FLEXIBILITY AND PERFORMANCE PROPERTIES OF BITUMEN STABILISED MATERIALS

by

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DECLARATION

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SUMMARY

This research investigates the flexibility and the performance properties of bitumen stabilised materials under the influence of mix variables. The laboratory testing consisted of two main phases. During the first phase (mix design), the strength and the flexibility of the mixes were assessed through ITS (Indirect Tensile Strength), UCS (Unconfined Compressive Strength), displacement at break, strain at break and fracture energy. The second phase consisted of a series of triaxial tests done to assess the performance properties (shear strength: cohesion and angle of internal friction; and stiffness: resilient modulus) of the mixes.

The mineral aggregates used in this study were milled from different locations of the R35, near Bethal. This was a blend of granular material (dolerite, from various locations of the existing base and subbase layer of the R35) and Reclaimed Asphalt (RA) milled from the existing surfacing. During the mix design phase, two types of bituminous binders were used (bitumen emulsion and foamed bitumen) at bitumen content ranges of 2%, 2.4% and 2.8% each. Two types of active filler were used separately and in combination at a proportion of 1% and 2%. Finally, specimens were tested in wet and dry conditions for each mix combination. During the triaxial testing phase, only the optimum bitumen content of 2.4% was used, both for bitumen emulsion and foamed bitumen, with only cement as active filler in a proportion 1% and 2%. The specimens were tested at different ranges of densities and saturation levels.

The flexibility of the mix was assessed through the fracture energy, the strain and the displacement at break parameters. An analysis of variance (ANOVA) was conducted on the data to assess the significance of experimental variables on this property. This property was found to be very sensitive to bitumen and cement content added to the mix. When assessing the combined effect and the significance of the variables on the flexibility of the mixes, it was found that fracture energy is mostly influenced by the cement content, followed by the bitumen content, then the type of treatment and finally the testing condition. However, the level of significance was not in the same order for the other two parameters (displacement and strain at break). It was also found that the combined effect of some independent variables (cement content + testing condition, type of treatment + cement content + bitumen content) had a significant effect on the fracture energy and the strain at break respectively.

From the ITS and UCS tests, an increase in strength was noticed with the increase of cement content. On the other hand, the increase in bitumen content led to a decrease in strength of the material. The statistical analysis on the ITS and UCS values show that the independent variable with the most

significant effect on the ITS is the cement content, followed by the testing conditions, then the bitumen content and finally the type of treatment. The combined effect of cement content + bitumen content was found to be significant both for ITS and UCS.

In the second phase triaxial tests were performed in order to evaluate the performance properties of the mixes. It was found that the increase of the active filler content significantly improves the shear strength of the material. It was also found that at a fixed cement content, specimens tested at low density and/or high level of saturation show low shear strength. The Mr-θ model was used to model the resilient modulus of the mixes and the model coefficients used to evaluate the effect of experimental variables on the resilient modulus. It was found that the resilient modulus of the mixes increases as the bulk stress increases. This confirms the stress dependent behaviour of bitumen stabilised materials. The analysis show that increasing the percentage of active fillers content results in a significant increase in the resilient modulus values. An increase in relative density also resulted in an increase in the resilient modulus of the mixes, while the opposite effect was observed with the increased of the saturation level.

Besides the engineering properties and the mechanical test parameters, other parameters such as the Tensile Strength Ratio (TSR) was calculated in order to evaluate the moisture sensitivity of the mixes. Weakening due to moisture was found to be more predominant in the mixes with less active filler. In addition, bitumen emulsion mixes were found to have a better resistance to moisture weakening effects compared to foamed bitumen. In addition, a comparison between the rapid curing and the accelerated curing was done. Higher ITS and UCS results were obtained for specimens cured using long term curing compared to specimens cured using the accelerated curing method.

In conclusion, flexibility is an important property of bitumen road construction material (bitumen stabilised material include) however, it is not an easy property to measure. Although, displacement/strain at break and fracture energy from ITS and UCS were able to give us some indications on the main factors governing the flexibility of bitumen stabilised materials (the bitumen and active filler content), more accurate and adequate tests are required to evaluate the parameter.

OPSOMMING

Die buigsaamheid en gedragseienskappe van bitumen gestabiliseerde materiale was getoets om sodoende die invloed van verskeie mengselveranderlikes te evalueer. Die ondersoek het uit twee fases bestaan. Tydens die eerste fase (mengfase) is die sterkte en buigsaamheid deur middel van indirekte treksterkte toetse (ITS), onbegrensde druksterkte toetse (UCS), verplasing – en vervorming by breekpunt sowel as breek-energie toetse gedoen en ondersoek. Die tweede fase het bestaan uit 'n reeks drie-assige triaksiaal toetse. Triaksiaaltoetse is uitgevoer om die gedragseienskappe soos die skuifsterkte, kohesie, hoek van interne wrywing, styfheid en weerstand modulus te ondersoek.

Die gemaalde mineraal-aggregaat wat in hierdie ondersoek gebruik is, was verkry op verskeie areas van die R35, geleë naby Bethal. Die materiaal is 'n mengsel van granulêre materiaal (van die bestaande kroonlaag en stutlaag van die pad) en herwonne asfalt (RA). Tydens die mengontwerp fase is twee tipes bitumen gebruik naamlik bitumenemulsie en skuimbitumen in hoeveelhede van 2%, 2.4% en 2.8%. Twee tipes aktiewe vulstof (hoeveelhede van onderskeidelik 1% en 2%) was saam met elk van die verskeie bitumen-hoeveelhede gebruik. Proefstukke van elk van hierdie mengsel kombinasies is onder beide nat en droë kondisies getoets. Tydens die tweede fase, is slegs die optimum binder inhoud (2.4%) gebruik vir beide emulsie- en skuimbitumen, gekombineer met 1% en 2% aktiewe vulstof. Proefstukke was getoets by 'n reeks van verskillende digthede en versadigingvlakke.

Die buigsaamheid was ondersoek deur middel van breek-energie, vervorming en die verplasing by breekpunt. 'n Analise van variasie (ANOVA) is uitgevoer op die toetsdata om sodoende die te evalueer of die veranderlikes beduidend is ten opsigte van buigsaamheid. Daar is gevind dat die buigsaamheideienskap sensitief is vir beide bitumen en sement inhoud. Met assessering van die gekombineerde effek en betekenis van die veranderlikes op die buigsaamheid van die mengsels, is daar gevind dat die hoogste beduidende veranderlike t.o.v breek-energie die sement inhoud is, gevolg deur die bitumeninhoud, tipe behandeling en laastens die toetskondisie. Die orde van belangrikheid verskil vir die ander twee parameters (verplasing en vervorming by breekpunt). Daar is ook gevind dat die gekombineerde effek van sommige veranderlikes (sement inhoud en toets kondisie, tipe behandeling en sement inhoud tesame met bitumen inhoud) ook beduidend was t.o.v breek-energie en vervorming by breekpunt.

Vanuit die ITS en UCS toetse was daar 'n toename in sterkte waargeneem soos die sementinhoud toeneem. Aan die anderkant, het 'n toename in bitumeninhoud 'n afname in sterkte veroorsaak. Die statistiese analise van ITS en UCS resultate, toon dat die grootste beduidende onafhanklike t.o.v ITS

waardes ook die sement inhoud was, gevolg deur toets kondisies die grootste effek, bitumen inhoud en die tipe behandeling. Die gekombineerde effek van sementinhoud en bitumeninhoud, was betekenisvol vir beide ITS en UCS.

Drie-assige triaksiaaltoetse was uitgevoer om die gedragseienskappe van die mengsels te evalueer. Daar is gevind dat die toename in sement inhoud, die skuif sterkte van die materiaal grootliks verbeter. By 'n konstante sementinhoud, wys toetsresultate van proefstukke wat getoets is by lae digthede en hoë vlakke van versadiging, lae skuif sterkte.

Die Mr – θ model was gebruik om die veerkragsmodulus van die mengsels te moduleer en die modelkoëffisiënte is gebruik om die effek van eksperimentele veranderlikes op die weerstand modulus te evalueer. Met toename in die omhullende spanning is 'n toename in die veerkragsmodulus waargeneem, wat bevestig dat die gedrag van bitumen gestabiliseerde materiale spannings afhanklik is. 'n Toename in die sement en relatiewe digtheid het 'n merkwaardige toename in die veerkragsmodulus tot gevolg gehad, terwyl die teenoorgestelde waargeneem is met toename in versadigingsvlakke.

Buiten die ingenieurseienskap en meganiese toetsfaktore, is ander faktore (soos die trekspanning verhouding) bereken om die vogsensitiwiteit van die mengsels te evalueer. Mengsels met laer sement inhoud het groter verswakking ervaar met blootstelling aan water. Bitumenemulsie proefstukke toon beter weerstand teen water as skuimbitumen. Vergelyking tussen versnelde en korttermyn nabehandelingsprosedure van proefstukke, toon hoër ITS en UCS waardes vir die versnelde nabehandelingsprosedure prosedure.

Buigsaamheid is 'n belangrike eienskap van bitumen in padkonstruksie materiale (insluitend bitumen gestabiliseerde materiale), maar word moeilik gemeet. Alhoewel verplasing/vervorming by breekpunt en breek energie, bepaal vanaf ITS en UCS, 'n indikasie toon van die hooffaktore (binder en sement) wat buigsaamheid van bitumen gestabiliseerde materiaal beïnvloed, word meer akkurate toetse benodig om die eienskap te ondersoek.

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LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO : American Association of Highway and Transportation Officials

BBR: Bending Beam Rheometer

BSM-emulsion: Bitumen stabilized materials with bitumen emulsion as binder

BSM-foam: Bitumen stabilized materials with foamed bitumen as binder

BSMs: Bitumen Stabilized Materials

CBR: Californian Bearing Ratio

CSIR: Council for Scientific and Industrial Research; based in Pretoria, South Africa

ERm: Expansion Ratio of the Foamed Bitumen

EMC : Equilibrium Moisture Content

FI: Foam Index

HMA: Hot Mix Asphalt

HVS: Heavy Vehicle Simulator

ICL: Initial consumption of lime

ITS: Indirect Tensile Strength

ITS_{dry}: Indirect tensile strength at equilibrium moisture content

ITS_{wet}: Indirect tensile strength after soaking the specimen in water

LVDT: Linear Variable Displacement Transducer

MESA: Million Equivalent Standard Axles, 80 kN axles

MTS : Material Testing and Simulation

PH: Power of Hydrogen

RAP : Reclaimed Asphalt Pavement

S.R. : Stress Ratio

SABITA : South African Bitumen Association

T $_{1/2}$: Half-live

TG2: Technical Guideline: Bitumen stabilized Materials. TG2 second Edition, May 2009

TMH: Technical Methods for Highways

UCS : Unconfined Compressive Strength

CHAPTER 1: INTRODUCTION

1.1 General introduction and background

For many years, lime and cement have been used for modification and stabilization of pavement materials in road construction and rehabilitation. The benefits associated with the technology are: the increase in strength and durability, the reduction of moisture damages and improved workability of the treated material. However, when subjected to trafficking over time, cement treated layers in pavements end up with shrinkage cracks due to the fully bound characteristics of cement treated layers and their high stiffness. Moreover, just as concrete, cemented layers yield a low tensile strength and are susceptible to crack under repeated flexure. Bitumen stabilisation is one solution to the above mentioned shortcomings.

Bitumen emulsion and foamed bitumen treated materials, also called Bitumen stabilised materials (BSM's), are the most used form of bitumen stabilisation used in pavement engineering across the world today. They are known to have the following characteristics:

- Combining rigidity and flexibility.
- Improving durability and moisture sensitivity of the treated material.

Although the technology is well known and has been successfully used worldwide for the past five decades, it is however believed that the lack of standard mix design procedures has limited its implementation in South Africa (Muthen, K.M. 1999).

In 2002, the Technical Guideline (TG2), which includes the design and use of foamed bitumen treated material, was published by the Asphalt Academy. However, the development of the guidelines did not include emulsion treated material and was limited to materials of moderate quality. Therefore, the guideline was revised in May 2009. In the new TG2, the classification of Bitumen Stabilised Materials (BSM) for design and use purposes in South Africa is based on ITS and UCS values, obtained at the mix design level in the dry and soaked condition. This implies that the principal parameter considered for BSM's structural design and material classification is strength. However, the behaviour of bitumen stabilized materials, relative to other pavement construction materials falls between cement treated materials which are stiff and brittle, and hot mix asphalt which is more flexible and has visco-elastic properties (i.e. is temperature dependent). Therefore, the need to take into account flexibility in BSM design and classification arises. This will provide a better understanding of BSM's properties and structural behaviour for design improvement.

The problem of accurate displacement measurements for flexibility assessment of bitumen stabilised materi als was raised in 2004 by the CSIR. Investigations were made in this regard by the CSIR on the four-point monotonic load beam test to measure flexibility and tensile strength of bitumen stabilized materials from strain at break values. However, it was recommended that more appropriate flexibility tests should be investigated and included in the mix design procedure of bitumen stabilised materials.

1.2 The need for flexibility properties for pavement materials

1.2.1 What is flexibility?

Generally, flexibility is defined as the ability for a particular material to deform elastically and return to its original shape after the applied force is removed. Flexibility is a material property that is linked to deflection and bending. In other words, flexibility is the measure of a material's ability to flex or bend without breaking or without being damaged. As illustrated in Figure1.1, flexibility is a combination of ductility (ability to deform plastically) and brittleness (ability to break without permanent distortion).
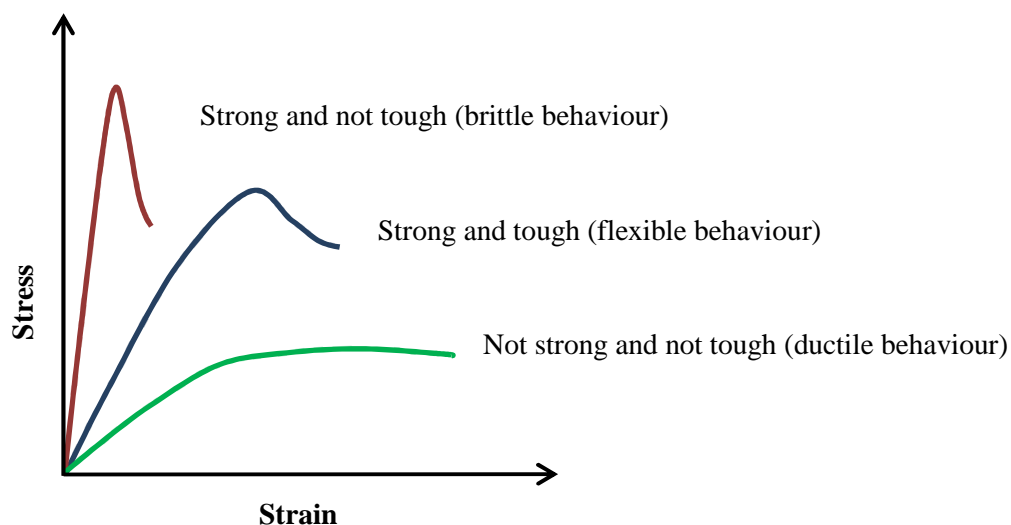
Figure 1.1 Illustration of engineering flexibility

For a material to be strong does not necessary guarantee that it is tough as well. A strong material will require a high load to be broken but not much energy (i.e. it does not allow much deformation before it breaks). They are brittle materials. On the other hand, some materials can undergo significant deformation or elongation without breaking but cannot support high load because they have very low modulus. There are ductile materials. Though it is good to use materials with high modulus because of their ability to withstand heavy loads, using a material that is not only strong, but which can also bend without breaking is better as it can ultimately withstand larger deformations

(including flexure) before it fails. Therefore, in designing flexible material, a little bit of strength is compromised and the amount of plastic deformation reduced in order to have a tough material.

It is clear from the definition, that flexibility is not a single material property, but has various parameters. For bituminous materials, flexibility is influenced by temperature. Under low temperature conditions, the bitumen within the material becomes hard and that leads to the stiffening of the material, which tend to behave more like brittle material. On the other hand, high temperates will soften to bitumen and the material will tend to exhibit increased ductile behaviour. Moreover, the stiffness of bitumen stabilized material is highly related to the percentage of active fillers added to the mix which the bitumen content influences their flexibility. Therefore, having a good balance between the cement and the bitumen content in the mix is indispensable to ensure the optimal strength and flexibility that will satisfy the mix design requirements.

1.2.2 Benefits of flexibility properties in pavement structure

Flexible pavements can be defined as pavements that are not considered to be a cement concrete pavement or concrete block pavement (Molenaar, A.A.A. 2011). Generally, their structure includes from the top to the bottom a bituminous-treated surface, one or more unbound or bitumen treated base layers, a subbase and a subgrade. The purpose of having such a layered system is to facilitate the load distribution from the traffic through the pavement by protecting each underlying layer including the subgrade from compressive shear failure. Loads from the traffic are distributed on an increasingly wide area between deep layers. Therefore, better materials are used from the top to the bottom of the pavement structure to resist higher near-surface stress condition caused by the traffic wheel loads.

The particularity of flexible pavement is that the total pavement structure, composed of several layers of materials (layer system), deflects under loading. This enables a good distribution of the loads through the layer system to the ground and provides safe and smooth movement of the vehicles. As said previously, bitumen stabilised materials are predominantly used for the construction of base and subbase layers of flexible pavements. On the other hand, flexibility in the pavement layer system is created by the nature of the materials that constitute the different layers. Therefore, it is crucial for BSMs to possess flexible properties in order to satisfy the need above stated. Though flexibility of bitumen stabilised materials is a difficult property to measure, knowing the factors that govern it and understanding their influences could have following benefits:

- Improve and optimise the mix design procedures;
- Better understand and anticipate the failure mechanisms;

- Predict the short and long term performance of the material.

1.2.3 Performance characteristics and properties required of pavement materials

In order to meet the properties required by the layers as mentioned earlier, basic questions such as: "what material should be used" or "how strong should the material be" must be answered. Materials for pavement layer construction are selected and evaluated with care in order to predict and ensure their performance when subjected to traffic loads. Engineering and mechanical tests are conducted in that regard: ITS, UCS, monotonic and dynamic triaxial testing. Through these tests material properties such as tensile and compression strength, shear parameters, resilient modulus and permanent deformation can therefore be determined and/or predicted.

1.3 Objectives and limitations of the research

1.3.1 Objectives

The primary objective of this study is to evaluate and understand the flexibility of bitumen stabilised materials. It includes the following:

- Investigate if flexibility indicator parameters such as displacement or strain at break and the fracture energy from the load displacement curve of ITS , UCS test are appropriate and robust enough to assess the flexibility of bitumen stabilised materials with reliability;
- Understand how the mix variables (i.e. the type of treatment, the percentage of bitumen added, the type and percentage of active fillers used as well as the moisture condition and material density when tested) influence the flexibility behaviour of bitumen stabilised materials.
- Determine the significance of each mix variable as well as their combined effect on the flexibility by means of statistical analysis.

The more pavement materials are understood, the better they can be used for their intended purpose. Therefore, the second objective is to evaluate the performance properties of bitumen stabilised material under the effect of the mix variable through triaxial testing. This includes:

- Determine the shear parameters by means of monotonic triaxial and the evaluation of the shear strength of the mixes;
- Evaluate the stiffness (resilient modulus) of the mixes under repeated loading by means of dynamic triaxial testing;

- Assess the influence of experimental variables on the shear strength and the stiffness of the mixes.

1.3.2 Limitations

The limitations of this study include the following:

- The determination of the strain at break from the ITS test; this is due to the configuration in which the specimen is positioned during the test. The displacement at break obtained at the maximum load cannot be used for strain at break calculation for it is not measured over the height of the specimen. Therefore, in evaluating the flexibility of the different mixes from the ITS test results, the displacement at break was considered instead of the strain at break.
- Investigations are limited to mixes that are made of one type of aggregate and the experimental testing is limited to ITS, UCS, monotonic triaxial and dynamic triaxial test (Mr).
- Flexibility evaluations from monotonic triaxial test were not included in the study, as this forms part of a separate study.
- It must be emphasized that this research project was undertaken with limited material resources taken from the R35. The range of repeat tests and variables being investigated were selected by the client in conjunction with the available material. Although an increased number repeat tests were desired, the unavailability of the material did not allow this.

1.4 Layout of the dissertation

The dissertation of this study is has been divided in five chapters described as it follows.

Chapter 1: Introduction – A background to the research is given followed by the shortcomings on the other mode of stabilisation and the importance of flexibility for bitumen stabilised materials. The objectives and limitations of the research are also presented as well as the layout of the dissertation.

Chapter 2: Literature Study – This section provides literature on bitumen emulsion and foamed bitumen stabilisation (definition, history and performance properties and use). Secondly, the concepts of flexibility as defined in general and in pavement materials with bitumen stabilised materials as a case study. Finally, the test evaluation methods of flexibility are presented and described.

Chapter 3: Methodology and Experimental Design – The experimental matrix and testing methodology (materials preparation, mixing and compaction, curing and testing) are the main focus of this section. The laboratory testing protocols and the methodologies used for data processing are

also described followed by the interrogation of whether or not displacement at break, strain at break and fracture energy can be used to evaluate the flexibility of bitumen stabilised materials.

Chapter 4: Material Testing Results, Interpretation, Findings, and Discussion – In this section, results of all the tests conducted are presented in tables and illustrated in graphs. The results presentations are followed by their interpretations with a focus on the influence of mix variables (i.e. type of treatment, the bitumen and active fillers content, the density and moistures condition) on the material properties (i.e. Strength, flexibility, shear parameters and stiffness under repeated load). Finally, the findings are highlighted and discussed.

Chapter 5: Conclusion and Recommendations – This last section of the dissertation provides syntheses on findings of the study with an emphasis on how they can contribute toward improving current design protocol of bitumen stabilised materials. Finally, recommendations are made for further studies on the topic.

CHAPTER 2: LITERATURE STUDY

This chapter provides a background on bitumen emulsion and foamed bitumen technology. Definition of main concepts such as Bitumen Stabilised Material and flexibility, being the main topic treated in this study, are given. It also includes an overview on the flexibility of bitumen stabilised materials, their performance properties, as well as their failure mechanisms. Finally, a survey of test evaluation methods of flexibility is provided and an answer is given to the question: Can strain-at-break and fracture energy be used to assess the flexibility of bitumen stabilised materials?

2.1 Introduction

Through the years, driving forces such as innovation, need of material properties improvement for better performance, construction cost reduction, limitation of negative impacts on the environment caused by material extraction and transport, and many others, have pushed pavement engineers to improve design procedures for construction and rehabilitation methods of road infrastructures. In this regard, the usage of bitumen, to modify and stabilise road construction materials, was found to be a suitable solution. The technology has been used successfully worldwide for the past four decades, to the satisfaction of the above listed requirements. Moreover, layers constructed from materials stabilised with bitumen are more flexible, as is required of flexible pavement layer system, and do not suffer from the shrinkage cracking phenomenon associated with cement stabilization (Wirtgen, 2012).

Bitumen Stabilised Materials (BSMs), being the focus in this study, are currently widely used in the industry of road construction and rehabilitation nowadays. Many countries have found a way through this technology, to limit the shortcomings of other methods of stabilisation, as well as the problems related to rehabilitation cost and environment conservation. Additionally, bitumen stabilisation improves the properties of the original material. Some benefits of bitumen stabilisation are:

- Improved strength;
- Increased stiffness;
- Improved durability (reduced moisture susceptibility); and
- Addition of flexibility.

Flexibility is an important property of bitumen stabilised materials. It is an indicator for flexural characteristics and for resistance of the two main sources of pavement deterioration (i.e. deformation and cracking). The property was mentioned in mix design of bitumen stabilised materials (TG2,

2009) for it enables layers made of bitumen stabilised materials to carry considerable traffic on relatively weaker support.

However, flexibility of BSMs is not an easy property to measure. The entire tests conducted so far in this regard are just used as indicators, because of the variation of the actual loading condition on the field. Therefore, they cannot easily qualify completely the long-term flexibility of bitumen stabilised materials.

2.2 Definition of bitumen stabilised materials

BSMs are pavement materials treated with either bitumen emulsion (BSM-emulsion) or foamed bitumen (BSM-foam), with the addition of a small percentage of active fillers. The treated material is used for the construction of pavement's base and subbase layers. The technology is currently used worldwide and literature on its production and usage is well documented.

A wide variety of pavement materials can be stabilised with bitumen emulsion and foamed bitumen ranging from sand through gravels to crushed stone and reclaimed asphalt (RA). This is usually the technology used in pavement engineering for the recycling of existing road. Therefore, the main type of aggregate used in BSMs production are reclaimed asphalt layers (RA, i.e. old seal or asphalt surfacing) and granular materials base layer, cement treated base included. In practice, the two materials (reclaimed asphalt layer and the underlying base layer) are mixed together and treated with bitumen emulsion or foamed bitumen to form a new base or subbase layer. The final product is slightly darker in colour but do not have a sticky feel unlike hot-mix asphalt.

During the treatment, the bitumen disperses among the aggregate and coats the fines aggregates particles. This leads to the formation of the mortar, which will then disperse through the coarse aggregate particles and bind them together. More often, a small percentage of active filler (lime or cement) is added to the mix in combination with bitumen emulsion or foamed bitumen. The active filler content usually varies from 1 to 2 % and is recommended not to exceed the percentage of bitumen content. The addition of the bitumen and active fillers to the granular materials results in an increase in strength, flexibility and resistance to moisture damages. Moreover, stabilization with bitumen results in an increase in the shear strength of the treated material (Ebels, L.J. 2008). Such increase in shear properties empowers the layer made of BSMs to withstand higher stresses from the heavy truck loads.

Bitumen stabilised materials are neither hot mix asphalt, nor cement treated materials. Previous studies have shown that the behaviour of pavement layers constructed from bitumen stabilized

materials, falls between rigidity and visco-elasticity. However, it is possible to create a BSMs mix with behaviour characteristics that are similar to those of granular material, hot mix asphalt or cement treated materials, by varying the proportion of the mix components (i.e. blending of aggregates, bitumen and active fillers). The behaviour of BSMs, compared to other pavement materials, is illustrated in Figure 2.1.
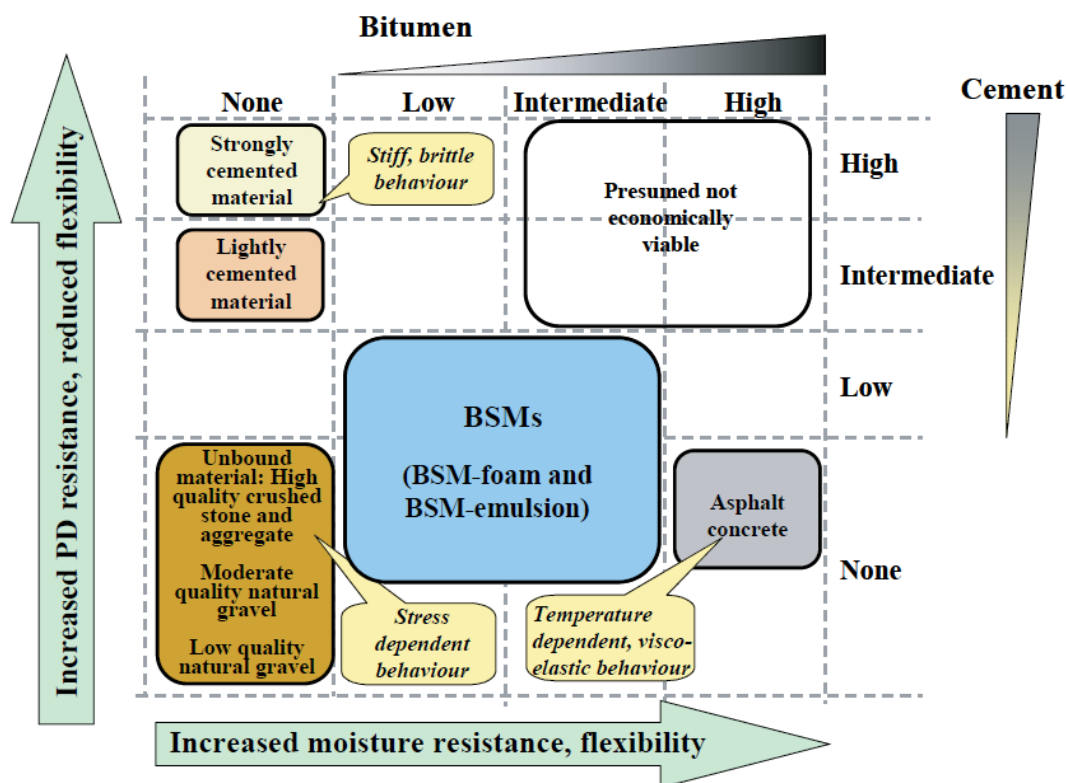
Figure 2. 1: Conceptual behaviour of pavement materials (TG2 second edition, 2009)

2.2.1   Emulsion Bitumen Stabilization

The first bitumen emulsions were developed in the 1900's. Their use in pavement engineering started in the early 1920's with spray application and control of dust generated by the traffic on gravel roads. It was only in the 1950's that their interest for road market picked up. Recently, development and new formulations have increased their performances, resulting in optimal application in road construction and rehabilitation.

2.2.1.1   *What is bitumen emulsion?*

The colloidal system in which fine droplets of one liquid are dispersed in another liquid is called emulsion. In other words, an emulsion is a stable dispersion of two immiscible liquids. Likewise, bitumen emulsion is a dispersion of small droplets of oil (bitumen) into water or vice-versa. The

quantity of water is generally in the order of 30 to 40% of the total solution. A small amount of additives (i.e. emulsifier agent and chemicals) is usually added. These additives are essential in manufacturing bitumen emulsions. When put together, one will be in the dispersed phase and the other in the continuous phase. The final product is a liquid with a consistency ranging from that of milk to heavy cream and can be used in cold processes for road construction and rehabilitation. Depending on the nature of the dispersed phase, three mains type of bitumen emulsions can be distinguished (James, 2006).

- Oil-in-water emulsion (O/W);
- Water-in-oil emulsion (W/O); and
- Multiple emulsion (W/O/W) .
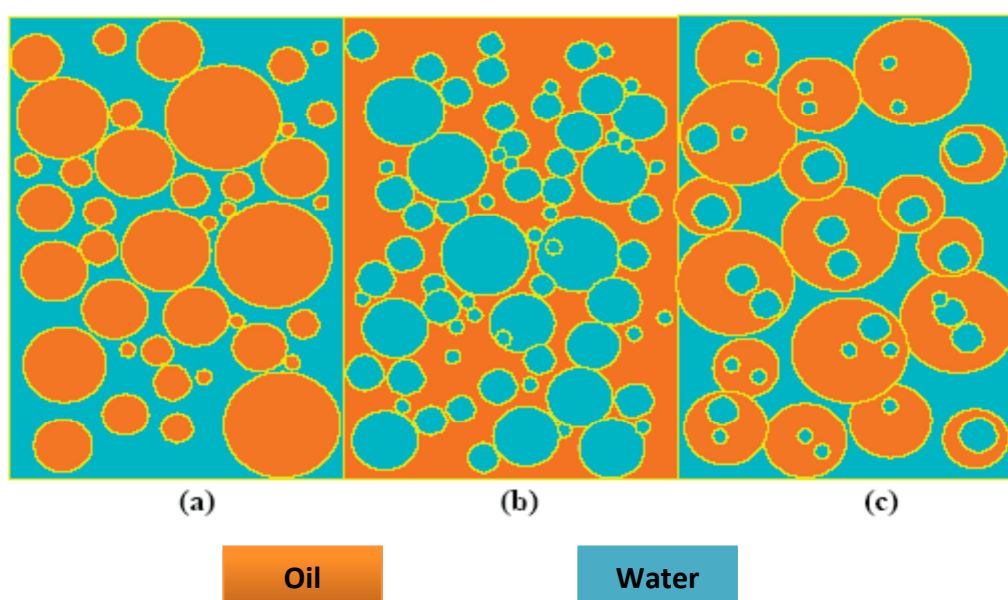
Figure 2. 2: Types of emulsion. (a) O/W emulsion, (b) W/O emulsion, (c) W/O/W emulsion (James, 2006)

The bitumen droplets particle dispersed in water differ in size. They are influenced by milling conditions (operating temperature, flow and shear rate) and chemistry (bitumen chemistry, emulsifier dosage). Their diameters typically range from 0.1 to 20 microns (Ebels, 2008). Particle size distribution of a particular emulsion affects its properties in the following way: smaller particle size distribution generally increase emulsion viscosity and improve storage stability (Coco Asphalt Engineering). The smaller the size of bitumen emulsion particles, the finer the dispersion will be, resulting in a slower breaking rate of the emulsion (Shell Bitumen Hand Book, pp.103).

2.2.1.2 *Bitumen emulsion production*

Bitumen is not very fluid at ambient temperatures. Therefore, in order to use it efficiently, there is a need to reduce its viscosity and bitumen emulsion technology is an effective way to do so. As said before, bitumen emulsions are made by mixing bitumen with water. First of all, the bitumen which is an unsoluble petroleum material in water, is pumped in the colloidal mill after being heated. Hot water is then added to the bitumen. This fusion turns to granules as shown in Figure 2.3.
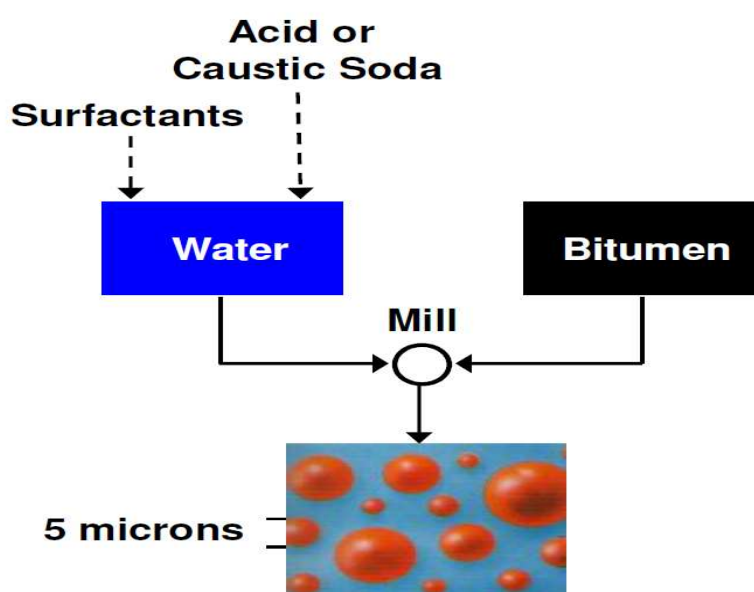
Figure 2. 3: Bitumen emulsion production illustration (Asphalt Academy, 2009)

The usage of sufficient mixing energy is required to break the bitumen into droplets. Chemical emulsifiers are also required to provide stability to the suspension of bitumen in water (i.e. not allowing bitumen to separate from water). Emulsifiers also give positive or negative charges to emulsions, depending on the type that is being used. Anionic emulsifiers are characterized by their negative charges while Cationic emulsifiers on the other hand are characterized by their positive charges. The choice of the emulsifier to be used during a particular bitumen emulsion production is essential, for it dictates the desired properties of the expected emulsion.

After being manufactured, bitumen emulsions could have a shelf life of several months, provided that the manufacturer's storage guidelines are strictly followed (TG2, 2009). They have to be handled and stored with care in order to ensure the quality of the mix. The following problems could occur when handling and storage are not done with care.

- **Flocculation**: it occurs when particles stick together.
- **Coalescence**: it occurs when particles that have stuck together becomes larger.

- **Settlement**: it occurs when they heavier bitumen particles settle due to prolonged storage, lack of mixing, problem with asphalt compatibility or incorrect chemical load.

2.2.1.3   *Emulsion aggregate reaction*

The interaction between emulsion bitumen and aggregates has two dimensions (i.e. physical and chemical). When mixed with aggregate, emulsifiers contained in emulsion lose their effectiveness. This causes the bitumen droplets to be attracted to the aggregate particles. Once in contact, aggregate take up droplets, depending on their nature (negative or positive droplet), the PH and the nature of the mineral aggregate.

2.2.1.4   *Advantages of bitumen emulsion*

- In most of the cases, bitumen emulsion can be used without additional heat;
- Bitumen emulsion offers flexibility to pavement construction materials;
- There are little or no hydrocarbon emissions with their use;
- One difference from foamed bitumen as far as application is concerned, is that bitumen emulsion can last for months before it is used (this can be seen as an advantage or a disadvantage) .

2.2.2   **Foamed Bitumen Stabilisation**

2.2.2.1   *Early days (brief history)*

The usefulness of foamed bitumen in pavement as soil binder was discovered in 1956 by Professor Ladis Csanyi at Iowa State University. Since then, foamed bitumen technology has been used successfully in many countries for road recycling and rehabilitation. Csanyi's original works demonstrated that by introducing saturated stream into heated bitumen, foamed bitumen can be produced and used efficiently to stabilise quite a few types of aggregate at relative low cost production. The steam foaming system was very convenient for bitumen plants where steam was readily available but it proved to be impractical for in situ foaming operations, because of the need for special equipment such as steam boilers. Mobil Oil (USA) later acquired the patent rights for Csanyi's invention. The Australian subsidiary of Mobil Oil applied the technology by adding cold water rather than stream in the hot bitumen. This method made the bitumen foaming process much more practical and less expensive for general use. Who could have imagined that anything good could come out by mixing bitumen with water? Professor Ladis Csanyi did and today, almost 50 years later, his discovery has revolutionised the way roads are recycled and rehabilitated all over the world.

2.2.2.2   *What is foamed bitumen stabilisation?*

Foamed bitumen stabilisation is a road construction and rehabilitation technique whereby the existing granular material (rehabilitation) or the imported granular material is mixed with foamed bitumen to produce a flexible non-continuously bound material, which is used for the construction of base and subbase layers of pavements. Currently, foamed bitumen is no more a new concept when it comes to pavement rehabilitation and stabilisation. Many countries across the world find in this technology a useful tool to help them maintain the integrity of their transportation infrastructure.

2.2.2.3   *What is foamed bitumen?*

Foamed bitumen is a binder made of a mixture of air, water and hot bitumen. It is produced by injecting a small quantity of cold water (approximately 2 to 3% by mass of bitumen), together with compressed air into hot bitumen (160 to 180^0 C) resulting in instantaneous foaming. When injected, the water in contact with the hot bitumen is turned into vapour, which is trapped in thousands of tiny bitumen bubbles. This results in the expansion of the hot bitumen to about fifteen to twenty times its original volume (Wirtgen, 2012). Figure 2.4 illustrates the foaming process of bitumen (how hot bitumen, cold water, and compressed air are mixed in the expansion chamber to produce foamed bitumen).
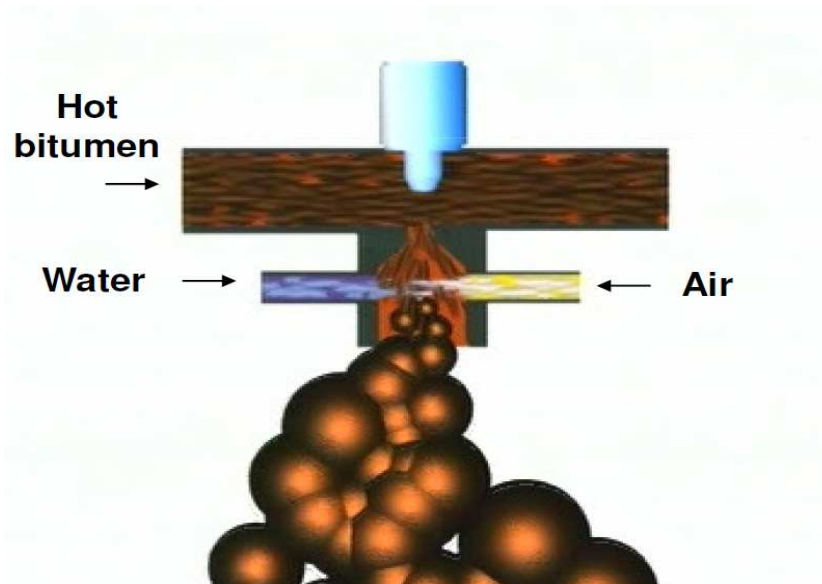
Figure 2. 4: Schematic nozzle for bitumen production (Wirtgen 2012)

2.2.2.4 Why foaming?

Liquid bitumen binder at high temperature without foaming would immediately become globules in contact with cold aggregates, and thus cannot be consistently dispersed. On the other hand, foamed bitumen, or bitumen bubbles, can be dispersed into the mix uniformly.

The purpose of foaming bitumen is to make it easier for bitumen to disperse into cold granular materials at ambient temperature. At its foamed state, bitumen has a very large surface area and a low viscosity and therefore can be easily mixed with aggregate particles. However, the foam dissipates in less than a minute and the bitumen resumes to its original properties. Therefore, in order to produce BSM-foam, the bitumen has to be incorporated into the aggregates while still in its foamed state.

When mixed with aggregate, the bitumen bubbles burst and produce tiny bitumen particles contained in the expended bitumen are attracted and coated with the fine particles of the aggregate. The contact creates bubble bursts and the coating of the finer particles (less than 0.075 mm). This process brings into being mastic that effectively binds the mixture together to form a strong stabilised pavement material. This mixture of foamed bitumen and aggregate is called BSM-foam. The expansion is very important during the mixing, for the greater the volume of the foam, the better the distribution of the bitumen in the aggregate. Also, the moisture in the mix prior to adhesion of the foamed bitumen plays an important role in dispersing the bitumen during the mixing (TG2, 2009).

2.2.2.5 Characterisation of foamed bitumen

Studies on foamed bitumen characteristics have shown that the quality of foamed bitumen is assessed through the two parameters. They are essential when considering the suitability of a particular foamed bitumen compared to another one before it is used to stabilize mineral aggregates.

- **The expansion ratio (ERm):** it is the maximum-foamed volume during the foaming process, divided by the volume of the bitumen after the foam has completely dissipated. It measures the viscosity of the foam and gives an indication of how well the binder will disperse in the mix.; and
- **The half-life time ($\zeta_{1/2}$):** it is the time in seconds, which the foam takes to collapse to half of its maximum volume, after expansion. The "half-life" quantifies the stability of the produced foam, as well as its rate of collapse during the mixing.

The two parameters need to be optimised for a particular bitumen type, in order to produce BSM-foam of good quality. This is achieved by measuring the half-life and the expansion ratio of the produced foamed bitumen, using various percentages of water. Usually five tests are conducted with a foaming water content varying from 1% to 3% at 0,5% increments. The expansion ratio and the half-life are inversely related. Increasing the foaming temperature has an effect on increasing the expansion ratio and decreasing the half-life. A higher expansion ratio has a larger surface area per unit area, a relatively low viscosity and it therefore gives foamed bitumen the ability to coat more and finer aggregates. On the other hand, the longer the "half-life", the more stable the foam is, and consequently has more effective time to interact with aggregate, resulting in better coating of the particles.

The two parameters are inter-dependent and their optimization is not simple. Because both parameters depend on the foaming water application rate, it is possible to determine the appropriate moisture content that optimizes both of them (Moftresh, 2004). However, this selection process is dependent on judgement of the trade-off between the expansion ratio and the half-life (Jenkins et al, 2000). In some cases, it could be difficult to determine the optimum expansion ratio and the half-life because there is a lack of numerical evaluation criteria. However, the following values are recommended by some BSM's literatures:

- ERm $\geq$ 10 and $\zeta_{1/2} \geq$ 12 seconds (CSIR, 1998)
- ERm $\geq$ 10 and $\zeta_{1/2} \geq$ 8 seconds (Wirtgen, 2012)

Moreover, Jenkins (2000) showed that expansion ratio and half-life only are not enough to effectively characterize and optimise foamed bitumen. Jenkins developed the foam index (FI) as a more useful measurement of asphalt foaming characteristics (Jenkins et al, 2000). More than expansion ratio and half-life, the foam index takes into account other factors such as binder type and temperature. This is calculated by measuring the area under the decay curve. In other words, instead of using just the two points that define the decay of the foam (ER and $T_{1/2}$), the foam index is calculated using the entire curve of foam expansion versus time, as measured in the bucket in the laboratory for example. Moreover, Jenkins' research on variety of bitumen has shown that the decay of foam can be successfully modelled by adapting equation for radioactive decay (Jenkins 2000). He then developed the foamed bitumen decay equation below.

$$ER\ (t) = ER_m\ exp\left[\frac{-\ln 2}{\tau_{1/2}} - t\right]$$ Equation 2.1

Where, $ER(t)$ = $expansion\ ratio\ with\ respect\ to\ time\ after\ foaming\ discharge,$

ER_m = $measured\ maximum\ expansion\ ration\ (immediatly\ after\ discharge,$

$\tau_{1/2}$ = $half\ live\ (seconds)$

t = $time\ measured\ from\ the\ moment\ all\ foam\ is\ discharged\ (seconds)$

2.2.3 Main Characteristics of Bitumen Stabilised Materials

Bitumen stabilised materials differ from other pavement materials. Their characteristics and behaviour vary significantly with the quantity of bitumen and active filler in the mix, the temperature, the moisture condition and type and quality of the parent material.

2.2.3.1 *Non-continuously bound materials*

Studies have shown that one particularity of BSMs is that they are non-continuously bound materials, unlike hot mix asphalt and cement-treated materials. This is due to the nature of bitumen dispersion in the aggregate which lead to the formation of bitumen droplets that are not joined together. This property gives BSMs mechanical characteristics and failure mechanisms that fall between those of unbound granular materials and bound material (i.e. Hot Mix Asphalt, Cemented material etc.). Stresses applied to continuously bound material result in bending, which lead to fatigue cracking. This is the reason why structural design procedures anticipate fatigue cracks at the bottom layers of those materials. Unlike continuous bound materials, stresses applied to non-continuously bound materials are better distributed, therefore minimizing the occurrence of permanent deformation, which is the main mode of failure of BSMs (See Figure 2.5).
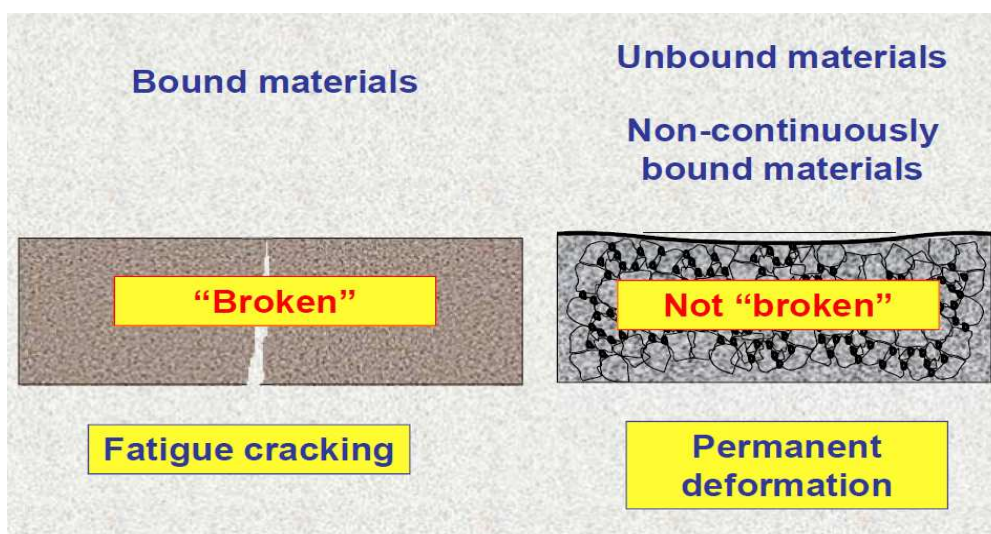
Figure 2. 5: Pavement condition at failure (Collings, 2012)

2.2.3.2   *Improved strength and stiffness*

The primary driving purpose behind the history of stabilisation is improvement of the strength and stiffness of the treated material. Studies have shown that there is a significant increase in the shearing properties of a material after it has been stabilized with bitumen (see Figure 2.6). The cohesion increases significantly, enabling the layer made of bitumen stabilised material to withstand high stress from heavy traffic load. On the other hand, the internal angle of friction of a material slightly decreases, causing the material to retain its inherent stability. The monotonic triaxial test conducted on granular material represented on Figure 2.6 gave a cohesion of 0.082 MPa and a friction angle of 53^0. After stabilising the material with foam bitumen, the new cohesion and friction angle obtained were 0.166 MPa and 44.7^0 respectively.
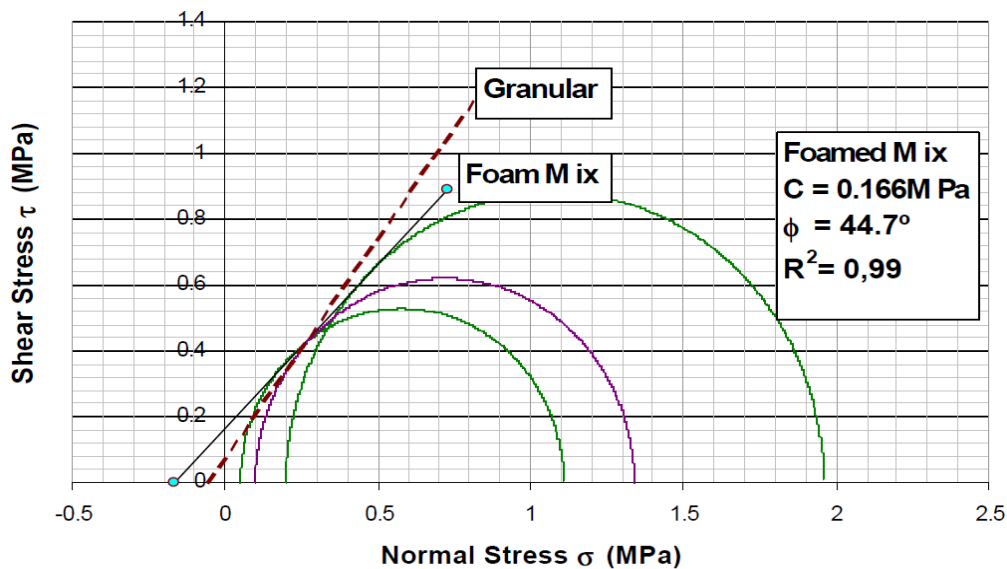
Figure 2. 6: **Mohr-Coulomb Circles for Foamed Mix with Failure Envelope for Granular Superposed of C=0.082 MPa and φ=53^0 (Jenkins, 2000)**

Moreover, studies have shown that, high cohesive strength of BSMs allows them to develop strength gain and to sustain a higher stiffness under traffic loads compared to unbound materials. This is due to curing (reduction in moisture) and cementation effect of active fillers. However, the stiffness of BSMs is function of the parent material stiffness; the density achieved during compaction, the moisture conditions, as well as the percentage of bitumen and active fillers added (*Asphalt Academy, 2002 & TG 2, 2009*).

2.2.3.3   *Improved durability (by a reduced moisture sensitivity)*

Durability is not an isolated property but has multiple parameters. However, scholars have mostly referred to durability of BSMs as the ability of the BSMs layer to resist moisture damages. The bitumen contained in BSMs and the surfacing layer placed on top of the BSMs layer play a significant role on preventing water infiltration.

Twagira (2010) carried out an investigation on the mechanisms that influence durability behaviour of bitumen stabilized materials in terms of moisture damage. This was achieved by simulating field moisture conditions through the moisture induction simulation test (MIST). It was found that the BSMs process improves durability due to the addition of active fillers. Cement or lime influence the cohesion and the adhesion of the binder even under adverse moisture conditions. Although reducing the moisture sensitivity of BSMs will significantly improve their durability, the durability of the mix components (i.e.: parent aggregate and bitumen used) have been found by scholars to plays a non-negligible role. (Jenkins 2000 and Paige-Green 2004)

2.2.4   Advantages and disadvantages of Bitumen Stabilised Materials

2.2.4.1   *Advantages identified*

The following advantages of bitumen stabilisation have been identified in previous research:
- Bitumen stabilisation increases the shear strength and reduces the moisture susceptibility of granular materials;
- The strength characteristics of BSMs is similar to those of cemented materials. However, BSMs are flexible and fatigue resistant;
- Bitumen stabilisation can be used with a wider range of aggregate types than other cold mix processes;
- Layers made of BSMs have a rapid strength gain and can therefore be opened to traffic almost immediately after compaction is completed;
- BSMs is a greener technology compared to other modes of stabilisation;
- BSMs strength can be improved significantly just by the addition of a small percentage of active fillers, especially cement;
- Bitumen stabilisation is a quick construction method and has a lower cost than reconstruction;
- Foamed bitumen stabilisation improves durability and material resistance to moisture infiltration.

2.2.4.2   *Disadvantages identified*

- **Grading:** based on the literature survey, the success of bitumen stabilization is very sensitive to the grading of the material to be treated. The percentage of passing the 0.075 mm sieve is an important requirement, especially with foam stabilization.
- **Skill requirement:** Skill and experience are essential in the mix design and the production of foamed bitumen.

2.3 Definition of flexibility and test evaluation methods

To gain better comprehension of the term 'flexibility', the understanding of material properties and load response (i.e. brittleness and ductility) relative to some fundamental material behaviour (i.e. elasticity, viscosity and plasticity) remains crucial.

2.3.1   Material Models: Elasticity, Plasticity and Viscosity

Elasticity

Elasticity is the property that allows a material to deform in direct response to a load and to recover its original shape when the load is removed. Within the elastic domain (from point 1 to point 2 on Figure 2.7), the strain response is proportional to stress applied to the material in small deformations. Most material can behave elastically up to a limit called elastic limit or yield limit (point 2 on Figure 2.7). Past that limit, the material will end up with permanent or plastic deformations if loaded. Few materials behave 100% elastically up to the failure point i.e. glass. Pure elastic materials can be characterised by a spring obeying the Hooke's Law:

$$\sigma = E\varepsilon \qquad\qquad\qquad \text{Equation 2.2}$$

Where: σ = stress

ε = strain

E = Modulus of elasticity

Plasticity

Plasticity in the other hand is the property that enables a material to undergo some extends of permanent deformation without failure. In other words, plasticity is the ability that a material has to deform when loaded and retain that deformation when unloaded (Ebels, 2008). Plastic deformations are not time dependent. They occur when the stress applied is sufficient to deform the material permanently. When material starts to behave plastically, a large amount a deformation can be

produced without a significant increase in stress. In plastic deformations, the strain can be separated into a recoverable elastic strain (e) and an inelastic strain (p), as shown in Figure 2.7.
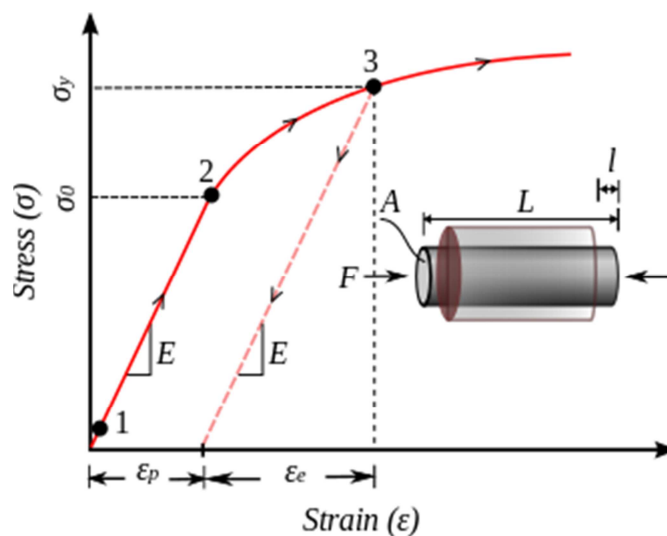
Figure 2. 7: Stress-strain curve showing typical elastic and plastic behaviour of a material in compression (Flow plastic theory, Wikipedia 2013)

When subjected to compressive testing, most materials used in road construction and rehabilitation (HMA, BSMs and compacted granular materials) display a stress-strain curve similar to the one of Figure 2.7.

Viscosity

Viscosity is more related to rheology, which is the science of deformation and flow characteristics of materials. Viscosity is more often defined as the measure of a fluid's resistance to flow. In other words, a fluid with a high viscosity resists motion while a fluid with low viscosity flows easily. Viscosity and plasticity have some similarities. However, some differences between the two properties do exist. Viscous deformations differ from plastic deformations by the way that plastic deformations only happen after the elastic limit is exceed while viscous deformations are not proportional to the applied stress because they are time and temperature dependent. Pure viscous materials can be characterised by a dashpot obeying the Newton's Law:

$$\overline{}$$

Where: = viscosity

t = time

= strain

Some materials such as bitumen exhibit the behaviour of semi-solid materials with high dependency on time and temperature. So when mixed with granular materials, the obtained material can exhibit both viscous and plastic behaviour. Such materials are called visco-elastic materials. That is the case with BSMs. Several mechanical models were developed to describe visco-elastic behaviour. Some are:

- Burgers Model
- Kelvin-Voigt Model
- Maxwell Model

In two the models the spring and the dashpot are combined in series (Burgers Model) or in parallel (Kelvin-Voigt Model), while in the Maxwell Model, the first two models are associated in series (see Figure 2.8).
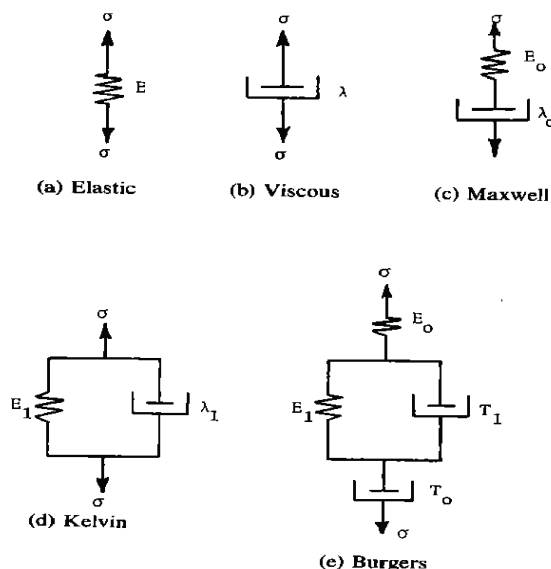
Figure 2. 8: Mechanical models for visco-elatic materials (Huang, 1993).

2.3.2   Brittleness and Ductility

One material characteristic of interest to this study that can be derived from the stress-strain diagram is the brittle vs. ductile behaviour, which can also be linked to the elastic vs. plastic behaviour.

Brittleness is a material property that allows bending or deformation without shattering. The brittleness property enables a material to break without considerable permanent distortion. It can also be defined as the absence of ductility and malleability. Brittle materials exhibit sudden cracking upon loading; such material types possess a low load response criterion. There are many materials that break or fail before much deformation take place: such materials are classified as brittle materials.

Ductility on the other hand is the property that enables a material to be stretched without breaking and to retain the changed shape after the stretching effort has been removed. In other words, ductile materials are able to undergo great deformation or elongation. Ductility gives an indication of the extent to which a sample of the material can be stretched before breaking. Ductility is also defined as the distance in centimetre to which a sample of material will be elongated without rupture. It can therefore be influenced by the size of the sample, the testing temperature, the rate of pulling etc. Figure 2.6 shows an example of ductility measurement of a sample of bitumen based on the elongation distance without breaking.
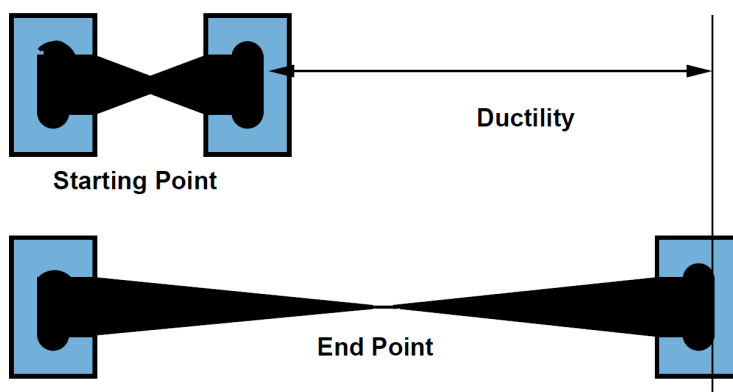
Figure 2. 9: Ductility Moulds (Pavement Materials, NTPEL, May 2006)

The boundary between brittle and ductile behaviour is not clearly identified. Most brittle materials exhibit linear elastic behaviour and very little plastic deformation before failure. Ductile materials on the other hand can undergo extensive plastic deformation before failure occurs. They deform more plastically under tensile stress.. Brittleness has a significant influence on the performance of road construction materials. In the past brittleness was defined qualitatively, but now a definition of brittleness for visco-elastic materials exists, enabling analysis of all types of polymer-based materials (J Mater, 2010). Brittle materials are known to possess low tensile strength and it may thus crack under excessive repeated flexure.

Brittleness is a parameter to be controlled with attention in bituminous mixes for it could lead to shrinkage, traffic associated cracks and deterioration of the BSMs layer. As said earlier it is believed that BSMs could become brittle as a result of aging or/and environmental conditions. Moreover, the amount of active fillers added to BSMs mixture can also have a significant input on developing brittleness property.

Brittle failures happen within the elastic domain of loading. In other words, brittle materials do not reach plastic deformation. In brittle failures, cracks may spread very fast with slight deformation which a typical load response of brittle materials. Bituminous materials can experience brittle failures. This happens when the thermal stress exceeds the tensile stress at low temperature, because of thermal shrinkage of the bitumen under freezing conditions (LU, Issacsson, and Ekblad, 2003). At low temperature, stiff bituminous mixtures exhibit brittle failure. Therefore, it is required for the binder to have a high ability of stress relaxation (good flexibility) at low temperature.

Unlike brittle materials, ductile materials are prone to exhibit plastic deformation when loaded. Their failure does not happen instantly, but rather are the result of a process that proceeds relatively slowly from the impact stress to the fracture. More often, plastic deformation occurs when the maximum stress exceeds the yield strength leading to more and uniform distribution of stress in the vicinity and stress raiser.

In pavement construction and rehabilitation where bitumen is extensively used, bitumen most be selected with care because the properties of the bituminous mixes used are strongly influenced by the characteristics of the binder. Mixing a binder that does not possess enough ductility with aggregate will result in a material not flexible enough and more susceptible to cracks when subjected to stresses. Therefore, the study of bitumen rheology is essential since bitumen largely reflects the performance of a bituminous mixes. Moreover, before a particular bitumen is selected for a mix, it is important that it meets the requirements of ductility and penetration index.

2.3.3 Flexibility

Having defined ductile and brittle materials, and having explained their failure mechanisms, it could now give us insight to a better understanding of what flexibility is. Flexibility is the material property that enables a particular material to return to its original shape after the force that has caused its temporally bending is removed. According to this definition, flexibility and elasticity could be mistaken for one to another; however, they are two distinct concepts. Elasticity is the ability to stretch and return to the original shape and is a material model; whilst flexibility is the ability to bend and it is a material property. In comparison with brittle and ductile materials, it could also be said of flexible materials that they are halfway-located in-between brittle and ductile materials. In other words, flexibility is a combination of brittleness and ductility. Figure 2.7 shows failure responses of brittle, flexible, and ductile materials.
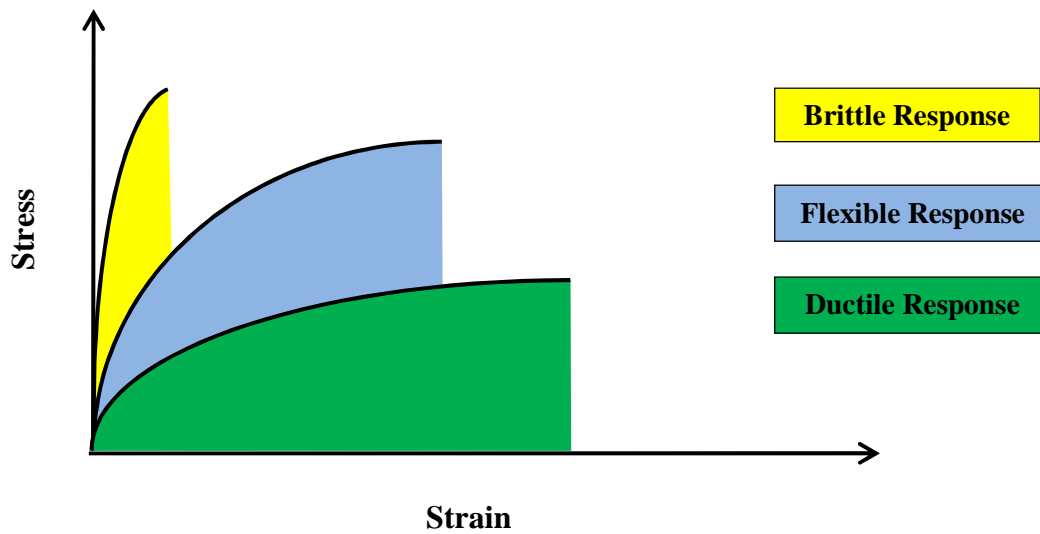
Figure 2. 10: Summary of the three properties

From an engineering point of view, flexibility has to do with the ability of the material to bend without breaking (cracking) when loaded. More often, the bending beam concept (defection) is used to illustrate the property. Consider a small element of a beam simply supported at its edges and loaded transversally to the long dimension (Figure 2.8), this could help to illustrate and understand the internal bending stresses within the beam which are also known as the flexural or bending stresses.
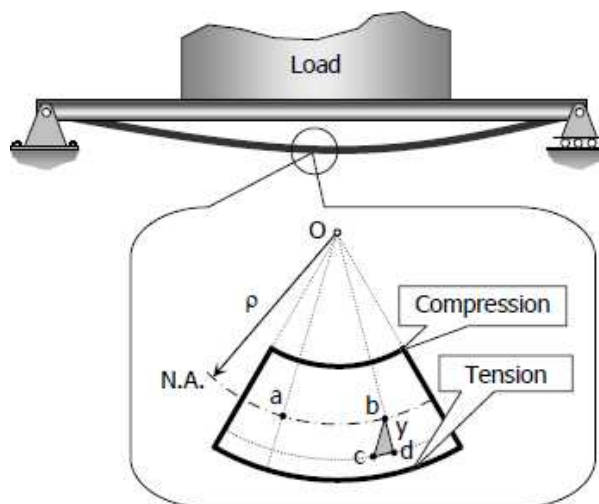
Figure 2. 11: Illustration of a beam flexure.

Considering a fibre located at a distance y from the neutral axis (NA) of the beam, it will stretch by an amount of cd as a result of bending. *bcd* and *Oba* can be assimilated to small triangles because of the curvature of the curve which is very small. The strain in this fibre can be expressed as:

$$\varepsilon = \frac{cd}{ab} = \frac{y}{\rho}$$
Eqation (2.4)

Where ρ = radius of the curvature of the beam and y the displacement from the neutral axis.

By using Hooke's law: $\varepsilon = \sigma/E$, where $\boldsymbol{\sigma}$ = stress and E= modulus of elasticity, we have:

$$\sigma = \frac{y}{\rho} E$$
Equation (2.5)

This equation shows that the stress is proportional to the distance *y* from the neutral axis.

In the following section, the notation f_b will be used instead of $\boldsymbol{\sigma}$

Considering now the differential area *dA* at a distance *y* from the neutral axis, the force acting above the area is given by:

$$dF = f_b \, dA = \frac{y}{\rho} E \, dA = \frac{E}{\rho} y \, dA$$
Equation (2.5)

Because the results of all the elemental moment about the neutral axis should be equal to bending moment on the section, we therefore have:

$$M = \int dM = \int y \, dF = \int y \left(\frac{E}{\rho} y \, dA\right)$$

$$M = \frac{E}{\rho} \int y^2 \, dA$$
Equation (2.7)

Since $\int y^2 dA = I$ = centroidal moment of inertia, then:

$$M = \frac{EI}{\rho} \ or \ \rho = \frac{EI}{M}$$

Substituting $\rho = Ey/f_b$

$$\frac{Ey}{f_b} = \frac{EI}{M} \leftrightarrow \boldsymbol{f_b} = \frac{\boldsymbol{My}}{\boldsymbol{I}} \ and \ (\boldsymbol{f_b})_{\boldsymbol{max}} = \frac{\boldsymbol{Mc}}{\boldsymbol{I}}$$
Equation (2.8)

Finally, the maximum bending stress due to beams curvature is given as:

$$f_b = \frac{Mc}{I} = \frac{\frac{EI}{\rho}c}{I} \leftrightarrow \boldsymbol{f_b} = \frac{\boldsymbol{Ec}}{\boldsymbol{\rho}} \qquad \text{Equation (2.9)}$$

And the beam curvature is:

$$\boldsymbol{k} = \frac{\boldsymbol{1}}{\boldsymbol{\rho}} \qquad \text{Equation (2.10)}$$

In pavement engineering, flexibility is a measure of the level of bending strength required by the layer system in order to withstand traffic load and avoid cracking. Flexile pavements usually consist of the top to the bottom of a bituminous surfacing layer, followed by a bituminous or granular base and a granular or cemented subbase. Bituminous materials are used in flexible pavement construction is to give more flexibility to the layer system. The absence of flexibility within the pavement layer has as primary consequence the development of fatigue and thermal cracks under the effects of repeated loading stress from the vehicles. This failure mechanism is one of the most important aspects that affect the design life and functional capacity of flexible pavements, for it decreases their structural effectiveness. Flexibility therefore happens to be a key parameter in the performance of roads construction materials. The flexibility property in pavement is improved by using bituminous materials i.e. hot mix asphalt or chip seals, for surfacing layers and bitumen stabilized materials treated by the means of bitumen emulsion or foamed bitumen for base layers.

Bitumen behaves as visco-elastic materials. Depending on the temperature and the loading time, bitumen has either the ability to behave somewhere between an elastic and a viscous liquid. At low temperatures, bitumen becomes hard and exhibit behaviour close to that of brittle materials. It can therefore crack easily with load application. The bending beam rheometer (BBR) test (AASHTO: T313-02) is commonly used to assess the creep response of bitumen at low temperature. It is a simple test whereby a load is applied to a bitumen beam in simple bending as shown in Figure 2.12. The creep stiffness obtained are plotted against the loading time.
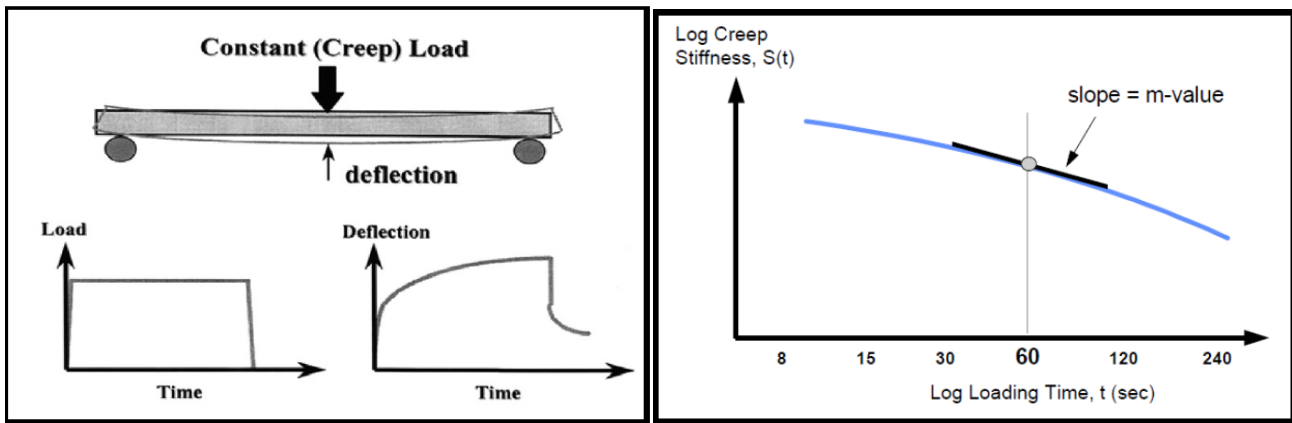
Figure 2. 12: Bending beam rheometer (left) and Determination of S(60) and m-value (right) (Rowe et al., 2001)

The two important parameters given by the test are S(t), which is the stiffness at any time t (usually t=60 is used for standard) and the m-value which is the slope of the curve as shown in Figure 2.12. As the loading time increases, the stiffness reduces. The same observation can be made with temperature. As the temperature increases, the stiffness will decrease. Therefore if the m-value is close to zero, it means the bitumen tested does not behave visco-elasticity. On the other hand, a high m-value will indicate a strong dependency to temperature and loading time of the treated bitumen.

2.3.4 Strength and flexibility

Strength enables a material has to withstand action of external forces without breaking. Strength and flexibility are two important properties required for road construction materials. However, it is important to have a perfect balance between the two properties within a material before it is used to construct the pavement layer. If the material is too strong, meaning very rigid, the layer will suffer from cracking under the stresses from the traffic loads. On the other hand, if the material is too flexible and have less strength, the layer could suffer rutting under the repeated impacts of the traffic loads. Therefore, the need of having a perfect balance between the two properties is very crucial for the performance of the constructed layer.

In the case of BSMs the Strength-Flexibility dependency of the mixes is governed by the cement/bitumen ratio (F. Long). Works in this regard were conducted by the CSIR on several BSMs mixes made at different percentages of residual bitumen content and cement. Based on Strain-at-break and Unconfined Compressive Strength, it was found that flexibility increases with the increasing the bitumen content while the strength of the mix increases with the increase of cement content as active filler. Both parameters are vital for the performance of the produced mix. Increase in cement content lend to an increase in stiffness. However, it should be noted that the increase in

mix stiffness is compromised significantly by the reduction in flexibility of the material. Hence, having a good balance between the cement and the bitumen content in the mix is indispensable to ensure the optimal strength and flexibility that will satisfy the mix design requirements. Figure 2.9 illustrates the interdependency of strength and flexibility in terms of cement/bitumen ratio.

Cement/Binder Ratio
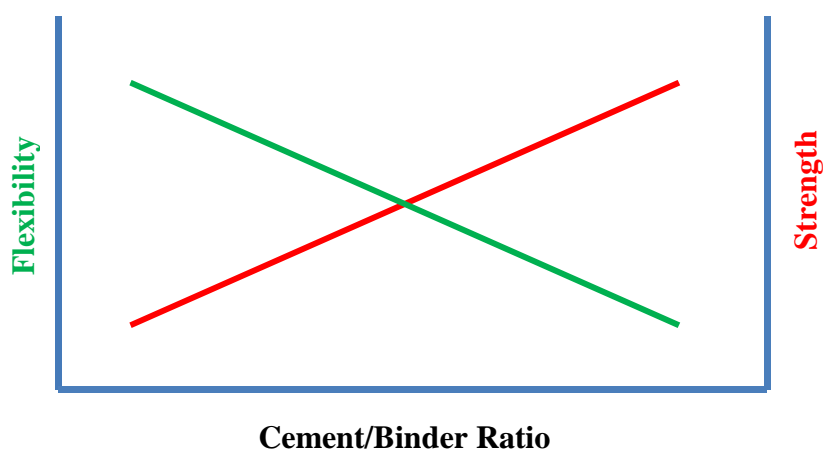
Figure 2. 13: Effect of Cement/bitumen Ratio on Strength and Flexibility for BSMs (Long, F. 2004)

2.3.5   Test evaluation methods of measuring flexibility

There is a wide range of laboratory test methods which could be used to evaluate the flexibility of a particular mix. Most of them are common both to flexibility and fatigue resistance (ability to resist cracking and fracture under repeated bending) because of similarities of the two mix properties.

- Simple flexure test;
- The bending test;
- Four point beam test (monotonic and dynamic);
- Supported flexure test;
- Fracture mechanics test; and
- Wheel track testing, to mention a few.

The flexibility of bitumen stabilised materials is not easy to measure. A wide range of tests have been carried out by researchers looking for the best way of measuring the flexibility of bitumen stabilized materials. Hereafter are some test evaluation methods of BSM's flexibility as well as their advantages and limitations.

2.3.5.1   Type of Test and Description

a)   The three-point beam flexural test

The three points bending flexural test provides values for the module of elasticity in bending, the flexural stress, flexural strain and the flexural stress-strain response of the material. This last parameter given by the test (i.e. flexural stress-strain response) is the point of focus for evaluation of flexibility of BMS's, where a particular attention is given to the strain at break. The main advantage of this test is the simulation of the in service flexure behaviour. However, this test also has some disadvantages such as the sensitivity of the results to the specimen, the loading geometry and the strain rate.

b)   Four Point Beam (4PB) or Three Points Beam (3PB) fatigue test

The engineering properties such as strain-at-break, flexural stiffness and fatigue behaviour can be obtained through the two tests, giving hence an indication of the performance properties such as flexibility and fatigue resistance.

c)   Strain-at-break from the four-point beam static test

The strain-at-break test is a four-point monotonically loaded flexural beam test that measures the flexibility and tensile strength of treated materials (Otte, 1972). From the literature, it can be seen that it is a well-known test when it comes to the characterisation of road construction materials. However, the flexural beam strain-at-break was found not to be a good parameter to normalize fatigue test results and the strain-at-break test would require further development before it could be considered in the characterization of flexibility properties in terms of fatigue resistance of BSMs.

Long F. & Theyse H. 2004, noticed that the determination of the strain-at-break through the monotonic four point beam test in the laboratory and its use in practice could be relatively easy. However, Twagira et al. (2006), followed by Ebels, L. & Jenkins, K.J. 2007, went in the contrary direction by recommending the flexural beam strain-at-break property not to be used as parameter for the design of bitumen stabilised materials.

Twagira *et al*. (2006) discovered an inconsistency in the monotonic strain-at-break results and from there emphasized on the crucial need of preparing BSM beam for monotonic strain-at-break test with great precautions. Ebels *et al*. (2009) also mentioned that the variability of the strain-at-break using the four-point beam was generally high. Implementing very good and strict specimen preparation protocols and good quality of the beams to be tested was therefore recommended. Unless this is done

and followed, the results of the strain-at-break from the four-point beam test may become untrustworthy.

d) The Indirect Tensile Strength (ITS) and the Unconfined Compressive Strength (UCS)

For years, Indirect Tensile Test and Unconfined Compressive Strength have been used to evaluate engineering properties, mechanical properties and the failure mechanisms of road construction materials. Performance properties such as resistance to cracks formation, crushing and disintegration can also be assessed through ITS and UCS tests respectively. Both tests are known to be quick, relatively simple and standardized. In South Africa, the TG2 Guideline used for BSM's classification is largely based on ITS and UCS tests. In this regard, the works of *Houston et al.* with regards to the correlation between different ITS and UCS test protocols for BSM foamed bitumen stabilized materials classification are also acknowledged.

The tensile strength of bitumen stabilized materials is an important property because the pavement layer made of BSMs will crack when the tensile stress at the bottom of the layer is exceeded. However, researchers have questioned the suitability of ITS and UCS tests when evaluating the flexibility of BSMs. This interrogation was emphasized with the works of *Bondietti et al., (2004)*. However, as far us flexibility is concerned there could be a potential link between the tensile and the compressive strength; for as the tensile strength increase, the compressive strength may increase as well. Therefore, in flexibility and flexure evaluation, the indirect tensile strength and the Unconfined Compressive Strength can give some indications. Furthermore, one of the material characteristics which can be derived from the stress-strain diagram of the ITS test is the ductile vs. brittle behaviour of the material, which can be then related to the flexibility as shown in section 2.4 of this chapter.

2.3.5.2 Can "Strain-at-break" or fracture "energy" be used to assess flexibility?

a) The strain-at-break

Flexibility is an important property of road construction material, however not easy to assess. As said in chapter one, one of the aims of this study is to attempt to evaluate the flexibility of BSM, using strain at break and fracture energy theory from the ITS, UCS, and Monotonic triaxial tests. Correlations of monotonic triaxial with dynamics triaxial tests (i.e. Resilient Modulus) responses are also to be investigated in that regard.

The strain is the characterisation of how much a material has been stretched (or compressed, or bent) when compared to its original length. Typical material characteristics of interest that can be derived from strain at break diagram plot include the following:

- Elastic Modulus , also known as Young Modulus,
- Yield and Ultimate Strength,
- Elastic vs. Plastic behaviour,
- Ductile vs. Brittle behaviour.

Early works on strain-at-break as a material properties were first conducted in South Africa in the 1970's by Freeme. Research by Freeme and Maree, 1981; indicated that crack initiation under traffic loading within layers made of cement treated materials is believed to be directly related to the strain-at-break property of these materials. From 1972 to 1978, Otte also dedicated some work to the link between strain break and the effective fatigue life. From the literature, the current method adopted in the South African Mechanistic Design Methods for calculating the fatigue life of cement treated material layers was proposed by Otte (Walket et al,1977; Otte, 1978; Freeme et al,1982). The calculations are based on the tensile strain ratio and the strain at break as shown in the following equation:

$$N_f = 10^{9.1\left(1 - \frac{\varepsilon_s}{\varepsilon_b}\right)}$$ ……………………………………………………………… Equation (2.11)

Where:

$N_f = Number\ of\ loads\ repetition\ at\ strain\ \varepsilon_s\ to\ cracks\ initiation$

$\varepsilon_s = Applied\ strain$

$\varepsilon_b = Strain\ at\ break$

For bituminous materials, break takes place under conditions of large stress which normally occurs at low temperature (Shell bitumen handbook, 2003). The strain-at-break is the parameter that measures how much the material is stretched at the time of breaking when compared to its original length. Since the tensile properties of bitumen are dependent on both temperature and loading time, the strain-at-break is highly connected to temperature and therefore to the stiffness of the material. At low temperature, the stiffness of the bituminous material will be high and the strain-at-break low. On the other hand, although the force required to break the specimen will reduce due to the low stiffness of the material at high temperature, the strain-at-break will be high.
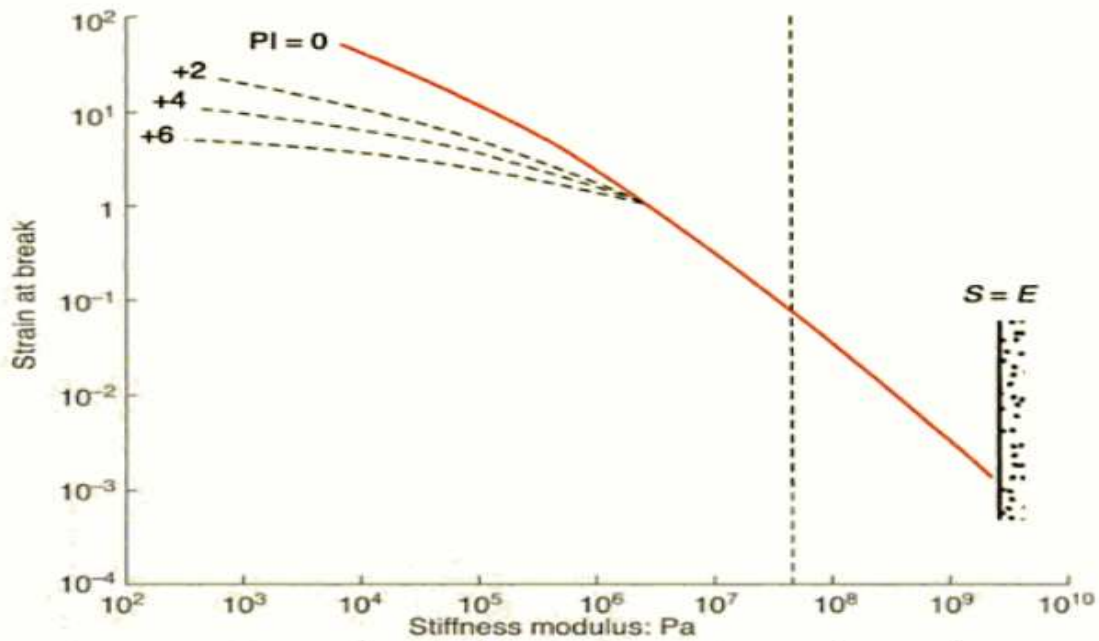
Figure 2. 14: Stain at break as function of stiffness (Shell bitumen handbook, 2003)

In their report on Mechanistic-Empirical Structural Design Model for emulsified bitumen treated materials, Long F. and Theyse, H. 2004, recommended that emulsified bitumen treated layers should be analysed in two phases: the first being the stiffness reduction phase and the second the permanent deformation phase. During the first phase live analysis, they established that: The stiffness reduction's transfer function is dependent on the strain ratio (which is the ratio between the tensile strain at the bottom of the pavement layer and the strain-at-break from the flexural beam test). The calculated strain ratio could give an indication of the measurement of the flexural bending of the material, which is the equivalent of the maximum flexibility obtained with the laboratory test (Long, F. and Theyse, H. 2004). The results show that low strains result in material that is more flexible.

According to literature, the life's phase of stiffness reduction of emulsion treated materials is a function of the strain ratio (ratio of the strain at the bottom of the layer and the strain-at-break from the flexural beam test). The behaviour of emulsion treated material can be divided into two phases (Ebels, 2008). The first phase is the fatigue life behaviour, whereby the emulsion treated material behaves like cement treated material. During this phase, emulsion treated materials develop a high resistance to permanent deformation and a high elastic modulus. On the other hand, the second phase behaviour can be related to the one of granular materials. The percentage of cement content used in the mix as fillers or active fillers plays a major role in this two-phase behaviour of cement treated material. The less the cement content, the more the material will behave as granular material and the more the cement content, the more the material will behave as cement treated material.

In the case of foamed bitumen treated materials, the TG2 (2002) of the asphalt academy which was latter on superposed and replaced by the TG2 (2009) used to describe the life phase of foamed bitumen treated material to be very similar to the one of cement treated materials implemented in the South African Mechanistic Design Method. For this reason, the strain at break was adopted as the parameter controlling the first phase, which is the effective fatigue life of foamed bitumen stabilised materials. Similar to cement treated materials, an equation of fatigue life as a function of strain at break was also developed and typical values of strain at break recommended for different category of foamed bitumen treated materials.

$$N_f = 10^{9.1\left(A - 0.708\frac{\varepsilon_s}{\varepsilon_b}\right)}$$ ……………………………………………….………. Equation (2.12)

Where:

$N_f = Number\ of\ loads\ repetition\ at\ strain\ \varepsilon_s\ to\ cracks\ initiation$

$\varepsilon_s = Applied\ strain$

$\varepsilon_b = Strain\ at\ break$

Layers made of BSM are designed and constructed to carry loading from traffic. Three types of loading could be acting within the layer under the tyre pressure i.e. Axial (Compressive and tensile), shear and bending. Therefore, one can distinguish:

Tensile strain at break: Tensile strain is one of the most important factors influencing the crack initiation and propagation in the layer under the action of the service loads. Tensile strain at break corresponds to the point of rupture.

Compressive or normal strain at break: it could be defined as the ratio of compressive deformation per unit length along the longitudinal axis. Unlike tensile forces that make the specimen longer, compressive forces make the specimen shorter.

Flexural strain at break: it can be measured from the three points or four points bending test, which measures the force required to bend a beam in the form of load-deflection response. During the test, bending moments occur in the beam when subjected to loads that are transversal to its longitudinal axe. Thus the top half of the beam is subjected to compression while its bottom part (under the neutral axis) is subjected to tension. The plane section of the beam at the neutral axis is assumed to remain plan during the test. The flexural modulus from these tests serve as an indicator to the stiffness of the beam when flexed (ASTM D790, ISO 178).

b) *The fracture energy theory/ dissipated energy*

According the first law of thermodynamics, the energy of an isolated system can be changed from one form to another or transferred into various types of energy. However, the entire energy always stays content, it can neither be created nor destroyed. Therefore, the amount of mechanical energy (load) applied to a material is generally transform into: internal energy (elastically stored energy), kinetic energy (result of material velocity) and dissipated energy (result of friction and plastic deformation), (P.J.G Schreurs, 2012). The latter is our point of interests as far as the deformation of material under load application is concerned.

Fracture energy is defined as the energy consumed in producing a crack within the material. The theory is used to predict the effect of cracks initiation and growth in a material based on the energy associated with the damage. For years, it has been used as a mix parameter to describe and model the fracture resistance of asphalt mixes for road construction; especially in the case mixes with ductile binder such as polymer-modified asphalt (ASTM D7313, 2013).

For asphalt mixes, it is believe that fracture energy of a material can be highly correlated to fatigue life (Saleh, M. 2006). Moreover, the failure of visco-elastic material is an energy dissipation process (Yuan, Zhan and Chen, 2013). In the case of HMA for instance, energy is dissipated during load and relaxation because of the visco-elastic behaviour of the material at ambient temperature (Rowe, M. 1996). As demonstrated in Figure2.15, the energy is represented by the area under the load-displacement curve, and the energy release by the material during the loading is the same as is recovered during the unloading phase (case of dynamic loading).
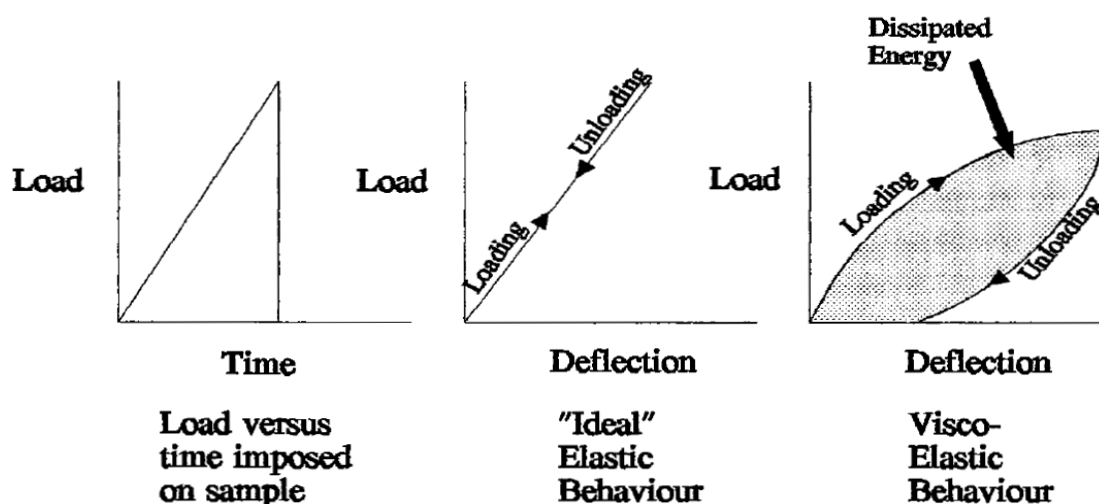
Figure 2. 15: Linear elastic versus visco-elastic behaviour (M. Rowe, 1996)

As shown in Figure 2.15 (case of visco-elastic behaviour), the dissipated energy produced during one loading cycle is determined by the area of the stress-strain hysteresis loop. In the case of monotonic loading, whereby the specimen is loaded until it fails, the energy stored in the material will be release when the specimen breaks under the application of a load. In this case, the dissipated energy is determined by the area under the load-displacement curve before the specimen fails. The amount of energy scored by the material before breaking is proportional to the amount of deformation that the material can undergo before it breaks. For brittle material, this amount of energy will be small compared to that of ductile materials

After comparing the fracture energy of foam and asphalt mixes by computing the area under the load-displacement curve from ITS test, Saleh found that the fracture energy of asphalt mixes was higher than that of foam mixes as shown in Figure 2.16. Meaning that the asphalt mixes show more flexibility than the foam mixes since the amount of deformation before failure is higher HMA than that of BSM-foam. This therefore indicates that the fatigue life of BSM-foam is expected to be less than that of dense grade HMA.
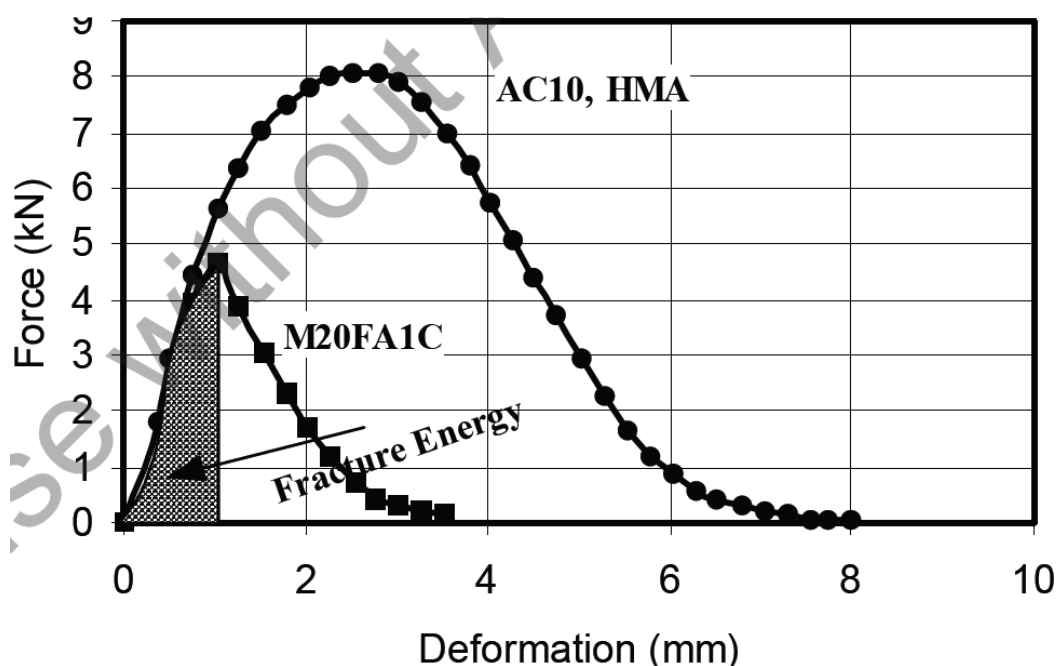
Figure 2. 16: Comparison between fracture energy of BS-foam and dense grade HMA, (Saleh, 2004)

Fracture energy is a fundamental material property and like any other material properties, it is determined through experiments. Load-displacement controlled tests are used in that regard i.e.

- The three point bend tests
- Indirect Tensile Strength

- Unconfined Compressive Strength

After the test is done and the load-displacement graph plotted, the fracture energy of the material given in Joules per meter square (J/m^2), is the measured area under the curve before the material fails (i.e. just before the maximum load is reached) as illustrated in Figure 2.17.
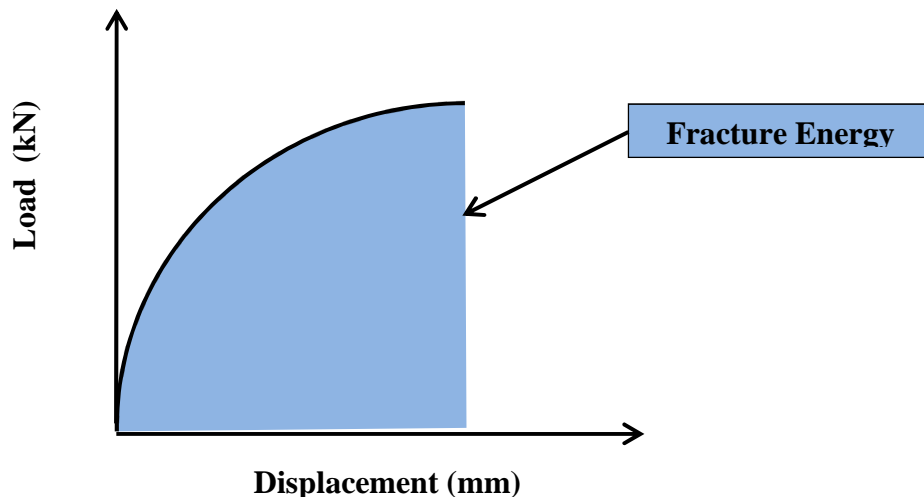
Figure 2. 17: Graphic illustration of fracture energy

The applications of fracture energy in pavement engineering are various. Scholars around the world conducted several works on fracture energy in relation with cracking in pavement layer (Hot Mix Asphalt in particular). Early works in this regard started in the late 1960's with K. Majidzadeh at Ohio State University, investigating on the prediction of fatigue damage in bituminous pavements. Wen and Kim's work on indirect tension test identified the fracture energy as a suitable indicator of the resistance to fatigue cracking in asphalt mixes. A research team of University of Florida also highlighted this theory while developing a fracture mechanic model for Hot Mix Asphalt. Moreover, Von Quintus et al. acknowledged the possibility of using failure strength and strain in evaluating thermal and fatigue cracking of asphalt mixes.

Fracture energy is related the fracture resistance. When determined for various BSM's mixes, fracture resistance could help to differentiate which of them will be easily affected by cracking when submitted to traffic loading. Moreover, the fracture energy parameter is known to be highly connected to the fatigue life and flexibility of bituminous mixes (M. Saleh, 2006). The theory is that: the higher the fracture energy of a material is, the higher the amount of deformation it can undergo before failure. Their higher fracture energy implies higher flexibility.

2.4 Performance properties, flexibility and dominating failure mechanism of BSM

2.4.1 Overview

Bitumen stabilized materials, being the main subject discussed in this research, are widely used for base layers in flexible pavement as mentioned in Section 2.4.3. The combination of non-continuously bound and flexible nature enable the layer made of BSM to transfert high load traffic without cracking. They are known to maintain flexibility in flexible pavements and for many years, they have been successfully used in flexible pavement rehabilitation. BSMs pocess several properties that are linked to their flexibility, performance and failure mechanisms.

2.4.1.1 Stress dependent materials

Previous research has shown that the behaviour of bitumen stabilised materials is stress dependent (Jenkins, 2000; Ebles, 2008). Jenkins (2000), through triaxial testing found that foamed treated materials with les that 4% bitumen up to 1% cement exhibit a stress dependency behaviour very close to that of granular materials. In other words, the resilient modulus of BSM increases during triaxial testing as the sum of the principal stresses applied increases. As shown in Figure 2.18, it can be seen that the resilient modulus of foamed bitumen mixes almost double as the sum of principal stresses increases from 100kPa to 900kPa.
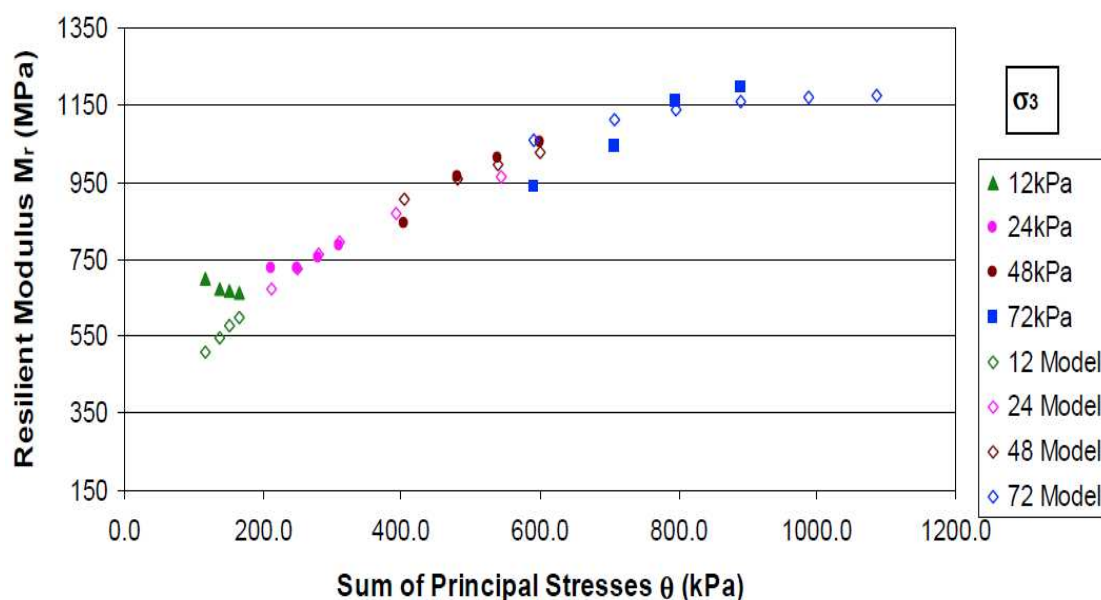
Figure 2. 18: Resilient Modulus as a Function of Total Stress from Triaxial Tests (2% Binder, T= 20ºC), Jenkins 2000.

2.4.1.2   Flexibility

Flexibility of bitumen stabilized materials is mostly seen in terms of mix property rather than material property. Flexibility in the BSM layer is the property that enables the layer to transit wheel load stress to the under layer without suffering from cracks and/or plastic deformation. The property is directly related to the load spreading ability of the layer under monotonic, cyclic and dynamic conditions. From the literature, the use of bituminous stabilizers (i.e. Bitumen emulsion or Foamed bitumen) increases flexibility and fatigue resistance of the mix. On the other hand, the absence of flexibility in the BSM's layer leads to premature cracks.

The idea of implementing flexibility as design property was newly introduced in bitumen stabilized materials guidelines. The design philosophy of TG 2 guidelines of the Asphalt Academy, contrarily to previous mix design guidelines, put an emphasis on stiffness and flexibility as key parameter to consider during the mix design of bitumen stabilized material. The concept was to limit the addition of cement, known to increase the compressive (UCS) and tensile (ITS) strength at the expense of the flexibility.

Long, F.M. *et al*. 2005, conducted a research on foamed bitumen treated sand materials with various bitumen contents and active filler content by adopting UCS, ITS, monotonic beam tests and erosion tests. It was found that increasing the cement content adds strength to the mix but reduces its flexibility. Moreover, the increase in cement content resulted in the decrease in the strain-at-break, and the higher the bitumen and filler content, the higher the durability of the mix (Long, F.M. et al. 2005).

2.4.1.3   Visco-elastic materials

BSMs contain bitumen, which is a visco-elastic binder. Therefore, they exhibit visco-elastic behaviours under load application. In other words, they can exhibit both viscos and elastic characteristics. At low temperature and high loading frequencies, the bitumen and the asphalt mix behave purely elastic. At high temperature and long loading times, the bitumen and asphalt mix will behave viscous. This shows that their stress-strain relationship is time and temperature dependent. Ebels (2008), after conducting a series of tests on BSMs made possible the determination of Burgers Model parameters describing visco-elastic behaviour. He also developed fatigue relationships for bitumen stabilized materials that he compared with the one of Hot Mix Asphalt. He found that: fatigue performance of BSMs is lower than the one of HMA and that BSM-foam have fairly better fatigue life and a lower flexural stuffiness compared to BSM-emulsion (see Figure 2.20).

The visco-elastic properties of bitumen droplets in the mix enable bitumen stabilised materials to gain flexural strength (TG2, 2009). The flexural strength of bitumen stabilised material is assessed by their ability to resist deformation (flexure failure) under loading (bending condition).

2.4.2 Flexural stiffness

The presence of bitumen in BSMs enables them to exhibit a visco-elastic behaviour, which can be determined by flexural stiffness tests. The fatigue behaviour and the flexural stiffness of bituminous mixes are related (Twagira, 2010). The stiffness of BSM reduces as a result of accumulated damages due to repeated loading over time i.e. fatigue. BSMs also acquire significant flexural strength due to the visco-elastic behaviour created by the bitumen. Flexural stiffness of BSM is time and temperature dependent. Ebels works shown that to a lesser extent that Hot Mix Asphalt, the flexural stiffness of BSM mixes increases with the increase of the loading rate and the decrease of the testing temperature (see Figure 2.19). In this regard, he noticed that the fatigue test performed on beams made of BSMs at a temperature of 5^0C led to shear failure at the edges of the beams rather than flexure at bottom of the central part of the beam.
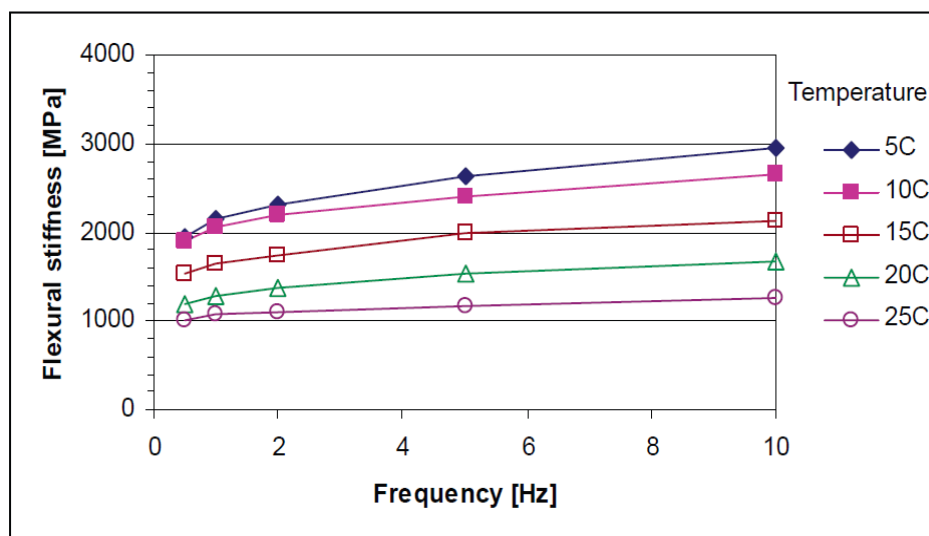
Figure 2. 19. **Illustration of flexural stiffness cure per temperature (Ebels, 2008)**

Analyses by Ebels (2008) have also shown that flexural stiffness of BSM is influenced by the percentage of RA in the mix, the percentage of active filler and bitumen content. He found that mixes with 1% cement content have high flexural stiffness compared to mixes with 0% cement and high percentage of RA.

Ebels work has shown that bitumen stabilized materials, to a lesser extent than hot mix asphalt exhibit visco-elastic behaviour. In other words, bitumen stabilized materials are time and temperature-dependent materials as illustrated in Figure 2.20
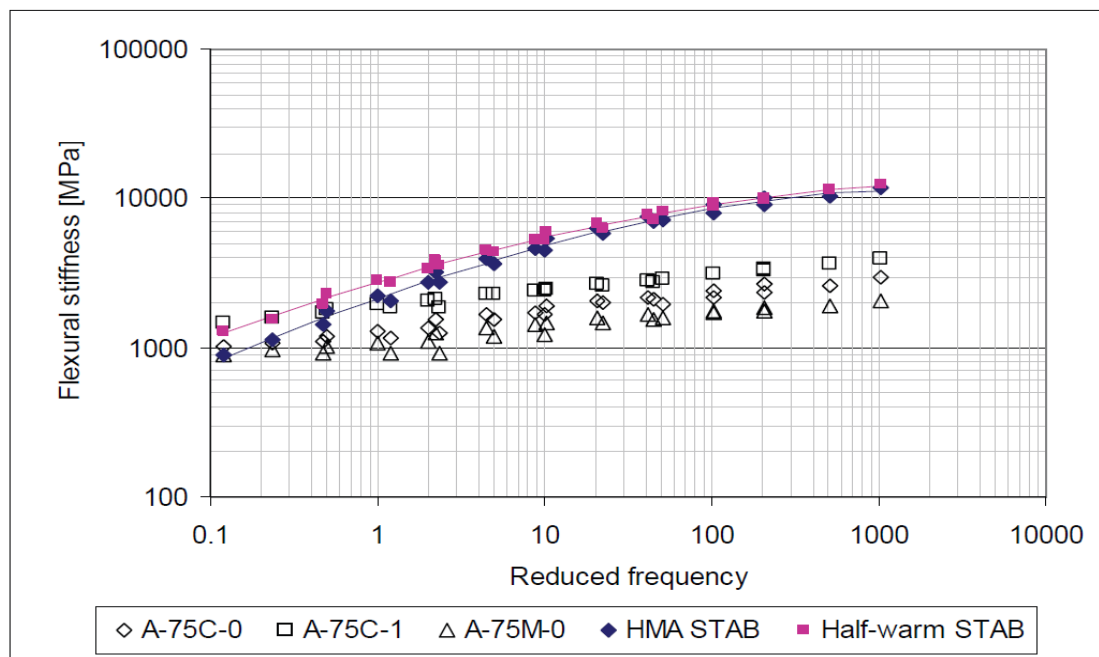
Figure 2. 20: Time and temperature dependency of BSM, half warm asphalt and hot mix asphalt (Ebels, 2008)

By comparing the initial stiffness of BSMs mixes during the fatigue testing with the flexural stiffness values obtained during the master curve testing, Ebels work showed that the flexural stiffness of BSMs depends on the stress/strain level applied. He also noticed that the flexural stiffness of foam BSMs was relatively low than that of emulsion BSMs. Therefore, foam-BSM will show a better fatigue life compared to emulsion BSM. This is partially due to the concept of bitumen diffusion which influences the mix property and behaviour.

2.4.3 Tensile and Compressive Strength vs. Flexural strength of BSM's
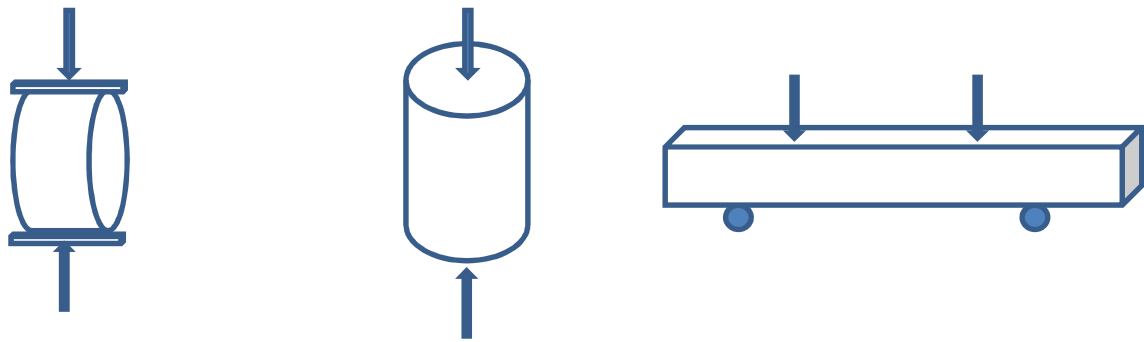
(a) Indirect Tensile loading   *(b) Compression Loading*   *(c) Flexure loading*

Figure 2. 21: Tensile, Compressive and Flexure Loading

The tensile, compressive and flexural strength are mechanical material properties that enable a particular material to withstand stresses that attempt to stretch it, reduce its size or bend it respectively. They could also be defined as the maximum amount of tensile, compressive and flexural stress that can be subjected to a material before it fails. They are all three determined based on the same principle: A load is applied progressively at a controlled displacement rate on a specimen till it fails completely while the stresses vs. strains measurements are being recorded. The tensile, compressive and flexure strengths therefore represent the highest stresses experienced by a material when submitted to tension, compression or flexure respectively. According to scholars, it's believed to be a relationship between the tensile, compressive and flexural strength (Kori F., 2004)

A cross section of a bound material in flexure shows that it experiences ranges of tresses along its depth. The fibres located on the bottom part of the material (below the neutral axis) will experience tensile stresses while the fibres located in the upper part of the material (above the neutral axis) will experience compressive stresses. These stresses increase as they move from the neutral axis to the edge of the material. The extreme fibres located at the top and bottom of the material therefore experience the maximum compressive and tensile stresses respectively. For many materials tested in flexure, one could notice that their failure usually happens under tensile stress before compressive stress. Therefore, the maximum tensile stress that the material can sustain before it breaks under flexure is flexural strength or modulus of rupture.

Based on their definition, the tensile strength could be seen as the opposite of the compressive strength. However, their values for the same material can be quite different: the tensile strength of

concrete being generally about 10 to 15% the value of the compressive strength. Contrary to compressive strength, tensile strength has been proved to correlate quite well with flexural strength (Popovics, 1998) despite the fact the failure mechanisms are different. It has been proven that some mix parameters such as: Cement content, cement- water ratio, voids content, the type of mineral aggregate, the curing and testing conditions etc., have significantly different effects on compressive strength measurements than on comparative flexural strength (Popovics 1998, pp.104 ff). Research has also shown that the type of coarse aggregate in the mix could considerably affect the compressive/tensile strength relationship. At all other mix parameter equal, mixes made with crushed aggregate usually have a higher tensile strength compared to mixes made with rounded aggregate.

In the case of rigid pavements, the design is based on the flexural strength (or modulus of rupture) of the concrete. However, the mix design of the concrete used at construction is based on the compressive strength (Sharad Y. and Deepak D., 2012). Moreover, that knowledge of the tensile strength at the bottom of the concrete layer is used to estimate the load under which cracking will develop due to its influence on cracks formation and propagation when the layer is subjected to flexure.

During the mix design of bitumen stabilised materials, the tensile strength from the ITS test is used to identify the suitable stabiliser agent, the required bitumen content, as well as the need for active fillers and the type and content. It's also believed that the flexural stiffness could be used to characterise their flexibility properties. In this regard, scholars recommended the use of the flexural beam strain-at-break as a mix design parameter for BSM's (Long, F. and Theyse, H. 2004, Twagira et al. 2006, Jenkins, K.J. and Ebels, L.J. 2008). However, Ebels (2008), found the flexural beam strain-at-break not to be reliable enough to normalize fatigue of BSMs, for real strain at break values were expected to deviate from the result he obtained based on linear elastic theory, since BSMs behave visco-elastic and not linear elastic.

2.4.4 Factors affecting the performance of BSMs and failure mechanism

The behaviour of bitumen stabilized materials is similar to that of unbound granular material, but with a significantly improved cohesive strength and reduced moisture sensitivity (TG2, 2009). On the other hand, bitumen stabilized materials differs from the unbound granular materials by the adhesive and cohesive bondage between binder (Bitumen emulsion/Foamed bitumen) and aggregates that makes them more flexible than unbound granular materials. Hence, understanding the interaction between the aggregate and bitumen as well as the factors that influence that interaction will give us an insight on performance and failure mechanisms of BSMs as far as flexibility is concerned.

2.4.4.1 Factors affecting the performance of BSMs: Flexibility included

BSMs are known to be sensitive to various parameters, which if not handled with care could affect their performance. They range from the design (i.e. selection of appropriate aggregate and grading, stabilizing agent and content, active fillers type and content, percentage of reclaimed asphalt (RA) in the mix) to construction (i.e. construction techniques, supporting layers) and environmental condition in service (i.e. traffic, climate). All these factors play a significant role in the performance of BSM's as well as their mode of distress. Here, some of them are listed.

a) Bitumen Type and Content

Foamed bitumen and emulsion bitumen used to manufacture BSMs are both produced from the penetration grade bitumen (TG 2, Asphalt academy). The selection of appropriate grade of bitumen for Bitumen Emulsion and Foamed bitumen production is important depending on the application. Table 2.1 below gives some indication on the bitumen requirement for the production of different type of bitumen.

Table 2. 1: Type of bitumen and bitumen requirements

Types of Bitumen	Bitumen Requirements
Bitumen Emulsion	Grade bitumen's with penetration values between 80 and 100 are generally selected. Softer and harder bitumen have been successfully used in the past and can be used Stable mix Grade (low set)
Foamed Bitumen	Soft binder bitumen can be used without compromising the stability of the Mix Bitumen's with penetration values between 80 and 100 are generally selected for BSM-Foam Softer and harder bitumen have been successfully used in the past and can be used when available, however harder bitumen's are generally avoided due to poor quality foam leading to poorer dispersion of the bitumen in the mix.

Research has shown that mixtures made with foamed bitumen and with bitumen emulsion have similar properties up to a bitumen content of 1.5 % by mass of aggregate (*Bowering et al*). Theyse, 2004 found that increasing the bitumen content of emulsion stabilized materials has undesirable

effects on their Indirect Tensile Strength (ITS) and Unconfined Compressive Strength (UCS). Moreover, in the case of bitumen emulsion where the breaking of emulsion is very important at high bitumen content, the bitumen particles are more likely to come into contact with each other, resulting in an increase in the rate of break *(Shell Bitumen Hand Book, pp. 103).*

F.M. Long and DGC Ventura, 2004 found that increasing the percentage of bitumen content resulted in an increase in flexibility of the foamed bitumen mixes tested. L.J, Ebels (2008) found that within the practical binder content range of 1.5%-3.0% the effects listed below were observed on the properties and behaviours of tested bitumen stabilised materials:

Table 2. 2: Effect of increasing the bitumen content on material properties (After Ebels, 2008)

Material property and behaviour	Effect
Cohesion	Increase
Friction angle	Decrease
Shear strength	Variable
Stiffness	Variable
Time-temperature dependenty	Increase
Permanent deformation resistance	Variable
Strain at break	Increase
Fatigue	Increase

Another important parameter to control when stabilising with bitumen is the bitumen dispersion in the mix. The parameter is influenced by the percentage of active filler in BSM-foam (Collings and Jenkins, 2011). In BSM-foam the bitumen the fine particles are coated by bitumen while in BSM-emulsion, the bitumen is initially attracted by the fine particles and then the larger particles (see Figure 2.22). These give a non-continuously bound nature to BSMs mixes when compacted.
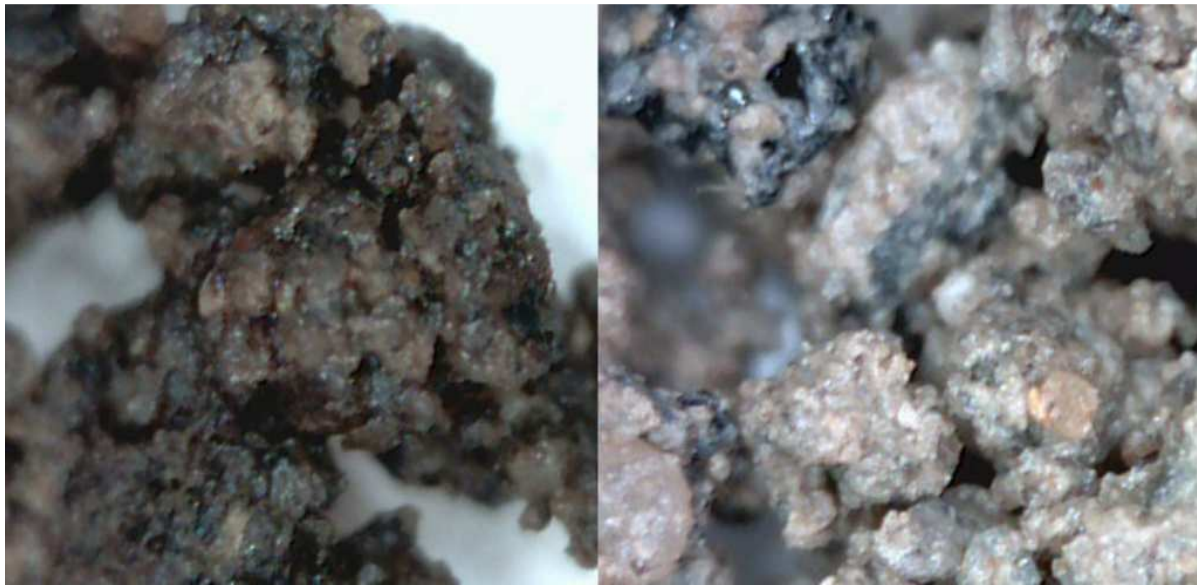
Figure 2. 22: Bitumen dispersion in the mix for BSM-emulsion (left) and BSM-foam (right), Jenkins, 2013.

b) *Active filler type and content*

The term active filler is used to define fillers that chemically alter the mix properties (TG2, Asphalt Academy 2009). They contribute to a good dispersion of the bitumen, to strength improvement and strength retained under saturated conditions. Various types of active fillers such as cement, lime, fly ash can be used separately or in combination during the production of BSMs depending on their availability, their cost and their efficacy with the actual component materials. However, rapid hardening cements are to be avoided. The higher the active filler content, the higher the tensile, compressive and structural strength of the mixture. This results in increasing susceptibility to cracking (Long et al., 2003).

When used as active filler, the application rate of cement should be limited to 1% by mass of dry material. In the case where hydrated lime is used, this rate might be increased to 1.5% or more especially if the lime is required to modify the plasticity (Wirtgen, 2010). The reason for limiting these values to 1 and 1.5% is to not create stiff and brittle materials, which would not perform as flexibly as expected from bitumen stabilized materials.

From the TG2 of the Asphalt Academy, the purpose of incorporating active fillers in BSM's mixes includes the followings:

- Improving the adhesion of the bitumen to the aggregate
- Improving the dispersion of the bitumen in the mix

- Modifying the plasticity of the natural material by reducing the PI (in this case, lime is recommended)
- Increasing the stiffness of the mix and the rate of the strength gain
- Accelerating the curing of the compacted mix.

Other reasons of using active fillers in BSM mixes are:

- To improve dispersion of bitumen in foam mixes;
- To improve the breaking time in emulsion mixes;
- To reduce moisture damage and improve the plasticity;
- To increase the stiffness and the strength of the mix
- To accelerate the curing time; and
- To improve the workability of the mix.

As far as flexibility is concerned, increasing the percentage of active filler will result is the reduction in flexibility and a more brittle material. F.M. Long and DGC Ventura, 2004 after conducting a series of test on a G2 material, treated using foamed bitumen found that the mixes were showing similar flexibility at 1 and 2 per cent cement content. However, the parent material that was stabilised had high strength already. Therefore, the mix component had a little effect on the material behaviour.

c) *Aggregate Type and Grading*

Despite the fact that a wide range of aggregate can be used for BSM's production, the nature of the parent material has a significant influence on the stiffness and the behaviour of the treated material when submitted to the stress from traffic wheels. Research have shown that properties such as stiffness, flexibility, durability, workability, compactability, permeability, fatigue resistance and others can be optimized by the grading of the aggregate used for the mix (Sakr and Manke, 1985; Roberts et al., 1996; Jenkins, K.J. 2000). This shows how important the concept of grading is in pavement mix design regardless of the type of mix: Hot Mix Asphalt, Seal, Cement Treated Materials or Bitumen Stabilized Materials.

According to literature and mix design guidelines, the suitability of the aggregate for BSM's mixes is more defined by its gradation rather than by its characteristics, nature or origin. In the TG2 (Second Edition) of the asphalt academy, the general grading requirements of aggregate for BSM's production are indicated in terms of zones of most suitable aggregate composition. Moreover, other specific grading requirements such as the percentage passing the 0.075 mm sieve, the maximum

stone size and the amount of coarse aggregate are vital for the performance of BSM's. They respectively have an influence on the dispersion of the bitumen through the mix and the mix compaction. Also, Sakr and Manke 1985 found that the angularity of fine aggregates is an excellent indicator of suitability for foam stabilization. They also showed that the stability of foamed asphalt mixes is affected to a greater extent by the aggregate interlock than by the viscosity of the binder.

The fine content in the material to be treated with foam has a significant influence on the quality of the mix produced. The ability of foamed bitumen to selectively mix with and coat the fines is well documented (Jenkins,K.J. 2000; Sakr and Manke, 1985; Ruckel et al, 1982). Previous research have shown that the mastic (mix of bitumen, fine and water) acts as a mortar between the coarse aggregate and therefore increases the strength of the mix. However, the relationship between the fines content and bitumen content is critical because excess bitumen in the mortar will tend to act as a lubricant and result in loss of strength, stability and workability (Muthen, 1999). Approximately 5% filler is required in the aggregate to produce BSM-foam that will perform (TG2 Second edition, 2009). Ruckel et al, 1982 suggested this fraction to be above 5%. Figure 2.23 shows the influence of fine blend on the tensile strength of a material treated with both bitumen emulsion and foamed bitumen.
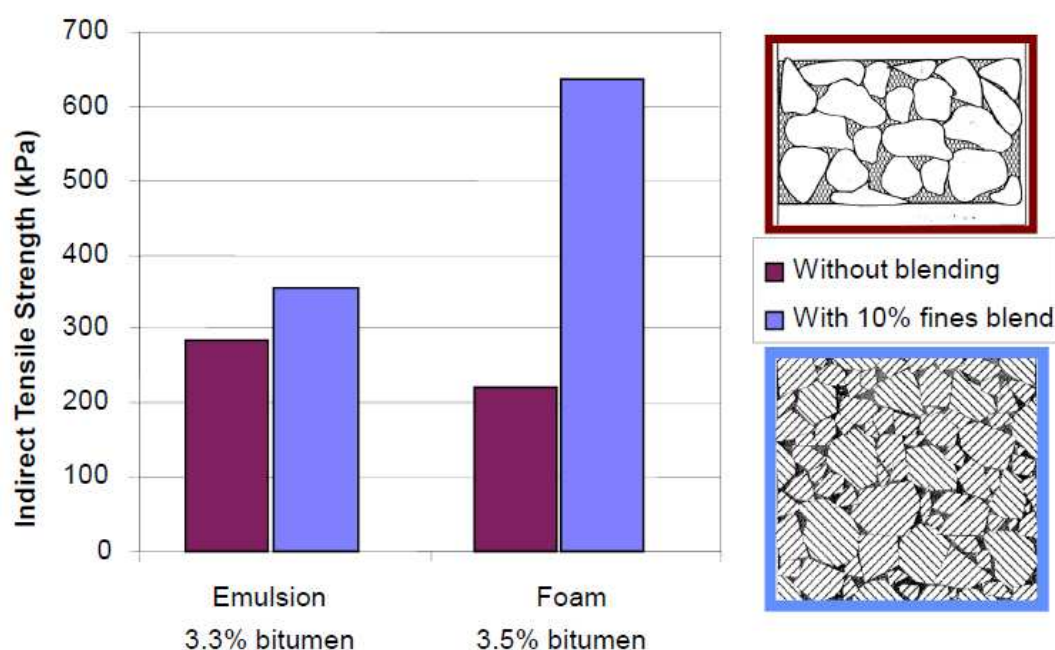
Figure 2. 23: influence of the grading curve optimisation on the ITS (Jenkins, 2013)

For BSM-emulsion, the fraction of fine is not as high as the one for BSM-foam because the nature of dispersion of the bitumen in BSMs which is different for BSM-foam and BSM-emulsion. Bitumen emulsion coats the larger aggregate particles to a greater extent than foamed bitumen. A minimum filler content of 2% is sufficient in aggregate treated with emulsion (TG2 Second edition, 2009).

d) Temperature

During the curing period of newly constructed BSM layer, temperature increases due to time and/or solar radiation could influence the lost in moisture and hence strength development of the layer. Twagira 2010, found that solar radiation, air temperature, wind speed and heat transfer coefficient of pavement material play significant roles in the temperature gradient in the BSM-emulsion layer. He listed the following points as advantages of predicting temperature distribution in the BSMs layer from local environmental conditions:

- Ability to understand the mechanism of bitumen ageing and/or mastic stiffening during curing process of compacted mix;
- The ability to predict water evaporation from the material during the curing period, which is an important aspect for the contractor/designer to be able to predict the early opening of the BSMs layer to traffic (For BSM-foam).

The flexibility of bituminous mixes is highly dependent on temperature. This is due to their visco-elastic behaviour and that of the bitumen, which is an essential component of the mix. At low temperature, the bitumen is susceptible to become hard, causing the bituminous material to loose of his flexibility and behave much like a brittle material. This may result in low temperature cracking with load application. On the other hand, increased temperature may improve the flexibility property of bituminous mixtures due to the visco-elastic property of the bitumen. However, high temperatures may also cause the bitumen to be more fluid, and this may result in rutting with if stress is applied on the material. Ebels (2008) noticed an increase in stiffness of BSMs by increasing the loading rate and keeping the temperature constant. In the other hand, the increase in temperature led to a decrease of the stiffness. This shows the BSMs' flexural stiffness dependency to temperature and loading rate.

e) Moisture condition

The importance of having the appropriate moisture content at the time of mixing and compaction of BSMs in order to insure its performance does not need to be proved. Besides, many researchers consider it to be one of the most important criteria of BSM's mix design, because it has a direct effect on the final product. During BSM production, the effectiveness of bitumen dispersion, the compaction effort required to achieve density, and the potential for surface cracking are all significantly influenced by not only the actual moisture content itself, but also by the uniformity of the moisture content throughout the recycled material (TG2). Insufficient moisture reduces the workability of the mix while too much moisture content lengthens the curing time, reduces the strength and density of the compacted mix and may reduce the coating of the aggregates (Muthen,

K.M. 1999). Therefore, the determination of the appropriate Optimum Moisture Content (OMC) of the material to be treated is crucial.

The mixing moisture content that will provide the best BSM mix is termed by the TG2 as the Optimum Mixing Moisture Content (OMMC). It is a combination of the moisture contained in the aggregate, any additional moisture and the moisture contained in emulsion in case of BSM-emulsion. In BSM-emulsion mixes, the OMC using the modified AASHTO compaction should be used for the total mixing fluid content. This is explained in the equation bellow.

$$OFC = FMC + EWC + RBC$$

$OFC = Optimum\ Fluid\ Content\ \%$

$OFC_{MOD-U} = Optimum\ Moisture\ Content\ using\ Mod.AASHTO\ Compaction\ on\ untreated\ material\ \%$

$FMC = Field\ Moisture\ Content\ of\ aggregate\ \%$

$EWC = Bitumen\ emulsion\ water\ content\ including\ water\ used\ for\ dilution\ \%$

$RBC = Residual\ Bitumen\ Content\ as\ percentage\ of\ dry\ aggregate\ \%$

In the case of BSM-foam, Mobile Oil devolved the "fluff point moisture content" of the aggregate concept. It is the moisture contain at which the maximum bulk volume of loose mineral aggregate is obtained. That point is believed to be the point where the minimum value of the mixing moisture content for BSM-foam lies. However, the fluff point may be too low to ensure adequate mixing (foam dispersion) and compaction, especially for finer materials (Muthen, K.M. 1999). Bowering (1970), observed that where inadequate foam dispersion occurred because of insufficient mixing moisture, the compacted densities were low and no benefit was gained from the foamed bitumen treatment. Lee (1981) found that the optimum mixing moisture content occurs in the range of 65 - 85 per cent of the modified AASHTO OMC for the aggregates. The TG2 recommends that 65 to 85% of the optimum moisture content using modified AASHTO compaction should be used for mixing moisture content when adding foamed bitumen.

As far as flexibility is concerned, considerable moisture reduction, take place in BSMs mixes during curing. Very low moisture content (dryness) could also compromise the flexibility of the mix. Fenton (2013), found that moisture content both at curing and at breaking (during the test) has a significant effect on the flexibility of bitumen stabilised materials. On the other hand, excessive

moisture can negatively influence the property by breaking the bound between the aggregate and the bitumen.

In summary, the role of moisture in bitumen stabilized mixtures. In several instances, the role of moisture plays is similar for BSM-emulsion and BSM-foam. Some differences do however exist. Some of them are given in the Table 2.3.

Table 2. 3: Role of fluid in BSM (Wirtgen, 2012)

Component	BSM-emulsion	BSM-foam
Bitumen	Contributes to fluids for compaction	Negligible contribution to fluid for compaction
Moisture in aggregate	Reduces absorption of bitumen emulsion water into aggregate	Separates and suspends fines making them available to bitumen during mixing
	Prevents premature cracking	Acts as carrier for bitumen splinters during mixing
	Extends curing time and reduces early strength	Reduces early strength
	Provides workability of the bitumen at ambient temperature	
	Reduces friction angle and lubricates for compaction	
	Provide shelf-life for the mix	

f) Curing

Previous works on the performance of BSM's have shown that they don't develop their full strength after compaction until a large percentage of the mixing water is lost. This process whereby the mixed and compacted bitumen stabilized materials discharges water through evaporation, particles charge repulsion or pore-pressure induced flow paths leading to the increase in strength (both tensile and compressive) as well as stiffness, could be defined as curing (Jenkins, 2000).

Reduction in moisture content constitutes the major part of the curing process of bitumen stabilised materials (Wirtgen, 2010). Curing is directly related to the stiffness of bitumen stabilised materials. The effect of curing leads to an increase in stiffness of BSM layers after construction (Ebels, 2008). It also leads to an increase in compressive and tensile strength, which affects the performance of the layer.
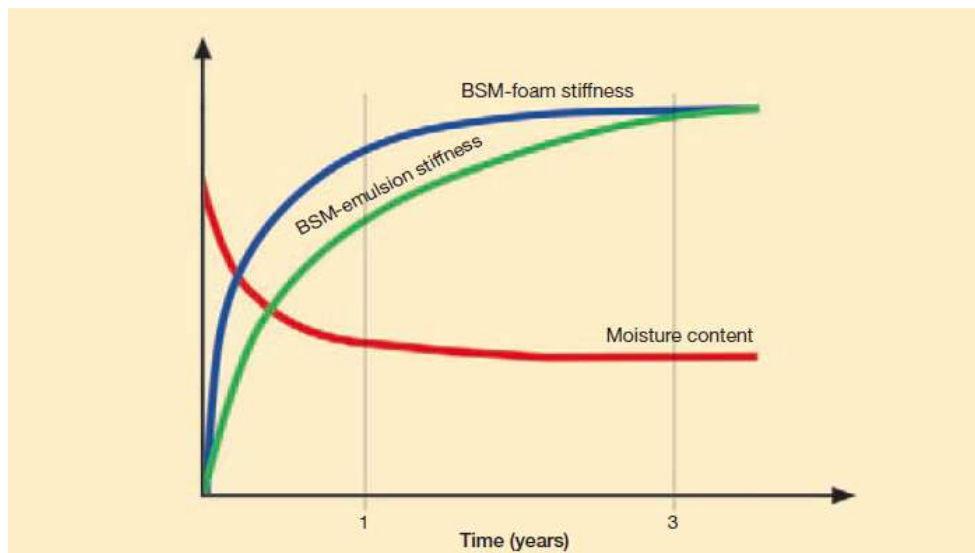
Figure 2. 24: Concept of curing and influence and the mix stiffness (Wirtgen, 2012)

This accelerated strength gain in the early days of the BSM layer could lead to stiffer and less flexible material. Although the addition of bitumen content may increase the flexibility of the mix, that same flexibility can also be compromised by the stiffening effect due to curing, especially in mixes with high percentage of active filler.

2.4.4.2   *Dominating failure mechanisms of BSM*

All materials are susceptible to failure. When a particular material is no longer able to achieve the functions for which it was originally designed, the phenomenon is referred to as failure. The failure of a material is not to be limited to a one-time action (i.e. fracture, total disintegration). It could also be a continuous process (i.e. change in shape, loss of material, changes in mechanical and engineering properties). This is more often the case with bitumen stabilized materials when submitted to operating conditions (i.e. trafficking, stress, impact) and environmental conditions (i.e. temperature, moisture conditions). Despite the fact that BSM's have been successfully used worldwide in road construction and rehabilitation for the past four decades, some area such as their failure mechanism are yet to be extensively documented. Knowing the failure mechanisms of BSM and understanding their occurrence will help engineers to ameliorate current design practice for better performance. From the literature, the performance of bitumen stabilized materials is affected by two fundamental failure mechanisms namely: permanent deformation and moisture susceptibility.

a)   *Permanent deformation*

As mentioned in section 2.2.3, the non-continuously bound property of BSMs gives them behaviours and failure mechanisms close to those of unbound granular material. Therefore, despite the flexible

property and the tensile strength that they possess, scholars have identified permanent deformation under load repetition and environmental conditions over time as the primary failure mechanism of bitumen stabilized materials. This failure mechanism results from the build-up of shear deformation due to repeated loading and environmental conditions and has been shown by scholars to be dependent on the material's shear properties, the densification achieved during compaction and also the temperature as shown in Figure 2.25.
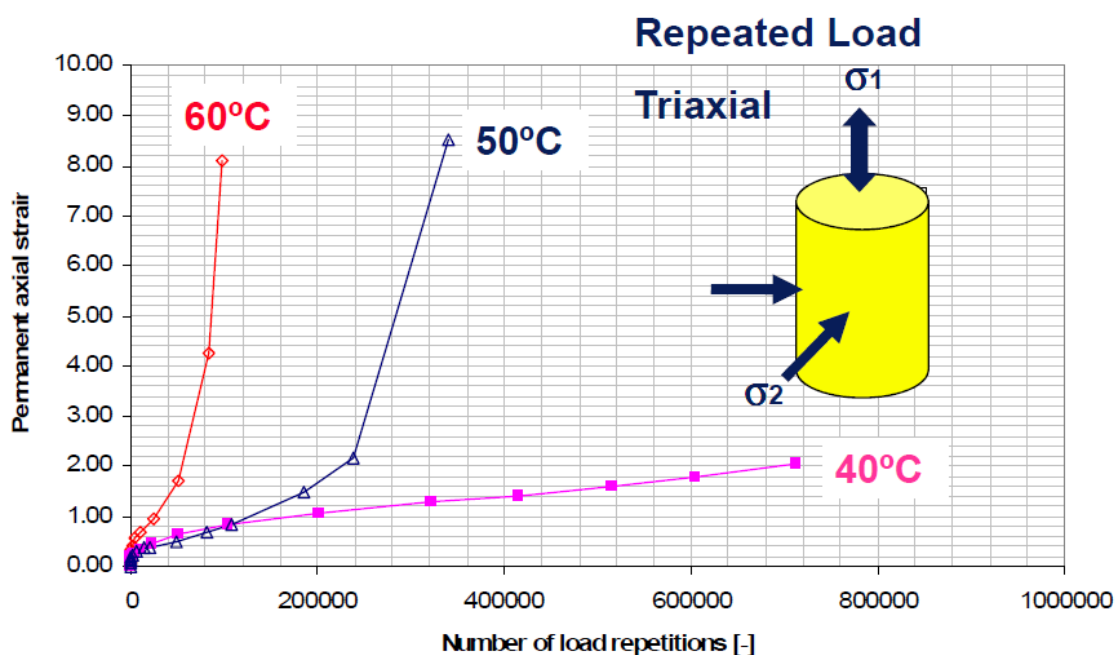
Figure 2. 25: Effect of temperature on permanent deformation (Jenkins, 2013)

One of the advantages of using bitumen stabilized material is early trafficking; for BSM's layers can carry traffic almost immediately after compaction is completed, provided that traffic volumes are not too high. However, it is in the early period of repeated loading that the majority of the permanent deformation takes place. Therefore, the rate of moisture loss from newly constructed BSM layers plays a significant role in determining the performance of the layer (TG2, Asphalt Academy 2009). One can prevent this from happening by minimizing moisture content during construction in the case where the BSM layer will be trafficked immediately after construction; for the lower the degree of saturation of the BSM, the greater the resistance to permanent deformation.

Researchers have shown that the permanent deformation of BSMs is a long-term phenomenon that usually takes place in three phases. The first one consists of bending due to accumulation of permanent axial strain. A second phase with a constant rate of strain accumulation and a third phase leading to failure initiated by the so-called flow point (Ebels and Jenkins, 2006).

From the general law of permanent deformation originally developed by Francken (1977) for hot-mix asphalt, Jenkins (2000) developed the permanent deformation law of BSM-foam materials showing that the failure mechanism is also mainly dependent on the number of load repetitions. According to the TG2 manual, the following actions can improve the resistance to permanent deformation of BSM.

- Improved aggregates characteristics (Angularity, Shape, roughness and particle size);
- High level of densification;
- Reduced moisture content through curing; and
- Addition of limited amount of bitumen and active fillers (less than 3.5% and 1% maximum respectively)

b) *Moisture Susceptibility*

One vital property of BSM's is the cohesion and adhesion of the binder or mastic with aggregate. Moisture susceptibility within the BSM layer could be defined as the damage caused by the exposure of BSMs to high moisture content because of poor drainage and/or pore-pressures caused by the traffic. This has as a primary consequence the loss of bond between bitumen and aggregate (see Figure 2.26). Despite the fact that bitumen emulsion and foamed bitumen binders, when mixed with aggregates react differently in presence of moisture, this exposure to high moisture content could have disadvantageous influences on the stiffness and the strength of the mix in both type of treatment. Hence the presence of water within the layer as well as the partially coated nature of the aggregate makes moisture susceptibility a major aspect to consider when it comes to BSM's performance evaluation (TG2, 2009).

Jenkins, Van der Riet and Twagira, 2008 recommended more investigations of moisture susceptibility in terms of mix variables such as: Binder and aggregate type, active filler, compaction, curing and harshness of moisture exposure. In this regards, Twagira (2010) developed the Moisture Inductive Sensitivity Test (MIST) apparatus. The apparatus was used to simulate moisture condition by water induction on triaxial specimens made of bitumen stabilized materials with various aggregate types at different binder and active filler content; in order to evaluate their resistance to moisture damages. Emulsion mixes with long curing condition were found to have a better performance with 100 000 loads repetition in wet conditions compared to the one tested at standard accelerated curing. This shows that mixes subjected to long curing have a high resistance to moisture damages compared to the ones subjected to accelerated curing. He also found that wet trafficking could lead to the disintegration of the BSM layer depending of the quality of aggregate used,

cohesion, adhesion between binder and aggregate, voids content, saturation level and even temperature.
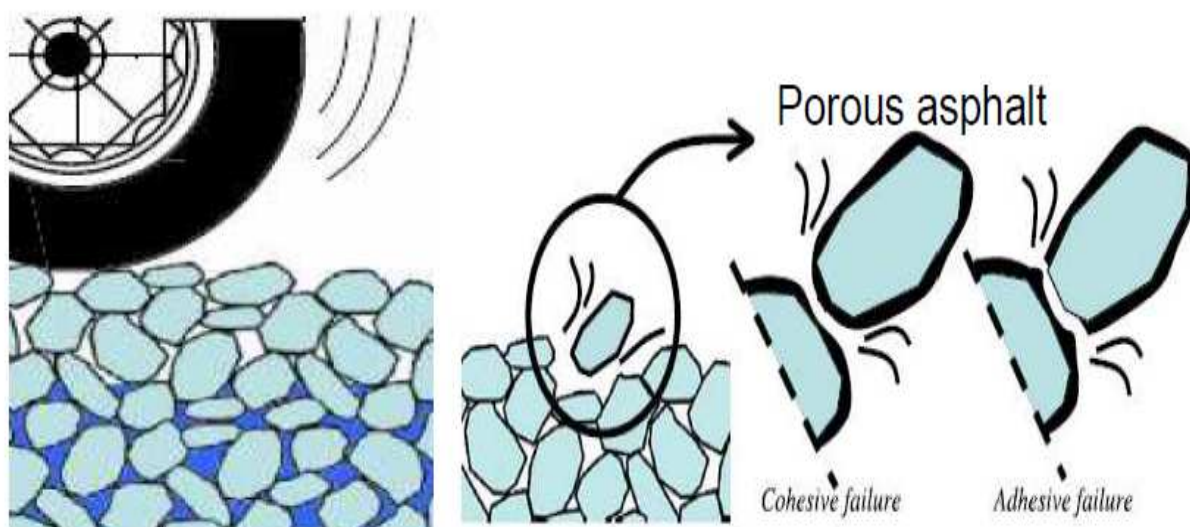
Figure 2. 26: Loss of bond between aggregated and mastic resulting in cohesive and adhesive failure (Erkens, 2002)

2.5 Synthesis

From the literature study presented in the sections of this chapter, it is clear that:

- Bitumen stabilisation is of great interest in the industry of road construction and rehabilitation across the world today. The technology possesses numerous advantages compared to other mode of pavement material stabilisation and proper mix design procedure is required to optimise their use and application.

- Flexibility is the property that enables a particular material to return to original state after the force that has caused its temporally bend is removed. To have better comprehension of the term 'flexibility', the understanding of some fundamental material behaviour (i.e. elasticity, viscosity and plasticity) in relationship to load response behaviours such as brittleness and ductility is necessary.

- For BSM, flexibility is mostly seen in terms of mix property than material property. The property enables the BSM layer to transit wheel load stress to the under layer without suffering from premature permanent deformation.

- In South Africa, BSM mix design and classification are mainly based on ITS and UCS values abstained in soak and dry condition. In the TG2, the importance of flexibility is mentioned, but it has not yet been adopted (TG2, 2009). The design philosophy of TG 2 guideline of the

Asphalt Academy, contrary to previous mix design guidelines put an emphasis on stiffness and flexibility as key parameter to consider during the mix design of bitumen stabilized material. However, too much flexibility could compromise the stiffness of the mix. Therefore, having an adequate balance between the two properties is important.

- The flexibility of BMS is not an easy property to measure. In the TG2, it is recommended that additional and more accurate tests should be identified to evaluate the stiffness and flexibility of bitumen stabilized materials at the mix design level. However, Twagira et al. (2006), followed by Ebels, L. & Jenkins, K.J. 2007, went in the contrary direction by recommending the flexural beam strain-at-break property not to be used as parameter for the design of bitumen stabilised materials.

- An attempt was made in order to evaluate the flexibility and fatigue of bitumen stabilised materials using the strain at break parameter (Twagira, 2006); but the data could not accurately reflect the fatigue of the mix tested and therefore, could not implement the mix design of BSM'S.

- After comparing the fracture energy of foam and asphalt mixes by computing the area under the load-displacement curve from ITS test, Saleh (2004) found that the fracture energy of asphalt mixes was higher than that of foam mixes. Meaning that the asphalt mixes show more flexibility than the foam mixes since the amount of deformation before failure is higher HMA than that of BSM-foam.

- Long, F. and Theyse, H. (2004) evaluated the stiffness reduction transfer function is of emulsion mixes as a function of the strain ratio (ratio between the tensile strain at the bottom of the pavement layer and the strain-at-break from the flexural beam test). Their work showed that the calculated strain ratio could give an indication of the measurement of the flexural bending of the material. This is the equivalent of the maximum flexibility obtained with the laboratory test. The results show that a low strain is an indication that the material is less flexible.

- The performance of BSMs is including flexibility is influence by mix components such as: the percentage and type of active filler, the bitumen content, the moisture condition the temperature, the quality of the parent material, the grading and compaction achieved during construction.

CHAPTER 3: METHODOLOGY AND EXPERIMENTAL DESIGN

3.1 Introduction

This chapter describes and explains the methodology used for investigations. The mix design is presented, the methodology outlined and the test procedures described. The experimental design of this research was developed in order to provide an understanding of the flexibility behaviour and other engineering properties of BSMs. The experimental variables considered were:

- The type of treatment (bitumen emulsion and foamed bitumen);
- The percentage of bitumen added;
- The percentage and type of active filler added;
- The density of the material; and
- The degree of saturation of the material.

To achieve this, a two phases experimental set up was designed including both specimen preparation (mixing, compaction, curing) and testing.

- Phase 1: Mix design (ITS and UCS tests)
- Phase 2: Mechanical Tests (Monotonic and dynamic triaxial tests)

3.2 Mix design function

The aim of the mix design is to select the proper proportions and quality of the mix components in order to optimize the properties (durability, stability, flexibility, void content, moisture, cost effectiveness etc.) of the produced mix. The number and type of the mix components (aggregates, water, binder, and active filler) as well as their own variability make mix design a challenging task.

All mix design procedures involve the preparation of a set of trial specimens using proposed material. Specimens of different sizes are required for the mix design of bitumen stabilized materials to be carried out at a level 1, 2 and 3 as explained in TG2. In the case of this research, optimization was done at level 2 mix design. The work proposal required two mix designs on the material with one and two per cent cement. Sets of bitumen emulsion and foamed bitumen mixes at three different bitumen contents were done. ITS and UCS specimens were made from the mixes and tested at equilibrium moisture condition and after moisture exposure (soaked 24 hours in water). When all tests have been completed and the results analysed, the selected variable combination is usually the one that provide the most economical mix, which satisfies at the same time the minimum required properties and performance. In other words, it is the compromise between the economic factor and all others mix properties.

The objective of the mix design is to produce a bituminous mix by identifying the suitable proportion of the various components to have:

- Sufficient bitumen to ensure the flexibility of the material,
- Sufficient strength to resist shear deformation under traffic at higher temperature,
- Sufficient air voids in the compacted bitumen to allow for additional compaction by traffic,
- Sufficient workability to permit easy placement without segregation,
- Sufficient flexibility to avoid premature cracking due to repeated bending by traffic, and shrinkage cracks at low temperature.

The purpose of doing both UCS and ITS tests at the mix design level of the research was also to have an indication of both compressive strength and flexibility properties of the produced mixes through USC, ITS, fracture energy, strain and displacement at break.

3.3 Mix Components

3.3.1 Mineral Aggregate

The bitumen stabilised material subjected to this research were made of one type of material from the R35, near Bethal. The material was brought to Stellenbosch University as part of an on-going research on the new South Africa Pavement Design Methods (SAPDM), which is also part of this study.

Approximately three tons of material milled during the rehabilitation of the R35 was separated into representative samples of 25kg to 30kg each and shipped from Bethal to Stellenbosch University. The material arrived sealed in thick plastic bags and was stored in an adequate space out of unsuitable conditions such as rainfall or stagnant water. The milled material received from the R35 road consisted of a mixture of two types of material and its composition was made of:

- Granular material i.e. dolerite (from the existing base and subbase made of with cement stabilised natural); and
- Reclaimed Asphalt Pavement (RA) milled from the existing surfacing.

The blending of the two materials was estimated to be at 80 to 85% of granular materials and 15 to 20% of RA per weight.

a. Grading:

After receipt of the material, a sieve analysis was done on three different samples chosen randomly. This was done in order to: firstly determine the percentage of fines (percentage passing the 0.075 mm sieve) which plays an important role when stabilising with bitumen. Secondly, to compare the

grading curve of the material with the one recommended by the TG 2 in order to ensure the quality and performance related to grading of the produced mix. The results are presented in Table 3.1 and the graphs are illustrated in Figure 3.1. It can be seen that the grading envelops are consistent. This shows that the bags received from the field had representative gradings. The percentage of fines was found to be 7.2%, 6.3% and 6.4% for Sample 1, 2 and 3, respectively. These values are within the range specified by the TG2.

Table 3. 1: Sieve analysis results done on three samples of material chosen randomly

Sieve size (mm)	Sample 1, Mass: 13349g		Sample 2, Mass: 13223 g		Sample 3, Mass: 12560 g	
	Mass Retained (g)	% Passing	Mass Retained (g)	% Passing	Mass Retained (g)	% Passing
53	0	100	0	100	0	100
37.5	0	100	0	100	0	100
26.5	686	94.9	547	95.9	612	95.1
19	528	90.9	672	90.8	732	89.3
13.2	479	87.3	550	86.6	605	84.5
9.5	635	82.6	671	81.6	750	78.5
6.7	602	78.1	1243	72.2	909	71.2
4.75	1037	70.3	1359	61.9	902	64
2.36	2270	53.6	2443	43.5	2115	47.1
1.18	2593	33.9	1858	29.5	1938	31.7
0.6	1562	22.3	1428	18.7	1120	22.7
0.425	723	16.9	572	14.4	539	18.4
0.3	435	13.6	348	11.7	517	14.3
0.15	551	9.6	329	9.3	584	9.7
0.075	305	7.2	394	6.3	407	6.4
receiver	965	0	831	0	803	0
Total	13371		13245		12533	
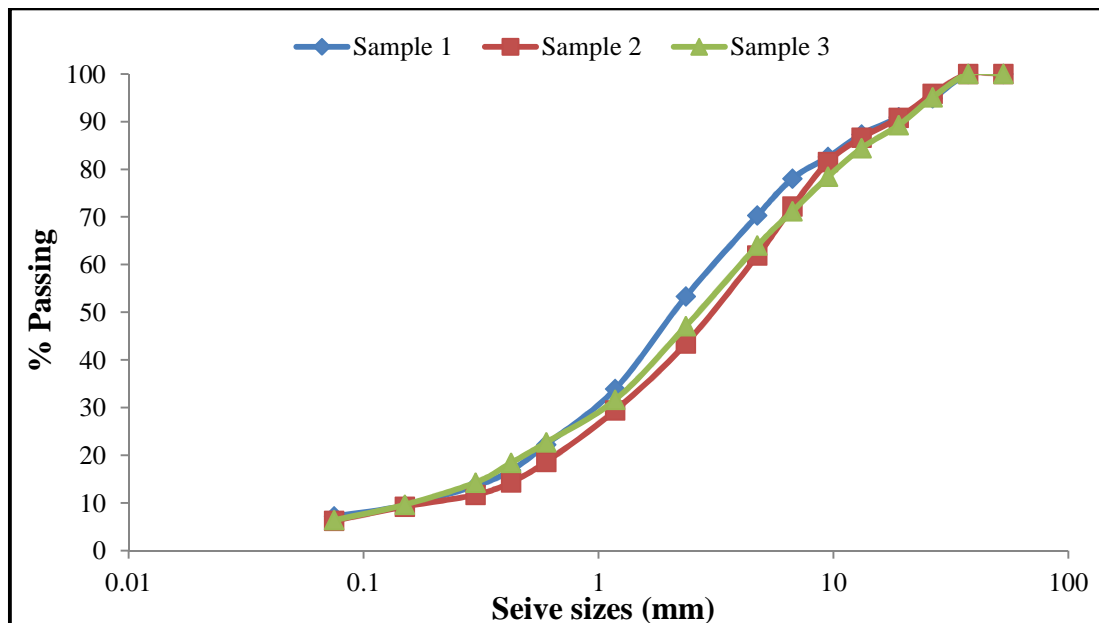
Figure 3. 1: Grading envelopes

The three grading curves obtained from the sieve analysis were then compared with the ranges recommended grading envelopes of the TG2. It can be seen from the Figure 3.2 that the grading envelopes of the three samples are shared between the ideal and the less suitable grading of material to be treated with bitumen as recommended by the TG2, but more close to the ideal limit than the upper limit. This may have a slight influence on the Voids in the Mineral Aggregate (VMA), which could slightly affect the porosity and the density of the mix after compaction.
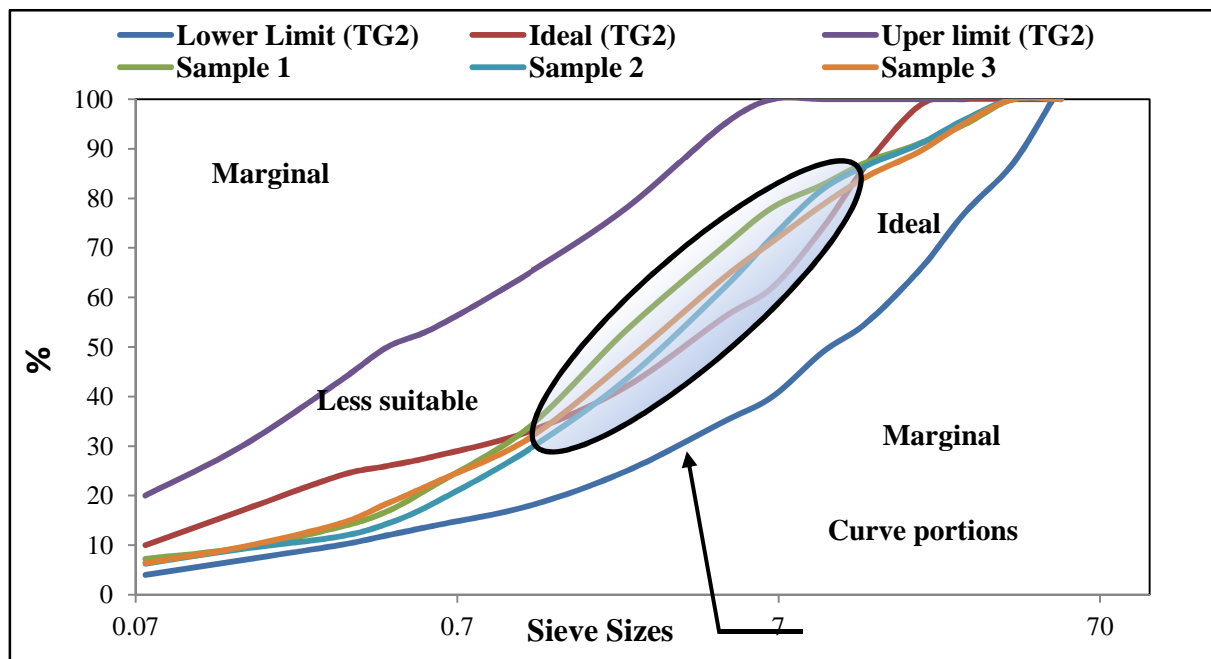
Figure 3. 2: Comparison of R35 material grading curves with the guidelines grading envelopes

In addition, a wet sieving was done on the material according to the standard procedure provided by the TMH 1(Method A5). This was done in order to have more accurate amount on the percentage of aggregate passing the 0.075mm sieve. Two samples of materials were taken randomly from the bags for wet sieving. The results are presented in Table 3.2 and the graphs are illustrated in Figure 3.3.

Table 3. 2: Wet Sieving Analysis Results

WET SIEVING ANALYSIS ON TWO SAMPLES OF MATERIALS				
Sieve (mm)	Sample 1, Dry Mass: 4097.7 g		Sample 2, Dry Mass: 3432.6 g	
	Mass Retained	% Passing	Mass Retained	% Passing
75	0	100	0	100
53	0	100	0	100
37.5	0	100	118.3	96.6
26.5	173.7	95.8	84.8	94
19	199.9	90.9	143.6	89.9
13.2	197.4	86.1	129.2	86.1
9.5	195.3	81.3	118.4	82.7
6.7	196.5	76.5	144	78.5
4.75	304.3	69.1	235.2	71.6
2.36	920.3	46.6	750.7	49.8
1.18	692.1	29.7	604.9	32.2
0.425	516.7	17.1	492.2	17.8
0.3	92.1	14.9	78	15.5
0.15	160.4	11.0	140.8	11.4
0.075	108.9	8.3	90.8	8.8
Receiver	340.1	0.0	301.7	0
Total	4097.7		3432.6	
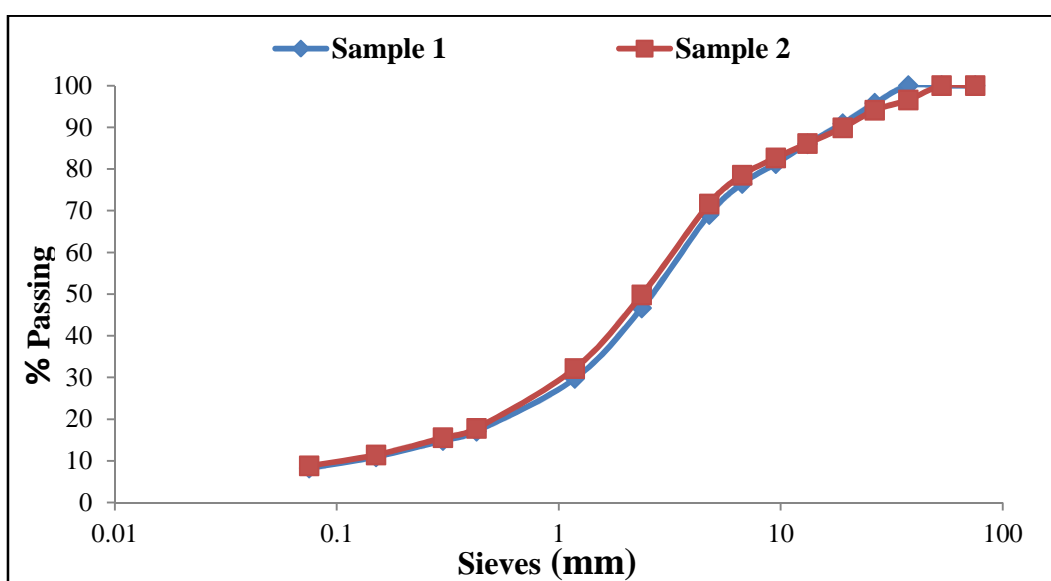
Figure 3. 3: Grading envelopes obtained after wet sieving analyses

It was found that of the percentages of fines obtained with the wet sieving (8.3% and 8.8% for Sample 1 and Sample 2 respectively) were greater than the ones obtained during the unwashed sieving; an increase of 15 to 20 per cent from the previous values obtained. This simply shows that there was a fraction of fines sticking on the coarse aggregates during the unwashed sieving.

b. *Moisture density relationship*

One important factor affecting the compaction and consequently the target density is the moisture content of the mix. The optimum moisture content and maximum dry density used in this research was determined by the CSIR through the moisture density relationship curve as shown in Figure 3.4. A maximum drydensity of 2100 kg/m^3 (100% of Modify AASHTO density) and an optimum moisture content (OMC) of 11% was obtained. These two parameters were used for the compaction of ITS and UCS specimens during the mix design phase. However, in the full testing phase, density was part of the mix variables and therefore, densities of 2160 kg/m^3 and 2050 kg/m^3 (102.8% and 97.6% of Modify AASHTO density respectively) were used for the compaction of triaxial specimens at an optimum moisture content of 11.2%. A detailed description of the test parameters is given further on.
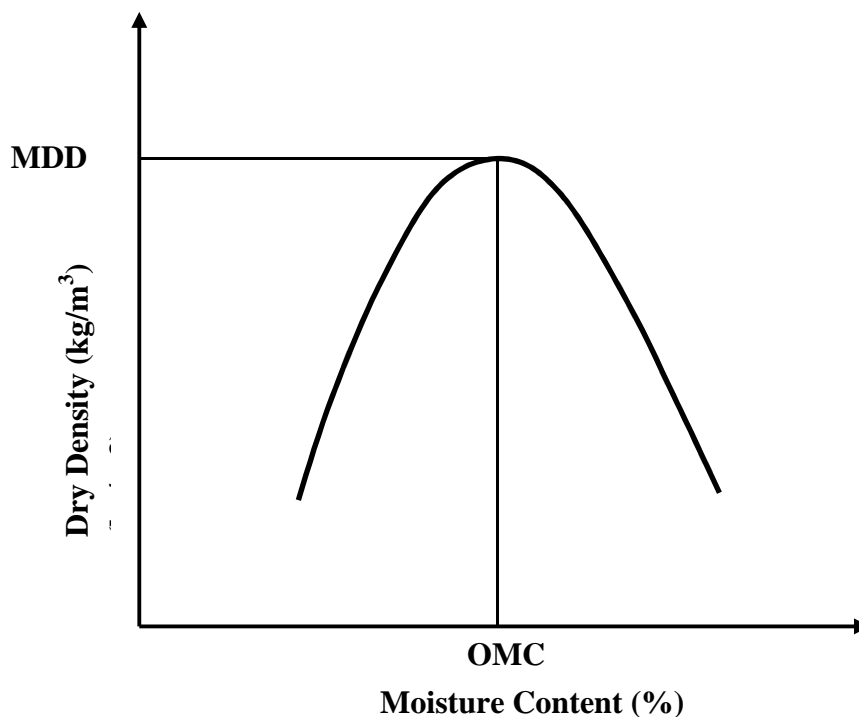
Figure 3. 4: Typical moisture density's relationship curve

3.3.2   Stabilizing agent selection

Two bituminous stabilizing agents were used in the research namely: Foamed bitumen and bitumen emulsion.

3.3.2.1 Bitumen emulsion

An Anionic Stable Grade mix emulsion 60/40 provided by Colas. Its composition is 60% of residual binder and 40% of emulsion water. Anionic emulsions are known to have a high PH (> 10.5) which is less detrimental to the PH of the stabilized material. The bitumen emulsion was received from the supplier in sealed containers of 20 litres each, stored at a temperature of $\pm\ 25^0$C and carefully stirred before being mixed with aggregate. As shown in Table 3.3, bitumen emulsion contents of 3.3%, 4% and 4.7% (i.e. 2% , 2.4% and 2.8% residual bitumen respectively), were used during the mix design phase while 4% bitumen emulsion content 2.4% (residual bitumen), was used during the second phase.

3.3.2.2 Foamed-Bitumen

The bitumen used for the foaming process was 70/100-penetration grade bitumen supplied by Colas. The reason for using bitumen of this grade is that soft bitumens have proved to have better foaming characteristics compared to hard ones. An optimum percentage of foamant water of 2% was determined at a temperature of 1600C and used for foam bitumen production. Figure 3.5 illustrates this.
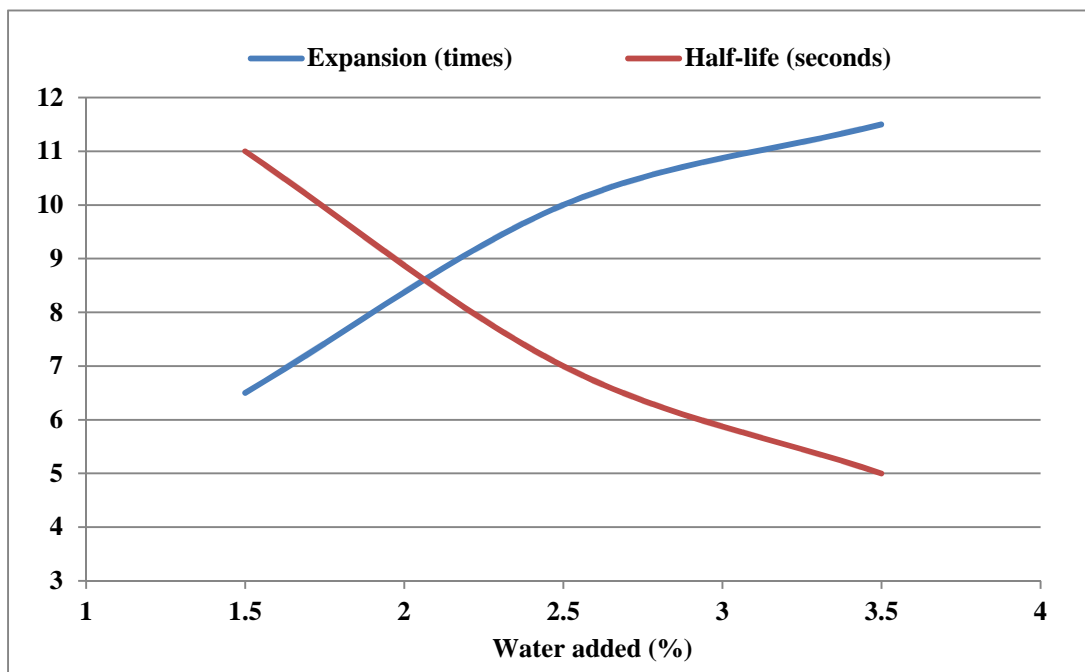
Figure 3. 5: Optimum foamant water

Three bitumen contents (i.e. 2%, 2.4% and 2.8%) were used during the mix design phase as shown in Table 3.3. However, one residual bitumen content (2.4%) was used during the second phase as in the case of bitumen emulsion mixes.

It can be seen from the graph above that increasing the foamant water leads to a greater expansion ratio but reduces the half-life. In this research, the water added to bitumen was 2% by mass. Based on this water injection rate, expansion ratio and half-life time values ranging from 8 to 10 times and 7 to 11 respectively, were obtained during the foamed bitumen production. These values fall within the ranges recommended by the bitumen stabilised material guidelines. Foamed bitumens were produced at foaming temperature varying between 155 and 165^0 C with acceptable foam properties. The air pressure was 600 kPa.

3.3.3 Active Fillers

Ordinary Portland Cement (OPC) labelled CEM II 32.5 N supplied by Afrisam and hydrated lime supplied by Cape Lime were used separately and in combination as the two type of active fillers.

3.4 Experimental design

The experimental design of this research was divided into two phases. The first consisted of the mix design where a series of ITS and UCS specimens were made and tested at different percentage of bitumen content and active filler both in wet and dry conditions. The main objective of this phase was firstly to understand the influences of mix components variation on the flexibility of the produce mix; and secondly to identify the optimal formulation of the mix variables for the second phase. The second phase was focused on the evaluation of the engineering properties of mixes produced with the optimal combination of mix variables. During the second phase, the following tests were done: ITS, UCS, Monotonic and Dynamic Triaxial Tests.

3.4.1 Experimental Variables

The following set of variables was incorporate at the mix design level (Phase 1):

- The type of bituminous binder : Foamed and Emulsion;
- The Bitumen Content: 2%, 2.4% and 2.8%;
- The type of active filler : Lime and Cement;
- The percentage of active filler: 1% and 2%
- Moisture condition at testing: Wet and Equilibrium. The term equilibrium here mains that specimen were curred according to the standard curing of the TG2 (see figure 3.12). The terms wet indicates the specimen was soaked in water for 24 hours after the standard curing before testing

Table 3. 3: Testing experimental matrix of the first phase (Mix Design)

Material Type	Type of Treatment	Bitumen Content	Active filler Type & content	TG2 Test conditions	Test type & number of specimens
Bled of Dolerite and RA	Bitumen Emulsion	2% residual	1% Lime	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			2% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Lime + 1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
		2.4% residual	1% Lime	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			2% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Lime + 1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
		2.8% residual	1% Lime	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			2% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Lime + 1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
	Foamed Bitumen	2%	1% Lime	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			2% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Lime + 1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
		2.4%	1% Lime	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			2% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Lime + 1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
		2.8%	1% Lime	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			2% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
			1% Lime + 1% Cement	Equilibrium	3 ITS & 3 UCS
				Wet	3 ITS & 3 UCS
Total number of specimens tested : 144 ITS & 144 UCS					

During the full testing phase (Phase 2), the experimental variables were as follow

- Type of treatment: Foamed and Emulsion

- The Bitumen Content: 2.4%

- The cement content : 1% and 2%

- The relative density at testing: High and Low (i.e.: 102.8% and 97.6% of Modify AASHTO density respectively)

- The saturation level at testing: Wet and Dry. The term dry here refers to moisture contents of 2.5 and 3.5% at testing while the term wet refers to moisture contents of 9.5 and 11.5% at testing.

Table 3. 4: Testing experimental matrix of the second phase (Full testing)

Material Type	Type of Treatment & bitumen content	Active filler Type & content	Type of Test	Test conditions		Number of specimens
				Density	**Saturation**	
Blend of Dolerite and RA	2.4 % Residual Bitumen Emulsion	1% Cement	ITS	MDD	Dry	3
			UCS	MDD	Dry	3
			Monotonic Triaxial test	High	High	3
					Low	3
				Low	High	3
					Low	3
			Dynamic Triaxial test (Mr)	High	High	3
					Low	3
				Low	High	3
					Low	3
		2% Cement	ITS	MDD	Dry	3
			UCS	MDD	Dry	3
			Monotonic Triaxial test	High	High	3
					Low	3
				Low	High	3
					Low	3
			Dynamic Triaxial test (Mr)	High	High	3
					Low	3
				Low	High	3
					Low	3
	2.4% Foamed Bitumen	1% Cement	ITS	MDD	Dry	3
			UCS	MDD	Dry	3
			Monotonic Triaxial test	High	High	3
					Low	3
				Low	High	3
					Low	3
			Dynamic Triaxial test (Mr)	High	High	3
					Low	3
				Low	High	3
					Low	3
		2% Cement	ITS	MDD	Dry	3
			UCS	MDD	Dry	3
			Monotonic Triaxial test	High	High	3
					Low	3
				Low	High	3
					Low	3
			Dynamic Triaxial test (Mr)	High	High	3
					Low	3
				Low	High	3
					Low	3
Total number of specimens tested : 12 ITS, 12 UCS, 48 Monotonic triaxial and 48 Dynamic triaxial (Mr)						

3.4.2 Laboratory Testing

After the material was received from the field, prepared, treated with bitumen emulsion and foamed bitumen at different bitumen content with the addition of active fillers, compacted and cured, the following tests were conducted.

- The Indirect Tensile Test (ITS);
- The Unconfined Compressive Test (UCS);
- Monotonic Triaxial Test; and
- Dynamic Triaxial Test: Resilient Modulus.

Figure 3.6 shows a summary of the methodology followed and the different tests conducted.
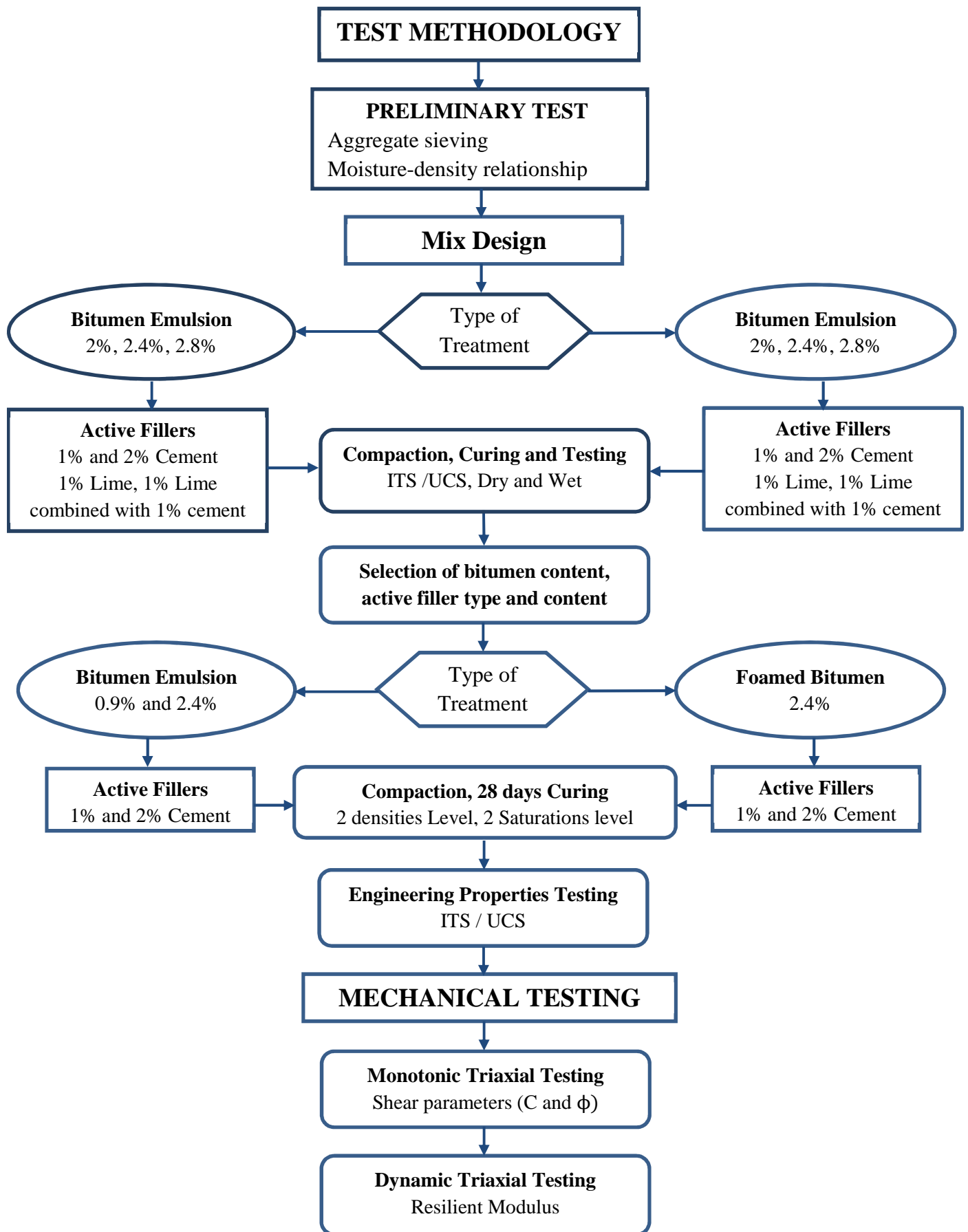
Figure 3. 6 : Research testing methodology diagram

3.5 Testing methodology and data processing

The testing methodology consisted of the following steps:

- Material preparation;

- Mixing of the material with water, active fillers and binder;

- Compaction of the specimens;

- Curing of the specimen compacted;

- Conditioning of the specimen when required;

- Testing of the specimens;

- Determination of moisture content and dry density at testing; and

- Determination of relative densities and saturation level at testing

3.5.1 Material and Specimen Preparation

3.5.1.1 Aggregates preparation

In compliance with the research requirements, all aggregate particles greater the 19 mm were put aside and only aggregates with a maximum size 19mm were used to prepare the mixes. Required sample sizes were obtained by riffling the material from the bags.

Figure 3. 7: Separation of aggregates in samples of representative grading

a) *Maximum dry Densities and Optimum moisture contents*

During this research, three dry densities and two moulding moisture content were used in compliance with the research project requirement. During the first phase (mix design), specimen were compacted to a target density of 2100 kg/m^3 at a moulding moisture content of 11%. During the second phase, triaxial specimens were compacted at a moulding moisture content of 11.2% to targets densities of 2050 kg/m^3 and 2160 kg/m^3.

Material	Moulding Moisture Content (%)	Density Target (kg/m^3)
Mix Design	11	2100
Full Testing	11.2	2050
	11.2	2160

Table 3. 5: Moulding moisture content and target densities

The dry mass of aggregate sample taken to the mixer for BSM-emulsion and BSM-foam specimens manufacturing were respectively calculated using Equations 3.1 and Equations 3.2 .

$$\boldsymbol{M_S = (MDD * V_{spec})/(1 + W_{hyg}/100 + RBC/100 + AFC/100)} \qquad \text{Equation 3.1}$$

$$\boldsymbol{M_S = (MDD * V_{spec})/(1 + W_{hyg}/100 + AFC/100)} \qquad \text{Equation 3.2}$$

Where: Ms   = mass of air dry aggregate to be taken to the mixer in kg

       MDD = maximum dry density of the material in kg/m^3

       Vspec = volume of the specimen in m^3

       W$_{hyg}$ = hygroscopic moisture content in % of dry mass of aggregate

       RBC   = residual bitumen content in % of dry mass of aggregate

       AFC   = filler content in % by mass of aggregate

3.5.2   Mixing and Compaction

3.5.2.1   The optimum mixing moisture content

The optimum mixing moisture content (OMMC) is different from the standard OMC for BSM-emulsion mixes. In this case, it is calculated as the sum of moisture in the aggregate, plus the bitumen emulsion water content and the residual emulsion binder content. Because water and bitumen emulsion act as lubricants in BSM-emulsion mixes, the OMMC was then determined for each mix, using the following equation:

$$OMMC = W_{agg} + EWC + RBC \qquad \text{Equation 3.3}$$

Where: OMMC = optimum mixing moisture content in percentage by mass of dry aggregate

W_{agg} = moisture content of the aggregate in percentage by mass of dry aggregate;

EWC = bitumen emulsion water content in grams in percentage by mass of dry aggregate;

RBC = residual bitumen content in percentage of dry mass of dry aggregate.

In the case of BSM-foam, the total OMMC is usually taken as 60 to 85% of the optimum moisture content of the dry aggregate. More often, it also corresponds to the fluff point moisture content at which the maximum bulk volume of aggregate is obtained (TG 2, 2009). The OMMC is greatly influenced by the grading of the material, especially the amount of fine fraction (smaller than 0.075 mm).

3.5.2.2 *Foamed bitumen characterisation*

During foaming, the temperature of bitumen and the percentage of foaming water were maintained at values of 155^0 C to 165^0 and 2% respectively. The foaming properties of the bitumen were measured during each production of foamed bitumen mixes. In evaluating foam characteristics, the values of the expansion ratio and half-life were in the range of 12 to 14 and 8 to 10 seconds respectively throughout this research. This gives an indication on the quality of the foamed bitumen's produced. The foam index parameter as proposed by Jenkins et al (2000) was not used for foam characterisation in this study.

3.5.2.3 *Aggregate mixing temperature*

Jenkins (2000) indicated that the temperature of the aggregates before mixing has a significant influence on the equilibrium binder-mix temperature. The aggregate should be warm enough to avoid undesirable effects such as poor collapsible rate of foam due to the transfer of heat from the foam at over 100^0C to cold aggregates. In the case of this research, the aggregate samples to be treated were placed in the oven overnight at a temperature of 25^0C the day preceding mixing, and removed the following morning just at the time of mixing with the binder. This was done in compliance with the research project specifications.

3.5.2.4 *Mixing*

All BSM-foam mixes were made using the Wirtgen WLB-10 S laboratory plant shown in Figure 3.8. The calibrations of the foam plant were done for each and every mix produced. During the calibration the percentage of water and bitumen rate was checked and adjusted if necessary. A percentage of

water of 2% (20 grams in 10 seconds) per mass was used at a bitumen rate of 500 gram in 5 seconds. Also the expansion ratio and half-life of each mixes was checked, measured and recorded.

Figure 3. 8: Laboratory foam plant WLB 10 S with WLM 30 mixer used for foam production

Each amount of material and binder required to produce samples were mixed at high speed in mechanical mixer (vertical shaft drum mixer for BSM-emulsion and twin shaft mixer WLM 30 for BSM-foam as shown in Figure 3.9) at the moisture content indicated earlier with the addition of active fillers. For both BSM-emulsion and BSM-foam, the procedure for mixing was as follows:

- Determine the hygroscopic moisture content of the material a day before the mixing by means of standard oven drying method (TMH 1)

$$W_{hyg} = \left[(M_{wet} - M_{dry})/M_{dry} \right] * 100 \qquad \text{Equation 3.4}$$

Where: W_{hyg} = hygroscopic moisture content of active filler % by mass;

M_{wet} = wet mass of the sample of material in grams

M_{dry} = dry mass of the sample of material in grams

- Determine the total amount of water to be added for optimum mixing purpose

$$W_{add} = W_{omc} - W_{hyg} \qquad \text{Equation 3.5}$$

$$M_{water} = (M_S + M_{AF}) * (W_{add}/100) \qquad \text{Equation 3.6}$$

Where: W_{add} = percentage of water to be added to the material in % by mass;

W_{omc} = optimum moisture content in % by mass;

W_{hyg} = hygroscopic moisture content in % by mass;

M_{water} = total mass of water to be added in grams;

M_S = mass of air dry aggregate in grams;

M_{AF} = mass of active filler in grams.

- Introduce into the mixer the required amount of aggregate determined from Equation 3.1 and Equations 3.2

Note: An extra 1500 grams of material was added to the calculated final mass of each specimen in order to perform moisture content check afterward.

- Add the required amount of active filler, determined as a percentage per mass of material;

$$M_{AF} = M_S * (AFC/100)$$ Equation 3.7

Where: M_{AF} = mass of active filler in grams;

Ms = dry mass of the sample in grams

AFC = active filler content in % of dry mass of aggregate

- Mix aggregate with active fillers;
- Add while mixing the optimum mixing moisture determined using Equation 3.3

$$M_{ommc} = (M_S + M_{AF}) * \left(\frac{W_{ommc}}{100}\right)$$ Equation 3.8

- Add while mixing the required amount of binder, also determined as a percentage of dry mass of aggregate.

$$M_{BC} = M_S * (BC/100)$$ Equation 3.9

Where: M_{BC} = mass of bitumen in grams;

Ms = dry mass of the sample in grams

BC = bitumen content in % of dry mass of aggregate

Note: for BSM-emulsion, the bitumen content is the residual bitumen content including water used for the dissolution.

- Add while mixing the compaction moisture content determined as follow:

$$M_{CMC} = M_{weter} - M_{OMMC}$$ Equation 3.10

Where: M_{CMC} = mass of the compaction moisture content;

M_{weter} = total mass of water to be added in grams;

M_{OMMC} = mass of the optimum mixing moisture in grams.

- Mix aggregate, cement, binder and water (see Figure 3.9)

After mixing, a minimum sample of 600 g is taken from the mixer and dried to a constant mass to determine the actual moulding moisture content using Equation 3.4

(a)   (b)

Figure 3. 9: (a) twin shaft mixer WLM 30 (BSM-foam) – (b) Vertical shaft drum mixer (BSM-emulsion)

A visual inspection was done at all stages to ensure that the mixing is done well. In the case of BSM-foam for instance, a mix of poor quality could be quickly identified by noticing the presence of bitumen concentration or clumps, exposed to elongation during mixing. In this of this research only mixes of good quality were used to compact the specimens.

3.5.2.5   *Compaction*

The performance of a constructed pavement layer mainly depends on the compaction quality applied during construction. As said in Section 2.3.3, laboratory compaction techniques are intended to simulate the in-place density of the mix after it has endured a couple of years of traffic. Several compaction procedures can be used on bitumen stabilized materials (i.e. Impact compactions, Kneading compaction, Gyratory compaction, Vibratory compaction). In South Africa, the most used methods for the compaction of BSM's are gyratory and vibratory compaction. In the case of this study, the vibratory Bosch hammer was used for the compaction of all specimens because of its ability to achieve density expected on the field and to emulate particles orientation after rolling (TG2 guidelines).

In compliance with the research project specification, ITS and UCS specimens were compacted in two layers, while triaxial specimens were compacted in five layers to the required height. After

compaction, the top of each layer except the last layer was scarified to an approximate depth of 10 mm before the material for the next layer is added. This was done in order to have continuity in bounding of the different layers.

After mixing the material with active filler, binder and water, the steps listed below were followed for the compaction of all specimen tested throughout this research:

- Cleaning and lubrication of the inner surfaces of the mould;
- Measurement of the amount of material required for each layer and sealing in plastic bags before compaction to avoid any moisture evaporation;
- Consecutive compaction of the different specimen layers;
- For triaxial specimens and especially specimens compacted at higher densities, the compaction of the 5th layer was done after adding an extension to the mould;
- Removal of the compacted specimen from the split mould with care to avoid any distortion;
- Measurement of the exact mass of the specimen;
- Measurement and recording of the height and diameter of the specimen at 3 positions of 120^{0} offset around the circumference of the specimen.

The specimen dimensions were as required by the terms of reference of the project 150 mm diameter by 75 mm height for ITS specimens, 150 mm diameter by 127 mm height for UCS specimens and 150 mm diameter by 300 mm height for ITS specimens

| (a) | (b) | (c) |

Figure 3. 10: (a) ITS specimens - (b) UCS specimens - (c) Triaxial specimens

3.5.3   Curing

During the mix design phase, specimens for ITS and UCS testing were cured following the procedure shown in Figure 3.12.

- Place the specimens in the oven unsealed for 26 hours (bitumen emulsion treated specimens) and for 20 hours (foamed bitumen treated specimens) at 30^0 C;
- Remove the specimen from the oven after the required time, seal them in plastic bags and place then back in the oven for 48 hours at 40^0 C;
- Cool down the specimens to the temperature of 25^0 C before testing.

(a)   (b)

Figure 3. 11: (a) Specimens unsealed at 30^0 C, (b) Specimens sealed at 40^0 C
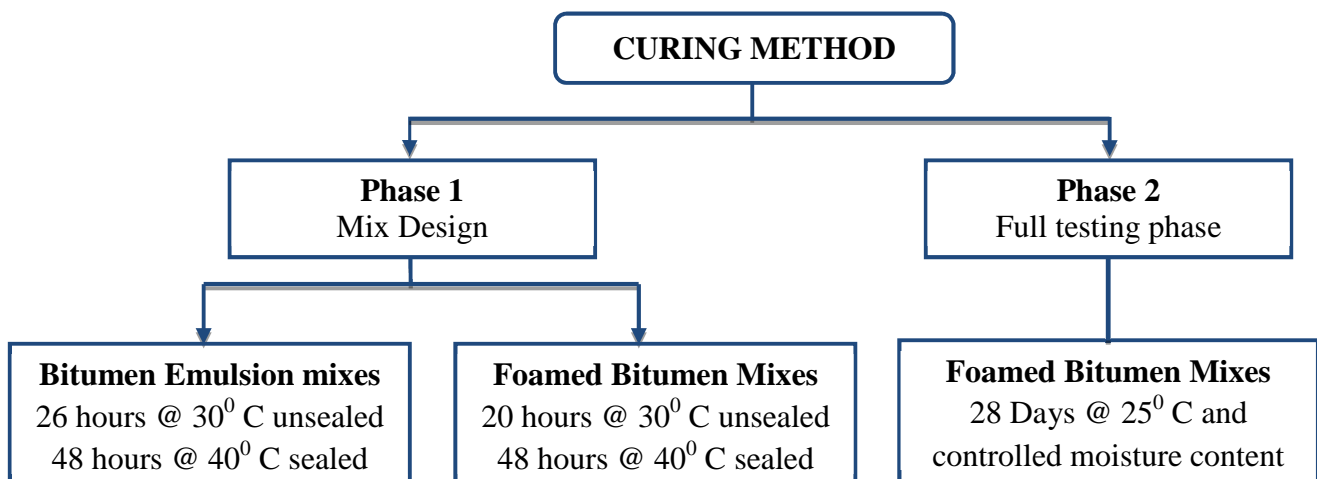

Figure 3. 12: Summary of the Curing protocol followed during the two phases of testing

It is however well known that bitumen stabilized mixes are sensitive to moisture conditions. For this reason, during the mix design phase, 3 ITS and UCS specimens were tested at equilibrium moisture content and 3 other in wet conditions. After the normal curing procedure, the specimen tested in wet

condition were soaked and conditioned in water for 24 hours at 25^0C before testing (see Figure 3.13). This was done in order to, firstly evaluate the moisture susceptibility of the mixes and secondly to investigate the effect of moisture on the flexibility of BSMs.

Figure 3. 13: Immersion of specimen in waterbath at 25^0C for ITS$_{wet}$ and UCS$_{wet}$ testing

During the second phase, long term curing was adopted in compliance with the research project requirements. Here specimens were all cured for 28 days at a control temperature of 25^0 C. The 28 days curing from the time of compaction included the followings phases:

- Store the specimen at room temperature;
- In compliance with the research project requirements, triaxial specimen were tested at target moisture content of 2.5% , 3%, 9.5% and 11.5%. To achieve this, specimens were first of all constantly weigh until it dries back to the target mass at of 2.5% moisture content, determined by Equation 3.11 and Equation 3.12 for BSM-emulsion emulsion and BSM-foam respectively.

$$M_{dry-back} = (Ms + M_{AF} + M_{Bit} + 2.5/100 * (Ms + M_{AF})) \qquad \text{Equation 3.11}$$

$$M_{dry-back} = (Ms + M_{AF} + M_{Bit} + 2.5/100 * (Ms + M_{AF})) \qquad \text{Equation 3.12}$$

Where: M$_{dry-back}$ = target mass of the specimen at 2.5% moisture content in grams;

      Ms   = dry mass of the sample in grams;

      M$_{Bit}$ = mass of the residual bitumen in grams;

      M$_{AF}$ = mass of active filler in grams;

Once the target mass of 2.5% moisture content was reached, the specimens were sealed and cured for the remaining time of the 28 days curing.

In case the dry-back mass of the specimen was less than the target mass at testing as determined by Equation 3.13 and Equation 3.14 for BSM-emulsion and BSM-foam respectively, water was sprayed onto the specimen as shown in Figure3.14 until its weight is equal to the target mass for testing.

$$M_{testing} = (Ms + M_{AF} + M_{Bit} * \frac{W_{testing}}{100} * (Ms + M_{AF})) \qquad \text{Equation 3.13}$$

$$M_{testing} = (Ms + M_{AF} + M_{Bit} + \frac{W_{testing}}{100} * (Ms + M_{AF})) \qquad \text{Equation 3.14}$$

Where: $M_{testing}$ = target mass of the specimen at testing in grams;

$\quad$ Ms $\quad$ = dry mass of the sample in grams;

$\quad M_{Bit}$ $\quad$ = mass of the residual bitumen in grams;

$\quad M_{AF}$ $\quad$ = mass of active filler in grams;

$\quad W_{testing}$ $\quad$ = target moisture content at testing.

3.5.4 Material Testing

3.5.4.1 ITS and UCS test

ITS and UCS tests were conducted in this research both at the mix design phase and during the full testing phase. They are both engineering tests for they give an indication of the engineering properties of the material. The indirect tensile strength and the unconfined compressive strength were both determined by measuring the ultimate applied load on the specimen before it fails (see Figure 3.14).

Figure 3. 14: ITS testing (left) and UCS testing (right)

3.5.4.2   Triaxial Testing

Phase 2 consisted mainly of triaxial testing:

- Monotonic triaxial test; to determine the shear parameters i.e. cohesion (C) and angle of internal friction (φ) of the different mixes; and
- Short term dynamic triaxial test to determine the resilient modulus (Mr);

a) The triaxial test apparatus

All triaxial tests were done at the University of Stellenbosch (Engineering laboratory). The testing device used was the hydraulic testing system (MTS 810- MODEL 318.10). It is a closed loop servo-hydraulic testing press system placed in a chamber where the temperature can be regulated from 0 to 60^0C if desired. It's equipped with an actuator of 10 metric tonnes, having a displacement range of 80 mm stroke (80 mm up and 80 mm down). The entire system is controlled and operated by the MTS Flextext 40 Digital Controller. This controller was newly acquired by Stellenbosch University and has the following advantages in comparison with the MTS Controller 407, which was previously used.

- Offers high speed and more channels to keep step with growing test demands;
- Is compatible with a full array of MTS application software for material testing;
- Processes centralized processors and test resource boards that can be easily upgraded.
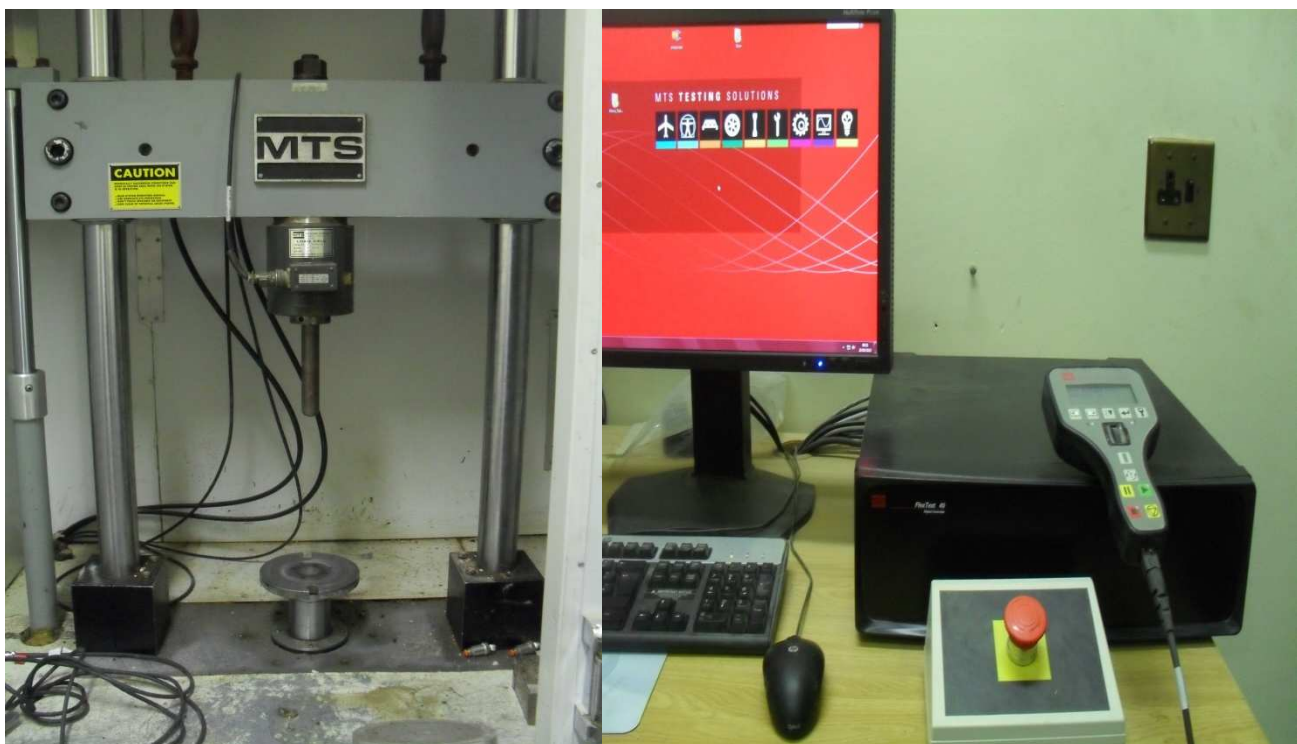

Figure 3. 15: Triaxial testing device (left); Flextext 40 Digital Controller and Computer (right)

The program used to perform triaxial testing was the MTS's Multiperpose TestWare, known for its great ability to meet demands of quick changing of test requirements. The load and displacement data from the test were separately captured by the software and saved in Excel format in a directory folder specified when writing the test algorithm.

b) Monotonic Triaxial Test

The monotonic triaxial test is a widely used test to determine the shear strength parameters (i.e. Cohesion C and internal angle of friction φ) of soils. Forty-eight monotonic triaxial tests were performed in total i.e. 12 tests for each of the four mixes.

All triaxial specimens were cured for twenty-eight days and tested at a temperature of 25^0C. Therefore, specimens were not conditioned after curing before testing. In compliance with the research project requirements, 3 specimens were compacted per mix and tested at the cell pressures of:

- 20 kPa;
- 110 kPa; and
- 200 kPa.

Is it important to mention that standard mix design of BSMs includes ten triaxial tests per mix (e.g. at BSM laboratories). It is unusual for research of BSM mixes to include only three equivalent tests, but material resources did not allow the desired additional tests to be carried out.

The tests were done at a displacement-controlled mode with a rate of 2.1% strain per minute. One specimen was tested at each confinement and the test procedure consisted of:

- Weigh the specimen to be tested and compare the weight with the target weight at testing determined from Equation 3.13 and Equation 3.14;
- Place a rubber membrane around the specimen and seal it with O rings rubber to avoid any air infiltration due to the confinement during the test;
- Put the specimen in the triaxial and make sure it is well centred;
- Close the cell and place it on the actuator in the MTS chamber;
- Introduce air in the cell until the required confinement pressure is reached;
- Take the load cell down until it comes in contact with the piston that pushes on the plate on top of the specimen and make sure it is well centred;
- The hydraulic ram is then locked, the pressure set high and the test in run;

Immediately after the test is completed, the specimen is removed from the tri-axil cell, broken, and a portion of it is taken back to the soil lab. This portion is weighed and dried in the oven to a constant mass for moisture content calculation.

c) *Short term Dynamic Triaxial Test (Mr)*

This test was done to determine the resilient modulus by measuring very small displacements over the middle part of the specimen. For each of the four mixes, twelve specimens were subjected to short term dynamic triaxial testing at two relative densities, two saturation levels, three confinement pressures, and five stress ratios as listed in Table 3.4 for each of the four mixes.

The MTS is equipped with a built-in Linear Variable Displacement Transducer (LVDT), which measures the vertical displacement of the specimen under loading during the test. However, its displacement measurements are not adequate for resilient modulus and permanent deformation calculations, which used displacements in the middle third and not over the whole specimen. Therefore, three LVDT's with a range of $\pm$ 5 mm each were fitted at three positions around the circumference of the specimen at an angle of 120^0 offset from one another as shown in Figure 3.16. Three LVDT are used, because with only two, if the specimen is moved to the side during the test (eccentricity effect), reading will still look the same but will not be correct.
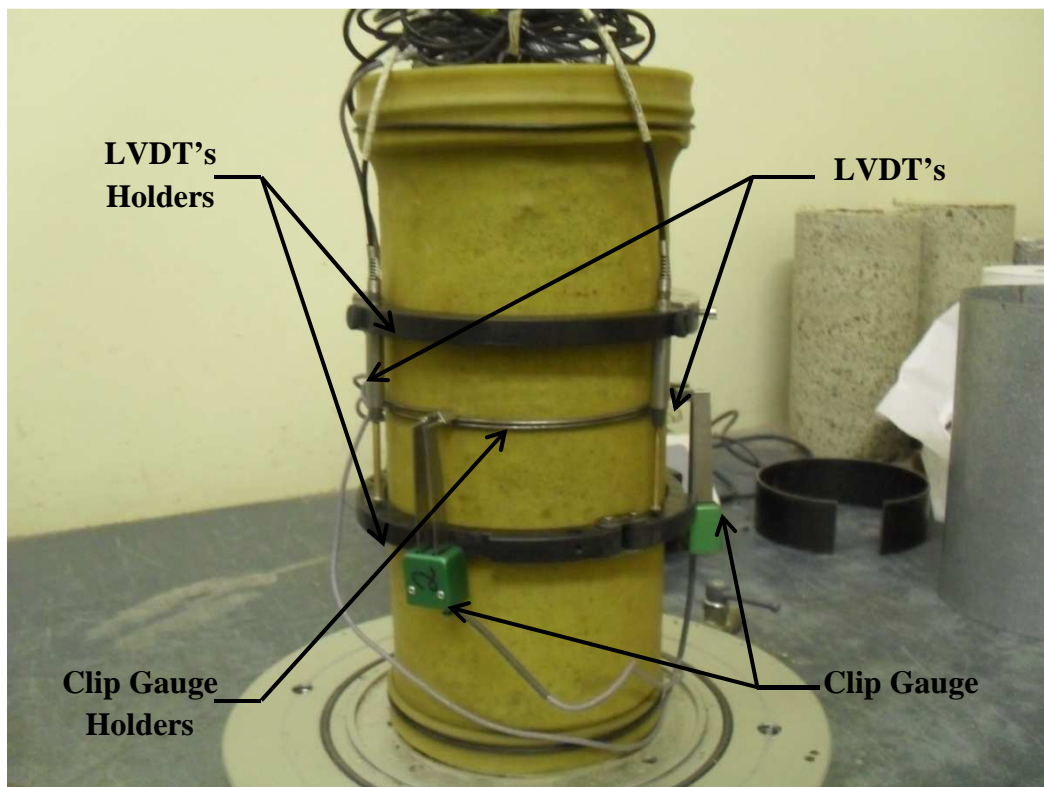

Figure 3. 16: Specimen setting for short term dynamic triaxial testing

Radial expansions were also measured during the short term dynamic triaxial testing. This was done by the means of two clip-gauges placed on rings around the specimen at approximately 150 mm from the top and the bottom of the specimen as shown in Figure3.16. This was done in order to take into account the dilation effect during calculations.

The short-term dynamic triaxial test was also done at 25^0C. A total number of 100 loading cycles were done for each stress ratio at a data-capturing rate of 512 Hz. However, only the first five and the last five cycles were recorded and used to calculate the resilient modulus. The loading pulse shape was an haversine type consisting of:

- A loading phase of 0.05 seconds;
- A unloading phase of 0.05 seconds; and
- A rest period of 0.9 seconds.

A seating load of 0.4 kN was constantly applied on the specimen during the test (see Figure 3.17).
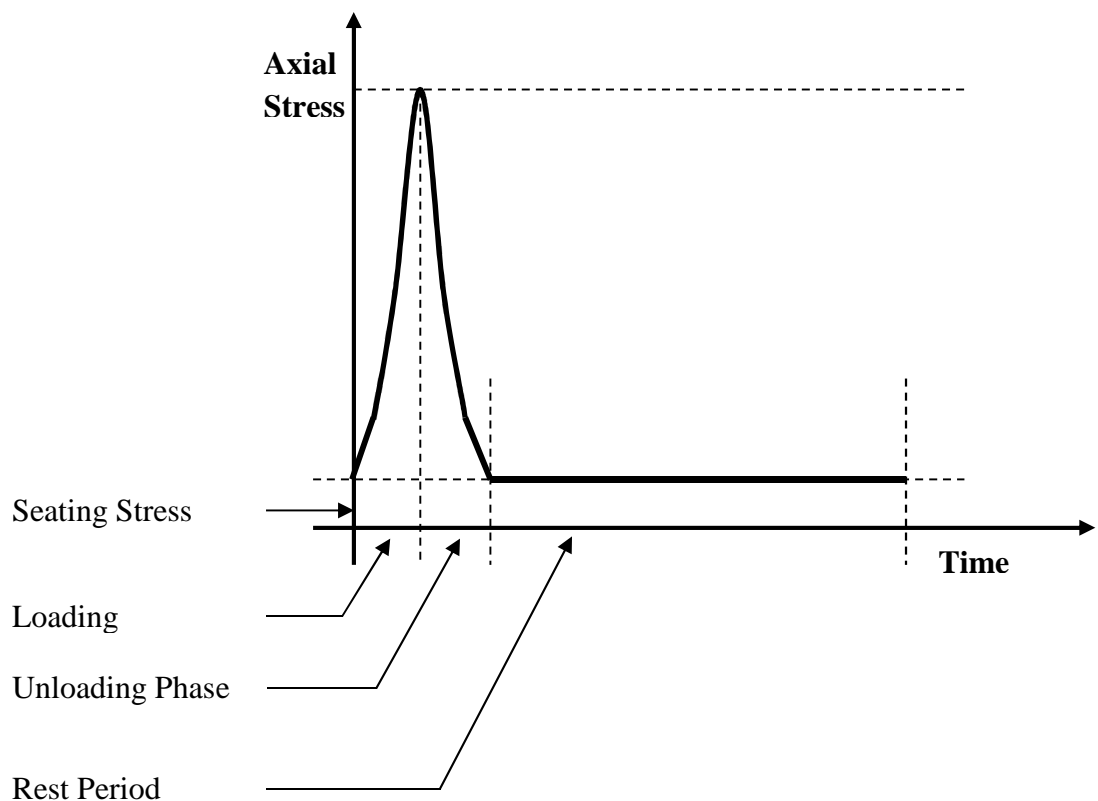

Figure 3. 17: Load-pulse for the resilient response triaxial test

The short-term dynamic triaxial test was conducted in a force-controlled mode. For each confinement and stress ratio, the load to be applied to the specimen was determined from the deviator stress ratio obtained from the shear parameters of the monotonic triaxial tests.

The protocol for the test consisted of an initial phase called conditioning, followed by a phase of specified stress regimes. During conditioning, 300 load cycles were applied to the specimens at 200 kPa and at three stress ratios: 20%, 50% and 70% of the failure stress from the monotonic test (i.e. 100 load cycles per stress ratio). The stress regimes phase consisted of a series of loading sequences done at five different confinement presures (200, 150, 100, 50 and 20 kPa) starting from the highest to the lowest. For each confinement of the loading sequence, the deviator stress is increased to induce stress ratios of 20, 40, 50, 60 and 70% of the failure stress at that particular confinement calibrated in advance from the monotonic triaxial test.

The test procedure consisted of:

- Identify the mix involved and its shearing parameters (C and φ)
- Determination of the failure stress at each confinement pressure using the shearing parameters from the monotonic triaxial test (see Table 3.6);

$$\sigma_d^f = \sigma_3 * \tan^2(\frac{\varphi}{2}) + 2C \tan\left(45 + \frac{\varphi}{2}\right)$$ Equation 3.15

Where: σ_d^f = Failure stress in kPa,

σ_3 = Confinement pressure in kPa

φ = internal angle of friction in degrees

C = cohesion in kPa

- Determination of the maximum load to be applied to the specimen during the test as percentages of the failure stress determined earlier (see Table 3.6);

$$L = SR * \sigma_d^f * \pi * r^2 \ /100$$ Equation 3.16

Where: L = maximum load in kN

SR = stress ratio in percentage of the failure stress

σ_d^f = Failure stress in kPa,

r = the radius of the specimen in meters

- Determination of the cyclic load (see Table 3.6);

$$CL = L - SL$$ Equation 3.17

Where: CL = actual cyclic load in kN

L = cyclic load in kN

SL = Seating load in kN

- The specimen to be tested is weighed and centrally placed on the base plate of the triaxial cell;
- A rubber membrane is placed around the specimen and around the top and bottom plate at the edges of the specimen. This is done carefully in order not to damage the edges of the specimen;
- Two O-rings are placed around the bottom and the cap plate to seal the membrane;
- The LVDT's frames are placed around the middle part of the specimen and held with elastic bands to ensure stability and good contact with the LVDT's throughout the test;
- Two clip gauge frames are placed in the middle part of the specimen,
- The 3 LVDTs and the two clip gauges are mounted on their respective holders;
- The specimen is put in the cell with care;
- The cell is then closed and placed in the chamber;
- The air is introduced into the cell until the required confinement. Listening and checking are done at this stage to ensure there is no leakage of air;
- The actuator is lowered until it encounters the piston on top of the specimen, but without loading the specimen. A visual inspection is done at this stage to ensure a perfect alignment of the piston of the actuator with the piston of the triaxial chamber;
- The hydraulic pressure is then locked and the pressure set high.
- All LVDTs and clip gauge channels are set to zero and the test is run.

Figure 3.19 shows an example of a prepared specimen placed in the triaxial cell and ready for testing

Figure 3. 18: Triaxial specimen in the cell, set and placed in the chamber, ready for testing

Table 3. 6: Example of loading schedule for the resilient modulus test

Cycles	Phase	Confinement pressure (kPa)	Stress Ratio	Deviator stress (A)	Load (kN) (B)	Seating load (kN)	Cyclic load (kN) (C)	Cycles to record
0 – 100			20%	$0.2*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	1 – 5, 96 – 100
101 – 200	Cond	200	50%	$0.5*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	101 – 105, 196 – 200
201 – 300			70%	$0.7*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	201 – 205, 296 – 300
301 – 400			20%	$0.2*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	301 – 305, 396 – 400
401 – 500			40%	$0.4*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	401 – 405, 496 – 500
501 – 600	1	200	50%	$0.5*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	501 – 505, 596 – 600
601 – 700			60%	$0.6*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	601 – 605, 696 – 700
701 – 800			70%	$0.7*\sigma_{df}(200)$	$A*\pi r^2$	0.4	B-0.4	701 – 705, 796 – 800
801 – 900			20%	$0.2*\sigma_{df}(150)$	$A*\pi r^2$	0.4	B-0.4	801 – 805, 896 – 900
901 – 500			40%	$0.4*\sigma_{df}(150)$	$A*\pi r^2$	0.4	B-0.4	901 – 905, 996 – 1000
1001 – 1100	2	150	50%	$0.5*\sigma_{df}(150)$	$A*\pi r^2$	0.4	B-0.4	1001 – 1005, 1096 – 1100
1101 – 1200			60%	$0.6*\sigma_{df}(150)$	$A*\pi r^2$	0.4	B-0.4	1101 – 1105, 1196 – 1200
1201 – 1300			70%	$0.7*\sigma_{df}(150)$	$A*\pi r^2$	0.4	B-0.4	1201 – 1205, 1296 – 1300
1301 – 1400			20%	$0.2*\sigma_{df}(100)$	$A*\pi r^2$	0.4	B-0.4	1301 – 1305, 1396 – 1400
1401 – 1500			40%	$0.4*\sigma_{df}(100)$	$A*\pi r^2$	0.4	B-0.4	1401 – 1405, 1496 – 1500
1501 – 1600	3	100	50%	$0.5*\sigma_{df}(100)$	$A*\pi r^2$	0.4	B-0.4	1501 – 1505, 1596 – 1600
1601 – 1700			60%	$0.6*\sigma_{df}(100)$	$A*\pi r^2$	0.4	B-0.4	1601 – 1605, 1696 – 1700
1701 – 1800			70%	$0.7*\sigma_{df}(100)$	$A*\pi r^2$	0.4	B-0.4	1701 – 1705, 1796 – 1800
1801 – 1900			20%	$0.2*\sigma_{df}(50)$	$A*\pi r^2$	0.4	B-0.4	1801 – 1805, 1896 – 1900
1901 – 2000			40%	$0.4*\sigma_{df}(50)$	$A*\pi r^2$	0.4	B-0.4	1901 – 1905, 1996 – 2000
2001 – 2100	4	50	50%	$0.5*\sigma_{df}(50)$	$A*\pi r^2$	0.4	B-0.4	2001 – 2005, 2096 – 2100
2101 – 2200			60%	$0.6*\sigma_{df}(50)$	$A*\pi r^2$	0.4	B-0.4	2101 – 2105, 2196 – 2200
2201 – 2300			70%	$0.7*\sigma_{df}(50)$	$A*\pi r^2$	0.4	B-0.4	2201 – 2205, 2296 – 2300
2301 – 2400			20%	$0.2*\sigma_{df}(20)$	$A*\pi r^2$	0.4	B-0.4	2301 – 2305, 2396 – 2400
2401 – 2500			40%	$0.4*\sigma_{df}(20)$	$A*\pi r^2$	0.4	B-0.4	2401 – 2405, 2496 – 2500
2501 – 2600	5	20	50%	$0.5*\sigma_{df}(20)$	$A*\pi r^2$	0.4	B-0.4	2501 – 2505, 2596 – 2600
2601 – 2700			60%	$0.6*\sigma_{df}(20)$	$A*\pi r^2$	0.4	B-0.4	2601 – 2605, 2696 – 2700
2701 – 2800			70%	$0.7*\sigma_{df}(20)$	$A*\pi r^2$	0.4	B-0.4	2701 – 2705, 2796 – 2800

3.5.5 Data Processing

This section gives a detailed description on how data from different tests were computed to determine material properties. In addition to material properties, other test variables such as relative density, and saturation level were determined for each specimen tested during this study.

$$RD = [(DD_{mix})/(ARD_{mix} * D_w)] * 100 \qquad \text{Equation 3.18}$$

$$S = \left[\frac{(MC/100) * DD_{mix}}{(1 - RD/100) * D_w} \right] * 100 \qquad \text{Equation 3.19}$$

Where: RD   = relative density in %

  DD_{mix} = dry density of the mix

  ARD_{mix} = apparent relative density of the mix including cement aggregate and bitumen

  D_w = density of water in kg/m^3

  S   = Level of Saturation

  MC   = Moisture Content in %

The apparent relative densities of the mix components used to calculate the apparent relative density of the mixes are summarised in Table 3.7

Table 3. 7: Apparent relative density values

Apparent Relative density of the Mix Components (ton/m^3)				
Aggregate	2.917			
Bitumen	1.01			
Cement	3.15			
Lime	1.02			
Apparent relative density of the mixes (ton/m^3)				
		Bitumen Content		
		2	2.4	2.8
Active filler content	1% Cement	2.815	2.795	2.776
	2% Cement	2.817	2.797	2.778
	1% Lime	2.765	2.746	2.728
	1% Lime + 1% Cement	2.792	2.773	2.754

3.5.5.1 ITS and UCS

ITS and UCS specimens of 150 mm diameter and 127 and 75 mm height respectively were tested at equilibrium and soaked conditions to determine the ITS$_{DRY}$, ITS$_{WET}$, UCS$_{DRY}$, and UCS$_{WET}$ values. The ITS$_{DRY}$ and UCS$_{DRY}$ values were determined by testing the specimen of 25^0 C after curing while the ITS$_{WET}$ and UCS$_{WET}$ values were determined after placing the specimen under water for 24 hours at 25^0 C after curing before testing.

The ITS values were calculated as followed:

$$ITS = (2 * P)/\pi * h * d) * 10000 \qquad \text{Equation 3.20}$$

Where: ITS = indirect tensile strength in kPa,

P   = maximum applied load in kN,

h   = average height of the specimen

d   = average diameter of the specimen in cm

The UCS values were calculated as followed:

$$UCS = (4 * P)/(\pi * d^2) * 10000 \qquad \text{Equation 3.21}$$

Where: UCS = Unconfined Compressive Strength in kPa

P   = maximum applied load at failure in kN

d   = average diameter of the specimen in cm

Three repeat specimens were tested per mix variable and average taken as ITS/UCS value of the mix.

$$ITS_{EQUIL} = \sum_{i=1}^{3} ITS_{EQUIL\,i} \Big/ 3 \qquad \text{Equation 3.22}$$

And

$$ITS_{SOAK} = \sum_{i=1}^{3} ITS_{SOAK\,i} \Big/ 3 \qquad \text{Equation 3.23}$$

Another important mix property that was determined at this level of testing was the Tensile Strength Ratio (TSR), which is the relationship between the average soak and dry ITS for specific mixes expressed in percentage.

$$TSR = ITS_{SOAK}/ITS_{EQUIL} * 100 \qquad \text{Equation 3.24}$$

3.5.5.2   *Displacement at break*

The displacement at break was determined as the maximum displacement value of the specimen at failure. It was determined from stress-displacement graphs of ITS and UCS tests.

In many cases, the load displacement curve does not start at the origin and does not have a consistent slope. Therefore, a slope that takes the major part of the curve was plotted manually on the graph and

its intercept with the horizontal axis was taken as the displacement starting point as shown on the Figure 3.19.
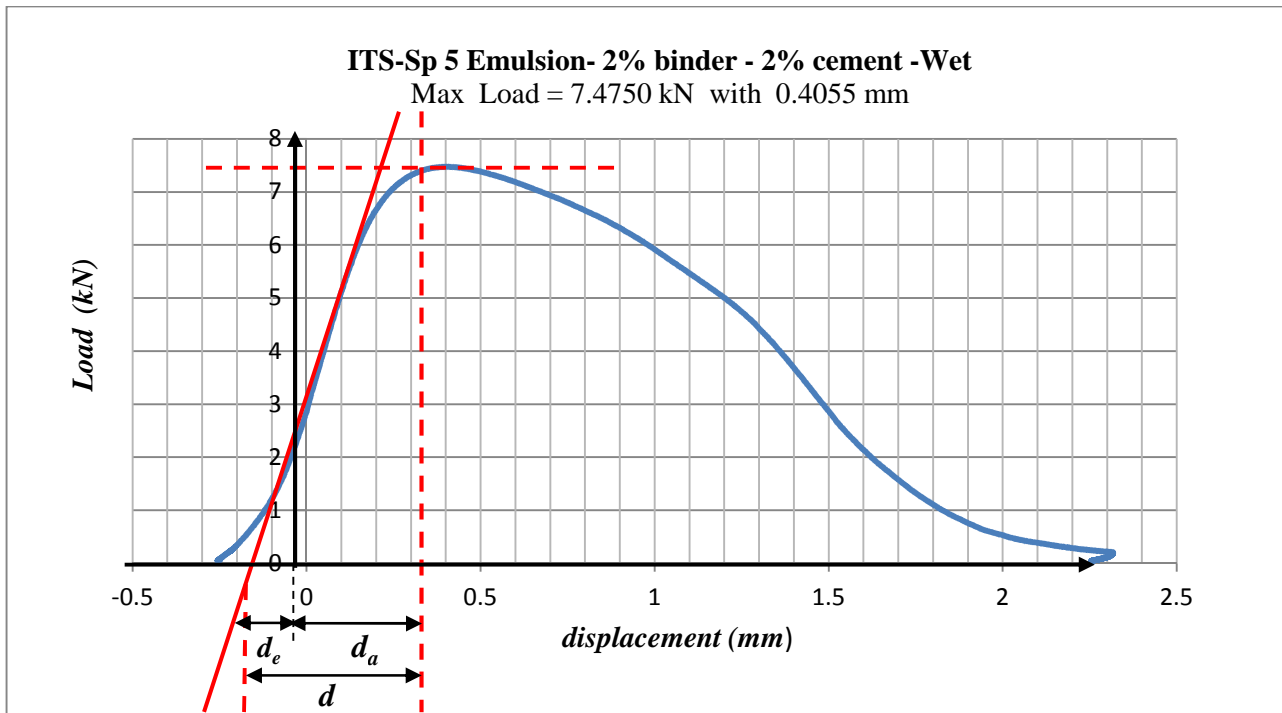
Figure 3. 19: Determination of the actual displacement at failure

The displacement at failure was then determined by adding to or subtracting from the apparent replacement obtained at the maximum load, the distance between the origin and the intersection of the slope with the axis of abscises; depending if the intercept falls in the positive or negative part of the axe.

$$d = d_a - d_e$$ Equation 3.25

Where: d = actual displacement at failure;

　　　d_a = apparent displacement from the test data; and

　　　d_e = distance from the origin to the intersection of the slope with the axis of abscises

3.5.5.3　Strain at break

The strain was determined as the ratio between the actual displacement at failure and the total height of the specimen. This was done only for UCS tests.

$$\varepsilon_b = d/h$$ Equation3.26

Where: d = actual displacement at failure; and

　　　h = average height of the specimen.

3.5.5.4   Fracture energy

The fracture energy from the ITS and UCS tests was determined from the load-displacement diagram by calculating the area under the load-displacement curve. The Simpson rule method of numerical integration was used in this regard. To achieve this, the load-displacement curve was first shifted so that the displacement will start at zero. This was done in order to simplify the integral calculations. Secondly, a polyline equation that best fits the curve was generated using Excel. Finally, the area under the curve that represents the fracture energy of the material was obtained by integrating the equation of the polynomial curve between 0 and the maximum displacement (see Figure 3.20).
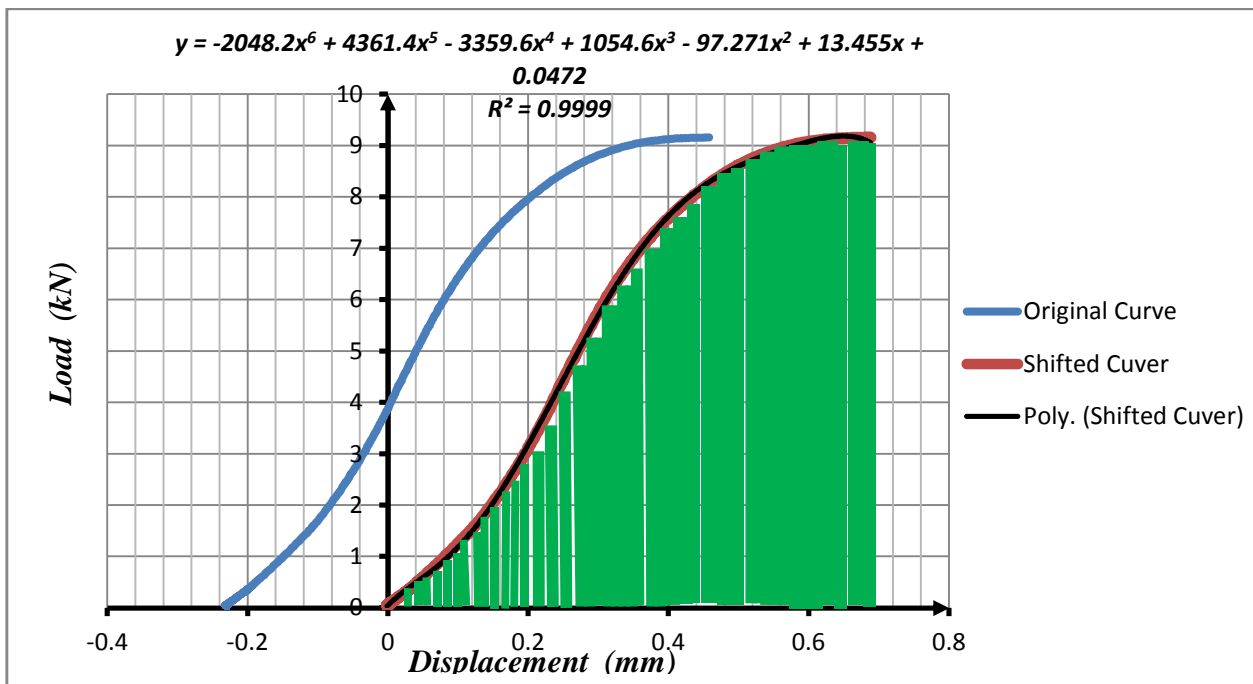
Figure 3. 20: Computation of fracture energy for Sp1-FB2.8-CM2-Dry

3.5.5.5   Monotonic triaxial testing-shearing parameters

Three specimens from each mix were tested at each confinement pressure of 20 kPa, 110 kPa and 200 kPa. Firstly, the maximum load and the corresponding displacement are determined after each test for failure stress and strain calculation.

$$\sigma_f = \frac{P_f}{A} * 10^{-3} \qquad\qquad \text{Equation 3.27}$$

Where σ_f = failure stress in kPa;

P_f = applied failure load in kN; and

A = area of the specimen in m^2

Secondly, the major stress at failure is determined as the stress at failure and the confinement pressure.

$$\boldsymbol{\sigma_{1f} = \sigma_f + \sigma_3} \hspace{4cm} \text{Equation 3.28}$$

Where σ_{1f} = major stress at failure in kPa

σ_f = the axial failure stress in kPa; and

σ_3 = the confinement pressure in kPa.

Thirdly, the shear strength parameters are computed by first determining the relationship between σ_{1f} and σ_3,

$$\boldsymbol{\sigma_{1f} = X.\sigma_3 + Y} \hspace{4cm} \text{Equation 3.29}$$

Where $\quad A = 1 + \sin\phi / 1 - \sin\phi \quad$ and $\quad B = 2\,C * \cos\phi / 1 - \sin\phi$

Finally, a linear regression analysis is done to determine the values of A and B. Once A and B are determined, C and ɸ are calculated as follows:

$$\boldsymbol{\phi = \sin^{-1}\left(\dfrac{X-1}{X+1}\right)} \hspace{3cm} \text{Equation 3.30}$$

$$\boldsymbol{C = \dfrac{Y(1-\sin\phi)}{2*\cos\phi}} \hspace{3cm} \text{Equation 3.31}$$

Where ɸ = the internal angle of friction of the material in degrees; and

C = the cohesion of the material in kPa.

3.5.5.6   *Short term dynamic triaxial testing-resilient modulus*

Data from the test comprises the applied cyclic load, the vertical displacements from the LVDT's and the radial dilatation form the clip gauges. For each of the last five cycles recorded per stress ratio, the Mr is determined from the load-displacement hysteresis loop as shown in Figure 3.22.
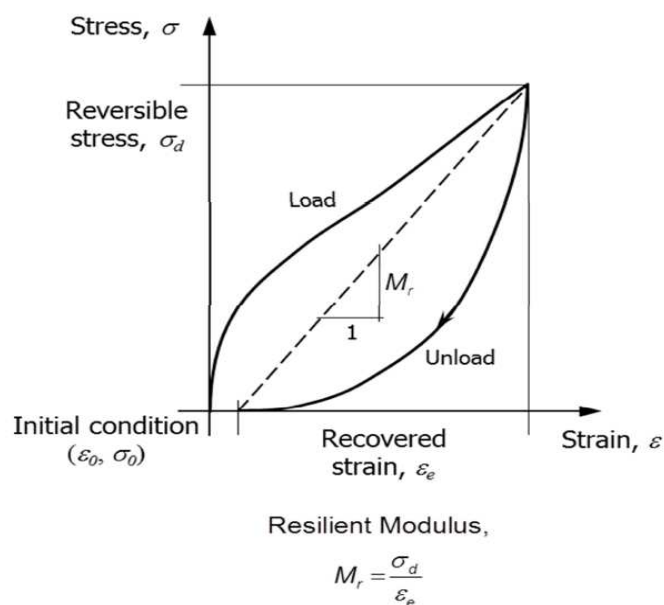
Figure 3. 21: Resilient modulus definition and calculation (Theyse, 2012)

In calculating the resilient modulus, the followings were determined for each of the 5 last cycles per stress ratio combination:

- The maximum and the minimum deformation of each of the three LVDTs to calculate the average axial deformation of the specimen per load cycle as follows:

$$\Delta\varepsilon_{a(N)} = \frac{\sum_{j=1}^{j=3}(LVDT_{j,max} - LVDT_{j,min})}{3}$$ Equation 3.32

Where: $\Delta\varepsilon_{a(N)}$ = average axial deformation per load cycle N in mm;

$LVDT_{j,max}$ = maximum deformation reading on the LVDT j in mm;

$LVDT_{j,min}$ = minimum deformation reading on the LVDT j in mm;

N   = the cycle number; and

J   = the LVDT number.

- The resilient axial strain per cycle,

$$\varepsilon_{a(N)} = \frac{\Delta\varepsilon_{a(N)}}{L_g}$$ Equation 3.33

Where: $\varepsilon_{a(N)}$ = resilient axial strain per load cycle;

$\Delta\varepsilon_{a(N)}$ = average axial deformation per load cycle N in mm; and

L_g = gauge length in mm

Note: in the case of this study, the gauge length, which is the distance between the measuring points of the vertical displacement, was taken as 100 mm for specimen of 300 mm height.

- The cyclic stress per load cycle

$$\sigma_{cyclic(N)} = \frac{L_{cyclic}}{A}$$ Equation 3.34

Where: $\sigma_{cyclic(N)}$ = cyclic stress in kPa at cycle N,

L_{cyclic} = cyclic load in kN;

A   = area of the specimen in m^2; and

N   = cycle number

- The resilient modulus per load cycle

$$M_{r(N)} = \frac{\sigma_{cycle(N)}}{\varepsilon_{a(N)}}$$ Equation 3.35

- The Mr of the of the specimen, as the average of the last five load cycles

$$M_r = \frac{\sum_{i=1}^{i=5} M_{r(N)}}{5}$$ Equation 3.36

CHAPTER 4: MATERIAL TESTING RESULTS, FINDINGS AND DISCUSSION

4.1 Introduction

This section of the report summarises the test results, the findings as well as the discussion and interpretation. ITS, UCS, and triaxial testing results are presented here and the effect of experimental variables (i.e. The type of treatment, the percentage of bitumen and active filler added and the density and saturation level at testing) is discussed. The following mix properties were investigated and interpreted in this research:

- The strength;
- The flexibility;
- The shear properties;
- The stiffness behaviour under dynamic loading

First of all, the ITS and UCS results are presented and the strength of the mixes are discussed. Secondly, the flexibility of the different mix is analysed in terms of strain at break and fracture energy. Thirdly, the shear parameters (cohesion and internal angle of friction) from the monotonic triaxial are analysed. Finally, the chapter ends with the modelling and discussion of resilient modulus results from the short term dynamic triaxial testing. The flow chart diagram of the results presentation is shown in Figure 4.1.
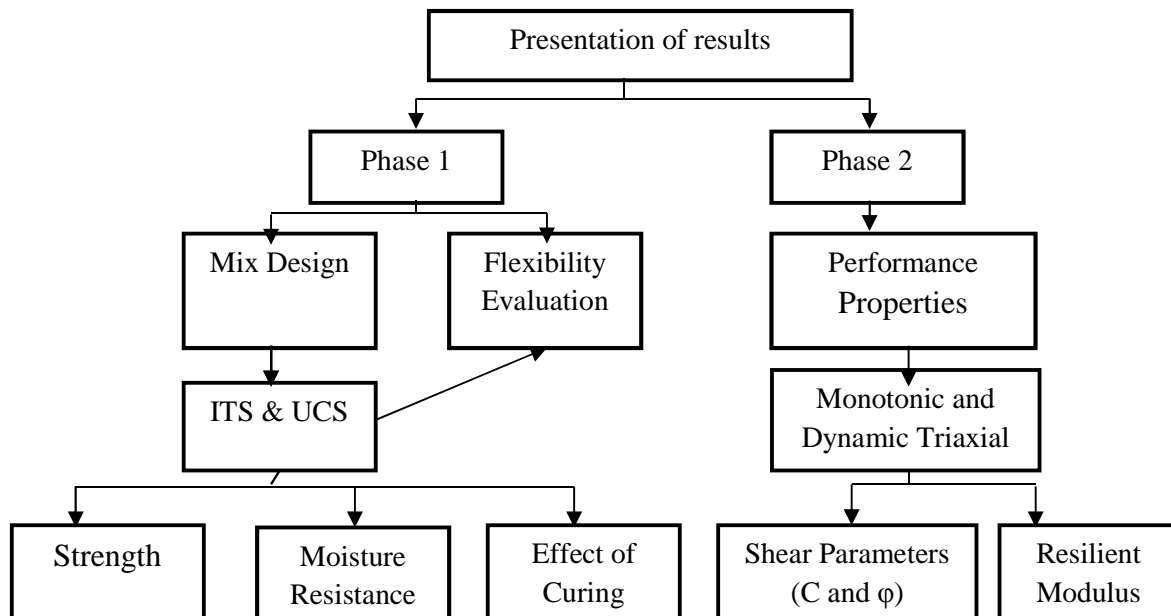
Figure 4. 1: Flow chart diagram of presentation of results

The nomenclature used to name the mixes is described in the two examples below:

Example 1: **FB2.4-CM1-LM1-Wet-nn,** where:

- FB2.4 indicates the foamed bitumen treated mix at 2.4 % bitumen content;
- CM1 indicates the addition of 1% cement in the mix;
- LM1 indicates the addition of 1% lime in the mix;
- Wet indicates the testing in wet conditions;
- nn indicates the specimen number.

Example 2: **EB0.9-CM2-LD-HS- C150-SR50,** where:

- EB0.9 indicates the bitumen emulsion treated mix at 0.9 % residual bitumen content;
- CM2 indicates the addition of 2% cement to the mix;
- LD indicates a low-density specimen;
- HS indicates a high-saturated specimen;
- C150 indicates a confinement pressure of 150 kPa;
- SR50 indicates a stress ratio 20%

4.2 Phase 1: Mix Design Results (ITS and UCS)

In this section, the mix design results are presented and analysed. Firstly, the influence of the mix variables (i.e. type of treatment, bitumen content, active filler content and the moisture conditions at testing) on the ITS, UCS, fracture energy, strain and displacement at break is evaluated based on the average values of the triplicate tests done for each mix. Secondly, a statistical analysis is conducted in order to validate the trends observed with the averages and evaluate the significance of the mix variable on mechanical parameters mentioned earlier.

4.2.1 Tensile and compressive Strength

Three replicates ITS and UCS tests were done on all mixes in dry and soak condition. The average ITS and UCS values of the mixes treated with the addition of cement as active fillers are presented in Table 4.1. ITS and UCS values of mixes treated with the addition of lime as active fillers are presented in Table A.2 of Appendix A.

Table 4. 1: ITS and UCS results, mix treated with 1% and 2% cement

Mix Type	Cement added (%)	Bitumen added (%)	Strength test results			
			ITS$_{EQUIL}$ (kPa)	ITS$_{WET}$ (kPa)	UCS$_{EQUIL}$ (kPa)	UCS$_{DRY}$ (Kpa)
BSM - emulsion	1	2.0	265.6	188.6	1323.2	1179.2
		2.4	226.3	205.1	1402.6	1232.2
		2.8	271.1	266.6	1280.8	1131.6
	2	2.0	544.6	431.6	2693.5	2408.0
		2.4	414.6	405.1	2132.8	1991.2
		2.8	430.4	375.7	1957.3	1774.8
BSM - foam	1	2..0	181.2	148.8	2061.3	1538.1
		2.4	256.4	202.7	2149.3	1539.0
		2.8	313.1	198.8	1920.7	1341.0
	2	2.0	337.8	318.6	2648.6	2446.5
		2.4	330.6	283.9	3130.4	2560.9
		2.8	525.9	426.6	2674.3	2199.0

Looking at the mixes where cement was used as active filler, all ITS$_{dry}$ and ITS$_{wet}$ values obtained are greater than 175 kPa and 100 kPa respectively. Moreover, except for mix EB2.0-CM1-wet and mix EB2.8-CM1-wet which have UCS values of 1179.16 kPa and 1131.60 respectively, all other UCS values obtained are greater than 1200 kPa. Therfore, these mixes can be classified as BSM1 (good quality) according to the material classification table of the TG2. On the other hand, mixes with lime as active filler did not have high ITS and UCS values compared to mixes with cement. Most of them fall into the BSM 2 class of TG2. Moreover, as we can see in Table 4.1, cement tends to react best with the parent material and was therefore selected for the second phase of the research (Triaxial testing).

4.2.1.1 *Influence of the type and percentage of active fillers on the strength*

From general observation of the results of Table 4.1, there is strong evidence that increasing the percentage of cement content in the mixes, regardless of the type of bitumen stabilisation (bitumen emulsion or foamed bitumen) leads to a significant increase of the tensile and the compressive strength. The trend is illustrated in Figure 4.2 and on the Figures A-3 and A-4 of Appendix A. This has been observed with several past research on BSMs. Hodgkinson, A. and Visser, A.T. 2004, stated that active filler (cement) influence the chemical and physical properties of bituminous mixes, even when used in small quantities. The gain in strength due to the addition and increase of the active filler content enabled BSMs to develop high stiffness modulus and excellent resistance to rutting (Giuliani, 2001).

Besides the increase in ITS and UCS values with the increase of cement content, the data also shows that the ITS displays greater sensitivity to active filler content variation compared to UCS.
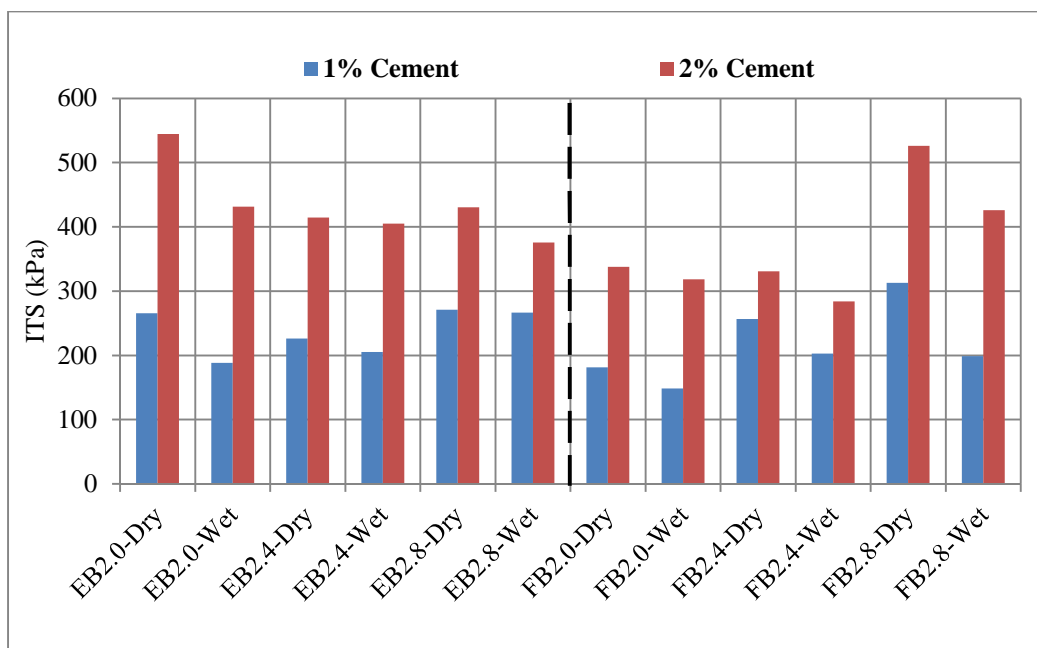

Figure 4. 2: Influence of the cement content on the ITS values

The increase in strength as a result of the increase in active filler content from 1% to 2% was computed. From Table 4.2 below, it can be seen that a 1% increase of cement content led to an increase of between 28% to 100% of the ITS and UCS value both in dry and wet conditions for bitumen emulsion and foamed bitumen mixes**.** However, the increase in cement content should be controlled with care in order not to end up with a tough and brittle material. If the BSM layer becomes too stiff, it will lose its flexural properties and will become susceptible to crack under loading and this could be detrimental to the layer.

Table 4. 2: Percentage of increase in ITS and UCS due to 1% increase in cement content

Percentage of increase of ITS and UCS values with the increase of 1% cement content						
UCS						
	EB2.0	EB2.4	EB2.8	FB2.0	FB2.4	FB2.8
Dry specimens	103.5%	52.1%	52.8%	28.5%	45.6%	39.2%
Wet specimens	104.2%	61.6%	56.8%	59.1%	66.4%	64%
ITS						
	EB2.0	EB2.4	EB2.8	FB2.0	FB2.4	FB2.8
Dry specimens	105%	83.2%	58.7%	86.4%	29%	68%
Wet specimens	128.8%	97.5%	40.9%	114.2%	40.1 %	114.4%

4.2.1.2   Influence of the moisture conditions at testing (moisture sensitivity)

It can be seen from Figure 4.3 and Figure A.1 and A.2 of Appendix A that there is a significant change in ITS and UCS values when specimens are tested in dry and in wet conditions; especially with mixes where lime was used as active filler. This shows that although the bond between mastic and aggregate in BSMs is improved during the curing phase of bitumen stabilised materials, excessive presence of water afterwards lead to the weakening of that bond and makes the treated material vulnerable.
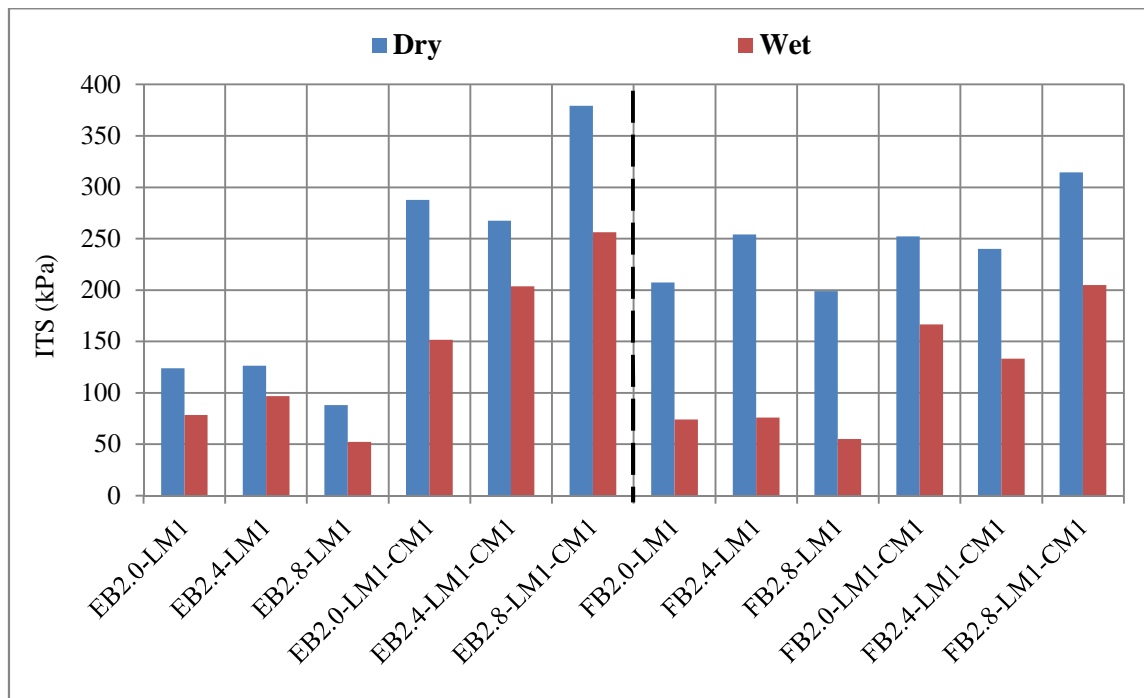
Figure 4. 3: ITS$_{Dry}$ and ITS$_{wet}$ values of mixes with lime

For a more in depth analysis of the effect of moisture on the mixes, the retained tensile strength (TSR) of all the mixes was determined. The obtained values are presented in Table 4.3 and illustrated in Figure 4.4. It is believe that the TSR (ratio between wet and dry ITS) is a suitable indicator in analysing the effect of active filler and the moisture susceptibility of the mix (Ebels and Jenkins, 2006). The results of the table confirm the preference of the parent material used in this study for cement as active during the treatment.

Table 4. 3: Tensile strength retained per mix vs. bitumen content

TSR-Emulsion bitumen mixes (%)				
Bitumen Content	EB-LM 1	EB-CM1	EB-LM-1 CM1	EB-CM2
2	63.2	71	52.7	79.2
2.4	76.5	90.6	76.1	97.7
2.8	59.5	98.3	67.6	87.3
TSR-Foamed Bitumen Mixes (%)				
Bitumen Content	FB-LM 1	FB-CM1	FB-L1 CM1	FB-CM2
2	35.7	82.1	66	94.3
2.4	29.9	79	55.5	85.8
2.8	27.7	63.5	65.2	81
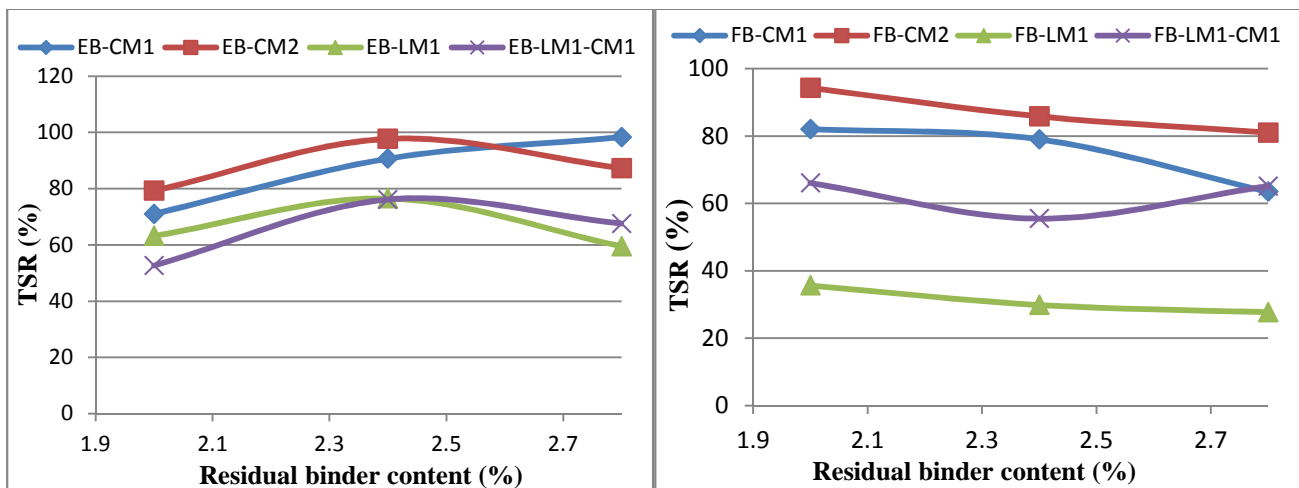
Figure 4. 4: Illustration of Tensile Strength Ratio per mix vs. bitumen content for emulsion mixes (left) and foam mixes (right)

The results show that for most of the mixes (especially mixes with cement) the effect of moisture on ITS/UCS values in wet and dry conditions is more pronounced with foamed bitumen mixes than bitumen emulsion mixes. This shows that bitumen emulsion mixes have better resistance to moisture effects compared to foamed bitumen mixes in terms of the strength of the mix. Twagira (2010), when looking at the moisture susceptibility and damage of BSMs made the same observation. This could be explained by the fact that, when mixing emulsion with aggregate, more of the larger aggregates are coated by the binder than foam mixes. This results in an improvement in bonding in the mix due to the significant binder coverage on the coarse aggregate fraction.

Results also show that the effect of moisture on ITS and UCS values is less pronounced on the mixtures where lime and cement were used in combination than mixtures where 2% cement was added. In other words, weakening due to moisture was found to be more predominant in the mixes with less active filler content; especially with foamed bitumen mixes. The same observation was

mode in past research, Long and Ventura (2003), Ebels (2008), Fu et al. (2009), Gonzalez (2009), Twagira and Jenkins (2009) without a clear definition of the mechanisms behind it. However, this observation could be explained by the fact that, increasing the amount of active filler from 1 to 2% increases the cementation effect that might reduce the diffusion of moisture within the material. In addition, more active fillers implies more fines which play an important role in the dispersion of foamed bubbles in the aggregate to form the mastic. Therefore, the far-reaching coating of binder with aggregate results in fewer voids content in the mixture that limits the diffusion of water through the mixture to limit the bound of bitumen and aggregate.

4.2.1.3   Influence of the type of treatment and the bitumen content

Though the test does not give highly repeatable results, the ITS is a cost-effective method that could be used to investigate the effectiveness of bitumen content in BSMs (Wirtgen,2012). In the case of this study, a reduction in ITS and UCS values was observed with the increase of the bitumen content as shown on Figure 4.5   for most of the mixes. This correlates with the findings of Liebenberg (2002). He found that at high cement content i.e. above the initial consumption of lime (ICL), there is a tendency that the cement dominate the ITS and the UCS whilst there is a reduction in UCS and ITS with the increase in bitumen content. On the other hand, he also noticed that when the ICL is not reached, the effect of cement is little and increasing the bitumen percentage tends to increase the ITS and UCS. (Hodgkinson and Visser, 2004).
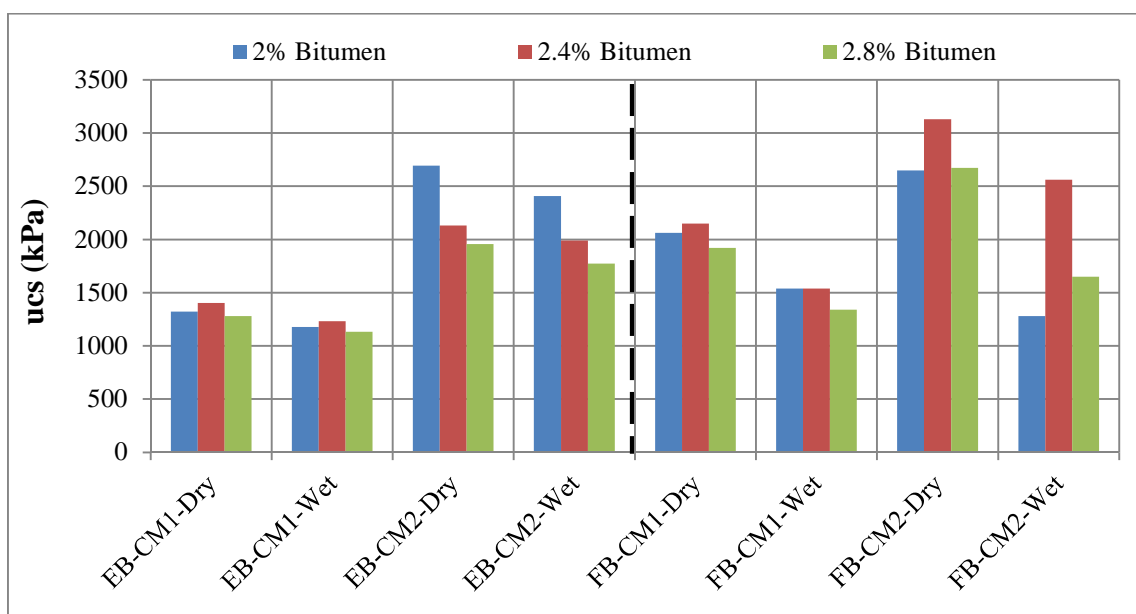
Figure 4. 5: Illustration of UCS vs bitumen content (mixes with cement as active filler)

The effect of the type of treatment (i.e. foam vs emulsion) shows that emulsion mixes have a slight higher strength compared the foam mixes (see Figure 4.5). At low bitumen content the wet and dry

ITS of emulsion treated mixes were higher than those of foamed treated mixes. But as the bitumen content increases, foamed bitumen mixes end up having higher dry and wet ITS. This could be attributed to the way in which the two type of bitumen bond with aggregate. It was furthermore observed that foamed bitumen stabilised mixes produce relatively low tensile strength retained emulsion treated mixes, especially mixes where lime was used as active filler. This means that they seem to be are more sensitive to moisture compared to emulsion treated mixes. In other words, it appears that bitumen emulsion stabilised mixes have better water proofing quality compared to foamed bitumen stabilised mixes. However, further investigation needs to be done in this regard.

4.2.1.4   Comparison of the curing method on the strength

The strength of specimens cured for 28 days was compared to that of specimens cured following the accelerated curing protocol from the mix design. 6 mixes out of the 8 (4 for ITS and 4 for UCS) that were compared, displayed that long term curing mixes have higher ITS and UCS results than mixes which were cured using the accelerated curing method (see Figure 4.6). This difference in strength between the accelerated curing and the 28 days curing seems to be larger with mixes containing 2% percent cement. This might be attributed to the cementation effect.
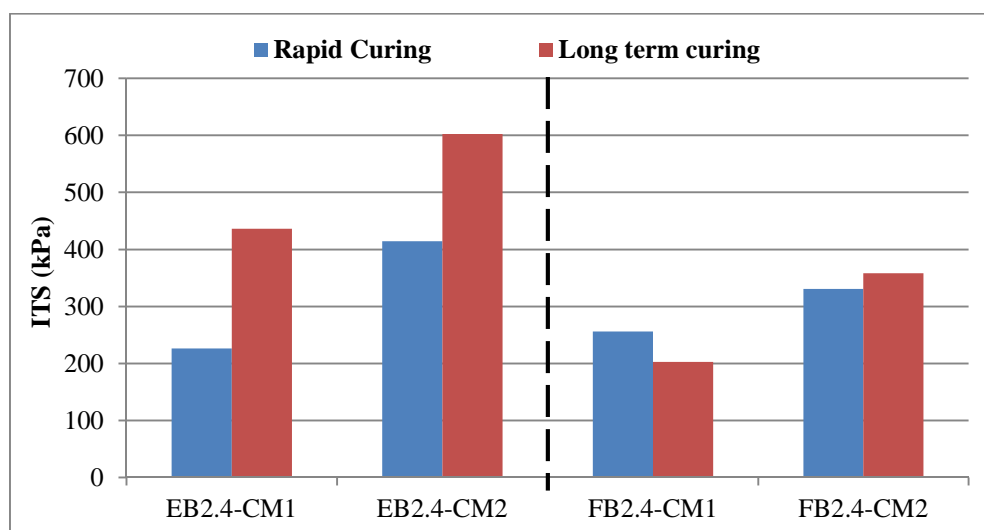
Figure 4. 6: Comparison of the curing method on the strength (ITS-values)

4.2.1.5   Correlation between ITS and UCS values

Previous research has shown an existing linear relationship between the ITS and the UCS of BSMs (Houston *et al*, 2004). In this regard, the data were analysed in order to investigate if there could be an eventual relationship between the ITS and UCS values of the different mixes tested. To achieve this, the UCS was plotted against the ITS. A linear regression was then fitted to the data and the R^2 values analysed as shown in Figure 4.7. A fairly good linear correlation of ITS and UCS values were

obtained both for mixes treated with the addition of cement and those treated with the addition of lime. This shows that one test among the two is sufficient to give indication on the strength of bitumen stabilised materials at the mix design level.
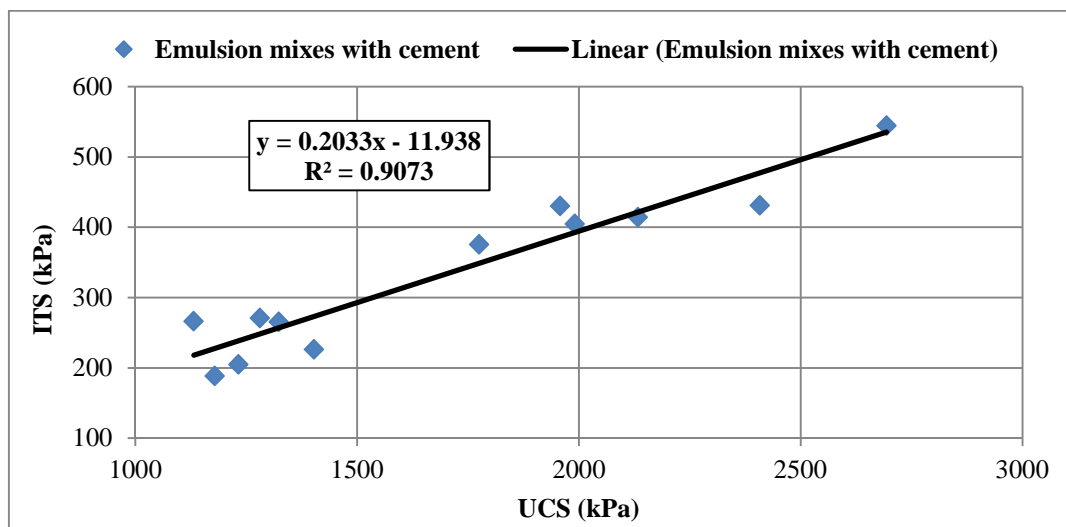
Figure 4. 7: ITS vs UCS at different cement/bitumen content (Emulsion mixes)

4.2.2 Displacement and strain at break results

The displacement and the strain at break were determined respectively from the ITS and UCS tests with the aim of investigating their use in evaluating the flexibility of the mixes. A low displacement or strain at break will indicate that the material has a low ability to flex under loading. In other words, the higher the value of displacement and strain at break, the more flexible it is. For a better analysis of the data, the average displacement and strain-at-break from the triplicate ITS and UCS test per mixes was determined. Results are summarised in Tables 4.4 bellow and in the tables of Appendix B.

As mentioned earlier, it was found that the increase of the percentage of active filler (using either lime or cement) results in an increase in strength of the material. However, it leads to a reduction of the displacement and the strain at break derived from the ITS and UCS test respectively. As shown in Figure 4.8, despite the benefit of strength increase that the increase in cement content causes, it has a secondary effect on the material, which is the increase of its brittleness behaviour. Therefore, the higher the cement content, the less flexible the material is.

Table 4. 4: Displacement and strain at break

Mix	Testing Condition	Average displacement at break from ITS test (mm)	Average strain at break from UCS test (m strain)
EB2.0-CM1	Dry	0.8	35
	Wet	0.8	33.6
EB2.4-CM1	Dry	0.6	19
	Wet	0.7	16.8
EB2.8-CM1	Dry	0.7	19.9
	Wet	0.8	18.2
EB2.0-CM2	Dry	0.6	20.4
	Wet	0.5	20.8
EB2.4-CM2	Dry	0.6	19.6
	Wet	0.6	19.3
EB2.8-CM2	Dry	0.6	16.3
	Wet	0.6	17.7
FB2.0-CM1	Dry	0.5	22.1
	Wet	0.7	15.8
FB2.4-CM1	Dry	0.6	22.5
	Wet	0.6	15.5
FB2.8-CM1	Dry	0.5	16.3
	Wet	0.6	13.3
FB2.0-CM2	Dry	0.5	23.8
	Wet	0.6	24.2
FB2.4-CM2	Dry	0.6	21
	Wet	0.6	17.8
FB2.8-CM2	Dry	0.6	16.4
	Wet	0.6	15.6
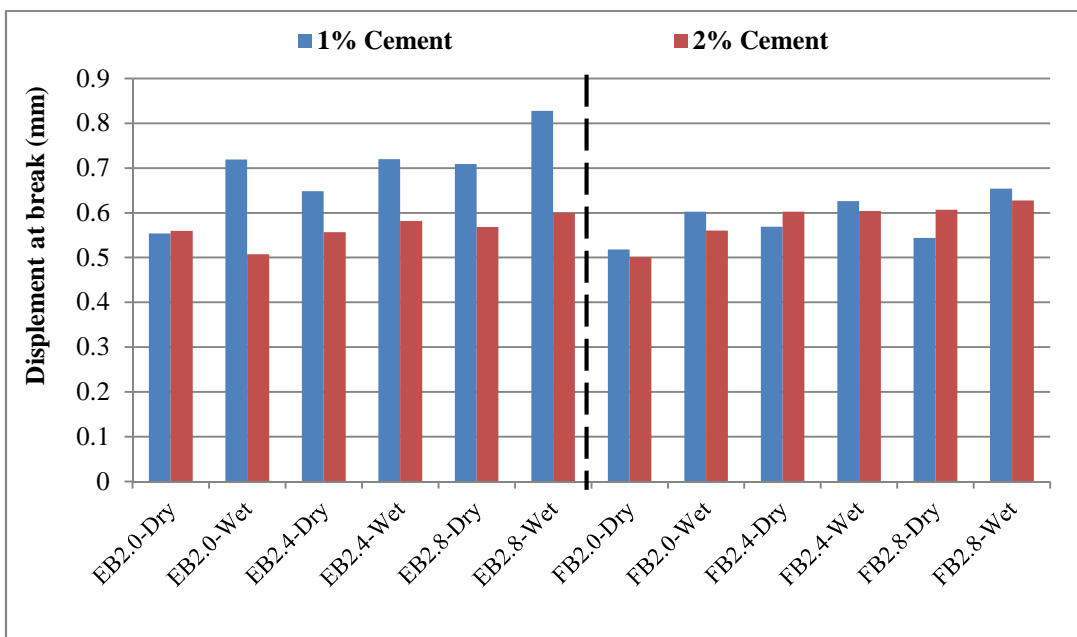
Figure 4. 8: Effect of the cement content on the displacement at break

The effect of the bitumen content on the displacement and strain at break was not consistent. However, for most of the mixes it was found that at constant active filler content, the displacement and strain at break values increase as the percentage of bitumen added to the mix increase. As shown in Figure 4.9 the displacement at break increases as the bitumen content is increased. This suggests that higher bitumen contents result in greater flexibility.
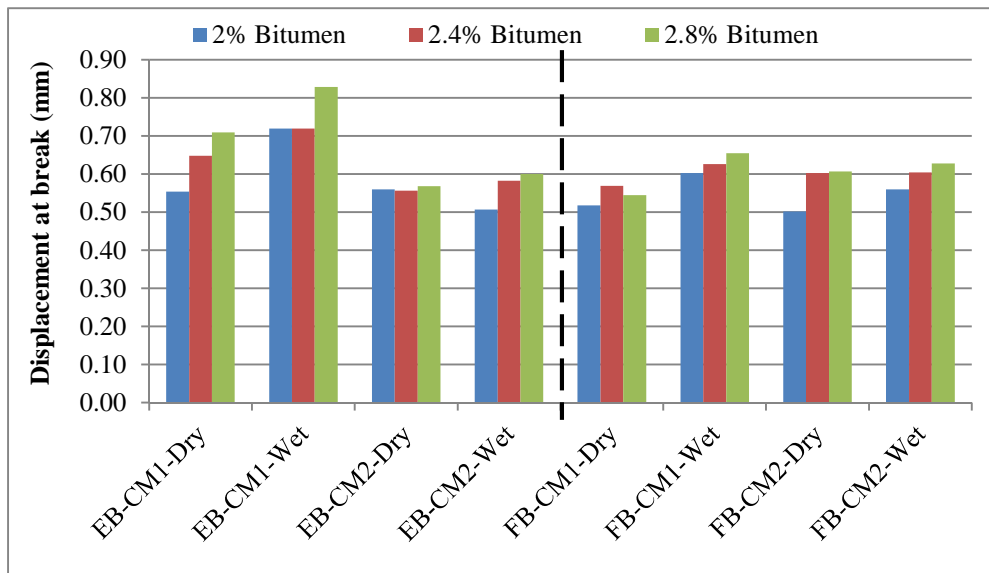
Figure 4. 9: Effect of the bitumen content on the displacement at break

4.2.3   Fracture energy

The fracture energy of the different mixes was computed by calculating the area under the load displacement curve from the ITS test as described in the previous chapter with the aim of using the parameter as an indicator of the flexibility of the mixes. When comparing two tested specimens, the one with the higher fracture energy value indicates that the specimen undergoes a greater amount of deformation before failure. In other words, the higher the fracture energy, the higher the combination of deformation at failure and maximum stress at failure.   The average fracture energy from the triplicate ITS specimen tested per mix combination are presented in Table 4.5. Only mixes where cement was used as active filler are presented in Table 4.5.

It is evident from Figure 4.10 that the fracture energy values of the majority of the mixes with 1% cement content active fillers are much higher than 2% cement content mixes. This shows that at 2% cement content mix undergoes less deformation before failure. In other words, the increase in cement content results in brittle behaviour of the material and therefore reduces its flexibility. The same observation was done on specimens tested in low and high saturation level, however to a lesser extent.

Table 4. 5: Dissipated energy values from ITS load displacement curve

Dissipated energy of mixes with cement (Joules)		
Mix	**Testing Condition**	**Average**
EB2.0-CM1	Dry	2.3
	Wet	2.7
EB2.4-CM1	Dry	3.2
	Wet	2.7
EB2.8-CM1	Dry	3.3
	Wet	3.2
EB2.0-CM2	Dry	2.1
	Wet	2.3
EB2.4-CM2	Dry	2.3
	Wet	1.8
EB2.8-CM2	Dry	2.3
	Wet	2.9
FB2.0-CM1	Dry	2
	Wet	2
FB2.4-CM1	Dry	2
	Wet	2.2
FB2.8-CM1	Dry	3.7
	Wet	2.8
FB2.0-CM2	Dry	1.2
	Wet	1.5
FB2.4-CM2	Dry	1.8
	Wet	1.6
FB2.8-CM2	Dry	2.7
	Wet	1.5
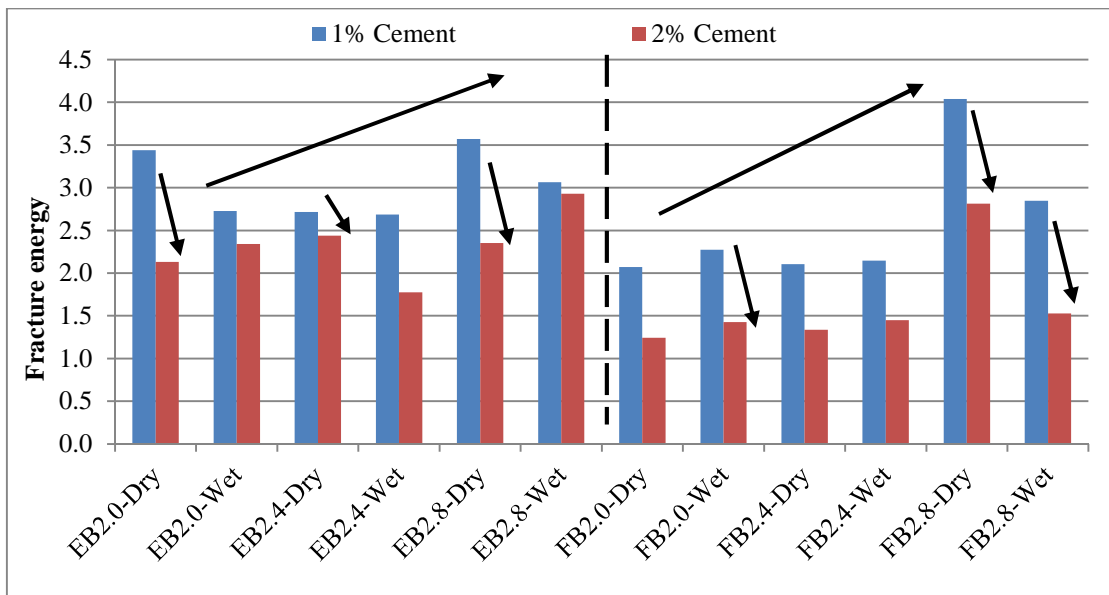
Figure 4. 10: Effect of cement content on dissipated energy values

It can be seen from the Figure 4.11 that at constant cement content, the fracture energy of mixes increases as the percentage of residual bitumen increases. Mixes with high bitumen content exhibit high fracture energy compared to mixes with lower bitumen content. This implies that increasing the percentage of bitumen content gives more flexibility to the mix. However, one should keep in mind that adding more bitumen to the mix might compromise the strength and result in rutting.
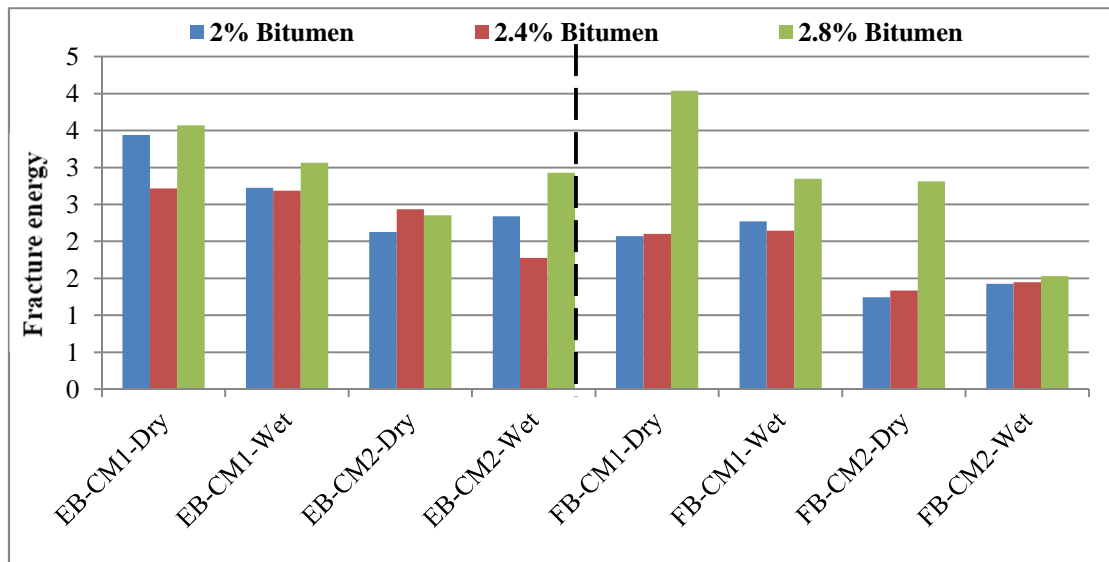
Figure 4. 11: Effect of bitumen content on the fracture energy

When comparing bitumen emulsion mixes with foamed bitumen mixes in the same condition as mentioned above, the results show that at constant cement and bitumen content, fracture energy of bitumen emulsion mixes is higher than the one of foam bitumen mixes. In other words, it could be said that at equal mix component, bitumen emulsion mixes tend to be more flexible compared to foamed bitumen mixes.

4.2.4   Statistical analysis

It is not enough to compare several sets of data to each other when looking at the influence of the test combination on the mechanical properties parameters. For instance, it is good to notice the improvement in strength due to the increase of cement content and reduction in saturation level, but one should consider the statistical significance of that improvement. Moreover, in a case of multiple sources of variation, the significance of one source of variation compared to another as well as the combined effect of sources of variation should be investigated.

The statistical analysis of this study investigates not only the effect of the type of treatment, the moisture condition, the bitumen and cement content (known as independent variables) on the ITS, UCS, displacement at break and strain-at-break (known as dependent variables) but also the effect and interaction of combined independent variables on the dependent variables. To achieve that, the

statistical method named analysis of variance (ANOVA) was performed using a statistical software programme . The assumptions associated with the method being that:

- The dependent variable is normally distributed within each cell ;

- The population variances are identical within each cell ;and

- The observations and groups are independent of each other.

The output data from the analysis comprises the following:

The R square: it is the correlation coefficient in the regression model. It indicates the percentage of variability of the dependent variable for a particular independent variable. The higher the value of R square, the better is the regression model, the highest value of R square being equal to 1 or 100%.

The adjusted R square: also expressed in percentage, the adjusted R square is used to determine the point at which the model would be best fitted (Asiimwe, 2013). The greater the number of significant independent variable, the higher the value of the adjusted R square will be.

The total sum of squares (SS_{Tot}): It is the sum of squares between treatment groups (SS_{Treat}) and the sum of squares within treatment groups (SS_{res}). It indicates the extent of variability in data point dispersion.

$$SS_{tot} = SS_{treat} + SS_{res}$$

The mean of squares (MS): Computed for each component of the total variability the mean of squares is calculated as the ratio between the individual component sum of squares and their respective degree of freedom. It can distinguish the mean of squares between the treatment group (MS_{treat}) and the mean of squares within the treatment group (MS_{res})

$$MS_{treat} = \frac{SS_{treat}}{df_{treat}}$$

$$MS_{res} = \frac{SS_{res}}{df_{res}}$$

The F-value: it is the ratio of the mean square between the treatment groups and the mean square within the treatment groups

$$F = \frac{MS_{treat}}{MS_{res}}$$

The P-value: it measures the significance of the effect of independent variables taken in isolation or in combination on the dependent variables. It is obtained from the F-value for a particular value of α

and degree of freedom. In this study, value of alpha equal to 5% was used for the analysis (i.e. analysis was done with 95% confidence interval). This also means that any independent variable with a p-value less than 0.05 will be considered not important when looking at its influence on a given dependent variable. On the other hand, the smaller the p-value of an independent variable when compared to 0.05 the more significant is its influence of the dependent variables assessed.

The results presented in Table 4.6 and Table 4.7 show the effect of independent variables on the strain at break and the ITS values respectively. The full set of data results of the statistical analysis performed is presented in Appendix C.

4.2.4.1 *Analysis on flexibility indicator parameters (Fracture energy, displacement and strain at break*

Table 4. 6: ANOVA Table for strain at break

ANOVA Table for strain at break					
Factor	**df**	**Sum of Squares**	**Mean Squares**	**F**	**p**
Type of treatment	1	16.496	16.496	6.346	0.029
Cement content	1	16.107	16.107	6.197	0.016
Bitumen content	2	19.742	9.871	3.797	0.018
testing condition	1	1.101	1.101	0.424	0.051
Type of treatment * Cement content	1	29.954	29.954	11.523	0.001
Type of treatment * Bitumen content	2	12.772	6.386	2.457	0.096
Type of treatment * Testing condition	1	2.764	2.764	1.064	0.308
Cement content * Bitumen content	2	36.552	18.276	7.031	0.002
Cement content * Testing condition	1	2.039	2.039	0.784	0.38
Bitumen content * Testing condition	2	20.48	10.24	3.939	0.026
Type of treatment * Cement content * Bitumen content	2	4.023	2.012	0.774	0.467
Type of treatment * Cement content * Testing condition	1	2.396	2.396	0.922	0.342
Type of treatment * Bitumen content * Testing condition	2	19.153	9.577	3.684	0.032
Cement content * Bitumen content* Testing condition	2	14.728	7.364	2.833	0.069
Type of treatment * Cement content* Bitumen content * Testing condition	2	3.42	1.71	0.658	0.523
Error	48	124.772	2.599		
Total	72	24193.66			

When analysing the single effect of the independent variables on the flexibility indicator parameters (i.e. displacement at break, stain at break and fracture energy), one can see from the statistical results that they all have p-values less than 0.05 (see Figure 4.12). This shows that each of the four independent variables (i.e. type of treatment, testing condition, cement and bitumen content), have a significant effect on the flexibility indicator parameters. Looking at the effect of bitumen content for

instance, p-values of 0.382E-5, 0.029 and 2.294E-17 were obtained for the displacement at break, the strain at break and the fracture energy, respectively. When assessing the significance of one independent variable compared to another, it was found that the independent variable with the most significant effect on the fracture energy is the cement content (p = 2.330E-18), followed by the bitumen content (P=2.294E-17), then the type of treatment (p = 3.340E-13) and finally the testing condition (0.011). It is important to mention that this level of significance was not observed in the same order for all the 3 parameters. In the case of the strain at break of instance, the type of treatment has the most significant effect, followed by the cement and the bitumen content. According to the hypothesis, the testing condition was found not to have an effect on the strain at break for it has a p-value of 0.518 which is far greater than 0.05.
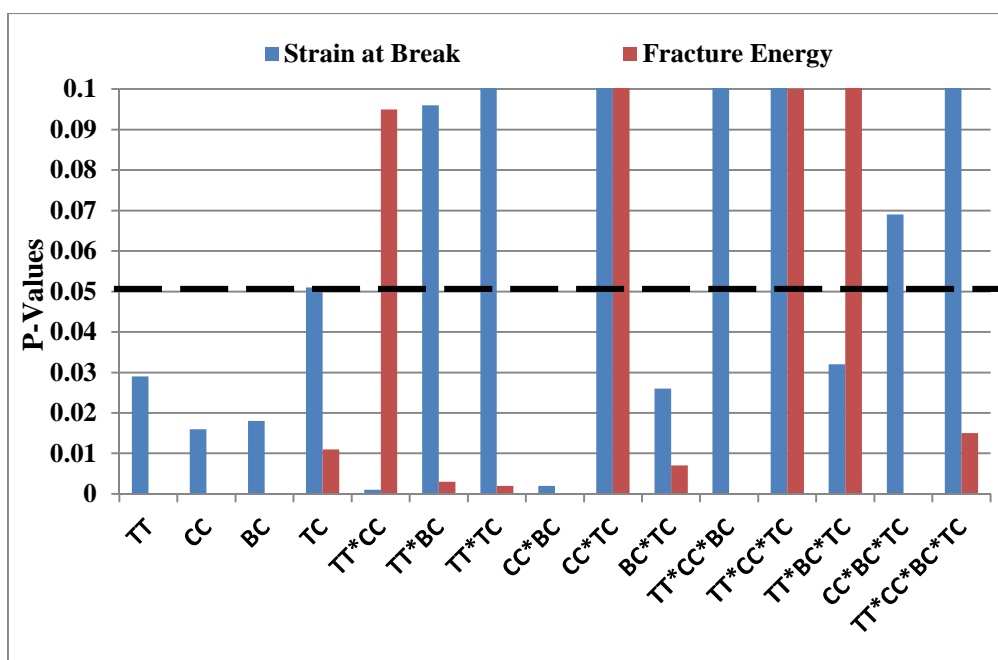
Figure 4. 12: Comparison of P-values of strain at break and fracture energy

At 2 level combination, all combined effects of independent variables, except the cement content * testing condition were found to have an effect on the strain at break and the fracture energy. The independent variables with the most significant effect on the stain at break being the type of treatment * cement content (p = 0.001), follow by cement content * bitumen content (p=0.002), and the bitumen content * type of treatment (p=0.026). At a 3 level combination, the type of treatment * cement content * bitumen content has a significant effect on both fracture energy and strain at break. Only the fracture energy was influenced by the combined effects of independent variables at 4 level combination. The effect of type of treatment * cement content * bitumen content * testing condition had a significance of 0.015.

4.2.4.2   *Analysis on strength indicators (ITS and UCS)*

Table 4. 7: ANOVA Table for ITS

ANOVA Table for ITS					
Factors	**df**	**Sum of Sq**	**Mean Sq**	**F**	**p**
Type of treatment	1	31449.162	31449.162	68.136	9.11E-11
Cement content	1	551529.394	551529.39	1194.919	1.41E-35
Bitumen content	2	49321.736	24660.868	53.429	6.19E-13
testing condition	1	52191.976	52191.976	113.077	3.26E-14
Type of treatment * Cement content	1	8225.847	8225.847	17.822	0.000107
Type of treatment * Bitumen content	2	59722.386	29861.193	64.696	2.37E-14
Type of treatment * Testing condition	1	932.781	932.781	2.021	0.162
Cement content * Bitumen content	2	17435.097	8717.549	18.887	8.9E-07
Cement content * Testing condition	1	196.445	196.445	0.426	0.517
Bitumen content * Testing condition	2	4178.096	2089.048	4.526	0.016
Type of treatment * Cement content * Bitumen content	2	37491.978	18745.989	40.614	4.76E-11
Type of treatment * Cement content * Testing condition	1	1491.569	1491.569	3.232	0.079
Type of treatment * Bitumen content * Testing condition	2	17065.18	8532.59	18.486	1.12E-06
Cement content * Bitumen content* Testing condition	2	602.701	301.351	0.653	0.525
Type of treatment * Cement content* Bitumen content * Testing condition	2	997.784	498.892	1.081	0.347
Error	48	22154.992	461.562		
Total	72	7978747.97			

According to the hypothesis, the analysis shows that all four independent variables have a great effect on the ITS and the UCS. For instance, the independent variable with the most significant effect on the ITS is the cement content; followed by the testing conditions, then the bitumen content and finally the type of treatment. On the other hand, the UCS was mostly influenced by the cement content; followed by the type of treatment, then the testing condition and finally the bitumen content. One could also notice that the UCS was most influenced by the type of treatment, the cement content and the testing condition compared to the ITS. The independent variables in the order above mentioned had p values of 3.337E-24, 4.188E-36 and 4.283E-18 respectively for UCS against p values of 9.109E-11, 1.414E-35 and 3.260E-14 respectively for ITS.

At two level combination, the combined effect of the cement content * bitumen content was found to be significant both for ITS and UCS with p values of 8.897E-7 and 7.971E-5 respectively. The type of treatment * cement content and the type of treatment * testing condition did not to have a significant effect on the UCS and the ITS respectively according to the hypothesis. Moreover, the

combined effect of cement content and testing condition did not have a significant effect on both ITS and UCS. At three level combination, only the type of treatment * cement content * bitumen content had a significant effect on both ITS and UCS with respective p values of 4.756E-11 and 1.988E-8. four level combination of type of treatment * cement content * bitumen content * testing condition did not have an effect on both ITS and UCS according to the hypothesis.

4.2.4.2   *Interaction between strength and flexibility*

On a general observation of strength and flexibility interaction, it can be concluded from the statistical analysis that the strength and the stiffness of the mixes are more influenced by the testing condition and the percentage of active filler added than the bitumen content or the type of treatment. On the other hand, the flexibility seems to be influenced more by the amount bitumen added and the percentage of active filler than the type of treatment or the testing condition. The combined effect of cement and bitumen content was found to be significant for both strength and flexibility.

At the end of this section, it is evident from the results obtained that flexibility of BSM is a property that can be measured during mix design. The analysis shows that strain at break and fracture energy are acceptable indicators of flexibility. The trends observed with the two parameters under the influence of the mix variables were correlating with what was expected i.e. a decrease and an increase of the strain at break values as a result of the increase of cement and bitumen content in the mix respectively; and a decrease in fracture energy when the percentage of cement is increased.

4.3 Phase 2 : Triaxial Test Results

4.3.1   Monotonic triaxial test

This test was used to determine the shearing properties (i.e. The Cohesion C and the internal angle of friction φ). In the case of this study, these parameters were determined for each of the four combinations of relative density and saturation level per mix.

4.3.1.1   *Presentation of the results*

A typical example of a stress-strain curve for mix EB2.4-CM1-HD-LS is shown in Figure 4.13 below. Curves of other mixes are presented in Appendix D.
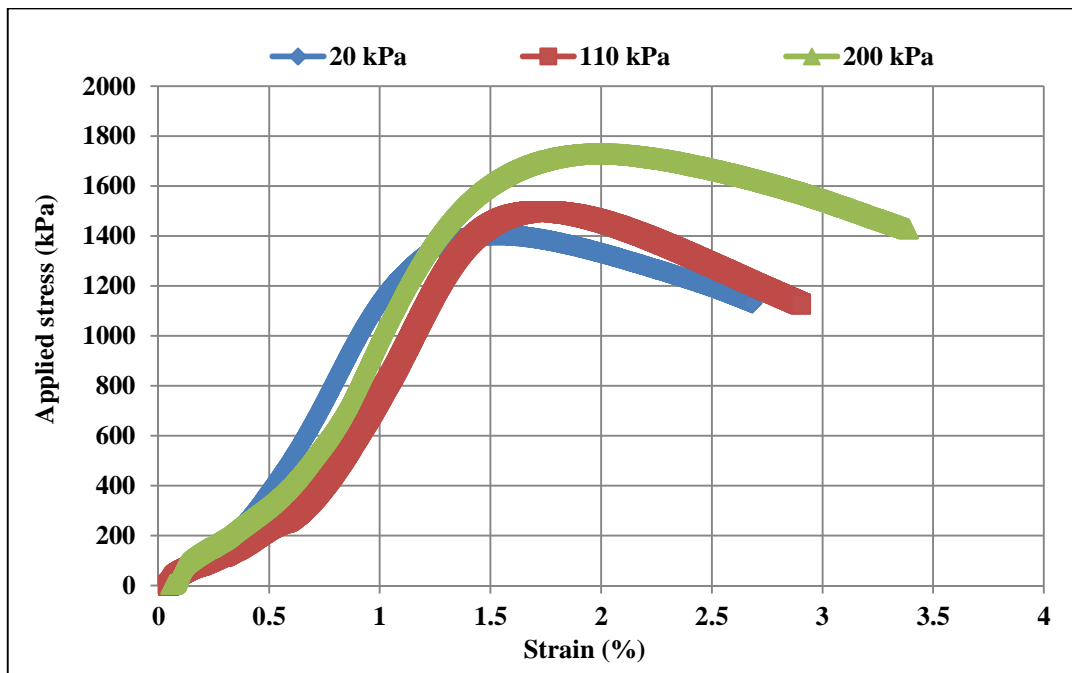
Figure 4. 13: Stress-strain diagram for mix EB2.4-CM1-HD-LS

The cell pressures and applied stresses at failure, measured at different confinement pressure during the monotonic test were used to produce a set of three Mohr's Coulomb circles. A resultant linear failure envelope was then fitted to the circles in order to determine the fundamental shearing properties of the different mixes: the intercept of the line with the axe of abscises representing the cohesion and its slope the internal angle of friction. A typical example is shown in the Figure 4.14.
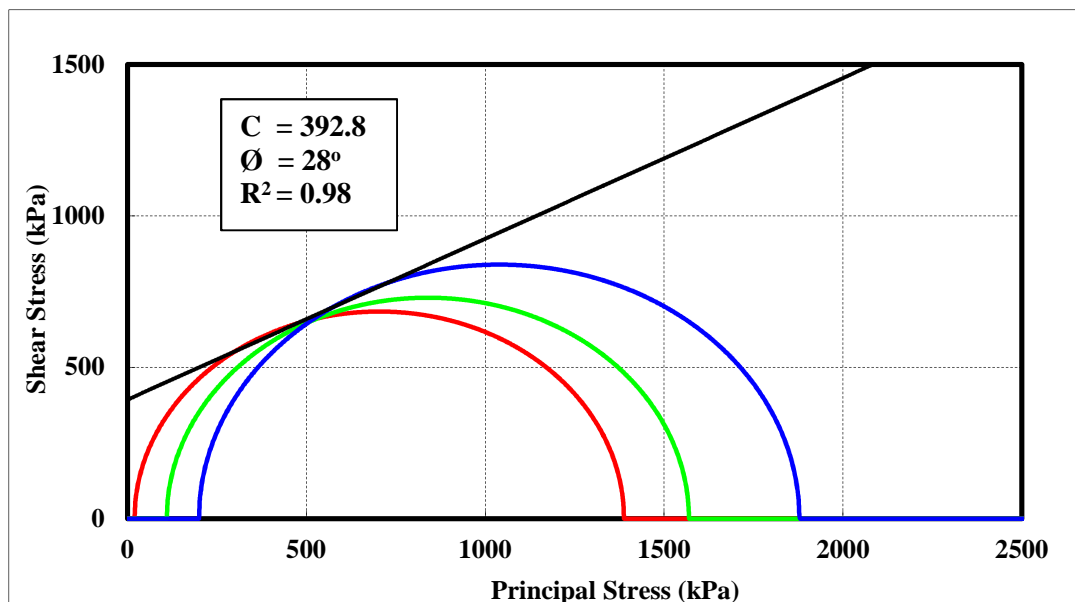
Figure 4. 14: Mohr Coulomb Plot for Mix EB2.4-CM1-HD-LS

The determined shearing parameters (cohesion and internal friction) for the four combinations of densities and saturation level following the method described earlier are presented in Table 4.8.

Table 4. 8: Summary of cohesion and angle of internal friction per mix

Additives		Average Density				Moisture Content			Cohesion (kPa)	Frition angle (°)	R²
Cement (%)	Bitumen (%)		RD	kg/m³	% Mod AASHTO [1]		Sat (%)	% by mass [2]			
1	0.9 EB	High	73.9	2121.9	101.0	High	77.8	9.6	315.4	19.3	0.97
		High	76.2	2189.0	104.2	Low	26.1	2.9	285.7	35.8	0.99
		Low	71.2	2044.7	97.4	High	79.5	11.2	283.3	26.1	0.99
		Low	70.9	2037.0	97.0	Low	25.1	3.6	259.9	45.5	1
1	2.4 EB	High	74.8	2093.1	99.7	High	75.4	9.1	367.5	22.4	0.92
		High	74	2069.9	98.6	Low	20.6	2.6	392.8	28	0.98
		Low	73.7	2060.4	98.1	High	85.6	10.9	205	32.5	0.99
		Low	72.9	2038.0	97.0	Low	26.9	3.6	296.2	33.6	0.97
	2.4 FB	High	75.1	2100.0	100.0	High	83.9	9.9	170.8	22.5	0.95
		High	74.9	2094.4	99.7	Low	25.16	3	495.8	19.9	0.98
		Low	74	2069.4	98.5	High	87.8	11	173.2	17.7	0.98
		Low	73.3	2050.0	97.6	Low	28	3.6	470.3	11.9	0.94
2	2.4 EB	High	75.3	2107.9	100.4	High	83.6	9.7	485.5	18.6	0.99
		High	74.4	2083.39	99.2	Low	20.7	2.5	595.2	25.9	0.99
		Low	73.6	2060.42	98.1	High	87.1	11.1	520	12.6	0.99
		Low	73	2042.6	97.3	Low	27.6	3.6	334.8	44.4	0.98
	2.4 FB	High	75.03	2099.6	100.0	High	79.3	9.4	374.4	33.4	0.98
		High	74.7	2091.3	99.6	Low	22.5	2.7	612.6	26.3	0.99
		Low	73.5	2055.6	97.9	High	86.38	11.2	372.9	22.7	0.98
		Low	73.4	2053.3	97.8	Low	23.9	3.1	460.7	34.2	0.95

[1] determined by back calculations using Equation 3.18 and relative densities of Table 3.7 and maximum dry density of 2100 kg/m³.

[2] determined by back calculations using Equation 3.19 and relative densities of Table 3.7 and the dry densities determined above.

Relatively good coefficients of variance (R^2-values) were obtained from the linear regression analysis. Except mixes EB2.4-CM1-HD-HS and FB2.4-CM1-LD-LS, which have R^2-values of 0.92 and 0.94 respectively, all other mixes have R^2-values greater the 0.95. This shows the accuracy with which the shearing parameters were determined from the Mohr Coulomb circle plotted at different confinement pressures.

Triaxial tests were done at 4 combinations of relative densities and saturation level. The calculations of the relative densities achieved and saturation levels were done according to Equation 3.18 and Equation 3.19 respectively. Table 4.9, contains a summary of the densities and saturation levels

achieved for the monotonic triaxial tests specimens and Figure 4.15 shows a plot of the 4 density-saturation combination.

Table 4. 9 Relative densities and saturation levels from monotonic triaxial testing

Additives		Average Density		Moisture Content	
Cement (%)	Bitumen (%)		RD		Sat (%)
1	0.9 EB	High	73.9	High	77.8
		High	76.2	Low	26.1
		Low	71.2	High	79.5
		Low	70.9	Low	25.1
1	2.4 EB	High	74.8	High	75.4
		High	74	Low	20.6
		Low	73.7	High	85.6
		Low	72.9	Low	26.9
	2.4 FB	High	75.1	High	83.9
		High	74.9	Low	25.16
		Low	74	High	87.8
		Low	73.3	Low	28
2	2.4 EB	High	75.3	High	83.6
		High	74.4	Low	20.7
		Low	73.6	High	87.1
		Low	73	Low	27.6
	2.4 FB	High	75.03	High	79.3
		High	74.7	Low	22.5
		Low	73.5	High	86.38
		Low	73.4	Low	23.9

The average and coefficient of variation were determined from three repeat specimens compacted per mix combination. From the table it can be seen that the coefficients of variation of mix combination are within small ranges. This shows a consistency in densities at testing. As for the saturation levels, only five mix combinations out of twenty have a coefficient of variation that is greater than 2 per cent with 5.8 per cent as maximum. This also shows a fairly small variation on the saturation levels. However, data shows that the variability in relative densities is small, compared to saturation levels.
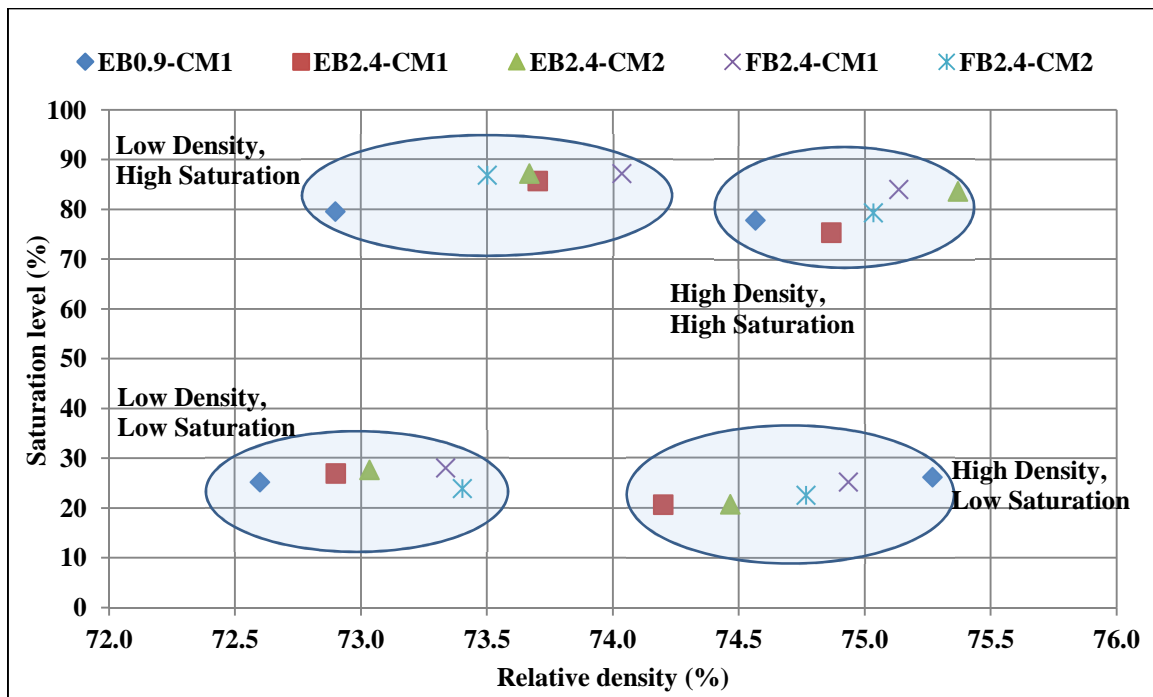
Figure 4. 15: Relative densities and saturation levels per testing combination

4.3.1.2   Interpretation of the shearing parameters results and influence of test variables

The cohesion and the internal angle of friction as determined from the Mohr Coulomb circles and presented in Table 4.8 are compared graphically in Figure 4.17 and Figure 4.18.
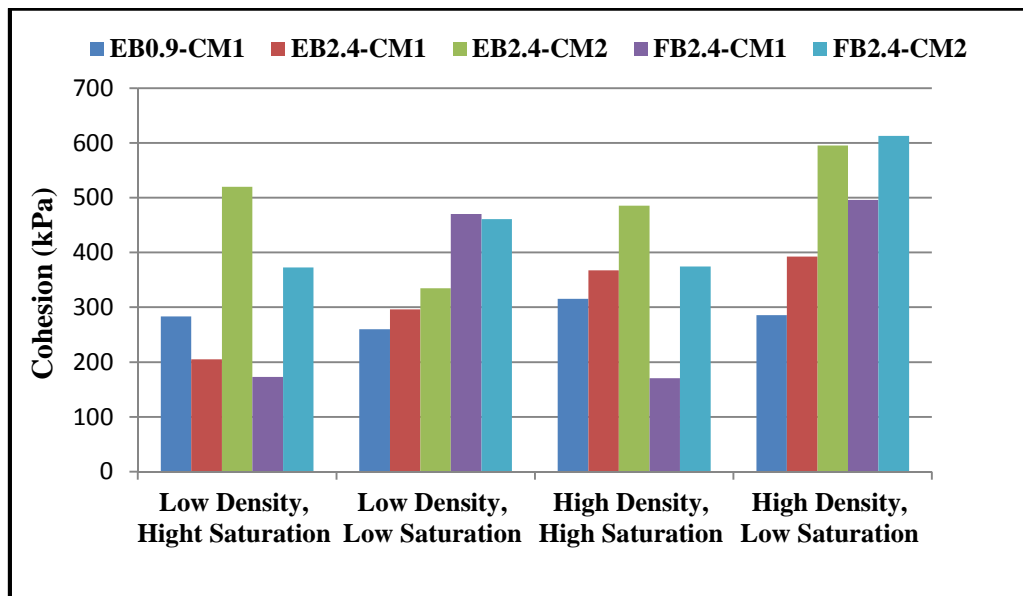
Figure 4. 16: Cohesion of BSMs mixes from R35

a)   Cohesion

The cohesion values range between 170.8 kPa and 612.6 kPa as shown in Table 4.9. From a general observation of Figire 4.17, it can be seen that the addition of a small percentage of active filler

(cement) in the mix results in a significant increase of cohesion values, regardless of the type of treatment (Foam or emulsion) and the four test combinations of density and saturation level. An increase of 1% cement caused an average increase of about 62 % and 65 % of cohesion values of bitumen emulsion and foamed bitumen mixes respectively.

It can also be seen that except mix EB2.4-CM2-LD-HS, the mixes of high density have cohesion values that are higher than those of the mixes of lower density. For most of the mixes, results also show a significant increase in cohesion when mixes are tested from high to low saturation level. Mixes tested in low saturation possess high cohesion compared to mixes tested in high saturation condition.
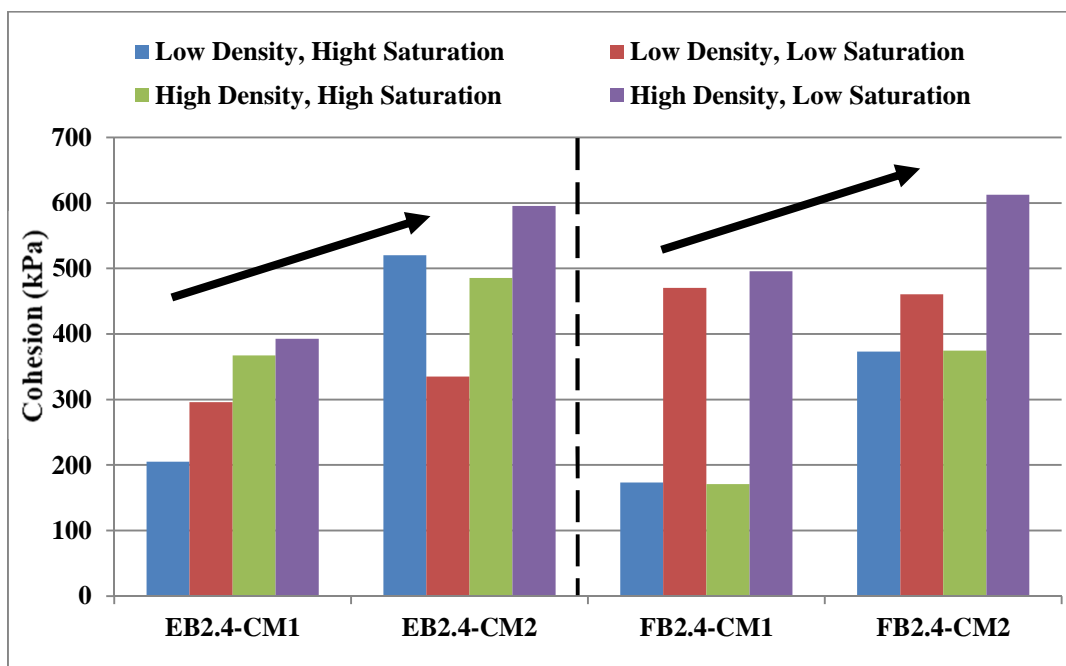
Figure 4. 17: Influence of density and saturation level on the cohesion

From the results we can see that the cohesion values of emulsion treated mixes with 0.9% residual bitumen are higher than those of the mixes treated with 2.4% residual bitumen. Moreover, at same mix combination, it can be seen that for most of the mixes the emulsion treatment results in higher cohesion values compared to foam treatment. However, a meticulous observation shows that at high density-Low saturation and Low density-Low saturation, BSM-foam mixes have greater cohesion values compared to BSM-emulsion mixes. On the other hand, at Low density-High saturation and High density-High saturation, it is the other way round (i.e. Cohesion values of BSM-emulsion are higher).

The effects of experimental variables (Type of treatment, addition of cement, and the 4 combinations of density and saturation level) on the cohesion are summarized in Table 4.10.

Experimental variables	Emulsion mixes	Foamed bitumen mixes
Adding 1% of cement	Increase ↑	Increase ↑
Increasing the density level	Increase ↑	Increase ↑
Increasing the saturation level	Decrease ↓	Decrease ↓
Type of treatment	Emulsion 0.9 < Emulsion 2.4 Emulsion 2.4-CM2-HS > foam 2.4-CM2-HS Emulsion 2.4-CM2-LS < foam 2.4-CM2-LS	

Table 4. 10: Effect of the experimental variables on the cohesion

b) *Internal angle of friction*

The angles of internal friction range between 12.6^o and 45.5^o as shown on Table 4.8. Figure 4.18 shows a plot of angle of friction against the four combinations of densities and saturation level.
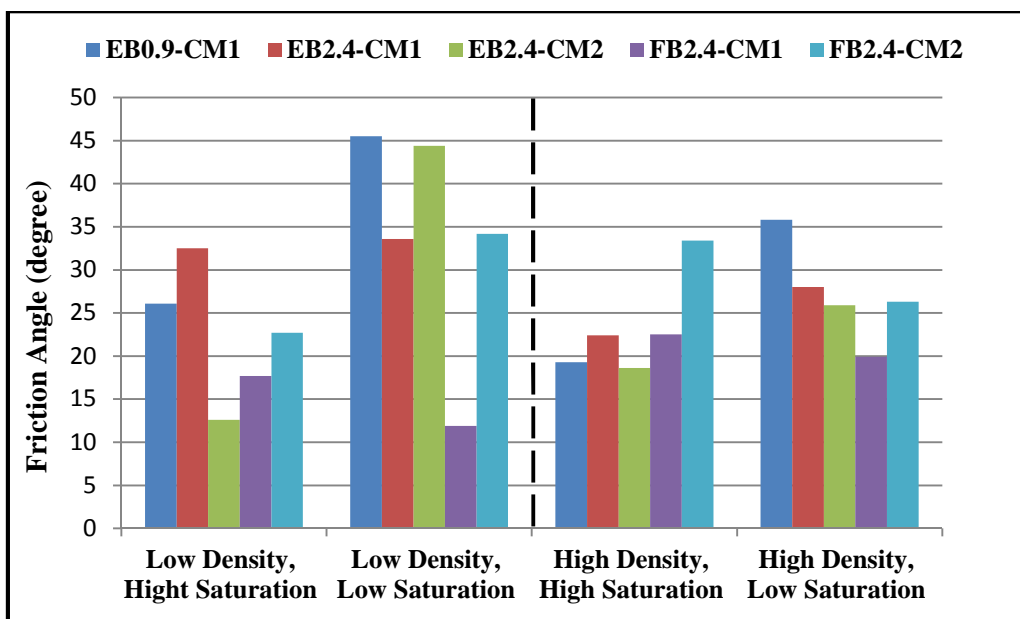
Figure 4. 18: Angle of friction for BSMs from R35

The friction angle is expected to decrease with the increase in cement content. The theory is confirmed in the study with BSM-emulsion mixes only. The same trend was not noticed in the case of BSM-foam mixes where an increase of the cement content led to an increase of the internal angle of friction values.

At other equal test variables, it was observed that most of the mixes compacted at low density possess internal angle of friction values higher than that of the mixes compacted at high density. This shows the effect of compaction on the shear properties of the material. Except for Mix FB2.4-CM1, the internal angles of friction of mixes of high density are lower compared to those of low density.

It was observed that in the case of emulsion mixes, the value of the angle of internal friction increases when the saturation level decreases. The same trend was not observed with foamed bitumen mixes where the values of angle of internal friction were reducing with the reduction of the saturation level.

Table 4. 11: Effect of the experimental variables on the angle of friction

Experimental variables	Emulsion mixes		Foamed bitumen mixes	
	φ	C	φ	C
Adding 1% of cement	Decrease ↓	Increase ↑	Increase ↑	Increase ↑
Increasing the density level	Decrease ↓	Increase ↑	-	Increase ↑
Increasing the saturation level	Decrease ↓	Decrease ↓	Increase ↑	Decrease ↓
Type of treatment	Emulsion 2.4-CM1 > foam 2.4-CM1 Emulsion 2.4-CM2 < foam 2.4-CM2			

c) *Interaction between cohesion and friction angle*

Interpretation of cohesion and friction angle was also made by analysing the interaction between the two parameters. More often, an increase in cohesion is usually associated with a reduction in friction angle. However, it was not always the case in this study as shown in Figure 4.19, where an example of normalised values of cohesion and friction angle for the mix with 2.4 per cent emulsion bitumen and 1 per cent cement is plotted. It can be seen that in both high and low-density condition, both cohesion and friction angle increase as the saturation level decreases: This show the effect of moisture on both parameters. In other words, at equal density the shear parameter values increase as the moisture condition decreases.

Note: The values were normalised by using the highest value of each parameter.
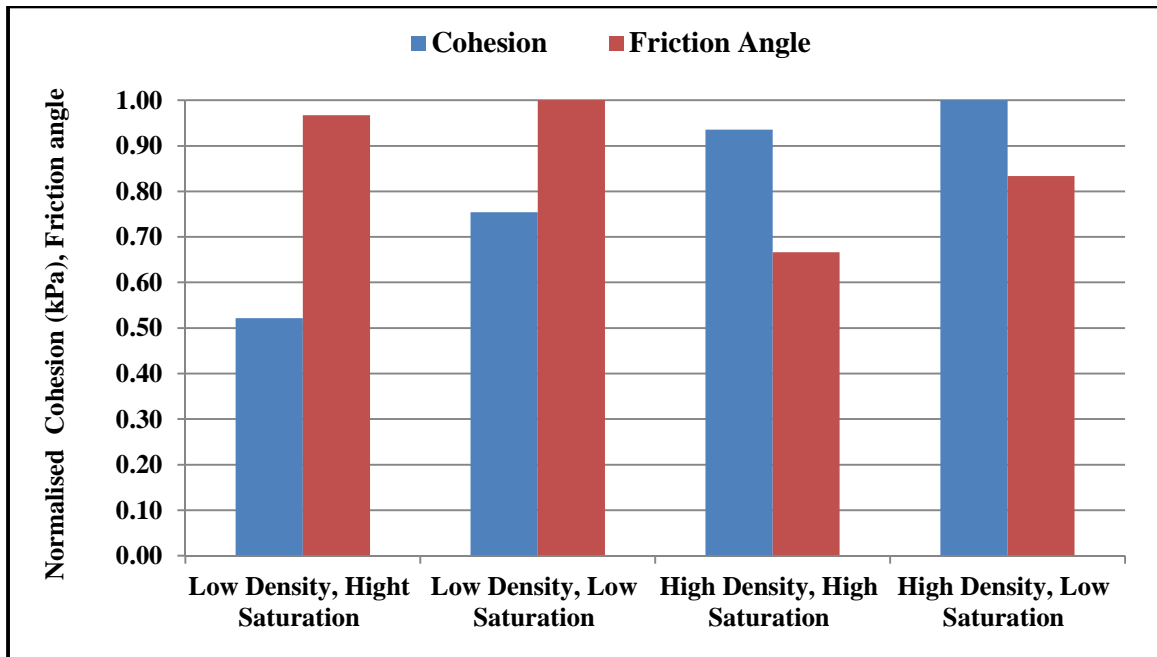
Figure 4. 19: Interaction between cohesion and friction Angle (Mix: EB2.4-CM1)

d) *Maximum allowable shear stress*

Analysing the mixes shearing parameters (cohesion of angle of friction) in isolation might not always give accurate interpretation of the properties of the mixes. Therefore, it is important to evaluate the performance of the mixes based on their allowable maximum shear stress, because the parameter is both dependent on the cohesion and the angle of friction as illustrated in Figure 4.20 for mix FB2.4-CM1. The illustrations of the maximum allowable shear stress of the other mixes are included in Appendix E.
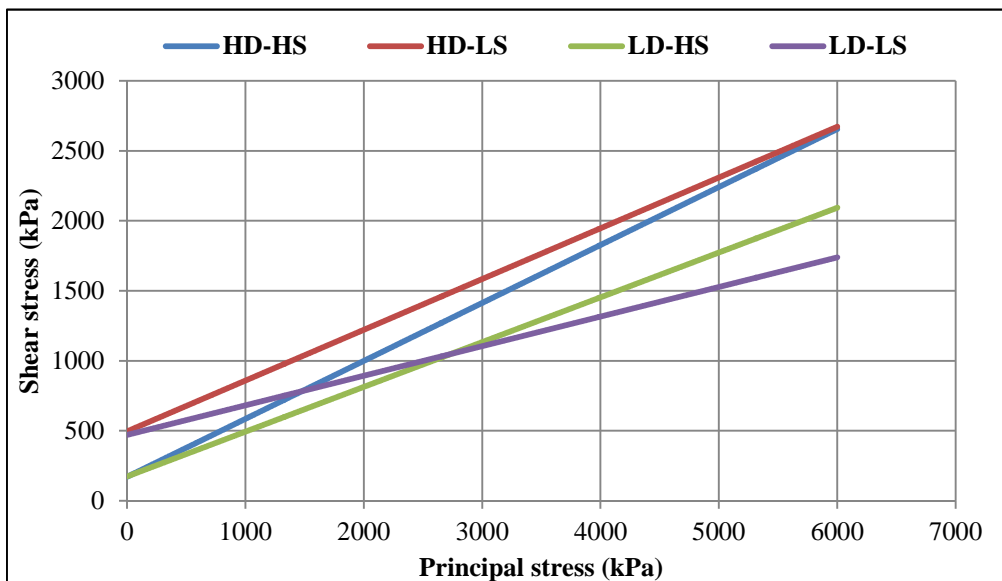
Figure 4. 20: Maximum shear stress for mix FB2.4-CM1

From the graphs, it can be seen that increasing the density result in an increase of the maximum allowable principal stress that the material can endure. The same effect is observed when the percentage of active fillers (cement) is increased from 1% to 2%. On the other hand, contrary to the effect of relative density, an increase in the saturation level results in a decrease of the maximum allowable principal stress of the material as shown in Figure 4.21.
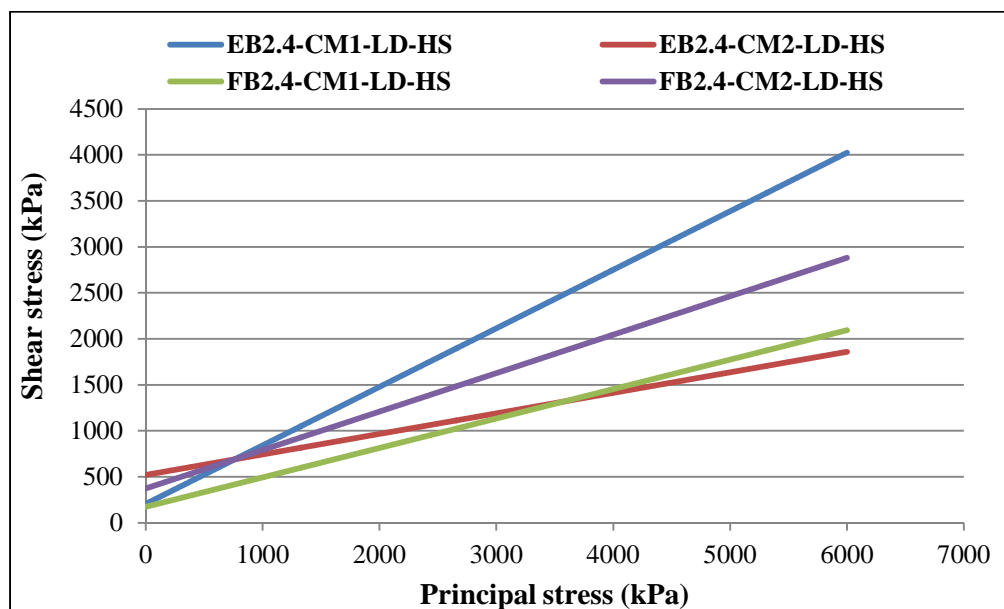
Figure 4. 21: Effect of the cement content on the maximum shear stress at low-density and high-saturation

Table 4. 12: Effect of the experimental variables on the shear stress

Experimental variables	Emulsion mixes	Foamed bitumen mixes
Adding 1% of cement	Increase ↑	Increase ↑
Increasing the density level	Increase ↑	Increase ↑
Increasing the saturation level	Decrease ↓	Decrease ↓
Type of treatment	Emulsion 0.9-CM1 > Emulsion 2.4-CM1 Emulsion > Foam	

e) *Critical analysis of the shear parameters*

It is important to note that the friction angles and cohesion values obtained in this study are respectively low and high compared to values obtained with bitumen stabilised materials in previous research and in the industry. In the CSIR report CR-2005/01 where similar tests were conducted on

crushed hornfels treated with emulsified bitumen and of sand treated with emulsified bitumen and foamed bitumen, cohesion values obtained are slightly lower whereas friction angle values are higher (greater than 35^0). The low angle of friction obtained in this research resulted from the relatively low stresses obtained at high confinement. A possible cause is confining air pressure infiltrating the particles of the specimen through leakage of the membrane. In addition, if the infiltrated air could not be drained out of the specimen, this could have reduced the effective confinement.

Is it also important to mention that standard mix design of BSMs includes 10 triaxial tests per mix (e.g. at BSM laboratories). It is unusual for research of BSM mixes to include only 3 equivalent tests, but material resources did not allow for the desired additional tests to be carried out.

In the light of this, the shear parameters obtained in this study should not be used for design or modelling purpose.

f) *Estimated compressive and tensile strength from Mohr Coulomb failure line*

Some other important parameters that can be derived from the monotonic triaxial test are the compressive and tensile strength. The failure line from the Mohr Coulomb diagram can be used for their estimation. As shown in Figure 4.22, two scenarios are possible: making σ_3 equal to zero in the equation to estimate the compressive strength, and making $\sigma_{1,f}$ equal to zero in the same equation to estimate the tensile strength.
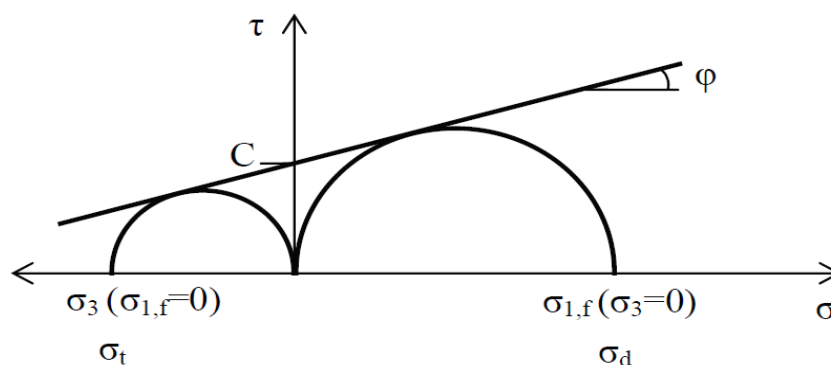
$$\sigma_{1,f} = \frac{1+\sin\varphi}{1-\sin\varphi}\sigma_3 + \frac{2 \cdot C \cdot \cos\varphi}{1-\sin\varphi}$$

Figure 4. 22: Compressive and tensile strength in Mohr Coulomb diagram (Ebels, 2008)

The compressive and tensile strength was estimated for all of the mixes at the four combinations of density and saturation. The results are presented in Table 4.13. Figure 4.23 shows the illustration of the specimens tested at high density and low saturation. The illustration of the other combination of density and saturation level is shown on the figures of Appendix F.

Table 4. 13: Estimated compressive and tensile strength from Mohr Coulomb diagram ($\sigma_3 = 0$)

Mixes	Cement (%)	Testing Combination		Compressive strength (kPa)	Tensile strength (kPa)
		Relative Density (kg/m3)	Saturation Level (%)		
EB0.9-CM1	1	High	High	888.86	447.63
		High	Low	1115.52	292.69
		Low	High	908.77	353.31
		Low	Low	1270.62	212.58
EB2.4-CM1	1	High	High	1098.62	491.61
		High	Low	1307.19	472.17
		Low	High	746.83	225.04
		Low	Low	1105.70	317.31
EB2.4-CM2	2	High	High	1351.22	697.83
		High	Low	1899.72	746.00
		Low	High	1297.55	833.44
		Low	Low	1594.44	281.18
FB2.4-CM1	1	High	High	511.17	228.23
		High	Low	1414.50	695.06
		Low	High	474.64	252.86
		Low	Low	1160.35	762.59
FB2.4-CM2	2	High	High	1391.71	402.85
		High	Low	1970.92	761.63
		Low	High	1120.03	496.56
		Low	Low	1741.31	487.59
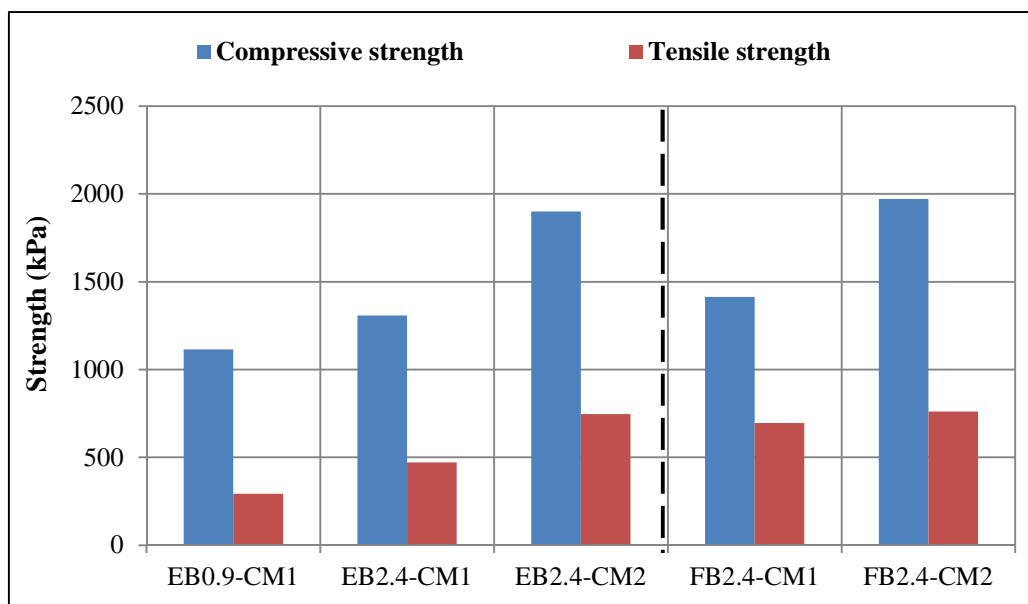
Figure 4. 23: Estimated compressive and tensile strength, mixes of high density-low saturation

The same trend of changing in shear parameters and the maximum allowable stress due to experimental variable were also observed with the estimated compressive and tensile strength from monotonic triaxial. It can be seen clearly that the addition of 1% cement results in an increase in the compressive and tensile strength of the mixes regardless of the type of treatment and the four test combinations. No special trend due to the type of treatment used was observed on the data. However, it was noticed that at low density, mixes treated with 0.9% emulsion and 1% cement had higher compressive and tensile strength than mixes treated with 2.4% emulsion and 1% cement. Looking at the test combination alone only, the compressive and tensile strength increase with the increase of density and the reduction of the saturation level as illustrated in Figure 4:24 for mix FB2.4-CM2 can be seen.
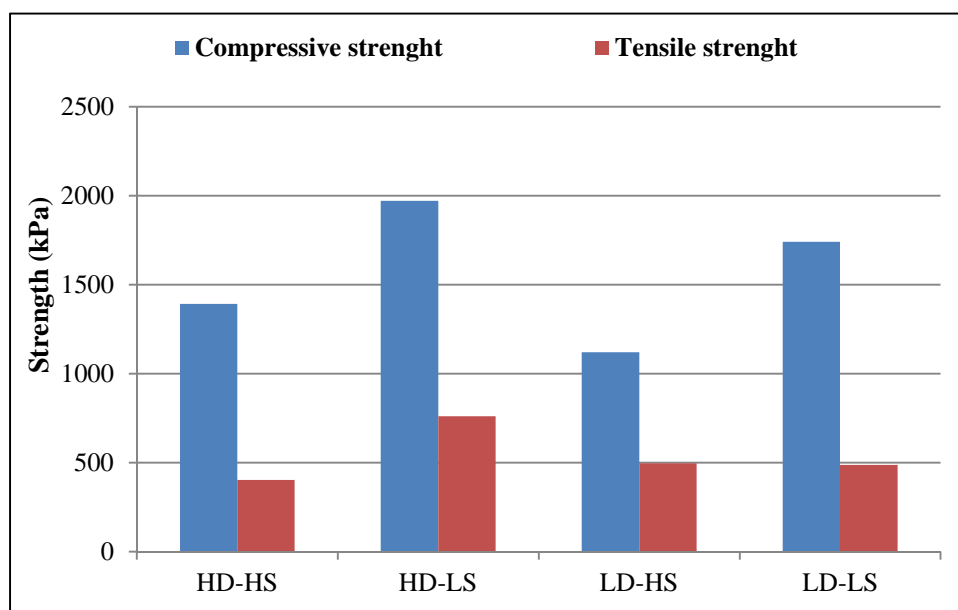
Figure 4. 24: Effect of the test combination the estimated compressive and tensile strength

Table 4. 14: Summary of the experimental variables of effects on the estimated compressive and tensile strength the monotonic triaxial test

Experimental variables	Emulsion mixes	Foamed bitumen mixes
Adding 1% of cement	Increase ↑	Increase ↑
Increasing the density level	Increase ↑	Increase ↑
Increasing the saturation level	Decrease ↓	Decrease ↓
Percentage and type of treatment	Emulsion 0.9-CM1-LD > Emulsion 2.4-CM1-LD	

4.3.2   The Dynamic Triaxial Test (Resilient Modulus)

4.3.2.1   *Presentation of the results*

Specimens for resilient modulus assessment were prepared at a range of relative densities and saturation level. The relative densities and saturation levels of specimens at the time of resilient modulus testing are presented in Table 4.15. Data comprises of the average and the coefficient of variation per test combination. The densities and saturation are fairly close to the one obtained during monotonic triaxial test. They also show the same trend of variability (i.e. a very small variability in relative density compared to the saturation level).

Table 4. 15: Relative densities and saturation level during dynamic triaxial testing

Mixes	Testing Combination		Relative Density Average (%)	Average Saturation Level (%)
	Relative Density (kg/m3)	Saturation Level (%)		
EB0.9-CM1	High	High	72.8	76.0
	High	Low	73.1	22.7
	Low	High	71.3	79.3
	Low	Low	71.6	23.5
EB2.4-CM1	High	High	75.4	85.2
	High	Low	75.7	25.1
	Low	High	72.8	80.7
	Low	Low	73.0	25.5
EB2.4-CM2	High	High	74.5	82.4
	High	Low	75.0	23.6
	Low	High	73.2	86.2
	Low	Low	73.1	23.9
FB2.4-CM1	High	High	74.3	82.5
	High	Low	75.2	24.3
	Low	High	73.8	86.2
	Low	Low	72.4	26.1
FB2.4-CM2	High	High	74.7	82.6
	High	Low	75.7	28.0
	Low	High	73.6	88.4
	Low	Low	73.3	26.8

The resilient modulus of each specimen was determined from the linear variable displacement transducer (LVDT's) measurements, at five confinement pressures for each deviator stress ratio, following the methodology described in in Chapter 3 Section 3.5.5. It must be noted that out of the three LVDT's mounted on the specimen, only two were used for resilient modulus calculations. This was due to the malfunctioning and the unrealistic results of the third LVDT's.

The Mr (resilient modulus) values obtained for all the mixes range between 165.3 MPa at low confinement and low stress ratio to about 1570.59 MPa at high confinement and high stress ratio. The test results of each specimen are presented in the tables of Appendix G. The tables also contain the average and the coefficient of variation of the Mr per mix at the four combinations of relative density and saturation level.

A sample of data obtained at 20 kPa confinement and 20% stress ratio for all the mixes is presented in Table 4.16. However, on the bar chart of Figure 4.25, only the Mr ranges of values obtained at low density and high saturation level of the confinement and stress ratio mentioned above, are plotted. The bars are shaded between the average ± the standard deviation, while the line represent the full range of data.

Table 4. 16: Average resilient modulus of all mixes at 20 kPa confinement and 20% stress ratio

Mixes	Relative Density (kg/m3)	Saturation Lavel (%)	Average resilient modulus (MPa)
EB0.9-CM1	High	High	278.1
	High	Low	615.4
	Low	High	259.1
	Low	Low	508.3
EB2.4-CM1	High	High	351.3
	High	Low	573.8
	Low	High	306.3
	Low	Low	605.2
EB2.4-CM2	High	High	845.6
	High	Low	1106.6
	Low	High	684.7
	Low	Low	735.2
FB2.4-CM1	High	High	426.3
	High	Low	669.3
	Low	High	389.0
	Low	Low	779.4
FB2.4-CM2	High	High	543.8
	High	Low	880.6
	Low	High	482.2
	Low	Low	1035.1
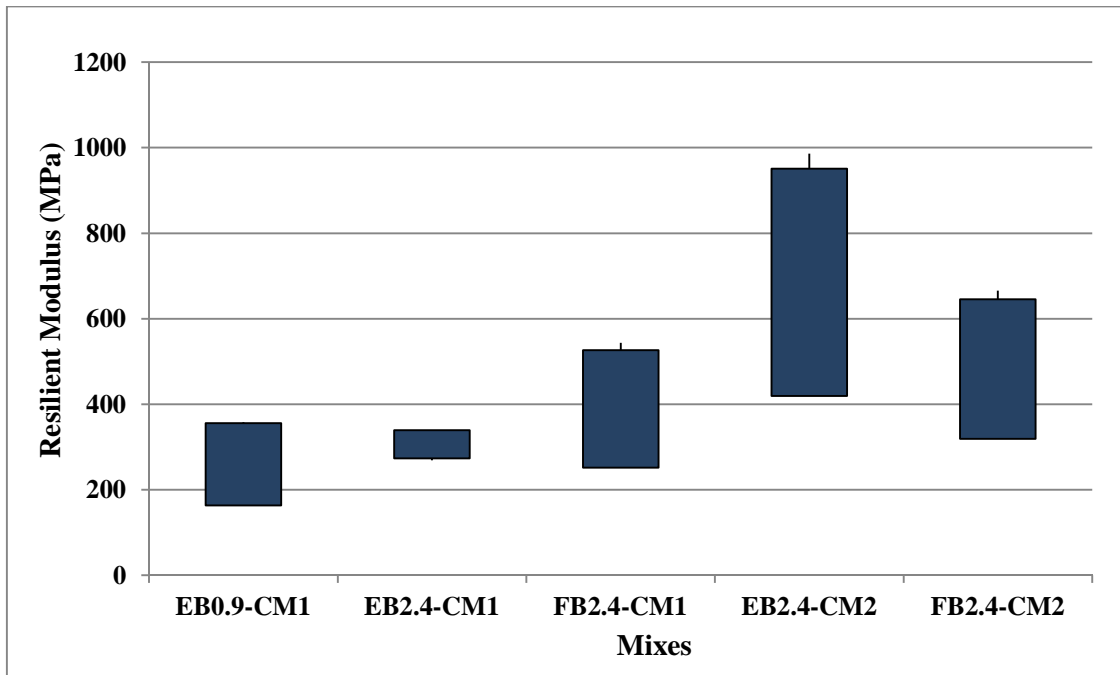
Figure 4. 25: Ranges of Mr Values, of all mixes at low confinement-low stress ratio (20 kPa confinement-20% stress ratio)

On a general observation, we could see that the resilient modulus of the mix increases as the stress ratio and the confinement pressure increases, as shown in Figure 4.26. This increase in stiffness was due to the increase of the stress ratio in relation to the confinement pressure that was observed for all the mixes regardless the type of treatment, the percentage of active fillers and the testing conditions.
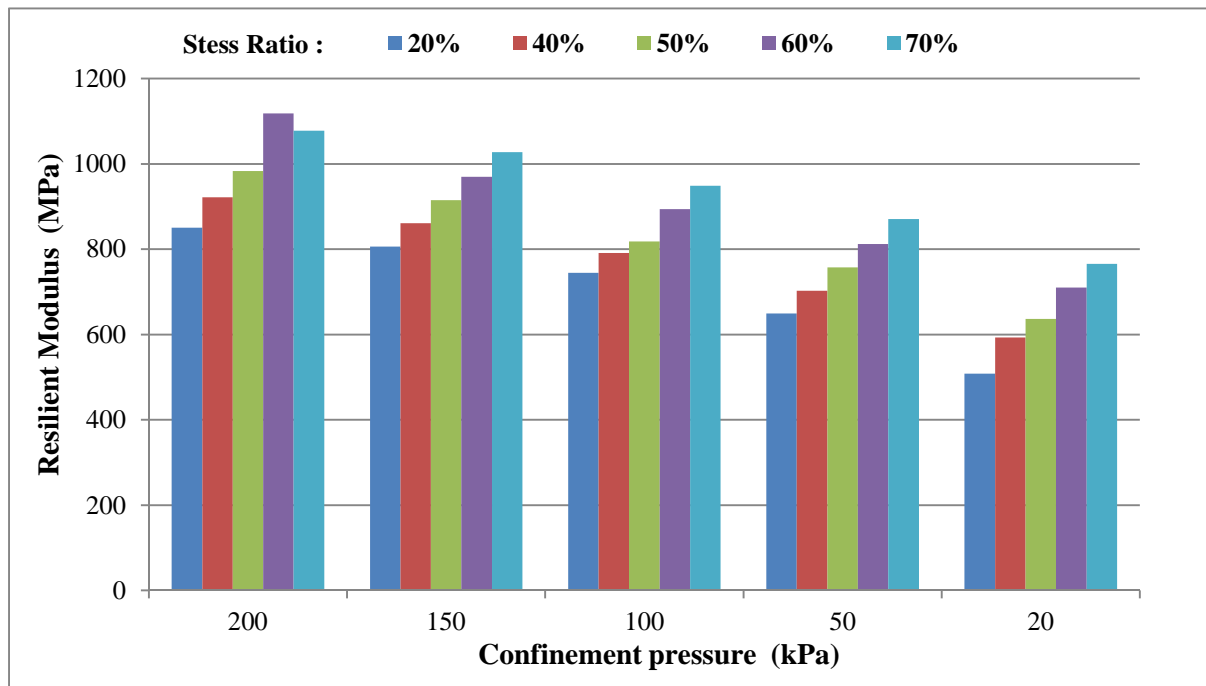
Figure 4. 26: Resilient modulus values-EB0.9-CM1-LD-LS

The short-term dynamic triaxial test was analysed by plotting the calculated resilient modulus versus the total stress on log-log scale. A typical example of this plot for mix FB2.4-CM2-HD-HS is

presented in Figure4: 27. It can be seen in Figure 4.27 and on the figures of Appendix H that the stiffness behaviour of the mix is greatly influenced by the deviator stress ratio. This shows the stress dependent behaviour of the mixes tested.
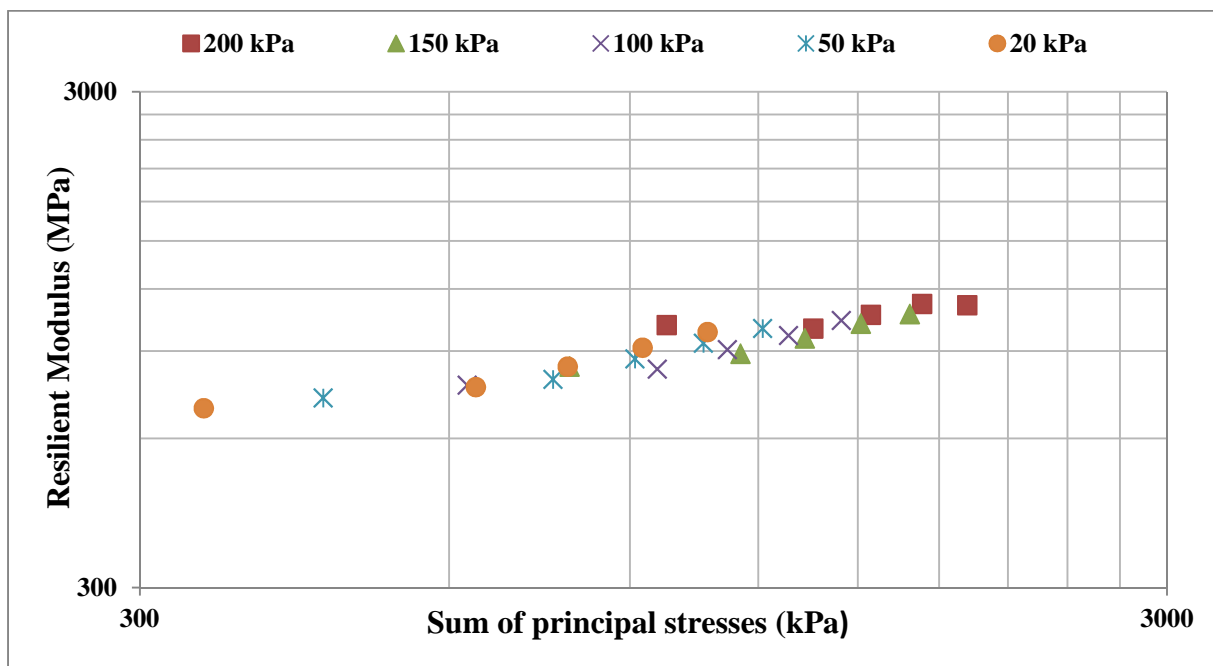
Figure 4. 27: Example of resilient modulus Vs. Sum of principal stress-FB2.4-CM2-HD-HS

In order to assess the resilient modulus response of the mixes at a range of relative densities, saturation levels, stress ratio and confinement, there was a need to build a model.

4.3.2.2   *Modelling of resilient modulus mechanical behaviour*

Many models exist to describe the stress dependent behaviour of material. However, the only model applied in this research is the M_r-θ model, which is also the most common and rudimentary model. The model is based on the following equation.

$$M_r = K_1 \left(\frac{\theta}{\theta_0}\right)^{K_2}$$

Equation 4.1

Mr = resilient modulus in MPa

Θ = sum of principal stresses ($\boldsymbol{\sigma}_1 + \boldsymbol{\sigma}_2 + \boldsymbol{\sigma}_3$) in kPa

Θ_0 = reference stress = 1 kPa

K_1, K_2 = model coefficients

The test data was fitted into the model by the means of non-linear regression analysis. The M_r-θ model plots of all mixed and specimens are presented in Appendix H. Figure 4.28 shows an example of the obtained M_r values from the test, plotted with the fitted Mr-θ model against the sum of principal stresses.
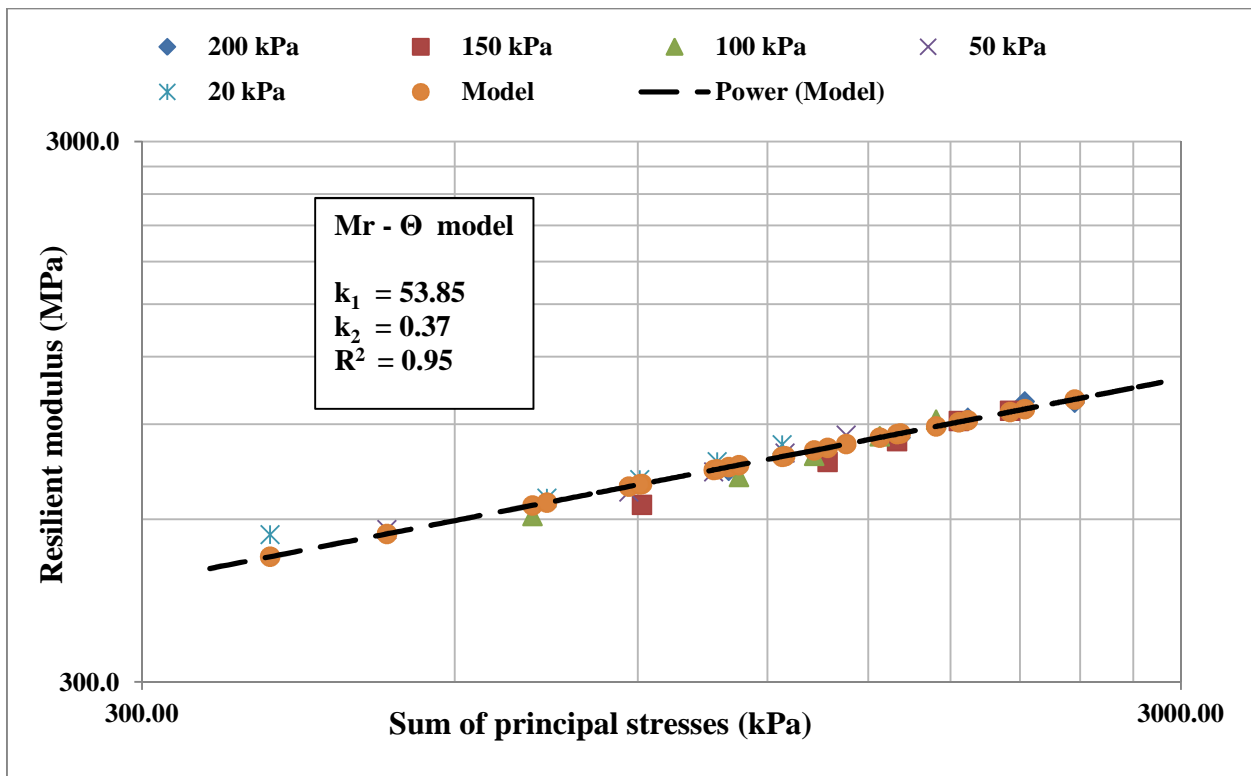
Figure 4. 28: EB 2.4–CM2–LD–LS-01, date and Mr-θ model

The M_r-θ model coefficients k_1 and k_2 were determined for all specimens by means of regression analysis. A summary of the model coefficients that give the best fit to the data per mix and test combinations is given in Table 4.17. The magnitude of the coefficient model obtained does not deviate significantly from the ones indicated in the past research, Jenkins (2000) and Ebels (2008). The R^2 values obtained from the analysis are also presented in Table 4.17. It can be seen that most of the mixes have a R^2 value are greater than 0.80. However, some mixes (EB2.4-CM1-LD-LS and FB2.4-CM1-LD-HS) have relatively low R^2 value (0.57 and 0.35 respectively). These are the mixes for which M_r-θ relationship gave a poor correlation. Though, their coefficient of correlation is low, it can be seen on the graphs in Appendix H that these two mixes do not oppose the Mr-θ model trend observed with others mixes of high coefficient of correlation which is a noticeable increase of the resilient modulus value with the increase of the bulk stress at constant confinement pressure.

Table 4. 17: Mr-θ model coefficients K_1 and K_2

Mixes	Testing Combination		Model coefficients		R^2
	Relative Density	Saturation Level	k_1	k_2	
EB0.9-CM1	High	High	14.05	0.52	0.82
	High	Low	36.05	0.47	0.77
	Low	High	9.14	0.51	0.83
	Low	Low	55.36	0.36	0.74
EB2.4-CM1	High	High	35.51	0.41	0.86
	High	Low	71.18	0.33	0.86
	Low	High	41.49	0.38	0.74
	Low	Low	29.59	0.48	0.57
EB2.4-CM2	High	High	78.84	0.39	0.93
	High	Low	177.58	0.29	0.92
	Low	High	77.77	0.33	0.85
	Low	Low	64.00	0.38	0.94
FB2.4-CM1	High	High	94.86	0.26	0.72
	High	Low	199.59	0.21	0.81
	Low	High	158.95	0.12	0.35
	Low	Low	153.11	0.22	0.79
FB2.4-CM2	High	High	50.55	0.38	0.83
	High	Low	217.79	0.22	0.85
	Low	High	71.49	0.38	0.92
	Low	Low	92.93	0.34	0.79

4.3.2.3   Effect of the experimental variables on the resilient modulus

The M_r-θ model's coefficients of Table 4.17 were used to predict the resilient modulus of the mixes at the four combinations of relative density and saturation level for specific values of bulk stresses. The obtained Mr values at low (500 kPa), medium (900 kPa) and high (1500 kPa) are presented in Table 4.18. However, only the values obtained at low density-high saturation are illustrated in figure 4.28. The illustration of other test combination is presented in Appendix I.

Table 4. 18: Predicted Mr Values from the Mr-θ model at low, medium and high bulk stress

Mixe	Testing combination		Values of Mr		
	Relative density	Saturation level	Mr (θ =500 kPa)	Mr (θ =900 kPa)	Mr (θ =1500 kPa)
EB0.9-CM1	High	High	355.75	482.93	629.86
	High	Low	668.99	881.86	1121.16
	Low	High	217.48	293.50	380.85
	Low	Low	518.59	640.80	770.17
EB2.4-CM1	High	High	453.87	577.55	712.11
	High	Low	553.37	671.83	795.18
	Low	High	440.10	550.24	668.13
	Low	Low	584.32	774.79	990.08
EB2.4-CM2	High	High	889.91	1119.18	1365.91
	High	Low	1076.70	1276.80	1480.68
	Low	High	604.61	734.03	868.80
	Low	Low	678.87	848.77	1030.61
FB2.4-CM1	High	High	477.33	556.14	635.14
	High	Low	736.08	832.78	927.08
	Low	High	335.07	359.56	382.29
	Low	Low	600.87	683.81	765.14
FB2.4-CM2	High	High	536.20	670.40	814.02
	High	Low	854.70	972.68	1088.37
	Low	High	758.32	948.11	1151.22
	Low	Low	768.79	938.85	1116.93

It was found that increasing the percentage of active filler content from 1 to 2% resulted in a significant increase of the resilient modulus values. When comparing the mix EB-2.4-CM1 with EB-2.4-CM2 at a bulk stress of 500 kPa, the Mr values range from 453.87 MPa to 584.32 MPa and from 604.61 MPa to 1076.70 MPa respectively. The same trend is also observed when comparing mix EB-2.4-CM1 with EB-2.4-CM2 both at low and high bulk stress. This increase of the Mr with the increase of cement content is illustrated in Figure 4.29 for Mr values obtained for a combination of confinement and stress ration and in Figure 4.30 for predicted Mr values obtained from the model equations are plotted at a specific bulk stress of 500 kPa. It can also be seen that the increase of the Mr due the increase of cement was more pronounced for bitumen emulsion mixes than foamed bitumen mixes.
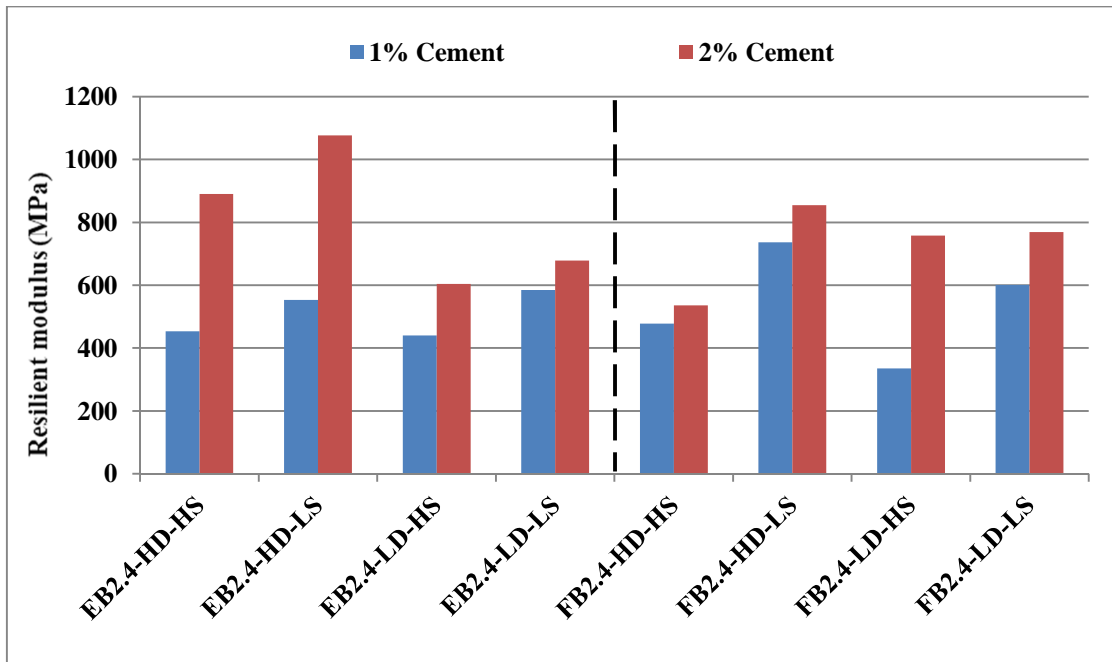
Figure 4. 29: Effect of the cement content on the Mr values obtained from the tests data
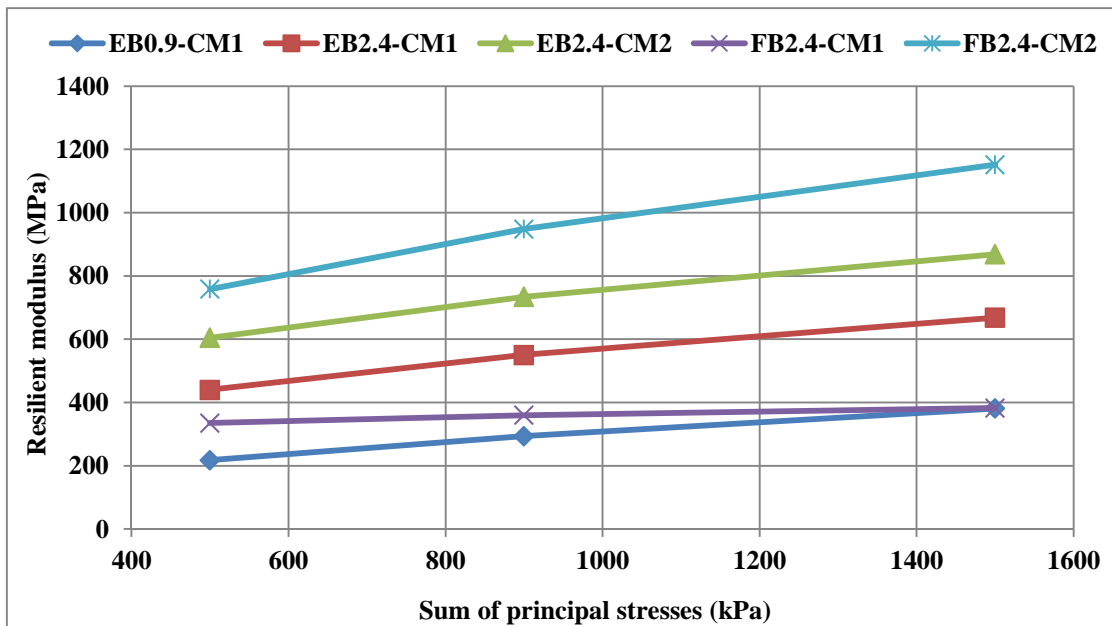
Figure 4. 30: Effect of the cement content on the Mr values obtained from the model at high density-high saturation

The increase in saturation levels caused a significant decrease of the M_r values as shown in Figure 4.31. It was also noticed that bitumen emulsion mixes were very sensitive to moisture increase compared to foamed bitumen mixes.
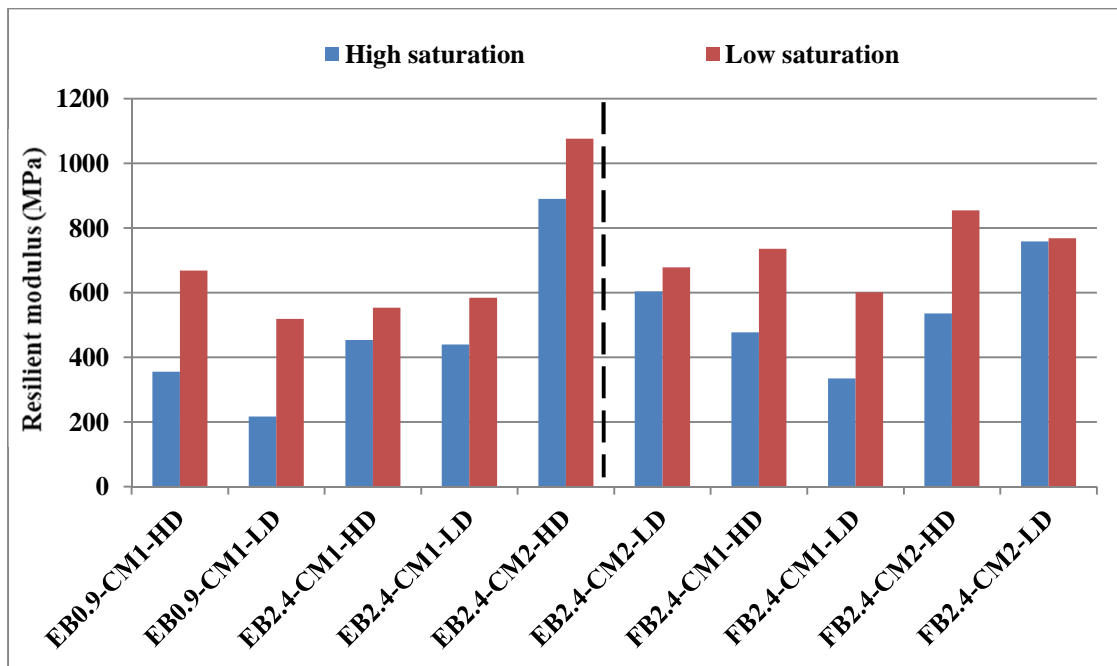
Figure 4. 31: Effect of the saturation level on the Mr values obtained from the tests data

It was not easy to identify the effect of the type of treatment on the resilient modulus response of the mixes. In addition, the susceptibility to moisture due to the type of treatment (emulsion or foam) to distinguish between mixes that are more resistant to moisture than those that are less resistant to moisture was not evident. However, emulsion mixes showed higher Mr value than foam mixes at 1% cement content.

Despite the small gap in the densities achieved between some specimens compacted at high density and those compacted at low density, the effect of the density on the Mr was evident. Except mix FB2.4-CM2-HD, all other mixes had high Mr values at high density as expected.

Influence of the density: As illustrated in Figure 4.32 , it can be seen that increasing the density result to an improvement of the shear properties of the material making it less susceptible to deformation.
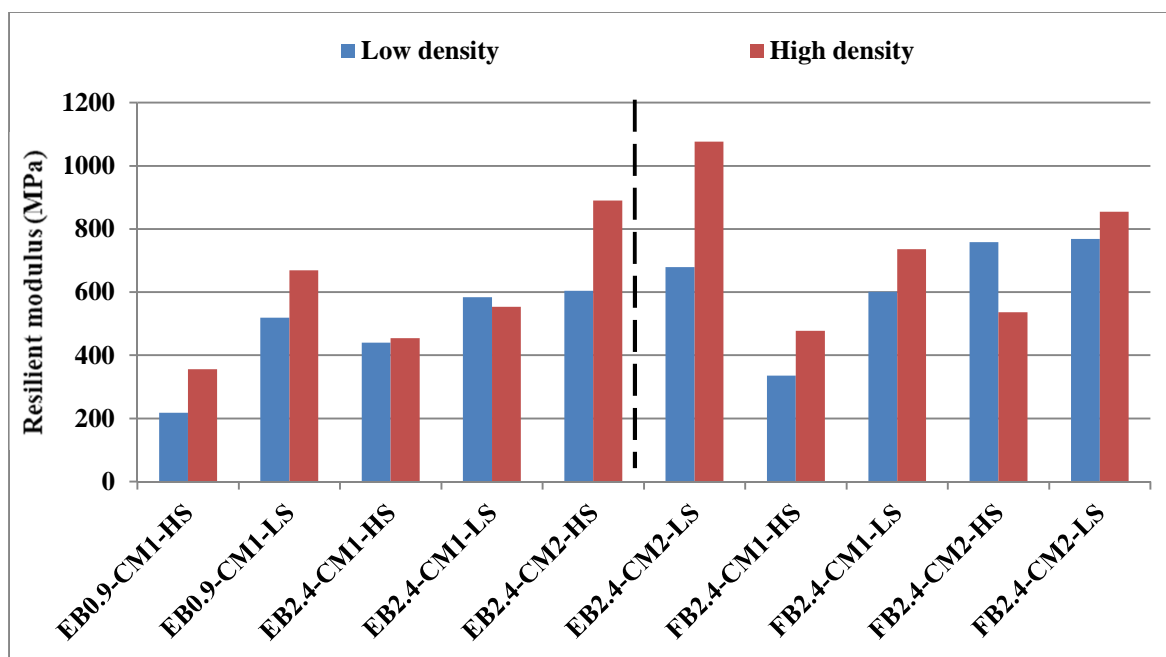
Figure 4. 32: Effect of the saturation level on the Mr values obtained from the tests data

It was clear from the test results data and from the model that the density and saturation level have an effect on the Mr. However, it can been seen from Figure 4.33 and the other figure of Appendix K that the Mr curve of mixes tested at low saturation are above those of mixes tested at high and low density. In other words, mixes of low density and low saturation have higher Mr values than mixes of high density and high saturation. This shows the significance of the saturation level variable. Though the density and saturation level both affect the Mr, the effect of the saturation level is more pronounced than the one of the density. This shows the susceptibility of bitumen stabilized materials to moisture conditions. The presence of excessive water creates a hydrodynamic effect within the BSM layer due to dynamic loading, even though the layer could have been well compacted beforehand. This could lead major moisture damages that affect the performance of the layer.
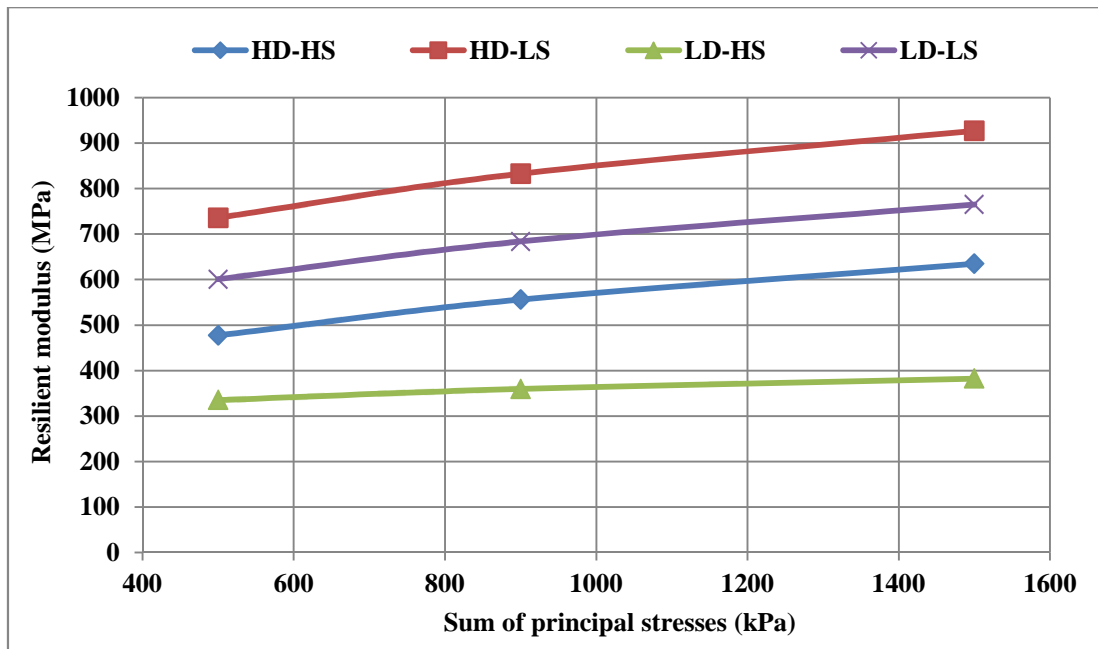
Figure 4. 33: Effect of density and saturation level on the predicted Mr values from the model, mix FB2.4-CM1

It was not easy to identify the effect of the type of treatment on the resilient modulus response of the mixes. However, emulsion mixes showed higher Mr values than foamed bitumen mixes at 1% cement content. But this same trend was not observed with the two mixes tested at low density with 2% cement content. The general effects of the experimental variables on the resilient modulus values are summarised in Table 4.19.

Table 4. 19: Effect of the experimental variables on the resilient modulus

Experimental variables	Emulsion mixes	Foamed bitumen mixes
Adding 1% of cement	Increase ↑	Increase ↕
Increasing the density level	Increase ↑	Increase ↕
Increasing the saturation level	Decrease ↓	Decrease ↓
Type of treatment	Emulsion 0.9-CM1 < Emulsion 2.4-CM1 Emulsion > Foam	

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

This chapter provides a series of conclusions on the study that has been conducted as well as some recommendations for further studies on the topic.

5.1 Introduction

Throughout the previous chapter (Material testing, results and findings), conclusions where provided after presentation and analysis of results. Therefore, this last chapter of the dissertation has to do with synthesis of the findings as well as the recommendations that follow out of the research conducted. A practical implication of the findings on road construction and rehabilitation using bitumen stabilised materials is also provided.

5.2 Conclusions

5.2.1 Influence of the BSM variables on the strength (ITS and UCS)

- At a constant bitumen content, it was found that increasing the percentage of active fillers from 1 to 2 % significantly increase the ITS and UCS values regardless of the type of treatment. Up to 128 % and 104 % increase was observed with ITS and UCS values of emulsion bitumen specimens tested in dry conditions. However too much active fillers in the mixes may result in poor resistance to rutting and the material may exhibit brittle behaviour at low temperature.

- Results show that the moisture content and the percentage of active filler have a significant effect on the strength of the treated material.

- It was found that increasing the cement content and the percentage of bitumen increases the moisture resistance of the mixes.

- The tensile strength retained comparing soak and unsoaked ITS test was determined for all the mixes.

- Bitumen emulsion mixes was found to have improved resistance to moisture effects compared to foamed bitumen mixes in terms of the strength of the mix.

5.2.2 Influence of the BSM variables on the flexibility indicator parameters

Flexibility indicator parameters (i.e. Fracture energy, strain and displacement break) were determined from the load-displacement curves of ITS and UCS test.

- An increase in flexibility was noticed at low cement content. Mixes with 1% cement and 1% lime were found to have higher displacement/strain at break and high fracture energy

compared to mixes with 2% cement and mixes where 1% cement and 1% lime were used in combination. In other words, increasing the percentage of bitumen increases the flexibility but reduces the strength while increasing the percentage of active fillers increase the strain but reduces the flexibility of the material.

- An increase in displacement and strain at break was noticed when increasing the percentage of bitumen content at a given active filler content. Increasing the percentage of bitumen at a fix filler content increases the flexibility but slightly reduces the strength. The same observation was made by Long (2005) after conducting a series of test on foamed bitumen sand treated materials.

- Statistical analysis revealed that the independent variable that have the most significant effect on the fracture energy with a direct effect on flexibility is the cement content followed by the bitumen content then the type of treatment and finally the testing condition.

5.2.3 Influence of the BSM variables on the shear parameters

Monotonic triaxial tests were performed to assess the shear strength of the mixes at different ranges and combinations of density, saturation level and active fillers content. The conclusions from the test results are as it follows:

- It is well known from the literature that bitumen emulsion and foamed bitumen stabilisation increase the shear strength of the parent material (TG2, 2009; Jenkins, K.J. 2002; Ebels, L.J. 2008). It was found that increasing the percentage of active filler in terms of cement from 1% that 2% by mass significantly increases the shear strength of the mix. This was characterised by the increase in cohesion values and the reduction of angle of internal friction.

- It is evident that the shear parameters are dependent on the level of density. The cohesion of specimens compacted at high density were found to be higher than that of the specimens compacted a low density.

- It was also found that the shear strength of the mixes are influenced by the moisture conditions. For in most of the cases, a significant increase in angle of internal friction was noticed as the moisture content at testing was increased.

5.2.4 Influence of the BSM variables on the resilient modulus

Dynamic triaxtial tests were performed to determine the resilient modulus of the mixes at different ranges and combination of density, saturation level and active fillers content. The $Mr\text{-}\theta$ model was computed on the data in other to determine the resilient modulus of the mixes as a function of the test variables and combination of variables. The followings observations were made:

- It was found that the resilient modulus of the mixes increases as the sum of the principal stresses increase. This shows the stress dependent behaviour of bitumen stabilised materials, which was accurately described by the *Mr-θ* model.

- Increasing the percentage of active fillers from 1% to 2% resulted in an increase of the resilient modulus. All mixes with 2% cement had higher resilient modulus than their correspondent mixes with 1% cement tested in the same conditions.

- Increasing the saturation levels caused a significant decrease of the Mr values, while increasing the density caused an increase in Mr values. Although the density and saturation level both affect the Mr, it was noticed the effect of the saturation level is more pronounced than the one of the density. For mixes of low density and low saturation had higher Mr values than mixes of high density and high saturation.

- The effect of the type of treatment on the resilient modulus response of the mixes was not perceptible. However, it was noticed that bitumen emulsion mixes had Mr values slightly larger than foamed bitumen mixes especially at 1% cement content.

5.2.5 General conclusions

At the end of this research, it can be concluded from the results that flexibility of BSM is a property that can be measured during the mix design. Fracture energy, strain and displacement at break from ITS and UCS test were able to give us an indication on the main mix components that govern the flexibility property in BSM mixes. In the case of fracture energy for instance, it was found that increasing the percentage of cement reduces the fracture energy. This implies that the material dissipates less energy before fracture (damage) occurs, which is the characteristic of less flexible materials. The contrary effect was noticed with the increase of bitumen content. Higher bitumen content improvement the flexibility but one should also keep in mind that apart from decreasing the strength of the mix, mixture with high bitumen content are costly and more susceptible to temperature.

On a general observation, it was found that the mix variables have contrary effects on the strength and the flexibility. The statistical analysis revealed that the strength and the stiffness of the mixes are more influenced by the testing condition (saturation level) and the percentage of active filler added than the bitumen content or the type of treatment. On the other hand, the flexibility seems to be more influenced by the amount bitumen added and the percentage of active filler than the type of treatment or the testing condition. Therefore, the bitumen and cement contents should be balanced in order to optimize the performance of the mix. As for the combined effect of independent variables, the

combined effect of cement and bitumen content was found to be significant for both strength and flexibility.

The triaxial test revealed that that increasing the percentage of active filler in terms of cement from 1% that 2% by mass significantly improved the shear strength and the resilient modulus of the mix. The same observation was made on mixes compacted at high density and tested at low saturation level. Moreover, an increase in resilient modulus values was noticed as the sum of the principal stresses increased. This shows the stress dependent behaviour of bitumen stabilised materials.

It was found that the retained tensile strength of mixes with 2% cement was higher than that of mixes with 1% cement. This indicates that increasing the percentage of active fillers reduces the moisture sensitivity of the mixes. Castedo made the same observations after he added 1 and 2 % Portland cement on foamed bitumen mixes (Castelo et al, 1986). Moreover, it was found that bitumen emulsion mixes are more resistant to moisture effects compared to foamed bitumen mixes.

It was found that the percentage of active fillers seems to have a significant influence on the moisture sensitivity. The moisture sensitivity, evaluated through the tensile strength retained (TSR) have shown that increasing the active filler content improved the moisture sensitivity of the mixes.

5.3 Recommendations

- Tests on flexibility are very sensitive to specimen preparation, handling and curing procedure. It's therefore important to ensure that specimens are well prepared (Grading, Maximum dry density, Optimum moisture content), moved with care, and a standard curing method used in order to have a consistency in test results.

- When stabilising with bitumen emulsion or foamed bitumen, the percentage of active filler is limited to specific amount. This is done in order to prevent cracking failure at low temperature. However, amounts recommended by specifications and guidelines across the world are different: 1% in South Africa (TG2, 2009), 1.5% in China (MOT, 2008) and 1% to 2% in USA (ARRA, 2002). Therefore, further research is required to assess the influence of the amount of active filler added to the mix on rut resistance and low temperature cracking.

- Although the flexibility of bituminous mixes is highly related to temperature as described in Chapter 2, the effect of temperature on flexibility of BSMs was not investigated in this study, Therefore further research are require in order to correlate the effects of temperature, bitumen and cement content to the flexibility of BSMs.

- Although strain at break and dissipated energy were able to give some indication of the flexibility of the mixes tested, there was still some variability with the attained data. There is

a need for alternate effective test method to accurately evaluate the flexibility of bitumen stabilised materials.

- A linear relationship between ITS and UCS test, with good correlation was established from the results. This implies that the two tests actually measure the same property, which is strength. However, ITS was found to be more sensitive to bitumen content compared to UCS. Therefore, ITS should receive priority in BSM mix design.

- In addition to strength, other material properties should be considered when evaluation and classifying BSMs mixes.

REFERENCES

1. ASTM D7313, 2013. *Standard Test Method for Determining Fracture Energy of Asphalt-Aggregate Mixtures Using the Disk-Shaped Compact Tension Geometry.* Available: http://www.astm.org/Standards/D7313.ht

2. Akzo Nobel 2000. *Bitumen Emulsion*, Technical Bulletin 03294, Akzo Nobel Asphalt Applications Stockholm, Sweden, 2000 Nobel, 2010.

3. Anastasios, M., 2007. *Fracture Mechanics in Pavement Engineering: The Specimen Size Effect,* Paper No 971312, Transportation Research Record: Journal of the Transportation Research Board, January 2007. Available:
http://trb.metapress.com/content/1k26578217556067/

4. Asphalt Academy, 2002. *The Design and Use of Foamed Bitumen Treated Materials.* Interim Technical Guideline 2 (TG2) , Pretoria, South Africa, 2002

5. Bissada, A, 1987. *Structural Response of Foamed Asphalt Sand Mixtures in Hot Environment*, Transportation Research Record, No. 1115, TRB, National Research Council, Washington D.C., 1987, pp. 134-149.

6. Bondietti, M., Murphy, D., Jenkins, K. and Burger, R., 2004. *Research on the stabilisation of two different materials using bitumen emulsion and cement,* Proceedings of the 8th Conference on Asphalt Pavements for Southern Africa, Sun City, South Africa, 2004.

7. Collings, D. AND Jenkins, K. J., 2011. *the long-term behaviour of bitumen stabilised materials,* Proceedings of the 10th Conference on Asphalt Pavements for Southern Africa, South Africa, 2011.

8. Council for Scientific and Industrial Research (CSIR), 1998–, *Foamed Asphalt Mixes – Mix Design Procedure,* Contract Report CR-98/077, SABITA and CSIR Transportek, Pretoria, South Africa, 1998.

9. Dowling, N.E*., Mechanical behaviour of materials: Engineering methods for deformation, fracture and fatigue*, 2nd edition, Prentice Hall, 1999.

10. Schreurs, P.J.G., 2012. *Fracture mechanics*, lecture notes, Department of Mechanical Engineering, Eindhoven University of Technology, September 6, 2012. Available:
http://www.mate.tue.nl/~piet/edu/frm/pdf/frmsyl1314.pdf

11. Ebels, L. J. and Jenkins, K. J., 2007. *Mix design of bitumen stabilised materials: Best practice and considerations for classification*, Proceedings of the 9th Conference on Asphalt Pavements for Southern Africa, Gaborone, Botswana, 2007.

12. Ebels, L. J., 2008. *Characterisation of Material Properties and Behaviour of Cold Bituminous Mixtures for Road Pavements*. Ph.D dissertation, University of Stellenbosch , South Africa, 2008.

13. Erkens, S., 2002. *Asphalt concrete response (ACRe): Determination, modelling, prediction*, PhD dissertation, published in Delft University of Technology, The Netherlands, 2002.

14. Fenton M., 2013. *Flexibility Assessment of Bitumen Stabilised Material*. B.Eng. Thesis, University of Stellenbosch, South Africa, 2013.

15. *Flow plastic theory*, Wikipedia 2013. Available:

http://en.wikipedia.org/wiki/Flow_plasticity_theory

16. Hibbleler, R.C., 2003. *Mechanics of materials*, SI second edition, 2003.

17. Hodgkinson, A. and Visser, A.T., 2004. *The role of fillers and cementitious binders when recycling with foamed bitumen or bitumen emulsion,* 8th Conference on Asphalt Pavements for Southern Africa, September 2004.

18. Houston, M. and Long, F.M., 2004. *Correlations between different ITS and UCS test protocols for foamed bitumen treated materials*, 8th Conference on Asphalt Pavements for Southern Africa (CAPSA'04), Sun City, South Africa, 2004.

19. Huang, Y. H., Pavement Analysis and Design, published by Prentice Hall, Englewood Cliff, New Jersey, USA, 1993.

20. James, A., 2006. *Overview of Asphalt Emulsions,* Asphalt Emulsion Technology, Transportation Research Circular E-C102, Transportation Research Board, Washington DC, USA, August 2006.

21. Jenkins, K J., 2010. *Material Science,* Pavement Materials I Lecture Notes, Stellenbosch University, April 2010.

22. Jenkins, K. J., Ebels, L. J. and Mathaniya E. T., 2006. *Updating Bituminous Stabilised Materials Guidelines: Mix Design Inception Study*, SABITA and Gauteng Department of Transport, Roads and Public Works. Pretoria, South Africa, 2006.

23. Jenkins, K.J and Collings D., 2003. *Key characteristics of materials stabilized with foam bitumen,* , Advanced Testing and Characterization of Bituminous Materials, 2003

24. Jenkins, K.J., 2002. *Mix Design Consideration of Cold and Half-warm Bituminous Mixes with Emphasis on Foam Bitumen*. Ph.D dissertation at University of Stellenbosch, South Africa, 2002.

25. Jenkins, K.J., 2013. *Pavement Materials Behaviour,* Pavement Materials III Lecture Notes, Stellenbosch University, 2013.

26. Jenkins, K.J., Van der Riet, J. and Twagira M.E., 2008. *Task 10: Moisture Sensitivity: Part II*, Technical Memorandum, Updating Bituminous Stabilized Materials Guidelines: Mix Design Report, Phase II, Final Report , September 2008.

27. Jenkins, K.J., Ebel, L.J., Kelfkens, R.W.C., Moloto, P.K., Molusa, W.K., 2008. *Updating Bituminous Stabilized Materials Guidelines: Mix Design Report, Phase II*, Research Report prepared for SABITA and the Department of Transport and Public Works, Gauteng, South Africa, 2008.

28. Koh, C. and Roque, R. *Use of Nonuniform Stress-State Tests to Determine Fracture Energy of Asphalt Mixtures Accurately*, Transportation Record Research 2121.

29. Kori F., 2004. *Relationship between compressive and tensile strength with special emphasis on flexural strength,* Special Project Report, Howard University, 2004.

30. Kosmatka, S., 1985. *Compression vs. Flexural Strength for Quality Control of Pavements*. CTT PL 854, Portland Cement Association, Skokie, 1985.

31. Loizos, A., Collings, D. C. and Jenkins, K. J., 2004. *Rehabilitation of a Major Greek Highway by Recycling/Stabilising with Foamed Bitumen*, 8th Conference on Asphalt Pavements for Southern Africa (CAPSA'04), Sun City, South Africa, 2004.

32. Long F., and Ventura, D.G.C, 2004. *Laboratory Testing for HVS Section on the N7*, Report CR-2003/56, Availaible:

http://www.gpdrthvs.co.za/popup/CR_2003_56/CR_2003_56,N7%20lab%20report,%20FINAL.pdf

33. Long, F. and Theyse, H., 2004. *Mechanistic-empirical structural design models for foamed and emulsified bitumen treated materials,* 8th Conference on Asphalt Pavements for Southern Africa, Sun City, South Africa, 2004.

34. Long, F. and Theyse, H., 2005. *The Engineering, Mechanical and Durability Properties of Crushed Hornfels Treated with Emulsified Bitumen and of Sand Treated with Emulsified Bitumen and Foamed Bitumen,* Report CR-2005/01, 2005.

35. Meinheit J., 2009. *Flexible-bounded bedding layer as an improvement for concrete block pavement* , 9th. International Conference on Concrete Block Paving, Buenos Aires, Argentina, 2009.

36. Molenaar, A.A.A., 2011. *Structural Pavement Design*, Flexible Pavement Design Lecture notes, Stellenbosch University, 2011.

37. Molenaar, A.A.A., 2011.*Cohesive and Non-Cohesive Soils and Undound Granular Materials For Base and Sub-Base in Roads, Pavement Materials 1 Lecture Notes,* Stellenbosch University, 2011.

38. Muthen K M, 1999. *Foamed Asphalt Mixes Mix Design Procedure*, Report CR-98/077, 1998, SIR Transportek, 1998.

39. Muthen, K.M., 1999. *Foam Asphalt Mixes: Mix Design Procedure,* Contract Report CR-98/077, June 1999.

40. Paige-Green, P. and Ventura D.F.C., 2004. *Durability of Foamed Bitumen Treated Basalt Base Courses*. CSIR report CR 2004/8

41. Rowe M., 1996. *Application of the dissipated energy concept to fatigue cracking in asphalt pavements*, Ph.D disertation, University of Nottingham, January 1966.

42. Sakr, H.A., and Manke, P.G., 1985. *Innovations in Oklahoma Foam mix Design Procedures*, Transportation Research Record, No. 1034, TRB, National Research Council, Washington D.C., 1985, pp. 26-34.

43. Saleh, M., 2006. *Characterisation of Foam Bitumen Quality and the Mechanical Properties of Foam Stabilised Mixes*, 10th International Conference on Asphalt Pavements (ICAP 2006), Quebec City, Canada, Aug 2006.

44. Sharad, Y.M. and Deepak, D.N., 2012. *Studies on correlation between flexural strength and compressive strength of concrete,* The Indian Concrete Journal, 2012.

45. Smith, C.W., Johnston, M.A., and Lorentz, S., 1997. *Assessing the compaction susceptibility of South African forestry soils*, The effect of soil type, water content and applied pressure on uni-axial compaction, Soil and Tillage Research,1997.

46. Theyse, H. L., 2000. *Overview of the South African Mechanistic Pavement Design Methods*, South African Transport Conference, Pretoria, South Africa, 2000.

47. Twagira, M. E., Jenkins, K. J. and Ebels, L. J., 2006. *Characterisation of Fatigue Performance of Selected Cold Bituminous Mixes*. Proceedings of the 10th International Conference on Asphalt Pavements. Quebec City, Canada, 2006

48. Twagira, M.E., 2006. *Characterisation of Fatigue Performance of Selected Cold Bituminous Mixes*. MSc. Thesis at University of Stellenbosch, South Africa, 2006

49. Twagira, M.E., 2010. *Influence of Durability Properties on Performance of Bitumen Stabilized Materials*. Ph.D dissertation**,** University of Stellenbosch**,** South Africa, 2010.

50. Wirtgen GmbH, 2004. *Cold Recycling Manual,* 2nd edition published by *Wirtgen* GmbH, Windhagen, Germany, 2004

51. Wittmann, F. H., *Crack formation and fracture energy of normal and high strength concrete*, Sadhana (India) 27, Part 4, 413-423 (2002). Available:
http://www.aedificat.de/274/Institute/publications.html

52. Lu, X., Ulf, I., and Ekblad J., 2003. *Influence of polymer modification on low temperature behaviour of bituminous binders and mixtures.* Material and Structure, Vol. 36, December 2003.

53. Yuan M., Zhang X., Chen W. and Zhang S., 2013. *Ratio of Dissipated Energy Change-based Failure Criteria of Asphalt Mixtures*, Research Journal of Applied Sciences, Engineering and Technology, August 2013.

APPENDICES

Appendix A: ITS and UCS Results

Table A. 1: ITS and UCS, mix treated with 1% and 2% cement

Mix Type	Cement added (%)	Bitumen added (%)	Strength test results			
			ITS_{DRY} (kPa)	ITS_{WET} (kPa)	UCS_{DRY} (kPa)	UCS_{DRY} (Kpa)
BSM - emulsion	1	2.0	265.6	188.6	1323.2	1179.2
		2.4	226.3	205.1	1402.6	1232.2
		2.8	271.1	266.6	1280.8	1131.6
	2	2.0	544.6	431.6	2693.5	2408.0
		2.4	414.6	405.1	2132.8	1991.2
		2.8	430.4	375.7	1957.3	1774.8
BSM - foam	1	2.0	181.2	148.8	2061.3	1538.1
		2.4	256.4	202.7	2149.3	1539.0
		2.8	313.1	198.8	1920.7	1341.0
	2	2.0	337.8	318.6	2648.6	2446.5
		2.4	330.6	283.9	3130.4	2560.9
		2.8	525.9	426.6	2674.3	2199.0

Table A. 2: ITS and UCS values mix treated with lime and cement

Mix Type	Active filler added	Bitumen added (%)	Strength test results			
			ITS_{DRY} (kPa)	ITS_{WET} (kPa)	UCS_{DRY} (kPa)	UCS_{WET} (Kpa)
BSM - emulsion	1% Lime	2.0	123.91	78.33	570.03	533.25
		2.4	126.53	96.81	528.38	335.86
		2.8	87.98	52.35	490.91	352.58
	1% Lime + 1% Cement	2.0	287.63	151.51	1611.03	1672.88
		2.4	267.33	203.52	1404.34	1232.73
		2.8	379.24	256.36	1075.64	1178.17
BSM - foam	1% Lime	2..0	207.30	73.91	1326.97	532.78
		2.4	253.94	75.84	1027.19	408.90
		2.8	199.10	55.15	860.38	428.71
	1% Lime + 1% Cement	2.0	252.28	166.69	1468.51	1279.84
		2.4	240.13	133.28	1949.93	1638.77
		2.8	314.58	204.99	1994.11	1649.56
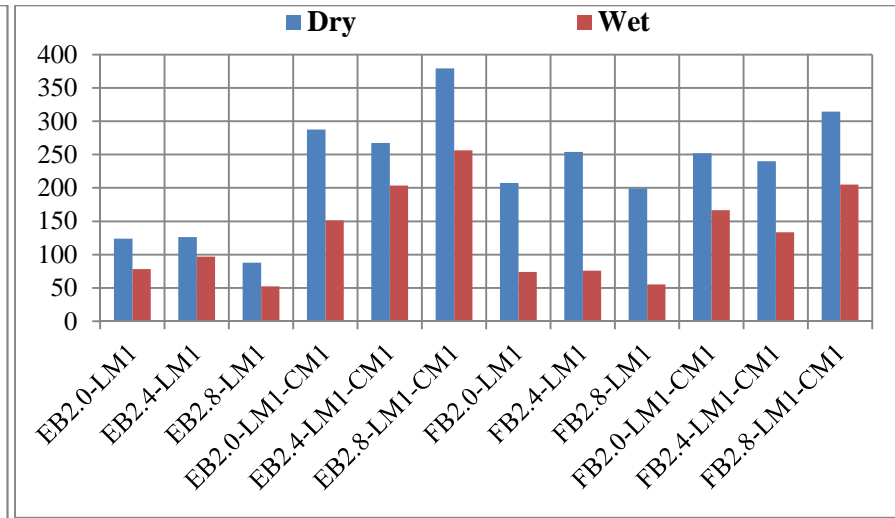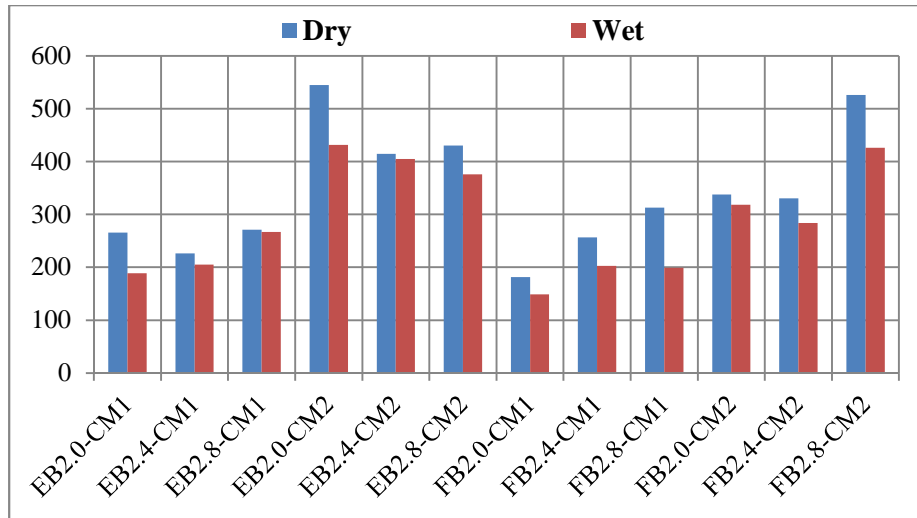
Figure A.1: Illustration of ITS wet and ITS dry of all the mixes
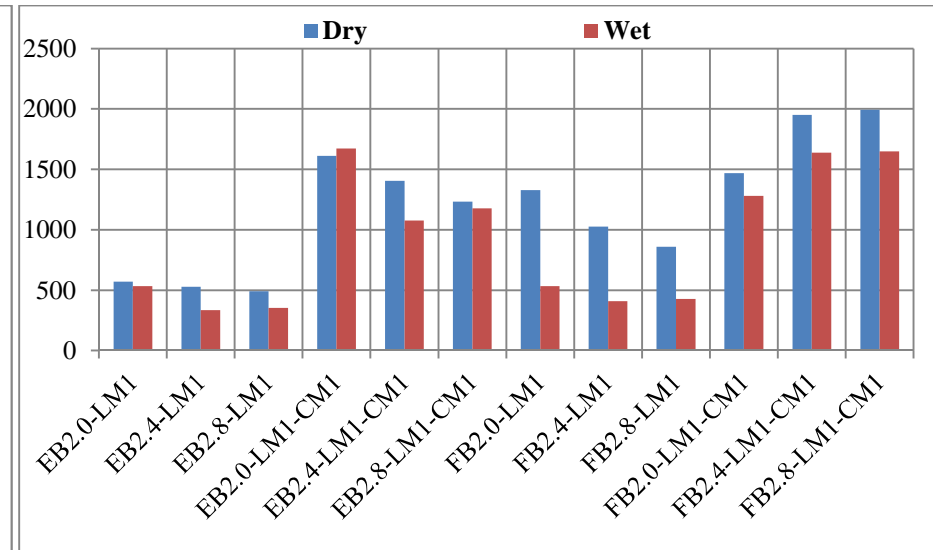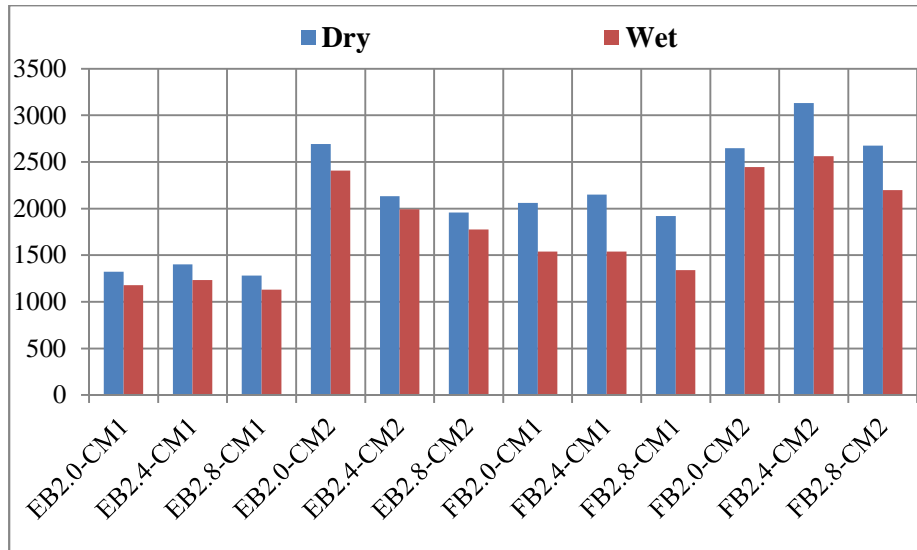
Figure A.2: Illustration of UCS wet and UCS dry of all the mixes
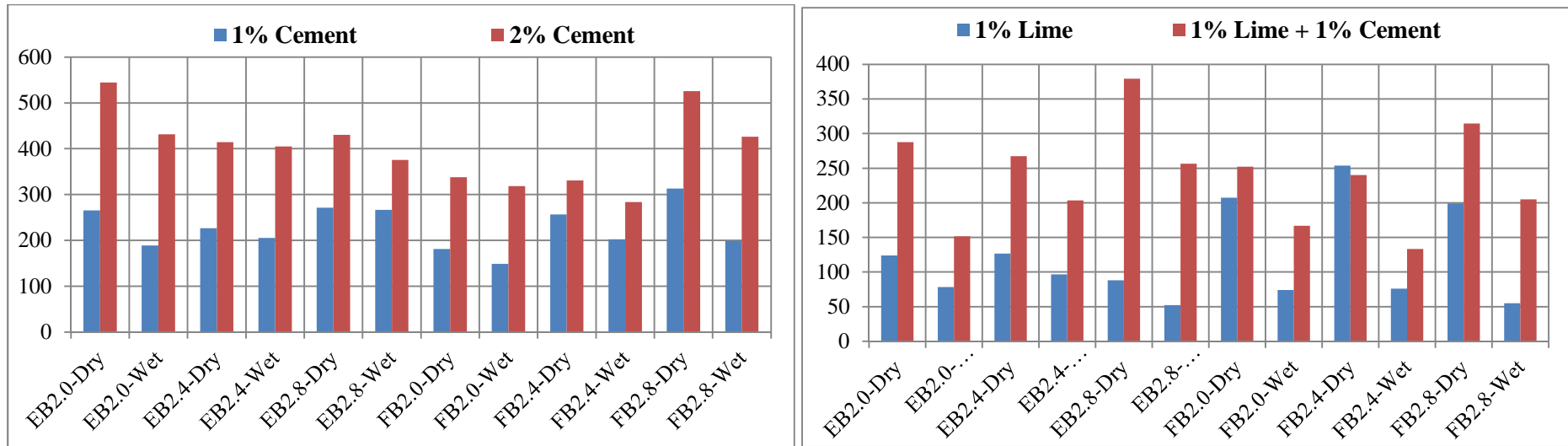
Figure A.3 Influence of the type and percentage of active fillers on ITS values
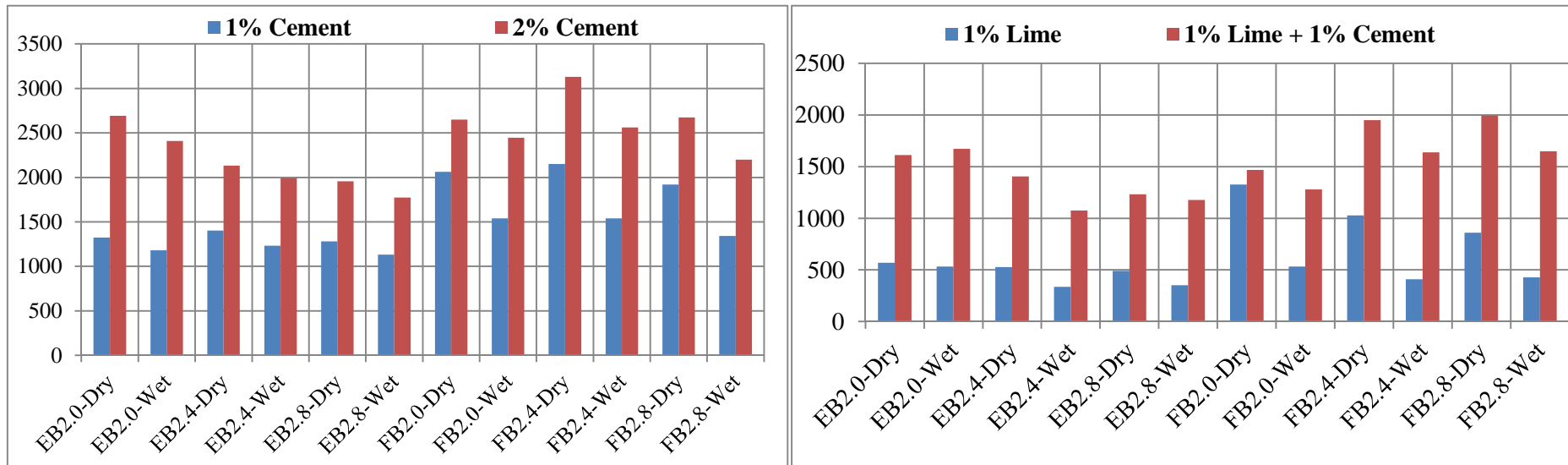
Figure A.4 Influence of the type and percentage of active fillers on UCS values
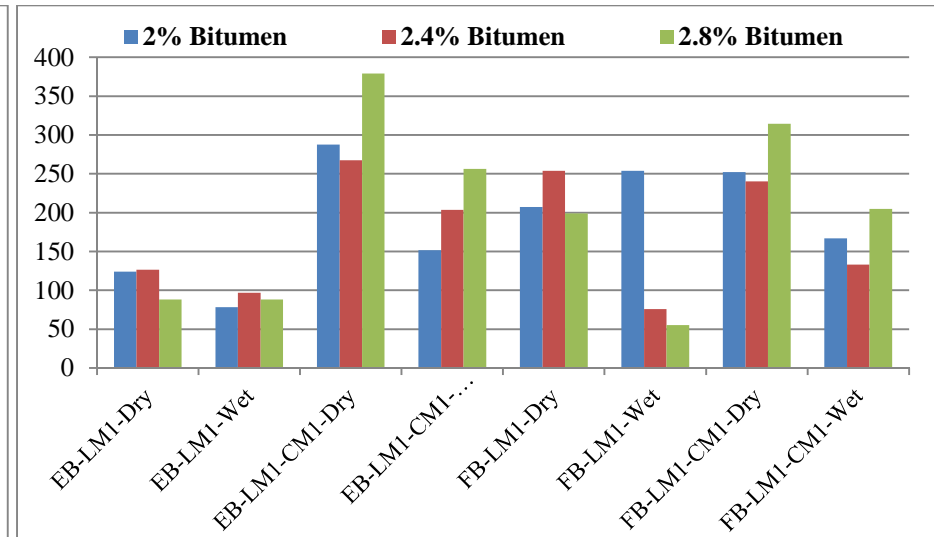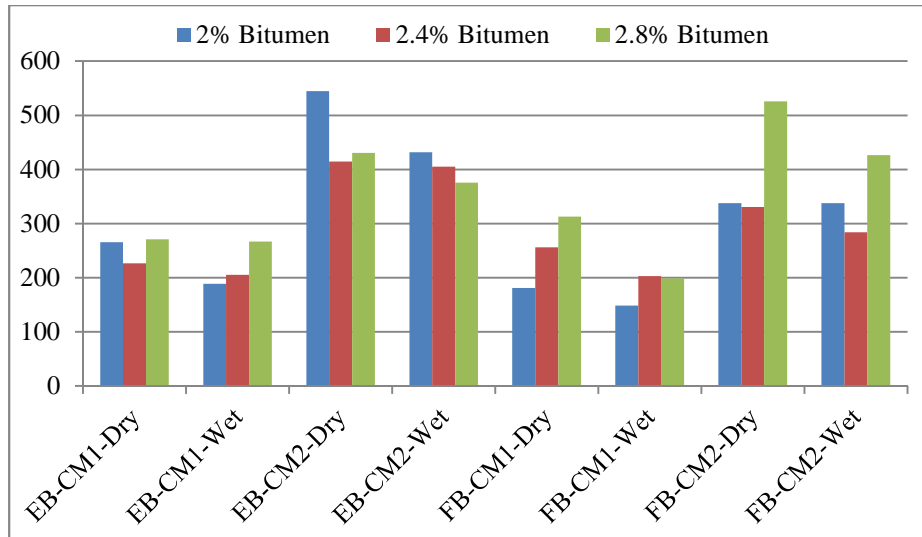
Figure A.5 Influence of the bitumen content ITS values
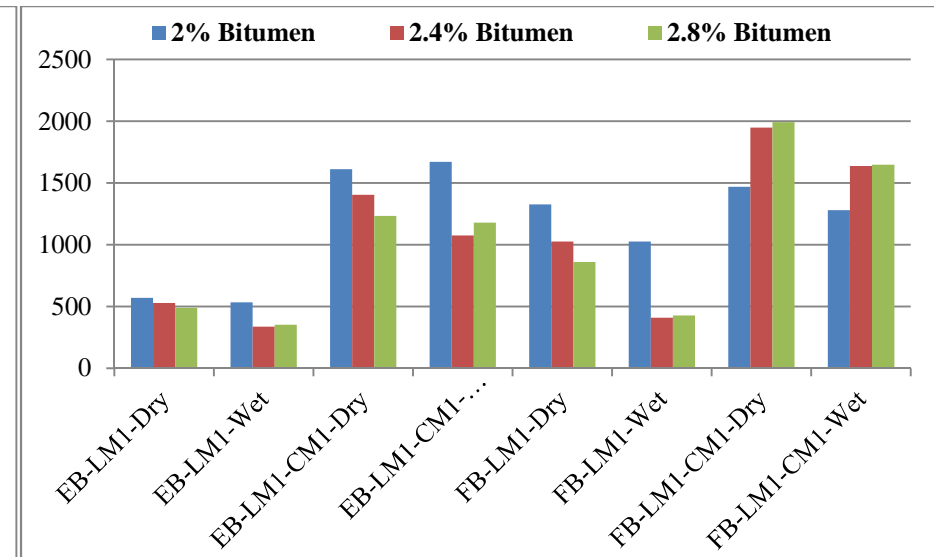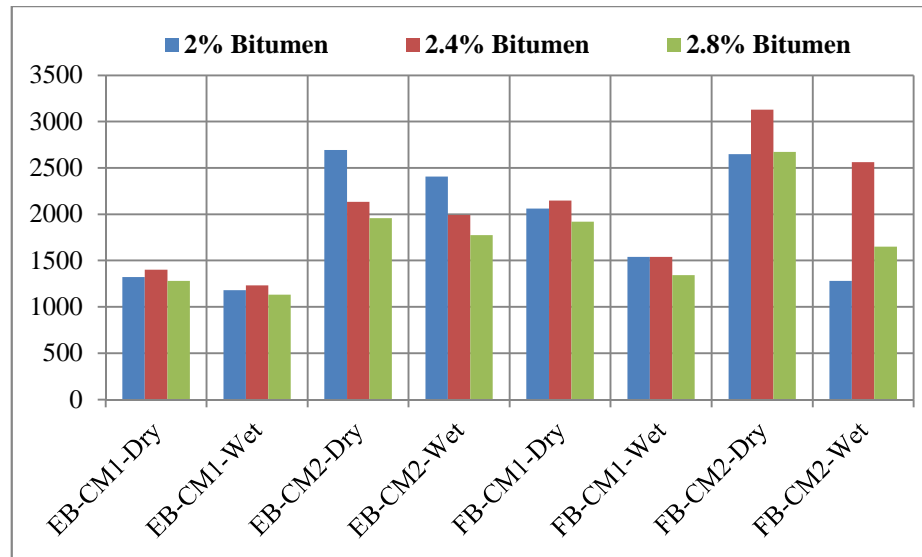
Figure A.6 Influence of the bitumen content UCS values

Appendix B: Displacement and strain at break results

Table B-1: Displacement and strain at break (mixes with cement)

Mixes	Testing Condition	Displacement at break from ITS test		Strain at break from UCS test	
		Average (mm)	COV (%)	Average (m strain)	COV (%)
EB2.0-CM1	Dry	0.77	4.44	34.97	5.35
	Wet	0.82	19.27	33.61	3.92
EB2.4-CM1	Dry	0.65	8.46	19.00	7.82
	Wet	0.72	0.24	16.76	0.54
EB2.8-CM1	Dry	0.71	3.48	19.90	13.50
	Wet	0.83	12.23	18.17	14.06
EB2.0-CM2	Dry	0.56	7.17	20.40	11.56
	Wet	0.51	4.97	20.80	3.25
EB2.4-CM2	Dry	0.56	10.21	19.62	9.53
	Wet	0.58	11.02	19.27	18.99
EB2.8-CM2	Dry	0.57	4.73	16.34	25.09
	Wet	0.60	5.31	17.75	15.05
FB2.0-CM1	Dry	0.52	12.05	22.13	19.83
	Wet	0.70	11.96	15.84	11.34
FB2.4-CM1	Dry	0.57	3.66	22.51	9.45
	Wet	0.63	13.16	15.53	3.51
FB2.8-CM1	Dry	0.54	6.23	16.27	4.07
	Wet	0.65	1.97	13.27	13.15
FB2.0-CM2	Dry	0.50	5.35	23.85	23.17
	Wet	0.56	9.30	24.18	7.14
FB2.4-CM2	Dry	0.60	10.61	20.93	15.33
	Wet	0.60	1.37	17.79	8.64
FB2.8-CM2	Dry	0.61	7.43	16.36	7.96
	Wet	0.56	9.13	15.58	6.95

Table B-2: Displacement and strain at break values (mixes with lime)

Mixes	Testing Condition	Displacement at break from ITS test		Strain at break from UCS test	
		Average (mm)	COV (%)	Average (m strain)	COV (%)
EB2.0-LM1	Dry	0.69	10.78	16.42	7.21
	Wet	0.73	8.91	24.56	14.37
EB2.4-LM1	Dry	0.73	9.23	24.82	5.07
	Wet	1.00	5.02	25.36	7.44
EB2.8-LM1	Dry	0.94	1.58	19.36	22.56
	Wet	1.01	13.65	20.01	20.41
EB2.0-LM1-CM1	Dry	0.57	15.97	18.43	15.99
	Wet	0.64	12.28	20.09	11.52
EB2.4-LM1-CM1	Dry	0.54	10.26	19.60	0.90
	Wet	0.64	8.10	21.53	4.95
EB2.8-LM1-CM1	Dry	0.61	3.97	16.74	5.98
	Wet	0.68	3.45	19.08	4.30
FB2.0-LM1	Dry	0.72	8.52	17.81	4.01
	Wet	0.74	20.65	18.40	20.80
FB2.4-LM1	Dry	0.74	5.39	19.97	5.09
	Wet	0.75	8.78	20.14	15.90
FB2.8-LM1	Dry	0.75	5.08	15.61	4.37
	Wet	0.80	10.11	19.37	5.59
FB2.0-LM1-CM1	Dry	0.60	6.44	15.36	4.87
	Wet	0.59	5.89	16.71	4.92
FB2.4-LM1-CM1	Dry	0.64	3.03	17.87	5.63
	Wet	0.75	8.97	17.03	12.06
FB2.8-LM1-CM1	Dry	0.70	13.13	16.36	1.11
	Wet	0.83	7.57	17.86	9.48
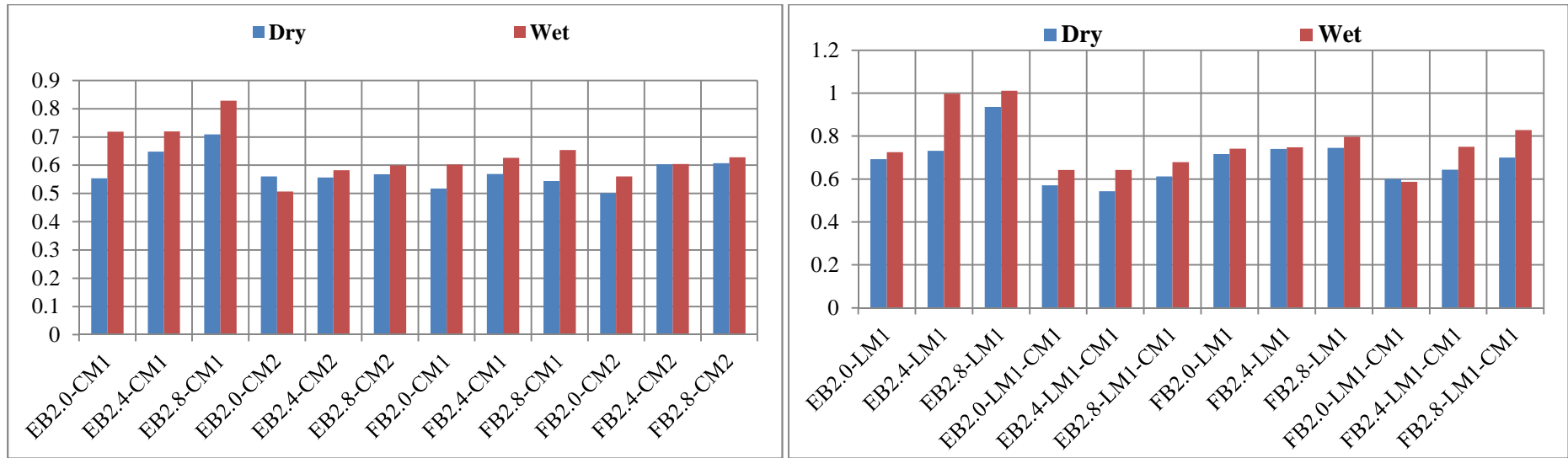
Figure B.1 Influence of the moisture condition on the displacement at break
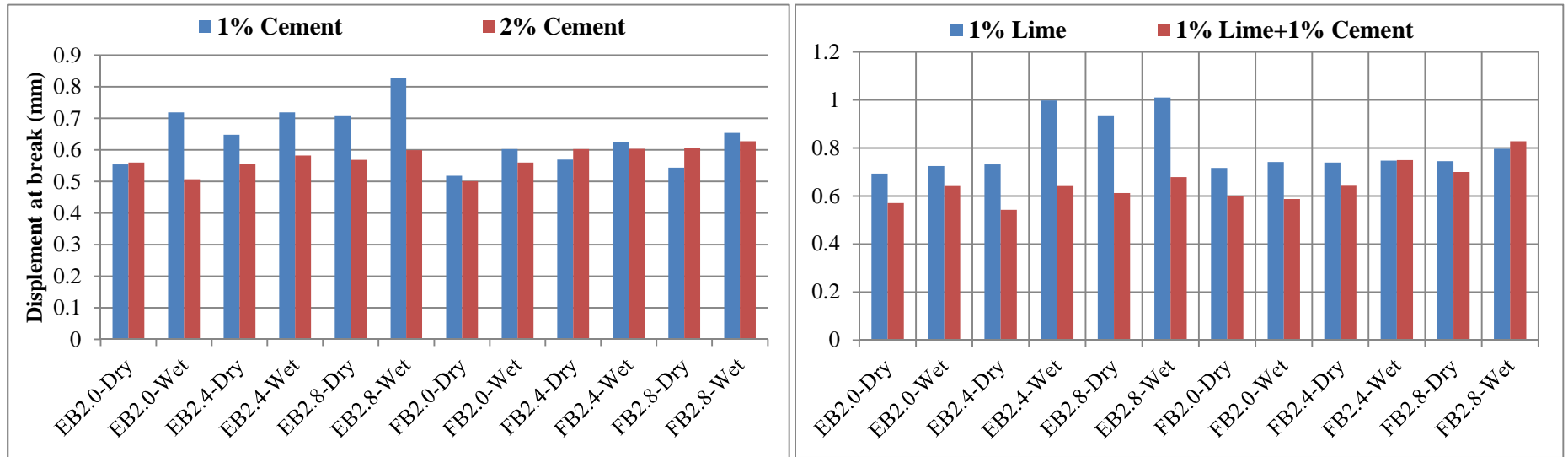
Figure B.2 Influence of the type and percentage of active fillers on the displacement at break
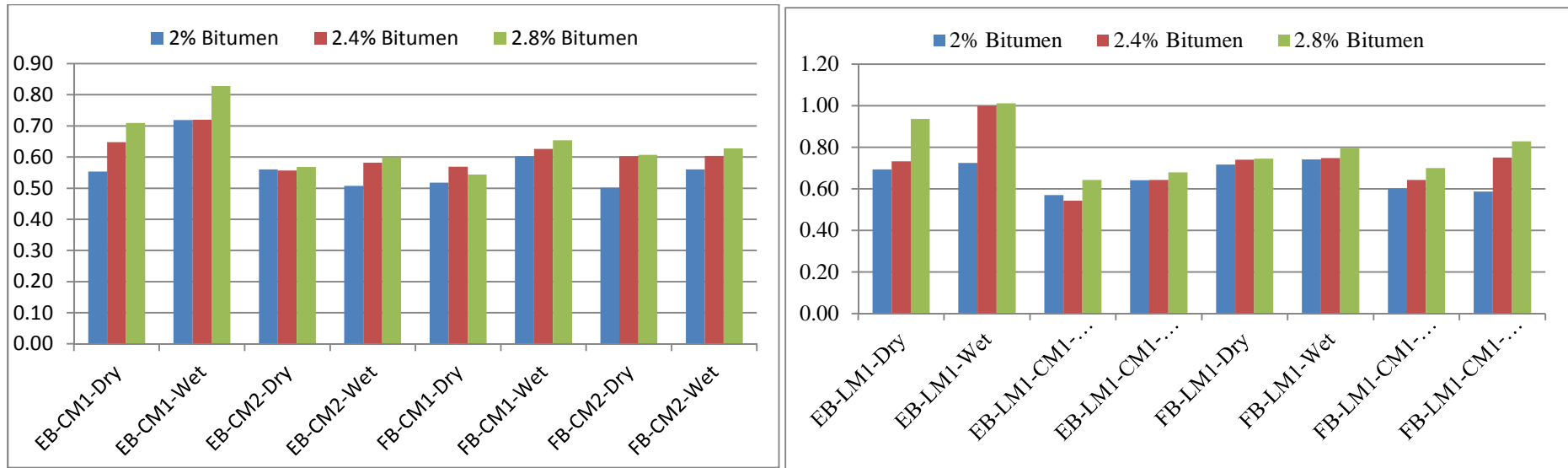
Figure B.3 Influence of the bitumen content on the displacement at break

Appendix C: Statistical results: factorial ANOVA output from SPSS

Between-Subjects Factors

		N
Type of treatment	Emulsion	36
	Foam	36
Cement content	1%	36
	2%	36
Bitumen content	2%	24
	2.4%	24
	2.8%	24
Testing condition	Dry	36
	Wet	36

Dependable variables	R-Square	Adjusted R square
ITS	0.997	0.996
UCS	0.998	0.997
Displacement at break	0.995	0.993
Strain at break	0.995	0.992
Fracture energy	0.995	0.993

Tests of Between-Subjects Effects

Source	Dependent Variable	Sum of Squares	df	Mean Square	F	Sig.
Model	ITS	7.957E6[a]	24	331524.707	718.266	1.789E-53
	UCS	2.956E8[b]	24	1.232E7	1121.710	4.180E-58
	Displacement at break	26.937[c]	24	1.122	439.133	2.262E-48
	Strain at break	24068.890[d]	24	1002.870	385.807	4.950E-47
	Fracture energy	426.318[e]	24	17.763	402.178	1.839E-47
Type of treatment	ITS	31449.162	1	31449.162	68.136	9.109E-11
	UCS	4063805.411	1	4063805.411	370.106	3.337E-24
	Displacement at break	.036	1	.036	13.948	4.987E-4
	Strain at break	16.496	1	16.496	6.346	0.0295
	Fracture energy	4.343	1	4.343	98.319	3.340E-13
Cement content	ITS	551529.394	1	551529.394	1194.919	1.414E-35
	UCS	1.383E7	1	1.383E7	1259.505	4.188E-36
	Displacement at break	.083	1	.083	32.515	7.123E-7
	Strain at break	16.107	1	16.107	6.197	0.016
	Fracture energy	8.443	1	8.443	191.147	2.330E-18
Bitumen content	ITS	49321.736	2	24660.868	53.429	6.185E-13
	UCS	944626.174	2	472313.087	43.015	1.981E-11
	Displacement at break	.073	2	.036	14.255	1.382E-5
	Strain at break	19.742	2	9.871	3.797	0.018
	Fracture energy	8.343	2	4.171	94.447	2.294E-17

Testing condition	ITS	52191.976	1	52191.976	113.077	3.260E-14
	UCS	2033379.665	1	2033379.665	185.188	4.283E-18
	Displacement at break	.060	1	.060	23.402	1.398E-5
	Strain at break	1.101	1	1.101	.424	0.0518
	Fracture energy	.311	1	.311	7.047	0.011
Type of treatment * Cement content	ITS	8225.847	1	8225.847	17.822	1.073E-4
	UCS	11080.792	1	11080.792	1.009	0.320
	Displacement at break	.079	1	.079	30.734	1.238E-6
	Strain at break	29.954	1	29.954	11.523	0.001
	Fracture energy	.128	1	.128	2.906	0.095
Type of treatment * Bitumen content	ITS	59722.386	2	29861.193	64.696	2.373E-14
	UCS	443527.196	2	221763.598	20.197	4.322E-7
	Displacement at break	.006	2	.003	1.079	0.348
	Strain at break	12.772	2	6.386	2.457	0.096
	Fracture energy	.568	2	.284	6.425	0.003
Type of treatment * Testing condition	ITS	932.781	1	932.781	2.021	0.162
	UCS	445027.160	1	445027.160	40.530	6.895E-8
	Displacement at break	.000	1	.000	.034	0.854
	Strain at break	2.764	1	2.764	1.064	0.308
	Fracture energy	.472	1	.472	10.678	0.002
Cement content * Bitumen content	ITS	17435.097	2	8717.549	18.887	8.897E-7
	UCS	253896.488	2	126948.244	11.562	7.971E-5
	Displacement at break	.003	2	.001	.496	0.612
	Strain at break	36.552	2	18.276	7.031	0.002
	Fracture energy	.489	2	.245	5.539	0.007
Cement content * Testing condition	ITS	196.445	1	196.445	.426	0.517
	UCS	12804.588	1	12804.588	1.166	0.286
	Displacement at break	.034	1	.034	13.328	6.446E-4
	Strain at break	2.039	1	2.039	.784	0.380
	Fracture energy	.005	1	.005	.117	0.734
Bitumen content * Testing condition	ITS	4178.096	2	2089.048	4.526	0.016
	UCS	22284.741	2	11142.370	1.015	0.370
	Displacement at break	.003	2	.002	.656	0.523
	Strain at break	20.480	2	10.240	3.939	0.026
	Fracture energy	1.251	2	.626	14.167	1.461E-5

Type of treatment * Cement content * Bitumen content	ITS	37491.978	2	18745.989	40.614	4.756E-11
	UCS	576384.402	2	288192.201	26.247	1.988E-8
	Displacement at break	.013	2	.006	2.520	0.091
	Strain at break	4.023	2	2.012	.774	0.467
	Fracture energy	.958	2	.479	10.845	1.299E-4
Type of treatment * Cement content * Testing condition	ITS	1491.569	1	1491.569	3.232	0.079
	UCS	46855.373	1	46855.373	4.267	0.044
	Displacement at break	.004	1	.004	1.592	0.213
	Strain at break	2.396	1	2.396	.922	0.342
	Fracture energy	.124	1	.124	2.816	0.100
Type of treatment * Bitumen content * Testing condition	ITS	17065.180	2	8532.590	18.486	1.115E-6
	UCS	66385.482	2	33192.741	3.023	0.058
	Displacement at break	.001	2	.000	.191	0.827
	Strain at break	19.153	2	9.577	3.684	0.032
	Fracture energy	2.462	2	1.231	27.874	9.253E-9
Cement content * Bitumen content * Testing condition	ITS	602.701	2	301.351	.653	0.525
	UCS	2999.747	2	1499.873	.137	0.873
	Displacement at break	.004	2	.002	.747	0.479
	Strain at break	14.728	2	7.364	2.833	0.069
	Fracture energy	.130	2	.065	1.474	0.239
Type of treatment * Cement content * Bitumen content * Testing condition	ITS	997.784	2	498.892	1.081	0.347
	UCS	40546.541	2	20273.270	1.846	0.169
	Displacement at break	.010	2	.005	1.914	0.159
	Strain at break	3.420	2	1.710	.658	0.523
	Fracture energy	.403	2	.202	4.563	0.015
Error	ITS	22154.992	48	461.562		
	UCS	527045.225	48	10980.109		
	Displacement at break	.123	48	.003		
	Strain at break	124.772	48	2.599		
	Fracture energy	2.120	48	.044		
Total	ITS	7978747.970	72			
	UCS	2.961E8	72			
	Displacement at break	27.060	72			
	Strain at break	24193.661	72			
	Fracture energy	428.438	72			
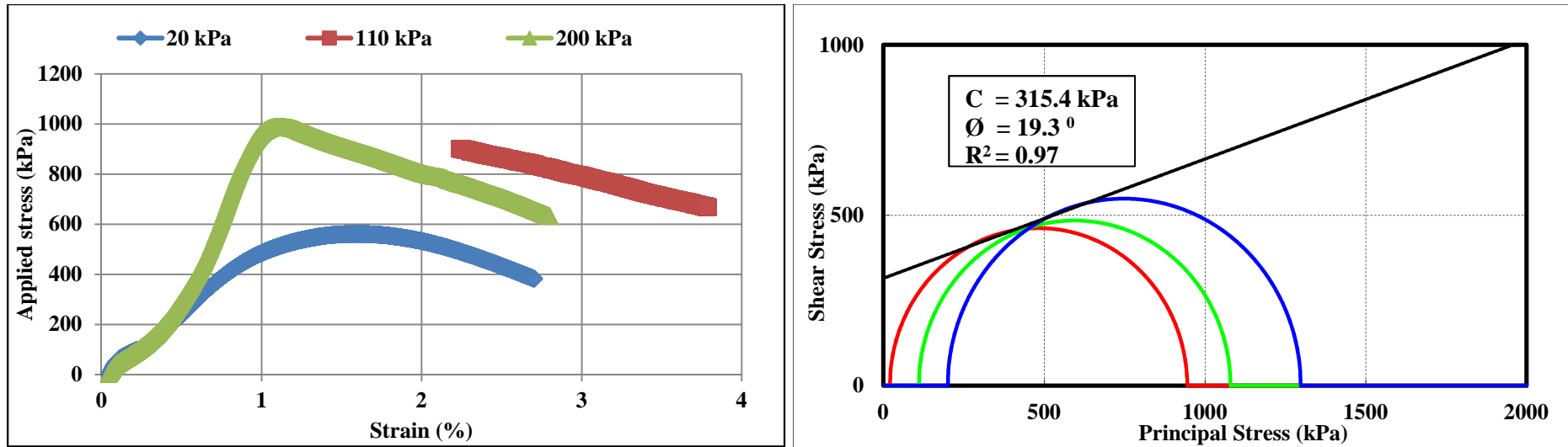
Figure D-1: Stress-Strain and Mohr-Coulomb Diagram, Mix EB0.9-CM1-HD-HS
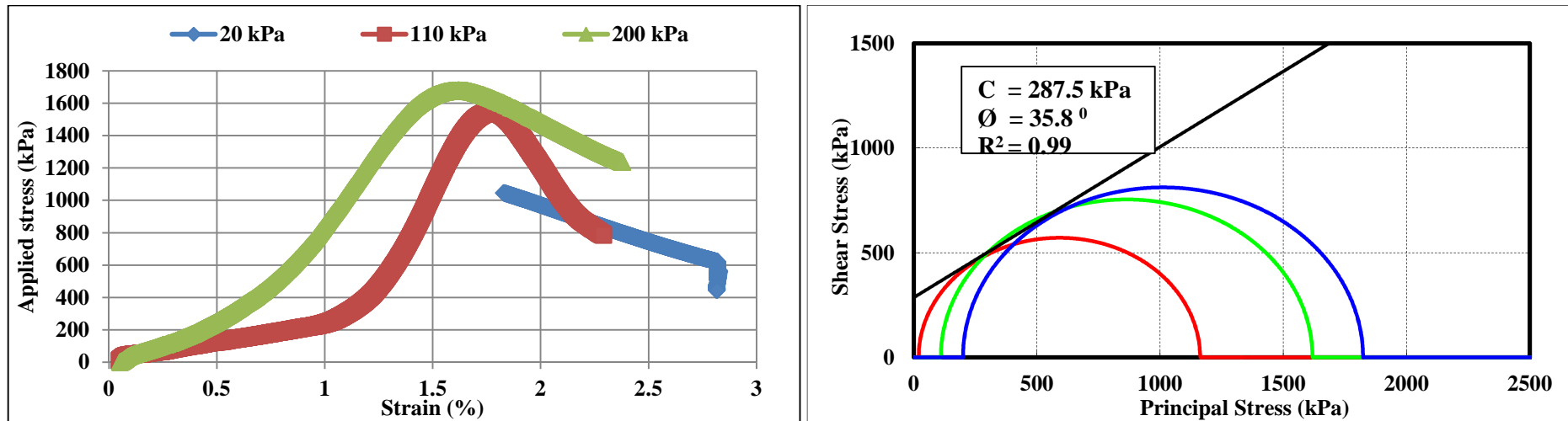
Fiigure D-2 Stress-Strain and Mohr-Coulomb Diagram, Mix EB0.9-CM1-HD-LS
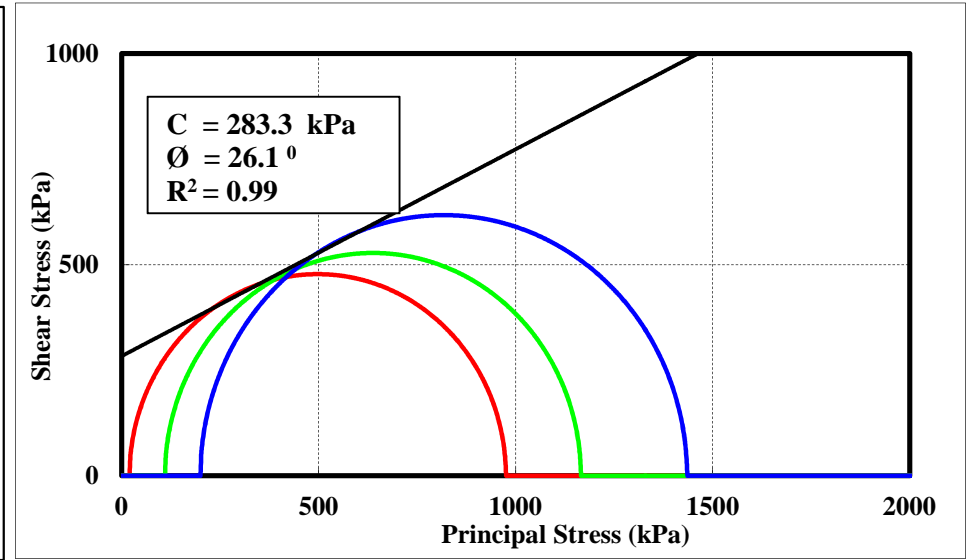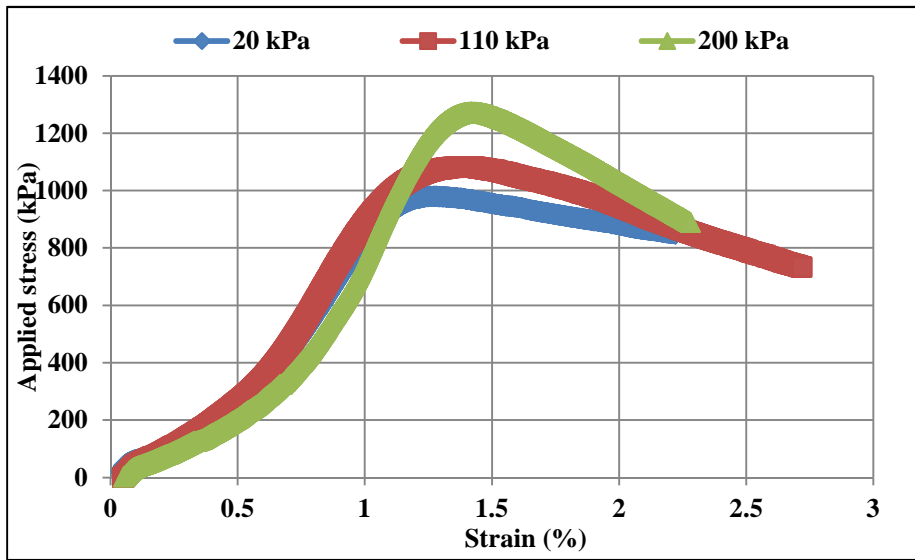
Figure D-3: Stress –strain curve and Mohr-Coulomb diagram; Mix EB0.9-CM1-LD-HS
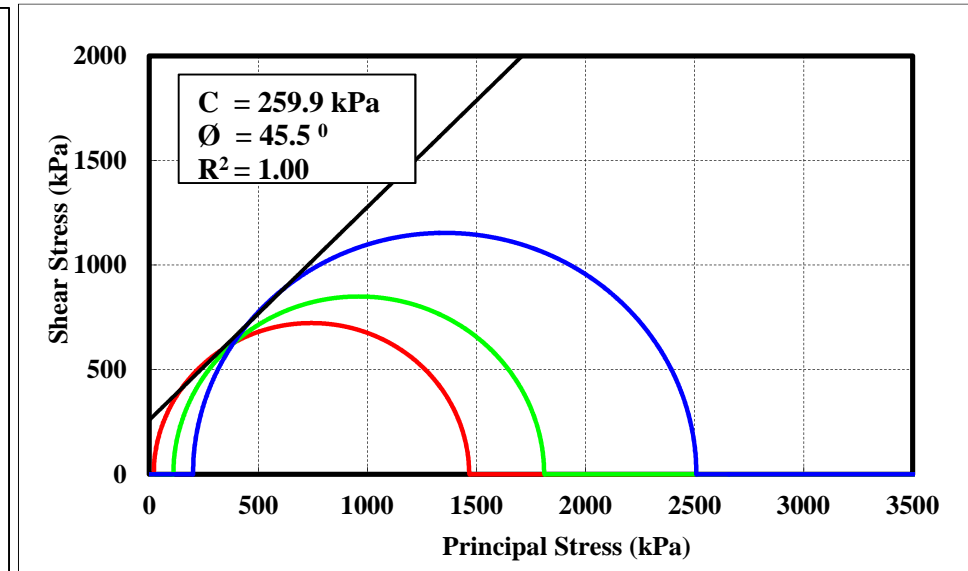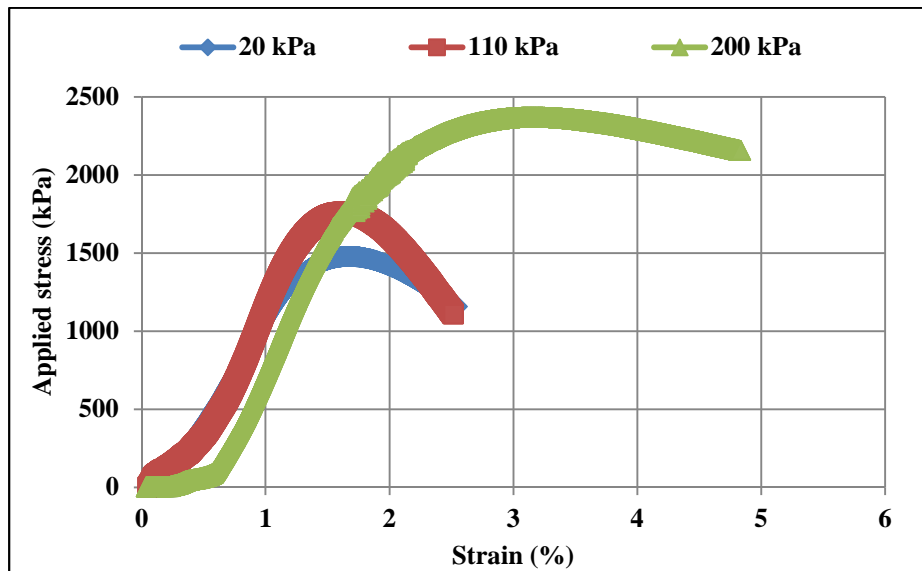
Figure D-4: Stress-Strain and Mohr-Coulomb Diagram, Mix EB0.9-CM1-LD-LS
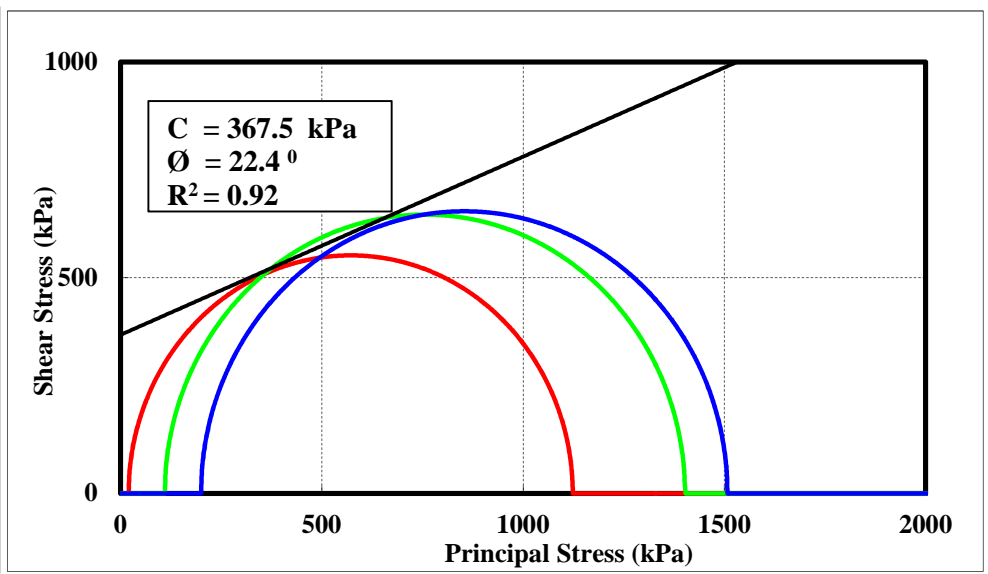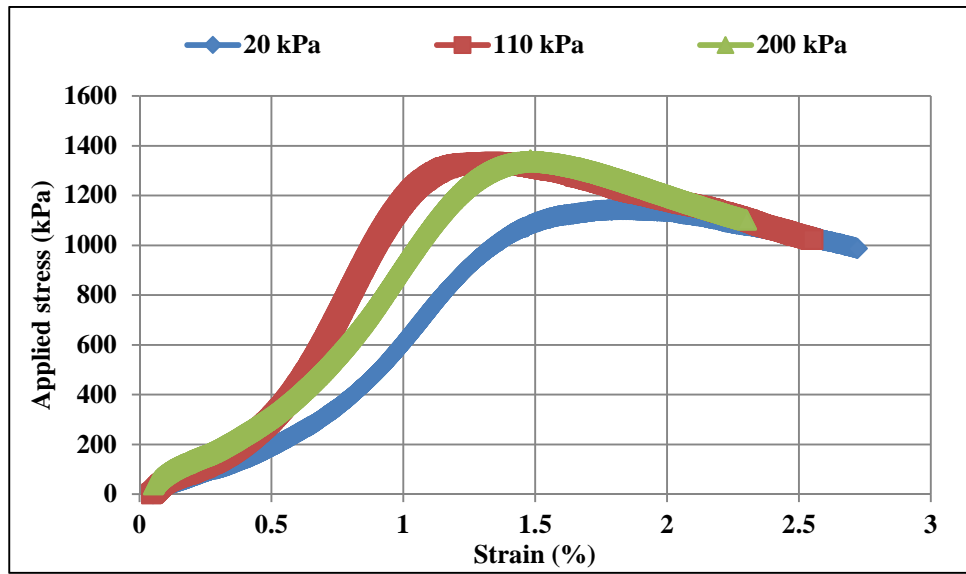
Figure D-5: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM1-HD-HS
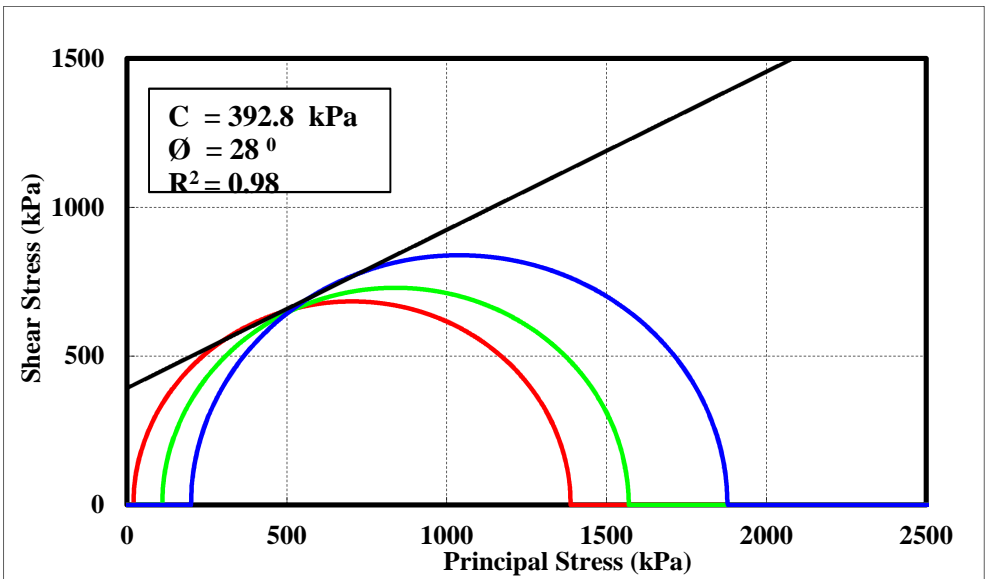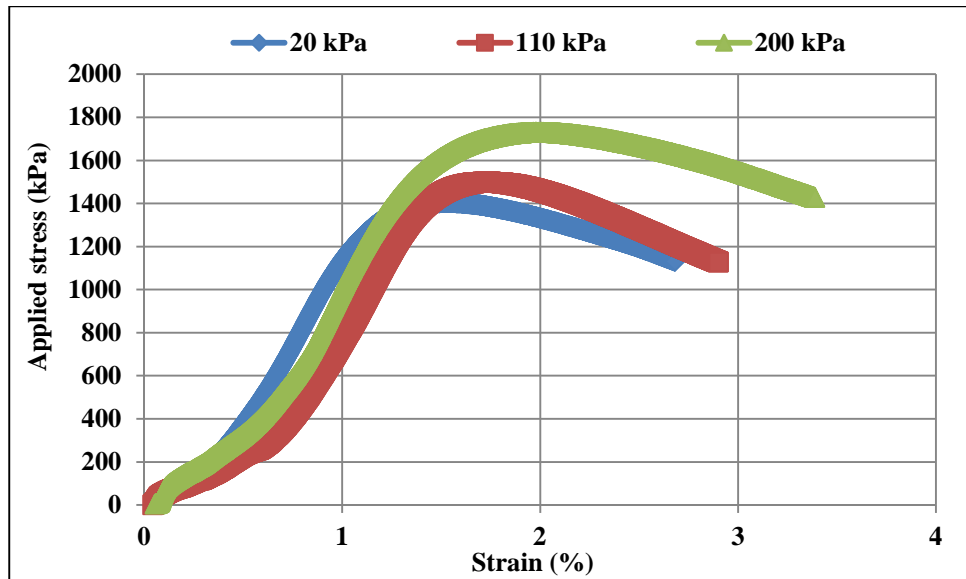
Figure D-6: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM1-HD-LS
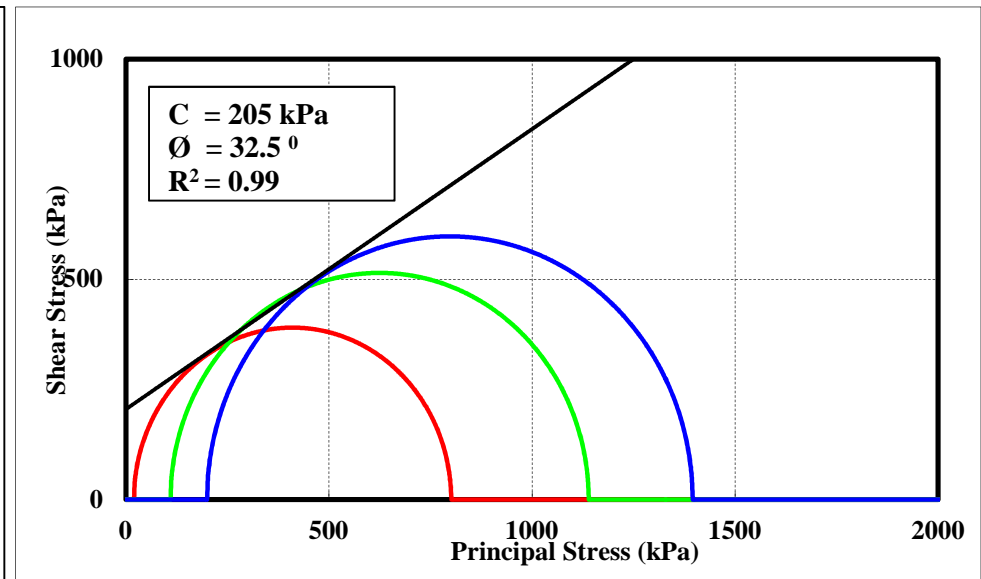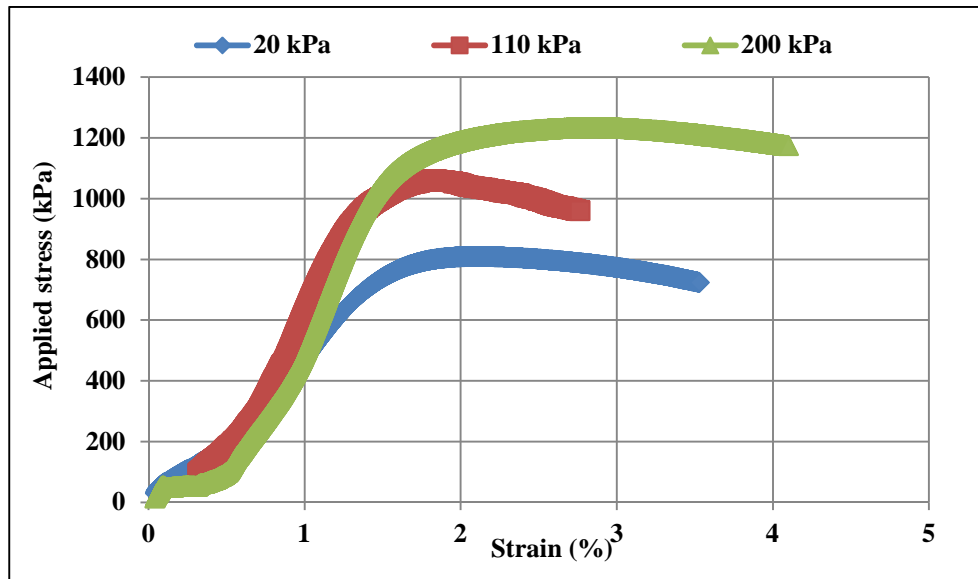
Figure C-7: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM1-LD-HS
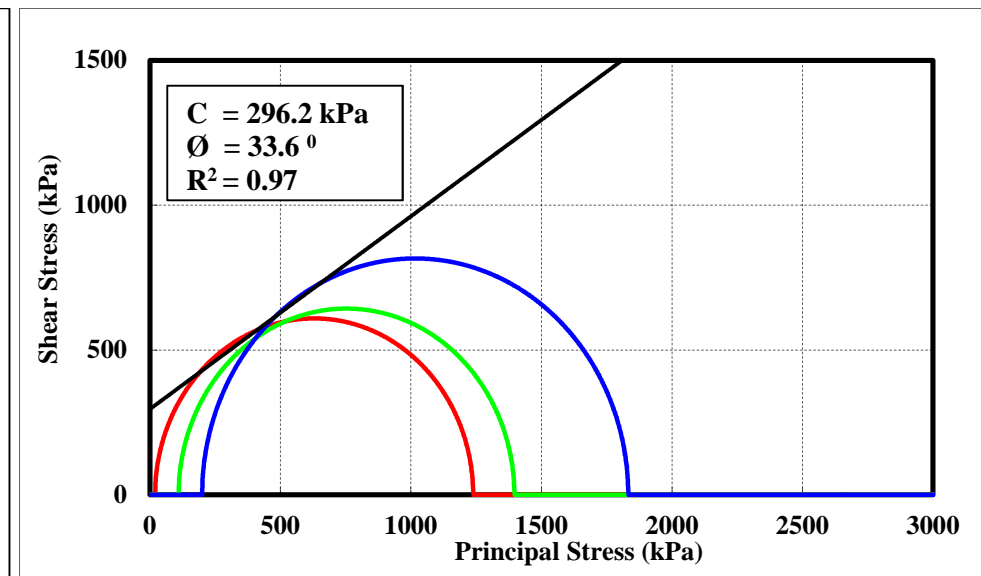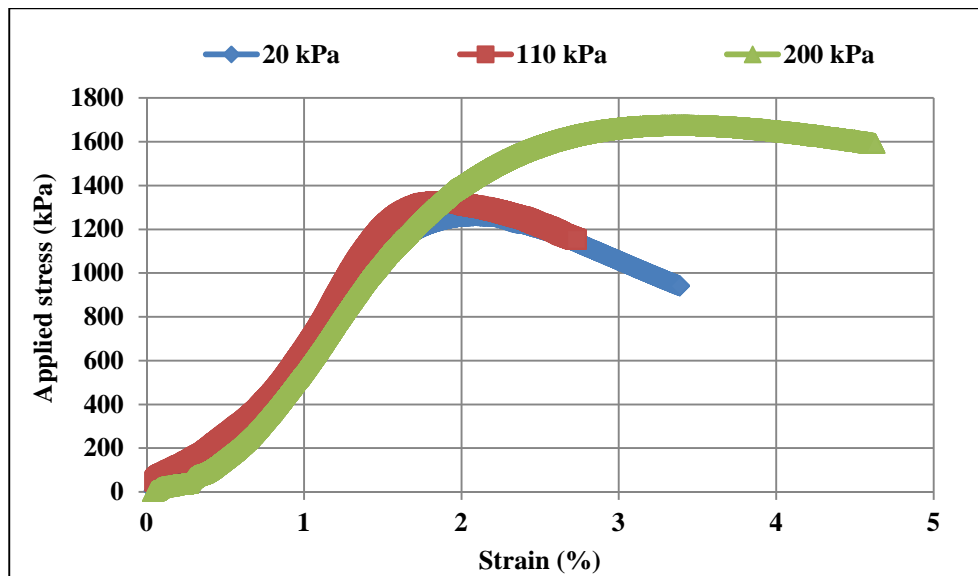
Figure D-8: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM1-LD-LS
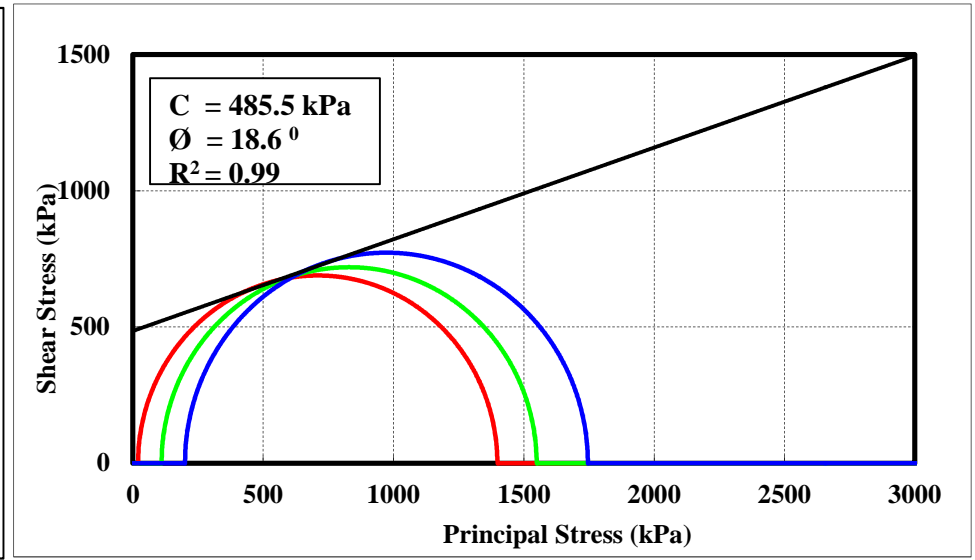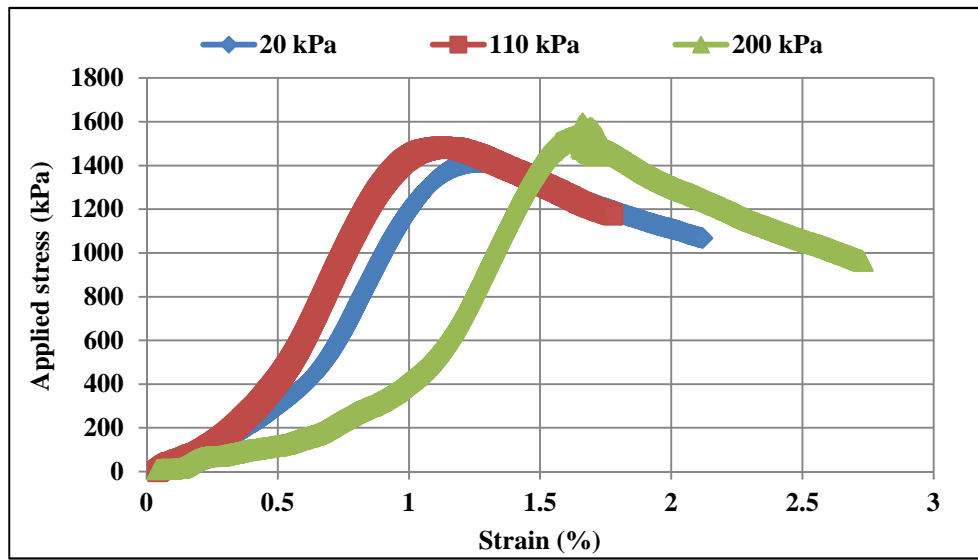
Figure D-9: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM2-HD-HS
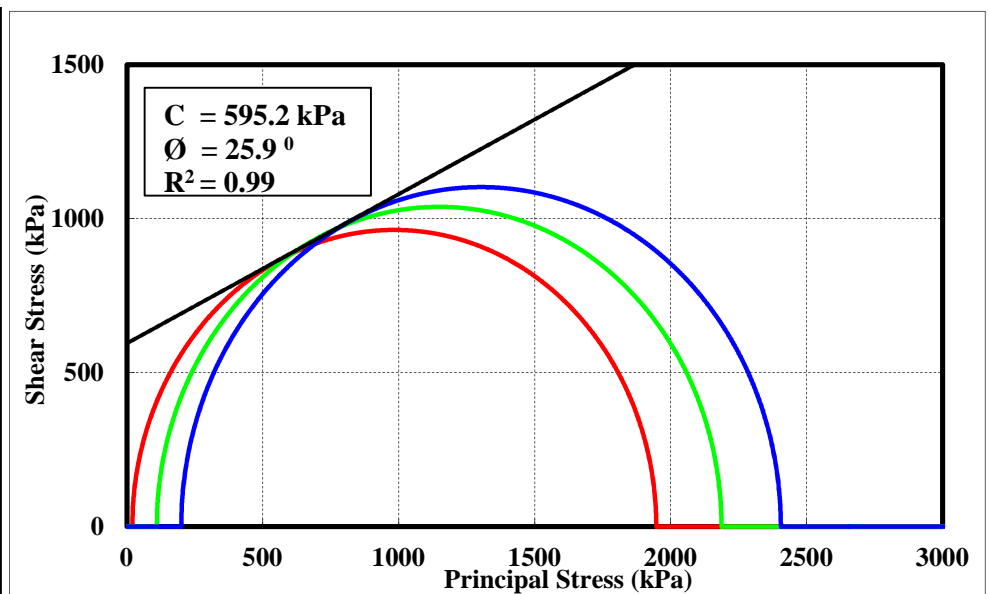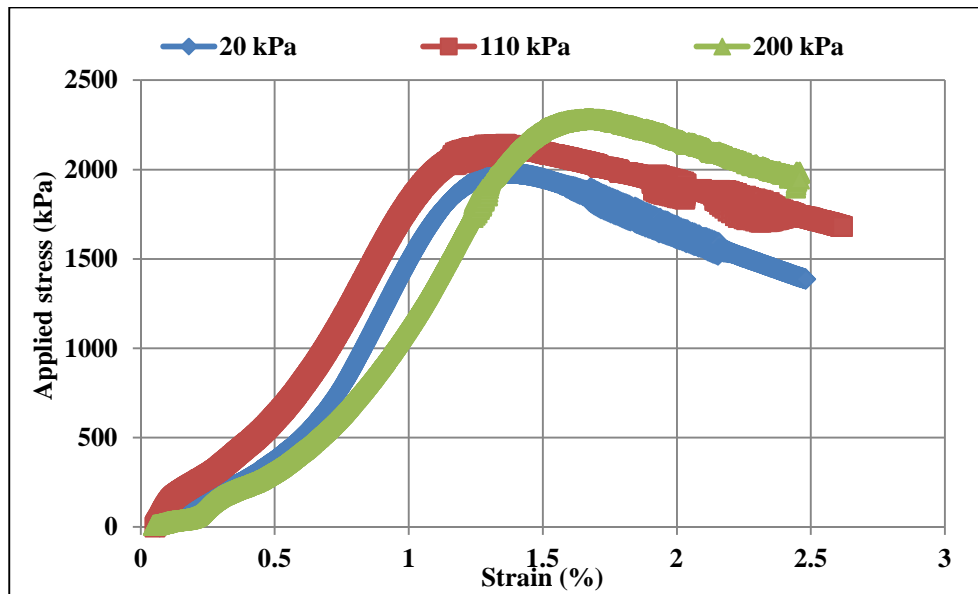
Figure D-10: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM2-HD-LS
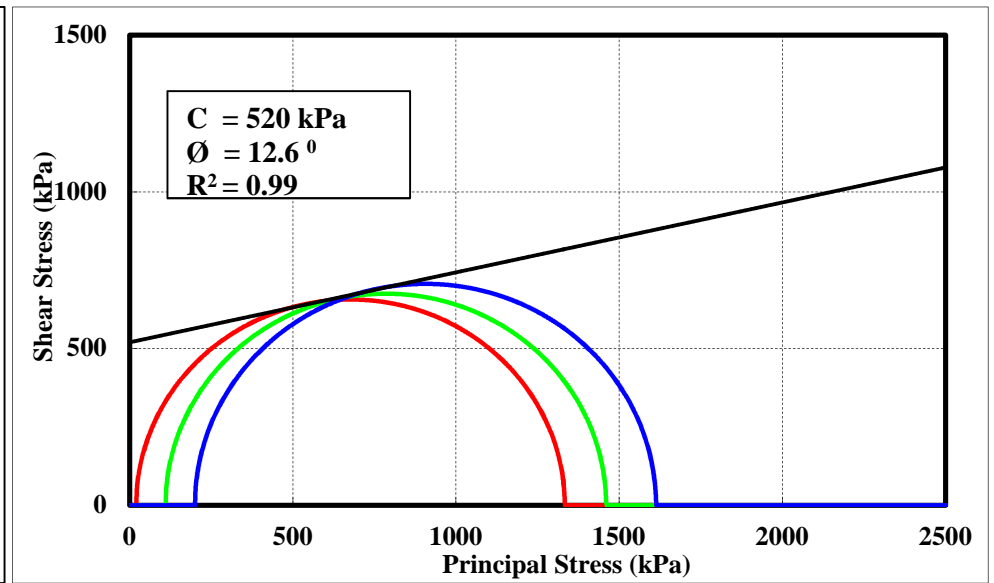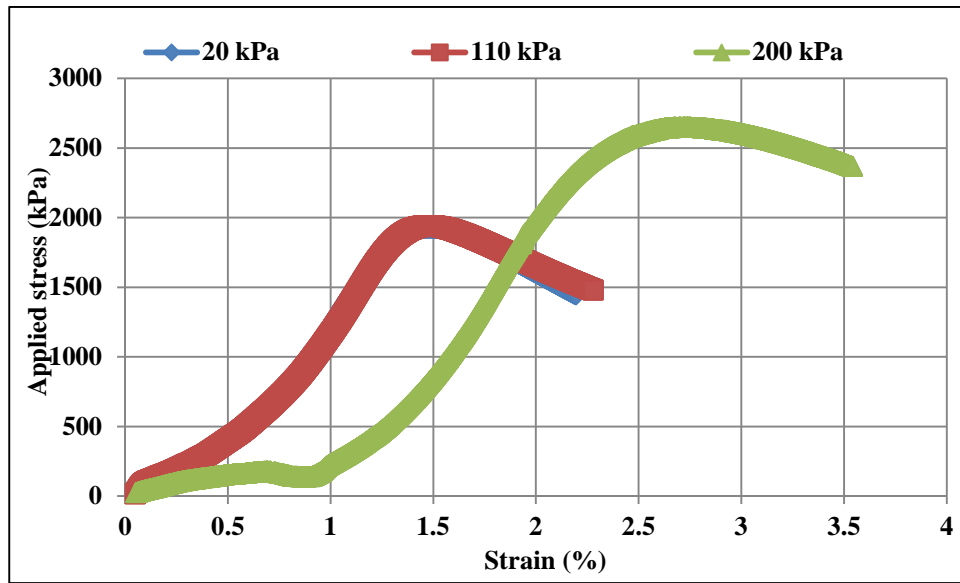
Figure D-11: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM2-LD-HS
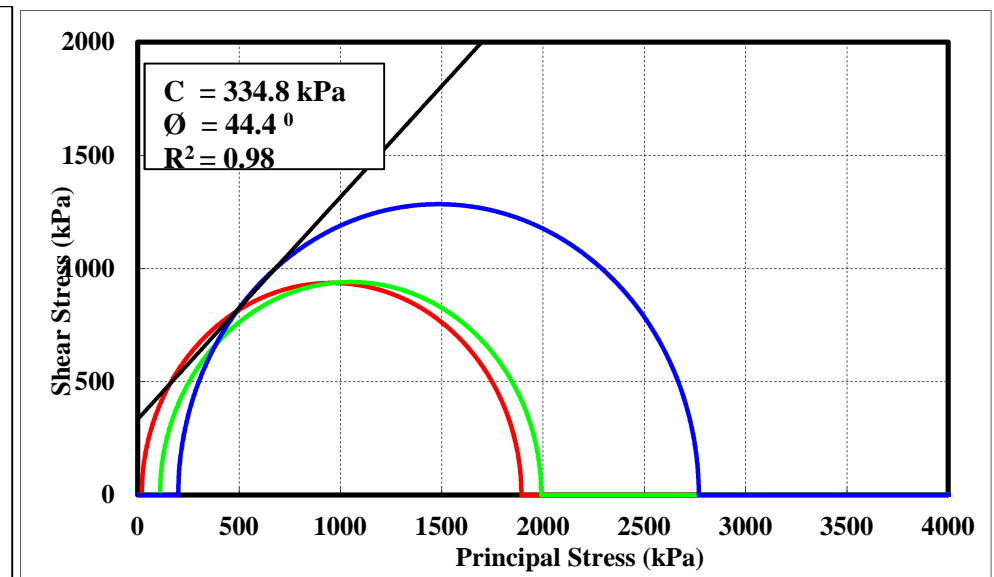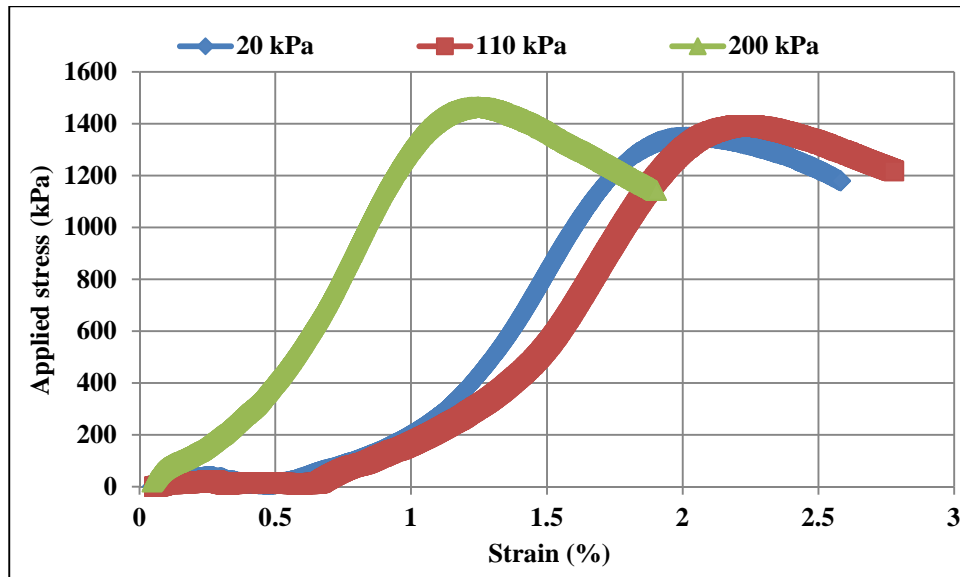
Figure D-12: Stress-Strain and Mohr-Coulomb Diagram, Mix EB2.4-CM2-LD-LS
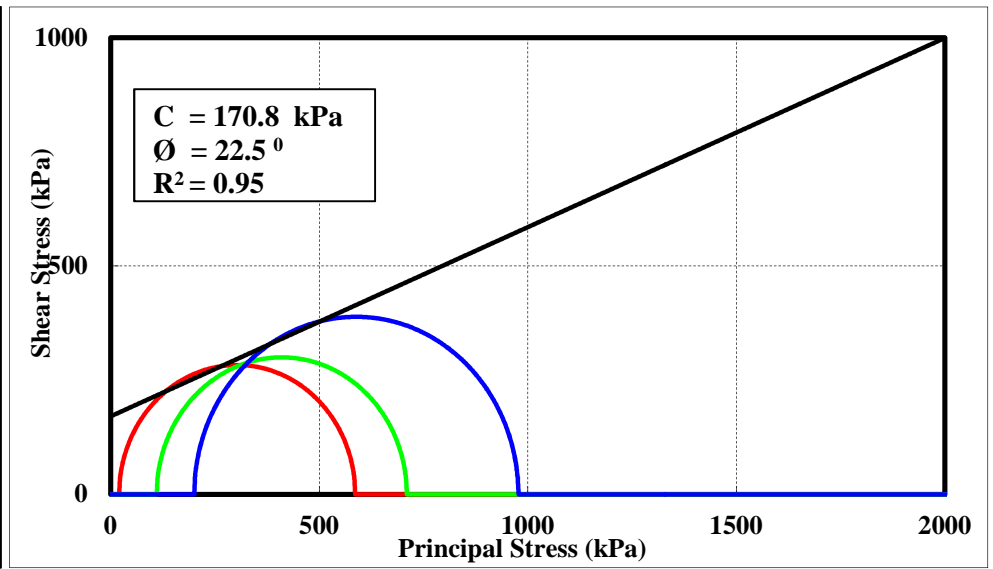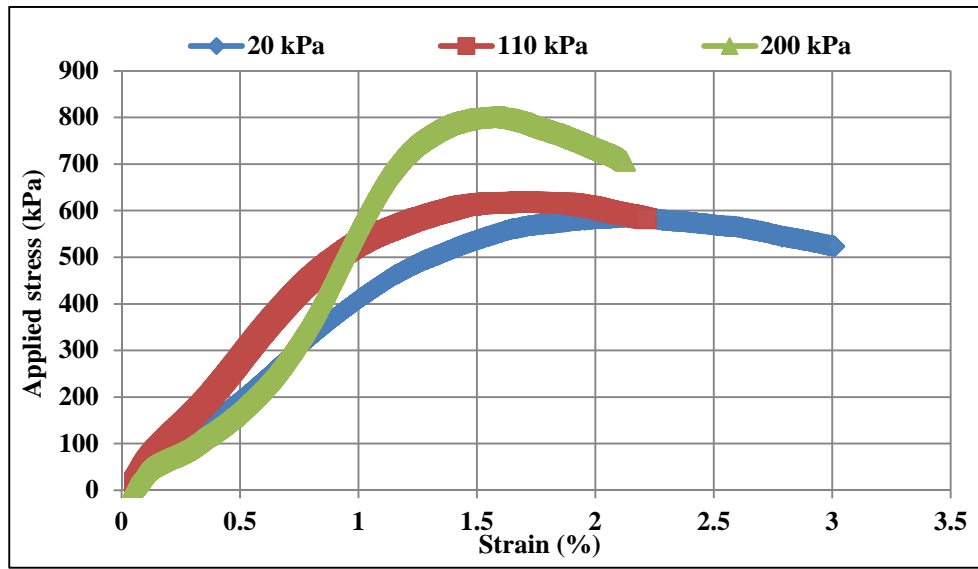
Figure D-13: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM1-HD-HS
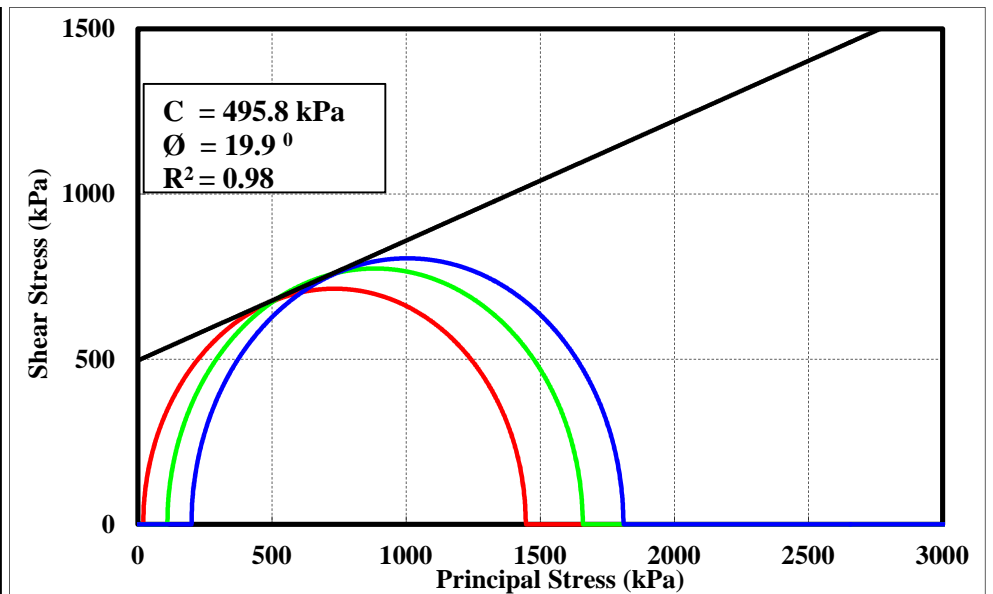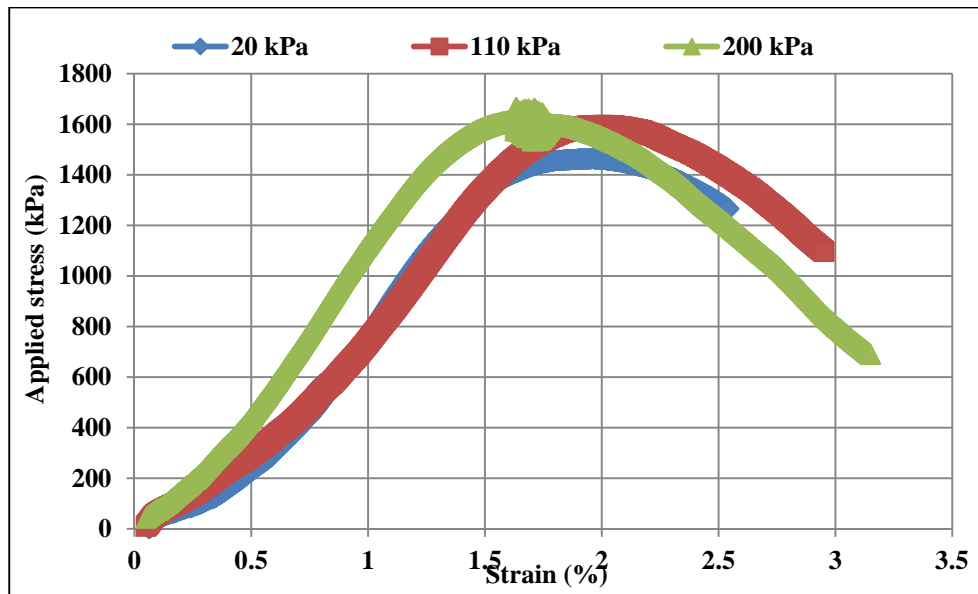
Figure D-14: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM1-HD-LS
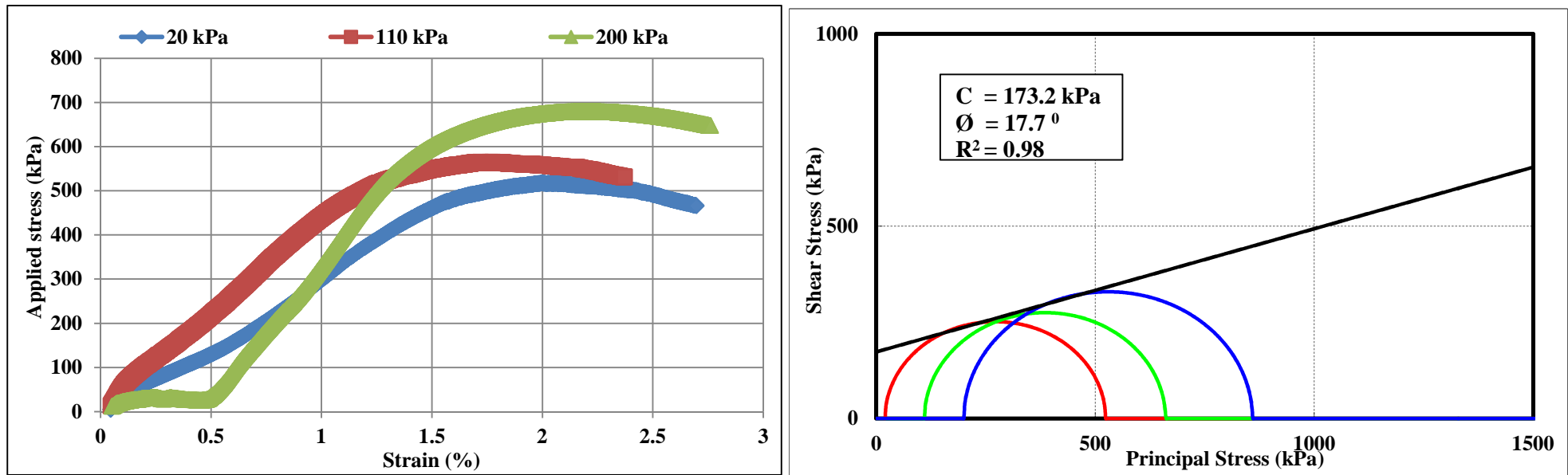
Figure D-15: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM1-LD-HS
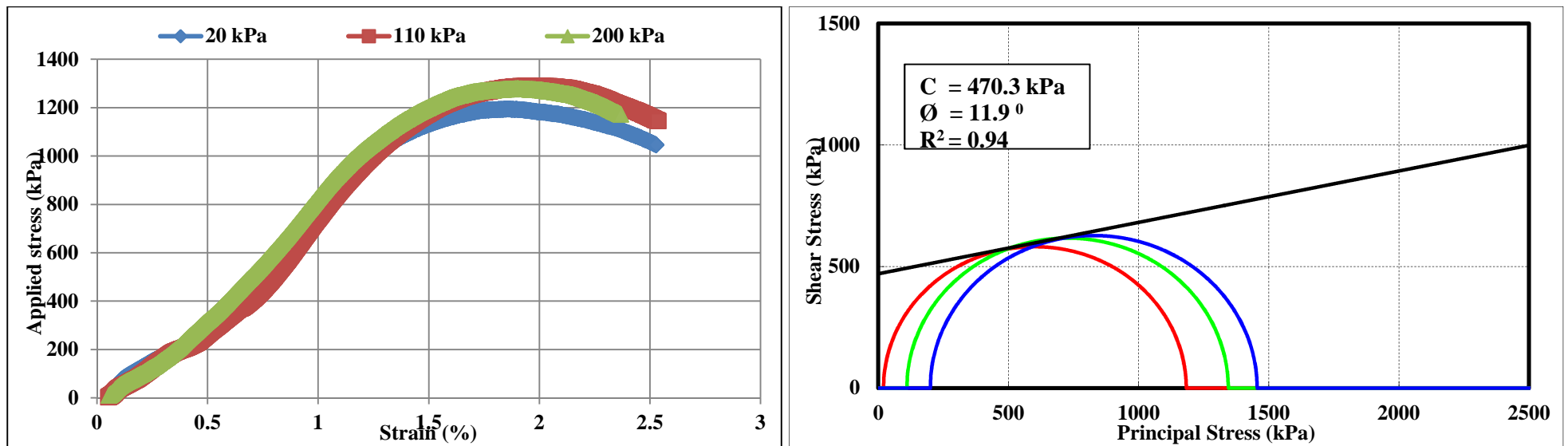
Figure D-16: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM1-LD-LS
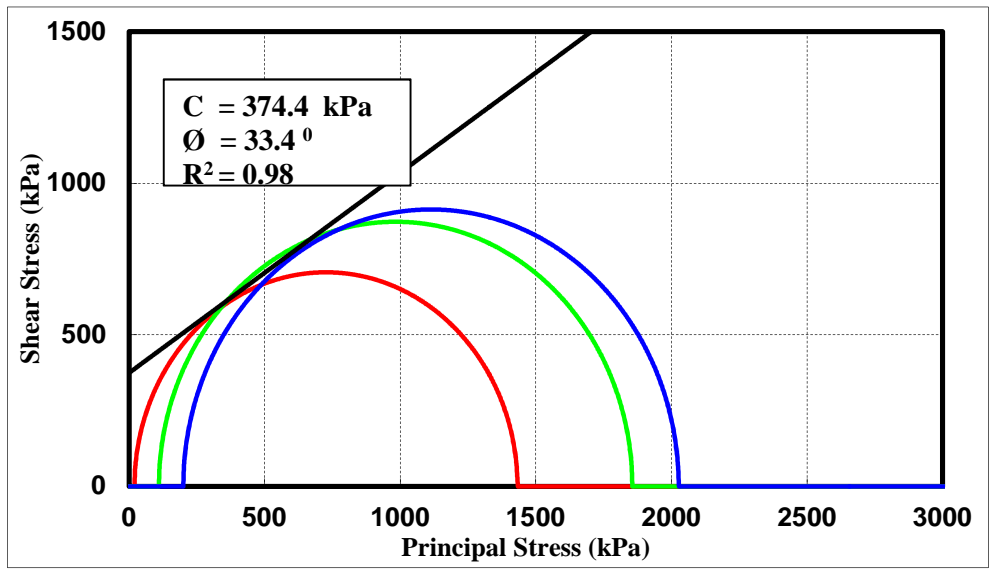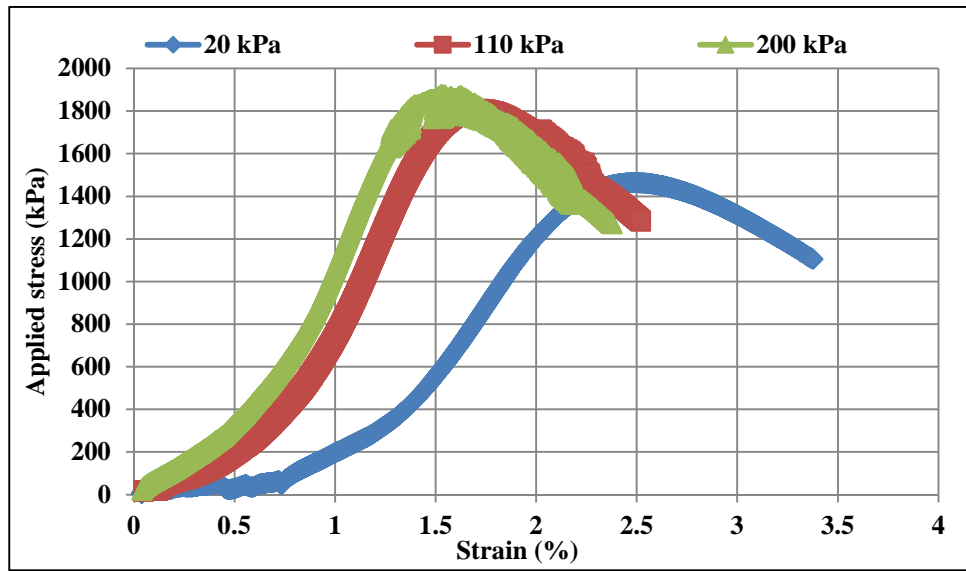
Figure D-17: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM2-HD-HS
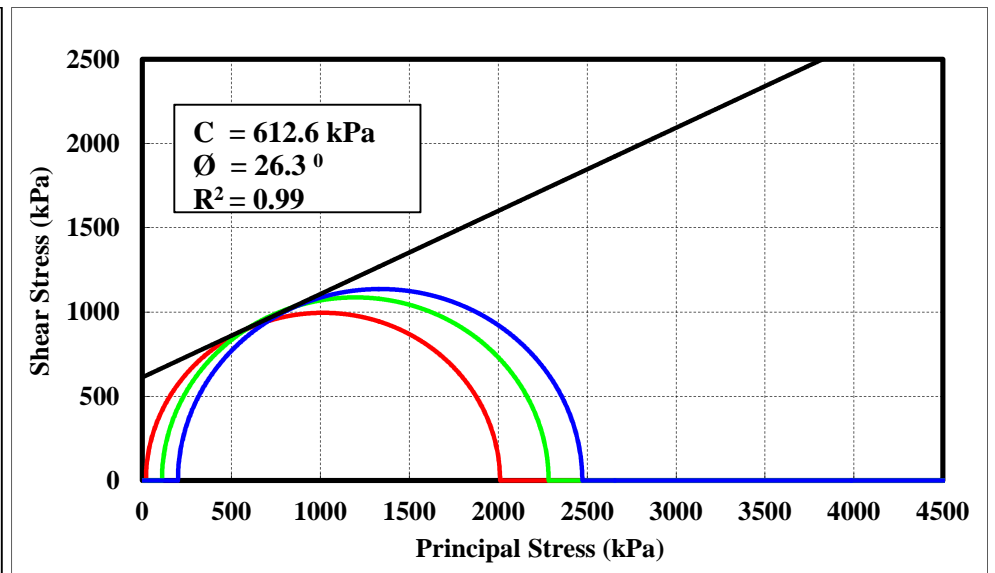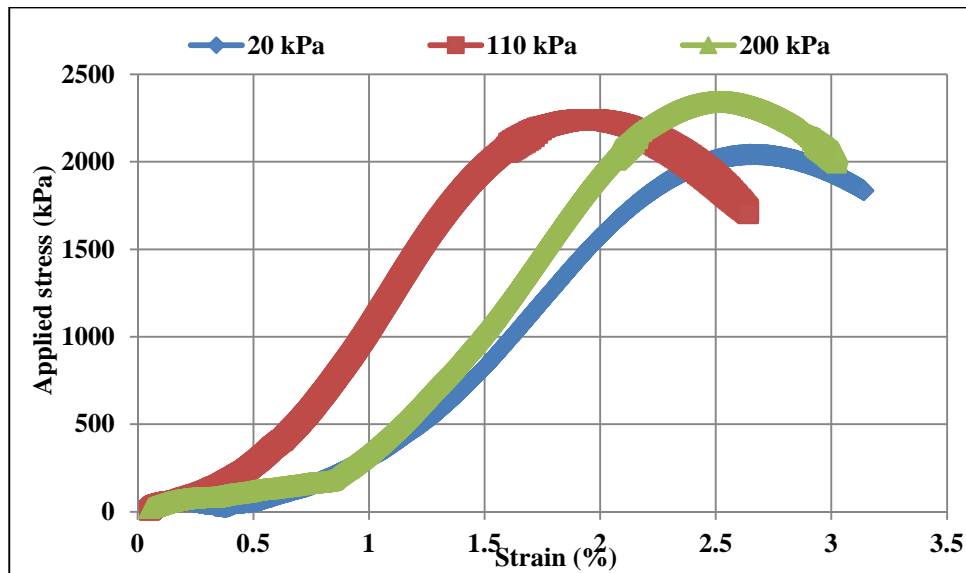
Figure D-18: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM2-HD-LS
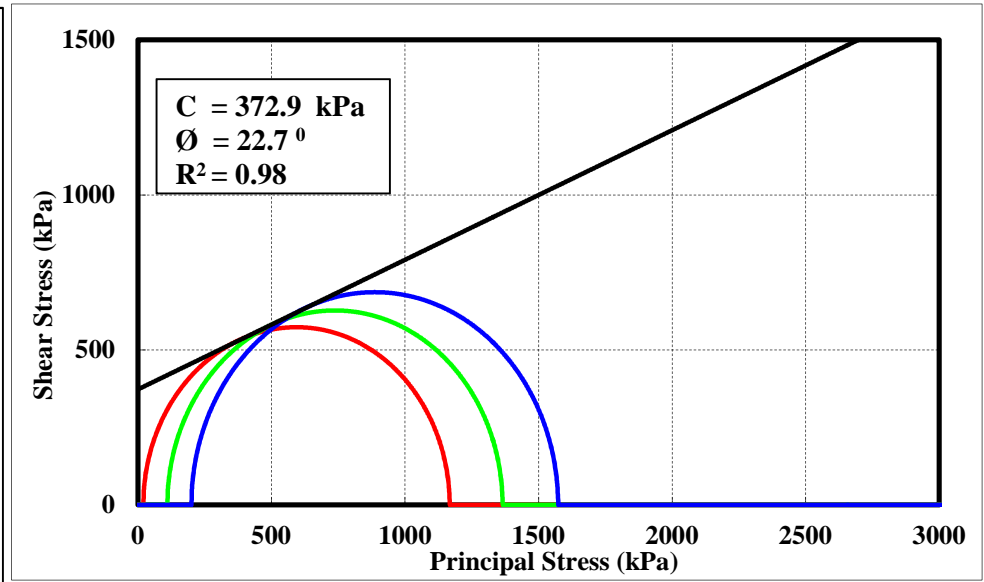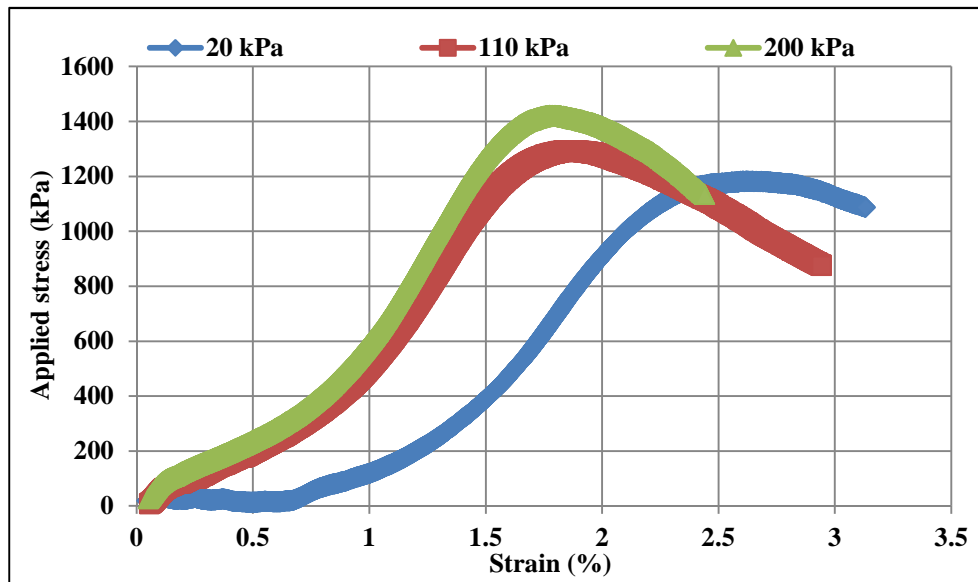
Figure D-19: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM2-LD-HS
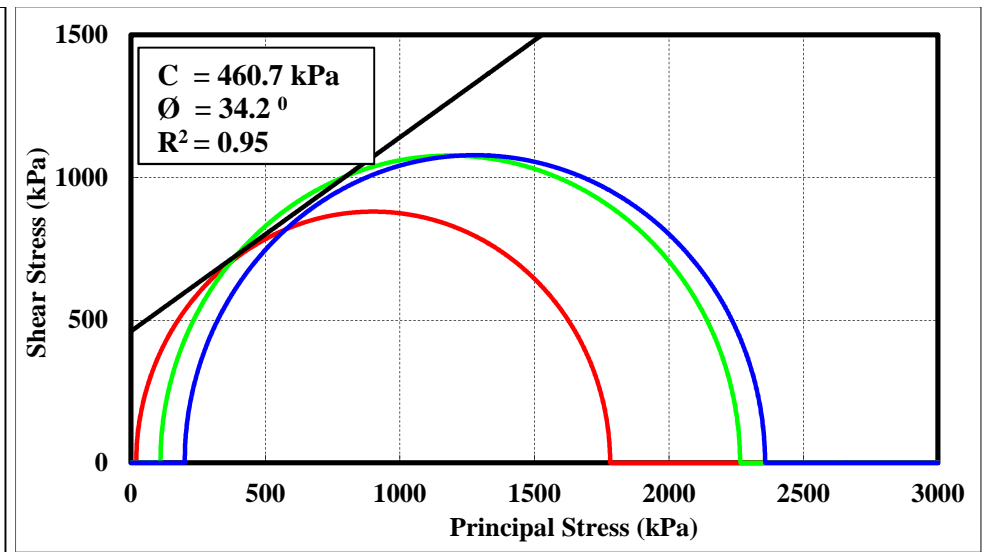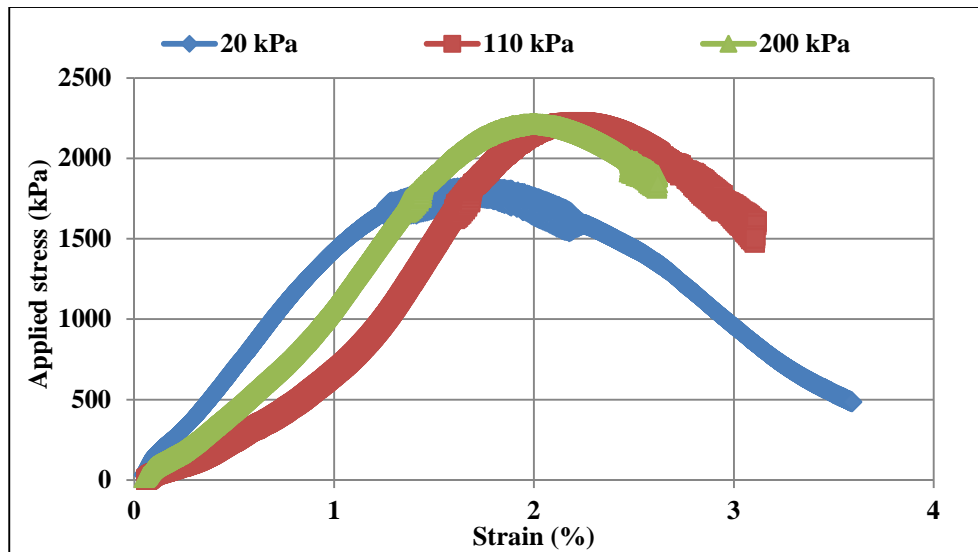
Figure D-20: Stress-Strain and Mohr-Coulomb Diagram, Mix FB2.4-CM2-LD-LS
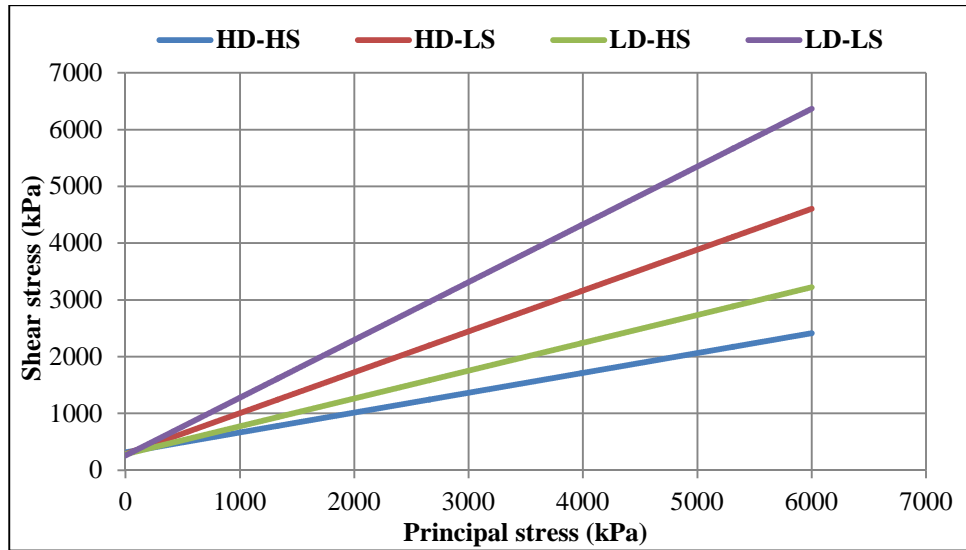
Figure D-1: Maximum shear stress for mix EB0.9-CM1
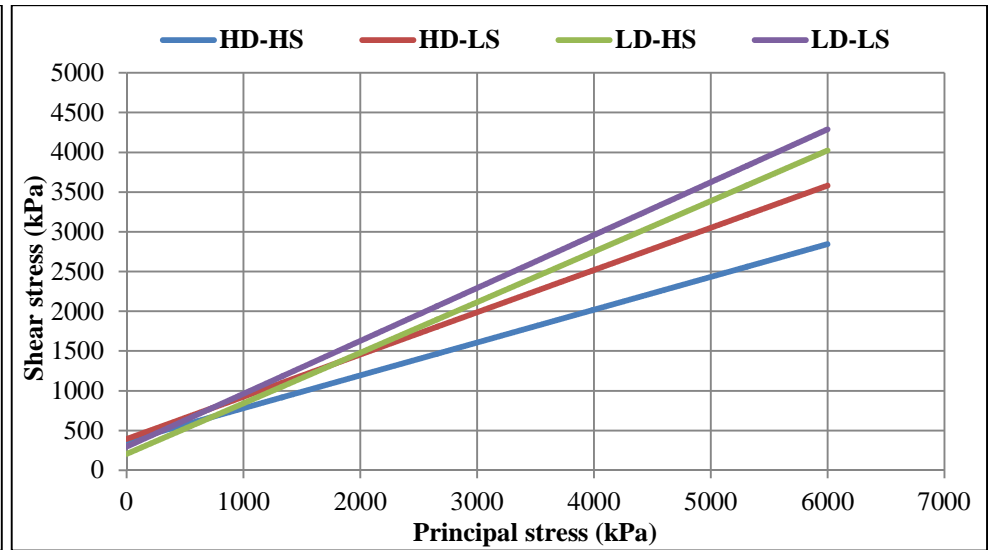
Figure D-2: Maximum shear stress for mix EB2.4-CM1
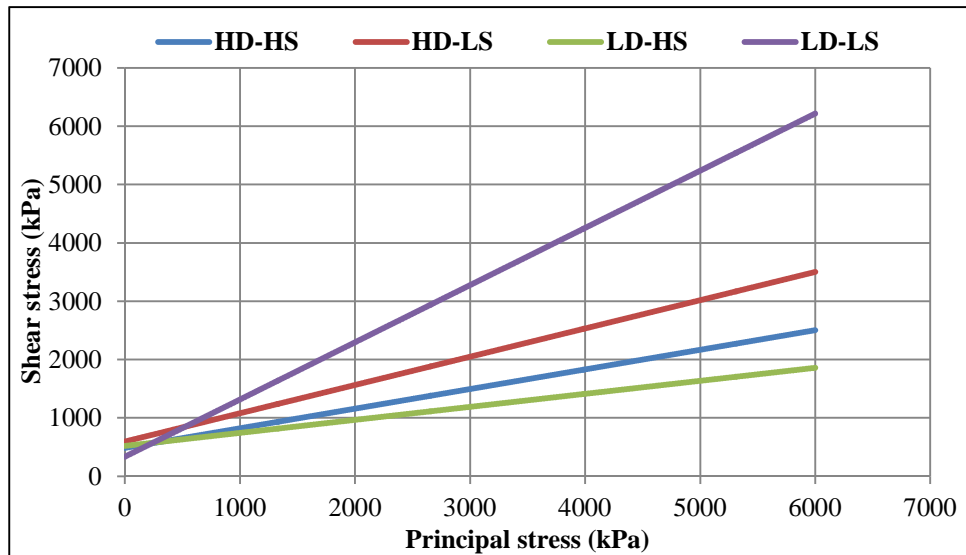
Figure D-3: Maximum shear stress for mix EB2.4-CM2
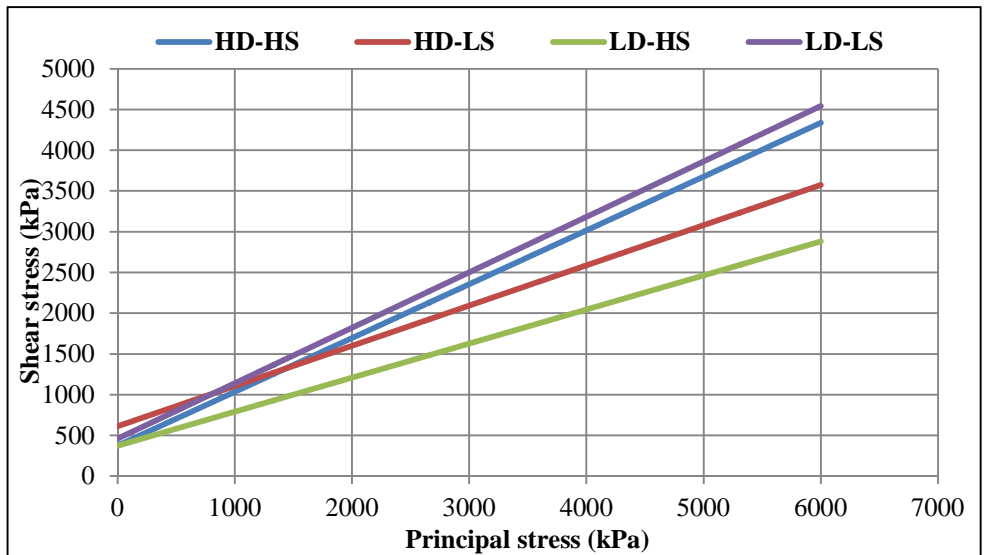
Figure D-4: Maximum shear stress for mix FB2.4-CM2

Appendix F: Estimated compressive and tensile strength from Mohr Coulomb diagram
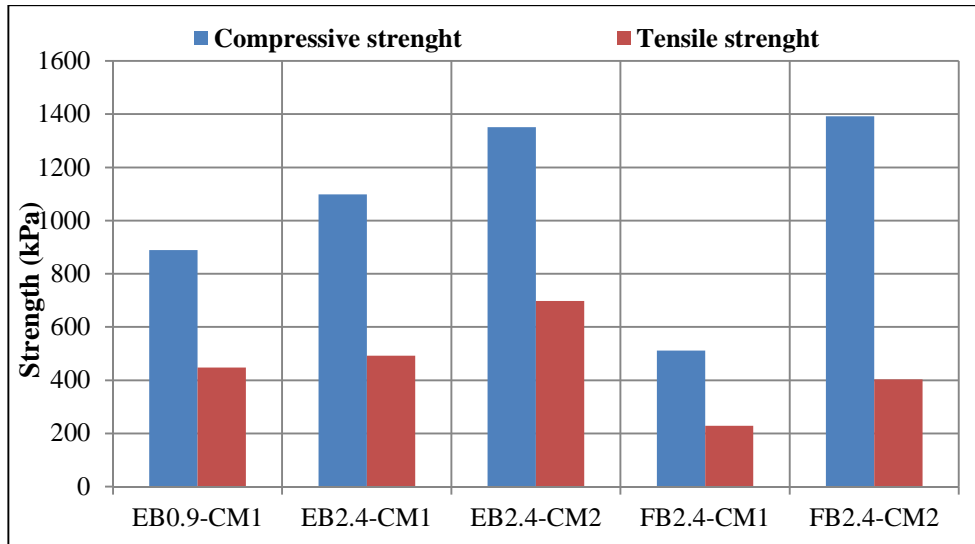
Figure F-1: Mixes of high density-high saturation
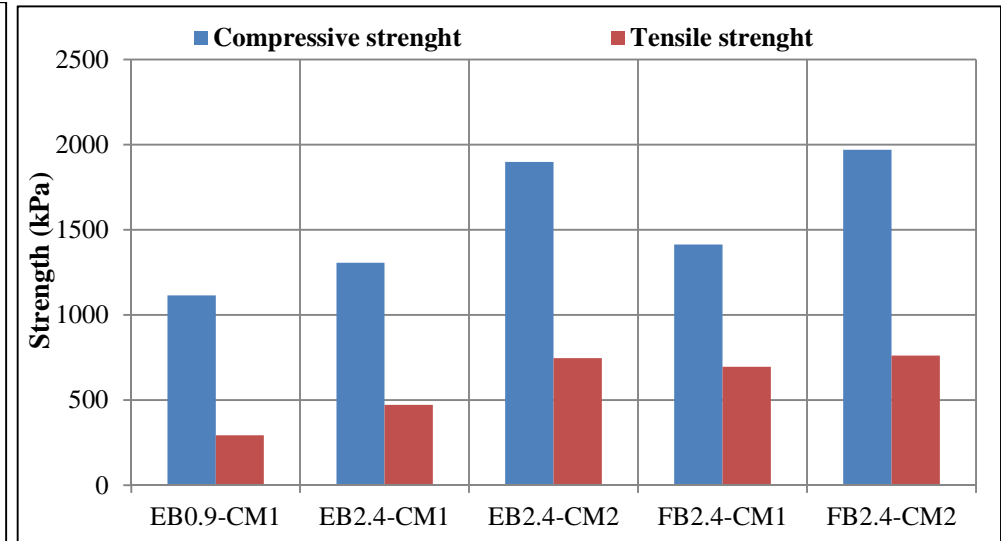
Figure F-2: Mixes of high density-low saturation
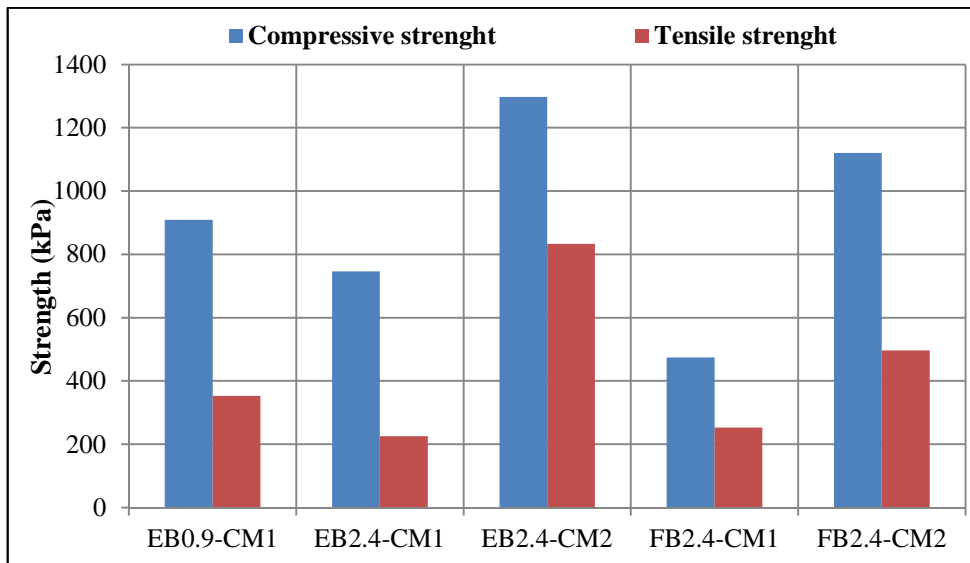
Figure F-3: Mixes of low density-high saturation
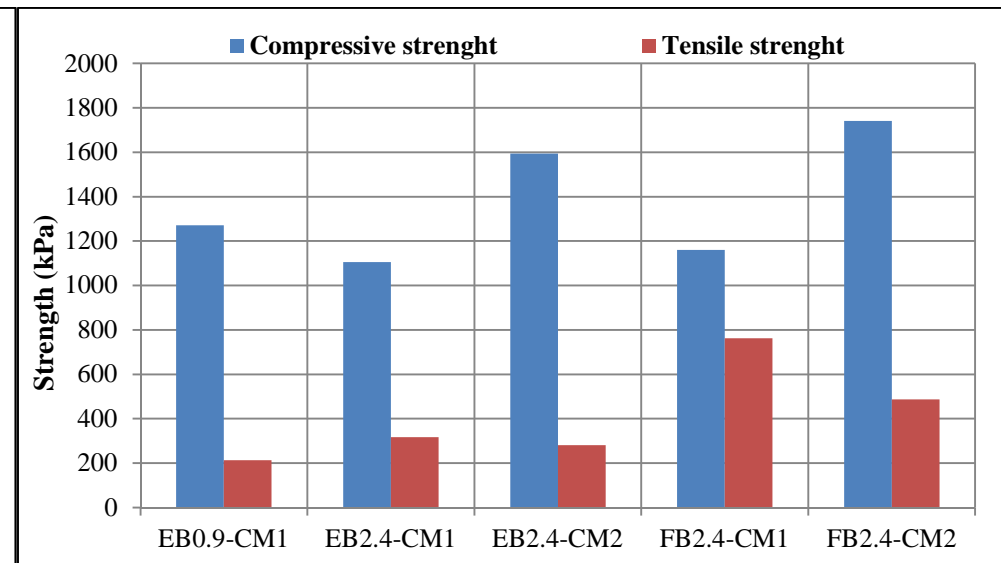
Figure F-3: Mixes of low density-low saturation

Appendix G: Short term dynamic triaxial test results per specimen

| Test conditions | | BITUMEN EMULSION 0.9%- 1% CEMENT | | | | | | | | | | | | | | | | | | | |
| Test conditions | | High Density-High Saturation | | | | | High Density-Low Saturation | | | | | Low Density-High Saturation | | | | | Low Density-Low Saturation | | | | |
Phases	SR	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 4	Sp 5	Sp 6	Aver*	COV**	Sp7	Sp 8	Sp 9	Aver*	COV**	Sp 10	Sp 11	Sp 12	Aver*	COV**
Cond (200 kPa)	20%	834.5	401.9		618.2	49.5	1121.6	1881.3	813.8	1272.2	43.2	1012.6	445.7	1030.3	829.5	40.1	1809.7	1274.2	1990.6	1691.5	22.0
	50%	749.7	581.5		665.6	17.9	1288.9	1689.2	964.0	1314.1	27.6	867.5	449.7	674.7	664.0	31.5	804.0	855.7	1632.2	1097.3	42.3
	70%	733.3	646.5		689.9	8.9	1162.0	1481.1	1744.5	1462.5	19.9	737.8	359.6	533.3	543.6	34.8	808.0	841.2	1505.0	1051.4	37.4
Phase 1 (200kPa)	20%	611.7	326.8		469.2	42.9	992.6	1352.2	1820.5	1388.4	29.9	496.3	204.7	328.0	343.0	42.7	531.2	535.2	1484.3	850.2	64.6
	40%	587.4	458.7		523.1	17.4	992.7	1298.4	1620.5	1303.9	24.1	535.4	285.3	398.7	406.5	30.8	644.0	661.2	1459.0	921.4	50.5
	50%	642.4	535.6		589.0	12.8	1050.7	1352.2	1660.2	1354.3	22.5	551.6	326.1	455.8	444.5	25.5	720.7	735.6	1493.4	983.2	44.9
	60%	720.9	605.7		663.3	12.3	1117.0	1410.5	1741.1	1422.9	21.9	531.7	344.5	572.6	482.9	25.2	855.0	860.4	1640.2	1118.6	40.4
	70%	619.1	647.8		633.4	3.2	1140.7	1420.8	1744.1	1435.2	21.0	449.4	329.0	503.3	427.2	20.9	829.4	839.9	1563.4	1077.6	39.0
Phase 1 (150kPa)	20%	375.9	310.4		343.2	13.5	795.3	1064.6	1413.9	1091.3	28.4	200.6	163.7	272.6	212.3	26.1	543.8	500.1	1374.0	805.9	61.1
	40%	424.5	434.4		429.5	1.6	1091.1	1055.7	1316.3	1154.4	12.2	276.4	250.3	360.7	295.8	19.5	640.4	630.0	1311.5	860.7	45.4
	50%	491.5	495.0		493.2	0.5	886.8	1138.5	1380.4	1135.2	21.7	318.1	289.7	409.7	339.2	18.5	705.7	698.9	1340.8	915.1	40.3
	60%	537.8	552.8		545.3	1.9	953.6	1186.2	1484.3	1208.0	22.0	355.7	313.1	449.0	372.6	18.6	773.1	769.1	1366.8	969.6	35.5
	70%	580.6	620.7		600.6	4.7	1015.2	1241.4	1539.4	1265.3	20.8	355.6	381.7	470.9	402.7	15.0	837.1	831.7	1412.6	1027.1	32.5
Phase 1 (100kPa)	20%	289.4	296.7		293.1	1.8	609.0	792.0	988.2	796.4	23.8	180.7	160.3	256.9	199.3	25.5	547.2	487.2	1200.5	745.0	53.1
	40%	433.3	400.4		416.8	5.6	686.8	866.1	1068.3	873.7	21.8	276.4	250.3	360.7	295.8	19.5	639.3	610.4	1123.6	791.1	36.4
	50%	425.0	476.0		450.5	8.0	746.2	939.9	1166.3	950.8	22.1	318.1	403.2	409.7	377.0	13.6	699.6	657.1	1098.1	818.3	29.7
	60%	503.5	548.5		526.0	6.0	813.6	1011.6	1271.7	1032.3	22.3	323.8	574.4	436.5	444.9	28.2	760.2	739.4	1182.5	894.0	28.0
	70%	542.1	602.5		572.3	7.5	875.3	1080.1	1370.0	1108.5	22.4	318.9	617.9	468.8	468.5	31.9	821.8	801.4	1222.9	948.7	25.1
Phase 1 (50 kPa)	20%	281.6	284.3		283.0	0.7	479.9	658.8	758.3	632.4	22.3	165.3	322.5	258.5	248.8	31.8	539.5	473.0	935.8	649.5	38.5
	40%	385.5	388.6		387.0	0.6	583.7	762.1	935.8	760.5	23.2	233.7	457.0	338.0	342.9	32.6	621.0	586.5	899.7	702.4	24.4
	50%	438.0	458.4		448.2	3.2	651.7	888.5	1142.7	894.3	27.5	266.0	596.4	382.8	415.0	40.4	678.9	647.6	945.2	757.2	21.6
	60%	485.0	523.0		504.0	5.3	714.9	877.4	1127.7	906.7	22.9	287.4	659.5	430.8	459.2	40.9	733.5	708.1	994.7	812.1	19.5
	70%	515.4	576.8		546.1	7.9	780.7	957.7	1220.3	986.2	22.4	269.3	707.3	469.8	482.1	45.5	797.1	768.1	1046.8	870.6	17.6
Phase 5 (20kPa)	20%	279.1	277.2		278.1	0.5	451.0	639.2	756.1	615.4	25.0	165.3	357.8	254.3	259.1	37.2	536.0	469.0	520.0	508.3	6.9
	40%	372.3	377.7		375.0	1.0	562.6	735.0	946.9	748.1	25.7	233.7	525.7	335.4	364.9	40.6	614.6	572.8	592.0	593.1	3.5
	50%	427.0	445.1		436.1	2.9	623.9	806.4	1051.8	827.4	26.0	266.0	808.0	382.8	485.6	58.7	665.9	632.4	612.0	636.8	4.3
	60%	474.7	513.7		494.2	5.6	688.4	872.8	1149.7	903.6	25.7	287.4	919.5	426.3	544.4	61.0	721.1	689.7	718.5	709.8	2.5
	70%	512.8	553.2		533.0	5.4	749.2	945.0	1239.2	977.8	25.2	269.3	465.0	464.0	399.4	28.2	780.9	746.9	768.4	765.4	2.2

* Average Mr in kPa; ** Coefficient of Variation in (%)

		High Density-High Saturation					High Density-Low Saturation					Low Density-High Saturation					Low Density-Low Saturation				
Test conditions																						
Phases	**SR**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**
Cond (200 kPa)	20%	1178.6	1098.9	1534.3	1270.6	18.2	744.6	679.5	1561.6	995.2	49.4	450.5	1034.4	852.4	779.1	38.3	1813.0	1492.2	2221.4	1842.2	19.8
	50%	860.2	833.8	1235.4	976.5	23.0	802.9	906.8	1211.7	973.8	21.8	616.1	750.9	578.8	648.6	14.0	1378.5	1344.2	1496.3	1406.3	5.7
	70%	760.7	635.4	1078.3	824.8	27.7	840.7	940.0	1083.1	954.6	12.8	691.3	711.5	499.0	633.9	18.5	1123.8	1261.0	1391.3	1258.7	10.6
Phase 1 (200 kPa)	20%	465.9	474.7	1018.4	653.0	48.5	597.4	610.8	1089.2	765.8	36.6	422.9	440.0	320.8	394.5	16.3	1226.0	1215.4	1472.5	1304.6	11.2
	40%	582.5	473.6	917.5	657.9	35.2	692.8	767.6	966.9	809.1	17.5	535.3	541.9	388.2	488.5	17.8	1085.4	1184.4	1341.5	1203.8	10.7
	50%	655.7	506.7	909.8	690.7	29.5	759.1	830.2	974.6	854.6	12.9	578.6	609.7	444.3	544.2	16.2	1094.3	1172.4	1327.7	1198.1	9.9
	60%	720.1	526.7	935.7	727.5	28.1	813.6	908.6	992.5	904.9	9.9	609.1	674.3	478.5	587.3	17.0	1114.5	1218.7	1364.2	1232.5	10.2
	70%	743.8	509.7	897.4	717.0	27.2	820.7	933.4	983.2	912.4	9.1	618.5	713.6	497.5	609.9	17.8	1105.2	1236.7	1358.4	1233.4	10.3
Phase 1 (150kPa)	20%	426.3	312.1	567.5	435.3	29.4	562.2	637.4	741.9	647.2	13.9	366.0	426.3	297.4	363.2	17.8	897.3	1061.3	1268.6	1075.7	17.3
	40%	548.5	387.4	634.8	523.6	24.0	660.1	749.1	752.6	720.6	7.3	454.2	531.3	367.2	450.9	18.2	836.4	1003.2	1147.8	995.8	15.6
	50%	611.0	433.1	668.6	570.9	21.5	717.1	822.3	786.6	775.3	6.9	507.3	587.9	417.5	504.2	16.9	861.7	1017.5	1164.8	1014.7	14.9
	60%	669.3	471.3	698.2	612.9	20.1	776.6	890.2	823.4	830.1	6.9	561.3	648.0	464.8	558.0	16.4	900.4	1061.2	1180.6	1047.4	13.4
	70%	690.2	481.0	712.0	627.7	20.3	802.6	940.8	832.1	858.5	8.5	602.1	702.6	501.1	601.9	16.7	904.9	1111.5	1210.7	1075.7	14.5
Phase 1 (100 kPa)	20%	399.3	294.0	416.1	369.8	17.9	544.7	640.5	552.1	579.1	9.2	356.4	410.9	288.7	352.0	17.4	607.5	863.6	983.5	818.2	23.5
	40%	510.1	367.9	524.7	467.6	18.5	633.5	752.1	632.3	672.6	10.2	441.5	500.3	360.6	434.1	16.2	636.4	832.0	942.7	803.7	19.3
	50%	572.7	412.8	584.9	523.5	18.3	690.8	812.6	680.9	728.1	10.1	488.3	555.4	405.2	483.0	15.6	683.3	865.7	968.9	839.3	17.2
	60%	625.6	450.8	640.9	572.4	18.4	747.1	879.2	723.3	783.2	10.7	537.4	609.7	450.9	532.7	14.9	720.5	915.5	1004.4	880.1	16.5
	70%	657.4	473.1	679.3	603.2	18.8	780.6	935.2	741.6	819.1	12.5	583.0	664.5	495.6	581.0	14.5	748.5	957.9	1036.6	914.3	16.3
Phase 1 (50 kPa)	20%	392.4	281.0	398.1	357.2	18.5	534.0	674.0	527.5	578.5	14.3	330.3	325.5	276.1	310.6	9.7	547.1	631.5	771.0	649.9	17.4
	40%	508.7	353.1	507.8	456.5	19.6	615.6	751.7	611.1	659.5	12.1	412.8	432.1	345.8	396.9	11.4	596.7	664.0	785.4	682.0	14.0
	50%	555.7	395.6	568.5	506.6	19.0	668.2	817.5	659.0	714.9	12.4	457.3	486.4	387.8	443.8	11.4	645.2	708.6	826.7	726.8	12.7
	60%	598.8	437.5	622.1	552.8	18.2	723.3	885.4	708.7	772.5	12.7	505.9	542.3	431.5	493.2	11.4	701.8	757.0	871.4	776.7	11.1
	70%	632.4	464.3	667.4	588.1	18.5	765.7	943.4	748.5	819.2	13.2	552.2	598.7	471.3	540.7	11.9	748.6	802.2	921.3	824.0	10.7
Phase 5 (20kPa)	20%	384.8	272.5	396.6	351.3	19.5	526.0	684.1	511.2	573.8	16.7	324.9	325.5	268.4	306.3	10.7	540.9	544.3	730.5	605.2	17.9
	40%	490.7	344.7	508.0	447.8	20.0	606.8	758.4	599.8	655.0	13.7	412.5	432.1	338.1	394.2	12.6	592.9	606.7	731.3	643.6	11.8
	50%	541.7	387.7	570.9	500.1	19.7	658.2	824.7	648.8	710.6	13.9	457.7	486.4	377.9	440.6	12.8	644.7	656.5	776.5	692.5	10.5
	60%	585.5	428.4	629.4	547.8	19.3	711.2	871.8	697.8	760.2	12.7	503.5	542.3	415.6	487.1	13.3	678.6	703.4	823.9	735.3	10.6
	70%	624.6	458.9	663.7	582.4	18.7	760.9	930.5	742.0	811.1	12.8	552.2	598.7	458.6	536.5	13.3	721.5	747.6	863.3	777.5	9.7

Title: **BITUMEN EMULSION 2.4%- 1% CEMENT**

		**BITUMEN EMULSION 2.4%-2% CEMENT**														
Test conditions		**High Density-High Saturation**					**High Density-Low Saturation**					**Low Density-High Saturation**					**Low Density-Low Saturation**				
Phases	**SR**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**
Cond (200 kPa)	20%	1522.3	2985.7	1793.4	2100.5	37.1		2621.7	2975.2	2798.5	8.9	7119.4	1477.1	2003.9	3533.5	88.2	2522.7	2019.6	2107.5	2216.6	12.1
	50%	1455.6	1553.8	1727.3	1578.9	8.7	2492.7	2033.1	1785.7	2103.8	17.1	2505.7	1083.6	1430.6	1673.3	44.3	1628.3	1576.1	1295.3	1499.9	11.9
	70%	1471.9	1355.1	1355.1	1394.0	4.8	1484.5	1809.7	1525.3	1606.5	11.0	1622.1	964.1	1041.5	1209.2	29.7	1443.2	1368.1	1092.6	1301.3	14.2
Phase 1 (200 kPa)	20%	989.1	1051.3	1068.3	1036.2	4.0	2151.9	1606.7	1268.4	1675.7	26.6	1914.7	621.6	831.2	1122.5	61.8	1134.1	985.3	738.2	952.6	21.0
	40%	1186.6	1099.4	1332.3	1206.1	9.8	1423.9	1532.8	1299.5	1418.8	8.2	1645.7	732.0	840.7	1072.8	46.5	1195.2	1096.0	850.2	1047.1	17.0
	50%	1296.0	1166.7	1368.1	1276.9	8.0	1414.9	1556.1	1371.5	1447.5	6.7	1536.6	808.7	885.3	1076.8	37.1	1273.0	1192.2	925.6	1130.3	16.1
	60%	1412.8	1248.6	1438.1	1366.5	7.5	1438.2	1596.4	1452.3	1495.6	5.9	1546.3	874.3	933.1	1117.9	33.3	1353.4	1270.8	992.6	1205.6	15.7
	70%	1467.5	1268.7	1461.6	1399.3	8.1	1373.9	1570.6	1446.0	1463.5	6.8	1458.0	912.7	912.7	1094.5	28.8	1377.2	1283.4	987.3	1216.0	16.7
Phase 1 (150kPa)	20%	929.1	898.0	1002.3	943.1	5.7	1461.4	1269.6	1117.0	1282.6	13.5	1444.2	557.6	661.9	887.9	54.6	1015.7	823.8	638.9	826.1	22.8
	40%	1121.5	982.9	1150.6	1085.0	8.3	1240.4	1310.4	1189.9	1246.9	4.9	1217.2	700.0	744.2	887.2	32.3	1110.4	949.3	766.5	942.1	18.3
	50%	1246.8	1086.1	1245.4	1192.8	7.7	1267.9	1385.0	1267.2	1306.7	5.2	1243.7	783.1	803.7	943.5	27.6	1187.8	1024.3	837.5	1016.5	17.2
	60%	1356.4	1160.4	1340.4	1285.7	8.5	1316.5	1456.4	1348.5	1373.8	5.3	1301.8	855.4	863.9	1007.0	25.4	1282.4	1107.4	912.4	1100.7	16.8
	70%	1434.0	1211.2	1412.7	1352.6	9.1	1352.3	1506.5	1397.4	1418.7	5.6	1326.3	906.7	896.1	1043.0	23.5	1361.7	1176.4	954.2	1164.1	17.5
Phase 1 (100 kPa)	20%	921.5	825.4	908.4	885.1	5.9	1341.4	1188.9	1050.1	1193.5	12.2	1207.1	530.2	605.8	781.0	47.5	991.2	755.4	609.2	785.3	24.5
	40%	1107.9	943.2	1065.3	1038.8	8.2	1146.7	1268.4	1148.1	1187.7	5.9	1121.5	679.6	697.8	833.0	30.0	1110.4	949.3	718.6	926.1	21.3
	50%	1220.1	1024.8	1178.9	1141.2	9.0	1193.4	1342.6	1223.7	1253.2	6.3	1165.7	759.9	761.9	895.8	26.1	1175.2	971.5	787.2	977.9	19.8
	60%	1339.6	1109.2	1289.9	1246.3	9.7	1243.3	1421.0	1304.4	1322.9	6.8	1234.8	834.8	829.1	966.2	24.1	1259.4	1050.8	855.0	1055.1	19.2
	70%	1425.3	1170.7	1376.0	1324.0	10.2	1299.0	1481.7	1355.2	1378.6	6.8	1275.4	890.3	874.3	1013.4	22.4	1349.1	1129.7	917.5	1132.1	19.1
Phase 1 (50 kPa)	20%	850.3	780.9	884.2	838.5	6.3	1211.3	1179.7	1001.9	1131.0	10.0	1068.4	496.1	565.8	710.1	44.0	967.3	719.0	574.9	753.7	26.3
	40%	1054.9	903.5	1054.6	1004.4	8.7	1118.9	1238.3	1120.0	1159.1	5.9	1048.4	654.5	675.3	792.7	28.0	1014.7	835.8	674.8	841.8	20.2
	50%	1176.6	995.2	1123.9	1098.6	8.5	1158.2	1306.6	1198.9	1221.2	6.3	1114.0	730.5	752.1	865.5	24.9	1088.1	908.7	734.5	910.4	19.4
	60%	1292.0	1073.9	1213.2	1193.0	9.3	1186.8	1387.1	1280.0	1284.6	7.8	1159.3	806.2	823.6	929.7	21.4	1166.5	980.5	797.0	981.3	18.8
	70%	1404.1	1139.2	1321.4	1288.2	10.5	1246.9	1458.5	1351.6	1352.3	7.8	1223.5	868.2	881.8	991.2	20.3	1241.3	1052.0	859.3	1050.8	18.2
Phase 5 (20kPa)	20%	922.7	761.9	852.1	845.6	9.5	1188.4	1147.9	983.5	1106.6	9.8	985.9	482.5	585.7	684.7	38.8	949.5	693.5	562.6	735.2	26.8
	40%	1104.9	888.7	1002.2	998.6	10.8	1074.8	1208.1	1099.6	1127.5	6.3	1009.0	641.7	685.5	778.7	25.8	987.8	810.8	657.5	818.7	20.2
	50%	1214.5	979.7	1107.6	1100.6	10.7	1127.7	1290.5	1176.4	1198.2	7.0	1082.7	719.2	746.7	849.6	23.8	1071.4	881.6	711.0	888.0	20.3
	60%	1300.6	1041.7	1184.6	1175.6	11.0	1188.0	1360.8	1258.9	1269.2	6.8	1140.9	794.5	812.4	915.9	21.3	1204.1	1013.7	767.2	995.0	22.0
	70%	1407.7	1126.9	1275.0	1269.9	11.1	1263.4	1437.0	1334.9	1345.1	6.5	1202.0	859.5	865.8	975.7	20.1	1204.1	1013.7	824.7	1014.2	18.7

* Average Mr in kPa; ** Coficient of Variation in (%)

			FOAMED BITUMEN 2.4%-1% CEMENT																			
			High Density-High Saturation					High Density-Low Saturation					Low Density-High Saturation					Low Density-Low Saturation				
Phase	Conf (kPa)	SR	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**
Cond	200	20%	1139.5	1165.1	727.9	1010.8	24.3	1088.1	1377.1		1232.6	16.6	697.7	1299.3	1112.5	1036.5	29.7	1673.3	1649.6	1943.9	1755.6	9.3
Cond	200	50%	728.4	760.5	653.4	714.1	7.7	836.9	1122.0		979.5	20.6	444.9	512.7	782.8	580.1	30.8	1143.4	1314.4	1406.1	1288.0	10.4
Cond	200	70%	684.2	665.5	674.1	674.6	1.4	739.0	1006.0		872.5	21.6	372.9	414.6	624.3	470.6	28.6	1105.2	1191.3	1167.9	1154.8	3.9
1	200	20%	515.3	566.2	457.4	513.0	10.6	623.1	868.6		745.8	23.3	304.0	374.1	611.2	429.8	37.5	870.3	1164.3	1044.0	1026.2	14.4
1	200	40%	546.0	564.1	545.0	551.7	2.0	650.2	863.6		756.9	19.9	326.3	367.0	533.7	409.0	26.9	803.4	1031.1	964.3	932.9	12.5
1	200	50%	590.4	593.5	585.7	589.9	0.7	703.3	912.9		808.1	18.3	357.3	388.3	541.4	429.0	23.0	804.4	1046.3	1023.3	958.0	13.9
1	200	60%	637.1	627.4	644.9	636.5	1.4	750.9	955.8		853.4	17.0	376.2	401.1	552.7	443.3	21.5	826.9	1077.9	1061.8	988.8	14.2
1	200	70%	662.6	634.7	657.8	651.7	2.3	749.6	961.3		855.4	17.5	377.0	404.3	547.7	443.0	20.7	809.5	1078.1	1061.4	983.0	15.3
2	150	20%	439.4	488.1	447.3	458.3	5.7	599.8	776.1	Irealistic results	688.0	18.1	293.2	330.2	530.3	384.6	33.2	636.3	947.5	911.6	831.8	20.5
2	150	40%	496.9	514.5	514.5	508.6	2.0	638.6	804.2		721.4	16.2	318.1	342.8	478.5	379.8	22.7	690.6	929.8	909.6	843.3	15.7
2	150	50%	542.2	551.9	559.3	551.2	1.6	684.1	853.1		768.6	15.6	346.7	366.9	506.6	406.7	21.4	728.4	974.0	957.1	886.5	15.5
2	150	60%	590.9	588.5	609.6	596.3	1.9	734.1	908.0		821.0	15.0	370.9	389.1	536.2	432.0	21.0	775.4	1025.7	1015.1	938.7	15.1
2	150	70%	627.3	614.3	654.4	632.0	3.2	750.1	935.4		842.7	15.5	382.0	400.2	548.9	443.7	20.6	790.6	1059.3	1046.0	965.3	15.7
3	100	20%	418.4	453.7	446.5	439.5	4.2	618.5	739.8		679.2	12.6	303.2	347.0	534.1	394.8	31.1	616.0	866.3	881.0	787.8	18.9
3	100	40%	476.6	526.0	514.5	505.7	5.1	656.9	787.5		722.2	12.8	317.5	346.8	475.9	380.1	22.2	658.2	893.4	894.0	815.2	16.7
3	100	50%	521.2	526.0	559.3	535.5	3.9	697.7	809.3		753.5	10.5	340.2	367.3	506.3	404.6	22.0	709.5	942.2	944.7	865.5	15.6
3	100	60%	570.0	564.9	595.5	576.8	2.8	739.9	896.4		818.1	13.5	366.0	390.5	531.4	429.3	20.8	757.1	999.9	1003.4	920.1	15.3
3	100	70%	617.5	593.2	642.5	617.7	4.0	766.7	940.3		853.5	14.4	384.8	402.3	550.0	445.7	20.4	781.4	1047.2	1050.5	959.7	16.1
4	50	20%	408.8	434.8	458.8	434.2	5.8	620.9	735.1		678.0	11.9	294.4	349.4	530.4	391.4	31.5	616.1	873.2	853.7	781.0	18.3
4	50	40%	462.6	474.7	499.0	478.8	3.9	651.7	778.4		715.0	12.5	309.7	349.3	473.0	377.3	22.6	658.2	889.7	880.7	809.5	16.2
4	50	50%	499.0	508.7	538.8	515.5	4.0	693.0	834.3		763.6	13.1	334.0	371.6	500.0	401.8	21.7	709.5	931.9	932.3	857.9	15.0
4	50	60%	533.0	546.0	582.3	553.7	4.6	739.8	884.8		812.3	12.6	361.2	386.4	530.7	426.1	21.5	757.1	979.9	989.2	908.8	14.5
4	50	70%	572.5	578.4	628.2	593.0	5.2	772.3	932.4		852.3	13.3	378.9	402.0	557.7	446.2	21.8	781.7	1018.8	1040.2	946.9	15.2
5	20	20%	369.3	434.7	475.0	426.3	12.5	622.8	715.7		669.3	9.8	279.7	344.1	543.1	389.0	35.3	609.5	846.9	881.9	779.4	19.0
5	20	40%	429.0	464.7	502.3	465.4	7.9	654.3	761.6		707.9	10.7	302.1	341.5	475.6	373.1	24.4	604.4	873.9	925.1	801.1	21.5
5	20	50%	469.5	497.1	535.7	500.8	6.6	691.9	814.5		753.2	11.5	327.8	363.2	502.7	397.9	23.2	647.2	925.2	946.9	839.8	19.9
5	20	60%	509.8	531.3	570.1	537.1	5.7	736.5	868.6		802.6	11.6	354.6	387.3	525.4	422.4	21.5	699.1	981.7	992.5	891.1	18.7
5	20	70%	553.4	562.6	615.1	577.0	5.8	767.7	921.3		844.5	12.9	374.4	404.1	555.0	444.5	21.8	735.0	1027.1	1030.4	930.9	18.2

* Average Mr in kPa; ** Coficient of Variation in (%)

|			High Density-High Saturation					High Density-Low Saturation					Low Density-High Saturation					Low Density-Low Saturation				
FOAMED BITUMEN 2.4%-2% CEMENT																						
Phase	Conf (kPa)	SR	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**	Sp 1	Sp 2	Sp 3	Aver*	COV**
Cond	200	20%	1195.8	1661.8	2783.5	1880.4	43.4	2361.0	1881.5	3450.4	2564.3	31.4	1471.9	1027.4	2193.4	1564.3	37.6	1604.1	1881.6	2836.1	2107.3	30.7
		50%	1053.0	1198.4	1818.3	1356.6	30.0	1766.1	1378.9	2209.1	1784.7	23.3	872.4	1152.2	1508.4	1177.7	27.1	2347.2	1448.2	1831.2	1875.6	24.1
		70%	949.9	963.7	1197.2	1036.9	13.4	1480.1	1195.2	1455.7	1377.0	11.5	710.9	697.8	1302.6	903.8	38.2	2741.8	1157.4	1575.3	1824.9	45.0
1	200	20%	918.4	639.6	1016.0	858.0	22.8	1168.5	981.8	1437.0	1195.8	19.1	491.5	428.0	946.1	621.9	45.4	2121.1	803.0	1307.7	1410.6	47.1
		40%	873.7	730.9	999.5	868.0	15.5	1218.5	996.2	1379.2	1198.0	16.1	577.9	547.4	1075.9	733.7	40.4	1928.9	917.9	1280.5	1375.8	37.2
		50%	887.9	798.6	1065.1	917.2	14.8	1282.0	1051.9	1368.6	1234.2	13.3	630.7	609.2	1143.4	794.4	38.1	2169.7	990.1	1348.2	1502.7	40.2
		60%	923.2	856.3	1120.8	966.8	14.2	1356.1	1101.5	1360.2	1272.6	11.6	675.7	663.7	1202.9	847.4	36.3	2406.7	1045.7	1407.3	1619.9	43.5
		70%	940.8	863.8	1113.4	972.7	13.1	1366.3	1084.7	1257.1	1236.0	11.5	683.7	682.8	1206.5	857.7	35.2	2413.7	1041.9	1376.7	1610.8	44.4
2	150	20%	600.9	511.3	837.0	649.7	25.9	1002.1	850.1	976.1	942.8	8.6	472.7	412.7	804.4	563.3	37.5	1835.6	721.6	917.2	1158.1	51.4
		40%	714.9	641.7	888.5	748.3	16.9	1102.2	910.2	1034.0	1015.5	9.6	531.2	508.7	941.6	660.5	36.9	1720.4	839.6	1037.7	1199.2	38.5
		50%	782.8	715.6	954.1	817.5	15.0	1184.7	978.2	1108.9	1090.6	9.6	584.6	576.1	1024.9	728.5	35.2	1923.5	914.9	1126.9	1321.8	40.2
		60%	860.0	785.1	1021.2	888.8	13.6	1264.0	1052.5	1194.4	1170.3	9.2	637.8	629.5	1094.7	787.3	33.8	2161.6	989.2	1214.1	1455.0	42.8
		70%	902.2	828.3	1069.1	933.2	13.2	1299.0	1078.7	1230.5	1202.7	9.4	668.2	664.5	1142.9	825.2	33.3	2422.1	1033.2	1252.0	1569.1	47.6
3	100	20%	567.3	473.7	767.8	602.9	24.9	925.6	856.5	957.6	913.2	5.7	444.2	391.6	748.4	528.1	36.5	1650.2	683.5	876.6	1070.1	47.8
		40%	679.1	607.9	827.9	705.0	15.9	1050.7	918.8	1013.5	994.3	6.8	512.3	490.6	886.1	629.7	35.3	1643.3	811.4	1000.2	1151.6	37.9
		50%	748.6	679.7	906.1	778.1	14.9	1128.1	984.4	1090.5	1067.7	7.0	568.1	548.7	964.1	693.6	33.8	1866.1	884.3	1075.9	1275.4	40.8
		60%	814.7	752.8	967.6	845.0	13.1	1209.9	1050.6	1174.2	1144.9	7.3	627.0	604.3	1040.7	757.3	32.4	2067.3	960.0	1162.6	1396.6	42.2
		70%	877.2	807.2	1036.8	907.1	13.0	1275.8	1088.1	1219.5	1194.5	8.1	667.6	648.1	1103.1	806.3	31.9	2220.1	1023.0	1227.5	1490.2	43.0
4	50	20%	530.2	445.2	724.4	566.6	25.3	899.2	858.6	961.6	906.4	5.7	441.7	366.7	697.7	502.0	34.6	1592.6	665.0	847.1	1034.9	47.5
		40%	646.3	583.5	789.7	673.2	15.7	1021.3	930.3	1023.8	991.8	5.4	511.6	464.4	835.0	603.7	33.4	1462.4	786.9	956.4	1068.6	32.9
		50%	713.2	654.9	868.2	745.4	14.8	1103.1	981.4	1091.7	1058.7	6.3	563.4	523.6	910.1	665.7	31.9	1528.9	859.2	1040.7	1142.9	30.3
		60%	782.3	724.5	933.6	813.5	13.3	1183.2	1034.1	1151.0	1122.8	7.0	606.6	575.2	983.7	721.8	31.5	1594.3	926.1	1112.2	1210.8	28.5
		70%	853.0	779.1	999.0	877.0	12.8	1252.2	1081.0	1205.5	1179.6	7.5	653.0	625.6	1055.0	777.9	30.9	1683.6	998.1	1185.5	1289.1	27.5
5	20	20%	512.7	428.7	690.0	543.8	24.5	879.4	842.5	919.9	880.6	4.4	427.0	353.8	665.9	482.2	33.8	1607.6	662.5	835.1	1035.1	48.6
		40%	630.1	567.5	761.9	653.2	15.2	1003.0	924.4	973.4	966.9	4.1	505.6	456.3	831.6	597.8	34.1	1457.7	779.7	942.2	1059.9	33.4
		50%	695.8	635.4	837.7	722.9	14.4	1085.0	983.1	1046.5	1038.2	5.0	558.1	513.0	904.7	658.6	32.5	1514.0	849.1	1013.9	1125.7	30.8
		60%	764.4	705.2	914.5	794.7	13.6	1162.9	1024.7	1122.1	1103.2	6.4	608.2	572.4	975.8	718.8	31.1	1580.4	923.7	1095.7	1199.9	28.4
		70%	830.8	765.7	983.7	860.1	13.0	1236.0	1068.4	1178.5	1161.0	7.3	652.9	617.5	1045.6	772.0	30.8	1660.1	993.0	1168.4	1273.8	27.1

# Appendix H: Mr-θ modelling graphs of the resilient modulus
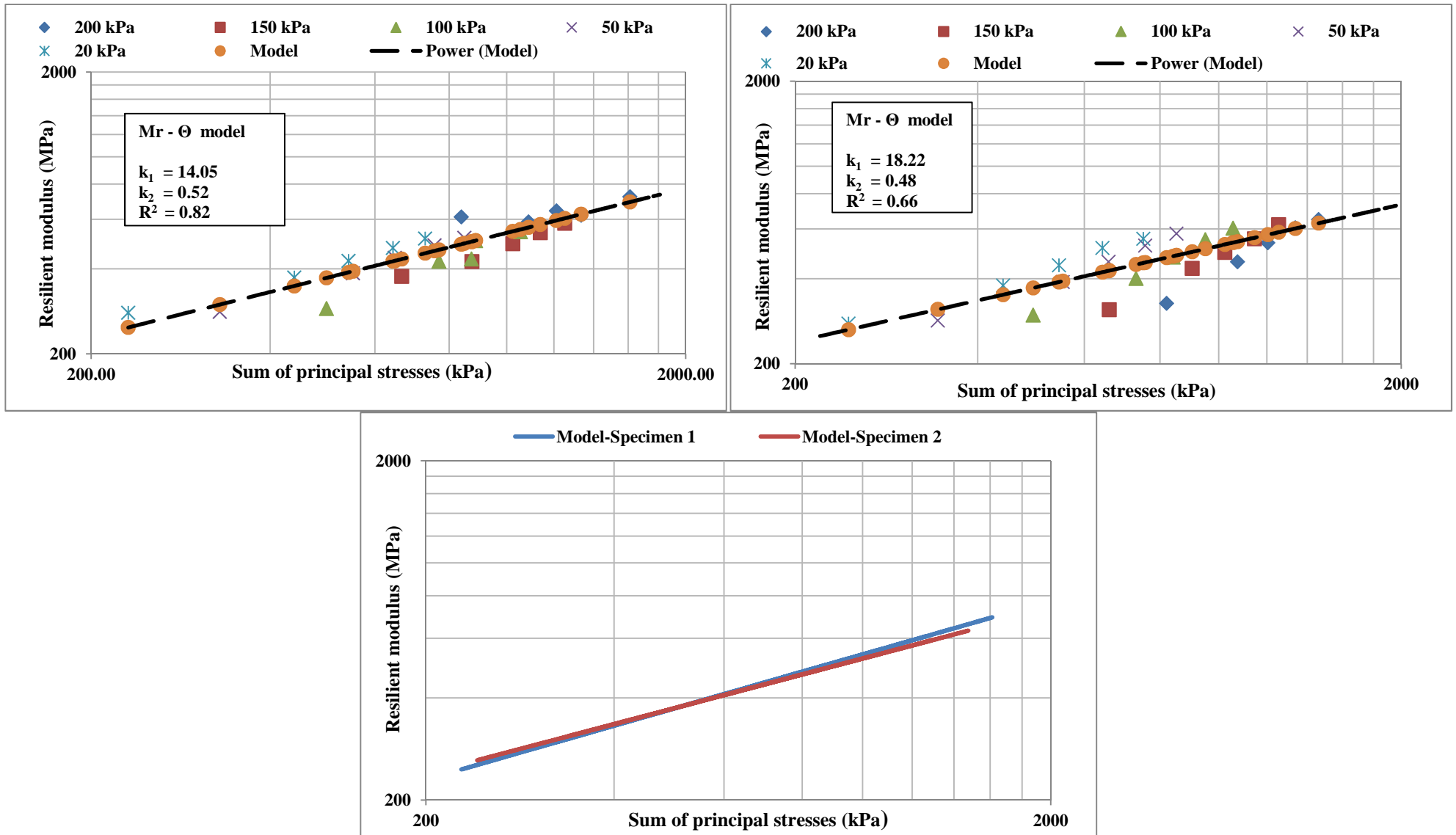
Figure H-1: M_r-θ modelling – Mix EB0.9-CM1-HD-HS
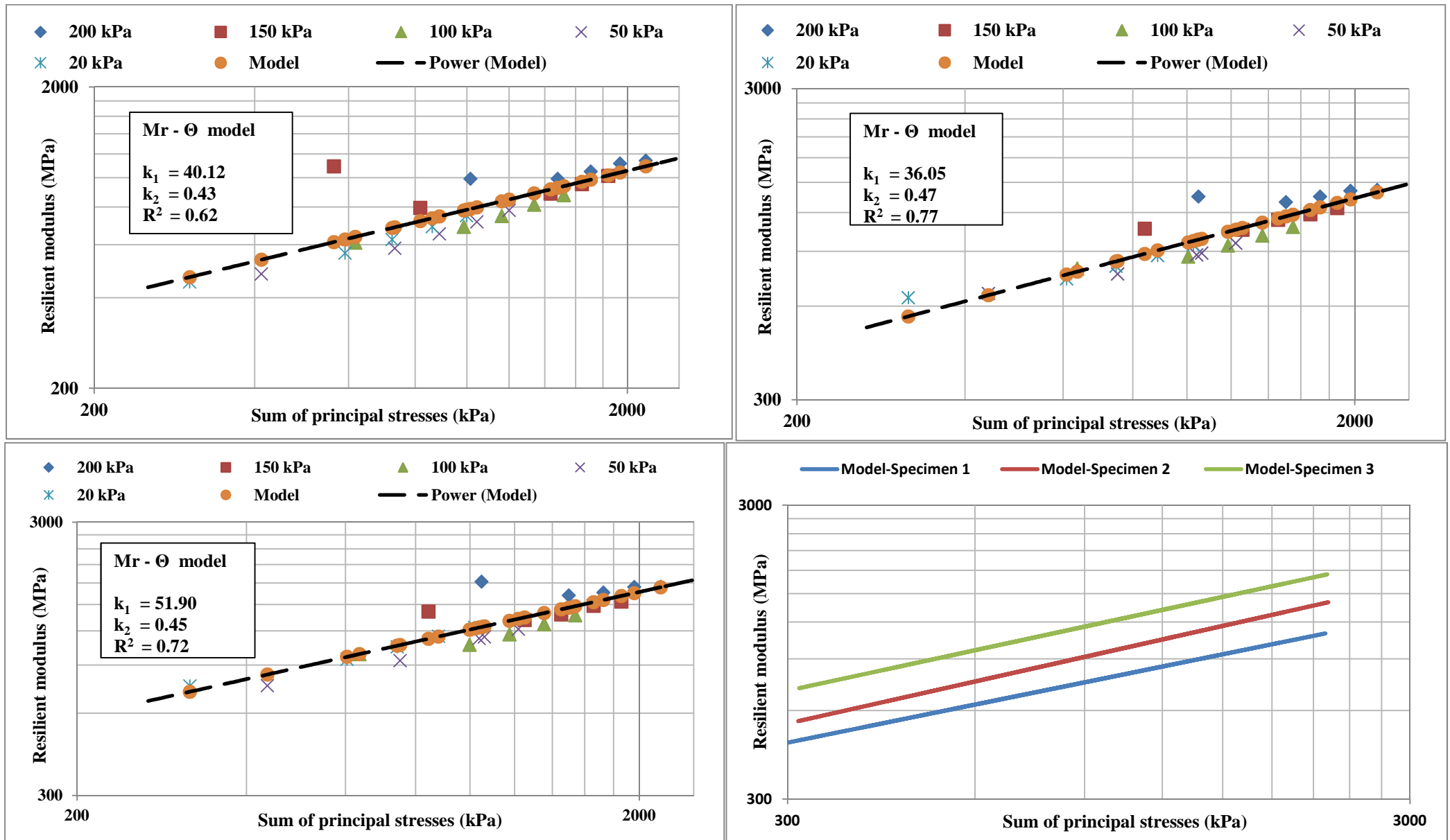
Figure H-2: M_r-θ modelling – Mix EB0.9-CM1-HD-LS
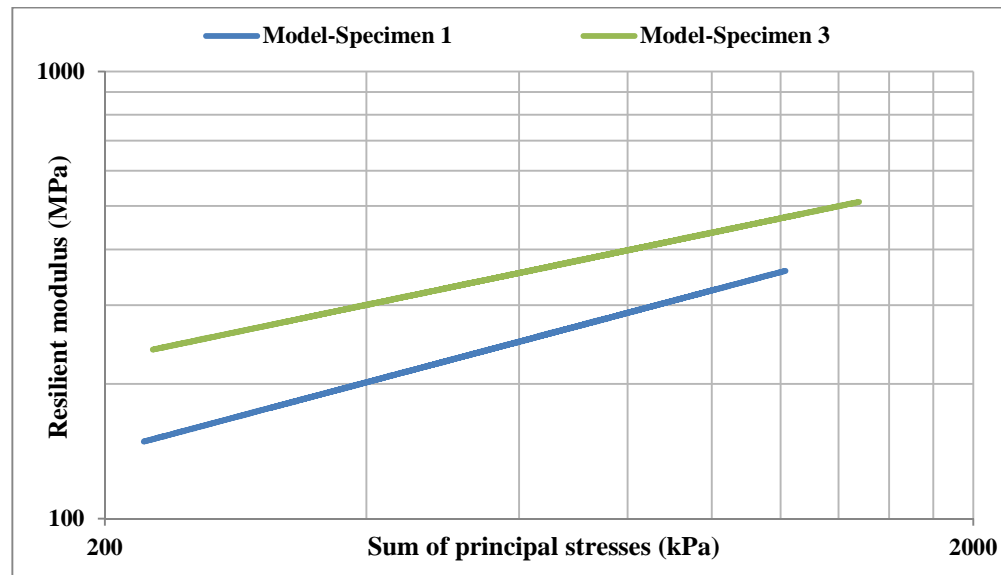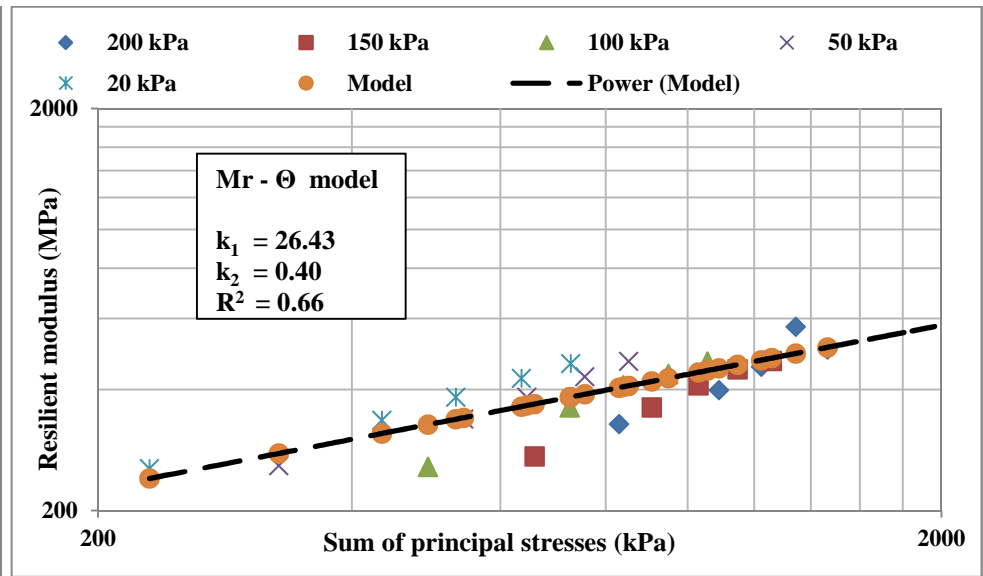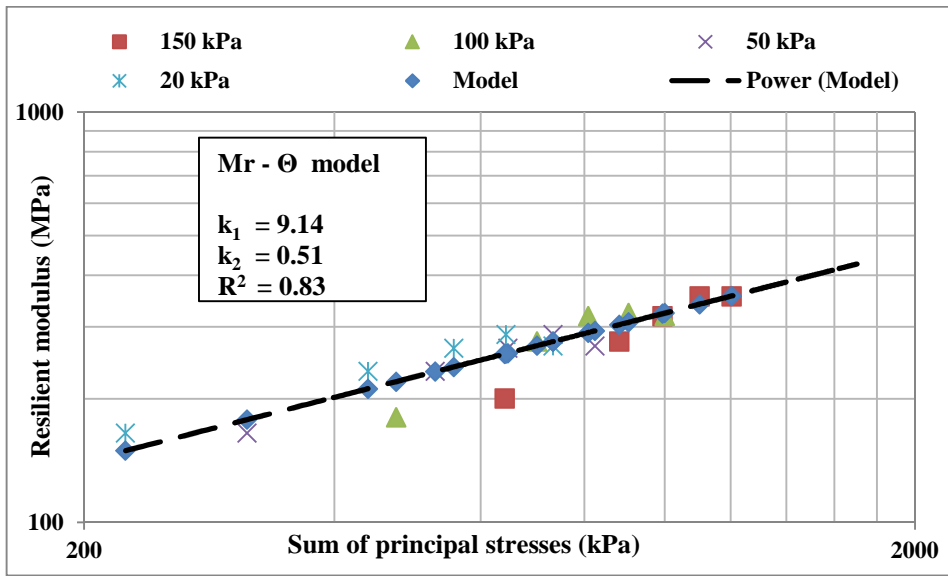
Figure H-3: M_r-θ modelling – Mix EB0.9-CM1-LD-HS
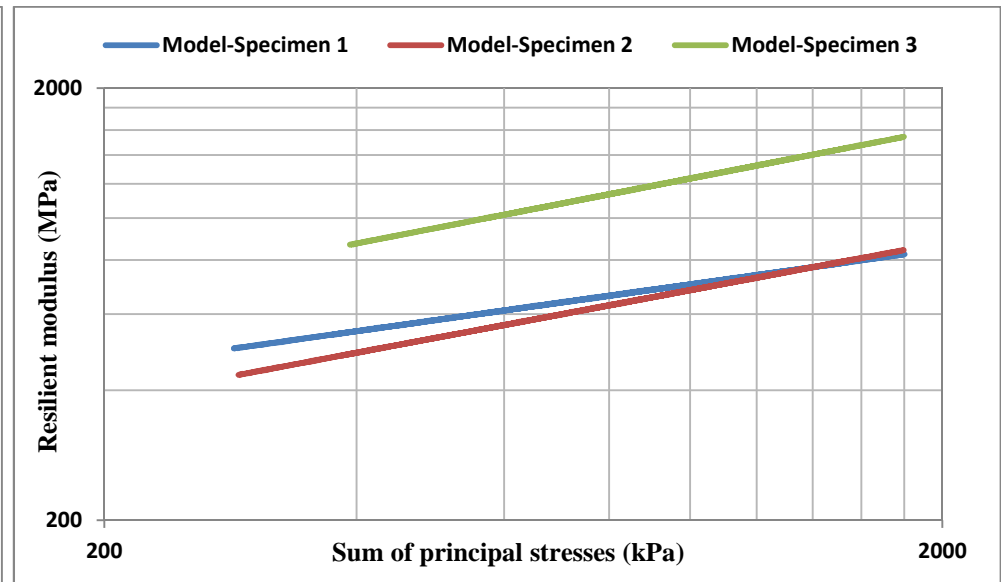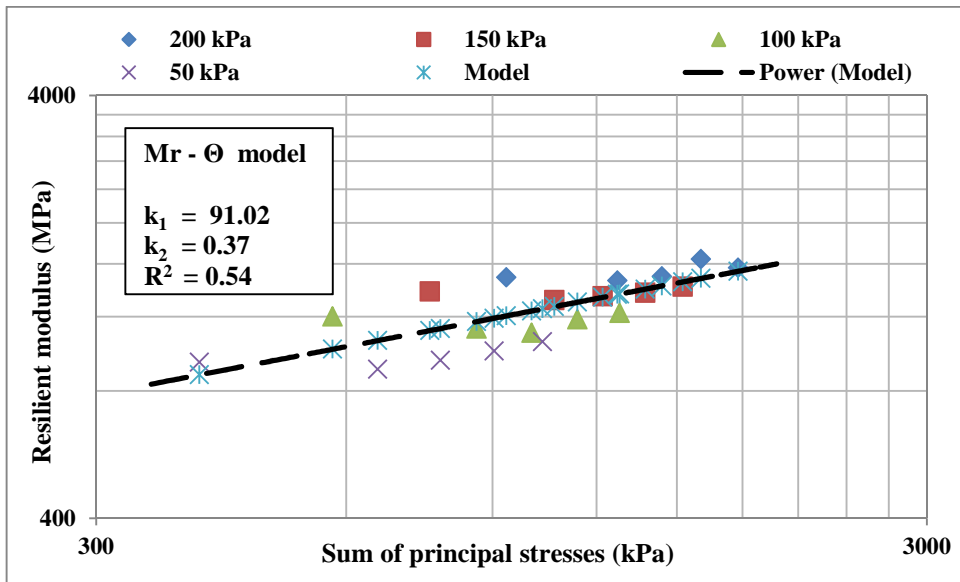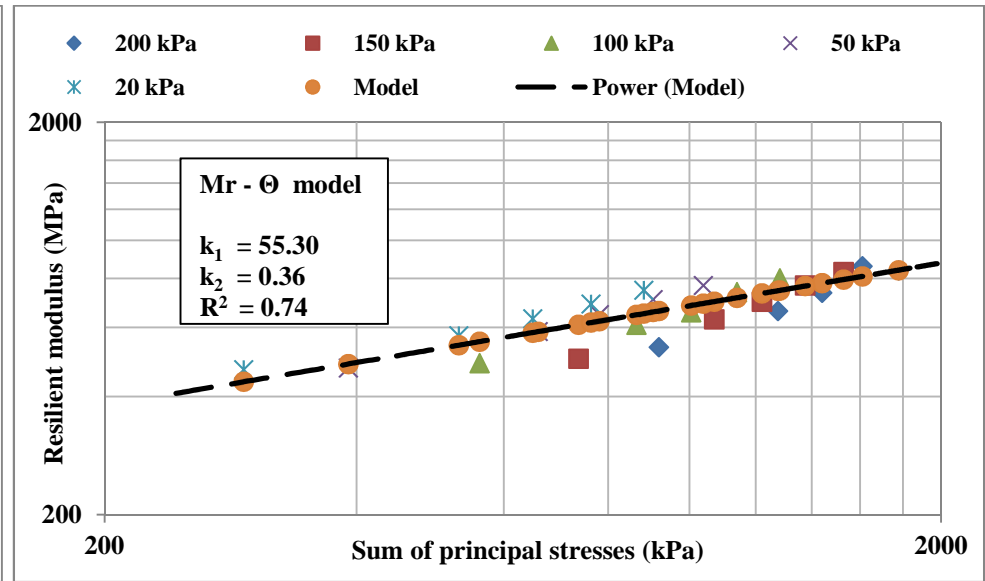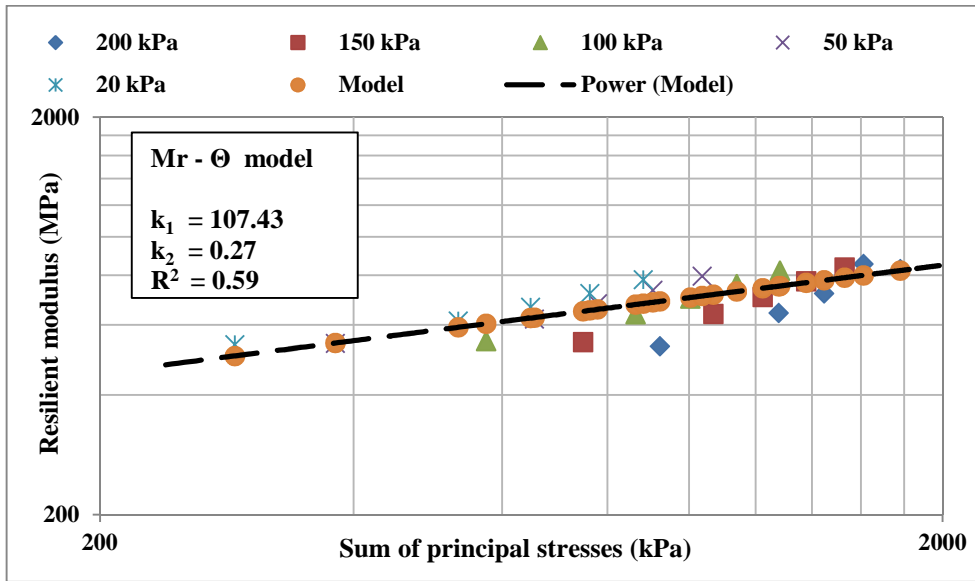
Figure H-4: M_r-θ modelling – Mix EB0.9-CM1-LD-LS
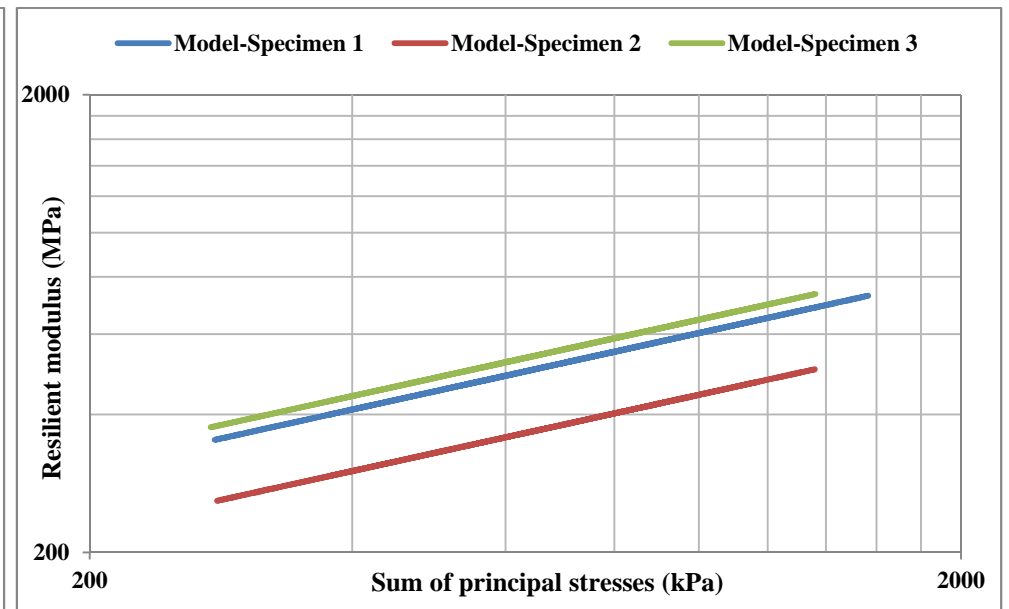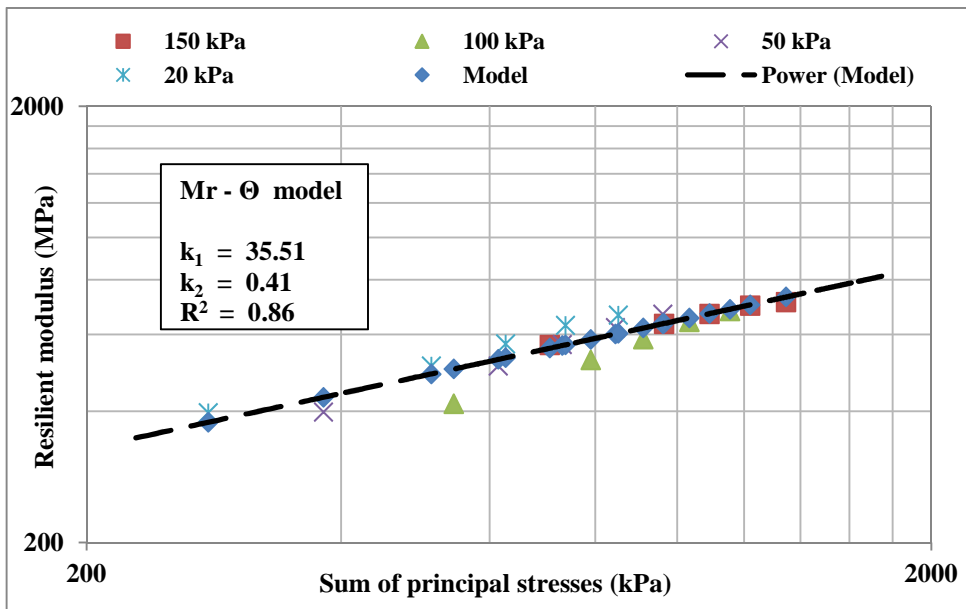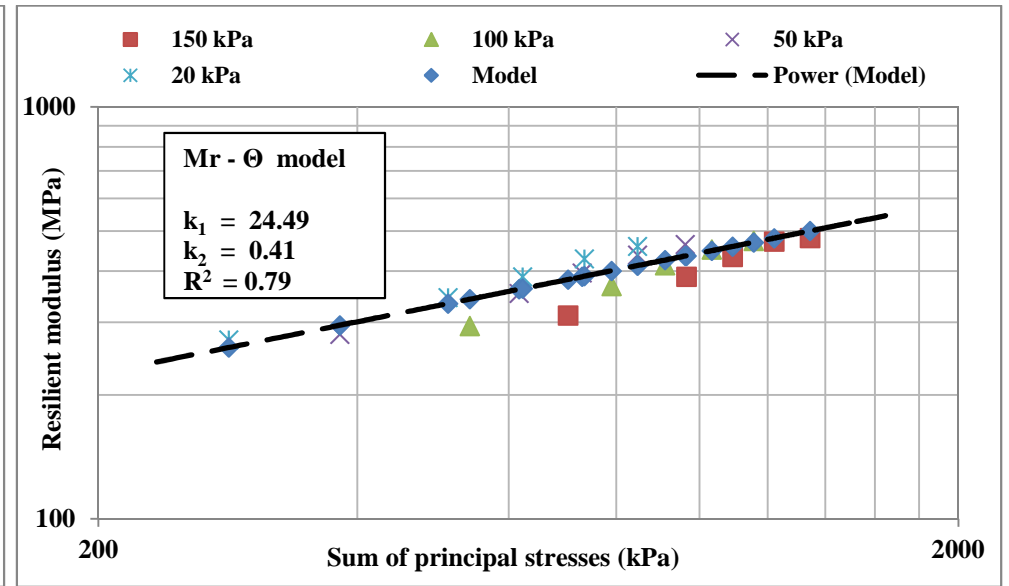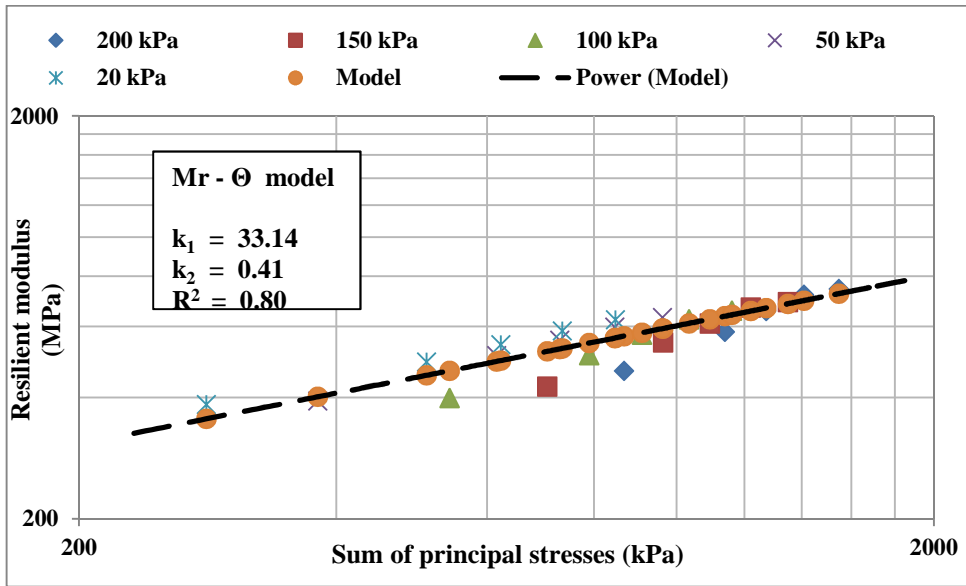
Figure H-5: M_r-θ modelling – Mix EB2.4-CM1-HD-HS
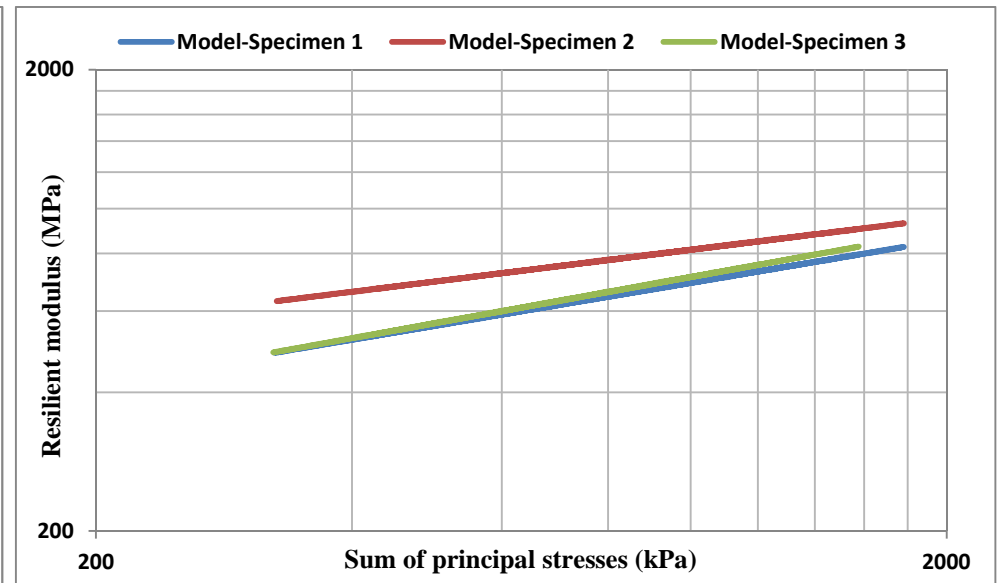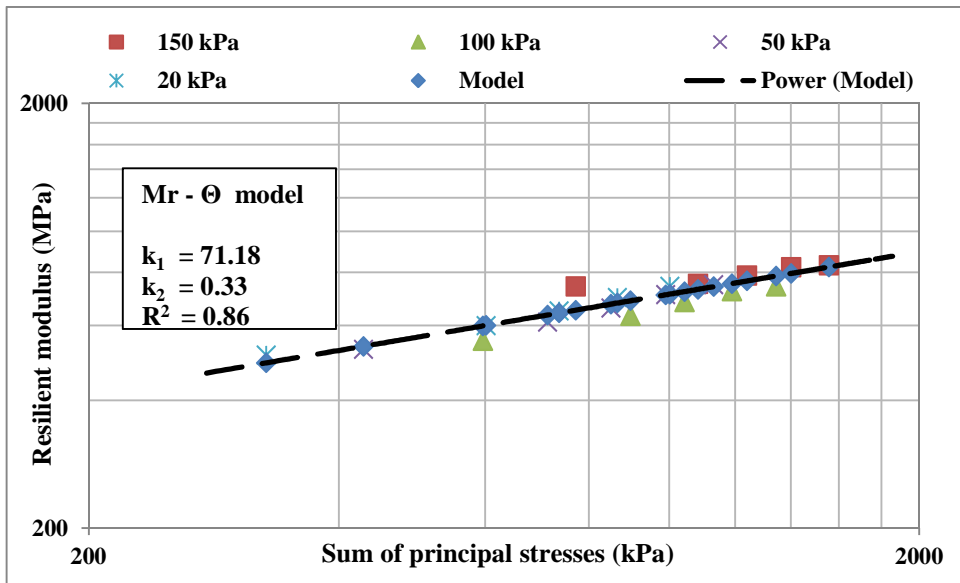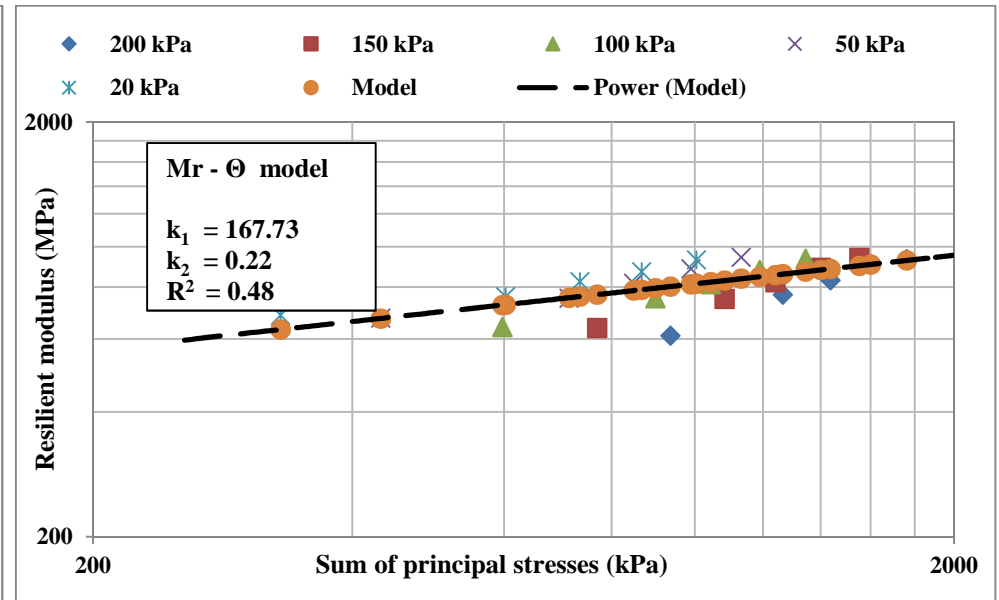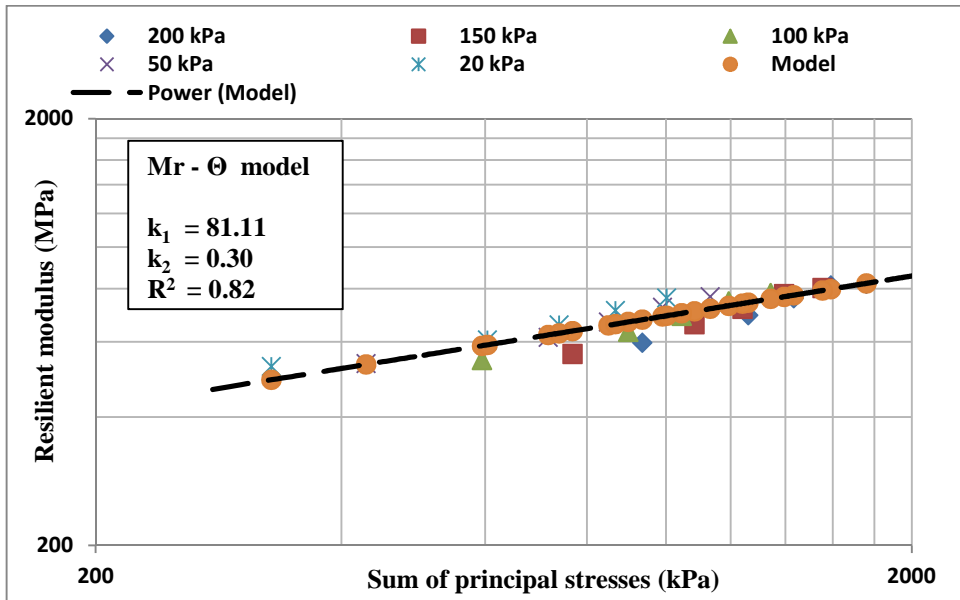
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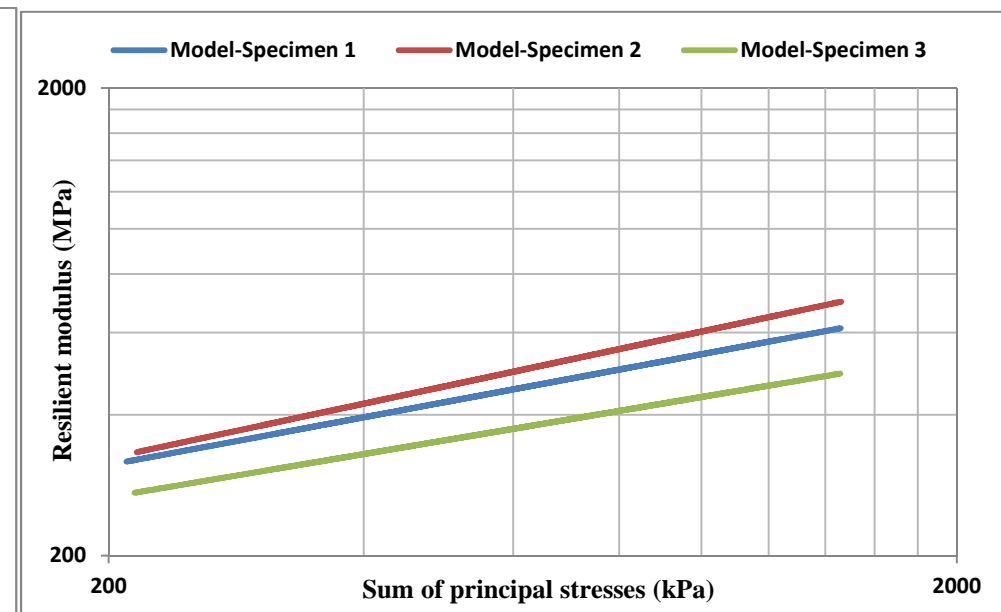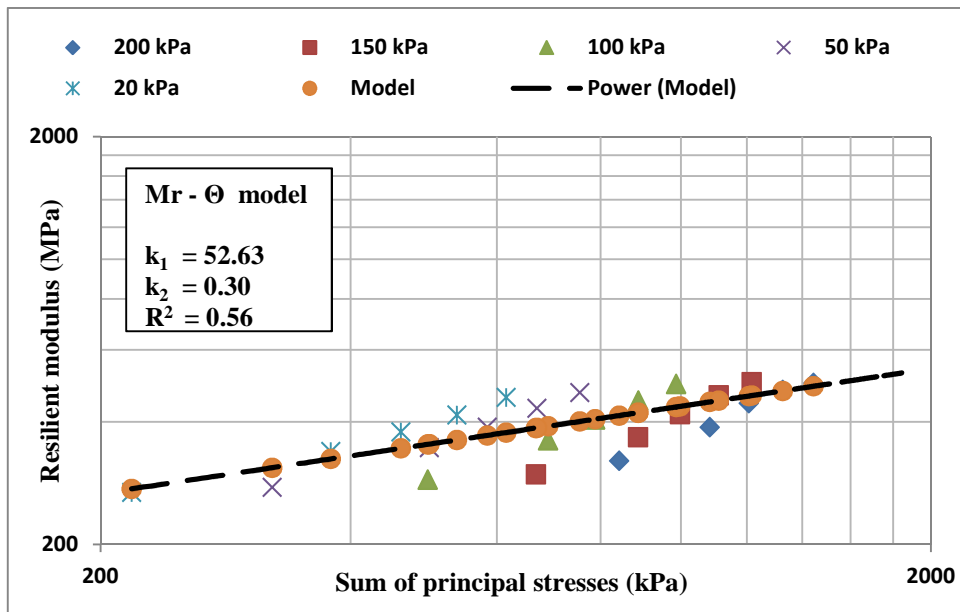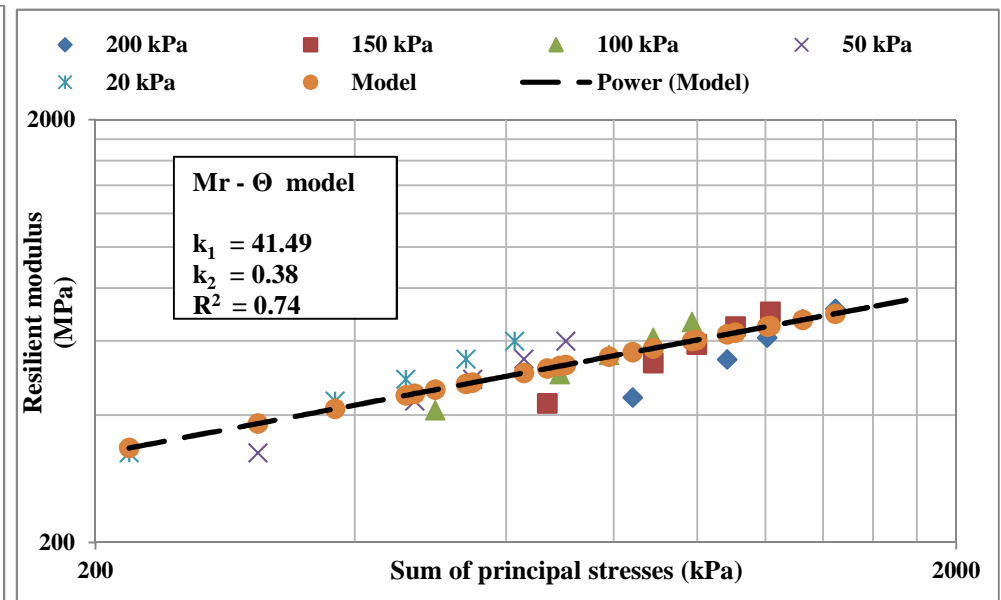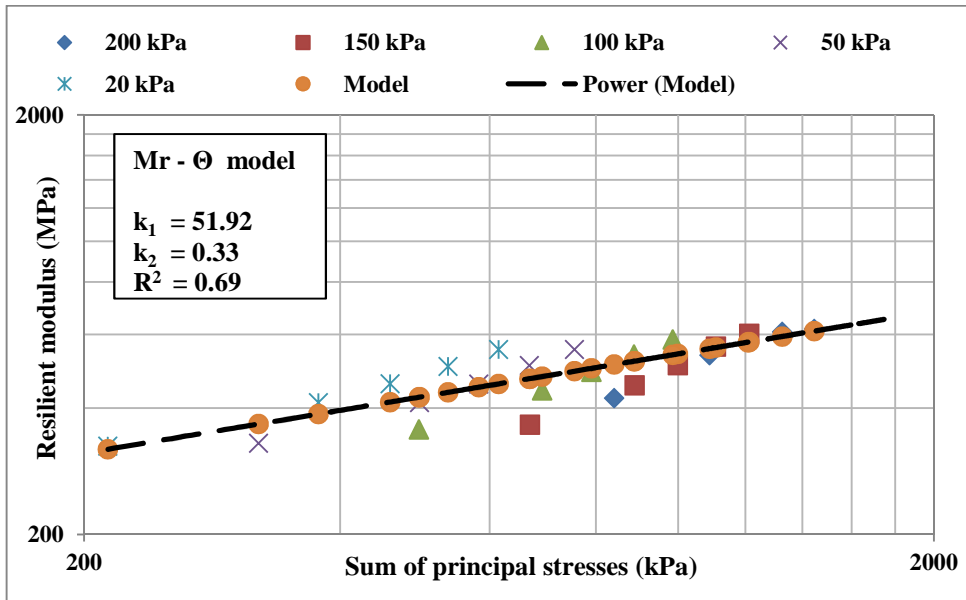
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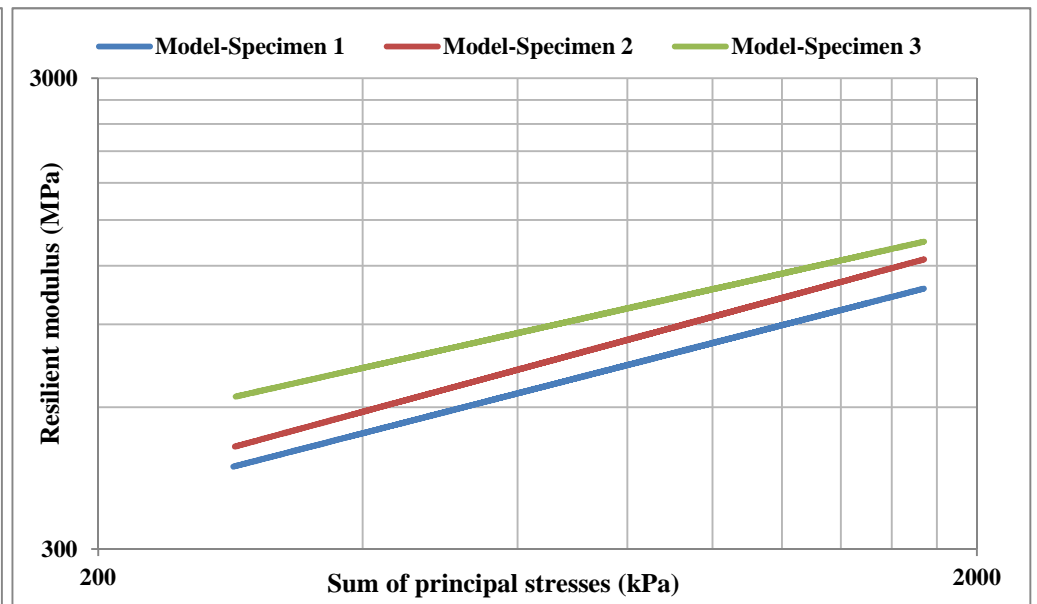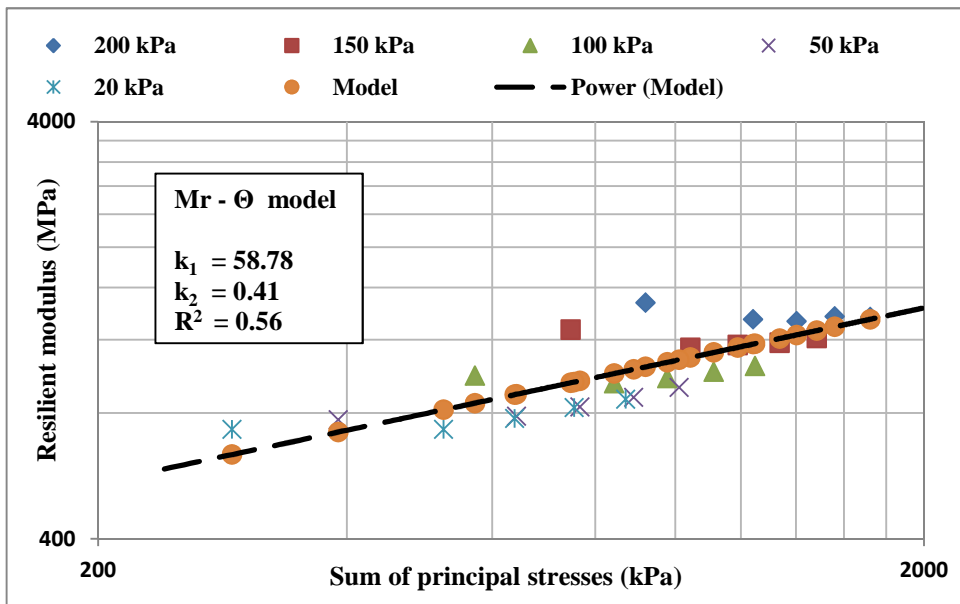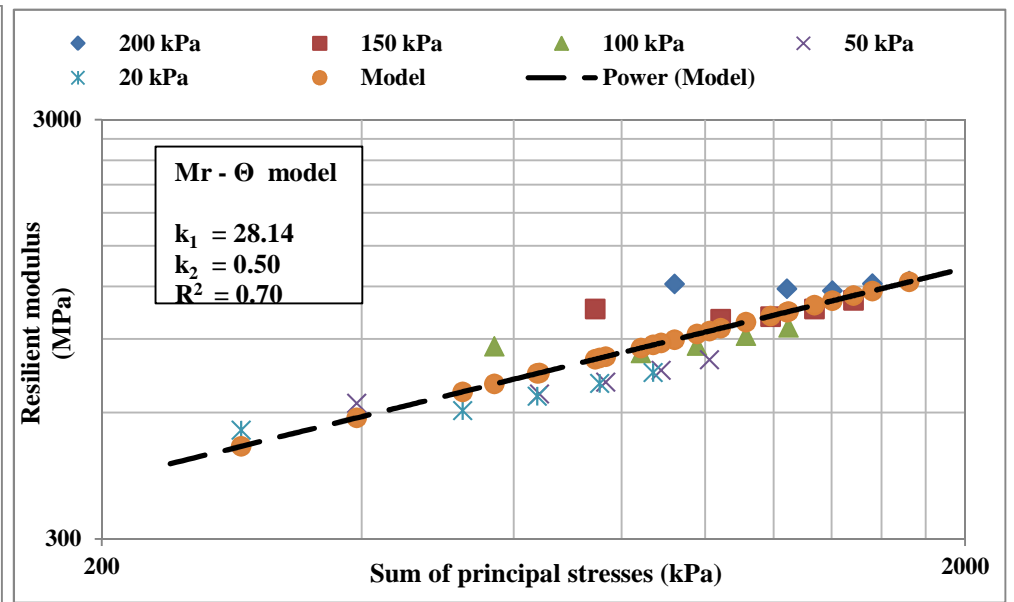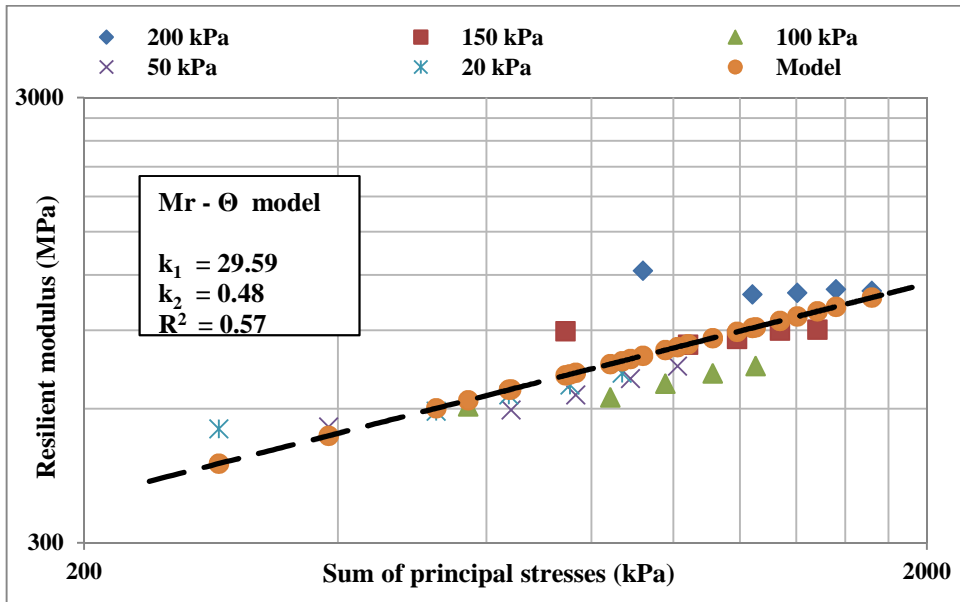
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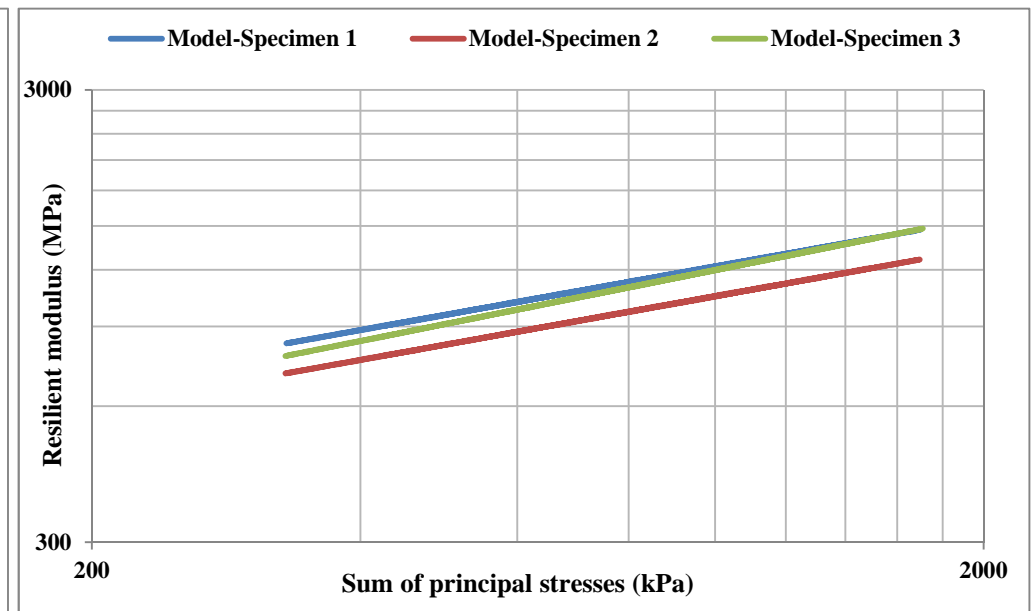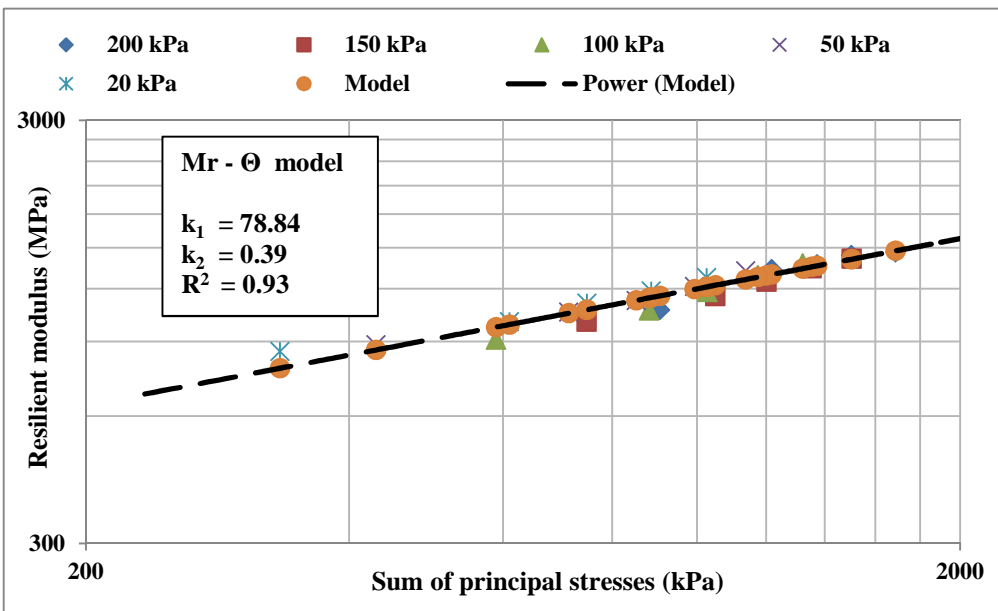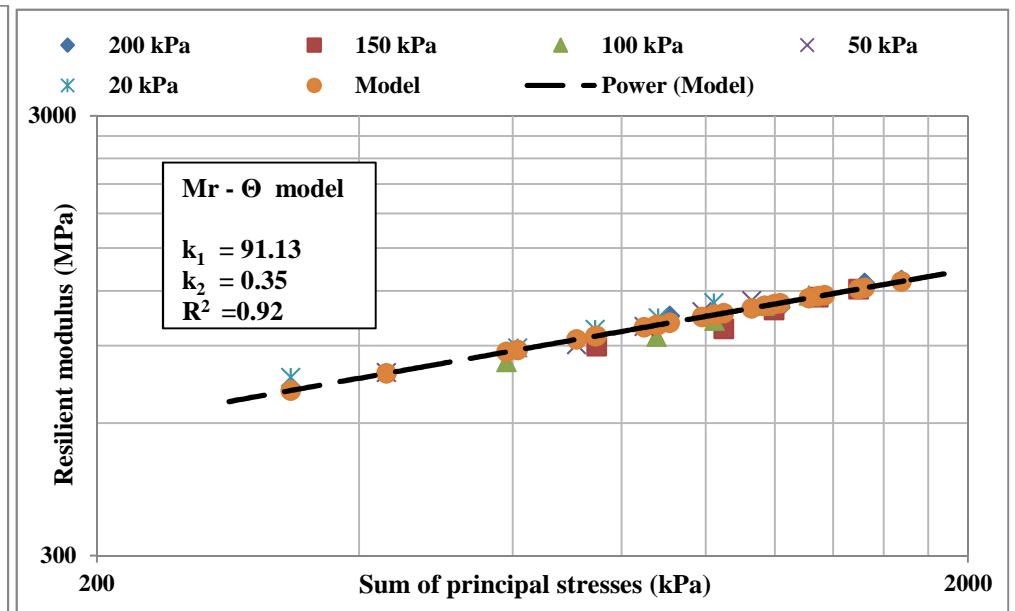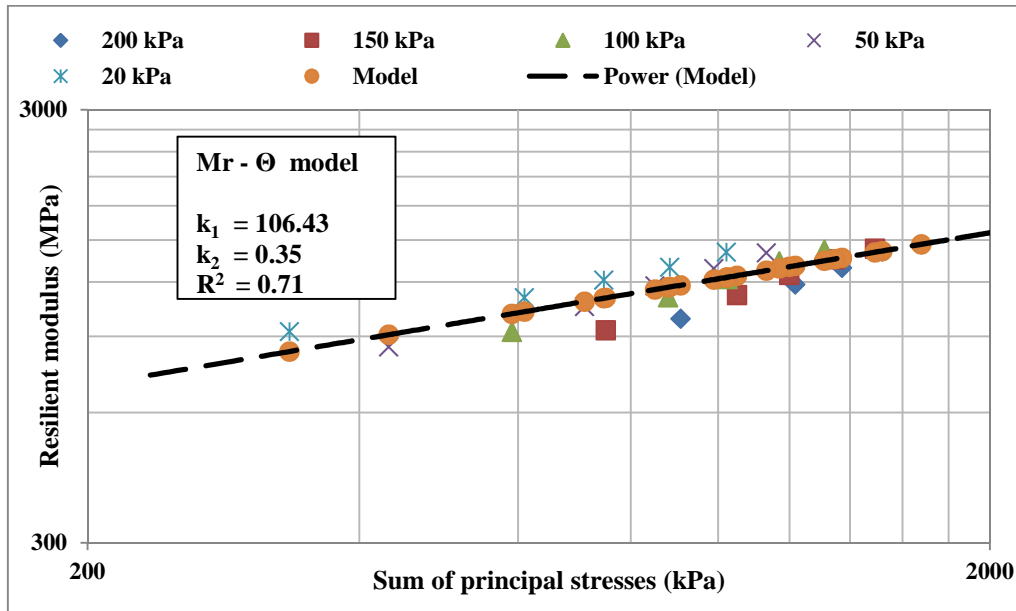
Figure H-9: M$_r$-θ modelling – Mix EB2.4-CM2-HD-HS
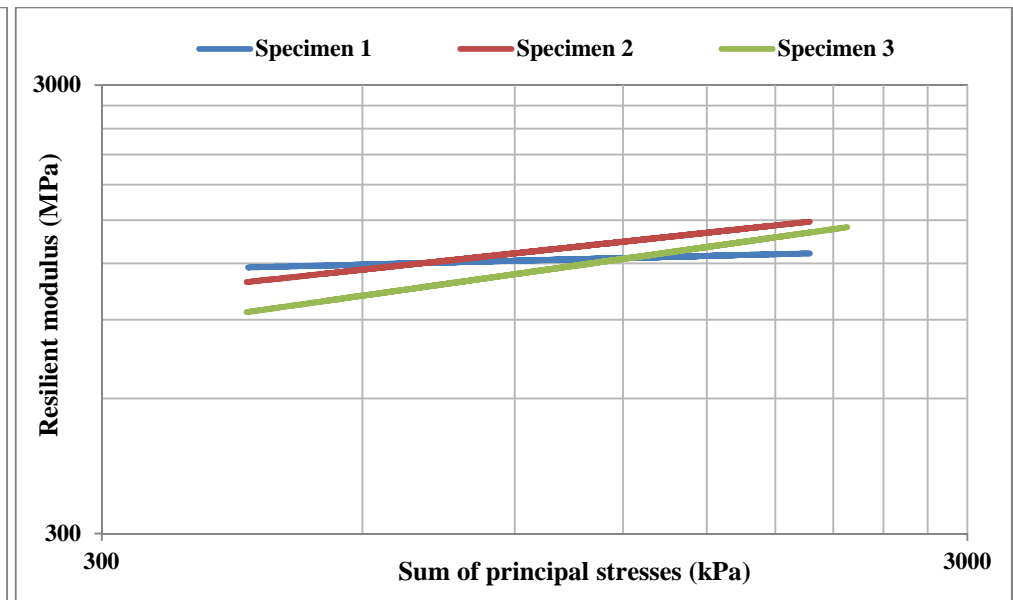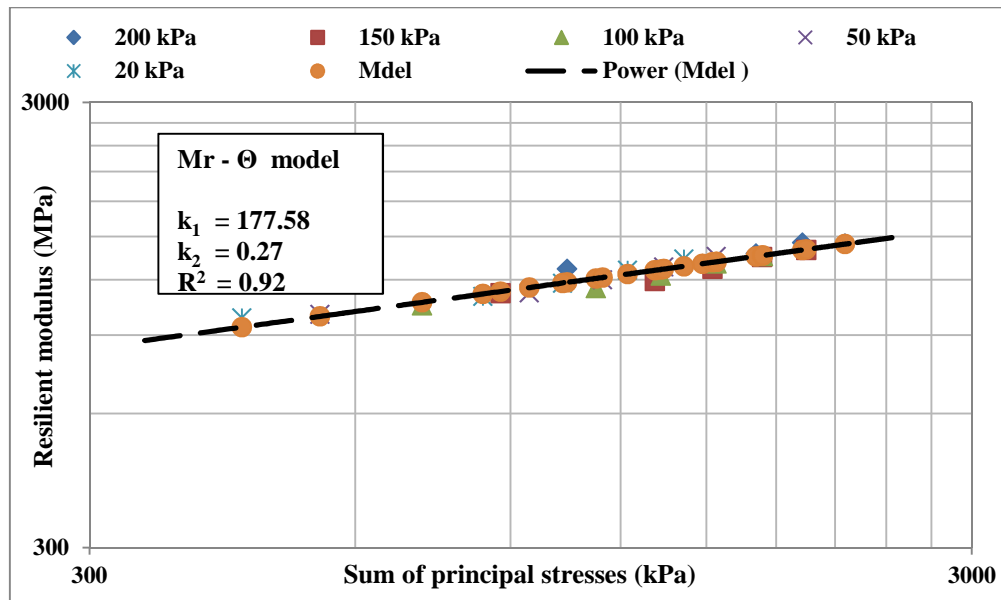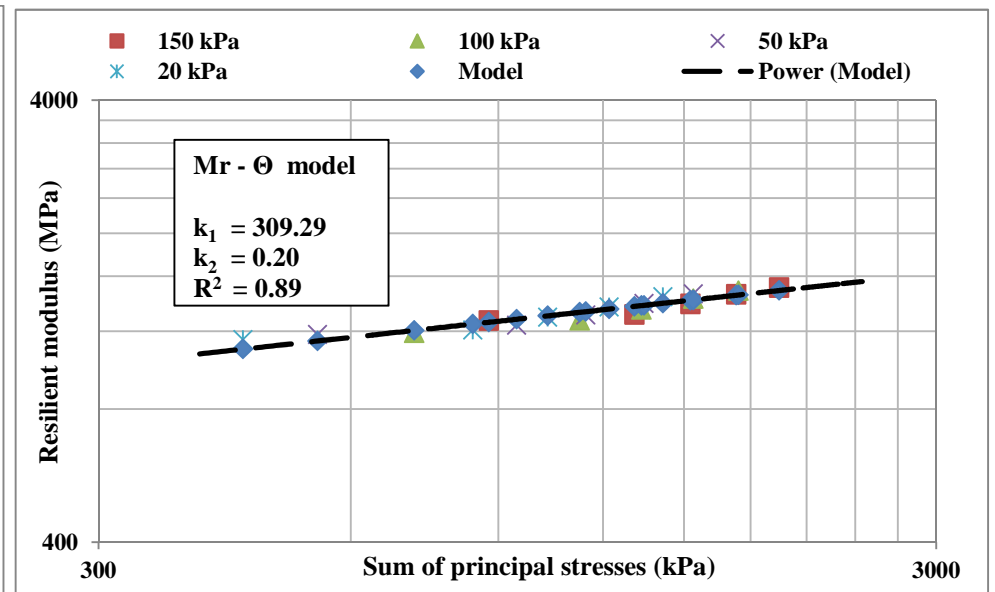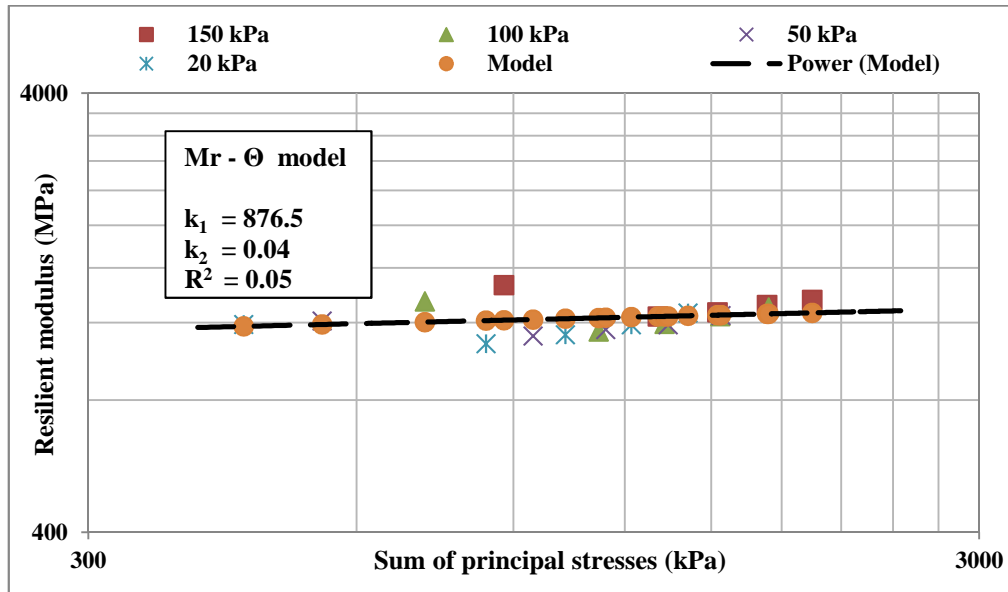
Figure H-10: M_r-θ modelling – Mix EB2.4-CM2-HD-LS
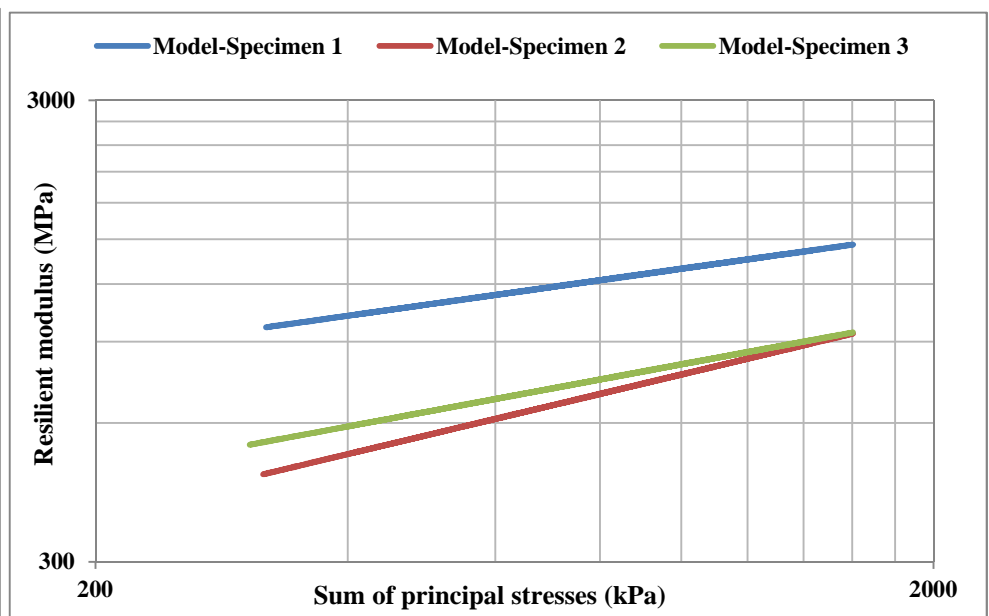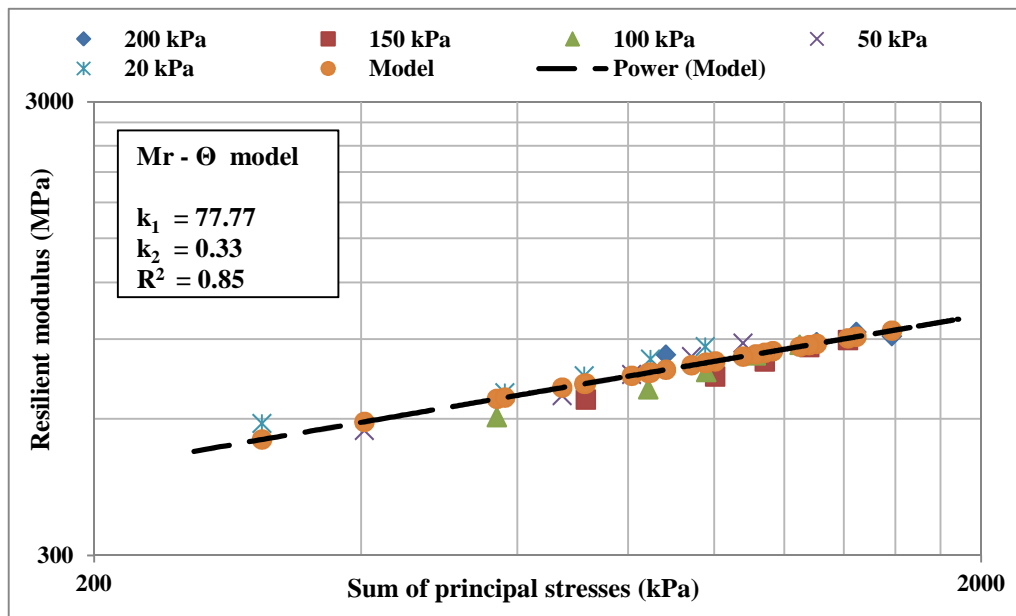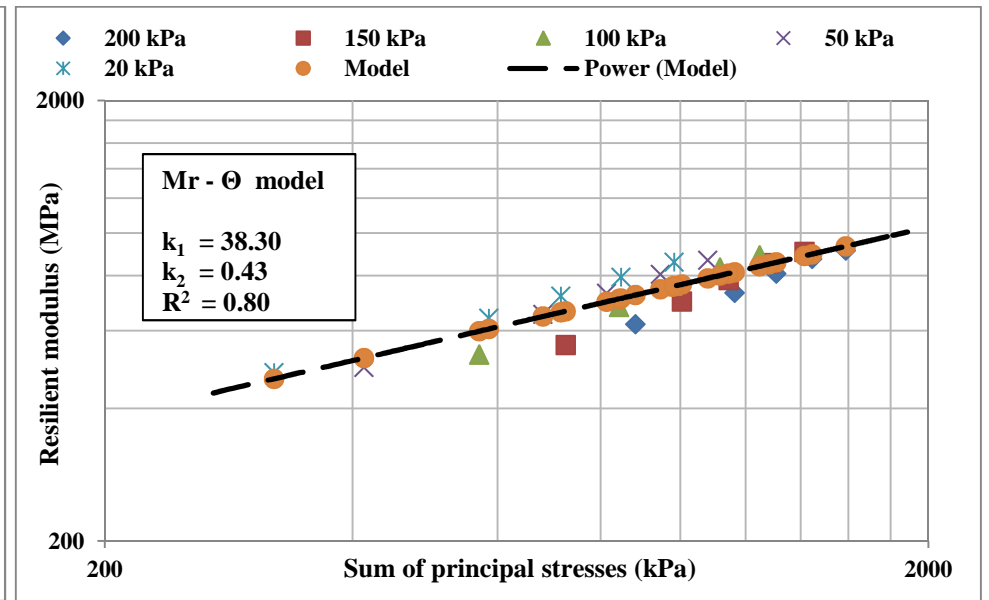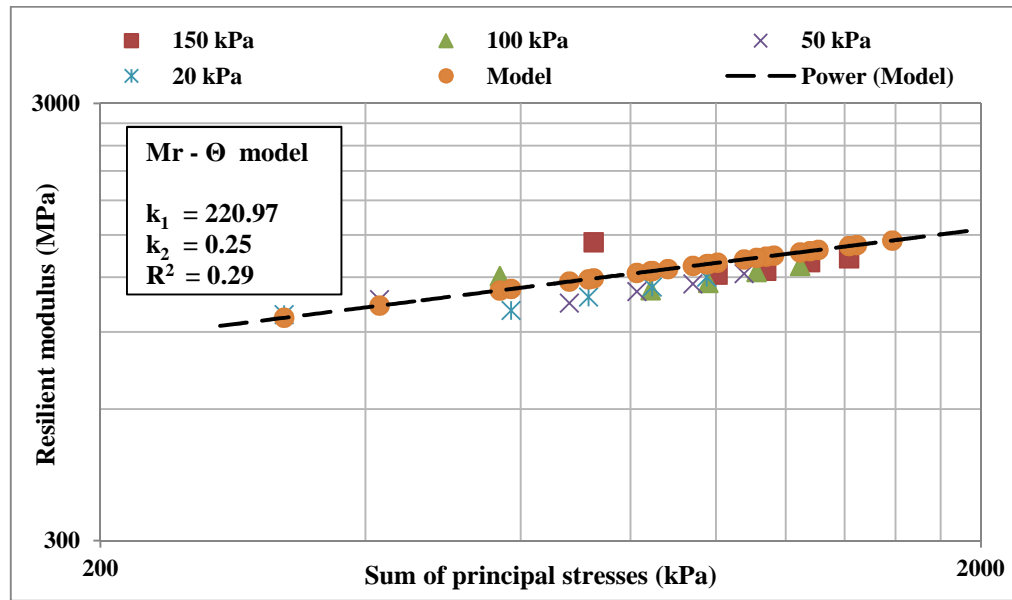
Figure H-11: Mr-θ modelling – Mix EB2.4-CM2-LD-HS
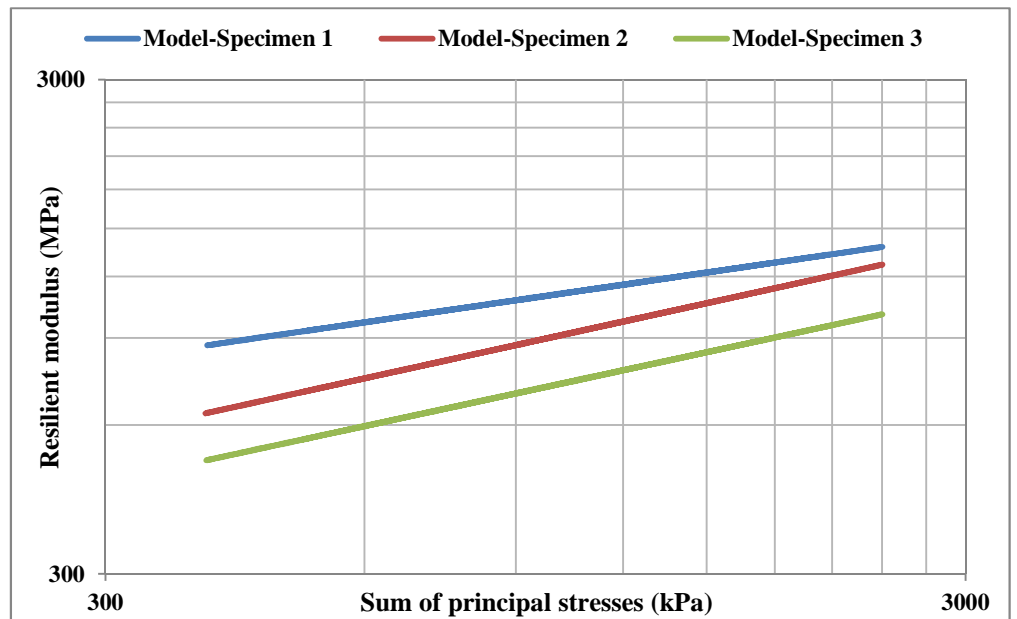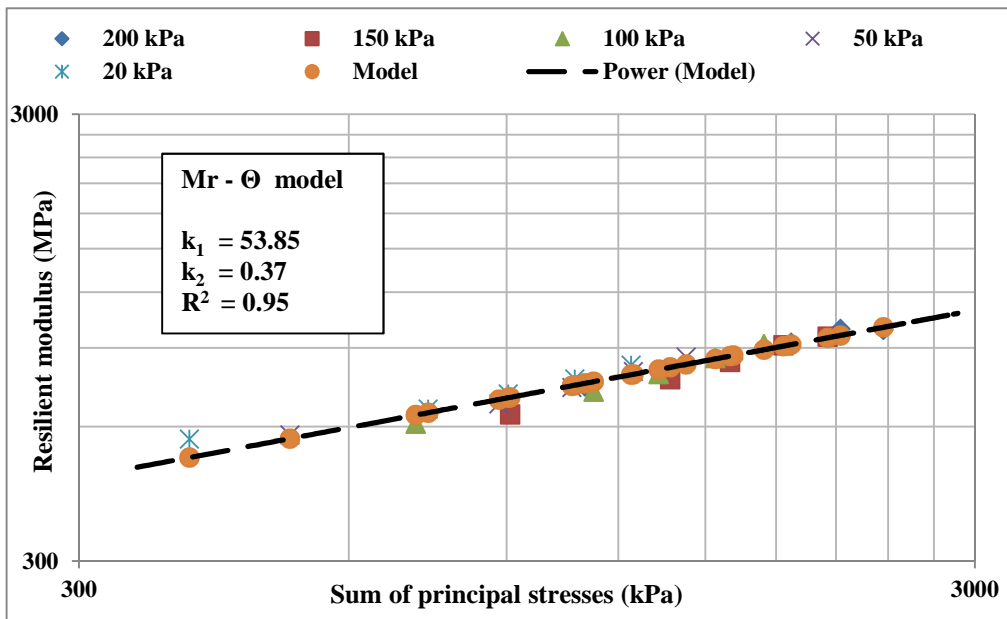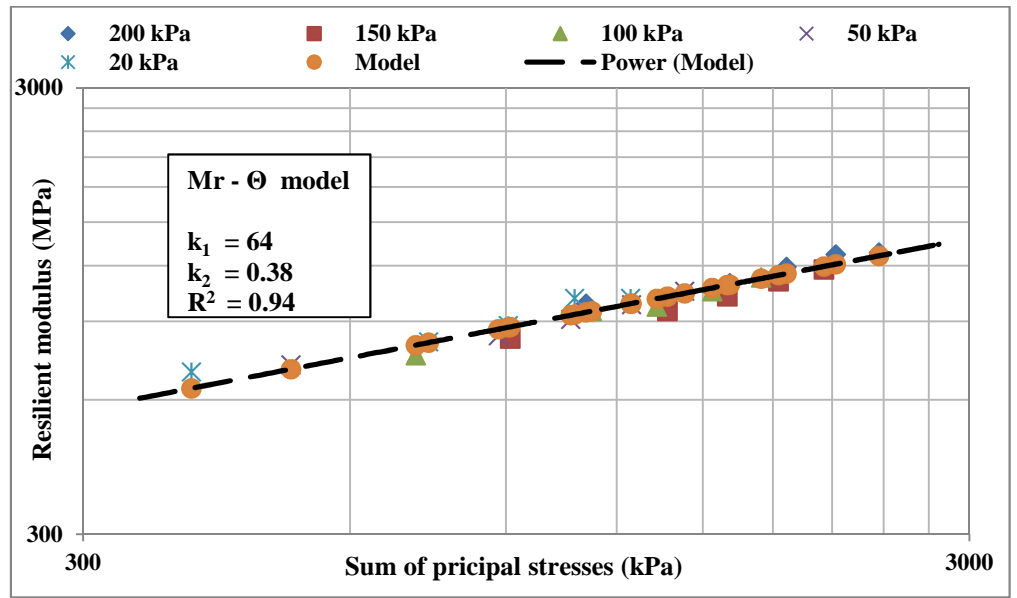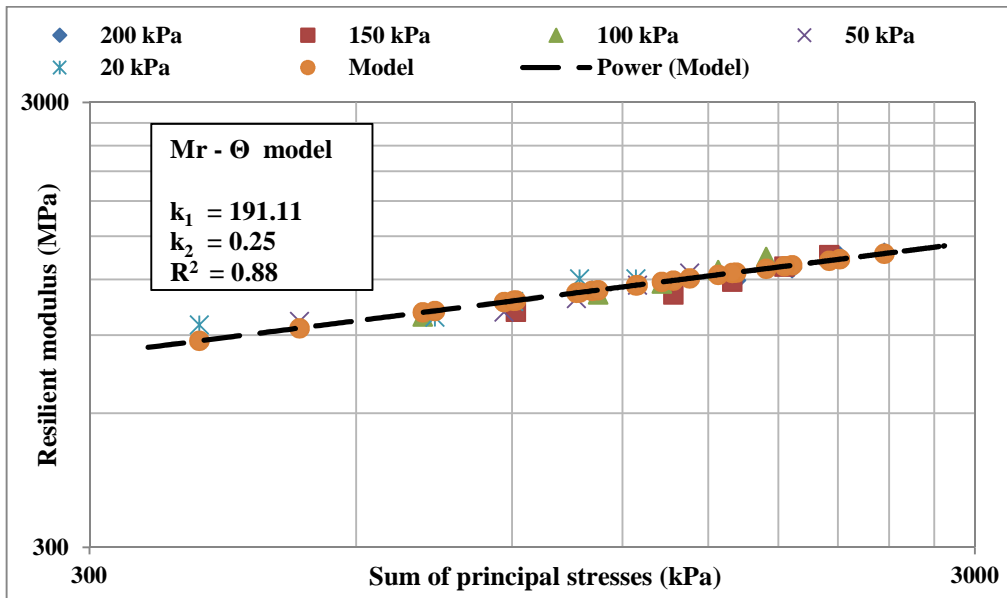
Figure H-12: M_r-θ modelling – Mix EB2.4-CM2-LD-LS
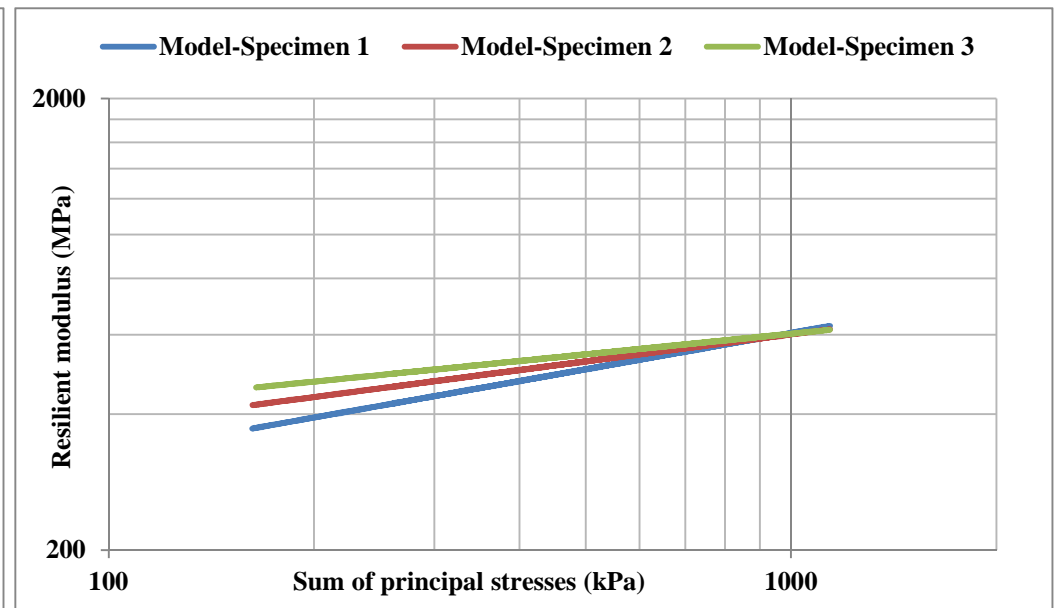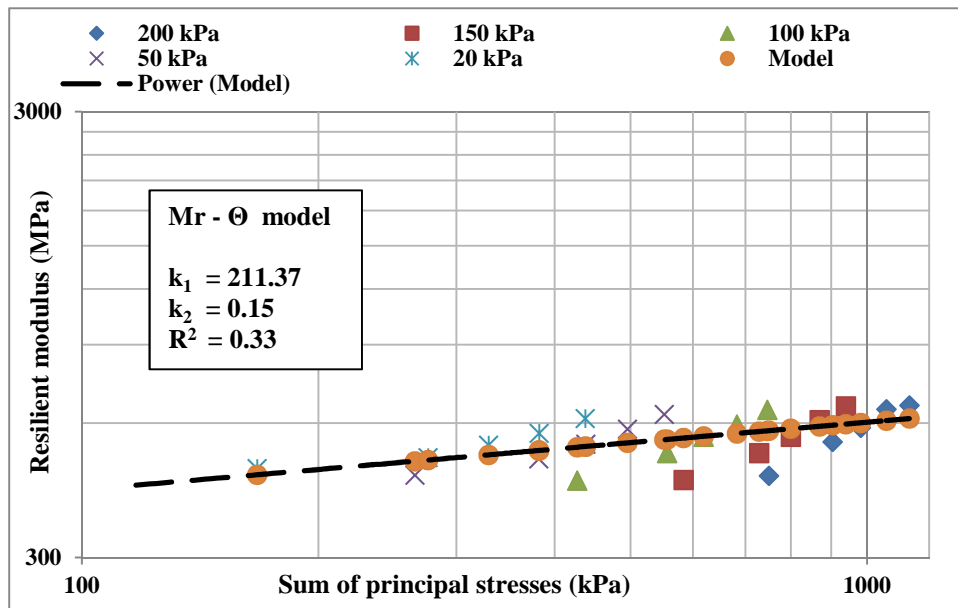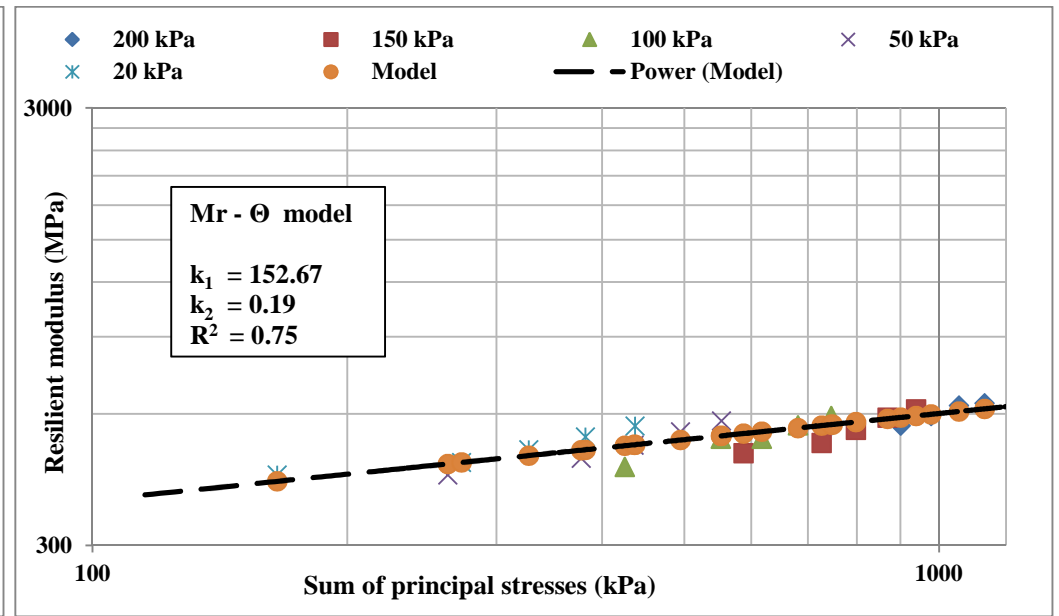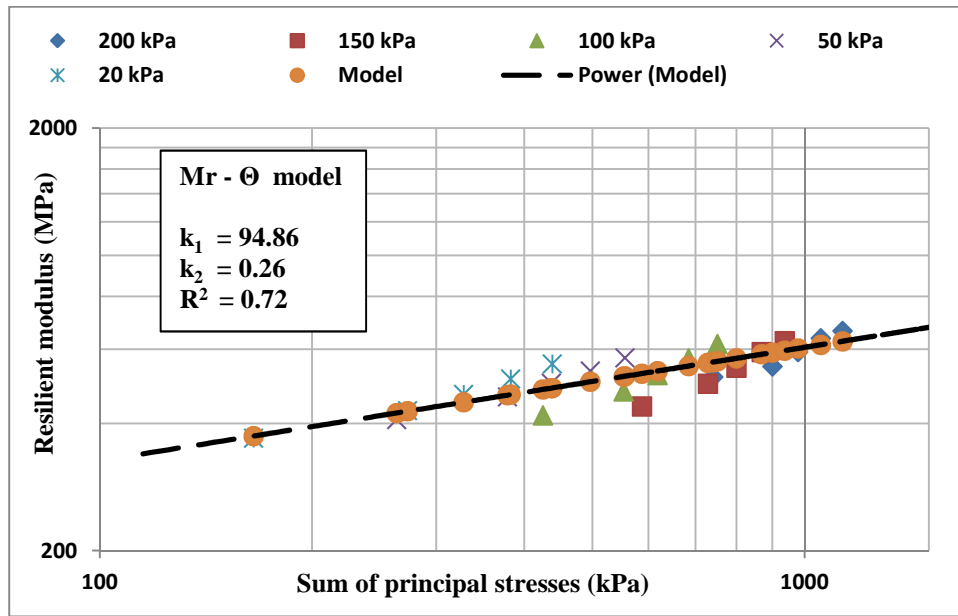
Figure H-13: M_r-θ modelling – Mix FB2.4-CM1-HD-HS
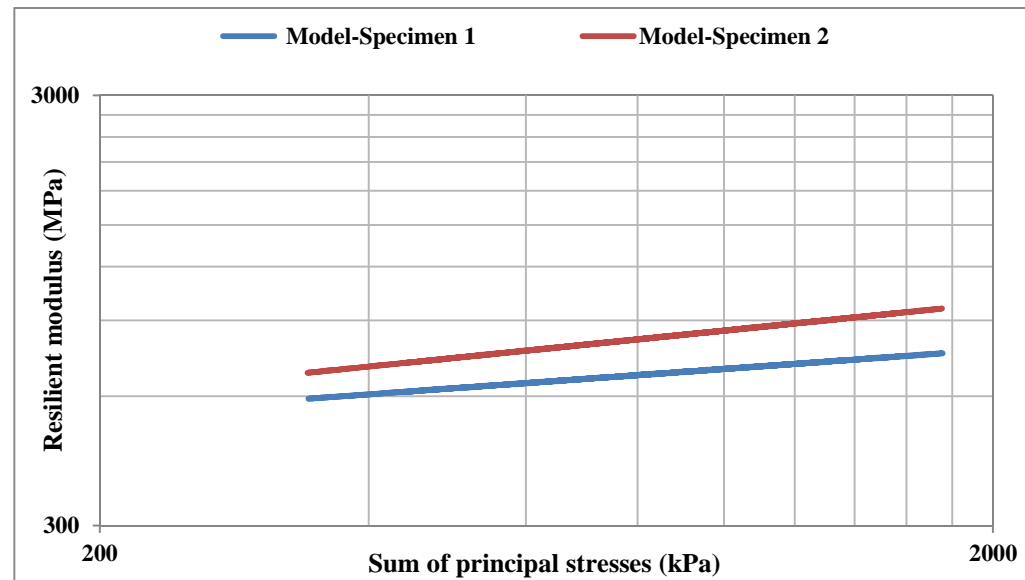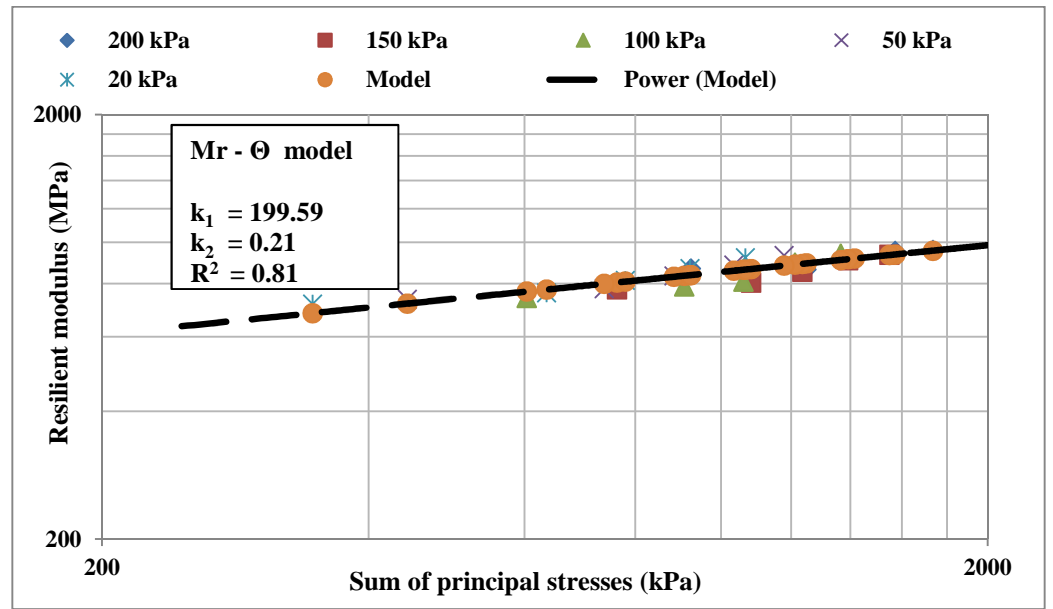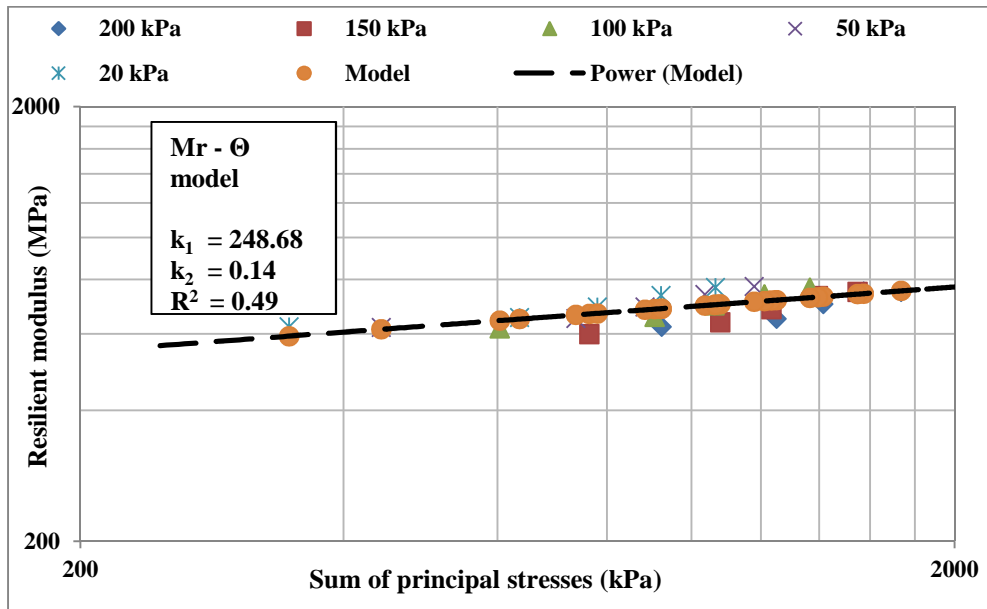
Figure H-14: M_r-θ modelling – Mix FB2.4-CM1-HD-LS
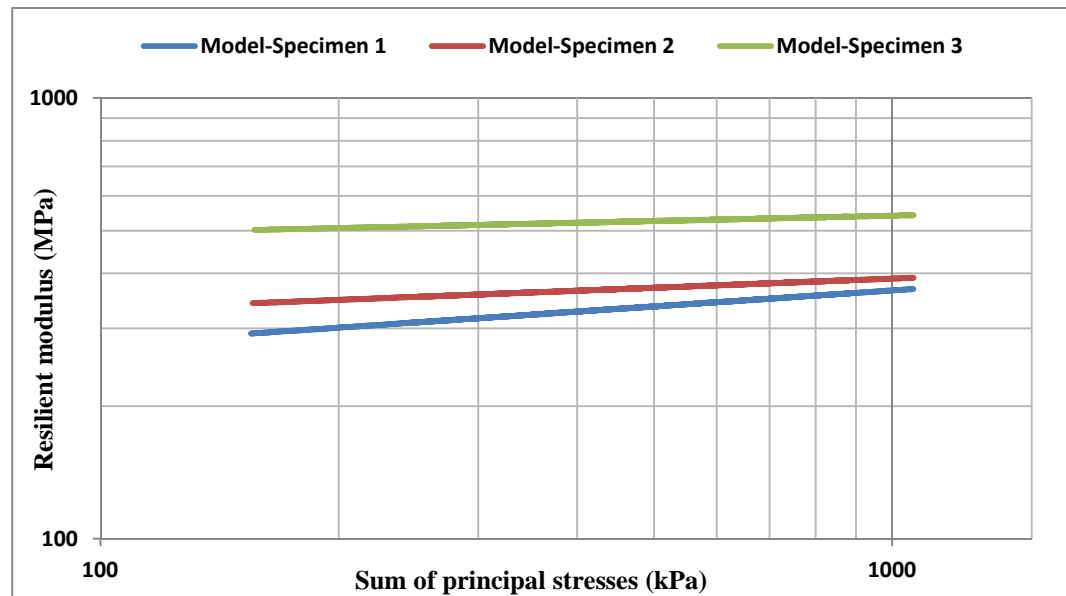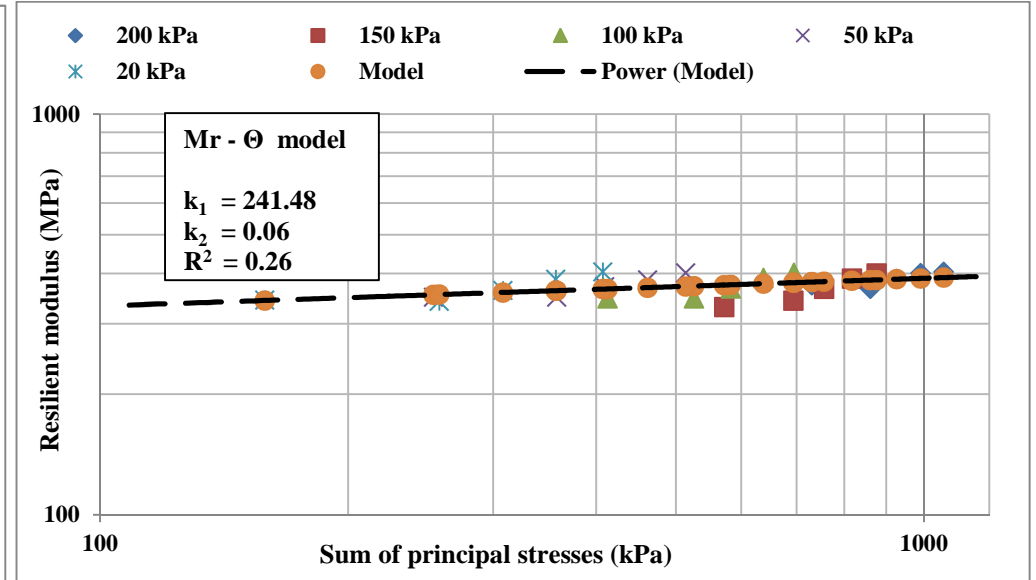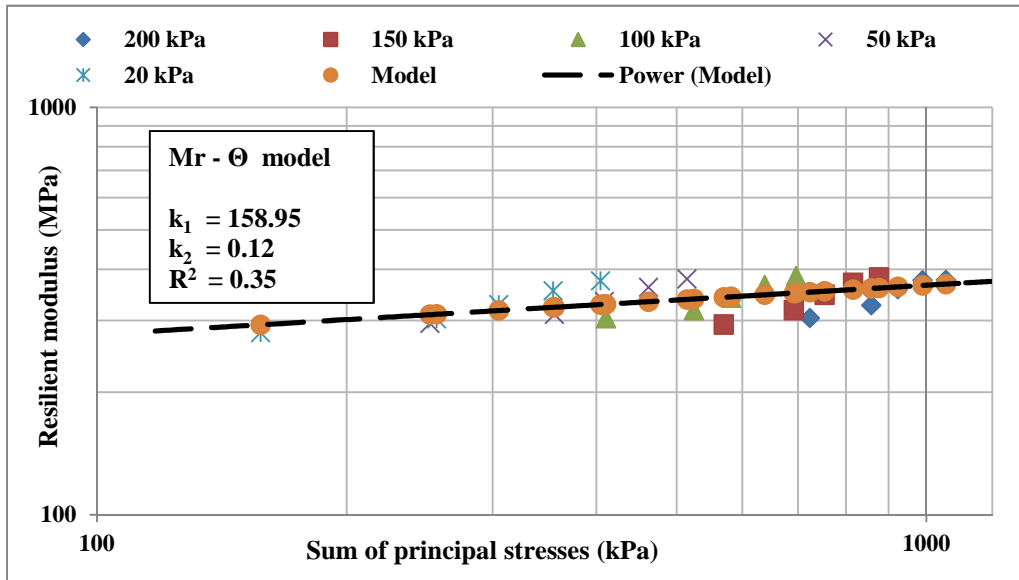
Figure H-15: Mr-θ modelling – Mix FB2.4-CM1-LD-HS
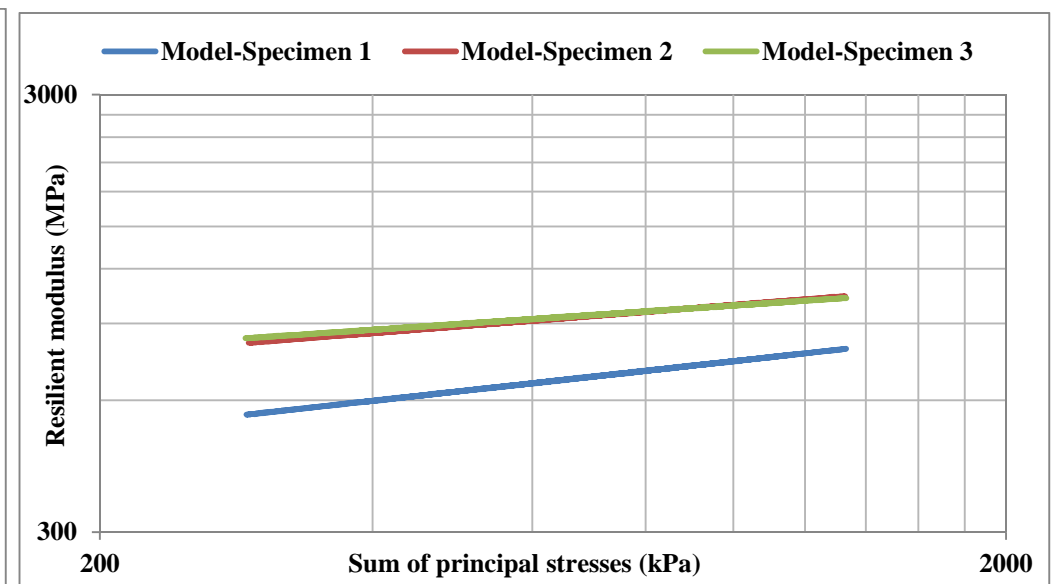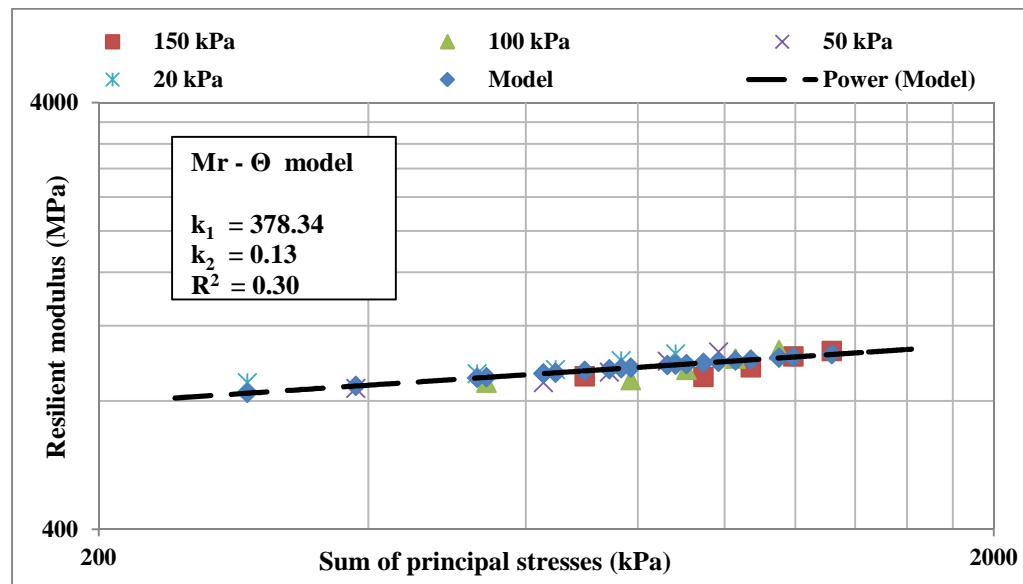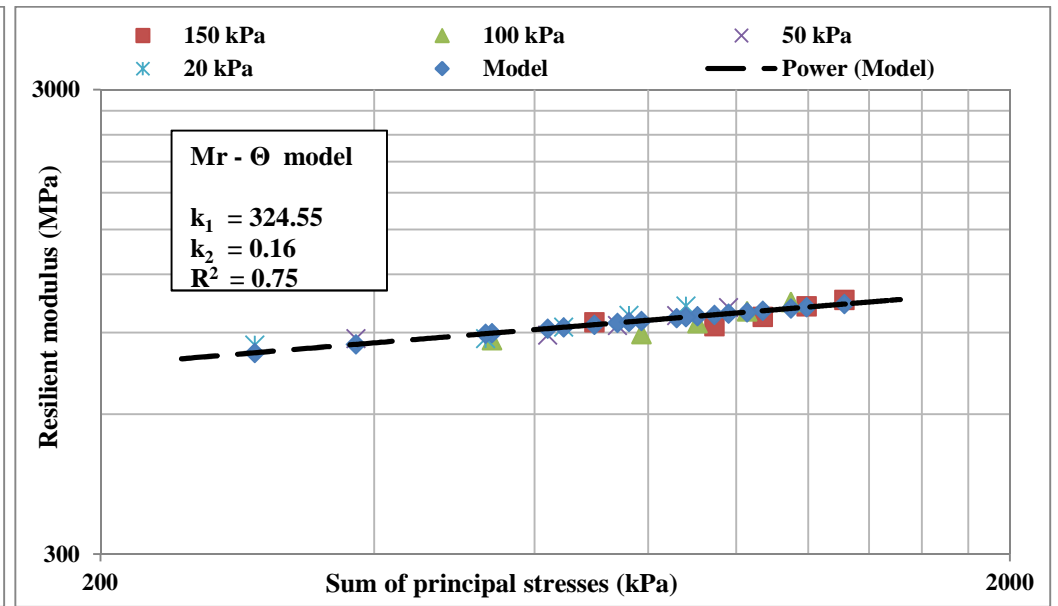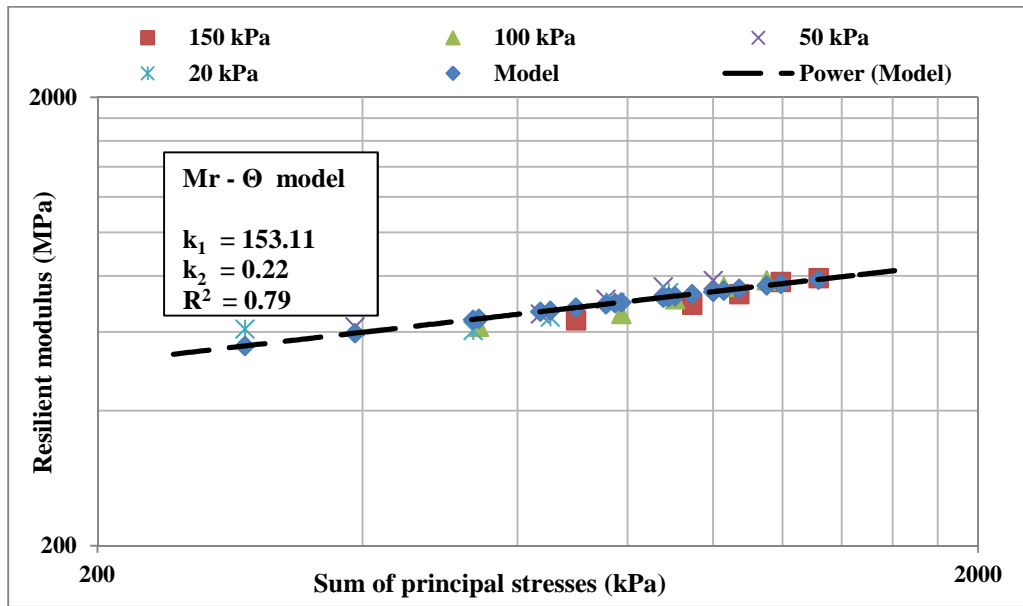
Figure H-16: M$_r$-θ modelling – Mix FB2.4-CM1-LD-LS
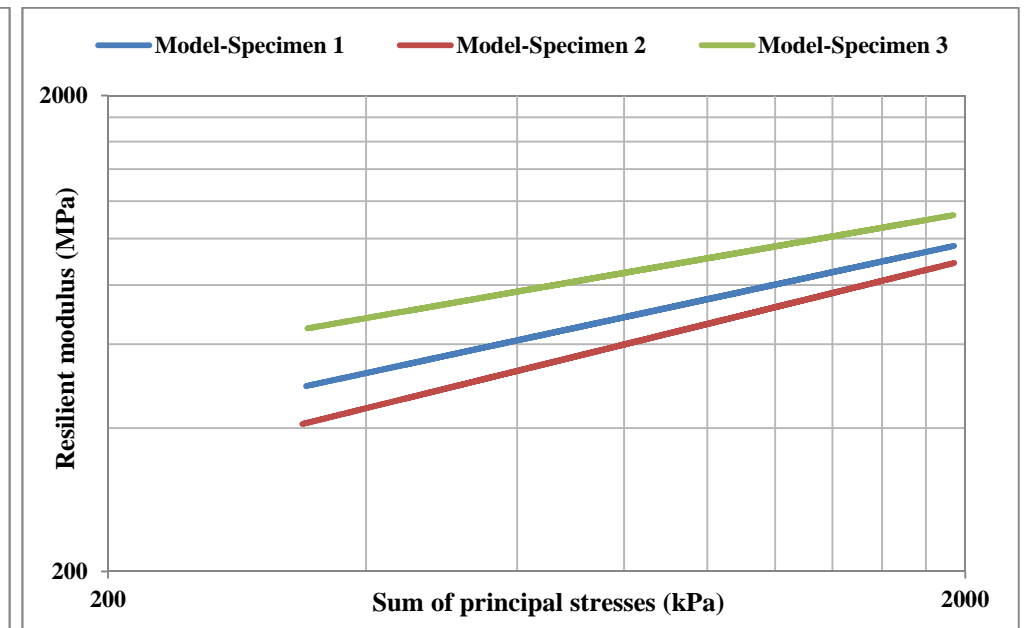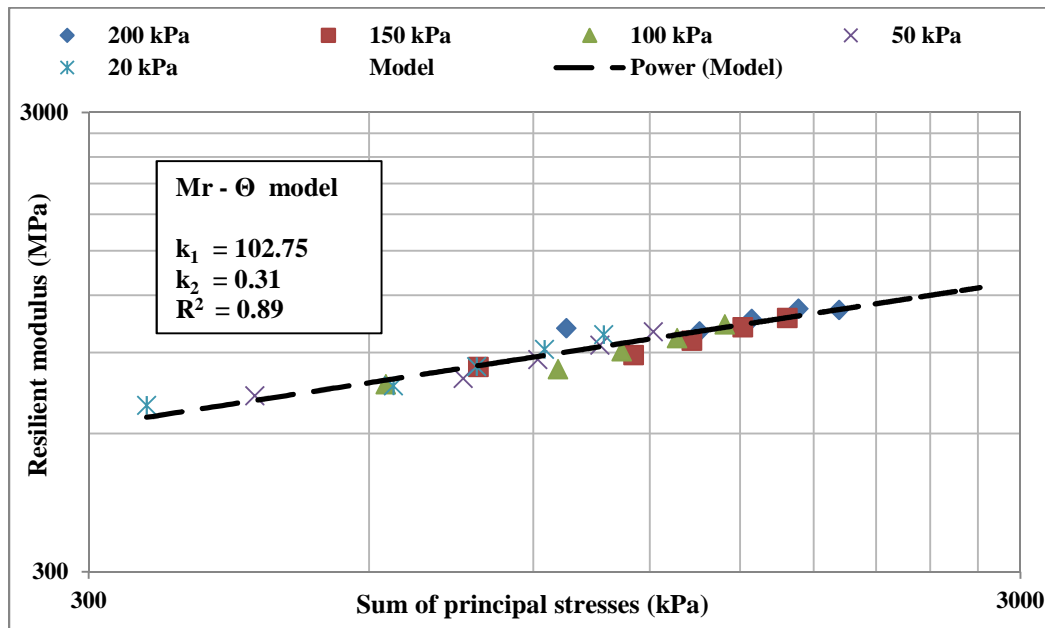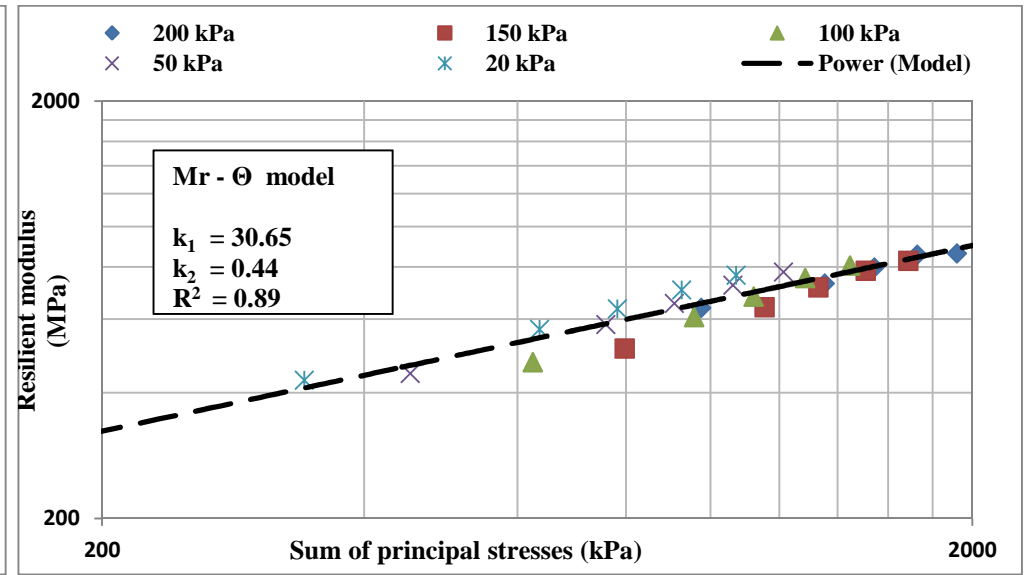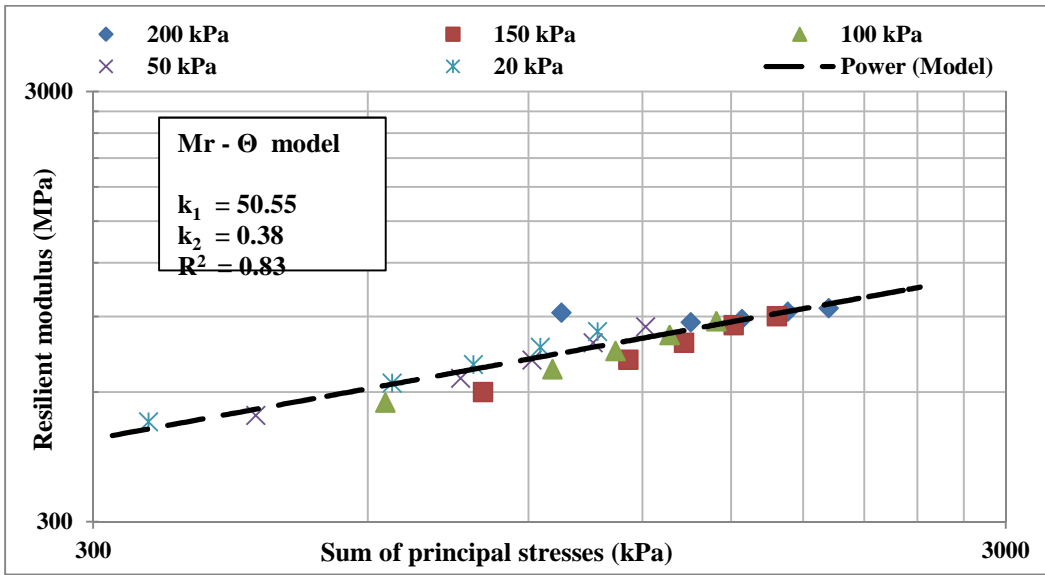
Figure H-17: M_r-θ modelling – Mix FB2.4-CM2-HD-HS
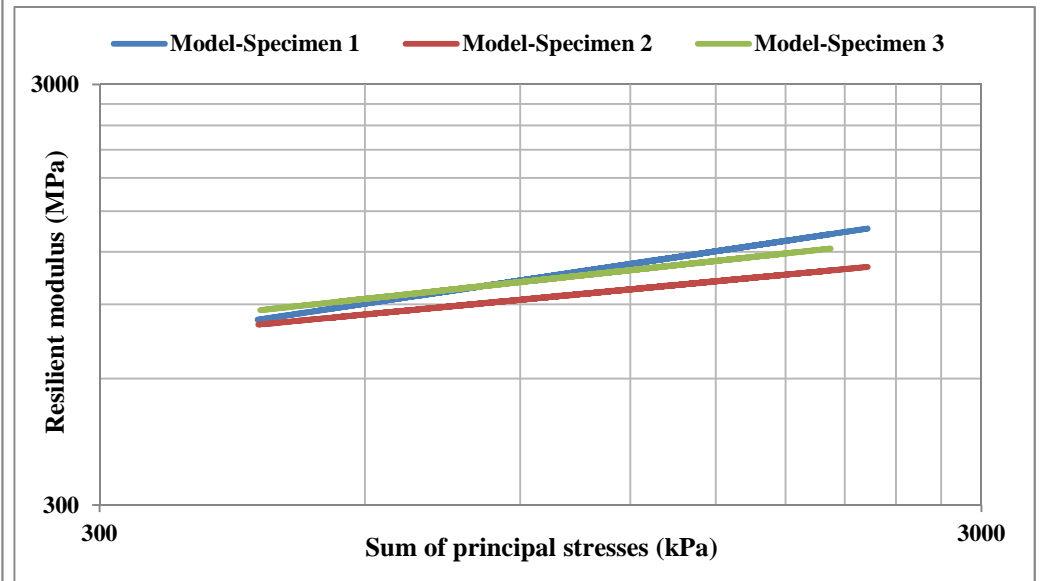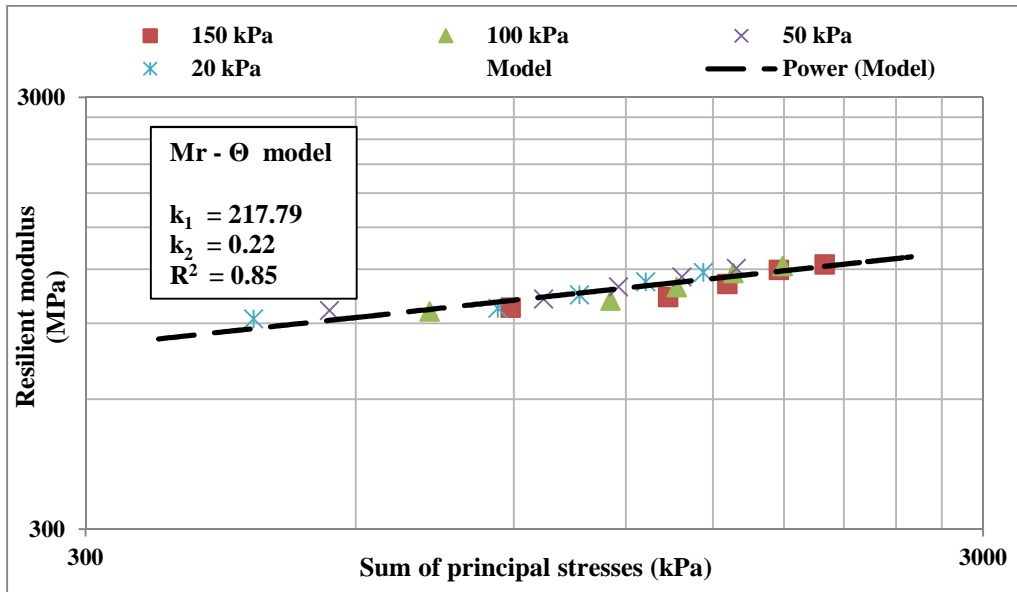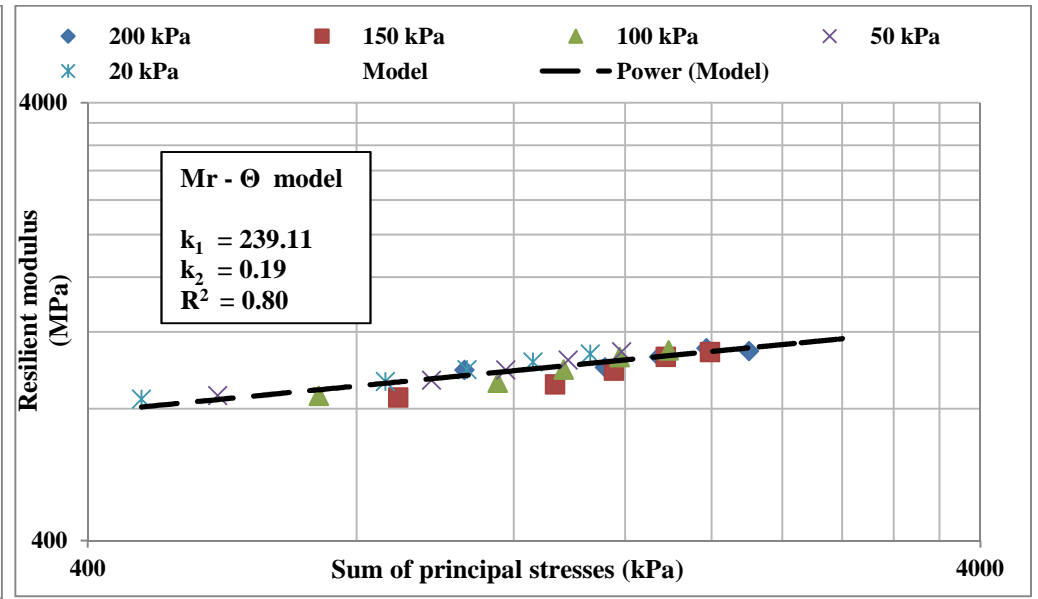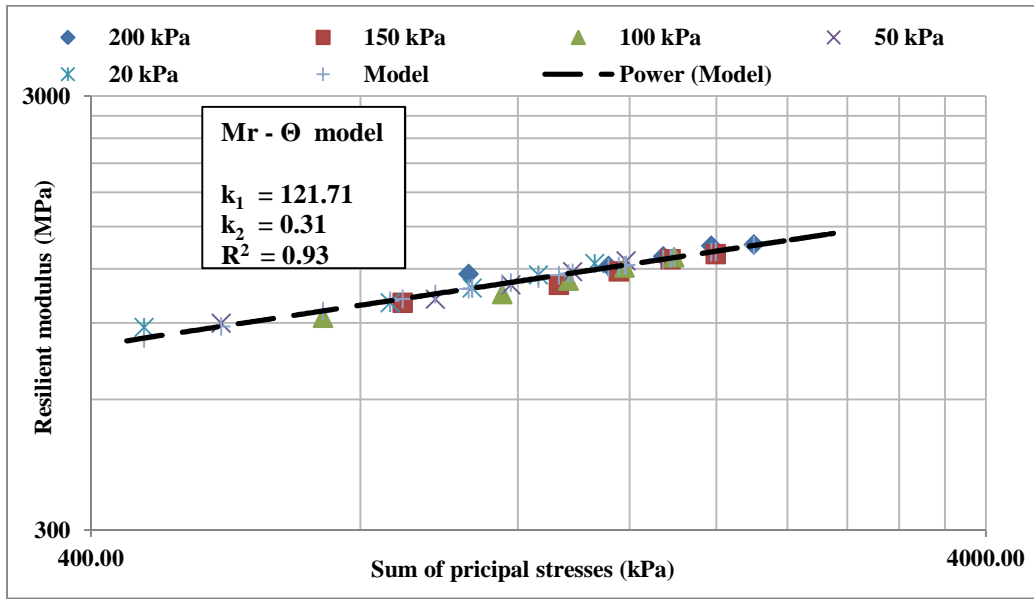
Figure H-18: M_r-θ modelling – Mix FB2.4-CM2-HD-LS
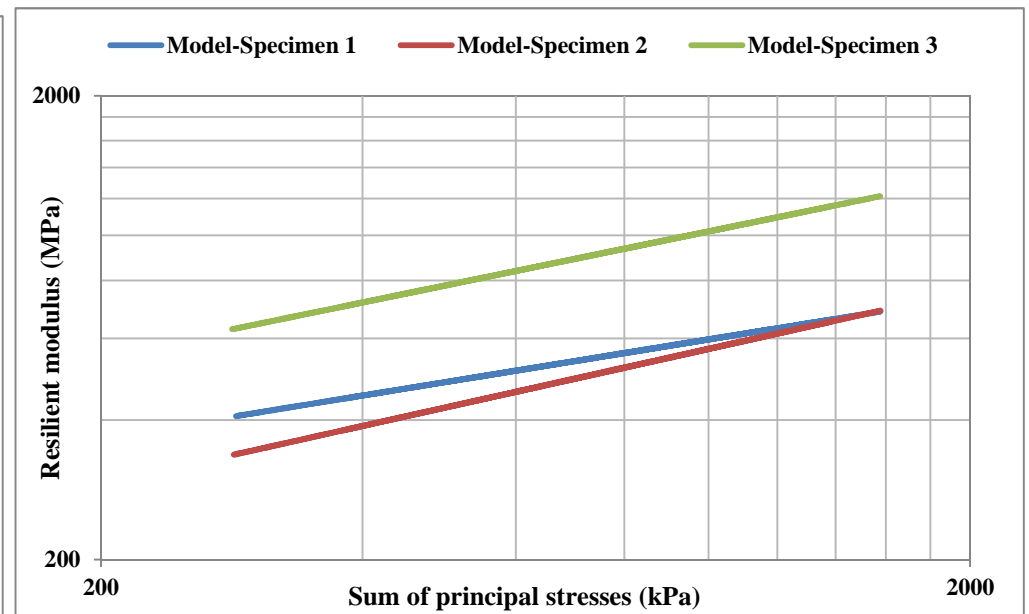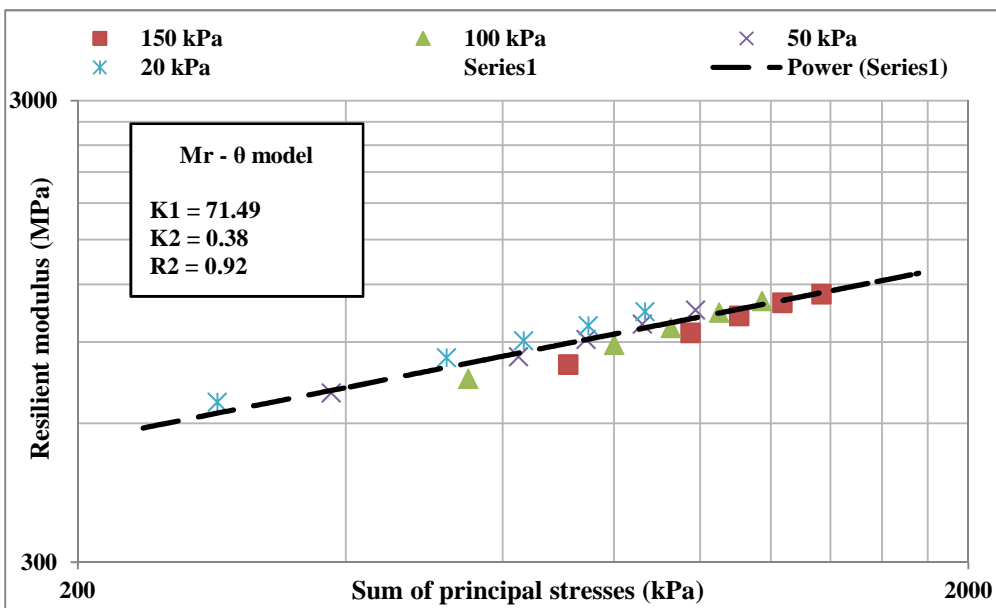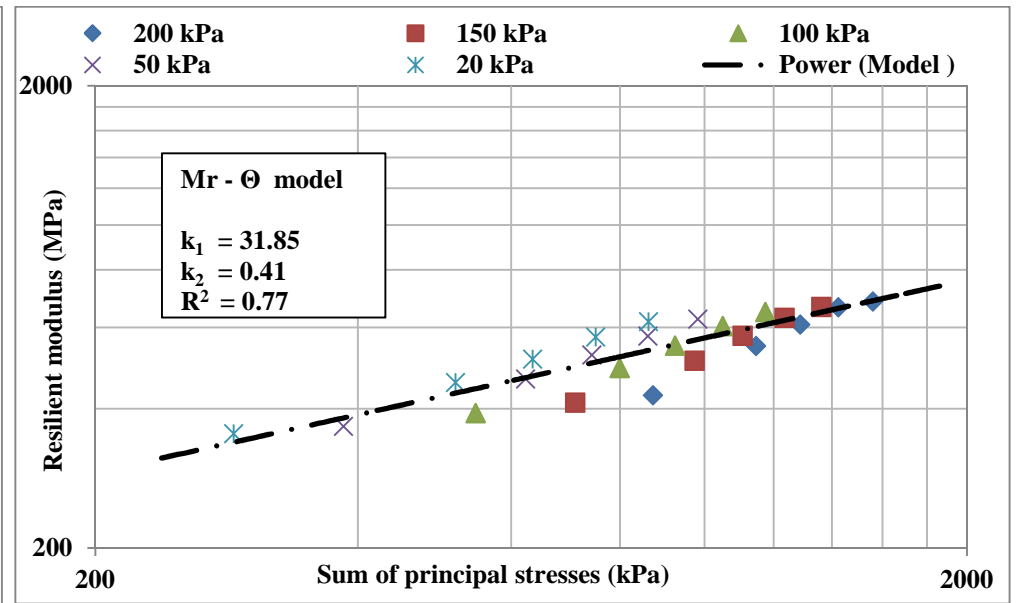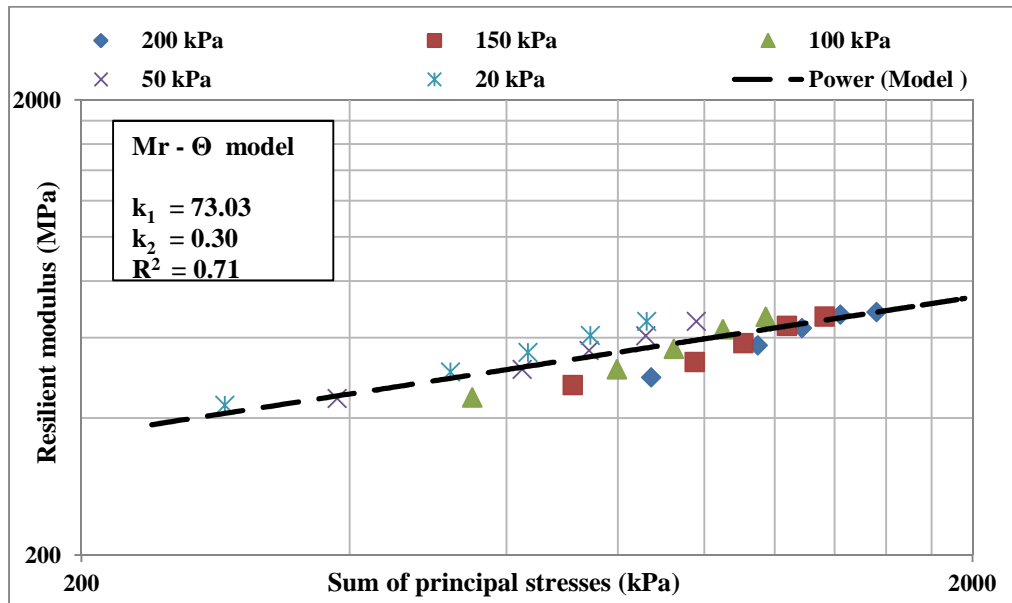
Figure H-19: M$_r$-θ modelling – Mix FB2.4-CM2-LD-HS
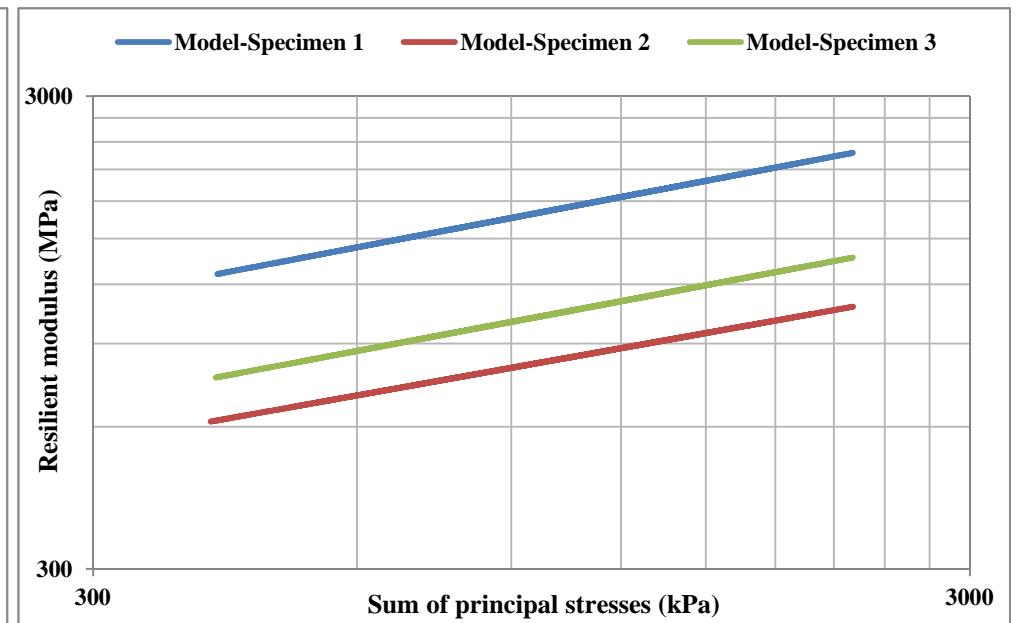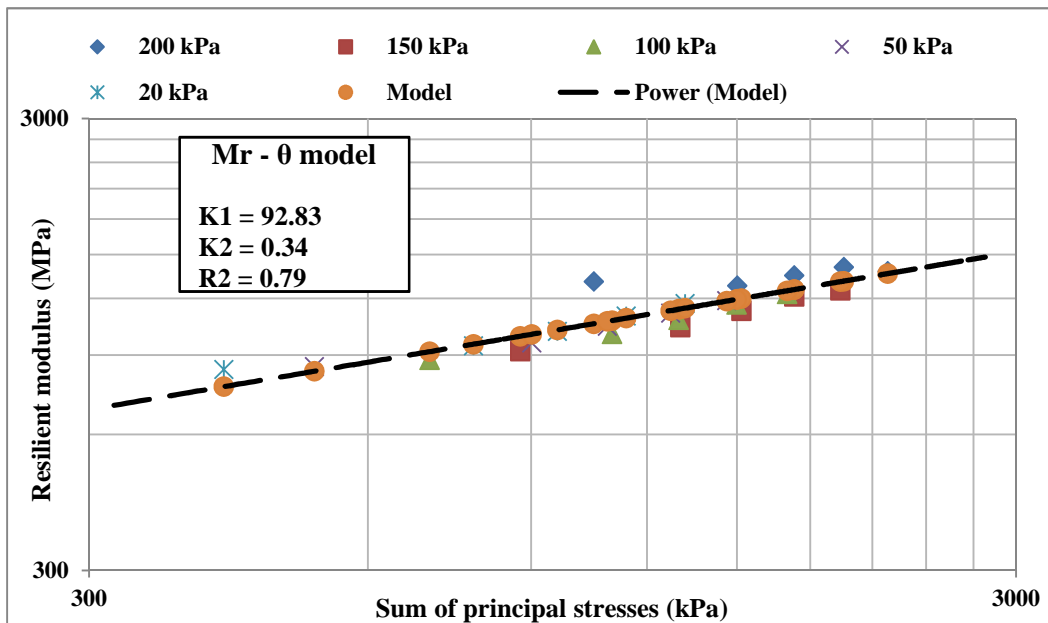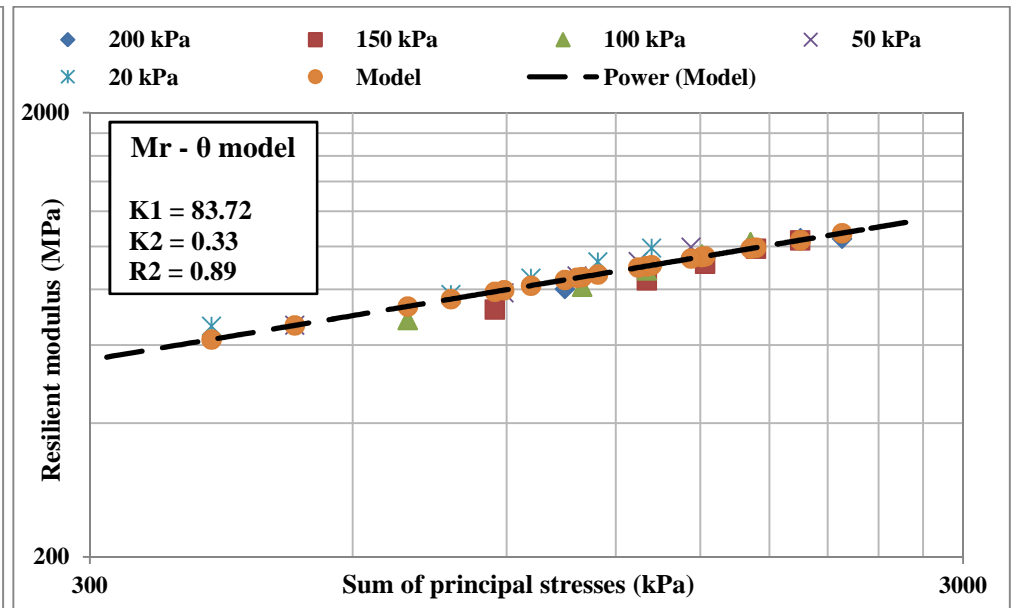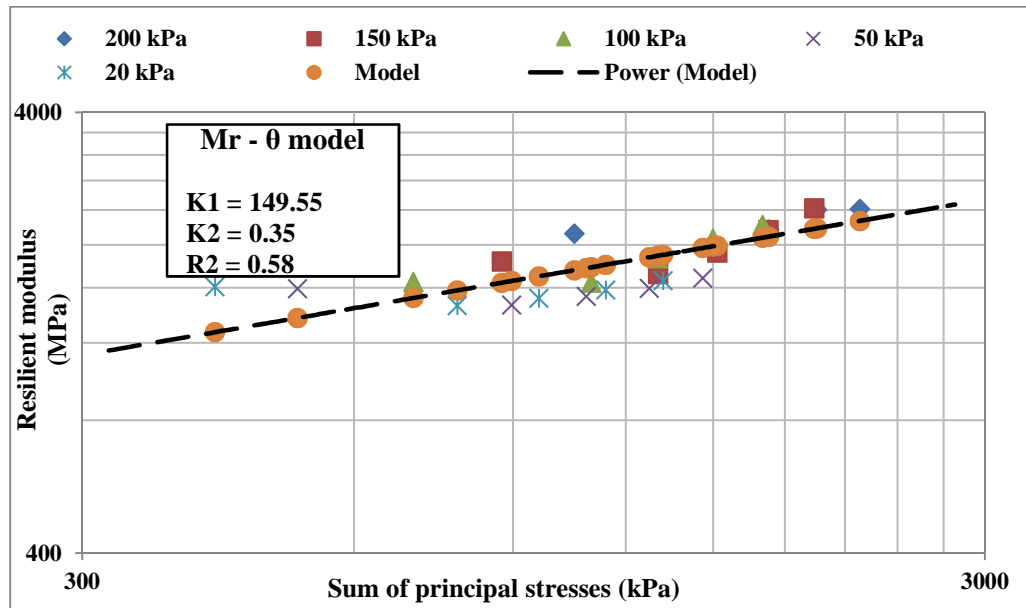
Figure H-20: M$_r$-θ modelling – Mix FB2.4-CM2-LD-HS

Appendix I: Effect of the relative density and saturation level on the predicted Mr values
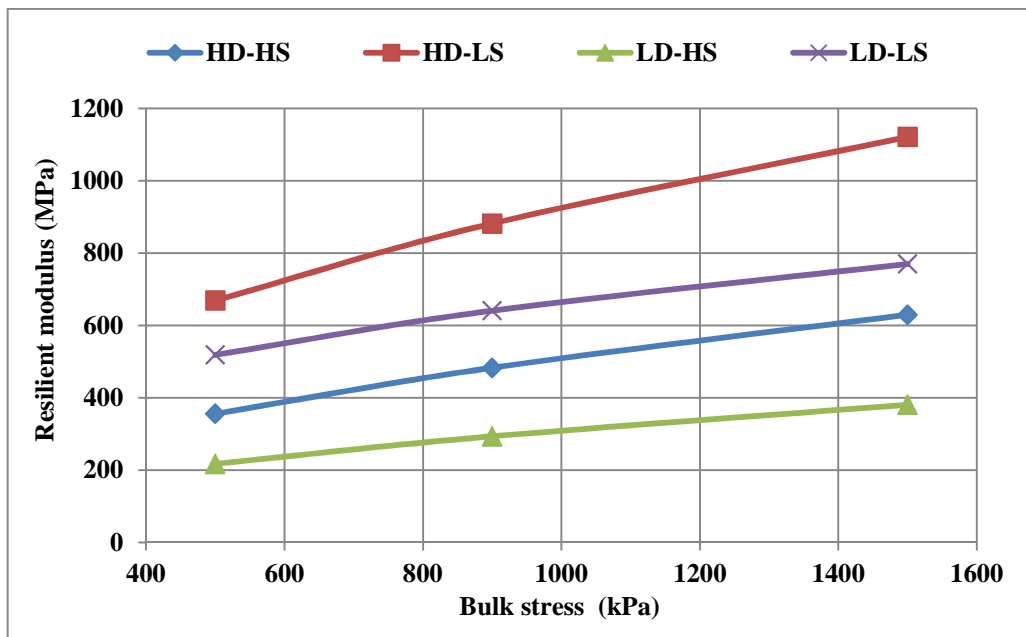
Figure I-1: Effect of the density and saturation on the Mr, mix EB0.9-CM1
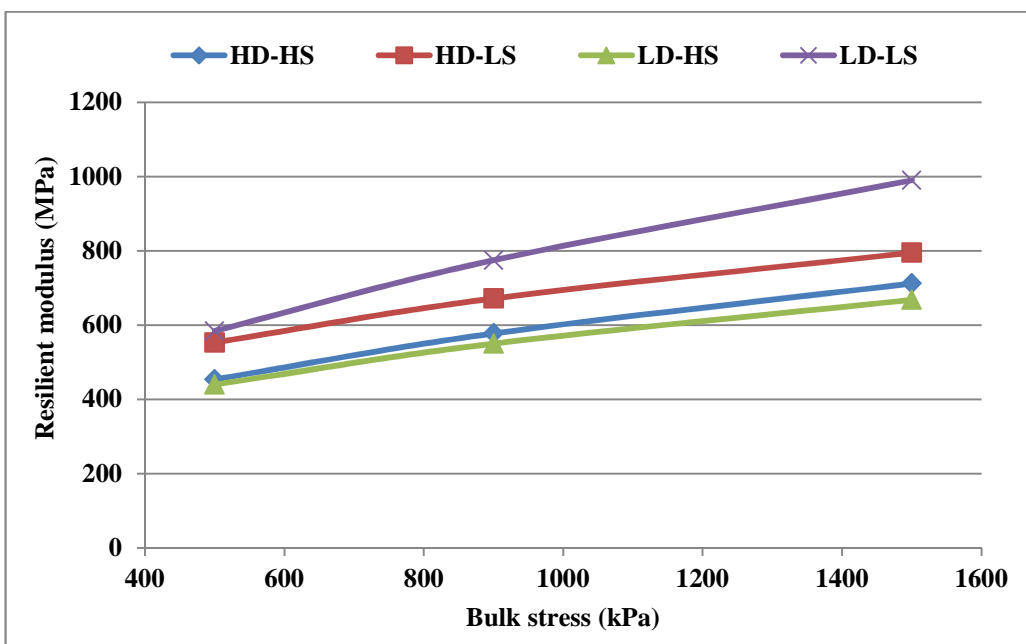
Figure I-2: Effect of the density and saturation on the Mr, mix EB2.4-CM1
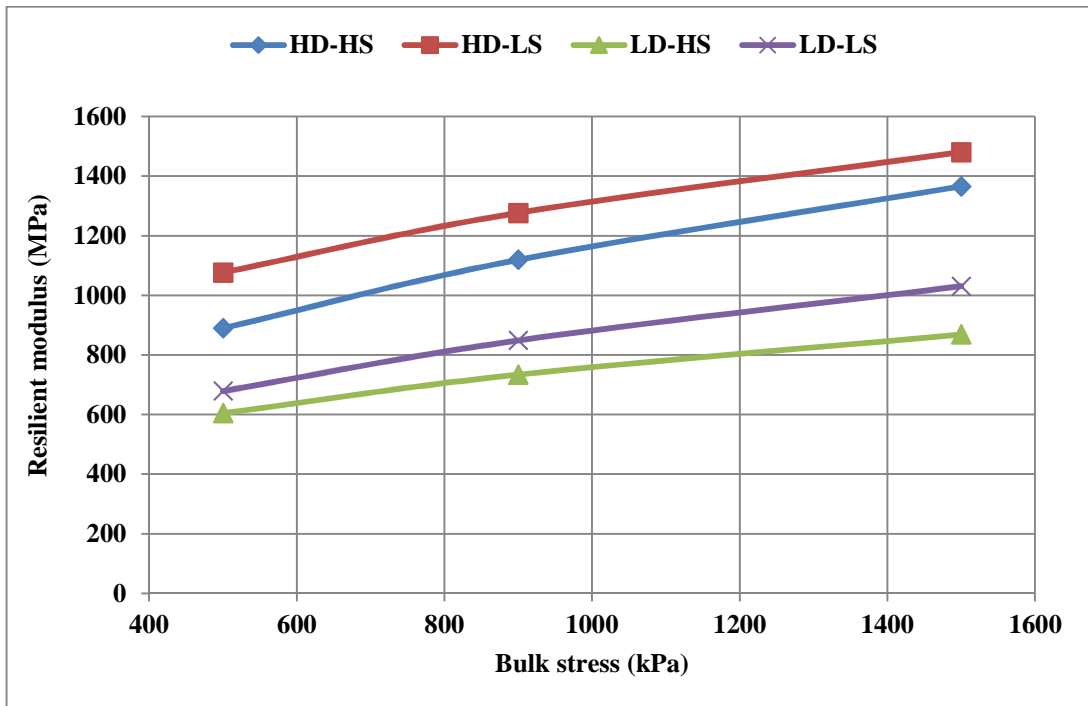
Figure I-3: Effect of the density and saturation on the Mr, mix EB2.4-CM2
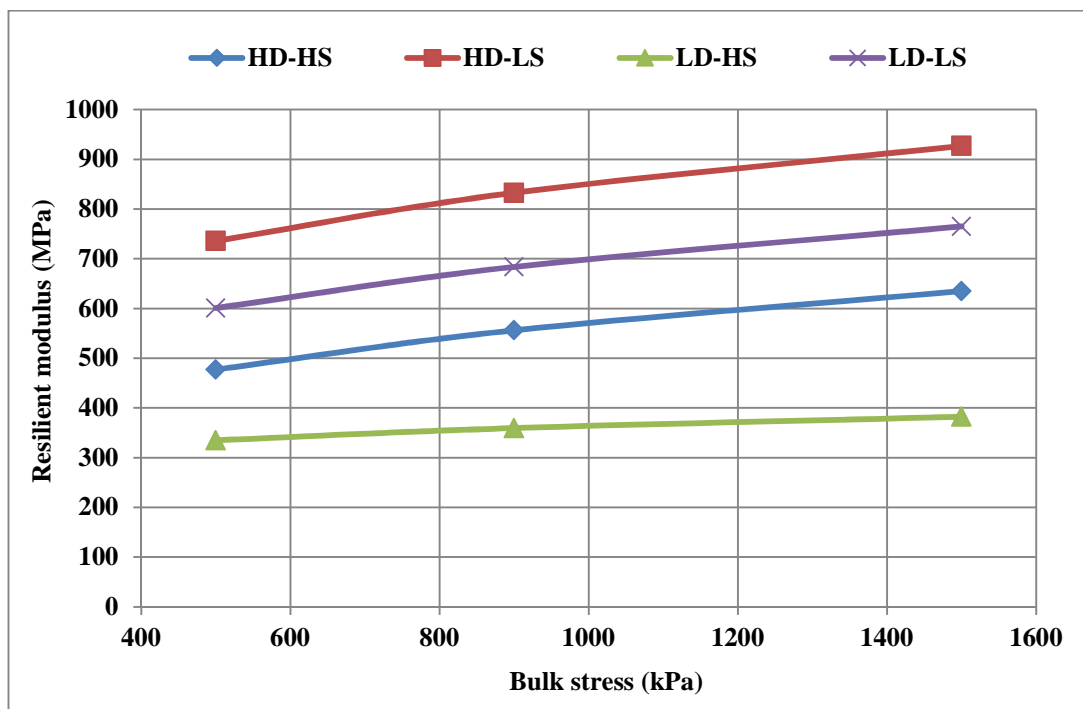
Figure I-4: Effect of the density and saturation on the Mr, mix FB2.4-CM1
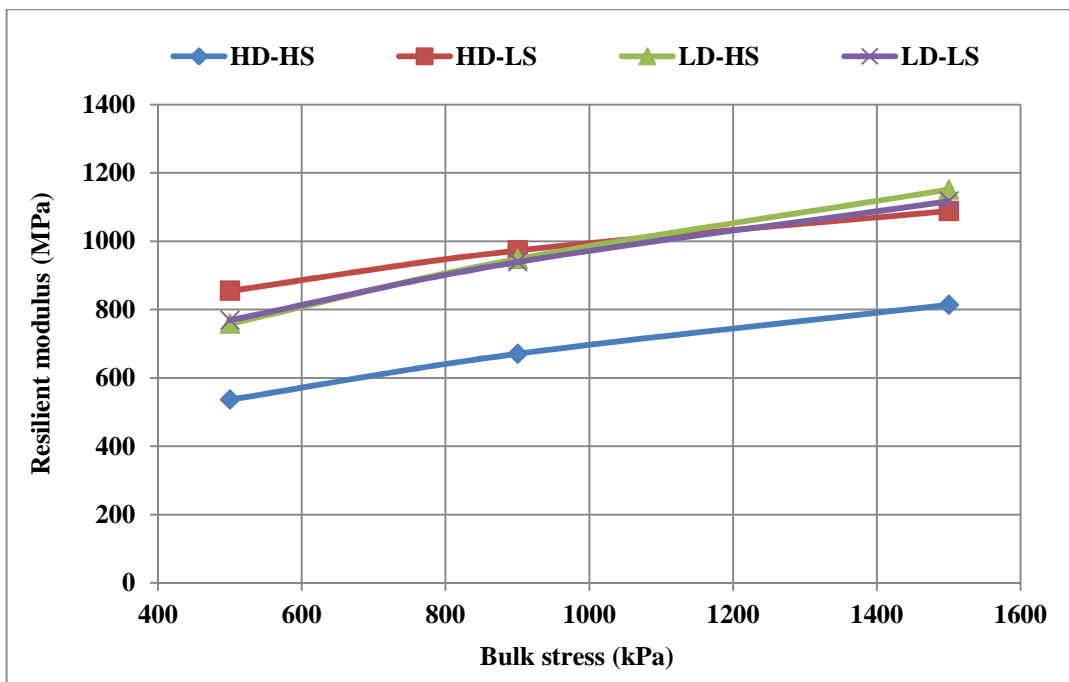
Figure I-5: Effect of the density and saturation on the Mr, mix FB2.4-CM2

Appendix J: Predicted Mr-values at chosen bulk stress

Predicted Mr at high density - high saturation in MPa					
Bulk stress (kPa)	EB0.9-CM1	EB2.4-CM1	EB2.4-CM2	FB2.4-CM1	FB2.4-CM1
500	355.75	453.87	889.91	477.33	536.20
900	482.93	577.55	1119.18	556.14	670.40
1500	629.86	712.11	1365.91	635.14	814.02
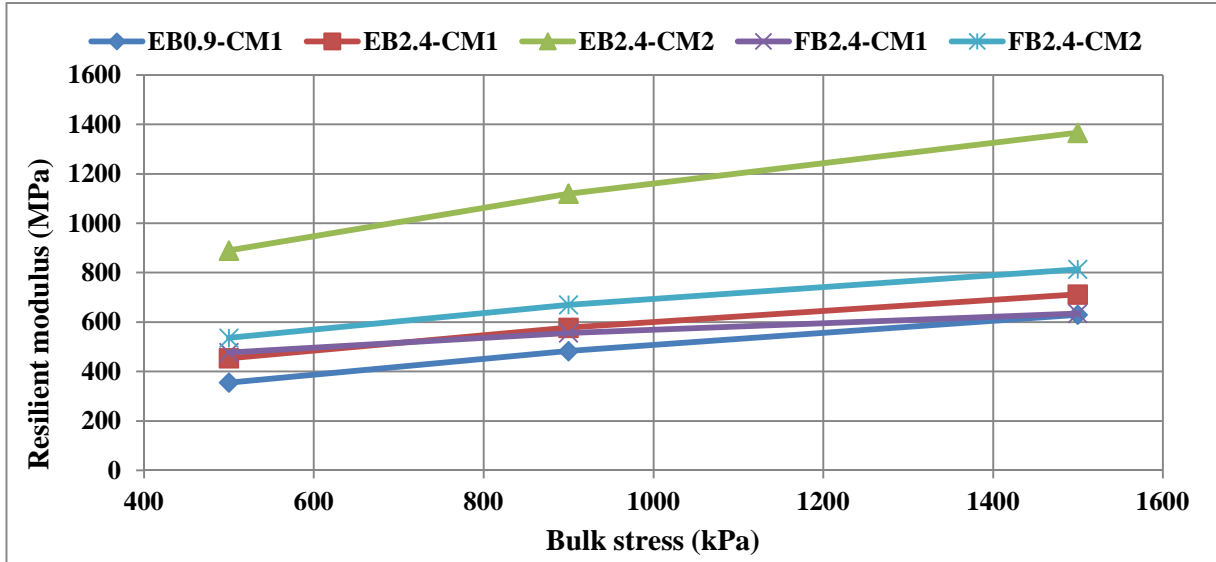
Figure J-1 Predicted Mr at high density - high saturation

Predicted Mr at high density - low saturation in MPa					
Bulk stress (kPa)	EB0.9-CM1	EB2.4-CM1	EB2.4-CM2	FB2.4-CM1	FB2.4-CM2
500	668.99	553.37	1076.70	736.08	854.70
900	881.86	671.83	1276.80	832.78	972.68
1500	1121.16	795.18	1480.68	927.08	1088.37
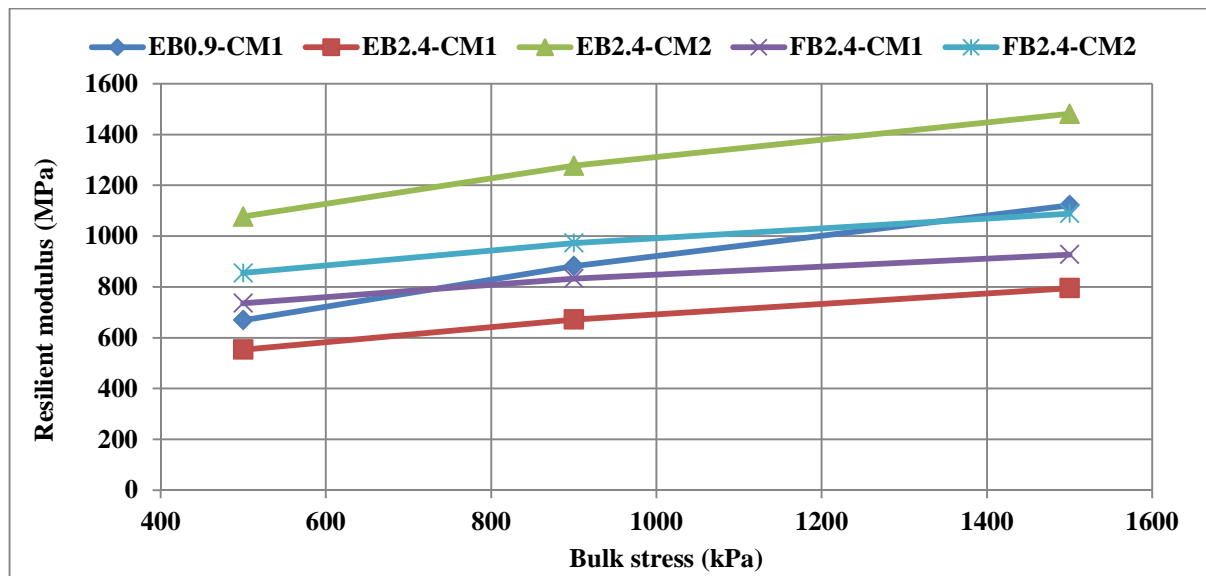
Figure J-2: Predicted Mr at high density - low saturation

Predicted Mr at low density - high saturation in MPa				
Bulk stress (kPa)	EB0.9-CM1	EB2.4-CM1	EB2.4-CM2	FB2.4-CM1	FB2.4-CM2
500	217.48	440.10	604.61	335.07	758.32
900	293.50	550.24	734.03	359.56	948.11
1500	380.85	668.13	868.80	382.29	1151.22
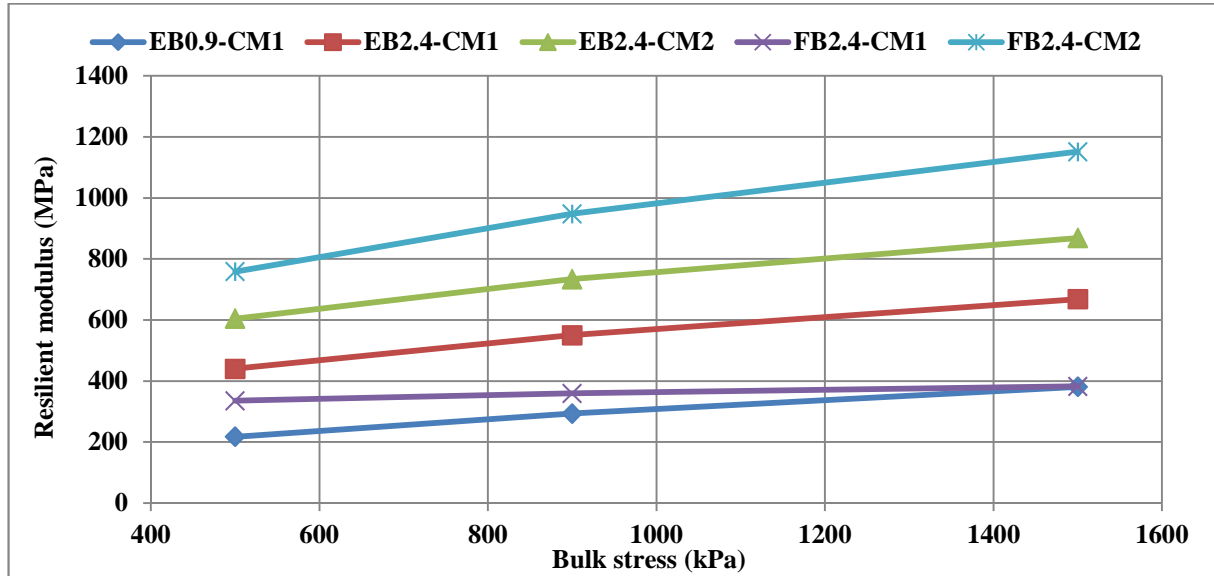
Figure J-3: Predicted Mr at low density - high saturation

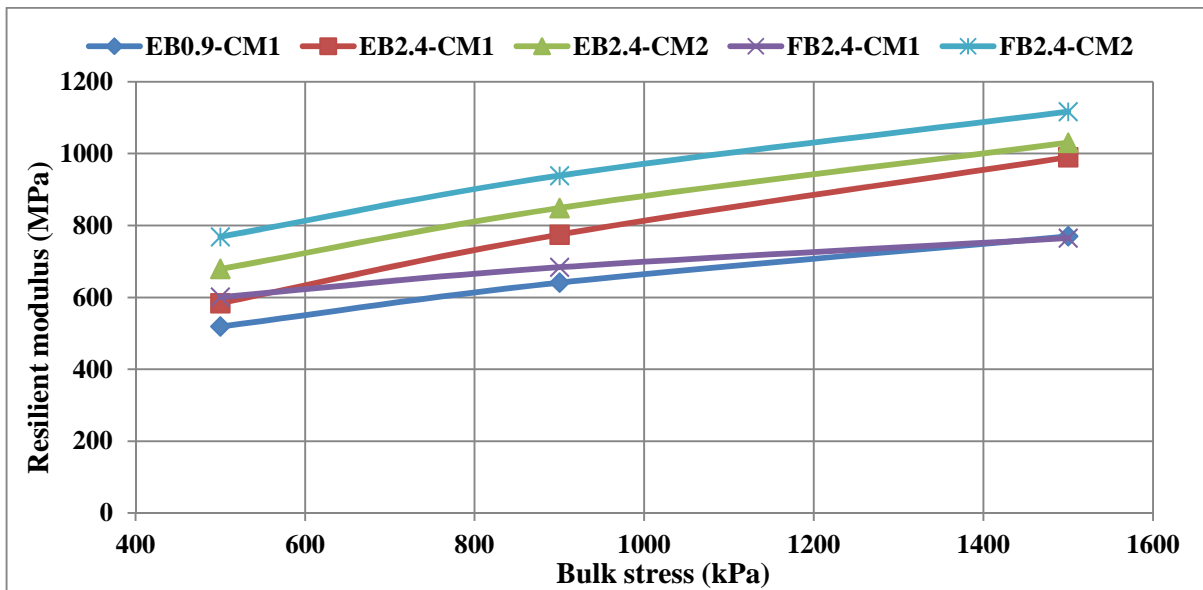| Predicted Mr at low density - low saturation in MPa				
Bulk stress (kPa)	EB0.9-CM1	EB2.4-CM1	EB2.4-CM2	FB2.4-CM1	FB2.4-CM2
500	518.59	584.32	678.87	600.87	768.79
900	640.80	774.79	848.77	683.81	938.85
1500	770.17	990.08	1030.61	765.14	1116.93

Figure J-4: Predicted Mr at low density - low saturation in

Appendix K: Effect of relative density and saturation level on the resilient modulus

Table K-1: Effect of saturation level on the resilient modulus EB0.9-CM1
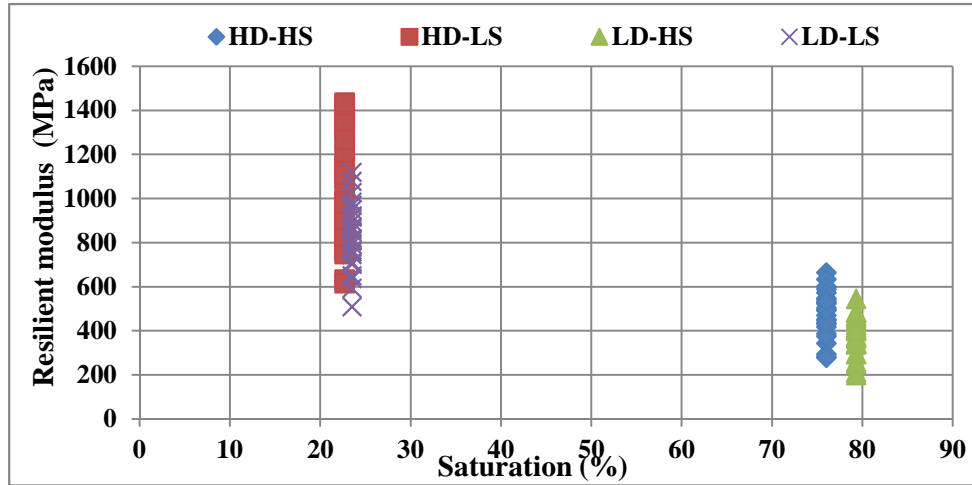

Table K-1: Effect of saturation level on the resilient modulus EB2.4-CM2
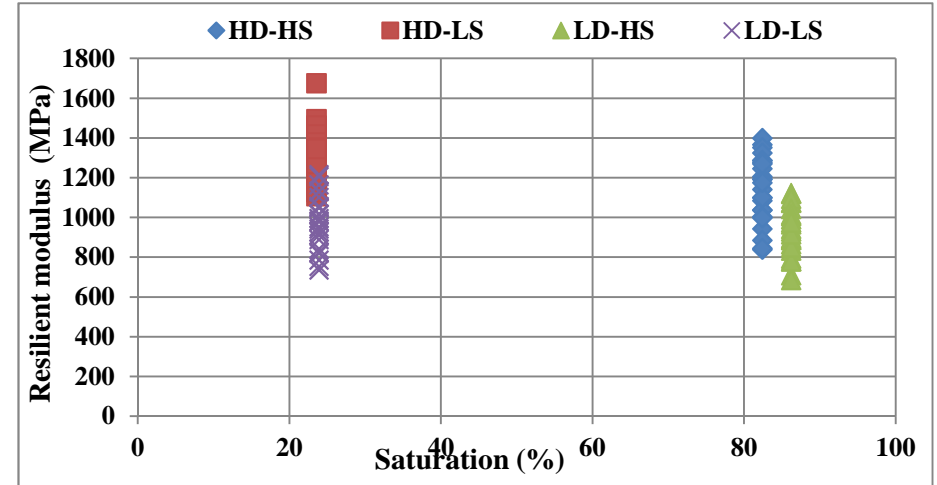

Table K-1: Effect of saturation level on the resilient modulus FB2.4-CM1
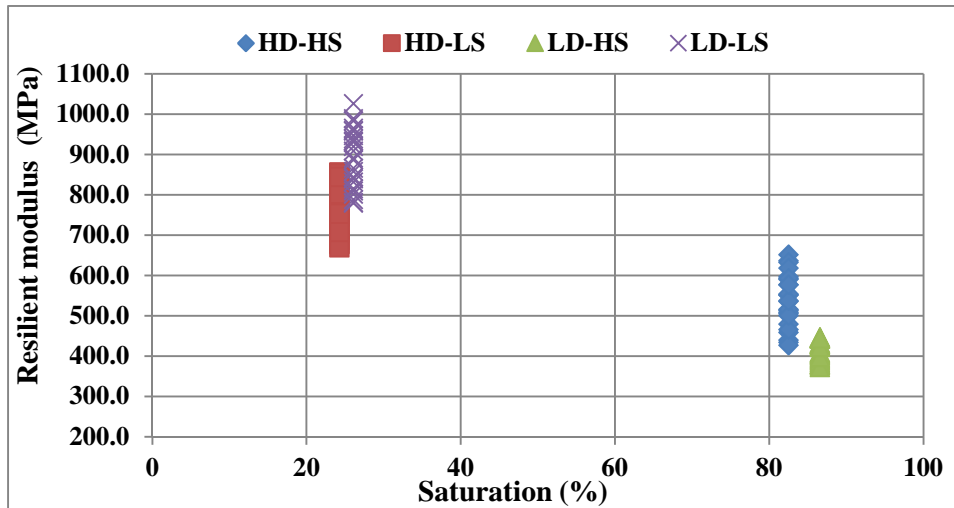

Table K-1: Effect of saturation level on the resilient modulus FB2.4-CM2