*. Cambridge: Department of Materials Science and Engineering.

Rubulusa. (2011). *The use of gypsum blocks to measure suction pressure in relation to grading of a crushed base material.* University of Stellenbosch.

SANRAL. (2013). *South African Pavement engineering Manual.* South African National Roads Agency.

Selverstone, J. (n.d.). *Exploring Earth.* Retrieved July 2014, from How do rocks respond to stress:https://www.classzone.com/books/earth_science/terc/content/investigations/es1102/es1102page01.cfm

Standard test procedures manual. (n.d.). Saskatchewan Highways and transportation.

Sudjianto, A. T., Suryolelono, K. B., Rifa'i, A., & Mochtar, I. B. (n.d.). The Effect of Water Content Change and Variation Suction in Behavior Swelling of Expansive Soil. *International Journal of Civil & Environmental Engineering, Vol: 11 No: 03.*

Sweere, G. T. (1990). *Unbound Granular Bases for Roads.* Technical University of Delft.

The Constructor. (2012). *The Constructor Civil Engineering home.* Retrieved July 2014, from Flexible pavement composition and structure: file:///H:/2014/FLEXIBLE%20PAVEMENT%20COMPOSITION%20AND%20STRUCTURE.htm

Theyse, H. (2010). *Roads pavement forum (SAPDM).* KwaZulu Natal.

Theyse, D. H. (July 2012). *SAPDM: Phase 2 laboratory test instructions.* Pavement modeling corporation.

TSHIVHASE, K. L. (2008). Moisture sensitivity of selected foamed bitumen-treated materials. Tshwane University of Technology.

University of Tolendo. (n.d.). *Soil suction.* Retrieved 11 9, 2013, from The University of Tolendo: www.eng.utoledo.edu/civil/.../unsaturated%20soils/unsat4.pdf

Wankhade, S. R. (2014). Improvement of Swelling-shrinkage Behaviour of Expansive Soil Using EPS Beads. *International Journal of Applied Engineering Research ., 224-228.*

Whiteoak, D. (1990). *The Shell Bitumen Handbook.* Chertsy: Shell bitumen U.K.

Wirtgen GmbH. (2012). *Wirtgen Cold Recycling Technology.* Germany: Wirtgen GmbH.

APPENDIX A – Mould designs

APPENDIX A - Mould designs

A.1. Beam moulds

The beam mould design was based on the beam mould design of Alex Mbagara (Stellenbosch University) which allows for easy, quick removal of beam specimens without causing damage to the specimens. A beam specimen with the following dimensions can be compacted using this mould:

- Length: 270 mm
- Width: 75 mm
- Height: 75 mm

The beam mould exists of a base plate (Figure A.2) and two angle plates (Figure A.3) made from galvanised steel to prevent rust, which could lead to an uneven surface. The mould also has two small side plates (Figure A.4) made from stainless steel.

Three of these moulds (Figure A.1) have been made to fasten up the compaction and testing process.

A.2.Cylindrical moulds

A cylindrical mould (Figure A.5) has been designed for easy quick removal of the specimen without causing harm to the specimen itself.

The cylindrical mould has been designed with hinges (note that Figure A.5 show the mould with bolts and not hinges) to remove the specimen quick and easily. The R35 material has some plastic properties and caused suction between the specimen and mould during de-moulding, causing major damage to the test specimens. A plastic sleeve was therefore used to line the inside of the mould to remove the specimen without any damage; however this slightly decreased the diameter of the cylinder. The scarifier used to roughen the material between layers loosely fitted into the cylindrical mould.

Figure A.8 provides the rigid framework that has been designed to test the cylindrical specimens with. It has been designed with a space allocated for both a LVDT and Dial gauge measuring tools (Figure A.8 – A.15 and Table A.1).

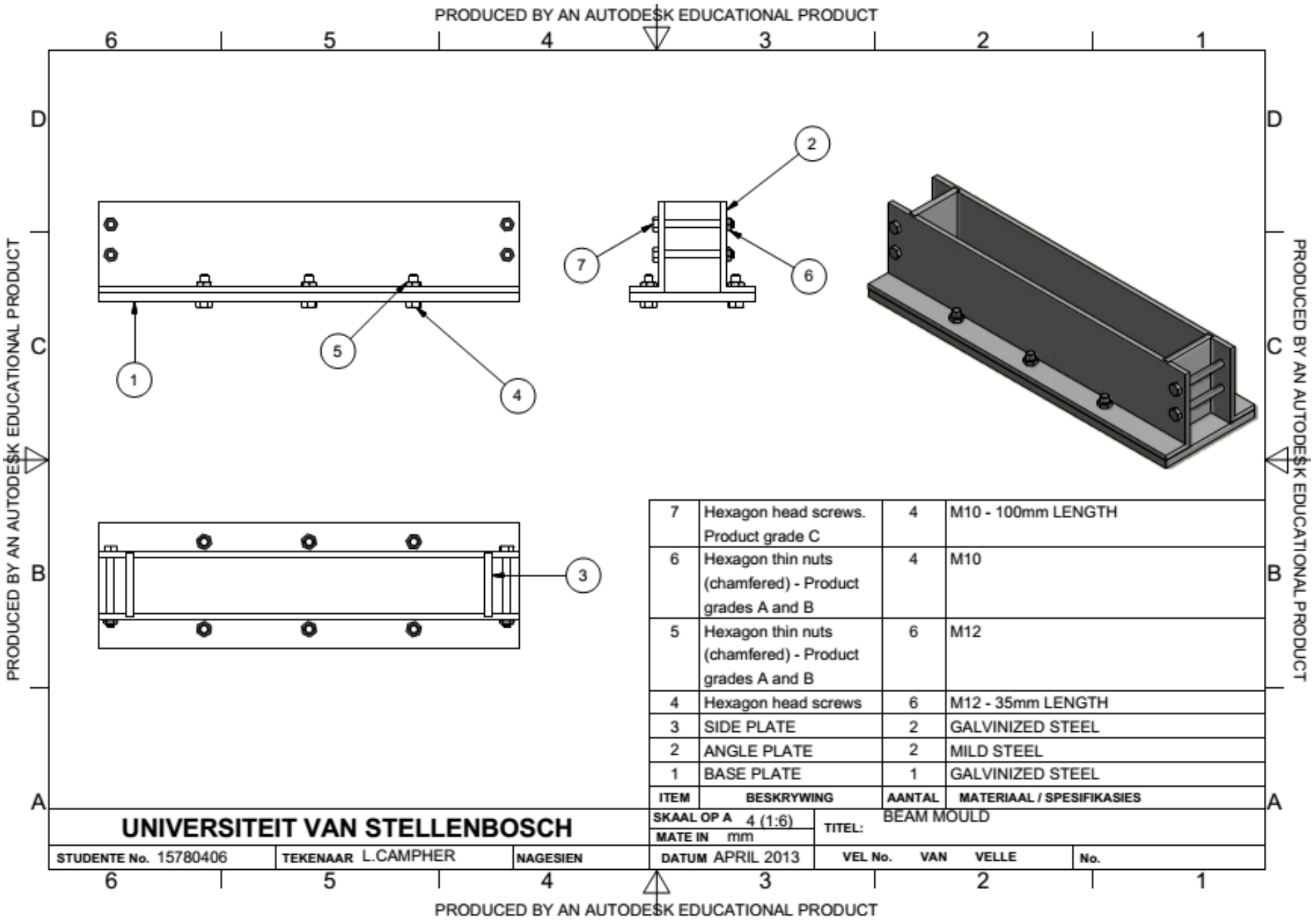


Figure A.1 Beam mould

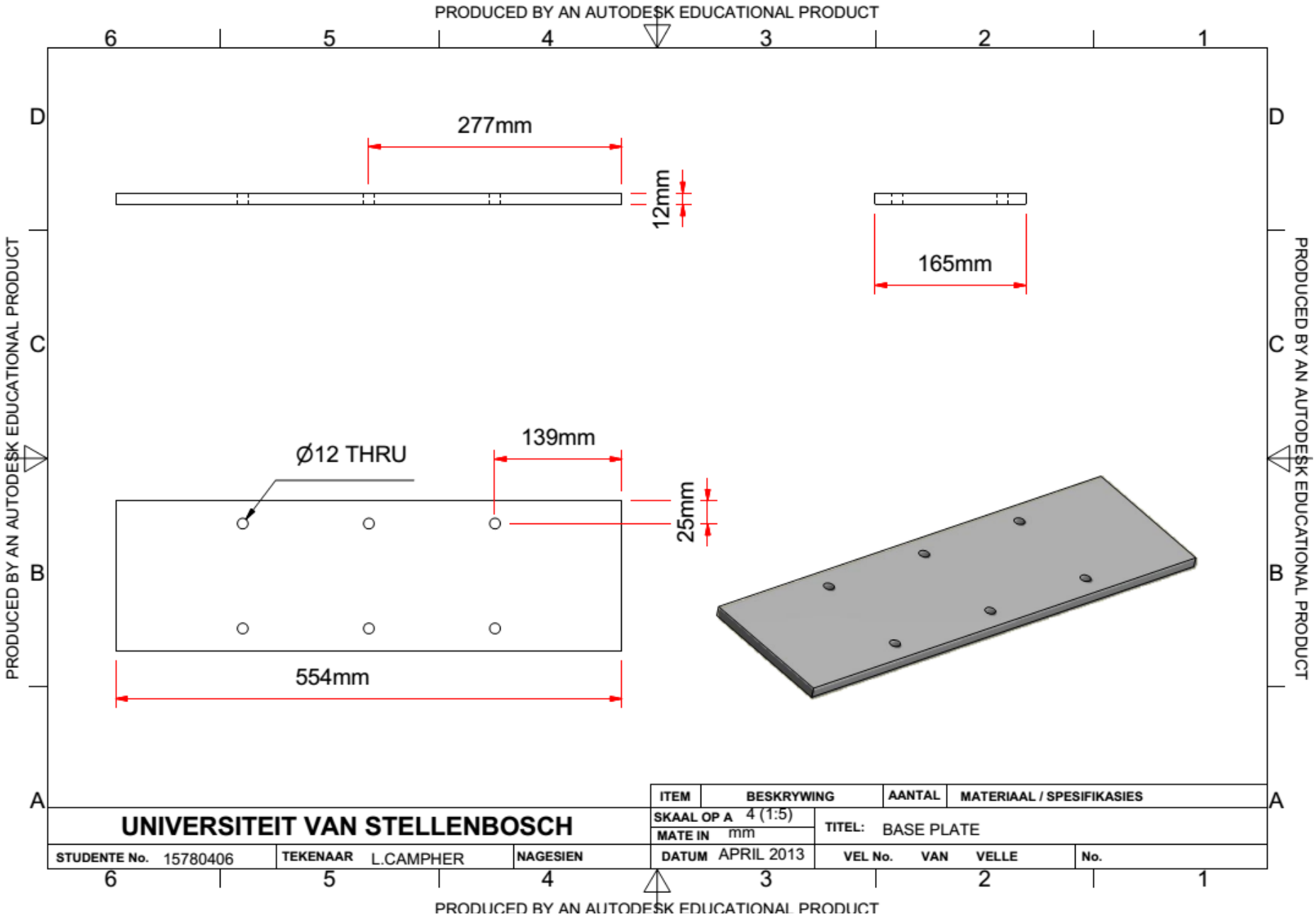


Figure A.2 Base plate of the beam mould

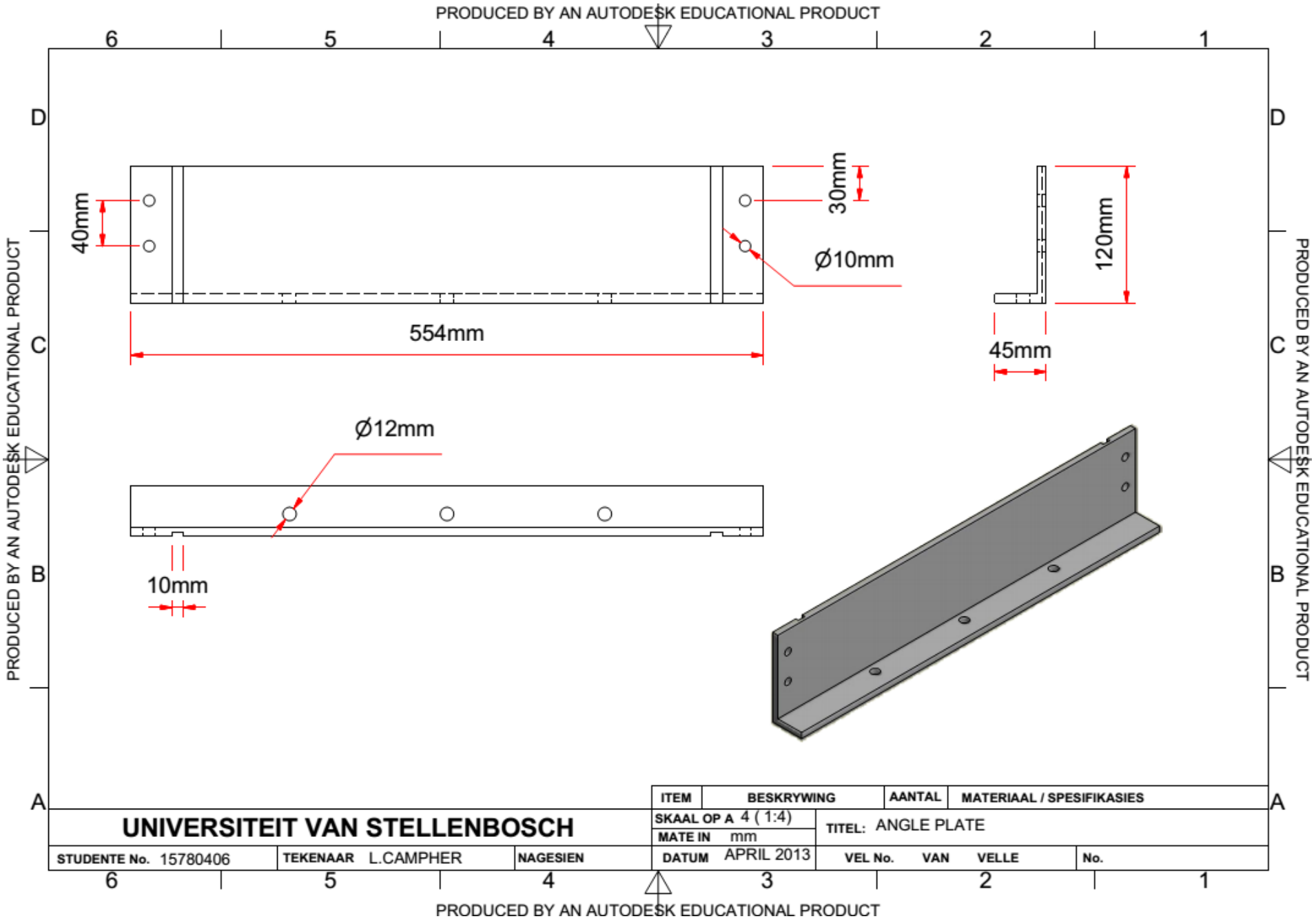


Figure A.3 Angle plate of beam mould

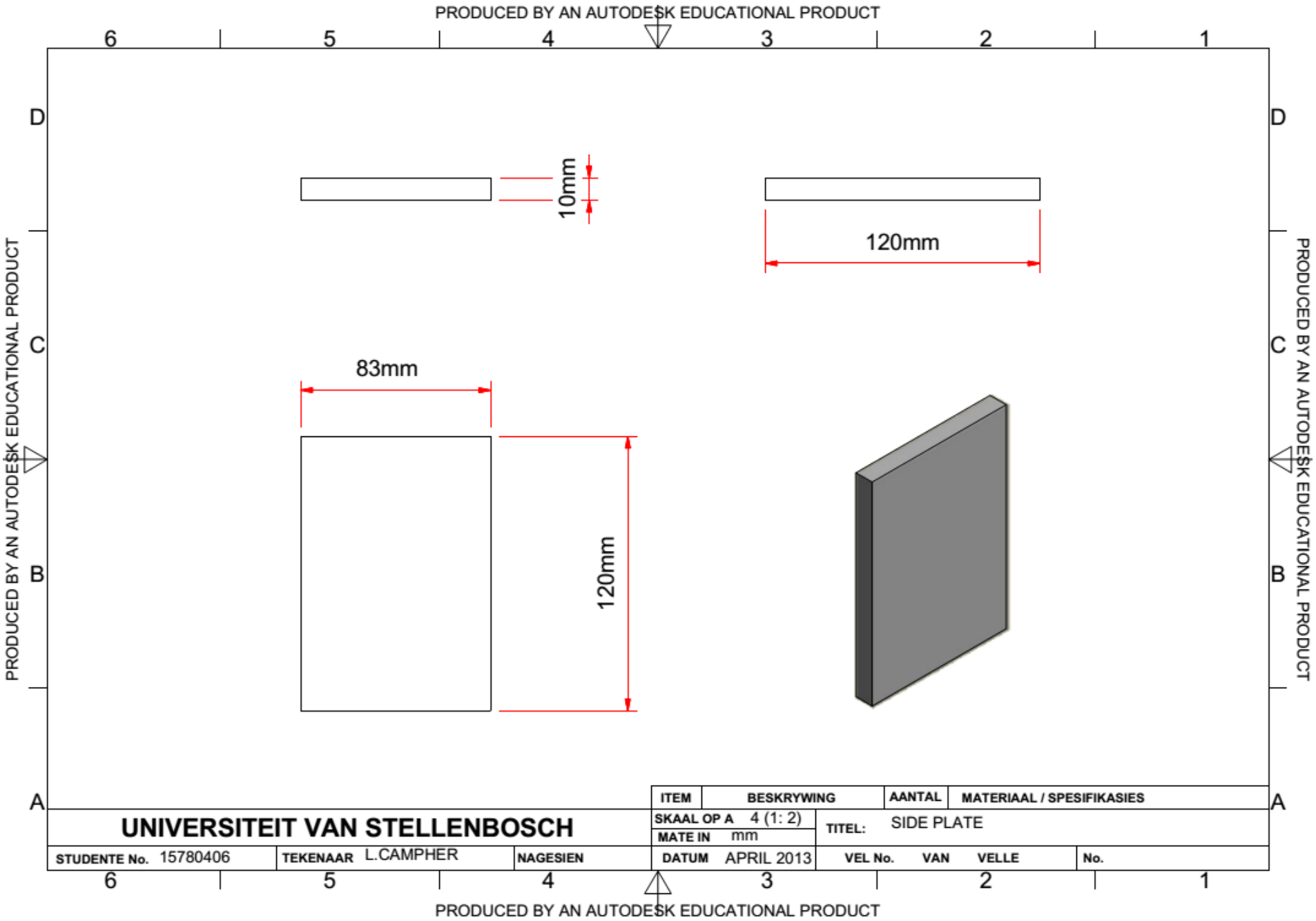


Figure A.4 Small side plate of beam mould

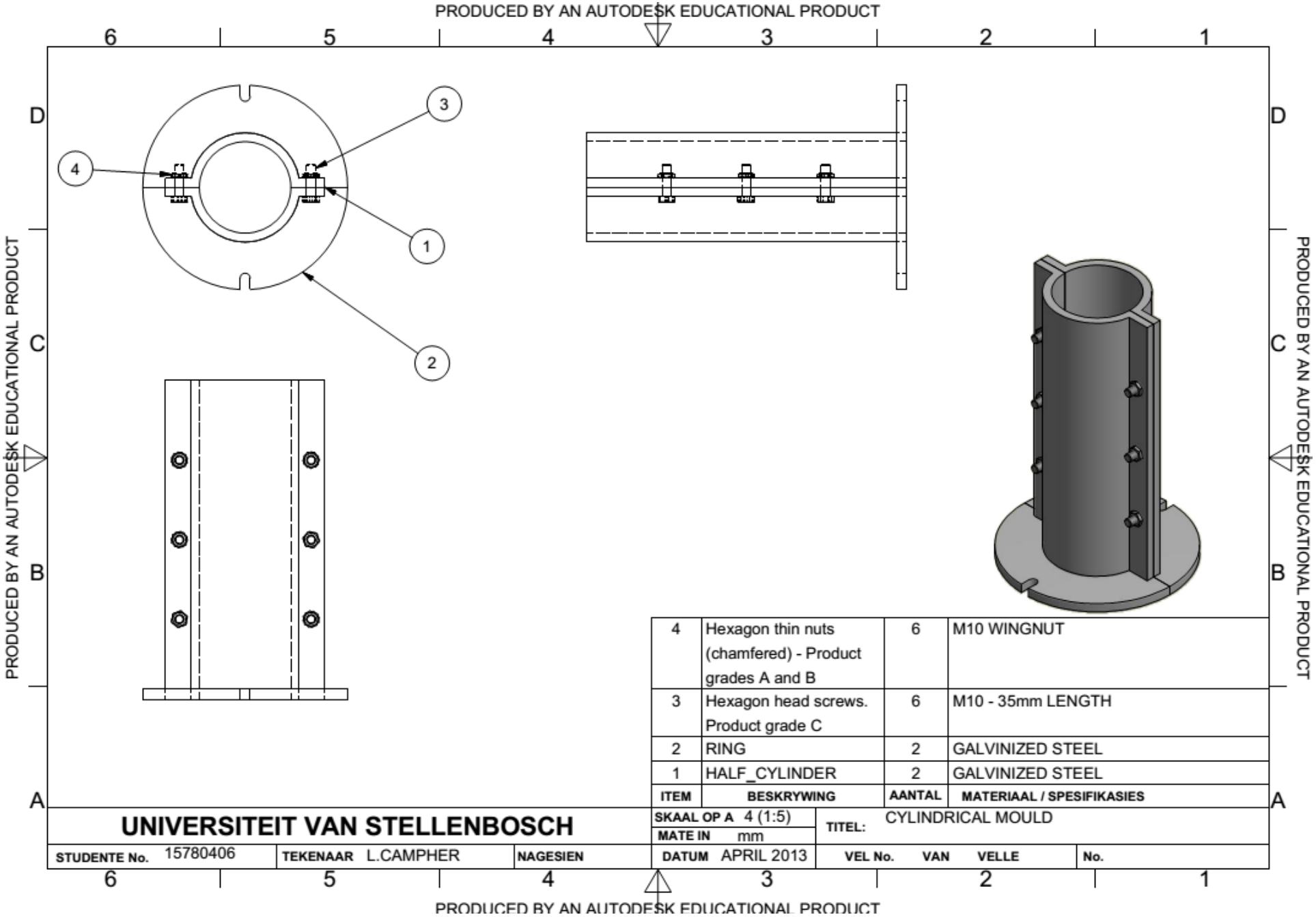


Figure A.5 Cylindrical specimen

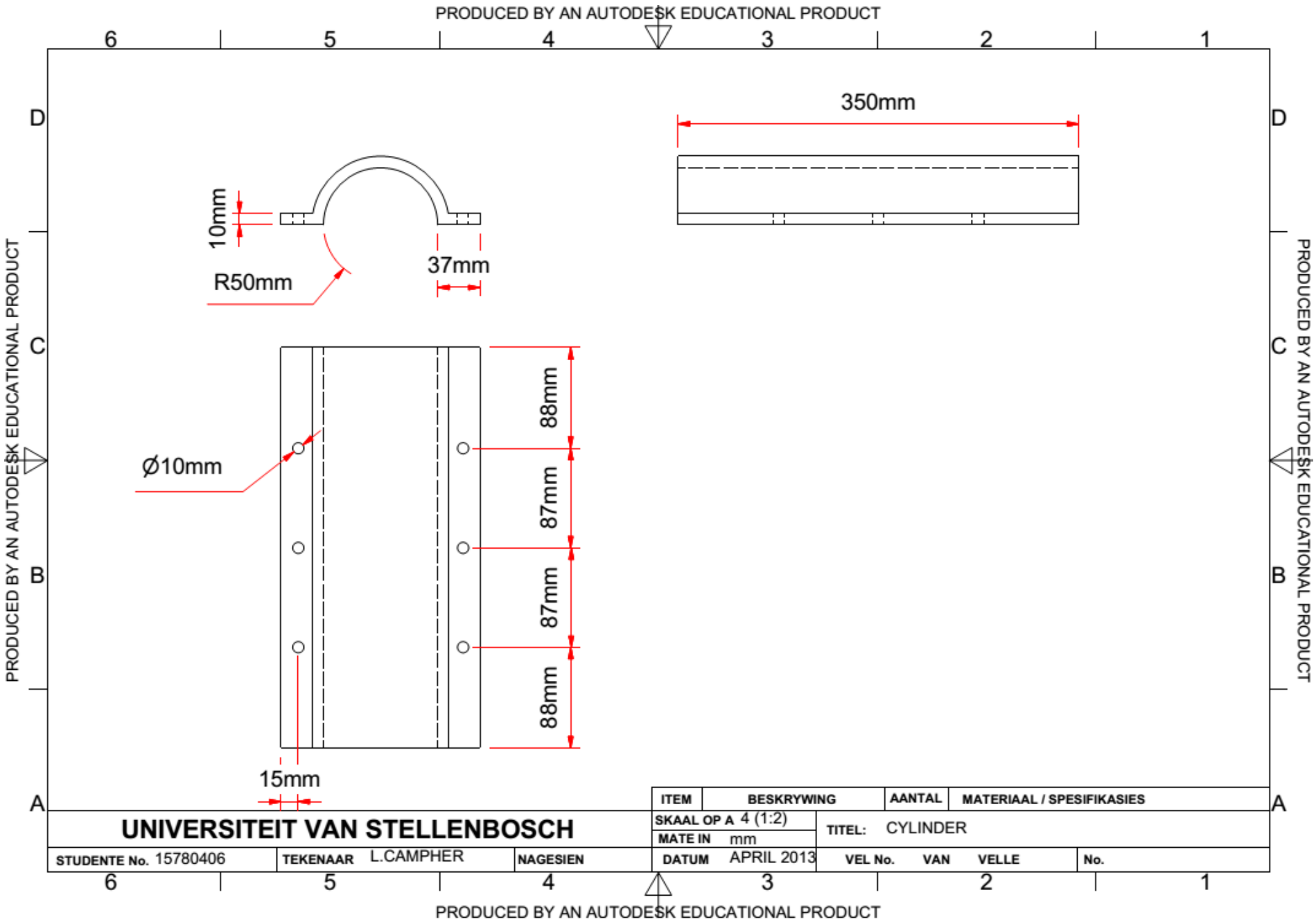


Figure A.6 Cylinder side

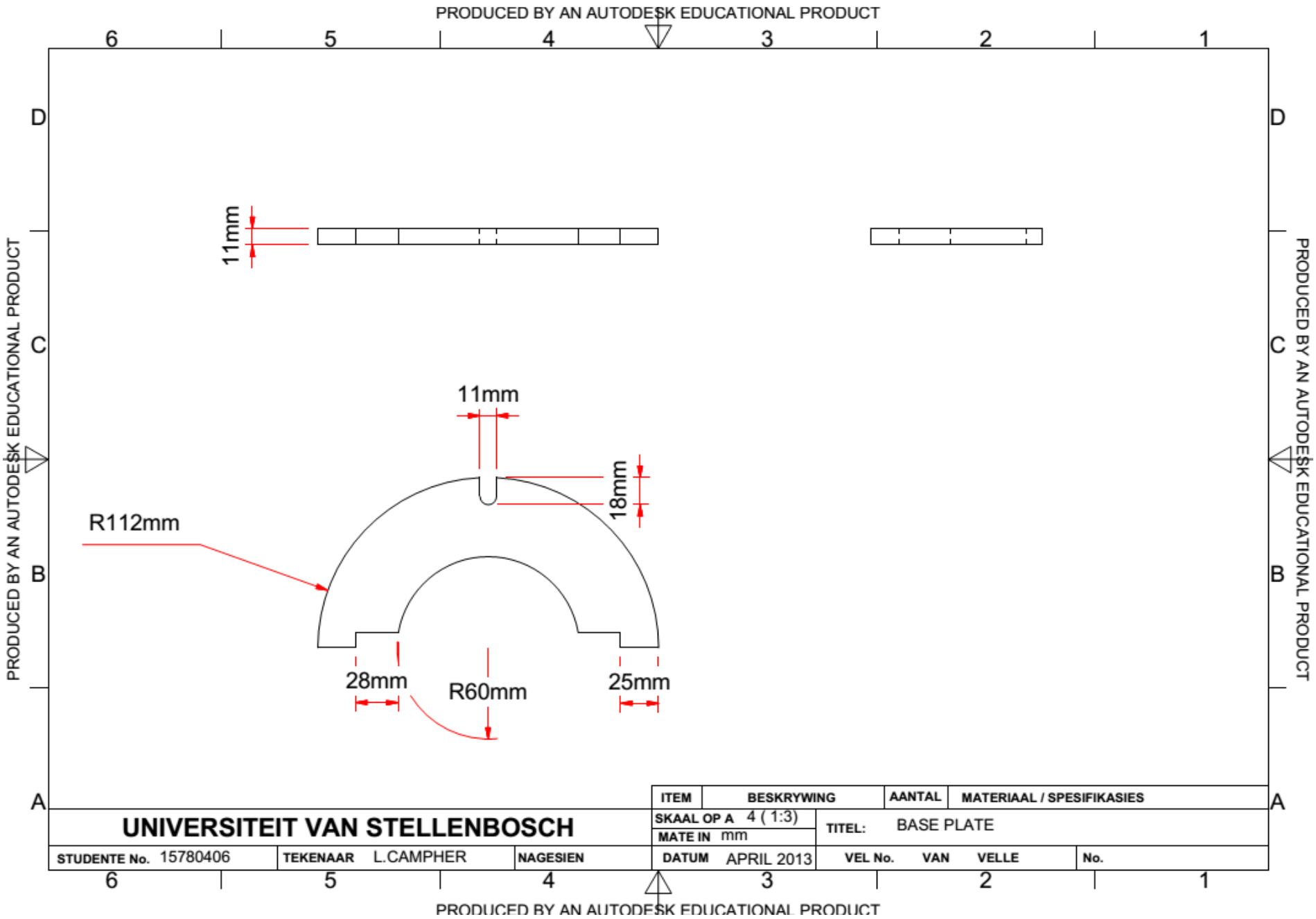


Figure A.7 Base plate of cylindrical mould

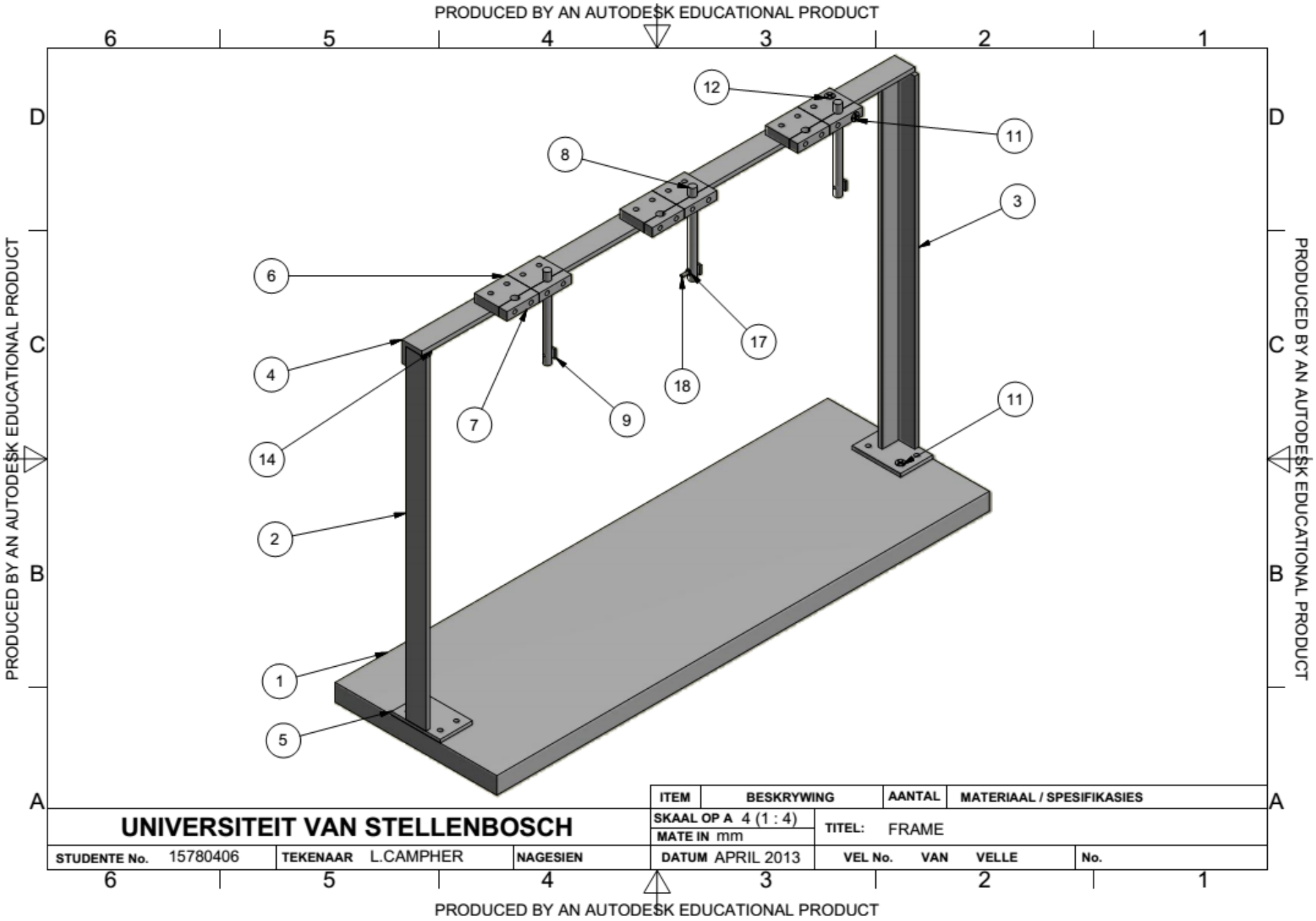
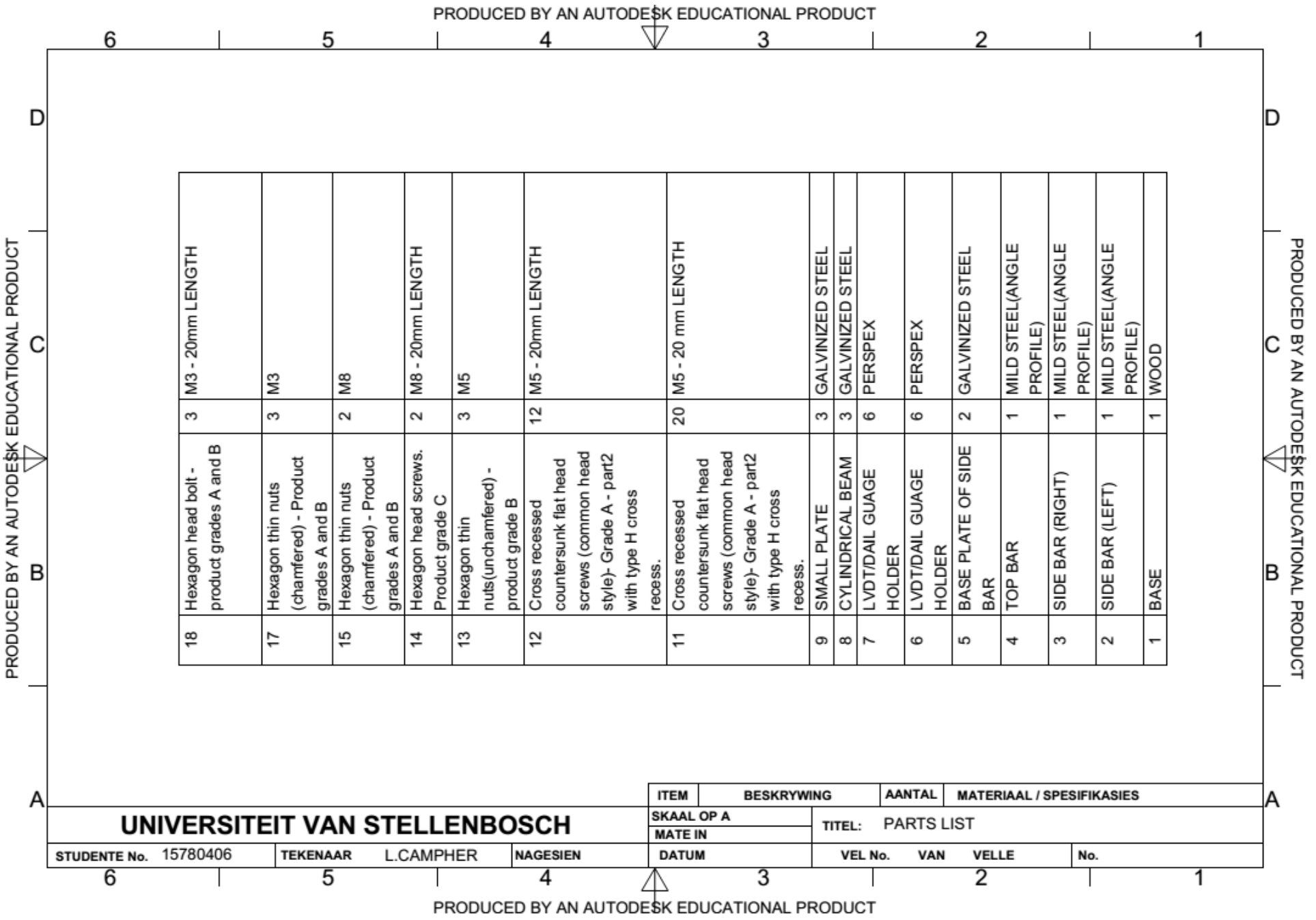


Figure A.8 Rigid framework for testing cylindrical specimens



ITEM	BESKRYWING	AANTAL	MATERIAAL / SPESIFIKASIES
18	Hexagon head bolt - product grades A and B	3	M3 - 20mm LENGTH
17	Hexagon thin nuts (chamfered) - Product grades A and B	3	M3
15	Hexagon thin nuts (chamfered) - Product grades A and B	2	M8
14	Hexagon head screws. Product grade C	2	M8 - 20mm LENGTH
13	Hexagon thin nuts(unchamfered) - product grade B	3	M5
12	Cross recessed countersunk flat head screws (common head style)- Grade A - part2 with type H cross recess.	12	M5 - 20mm LENGTH
11	Cross recessed countersunk flat head screws (common head style)- Grade A - part2 with type H cross recess.	20	M5 - 20 mm LENGTH
9	SMALL PLATE	3	GALVINIZED STEEL
8	CYLINDRICAL BEAM	3	GALVINIZED STEEL
7	LVDT/DAIL GUAGE HOLDER	6	PERSPEX
6	LVDT/DAIL GUAGE HOLDER	6	PERSPEX
5	BASE PLATE OF SIDE BAR	2	GALVINIZED STEEL
4	TOP BAR	1	MILD STEEL(ANGLE PROFILE)
3	SIDE BAR (RIGHT)	1	MILD STEEL(ANGLE PROFILE)
2	SIDE BAR (LEFT)	1	MILD STEEL(ANGLE PROFILE)
1	BASE	1	WOOD

Table A. 1 Parts list for rigid testing frame

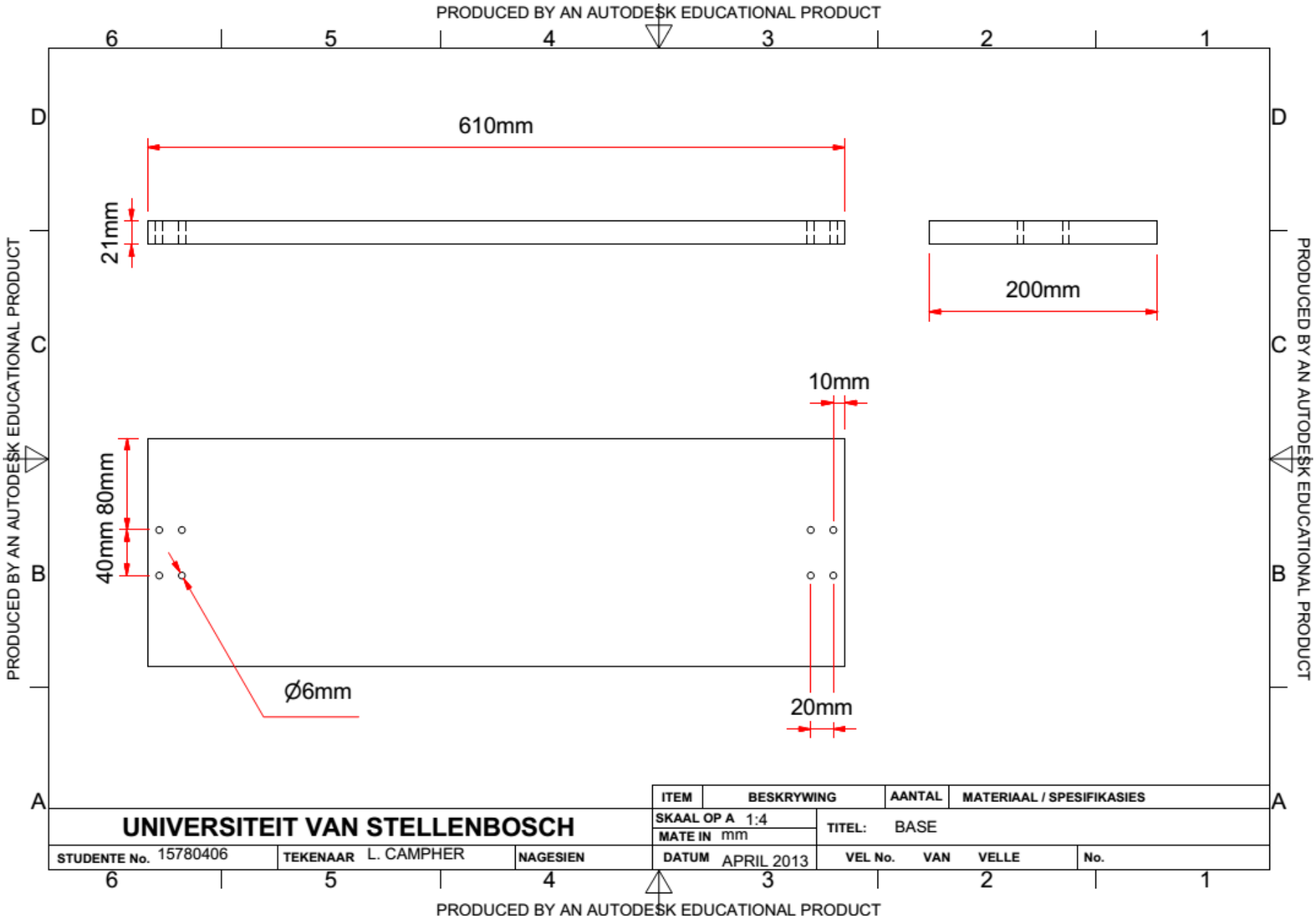


Figure A.9 base plate of rigid frame

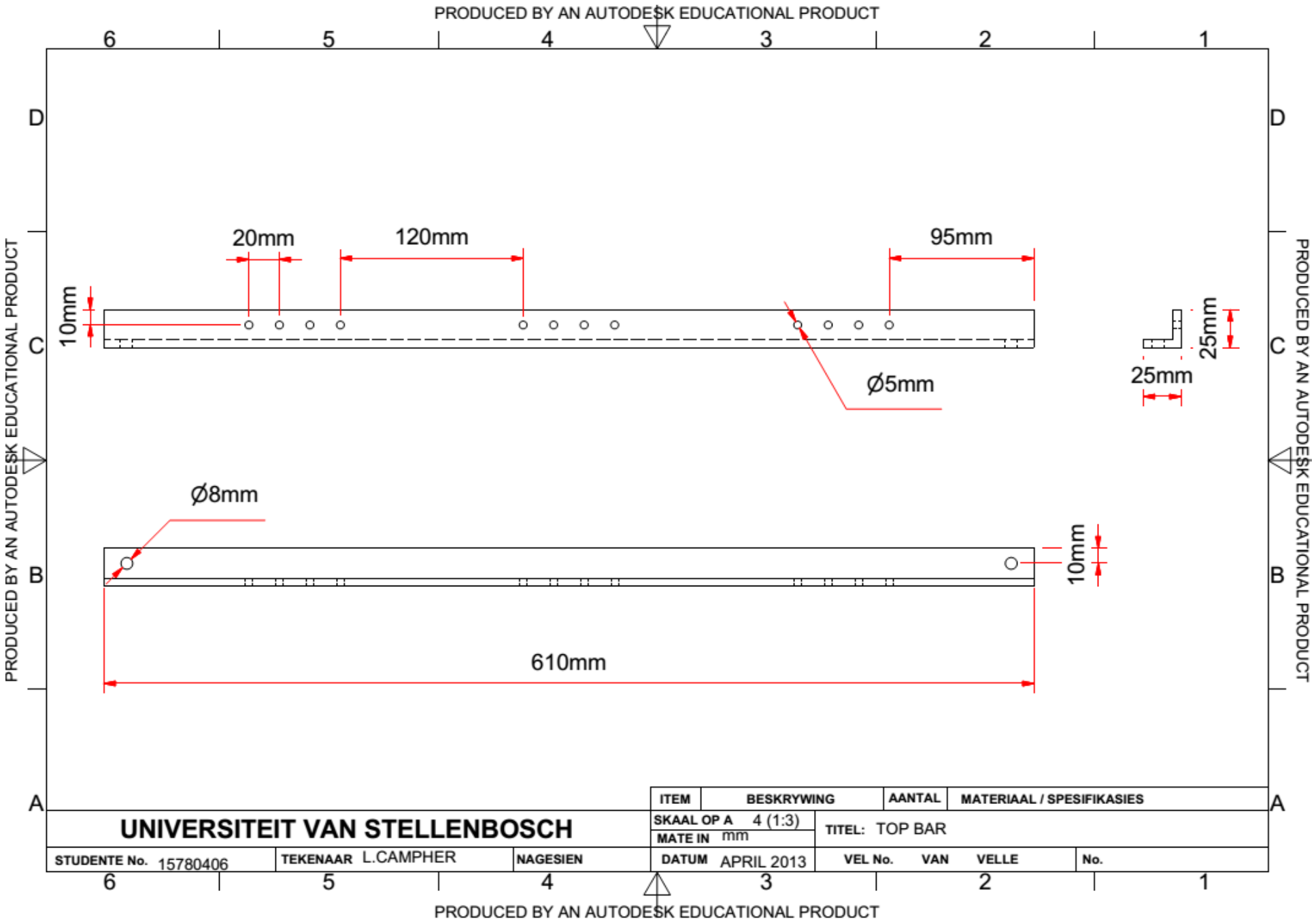


Figure A.10 Top bar of rigid testing frame.

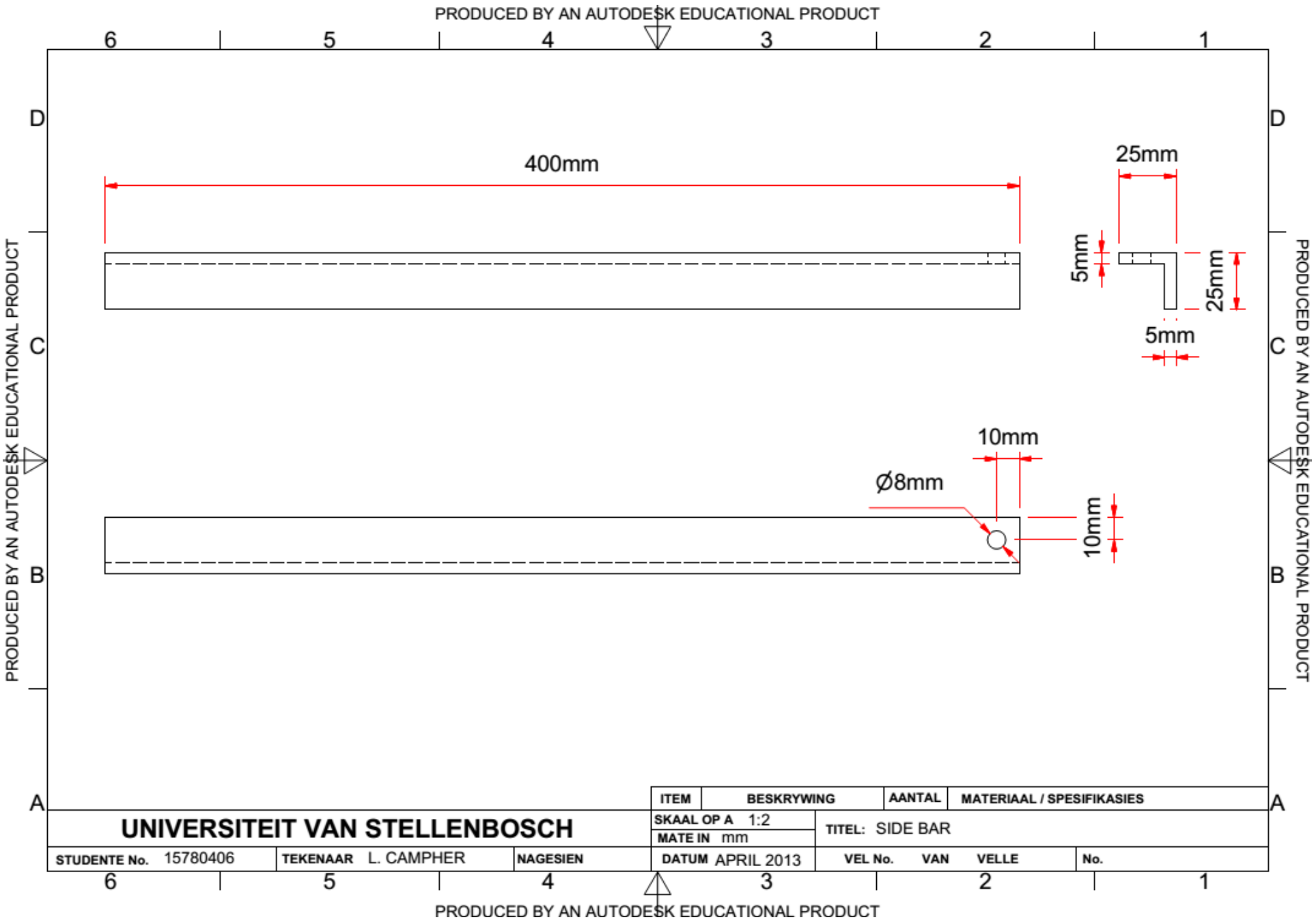


Figure A.11 Side bar of the rigid testing frame.

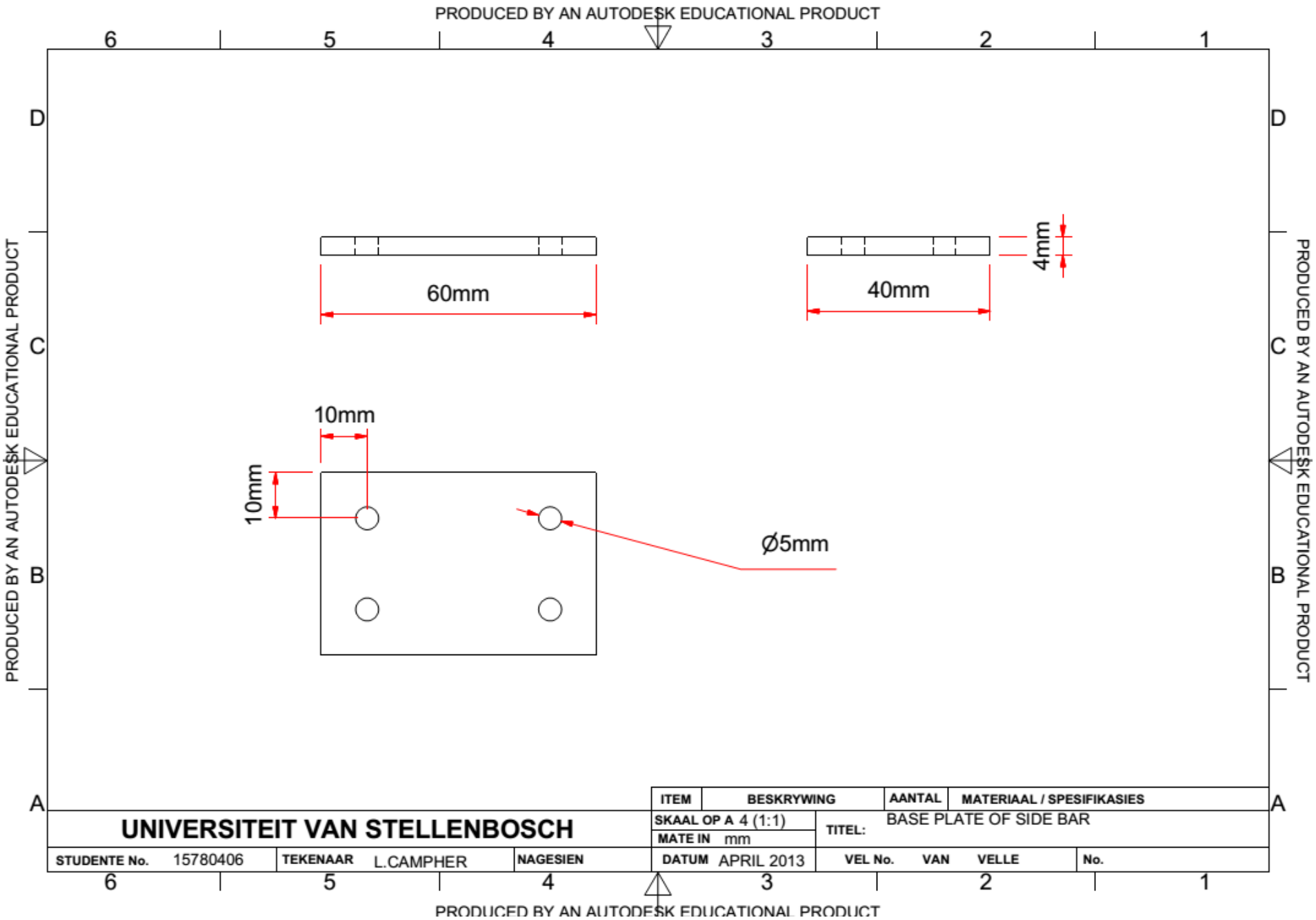


Figure A.12 Base plate of side bar, for rigid testing frame.

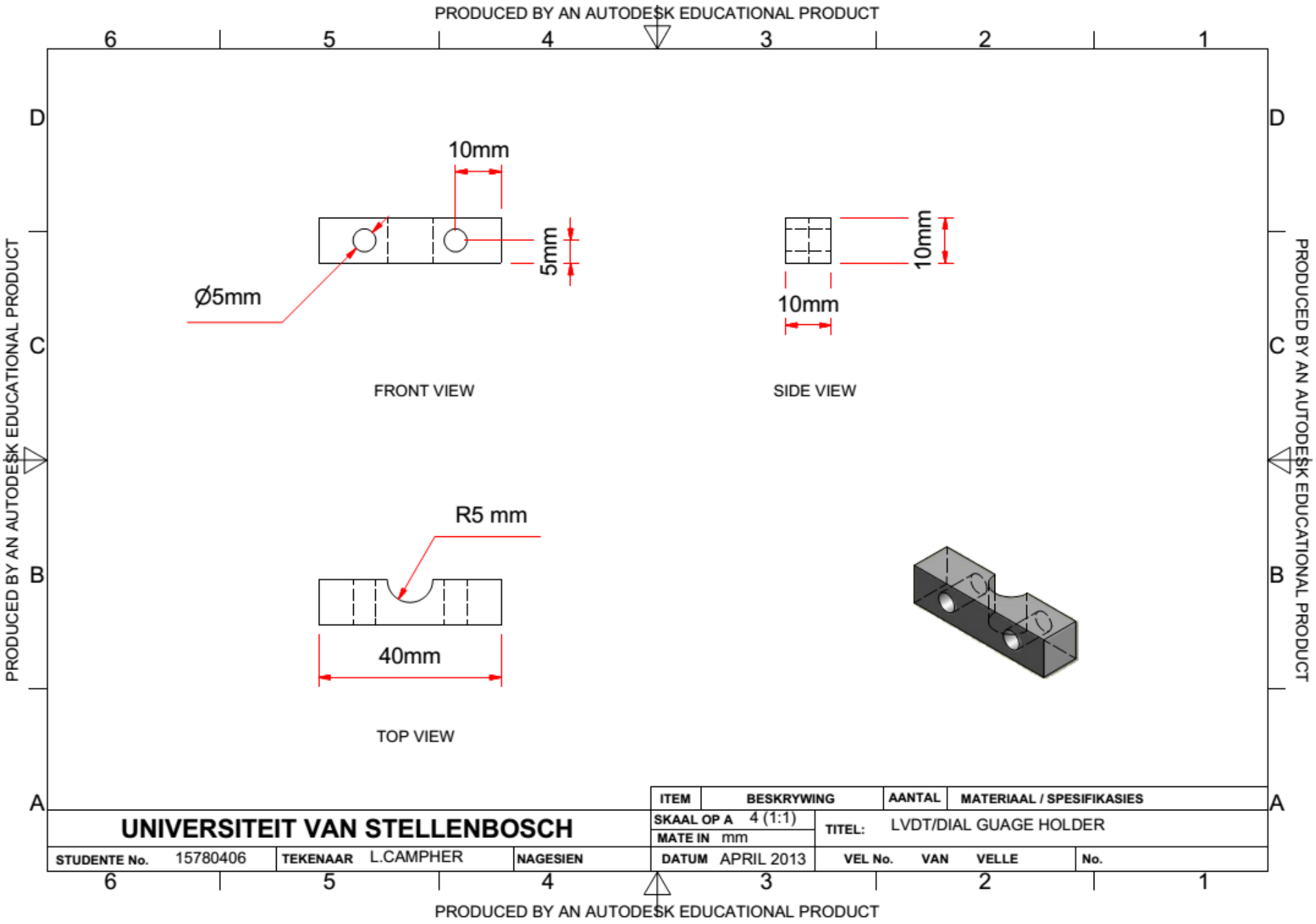


Figure A.13 LVDT/Dial gauge holder (Part 1) used on rigid testing frame

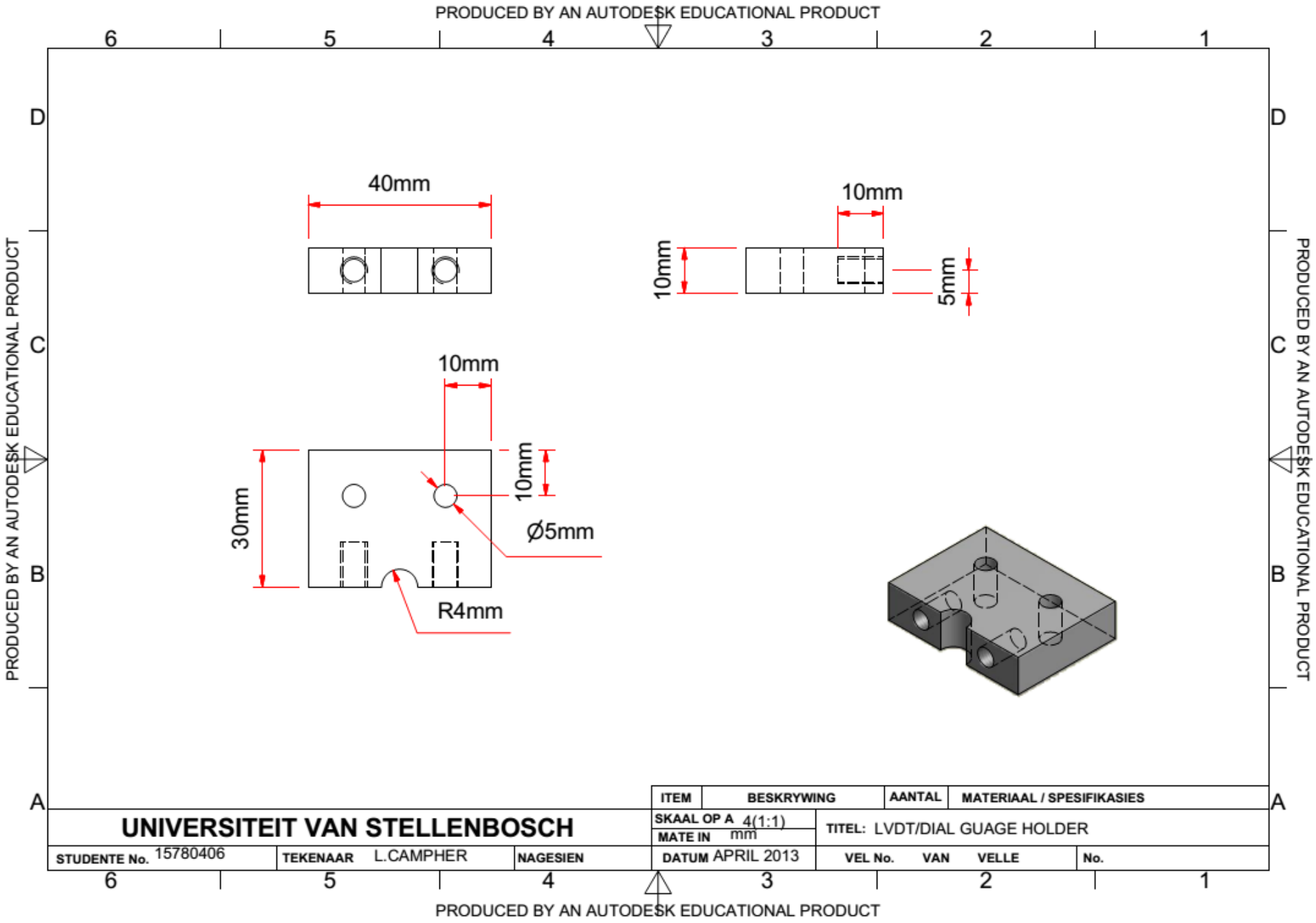


Figure A.14 LVDT/Dial gauge holder (Part 2) used on rigid testing frame.

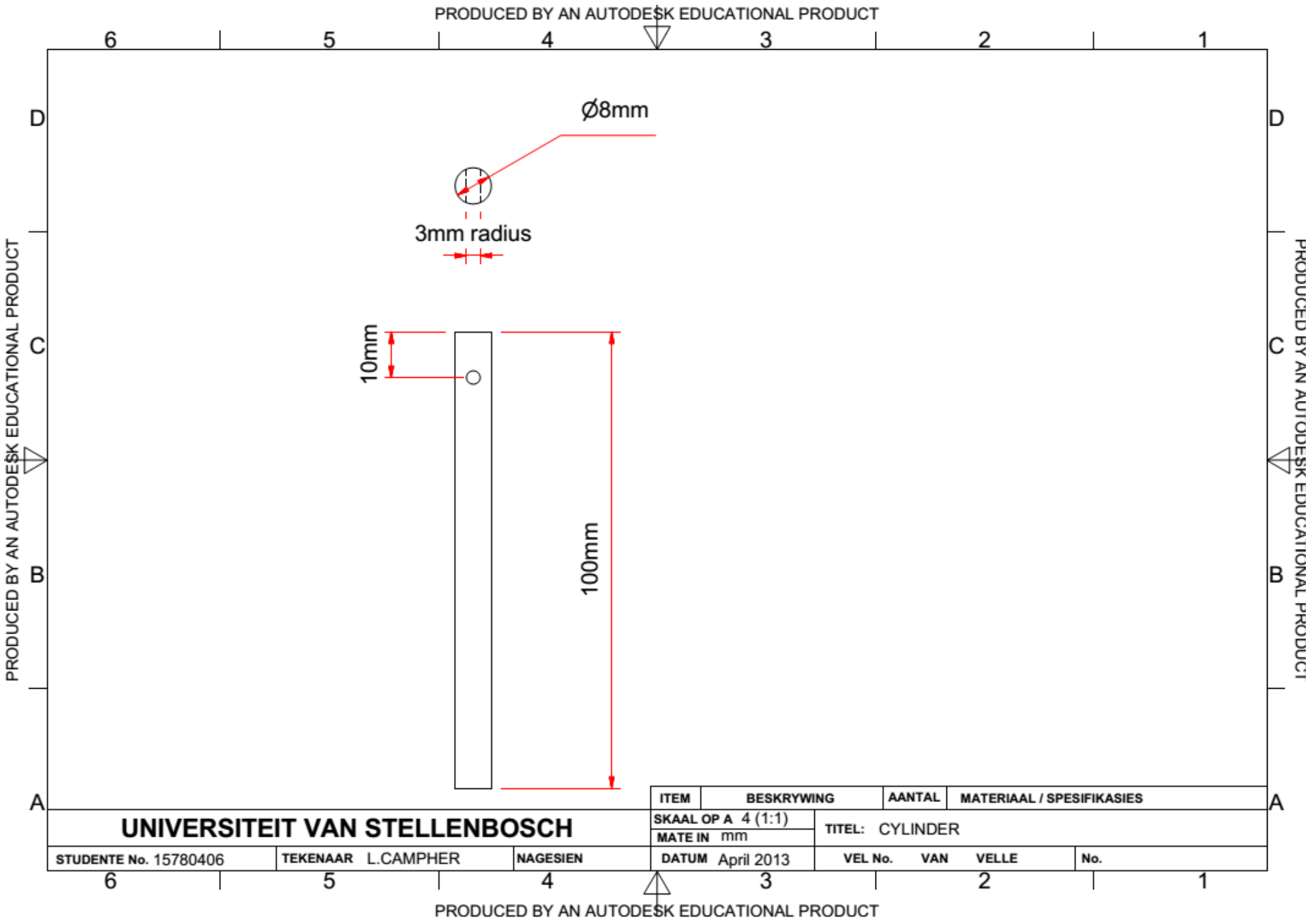


Figure A.15 Cylindrical extension for dial gauge, used in the rigid testing frame.

APPENDIX B – Laboratory shrinkage results

APPENDIX B - Laboratory shrinkage results

B.1. Data from R35 material

R35 material characteristics determined in phase one:

Optimum moisture content:	11.2	%
Maximum dry density of R35 material	2100	kg/m^3

Design beam dimensions determined during phase two:

Height:	75	mm
Width:	75	mm
Length:	470	mm

Design cylindrical dimensions determined during phase two:

Height:	300	mm
Diameter:	100	mm

Emultion 60/40 (bitumen/water)
2.4% / 0.6% = 4% bitumen has to be added to add 2.4% pure bitumen.
(4% * 40%)/100 = 1.6% water will be added to the mixture when adding 4% emultion.

B.2. Beam specimens:

Specimens containing 0% bitumen, 0% Cement:

Aggregate moiture:

Bowl weight:	228	g
Bowl + agg.:	787.7	g
After 24 h:	768	g
	2.50095	% moisture in agg.

Mixture quantities:

Material:	17000	g
Bitumen	0	g
Cement:	0	g
Water:	1478.84	g

=> 0% Bitumen
=> 0% Cement

Moisture after mixing:

Bowl43 weight:	228	g
Bowl43 + agg.:	806	g
After 24 h:	744	g
	7.69231	% moisture in specimen

Beam specimen name	Beam dimensions: Hight (mm): Height (mm)	Hight (mm): Average Height (mm)	Width (mm)	Length (mm)	Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
Beam 1 - B6	72 70	71.75	75	470	5.6572	0.002529	7.69231	2236.77	2077
	71 70								
	70 69								**98.90 %** of MOD density
	73 73								
	73 72								
	74 74								
Beam 2 - C6	66 76	73.65833	75	468	5.7196	0.002585	7.69231	2212.26	2054.2
	67 75								
	67 76								**97.82 %** of MOD density
	73 70								
	74 68								
	74 67								

APPENDIX B - Laboratory shrinkage results

Beam 3 - P6	74	72	72.41667	75	469	5.7212	0.002547	7.69231	2246.02	2085.6
	71	73								
	73	71								**99.31 %** of MOD density
	70	73								
	72	72								
	73	75								

Table B 1) **Beam specimens properties - 0% bitumen, 0% Cement**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.26302	2.75492	4.01794	0
0.5	1.21725	2.77367	3.99092	-0.02702
1.5	1.15987	2.81007	3.96994	-0.048
3	1.04522	2.79724	3.84246	-0.17548
6	0.89708	2.74981	3.64689	-0.37105
12	0.72894	2.66404	3.39298	-0.62496
24	0.5617	2.56593	3.12763	-0.89031
48	0.39553	2.54277	2.9383	-1.07964
72	0.32982	2.55952	2.88934	-1.1286

Table B 2) **Beam shrinkage results - Beam B6**

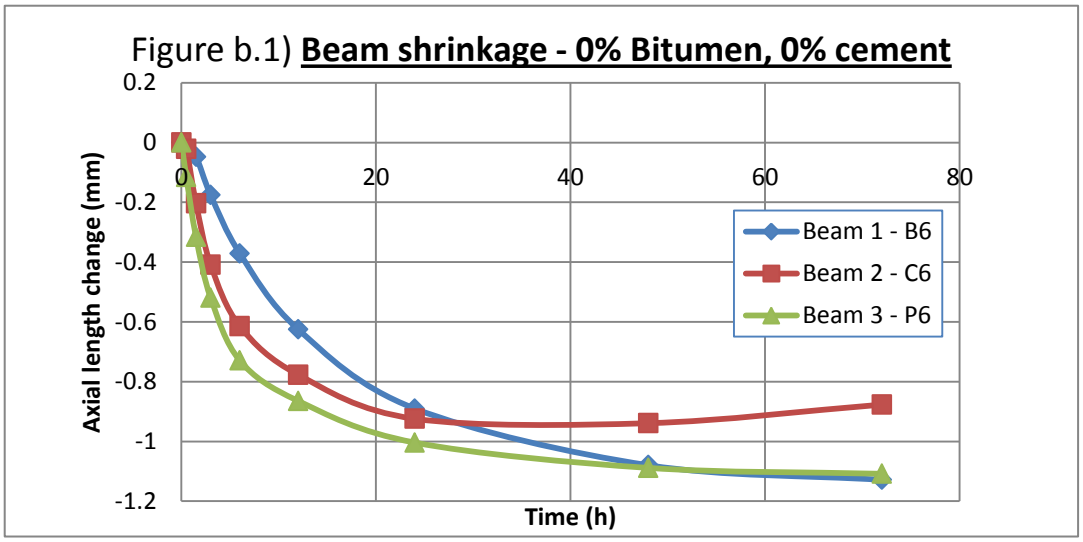
Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.35215	1.49455	2.8467	0
0.5	1.3321	1.49306	2.82516	-0.02154
1.5	1.24322	1.39978	2.643	-0.2037
3	1.16415	1.27389	2.43804	-0.40866
6	1.10358	1.12877	2.23235	-0.61435
12	1.05295	1.017	2.06995	-0.77675
24	0.97181	0.95058	1.92239	-0.92431
48	0.94595	0.96169	1.90764	-0.93906
72	0.96554	1.00431	1.96985	-0.87685

Table B 3) **Beam shrinkage results - Beam C6**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.23932	2.13984	4.37916	0
0.5	2.19329	2.07042	4.26371	-0.11545
1.5	2.12583	1.93737	4.0632	-0.31596
3	2.06262	1.79722	3.85984	-0.51932
6	1.98391	1.66591	3.64982	-0.72934
12	1.92123	1.59343	3.51466	-0.8645
24	1.89664	1.47848	3.37512	-1.00404
48	1.92091	1.36962	3.29053	-1.08863
72	1.95575	1.31506	3.27081	-1.10835

Table B 4) **Beam shrinkage results - Beam P6**

APPENDIX B - Laboratory shrinkage results



Specimens containing 2.4% bitumen emulsion, 1% Cement:

Aggregate moiture:

Bowl weight:	185	g
Bowl + agg.:	790	g
After 24 h:	775	g
	1.89873	% moisture in agg.

Mixture quantities:

Material:	17500 g	
Bitumen	700 g	=> 2.4% Bitumen
Cement:	175 g	=> 1% Cement
Water:	1347.72 g	

Moisture after mixing:

Bowl weight:	153	g
Bowl + agg.:	714	g
After 24 h:	655	g
	8.26331	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)		Width (mm)	Length (mm)					
	Height (mm)	Average Height (mm)							
Beam 1 - B7	76 73	75.75	75	470	5.4972	0.00267	8.26331	2058.73	1901.6
	75 74								
	77 74								**90.55 %** of MOD density
	77 74								
	78 75								
	79 77								
Beam 2 - C7	77 69	72.16667	75	470	5.5242	0.002544	8.26331	2171.57	2005.8
	76 69								
	70 70								**95.52 %** of MOD density
	70 72								
	71 74								
	74 74								

APPENDIX B - Laboratory shrinkage results

Beam 3 - P7	72	73	73.33333	75	470	5.6417	0.002585	8.26331	2182.48	2015.9
	75	73								
	76	73								**96.00 %** of MOD density
	74	72								
	73	73								
	73	73								

Table B 5) **Beam specimens properties - 2.4% bitumen emulsion, 1% Cement**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.32136	2.29469	3.61605	0
0.5	1.3514	2.3159	3.6673	0.05125
1.5	1.42236	2.32545	3.74781	0.13176
3	1.4256	2.32193	3.74753	0.13148
6	1.40223	2.28823	3.69046	0.07441
12	1.38211	2.28697	3.66908	0.05303
24	1.24896	2.21554	3.4645	-0.15155
48	1.23247	2.17153	3.404	-0.21205
72	1.31829	2.22334	3.54163	-0.07442

Table B 6) **Beam shrinkage results - Beam B7**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.54161	1.8037	3.34531	0
0.5	1.57962	1.76716	3.34678	0.00147
1.5	1.61955	1.70393	3.32348	-0.02183
3	1.62249	1.68975	3.31224	-0.03307
6	1.60794	1.61114	3.21908	-0.12623
12	1.54131	1.52408	3.06539	-0.27992
24	1.51024	1.39883	2.90907	-0.43624
48	1.52443	1.4093	2.93373	-0.41158
72	1.58932	1.44127	3.03059	-0.31472

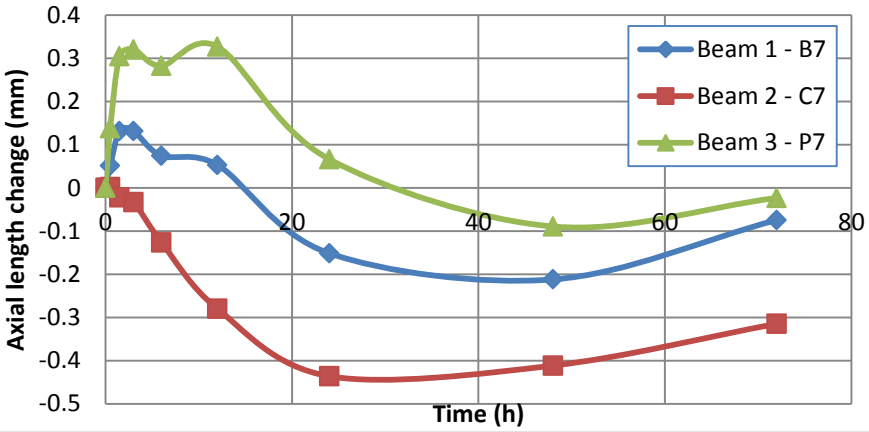
Table B 7) **Beam shrinkage results - Beam C7**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.06223	2.54874	4.61097	0
0.5	2.14714	2.60177	4.74891	0.13794
1.5	2.25302	2.66285	4.91587	0.3049
3	2.26003	2.6715	4.93153	0.32056
6	2.22278	2.67057	4.89335	0.28238
12	2.24339	2.69447	4.93786	0.32689
24	2.09342	2.58372	4.67714	0.06617
48	1.97865	2.54285	4.5215	-0.08947
72	2.00345	2.58424	4.58769	-0.02328

Table B 8) **Beam shrinkage results - Beam P17**

APPENDIX B - Laboratory shrinkage results

Figure b.2) **Linear beam shrinkage - 2.4% bitumen emulsion, 1% cement**



Specimens containing 2.4% bitumen emulsion, 2% Cement:

Aggregate moiture:

Bowl weight:	236	g
Bowl + agg.:	939	g
After 24 h:	917	g
	2.34292	% moisture in agg.

Mixture quantities:

Material:	17500 g	
Bitumen	700 g	=> 2.4% Bitumen
Cement:	350 g	=> 2% Cement
Water:	1269.99 g	

Moisture after mixing:

Bowl E weight:	236	g
Bowl E + agg.:	765	g
After 24 h:	711	g
	7.05882	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)		Width (mm)	Length (mm)					
	Height (mm)	Average Height (mm)							
Beam 1 - B8	82 78 80 77 78 76 78 82 79 75 78 79	78.5	75	470	5.804	0.002767	7.05882	2097.48	1959.2 **93.29 %** of MOD density
Beam 2 - C8	74 77 73 74 71 75 75 75 78 78 80 80	73.65833	75	468	5.6256	0.002585	7.05882	2175.9	2032.4 **96.78 %** of MOD density

APPENDIX B - Laboratory shrinkage results

Beam 3 - P8	80	78	75.25	75	469	5.684	0.002647	7.05882	2147.4	2005.8
	79	75								
	78	73								**95.52 %** of MOD density
	76	72								
	74	72								
	75	71								

Table B 9) **Beam specimens properties - 2.4% bitumen emulsion, 2% Cement**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.6	2.86	4.46	0
0.5	1.65219	2.86623	4.51842	0.05842
1.5	1.71431	2.87085	4.58516	0.12516
3	1.73337	2.89237	4.62574	0.16574
6	1.68558	2.86727	4.55285	0.09285
12	1.55837	2.79702	4.35539	-0.10461
24	1.46695	2.75141	4.21836	-0.24164
48	1.42276	2.75831	4.18107	-0.27893
72	1.42512	2.75831	4.18343	-0.27657

Table B 10) **Beam shrinkage results - Beam B8**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	0.54	2.81123	3.35123	0
0.5	0.48018	2.92134	3.40152	0.05029
1.5	0.51879	2.93841	3.4572	0.10597
3	0.63414	2.92931	3.56345	0.21222
6	0.54431	2.90451	3.44882	0.09759
12	0.4302	2.85161	3.28181	-0.06942
24	0.32316	2.83237	3.15553	-0.1957
48	0.33571	2.84216	3.17787	-0.17336
72	0.35398	2.85112	3.2051	-0.14613

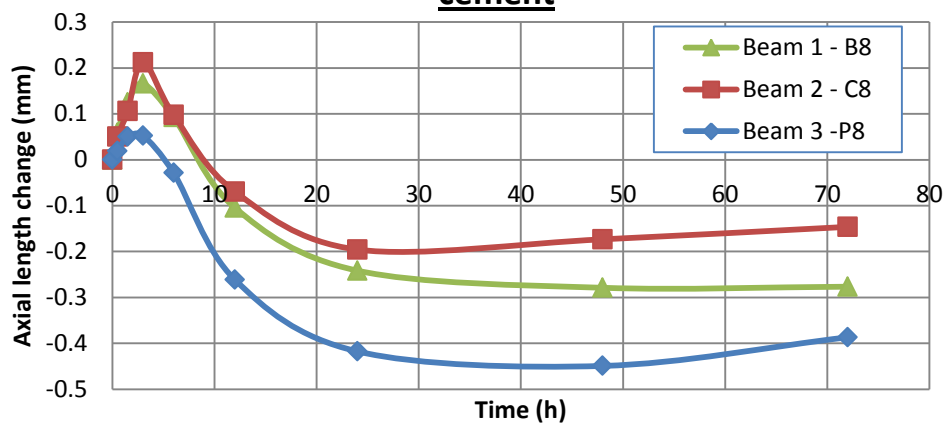
Table B 11) **Beam shrinkage results - Beam C8**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.72	2.44	5.16	0
0.5	2.74862	2.43058	5.1792	0.0192
1.5	2.77408	2.43587	5.20995	0.04995
3	2.7771	2.43561	5.21271	0.05271
6	2.73346	2.39789	5.13135	-0.02865
12	2.54935	2.34935	4.8987	-0.2613
24	2.42118	2.32118	4.74236	-0.41764
48	2.35538	2.35538	4.71076	-0.44924
72	2.38657	2.38657	4.77314	-0.38686

Table B 12) **Beam shrinkage results - Beam P8**

APPENDIX B - Laboratory shrinkage results

Figure b.3) **Beam shrinkage - 2.4% emulsion, 2% cement**



Specimens containing 0.9% bitumen emulsion, 1% Cement:

Aggregate moiture:		
Bowl weight:	228	g
Bowl + agg.:	782	g
After 24 h:	764	g
	2.30179	% moisture in agg.

Mixture quantities:		
Material:	17000 g	
Bitumen	255 g	=> 0.9% Bitumen
Cement:	170 g	=> 1% Cement
Water:	1410.70 g	

Moisture after mixing:		
Bowl43 weight:	228	g
Bowl43 + agg.:	795	g
After 24 h:	736	g
	7.42138	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)		Width (mm)	Length (mm)					
	Height (mm)	Average Height (mm)							
Beam 1 - B9	76 76 74 74 73 74 75 73 76 73 75 75	74.5	75	470	5.7766	0.002626	7.42138	2199.67	2047.7 **97.51 %** of MOD density
Beam 2 - C9	79 78 78 77 77 77 76.5 76 76.5 75 78 77	77.08333	75	470	6.097	0.002717	7.42138	2243.86	2088.8 **99.47 %** of MOD density

APPENDIX B - Laboratory shrinkage results

Beam 3 - P9	78	77	74.16667	75	470	5.754	0.002614	7.42138	2200.91	2048.9
	75	76								
	72	73								**97.56 %** of MOD density
	73	75								
	72	73								
	73	73								

Table B 13) **Beam specimens properties -0.9% bitumen emulsion, 1% Cement**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.28641	3.06468	5.35109	0
0.5	2.29339	3.06291	5.3563	0.00521
1.5	2.29476	3.00657	5.30133	-0.04976
3	2.29816	2.97942	5.27758	-0.07351
6	2.27628	2.95156	5.22784	-0.12325
12	2.22979	2.90562	5.13541	-0.21568
24	2.21536	2.81074	5.0261	-0.32499
48	2.24403	2.72917	4.9732	-0.37789
72	2.28523	2.70206	4.98729	-0.3638

Table B 14) **Beam shrinkage results - Beam B9**

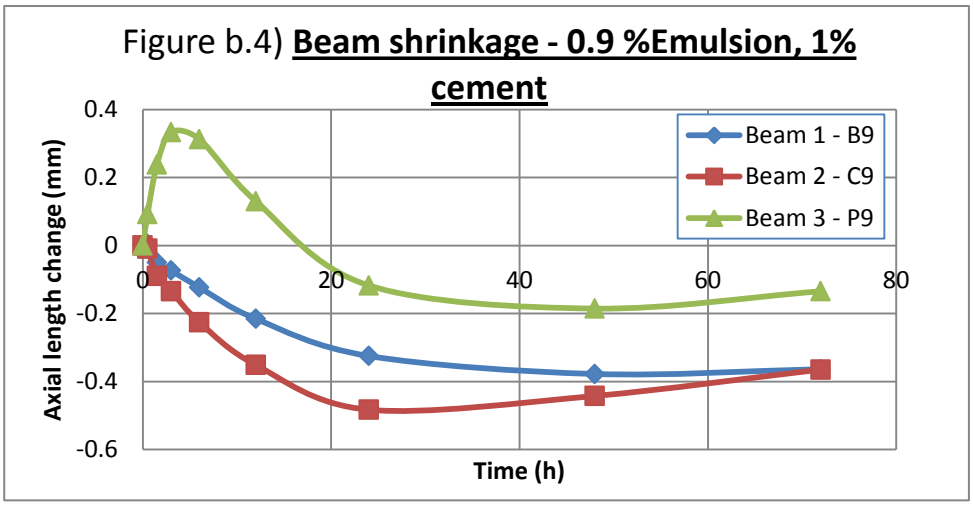
Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.21485	0.78635	3.0012	0
0.5	2.23863	0.75322	2.99185	-0.00935
1.5	2.24595	0.66573	2.91168	-0.08952
3	2.25883	0.60808	2.86691	-0.13429
6	2.25061	0.52489	2.7755	-0.2257
12	2.20614	0.44364	2.64978	-0.35142
24	2.13547	0.38259	2.51806	-0.48314
48	2.15295	0.40577	2.55872	-0.44248
72	2.1921	0.44436	2.63646	-0.36474

Table B 15) **Beam shrinkage results - Beam C9**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.70199	1.92075	4.62274	0
0.5	2.73256	1.98275	4.71531	0.09257
1.5	2.78984	2.07119	4.86103	0.23829
3	2.82724	2.12928	4.95652	0.33378
6	2.82089	2.11451	4.9354	0.31266
12	2.75205	2.00105	4.7531	0.13036
24	2.6732	1.83197	4.50517	-0.11757
48	2.663	1.77436	4.43736	-0.18538
72	2.68076	1.80749	4.48825	-0.13449

Table B 16) **Beam shrinkage results - Beam P9**

APPENDIX B - Laboratory shrinkage results



Specimens containing 2.4% foam bitumen, 1% Cement:

Aggregate moiture:

Bowl weight:	236	g
Bowl + agg.:	956	g
After 24 h:	905	g
	5.33473	% moisture in agg.

Mixture quantities:

Material:	17000 g	
Bitumen	408 g	=> 2.4% Bitumen
Cement:	170 g	=> 1% Cement
Water:	997.10 g	80% = 797.677 g
		20% = 199.419 g

Moisture after mixing:

Bowl T weight:	236	g
Bowl T + agg.:	814	g
After 24 h:	757	g
	7.00246	% moisture in specimen

Foamed bitumen characteristics: Expansion = 13; Half life = 10 seconds

Beam specimen name	Beam dimensions: Hight (mm) Height (mm)		Hight (mm) Average Height (mm)	Width (mm)	Length (mm)	Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
Beam 1 - B10	74	73	74.20833	75	470	5.85994	0.002616	7.00246	2240.17	2093.6
	73	75								
	74	74								**99.69 %** of MOD density
	74	73								
	74.5	75								
	75	76								
Beam 2 - C10	75	76	75.45833	75	470	5.7707	0.00266	7.00246	2169.51	2027.5
	76.5	76								
	76	77								**96.55 %** of MOD density
	75	76								
	75	75								
	74	74								

APPENDIX B - Laboratory shrinkage results

Beam 3 - P10	73	76	73.625	75	470	5.6869	0.002595	7.00246	2191.25	2047.8
	72	75								
	73	73								**97.52 %**
	73.5	73								of MOD
	74	74								density
	73	74								

Table B 17) **Beam specimens properties -2.4% Foam bitumen, 1% Cement**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.55604	1.81934	4.37538	0
0.5	2.68332	1.81818	4.5015	0.12612
1.5	2.72278	1.81401	4.53679	0.16141
3	2.71225	1.77849	4.49074	0.11536
6	2.67455	1.68886	4.36341	-0.01197
12	2.66762	1.61484	4.28246	-0.09292
24	2.66516	1.55006	4.21522	-0.16016
48	2.68678	1.53319	4.21997	-0.15541
72	2.69	1.52038	4.21038	-0.165

Table B 18) **Beam shrinkage results - Beam B10**

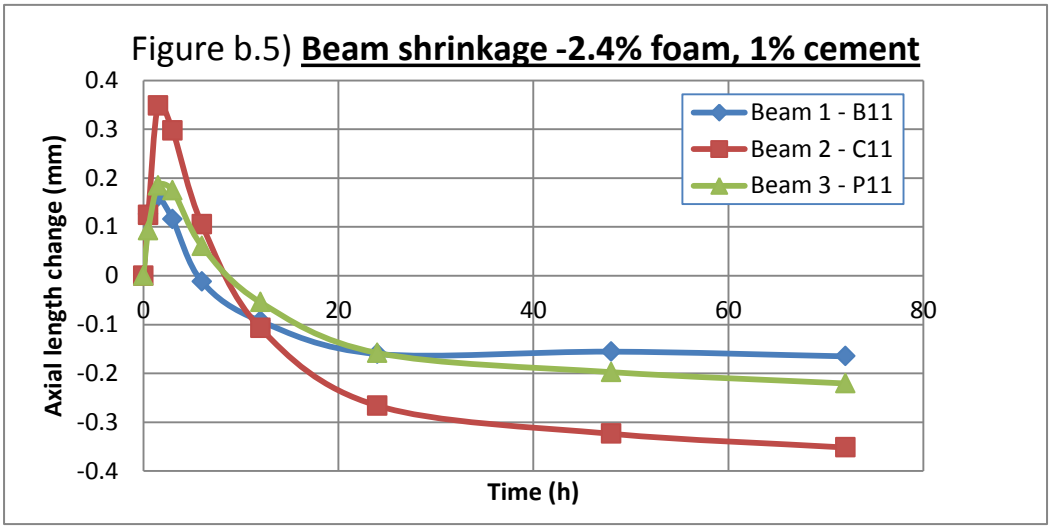
Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.03864	2.68776	4.7264	0
0.5	2.1216	2.72874	4.85034	0.12394
1.5	2.27746	2.79732	5.07478	0.34838
3	2.28398	2.73981	5.02379	0.29739
6	2.15287	2.67896	4.83183	0.10543
12	2.00226	2.61676	4.61902	-0.10738
24	1.85882	2.6015	4.46032	-0.26608
48	1.81412	2.58864	4.40276	-0.32364
72	1.78593	2.58875	4.37468	-0.35172

Table B 19) **Beam shrinkage results - Beam B10**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.8385	1.0584	2.8969	0
0.5	1.93168	1.05815	2.98983	0.09293
1.5	2.03115	1.04995	3.0811	0.1842
3	2.03931	1.03228	3.07159	0.17469
6	2.01254	0.94532	2.95786	0.06096
12	1.97911	0.86356	2.84267	-0.05423
24	1.93966	0.79916	2.73882	-0.15808
48	1.93246	0.76709	2.69955	-0.19735
72	1.92662	0.74974	2.67636	-0.22054

Table B 20) **Beam shrinkage results - Beam B10**

APPENDIX B - Laboratory shrinkage results



Specimens containing 2.4% foam bitumen, 2% Cement:

Aggregate moiture:

Bowl weight:	236	g
Bowl + agg.:	1015	g
After 24 h:	987	g
	2.75862	% moisture in agg.

Mixture quantities:

Material:	17000	g
Bitumen	408	g
Cement:	340	g
Water:	1435.03	g

=> 2.4% Bitumen
=> 2% Cement

Moisture after mixing:

Bowl X1 weight:	237	g
Bowl X1 + agg.:	785	g
After 24 h:	724	g
	7.7707	% moisture in specimen

Foamed bitumen characteristics:

Expansion = 13
Half life = 9 seconds

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)		Width (mm)	Length (mm)					
	Height (mm)	Average Height (mm)							
Beam 1 - B11	74 73 73 74 72 74 71 72 71 72 73 73	72.66667	75	470	5.7372	0.002562	7.7707	2239.78	2078.3 **98.97 %** of MOD density
Beam 2 - C11	74 74 76 74 75 73 74 73 74 75 75 76	74.41667	75	470	5.7711	0.002623	7.7707	2200.03	2041.4 **97.21 %** of MOD density

APPENDIX B - Laboratory shrinkage results

Beam 3 - P11	76 75 74 74 75 75 74 73 73 73 74 74	74.16667	75	470	5.8078	0.002614	7.7707	2221.49	2061.3 **98.16 %** of MOD density

Table B 21) **Beam specimens properties -2.4% Foam bitumen, 2% Cement**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	1.87993	2.27271	4.15264	0
0.5	2.06223	2.47385	4.53608	0.38344
1.5	2.18353	2.55493	4.73846	0.58582
3	2.17683	2.55325	4.73008	0.57744
6	2.0228	2.46401	4.48681	0.33417
12	1.90461	2.36537	4.26998	0.11734
24	1.79416	2.2684	4.06256	-0.09008
48	1.74897	2.23585	3.98482	-0.16782
72	1.72369	2.2268	3.95049	-0.20215

Table B 22) **Beam shrinkage results - Beam B11**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	0.74934	1.0431	1.79244	0
0.5	0.81868	1.1353	1.95398	0.16154
1.5	0.82925	1.19037	2.01962	0.22718
3	0.81002	1.17481	1.98483	0.19239
6	0.75328	1.07274	1.82602	0.03358
12	0.68968	0.9997	1.68938	-0.10306
24	0.58146	0.97153	1.55299	-0.23945
48	0.54921	0.96539	1.5146	-0.27784
72	0.54497	0.96992	1.51489	-0.27755

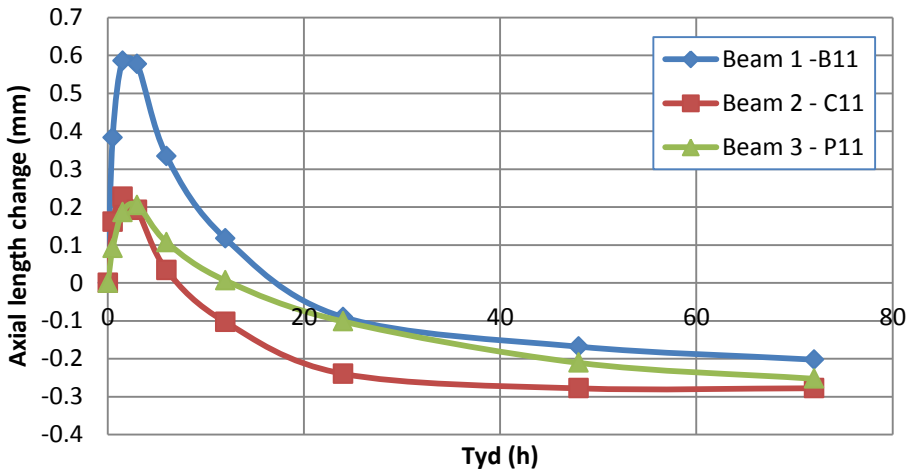
Table B 23) **Beam shrinkage results - Beam C11**

Time (h)	Reading (mm) from LVDT 1	Reading (mm) from LVDT 2	Sum of LVDT readings (mm)	Shrinkage of the beam (mm)
0	2.02566	2.41147	4.43713	0
0.5	2.0668	2.46202	4.52882	0.09169
1.5	2.11532	2.50853	4.62385	0.18672
3	2.12884	2.51305	4.64189	0.20476
6	2.10696	2.43763	4.54459	0.10746
12	2.07464	2.36896	4.4436	0.00647
24	2.03814	2.2975	4.33564	-0.10149
48	2.05857	2.16801	4.22658	-0.21055
72	2.04173	2.14286	4.18459	-0.25254

Table B 24) **Beam shrinkage results - Beam P11**

APPENDIX B - Laboratory shrinkage results

Figure b.6) **Beam shrinkage - 2.4 % Foam, 2% cement**



B.3. Cylindrical specimens:

Specimens containing 0% bitumen, 0% Cement:

Aggregate moiture:

Bowl24 weight:	186.2	g
Bowl + agg.:	807.7	g
After 24 h:	772	g
	4.41996	% moisture in agg.

Mixture quantities:

Material:	11284.5 g	
Bitumen	0 g	=> 0% Bitumen
Cement:	0 g	=> 0% Cement
Water:	765.094 g	

Moisture after mixing:

Bowl24 weight:	186.2	g
Bowl24 + agg.:	756.1	g
After 24 h:	708	g
	6.36159	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Height (mm)		Diameter (mm)						
	Height (mm)	Average Height (mm)	Diameter (mm)	Average diameter					
Cylinder 1 - B12	296	295.8333	98.7	98.9667	5.033	0.002276	6.36159	2211.63	2079.3
	295.5		98.2						
	296		100						**99.02** % of MOD density
Cylinder 2 - B12	289	291.3333	98.8	99.4333	5.033	0.002262	6.36159	2224.76	2091.7
	293		99.5						
	292		100						**99.60** % of MOD density

Table B 25) **Cylindrical specimens properties - 0% bitumen, 0% Cement**

APPENDIX B - Laboratory shrinkage results

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	10.820	0	2.65173	0
0.5 h	10.670	-0.150	2.49371	-0.1580
1.5 h	10.320	-0.500	2.27801	-0.3737
3 h	10.299	-0.521	2.19337	-0.4584
6 h	10.380	-0.440	2.19325	-0.4585
12 h	10.500	-0.320	2.36562	-0.2861
24 h	10.670	-0.150	2.43738	-0.2144
48h	10.650	-0.170	2.33676	-0.3150
72 h	10.600	-0.220	2.25194	-0.3998

Table B 26) **Cylindrical shrinkage results - Cylinder B12**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	4.12	0	2.70521	0
0.5 h	3.98	-0.14	2.55168	-0.15353
1.5 h	3.76	-0.36	2.33874	-0.36647
3 h	3.69	-0.43	2.25146	-0.45375
6 h	3.684	-0.436	2.25878	-0.44643
12 h	3.804	-0.316	2.37073	-0.33448
24 h	3.805	-0.315	2.32119	-0.38402
48h	3.66	-0.46	2.19759	-0.50762
72 h	3.578	-0.542	2.14164	-0.56357

Table B 27) **Cylindrical shrinkage results - Cylinder P12**

Figure b.7) **Dial gauge shrinkage - 0%Bitumen, 0%Cement**

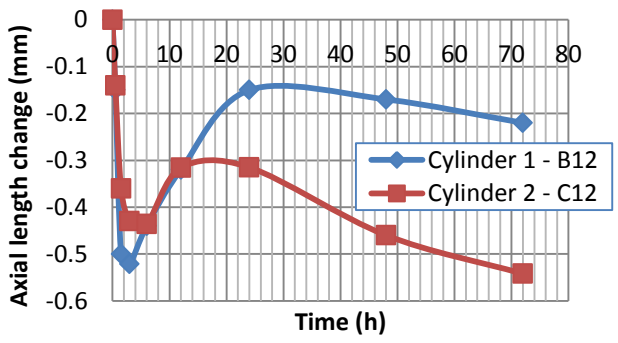
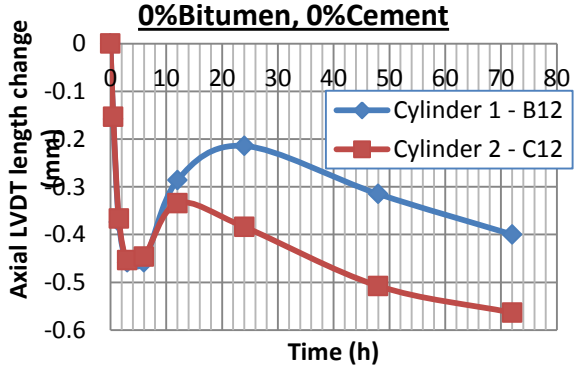


Figure b.8) **LVDT shrinkage - 0%Bitumen, 0%Cement**



APPENDIX B - Laboratory shrinkage results

Specimens containing 2.4% bitumen emulsion, 1% Cement:

Aggregate moiture:

Bowl54 weight:	186.2	g
Bowl + agg.:	807.7	g
After 24 h:	772	g
	4.41996	% moisture in agg.

Mixture quantities (Beam P13):

Material:	5600	g
Bitumen	224	g => 2.4% Bitumen
Cement:	56	g => 1% Cement
Water:	290.082	g

Aggregate moiture:

Bowl weight:	197.9	g
Bowl + agg.:	816.9	g
After 24 h:	788.4	g
	3.4888	% moisture in agg.

Mixture quantities (Beam B13 & C13):

Material:	16795	g
Bitumen	671.8	g => 2.4% Bitumen
Cement:	167.95	g => 1% Cement
Water:	1026.38	g

Moisture after mixing (Beam P):

Bowl83 weight:	223.9	g
Bowl83 + agg.:	770.7	g
After 24 h:	709.5	g
	7.940833	%moist.

Beam B and C:

Bowl225 weight:	212	g
Bowl225 + agg.:	748	g
After 24 h:	687.2	g
	8.12834	%moist.

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Height (mm)		Diameter (mm)						
	Height (mm)	Average Height (mm)	Diameter (mm)	Average diameter					
Cylinder 1 - B11	291	291.1667	100	99.3333	5.046	0.002256	8.12834	2236.28	2068.2
	291.5		99						
	291		99						**98.48** % of MOD density
Cylinder 1 - B11	289	291.3333	98.8	99.4333	5.09	0.002262	8.12834	2249.95	2080.8
	293		99.5						
	292		100						**99.09** % of MOD density
Cylinder 1 - B11	289	291.3333	98.8	99.4333	5.043	0.002262	7.94083	2229.18	2065.2
	293		99.5						
	292		100						**98.34** % of MOD density

Table B 28) **Cylindrical specimens properties - 2.4% bitumen emulsion, 1% Cement**

APPENDIX B - Laboratory shrinkage results

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	8.804	0	1.35	0
0.5 h	8.5	-0.3040	1.09995	-0.25005
1.5 h	8.2	-0.6040	0.89284	-0.45716
3 h	8.1032	-0.7008	0.83934	-0.51066
6 h	7.9	-0.9040	0.80212	-0.54788
12 h	8.21	-0.5940	0.91040	-0.4396
24 h	8.24	-0.5640	0.92879	-0.42121
48h	8.135	-0.6690	0.84246	-0.50754
72 h	-	-	-	-

Table B 29) **Cylindrical shrinkage results - Cylinder B13**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	10.4012	0	2.09061	0
0.5 h	10.2	-0.2012	2.01889	-0.07172
1.5 h	9.992	-0.4092	1.8535	-0.23711
3 h	9.86	-0.5412	1.74649	-0.34412
6 h	9.826	-0.5752	1.73958	-0.35103
12 h	10.045	-0.3562	1.87787	-0.21274
24 h	10.2	-0.2012	1.95500	-0.13561
48h	10.14	-0.2612	1.89499	-0.19562
72 h	-		-	

Table B 30) **Cylindrical shrinkage results - Cylinder C13**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	11.67000	0.00000	2.46068	0
0.5 h	11.55000	-0.12000	2.22914	-0.23154
1.5 h	11.432	-0.23800	2.07784	-0.38284
3 h	11.38	-0.29000	1.98931	-0.47137
6 h	11.37	-0.30000	1.93991	-0.52077
12 h	11.51	-0.16000	2.08067	-0.38001
24 h	11.61	-0.06000	2.20939	-0.25129
48h	11.53	-0.14000	2.12829	-0.33239
72 h	11.347	-0.32300	1.94857	-0.51211

Table B 31) **Cylindrical shrinkage results - Cylinder P13**

APPENDIX B - Laboratory shrinkage results

Figure b.9) **Dial Guage shrinkage -2.4% bitumen emulsion, 1% cement**

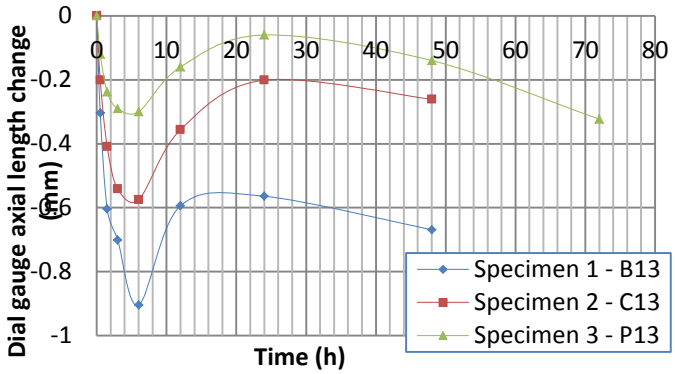
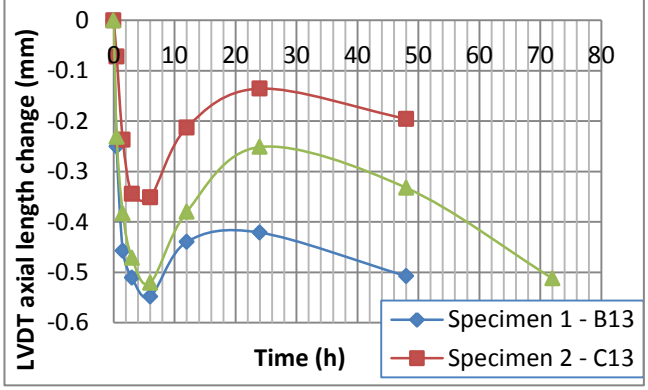


Figure b.10) **LVDT shrinkage- 2.4% bitumen emulsion, 1% cement**



Specimens containing 2.4% Emulsion bitumen, 2% Cement:

Aggregate moiture:

Bowl54 weight:	162.9	g
Bowl + agg.:	819.7	g
After 24 h:	799	g
	2.52531	% moisture in agg.

Mixture quantities:

Material:	17280	g
Bitumen	691.2	g
Cement:	345.6	g
Water:	1222.51	g

=> 2.4% Bitumen
=> 2% Cement

Moisture after mixing:

Bowl60 weight:	170	g
Bowl60 + agg.:	731.8	g
After 24 h:	678.7	g
	7.25608	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Height (mm)		Diameter (mm)						
	Height (mm)	Average Height (mm)	Diameter (mm)	Average diameter					
Cylinder 1 - B14	300 300 300	300	99 98.6 98.5	98.7	5.1155	0.002295	7.25608	2228.65	2077.9 **98.95** % of MOD density
Cylinder 2 - C14	296 295.5 296	295.8333	98.6 99 98.6	98.7333	5.0517	0.002265	7.25608	2230.35	2079.5 **99.02** % of MOD density
Cylinder 3 - B14	294 294.5 296	294.8333	98.5 99 98.6	98.7	5.0318	0.002256	7.25608	2230.6	2079.7 **99.03** % of MOD density

Table B 32) **Cylindrical specimens properties - 2.4% Emulsion bitumen, 2% Cement**

APPENDIX B - Laboratory shrinkage results

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	11.505	0	1.81	0
0.5 h	11.01	-0.495	1.4487	-0.3613
1.5 h	10.95	-0.555	1.37456	-0.43544
3 h	10.85	-0.655	1.3009	-0.5091
6 h	10.8	-0.705	1.26126	-0.54874
12 h	10.9	-0.605	1.34877	-0.46123
24 h	10.98	-0.525	1.41499	-0.39501
48h	10.96	-0.545	1.36994	-0.44006
72 h	-	-	-	-

Table B 33) **Cylindrical shrinkage results - Cylinder B14**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	14.64	0	1.27	0
0.5 h	14.265	-0.375	1.01754	-0.25246
1.5 h	14.08	-0.56	0.8682	-0.4018
3 h	13.95	-0.69	0.78883	-0.48117
6 h	13.885	-0.755	0.78086	-0.48914
12 h	13.962	-0.678	0.85034	-0.41966
24 h	14.05	-0.59	0.90710	-0.3629
48h	14.02	-0.62	0.88576	-0.38424
72 h	-	-	-	-

Table B 34) **Cylindrical shrinkage results - Cylinder C14**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	4.85000	0.00000	2.04507	0
0.5 h	4.69500	-0.15500	1.8971	-0.14797
1.5 h	4.41	-0.44000	1.70997	-0.3351
3 h	4.325	-0.52500	1.64212	-0.40295
6 h	4.3	-0.55000	1.6335	-0.41157
12 h	4.315	-0.53500	1.65673	-0.38834
24 h	4.32	-0.53000	1.62254	-0.42253
48h	4.28	-0.57000	1.48583	-0.55924
72 h	-	-	-	-

Table B 35) **Cylindrical shrinkage results - Cylinder P14**

APPENDIX B - Laboratory shrinkage results

Figure b11) **Dial gauge shrinkage - 2.4% Emulsion, 2% cement**

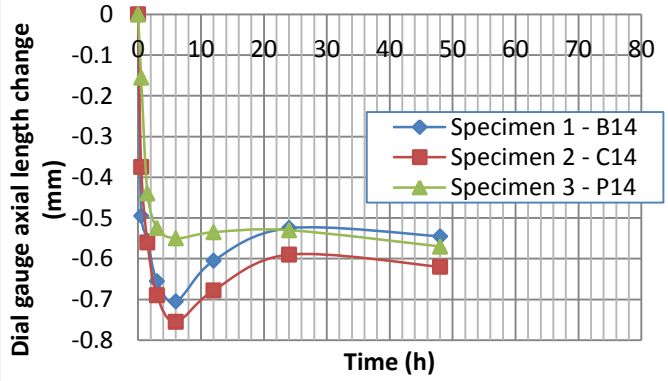
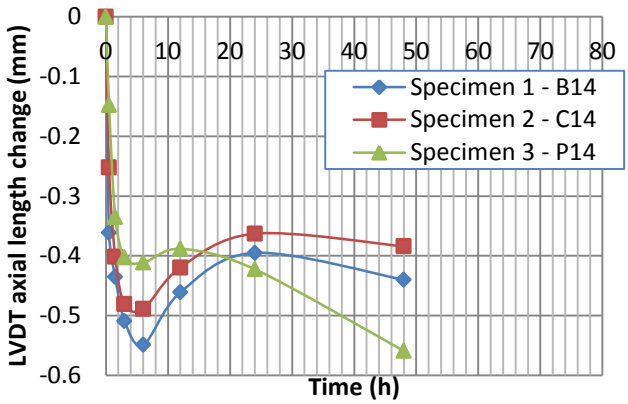


Figure b 12) **LVDT shrinkage - 2.4% Emulsion,2% cement**



Specimens containing 0.9% Emulsion bitumen, 1% Cement:

Aggregate moiture:

Bowl67 weight:	193.6	g
Bowl + agg.:	760.9	g
After 24 h:	739.3	g
	2.83874	% moisture in agg.

Mixture quantities:

Material:	15906	g
Bitumen	238.59	g
Cement:	159.06	g
Water:	1234.51	g

=> 0.9% Bitumen
=> 1% Cement

Moisture after mixing:

Bowl60 weight:	170	g
Bowl60 + agg.:	731.8	g
After 24 h:	678.7	g
	7.25608	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Height (mm)		Diameter (mm)						
	Height (mm)	Average Height (mm)	Diameter (mm)	Average diameter					
Cylinder 1 - B15	279.5	280.1667	99.8	99.85	4.8668	0.002194	7.25608	2218.41	2068.3
	280		99.9						
	281		99.85						**98.49** % of MOD density
Cylinder 2 - C15	306	306	98.6	98.8667	5.2375	0.002349	7.25608	2229.53	2078.7
	305.5		99						
	306.5		99						**98.99** % of MOD density
Cylinder 3 - P15	305	305.3333	99.5	99.1667	5.2801	0.002358	7.25608	2238.96	2087.5
	306		98.5						
	305		99.5						**99.40** % of MOD density

Table B 36) **Cylindrical specimens properties - 0.9% Emulsion bitumen, 1% Cement**

APPENDIX B - Laboratory shrinkage results

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	6.62	0	2.77397	0
0.5 h	6.45	-0.17	2.6652	-0.10877
1.5 h	6.265	-0.355	2.51831	-0.25566
3 h	6.198	-0.422	2.40463	-0.36934
6 h	6.18	-0.44	2.33817	-0.4358
12 h	6.245	-0.375	2.39706	-0.37691
24 h	6.26	-0.36	2.43576	-0.33821
48h	6.1	-0.52	2.26766	-0.50631
72 h	5.92	-0.7	2.10811	-0.66586

Table B 37) **Cylindrical shrinkage results - Cylinder B15**

Time interval (h) (After initial testing time)	Dial Guage		LVDT		Circumferential LVDT
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)	Shrinkage (mm)
0 h	20.126	0	2.6262	0	0
0.5 h	19.86	-0.266	2.43561	-0.19059	-0.33
1.5 h	19.57	-0.556	2.12282	-0.50338	-0.598
3 h	19.4	-0.726	1.95241	-0.67379	-0.635
6 h	19.31	-0.816	1.87015	-0.75605	-0.659
12 h	19.445	-0.681	1.96336	-0.66284	-0.674
24 h	19.579	-0.547	2.05597	-0.57023	-0.697
48h	19.395	-0.731	1.88859	-0.73761	-0.735
72 h	19.165	-0.961	1.65553	-0.97067	-0.762

Table B 38) **Cylindrical shrinkage results - Cylinder C15**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	9.376	0.00000	2.13152	0
0.5 h	9.14	-0.23600	1.84356	-0.28796
1.5 h	8.86	-0.51600	1.56257	-0.56895
3 h	8.74	-0.63600	1.44492	-0.6866
6 h	8.665	-0.71100	1.40685	-0.72467
12 h	8.65	-0.72600	1.45229	-0.67923
24 h	8.57	-0.80600	1.4454	-0.68612
48h	8.395	-0.98100	1.21193	-0.91959
72 h	8.27	-1.10600	1.0725	-1.05902

Table B 39) **Cylindrical shrinkage results - Cylinder P15**

APPENDIX B - Laboratory shrinkage results

Figure b 13) **Dial gauge shrinkage - 0.9% Emulsion, 1% cement**

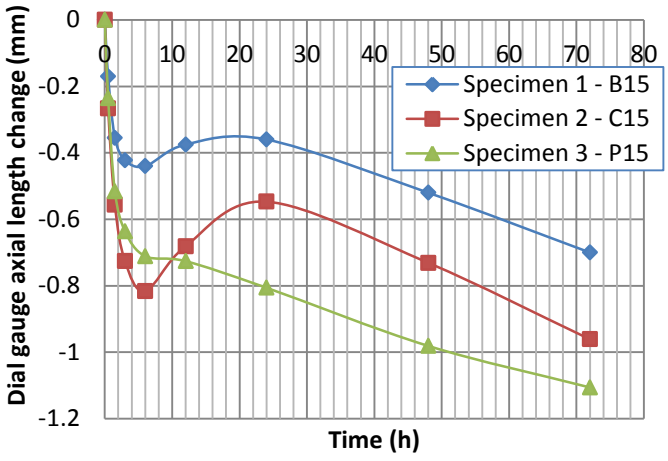


Figure b 14) **LVDT shrinkage - 0.9% Emulsion, 1% cement**

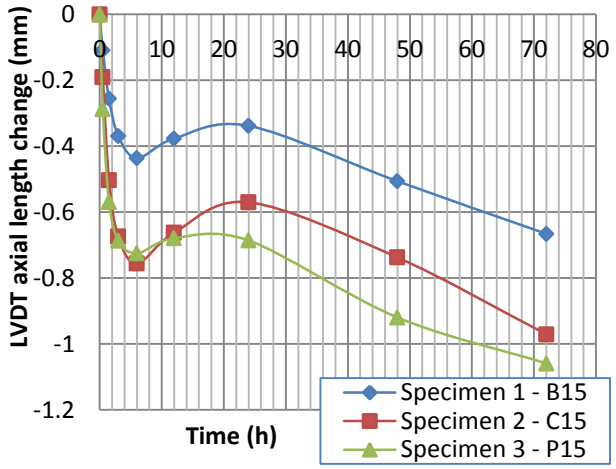
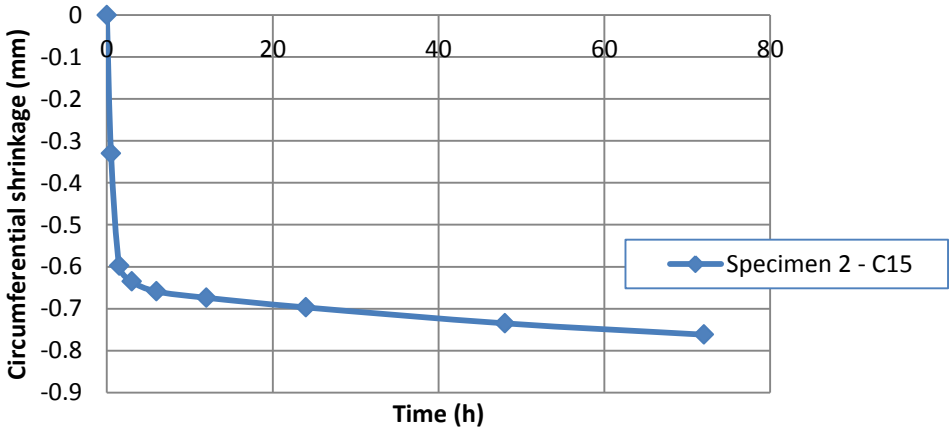


Figure b 15) **Circumferential LVDT shrinkage (specimen containing 0.9% Emulsion; 1% cement)**



Specimens containing 2.4% Foamed bitumen, 1% Cement:

Aggregate moiture:

Bowl3weight:	206.3	g
Bowl + agg.:	964.7	g
After 24 h:	895.8	g
	7.14212	% moisture in agg.

Mixture quantities:

Material:	16901.6 g	
Bitumen	405.6384 g	=> 2.4% Bitumen
Cement:	169.016 g	=> 1% Cement
Water:	685.847 g	80% = 548.678 g
		20% = 137.169 g

Moisture after mixing:

Bowl X weight:	236.57	g
Bowl X + agg.:	818	g
After 24 h:	746.62	g
	8.72616	% moisture in specimen

APPENDIX B - Laboratory shrinkage results

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Height (mm)		Diameter (mm)						
	Height (mm)	Average Height (mm)	Diameter (mm)	Average diameter					
Cylinder 1 - B16	298.5 299 298	298.5	100 99.9 99.9	99.9333	5.332	0.002341	8.72616	2277.38	2094.6 **99.74** % of MOD density
Cylinder 2 - C16	299.5 300.5 299	299.6667	99 100 100	99.6667	5.205	0.002338	8.72616	2226.35	2047.7 **97.51** % of MOD density
Cylinder 3 - B16	300 298.5 299.5	299.3333	99.6 98.5 100	99.3667	5.118	0.002321	8.72616	2204.82	2027.9 **96.57** % of MOD density

Table B 40) **Cylindrical specimens properties - 2.4% Foam bitumen, 1% Cement**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	9.26	0	1.41823	0
0.5 h	9.22	-0.04	1.38744	-0.03079
1.5 h	9.12	-0.14	1.29501	-0.12322
3 h	9.1	-0.16	1.19347	-0.22476
6 h	8.96	-0.3	1.147119	-0.271111
12 h	8.99	-0.27	1.26948	-0.14875
24 h	9.15	-0.11	1.36495	-0.05328
48h	9.2	-0.06	1.44493	0.0267
72 h	9.175	-0.085	1.41156	-0.00667

Table B 41) **Cylindrical shrinkage results - Cylinder B16**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	18.28	0	2.23379	0
0.5 h	18.245	-0.035	2.22476	-0.00903
1.5 h	18.19	-0.09	2.21418	-0.01961
3 h	18.15	-0.13	2.20214	-0.03165
6 h	18.14	-0.14	2.20172	-0.03207
12 h	18.195	-0.085	2.25756	0.02377
24 h	18.205	-0.075	2.24601	0.01222
48h	18.168	-0.112	2.15041	-0.08338
72 h	18.13	-0.15	1.99931	-0.23448

Table B 42) **Cylindrical shrinkage results - Cylinder C16**

APPENDIX B - Laboratory shrinkage results

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	11.21	0.00000	2.24013	0
0.5 h	11.1	-0.11000	2.22654	-0.01359
1.5 h	11.05	-0.16000	2.1303	-0.10983
3 h	10.925	-0.28500	1.9997	-0.24043
6 h	10.85	-0.36000	1.95614	-0.28399
12 h	10.98	-0.23000	2.03681	-0.20332
24 h	11.108	-0.10200	2.05796	-0.18217
48h	11.17	-0.04000	2.05034	-0.18979
72 h	11.115	-0.09500	1.98691	-0.25322

Table B 43) **Cylindrical shrinkage results - Cylinder P16**

Figure b 16) **Dial gauge shrinkage - 2.4% Foam, 1% cement**

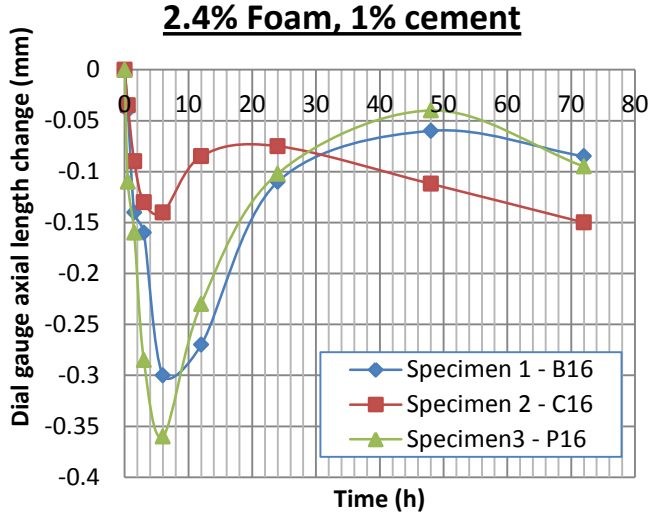
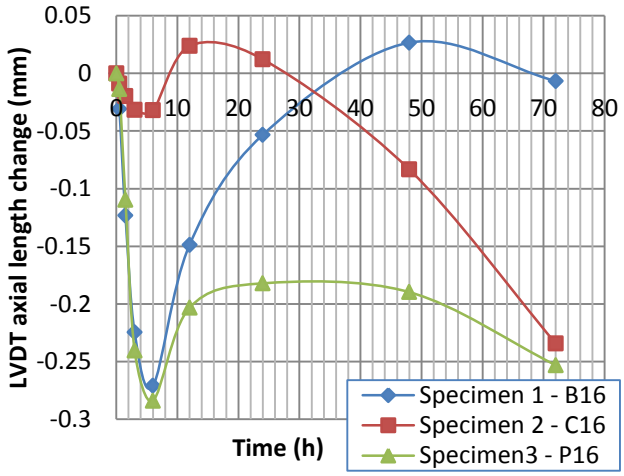


Figure b 17) **LVDT shrinkage - 2.4% Foam, 1% cement**



Specimens containing 2.4% Foam bitumen, 2% Cement:

Aggregate moiture:

Bowl44weight:	168.3	g
Bowl + agg.:	910.8	g
After 24 h:	854.1	g
	6.2253	% moisture in agg.

Mixture quantities:

Material:	16320.5 g	
Bitumen	391.692 g	=> 2.4% Bitumen
Cement:	326.41 g	=> 1% Cement
Water:	811.896 g	80% = 649.517 g
		20% = 162.379 g

Moisture after mixing:

Bowl 44 weight:	168.29	g
Bowl 44 + agg.:	703.63	g
After 24 h:	651.4	g
	7.42294	% moisture in specimen

APPENDIX B - Laboratory shrinkage results

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Height (mm)		Diameter (mm)						
	Height (mm)	Average Height (mm)	Diameter (mm)	Average diameter					
Cylinder 1 - B16	300 301 299	300	99.5 99.9 99.9	99.7667	5.114	0.002345	7.42294	2180.61	2029.9 **96.66** % of MOD density
Cylinder 2 - C16	299.5 300.5 299	299.6667	100 99 100	99.6667	4.847	0.002338	7.42294	2073.22	1930 **91.90** % of MOD density
Cylinder 3 - B16	297 298 297	297.3333	98 98.5 99	98.5	4.929	0.002266	7.42294	2175.47	2025.1 **96.44** % of MOD density

Table B 44) **Cylindrical specimens properties - 2.4 Foam bitumen, 2% Cement**

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	22.25	0	3.16969	0
0.5 h	22.5	0.25	3.13685	-0.03284
1.5 h	22.72	0.47	3.10554	-0.06415
3 h	23.91	1.66	3.08669	-0.083
6 h	23.16	0.91	3.07633	-0.09336
12 h	23.33	1.08	3.06714	-0.10255
24 h	23.12	0.87	3.09452	-0.07517
48h	22.43	0.18	3.14028	-0.02941
72 h	22.1	-0.15	3.25028	0.08059

Table B 45) **Cylindrical shrinkage results - Cylinder B17**

Time interval (h) (After initial testing time)	Dial Guage		LVDT		Circumferential LVDT
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)	Shrinkage (mm)
0 h	52.41	0	2.31239	0	0
0.5 h	52.83	-0.42	2.2209	-0.09149	-0.024
1.5 h	53.372	-0.962	2.13544	-0.17695	-0.055
3 h	53.55	-1.14	2.08162	-0.23077	-0.076
6 h	53.88	-1.47	2.02048	-0.29191	-0.11
12 h	54.33	-1.92	1.91187	-0.40052	-0.138
24 h	53.56	-1.15	2.06577	-0.24662	-0.17
48h	52.39	0.02	2.31769	0.0053	-0.204
72 h	52.86	-0.45	2.42966	0.11727	-0.23

Table B 46) **Cylindrical shrinkage results - Cylinder C17**

APPENDIX B - Laboratory shrinkage results

Time interval (h) (After initial testing time)	Dial Guage		LVDT	
	Reading (mm)	Shrinkage (mm)	Reading (mm)	Shrinkage (mm)
0 h	9.12	0.00000	-1.99823	0
0.5 h	9	-0.12000	-1.99823	0
1.5 h	8.922	-0.19800	-1.99823	0
3 h	8.885	-0.23500	-1.99823	0
6 h	8.865	-0.25500	-1.99823	0
12 h	8.816	-0.30400	-1.99823	0
24 h	8.96	-0.16000	-1.99823	0
48h	9.174	0.05400	-1.99823	0
72 h	9.265	0.14500	-1.99823	0

Table B 47) **Cylindrical shrinkage results - Cylinder P17**

Figure b 18) **Dial gauge shrinkage - 2.4% Foam bitumen, 2% cement**

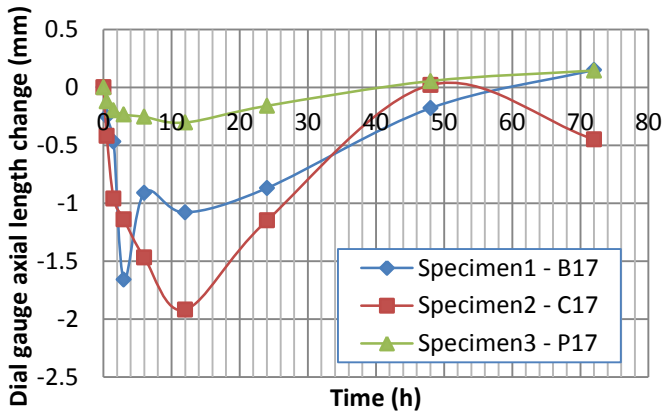


Figure b 19) **LVDT shrinkage - 2.4% Foam bitumen, 2% cement**

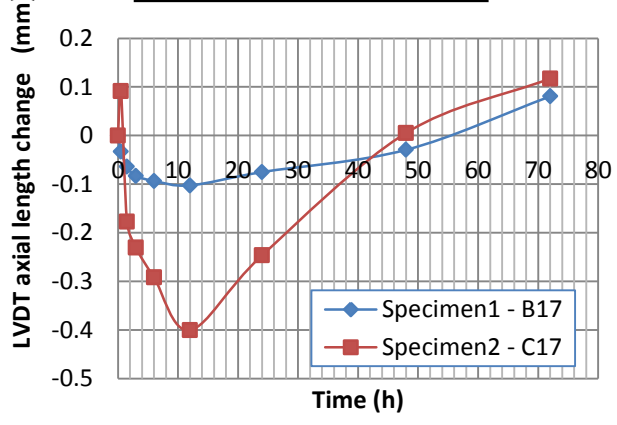
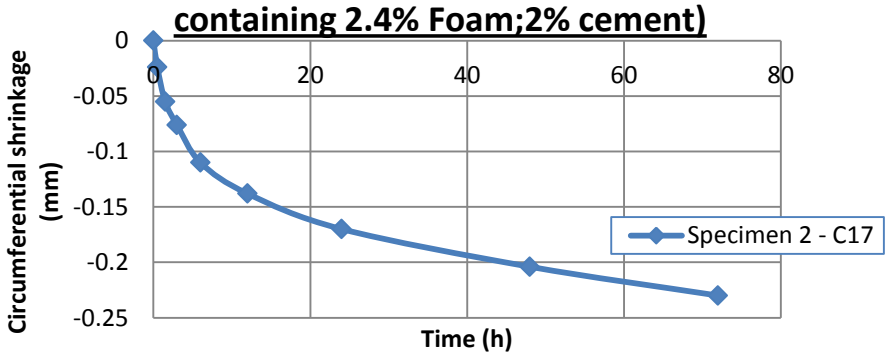


Figure b 20) **Circumferential LVDT shrinkage (specimen containing 2.4% Foam;2% cement)**



APPENDIX C – Laboratory strain-at-break results

APPENDIX C - Laboratory strain-at-break results

C.1. Data from R35 material

R35 material characteristics determined in phase one:

Optimum moisture content:	11.2	%
Maximum dry density of R35 material	2100	kg/m^3

Design beam dimensions determined during phase two:

Height:	75	mm
Width:	75	mm
Length:	470	mm

Emultion 60/40 (bitumen/water)

2.4% / 0.6% = 4% bitumen has to be added to add 2.4% pure bitumen.

(4% * 40%)/100 = 1.6% water will be added to the mixture when adding 4% emultion.

C.2. Strain-at-break specimens:

Specimens containing 2.4% Emulsie bitumen, 1% Cement:

Aggregate moiture:

Bowl weight:	210	g
Bowl + agg.:	837	g
After 24 h:	820	g
	2.03106	% moisture in agg.

Mixture quantities:

Material:	17820 g	
Bitumen	712.8 g	=> 2.4% Bitumen
Cement:	178.2 g	=> 1% Cement
Water:	1348.78 g	

Moisture after mixing:

Bowl17 weight:	238	g
Bowl17 + agg.:	777.8	g
After 24 h:	708	g
	8.97403	% moisture in specimen

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)		Width (mm)	Length (mm)					
	Height (mm)	Average Height (mm)							
Beam 1 - B1	76 77.7 73 75.4 71 73 69.5 70.5 67 69 67 69	71.5083	75	470	5.766	0.002521	8.97403	2287.49	2099.113 **99.96 %** of MOD density
Beam 2 - C1	77 77 75 75 74.2 73 74 71.7 73 71 72 71	73.6583	75	470	5.939	0.002596	8.97403	2287.35	2098.985 **99.95 %** of MOD density

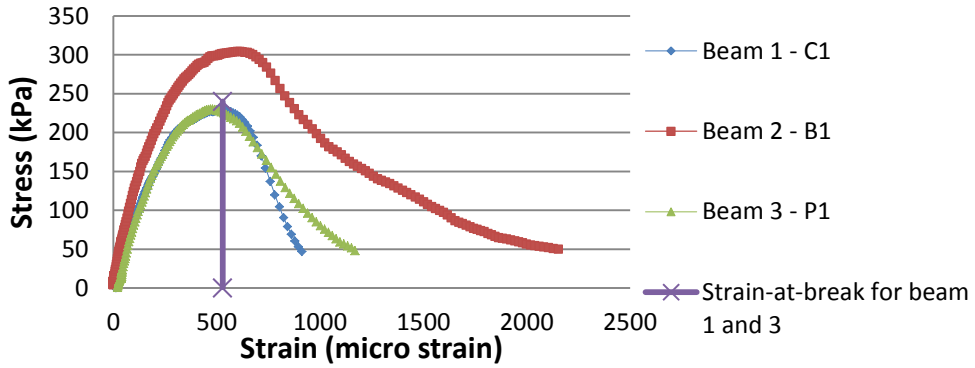
APPENDIX C - Laboratory strain-at-break results

Beam 3 - P1	73 75 71 73 70 71.6 70 70.5 70.6 71.7 72 72.2	71.7167	75	470	5.773	0.002528	8.97403	2283.61	2095.556 **99.79 %** of MOD density

Table C 1) **Beam specimens properties - 2.4% bitumen emulsion, 1% Cement**

Stress vs. strain graphical results:

Figure c.1) **Stress vs. strain of 2,4% Emulsion, 1%Cement**



Beam specimen name	Stress- at-break (kPa)	Strain-at-break (με)	Stiffness (Mpa)	Dissipation energy (N/m^2) - Pa	Yield strength (kpa)
Beam 1 - B1	304.06239	601.47181	505.53058	110.77692	293.9748
Beam 2 - C1	230.47868	528.93038	435.74483	94.52532	216.9111
Beam 3 - P1	230.05445	481.16837	478.11632	64.10061	219.7877
Average	254.86517	537.19019	473.13058	89.80095	243.5579

Table C.2) **Data obtained from Stress vs. Strain curve**

Specimens containing 2.4% bitumen emulsion, 2% Cement:

Aggregate moiture:

Bowl54 weight:	210	g
Bowl + agg.:	837	g
After 24 h:	820	g
	2.03106	% moisture in agg.

Mixture quantities (Beam B & P):

Material:	12300 g	
Bitumen	492 g	=> 2.4% Bitumen
Cement:	246 g	=> 2% Cement
Water:	930.98 g	

Mixture quantities (Beam C):

Material:	6000 g	
Bitumen	240 g	=> 2.4% Bitumen
Cement:	120 g	=> 2% Cement
Water:	454.14 g	

APPENDIX C - Laboratory strain-at-break results

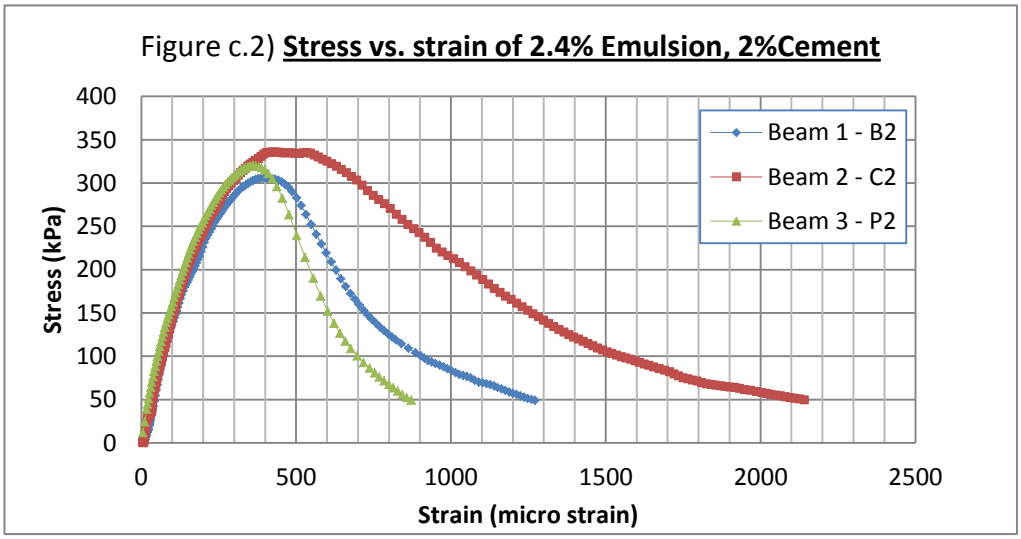
Moisture after mixing (Beam B & P):	Bowl65 weight:	228.6	g	Beam C:	Bowl54 weight:	209.7	g
	Bowl65 + agg.:	804.6	g		Bowl54 + agg.:	760	g
	After 24 h:	749	g		After 24 h:	703	g
		6.910266	%moist.			7.5	%moist.

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)		Width (mm)	Length (mm)					
	Height (mm)	Average Height (mm)							
Beam 1 - B2	73 74 73 72 71 71 72 72 74 73 76 75	73	75	470	5.772	0.002573	6.91027	2243.08	2098.094 **99.91 %** of MOD density
Beam 2 - C2	75 75 73 75 73 74 72 73 73 73 73 73	73.5	75	470	5.842	0.002591	7.5	2254.84	2097.522 **99.88 %** of MOD density
Beam 3 - P2	76 78 74 74 71 74 70 73 70 72 72 72	73	75	470	5.773	0.002573	6.91027	2243.47	2098.457 **99.93 %** of MOD density

Table C 3) **Beam specimens properties - 2.4% bitumen emulsion, 2% Cement**

Stress vs. strain graphical results:

Figure c.2) **Stress vs. strain of 2.4% Emulsion, 2%Cement**



APPENDIX C - Laboratory strain-at-break results

Beam specimen name	Stress- at-break (kPa)	Strain-at-break (με)	Stiffness (Mpa)	Dissipation energy (N/m^2) - Pa	Yield strength (kpa)
Beam 1 - B2	305.61877	386.95794	789.79842	71.92973	298.8254
Beam 2 - C2	335.52454	425.38311	788.75848	88.80845	333.8967
Beam 3 - P2	320.02149	361.07661	886.29802	75.56141	304.8937
Average	320.38826	391.13922	821.61831	78.76653	312.5386

Table C.4) **Data obtained from Stress vs. Strain curve**

Specimens containing 0.9% bitumen emulsion, 1% Cement:

Aggregate moiture:

Bowl weight:	132.88	g
Bowl + agg.:	513.9	g
After 24 h:	496.39	g
	3.40728	% moisture in agg.

Mixture specimens:

Material:	17500 g	
Bitumen	262.5 g	=> 2.4% Bitumen
Cement:	175 g	=> 2% Cement
Water:	1258.73 g	

Moisture after mixing:

Bowl57 weight:	199	g
Bowl57 + agg.:	789	g
After 24 h:	727	g
	7.85805	% moisture in specimen

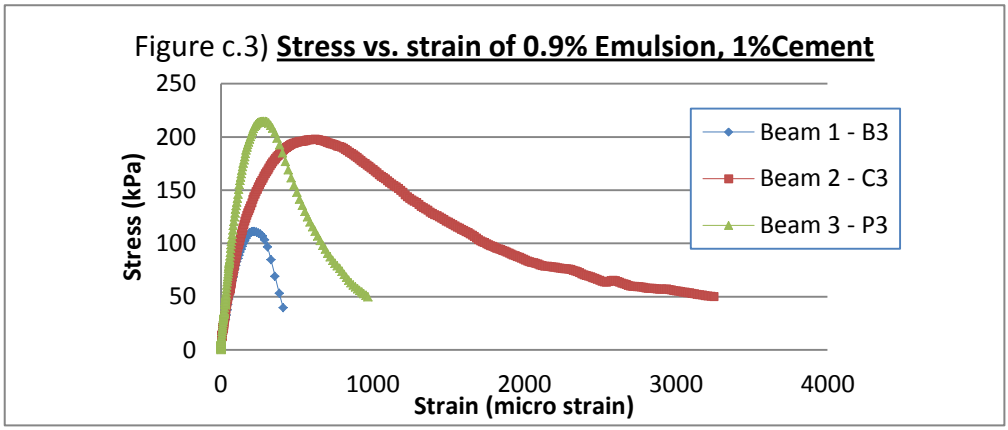
Beam specimen name	Beam dimensions: Hight (mm) Height (mm)		Average Height (mm)	Width (mm)	Length (mm)	Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
Beam 1 - B3	75	73	72.5833	75	470	5.78	0.002559	7.85805	2259.08	2094.495
	74	72								
	72	71								**99.74 %** of MOD density
	71	72								
	71	73								
	72	75								
Beam 2 - C3	73	74	71.75	75	470	5.722	0.002529	7.85805	2262.39	2097.559
	71	71								
	70	72								**99.88 %** of MOD density
	70	72								
	71	72								
	72	73								
Beam 3 - P3	70	74.5	71.9583	75	470	5.737	0.002537	7.85805	2261.75	2096.969
	72	73								
	70	71								**99.86 %** of MOD density
	70	72								
	71	73								
	73	74								

Table C 5) **Beam specimens properties - 0.9% bitumen emulsion, 1% Cement**

APPENDIX C - Laboratory strain-at-break results

Stress vs. strain graphical results:

Figure c.3) **Stress vs. strain of 0.9% Emulsion, 1%Cement**



Beam specimen name	Stress- at-break (kPa)	Strain-at-break (με)	Stiffness (Mpa)	Dissipation energy (N/m^2) - Pa	Yield strength (kpa)
Beam 1 - B3	110.9881270	212.5943970	522.065156	15.6086871	109.6497
Beam 2 - C3	197.6236570	624.7633910	316.317601	55.5861868	192.5964
Beam 3 - P3	214.5122350	292.1522190	734.248180	45.9933618	214.1776
Average	174.3746730	376.5033357	524.210312	39.0627452	172.1412

Table C.6) **Data obtained from Stress vs. Strain curve**

Specimens containing 2.4% Foam bitumen, 1% Cement:

Aggregate moiture:

Bowl weight:	132.88	g
Bowl + agg.:	513.9	g
After 24 h:	496.9	g
	3.308	% moisture in agg.

Mixture specimens:

Material:	17500 g	
Bitumen	420 g	=> 2.4% Bitumen
Cement:	175 g	=> 2% Cement
Water:	1381.09 g	80% 1104.87 g
		20% 276.219 g

Moisture after mixing:

Bowl17 weight:	153	g
Bowl17 + agg.:	710	g
After 24 h:	650	g
	8.4507	% moisture in specimen

Foamed bitumen characteristics:

Expansion = 12

Half life = 9 seconds

APPENDIX C - Laboratory strain-at-break results

Beam specimen name	Beam dimensions				Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
	Hight (mm)			Width (mm)	Length (mm)					
	Height (mm)		Average Height (mm)							
Beam 1 - B4	74	73	72.5833	75	470	5.729	0.002559	8.4507	2239.15	2064.669
	73	71								
	73	71								**98.32 %**
	73	71								of MOD
	73	72								density
	74	73								
Beam 2 - C4	74	75	72.5833	75	470	5.741	0.002559	8.4507	2243.84	2068.994
	71	73								
	70	74								**98.52 %**
	70	72								of MOD
	71	72								density
	74	75								
Beam 3 - P4	73	75	73.3333	75	470	5.796	0.002585	8.4507	2242.17	2067.452
	71	72								
	70	73								**98.45 %**
	71	73								of MOD
	74	74								density
	78	76								

Table C 7) **Beam specimens properties - 2.4% Foam bitumen, 1% Cement**

Stress vs. strain graphical results:

Figure c 4) **Stress vs. strain for 2.4% Foam, 1%Cement**

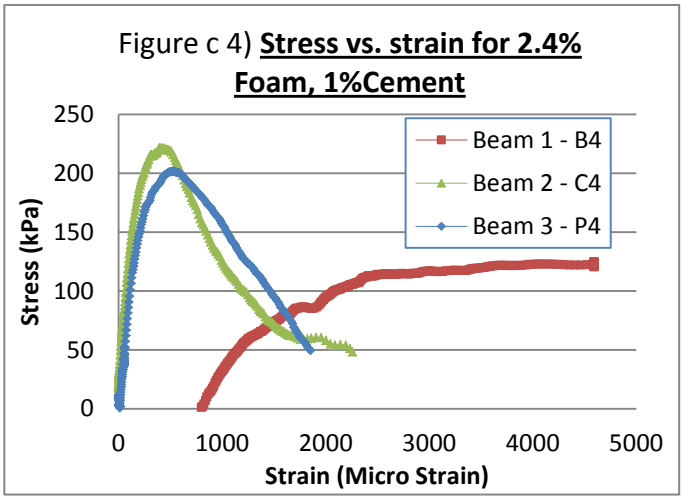
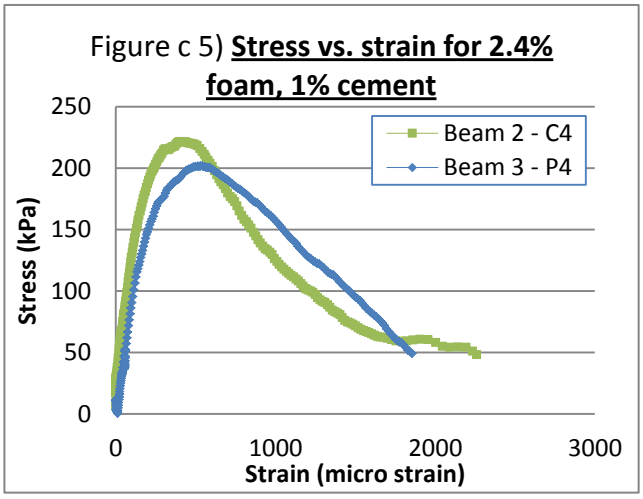


Figure c 5) **Stress vs. strain for 2.4% foam, 1% cement**



* Note that Beam 1 in Figure b.4 show unreliable results, therefore it was assumed as an outlier and has been removed from the data. Figure b.5 will thus be the final results.

APPENDIX C - Laboratory strain-at-break results

Beam specimen name	Stress- at-break (kPa)	Strain-at-break (με)	Stiffness (Mpa)	Dissipation energy (N/m^2) - Pa	Yield strength (kpa)
Beam 1 - B4	124.48253	4595.23187	27.08950	375.39290	-
Beam 2 - C4	221.50391	429.24970	516.02577	66.61472	217.9589
Beam 3 - P4	201.64754	532.33135	378.80080	70.80571	196.2163
Average	211.57573	480.79053	447.41329	68.71021	207.0876

Table C.8) **Data obtained from Stress vs. Strain curve**

Specimens containing 2.4% Foam bitumen, 2% Cement:

Aggregate moiture:

Bowl weight:	132.88	g
Bowl + agg.:	513.9	g
After 24 h:	494.9	g
	3.69722	% moisture in agg.

Mixture specimens:

Material:	17500	g	
Bitumen	420	g	=> 2.4% Bitumen
Cement:	175	g	=> 2% Cement
Water:	1312.99	g	80% 1050.39 g
			20% 262.597 g

Moisture after mixing:

Bowl12 weight:	202	g
Bowl12 + agg.:	737	g
After 24 h:	677	g
	8.14111	% moisture in specimen

Foamed bitumen characteristics: Expansion = 12
Half life = 10 seconds

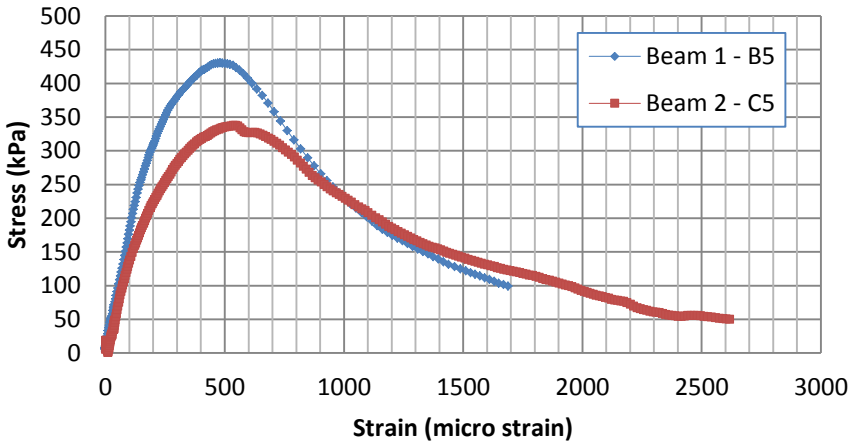
Beam specimen name	Beam dimensions: Hight (mm) Height (mm)	Average Height (mm)	Width (mm)	Length (mm)	Mass (kg)	Volume (m^3)	Moisture content (%)	Bulk Density (kg/m^3)	Dry density (kg/m^3)
Beam 1 - B5	75 73 74 72 72 71 71 72 71 73 72 75	72.5833	75	470	5.751	0.002559	8.14111	2247.75	2078.531 **98.98 %** of MOD density
Beam 2 - C5	73 74 72 73 71 72 72 72 73 73 73 74	72.6667	75	470	5.768	0.002562	8.14111	2251.81	2082.284 **99.16 %** of MOD density
Beam 3 - P5	70 74.5 72 73 70 71 70 72 71 73 73 74	71.9583	75	470	5.737	0.002537	8.14111	2261.75	2091.48 **99.59 %** of MOD density

Table C 9) **Cylindrical specimens properties - 2.4 Foam bitumen, 2% Cement**

APPENDIX C - Laboratory strain-at-break results

Stress vs. strain graphical results:

Figure c.6) **Stress vs. strain of 2.4% foam, 2% cement**



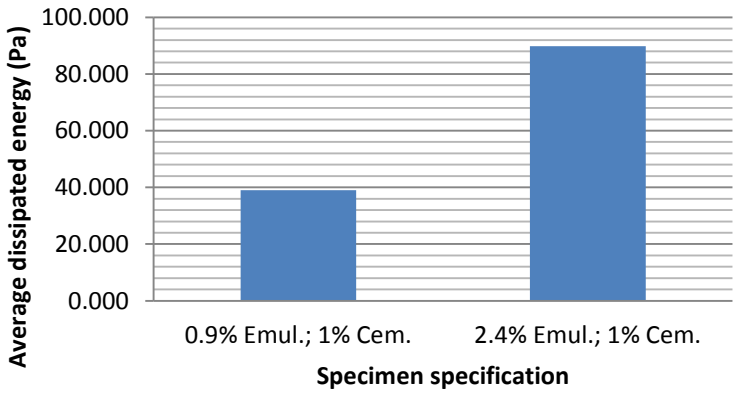
* Note that Beam 3 in Figure b.6 have been removed from the graph since results show to be unusable.

Beam specimen name	Stress- at-break (kPa)	Strain-at-break (με)	Stiffness (Mpa)	Dissipation energy (N/m^2) - Pa	Yield strength (kpa)
Beam 1 - B5	430.301257	480.366473	895.77704	145.3343311	419.3013
Beam 2 - C5	337.310482	537.033064	628.10003	157.1970836	326.2375
Beam 3 - P5	502.246752	4491.140876	111.83055	-	-
Average (of beam 1 and 3)	383.8058695	508.6997685	761.93853	151.2657073	372.7694

Table C.10) **Data obtained from Stress vs. Strain curve**

Influence of additives on the dissipated energy of BSMs

Figure c7) **Influence on average Dissipated energy (Pa) with change in bitumen content**



APPENDIX C - Laboratory strain-at-break results

Figure c 8) **Influence on average Dissipated energy (Pa) with change in cement for emulsion treated specimens**

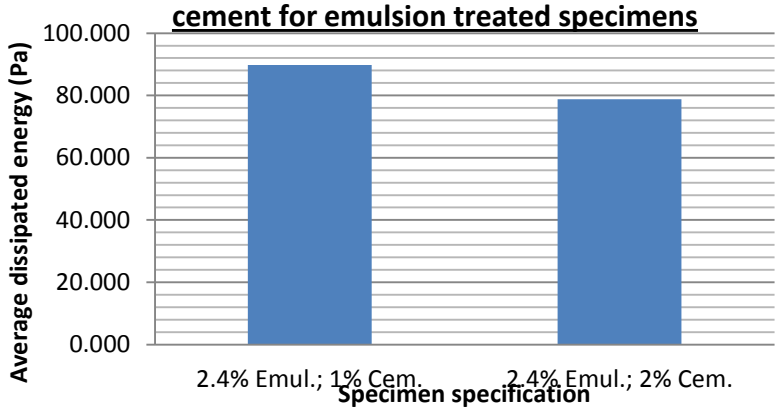


Figure c 9) **Influence on average Dissipated energy (Pa) with change in cement for foam treated specimens**

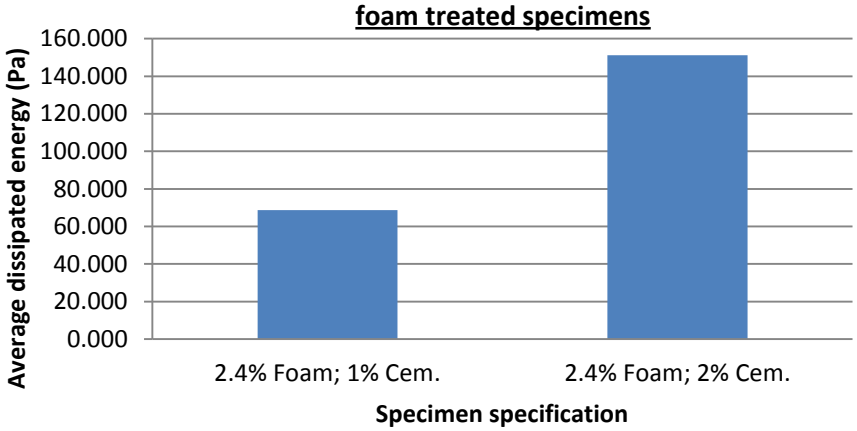


Figure c 10) **Influence of bitumen type on the average Dissipated energy (Pa) with 1% cement**

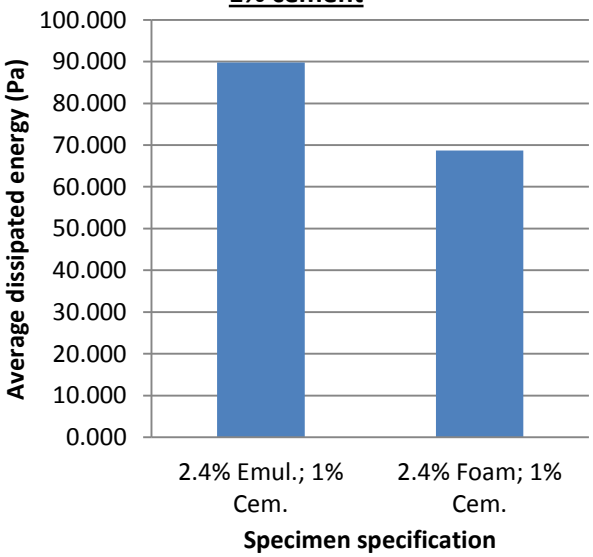


Figure c 11) **Influence of bitumen type on the average Dissipated energy (Pa) with 2% cement**